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ALAMEDA AND RIVERSIDE DRAINS ENGINEERING ANALYSIS

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8053.42

VOLUME I

Prepared for
CITY OF ALBUQUERQUE

MAY 1991

LEEDSHILL HERKENHOFF, INC.
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P.O. Box 1217 - Phone (505) 247-0294
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ENGINEERS ARCHITECTS

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VOLUME I

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ENVIRONMENTAL ASSESSMENT FOR ALAMEDA AND RIVERSIDE DRAINS
ENGINEERING ANALYSIS

1.0. INTRODUCTION

1.1 Purpose

The Alameda and Riverside Drains Engineering Analysis has been performed to evaluate and determine the most effective utilization of the drains to convey stormwater from the Albuquerque East Valley area to the Rio Grande. Project analysis includes hydrologic and hydraulic evaluations, and water quality and environmental assessments for existing and improved drain conditions based on both existing and future introductions of stormwater to the drains.

1.2 Authorization and Objective

The official name of Riverside Drain is the Albuquerque/Barr Riverside Drain, but it will hereinafter be referred to simply as Riverside Drain. The Alameda and Riverside Drains are owned and operated under the jurisdiction of the Middle Rio Grande Conservancy District (MRGCD). They are located in the eastern Rio Grande Valley area of Albuquerque, New Mexico. The drains have received stormwater flows from the Valley area since their initial construction simply because they are a low point in the valley. In addition, legislation which officially established MRGCD provided for the agency to have a role in flood protection of the East Valley area. In recent years, the City of Albuquerque (City), Bernalillo County and private developers have intensified the use of these drains as an outfall facility for conveyance of stormwater runoff from the East Valley to the Rio Grande.

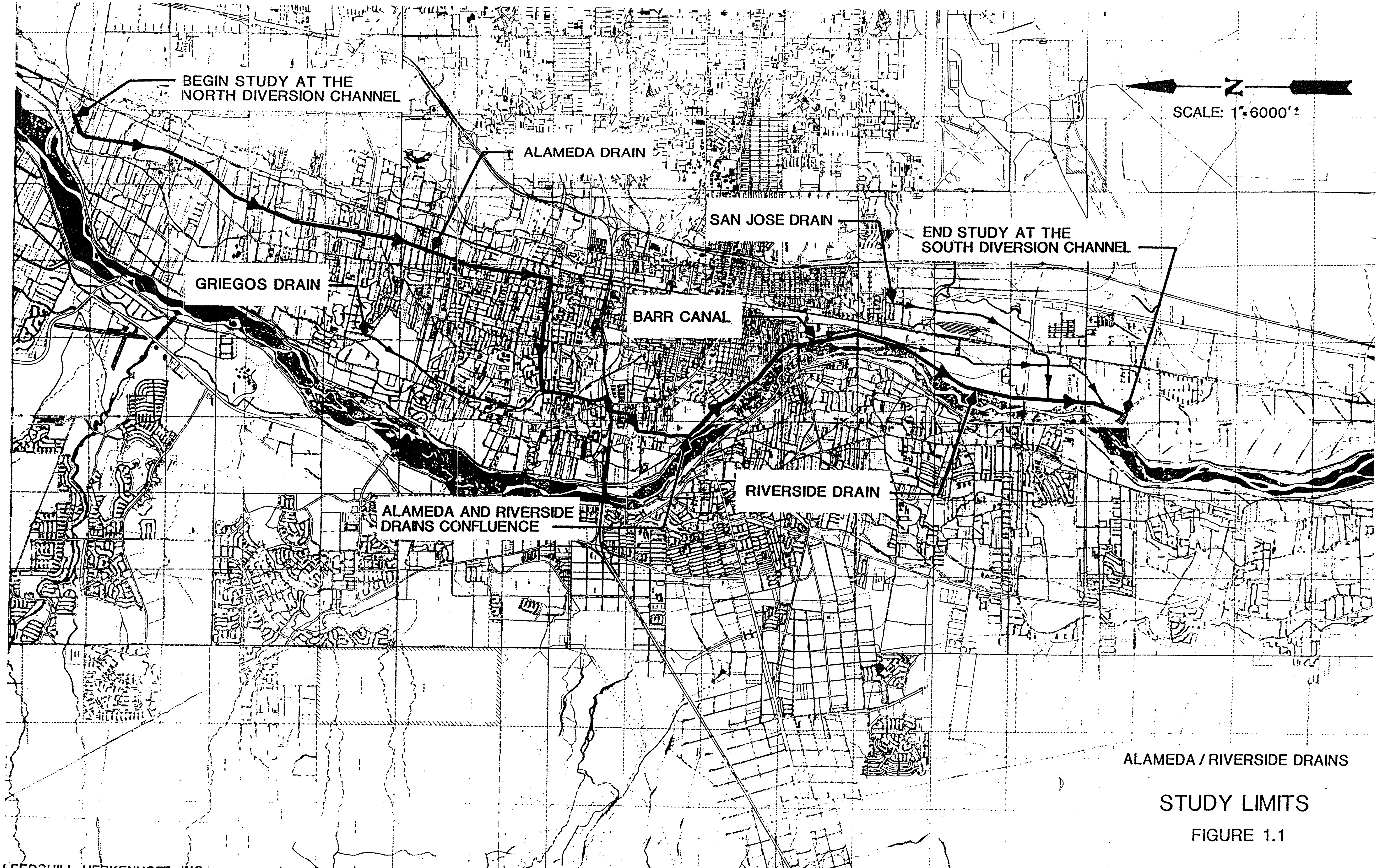
MRGCD requires applications for a license approval before allowing the discharge of urban stormwater into the drains. A summary of those stormwater discharges entering the Alameda and Riverside Drains which are licensed by the MRGCD is included in Appendix C, Volume II. In 1987, the MRGCD Board placed a moratorium on any additional license approvals until a resolution is made between the City and MRGCD regarding a plan for improvements to the Alameda and Riverside Drains in order to safely convey stormwater flows in the drains without detrimentally impacting operations of the MRGCD irrigation and drainage system.

As a result, in December 1988, Leedshill-Herkenhoff, Inc. entered into a contract with the City to perform a conceptual design analysis to determine the feasibility and effectiveness of using the Alameda and Riverside Drains for conveying existing and additional urban storm runoff from the East Valley metropolitan area to the Rio Grande. Analysis includes considerations for increasing the capacity of the drain facility.

The analysis and design for this project are the result of the combined efforts of Leedshill-Herkenhoff, Inc. as the lead consultant along with Metric Corporation heading up the water quality evaluation and environmental assessment, assisted by Kramer and Associates for the performance of laboratory testing of the water quality samples.

The conceptual design analysis summarized in this report includes a preliminary environmental assessment of the drains as well as a hydraulic evaluation of the conveyance capacity of the existing drain system. In addition, evaluation of potential improvements which increase the effectiveness of the drains for conveying stormwater runoff are presented.

The study limits for hydraulic analyses of the drains begin at the Alameda Drain's intersection with the North Diversion Channel, continue southerly to its confluence with Riverside Drain, and include the Riverside Drain from its confluence with the Alameda Drain southerly to its intersection with the South Diversion Channel. Figure 1.1 illustrates the limits of study for hydraulic analyses. Water quality analyses extended beyond these limits on Riverside Drain south to its intersection with Interstate 25 because of the concern regarding contributions from the San Jose Drain, the Barr Drains and the total effect on water quality at the north boundary of the Isleta Pueblo. As well, water quality sampling and evaluations where performed on Riverside Drain just north of its confluence with the Alameda Drain. Although the Griegos Drain and the San Jose Drain are contributing elements, water quality evaluations were not included for these drains as part of this project.



ALAMEDA / RIVERSIDE DRAINS

STUDY LIMITS

FIGURE 1.1

LEEDSHILL-HERKENHOFF, INC.

1.3 Project Scope

The scope of the Alameda and Riverside Drains Engineering Analysis specifically includes the following tasks:

A. Inventory of existing drain conditions including inlets, crossing structures, vegetation, and any other appurtenances which impact flow conditions in the drains.

B. Field survey sufficient to characterize the existing system as to:

   1. Available capacities

   2. Conveyance deficiencies for a range of storm flow rates

   3. Comparison of existing drain conditions to recorded data

   4. Quality of existing and anticipated stormwater inflow into the drains

   5. Drain management practices and needs

   6. Operation and maintenance practices and needs.

C. Conceptual design of improvements necessary to achieve 100-year storm event conveyance capacity, based on a predefined limited quantity of storm water introductions from certain contributing areas.

D. Environmental assessment of existing conditions and impact analysis for improvement alternates.

E. Construction cost estimates and operation and maintenance costs for improvements and environmental mitigation.

1.4 Other Agency Involvement

Due to the interest and inherent involvement of other parties in the project, a "Coordination Panel" was designated to establish design guidelines and to determine and review environmental concerns. The Panel met periodically throughout the project to accomplish these tasks. The Panel was comprised of representatives from the following agencies:

- City of Albuquerque (City)
- Middle Rio Grande Conservancy District (MRGCD)
- Bureau of Reclamation (Bureau)
- County of Bernalillo (County)
- Albuquerque Metropolitan Flood Control Authority (AMAFCA)

1.5 Description of Report Volumes

The report is comprised of three volumes. Volume I presents a summary of the hydraulic analysis, water quality testing and environmental mitigation measures including text, tables and figures. The text presents an overview of project objectives, analyses and results. Associated figures and tables summarize hydraulic evaluations and water quality results. Computer printouts, and other technical calculations which support the engineering analysis are presented as appendices in a separately bound Volume II. The detailed Environmental Assessment is presented in a separately bound Volume III. The table of contents for Report Volumes II and III are listed on page iii through v of this report.

2.0 GENERAL INFORMATION

2.1 Project Area Description

The study reach for hydraulic analyses on this project was limited to the Alameda Drain from the AMAFCA North Diversion Channel at the north end, south to its confluence with Riverside Drain and continuing southerly along Riverside Drain to the AMAFCA South Diversion Channel. The analysis was confined to the drain reaches themselves and specifically identified inflows, both existing and planned, which partially serve the watershed area surrounding the drains. The Griegos Drain, Riverside Drain north of the confluence and the San Jose Drain hydraulic characteristics were not studied. Refer to Figure 1.1 for a map of the project limits.

2.2 History and Operation

The establishment of communities along the Rio Grande centered around an agrarian culture. Acequias (ditches) were built to convey the river water to crops. However, over time, the land in the Valley area became boggy and alkaline. In 1925, a community group formed the Middle Rio Grande Conservancy District to administrate flood control, drainage and irrigation operations. Jurisdiction for the MRGCD extends from Cochiti Pueblo on the north to near Socorro on the south. The MRGCD completed a series of drainage ditches in the early 1930's that began lowering the water table along the Rio Grande. This, combined with dry periods in the 1930's and 1940's, made it possible for much reclaimed swampland in the Valley area to be occupied. The drains reportedly lowered the water table over 5 feet in 70% of this area. In the 1950's, the Bureau of Reclamation was contracted by MRGCD to completely rehabilitate the system. As a result, the MRGCD became financially indebted to the Bureau and is still financially connected to this agency.

The Riverside Drain receives diverted water from the Rio Grande at the Angostura Diversion, north of Albuquerque. Typically this occurs between mid March and mid October. Riverside Drain parallels the river and provides the source of flow for diversions to the Albuquerque Main Canal. The remaining flows are divided at the Atrisco Diversion just north of Central Avenue, to augment the supply on the west side of the Rio Grande. A portion of the irrigation water remains in the Riverside Drain to continue downstream combining with flows from the Alameda Drain, and supplying another network of canals at the Barr Canal Diversion, just south of Bridge Blvd. The remaining flows in the Riverside Drain continue downstream, combining with flows at the San Jose Drain confluence. These flows continue south, under AMAFCA's South Diversion Channel via a siphon and eventually divert back into the Rio Grande within the boundaries of Isleta Pueblo, south of Isleta Lakes. The Riverside Drain not only conveys irrigation and return flows, but also conveys groundwater for the cases where it is above the drain invert and controls the water surface elevation of Tingley Lake and the Isleta Lakes. Riverside Drain continues to aid in lowering the water table in the east valley along the Rio Grande.

The Alameda Drain serves as an inland or "interior" drain and receives unused or "return" irrigation water, via wasteways. The Drain has historically received stormwater runoff as well. The Alameda Drain begins at AMAFCA's North Diversion Channel and parallels North 2nd Street south to Matthew Blvd., and continues westerly. The Griegos Drain, which functions similarly to the Alameda Drain, ties into the Alameda Drain just west of San Isidro St. The Alameda Drain continues in a southerly direction, receiving return tailwater from various wasteways, to its confluence with the Riverside Drain just south of Central Ave. The Alameda Drain at one time had also served to lower the water table in the Valley area. The groundwater is now well below the bottom of the drain and has not served this purpose in recent years. The occurrence of successive wet years could, however, potentially change this condition.

The San Jose Drain functions similarly to the Alameda and Griegos Drains, receiving return tailwater flows and conveying them to the Riverside Drain just north of the South Diversion Channel.

Since World War II, urbanization of the North Valley east of the Rio Grande has intensified and thus created greater problems for dealing with drainage and flood control in the area. Both the Alameda and Riverside Drains are situated such that they are potentially effective for conveying increased stormwater runoff from the eastern Rio Grande Valley to the Rio Grande.

2.3 Related Studies

Several studies have previously been performed which directly impact the design criteria, assumptions and results of the present analysis. Those reports and design projects which are pertinent to this study are identified in the Bibliography, Section 12.0, at the end of this report.

2.4 Procurement of Information

Due to the interest and interaction of the various agencies affected by this project, agency conferrals were conducted to assimilate relevant information. Discussions, studies, reports, master plans and as-built drawings were accumulated and are specifically referenced throughout this report and in the Bibliography, as appropriate. The agencies and specific personnel contacted in order to procure a complete data base of information are identified in Appendix A, Volume II.

2.5 Computer Model

Since the Albuquerque Master Drainage Study (AMDS) was conducted in 1981, hydraulic analyses of the Valley have been performed utilizing the Rainfall and Drainage Simulation Model (RADS). However, due to the loss of technical support from the author of RADS and the development of other hydraulic modeling software in recent years, alternate hydraulic models were evaluated for performance of this study. Selection of a substitute simulation model was predicated on the assumption that the program selected have an equal or greater level of sophistication as RADS relative to the mathematical theory used to calculate backwater, surcharging, free surface flow, pressure flow and reverse flow.

The U.S. Environmental Protection Agency's "Stormwater Management Model - Extended Transport Block" - (SWMM-EXTRAN) was selected as the appropriate hydraulic simulation model for this study. The most recent software, EXTRAN Version 4.04, was acquired from the University of Florida in February, 1989. A series of test data files were developed by Leedshill-Herkenhoff, Inc. (LH) and executed using SWMM to ensure that the model would correctly react to the dynamic changes of culvert headwater conditions, submerged orifice flow, free discharge weir flow and pump station interaction. Each of the existing major hydraulic structures (Paseo del Norte Diversion, Barr Canal Diversion and the South Diversion Channel Siphon) were also modeled separately and simulation results were verified against hand calculations.

The program disks acquired were compiled for use on a personal computer. LH recompiled the program for a VAX mini-computer to minimize running time. A copy of the preface from the SWMM-EXTRAN Version 4.04 User's Manual can be found in Appendix B, Volume II. The hydraulic modeling map used for the Alameda and Riverside Drains analysis is contained in Appendix D, Volume II, along with a description of the identification numbers assigned to channel features.

3.0 EXISTING CONDITIONS ANALYSES

3.1 Baseline Irrigation Flows

During the irrigation season, flow conveyed along the drains is comprised of irrigation return flows and sub-surface drainage flows (groundwater). These combined flows are hereinafter referred to as "baseline irrigation flows". The irrigation season generally occurs from mid-March through mid-October. Convective thunderstorms, which occur in the summer, generate the highest peak discharges for storm runoff. Consequently, peak storm runoff occurs coincident with the period when irrigation flows are being conveyed in the drains.

To accurately evaluate the capacity available in the drains to convey stormwater runoff, the baseline irrigation flow rates had to first be quantified. Any capacity exceeding that needed for conveyance of irrigation return flows and subsurface drainage flows was then considered available for conveyance of stormwater runoff.

MRGCD maintains a recording gage located on the Riverside Drain near the Albuquerque Country Club. Flow depth readings are regularly obtained from the gage by MRGCD. Although, recording gage measurements are not actually representative of baseline irrigation flow conditions because of the inability to account for what portion of the flow may include stormwater runoff. Therefore, the District Engineer at MRGCD was consulted to determine what baseline irrigation flow rates are required to satisfy system needs.

Following several discussions, the design basis for irrigation return flow and subsurface drainage flow rates estimated to be entering the drains was agreed upon between the City and MRGCD. A summary of the accumulative design flow rates is presented in Table 3.1. As shown in the Table, the baseline irrigation flow rate in the Riverside Drain at the South Diversion Channel is 115 cfs with and 185 cfs without the diversion at the Barr Canal. Analysis included in this study assumes that a 70 cfs diversion is occurring at the Barr Canal, which creates the worst condition for tailwater depths upstream. This results in a 115 cfs

TABLE 3.1
BASELINE IRRIGATION DESIGN FLOW RATES

ALAMEDA DRAIN

Wasteway I.D.	Approximate Location Where Flow Enters Drain	Estimated Flow Entering Drain (CFS)	Baseline Flow in Alameda Drain at the Respective Approx. Location (CFS)
1. Alameda Wasteway	Alameda Road	5	5
2. La Deramedera	El Pueblo Road	5	10
3. Chamisal Lateral	So. of Willow	15	25
4. Stotts Lateral	So. of Vineyard	5	30
5. Gallegos Acequia	So. of Vineyard	5	35
6. Alameda Lateral	Candelaria Road	5	40
7. Harwood Lateral	West of 4th Street	10	50
8. Griegos Acequia	West of 4th Street	10	60
9. Menaul Acequia	San Ysidro/Matthew	5	65
10. Griegos Drain	So. of Matthew	25	90
11. Albuquerque Acequia	So. of Matthew	10	100
12. Zearing Lateral	So. of I-40 & Zearing	5	105

RIVERSIDE DRAIN

Wasteway I.D.	Approximate Location Where Flow Enters Drain	Estimated Flow Entering Drain (CFS)	Baseline Flow in Riverside Drain at the Respective Approx. Location (CFS)
1. Feeder Canal	Candelaria Road	35	35
2. Pierce Lateral	No. of I-40	5	40
3. Anaya Wasteway	No. of I-40	5	45
4. Duranes Lateral	No. of Central Ave.	10	55
5. Alameda Drain	Confluence near New York/Central inter.	105	160
6. Barr Diversion	So. of Bridge Blvd.	0/-70 (out)	160/90
7. San Jose Drain (incl. San Jose Lateral)	So. of Rio Bravo Blvd.	25	185(a);115(b)

(a) Assuming no diversion to Barr Canal = 185 cfs.
(b) Assuming 70 cfs diversion to Barr Canal = 115 cfs.(Used for this Study)

baseline flow rate in the Riverside Drain downstream of the Barr Canal and a 185 cfs baseline flow rate upstream of the Barr Canal. Subsequent hydraulic analyses are based on reserving drain capacities for the baseline irrigation flows before the addition of any stormwater runoff.

3.2 Existing Conditions Hydrology

A hydrologic analysis was not performed for this study. Rather, the City agreed to provide all of the storm inflow rates to the extent possible, based on existing RADS and HYMO output files, as-built construction plans, drainage studies and MRGCD permits. The rainfall distribution utilized in the referenced RADS and HYMO analysis is based on that recommended in the City of Albuquerque, 1982 Design Procedure Manual (DPM), including updates through 1988. The City is currently considering a modification to design storm rainfall amounts and distributions based on recent analyses of long term weather records. If, in the future, a modified design rainfall is officially promulgated by the City, analysis would be required to determine impacts on the evaluation and results of this study.

Existing condition hydrology is based on those stormwater runoff contributions to the drains which existed on or before November, 1988. All inflows to the drains were based on either constant inflows, as in the case of baseline irrigation flows, or on inflow hydrographs which represent time dependent inflows which occur during a storm event. The 100-year storm hydrology used in the model was derived from three primary sources:

A. Uplands area hydrographs used in the latest RADS analysis were recently developed by the City's Planning Group for the October, 1987 report, "Flexibility, Impacts and Costs of Using the Alameda and Riverside Drains to Control Urban Stormwater Flows" prepared by Weiss-Hines for MRGCD. These hydrographs were imported into the SWMM model data file. They included the Osuna Road Storm Drain, Candelaria Road Storm Drain and the Menaul High School Detention Basin.

B. Storm drain flow rate data generated within RADS was extrapolated to describe hydrographs for existing storm drain systems which outfall to the drains. The majority of the storm drain systems are included in this category.

C. Some existing storm drain facilities have been incorporated into the drainage system subsequent to the latest RADS analysis. A majority of these were a result of recent improvements along north 2nd Street, through the Bernalillo County project BC-85-3. These storm drain inflows were incorporated by generating a triangular hydrograph based on the peak design flow shown on the construction plans. Peak design flow rates for other storm drain facilities not contained in the RADS data were obtained from either as-built drawings, drainage studies, MRGCD licensed flow rates, or calculations based on full pipe flow.

   The peak flow rate for the San Jose Drain was obtained from the drainage study entitled "Southeast Valley Drainage Management Plan, San Jose Drain and Vicinity" prepared by Wilson & Co. for AMAFCA (36).

Some storm drain facilities in the existing condition hydrology are restricted to temporarily reduced peak flow rates, based on agreements previously made between MRGCD and the City. These temporarily reduced peak flow rates will remain in effect until improvements are made to the drains which will allow sufficient conveyance capacity in the drains to accept increased flow rates. Specifically, these included the following:

A. Montano Road Storm Drain (East): The allowable discharge from the Montano Road storm drain system into the Alameda Drain is limited to 25 cfs. Although the ultimate design discharge for the storm drain system is 139 cfs, the City has throttled the discharge down to 23 cfs.

B. Griegos Road Storm Drain (East): An interim discharge from Griegos Road storm drain (east) into the Alameda Drain of 22 cfs is permitted, although the peak flow rate generated for existing conditions is only 6 cfs. The storm drain is designed for an ultimate system capacity of 138 cfs.

C. Griegos Road Storm Drain (West): The permitted discharge for the Griegos Road storm drain (west) is limited to 35 cfs. The ultimate design capacity of the system is also 35 cfs.

In total, 56 individual inflow hydrographs were used to model the existing storm drain systems tied into the Alameda and Riverside Drains. Some of these hydrographs represent the combined flows from groups of small storm drains. The majority of the hydrographs used represent individual storm drain systems. For a complete listing of existing baseline irrigation inflows and 100-year storm inflows and locations, refer to Figure 3.2, Sheets 1 to 4 and Appendix C of Volume II.

3.3 Existing Conditions Drain Characteristics

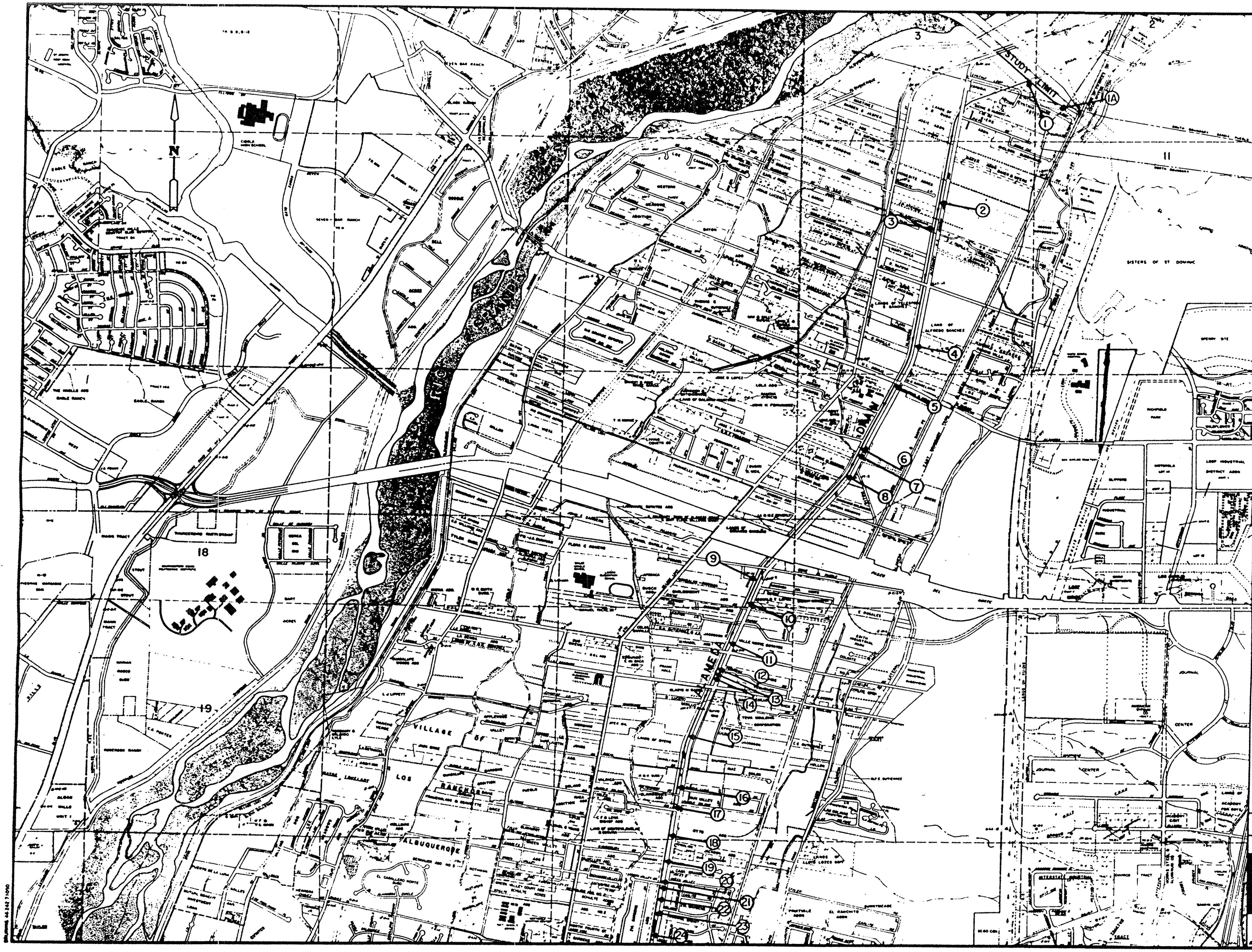
Because of the previous study of the Alameda and Riverside Drains in 1987, a separate field survey was not performed for this study. Instead, the City requested that the survey data contained in the latest RADS model be used as a basis for developing the SWMM model. Field reconnaissance trips were performed to verify culvert crossings, known storm drain inlets and drain depths and cross sections at several selected locations. Actual elevations contained in the original RADS file were not field verified in this study. Right-of-way and topographic mapping, as well as the original 1930 plan and profile drawings were researched. A series of photographs and a videotape of the drains were taken in September, 1988 to record the existing conditions.

A summary of the existing conditions drain characteristics is shown in Table 3.3. Detailed descriptions of existing hydraulic structures, field

TABLE 3.3

EXISTING CONDITIONS OF DRAIN CHARACTERISTICS AND DESIGN ANALYSIS ASSUMPTIONS

<u>Alameda Drain</u>	<u>Typical Channel Characteristics</u>
Sideslope	1.4:1.0 to 2.0 to 1.0 (H:V)
Bottom Width	6 Ft. to 17 Ft.
Depth	7 Ft. to 11 Ft.
Bottom Slope	0.001 Ft./Ft. to 0.0001 Ft./Ft.
Reach Length	11 Miles
Number of Culvert Crossings	65
Channel 'n' Value	0.035
<u>Riverside Drain</u>	
Sideslope	1.0:1.0 to 1.8:1.0 (H:V)
Bottom Width	20 Ft. to 33 Ft.
Depth	7 Ft. to 12 Ft.
Bottom Slope	0.002 Ft./Ft. to 0.0004 Ft./Ft.
Reach Length	6 miles
Number of Culvert Crossings	8
Channel 'n' Value	0.035
<u>Culvert Data</u>	
RCP 'n' Value	0.013
CMP 'n' Value	0.024
CMPA 'n' Value	0.030
CBC 'n' Value	0.015
Entrance & Exit Loss (RCP)	$K_{en} = 0.15$ $K_{ex} = 1.0$
Entrance & Exit Loss (CMP)	$K_{en} = 0.9$ $K_{ex} = 1.0$
Entrance & Exit Loss (CMPA)	$K_{en} = 0.9$ $K_{ex} = 1.0$
Entrance & Exit Loss (CBC)	$K_{en} = 0.15$ $K_{ex} = 1.0$



INLET ID NO.	PIPE SZ & TYP	100 YR. PEAK FLOW (CFS)	REFERENCE
1	24" RCP	10	PERMIT
1A	—	N/A	AMAFCA - NO. DIV CHANNEL
2	18" CMP	27	PERMIT
3	24" CMP	5	ALAMEDA WASTEWAY (IRRIG.)
4	24" CMP	5	PERMIT
5	24" CMP	5	PERMIT
6	24" CMP	7	PERMIT
7	36" CMP	15	BC-4035(200)
8	24" CMP	5	BC-4035(200)
9	18" CMP	5	LA DERAMEDERA WASTEWAY (IRRIGATION)
10	18" RCP	24	BC-85-3
11	18" RCP	6	BC-85-3
12	24" CMP	15	BC-85-3
13	18" RCP	3	BC-85-3
14	18" RCP	3	BC-85-3
15	18" RCP	21	BC-85-3
16	18" CMP	23	BC-85-3
17	18" RCP	2	BC-85-3
18	18" RCP	17	BC-85-3
19	18" RCP	15	BC-85-3
20	18" RCP	10	BC-85-3
21	18" RCP	21	BC-85-3
22	18" CMP	5	BC-85-3
23	24" RCP	3	FAC. MAPS
24	18" RCP	26	BC-85-3

NOTE:

DIRECTION OF INLET DISCHARGE INTO DRAIN.

GRAPHIC SCALES

0 500' 1000' 2000' 3000'

ALBUQUERQUE, NEW MEXICO

ALAMEDA/RIVERSIDE DRAIN ANALYSIS

FIGURE 3.2
EXISTING STORMWATER AND IRRIGATION INFLOWS TO DRAINS

DRAWN BY M.L.
DATE JAN. 1989
SCALE AS SHOWN

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ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE PHOENIX SAN DIEGO SAN FRANCISCO

SHEET 1 OF 4

INLET ID NO.	PIPE SZ & TYP	100 YR. PEAK FLOW (CFS)	REFERENCE
25	48" RCP	40	BC-85-3
26	24" RCP	20	BC-85-3
27	18" RCP	17	BC-85-3
28	18" RCP	20	BC-85-3
28A	—	15	CHAMISA LATERAL (IRRIG.)
29	18" RCP	6	BC-85-3
30	18" RCP	21	BC-85-3
31	24" RCP	5	STOTTS LATERAL
31A	—	5	GALLEGOS ACEQUIA (IRRIG.)
32	18" CMP	5	PERMIT
33	18" CMP	5	FAC. MAPS
34	36" RCP	14	FAC. MAPS
35	12" RCP	2	FAC. MAPS
36	60" RCP	23	PROJ. 2461
37	18" CMP	5	FAC. MAPS
38	12" CMP	2	FAC. MAPS
39	12" CMP	2	FAC. MAPS
40	12" CMP	5	FAC. MAPS
41	18" CMP	5	FAC. MAPS
42	12" CMP	3	FAC. MAPS
43	42" RCP	35	PROJ. 2461
44	60" RCP	6	PROJ. 2461
45	24" RCP	3	FAC. MAPS
46	24" RCP	4	FAC. MAPS
47	18" RCP	3	FAC. MAPS
48	18" RCP	4	FAC. MAPS
49	18" RCP	2	FAC. MAPS
50	18" RCP	6	FAC. MAPS
51	24" RCP	4	FAC. MAPS
52	18" RCP	2	FAC. MAPS
53	18" RCP	2	FAC. MAPS
54	24" RCP	2	FAC. MAPS
55	24" RCP	2	FAC. MAPS
56	24" RCP	2	FAC. MAPS
56A	—	5	ALAMEDA LATERAL (IRRIG.)
57	18" RCP	3	FAC. MAPS
58	48" RCP	84	FAC. MAPS
59	18" RCP	3	FAC. MAPS
60	48" RCP	13	FAC. MAPS
61	48" RCP	13	FAC. MAPS
62	48" RCP	13	FAC. MAPS
63	36" RCP	28	FAC. MAPS
63A	—	10	HARWOOD LATERAL (IRRIG.)
63B	—	10	GRIEGOS ACEQUIA (IRRIG.)
64	60" RCP	63	FAC. MAPS
64A	—	5	MENAUL ACEQUIA (IRRIG.)
65	36" RCP	7	FAC. MAPS
65A	—	25	GRIEGOS DRAIN (IRRIG.)
65B	—	10	ALBUQ. ACEQUIA (IRRIG.)
66	36" RCP	52	FAC. MAPS
67	30" RCP	69	FAC. MAPS
68	18" RCP	3	FAC. MAPS
69	24" RCP	5	FAC. MAPS
70	18" RCP	4	FAC. MAPS
71	18" RCP	5	FAC. MAPS
72	24" RCP	3	FAC. MAPS
73	15" RCP	2	FAC. MAPS
73A	—	5	ZEARING LATERAL (IRRIG.)
74	24" RCP	3	FAC. MAPS
75	15" RCP	3	FAC. MAPS

GRAPHIC SCALES

0 500' 1000' 2000' 3000'

ALBUQUERQUE, NEW MEXICO

ALAMEDA/RIVERSIDE DRAIN ANALYSIS

LH

FIGURE 3.2
EXISTING STORMWATER AND IRRIGATION INFLOWS TO DRAINS

DRAWN BY	DATE	SCALE
M.L.	JAN. 1989	AS SHOWN

LEEDSHILL-HERKENHOFF, INC.
ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE PHOENIX SAN DIEGO SAN FRANCISCO

SHEET 2 OF 4

INLET ID NO.	PIPE SZ & TYP	100 YR. PEAK FLOW (CFS)	REFERENCE
76	DROP INLET	3	FAC. MAPS
76A	—	55	RIVERSIDE DRAIN (FEEDER CANAL, PIERCE LATERAL, ANAYA WASTEWAY, DURANES LATERAL)(IRRIGATION)
76B	12" RCP	8	PERMIT
77	18" RCP	5	FAC. MAPS
78	24" RCP	5	FAC. MAPS
79	18" RCP	5	FAC. MAPS
80	18" RCP	5	FAC. MAPS
81	18" RCP	5	FAC. MAPS
81A	18" RCP	-70	BARR CANAL DIV. (IRRIG.)
82	48" RCP	45	FAC. MAPS

NOTE:

DIRECTION OF INLET DISCHARGE INTO DRAIN

GRAPHIC SCALES

0 500' 1000' 2000' 3000'

ALBUQUERQUE, NEW MEXICO

ALAMEDA/RIVERSIDE DRAIN ANALYSIS

FIGURE 3.2
EXISTING STORMWATER AND IRRIGATION INFLOWS TO DRAINS

DRAWN BY M.L.
DATE JAN. 1989
SCALE AS SHOWN

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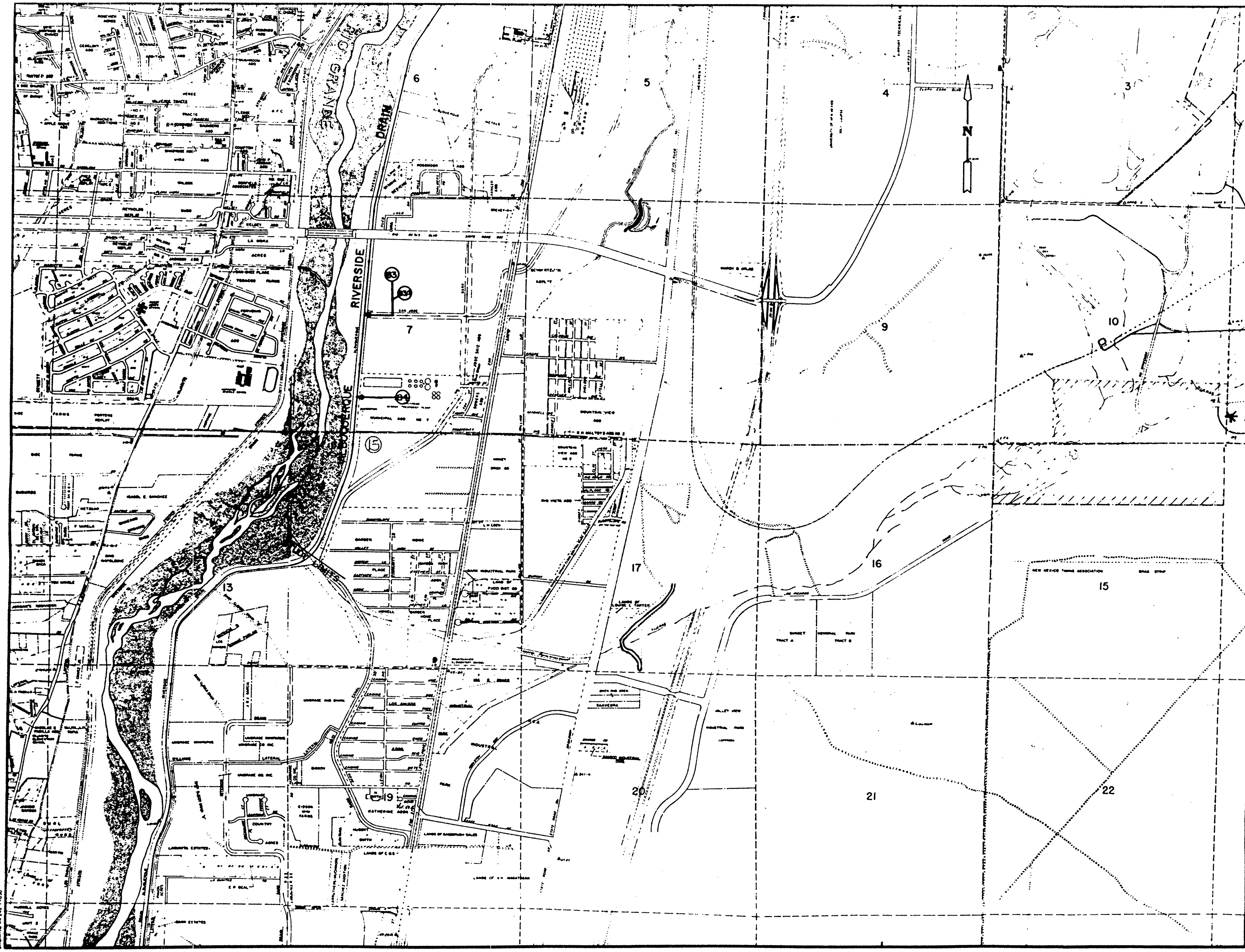
ALBUQUERQUE SANTA FE PHOENIX SAN DIEGO SAN FRANCISCO

SHEET 3 OF 4



INLET ID NO.	PIPE SZ & TYP	100 YR. PEAK FLOW (CFS)	REFERENCE
83	CHANNEL	304	SAN JOSE DRAIN (STORM)
83A	—	25	SAN JOSE DRAIN (IRRIG.)
84	30" RCP	30	FAC. MAPS

NOTE:

DIRECTION OF INLET DISCHARGE INTO DRAIN

GRAPHIC SCALES

0 300' 1000' 2000' 3000'

ALBUQUERQUE, NEW MEXICO

ALAMEDA/RIVERSIDE DRAIN ANALYSIS

FIGURE 3.2
EXISTING STORMWATER AND IRRIGATION INFLOWS TO DRAINS

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LEEDSHILL-HERKENHOFF, INC.
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surveys, channel cross-sections, and crossing structure sizes are presented in Appendix D, Volume II of this report. This Appendix also describes assumptions and interpretation of inflows utilized to develop the input data files.

Soils along the Alameda and Riverside Drains are alluvial and are of the Entisol Soil Order (UDA-SCS, 1975). Soils are comprised of sandy and silty clay loams with low to moderate shrink-swell potential.

Vegetation along channel sideslopes consists of grass, weeds, low bushes and small saplings (primarily Russian Olive, Elm and some Salt Cedar along Riverside Drain) (31).

Plan and profile sheets which show the existing physical characteristics of the drains, as well as the water surface profiles for existing conditions were developed as a result of the analyses. Refer to the plan and profile sheets, Figure 3.3, EX-1 to EX-4 and detail sheets Figure 10.2, D-1 and D-2 for existing hydraulic conditions. These sheets indicate the lowest top of bank as a light dashed line. This lowest bank shown is the lower of the two side banks, which would allow water to spill out of the confined channel section if overtopped.

3.4 EXISTING CONDITIONS HYDRAULIC ANALYSES

Water surface profiles were computed for the two separate existing conditions analyzed. These are described as:

A. Baseline irrigation flow conditions.

B. Baseline irrigation + 100-year storm flow conditions (based on existing points identified where stormwater is introduced to the drains).

Simulation time periods were performed for up to 60 hours.

The initial scope of work required analysis of 10-year storm flow conditions. However, a recent policy issued by the Bureau of Reclamation concerning discharge of stormwater into their agricultural drains, states (in part) "...Drain capacity should be sufficient to accommodate the 100-year storm event." As a result of this policy, the City requested that the analysis focus on the 100-year storm event. If a level of protection for a 100-year storm can be achieved, then 10-year event conveyance capacities will be satisfied as well. Therefore, a 10-year storm analysis was not performed.

3.4.1 Baseline Irrigation Flow Conditions

In general, the baseline flows alone occupy 20% to 50% of the available channel depth along the drains. Headwater depth for a majority of the culverts at crossings is controlled by tailwater conditions, and downstream open channel flow. All of the existing culverts are operating under partial flow conditions. Average flow depths and velocities characteristic of this condition are summarized in Table 3.4. The water surface profile for the baseline irrigation flow conditions is shown on plan and profile sheets Figure 3.3, EX-1 to EX-4.

3.4.2 Baseline Irrigation + 100-Year Storm Flow Conditions

With the exception of four reaches of the study length, the existing drain is of sufficient capacity to convey the 100-year storm event runoff presently being introduced to the drains in addition to baseline irrigation flows. The four exceptions are along the following reaches:

A. Along Alameda Drain, from Roehl Road south to Green Valley Drive, peak flows overtop the lowest channel bank by approximately 0.5 to 2.5 feet.

B. Along Alameda Drain, from Griegos Road downstream to Mescalero Rd., peak flows overtop the lowest channel bank by approximately 1.0 to 2.0 feet.

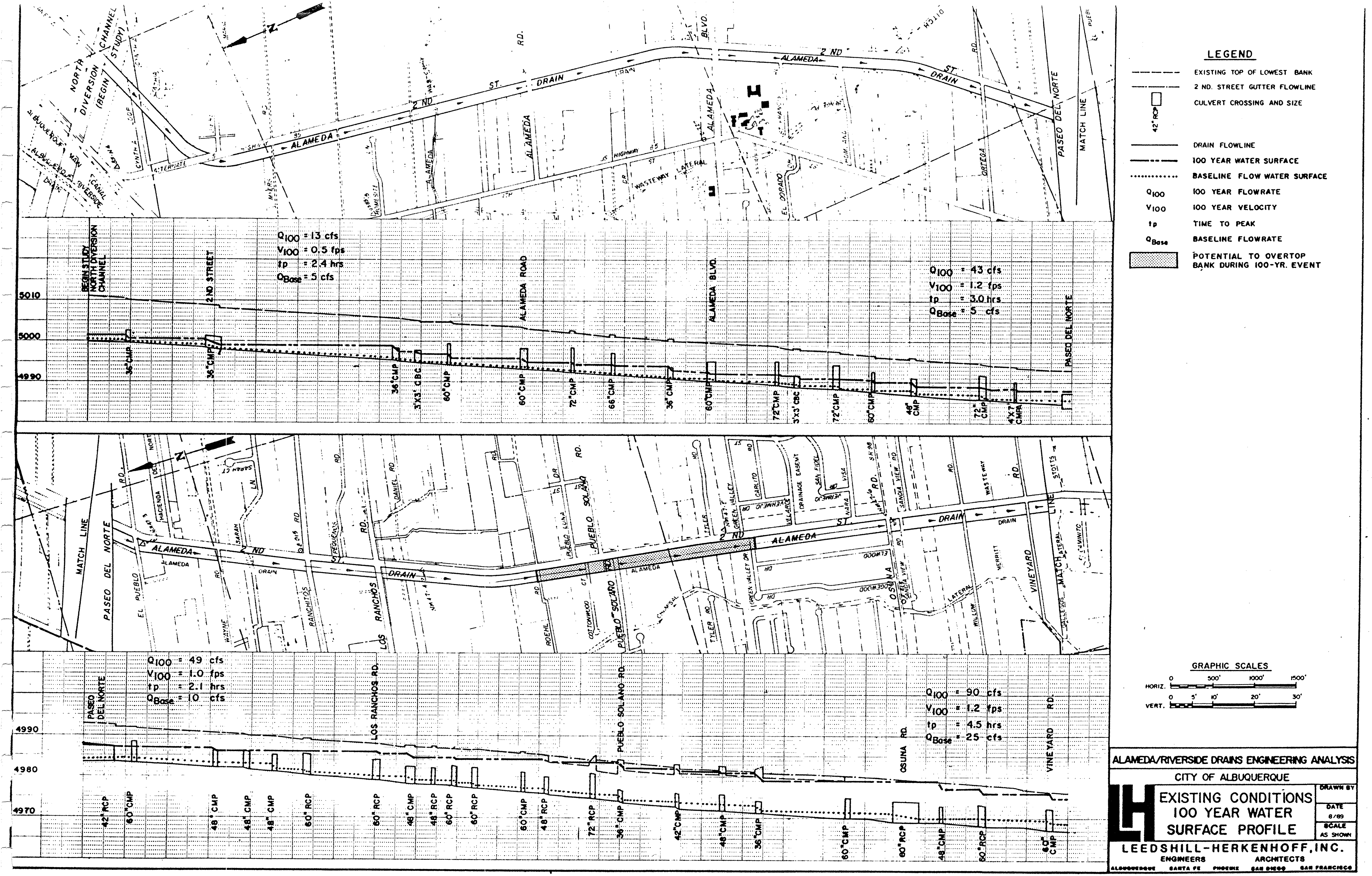
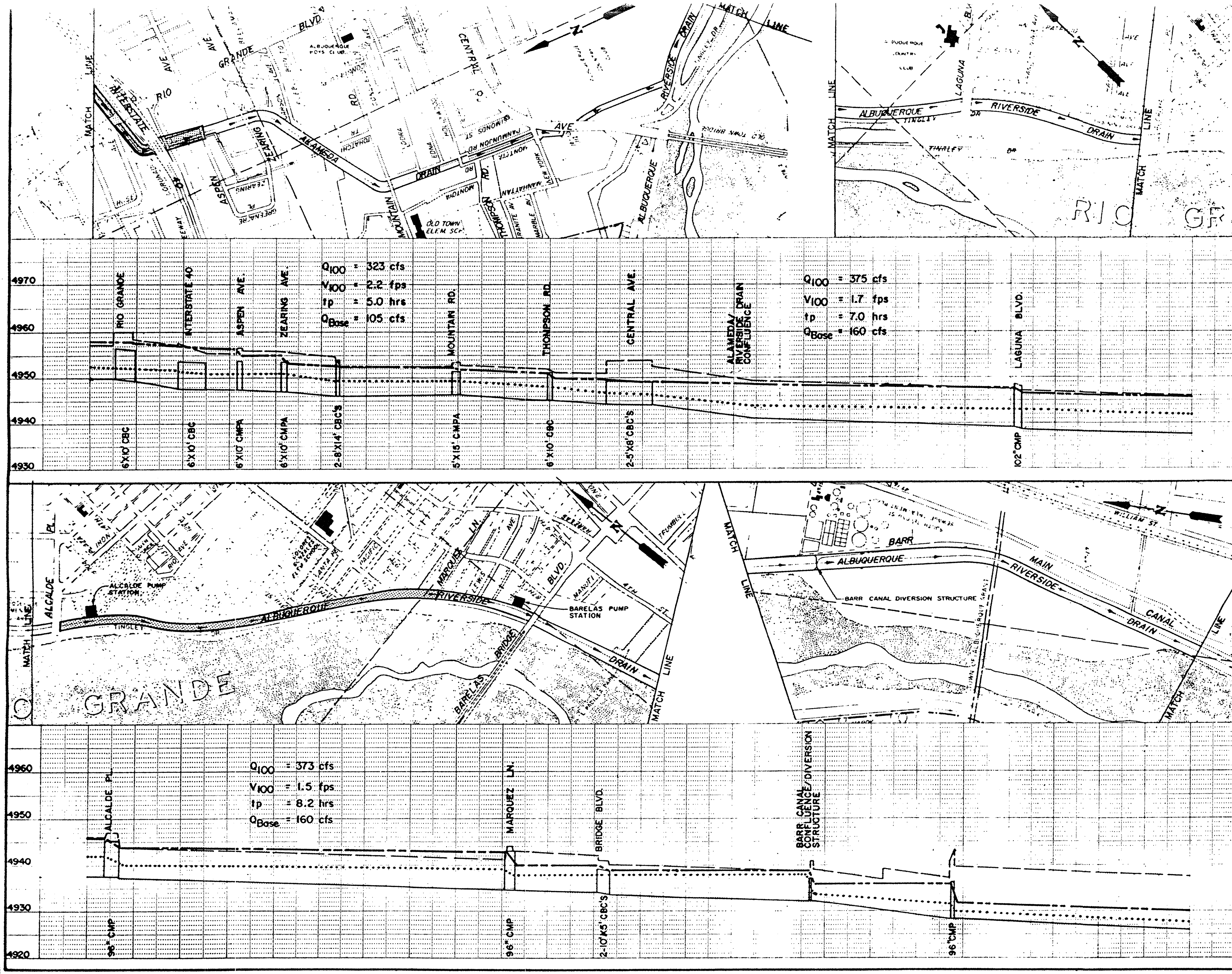


FIGURE 3.3 SHEET EX-1 OF EX-4

FIGURE 3.3 SHEET EX-2 OF EX-4



LEGEND

- EXISTING TOP OF LOWEST BANK
- 2 ND. STREET GUTTER FLOWLINE
- CULVERT CROSSING AND SIZE (42" RCP)
- DRAIN FLOWLINE
- 100 YEAR WATER SURFACE
- BASELINE FLOW WATER SURFACE
- Q_{100} 100 YEAR FLOWRATE
- V_{100} 100 YEAR VELOCITY
- tp TIME TO PEAK
- Q_{Base} BASELINE FLOWRATE
- POTENTIAL TO OVERTOP BANK DURING 100-YR. EVENT

GRAPHIC SCALES

HORIZ. 0 500' 1000' 1500'

VERT. 0 5' 10' 20' 30'

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS

CITY OF ALBUQUERQUE

EXISTING CONDITIONS 100 YEAR WATER SURFACE PROFILE

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DATE 8/89

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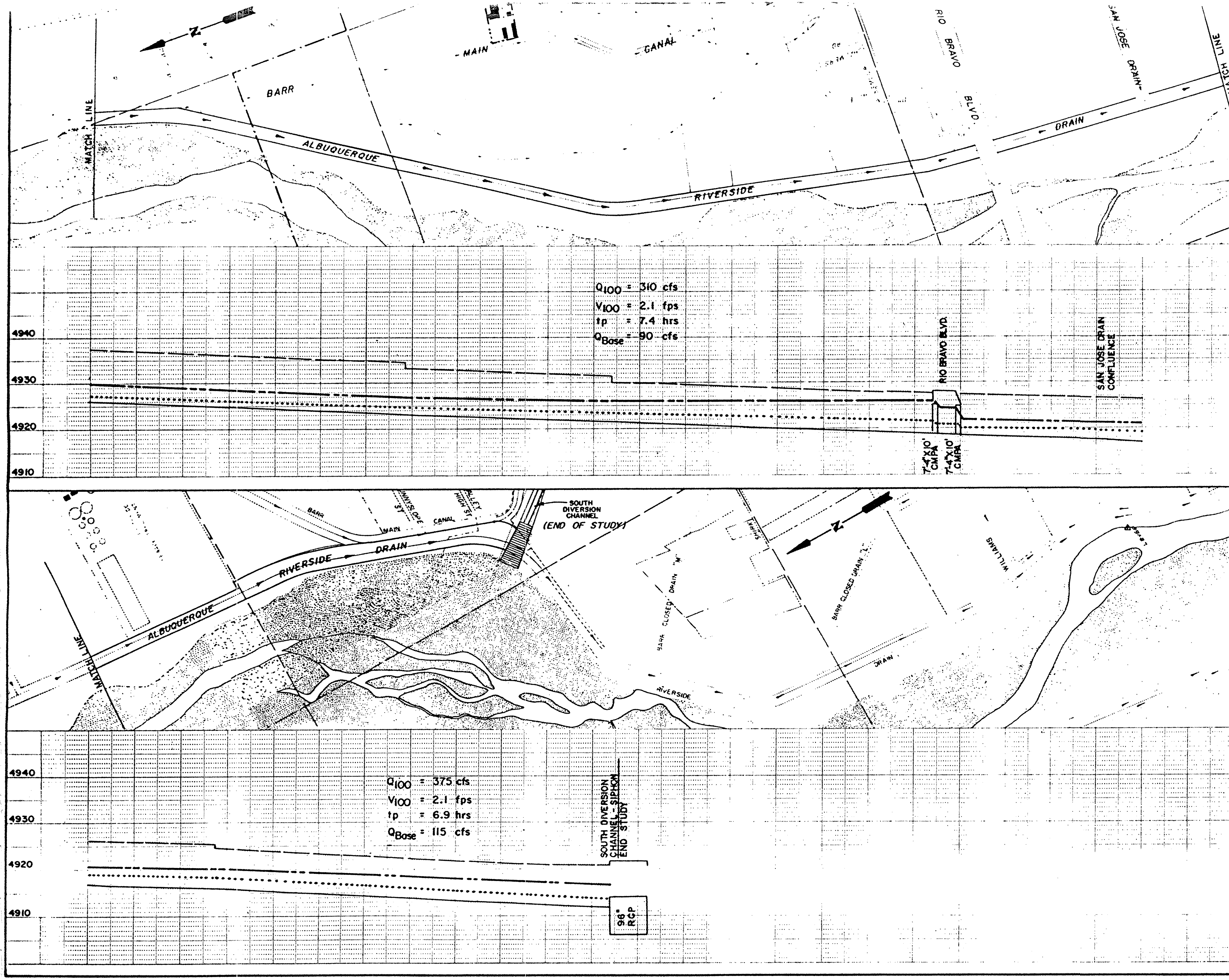
FIGURE 3.3 SHEET EX-3 OF EX-4



LEGEND

— — —	EXISTING TOP OF LOWEST BANK
—·—·—	2 ND. STREET GUTTER FLOWLINE
42" RCP ▯	CULVERT CROSSING AND SIZE
———	DRAIN FLOWLINE
—— - ——	100 YEAR WATER SURFACE
....	BASELINE FLOW WATER SURFACE
Q_{100}	100 YEAR FLOWRATE
V_{100}	100 YEAR VELOCITY
tp	TIME TO PEAK
Q_{Base}	BASELINE FLOWRATE
(shaded)	POTENTIAL TO OVERTOP BANK DURING 100-YR. EVENT

GRAPHIC SCALES

HORIZ. 0 500' 1000' 1500'

VERT. 0 5' 10' 20' 30'

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS

CITY OF ALBUQUERQUE

LH

EXISTING CONDITIONS
100 YEAR WATER
SURFACE PROFILE

DRAWN BY

DATE 6/89

SCALE AS SHOWN

LEEDSHILL-HERKENHOFF, INC.
ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE PHOENIX SAN DIEGO SAN FRANCISCO

FIGURE 3.3 SHEET EX-4 OF EX-4

C. Along Alameda Drain, from Lilac Avenue downstream to Aspen Ave., peak flows overtop the lowest channel bank by approximately 1.0 foot. Overtopping does not occur between Rio Grande Blvd., and I-40, however.

D. Along Riverside Drain, from Alcalde Place downstream to Marquez Ln., peak flows overtop the lowest channel bank by approximately 2.0 feet.

It should be noted, however, that along the majority of the drains where no overtopping occurs, minimal to no freeboard remains during the existing conditions 100-year storm. The exceptions to this are north of Paseo del Norte (where minor storm flows enter) and south of the Barr Canal diversion (where tailwater depths downstream of the diversion structure are not increased by the storm).

The results of the analysis of existing conditions indicate some interesting aspects of the operation of the drains during a storm event. First, due to the numerous culvert crossings which act as constrictions, the drain operates as a series of ponds behind each of the culverts, which helps to attenuate the peak flows. This ponding accounts for the higher water surface elevations at the upstream end of the culverts. Second, because the varied times to peak of the inflow hydrographs and the dynamics of the drain system, the peak flows in the drain are not coincident and therefore not directly additive. This can be seen by the fact that the peak flow times occurring along the drains vary from reach to reach. Third, because of the many culvert crossings, the total time required for the drains to completely convey the entire storm volumes and "drain down" to baseline flow conditions, is about 40 hours. Table 3.4 shows the average peak flow rates, velocities and drain down times at various locations along the drain for the 100-year event. Refer to plan and profile sheets Figure 3.3, EX-1 to EX-4 for 100-year water surface elevations.

TABLE 3.4

EXISTING CONDITIONS HYDRAULIC ANALYSIS RESULTS

BASELINE FLOWS

Location on Drain	Peak Flow (cfs)	Velocity (fps)	Depth (ft)
N. of Vineyard	25	1.0	2.5
N. of Candelaria	35	1.3	2.6
N. of Rio Grande	90	2.0	2.8
N. of Alcalde Pl.	160	1.2	4.5
N. of Bridge Blvd.	160	3.8	3.4
Siphon at S. Div. Ch.	115	2.9	1.6

100-YEAR + BASELINE FLOWS

Location on Drain	Peak Flow(CFS)	Peak Velocity(fps)	Maximum Depth(ft)	Drain Down Time(hrs)
N. of Vineyard	118	1.7	8.3	26
N. of Candelaria	146	1.2	5.6	30
N. of Rio Grande	309	2.5	8.4	32
N. of Alcalde Pl.	373	1.3	8.7	34
N. of Bridge Blvd.	374	2.2	5.6	36
Siphon at S. Div. Ch.	375	2.4	4.8	40

3.5 EXISTING CONDITIONS - LIST OF ASSUMPTIONS

3.5.1 Hydrologic Assumptions

A. Rainfall/Runoff Data: Hydrological Data as used in the latest RADS analysis (September 1987). 100-year, 24 hour storm distribution accumulative amounts are:

5 min. = 0.59 in.
30 min. = 1.62 in.
6 hr. = 2.4 in.
24 hr. = 2.8 in.

B. The area north of Paseo Del Norte is largely undeveloped. Storm inflows are limited to local 2nd St. runoff.

C. The area south of Paseo Del Norte includes the 2nd St. improvements (BC-85-3).

D. The proposed Edith Blvd. storm drain improvements and detention basins are <u>not</u> in place.

E. The Osuna Road storm drain improvements (east) are in place.

F. The Montano Road (east) storm drain includes limited discharge from Renaissance and AGP-Montbel detention basins and local street runoff, but no contribution from Edith Blvd. Flows are limited to the MRGCD permitted rate of 25 cfs.

G. Griegos Road storm drain (west) includes 4th St. tie-in, and flow rates are limited to the MRGCD permitted rate of 35 cfs.

H. Griegos Road storm drain (east) does not include the proposed Griegos-Commanche Road extension or detention basin flows. MRGCD permitted flows at this intersection are limited to 22 cfs, but only 6 cfs is used per Scanlon & Assoc. plans. (26)

I. Menaul High School detention basin does not include any flow contributions from Odelia Pond or street runoff from downstream of the basin.

J. A majority of the storm drain flows are obtained directly from RADS imported hydrographs, element and link outputs.

K. San Jose Drain flows include contributions north of Woodward Road at a peak flow rate of 304 cfs (obtained from Wilson & Co. Report (36).

L. Storm drain flows entering the Griegos Drain include contributions from Griegos Road, Headingly Ave. and Candelaria Road (obtained from RADS (31)).

3.5.2 Hydraulic Analysis Assumptions

A. Normal depth is achieved in channel immediately downstream from the 96" diameter siphon under the South Diversion Channel.

B. Existing radial gates (2) in Riverside Drain at Barr Canal Diversion Structure remain fully closed (worst case).

C. 70 cfs is diverted from Riverside Drain to the Barr Canal at the Barr Canal Diversion Structure.

D. Channel banks are extended (in the SWMM model) beyond the existing top of bank at the same sideslopes to keep flow in the confined channel cross-section when overtopping occurs.

E. Culvert entrance and exit headloss factors used are based on full-flow conditions.

4.0 FUTURE CONDITIONS ANALYSIS

4.1 Drain Improvements Design Criteria

The following criteria were established for the design of system improvements. These criteria were a result of inter-agency input through the various panel meetings held (refer to Appendix A for Agency representatives and contacts).

A. Freeboard. A minimum of 1.5 feet between the maximum water surface elevation (WSEL) and the critical channel spill elevation was used as freeboard. The critical channel spill elevation used was typically the lowest bank on the existing channel cross-section, which would allow a spill-over. The exception to this was along north 2nd Street where the west gutter flowline is below both channel bank elevations. If the WSEL rises above the gutterline, then spill-out could occur through storm drain inlets connected directly into the drain. Therefore, in this area the existing gutterline is considered the critical spill elevation from which 1.5′ of freeboard will be maintained. All future gutter flow lines along Second Street are assumed to be equal to or higher than existing elevations.

B. Drain Sideslopes. Because of the existing soil conditions along the drains, channel side slopes of 1.5 to 1.0 (H to V) were used for the Alameda Drain. For the Riverside Drain, sideslopes of 2.0 to 1.0 (H to V) were set due to the infiltration of groundwater causing a less stable cut slope.

C. Channel Bottom Width, Depth and Equipment Constraints. The operational constraints of MRGCD′s current maintenance equipment is not considered an improvement design constraint with respect to channel bottom widths, top widths and depths. Rather, if a proposed improvement exceeds the operational constraints of MRGCD′s maintenance equipment, an additional cost for new or rental equipment will be included in that alternate.

D. Maintenance/Service Roads. For maintenance and access purposes, a minimum 14 ft. wide road will be provided on the west channel bank on the Alameda Drain (where right-of-way is limited along 2nd St.). For improvements along the Alameda Drain (where adequate ROW exists) and Riverside Drain, two maintenance roads will be provided, each at a minimum width of 14 ft.

E. Channel Widening Within Right of Way. Improvements which include channel widening will contain the improved channel section within the existing right of way. The channel section will be widened as necessary hydraulically, with consideration for maintenance roads, slope criteria, and the potential expansion of Second Street.

F. Baseline Flow Conditions - Steady State. All improvements must allow the unimpeded conveyance of baseline flows, as outlined in Section 3.1. All analyses will consider the steady-state condition of the baseline flows, prior to improvement.

G. Berming of Channel Banks. Berming of channel banks will be considered only as a last resort, on an "as-needed" basis, where freeboard criteria is not satisfied.

H. Concrete Lining. Concrete lining is limited to the Alameda Drain due to groundwater infiltration on the Riverside Drain. It was assumed that lining will only be applied to channel sideslopes, not the channel bottom.

A summary of the design criteria is shown in Table 4.1.

TABLE 4.1

DRAIN IMPROVEMENTS DESIGN CRITERIA

ITEM	ALAMEDA DRAIN	RIVERSIDE DRAIN
FREEBOARD	1.5'	1.5'
SIDESLOPES (H:V)	1.5:1.0	2.0:1.0
BOTTOM WIDTH	as required	as required
DEPTH	as required	as required
MAINTENANCE ROADS	Two 14 FT. wide (One along 2nd St.)	Two 14 FT. Wide
CHANNEL WIDENING	Up to 100' ROW (60' along 2nd St.)	Up to 100' ROW

4.2 Future Conditions Hydrology

Future conditions hydrology considers the impacts of increased stormwater runoff entering into the existing drains as a result of projects now in the planning, design or construction phases. The future conditions hydrology also includes all of the remaining AMDS recommended projects which impact the drains.

Thus, future conditions analysis represents ultimate development of the North Valley area, east of the Rio Grande.

The following is a listing of those projects considered in the future conditions hydrology, along with the impacts to existing storm inflows. Refer to Appendix E, Volume II for a detailed listing of future conditions flow rates.

A. North of Paseo Del Norte. Future conditions hydrology for the area north of Paseo Del Norte is in accordance with the drainage report entitled "North Valley Drainage in the Proximity of Paseo Del Norte" (3). The future flows are based on fully developed conditions.

B. Edith Boulevard Widening/Detention Basins. This project is currently in the design phase and will provide 3 detention basins in the vicinity of Edith Blvd. from Osuna Road to Candelaria Road (6). The basins will detain flows within the upstream watersheds, and reduce developed 100-year storm peak flow rates into the Alameda Drain from Wayne Rd. to La Plata Rd. to approximately the developed 10-year amounts. Because the project will be adding storm flows from Edith Blvd. into the Montano Road Storm Drain System (east), the future flows into the Montano system will be increased. Therefore, the total inflow hydrograph used at Montano Road for future conditions is 139 cfs, as outlined in the drainage report (6). Additionally, the outfalls from the two southern detention basins will increase flow rates in their respective systems by about 10 cfs (north of Velaride Rd. and Headingly Ave. storm drains).

C. Comanche Road/Griegos Road Storm Drain Extension. The extension of Griegos Road storm collecting system (east) is currently under design. This project includes a detention basin located in the vicinity of Edith Blvd. and Griegos Road (26), which will detain upstream watershed flows. The outfall from the basin will tie into the existing Griegos Rd. storm drain system, and discharge at a peak flow rate of 22 cfs into the Alameda Drain. This peak flow rate is based on the interim permit agreement between the MRGCD and the COA.

D. Proposed North 2nd Street Widening. The long range major street plan of the MRGCOG calls for North 2nd Street to be classified as a principal arterial roadway, thus, ultimately requiring widening beyond the existing width. Because the Alameda Drain right-of-way is adjacent and parallel to 2nd St., any additional widening of 2nd St. will encroach upon the ROW available for widening Alameda Drain. The final ultimate width of 2nd St. has not been set, but based on the City's request, an encroachment width of 40 ft. into the MRGCD right-of-way was assumed. This leaves 60 ft. of total ROW remaining for the Alameda Drain along 2nd St. for future drain improvements.

E. Proposed Montano Road Pump Station. A storm water pump station is proposed to be located at the intersection of Rio Grande Blvd. and Montano Rd., which accepts flows from the Montano Rd. storm drain (east). The pump station is designed to pump 95 cfs to the Rio Grande, from the Montano storm drain system (33). A diversion structure within the storm drain is planned to divert 35 cfs into the Alameda Drain. The pump station and diversion were modeled based on the Montano Road storm drain hydrograph discussed in Item B. above. The Montano Pump Station is proposed in conjunction with the Montano Bridge. Because of the controversy associated with this project it is uncertain whether the pump station will actually be constructed. Therefore, rather than including the pump station in all future conditions analyses, it was only considered as a viable option for one improvement alternate.

F. Menaul Basin - Odelia Pond Connection. The AMDS recommends the connection of the Odelia Pond outfall into the existing Menaul Detention Basin outfall (AMDS System 129-07B). The additional flows from Odelia Pond were combined with the Menaul Basin flows to develop a future conditions peak flow of 115 cfs.

G. 4th Street Drainage Improvements. At the time of this study, only preliminary hydrology had been performed on the 4th Street Drainage Study by Leedshill-Herkenhoff, Inc (16). As a result, an inflow hydrograph at 4th Street of 100 cfs maximum peak flow was assumed.

H. Los Anayas Detention Basin. The proposed Los Anayas Detention Basin will discharge into the Rio Grande Blvd. storm drain, north of Indian School Road. The basin is being designed by City staff, and the final discharge rate had not been determined at the time of this study. A preliminary basin outflow rate of a constant 1 cfs was assumed.

I. New 24" storm drain in Carson Rd. (AMDS System 124-02D). The proposed 24" storm drain will discharge an additional 18 cfs into the Alameda Drain.

J. New 36" storm drain in Edna Ave. (AMDS System 124-04B). This proposed storm drain will discharge an additional 27 cfs into the Alameda Drain.

K. New 24" storm drain in Arbordale Ln. (AMDS system 141-01B). This proposed storm drain will discharge 9 cfs into the Griegos Drain, which outfalls into the Alameda Drain.

L. San Jose Drain Outfall to River. The "Southeast Valley Drainage Management Plan, San Jose Drain and Vicinity" report (37) recommends a future outfall for the San Jose Drain into the Rio Grande. Based on this plan, no stormwater contributions from the San Jose Drain are considered for future conditions hydrology.

M. The following AMDS system improvements will be constructed which are accounted for in the future conditions hydrology:

1.	111-01C	10.	124-02D
2.	111-04B	11.	124-04B
3.	111-05B	12.	128-03D
4.	112-01C	13.	129-02C
5.	116-01C	14.	129-07B
6.	117-01B	15.	133-01C
7.	117-02B	16.	140-01B
8.	119-01D	17.	141-01B
9.	121-02D		

4.3 Future Conditions Hydraulic Analysis

As discussed in the previous section, implementation of future conditions projects greatly affects the stormwater inflows to the drains. In general, stormwater inflows will be increased north of Paseo Del Norte as a result of increased development. From Paseo Del Norte to Montano Blvd., the Edith Blvd. Detention Basins will decrease inflows to the Alameda Drain. However, flows in the Montano storm drain system will be increased. Flows will be significantly increased from Mildred Avenue to the Riverside Drain confluence, primarily due to the Odelia Pond - Menaul Basin outfall connection and 4th Street improvements, as well as the other storm drain projects mentioned. The San Jose Drain outfall diversion greatly reduces downstream stormwater flows within the Riverside Drain.

As a result of these increased flows into the drains, significant overtopping of the banks occurs during a 100-year storm event. Spill-over occurs for the entire length from Montano Blvd. south to the Barr Diversion Structure under future development conditions.

4.4 Future Conditions Improvement Concepts

As the future conditions analysis revealed, improvements to the drains will be required to safely convey the anticipated future stormwater inflows with a minimum of 1.5 feet of freeboard. Improvements that were investigated consisted of various combinations of the following concepts:

A. On-line Detention. Enlargement of the existing channel by allowing existing culvert constrictions under roadway crossings to act as discharge controlled detention basins.

B. Off-line Detention. Location of detention basins in parallel with drains to attenuate peak flow in the drain and thus, optimize drain utilization.

C. Drain Improvements. Identification of channel constrictions and implementation of flow capacity improvements.

D. Bypass Storm Drains. Parallel storm drain facilities located at areas where the channel has limited flow conveyance capacity and limited ROW.

E. Alternate Outfall. Diversion of storm runoff away from the drains.

F. Stormwater Pumping Facilities. Collection and pumping of stormwater runoff directly to the Rio Grande.

Using these 6 major improvement concepts as a basis, 7 alternate improvement scenarios for the drains were developed. These are discussed in detail in the next chapter.

5.0 FUTURE CONDITIONS ALTERNATE IMPROVEMENTS ANALYSIS

5.1 Hydraulic Reaches

The sophistication of the modeling software utilized allows a determination of the dynamic and time-dependent operation of the drains during storm events. For example, peak flow rates and time to peaks vary at every location along the drains. Because this characteristic is exhibited, significant flow depth reductions associated with improvements to the drains are generally limited to channel areas within the proximity of the improvement. For instance, concrete lining a section of drain may drop the water surface level within that area but not necessarily a great distance upstream or downstream. As a result of this, and for the convenience of discussing various improvements along the drain, the drain length was divided into a series of reaches. These hydraulic reaches are defined based on similar hydraulic characteristics, limits of improvements and benefits.

Table 5.1 defines the 12 hydraulic reaches, and Figure 5.1 is a graphic representation of the same.

TABLE 5.1

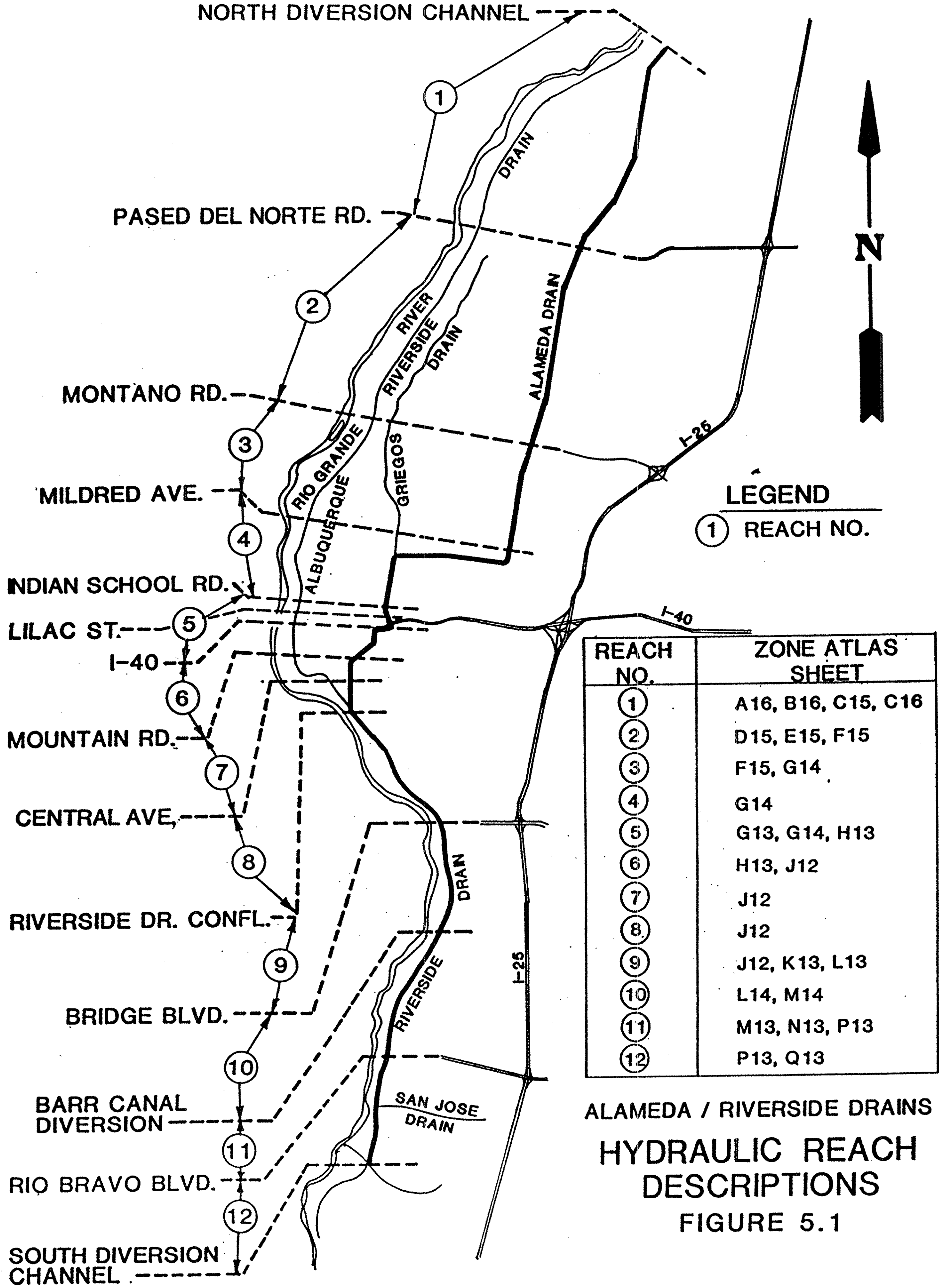
HYDRAULIC REACH DESCRIPTIONS

REACH LIMITS

Reach No.	Reach Begins	Reach Ends
1	North Diversion Channel	Paseo Del Norte Rd.
2	Paseo Del Norte Rd.	Montano Road
3	Montano Road	Mildred Avenue
4	Mildred Avenue	Indian School Road
5	Indian School Road	Interstate 40
6	Interstate 40	Mountain Road
7	Mountain Road	Central Avenue
8	Central Avenue	Alameda/Riverside Drain Confluence
9	Alameda/Riverside Drain Confluence	Bridge Blvd.
10	Bridge Blvd.	Barr Canal Diversion
11	Barr Canal Diversion	Rio Bravo Blvd.
12	Rio Bravo Blvd.	South Diversion Channel

5.2 Alternate Improvement Scenarios

As previously discussed in Section 4.3, six major improvement concepts were considered in developing improvement scenarios. From these, seven scenarios were evaluated, which consisted of various combinations of system improvements. Each alternate improvement scenario emphasized one or more of the major improvement concepts in order to provide a comprehensive list of improvement possibilities from which to select the most cost-effective and feasible improvement option. The various improvement combinations were analyzed in accordance with the design criteria listed in Section 4.1.



REACH NO.	ZONE ATLAS SHEET
1	A16, B16, C15, C16
2	D15, E15, F15
3	F15, G14
4	G14
5	G13, G14, H13
6	H13, J12
7	J12
8	J12
9	J12, K13, L13
10	L14, M14
11	M13, N13, P13
12	P13, Q13

ALAMEDA / RIVERSIDE DRAINS

HYDRAULIC REACH DESCRIPTIONS

FIGURE 5.1

5.2.1 Improvements Common to All Scenarios

The results of the hydraulic analysis indicated certain improvements were unquestionably cost effective. Because of the relatively low cost - high benefit ratio associated with these improvements, they were assumed common to all of the alternate improvement scenarios. These improvements are:

A. Paseo Del Norte Diversion Structure Orifice. The existing culvert under Paseo Del Norte will be restricted to allow the passage of primarily baseline irrigation flows. The majority of the storm flows will be diverted into the existing detention basin facilities paralleling Paseo del Norte. Channel widening and a gated structure installation will be necessary between the Alameda Road crossing and Paseo Del Norte along the Alameda Drain, so as to not exceed storage volume capacities of the basins. This improvement also minimizes storm flows continuing downstream in Alameda Drain.

B. Barr Canal Diversion Structure. The existing Barr Diversion includes an overflow weir in the Riverside Drain. To allow the passage of future storm flows with the minimum WSEL, the overflow weir will be lengthened as required. Hydraulic conditions are controlled such that diversion of irrigation water into the Barr Canal is relatively unaffected as compared to existing conditions.

C. San Jose Drain Outfall. As recommended in a previous study, the San Jose Drain will have a direct outfall to the Rio Grande (37). Because this requires the invert of the San Jose Drain to be above the Riverside Drain, culvert siphons are necessary in the Riverside Drain.

5.2.2 Description Summaries of Seven Scenarios

Each of the seven alternate improvement scenarios is discussed in detail below. The improvements described in Section 5.2.1 are common to each although not repeated below. For a more detailed description of alternate improvements, refer to Appendix F in Volume II. SWMM-EXTRAN input and output data files are available for review from the City but are not included in the Appendix for Scenarios 1, 3, 4, 5, 6 and 7.

A. Alternate Improvement Scenario No. 1. Refer to plan and profile sheets 1-1 to 1-4 (Appendix F, Volume II) and Figure 10.2 detail sheets D-1 and D-2 for detailed improvement requirements and the associated 100-year WSEL profile for this scenario. Under this scenario the Alameda Drain would be widened from Paseo Del Norte south to Mildred Avenue, and from Interstate 40 to Mountain Road. A 16 acre-ft. off-line detention basin would be required between Montano Road and Delamar Road. To bring the peak water surface elevation (WSEL) down below the 2nd Street gutter flowline, a parallel 4 ft. by 10 ft. concrete box culvert would be used between Griegos Road and Mildred Ave. Some culvert replacements would also be necessary between Griegos Rd. and Mildred Ave., as well as between I-40 and Mountain Rd. The Alameda Drain would be concrete lined between Mountain Rd. and the confluence of the Riverside Drain.

   On the Riverside Drain, widening would be required from the confluence to the Barr Diversion. Culvert replacement in these reaches would also be necessary. The Rio Bravo Blvd. culverts would also need replacement.

B. Alternate Improvement Scenario No. 2. Refer to Figure 10.1, plan and profile sheets 2-1 to 2-4 and Figure 10.2, detail sheets D-1 and D-2, for detailed improvements and the associated 100-year water surface profile for this scenario. Under this Scenario, the Alameda Drain would be widened from Paseo Del Norte to Mildred Ave., and from Interstate 40 to Mountain Road. A new storm drain collection system would be installed between Delamar Road and Mildred Ave., parallel to and within the adjacent 40 ft. drain ROW. All of the curb storm inlets along 2nd St. (between Delamar and Mildred, on both sides) would be connected to this storm drain, which will terminate at a small buried pump station just south of Mildred Ave. The existing connections between the 2nd Street storm inlets and the Alameda Drain would be eliminated. The pump station would collect these local street flows and discharge them into the Alameda Drain just south of Mildred Ave. There would be no direct connection between the Alameda Drain and 2nd St. gutterline within this section. Therefore, this system would eliminate the need to keep the WSEL in the drain below the 2nd St. gutterline, thereby increasing the allowable depth and capacity of the drain within this section. Three culvert replacements would be required between I-40 and Mountain Rd. The Alameda Drain would be concrete lined from Mountain Rd. to the Riverside Drain confluence.

   On the Riverside Drain, channel widening would not be required. Instead, a diversion structure would be installed just south of Alcalde Place. This structure would allow baseline irrigation flows to pass, but would divert the stormwater peak flows into the existing Alcalde Pump Station, to be discharged into the Rio Grande. The first priority for the discharge capacity of Alcalde Pump Station is reserved for conveying local storm runoff from the area. Yet the timing of peak flows in the Riverside Drain compared to peak flows entering Alcalde pump station is such that pumping capacity is available at the station for conveying flows from Riverside Drain to the Rio Grande. As a result, less stormwater would remain within the Riverside Drain. The Barr Diversion overflow weir length would be extended from the existing 16 feet to a length of 100 feet. The siphon culvert under the San Jose Drain outfall would also be reduced in size to a single 6' x 10' CBC as a result of the diversion to Alcalde Pump Station. The Alcalde Pump Station Diversion improvement could be utilized in any of the other scenarios, although it was only analyzed within this one.

The total time to evacuate stormwater runoff from the drains for the 100-year design storm, at the southern study limits (South Diversion Channel) would be approximately 24 to 36 hours. The SWMM-EXTRAN input data file and output summary are presented in Appendix F, Volume II.

C. <u>Alternate Improvements Scenario No. 3</u>. Refer to plan and profile sheets 3-1 to 3-4 (Appendix F, Volume II) and Figure 10.2 detail sheets D-1 and D-2 for detailed improvements and the associated 100-year water surface profile for this scenario. In Scenario No. 3, the Alameda Drain would be widened from Paseo Del Norte to Mildred Avenue, and from Interstate 40 to Mountain Road. The storm drain collection and pumping system as described in Scenario No. 2 would be used here as well, however, between Griegos Road and Mildred Avenue only. It is assumed that the currently proposed Montano Pump Station would be in place for Scenario No. 3. The cost associated to build the pump station is assumed to be the burden of the Montano Blvd. roadway improvement project. Only the cost to construct the flow splitting modification is assigned to this scenario. A diversion structure would be placed just upstream of Montano Road, in the Alameda Drain. This diversion structure would divert a portion of the peak drain flows into the proposed Montano Pump Station storm drain system. To maximize the effectiveness of the pump station, this improvement would eliminate splitting flows from the Montano Storm drain into the Alameda Drain, as the system was originally designed to do. Instead, when storm drain flows exceed the pump station capacity, the resulting hydraulic grade line would force flow to be diverted into the Alameda Drain at the diversion structure. The total improvement package would reduce the amount of storm water flows conveyed in the Alameda Drain because of the additional outfall directly to the Rio Grande via pumps from the Montano system. Only two culvert replacements would be required between Paseo del Norte and Mildred Ave. Culvert replacements would be necessary between I-40 and Mountain, as well as concrete lining of the drain between Mountain Rd. and the Riverside Drain confluence.

On the Riverside Drain, widening would be required from the confluence to the Barr Diversion. Culvert crossings would also be replaced within this section of the drain. The Rio Bravo Blvd. culverts would be replaced. The Alcalde pump station diversion structure could also be implemented in this scenario. The associated benefits in Scenario No. 2 would be applicable, and would result in approximately $150,000 savings in total improvements cost for Scenario No. 3.

D. <u>Alternate Improvement Scenario No. 4</u>. Refer to plan and profile sheets 4-1 to 4-4 (Appendix F, Volume II) and Figure 10.2 detail sheets D-1 and D-2 for detailed improvements and the associated 100-year water surface profile for this scenario. Under this Scenario, the Alameda Drain would be widened from Paseo Del Norte to Mildred Avenue, and the storm drain collection and pumping system between Delamar Rd. and Mildred Ave. would be constructed and used. Two culverts would be replaced in this reach. A new 350 cfs pump station would be located at the confluence of the Alameda Drain and Griegos Drain. This pump station would replace the existing Matthews Pump Station at the same location, which is no longer in service. The outfall for the new Matthew Pump Station would be the Rio Grande. Because of the large pumping capacity of the new station, minimal improvements would be required downstream. The Alameda Drain would not be widened downstream, nor would any culvert replacements be necessary.

On the Riverside Drain, only two culvert replacements would be required. The drain would not need to be widened and the Rio Bravo culverts would not need replacement.

E. Alternate Improvement Scenario No. 5. Refer to the plan and profile sheets 5-1 to 5-4 (Appendix F, Volume II) and Figure 10.2 detail sheets D-1 and D-2 for detailed improvements and the associated 100-year water surface profile for this scenario. Under Scenario No. 5, drain improvements would be identical to Scenario No. 4 up to the new Matthew Pump Station. The capacity of the new Matthew Pump Station would be 170 cfs, about one-half the capacity of that in Scenario No. 4. The Alameda Drain would require widening from Interstate 40 to Mountain Road. Three culverts would be replaced in this reach, but with smaller sizes than previous scenarios. Concrete lining of the drain would also be required from Mountain Rd. to the Riverside Drain confluence.

The Riverside Drain would be widened, but only from Alcalde Place to the Barr Diversion. Two culvert replacements would be needed in this section but the Rio Bravo culverts would not be replaced.

F. Alternate Improvement Scenario No. 6. Refer to the plan and profile sheets 6-1 to 6-4 (Appendix F, Volume II) and Figure 10.2 detail sheets D-1 and D-2 for detailed improvements and the associated 100-year water surface profile for this scenario. Under this Scenario, off-line detention would be the major improvement. This alternate would maximize the use of off-line detention basins by requiring a 150 acre-ft. basin between Montano Road and Griegos Road, and a 200 acre-ft. basin between Indian School Road and Lilac Avenue. Due to the benefits associated with the off-line detention ponding, no channel widening or culvert replacement would be necessary along the Alameda Drain.

On the Riverside Drain, only one culvert crossing, at Marquez Lane, would require replacement.

G. Alternate Improvement Scenario No. 7. Refer to plan and profile sheets 7-1 to 7-4 (Appendix F, Volume II) and Figure 10.2 detail sheets D-1 and D-2 for detailed improvements and the associated 100-year water surface profile for this scenario. A reduced capacity, off-line detention concept would be used in Scenario No. 7. The Alameda Drain would be widened from Paseo Del Norte to Griegos Drain, and from I-40 to Mountain Road. A 40 acre-ft. detention basin would be required between Montano Road and Griegos Road, and an 80 acre.-ft basin would be needed between Indian School Road and Lilac Avenue. Six culverts would be replaced between Griegos Rd. and Candelaria Rd. and three culverts between I-40 and Mountain Rd. The Alameda Drain would contain concrete lined sideslopes between Mountain Rd. and the Riverside Drain confluence.

On the Riverside Drain, three culvert replacements between the confluence and Bridge Blvd. would be the only improvements required.

5.3 Alternate Improvements Summary/Costs

A summary of the alternate scenarios improvements, on a per reach basis, is graphically shown in Figure 5.3. A summary of improvement costs per reach is presented in Table 5.3. The estimated construction costs shown are in 1989 dollars and include a 20% construction contingency factor, and an additional 20% for engineering, construction inspection and testing.

5.4 Future Conditions List of Assumptions

5.4.1 Hydrologic Assumptions

A. Fully developed flows north of Paseo Del Norte.

B. Edith Blvd. detention basins are constructed.

C. Detention basin at Edith Blvd. and Griegos Rd. is constructed and outflow limited to 22 cfs peak discharge.

SUMMARY OF ALTERNATE IMPROVEMENTS PER REACH

<table>
<tr><th>REACH NO.</th><th>SCENARIO NO. 1</th><th>SCENARIO NO. 2</th><th>SCENARIO NO. 3</th><th>SCENARIO NO. 4</th><th>SCENARIO NO. 5</th><th>SCENARIO NO. 6</th><th>SCENARIO NO. 7</th></tr>
<tr><td>NO. DIV. CHNL.
1</td><td colspan="7">PASEO DEL NORTE ORIFICE CHOKE, CHANNEL WIDENING AND ORIFICE WALLS</td></tr>
<tr><td rowspan="2">PASEO DEL. NO.
2</td><td colspan="5">CHANNEL WIDENING</td><td rowspan="2"></td><td rowspan="2">CHANNEL WIDENING</td></tr>
<tr><td></td><td></td><td>MONTANO P.S. DIVER.</td><td></td><td></td></tr>
<tr><td rowspan="3">MONTANO
3</td><td colspan="5">CHANNEL WIDENING</td><td rowspan="3">150 AC–FT BASIN</td><td rowspan="3">CHANNEL WIDENING
40 AC–FT BASIN
6 CULVERT REPLM'T.</td></tr>
<tr><td rowspan="2">5 CULVERT REPLM'T.
16 AC–FT DET. BASIN
4x10 CBC BY–PASS</td><td>5 CULVERT REPLM'T.</td><td>2 CULVERT REPLM'T.</td><td colspan="2">5 CULVERT REPLACEMENTS</td></tr>
<tr><td colspan="4">STORM DRAIN COLLECTION SYSTEM, CHANNEL WIDENING, & RET. WALLS</td></tr>
<tr><td rowspan="2">MILDRED
4</td><td>CHANNEL WIDENING</td><td colspan="2">MINOR BERMING</td><td rowspan="2">350 CFS
PUMP STATION</td><td rowspan="2">170 CFS
PUMP STATION</td><td rowspan="2"></td><td rowspan="2">CHANNEL WIDENING
1 CULVERT REPLM'T.</td></tr>
<tr><td colspan="3">1 CULVERT REPLACEMENT</td></tr>
<tr><td>INDIAN SCH.
5</td><td></td><td>MINOR BERM AND HEADWALLS</td><td></td><td></td><td></td><td>200 AC–FT BASIN</td><td>80 AC–FT BASIN</td></tr>
<tr><td>I–40
6</td><td colspan="3">CHANNEL WIDENING, 3 CULVERT REPLACEMENTS</td><td></td><td>CHANNEL WIDENING
4 CULVERT REPLM'T.</td><td></td><td>CHANNEL WIDENING
4 CULVERT REPLM'T.</td></tr>
<tr><td>MOUNTAIN
7</td><td colspan="3">1 CULVERT REPLACEMENT, CONCRETE LINE DRAIN</td><td></td><td>CONCRETE LINE DRAIN</td><td></td><td>CONCRETE LINE DRAIN</td></tr>
<tr><td>CENTRAL
8</td><td colspan="3">CONCRETE LINE DRAIN</td><td></td><td>CONCRETE LINE DRAIN</td><td></td><td>CONCRETE LINE DRAIN</td></tr>
<tr><td rowspan="2">RIVERSIDE DR.
9</td><td>TOTAL REACH WIDEN</td><td>ALCALDE P.S. DIVERSION</td><td>PARTIAL REACH WIDEN</td><td rowspan="2">2 CULVERT REPLM'T.</td><td rowspan="2">PARTIAL REACH WIDEN
3 CULVERT REPLM'T.</td><td rowspan="2">1 CULVERT REPLM'T.</td><td rowspan="2">3 CULVERT REPLM'T.</td></tr>
<tr><td colspan="3">3 CULVERT REPLACEMENTS</td></tr>
<tr><td rowspan="2">BRIDGE
10</td><td>CHANNEL WIDENING</td><td></td><td>CHANNEL WIDENING</td><td></td><td>CHANNEL WIDENING</td><td></td><td></td></tr>
<tr><td colspan="7">LENGTHEN BARR DIVERSION WEIR STRUCTURE</td></tr>
<tr><td rowspan="2">BARR CANAL
11
DIVERSION</td><td colspan="7">REMOVE 96" CMP CULVERT DOWNSTREAM OF BARR DIVERSION</td></tr>
<tr><td>RIO BRAVO CULVERT REPLACEMENT</td><td></td><td>RIO BRAVO CULVERT REPLACEMENT</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">RIO BRAVO SO.
12
DIVER.CHANNEL</td><td colspan="7">SAN JOSE DRAIN DIVERSION STRUCTURE</td></tr>
<tr><td>2–6x10 SIPHON</td><td>1–6x10 SIPHON</td><td>2–6x10 SIPHON</td><td>1–6x10 SIPHON</td><td>2–6x10 SIPHON</td><td>1–6x10 SIPHON</td><td>2–6x10 SIPHON</td></tr>
</table>

TABLE 5.3
SYSTEM IMPROVEMENT ALTERNATES SCENARIOS - TOTAL IMPROVEMENT COSTS PER REACH

	SCENARIO NO.						
REACH NO.	1 - Channel Widening with off-line detention and 4x10 CBC by-pass	2 - Channel Widening with s.d. collection and Alcalde diversion	3 - Montano Pump Sta. with s.d. collection and channel widening	4 - Matthew Pump Sta. with max size pump and s.d. coll. system	5 - Matthew Pump Sta. with min size pump and s.d. coll. system	6 - Off-Line Detention with max size basins and minimum improvement	7 - Off-Line Detention with min size basins and medium improvement
1	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000
2	$730,000	$730,000	$810,000	$710,000	$710,000	$0	$730,000
3	$4,800,000	$4,000,000	$2,000,000	$4,000,000	$4,000,000	$10,800,000	$6,300,000
4	$710,000	$230,000	$110,000	$6,500,000	$4,500,000	$0	$710,000
5	$0	$96,000	$96,000	$0	$0	$11,000,000	$3,000,000
6	$510,000	$580,000	$580,000	$0	$520,000	$0	$980,000
7	$640,000	$640,000	$640,000	$0	$560,000	$0	$560,000
8	$380,000	$380,000	$380,000	$0	$380,000	$0	$380,000
9	$780,000	$540,000	$570,000	$330,000	$580,000	$93,000	$450,000
10	$190,000	$64,000	$190,000	$97,000	$190,000	$92,000	$91,000
11	$120,000	$10,000	$170,000	$10,000	$180,000	$0	$10,000
12	$1,300,000	$1,100,000	$1,300,000	$1,100,000	$1,300,000	$960,000	$1,300,000
TOTAL IMPROVEMENTS COST	$10,360,000	$8,570,000	$7,046,000	$12,947,000	$13,120,000	$23,145,000	$14,711,000
USE	$10,400,000	$8,600,000	$7,100,000	$13,000,000	$13,100,000	$23,200,000	$14,700,000

NOTES:

1. Cost estimates do not include any necessary right-of-way costs.
2. Improvement costs per reach are presented to two significant figures only.
3. See appendix G, Volume II, for detailed breakdown of costs per reach.

D. Inflow hydrograph to Alameda Drain is limited to 139 cfs peak discharge at Montano Road (east).

E. Odelia Pond is connected to Menaul Detention Basin outfall and inflow hydrograph is limited to 115 cfs peak discharge.

F. 4th Street inflow hydrograph is limited to 100 cfs peak discharge.

G. Various storm drain projects on Carson Rd., Edna Ave., and Arbordale Ln. are constructed, as is Los Anayas detention basin.

H. San Jose Drain outfall to Rio grande is constructed.

5.4.2 Hydraulic Analysis Assumptions

A. Same assumptions as Section 3.5.2.

B. Majority of storm flows in Alameda Drain north of Paseo Del Norte are diverted into existing detention basins west of the drain at Paseo Del Norte.

C. Gated structures allow passage of flows through small diameter openings, and are not clogged.

D. Detention basins have pumped outfall into Alameda Drain and allow complete drain-down times of less than 96 hours.

E. Second Street encroachment on Alameda Drain allows only 60 ft. ROW for drain improvements (not including perched canals ROW).

F. Detention basin and pump station diversion structures are overflow weirs.

G. Concrete drain lining is on sideslopes only.

H. Flow rate in Riverside Drain upstream of confluence with Alameda Drain remains at a constant rate of 55 cfs during storm events.

6.0 OPERATION AND MAINTENANCE

6.1 Existing O & M of Drains

Historically, responsibility for operating and maintaining MRGCD's irrigation and drainage system has varied. Initially, the system was not adequately maintained. As a consequence, the deteriorated irrigation and drainage system was rehabilitated in the 1950's by the Bureau of Reclamation.

In 1951, MRGCD entered into an operation and maintenance contract with the Bureau of Reclamation. The contract was terminated in February, 1975, and MRGCD resumed O & M responsibilities with the exception of reserved works.

Due to increasing its use of the drains for stormwater conveyance, the City of Albuquerque agreed to maintain only the Alameda Drain within the City limits from 1983 to 1988. Responsibility was returned to MRGCD in 1988 as part of a joint agreement under which the City would pay $113,000 per year to MRGCD specifically for the MRGCD to perform drain maintenance.

Subhas Shah, District Engineer for MRGCD stated that the following maintenance practices are typical:

A. Annual inspections are performed in the fall to determine whether dredging is needed at specific locations along the drains. In general, Alameda Drain sufficiently slopes to be self-cleaning and seldom requires dredging.

B. Weed cutting and road maintenance are conducted annually along Alameda Drain.

C. Respective division managers schedule the maintenance of a particular system depending on needs and the availability of equipment, supplies and funding.

D. Channel side-slope stabilization is a problem along portions of Riverside Drain due to the higher water table in the area.

Operation and maintenance costs performed by the City on 5.6 miles of Alameda Drain from 1985 to 1988 were reported at an annual average of $5,787 per mile. This cost excluded dredging costs, estimated at $3,000 to $7,000 per mile every 5 to 10 years (1987 dollars). MRGCD maintained 5.26 miles of Alameda Drain between January 1985 and December 1986 at an average annual cost of $6,486 per mile.

According to the report prepared by Weiss-Hines for MRGCD (31) "Riverside Drain is estimated to be most costly to maintain (per reach), due to its larger cross section. However, proportionately, fewer reaches along Riverside Drain require annual maintenance to insure safe storm water conveyance".

In the past, operation and maintenance of the drains has been based on the needs and demands of its irrigation customers. However, with the increased usage of the drains for conveyance of stormwater, past practices are not necessarily sufficient. Potential problems associated with the established management, operation and maintenance program relative to using the drains for stormwater conveyances were identified by various agencies. Following is a summary of those concerns:

A. Historically, drain maintenance by MRGCD tended to follow a reactionary approach based on response to rainfall/runoff events and irrigation problems.

B. Non-maintenance of the channel results in reducing the infiltration potential due to silts sealing the drain bottom. The City prefers as much infiltration and groundwater recharge to occur as is possible.

C. Trees and dense vegetation which are allowed to remain within the channel effectively result in reducing conveyance capacities along the drains.

D. It is apparent, based on water quality test results, that certain discharges into the drains exist which detrimentally impact water quality, but there is no effort in force to eliminate or control these discharges.

E. MRGCD has an insufficient number of gaging stations to allow for determination of the quantity of irrigation return flows and groundwater drainage flows in the drain.

6.2 Future O & M Assumptions/Criteria

The operation and maintenance programs for each alternate improvement scenario differ. A list of general assumptions relating to the future operation and maintenance of the drains follows.

A. Drain bottom and detention basins are estimated to accumulate silt at a rate of about 2 inches per year. Dredging will be required every 3 years along both the Alameda and Riverside Drains.

B. Vegetative growth along the drain banks and detention basin slopes will be controlled by mowing at least once a year.

C. Silt, trash and debris will be removed from culverts, orifice walls and diversion structures periodically, so as not to affect their performance during a storm event.

D. Maintenance costs for dredging of the widened drain include $200,000 amortized for new equipment to be purchased initially and every 10 years.

E. Maintenance costs for new pump stations are based on Alcalde Pump Station maintenance costs prorated by capacity.

F. Maintenance costs for Alcalde Diversion include only the cost of energy used at Alcalde Pump Station to pump additional stormwater and to clean the wet well.

G. All maintenance costs include a 40% contingency, and are in 1989 dollars.

H. Life cycle costs based on 25 years and 7% compound interest are used to adjust annual maintenance costs within the project life to a present worth value.

6.3 O & M Costs for Alternate Improvement Scenarios

Table 6.3 summarizes the estimated maintenance costs for each alternate improvement scenario. Assumptions for developing the costs are as outlined in Section 6.2.

TABLE 6.3
SYSTEM IMPROVEMENT ALTERNATES - ESTIMATED ANNUAL MAINTENANCE COSTS

SCENARIO NO.

MAINTENANCE ITEMS	ESTIMATED ANNUAL MAINTENANCE UNIT COST	1 - Channel Widening with off-line detention and 4x10 CBC by-pass	2 - Channel Widening with s.d. collection and Alcalde diversion	3 - Montano Pump Sta. with s.d. collection and channel widening	4 - Matthews Pump Sta. with max size pump and s.d. coll. system	5 - Matthews Pump Sta. with min size pump and s.d. coll. system	6 - Off-Line Detention with max size basins and minimum improvement	7 - Off-Line Detention with min size basins and medium improvement
1. Existing drain section dredge, weed control, road maintenance Alameda Drain, N. Div. Ch. to I-40	$7,000 per mile	$6,300	$16,800	$16,800	$16,800	$16,800	$62,300	$6,300
2. Widened drain section dredge, weed control, road maintenance Alameda Drain, N. Div. Ch. to I-40	$11,200 per mile	$89,600	$72,800	$72,800	$72,800	$72,800	$0	$89,600
3. Existing drain section dredge, weed control, road maintenance Alameda Drain, I-40 to Confl.	$7,700 per mile	$0	$0	$0	$7,700	$0	$7,700	$0
4. Widened drain section dredge, weed control, road maintenance Alameda Drain, I-40 to Confl.	$17,500 per mile	$8,750	$8,750	$8,750	$0	$8,750	$0	$8,750
5. Existing drain section dredge, weed control, road maintenance Riverside Drain	$11,500 per mile	$46,000	$72,450	$58,650	$72,450	$56,350	$72,450	$72,450
6. Widened drain section dredge, weed control, road maintenance Riverside Drain	$14,000 per mile	$32,200	$0	$16,800	$0	$19,600	$0	$0
7. New dredging equipment for Alameda drain widening	$54,000 lump sum	$54,000	$54,000	$54,000	$54,000	$54,000	$0	$54,000
8. Culvert cleaning, trash removal, repairs	$14,000 lump sum	$14,000	$14,000	$14,000	$14,000	$14,000	$14,000	$14,000
9. Orifice Wall,Diversion Structure, clean, repair	$1,400 each	$9,800	$9,800	$8,400	$8,400	$8,400	$9,800	$11,200
10. Storm Drain Collection System, clean and repair.	$10,000 lump sum	$0	$10,000	$10,000	$10,000	$10,000	$0	$0
11. 4x10 By-Pass Culvert cleaning and repairs	$10,000 lump sum	$10,000	$0	$0	$0	$0	$0	$0

TABLE 6.3
SYSTEM IMPROVEMENT ALTERNATES - ESTIMATED ANNUAL MAINTENANCE COSTS

SCENARIO NO.

MAINTENANCE ITEMS	ESTIMATED ANNUAL MAINTENANCE UNIT COST	1 - Channel Widening with off-line detention and 4x10 CBC by-pass	2 - Channel Widening with s.d. collection and Alcalde diversion	3 - Montano Pump Sta. with s.d. collection and channel widening	4 - Matthews Pump Sta. with max size pump and s.d. coll. system	5 - Matthews Pump Sta. with min size pump and s.d. coll. system	6 - Off-Line Detention with max size basins and minimum improvement	7 - Off-Line Detention with min size basins and medium improvement
12. In-drain Pump Stations, energy use, maintenance	$25,000 % Alcalde	$0	$19,000	$6,000	$34,250	$15,750	$0	$0
13. Pump Station used with storm drain collection sys	$25,000 % Alcalde	$0	$2,500	$2,500	$2,500	$2,500	$0	$0
14. Pump Stations used with detention basin outfalls	$25,000 % Alcalde	$1,500	$0	$0	$0	$0	$3,000	$3,000
15. Detention Basins, clean dredge and maintain.	$550 per ac-ft	$8,800	$0	$0	$0	$0	$192,500	$66,000
16. Concrete Channel Lining repairs, cleaning	$3,500 per mile	$1,750	$1,750	$1,750	$0	$1,750	$0	$1,750
TOTAL ANNUAL MAINT. COSTS		$282,700	$281,850	$270,450	$292,900	$280,700	$361,750	$327,050
USE		$283,000	$282,000	$270,000	$293,000	$281,000	$362,000	$327,000
PRESENT WORTH VALUES OF MAINTENANCE COSTS USING 7 PERCENT INTEREST AND 25 YR LIFE		$3,300,000	$3,290,000	$3,150,000	$3,410,000	$3,270,000	$4,220,000	$3,810,000
USE		$3,300,000	$3,300,000	$3,200,000	$3,400,000	$3,300,000	$4,200,000	$3,800,000

ASSUMPTIONS USED IN FORMULATING MAINTENANCE COSTS:

1. All unit costs include 40 % contingency factor, which includes construction, engineering, survey, inspection and testing. All costs are in 1989 dollars.
2. Drain and detention basin costs assume silt accumulation rate of 2 inches per year, and dredging every 3 years.
3. Drain and basin costs assume bank mowing as weed control.
4. Pump station costs are based on Alcalde Pump Station maintenance record data, and are determined as a % of Alcalde pumping capacity.
5. New dredging equipment assumed for drain maintenance costs. Total equipment cost is $200,000 amortized annually with additional maintenance costs (gas, oil etc.), and equipment replacement in the 10th and 20th years, and amortized.
6. Culvert maintenance costs are assumed approximately $150 each/year.
7. Storm drain collection system and 4 x 10 by-pass culvert cleaning and repair costs assume 3 man-weeks per year.
8. Alcalde pump station diversion costs include only the cost of energy associated with pumping the additional diverted storm water, and an additional wet well cleaning.

7.0 WATER QUALITY

7.1 Introduction

A primary objective of the Alameda/Riverside Drains project was to characterize the water quality of storm flows presently occurring in the drains and the stormwater typical of the North Valley area, as well as to evaluate water quality changes and impacts associated with discharging additional storm flows into the drain system. The characterization was based on primary water quality data collected from storm flows along the drains and from storm sewer flows in the North Valley. In order to supplement the description of drain flow water quality for prospective regulatory and management needs, primary data on drain water quality included non-storm drain flows during irrigation season, and flows representing ground water during the non-irrigation season.

Applicability of federal and state discharge and environmental regulations were considered with regard to the Alameda/Riverside Drains project. According to New Mexico Water Quality Control Commission Regulations (Section 1-201), a Notice of Intent to Discharge is to be filed with the New Mexico Environmental Division. However, the intent of the current regulations is not to regulate municipal stormwater discharge. Therefore, the required discharge plan approval (Section 3-104) would likely be exempted (Section 3-105 G and H) under the regulations.

The National Pollutant Discharge Elimination System (NPDES) program regulates point source discharges of industrial process wastewater and municipal sewage. Under this program, requirements are emerging for subsequent regulation of stormwater discharges from municipal storm sewer systems. The Alameda/Riverside Drains project provides water quality data and strategies for local management of stormwater runoff and for addressing stormwater discharge regulations which may become applicable.

Utilization of the drain system to convey stormwater runoff, as it relates to rehabilitation, operation and maintenance interests of the Bureau of Reclamation, as well as subsequent applicability of the NPDES municipal stormwater amendments, requires implementation of the National Environmental Policy Act (NEPA). An environmental assessment was developed as a component of the Alameda/Riverside Drains project to clarify the issues and environmental effects of the proposed project.

7.2 Water Sampling Program

7.2.1 Sampling Sites

In order to characterize water currently conveyed in the drain system and stormwater flows generated in the North Valley, primary data relating to water quality was collected in the field. Water sample site locations were established along reaches of the Alameda and Riverside Drains, as delineated on Figure 7.2.1. Each sampling point was sited in a uniform straight channel section to validate the use of uniform flow equations to the greatest extent possible. Rationale for site location was as follows:

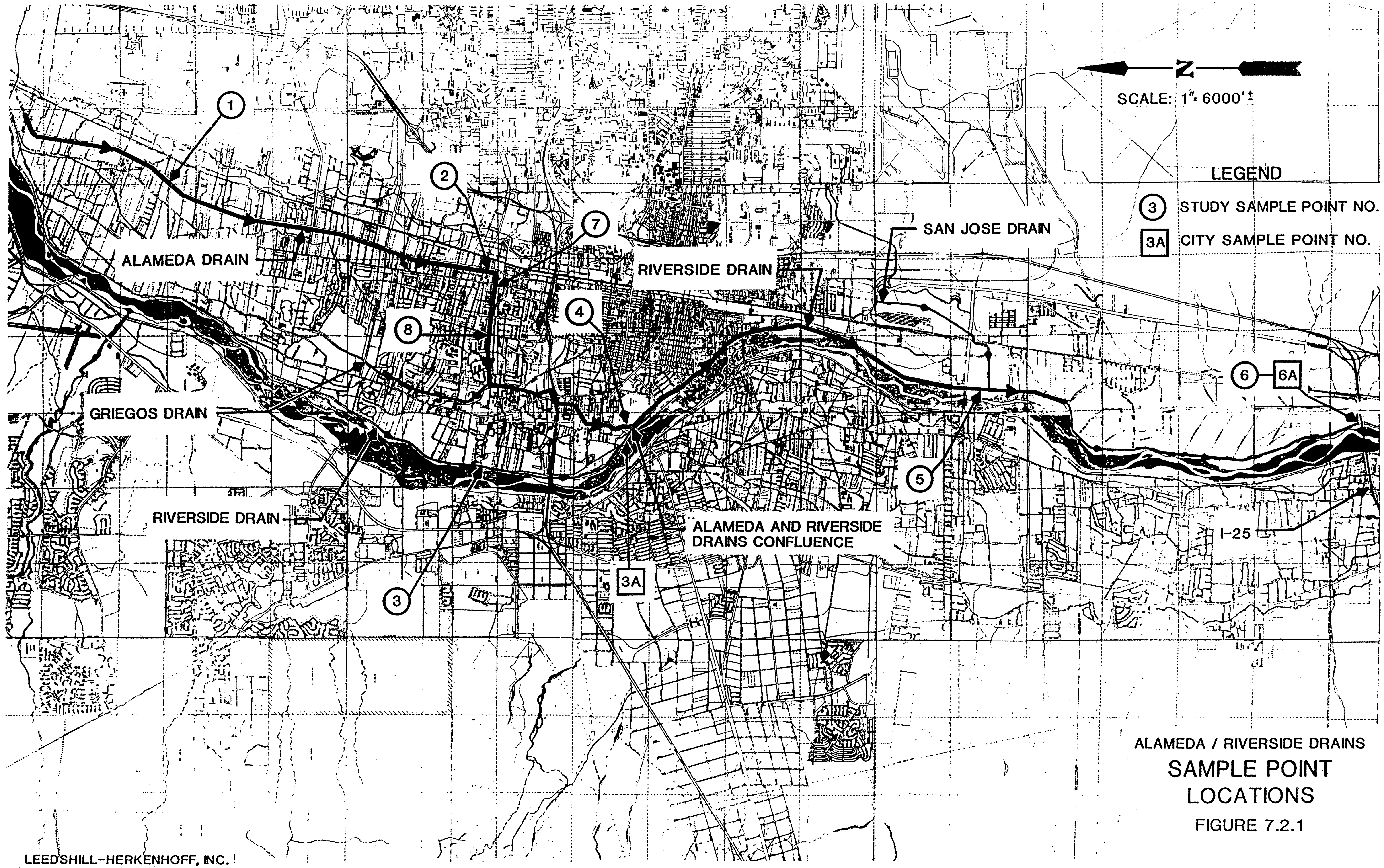
Sample Site #1 was located in the upper reach of the Alameda Drain north of any influence of known stormwater connections to the drain.

Sample Site #2 was located in the Alameda Drain upstream of the Menaul Detention Basin outlet and south of all 2nd Street storm sewer connections. Site #2 was intended to indicate the influence on water quality of the light industrial and commercial land use activity along 2nd Street.

Sample Site #3 was located on the Riverside Drain north of its confluence with the Alameda Drain.

Sample Site #4 was located in the Alameda Drain immediately above its confluence with the Riverside Drain, in order to characterize the effects of flows in the Alameda Drain, as well as contributing flows from the Menaul Detention Pond, Griegos Drain, and 4th and 12th Street storm drain systems.

Sample Site #5 was located in the Riverside Drain as a intermediate point downstream of the Albuquerque City limits.



ALAMEDA / RIVERSIDE DRAINS

SAMPLE POINT LOCATIONS

FIGURE 7.2.1

Sample Site #6 was located as the most downstream data point evaluated in order to characterize the effect of all inflows upstream of that point.

Sample Site #7 was located in the 4th Street storm drain for the purpose of characterizing stormwater quality from the generally urbanized light commercial area prior to discharge to the Alameda Drain.

Sample Site #8 was located in the 12th Street storm drain for the purpose of characterizing stormwater quality from medium density residential development prior to discharge to the Alameda Drain.

7.2.2 Sampling Station Configuration

Water sampling stations were established at each of the eight sampling sites. Each station consisted of a crest-stage gage and four single stage sediment samplers for the purpose of unmanned stormwater sample collection. The crest-stage gage consisted of a 5-foot (3-foot at storm sewer station #8) length of plastic pipe with a cap at each end and a graduated staff contained within the pipe. Figure 7.2.2 illustrates the construction of a typical crest-stage gage.

Each single-stage sediment sampler, based on a U.S. Geological Survey design, consisted of a one-liter bottle housed in a 4-inch plastic pipe, capped at both ends. Water intake and air exhaust tubes were provided. Figure 7.2.3 illustrates the single-stage sediment sampler design. In order to collect adequate stormwater samples for both inorganic and organic analysis, a modification of the sampler materials design was implemented. For collection of stormwater to be used for organic analyses, the design modification included a wide mouth glass bottle, a teflon bottle cap, and copper intake and exhaust tubing.

The configuration of a typical stormwater sample station, as installed at Sampling Sites 1 through 6 is illustrated in Figure 7.2.4. The crest-stage gages and single-stage sediment samplers were secured to steel fence posts driven into the channel bottom. Four samplers were installed in pairs at

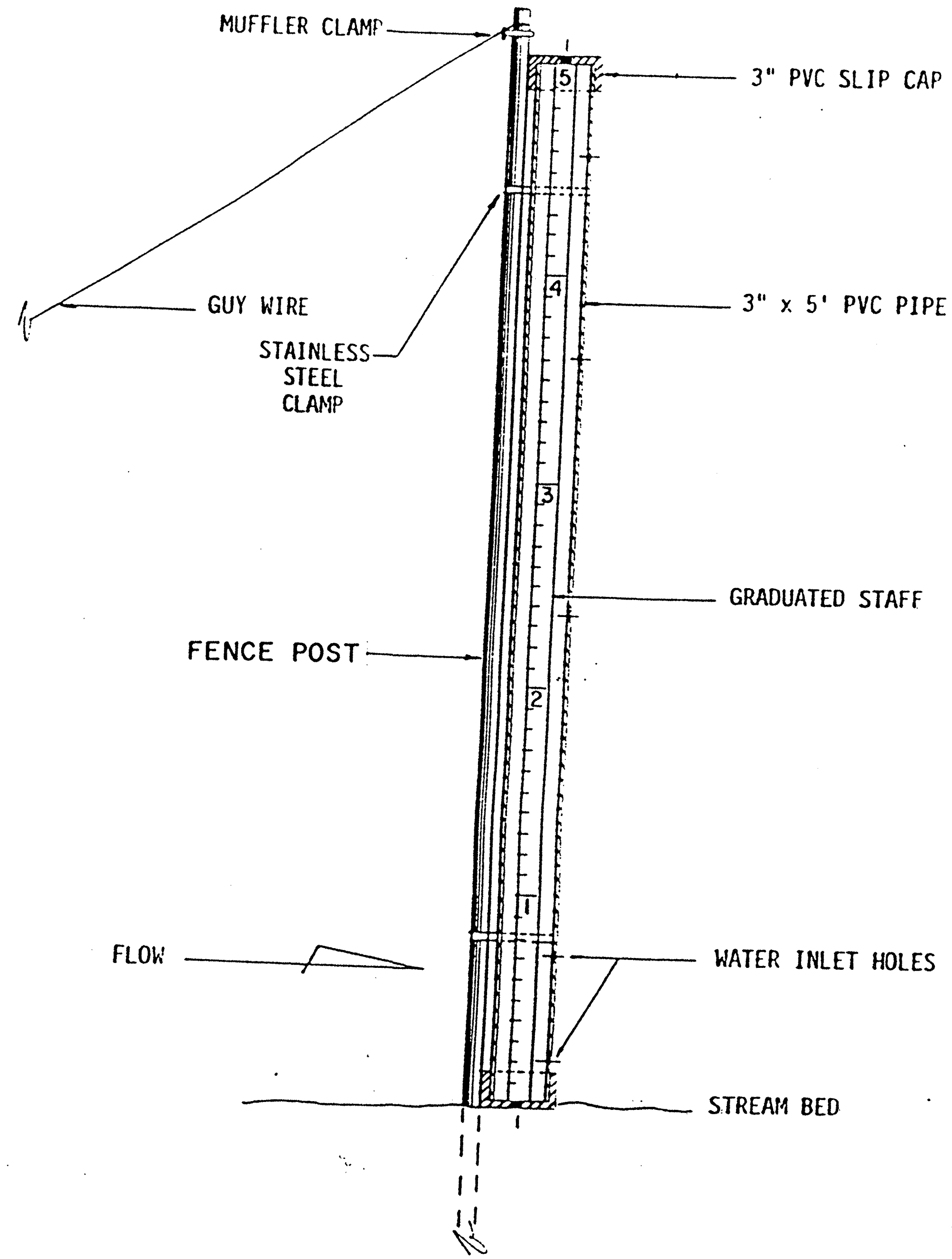


FIGURE 7.2.2

CREST-STAGE GAGE

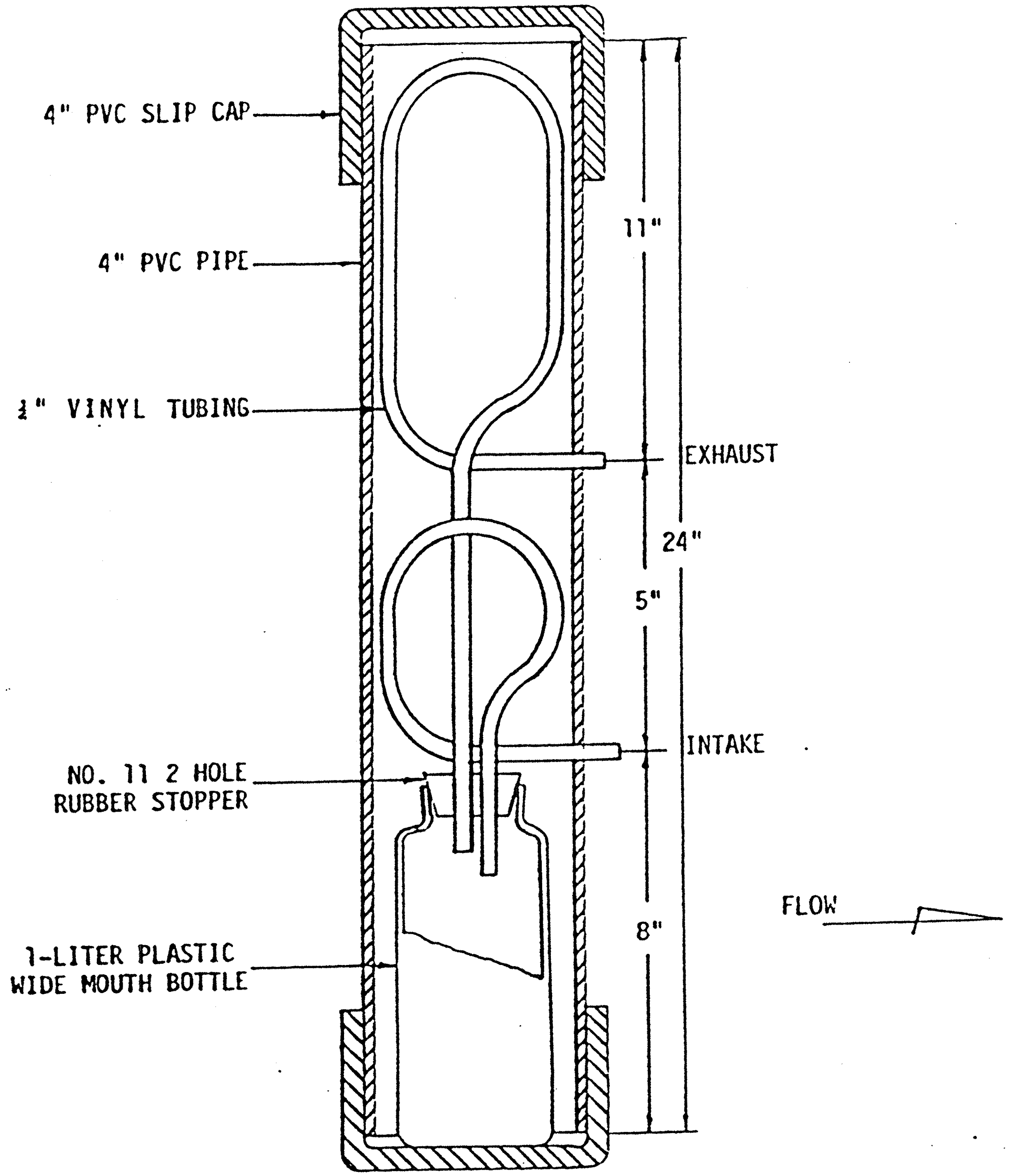


FIGURE 7.2.3

SINGLE-STAGE SEDIMENT SAMPLER

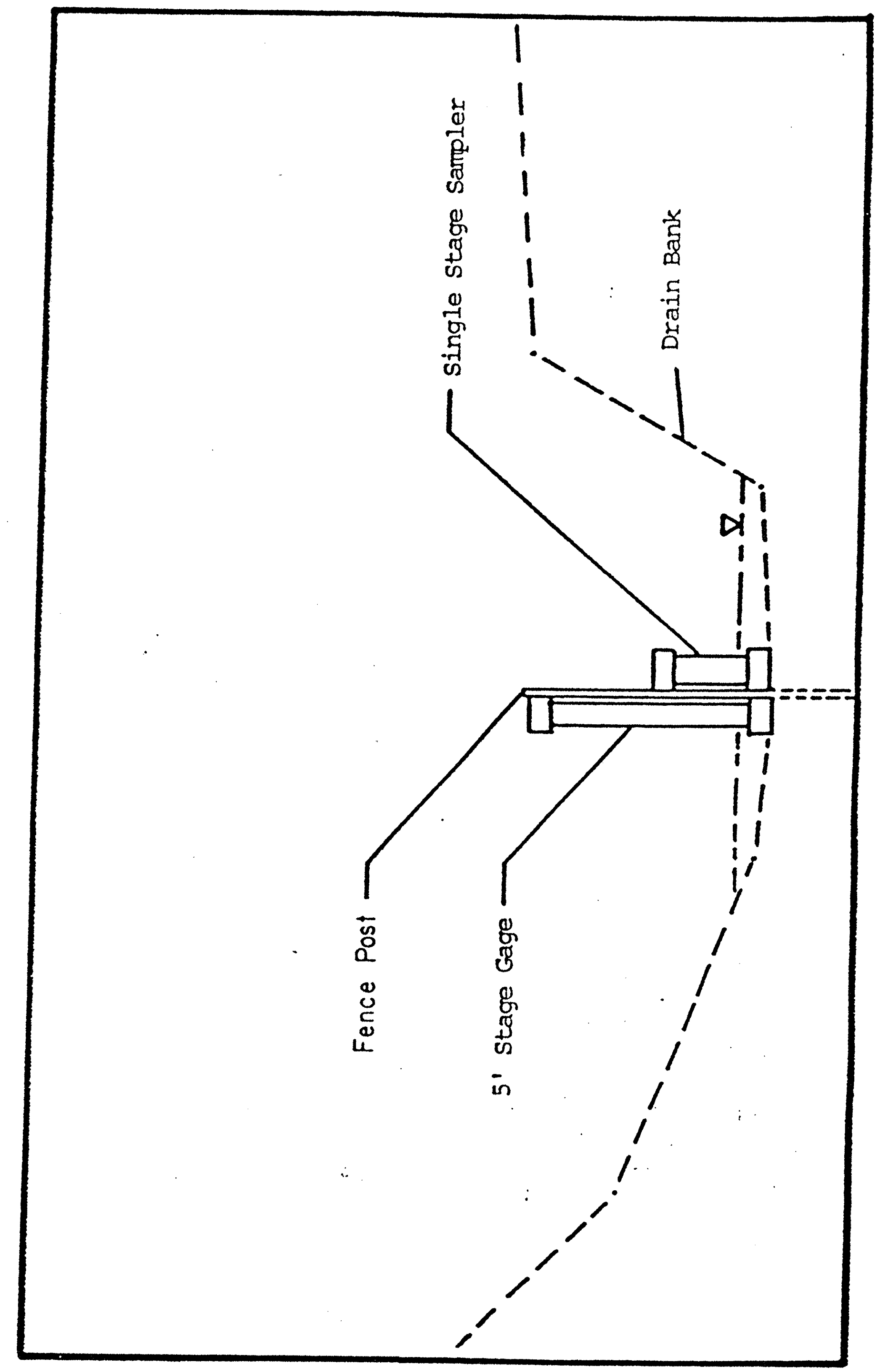


FIGURE 7.2.4
TYPICAL SAMPLING STATION CROSS-SECTION

each station to collect stormwater separately for inorganic and organic analyses at high and low flow stages. Sampler heighths were set such that a pair of single-stage sediment samplers would fill with an initial storm flow depth of about 3 inches above base flow (low stage), and the second pair of samplers would fill at an 8-inch rise in storm flow (high stage). Similar sample station configurations were installed at Sites 7 and 8 within 4th and 12th Street storm sewers. Low and high stage samplers were used in an attempt to detect a difference between "first flush" stormwater and runoff generated later in the storm event.

All sampling stations were visited by a technician at a frequency of at least once per week during the rainfall season in order to check condition of the equipment and to adjust the heighth of the samplers in anticipation of a storm flow stage use.

For the purpose of providing a record of rainfall within the study area during the field sampling period, rain gages were also installed at each sample site in the drain.

7.2.3 Sampling Procedures

Water sampling in the Alameda and Riverside Drains was conducted during the month of September, October and November, 1988. Water samples collected under the project scope are as follows:

a) Low and high water flows within the drains resulting from rainfall events during irrigation season. These waters included irrigation flow, ground water, direct precipitation, and storm sewer flow.

b) Low and high water flows generated by rainfall events in storm sewers.

c) Normal irrigation season flows when river water is diverted into the drain system, but not influenced by rainfall events. These waters include ground water contributions.

d) Normal flows during the non-irrigation season when no river water is diverted into the drain system. These waters are typically ground water.

Following known stormwater flow events during the sampling period, each sampling station was visited within 24 hours to retrieve samples which may have been collected in the unmanned samplers, to record flow stages and rainfall which occurred, to conduct field tests of conductivity, pH, temperature, and chlorine residual of samples collected, and to record relevant observations made at each site. Field information was recorded on forms as illustrated in EXHIBIT 7.2.3.1, Appendix H, Volume II.

Filled sample bottles were retrieved from each sampler, and placed in a cooler for delivery on that day to the lab. Clean sample bottles were reinstalled in the samplers for collection during a subsequent event.

Water representing irrigation season flow in the drain was collected at each sample site along the drain. Samples were collected by grab sampling method following at least one week from the previous storm event. At each site, one liter glass and plastic bottles were submerged about 4 inches below the water surface in order to allow collection of a representative suspended sediment and dissolved constituent concentration while excluding floating debris and bed load sediment. Samples collected were placed in a cooler for delivery on that day to the lab.

Similar grab sampling and recording operations were conducted for flowing waters during non-irrigation season.

During the sampling season, flow velocity measurements were taken at varying stages of flow in the drain (storm, irrigation, and non-irrigation flows). Field tests for water temperature and dissolved oxygen were conducted. Table 7.2.3 summarizes the number, type and location of samples obtained by this project.

TABLE 7.2.3
NUMBER AND DATES OF SAMPLES OBTAINED
*-INDICATES SAMPLES WHICH WERE ANALYZED BY GAS CHROMATOGRAPHY

DATES / STATION V	AUG 28	SEPT 2-3	SEPT 12	SEPT 14	SEPT 20	SEPT 22	SEPT 23	OCT 12-13	OCT 26-27	NOV 11	NOV 22	DEC 1	DEC 8	APR 4	APR 14	APR 18
1 HIGH		X														
1 LOW		X					X									
2 HIGH				X			X									
2 LOW			X	X	X											
3 HIGH																
3 LOW		X		X			X									
4 HIGH	X	X		X			X									
4 LOW	X	X	X	X	X											
5 HIGH			X													
5 LOW		X	X													
6 HIGH				X												
6 LOW		X	X													
7 HIGH			X	X			X			X*						
7 LOW			X	X			X	X			X					
8 HIGH			X	X		X				X						
8 LOW			X	X		X				X*						
1 GRAB					X			X	X							
2 GRAB					X			X	X							
3 GRAB					X			X	X	X*	X	X		X	X	X
4 GRAB					X			X	X					X	X	X
5 GRAB					X			X	X	X	X	X				
6 GRAB					X			X	X	X*	X	X*		X	X	X
N DIV CH													X*			
	2	7	9	10	8	2	6	7	6	6	4	3	1	3	3	3
TOTAL	80															

Subsequent to the field sampling period, it was determined that sampling site No. 3 had been located in the Overlap Drain in proximity to the Riverside Drain. Valid data description of ground water was retained for sampling site No. 3. In order to provide water quality data for storm flows in the Riverside Drain above the confluence with the Alameda Drain, results of an independent stormwater quality study concurrently conducted by the City Laboratory at the Southside Water Reclamation Plant was utilized. The City Laboratory sampling included data points coinciding with sampling sites No. 5 and 6 and a data point on the Riverside Drain above Central Avenue (upstream from the confluence with the Alameda Drain). As the data results for the City Laboratory study closely correlated with the Alameda/Riverside Drains project, results at sampling sites 5 and 6 for many of the same sampling date and most of the parameters, the City Laboratory data results for the Riverside Drain at Central Avenue as well as results near sampling site 6, were incorporated into the data base as sites 3A and 6A, respectively.

7.2.4 Water Quality Parameters

Water samples collected from unmanned samplers and by grab sample techniques were submitted to Kramer and Associates Laboratory in Albuquerque for analysis. Table 7.2.4. outlines the parameters for which laboratory tests were conducted, as well as indicating the field tests performed for each sample. The list of parameters represents attributes which characterize municipal stormwater runoff, and is derived from water quality standards for interstate and intrastate streams in New Mexico and from a proposed rule (December 7, 1988) for NPDES Permit Application Regulations for Stormwater Discharges.

7.3 Existing Quality of Alameda/Riverside Drains Water and Storm Sewer Water

The water samples collected during this investigation were delivered to Kramer and Associates Laboratory in Albuquerque. The original laboratory data sheets are available for review in the City Hydrology Section of the Public Works Division.

TABLE 7.2.4

<u>TESTED WATER QUALITY PARAMETERS</u>

<u>Field Tests</u>

- pH
- Temperature
- Conductivity
- Dissolved Oxygen
- Residual Chlorine

<u>Laboratory Tests</u>

- Metals
  - Cadmium
  - Chromium
  - Copper
  - Lead
  - Silver
  - Nickel
  - Zinc
- Total Dissolved Solids
- Suspended Solids
- Biological Oxygen Demand 5
- Chemical Oxygen Demand
- Total Organic Carbon
- Nitrate and Nitrite as Nitrogen
- Phosphate
- Ammonia
- Kjeldahl Nitrogen
- Oil and Grease
- Total Phenol
- Cyanides
- Surfactants (MBAS)
- Volatile Organic Carbon (VOC)
- Fecal Streptococcus
- Chlorides

An analysis was performed on the water quality data to determine whether or not a significant statistical difference exists between the samples collected at the upstream or downstream stations or between the samples collected in the high and low stages. See Table 7.3.1 summary of sample results by parameter and location. The results of the analysis indicated that there are no significant statistical differences in the data. As a result, the data for the high and low stages for all stations were averaged, and the parameter averages were used to make the comparison between the existing conditions during storm flows in the drains and future conditions with additional stormwater

TABLE 7.3.1
SUMMARY OF SAMPLES OBTAINED

RIVERSIDE ALAMEDA DRAINS — RANGE OF SAMPLE DATA FOR SAMPLE POINT

PARAMETER	SAMPLE	1	2	3	4	5	6	7	8
AMMONIA	STORM HIGH	0.13	0.01-0.20	-	0.13-0.36	0.15	0.24	0.18-0.63	0.15-1.0
	STORM LOW	0.13	0.19-0.60	0.46	0.12-0.40	0.16-0.35	0.10-0.28	0.13-0.50	0.09-0.39
	IRRIGATION	0.09-0.21	0.06-0.20	0.10-2.62	0.06-0.25	0.05-0.11	0.03-1.43	-	-
	DRY	-	-	0.50	-	0.05-0.24	0.09-0.50	-	-
BOD	STORM HIGH	0.90	6.60-9.00	-	2.40-14.00	3.80	3.30	5.00-12.00	14.00-53.00
	STORM LOW	2.40	6.60-12.00	1.20	0.90-6.00	1.80-3.00	0.90-6.30	1.00-5.90	1.00-40.00
	IRRIGATION	1.00-3.00	0.50-3.00	0.10-1.20	0.50-2.50	0.70-3.00	0.50-7.50	-	-
	DRY	-	-	0.40	-	0.40-0.50	0.50-0.90	-	-
CADMIUM (MG/L)	STORM HIGH	*0.02	*0.02	-	*0.02	*0.02	*0.02	*0.02	*0.02
	STORM LOW	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02
	IRRIGATION	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02	-	-
	DRY	-	-	*0.02	-	*0.02	*0.02	-	-
CHLORIDE	STORM HIGH	13.0	4.0-8.8	-	3.5-10.0	11.0	19.4	0.5-11.0	0.5-2.0
	STORM LOW	13.0	7.0-12.5	7.8	6.0-12.0	10.5-12.5	12.4-18.5	5.4-14.0	2.0-12.0
	IRRIGATION	9.0-13.0	9.0-13.0	7.8-12.9	8.0-12.0	9.0-12.5	10.0-17.9	-	-
	DRY	-	-	13.1	-	14.0-15.0	17.9-21.5	-	-
CHROMIUM (MG/L)	STORM HIGH	0.02-0.10	0.06	-	0.02-0.04	0.02	0.02	0.02	0.02
	STORM LOW	0.02-0.06	0.02	-	0.03-0.09	0.02	0.02	0.02	0.02
	IRRIGATION	0.02	0.02	-	0.02	0.02	0.00-0.02	-	-
	DRY	-	-	-	-	0.02	0.00-0.02	-	-
COD (MG/L)	STORM HIGH	4	26-38	-	20-66	57	58	23-804	88-226
	STORM LOW	17	24-64	-	14-39	4-10	11-40	52-168	36-52
	IRRIGATION	8-35	10-25	1-16	14-30	1-17	1-30	-	-
	DRY	-	-	6	-	7-14	2-17	-	-
CONDUCTIVITY (MG/L)	STORM HIGH	-	230	-	160-340	480	450	132-168	125-190
	STORM LOW	550	250-455	360	360-480	475-490	525-560	215	165-225
	IRRIGATION	400-485	345-510	340-360	385-485	425-530	340-560	-	-
	DRY	-	-	400	-	495-545	430-595	-	-
COPPER (MG/L)	STORM HIGH	0.02	0.02	-	0.02-0.08	0.02	0.02	0.02	0.02-0.04
	STORM LOW	0.02-0.03	0.02-0.05	0.01	0.02-0.08	0.02	0.02-0.03	0.02	0.02-0.05
	IRRIGATION	0.02	0.02	0.01	0.02	0.02	0.01-0.02	-	-
	DRY	-	-	0.01	-	0.02-0.80	0.01-0.02	-	-
CYANIDE (MG/L)	STORM HIGH	0.005	0.005	-	0.005-0.011	0.005	0.008	0.005-0.011	0.005-0.008
	STORM LOW	0.005	0.005-0.009	-	0.005-0.006	0.005	0.005	0.005-0.008	0.005-0.009
	IRRIGATION	0.005	0.005	0.005	0.005	0.005	0.005-0.006	-	-
	DRY	-	-	-	-	0.005	0.005	-	-
DISSOLVED OXYGEN (MG/L)	STORM	6.80	7.50	7.20	7.85	6.75	7.20	-	-
	IRRIGATION	8.40-9.65	8.60-9.85	7.20-8.20	8.40-9.75	7.40-9.70	7.20-9.80	-	-
	DRY	-	-	6.90	-	9.00-9.70	6.80-9.20	-	-
NICKEL (MG/L)	STORM HIGH	*0.02	*0.02	-	*0.02	*0.02	*0.02	*0.02	*0.02
	STORM LOW	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02
	IRRIGATION	*0.02	*0.02	*0.02	*0.02	*0.02	*0.02	-	-
	DRY	-	-	*0.02	-	*0.02	*0.02	-	-

TABLE 7.3.1
SUMMARY OF SAMPLES OBTAINED

RIVERSIDE ALAMEDA DRAINS — RANGE OF SAMPLE DATA FOR SAMPLE POINT

PARAMETER	SAMPLE	1	2	3	4	5	6	7	8
FECAL COLIFORM (COLONIES/100ML)	STORM HIGH	TNTC	100-200	-	10-TNTC	TNTC	100	70-TNTC	TNTC
	STORM LOW	100-TNTC	100-TNTC	330	30-TNTC	240-TNTC	60-TNTC	100-TNTC	40-200
	IRRIGATION	10-1100	100-700	100-330	490-1100	20-400	10-870	-	-
	DRY	-	-	10	-	20-200	10-300	-	-
FECAL STREPTOCOCCUS (COLONIES/100ML)	STORM HIGH	170	10-1060	-	150-TNTC	1000	110	10-1200	190-TNTC
	STORM LOW	50-390	10-120	-	20-1040	180-240	50-100	20-TNTC	30-TNTC
	IRRIGATION	230-1060	100-500	40-320	310-660	340-520	540-TNTC	-	-
	DRY	-	-	-	-	80-160	140-220	-	-
NITROGEN (MG/L)	STORM HIGH	0.65	0.69-2.70	-	0.68-1.70	0.64	1.40	0.28-1.70	1.09-5.90
	STORM LOW	0.99	0.87-3.80	-	0.49-1.59	0.51-0.71	0.67-0.80	0.63-1.30	0.64-10.00
	IRRIGATION	0.39-0.72	0.23-1.20	-	0.11-0.58	0.11-1.60	0.21-1.30	-	-
	DRY	-	-	-	-	0.12-0.30	0.14-0.54	-	-
LEAD (MG/L)	STORM HIGH	0.41	0.44-0.46	-	0.10-0.41	0.10	0.05	0.10-0.22	0.20-0.53
	STORM LOW	0.30-0.59	0.20-0.49	0.01	0.10-0.20	0.10-0.28	0.10	0.10-0.31	0.10-0.20
	IRRIGATION	0.20	0.20-0.30	0.01	0.20-0.30	0.20-0.30	0.01-0.20	-	-
	DRY	-	-	0.10	-	0.20	0.01-0.20	-	-
DETERGENTS (MG/L)	STORM HIGH	0.009	0.017-0.081	-	0.015-0.087	0.230	0.005	0.010-0.056	0.035-0.110
	STORM LOW	0.021	0.011-0.230	-	0.009-0.170	0.015-0.280	0.015-0.620	0.007-0.260	0.014-0.070
	IRRIGATION	0.005-0.011	0.003-0.010	0.100	0.004-0.014	0.007-0.018	0.006-0.100	-	-
	DRY	-	-	0.100	-	0.005-0.027	0.016-0.100	-	-
NITRATE (MG/L)	STORM HIGH	0.90	0.10-0.90	-	0.10-1.10	1.10	0.10	0.30-2.00	0.70-2.00
	STORM LOW	0.10-0.60	0.10-1.10	-	0.20-1.20	0.70-1.00	0.70-1.20	0.10-1.40	0.10-1.70
	IRRIGATION	0.10-0.20	0.10-0.20	0.10-0.17	0.10	0.10	0.10-0.30	-	-
	DRY	-	-	0.10	-	0.10-0.20	0.10-0.60	-	-
OIL & GREASE (MG/L)	STORM HIGH	19.9	15.0-19.0	-	13.6-19.0	21.0	17.6	17.0-42.0	11.0-48.0
	STORM LOW	20.4	8.4-36.0	-	8.0-34.0	19.0-32.0	20.0-26.0	15.0-46.0	15.0-64.0
	IRRIGATION	6.7-11.5	6.4-13.3	-	7.4-12.6	6.8-13.6	7.1-13.6	-	-
	DRY	-	-	-	-	13.6-25.2	13.2-22.0	-	-
PH	STORM HIGH	-	7.76	-	7.75-7.77	-	7.88	7.71	7.43-7.77
	STORM LOW	-	7.68-8.15	-	7.62-8.24	8.15-8.34	8.15-8.38	7.78-8.58	7.76-7.77
	IRRIGATION	7.60-8.26	8.00-8.35	8.40	7.80-8.37	7.50-8.18	7.70-8.20	-	-
	DRY	-	-	7.90	-	7.30-7.70	7.35-7.90	-	-
PHENOLS (MG/L)	STORM HIGH	0.009	0.019-0.207	-	0.019-0.258	0.017	0.016	0.019-0.054	0.012-0.028
	STORM LOW	0.005	0.003-0.021	-	0.003-0.025	0.002-0.020	0.001-0.016	0.019-0.057	0.018-0.062
	IRRIGATION	0.003-0.020	0.003-0.020	-	0.003-0.024	0.003-0.023	0.003-0.023	-	-
	DRY	-	-	-	-	0.005	0.005	-	-
ORTHO-PHOSPHATE (MG/L)	STORM HIGH	0.29	0.33-0.54	-	0.16-1.01	1.68	1.03	0.13-0.64	0.56-1.70
	STORM LOW	0.61	0.48-5.16	0.10	0.12-2.41	1.12-2.19	1.06-1.78	0.27-0.62	0.20-0.95
	IRRIGATION	0.07-2.52	0.13-0.64	0.10	0.10-1.50	0.17-0.38	0.10-1.84	-	-
	DRY	-	-	0.10	-	0.16-0.30	0.02-1.04	-	-

TABLE 7.3.1
SUMMARY OF SAMPLES OBTAINED

RIVERSIDE ALAMEDA DRAINS — RANGE OF SAMPLE DATA FOR SAMPLE POINT

PARAMETER	SAMPLE	1	2	3	4	5	6	7	8
RESIDUAL CHLORINE	STORM HIGH	-	0.06	-	0.06	-	0.03	0.03-0.06	0.03
	STORM LOW	-	0.03-0.06	0.10	0.03	-	-	0.02 *	0.03-0.04
	IRRIGATION	0.02-0.03	0.05-0.06	0.10	0.02-0.06	0.03-0.13	0.03-0.10	-	-
	DRY	-	-	0.10	-	0.04-0.07	0.05-0.13	-	-
SILVER (MG/L)	STORM HIGH	0.12	0.02-0.05	-	0.02-0.10	0.02	0.02	0.02-0.05	0.02-0.05
	STORM LOW	0.02-0.05	0.02-0.05	0.02	0.02-0.12	0.02-0.09	0.02-0.13	0.02-0.05	0.02-0.05
	IRRIGATION	0.05	0.05	0.02	0.05	0.05	0.02-0.05	-	-
	DRY	-	-	0.02	-	0.05	0.02	-	-
TOTAL DISSOLVED SOLIDS (MG/L)	STORM HIGH	247	163	130-298	-	293	240	130-243	40-138
	STORM LOW	463	155-273	330	230-310	240-318	258-358	135-240	72-248
	IRRIGATION	298-315	273-308	284-330	285-335	288-313	268-1175?	-	-
	DRY	-	-	315	-	303-310	333-353	-	-
TSS (MG/L)	STORM HIGH	168	874-1060	-	667-1201	2968	636	217-848	252-352
	STORM LOW	270	203-3768	60	294-2370	268-2142	968-1722	294-710	417-1784
	IRRIGATION	113-314	64-237	31-60	57-296	32-148	19-238	-	-
	DRY	-	-	3	-	7-9	10-52	-	-
TOC (MG/L)	STORM HIGH	5.76	8.90	-	6.60-17.00	10.90	16.50	11.10-19.00	10.20-12.00
	STORM LOW	9.52	5.60-16.90	-	6.30-10.00	8.80-15.60	8.40-12.20	3.60-17.30	6.50-19.80
	IRRIGATION	5.40-6.50	5.60-8.90	-	4.70-7.40	3.90-5.50	4.10-6.50	-	-
	DRY	-	-	-	-	3.50-16.10	4.80-10.70	-	-
VOC (MG/L)	STORM HIGH	0.002	0.002-0.005	-	0.002-0.035	0.004	0.156	0.002-0.094	0.002-0.098
	STORM LOW	0.002-0.010	0.003-0.183	-	0.002-0.286	0.002-0.004	0.002-0.216	0.002-5.830	0.002-0.286
	IRRIGATION	0.003-0.005	0.003-0.004	-	0.003-0.004	0.003-0.099	0.003-0.018	-	-
	DRY	-	-	-	-	0.003-0.004	0.003-0.016	-	-
TEMPERATURE (DEG. CELSIUS)	STORM	20.0	19.5	19.0	18.5-19.0	17.5-19.0	17.5-19.0	-	-
	IRRIGATION	10-0-16.0	10.5-16.0	16.0-19.0	14.0-16.5	13.5-16.5	14.0-18.0	-	-
	DRY	-	-	16.0	-	13.0-14.5	13.0-15.0	-	-
ZINC (MG/L)	STORM HIGH	0.02	0.02-0.14	-	0.02-0.31	0.12	0.15	0.02-0.37	0.02-0.62
	STORM LOW	0.02-0.07	0.09-0.23	0.03	0.07-0.22	0.02-0.10	0.04-0.19	0.02-0.18	0.02-0.73
	IRRIGATION	0.02-0.08	0.02-0.07	0.01-0.03	0.02-0.04	0.02-0.03	0.01-0.03	-	-
	DRY	-	-	0.01	-	0.02-0.18	0.01-0.02	-	-

* VALUE INDICATED EQUALS LIMIT OF DETECTION FOR TESTING PROCEDURE.

TNTC = TOO NUMEROUS TO COUNT

discharging into the drains. High and low values were noted for several parameters, however, they do not appear to represent a trend. Each individual water sample collected does not represent a storm weighted average, however, the average computed from the data represent a reasonable determination of stormwater quality.

Baseline or "existing conditions" were represented by averaging the data for existing conditions during storm flows in the drains in both the high and low stages at stations 1, 2, 3A, 4, 5, 6 and 6A as presented in the summary tables contained in Appendix H, Volume II. The "existing conditions" flows include irrigation water, stormwater and groundwater when present.

Future conditions with additional stormwater discharging to the drains are, as a result, represented by the average of all data collected in the high and low stages at stations 7 and 8 which are storm drain stations sampled during storm events. This data is also contained in Appendix H, Volume II. All averages were computed ignoring any analyses which were reported as "less than" the detection limit.

Since the proportion of additional stormwater to be discharged to the drains for future conditions with respect to the quantity of "existing conditions" water already present is not known, it was conservatively assumed that future conditions would be represented by 100% storm drain water. The summarized data contained in Appendix H, Volume II was plotted for each parameter, and the plots are presented in FigureS 7.4.1 to 7.4.8.

In addition to the water quality comparison between existing and future conditions, a comparison was also made with available water quality criteria and guidelines. The criteria and guidelines were obtained from the following sources:

1. (WQCC (ST) Water Quality Standards for Interstate and Intrastate Streams in New Mexico, as amended through March 8, 1988.

2. (WQCC (WC), (HH),(WS)) New Mexico Water Quality Control Commission Regulations, as amended November 25, 1988.

3. (EPA Guideline (LS)) Environmental Protection Agency, A Report of the Committee on Water Quality Criteria, Water Quality Criteria, 1972, Water for Livestock Enterprises.

4. (EPA Guideline (IR)) Environmental Protection Agency, A Report of the Committee on Water Quality Criteria, Water Quality Criteria, 1972, Water for Irrigation.

5. (EPA Guideline (LS)) Federal Water Pollution Control Administration, Report of the National Technical Advisory Committee, Water Quality Criteria, 1968, Livestock Water Supplies.

6. (EPA Guideline (AL)) Environmental Protection Agency, Quality Criteria for Water 1986.

While none of the criteria or guidelines appear to be required standards at this time, for discharge of floodwater, the comparisons with the criteria or guidelines were included to provide an indication of the potential for future regulatory concerns. It is assumed that the enforcement of NPDES would adopt similar criteria or guidelines once floodwater discharges become regulated. Table 7.3.2 provides a summary of the available criteria and guidelines from the previously listed documents. The criteria or guideline selected for comparison with the water quality data collected is underlined in Table 7.3.2. The selection was based principally on the designated uses of the Rio Grande between the headwaters of Elephant Butte Reservoir and the Angostura Diversion Works which are irrigation, limited warm water fishing, livestock and wildlife watering, and secondary contact recreation (see Water Quality Standards for Interstate and Intrastate Streams in New Mexico, March 8, 1988). As the final NPDES program for regulation of stormwater discharges from municipal storm sewer systems is not yet certain, no standards or guidelines for this emerging program are provided in Table 7.3.2.

Table 7.3.2 Water Quality Standards and Guidelines (mg/l unless noted)

	Stream (ST)	Discharge to Water Course (WC)	Discharge to GW: Human Health (HH)	Discharge to GW: Water Supply (WS)	EPA Guidelines: Irrig. (IR)	EPA Guidelines: Live Stock (LS)	EPA Guidelines: Aquatic Life (AL)
Ammonia							1.09[6]
BOD		30[2]					
Cadmium			0.01[2]		0.05[4]	0.05[3,5]	
Chloride	250[1]			250[2]		1500[3,5]	
Chromium			0.05[2]		1.0[4]	1.0[3,5]	
COD		125[2]					
Conductivity					750		
Copper				1.0[2]	.2[4]	.5[3,5]	0.024[6]
Cyanide			0.22				0.022[6]
D.O.	>4.0[1]						
Fecal Coliform	2000/100ml[1]	500/100ml[2]					
Fecal Shep							
TKN							
Lead			0.05[2]		5[4]	0.1[3,5]	
Detergents							
Nickel					0.2[4]		
Nitrate			10.0[2]			100[3,5]	
Oil & Grease							
pH	6.0-9.0[1]	6.6-8.62		6.0-9.0[2]	4.5-9[4]		
Phenols			3.5	0.005			
Ortho-Phosphate							
Silver			0.05[2]				
Temperature	<32.2°C[1]						
Total Chlorine							.011
TDS	1500[1]			1000[2]	1000[4]	3000[3,5]	
TSS							
TOC							
VOC							
Zinc				10.0	2.0[4]	25[3,5]	

[1] WQCC (ST)
[2] WQCC (WC) (HH) or (WS)
[3] EPA Guideline (LS)
[4] EPA Guideline (IR)
[5] EPA Guideline (LS)
[6] EPA Guideline (AL)
(see Section 7.3 for full citation)

7.4 Specific Parameter Results

The following discussion provides short summaries of the indication of each of the water quality parameters tested in this study.

7.4.1 Ammonia

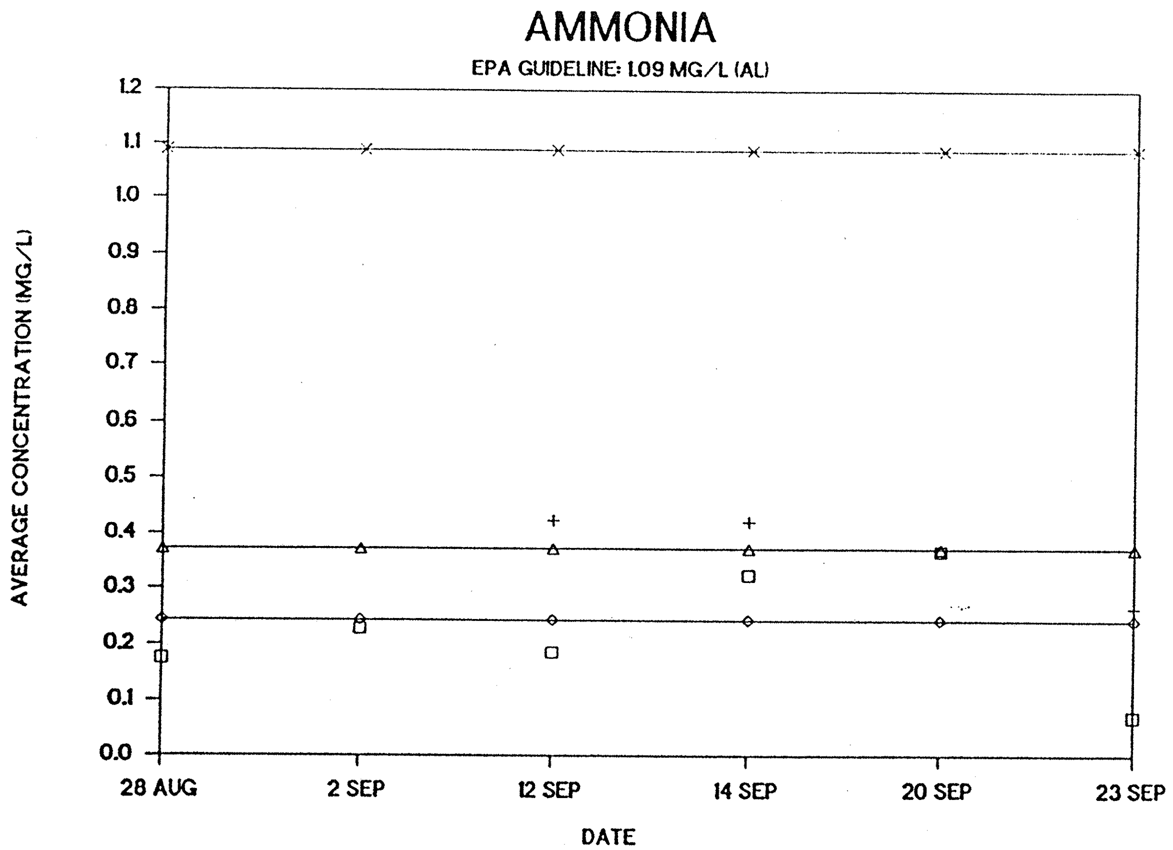
Ammonia is naturally present in surface water. It is a form of nitrogen which is produced in the nitrogen cycle and by the hydrolysis of urea. The recommended guideline is 1.09 mg/l. This is the aquatic life guideline for maximum 4-day average concentration. The average concentration for existing storm conditions in the drains is 0.25 mg/l whereas the average concentration for the storm sewer samples is 0.37 mg/l (see Table 7.4.1, Appendix H, Volume II). This represents an increase, but with additional stormwater should still exhibit a concentration of ammonia less than the guideline of 1.09 mg/l (see Figure 7.4.1).

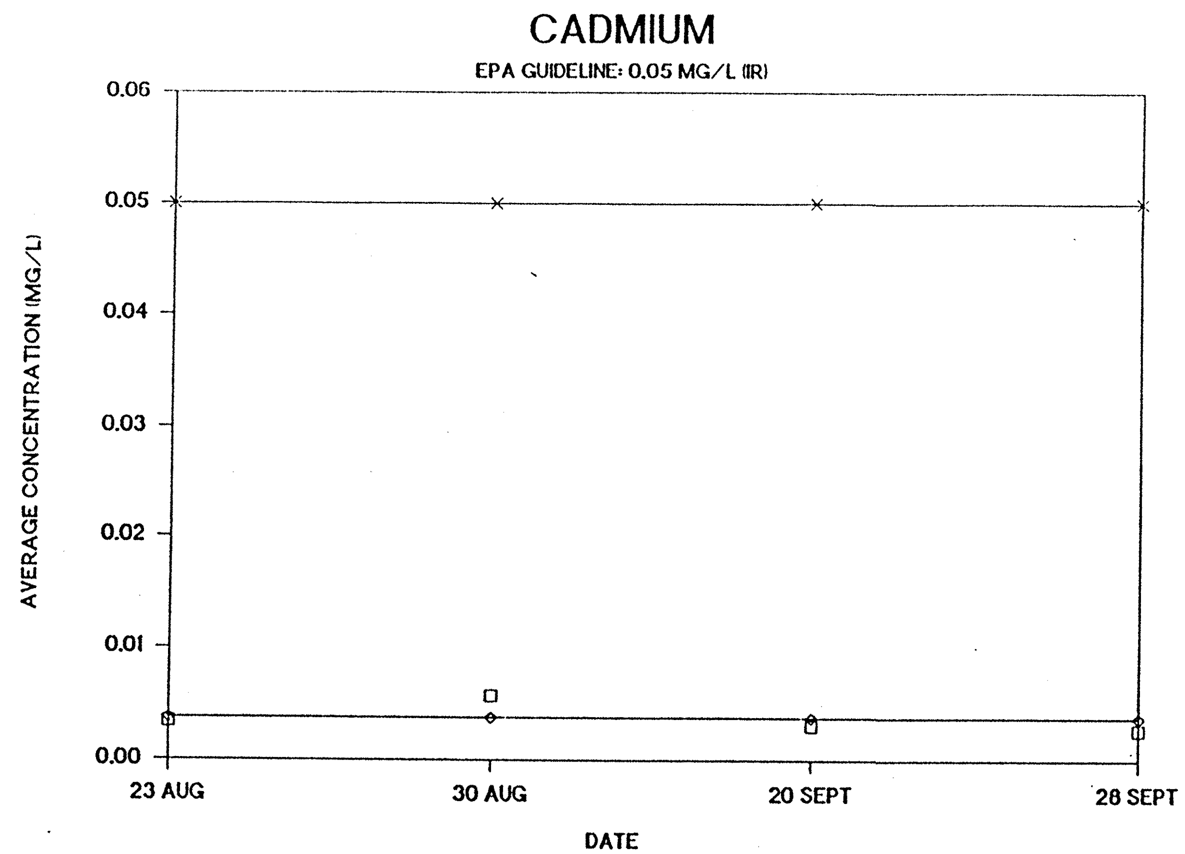
7.4.2 Biochemical Oxygen Demand

Biochemical Oxygen Demand or BOD-5 is a test to determine the oxygen requirements of organic material and inorganic material such as sulfides and iron in waters. The standard for discharges to watercourses in the State of New Mexico is 30 mg/l. The average concentration for existing storm conditions in the drain is 4.84 mg/l whereas the average concentration for the storm sewer samples is 14.23 mg/l (see Table 7.4.2, Appendix H, Volume II). This represents an increase, but the future conditions with additional stormwater should still exhibit a concentration of BOD-5 less than the guideline of 30 mg/l (see Figure 7.4.1).

7.4.3 Cadmium

Cadmium is a toxic metal whose source in water may be discharges from plating or battery manufacture, fungicides, or the deterioration of galvanized metal. The recommended guideline for both irrigation and livestock water is 0.05 mg/l. The average concentration for existing storm conditions in the drains is 0.004 mg/l whereas the average concentration for the storm sewer samples is <0.02





LEGEND

IRRIGATION AND STORM, POINTS 1 THROUGH 6 AVERAGE □

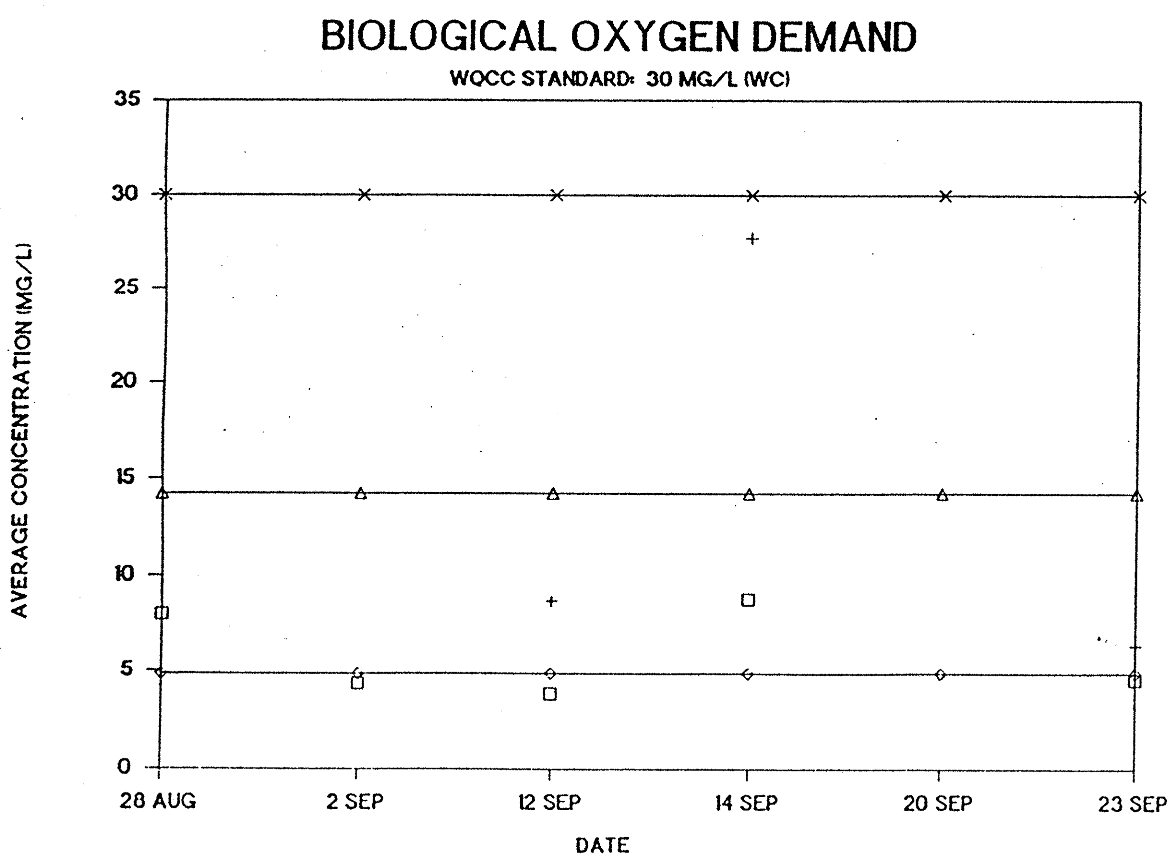
STORM, POINTS 7 AND 8 AVERAGE +

IRRIGATION AND STORM POINTS 1 THROUGH 6, OVERALL AVERAGE ◇—◇

STORM POINTS 7 AND 8, OVERALL AVERAGE △—△

RECOMMENDED GUIDE-LINE/WQCC STANDARD, UPPER LIMIT ×—×

RECOMMENDED GUIDE-LINE/WQCC STANDARD, LOWER LIMIT ▽—▽



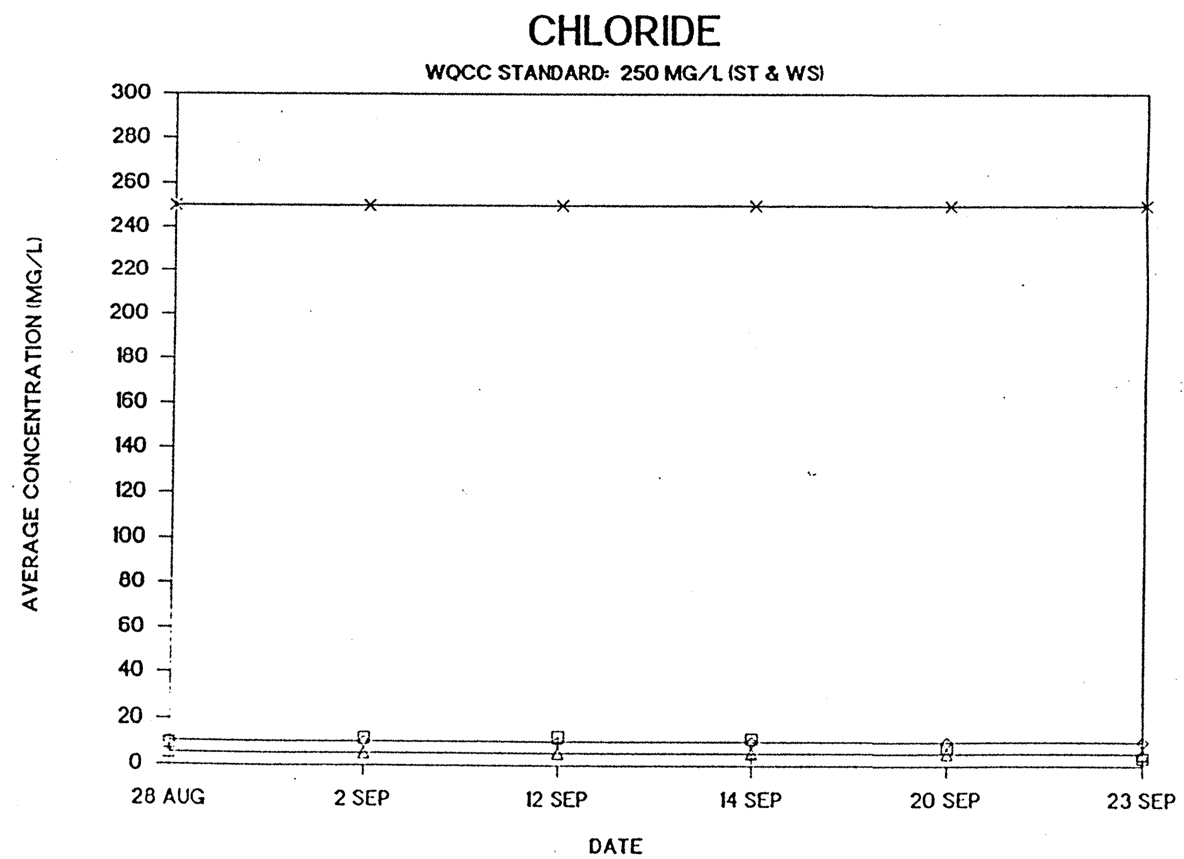


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FIGURE 7.4.1

LEEDSHILL-HERKENHOFF

mg/l (see Table 7.4.3, Appendix H, Volume II). It is not clear whether an increase or decrease in cadmium concentration can be expected for the future conditions with additional stormwater. In either case, the future conditions should exhibit a concentration of cadmium less than the guideline of 0.05 mg/l (see Figure 7.4.1).

7.4.4 Chloride

Chloride is one of the most common inorganic ions found in water. A high chloride content in water may be detrimental to its use for drinking water, irrigation water, or livestock water.

The New Mexico stream standard is 250 mg/l. The average concentration for existing storm conditions in the drains is 10.7 mg/l whereas the average concentration for the storm sewer sample is 5.6 mg/l (see Table 7.4.4, Appendix H, Volume II). Future conditions with additional storm sewer water should exhibit a decrease in chloride concentrations (see Figure 7.4.1).

7.4.5 Chromium

Sources of chromium in the drains and storm sewers could include discharges from plating processes, paint manufacturing or paint shipping operations. Chromium may also be present on roadways as a result of wear of engine parts and brake linings of vehicles. The guideline for chromium in irrigation or livestock water is 1.0 mg/l. The average concentration for existing storm conditions in the drains is 0.03 mg/l (see Table 7.4.5, Appendix H, Volume II). Future conditions with additional storm sewer water should exhibit a decrease in chromium concentrations (see Figure 7.4.2).

7.4.6 Chemical Oxygen Demand

Chemical Oxygen Demand (COD) is a measure of the amount of organic matter which is not subject to biological decomposition. The standard for discharges to a water course is a maximum COD of 125 mg/l.

The average concentration for existing storm conditions in the drains is 34 mg/l whereas the average concentration for the storm sewer samples is 150 mg/l (see Table 7.4.6, Appendix H, Volume II). This represents an increase for future conditions to a concentration possibly greater than the stream standard during storm events (see Figure 7.4.2).

7.4.7 Conductivity

Conductivity is a measurement of the ability of water to carry an electrical current. It is an indicator of the presence and types of ions in solution. There is no standard or criteria for conductivity, however irrigation waters are classified as "good" by agricultural specialists if the conductivity is less than 750 umhos/cm.

The average conductivity for existing storm conditions in the drains is 408 umhos/cm whereas the average conductivity for the storm sewer samples is 174 umhos/cm (see Table 7.4.7, Appendix H, Volume II). This represents a decrease in conductivity for future conditions (see Figure 7.4.2).

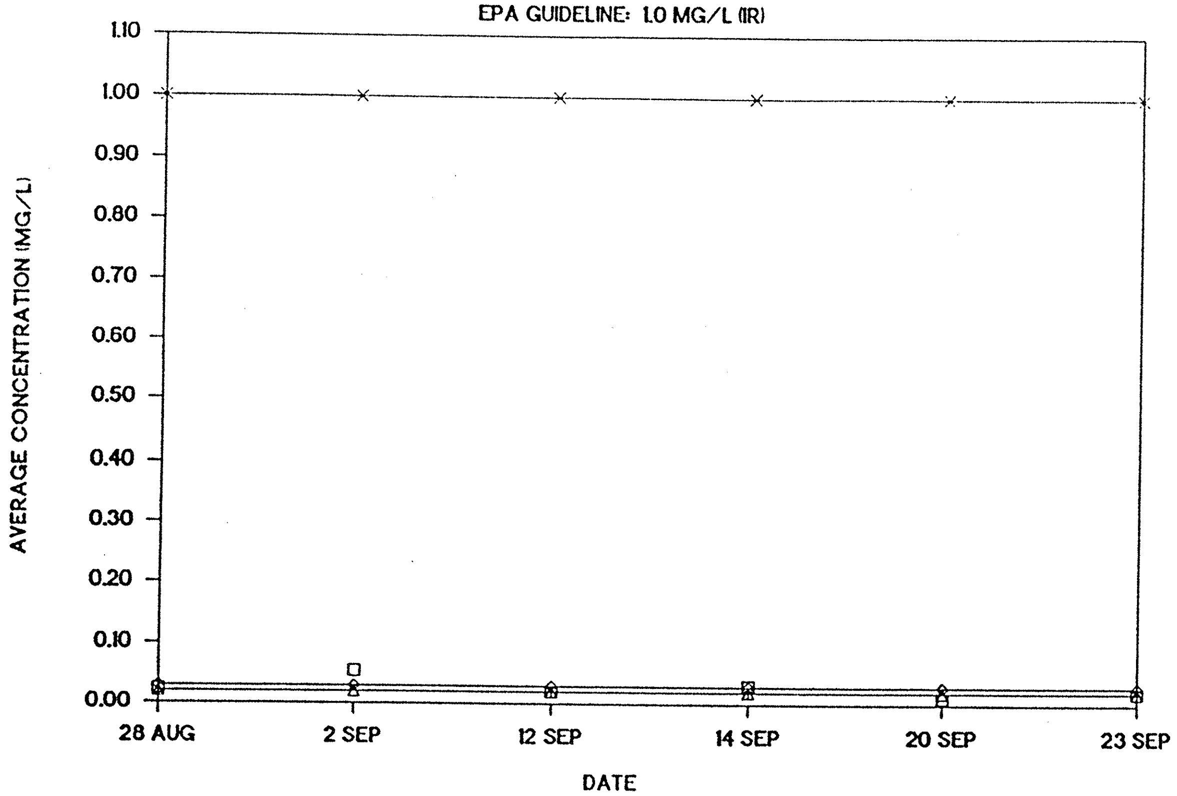
7.4.8 Copper

Copper is an essential metal for humans, but can be toxic to some aquatic life. The sources of copper in water may be corrosion of copper-containing alloys, discharges from plating operations, fungicides or pesticides. Copper has a guideline criteria for aquatic life of 0.024 mg/l for a 4-day average, and 0.039 mg/l for a 1-hour average.

The average concentration for existing storm conditions in the drains is 0.03 mg/l whereas the concentration for storm sewer samples is 0.02 mg/l (see Table 7.4.8, Appendix H, Volume II). This represents a decrease in copper concentrations from above the guideline to below the guideline for future conditions with additional stormwater (see Figure 7.4.2).

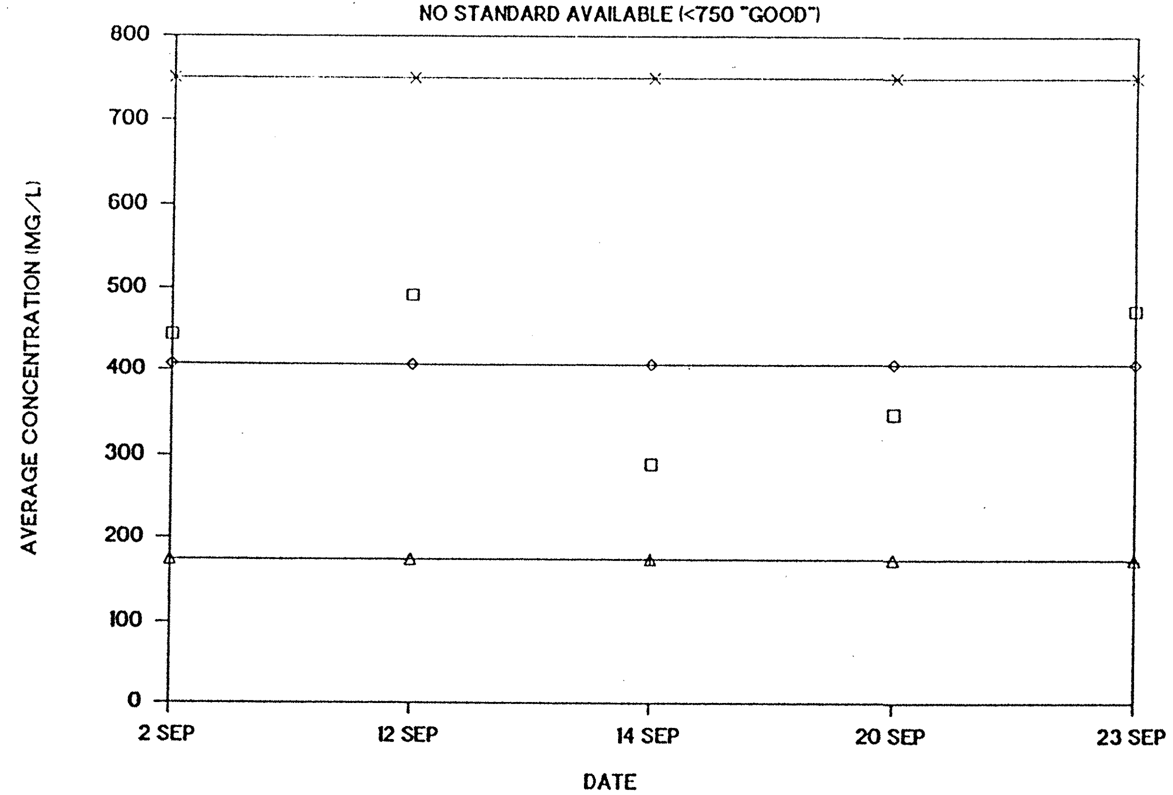
CHROMIUM

EPA GUIDELINE: 1.0 MG/L (IR)



CONDUCTIVITY

NO STANDARD AVAILABLE (<750 "GOOD")



LEGEND

IRRIGATION AND STORM, POINTS 1 THROUGH 6 AVERAGE

STORM, POINTS 7 AND 8 AVERAGE

IRRIGATION AND STORM POINTS 1 THROUGH 6, OVERALL AVERAGE

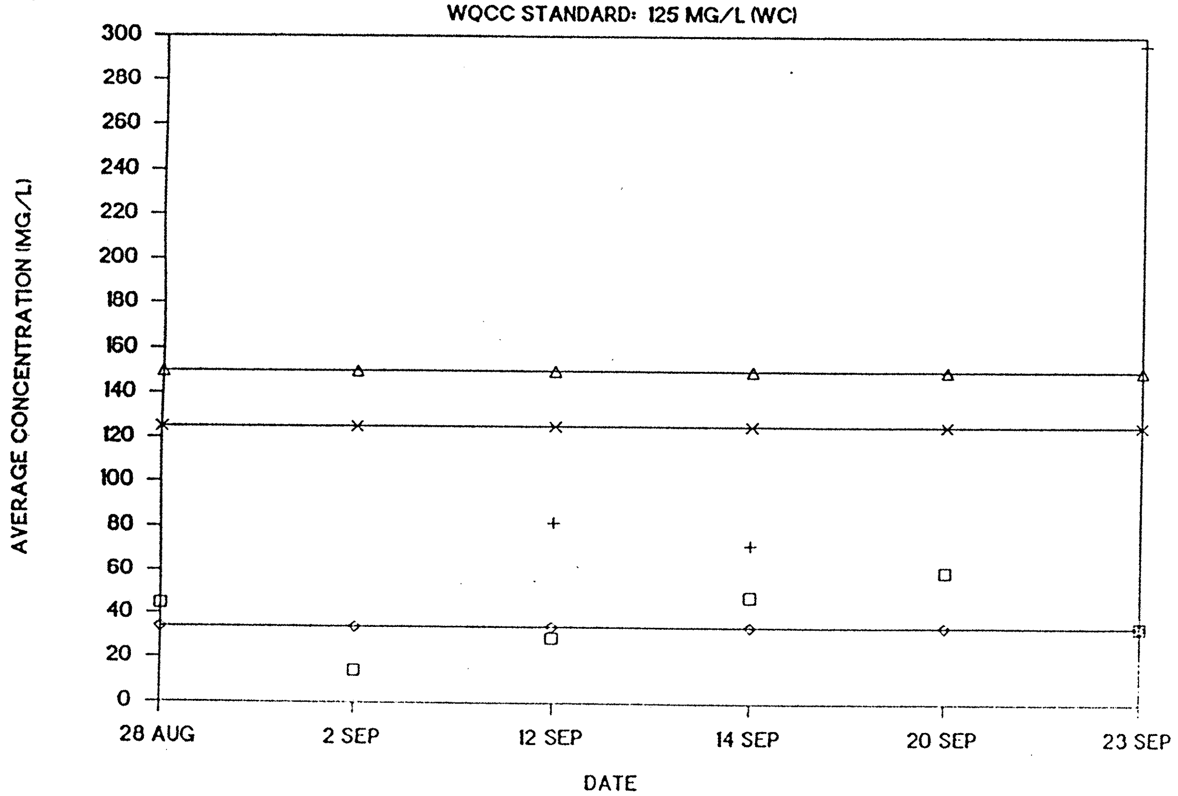
STORM POINTS 7 AND 8, OVERALL AVERAGE

RECOMMENDED GUIDE-LINE/WQCC STANDARD, UPPER LIMIT

RECOMMENDED GUIDE-LINE/WQCC STANDARD, LOWER LIMIT

CHEMICAL OXYGEN DEMAND

WQCC STANDARD: 125 MG/L (WC)



COPPER

EPA GUIDELINE: 0.024 MG/L @ 4 DAY (AL)

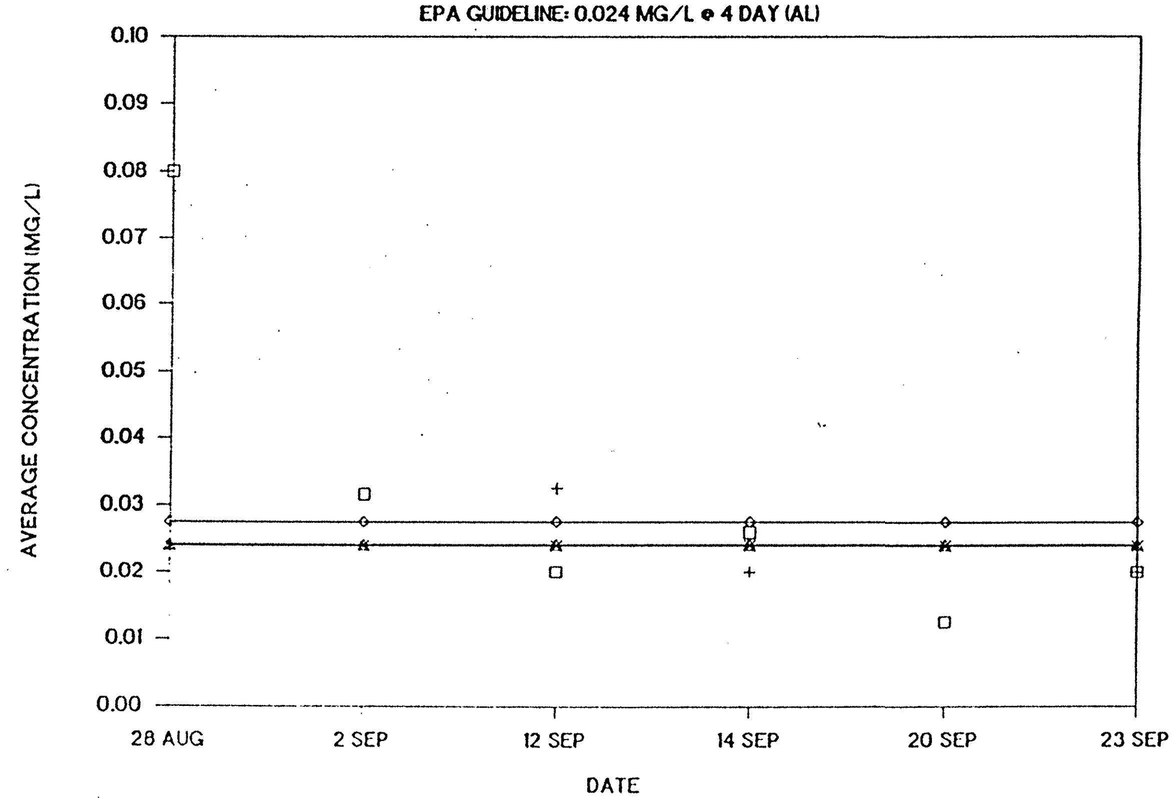


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FIGURE 7.4.2

7.4.9 Cyanide

Cyanide is a toxic element which is used as a fumigant for insects and rodents. It is very toxic to fish. The WQCC standard for cyanide in discharges is less than 0.2 mg/l, however the recommended aquatic life criteria is a maximum 1-hour average of 0.022 mg/l.

The average concentrations for both existing and future conditions is 0.006 mg/l (see Table 7.4.9, Appendix H, Volume II). Additional contributions of stormwater to the drains should not affect the cyanide concentrations (see Figure 7.4.3).

7.4.10 Dissolved Oxygen

Deficient dissolved oxygen can be detrimental to aquatic life. The New Mexico stream standard for dissolved oxygen in the section of the Rio Grande near Albuquerque is >4.0 mg/l. The average concentration for existing storm conditions in the drains is 7.22 mg/l whereas the average concentration for the storm sewer samples is 8.46 mg/l (see Table 7.4.10, Appendix H, Volume II). This represents an improvement in the dissolved oxygen concentration for future conditions (see Figure 7.4.3).

Dissolved oxygen concentration in the storm sewers were higher than in the drains even though BOD-5 and COD were higher, because the dissolved oxygen determinations were performed in the field on the turbulent flows which were continuously entraining oxygen from the atmosphere.

7.4.11 Fecal Coliforms

Fecal coliforms are a group of bacteria present in the feces of warm blooded animals and are frequently used as an indicator of pollution of water by contact with fecal matter. Standards for fecal coliforms have been set for the Rio Grande near Albuquerque of an average of 1000 colonies/100 ml or a single sample maximum of 2000 colonies/100 ml. The sample results shown on Figure 7.3.11 reflect the sample results in number per 10 ml sample. The maximum number of coliform colonies which can be distinguished in the method used is 200. Numbers higher than 200 are reported as "too numerous to count" or TNTC. Two hundred colonies in 10 ml is theoretically equivalent to 2000/100 ml.

Fecal coliform exceed the standard in virtually all of the storm season samples which were taken during the project period (see Table 7.4.11, Appendix H, Volume II and Figure 7.4.3). This has been observed in other studies of storm-water runoff including the Federal Highway Administration report "Sources and Migration of Highway Runoff Pollutants", FHWA/RD-84/057 through 84/060. A study conducted by the Water Pollution Control Segment of the New Mexico Health and Environment Department "Pollutant Loads in Stormwater Runoff from Albuquerque, New Mexico" also observed elevated Fecal coliform counts in stormwater runoff.

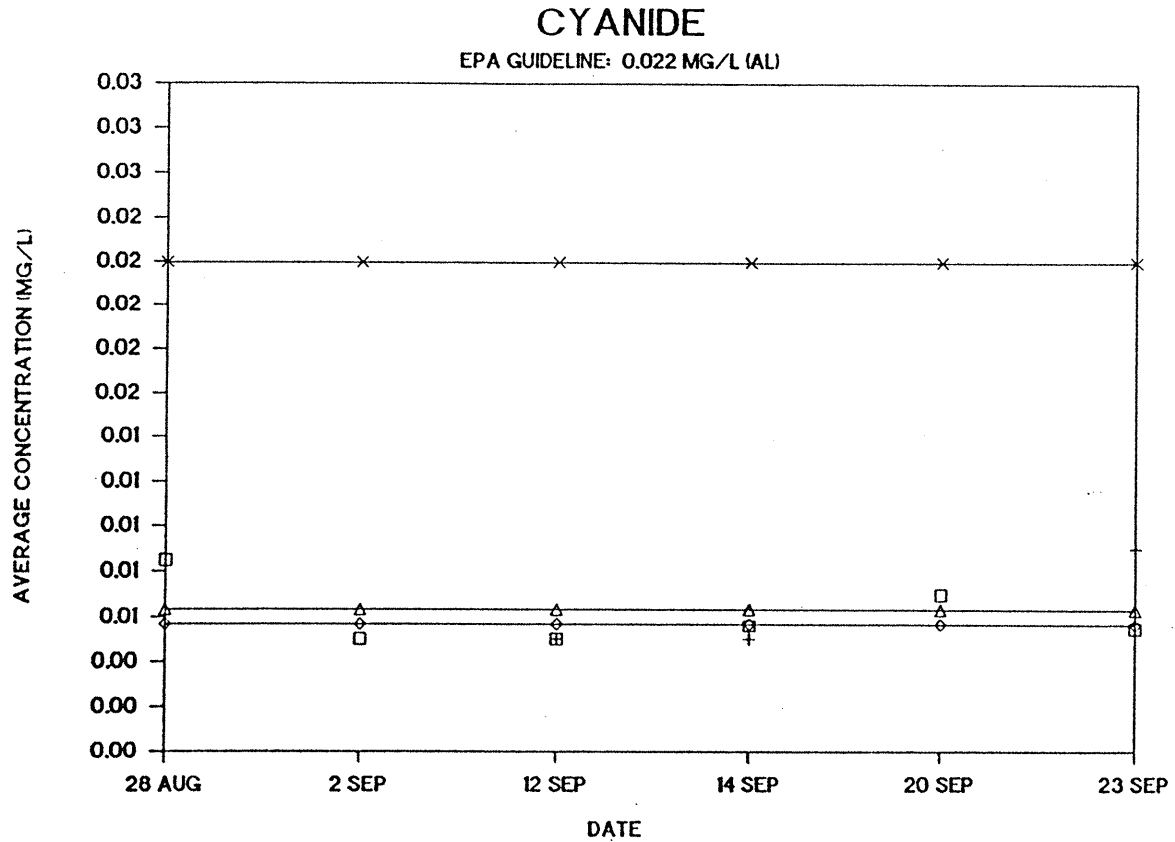
The FHWA study speculates that the source of coliforms on highways may be stock trailers, as well as birds and other animals travelling along the right-of-way. Counts tended to be higher in piped systems than in surface runoff. This was thought to be due to pockets of stagnant warm water which may harbor and grow the bacteria in the sewers.

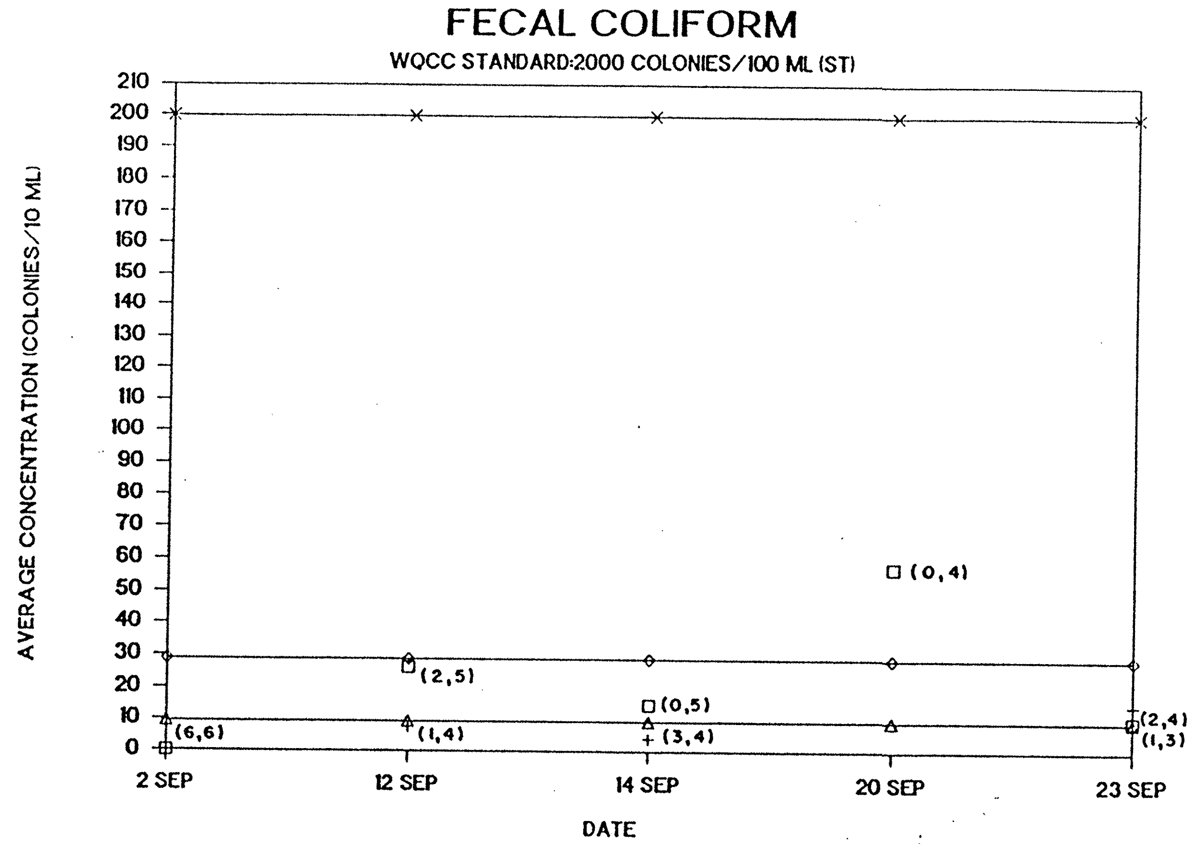
The source of coliforms for this project cannot be limited to piped stormwater systems. Runoff from agricultural areas adjacent to the drain as well as the possibility of infiltration or discharge from private septic systems are just as likely sources.

7.4.12 Fecal Streptococcus

Fecal streptococcus is a specific group of fecal coliform bacteria which may be used to further identify the source of fecal contamination of water sources. Specifically, the ratio of fecal coliform to fecal strep may be used to determine whether the source is man or animal. A ratio of 4.4 or greater is indicative of human waste. Ratios less than 0.7 suggest animal sources and ratios between 0.7 and 4.4 indicate mixed sources.

Figure 7.4.3 and Table 7.4.12, Appendix H, Volume II show the average fecal strep counts during the project sampling period. Ratios for the project period indicate primarily animal sources with the potential of some human wastes.





LEGEND

IRRIGATION AND STORM, POINTS 1 THROUGH 6 AVERAGE □

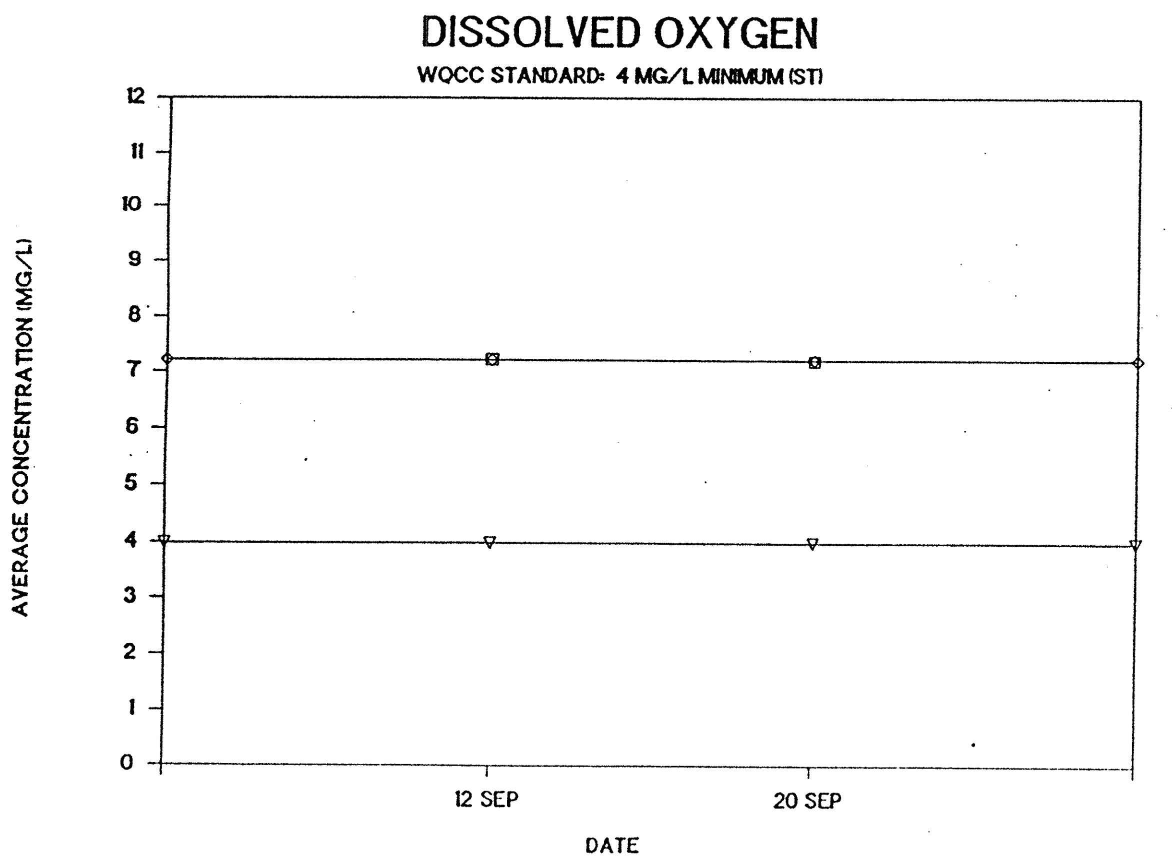
STORM, POINTS 7 AND 8 AVERAGE +

IRRIGATION AND STORM POINTS 1 THROUGH 6, OVERALL AVERAGE

STORM POINTS 7 AND 8, OVERALL AVERAGE

RECOMMENDED GUIDE-LINE/WQCC STANDARD, UPPER LIMIT

RECOMMENDED GUIDE-LINE/WQCC STANDARD, LOWER LIMIT



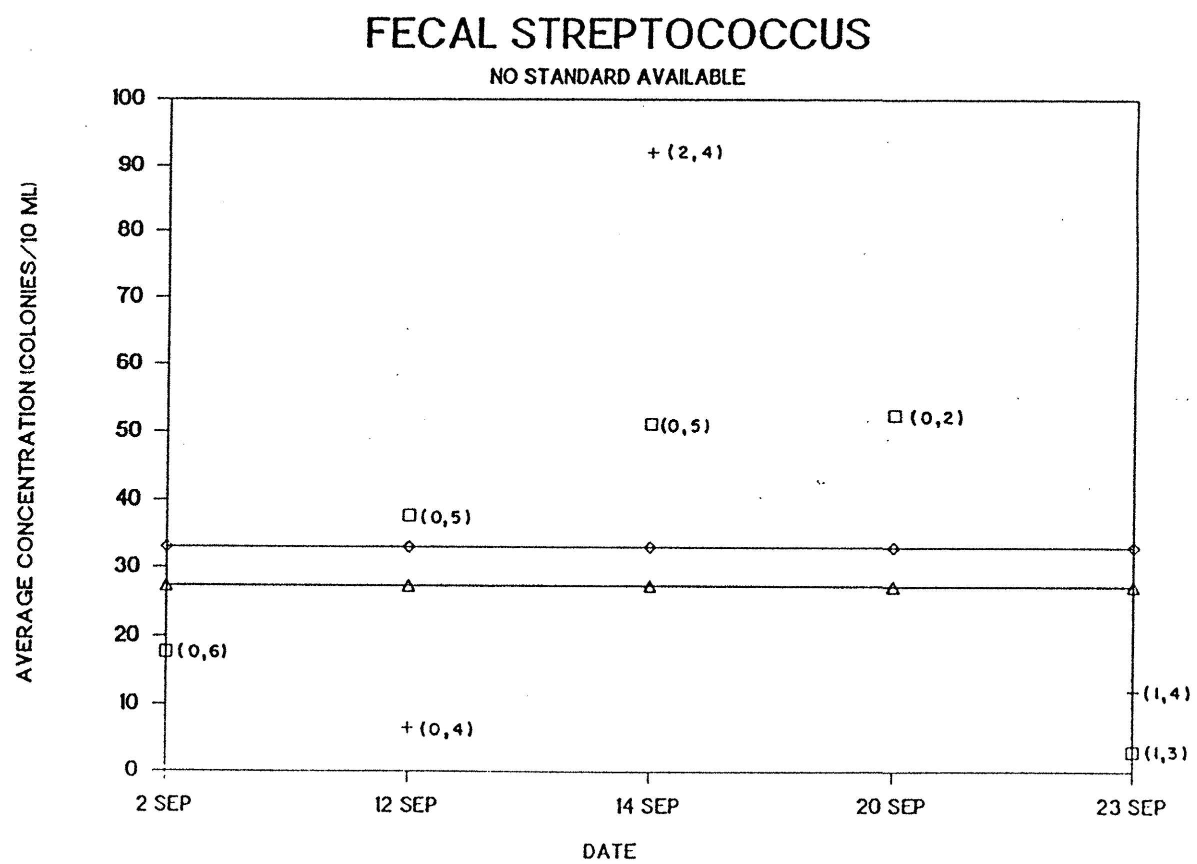


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FIGURE 7.4.3

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The average ratio for the irrigation baseline was 1.1. The average ratio for the storm sewer was 2.7 and the average ratio in the drain during storms was 3.0. The dry weather samples in points 5 and 6 alone had an average ratio of 0.8.

7.4.13 Kjeldahl Nitrogen

Kjeldahl nitrogen is the total of ammonia nitrogen and organic nitrogen. The amount of organic nitrogen may be determined by subtracting the concentration of ammonia as N from the Kjeldahl nitrogen concentration. Organic nitrogen includes natural materials such as proteins and urea and synthetic organic materials as might be found in fertilizers. Kjeldahl nitrogen is not a regulated constituent, but serves to indicate the amount of nutrients in the water source.

Table 7.4.13, Appendix H, Volume II and Figure 7.4.4 illustrate the averages for this constituent.

7.4.14 Lead

Lead is a toxic metal whose sources in water may be from industrial discharges or dissolution of lead plumbing. Automobile exhaust precipitates, tire wear and lubricating oil and grease compounds may contribute lead to the surface of roadways. The livestock water guideline for lead is 0.10 mg/l. The average concentration for both existing storm conditions and the storm drain samples is 0.22 mg/l (see Table 7.4.14, Appendix H, Volume II). Additional concentrations of storm-water should not affect the concentration of lead although it is greater than the guideline (see Figure 7.4.4).

7.4.15 Detergents (MBAS)

MBAS are Methylene-Blue-Active Substances. This test indicates the presence of anionic surfactants which are a component of detergents. Detergents are not regulated.

The average concentration of MBAS for the existing storm condition in the drain is 0.096 mg/l whereas the average concentration for the storm drain samples is 0.060 mg/l (see Table 7.4.15, Appendix H, Volume II). This represents a decrease for future conditions (see Figure 7.4.4).

7.4.16 Nickel

Sources of nickel in water may be precipitates from diesel exhaust, wear of nickel alloys, and asphalt paving. The irrigation guideline for nickel is 0.2 mg/l. Figure 7.4.4 illustrates that all sample results for this project indicated less than 0.2 mg/l of nickel, the limit of detection for the method used for analysis (see Table 7.4.16, Appendix H, Volume II).

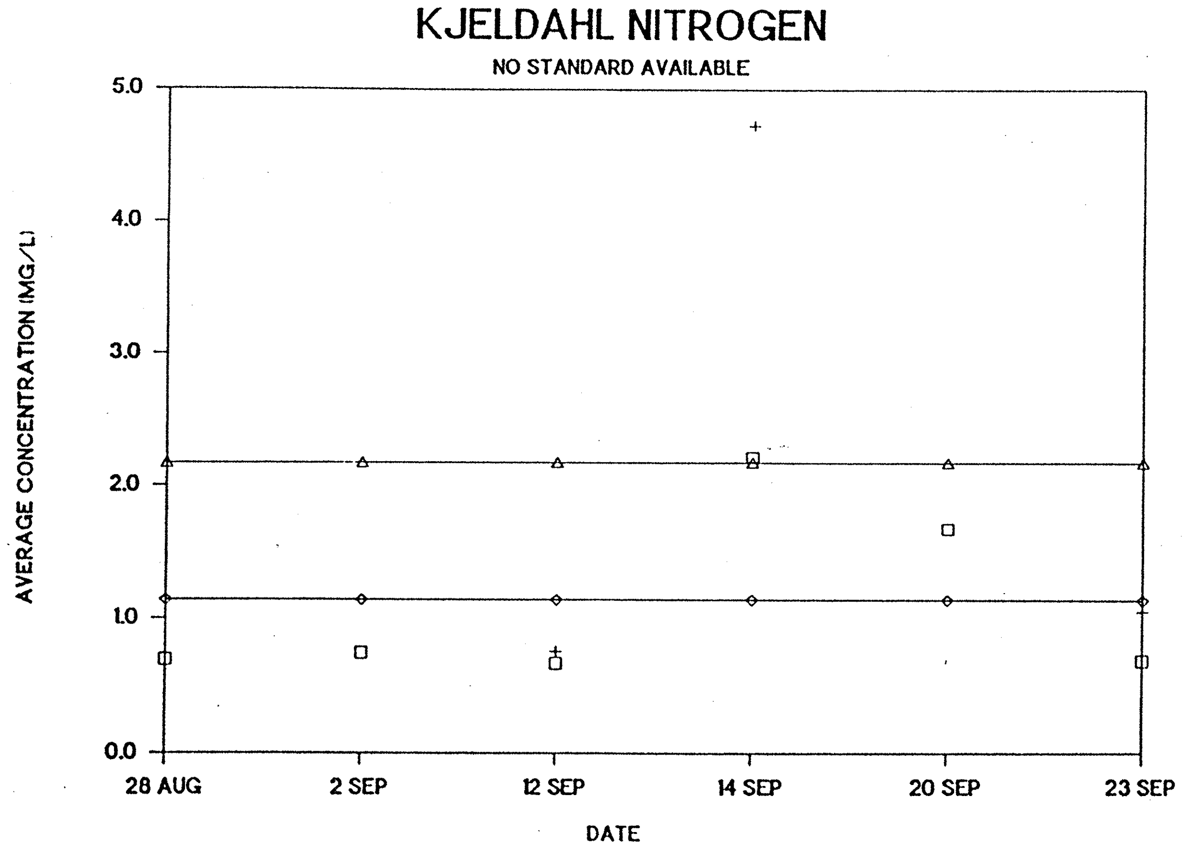
The data suggests that additional stormwater contributions will not exceed the guidelines.

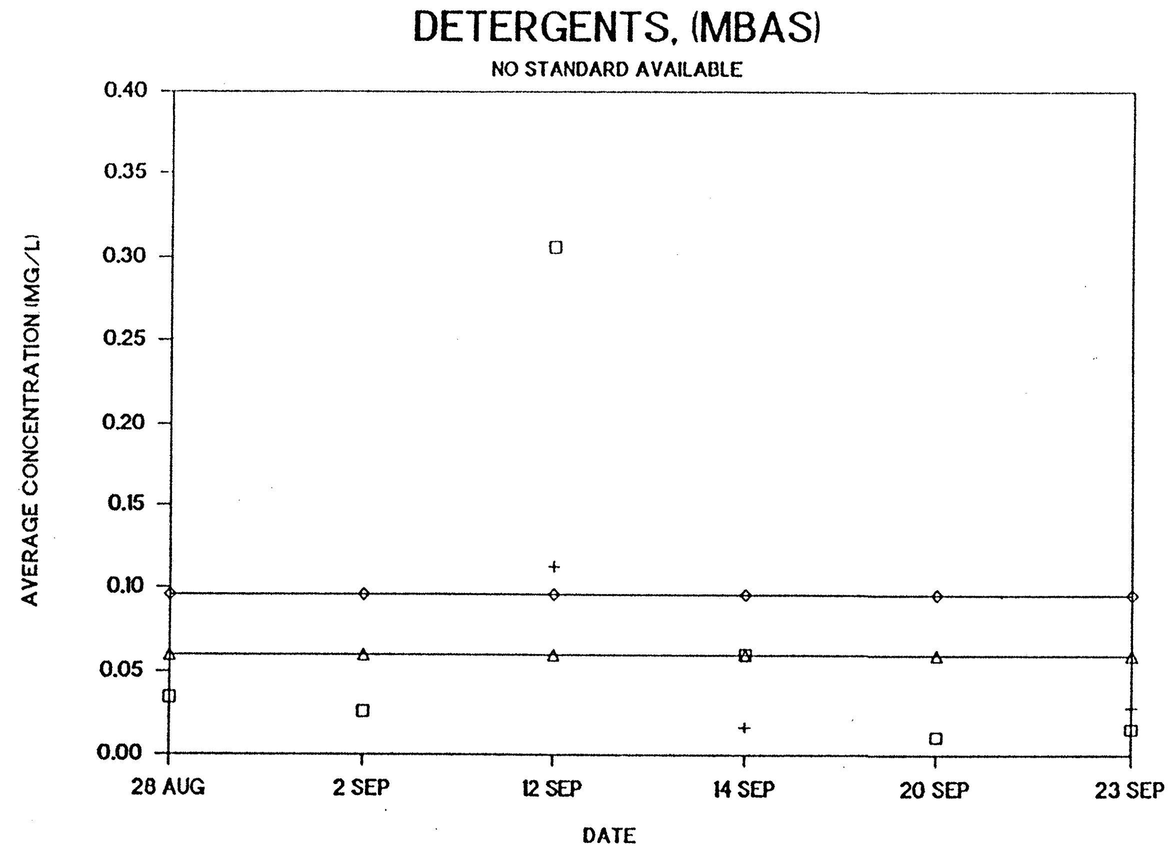
7.4.17 Nitrate

Nitrate is a form of nitrogen formed by decomposition of organic material. The livestock water guideline for nitrate is 100 mg/l. The average concentration for the existing storm condition in the drains is 0.73 mg/l whereas the average concentration for the storm sewers is 1.09 mg/l (see Table 7.4.17, Appendix H, Volume II and Figure 7.4.5). This represents an increase, but the future conditions with additional stormwater should still exhibit a concentration of nitrate less than even the human health standard which is 10 mg/l.

7.4.18 Oil and Grease

Oil and grease sample results indicate the presence of petroleum, animal or vegetable oils and greases. The only standard for oil and grease is that there be none visible on the surface of waters used as water supply sources. It was noted during field data collection that there were no oil or grease sheens visible during the project period. The EPA Water Quality Criteria document states that animal and vegetable oils do not have any toxicity to aquatic life if there are not floating scums or sheens visible. Petroleum oils can be toxic to aquatic life at levels as low as 0.01 mg/l depending on the type of oil and the type of aquatic life in concern. Since the testing conducted in this project





LEGEND

IRRIGATION AND STORM, POINTS 1 THROUGH 6 AVERAGE

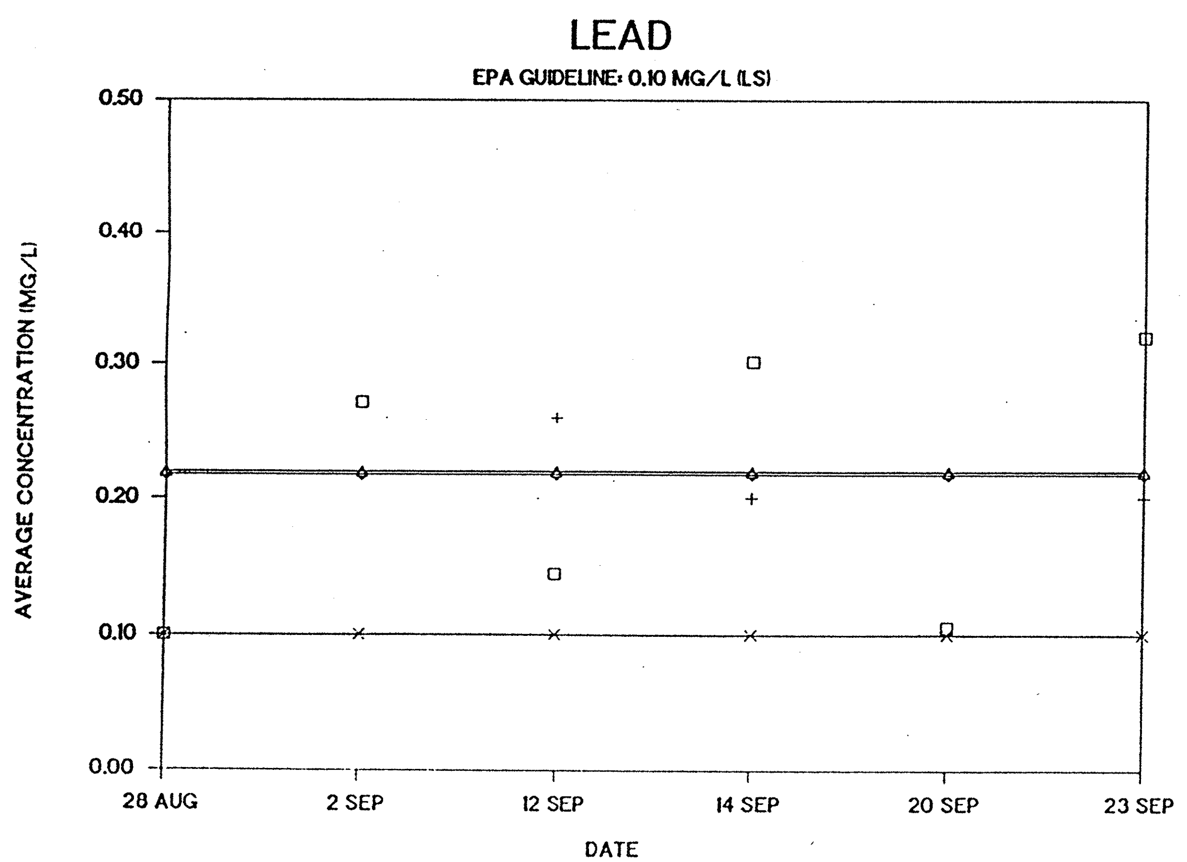
STORM, POINTS 7 AND 8 AVERAGE

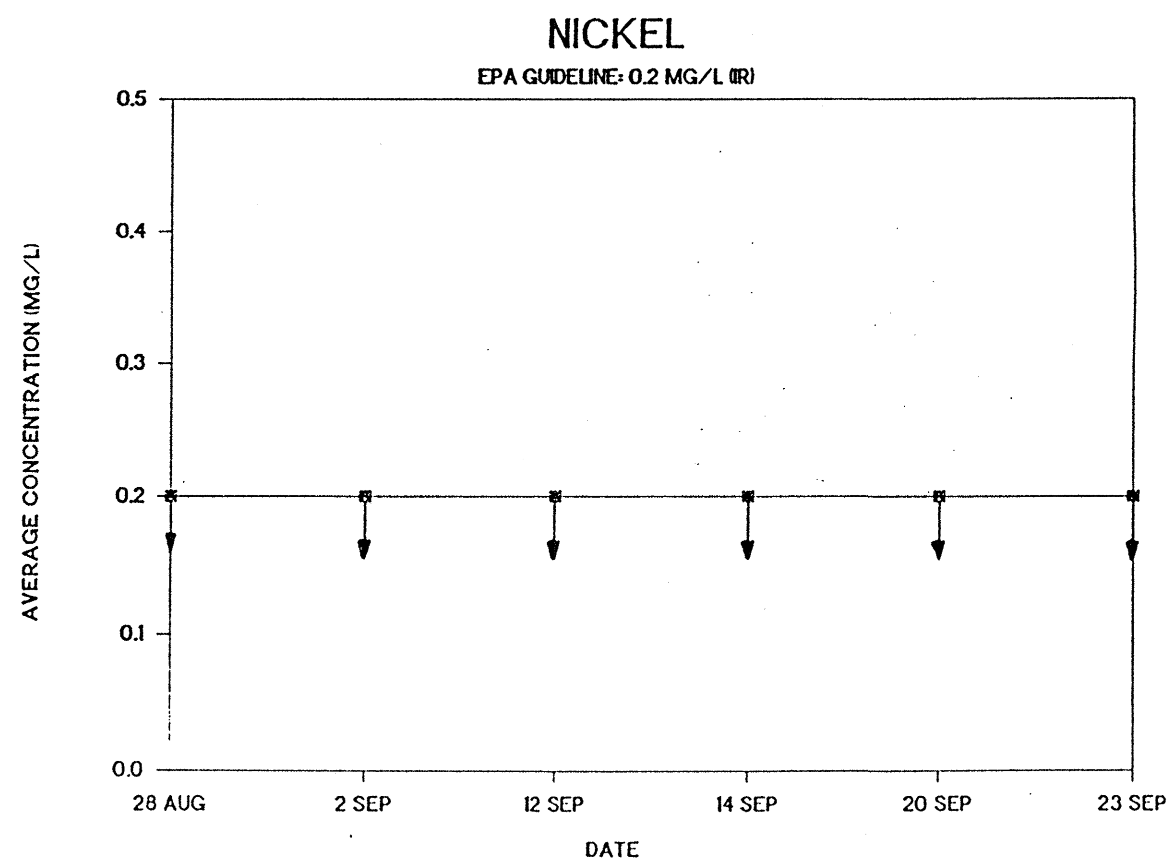
IRRIGATION AND STORM POINTS 1 THROUGH 6, OVERALL AVERAGE

STORM POINTS 7 AND 8, OVERALL AVERAGE

RECOMMENDED GUIDE-LINE/WQCC STANDARD, UPPER LIMIT

RECOMMENDED GUIDE-LINE/WQCC STANDARD, LOWER LIMIT





ALAMEDA/RIVERSIDE DRAINS
WATER QUALITY
SUMMARY OF COLLECTED DATA

FIGURE 7.4.4

did not differentiate between types of oils and grease, further study may be warranted in this area.

The average concentration for existing storm conditions in the drains is 20.7 mg/l whereas the average concentration for the storm sewers is 31.8 (see Table 7.4.18 and Appendix H, Volume II, and Figure 7.4.5). This represents an increase in the oil and grease concentrations for future conditions which may or may not be detrimental to aquatic life.

7.4.19 pH

The stream standard for the Rio Grande near Albuquerque for pH is 6.0 to 9.0. Both the average existing storm conditions in the drain and the average of the storm sewer samples fall within this range (see Table 7.4.19, Appendix H, Volume II and Figure 7.4.5), indicating additional stormwater contributions should not adversely affect pH.

7.4.20 Phenols

Phenols are derivatives of benzene and may be found in wastewaters, natural waters and water supplies. The presence of phenols in water supply sources may cause objectionable odors and tastes when the water is chlorinated. The standard for water supply sources is 0.005 mg/l. The recommended guideline for human health is 3.5 mg/l.

The average concentration for existing storm conditions is 0.038 mg/l whereas the average concentration for the storm sewers is 0.032 (see Table 7.4.20, Appendix H, Volume II).

This represents a decrease in phenols for future conditions (see Figure 7.4.5).

7.4.21 Ortho-phosphate

Ortho-phosphates are used as fertilizers and enter water courses during runoff events. There are not standards for ortho-phosphates. The average concentration for the existing storm flows in the drains is 0.98 mg/l whereas the average concentration for the storm sewers is 0.56 mg/l (see Table 7.4.21, Appendix H, Volume II). This represents a decrease for future conditions (see Figure 7.4.6).

7.4.22 Silver

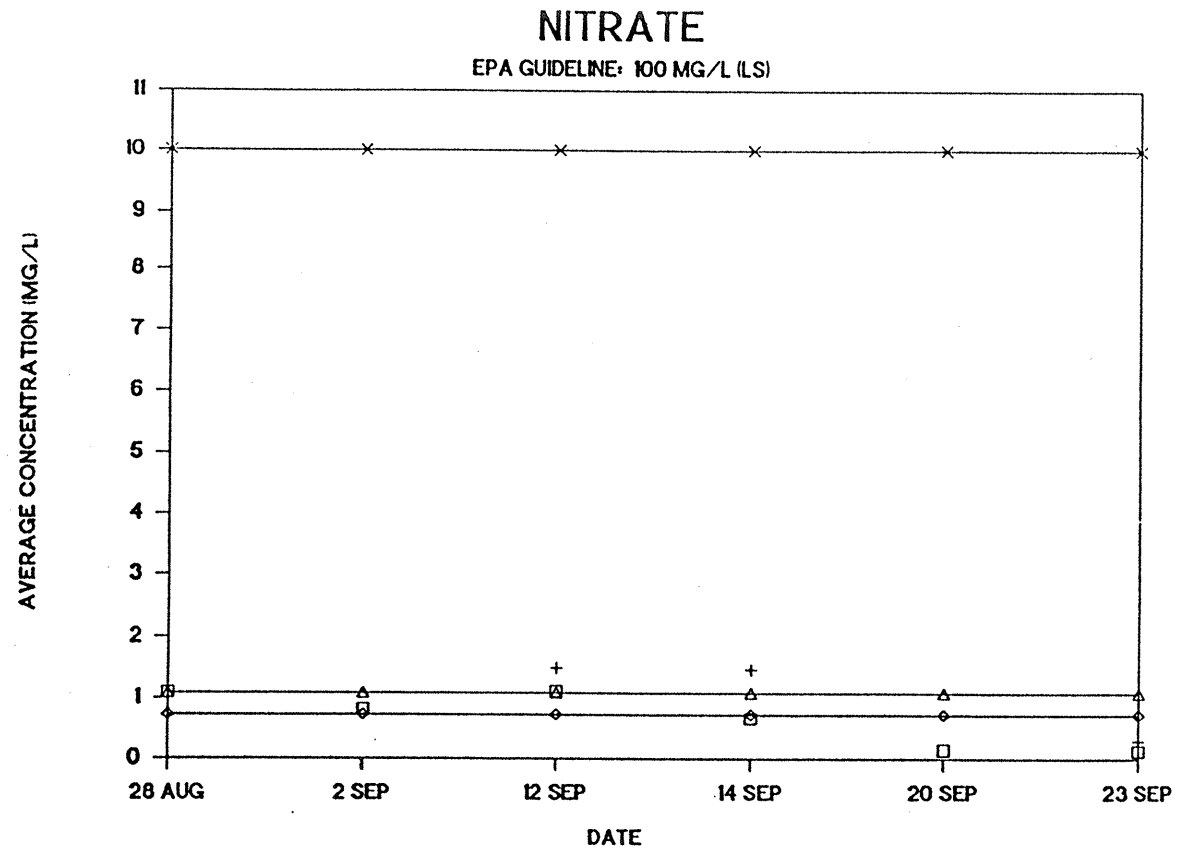
Silver is a metal which can enter water courses by discharges from photographic or electronic industries. The only available standard for silver is the human health standard of 0.05 mg/l. The average concentrations for existing storm conditions in the drains is 0.05 mg/l whereas the average concentration for the storm sewer samples is 0.03 mg/l (see Table 7.4.22, Appendix H, Volume II). This represents a decrease to below the human health standard for future conditions with additional stormwater contributions (see Figure 7.4.6).

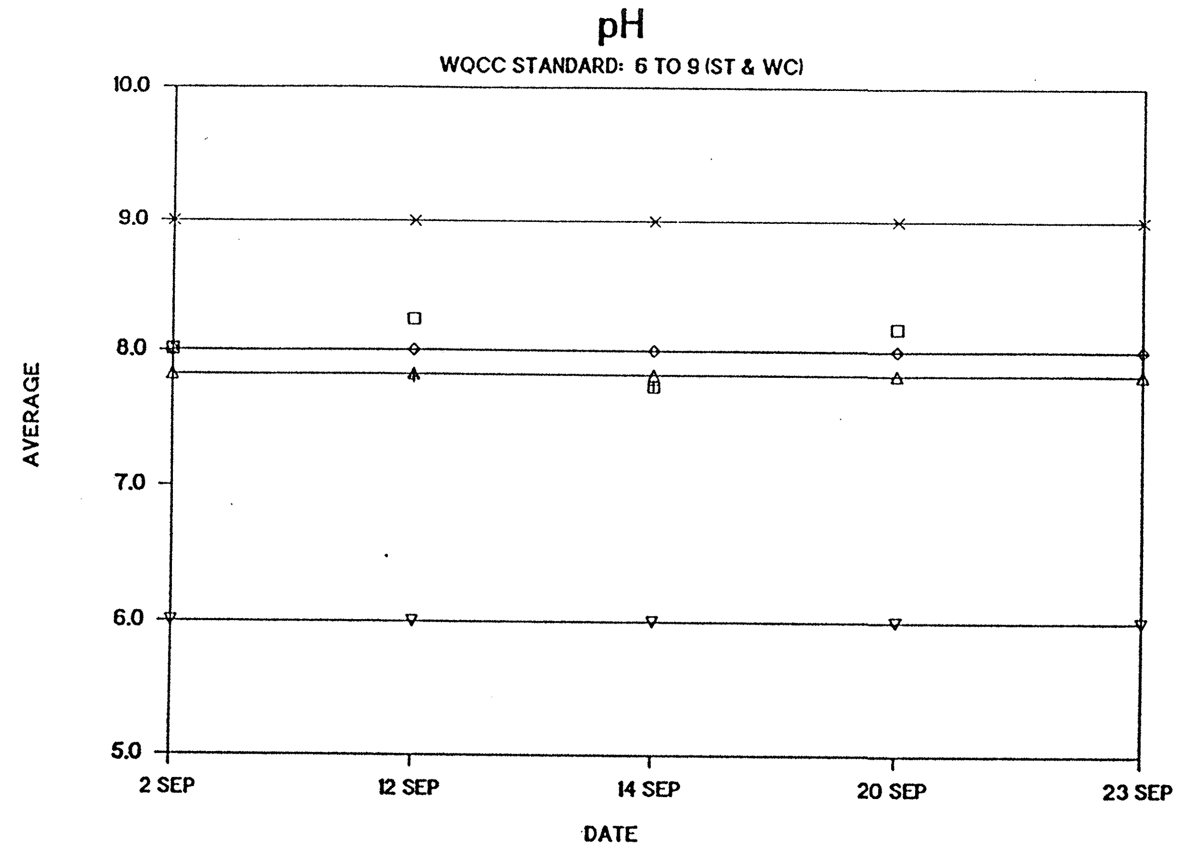
7.4.23 Temperature

The stream standard for temperature in the Rio Grande near Albuquerque is 32.2°C or 90°F. The average temperature for the existing storm condition in the drains is 18.7°C (see Table 7.4.23, Appendix H, Volume II and Figure 7.4.6). Temperature was not taken in the storm sewer samples due to the time lapse between sample collection and sample removal.

7.4.24 Total Chlorine

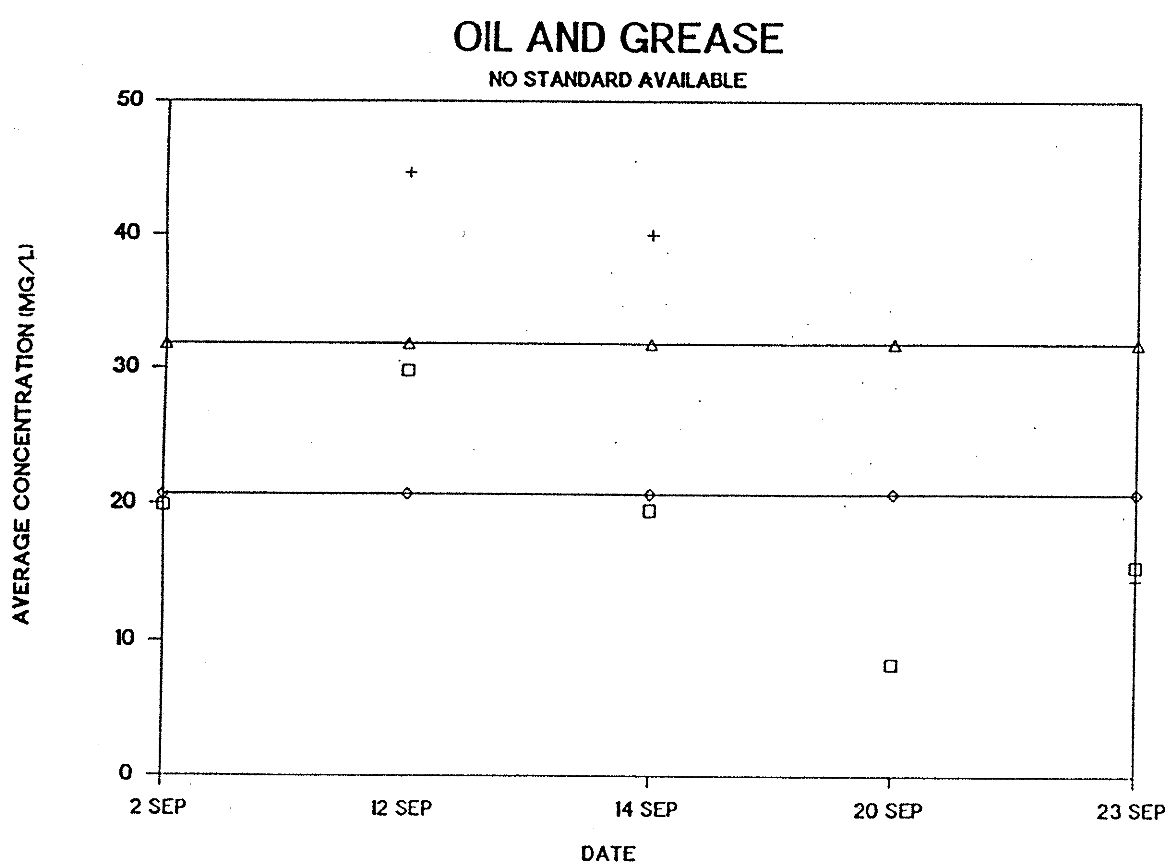
The presence of chlorine in a water source can indicate contamination by chlorinated sewage effluent, discharge of municipal drinking water, or discharge from swimming pools. The recommended guideline for chlorine with regard to aquatic life is a 4-day average of 0.011 mg/l. The concentration for existing storm flows in the drains is 0.06 mg/l whereas the average concentration for the storm sewer samples is 0.03 mg/l (see Table 7.4.24, Appendix H, Volume II). Figure 7.4.6 shows that the averages for all of the samples obtained exceeded these guidelines.

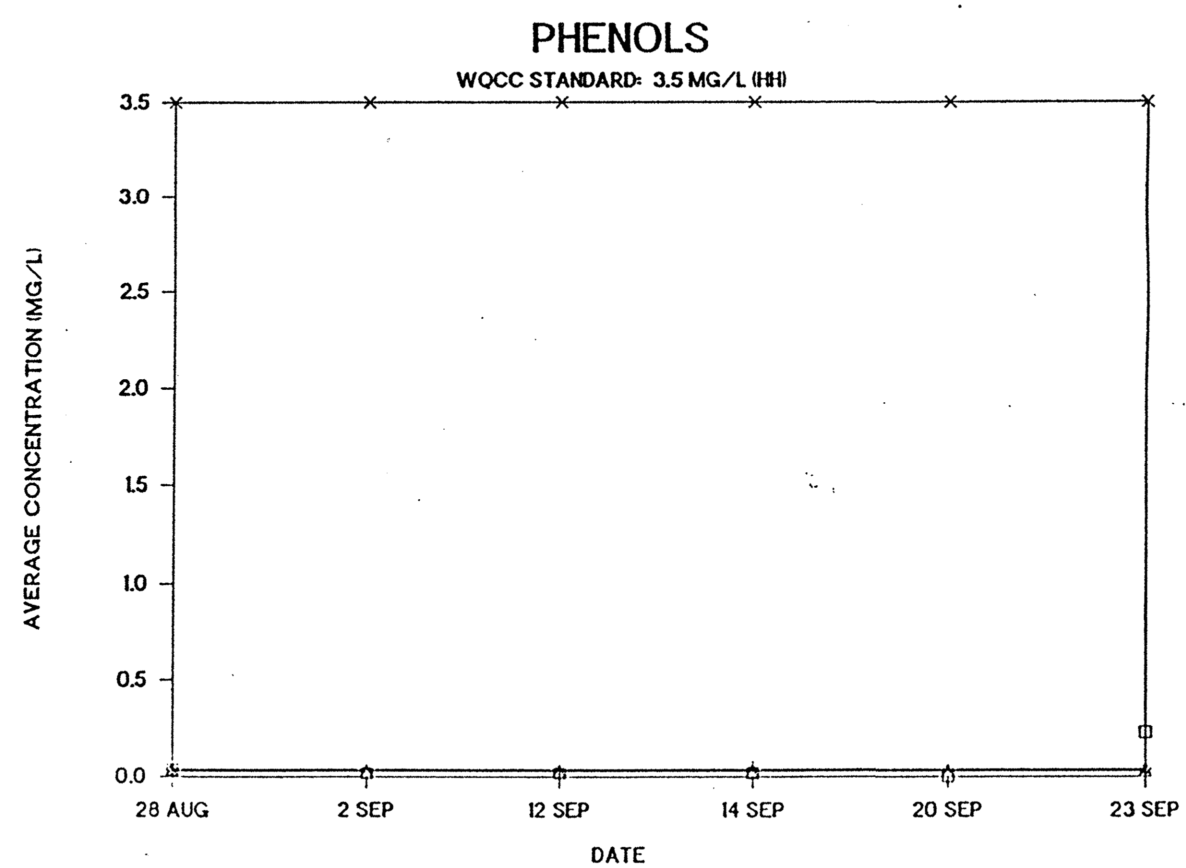




LEGEND

- IRRIGATION AND STORM, POINTS 1 THROUGH 6 AVERAGE □
- STORM, POINTS 7 AND 8 AVERAGE +
- IRRIGATION AND STORM POINTS 1 THROUGH 6, OVERALL AVERAGE —◇—◇—
- STORM POINTS 7 AND 8, OVERALL AVERAGE —△—△—
- RECOMMENDED GUIDE-LINE/WQCC STANDARD, UPPER LIMIT —×—×—
- RECOMMENDED GUIDE-LINE/WQCC STANDARD, LOWER LIMIT —▽—▽—





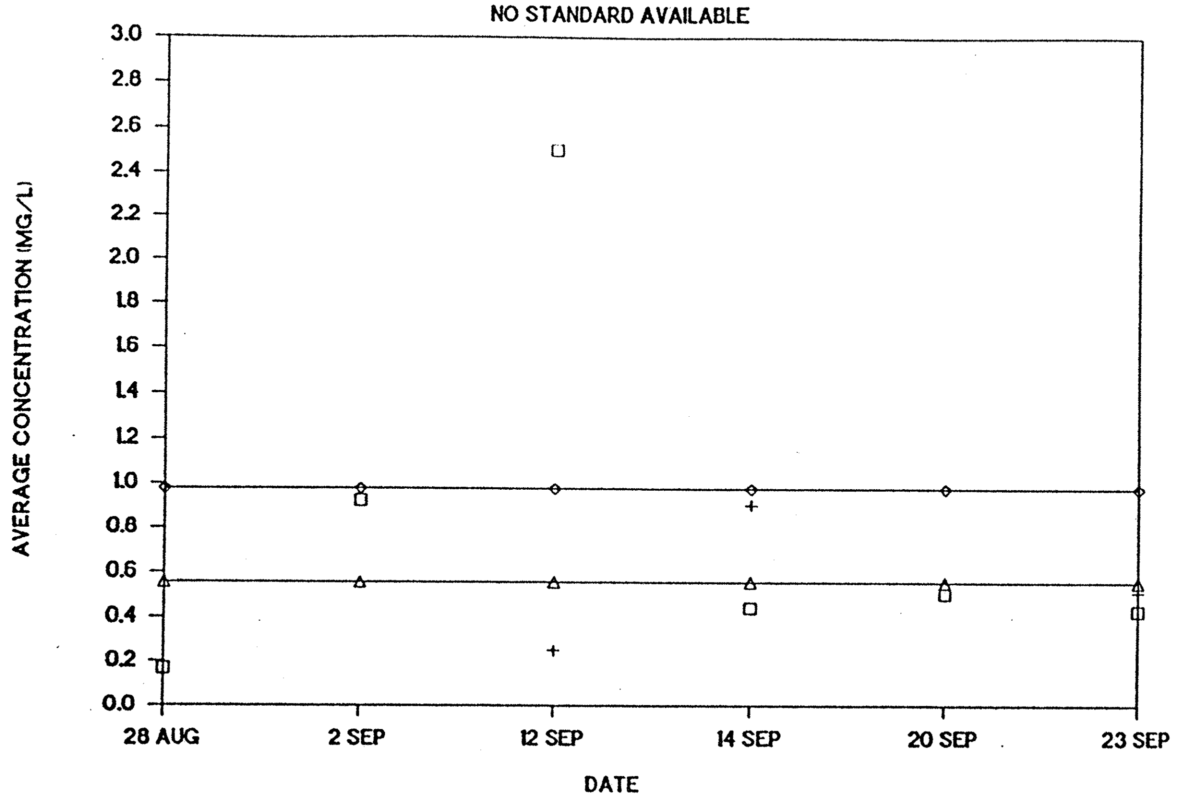
ALAMEDA/RIVERSIDE DRAINS
WATER QUALITY
SUMMARY OF COLLECTED DATA

FIGURE 7.4.5

LEEDSHILL-HERKENHOFF

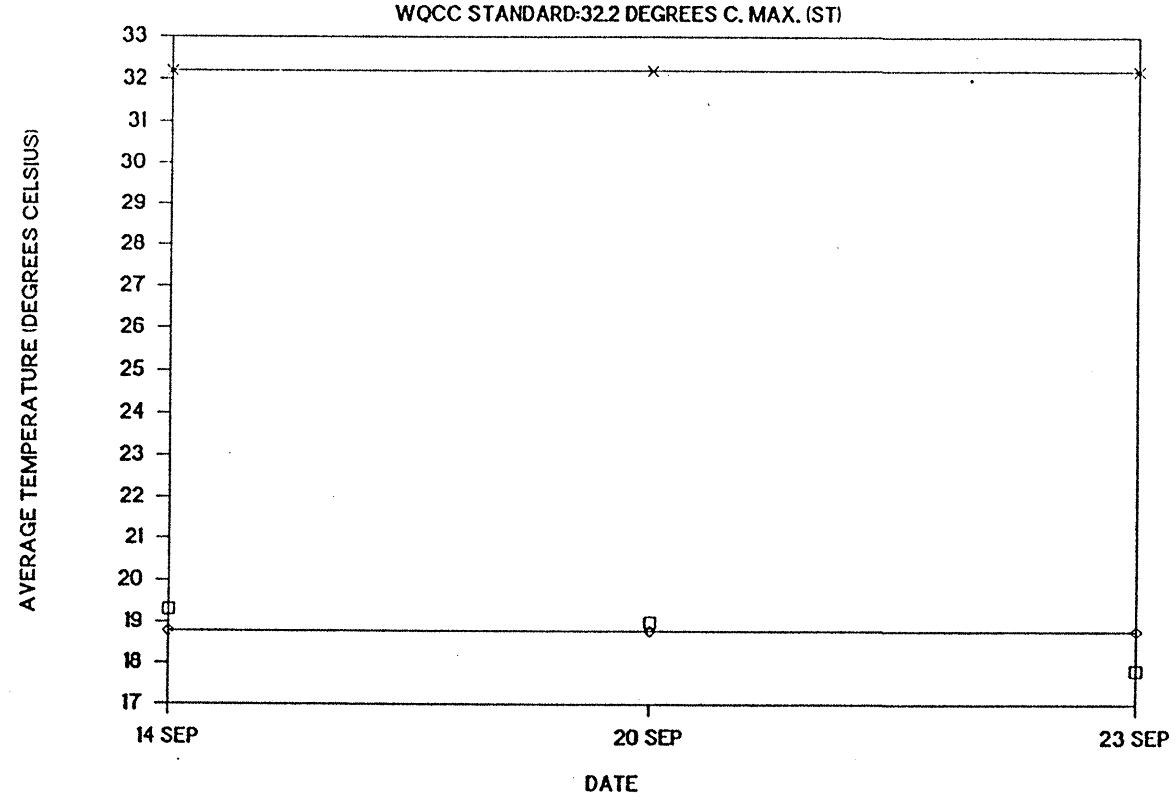
ORTHO-PHOSPHATE

NO STANDARD AVAILABLE



TEMPERATURE

WQCC STANDARD:32.2 DEGREES C. MAX. (ST)



LEGEND

IRRIGATION AND STORM, POINTS 1 THROUGH 6 AVERAGE □

STORM, POINTS 7 AND 8 AVERAGE +

IRRIGATION AND STORM POINTS 1 THROUGH 6, OVERALL AVERAGE —◇—◇—

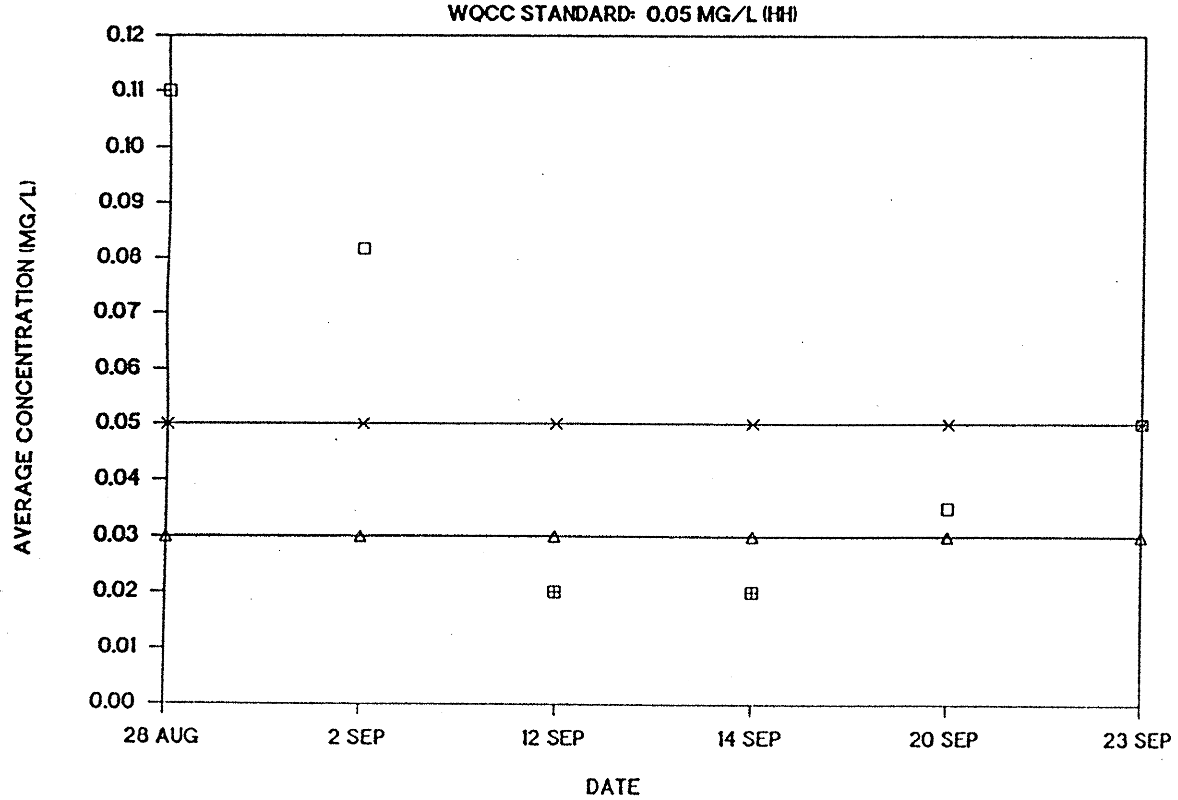
STORM POINTS 7 AND 8, OVERALL AVERAGE —△—△—

RECOMMENDED GUIDE-LINE/WQCC STANDARD, UPPER LIMIT —×—×—

RECOMMENDED GUIDE-LINE/WQCC STANDARD, LOWER LIMIT —▽—▽—

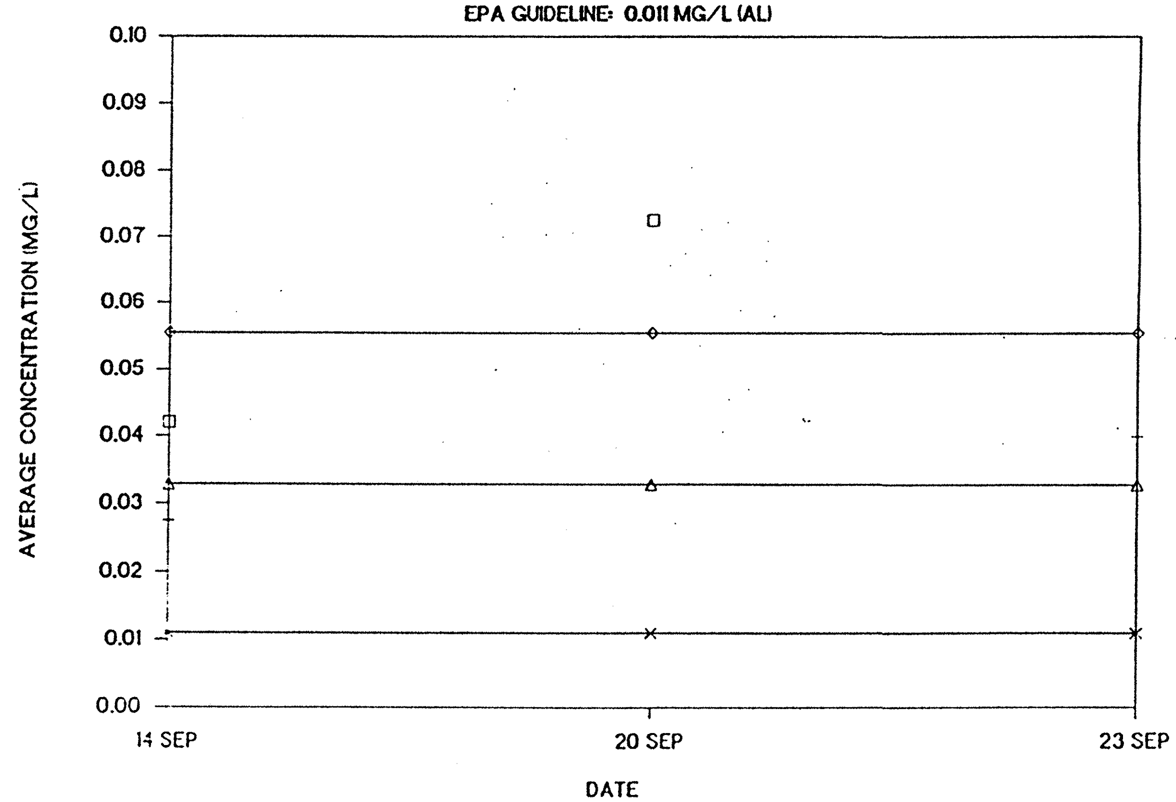
SILVER

WQCC STANDARD: 0.05 MG/L (HH)



TOTAL CHLORINE

EPA GUIDELINE: 0.011 MG/L (AL)



ALAMEDA/RIVERSIDE DRAINS
WATER QUALITY
SUMMARY OF COLLECTED DATA

LEEDSHILL-HERKENHOFF

FIGURE 7.4.6

Further investigation is recommended to verify the results of this study and/or to determine the source or sources of chlorine to the drain system if aquatic life is of concern in the drain, however, it does not appear that the contribution of additional stormwater to the drains will significantly alter the averages for this parameter.

7.4.25 Total Dissolved Solids

Total dissolved solids (TDS) is a measure of the quantity of all dissolved ions in a water. The irrigation guideline for TDS is 1000 mg/l. The average concentration for storm flows in the drain is 264 mg/l whereas the average concentration for the storm sewer samples is 149 mg/l (see Table 7.4 25, Appendix H, Volume II and Figure 7.4.7). This represents a decrease in TDS for future conditions with additional stormwater in the drains.

7.4.26 Total Suspended Solids

Total suspended solids (TSS) is a measure of the quantity of undissolved solids (sediment) contained in the flow. Total suspended solids is not regulated. The average concentration for existing storm conditions in the drains is 1086 mg/l whereas the average concentration for the storm sewer samples is 576 mg/l (see Table 7.4.26, Appendix H, Volume II). This represents a decrease in TSS for future conditions with additional stormwater in the drain (see Figure 7.4.7).

7.4.27 Total Organic Carbon

Total organic carbon is a measure of total organic content in the water. It is different than BOD or COD in that BOD and COD do not reflect total organic carbon since some carbon compounds cannot be oxidized. TOC can be an indicator of discharges from industrial activities. TOC has no standard or guideline.

The average concentration for existing storm conditions in the drains is 10.30 mg/l whereas the average concentration for the storm sewer sample is 12.40 mg/l (see Table 7.4.27, Appendix H, Volume II). This represents an increase in total organic carbon (see Figure 7.4.7).

7.4.28 Volatile Organic Carbon

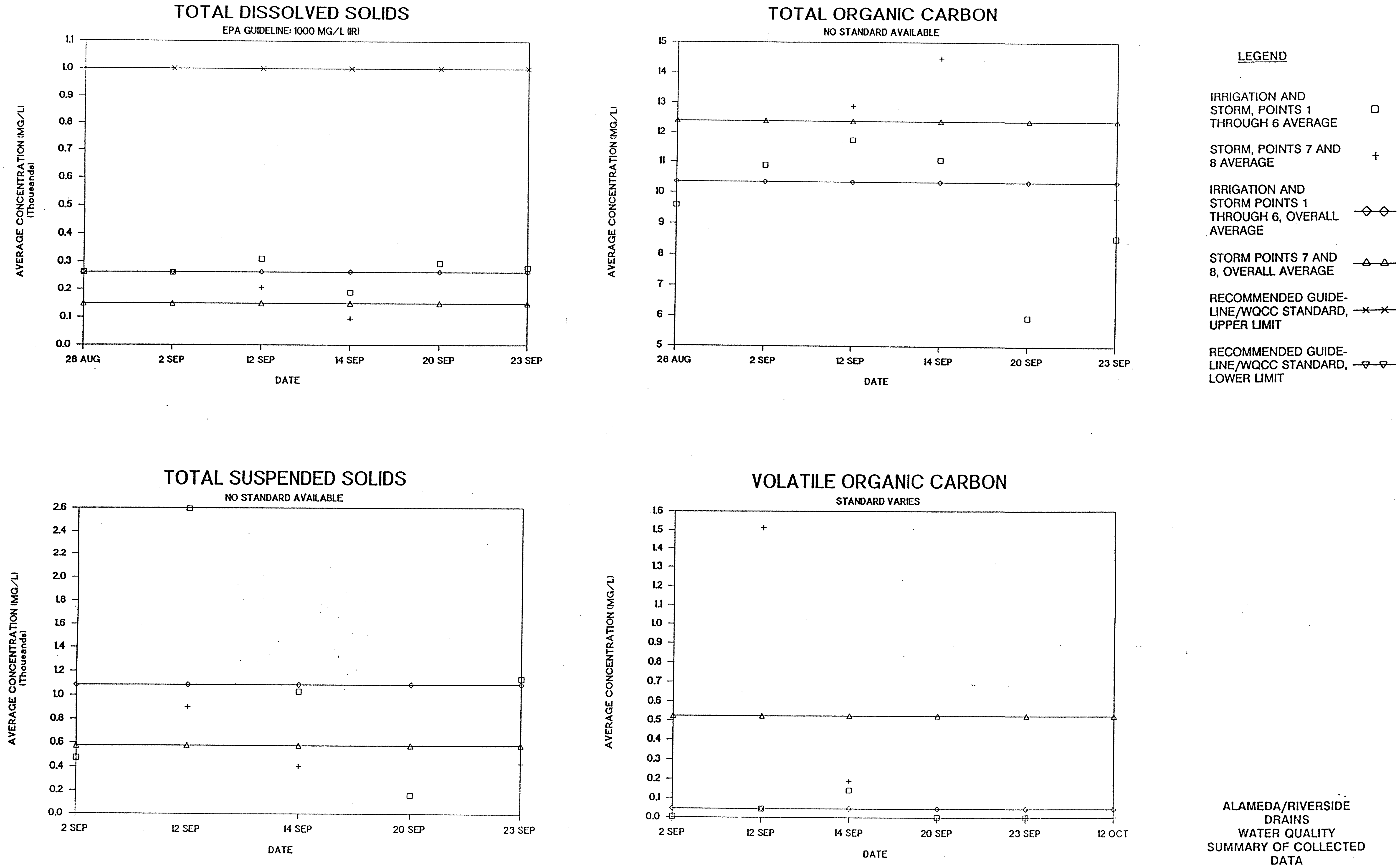
Volatile organic carbon (VOC) is used as an indicator of the presence of solvents, gasoline or other organic compounds. VOC is not a regulated parameter but a screen for compounds that may be regulated as well as many compounds which are not regulated. Detection of VOC in water warrants further testing for specific compounds which can then be compared with applicable standards. VOC was included as a parameter for this project since testing for the many specific organic pollutants is very expensive.

The average concentration for existing storm conditions in the drains is 0.046 mg/l whereas the average concentration for the storm sewer samples is 0.525 mg/l (see Table 7.4.28, Appendix H, Volume II and Figure 7.4.7). This is a significant increase which could require regulatory management of any additional stormwater contributions to the drains, if stormwater discharges become regulated in the future. Whether or not such management will be required depends on the levels of any standards which may be established for these compounds.

7.4.29 Zinc

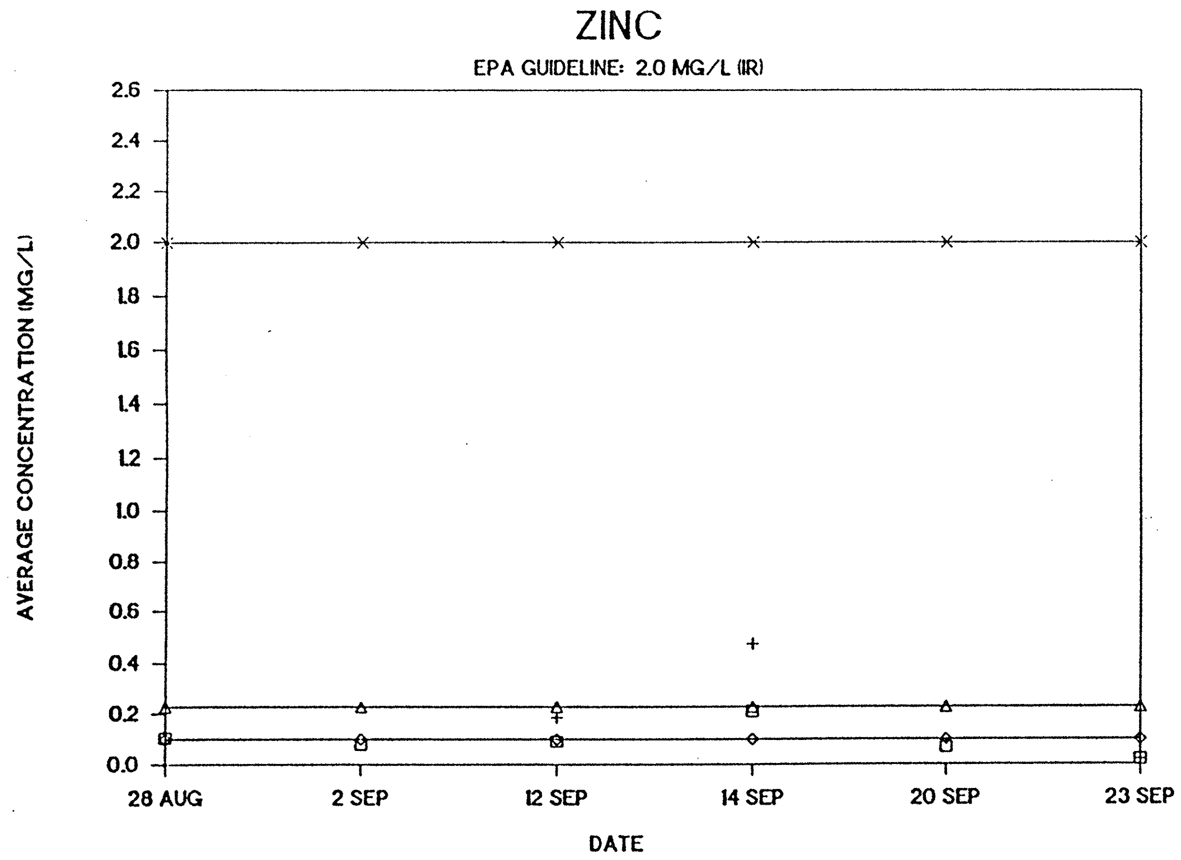
Zinc is an essential element which can enter a water source through deterioration of galvanized piping, industrial wastes, or automobile grease, tire wear or motor oil.

The irrigation water guideline for zinc is 2.0 mg/l (see Table 7.4 29, Appendix H, Volume II). The average concentration for existing storm conditions in the drains is 0.10 mg/l whereas the average concentration for the storm sewer samples is 0.23 mg/l. This represents an increase in zinc concentration for future conditions with additional stormwater in the drains (see Figure 7.4.8), but the concentration will still be below the guideline.



ALAMEDA/RIVERSIDE DRAINS
WATER QUALITY
SUMMARY OF COLLECTED DATA

FIGURE 7.4.7



LEGEND

IRRIGATION AND STORM, POINTS 1 THROUGH 6 AVERAGE □

STORM, POINTS 7 AND 8 AVERAGE +

IRRIGATION AND STORM POINTS 1 THROUGH 6, OVERALL AVERAGE

STORM POINTS 7 AND 8, OVERALL AVERAGE

RECOMMENDED GUIDELINE/WQCC STANDARD, UPPER LIMIT

RECOMMENDED GUIDELINE/WQCC STANDARD, LOWER LIMIT

ALAMEDA/RIVERSIDE DRAINS
WATER QUALITY
SUMMARY OF COLLECTED DATA

FIGURE 7.4.8

7.5 Water Quality Impacts

Table 7.5.1 contains a summary of the water quality comparisons discussed in detail in Section 7.4, including the guideline for each parameter, the average for existing storm conditions in the drains, and the average for the storm sewer samples. Table 7.5.2 summarizes the water quality impacts of introducing additional stormwater into the Alameda and Riverside Drains based on the results of the water sampling program conducted for this study. Of the 29 parameters studied, no apparent change is indicated for eight parameters. Apparent positive impacts are indicated for twelve parameters, and apparent negative impacts are indicated for nine parameters. Of the nine parameters for which apparent negative impacts are indicated, only chemical oxygen demand (COD) exceeds a selected guideline for future conditions. If stormwater discharges do not become regulated, the introduction of additional municipal stormwater into the drains will not significantly impact the existing water quality, based on the data collected in this study.

At the present time stormwater discharges are not regulated. If in the future, however, stormwater discharges become regulated, it may be necessary to mitigate these factors which contribute to negative water quality impacts to help meet any standards which may be imposed. Elevated values for metals were noted during some storm flows in the drains. These elevated values do not appear to represent continuous discharges but instead, the possibility of isolated indiscrete or accidental discharges.

Table 7.5.1 Water Quality Comparisons

Parameters	Guideline (mg/l)	Existing Storm Flows in Drains (mg/l)	Storm Drain Flows (mg/l)
Ammonia	1.09	0.25	0.37
BOD-5	30	4.84	14.23
Cadmium	0.05	0.004	<0.02
Chloride	250	10.7	5.6
Chromium	1.0	0.03	0.02
COD	125	34	150
Conductivity	750	408	174
Copper	0.024	0.03	0.02
Cyanide	0.022	0.006	0.006
DO	>4.0	7.2	8.46
Fecal Coliform	2000/100ml	>2000/100ml	>2000/100ml
Fecal Strep.	-	-	-
TKN	-	1.13	2.18
Lead	0.1	0.22	0.22
Detergent (MBAs)	-	0.096	0.060
Nickel	0.2	<0.2	<0.2
Nitrate	100	0.73	1.09
Oil & Grease	-	20.7	31.8
pH	6.0-9.0 units	8.01 units	7.83 units
Phenols	3.5	0.038	0.032
Ortho-Phosphate	-	0.98	0.56
Silver	0.05	0.05	0.03
Temperature	32.2°C	18.7	-
Total Chlorine	0.011	0.06	0.03
TDS	1000	264	149
TSS	-	1085	576
TOC	-	10.35	12.40
VOC	-	0.046	0.525
Zinc	2.0	0.10	0.23

Table 7.5.2 Water Quality Impacts

No Apparent Change

Cadmium - Existing and future less than guideline
Cyanide - Existing and future less than guideline
Fecal Coliform - Existing and future greater than guideline
Fecal Strep. - No guideline
Lead - Existing and future greater than guideline
Nickel - Existing and future less than guideline
pH - Existing and future within guideline
Temperature - Existing and future less than guideline

Apparent Positive Impact

Chloride - Existing and future less than guideline
Chromium - Existing and future less than guideline
Conductivity - Existing and future less than guideline
Copper - Existing greater than guideline, future less than guideline
Dissolved Oxygen - Existing and future greater than guideline
Detergents - No guideline
Phenols - Existing and future less than guideline
Ortho-Phosphate - No guideline
Silver - Existing at guideline, future less than guideline
Total Chlorine - Existing and future greater than guideline
TDS - Existing and future less than quideline
TSS - No guideline

Apparent Negative Impact

Ammonia - Existing and future less than guideline
BOD-5 - Existing and future less than guideline
COD - Existing less than guideline, future greater than guideline
TKN - No guideline
Nitrate - Existing and future less than guideline
Oil and Grease - No guideline
TOC - No guideline
VOC - No guideline
Zinc - Existing and future less than guideline

8.0 ENVIRONMENTAL IMPACTS AND MITIGATION

8.1 Summary of Projected Environmental Impacts

An assessment of environmental impacts of the proposed Alameda/Riverside Drain Project is presented in a separately bound report, Volume III. Environmental impacts are outlined in Table 8.1 for improvement concepts; on-line detention, off-line detention, and stormwater pumping facilities. Overall, impacts determined are not of a significant nature and constitute only minor changes between existing conditions and projected conditions with project implementation.

Among the various impacts associated with the project, several are expected to be of public concern. In particular, drain widening will reduce the available recreation area in the existing right-of-way and easements for development of bike, walking, and equestrian trails and associated recreation amenities. Additionally, drain widening will increase widths of flow, although only temporarily during periods of stormwater runoff. This may also increase the public safety hazard associated with the drain system although hazards are reduced elsewhere within the contributing watershed. Among adverse impacts of lesser concern, the use of the stormwater pumping facilities exclusively along the Riverside Drain could result in the destruction of a minor proportion of sport fishes which remain in the drain environment during the rainfall season. The Alameda Drain is considered to support a very limited amount of sport fishes, as it does not experience annual stocking. Utilization of concrete lining along the lower Alameda Drain could also be of some public concern as it would result in a reduction of wildlife habitat, and is aesthetically unnatural looking.

With these environmental impacts in mind, the seven alternate improvement scenarios identified in Chapter 5.0, are rated. Alternate Scenario Nos. 1, 3, and 5 are somewhat less preferable than the remaining four scenarios, as they require widening of both the Alameda and Riverside Drains, restricting available undeveloped recreation area and slightly increasing public safety hazard specifically along the drains, and they incorporate concrete lining. Alternate Scenario Nos. 2, 4, and 7 are moderately preferable on an environmental basis, as they result in less adverse impact in the Riverside Drain, but incorporate concrete lining along the lower Alameda Drain to its confluence with the Riverside Drain to its confluence with the Riverside Drain. Scenario No. 2 includes use of the Alcalde Pump Station to divert Riverside Drain stormwater to the river. Alternate Scenario No. 6, utilizing detention basins, is considered most preferable on an environmental impact basis as available recreation area is not restricted within the rights-of-way, public safety hazards are essentially not increased over existing conditions, and adversities associated with concrete lining do not arise. However, as final sites are not identified for detention basins, some environmental adversities could be subsequently defined for this scenario.

All seven alternate improvement scenarios are considered to have minor adverse environmental impacts which are outweighed by advantages each exhibits in effectively addressing the purpose and need of the proposed project. Distinction between scenarios for rating purposes was based on differences drawn between these defined minor adverse impacts.

Table 8.1 Summary of Projected Environmental Impacts with Improvement Concepts

Environmental Component
    Improvement Concepts
        Projected Impacts

A. Climate and Air Quality
    1. On-line Detention, Off-line Detention, and Stormwater Pumping Facilities
        - temporary, minor decrease in air quality during construction

B. Topography
    1. On-line Detention and Off-line Detention
        - minor change attributable to excavation
    2. Pump Stations
        - essentially no change

C. Geology
    1. On-line Detention, Off-line Detention, and Stormwater Pumping Facilities
        - essentially no change

D. Surface Water
   1. On-line Detention
      - increase stormwater delivery to river
      - increase total sediment delivered to the drain system
      - increase in maintenance expense for sediment removal
   2. Off-line Detention
      - increase stormwater delivery to the river
      - increase total sediment delivery to the drain system
      - increase in maintenance expense for sediment removal
   3. Stormwater Pumping Facilities
      - increase stormwater delivery to the river
      - increase total sediment delivery to the drain system
      - increase in maintenance expense for sediment removal
      - introduction of additional flow into river at upstream locations from drain outlet

E. Ground Water
   1. On-line Detention
      - wider channel bottom provides for increased ground water recharge
   2. Off-line Detention
      - provide opportunity for ground water recharge
   3. Stormwater Pumping Facilities
      - reduces opportunity for ground water recharge

F. Land Use and Socioeconomics
   1. On-line Detention
      - little or no additional land purchase required
      - increased traffic service on formerly flooded street sections
      - increase in land value, and inducement for development
      - increased safety hazard during stormwater runoff periods
      - reduction in flood damage
      - beneficial expenditures in local economy during construction
      - increase employment opportunity during construction
   2. Off-line Detention
      - need additional land purchase
      - increase traffic service on formerly flooded street sections
      - increase in land value and inducement for development
      - reduction in flood damage
      - beneficial expenditures in local economy during construction
      - increased employment during construction
   3. Stormwater Pumping Facilities
      - may need additional right-of-way or easement for new lines and stations
      - increased traffic service on formerly flooded street sections
      - increase in land value, and inducement for development
      - reduction in flood damage
      - beneficial expenditures in local economy during construction
      - increased employment during construction

G. Fish and Wildlife
   1. On-line Detention
      - no fish displacement
   2. Off-line Detention
      - possible minor entrapment of aquatic life at ponds along Alameda Drain upon stormwater release
   3. Stormwater Pumping Facilities
      - some loss of fish and invertebrates during periods of operation

H. Threatened and Endangered Species
   1. On-line Detention, Off-line Detention, and Stormwater Pumping Facilities
      - No adverse impact

I. Vegetation
   1. On-line Detention
      - temporary disturbance of vegetative communities during construction
      - disturbance of cottonwood root systems in proximity to drain system during construction
      - elimination of wildlife habitats in concrete lined areas
   2. Off-line Detention
      - temporary disturbance of vegetation communities in drain right-of-way during construction
   3. Stormwater Pumping Facilities
      - essentially no adverse impact

J. Soils
   1. On-line Detention
      - temporary soil/slope instability during construction
   2. Off-line Detention
      - likely disturbance of natural soil horizons as a result of excavation
   3. Stormwater Pumping Facilities
      - essentially no impacts

K. Recreation
   1. On-line Detention
      - reduction of available recreation land area within right-of-way
   2. Off-line Detention
      - opportunity for pond areas to be used for recreation grounds
   3. Stormwater Pumping Facilities
      - essentially no impact

L. Noise
   1. On-line Detention and Off-line Detention
      - temporary increase in noise levels during construction

2. Stormwater Pumping Facilities
   - minor increase in ambient noise during operating periods, but within City noise ordinance criteria

M. Cultural Resources
   1. On-line Detention
      - may need archaeological monitoring during construction
      - no adverse impact on known resources
   2. Off-line Detention
      - need additional survey prior to construction
      - no adverse impact on known resources
   3. Stormwater Pumping Facilities
      - essentially no additional survey necessary
      - no adverse impact on known resources

8.2 Mechanisms for Environmental Mitigation

Assessment of environmental impacts associated with proposed discharge of additional stormwater to the Alameda/Riverside Drain system has been outlined.

Mitigation measures are proposed to reduce or eliminate certain avoidable adverse impacts associated with the construction and implementation of drain improvements. The following measures will be required for project implementation, and where they relate to construction, will be made a part of the construction contract specifications.

a) Local environmental ordinances apply to this project and compliance is required. A list of applicable ordinances will be included in the contract documents.

b) Any solid waste materials generated during construction must be disposed of in a municipal or county sanitary landfill. Burning of wastes is not permitted.

c) Areas of construction disturbance must be watered down and fill material kept out of traffic lanes to prevent unnecessary dust.

d) Vehicle and equipment traffic will be restricted to the immediate project area (right-of-way and easement) to prevent destruction of natural vegetation and unnecessary soil disturbance.

e) The contractor is required to halt work immediately upon discovery of any archaeological remains. He must advise the Public Works Department Project Manager, who will confer with the State Historic Preservation Officer regarding guidance on archaeological mitigation.

f) Excavated material removed in the course of construction will be either utilized for other construction, or placed in approved fill areas and stabilized to prevent blowing dust.

Some mitigation may be possible for other operation impacts which have been defined.

a) Fencing of drains and provision of increased numbers of pedestrian crossings along the drain system could be incorporated into improvements associated with alternate scenarios which require drain widening, in order to reduce public safety hazards.

b) A screening mechanism could be provided at Alcalde Pump Stations, to prevent sport fishes from being pulled into the pump system.

8.3 Conclusion

No significant adverse impacts were determined among the seven alternate improvement scenarios. Differences in impact between alternates are considered to be of a minor nature. These minor differences, though, were used in ranking the alternates, producing a preference for the off-line detention scenario (No. 6). Least preferable, environmentally, were Scenario Nos. 1, 3, and 5.

With consideration of the overall environmental acceptability of the seven alternate improvement scenarios, it is recommended that selection of a preferred alternate for project implementation be primarily based on cost effectiveness. Environmental factors should be used as a secondary guide for final selection.

Final design of any improvement scenario should include a Phase I Environmental Property Evaluation for that portion of the Alameda Drain right-of-way which parallels Second Street. Potentially gasoline contamination from parcels located along the east right-of-way of Second Street may impact the project scope for mitigation.

9.0 ALTERNATE IMPROVEMENT SCENARIO COST SUMMARY

The following Table 9.1 shows the total estimated project costs for each Alternate Improvement Scenario. The estimates reflect life cycle cost analysis based on a 25-year life and 7% compound interest. Included in the estimates are initial construction costs, annual maintenance costs, environmental mitigation costs and maintenance and operation equipment purchase and replacement within the project life. The environmental mitigation cost includes additional archeology survey and testing for detention basin and pump station sites, as well as a screen at Alcalde Pump Station to protect sport fish from contact with the pump. Detailed cost estimate calculations are presented in Appendix G, Volume II, for each of the seven scenarios on a per reach basis.

TABLE 9.1

SYSTEM IMPROVEMENT ALTERNATES SCENARIOS - TOTAL PROJECT COSTS

	SCENARIO NO.						
	1 - Channel Widening with off-line detention and 4x10 CBC by-pass	2 - Channel Widening with s.d. collection and Alcalde diversion	3 - Montano Pump Sta. with s.d. collection and channel widening	4 - Matthew Pump Sta. with max size pump and s.d. coll. system	5 - Matthew Pump Sta. with min size pump and s.d. coll. system	6 - Off-Line Detention with max size basins and minimum improvement	7 - Off-Line Detention with min size basins and medium improvement
TOTAL IMPROVEMENTS COST (a)	$10,400,000	$8,600,000	$7,100,000	$13,000,000	$13,100,000	$23,200,000	$14,700,000
ANNUAL MAINTENANCE COSTS (b)	$283,000	$282,000	$270,000	$295,000	$281,000	$362,000	$327,000
PRESENT WORTH OF MAINTENANCE COSTS FOR 25 YRS (b)	$3,300,000	$3,300,000	$3,200,000	$3,400,000	$3,300,000	$4,200,000	$3,800,000
ENVIRONMENTAL MITIGATION COSTS	$2,900	$2,400	$2,900	$2,900	$2,900	$6,200	$3,500
TOTAL PROJECT COST	$13,702,900	$11,902,400	$10,302,900	$16,402,900	$16,402,900	$27,406,200	$18,503,500

(a) FROM TABLE 5.3
(b) FROM TABLE 6.3

10.0 RECOMMENDATIONS

10.1 Recommended Alternate Improvement

Upon review and consideration for cost, maintenance, environmental, and operational impacts of the seven scenarios presented, Alternate Improvement Scenario No. 2 is the recommended Plan. Scenario No. 3, including the Alcalde Pump Station Diversion, is actually the most preferred alternate; however, Scenario No. 3 is dependent upon Montano Pump Station being in place. The present political climate does not indicate that construction for Montano Boulevard improvements, which includes the pump station, will occur in the near future. Therefore, Scenario No. 2 improvements should be implemented unless Montano Pump Station becomes operational prior to construction of Scenario No. 2 improvements. There is no cost benefit derived in modifying the Montano pump station design if it occurs after Alameda Drain improvements are constructed according to Scenario No. 2.

10.2 Description of Recommended Improvements

Figure 10.1 (plan and profile sheets 2-1 to 2-4) and Figure 10.2 (detail sheets D-1 and D-2) present a graphical summary of all improvements associated with Scenario No. 2. Following is a detailed discussion of improvements required within each hydraulic reach.

Reach 1:

Between Alameda Road and Paseo del Norte, the existing channel is widened. The improved bottom width is 14 feet, with an average depth of 9 feet. Based on 1.5:1 sideslopes, this results in an average top width of 41 feet. Depending on the amount of right-of-way reserved for the expansion of Second Street, at least one maintenance road will be provided at a minimum width of 14 feet. No culvert improvements are required in this reach.

One gated structure with an orifice opening of 2 square feet is placed just north of Alameda Road and another of equal dimensions north of Paseo del Norte. South of Alameda Boulevard a 4 square foot orifice is required. With these, peak flows are attenuated in the form of on-line detention such that discharge rates downstream of the Paseo del Norte diversion in the Alameda Drain are similar to baseline irrigation conditions. The inlet side of the existing 42-inch culvert under Paseo del Norte is restricted to an 0.8 square foot opening with an orifice plate. A majority of the storm flows are diverted into the existing Paseo del Norte detention ponds, ultimately to discharge into the Rio Grande through the existing Paseo del Norte Pump Station. Peak discharges conveyed southerly in Alameda Drain are no less than the baseline irrigation design peak flow of 5 cfs.

Reach 2:

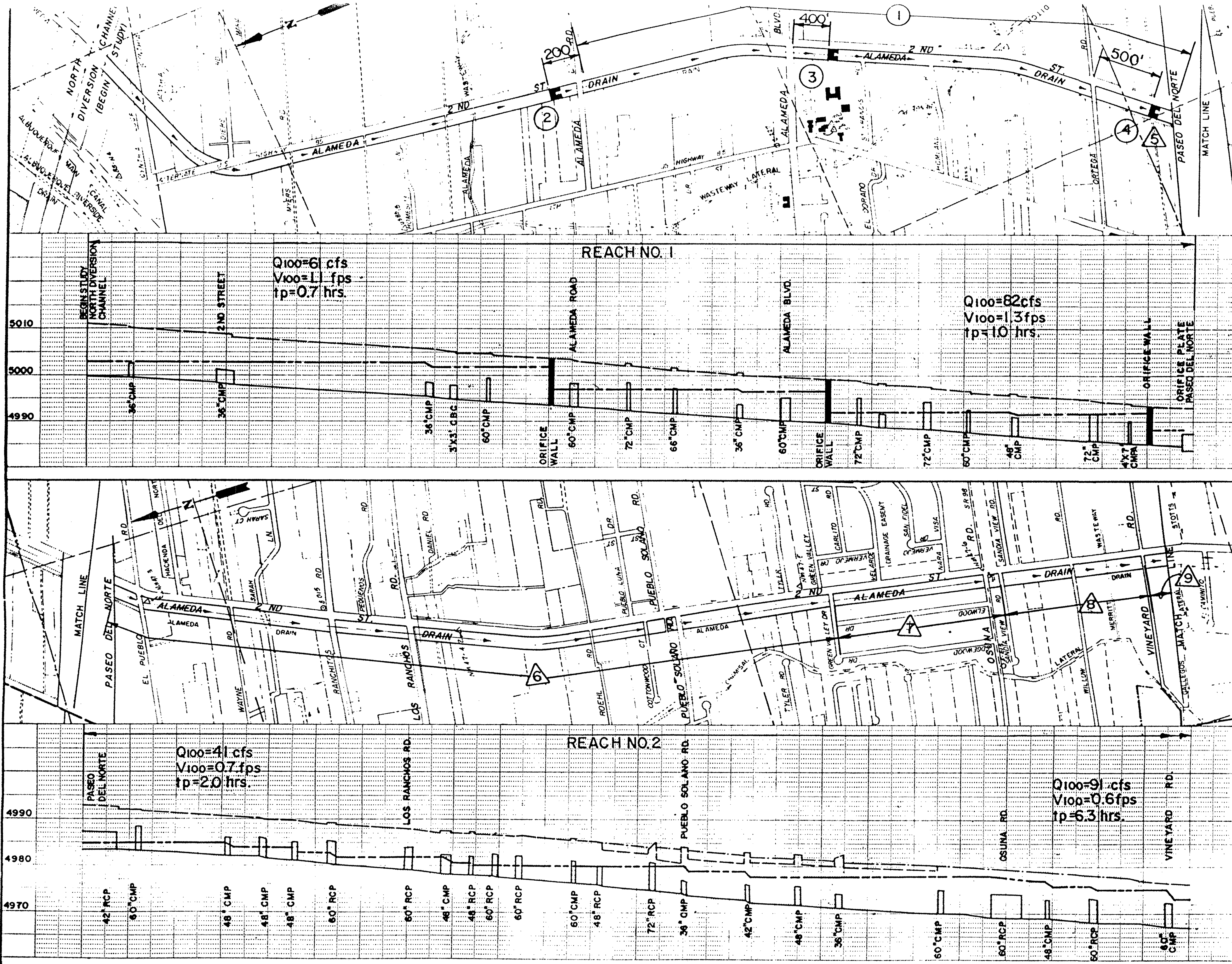
Between Paseo del Norte and Montano Road, the existing channel is widened to bottom widths ranging from 20 feet to 14 feet. Channel depths range from 8.5 feet to 9.0 feet, respectively, at various locations along the reach. If available right-of-way for channel improvements is held to 60 feet, then the bottom width must vary depending on the varying channel depth requirements so as not to exceed a maximum 46 foot top width. However, if more right-of-way is available, it would be preferable to maintain a constant bottom width of 20 feet in this reach. One 5 square foot opening gated structure is needed just upstream of Montano Road to attenuate the peak flow and to reduce downstream peak discharges. No culvert improvements are required in this reach. No diversion of flow from Alameda Drain to the proposed Montano Pump Station storm drain collection system is planned for this scenario.

Reach 3:

Extensive improvements are necessary between Montano Road and Mildred Avenue. Channel widening is needed throughout the reach to bottom widths ranging from 15 feet to 23 feet with depths ranging from 7 feet to 8.5 feet. Similar to Reach 2, the top width must not exceed 46 feet, so bottom widths must be adjusted depending upon channel depth requirements, unless more than 60 feet of right-of-way is available for drain improvements.

Culvert improvements and replacements include:

Lorenzo Avenue - new 6' x 10' CBC
Mescalero Road - new 6' x 10' CBC



LEGEND

- EXISTING TOP OF LOWEST BANK
- 2 ND STREET GUTTER FLOWLINE
- 42" RCP — CULVERT CROSSING AND SIZE
- DRAIN FLOWLINE
- 100 YR. WATER SURFACE
- Q_{100} 100 YR. DISCHARGE
- V_{100} 100 YR. VELOCITY
- tp TIME TO PEAK
- IMP RECOMMENDED IMPROVEMENTS
- △ PRIORITY 1 IMPROVEMENT
- ○ PRIORITY 2 IMPROVEMENT

KEY TO IMPROVEMENTS

CHANNEL WIDENING

IMP.	bw	DEPTH	DETAIL
1	14'	9.0'	A
6	20'	8.5'	A
7	17'	9.0'	A
8	19'	9.0'	A
9	14'	9.0'	A

GATED STRUCTURE- DETAIL E

IMP.	Ao	H	L
2	2	8.0'	50'
3	2	7.5'	50'
4	4	6.5'	50'

MISC. IMPROVEMENTS

IMP.	DESCRIPTION
5	ORIFICE PLATE ON 42", Ao=0.78 FT.^{2}

GRAPHIC SCALES

HORIZ. 0 500' 1000' 1500'

VERT. 0 5' 10' 20' 30'

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS

CITY OF ALBUQUERQUE

SCENARIO NO. 2
100 YEAR WATER SURFACE PROFILE

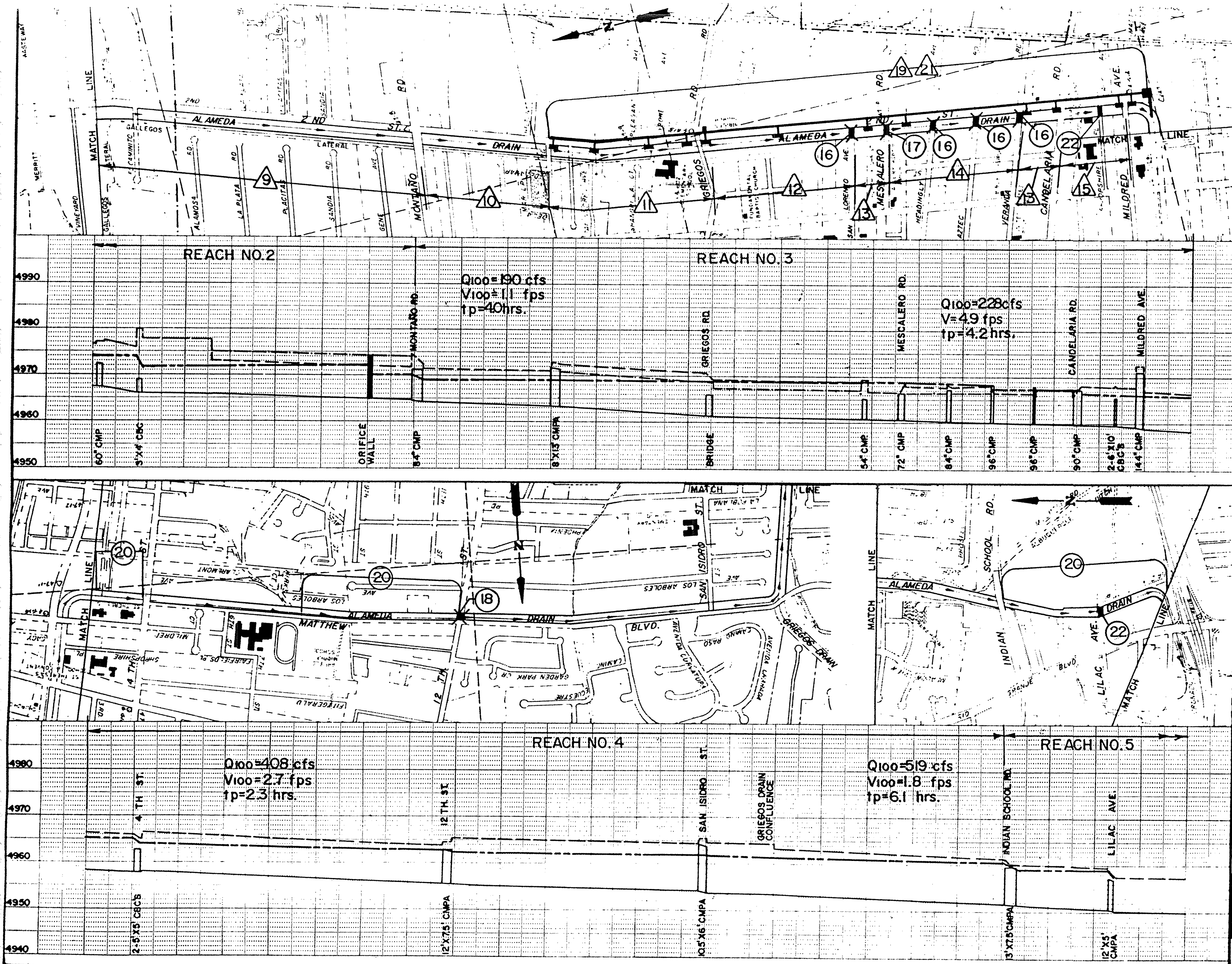
DRAWN BY

DATE 8/89

SCALE AS SHOWN

LEEDSHILL-HERKENHOFF, INC.
ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE SAN DIEGO SAN FRANCISCO

FIGURE 10.1 SHEET 2-1 OF 2-4



LEGEND

IMP	RECOMMENDED IMPROVEMENTS
△	PRIORITY 1 IMPROVEMENT
○	PRIORITY 2 IMPROVEMENT
- - - -	EXISTING TOP OF LOWEST BANK
— · —	2 ND. STREET GUTTER FLOWLINE
▯ 42" RCP	CULVERT CROSSING AND SIZE
——	DRAIN FLOWLINE
— — —	100 YR. WATER SURFACE
Q100	100 YR. DISCHARGE
V100	100 YR. VELOCITY
tp	TIME TO PEAK

KEY TO IMPROVEMENTS

CHANNEL WIDENING			
IMP.	bw	DEPTH	DETAIL
9	14'	9.0'	A
10	15'	7.0'	A
11	17'	8.5'	AA
12	22'	7.5'	AA
13	23'	7.5	AA
14	20'	7.5	AA
15	18'	7.0	AA

CULVERT REPLACEMENT			
IMP.	DIA.	L	TYPE
16	6'x10'	50'	CBC
17	6'x10'	66'	CBC
18	2-6'x10'	100'	CBC

MISC. IMPROVEMENTS	
IMP.	DESCRIPTION
19	42" STORM DRAIN COLLECTION SYSTEM & 41 cfs CAPACITY PUMPSTATION, CONNECT EXISTING INLETS.
20	BERM 2' HIGH BANKS, DETAIL B
21	CHANNEL WIDENING RETAINING WALL, DETAIL AA
22	2' HEADWALL EXTENSION

GRAPHIC SCALES

HORIZ. 0 500' 1000' 1500'

VERT. 0 5' 10' 20' 30'

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS

CITY OF ALBUQUERQUE

SCENARIO NO. 2
100 YEAR WATER SURFACE PROFILE

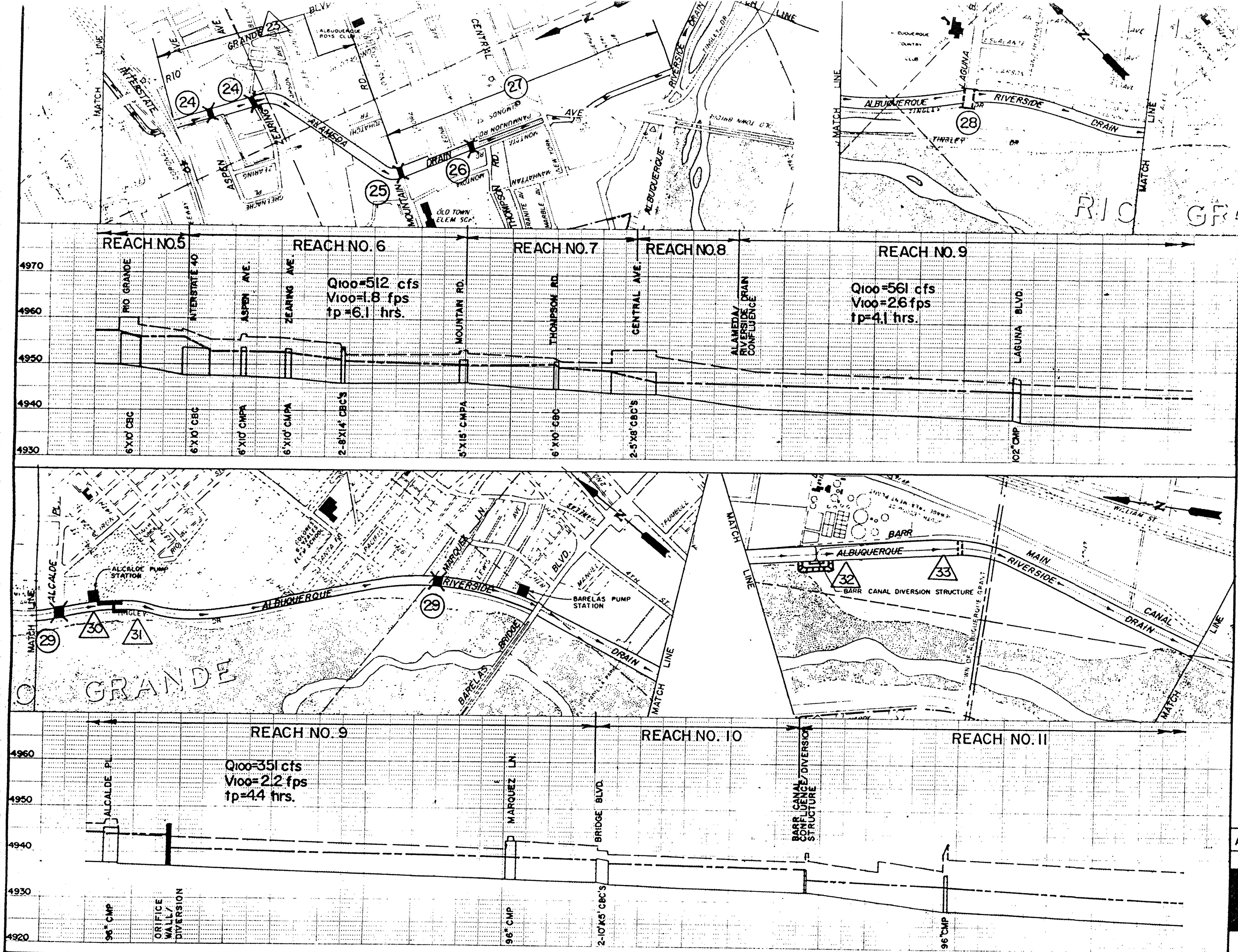
DRAWN BY

DATE 6/89

SCALE AS SHOWN

LEEDSHILL-HERKENHOFF, INC.
ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE SAN DIEGO SAN FRANCISCO

FIGURE 10.1 SHEET 2-2 OF 2-4



LEGEND

- IMP — RECOMMENDED IMPROVEMENTS
- △ — PRIORITY 1 IMPROVEMENT
- ○ — PRIORITY 2 IMPROVEMENT
- — EXISTING TOP OF LOWEST BANK
- — 2 ND. STREET GUTTER FLOWLINE
- 42" RCP — CULVERT CROSSING AND SIZE
- — DRAIN FLOWLINE
- — 100 YR. WATER SURFACE
- Q100 — 100 YR. DISCHARGE
- V100 — 100 YR. VELOCITY
- tp — TIME TO PEAK

KEY TO IMPROVEMENTS

CHANNEL WIDENING			
IMP.	bw	DEPTH	DETAIL
23	46'	8'	B

CULVERT REPLACEMENT			
IMP.	DIA.	L	TYPE
24	2-6'x10'	60'	CBC
25	2-6'x10'	90'	CBC
29	2-6'x10'	100'	CBC
28	REMOVE EXISTING CMP; ADD PEDESTRIAN BRIDGE		

GATED STRUCTURE-DETAIL E			
IMP.	Ao	H	L
31	40	6.7'	100'

MISC. IMPROVEMENTS	
IMP.	DESCRIPTION
26	ADD 6'x10' CBC, L=50'
27	CONCRETE LINE, DETAIL BB
30	ALCALDE DIVERSION, DETAIL F
32	EXTEND WEIR TO 100', DETAIL G
33	REMOVE EXISTING CULVERT

GRAPHIC SCALES

HORIZ. 0 500' 1000' 1500'

VERT. 0 5' 10' 20' 30'

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS

CITY OF ALBUQUERQUE

SCENARIO NO. 2
100 YEAR WATER
SURFACE PROFILE

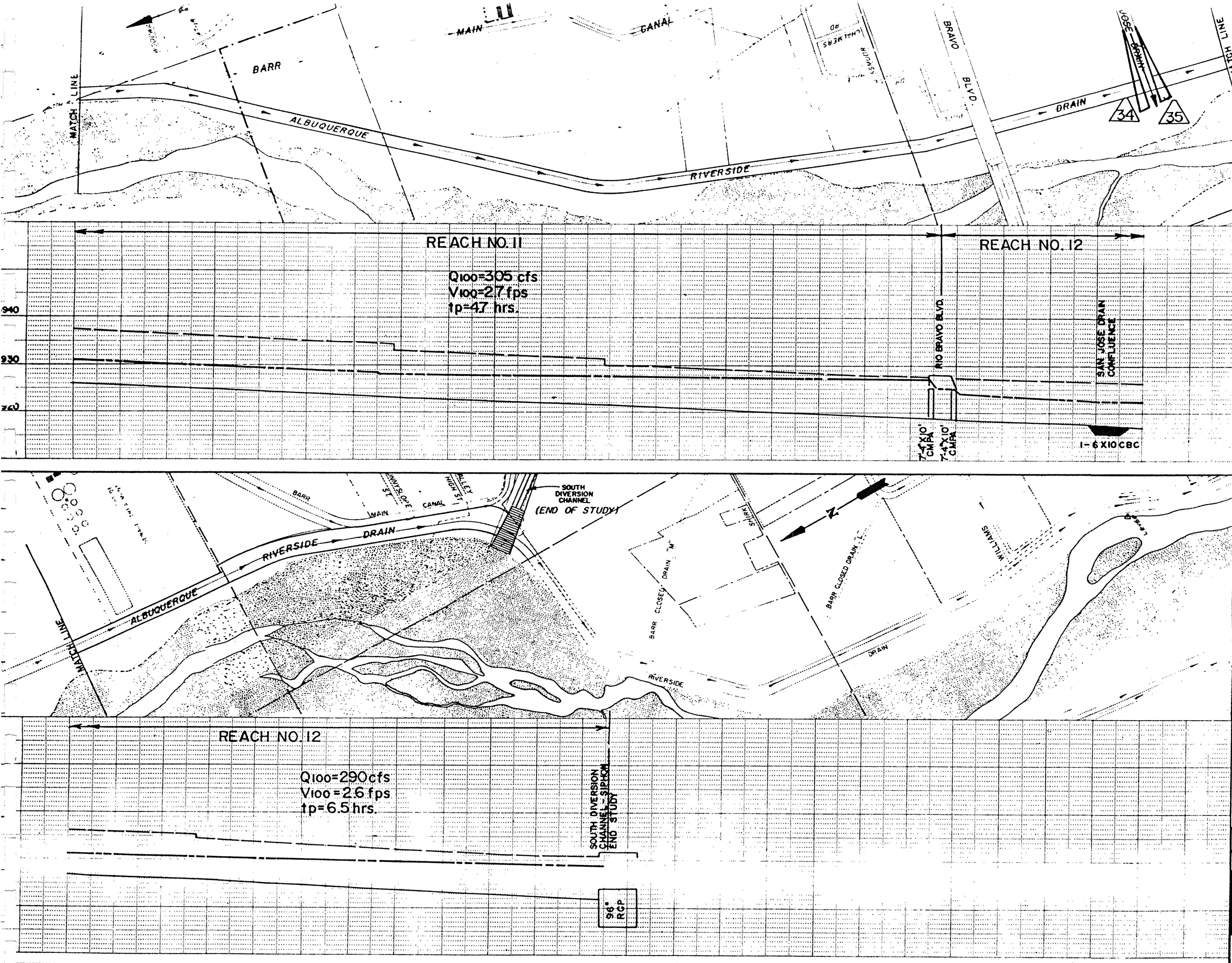
DRAWN BY

DATE 6/89

SCALE AS SHOWN

LEEDSHILL-HERKENHOFF, INC.
ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE SAN DIEGO SAN FRANCISCO

FIGURE 10.1 SHEET 2-3 OF 2-4



LEGEND

- EXISTING TOP OF LOWEST BANK
- 2 NO. STREET GUTTER FLOWLINE
- 42" RCP — CULVERT CROSSING AND SIZE
- DRAIN FLOWLINE
- 100 YR. WATER SURFACE
- Q_{100} — 100 YR. DISCHARGE
- V_{100} — 100 YR. VELOCITY
- tp — TIME TO PEAK
- IMP — RECOMMENDED IMPROVEMENTS
- △ — PRIORITY 1 IMPROVEMENT
- ○ — PRIORITY 2 IMPROVEMENT

KEY TO IMPROVEMENTS

MISC. IMPROVEMENTS	
IMP.	DESCRIPTION
△34	6'x10'x250' SIPHON
△35	SAN JOSE DRAIN OUTFALL

GRAPHIC SCALES

HORIZ. 0 500' 1000' 1500'

VERT. 0 5' 10' 20' 30'

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS

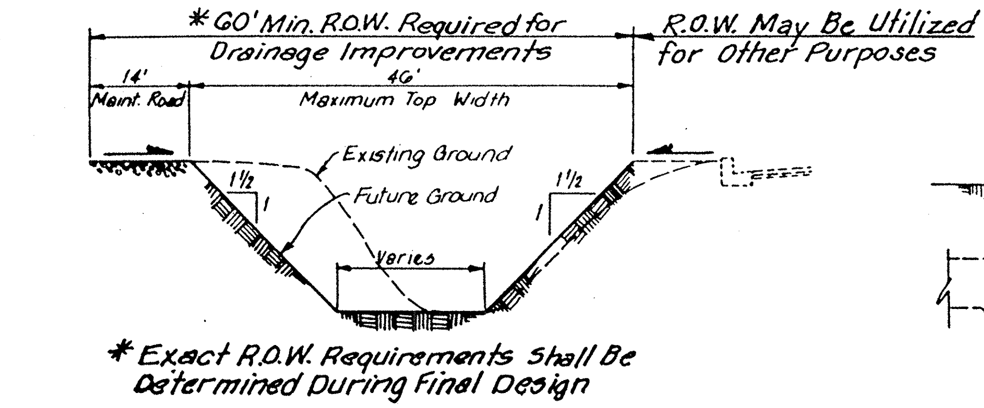
CITY OF ALBUQUERQUE

SCENARIO NO. 2
100 YEAR WATER SURFACE PROFILE

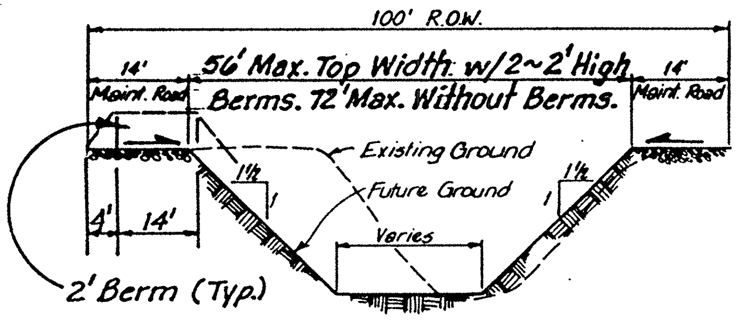
DRAWN BY	
DATE	6/89
SCALE	AS SHOWN

LEEDSHILL-HERKENHOFF, INC.
ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE SAN DIEGO SAN FRANCISCO

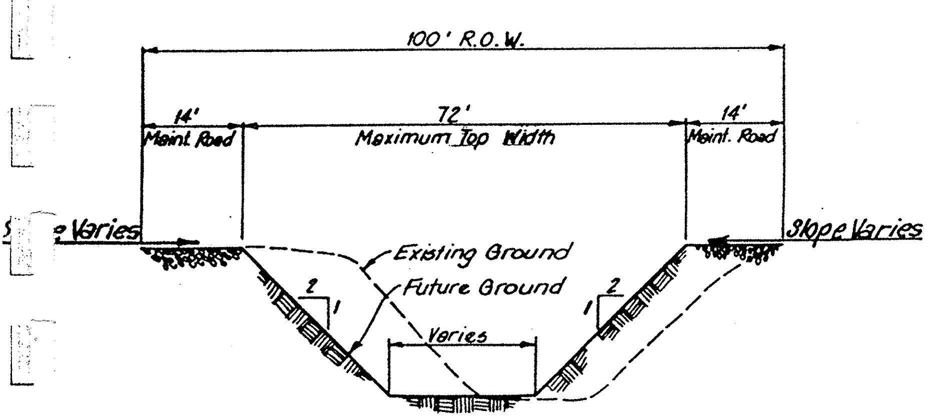
FIGURE 10.1 SHEET 2-4 OF 2-4



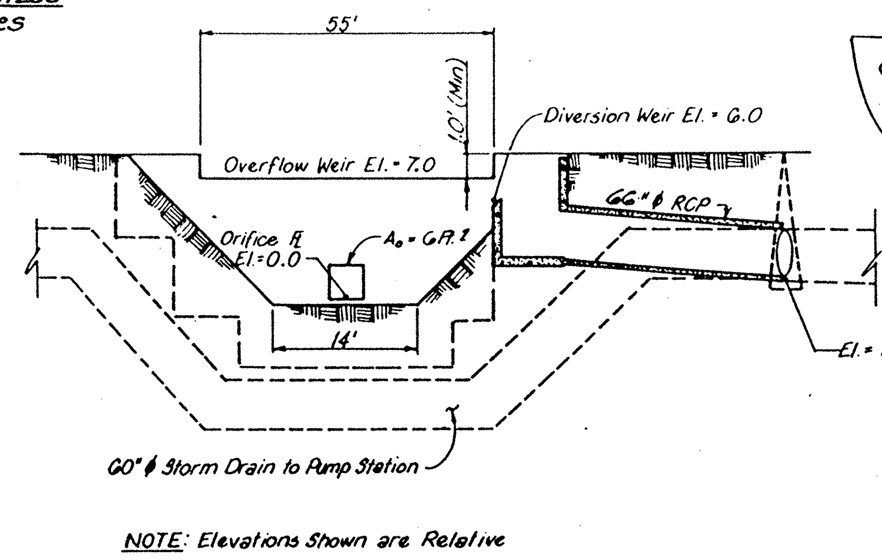
A TYPICAL SECTION - ALAMEDA DRAIN ALONG NORTH 2 ND. STREET DELAMAR RD. TO ALAMEDA RD.
NOT TO SCALE



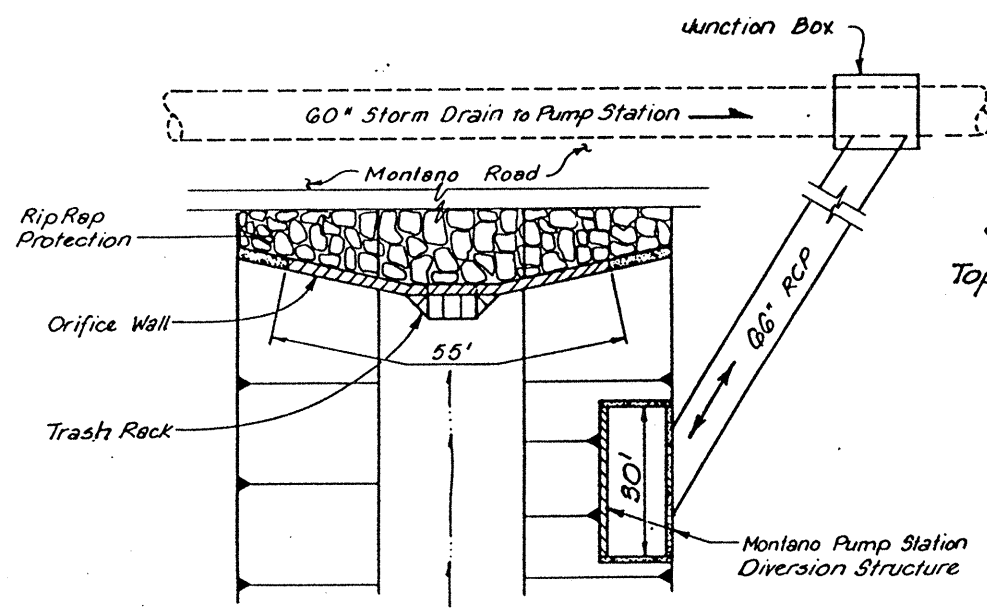
B TYPICAL SECTION - ALAMEDA DRAIN, MILDRED AVE. TO 12 TH. STREET AND I-40 TO MOUNTAIN ROAD
NOT TO SCALE



C TYPICAL SECTION - RIVERSIDE DRAIN, CONFLUENCE TO BARR DIVERSION
NOT TO SCALE

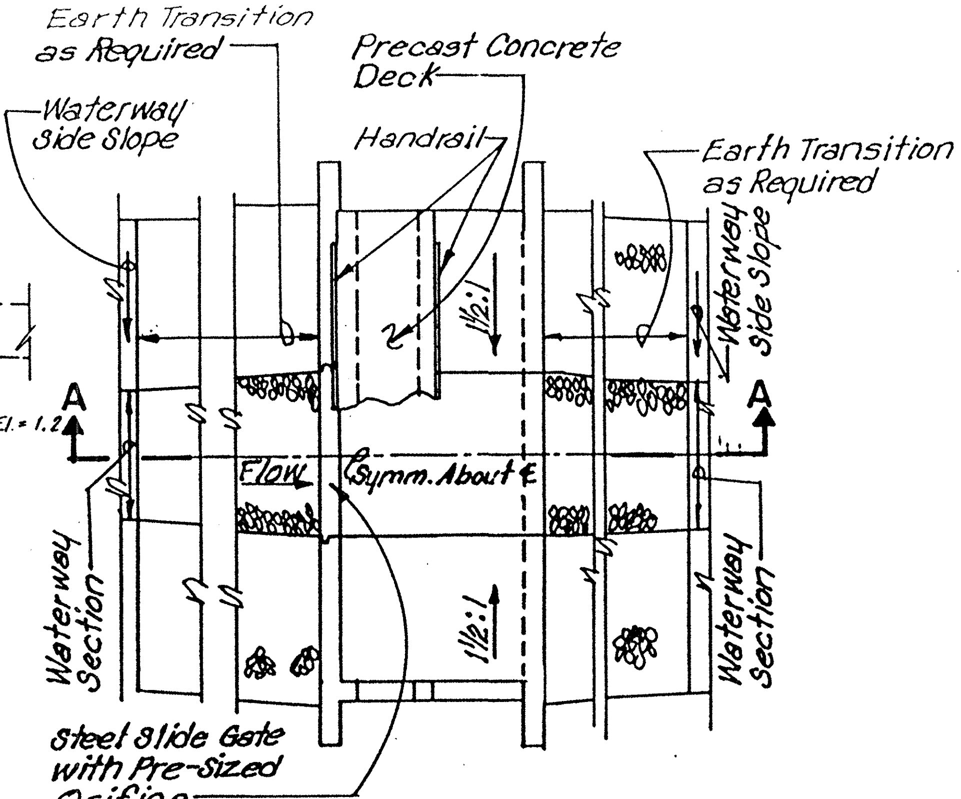


ELEVATION

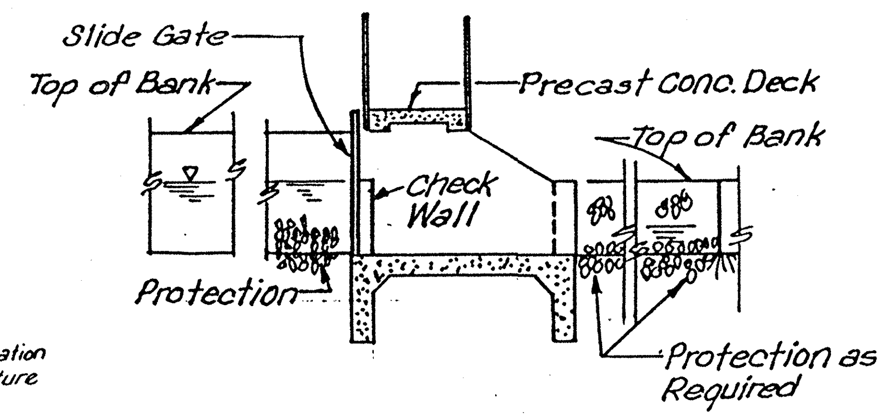


PLAN

D MONTANO PUMP STATION DIVERSION
NOT TO SCALE

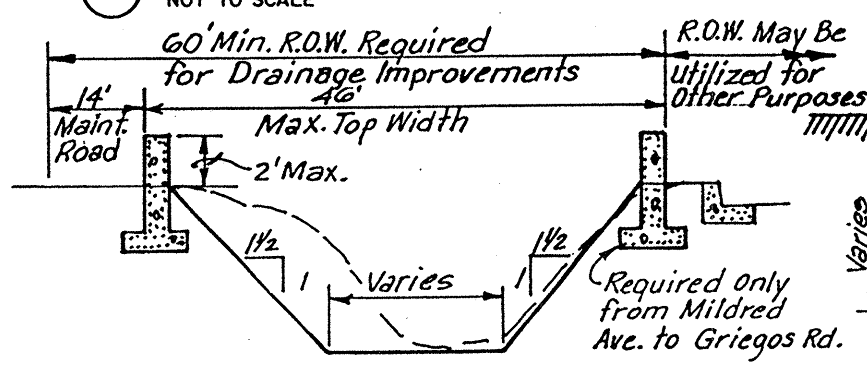


PLAN

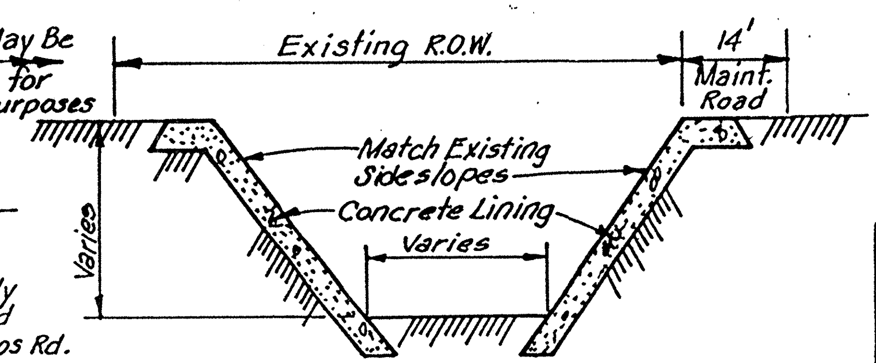


SECTION A-A

E GATED STRUCTURE
NOT TO SCALE



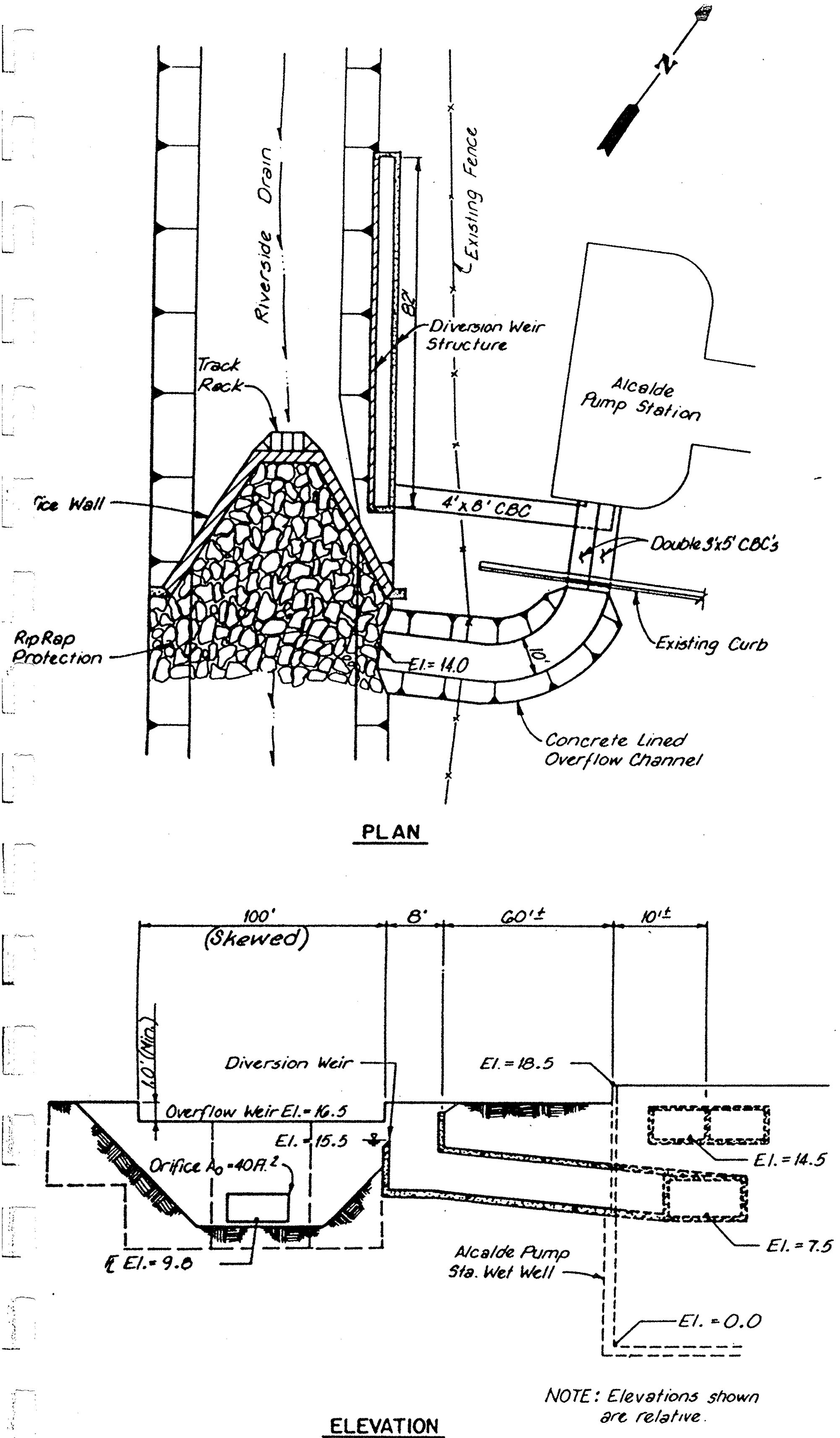
AA TYPICAL SECTION - ALAMEDA DRAIN ALONG 2 ND. STREET - MILDRED AVE. TO DELAMAR RD.
NOT TO SCALE



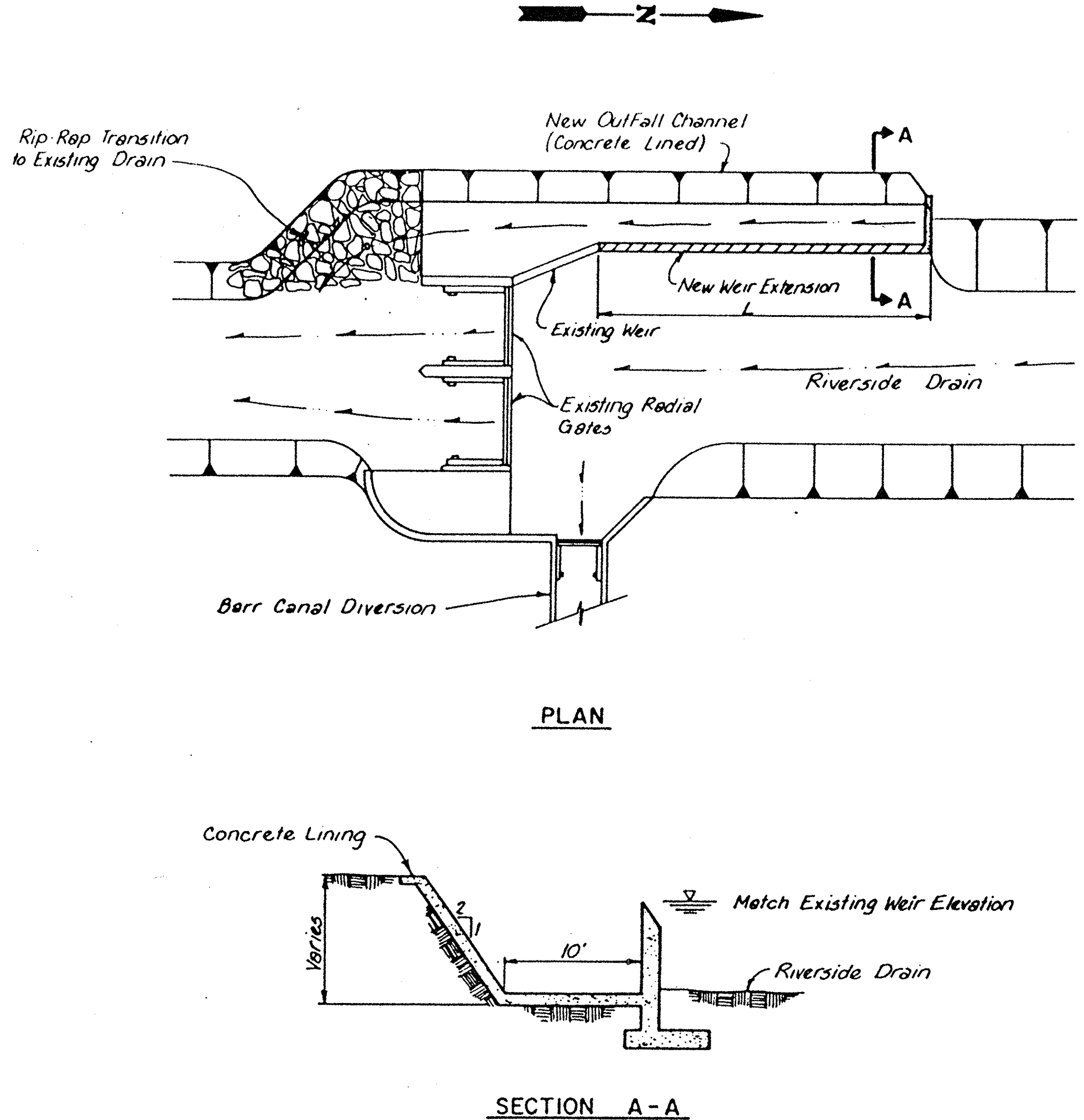
BB TYPICAL SECTION - CONCRETE LINING FROM THOMPSON TO THE CONFLUENCE OF THE ALAMEDA AND RIVERSIDE DRAINS
NOT TO SCALE

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS	
CITY OF ALBUQUERQUE	
MISCELLANEOUS DETAILS	DRAWN BY M.R.L. DATE 9-89 SCALE AS SHOWN
LEEDSHILL-HERKENHOFF, INC. ENGINEERS ARCHITECTS ALBUQUERQUE SANTA FE SAN DIEGO SAN FRANCISCO	

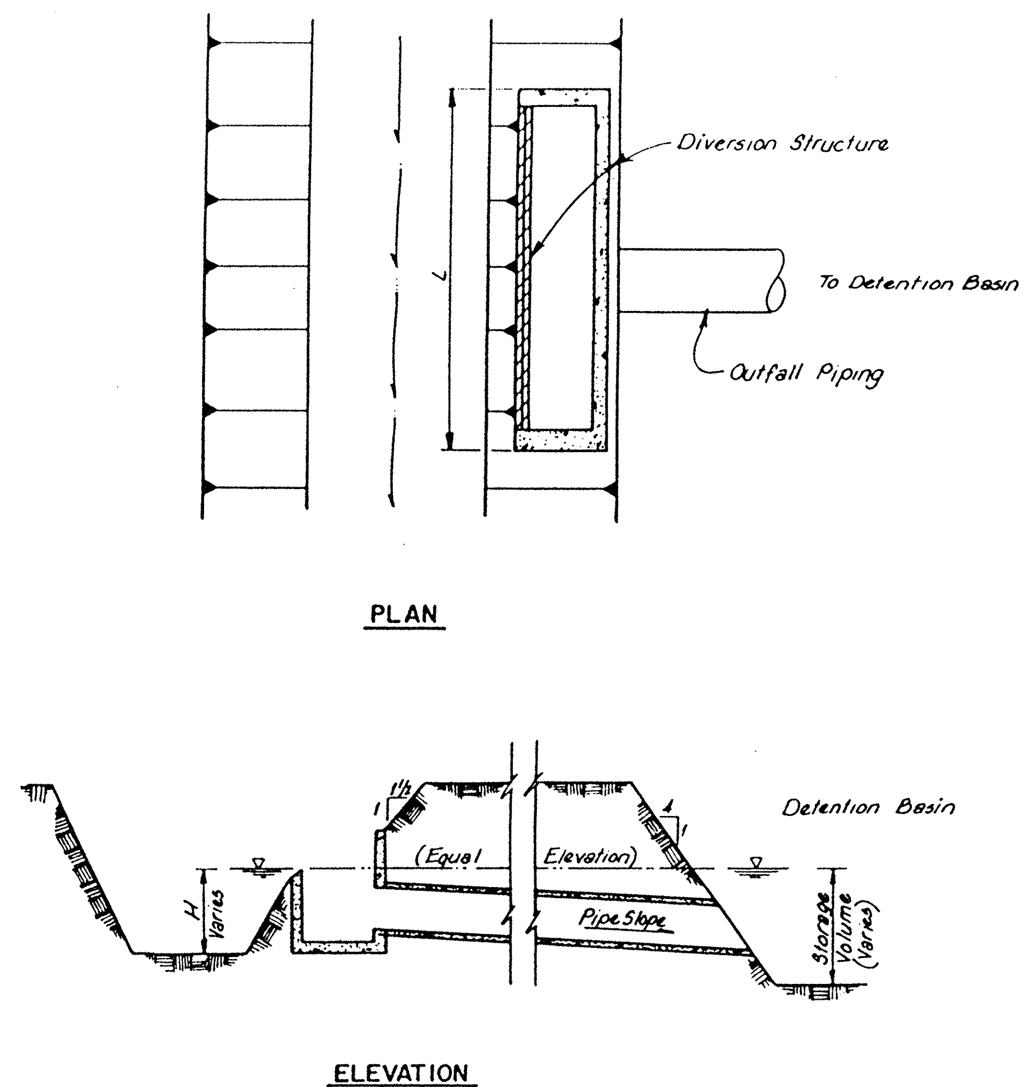
FIGURE 10.2 SHEET D-1 OF D-2



(F) ALCALDE PUMP STATION DIVERSION
NOT TO SCALE



(G) BARR CANAL DIVERSION STRUCTURE
NOT TO SCALE



(H) DETENTION BASIN DIVERSION (TYP.)
NOT TO SCALE

ALAMEDA/RIVERSIDE DRAINS ENGINEERING ANALYSIS

CITY OF ALBUQUERQUE

MISCELLANEOUS DETAILS

DRAWN BY	M.R.L.
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ENGINEERS ARCHITECTS
ALBUQUERQUE SANTA FE SAN DIEGO SAN FRANCISCO

FIGURE 10.2 SHEET D-2 OF D-2

Headingly Avenue - new 6' x 10' CBC
Aztec Road - new 6' x 10' CBC
Veranda Road - new 6' x 10' CBC
Shropshire Avenue - extend existing headwall height by 2 feet

A parallel 42-inch diameter storm drain system is needed along Second Street between Delamar Road and Mildred Avenue to hydraulically separate Second Street from the Alameda Drain. All existing and proposed storm inlets along Second Street between Delamar Road and Mildred Avenue are connected to the storm drain. The storm drain system is located within existing Drain right-of-way, east of the improved channel at a hydraulic grade line slope of 0.002 ft/ft. The line terminates at a small buried pump station just south of Mildred Avenue. The buried pump station collects local street runoff and discharges the flows at a peak rate of 41 cfs through a 30-inch discharge line into the Alameda Drain just south of Mildred Avenue.

This solution eliminates all connections between the Second Street storm inlets and the Alameda Drain. As a result, the water surface elevation in the Drain no longer is required to be below the gutter flowline elevations along Second Street. Instead the allowable flow depth in the Drain can exceed the gutter flowline and still be contained within the channel boundaries. A two foot high berm or retaining wall is required along both banks of the channel from Griegos Road to Mildred Avenue to confine the higher water surface within a 46-foot top width. Only the west bank requires a berm or retaining wall from Delamar Road to Griegos Road. The amount of right-of way reserved for the expansion of Second Street will determine whether space is available for berming and provision of a 14 foot wide maintenance road. If not, retaining walls will be the required improvement instead of berming. Design for this reach is predicated on gutter flowline elevations remaining equal to or higher than existing north of Avenue Road, if Second Street is widened or expanded.

Reach 4:

No channel widening is required in this reach, thus allowing sufficient space for two maintenance roads within the existing right-of-way. Widths of these roads vary depending on existing channel bottom widths and depths as well as the location of proposed berm improvements. Just upstream of 4th Street and 12th Street, the existing freeboard does not meet design criteria. Berming on both banks, approximately 2 feet high, is required to insure that a minimum freeboard of 1.5 feet is provided throughout the reach.

A new double 6' x 10' CBC is needed at 12th Street.

Reach 5:

Two improvements are required from Indian School Road to Interstate 40. One is to improve the existing culvert at Lilac Avenue by extending the height of the headwall by approximately two feet to prevent flow in the Drain from overtopping the street. The other is to provide a two foot high berm along both channel banks to insure that sufficient freeboard is attained in the reach.

Reach 6:

Channel widening is required from Interstate 40 to Mountain Road to provide a bottom width of 46 feet at a minimum depth of 8 feet, based on maximum top width of 72 feet. This allows for two maintenance roads at minimum widths of 14 feet, one each side along the channel. New double 6' x 10' CBC's are needed at Aspen Avenue, Zearing Avenue, and Mountain Road.

Reach 7:

Concrete lining of the channel sideslopes is necessary from Mountain Road to Central Avenue. Right-of-way is limited for the lower portion of this reach so space is inadequate for two maintenance roads. Bottom widths are assumed to be similar to existing conditions resulting in a 13 foot bottom width at a minimum depth of 6.5 feet, one 14 foot wide maintenance road and a maximum top width of 34 feet. The channel bottom is not lined to allow for continued infiltration and exfiltration to the water table.

A new 6' x 10' CBC is needed parallel to the existing 6' x 10' CBC at Thompson Road to increase the culvert capacity and to minimize the impacts of backwater in the upstream reach.

Reach 8:

Concrete lining of the channel sideslopes is also necessary from Central Avenue to the confluence of Alameda Drain and Riverside Drain. Right-of-way is limited in this reach. Since it is located adjacent to the Albuquerque Country Club property, no maintenance road is provided. Maintenance access will be from the adjacent area. Bottom widths are assumed to be similar to existing conditions resulting in a 12 foot bottom width at a minimum depth of 7.5 feet and a maximum top width of 34.5 feet. The channel bottom is not lined to allow for continued infiltration and exfiltration to the water table.

Reach 9:

Channel widening is not needed along Riverside Drain in this reach. A diversion structure is required just south of Alcalde Place. This structure allows baseline irrigation flows to continue downstream in the Drain, but diverts the stormwater peak flows into the existing Alcalde Pump Station, to be discharged into the Rio Grande. The station is designed to accept stormwater runoff from streets and storm drain systems in the surrounding area. The design inflow hydrograph for the pump station is such that a significant amount of pumping capacity is available to divert flows from Riverside Drain to the Rio Grande without inhibiting the station's effectiveness in conveying the local runoff. By diverting up to 270 cfs to Alcalde Pump Station, costs associated with downstream improvements needed to provide 1.5 feet of freeboard are reduced significantly. Due to the close proximity of Barelas Pump Station to Alcalde Pump Station, little benefit is derived from diverting flows at the Barelas location.

Two culvert replacements are needed in this reach:

Alcalde Place - new double 6' x 10' CBC
Marquez Lane - new double 6' x 10' CBC

In addition, the existing 102" CMP at Laguna Boulevard is recommended to be removed to minimize tailwater impacts upstream. No replacement is required since Laguna Boulevard was vacated in 1980. Coordination with appropriate parties should be made to determine if pedestrian access is needed at this crossing. As well, "The Albuquerque Biological Park Master Plan," Bohannan-Huston Inc., draft, February 15, 1991, should be referenced to insure compatibility between Riverside Drain improvements and the Master Plan in Reach 9.

Reach 10:

The single improvement required for this reach is to extend the weir length of the existing diversion structure in Riverside Drain at the Barr Canal. This results in minimizing the water surface upstream of the structure, while still providing for a diversion of up to 70 cfs to the Barr Canal. The existing radial gates may need rehabilitation, but the primary improvement is to extend the weir structure length from 16 feet to a total of 100 feet. A new outfall channel must be constructed to convey the weir flow around the radial gates safely, without eroding the adjacent channel banks. The diversion structure to Barr Canal may also need minor rehabilitation.

Reach 11:

An existing 96-inch diameter CMP culvert is located approximately 1500 feet downstream of the Barr Canal Diversion in Riverside Drain. To minimize tailwater impacts on the diversion structure, it is necessary to remove this culvert. There is no apparent access need satisfied by this crossing, so it was assumed that it need not be replaced with a larger sized culvert.

Riverside Drain has adequate conveyance capacity and freeboard throughout this reach length until reaching Rio Bravo Boulevard. Construction of a two foot high berm is recommended from approximately 100 feet upstream of Rio Bravo to the upstream headwall to insure that freeboard requirements are met. No other improvements are required in this reach.

Reach 12:

A critical improvement of the recommended Plan is to provide a direct outfall from the San Jose Drain to the Rio Grande. Although hydraulic conditions upstream of this confluence are not improved, the peak flows allowed to continue downstream are significantly reduced as a result of a new outfall. In addition, the potential exposure of poor water quality originating from San Jose Drain flows is eliminated by providing an outfall at San Jose Drain. It is anticipated that the invert of the San Jose Drain will be above the Riverside Drain invert. Therefore, a single 6' x 10' CBC siphon is needed to convey Riverside Drain flows under the San Jose Drain Outfall. The outfall is presently under design by Wilson & Co. Final design of Riverside Drain improvements will depend on the detailed design of the outfall.

No other improvements are needed in this reach.

10.3 LIST OF ASSUMPTIONS

A. Channel improvements are based on channel slope equal to existing conditions.

B. New culverts shall be installed at similar invert elevations as existing conditions.

C. The centerline of improved channel sections is coincident with the centerline of right-of-way, unless otherwise noted.

D. All improvements will occur within the existing right-of-way limits, unless otherwise noted.

E. Minor variations in the recommended channel bottom widths are acceptable. As such, transitions in channel bottom widths and top widths will be necessary to accommodate culvert crossings.

10.4 Priority of Improvements

Recommended improvements are based on future development conditions. As a result, it is not imperative that all improvements be constructed immediately. Complete implementation of improvements is not required until all of the planned future projects described in Section 4.2 are implemented.

Two improvement priorities have been assigned to the recommended Plan. Priority 1 improvements represent those necessary to prevent overtopping of the channel banks but which do not necessarily result in creating freeboard sufficient to meet the design criteria. Certain improvements are also assigned a Priority 1 based on projects already planned and/or funded by the City, County or developers which will have a direct impact on stormwater discharge into the Drains. Priority 2 improvements include all remaining improvements needed to satisfy the channel design criteria and freeboard requirements for ultimate development conditions. Although the timing for completion of Priority 2 improvements is less critical, they must precede the construction of planned projects which would further increase the introduction of stormwater to the drains.

Improvements priorities are illustrated graphically on Figure 10.1. On each of the four plan and profile sheets, Priority 1 keyed improvements are shown within a triangle. Priority 2 keyed improvements are indicated by a circle on the same sheets.

Table 10.4.1 summarizes those improvements which have the highest construction priority. An estimate of cost for each improvement is included in the table.

Table 10.4.2 summarizes those improvements which have a Priority 2 for implementation. An estimate of cost for each improvement is included in the table.

TABLE 10.4.1
ALTERNATE SCENARIO NO. 2
PRIORITY 1 IMPROVEMENTS

IMPROVEMENT NO.	DESCRIPTION	SHEET NO.	REACH	LOCATION	CONSTRUCTION COST EST.
5	Orifice Plate	2-1	1	Paseo del Norte 42" culvert	$14,000
6	Channel Widening	2-1	2	From: Paseo del Norte To: Green Valley Drive	$380,000
7	Channel Widening	2-1	2	From: Green Valley Drive To: Sandia View Road	$90,000
8	Channel Widening	2-1	2	From: Sandia View Road To: Vinyard Road	$80,000
9	Channel Widening	2-1,2	2	From: Vinyard Road To: Montano Road	$180,000
10	Channel Widening	2-2	3	From: Montano Road To: Delamar Road	$60,000
11	Channel Widening	2-2	3	From: Delamar Road To: Griegos Road	$65,000
12	Channel Widening	2-2	3	From: Griegòs Road To: Loreno Avenue	$60,000
13	Channel Widening	2-2	3	From: Loreno Avenue To: Mescalero Road	$16,000
14	Channel Widening	2-2	3	From: Mescalero Road To: Veranda Road	$56,000
13	Channel Widening	2-2	3	From: Veranda Road Candelaria Road	$20,000
15	Channel Widening	2-2	3	From: Candelaria Road To: Mildred Avenue	$30,000
19	Storm Drain & Pump Station	2-2	3	From: Delamar Road To: Mildred Avenue	$2,076,000
21	Retaining Walls Along Channel	2-2	3	From: Delamar Road To: Mildred Avenue	$1,310,000
23	Channel Widening	2-3	6	From: Interstate 40 To: Mountain Road	$260,000
30	Alcalde Diversion	2-3	9	Alcalde Pump Station	$128,000
31	Gated Structures	2-3	9	Alcalde Pump Station	$25,000
32	Side-spill Weir Extension	2-3	10	Barr Canal Diversion Structure	$64,000
33	Remove Existing Culvert	2-3	11	500' Dwnstrm - Barr Canal Div.	$10,000
34	Riverside Drain Siphon	2-4	12	Confluence with San Jose Drain	$150,000
35	San Jose Drain Outfall	2-4	12	Confluence with San Jose Drain	$950,000

TOTAL COST OF PRIORITY 1 IMPROVEMENTS = $6,024,000

TABLE 10.4.2
ALTERNATE SCENARIO NO. 2
PRIORITY 2 IMPROVEMENTS

IMPROVEMENT NO.	DESCRIPTION	SHEET NO.	REACH	LOCATION	CONSTRUCTION COST EST.
1	Channel Widening	2-1	1	From: Alameda Road To: Paseo del Norte	$120,000
2	Gated Structure	2-1	1	200' Upstream of Alameda Road	$22,000
3	Gated Structure	2-1	1	400' Downstream of Alameda Blvd.	$22,000
4	Gated Structure	2-1	1	500' Downstream of Ortega Road	$22,000
16	New 6' x 10' CBC	2-2	3	Lorenzo Avenue	$55,000
17	New 6' x 10' CBC	2-2	3	Mescalero Road	$75,000
16	New 6' x 10' CBC	2-2	3	Headingly Avenue	$55,000
16	New 6' x 10' CBC	2-2	3	Aztec Road	$55,000
16	New 6' x 10' CBC	2-2	3	Veranda Road	$55,000
22	Extend Height of Headwall 2'	2-2	3	Shropshire Place-upstream headwall	$12,000
18	New Double 6' x 10' CBC	2-2	4	12th Street	$150,000
20	Add 2' High Berm to Banks	2-2	4	From: 500' Upstream of 4th Street To: 4th Street	$19,000
20	Add 2' High Berm to Banks	2-2	4	From: 1600' Upstream of 12th Street To: 12th Street	$61,000
20	Add 2' High Berm to Banks	2-2	5	From: Indian School Road To: Lilac Avenue	$55,000
22	Extend Height of Headwall 2'	2-2	5	Lilac Avenue-upstream & downstream	$41,000
24	New Double 6' x 10' CBC	2-3	6	Aspen Avenue	$90,000
24	New Double 6' x 10' CBC	2-3	6	Zearing Avenue	$90,000
25	New Double 6' x 10' CBC	2-3	6	Mountain Road	$140,000
26	Add parallel 6' x 10' CBC	2-3	7	Hollywood Avenue	$80,000
27	Concrete Line Channel Sideslopes	2-3	7,8	From: Mountain Road To: Alameda/Riverside Drn. Confl.	$940,000
28	Remove Existing 102" CMP/ Construct Pedestrian Bridge	2-3	9	Laguna Boulevard	$10,000
29	New Double 6' x 10' CBC	2-3	9	Alcalde Place	$138,000
29	New Double 6' x 10' CBC	2-3	9	Marquez Lane	$138,000

TOTAL COST OF PRIORITY 2 IMPROVEMENTS = $2,445,000

11.0 ADOPTION, IMPLEMENTATION, AND MAINTENANCE OF DRAIN IMPROVEMENTS

11.1 Adoption of Recommended Drain Improvements

Alternate Improvement Scenario No. 2 or No. 3 drain improvements are favored by the City of Albuquerque, Hydrology Division as the preferred options. As previously discussed, Scenario No. 3 is dependent upon the planned Montano Pump Station being in place. Due to the uncertainty of this project, Scenario No. 2 appears to be the most feasible alternate to implement. The recommended improvements may be altered depending upon actual future conditions and unforeseen circumstances.

A proposal has been developed by the City of Albuquerque for the approval by the MRGCD Board which shall serve as the basis for future improvements along the Alameda and Riverside Drains. This proposal includes the following considerations:

- MRGCD's and the Bureau of Reclamation's approval of Alternate Improvement Scenario No. 2 as the basis for implementing construction improvements along the drains.

- Assignment of maintenance responsibilities for Alameda Drain from its north terminus near the North Diversion Channel to its confluence with the Riverside Drain near Central Avenue and for Riverside Drain from the existing force main of the Duranes Storm Pump Station to the South Diversion Channel north right-of-way line.

- Construction and maintenance by the City of the San Jose Drain Outfall to the Rio Grande.

- Compliance by the City with any stormwater regulations imposed by the federal government or the State of New Mexico.

- Cooperation between City maintenance personnel and MRGCD to insure that the integrity of the MRGCD operations is not jeopardized.

- First rights of refusal given to the City prior to the vacation of any easements or right-of-way along the drains by the MRGCD.

- Flow rates to be minimized in the drains, by the MRGCD, during the normal rainy season to optimize their effectiveness to conveyance of storm flows.

11.2 Implementation of Alternate Improvement Scenario No. 2

The improvements presented in this report were evaluated on a conceptual basis only. Prior to construction, a detailed design effort must be performed which will include preparation of design drawings, contract documents, specifications and a detailed construction cost estimate.

Construction phasing of improvements will be based on the assigned priority, the schedule of other valley drainage projects which rely on the Alameda and Riverside Drains, and the availability of funds from the City's general obligation bond program.

11.3 Maintenance of Drains

Providing an agreement is reached between the City and the MRGCD, the City will take responsibility for maintenance of the Alameda and Riverside Drains within the reaches described in Section 11.1. Regular maintenance of the drains will be in accordance with City maintenance standards as outlined in Volume 2, Design Criteria, of the City's Development Process Manual (DPM). Particular attention must be given to the small openings in the orifice walls to insure that they are not plugged while the drains are flowing. Section 8, Maintenance Standards of the DPM is excerpted below:

City Maintenance Standards

The City shall regularly maintain the drainage control, flood control and erosion control facilities for which it has responsibility based on the following schedule:

Facility	Maintenance	Inspection
Channels	Monthly Jun-Oct	Semi-Annual
Channel Joints	Monthly Jun-Oct	Semi-Annual
Crossing Structures & Gated Structures	Monthly Jun-Oct	Semi-Annual
Pump Stations	Monthly Jun-Oct	Semi-Annual
Detention Facilities	Silt removal and weed control	After any major operation or monthly during flood season
Storm Pumps	Periodic cycling	Semi-annually in April and October
	Vibration analysis	3-5 Years
Storm Sewer Systems	Annual	Bi-Annual
Storm Sewer Inlets	Ongoing process during flood season.	Semi Annual

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