HDR

April 1, 1993

Honorable Mayor and City Commission
City of Amarillo
Post Office Box 1971
Amarillo, Texas 79186-0001

RE: Storm Water Management Master Plan for
The City of Amarillo, Texas

Gentlemen:

In accordance with the Scope of Services outlined in our Engineering Agreement dated October 15, 1991, HDR Engineering, Inc. is pleased to submit herewith twenty-five (25) copies of the final Storm Water Management Master Plan Report for the City of Amarillo. In addition to the Report, we are providing you with one (1) reproducible copy and five (5) blueline copies of the accompanying Master Plan Drawing Set, as well as one (1) diskette copy of the ASAPP Playa Simulation Model.

We wish to thank you, your City Staff, and members of the Storm Water Management Master Plan Advisory Committee for the assistance and cooperation provided throughout the development of this Master Plan. We sincerely believe that this Master Plan, together with the previously delivered Storm Water Management Criteria Manual, will be the foundation for improved storm water management in the City of Amarillo.

Respectfully submitted,

HDR Engineering, Inc.

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SECTION 3 STUDY AREAS AND IMPROVEMENT ALTERNATIVES

Development of this Storm Water Management Master Plan has involved the cooperation and assistance of numerous capable individuals. HDR Engineering, Inc. would like to thank Mr. Mike Kennedy, Mr. Mike Smith, Mr. Michael Rice, Mr. Rob Jetter, and remaining members of the City Public Works and Engineering staffs for their efforts throughout the project. HDR would also like to thank the citizen members of the Storm Water Management Master Plan Advisory Committee for their willingness to serve in their appointed roles.

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The purpose of this Storm Water Management Master Plan was to establish a basis for addressing existing and future drainage problems in the City of Amarillo. Specifically, with this Master Plan the City should be able to accomplish the following:

- Adopt the Storm Water Management Criteria Manual and use the manual as the basis for technical standards and criteria; and
- Use the storm water management policy adopted (Section 1 of the Storm Water Management Criteria Manual) to develop a comprehensive storm water ordinance;
- Prioritize and establish an implementation schedule for the Capital Improvements Program Project List.

The first step of the Master Plan process was to establish a Storm Water Policy. This policy appears as Section 1 of the Criteria Manual and is intended to serve as the basis for development of a comprehensive storm water ordinance. The policies and standards defined in the Criteria Manual were then used in evaluating and identifying solutions for specific City storm water problems. The results of this process are summarized in the Capital Improvements Program Project List, which appears in Section 4 of this Master Plan Report.

The full-sized Drawing Set which accompanies this Master Plan Report presents schematic improvement designs for 14 playa projects, 17 storm sewer projects, and 14 channel projects. These proposed improvements are depicted graphically with an accompanying tabular summary describing the improvement details. The Drawing Set should be referenced along with this Master Plan Report, as the report provides a written description of the improvements as well as estimated construction costs.

Construction costs for the proposed playa improvement projects totalled approximately $18 million compared to approximately $15 million for storm sewer improvement projects, and just over $3 million for channel improvement projects. Improvement designs were based on the design standards adopted and included in the Criteria Manual. Improvement designs for playas required that the 100-year water surface elevation under ultimate basin development conditions not reach the primary damage elevation, generally defined as the lowest residential or commercial property at the playa fringe. Designs for storm sewer improvements required that streets be capable of conveying the two-year frequency storm event with the storm water level no higher than the top-of-curb elevation. An additional standard required that the storm water level for the 100-year frequency event not exceed 18 inches above the top-of-curb. Channel projects, which involved improvements to culverts, bridges, street crossings, and the channels themselves, were based on standards requiring passage of the 25-year frequency event without overtopping of the street, with an additional requirement that the 100-year water surface elevation not exceed 18 inches above the street. In addition, at no location along a channel reach was the water surface elevation allowed to exceed the slab elevation of a permanent structure.

The proposed improvement designs identified in this report and depicted graphically in the Drawing Set are preliminary. The project descriptions and cost estimates are intended to be sufficient to provide the City with the information necessary to prioritize and schedule the improvement projects. Storm sewer sizes and configuration; inlet number, size, and location; channel and culvert dimensions; playa excavation quantities and pumping capacities; and other design-related information indicated are all subject to adjustment. When a project is ready for implementation, a thorough final design will be required involving detailed surveys, refined hydrologic and hydraulic analyses, and consideration of other factors not considered in this Master Plan.

In implementing this Storm Water Management Master Plan, the City must carefully consider the impacts of the new Environmental Protection Agency (EPA) NPDES storm water quality regulations. The City's EPA municipal permit will require the adoption of certain best management practices (BMPs) intended to reduce the amount of pollution in storm water runoff. Although this Storm Water Management Master Plan does not recommend specific BMPs, the Drawing Set will serve as convenient workmaps for incorporating any required BMP's with the proposed storm water improvement projects due to the wide aerial coverage of both developed and undeveloped areas.

1.0 INTRODUCTION

1.1 Background

The City of Amarillo is located along the watershed divide between the Canadian River Basin (to the north) and the Red River Basin (to the south) in the northern high plains panhandle region of west Texas. The dominant natural drainage features in the southwest, southeast, and northeast quadrants of the City are playas, shallow indentations in the land surface which serve as central, terminal points for runoff from the respective drainage basins. Seventeen playas currently lie within the City limits; approximately 45 additional playas exist within five miles of the City limits. Land slopes in these areas are generally flat. Only in the northwest quadrant of the City, where slopes are steeper, does runoff collect and flow in a conventional tributary and creek channel system. In this area of the City, runoff is conveyed to the Canadian River through East Amarillo Creek, West Amarillo Creek, and their tributaries.

Storm sewer installation began in Amarillo in 1927. The original system served the downtown area and additional systems were built as the City was developed. Today, about 25 percent of the developed portion of the City is served by storm sewers. Remaining areas drain to either playas or to the creeks in the northwest quadrant of the City.

Although located in a semi-arid region, Amarillo can have significant flooding problems. These problems typically manifest themselves in the form of high playa levels, which flood homes and businesses located at the playa fringe; and street and intersection flooding, which floods homes and businesses located along flood-prone streets, interfering with traffic. Playa flooding is caused by (1) development in the watershed which, due to the increase in the percent of impervious cover, causes greater amounts of runoff to reach the playas and (2) development encroachment upon the playa fringes, which subjects greater amounts of property to flooding when playa levels do rise. Street flooding is caused by undersized storm sewer systems, lack of storm sewers, inadequate inlet capacity, or, in some cases, backwater resulting from high playa levels.

In the City's Comprehensive Plan published in 1989, flooding problems were addressed and three recommendations offered:

1) Prepare and implement a long-range Storm Water Master Plan;
2) Strictly enforce the Flood Hazard Ordinance; and
3) Amend the Building Code to require elevated structural foundations and grading plans.

1.2 Goals and Objectives

Flooding problems in Amarillo can be attributed to two chief causes: a natural topography not conducive to flood relief and the lack of a comprehensive and systematically-applied drainage policy. This Storm Water Management Master Plan addresses both of these issues. Although little can be done about Amarillo's flat topography and playa-based drainage, the Master Plan identifies and describes proposed projects to provide flooding relief for 45 problem areas, channel reaches, and playas. In addition, to avoid future flooding problems, a Storm Water Management Criteria Manual accompanies this Master Plan. The Manual includes a Storm Water Policy, which will serve as the framework for development of a Storm Water Ordinance; and multiple technical sections, which provide the standards and criteria needed to properly design storm water systems.

The following are the major goals of this Storm Water Master Plan:

1) development of a storm water policy/ordinance;
2) development of technical standards and criteria;
3) identification and assessment of existing flood problems; and
4) identification of projects to correct existing flood problems.

1.3 Organization of the Master Plan Report and Drawing Set

Section 3 of the Master Plan Report, as well as the accompanying Drawing Set, is organized by playa or creek basin. All of the playas, problem areas/storm sewers, and channel reaches investigated fall into one of 37 study areas: 35 playa basins or two creek basins (East Amarillo Creek and West Amarillo Creek). Playa study areas are designated with a "P" followed by the playa number (i.e. P16), or, in the case of a creek study area, by "CW", for West Amarillo Creek, and "CE", for East Amarillo Creek. Specific playas, storm sewer systems, and channel reaches are identified by a two- or three-character designation such as "P16" for Playa No. 16, "RH" for Rushmore/Hayden storm sewer system, and "M1" for the lower East Amarillo Creek channel reach. Designations and nomenclature are summarized in Legend Sheet L-1 of the Drawing Set.

To find information about a particular area or improvement project, it is necessary to identify the appropriate playa or creek basin. This is most easily done using the Legend Sheet (Sheet L-1) of the Drawing Set. Section 3 of this report describes existing problems and proposed improvements for all of the playas, storm sewer systems, or channel reaches which fall within the particular playa or creek basin. For example, Section 3.14 Playa No. 16 (Study Area P16), would cover existing problems and proposed improvements for not only the Willow Grove Lake playa, but also for the Rushmore/Hayden storm sewer system, which falls within the Playa No. 16 basin boundaries.

2.0 ANALYSIS METHODOLOGIES

2.1 Playas

When evaluating drainage systems or designing drainage improvements in Amarillo, consideration of playas is essential. Playas serve as terminal storage for runoff from most Amarillo drainage basins and, therefore, determine tailwater conditions for drainage collection systems. Furthermore, flood levels around playas must be examined prior to locating any structure near a playa to assess the risk of flooding and to evaluate potential flood control measures.

2.1.1 Playa Simulation Using ASAPP

The Amarillo Simulation Analysis of Playa Performance (ASAPP) model was developed to assess the performance of playas and pumping systems as they exist today and evaluate storm water management alternatives involving playas. ASAPP is an interactive program written in the Basic programming language specifically for this Master Plan. Playa storage fluctuations are simulated in ASAPP by mass balance calculations considering daily inflow, direct precipitation, infiltration, evaporation, and pumpage rates. Playa performance simulations conducted for a series of historical storm events at Lawrence Lake serve as the basis for model verification.

Playa performance subject to existing and future watershed development, playa dimensions, and pumping operations are evaluated rapidly using ASAPP. ASAPP tabulates annual maximum storage attained in each playa through daily contents simulation over the 93 years of available precipitation data. A Pearson type III distribution is fitted to the annual data and used to estimate the maximum flood levels for the two-, five-, 10-, 25-, 50-, and 100-year return intervals. ASAPP assists in the design of hydrologic structures by providing the tailwater condition at the playa for the design return period. Finally, ASAPP is used to evaluate the effects of encroachment and/or modifications to the playas for the purpose of reducing the frequency of flooding at predetermined elevations.

2.1.2 General Playa Operations

Playa storage volume increases as a result of runoff from the contributing drainage basin and precipitation falling on the lake surface. Runoff is modeled by ASAPP using historical daily precipitation and U. S. Soil Conservation Service runoff methods. In this manner, the basin-specific runoff characteristics are used to define a runoff curve number. Curve numbers for the area contributing to each of the 35 playas studied were determined from soil surveys and land use maps. These curve numbers, which are tabulated in Appendix B, have been incorporated into the ASAPP data files.

A third and relatively minor source of playa inflow is ground water intrusion. Ground water intrusion has been observed at Lawrence Lake and has been attributed to deep excavation of the playas. Intrusion has not been observed to be a significant contributor at other playas, however. Therefore, the ASAPP model does not consider ground water intrusion, although groundwater levels should be taken into consideration when planning the depth of a playa excavation.

The natural mechanisms for playa storage depletion are evaporation and infiltration. Evaporation is computed in the ASAPP model using historical monthly evaporation for the Amarillo area and simulated daily surface areas. An infiltration rate of 0.038 acre-feet per inundated acre per day was computed from a statistical analysis of playa level observations provided by the City. Observations of playa levels indicate that seepage rates are greater when playas are full than when they are at lower elevations. A possible physical explanation of this phenomenon is clay sediment accumulation in the bottom of playas which creates a region of lower permeability. Simulation runs using ASAPP revealed that the model tracked historical levels more closely if infiltration was only considered when the water level in a playa was more than three feet deep.

Other mechanisms for playa storage depletion are pumping, and drainage through tunnels or open channels. Pumping is currently practiced at four playas. The ASAPP model simulates pumpage based on the practice of not pumping during rain days to avoid overloading drainage conduits and channels. Pumping rates are varied with head based on a user-supplied rating curve. Tunneling and ditching options are not directly simulated by ASAPP.

2.1.3 Evaluating Playa Improvements Using ASAPP

Performance of each of the 35 study playas was simulated with ASAPP under existing and ultimate basin development and playa physical characteristics. Statistical summaries of these simulations are presented in Tables A-1, A-2, and A-3 in Appendix A.

2.1.4 Criteria for Selecting Playa Improvements

Each playa was assigned a primary and secondary damage elevation. These elevations were determined by City staff from a field survey of low lying building slab elevations or other significant damageable property near the playa. Based on ASAPP simulations of each of these playas under ultimate basin development conditions, playas were targeted for improvements when the 100-year water level exceeded the primary damage elevation. Using ASAPP, alternative flood control measures such as additional pump capacity, excavation, and a combination of the two measures were considered to identify appropriate playa improvement projects. Where arterial roads were inundated by a playa lake, a recommendation was made to raise the roads to a minimum elevation corresponding to the 25-year playa level.

In some cases, the improvements necessary to meet the 100-year criterion are quite costly. In these cases, City staff may want to review the value of the structures and properties which establish the damage elevations to decide whether a relaxed criterion is appropriate or whether the value of the benefits provided outweigh the costs of protection. For purposes of consistency in this Master Plan, however, all proposed playa improvements identified are based on the 100-year flood criterion noted above. Project descriptions and cost estimates for those playas needing improvements are described in Section 3. A summary of the resulting flood levels for each of the "problem playas" is found in Table A-3 in Appendix A.

2.2 Storm Sewer Systems

A significant portion of the runoff generated within the City of Amarillo is intercepted by storm sewers and curb inlets. Storm sewers were first added to the downtown Amarillo area in the early 1920s. These sewers drained north to East Amarillo creek into a 96-inch diameter outfall. Since then, several extensions and new storm sewer systems have been constructed as Amarillo has grown and new streets and highways have been constructed. About 40 storm sewer systems, ranging in size from one inlet and lateral to several miles of mains and laterals, drain the City of Amarillo. Many of these storm sewers drain to playas or East or West Amarillo Creeks.

Initially, 19 storm sewer systems with flooding problems were identified for detailed study. Of these 19 problem areas, two areas were studied using a HEC-2 water surface profile model, three underpass areas were studied considering some type of pump system arrangement, existing storm sewer systems were analyzed for 11 areas, and three areas with no storm sewer systems were analyzed regarding the addition of such systems.

2.2.1 Flood Hydrology

In order to evaluate the flood conditions in each of the 19 problem areas, base maps were prepared which show the extent and characteristics of the storm sewer systems, including existing drainage areas, land use, topography, discharge points, junction boxes, outfalls, and other drainage features. From these working maps, drainage areas were assigned to inlets and selected points in the storm sewer system. Flows were determined using nomographs described in the Storm Water Management Criteria Manual, a method which required data on the watershed slope, runoff curve number, and drainage area. An inlet flow summary table was prepared for each study area and inlet capacities checked. A separate system flow summary table was prepared which established accumulated flows in the storm sewer systems for the two-year and 100-year storm events. These flows were then used to analyze system capacities and required improvements.

2.2.2 Hydraulic Grade Lines

The storm sewer modeling program "HYDRA" was used to analyze the existing storm sewer systems and establishing their capacities. "HYDRA" is described in detail in the Storm Water Management Criteria Manual for Amarillo. Essentially, a storm sewer system is physically modelled by describing a network of links and nodes. Parameters are attached to each link (pipe) to define the pipe material type, diameter, Manning's "n" value, invert elevation up and down, natural ground elevations up and down, etc. Additional global parameters, such as limiting velocities, acceptable diameters, slopes, flows, etc. are used when analyzing the modelled system.

Another useful feature of the "HYDRA" program is the surcharge module, which determines an acceptable hydraulic grade line for storm sewers flowing at capacity. Storm sewer inlets can be surcharged to some extent and still pass a given frequency storm event. The program determines, assuming a gravity flow condition only, whether an existing storm sewer is adequate or inadequate to pass a given design flow. The "HYDRA" output generally defines a replacement or parallel pipe to meet the flow conditions requested.

2.2.3 Criteria Used for Analysis

Criteria used for analyzing the storm sewer systems is based on the Storm Water Management Criteria Manual for Amarillo and current design practices used by the City Engineering Department. Specifically, storm sewers were analyzed to check the two-year top-of-curb and the 100-year, 18 inches over top-of-curb conditions. Street capacities were determined from existing plans or from street slopes measured from topographic maps. For a standard 37-foot wide residential street at a 0.3 percent grade, the street capacity was about 10 to 12 cubic feet per second. Street capacities were deducted from the runoff values determined at selected discharge points. Other criteria used for analysis included using a minimum storm sewer slope of 0.1 percent, minimum diameter of 18 inches, minimum allowable velocity of two feet per second, and a minimum cover of two feet. This criteria is commonly used as a guide when designing new storm sewer systems.

2.2.4 Evaluating Proposed Improvements

Proposed improvements involved replacing or paralleling existing laterals and trunklines where appropriate, and adding new storm sewer lines where necessary to intercept runoff and reduce street flooding. In some cases, it was more cost-effective to divert upstream areas away from downstream areas in large storm sewer systems. Construction cost estimates for proposed storm sewer improvements were prepared for all problem areas. These estimates were based in part on Texas Department of Transportation (TxDOT) statewide summaries and TxDOT District 4 updates. Estimates for storm sewer replacement, inlet replacement, channel excavation, street regrading, rip-rap protection, and seeding were tabulated for each problem area.

It is suggested that the City of Amarillo consider cost apportionment with TxDOT on some of the proposed storm sewer improvements. This cost sharing worked successfully with the 102-inch outfall to Gooch Lake. Three of the proposed storm sewer improvement projects involve upgrading or tying into TxDOT storm sewers. Other revenue sources may include Potter or Randall Counties.

2.3 Open Channels

The study areas for East Amarillo Creek and West Amarillo Creek cover a large portion of northwest Amarillo. The watershed area for the two study areas encompasses 30 square miles; 16.5 square miles in the East Amarillo Creek basin and 13.5 square miles in the West Amarillo Creek basin. Selected channel reaches where flood conditions were evaluated total 24 miles in length and include over 80 bridges, culverts, and other special hydraulic structures.

2.3.1 Flood Hydrology

For the purpose of evaluating flood conditions in each of the two study areas, a hydrologic model was developed. Flood hydrographs were computed at various locations for ultimate land use conditions using a rainfall-runoff model which simulates a watershed's response to precipitation. The U.S. Army Corps of Engineers Flood Hydrograph Package, HEC-1 (HEC, 1990), was used to model the flood hydrology of the watersheds. The model simulates the rainfall-runoff process and computes runoff hydrographs, peak discharges, and runoff volumes at various locations in the watershed.

The HEC-1 model has numerous options for generating and routing flood hydrographs. The Soil Conservation Service (SCS) methodology was selected as the most appropriate option to generate flood hydrographs for the City of Amarillo. Information required by the HEC-1 model includes:

- watershed area
- precipitation amounts
- runoff curve number
- basin lag time; and
- channel and reservoir routing parameters.

2.3.2 Watershed Area

The watersheds for East Amarillo Creek and West Amarillo Creek were subdivided into smaller areas called subbasins in order to create a more detailed hydrologic model and to compute peak flows at a variety of locations. The East Amarillo Creek watershed was subdivided into 36 subbasins ranging in size from 0.03 square miles to 2.76 square miles. The West Amarillo Creek watershed was subdivided into 46 subbasins ranging in size from 0.02 square miles to 2.24 square miles.

2.3.3 Precipitation Amounts

In order to develop flood hydrographs for various return interval events, precipitation amounts that correspond with each of these events were modelled using HEC-1. Point rainfall amounts for the two-year, five-year, 10-year, 25-year, 50-year, and 100-year flood events were developed. These values were used in HEC-1 to develop design storms for determining runoff hydrographs. The storm rainfall was distributed using the "balanced storm" procedure in HEC-1, which creates a triangular shaped hyetograph from the given rainfall depths. Areal rainfall reduction factors were used in the model to reduce the point rainfall amounts to an average depth for each study area. HEC-1 reduces the point rainfall amounts according to recommendations in Weather Bureau TP-40. A point rainfall depth versus duration summary for Amarillo is given in Table 2.3-1.

TABLE 2.3-1
DEPTH-DURATION-FREQUENCY DATA FOR AMARILLO, TEXAS

Duration (min)	Rainfall Depth (inches)					
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr
5	0.45	0.54	0.60	0.69	0.77	0.84
10	0.69	0.86	0.97	1.14	1.28	1.41
15	0.85	1.08	1.23	1.46	1.64	1.81
30	1.18	1.53	1.77	2.12	2.38	2.65
60	1.52	2.00	2.33	2.80	3.16	3.52
120	1.65	2.15	2.48	3.00	3.37	3.80
180	1.80	2.30	2.65	3.20	3.63	4.01
360	2.15	2.75	3.15	3.65	4.20	4.70
720	2.40	3.10	3.55	4.05	4.70	5.20
1,440	2.72	3.45	3.96	4.49	5.23	5.79

2.3.4 Runoff Curve Number

The runoff curve number indicates the runoff potential of a hydrological soil-cover complex. The curve number is based on soil type, antecedent moisture condition, and land use. For each of the drainage basins, a runoff curve number was determined based on the soil type and ultimate land use condition. The soils in the study area are predominantly characterized as hydrologic soil group D (SCS, 1972), which have high runoff potential and very low infiltration rates. Generally, these soils consist chiefly of clay soils or soils with a claypan or clay layer at or near the surface. For the two-year through 100-year flood events, average antecedent moisture conditions (AMC-II) were assumed.

2.3.5 Basin Lag Time

The lag time, which is the length of time from the centroid of rainfall excess to the peak of the runoff hydrograph, determines the shape of the runoff hydrographs. The lag time is related to the basin length, shape, and slope. Lag times were computed for each of the subbasins using procedures described in SCS TR-55(SCS, 1986).

2.3.6 Channel and Reservoir Routing Parameters

Routing of flood flows through channel reaches and reservoirs was accomplished using the Modified Puls method in HEC-1. Stage-storage-discharge relationships were developed for each channel routing reach using results obtained from the computed water surface profiles. Stage-storage-discharge relationships were also developed for each reservoir by computing a stage-outflow relationship for each dam and combining it with the stage-storage relationship for the upstream reservoir.

2.3.7 Water Surface Profiles

Water surface profiles for each selected channel reach were computed for the two-year, five-year, 10-year, 25-year, 50-year, and 100-year flood events. The U.S. Army Corps of Engineers HEC-2 Water Surface Profiles model was utilized to compute each of these water surface profiles. The HEC-2 model assumes one-dimensional, gradually-varied, steady flow and requires:

- channel cross section geometry;
- bridge/culvert geometry;
- flow lengths;
- Manning's roughness coefficients (*n*);
- dam and reservoir characteristics; and
- discharges.

Channel cross section geometry and flow lengths were obtained from survey data provided by the City of Amarillo and from two-foot contour maps. Bridge and culvert geometry was surveyed at each stream crossing in the study reach or obtained from design plans if recently constructed. Manning's roughness coefficients were selected based on field observations and interpretation of aerial photographs. Dam and reservoir characteristics were based on survey information for the dam and spillway structures, while data for the upstream reservoir was developed from the two-foot contour maps. Discharge values used in the model for the two-year through 100-year flood events were taken from the results of the HEC-1 model.

2.3.8 Evaluating Channel Improvements

Evaluating flood conditions for each channel reach involved reviewing the calculated flood levels and comparing them to the existing geometry of bridges and culverts, top of roadway elevations, and existing permanent structure damage elevations. Damage elevations for structures were determined using survey data at most sites. Where survey data was not available, damage elevations for the structures were estimated by using the two-foot contour maps. In general, the following criteria was applied to determine if improvements were required at specific locations in a channel reach:

Open Channels: Open channels must be designed so that residential dwellings or public, commercial, and industrial buildings are not inundated at the lowest finished floor elevation for the 100-year storm event unless the building is flood-proofed.

Culverts and Bridges: Culverts and bridges must be designed so that there is no overtopping of the associated roadway for the 25-year storm event. For the 100-year storm event, overtopping of local and collector streets must not exceed 18 inches and overtopping of arterial streets must not exceed 12 inches. In addition, for the 100-year storm event, residential dwellings or public, commercial, and industrial buildings must not be inundated at the lowest finished floor elevation unless the building is flood-proofed.

If any of the criteria was violated at a specific location, improvement alternatives were evaluated at that location in order to comply with the criteria.

3.0 STUDY AREAS AND IMPROVEMENT ALTERNATIVES

3.1 Playa No. 1 Study Area (P1)

3.1.1 Existing Problems

Playa No. 1

Playa No. 1 is located on Helium Road just south of S.W. 34th Avenue. The playa drainage basin, which totals 918 acres, is essentially undeveloped. Playa No. 1 occasionally overflows to Playa No. 3, located approximately one mile to the southeast. The Playa No. 1 overflow elevation is estimated at 3,738.5 feet. Under existing basin development conditions, the ASAPP model indicates that this elevation is reached under the 25-year storm event. Since the ASAPP model is not equipped to perform storage routing of a storm hydrograph, precise flood elevations for events in excess of the 25-year have not been determined. However, it can be stated that, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

A summary of existing levels for Playa No. 1 is found in Table A-1 in Appendix A.

3.1.2 Proposed Improvements

Playa No. 1

Under ultimate basin development conditions, the ASAPP model indicates that the overflow elevation of 3,738.5 feet would be reached under the 10-year event. Due to the lack of existing or proposed development in the Playa No. 1 area, no improvements for the basin are recommended in this Master Plan. Should development pressure occur, a hydrologic model of the basin should be prepared which accounts for the hydraulic behavior of the playa overflow. Accurate playa flood elevations can then be established by storage routing a full series of storm events through the playa.

A summary of ultimate levels for Playa No. 1 is found in Table A-2 in Appendix A.

3.2 Playa No. 3 Study Area (P3)

3.2.1 Existing Problems

Playa No. 3

Playa No. 3 is located just north of S.W. 45th Avenue between Helium Road and Soncy Road. The playa drainage basin, which totals 1,058 acres, is essentially undeveloped. Playa No. 3 occasionally collects overflow from Playa No. 1, located approximately one mile to the northwest at Helium and S.W. 34th Avenue. The Playa No. 1 basin area of 918 acres is not part of the Playa No. 3 area of 1,058 acres. A survey of the Playa No. 3 perimeter revealed a primary damage elevation of 3,723.5 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,706.9 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year flood event, including events in excess of the 25-year which result in overflows from Playa No. 1.

3.2.2 Proposed Improvements

Playa No. 3

Under ultimate basin development conditions, the 100-year flood level for Playa No. 3 is 3,710 feet, well below the existing primary damage elevation of 3,723.5 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 3 area, no improvements for the basin are recommended in this Master Plan. Should development pressure occur, a hydrologic model of the Playa No. 1/Playa No. 3 system should be prepared which accounts for the hydraulic behavior of the Playa No. 1 overflow. A backwater profile model should then be prepared to estimate the water surface elevations of the overflows from Playa No. 1 to Playa No. 3 for a full range of flood events.

A summary of ultimate levels for Playa No. 3 is found in Table A-2 in Appendix A.

3.3 Playa No. 4 Study Area (P4)

3.3.1 Existing Problems

Playa No. 4

Playa No. 4 is located on Helium Road just south of Hillside Road. The playa drainage basin, which totals 1,801 acres, is essentially undeveloped and a survey of the playa perimeter revealed no significant damage points. Under existing basin development conditions, the 100-year playa level was found to be 3,698.3 feet, below the playa top elevation of 3,702 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year flood event.

A summary of existing levels for Playa No. 4 is found in Table A-1 in Appendix A.

3.3.2 Proposed Improvements

Playa No. 4

Under ultimate basin development conditions, the 100-year level for Playa No. 4 is 3,698.9 feet, well below the playa top elevation of 3,702 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 4 area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 4 is found in Table A-2 in Appendix A.

3.4 Playa No. 5 Study Area (P5)

3.4.1 Existing Problems

Playa No. 5 (McDonald Lake)

Playa No. 5 is located on Coulter Street just south of S.W. 45th Avenue. The playa drainage basin, which totals 1,539 acres, is primarily undeveloped with some low-density residential development (LDR). A survey of the playa perimeter revealed a single damage point at elevation 3,696.8 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will not be reached by the 100-year flood event. However, Coulter Street, with a low elevation of approximately 3,687 feet, would be inundated by any event greater than the 10-year event.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

Coulter Storm Sewer Problem Area (CO)

The Coulter Street storm sewer system is located in southwest Amarillo between Hillside Avenue and S.W. 45th Avenue along Coulter Street. At the time of this writing (Spring 1993), curbs and gutters exist only on the east side of Coulter; therefore, the storm sewer collects runoff from inlets along that side of the street. The storm sewer has been oversized to add inlets in the future on the west side of Coulter, whenever curbs and gutters are built. The outfall is a 72-inch reinforced concrete pipe to McDonald Lake.

The contributing drainage area intercepts about 1,056 acres prior to the outfall to McDonald Lake. Development is mixed with some residential areas north of S.W. 45th Avenue and west of Coulter and a largely undeveloped area west of Coulter and south of S.W. 45th Avenue Watershed slopes vary from zero to one percent.

3.4.2 Proposed Improvements

Playa No. 5 (McDonald Lake)

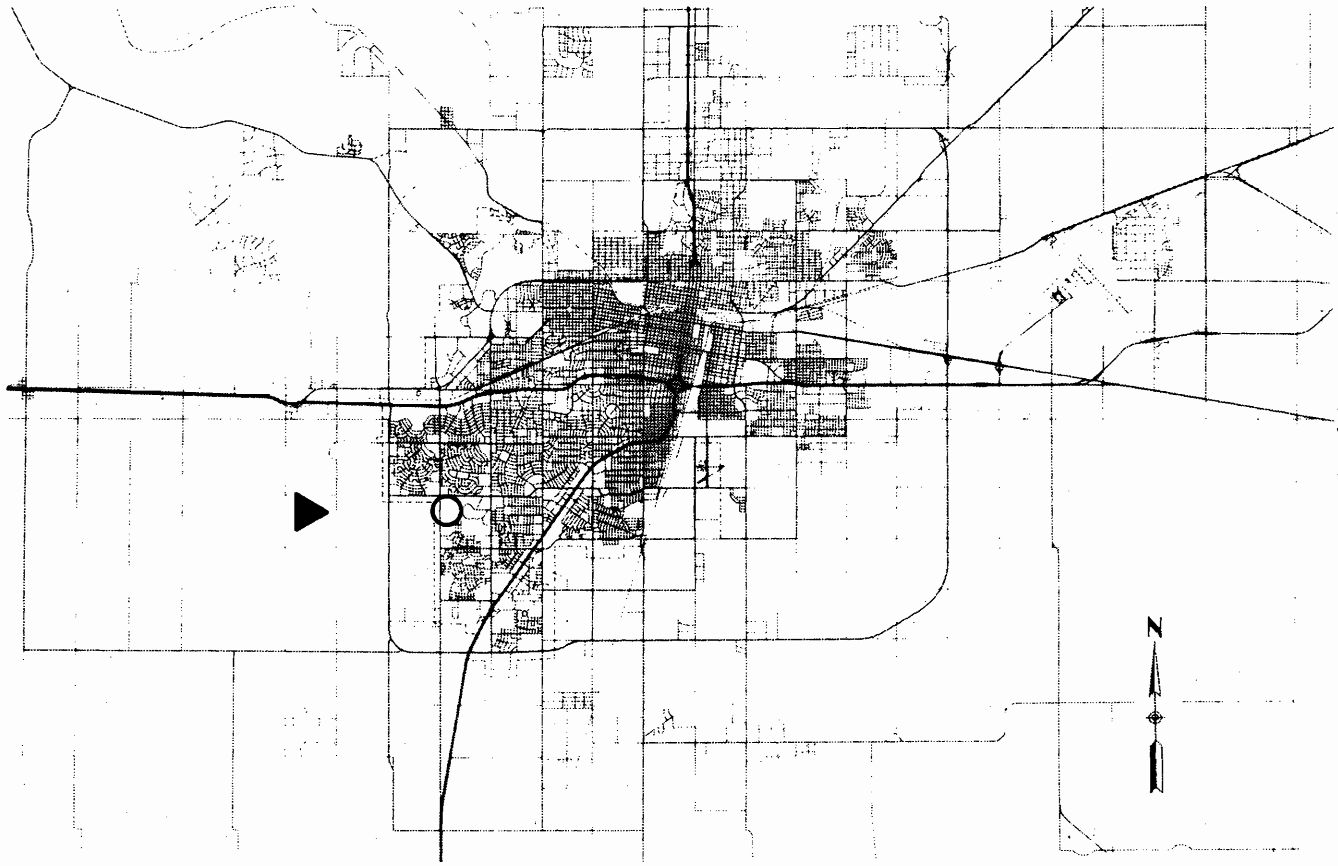
Under ultimate basin development conditions, the primary damage elevation will not be reached by the 100-year flood event. However, Coulter Street will continue to be inundated by any event greater than the 10-year event. A summary of ultimate levels for Playa No. 5 is found in Table A-2 in Appendix A.

Improvements are scheduled to convert the McDonald Lake site to a city park. The "Southwest Regional Park Master Plan" calls for the excavation of a portion of the playa in order to establish a permanent park lake. In order to maintain the lake pool as desired by the City and protect Coulter Street from the 100-year flood event, a new 2,000 gpm playa pump station is proposed for McDonald Lake. Proposed improvements for Playa No. 5 are summarized in Table 3.4-1 and their location shown in Figure 3.4-1.

TABLE 3.4-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McDONALD LAKE STUDY AREA (P5)
McDONALD LAKE (PLAYA NO. 5)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P5-A		Excavate playa per "Southwest Regional Park Master Plan"	350,000	CY	$4.00	$1,400,000
P5-B	P5-C	New 2,000 gpm pump station with 14" suction line		LS		$150,000
P5-C	P5-D	New 12" force main	9,700	LF	$40.00	$388,000
					TOTAL	$1,938,000

(See Drawing Set Sheet No. P5-1)



STUDY AREA: PLAYA NO. 5 "P5"

PROJECT NAME: McDONALD LAKE

DESCRIPTION: NEW PUMP STATION AND FORCE MAIN/EXCAVATE PLAYA

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P5)

FIGURE 3.4-1

A summary of ultimate levels for Playa No. 5 under the proposed improvements scenario is found in Table A-3 in Appendix A.

Coulter Storm Sewer Problem Area (CO)

Proposed improvements to the Coulter Street storm sewer are the extension of a main around the proposed Wal-Mart site and up Van Winkle Drive to the vicinity of Tripp Avenue. This line will reduce the areas intercepted by existing inlets and take runoff away from the intersection of Coulter and S.W. 45th Avenue. Remaining improvements consist of paralleling and replacing portions of the existing storm sewer along Coulter Street between S.W. 45th Avenue and Hillside Avenue.

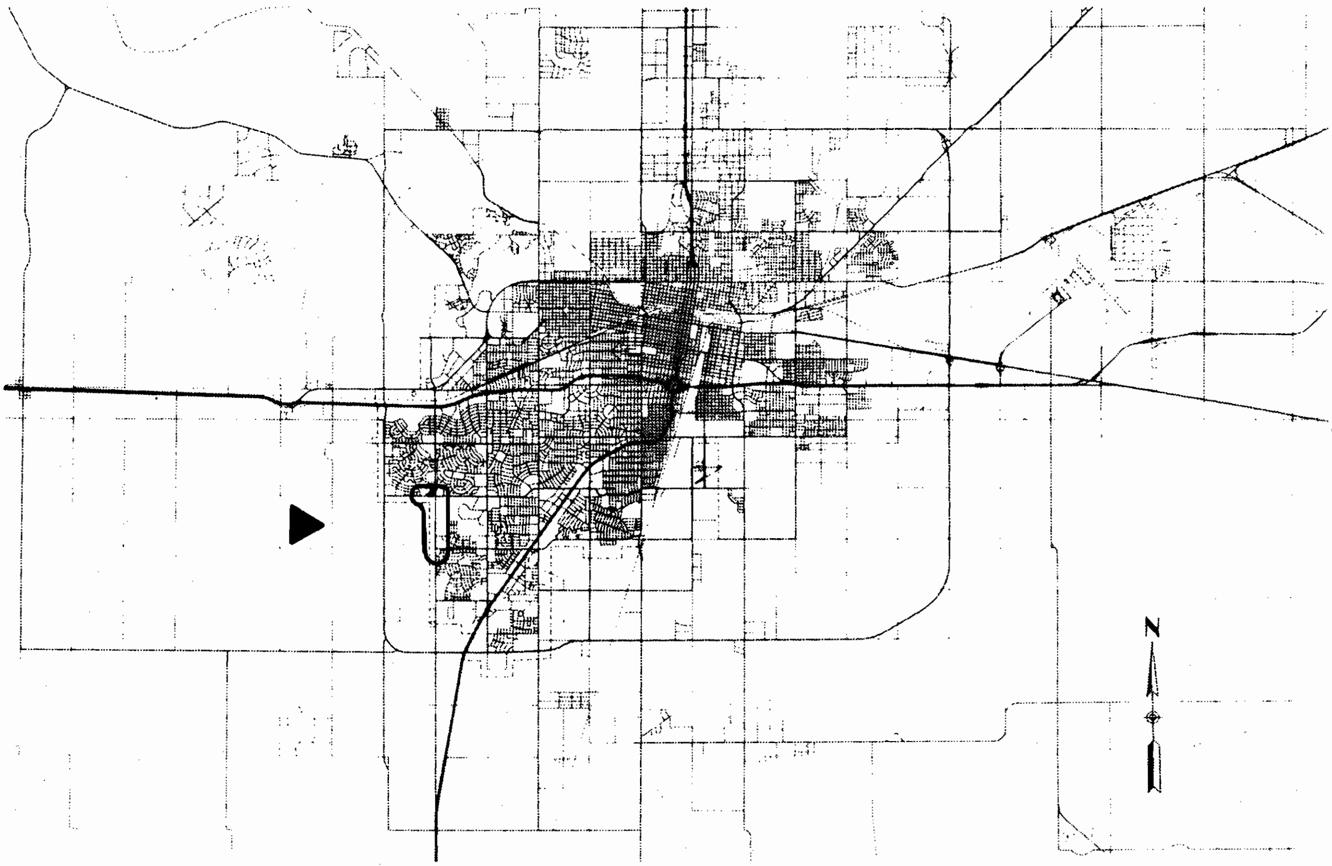
The existing storm sewers were analyzed using the "HYDRA" storm sewer modelling program to meet the two year top-of-curb condition. Runoff to the system was analyzed using flow curves developed for the City of Amarillo based on drainage area, curve numbers, and watershed slope. The estimated top-of-curb hydraulic capacity of streets as compiled by the City of Amarillo was computed assuming streets are 65-foot-wide collectors, with slopes at 0.003 ft/ft. Only one-half of Coulter Street has been completed, so flows for one-half of a 65-foot collector were computed. Other assumptions included using a minimum storm sewer slope of 0.001, minimum diameter of 18 inches, minimum allowable velocity of two feet per second, and a minimum cover of two feet. It was assumed that what the street does not carry, the storm sewer will. Inlet capacities were increased in some cases at the upstream end of the storm sewer system to capture runoff and put it into the system.

The proposed improvements for the Coulter Storm Sewer System are summarized in Table 3.4-2 and their location shown in Figure 3.4-2.

TABLE 3.4-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McDONALD LAKE STUDY AREA (P5)
COULTER STORM SEWER (CO)

INLET/NODE		PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
FROM	TO					
STORM SEWER IMPROVEMENTS						
CO-B	CO-C	---	1267	LF	$0	$0
CO-C	CO-D	---	925	LF	$0	$0
CO-D	CO-E	Parallel w/36" RCP	383	LF	$79	$30,257
CO-E	CO-F	Parallel w/36" RCP	258	LF	$79	$20,382
CO-F	CO-G	Parallel w/36" RCP	453	LF	$79	$35,787
CO-G	CO-H	Parallel w/36" RCP	400	LF	$79	$31,600
CO-H	CO-I	Parallel w/36" RCP	400	LF	$79	$31,600
CO-KF	CO-KE	New 42" RCP	1000	LF	$94	$94,000
CO-KE	CO-KD	New 48" RCP	350	LF	$111	$38,850
CO-KD	CO-KC	New 48" RCP	800	LF	$111	$88,800
CO-KC	CO-KB	New 54" RCP	400	LF	$135	$54,000
CO-KB	CO-I	New 54" RCP	2,600	LF	$135	$351,000
CO-KA	CO-K	New 30" RCP	500	LF	$58	$29,000
CO-K	CO-J	--	800	LF	$0	$0
CO-J	CO-I	--	400	LF	$0	$0
CO-I	CO-L	Replace w/84" RCP	130	LF	$394	$51,220
					SUBTOTAL	$856,496
INLET IMPROVEMENTS						
		4' Inlets	3	EA	$1,900	$5,700
		5' Inlets	2	EA	$2,200	$4,400
		10' Inlets	4	EA	$2,700	$10,800
		15' Inlets	1	EA	$3,100	$3,100
		20' Inlets	4	EA	$3,600	$14,400
		25' Inlets	2	EA	$4,100	$8,200
		30' Inlets	2	EA	$4,600	$9,200
					SUBTOTAL	$55,800
MANHOLES						
		Manholes	8	EA	$3,500	$28,000
					SUBTOTAL	$28,000
					TOTAL	$940,296

(See Drawing Set Sheet No. P5-1)



STUDY AREA: McDONALD LAKE "P5"

PROJECT NAME: COULTER STREET "CO"

DESCRIPTION: EXTEND STORM SEWER/REPLACE EXISTING STORM SEWER/ADD NEW INLETS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P5-CO)

FIGURE
3.4-2

3.5 Playa No. 6 Study Area (P6)

3.5.1 Existing Problems

Playa No. 6 (Lawrence Lake)

Playa No. 6 is located southeast of the intersection of Interstate 40 and Western Street. The playa drainage basin, which totals 5,838 acres, is fully developed, with the primary use being low-density residential (LDR). Flooding problems have been reported at Lawrence Lake. Currently, two vertical turbine pumps, one at the west end of the playa (Western Plaza) and one at the east end (Royal Inn), are used to lower the playa level during dry weather. Pumped discharges from Bennett Lake (Playa No. 17) are routed to Lawrence Lake. In addition, approximately 1,100,000 cubic yards of material were recently excavated from Lawrence Lake to create additional storage volume. A survey of the playa perimeter revealed that damages occur when playa levels reach elevation 3,624.5 feet. Results of the ASAPP model indicate that, under existing watershed and pumping conditions, this damage elevation will be reached by the 47-year flood event.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

Seven major storm sewer systems drain to the Lawrence Lake playa. Five of these systems were studied in detail and are described in the following sections.

Fleetwood Storm Sewer Problem Area (FW)

The Fleetwood/Terrace storm sewer system drains a large part of western Amarillo. With the exception of the downtown storm sewer system, it is one of the City's most extensive systems. The Fleetwood system originates near Coulter Street and S.W. 34th Avenue, draining east down Fulton Drive across Bell Street. At Bell Street, another 24-inch lateral connects from the north. The trunk line turns southeast at Teckla Boulevard and northeast at Ridgecrest Circle and continues across Western before turning northeast down Terrace Drive. A junction box near Terrace and Western connects 48-inch and 42-inch laterals to 72-inch and 60-inch outfalls. The 72-inch and 60-inch outfalls parallel each other to S.W. 34th Avenue and connect another 24-inch lateral. At S.W. 34th Avenue, the storm sewer turns due north down Fleetwood to Lawrence Lake. The 72-inch line increases in size to 78 inches and 84 inches before draining to the outfall at Lawrence Lake. The 60-inch line is continuous from Western and Terrace to Lawrence Lake.

Originally, the system consisted of only the 60-inch line to Lawrence Lake. In the late 1970's, this line was paralleled by the 72-inch, 78-inch, and 84-inch lines. From as-built plans, it appears the two trunklines were interconnected at three points to provide hydraulic equalization.

The Fleetwood/Terrace storm sewer system intercepts a contributing drainage area of about 1,024 acres at the outfall to Lawrence Lake. Almost all of the watershed is fully developed as residential areas with some commercial development around Lawrence Lake. Watershed slopes vary from 0.5 to two percent.

Flooding problems occur along Terrace and Fleetwood to Lawrence Lake. High lake levels cause a surcharge situation at inlets connected to the system, resulting in street flooding which can back up storm water into buildings with low slab elevations near Lawrence Lake.

S.W. 26th Avenue Storm Sewer Problem Area (TS)

The existing S.W. 26th Avenue storm sewer outfalls to Lawrence Lake as a 78-inch reinforced concrete pipe (RCP) and extends from Lawrence Lake up S.W. 26th Avenue to

Georgia Street, turning into a 60-inch RCP and continuing easterly to Elmwood Drive. Major laterals tying into the storm sewer consist of a 30-inch RCP at Patterson, a 30-inch RCP at Brittian, a 42-inch RCP from the north, a 48-inch RCP from the south at Georgia St., and a 24-inch RCP from the north and south at Elmwood Drive. Another drainage feature is S.W. 26th Avenue, which has been constructed with an inverted crown from Georgia Street to Elmwood Drive and basically serves as a channel to funnel runoff to Lawrence Lake. Street slopes are about 0.3 percent. Grate inlets have been added to the system east of Georgia Street.

The contributing drainage area is approximately 933 acres at the outfall to Lawrence Lake. Development in the watershed is primarily light industrial adjacent to Lawrence Lake and extending east to areas along Georgia Street. Areas east of Georgia Street are primarily residential with some scattered businesses. Watershed slopes are one to two percent for the most part.

Major problems occur as Lawrence Lake levels rise, blocking flow in the S.W. 26th storm sewer outfall. Runoff backs up into the inlets to Elmwood Drive, east of Georgia Street, during a 25-year or greater flood event. The result has been ponded water in many areas as runoff recedes. The invert elevation of the outfall is 3,609 feet at Lawrence Lake. When lake levels rise to elevation 3,616, problems in the S.W. 26th Avenue storm sewer occur. In the upper watershed, inadequate inlet capacity exists to carry the two-year and greater flows.

Another localized problem occurs at the intersection of S.W. 26th Avenue and Georgia Street. Since S.W. 26th Street has an inverted crown east of Georgia St., runoff "stacks up" at the intersection. Georgia Street has a regular crown for a 65' street. This "damming" effect backs up water easterly on S.W. 26th Avenue.

Olsen/Emil Storm Sewer Problem Area (OE)

The Olsen/Emil storm sewer begins as a 24-inch RCP near Western Street and S.W. 21st Avenue and drains south along Western until it reaches Olsen Boulevard At Olsen, the storm sewer picks up additional inlets and turns into a 60-inch RCP at Olsen and Western. The storm sewer drains to the east down Olsen until it joins an additional system coming in from Hobbs Road. From the junction box at Olsen and Hobbs Road, the system drains to an outfall on the southwest corner of Lawrence Lake.

The existing drainage area to the outfall is approximately 382 acres. Development in the watershed is mostly residential, with some commercial development along Western and Olsen near Lawrence Lake. Watershed slopes vary from 0.5 to two percent.

Flooding problems occur along Western Street when lake levels rise, causing street flooding. Considerable amounts of storm water runoff also occur west of Western Street, with several streets sloping down to Western Street, conveying substantial amounts of gutter flow. This effect adds to the street flooding during periods of high lake levels.

Julian Storm Sewer Problem Area (JU)

The Julian problem area is located in west Amarillo, north of Interstate 40 (I40). This area, which is not currently served by storm sewers, has experienced shallow street flooding along the westbound I40 service road. Rapid street flow from the north along Julian, Virginia, and Kentucky Streets to I40 cause flooding problems along the service road. The flooding problem is compounded by overloading of the storm sewer system along I40 and high lake levels in Lawrence Lake.

At the outfall to Lawrence Lake, the drainage area is approximately 1,269 acres (this area includes Dilday Draw, Julian, and areas along I40). Development in the watershed is primarily residential, with some commercial development along I40. Watershed slopes vary from 0.5 to two percent.

Dilday Draw Problem Area (DD)

The Dilday Draw storm sewer system is located in west Amarillo, north of I40. It starts as a 30-inch RCP at the intersection of 11th and Avondale. The storm sewer continues south down Avondale to 16th Avenue. At 16th Avenue, the storm sewer turns east down 16th to Broadmoor Street. At Broadmoor, the storm sewer turns south under the Burlington-Northern Railroad (BNRR) right-of-way and continues down Bellaire, to a natural depression called Dilday Draw. Minor channel improvements have been made to this natural depression. The storm sewer follows this depression from Bellaire to La Salle Streets. At La Salle Street, the storm sewer follows La Salle to Crouch to Western Street. At Western Street, the storm sewer turns south down Western as a 54-inch RCP and continues to I40, eventually connecting to a 72-inch RCP trunkline along I40. The 72-inch RCP trunkline joins with the Julian storm sewer before it outfalls to the northwest corner of Lawrence Lake.

The contributing drainage area is approximately 406 acres at the storm sewer connection of Dilday Draw to the I40 trunkline. At the outfall to Lawrence Lake, the drainage area is approximately 1,269 acres (this includes Dilday Draw, Julian, and areas along I40). Development in the watershed is primarily residential, with some commercial development along I40. Watershed slopes vary from 0.5 to two percent.

Flooding problems occur at the BNRR and at I40. Low points at the railroad embankment and shallow street flooding at Western and I40 have been experienced. This occurrence is due in part to overloading the storm sewer system along I40 and to high lake levels in Lawrence Lake.

3.5.2 Proposed Improvements

Playa No. 6 (Lawrence Lake)

Under ultimate basin development conditions, the primary damage elevation will be reached at the 47-year flood event. Under the 100-year event, the flood level in Lawrence Lake would reach 3,626.7 feet, 2.2 feet above the primary damage elevation. A summary of ultimate levels for Playa No. 6 is found in Table A-2 in Appendix A.

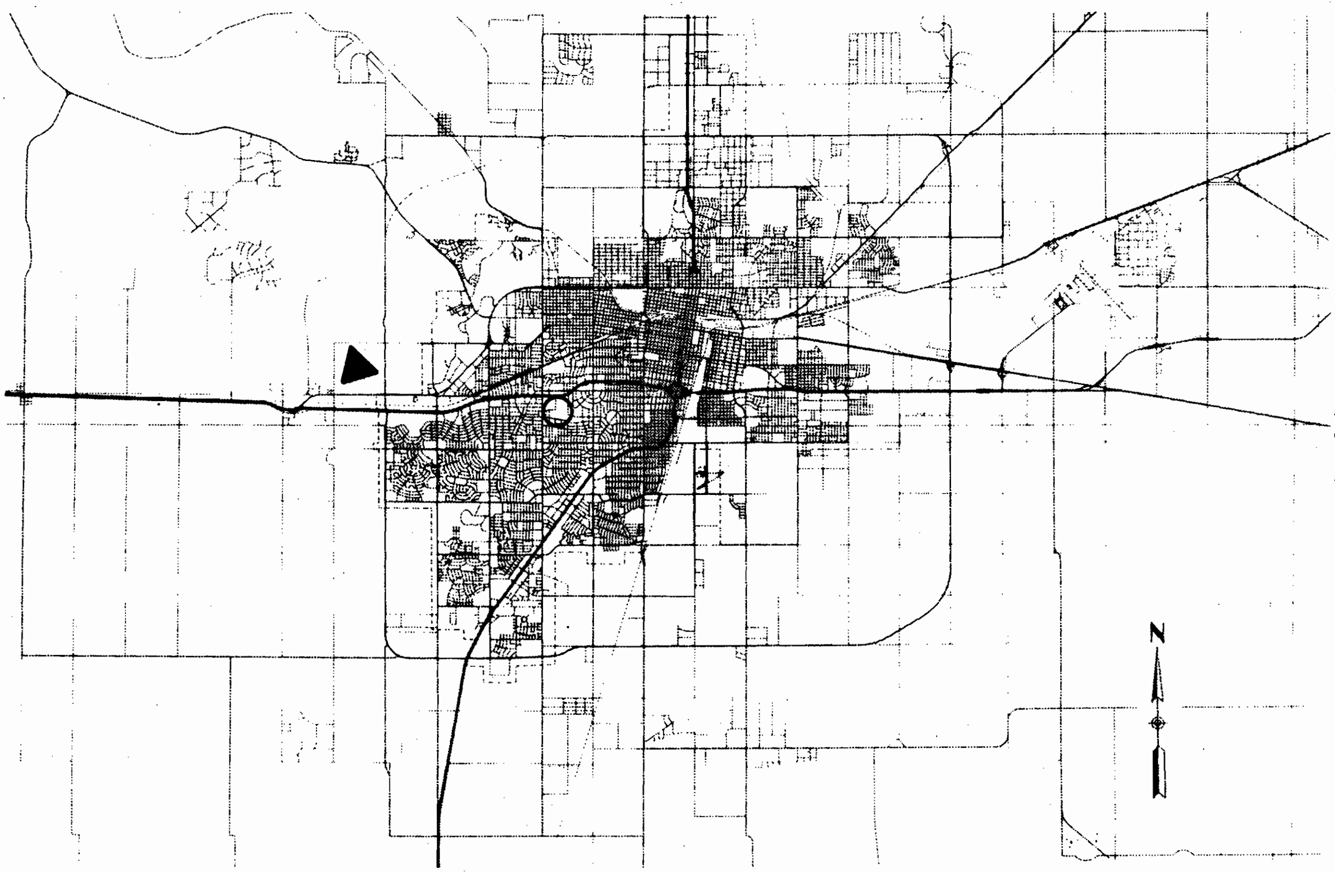
In order to provide 100-year protection at Lawrence Lake, the addition of two new 3,000 gpm pumps, one at the Western Plaza Pump Station and one at the Royal Inn Pump Station, is proposed. In addition, pumped discharges from Bennett Lake (Playa No. 17) would be diverted from Lawrence Lake and piped to the Spring Draw outfall in Southwest Amarillo (see Section 3.15.2). Proposed improvements for Playa No. 6 are summarized in Table 3.5-1 and their location shown in Figure 3.5-1.

A summary of ultimate levels under the proposed playa improvement scenario is found in Table A-3 in Appendix A.

TABLE 3.5-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
LAWRENCE LAKE (PLAYA NO. 6)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P6-A	P6-B	Add new 3,000 gpm pump at Western Plaza		LS		$140,000
P6-B	P6-C	Replace existing force main with 20"	6,500	LF	$65	$422,500
P6-D	P6-E	Add new 3,000 gpm pump @ Royal Inn		LS		$140,000
P6-E	P6-F	Replace existing force main with 24"	6,400	LF	$75	$480,000
					TOTAL	$1,183,000

(See Drawing Set Sheet No. P6-3)



STUDY AREA: PLAYA NO. 6 "P6"

PROJECT NAME: LAWRENCE LAKE

DESCRIPTION: UPGRADE PUMP STATIONS AND FORCE MAINS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P6)

FIGURE
3.5-1

Discharges from the Royal Inn Pump Station are currently directed to the downtown storm sewer system and hence to Thompson Park. When the Ong-Lipscomb storm sewer improvements are constructed, these discharges will continue to be directed to Thompson Park through a diversion structure at node OL-F (see Section 3.21.2). As an alternative to this diversion structure, the purpose of which is to prevent dry-weather pumped discharges from entering Wild Horse Lake, the City may wish to extend the force main from the Royal Inn Pump Station west to one of the tributaries to West Amarillo Creek, or east to another portion of the downtown storm sewer system.

Fleetwood Storm Sewer Problem Area (FW)

Proposed improvements for the Fleetwood storm sewer system include diverting the Fulton Street lateral to the Hillside/Hampton storm sewer system. This diversion would relieve some of the overload on the Fleetwood system by taking all of the Fulton system from Coulter and S.W. 34th Avenue and Coulter to Fulton and Bell and putting this flow into the Hillside/Hampton system near Bell and Fulton. About 360 acres are diverted from Lawrence Lake (Playa 6) to McCarty Lake (Playa 2) for the two-year and smaller events.

Other improvements would consist of upgrading the Bell Street lateral, replacing the Teckla Boulevard/Ridgecrest storm sewer, rerouting laterals on Western and Teckla Boulevard, and interconnecting the 60-inch and 72-inch RCP at three locations on Terrace Drive/Fleetwood. New and replacement inlets are proposed in several locations. The proposed improvements are summarized in Table 3.5-2 below and their location shown in Figure 3.5-2.

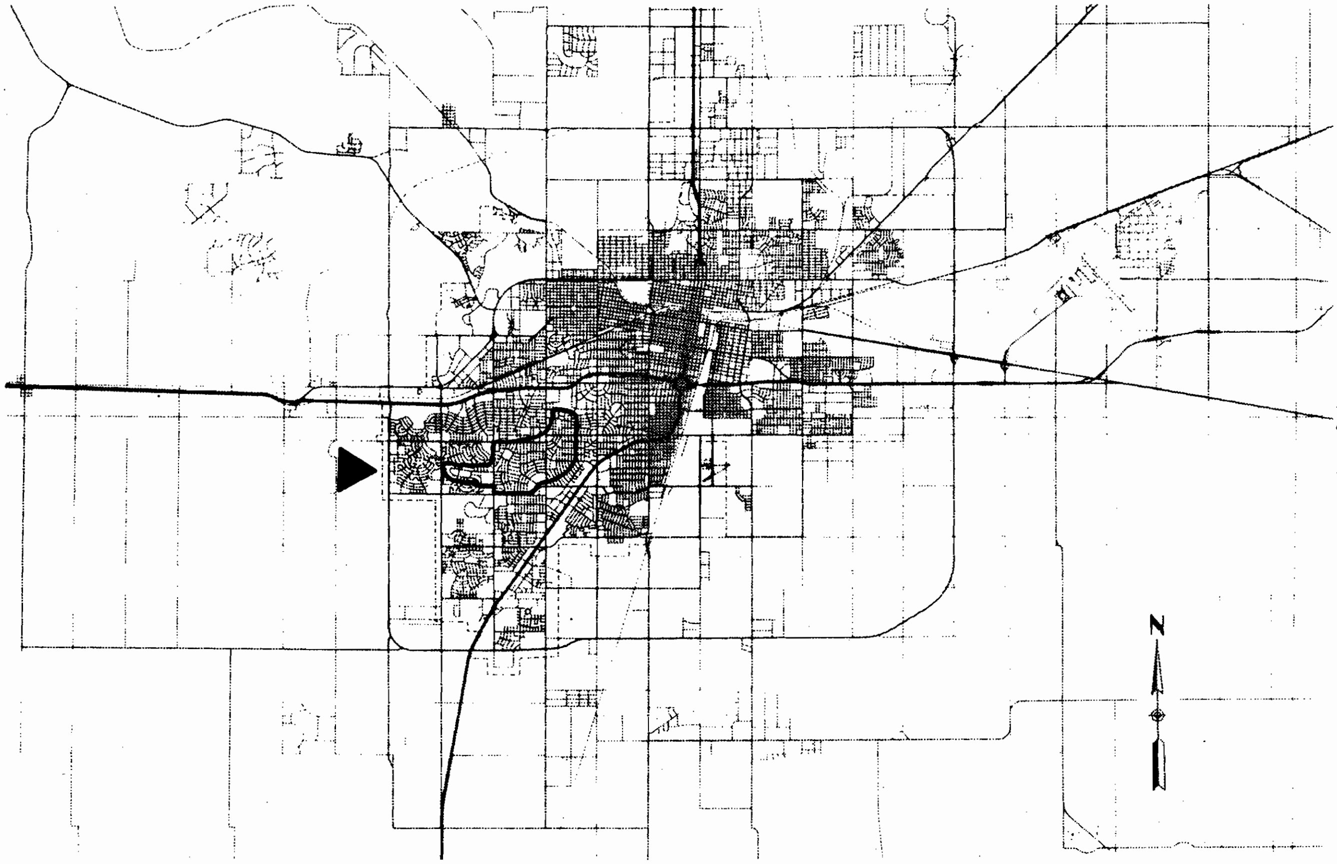
TABLE 3.5-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
FLEETWOOD STORM SEWER (FW)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
(Fulton Lateral FW-A to FW-K See Hillside/Hampton Table 3.13-2)						
FW-K	FW-L	Replace w/ 36" RCP	425	LF	$99	$42,075
FW-L	FW-J	Replace w/ 36" RCP	1,250	LF	$99	$123,750
FW-J	FW-M	Replace w/ 42" RCP	230	LF	$118	$27,140
FW-M	FW-N	---	400	LF	$0	$0
FW-N	FW-O	---	310	LF	$0	$0
FW-O	FW-P	---	320	LF	$0	$0
FW-P	FW-Q	---	1,320	LF	$0	$0
FW-Q	FW-R	---	240	LF	$0	$0
FW-R	FW-S	Replace w/ 60" RCP	286	LF	$204	$58,344
FW-S	FW-T	Replace w/ 60" RCP	320	LF	$204	$65,280
FW-T	FW-U	Replace w/ 60" RCP	350	LF	$204	$71,400
FW-T	FW-U	Replace w/ 60" RCP	120	LF	$204	$24,480
FW-U	FW-V	Replace w/ 60" RCP	1,080	LF	$204	$220,320
FW-W	FW-X	---	550	LF	$0	$0
FW-X	FW-Y	---	530	LF	$0	$0
FW-Y	FW-JB	---	633	LF	$0	$0
FW-Z	FW-JB	---	1,075	LF	$0	$0
FW-JB	FW-V	---	365	LF	$0	$0
FW-V	FW-A/O-N	---	811	LF	$0	$0
FW-Z	FW-U	New 30" RCP	300	LF	$58	$17,400
FW-W	FW-X	New 60" RCP	550	LF	$163	$89,650
FW-X	FW-N	New 60" RCP	900	LF	$163	$146,700
FW-A/O-N	FW-B/O-N	Replace w/ 72" RCP	345	LF	$266	$91,770
FW-B/O	FW-C/O	---	245	LF	$0	$0
FW-C/O	FW-H/O	---	1,324	LF	$0	$0
FW-H/O	FW-I/O	---	320	LF	$0	$0
FW-I/O	FW-J/O	---	320	LF	$0	$0
FW-J/O	FW-K/O	---	320	LF	$0	$0
FW-K/O	FW-L/O	---	320	LF	$0	$0

TABLE 3.5-2 (CONTINUED)
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
FLEETWOOD STORM SEWER (FW)

INLET/NODE FROM	INLET/NODE TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
FW-L/O	FW-M/O	--	1,148	LF	$0	$0
FW-M/O	FW-N/O	--	1,570	LF	$0	$0
FW-N/O	FW-O/O	---	800	LF	$0	$0
FW-O/O	OUTFALL	---	310	LF	$0	$0
FW-B/O-N	FW-D/N	---	82	LF	$0	$0
FW-B/O-N	FW-D/N	---	293	LF	$0	$0
FW-D/N	FW-E/N	---	439	LF	$0	$0
FW-E/N	FW-F/N	---	185	LF	$0	$0
FW-F/N	FW-G/N	Connect 60" & 72"	161	LF	$0	$0
FW-G/N	FW-H/N	---	314	LF	$0	$0
FW-H/N	FW-I/N	---	320	LF	$0	$0
FW-I/N	FW-J/N	---	318	LF	$0	$0
FW-J/N	FW-K/N	---	323	LF	$0	$0
FW-K/N	FW-L/N	---	368	LF	$0	$0
FW-L/N	FW-M/N	Connect 60" & 72"	1,150	LF	$0	$0
FW-M/N	FW-N/N	Connect 60" & 72"	1,610	LF	$0	$0
FW-N/N	FW-O/N	---	840	LF	$0	$0
FW-O/N	OUTFALL	---	310	LF	$0	$0
					SUBTOTAL	$978,309
INLET IMPROVEMENTS						
		10' Inlets w/ Lat.	7	EA	$2,700	$18,900
		15' Inlets w/ Lat.	5	EA	$3,100	$15,500
		20' Inlets w/ Lat.	3	EA	$3,600	$10,800
		25' Inlets w/ Lat.	2	EA	$4,100	$8,200
		30' Inlets w/ Lat.	3	EA	$4,600	$13,800
					SUBTOTAL	$67,200
MANHOLES/JUNCTION BOXES						
		Manholes	11	EA	$4,500	$49,500
		Junction Boxes	10	EA	$10,000	$100,000
					SUBTOTAL	$149,500
					TOTAL	$1,195,009

(See Drawing Set Sheet No. P6-4)



STUDY AREA: LAWRENCE LAKE "P6"

PROJECT NAME: FLEETWOOD DRIVE "FW"

DESCRIPTION: REPLACE STORM SEWERS/ADD NEW LATERALS
DIVERT FULTON LATERAL TO HILLSIDE/
HAMPTON SYSTEM

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P6-FW)

FIGURE
3.5-2

S.W. 26th Avenue Storm Sewer Problem Area (TS)

Proposed improvements for the S.W. 26th Avenue storm sewer system consist of enlarging the major trunkline from Elmwood Drive west to Georgia Street, paralleling the existing 84-inch storm sewer with an additional 60-inch storm sewer, and extending the existing laterals to intercept smaller contributing areas in the upper watershed. An overflow channel from Lakeview Street to Lawrence Lake is proposed. An additional improvement would consist of regrading the intersection of S.W. 26th Avenue and Georgia Street to eliminate the "damming" effect east of Georgia Street. These proposed improvements are summarized in Table 3.5-3 and their location shown in Figure 3.5-3.

Olsen/Emil Storm Sewer Problem Area (OE)

Proposed improvements to the Olsen/Emil storm sewer system consist of extending the existing system to intercept more runoff before it reaches Western Street. Proposed laterals would extend west from Olsen and Western to Olsen and Erik Avenue. Another lateral is proposed from Olsen and Harmony down Harmony and Emil to Emil and Anna Streets. Upstream interception of storm water should alleviate some of the downstream flooding for the two-year storm event. Additional inlet capacity is proposed, along with extension of the laterals into the upper watershed. These proposed improvements are summarized in Table 3.5-4 and their location shown in Figure 3.5-4.

Julian Storm Sewer Problem Area (JN)

Proposed improvements for the Julian storm sewer system consist of adding a major trunkline from Julian and I40 to Julian and West 15th Avenue, and a lateral and inlets along West 15th Avenue. Proposed improvements are summarized in Table 3.5-5 and their location shown in Figure 3.5-5.

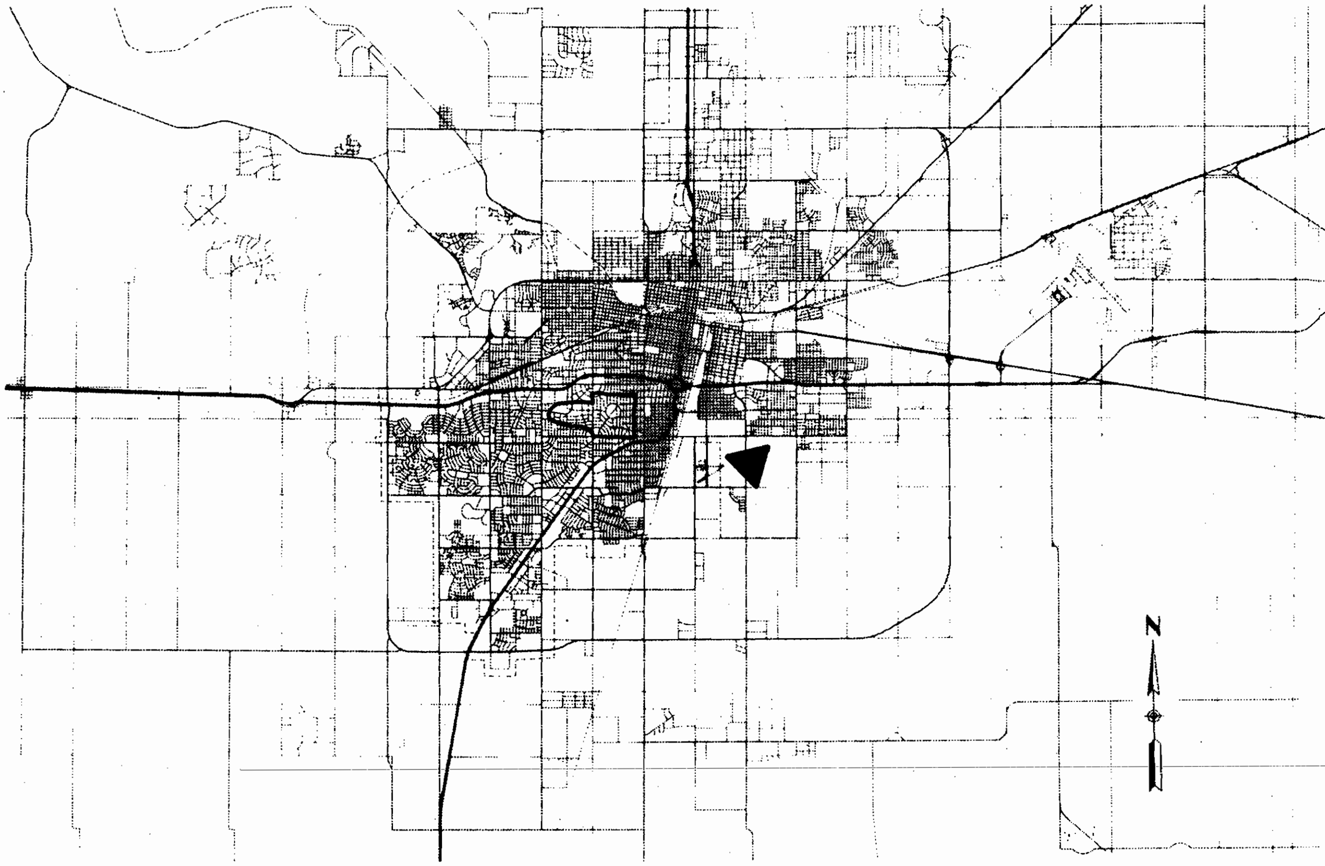
TABLE 3.5-3
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
S.W. 26th AVENUE STORM SEWER (TS)

INLET/NODE						
FROM	**TO**	**PROPOSED IMPROVEMENTS**	**NO. UNITS**	**UNIT**	**UNIT COST**	**TOTAL COST**
STORM SEWER IMPROVEMENTS						
TS-AC	TS-AB	New 36" diameter S.S.	850	LF	$79	$67,150
TS-AB	TS-AA	New 36" diameter S.S.	280	LF	$79	$22,120
TS-AA	TS-A	New 42" diameter S.S.	800	LF	$94	$75,200
TS-A	TS-B	Parallel w/48" diameter S.S.	325	LF	$111	$36,075
TS-B	TS-C	Parallel w/54" diameter S.S.	325	LF	$138	$44,850
TS-C	TS-D	Parallel w/54" diameter S.S.	325	LF	$138	$44,850
TS-EB	TS-EA	New 30" diameter S.S.	400	LF	$58	$23,200
TS-EA	TS-E	New 42" diameter S.S.	800	LF	$94	$75,200
TS-E	TS-D	Parallel w/48" diameter S.S.	621	LF	$111	$68,931
TS-D	TS-F	Parallel w/60" diameter S.S.	600	LF	$163	$97,800
TS-F	TS-G	Parallel w/60" diameter S.S.	465	LF	$163	$75,795
TS-G	TS-H	Parallel w/60" diameter S.S.	180	LF	$163	$29,340
TS-PE	TS-PD	New 36" diameter S.S.	600	LF	$79	$47,400
TS-PD	TS-PB	New 42" diameter S.S.	680	LF	$94	$63,920
TS-PC	TS-PB	New 36" diameter S.S.	1,040	LF	$79	$82,160
TS-PB	TS-PA	New 54" diameter S.S.	760	LF	$138	$104,880
TS-PA	TS-P	New 60" diameter S.S.	650	LF	$163	$105,950
TS-P	TS-O	Parallel w/60" diameter S.S.	160	LF	$163	$26,080
TS-O	TS-N	Parallel w/60" diameter S.S.	145	LF	$163	$23,635
TS-N	TS-M	Parallel w/60" diameter S.S.	365	LF	$163	$59,495
TS-M	TS-H	Parallel w/60" diameter S.S.	325	LF	$163	$52,975
TS-I	TS-J	---	260	LF	$0	$0
TS-J	TS-K	---	213	LF	$0	$0
TS-KA	TS-K	New 30" diameter S.S.	960	LF	$58	$55,680
TS-K	TS-L	---	455	LF	$0	$0
TS-LC	TS-LB	New 24" RCP	350	LF	$31	$10,850
TS-LB	TS-LA	New 24" RCP	350	LF	$31	$10,850
TS-LA	TS-L	New 24" RCP	400	LF	$31	$12,400
TS-L	TS-H	---	868	LF	$0	$0
TS-H	TS-Q	Parallel w/60" diameter S.S.	470	LF	$163	$76,610

TABLE 3.5-3 (CONTINUED)
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
S.W. 26th AVENUE STORM SEWER (TS)

INLET/NODE						
FROM	**TO**	**PROPOSED IMPROVEMENTS**	**NO. UNITS**	**UNIT**	**UNIT COST**	**TOTAL COST**
TS-Q	TS-R	Parallel w/60" diameter S.S.	625	LF	$163	$101,875
TS-S	TS-4	---	1,290	LF	$0	$0
TS-R	TS-T	Parallel w/60" diameter S.S.	400	LF	$163	$65,200
TS-T	TS-U	Parallel w/60" diameter S.S.	220	LF	$163	$35,860
TS-V	TS-U	---	1,300	LF	$0	$0
TS-U	TS-W	Parallel w/60" diameter S.S.	370	LF	$163	$60,310
TS-W	OUT-FALL	Parallel w/60" diameter S.S.	485	LF	$163	$79,055
					SUBTOTAL	$1,735,696
INLET IMPROVEMENTS						
		4' Inlets	12	EA	$1,900	$22,800
		5' Inlets	6	EA	$2,200	$13,200
		10' Inlets	12	EA	$2,700	$32,400
		15' Inlets	5	EA	$3,100	$15,500
		20' Inlets	4	EA	$3,600	$14,400
		25' Inlets	2	EA	$4,100	$8,200
		30' Inlets	2	EA	$4,600	$9,200
					SUBTOTAL	$115,700
MANHOLES						
		Manholes	12	Each	$3,500	$42,000
					SUBTOTAL	$42,000
STREET RECONSTRUCTION						
TS-G	TS-H	Regrading-SW 26th & Ga.		LS	$100,000	$100,000
					SUBTOTAL	$100,000
					TOTAL	$1,993,396

(See Drawing Set Sheet No. P6-3)



STUDY AREA: LAWRENCE LAKE "P6"

PROJECT NAME: S.W. 26TH AVE. "TS"

DESCRIPTION: PARALLEL EXISTING STORM SEWER/EXTEND LATERALS/ ADD NEW INLETS/REPLACE INLETS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

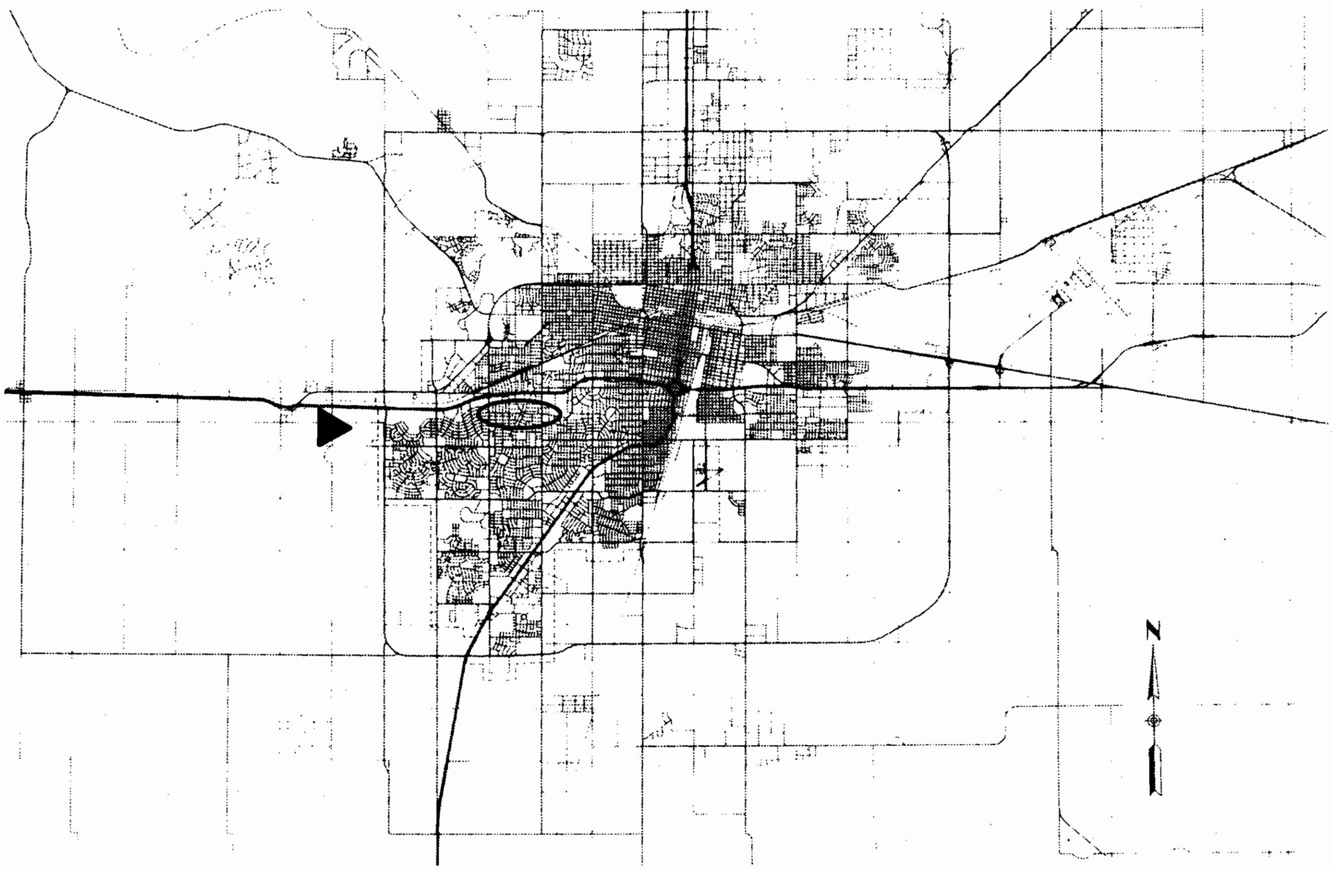
PROJECT IDENTIFICATION MAP (P6-TS)

FIGURE
3.5-3

TABLE 3.5-4
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
OLSEN/EMIL STORM SEWER (OE)

INLET/NODE FROM	INLET/NODE TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
OE-A	OE-B	New 42" RCP	420	LF	$94	$39,480
OE-B	OE-C	New 42" RCP	770	LF	$94	$72,380
OE-C	OE-D	New 42" RCP	300	LF	$94	$28,200
OE-E	OE-D	New 36" RCP	760	LF	$79	$60,040
OE-D	OE-F	New 54" RCP	370	LF	$135	$49,950
OE-F	OE-G	New 54" RCP	900	LF	$135	$121,500
OE-H	OE-G	New 30" RCP	320	LF	$58	$18,560
OE-G	OE-I	New 54" RCP	1000	LF	$135	$135,000
OE-P	OE-O	---	418	LF	$0	$0
OE-O	OE-N	Replace w/30" RCP	259	LF	$73	$18,907
OE-N	OE-M	Replace w/48" RCP	167	LF	$139	$23,213
OE-M	OE-L	Replace w/48" RCP	345	LF	$139	$47,955
OE-L	OE-I	Replace w/48" RCP	655	LF	$139	$91,045
OE-I	OE-J	Replace w/78" RCP	370	LF	$315	$116,550
OE-J	OE-K	Replace w/78" RCP	305	LF	$315	$96,075
OE-K	OUTFALL	Replace w/78" RCP	288	LF	$315	$90,720
				SUBTOTAL		$1,009,575
INLET IMPROVEMENTS						
		4' Inlets	11	EA	$1,900	$20,900
		5' Inlets	6	EA	$2,200	$13,200
		10' Inlets	4	EA	$2,700	$10,800
		15' Inlets	3	EA	$3,100	$9,300
		20' Inlets	3	EA	$3,600	$10,800
		30' Inlets	1	EA	$4,600	$4,600
				SUBTOTAL		$69,600
MANHOLES						
		Manholes	7	EA	$3,500	$24,500
				SUBTOTAL		$24,500
				TOTAL		$1,103,675

(See Drawing Set Sheet No. P6-2)



STUDY AREA: LAWRENCE LAKE "P6"

PROJECT NAME: OLSEN/EMIL "OE"

DESCRIPTION: ADD NEW STORM SEWERS/INLETS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

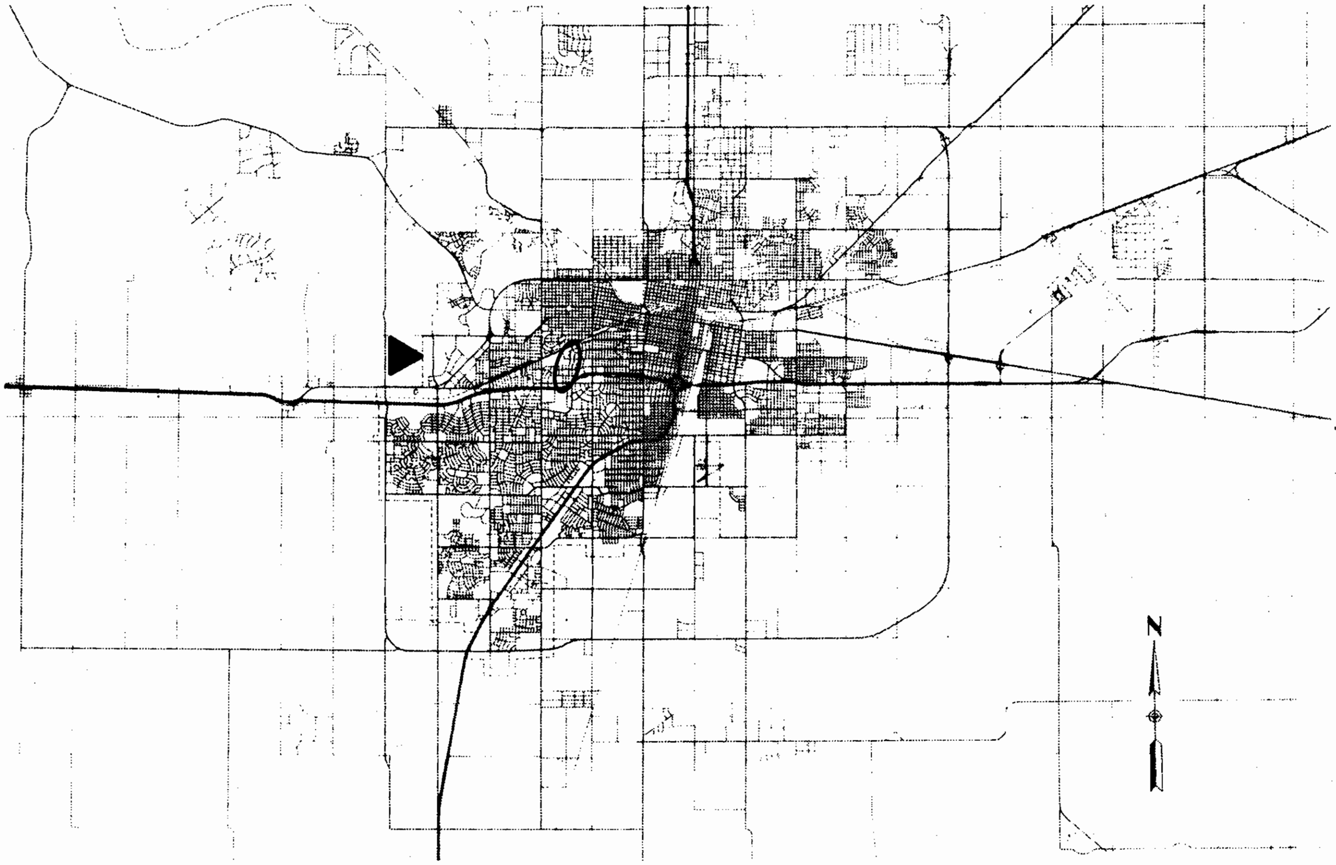
PROJECT IDENTIFICATION MAP (P6-OE)

FIGURE 3.5-4

TABLE 3.5-5
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
JULIAN STORM SEWER (JN)

INLET/NODE FROM	INLET/NODE TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
JN-A	JN-B	New 24" RCP	440	LF	$31	$13,640
JN-C	JN-B	New 24" RCP	400	LF	$31	$12,400
JN-B	JN-D	New 42" RCP	1,20	LF	$94	$105,280
JN-E	JN-D	New 30" RCP	350	LF	$58	$20,300
JN-D	JN-F	New 42" RCP	600	LF	$94	$56,400
JN-F	JN-G	New 48" RCP	670	LF	$111	$74,370
JN-G	JN-H	---	460	LF	$0	$0
JN-H	JN-I	---	635	LF	$0	$0
JN-I	JN-T	---	650	LF	$0	$0
JN-T	OUT-FALL	---	165	LF	$0	$0
					SUBTOTAL	$282,390
INLET IMPROVEMENTS						
		4' Inlets	4	EA	$1,900	$7,600
		5' Inlets	5	EA	$2,200	$11,000
		10' Inlets	1	EA	$2,700	$2,700
		15' Inlets	2	EA	$3,100	$6,200
		20' Inlets	1	EA	$3,600	$3,600
		25' Inlets	1	EA	$4,100	$4,100
					SUBTOTAL	$35,200
MANHOLES						
		Manholes	3	EA	$10,500	$10,500
					SUBTOTAL	$10,500
OTHER INCIDENTAL CONSTRUCTION						
		Connect to Junction Box at I40	1	LS	$10,000	$10,000
					SUBTOTAL	$10,000
					TOTAL	$338,090

(See Drawing Set Sheet No. P6-3)



STUDY AREA: LAWRENCE LAKE "P6"

PROJECT NAME: JULIAN BLVD "JN"

DESCRIPTION: ADD NEW STORM SEWER SYSTEM

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P6-JN)

FIGURE
3.5-5

Dilday Draw Problem Area (DD)

Proposed improvements for the Dilday Draw storm sewer system consist of replacing the lower segments of the major trunkline on I40, replacing some of the inlets, and making primarily cosmetic channel improvements from La Salle Street to Plains Boulevard. Proposed improvements are summarized in Table 3.5-6 and their location shown in Figure 3.5-6.

3.6 Playa No. 7 Study Area (P7)

3.6.1 Existing Problems

Playa No. 7

Playa No. 7 is located on Arden Road just east of Soncy Road. The playa drainage basin, which totals 1,631 acres, is essentially undeveloped. A survey of the playa perimeter revealed a primary damage elevation of 3,672.7 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,673.6 feet. Results of the ASAPP Model indicate that the primary damage elevation would be reached under the 25-year event.

A summary of existing levels for Playa No. 7 is found in Table A-1 in Appendix A.

3.6.2 Proposed Improvements

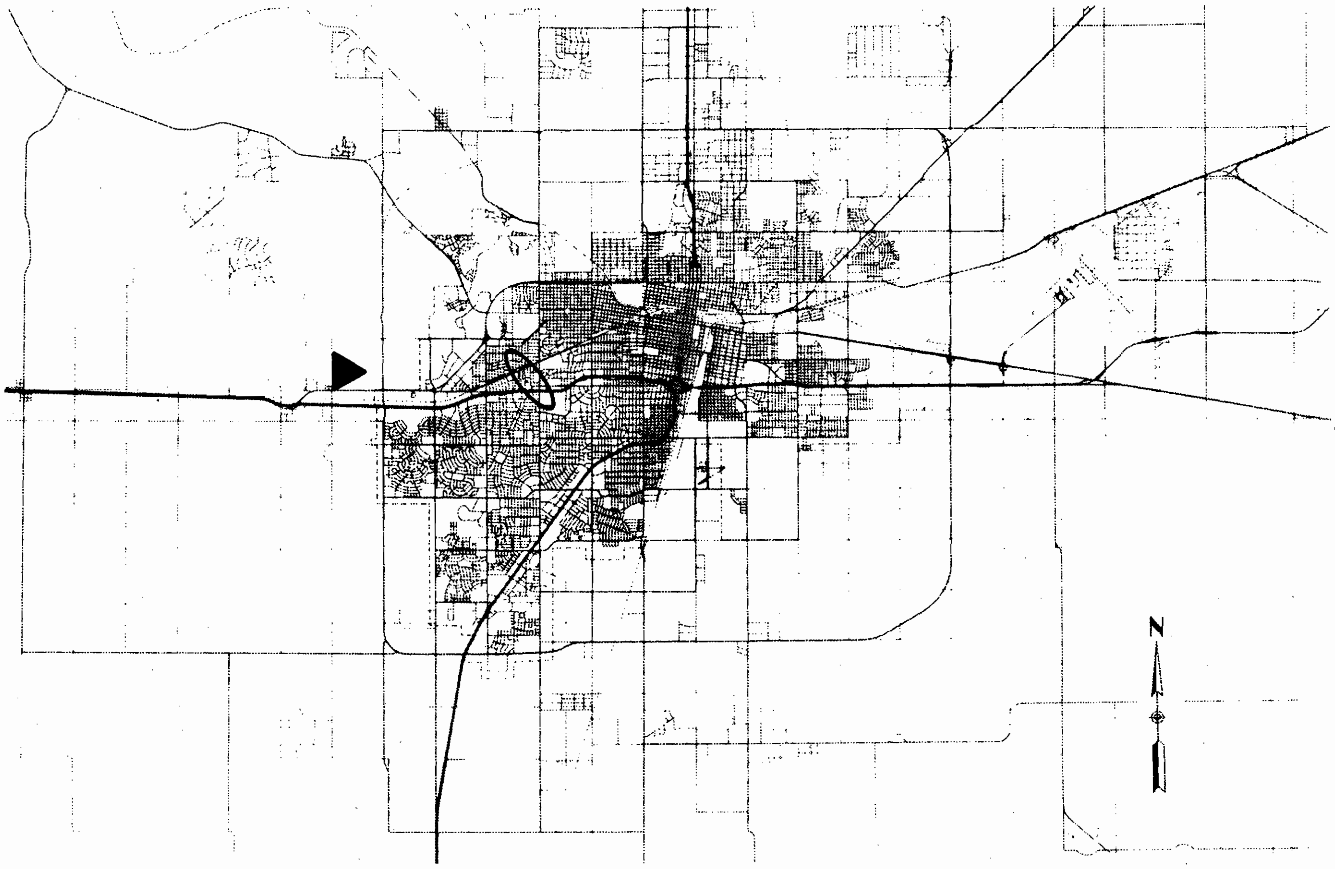
Playa No. 7

Under ultimate basin development conditions, the 100-year level for Playa No. 7 is 3,674 feet. This is 1.3 feet above the primary damage elevation, which would be protected against the 16-year storm event. A summary of ultimate levels for Playa No. 7 is found in Table A-2 in Appendix A.

TABLE 3.5-6
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
LAWRENCE LAKE STUDY AREA (P6)
DILDAY DRAW STORM SEWER (DD)

INLET/NODE FROM	INLET/NODE TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER/CHANNEL IMPROVEMENTS						
DD-A	DD-B	----	714	LF	$0	$0
DD-B	DD-C	----	701	LF	$0	$0
DD-C	DD-D	----	567	LF	$0	$0
DD-D	DD-E	----	150	LF	$0	$0
DD-E	DD-F	----	243	LF	$0	$0
DD-F	DD-G	----	320	LF	$0	$0
DD-G	DD-H	----	330	LF	$0	$0
DD-H	DD-I	----	350	LF	$0	$0
DD-I	DD-J	----	315	LF	$0	$0
DD-J	DD-K	----	447	LF	$0	$0
DD-K	DD-L	Channel Improvements	510	LF	$250	$127,500
DD-L	DD-M	Channel Improvements	420	LF	$255	$107,000
DD-M	DD-N	Channel Improvements	445	LF	$260	$115,700
DD-N	DD-V	----	409	LF	$0	$0
DD-V	DD-O	----	293	LF	$0	$0
DD-O	DD-P	----	507	LF	$0	$0
DD-P	DD-U	Replace w/60" RCP	374	LF	$204	$76,296
DD-U	DD-Q	Replace w/60" RCP	240	LF	$204	$48,960
DD-Q	DD-R	Parallel w/30" RCP	780	LF	$58	$45,240
DD-R	DD-S	Parallel w/30" RCP	350	LF	$58	$20,300
DD-S	DD-T	Parallel w/30" RCP	560	LF	$58	$32,480
DD-T	OUT-FALL	----	165	LF	$0	$0
					SUBTOTAL	$573,576
INLET IMPROVEMENTS						
		10' Inlets	4	EA	$2,700	$10,800
		15' Inlets	1	EA	$3,100	$3,100
		20' Inlets	1	EA	$3,600	$3,600
		25' Inlets	1	EA	$4,100	$4,100
					SUBTOTAL	$21,600
MANHOLES/JUNCTION BOXES						
		Manholes	6	EA	$4,500	$27,000
		Junction Boxes	4	EA	$10,000	$40,000
					SUBTOTAL	$67,000
					TOTAL	$662,176

(See Drawing Set Sheet No. P6-2)



STUDY AREA: LAWRENCE LAKE "P6"

PROJECT NAME: DILDAY DRAW "DD"

DESCRIPTION: REPLACE STORM SEWERS/ADD AND REPLACE INLETS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P6-DD)

FIGURE 3.5-6

In order to provide protection against the 100-year event under ultimate basin development conditions, it is proposed to excavate 100,000 cubic yards of earth from Playa No. 7 and install a new 3,000 gpm pump station. Pumped discharges from Playa No. 7 would be piped to the Spring Draw outfall southwest of Amarillo. Proposed improvements for Playa No. 7 are summarized in Table 3.6-1 and their location shown in Figure 3.6-1.

TABLE 3.6-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 7 STUDY AREA (P7)
PLAYA NO. 7

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P7-A		Excavate playa	100,000	CY	$4.50	$450,000
P7-B	P7-C	New 3,000 gpm pump station with 18" suction line		LS		$180,000
P7-C	P17-G	New 16" force main	400	LF	$50.00	$20,000
					TOTAL	$650,000

(See Drawing Set Sheet No. P7-2)

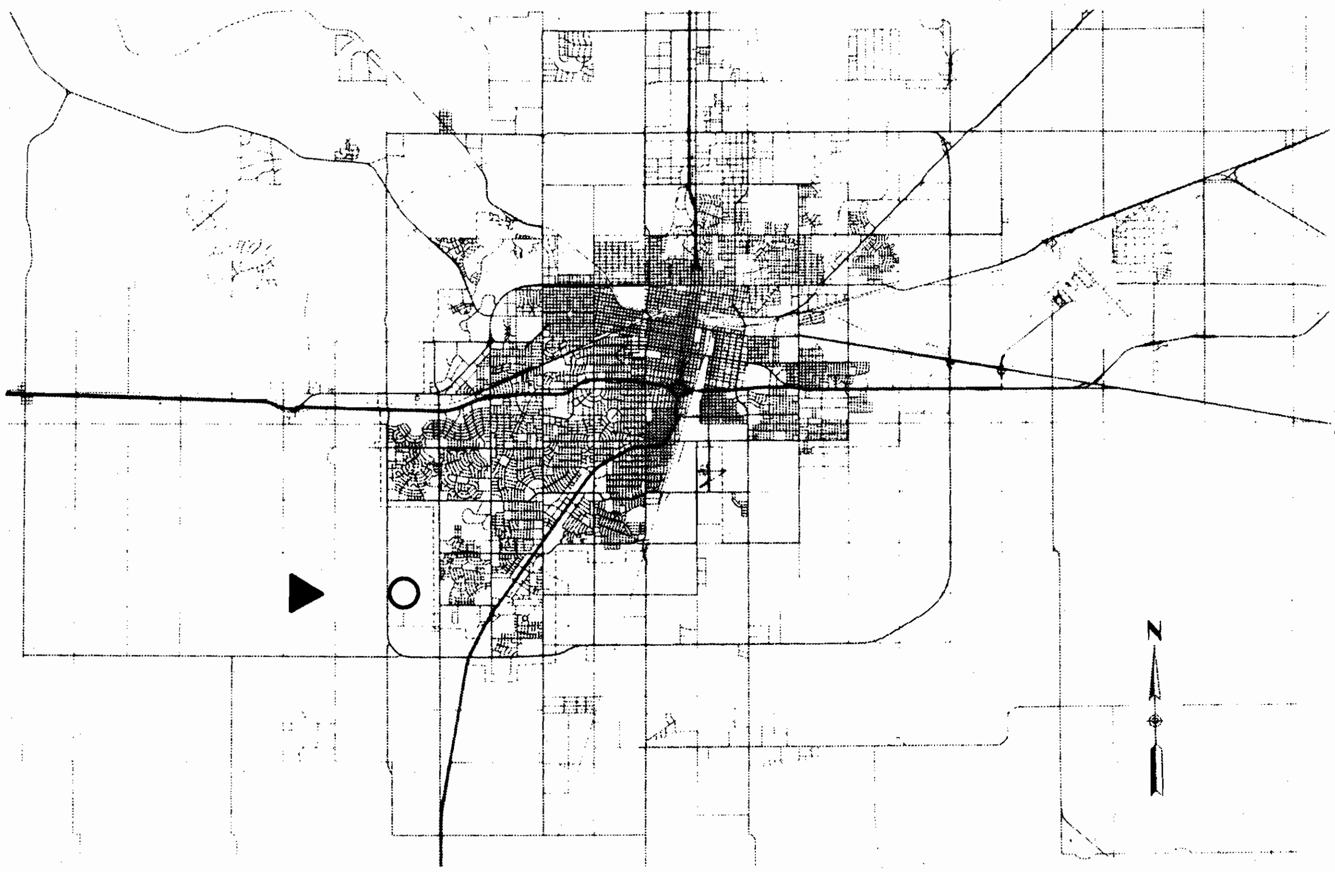
A summary of ultimate levels under the proposed playa improvement scenario is found in Table A-3 in Appendix A.

3.7 Playa No. 8 Study Area (P8)

3.7.1 Existing Problems

Playa No. 8

Playa No. 8 is located just south of the intersection of Hollywood Road and Soncy Road. The playa drainage basin, which totals 1,224 acres, is essentially undeveloped. A survey of the Playa No. 8 perimeter revealed a primary damage elevation of 3,681.4 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,678.7 feet. Therefore, due to the lack of development in the area, no significant damages



STUDY AREA: PLAYA NO. 7 "P7"

PROJECT NAME: PLAYA NO. 7

DESCRIPTION: NEW PUMP STATION/EXCAVATE PLAYA

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P7)

FIGURE
3.6-1

result for any events up through the 100-year flood event.

Playa levels for existing conditions are found in Table A-1 in Appendix A.

3.7.2 Proposed Improvements

Playa No. 8

Under ultimate basin development conditions, the 100-year flood level for Playa No. 8 is 3,679.1 feet, well below the existing primary damage elevation of 3,681.4 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 8 area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 8 is found in Table A-2 in Appendix A.

3.8 Playa No. 10 Study Area (P10)

3.8.1 Existing Problems

Playa No. 10

Playa No. 10 is located west of Coulter Street at Sundown Lane. The playa drainage basin, which totals 1,567 acres, is undeveloped. A survey of the Playa No. 10 perimeter revealed a primary damage elevation of 3,667.7 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,664.7 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

Playa levels for existing conditions are found in Table A-1 in Appendix A.

3.8.2 Proposed Improvements

Playa No. 10

Under ultimate basin development conditions, the 100-year level for Playa No. 10 is 3,665.2 feet, well below the existing primary damage elevation of 3,667.7 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 10 area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 10 is found in Table A-2 in Appendix A.

3.9 Playa No. 11 Study Area (P11)

3.9.1 Existing Problems

Playa No. 11

Playa No. 11 is located north of Sundown Lane and east of Interstate 27. The playa drainage basin, which totals 1,875 acres, is essentially undeveloped. A survey of the Playa No. 11 perimeter revealed a primary damage elevation of 3,661.5 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,644.1 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

A summary of levels for Playa No. 11 is found in Table A-1 in Appendix A.

3.9.2 Proposed Improvements

Playa No. 11

Under ultimate basin development conditions, the 100-year level for Playa No. 11 is 3,644.5 feet, well below the existing primary damage elevation of 3,661.5 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 11 area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 11 is found in Table A-2 in Appendix A.

3.10 Playa No. 12 Study Area (P12)

3.10.1 Existing Problems

Playa No. 12

Playa No. 12 is located west of Western Street between Sundown Lane and McCormick Road. The playa drainage basin, which totals 1,156 acres, is undeveloped. Playa No. 12 occasionally collects overflows from Playa No. 30, located approximately one mile northeast on the opposite side of Western Street. The Playa No. 30 basin area of 310 acres is not part of the Playa No. 12 area of 1,156 acres. A survey of the Playa No. 12 perimeter revealed a primary damage elevation of 3,629 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,624.5 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year flood event, including events in excess of the 50-year event which result in overflows from Playa No. 30.

A summary of existing levels for Playa No. 12 is found in Table A-1 in Appendix A.

3.10.2 Proposed Improvements

Playa No. 12

Under ultimate basin development conditions, the 100-year level for Playa No. 12 is 3,625.3 feet, well below the existing primary damage elevation of 3,629 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 12 area, no improvements for the basin are recommended in this Master Plan. Should development pressure occur, a hydrologic model of the Playa No. 30/Playa No. 12 system should be prepared to account for the hydraulic behavior of the Playa No. 30 overflow. A backwater profile model should then be prepared to estimate the water surface elevations of the overflows from Playa No. 30 to Playa No. 12 for a full range of flood events.

A summary of ultimate levels for Playa No. 12 is found in Table A-2 in Appendix A.

3.11 Playa No. 13 Study Area (P13)

3.11.1 Existing Problems

Playa No. 13

Playa No. 13 is located on Western Street just south of Hollywood Road. The playa drainage basin, which totals 1,833 acres, is essentially undeveloped. A survey of the Playa No. 13 perimeter revealed a primary damage elevation of 3,624.2 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,624.1 feet. Therefore, under existing conditions, no significant damages result for any events up through the 100-year flood event.

A summary of existing playa levels for Playa No. 13 is found in Table A-1 in Appendix A.

3.11.2 Proposed Improvements

Playa No. 13

Under ultimate basin development conditions, the 100-year level for Playa No. 13 is 3,624.3 feet, just over the existing primary damage elevation. However, due to the lack of existing or planned development in the area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 13 is found in Table A-2 in Appendix A.

3.12 Playa No. 14 Study Area (P14)

3.12.1 Existing Problems

Playa No. 14 (Diamond Horseshoe Lake)

Playa No. 14 is located east of the intersection of Interstate 27 and Bell Street. The playa drainage basin, which totals 1,102 acres, is primarily low-density residential and commercial development with about one-fourth of the basin undeveloped. A survey of the playa perimeter revealed a primary damage elevation of 3,658.1 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will be reached by the 47-year flood event.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

3.12.2 Proposed Improvements

Playa No. 14 (Diamond Horseshoe Lake)

Under ultimate basin development conditions, the primary damage elevation will be reached at the 39-year flood event. Under the 100-year event, the flood level in Playa No. 14 would reach 3,659.4 feet, 1.3 feet above the primary damage elevation. A summary of ultimate levels for Playa No. 14 is found in Table A-2 in Appendix A.

In order to provide 100-year flood event protection at Diamond Horseshoe Lake, a new 750 gpm playa pump station is proposed. Pumped discharges from Playa No. 14 would be directed to the Spring Draw Outfall southwest of Amarillo. Proposed improvements for Playa No. 14 are summarized in Table 3.12-1 and their location shown in Figure 3.12-1.

TABLE 3.12-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 14 STUDY AREA (P14)
DIAMOND HORSESHOE LAKE (PLAYA NO. 14)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P14-A	P14-B	New 750 gpm pump station with 8" suction line		LS		$120,000
P14-B	P17-F	New 8" force main	1,200	LF	$35.00	$42,000
					TOTAL	$162,000

(See Drawing Set Sheet No. P14-1)

A summary of ultimate levels for Playa No. 14 under the proposed improvements scenario is found in Table A-3 in Appendix A.

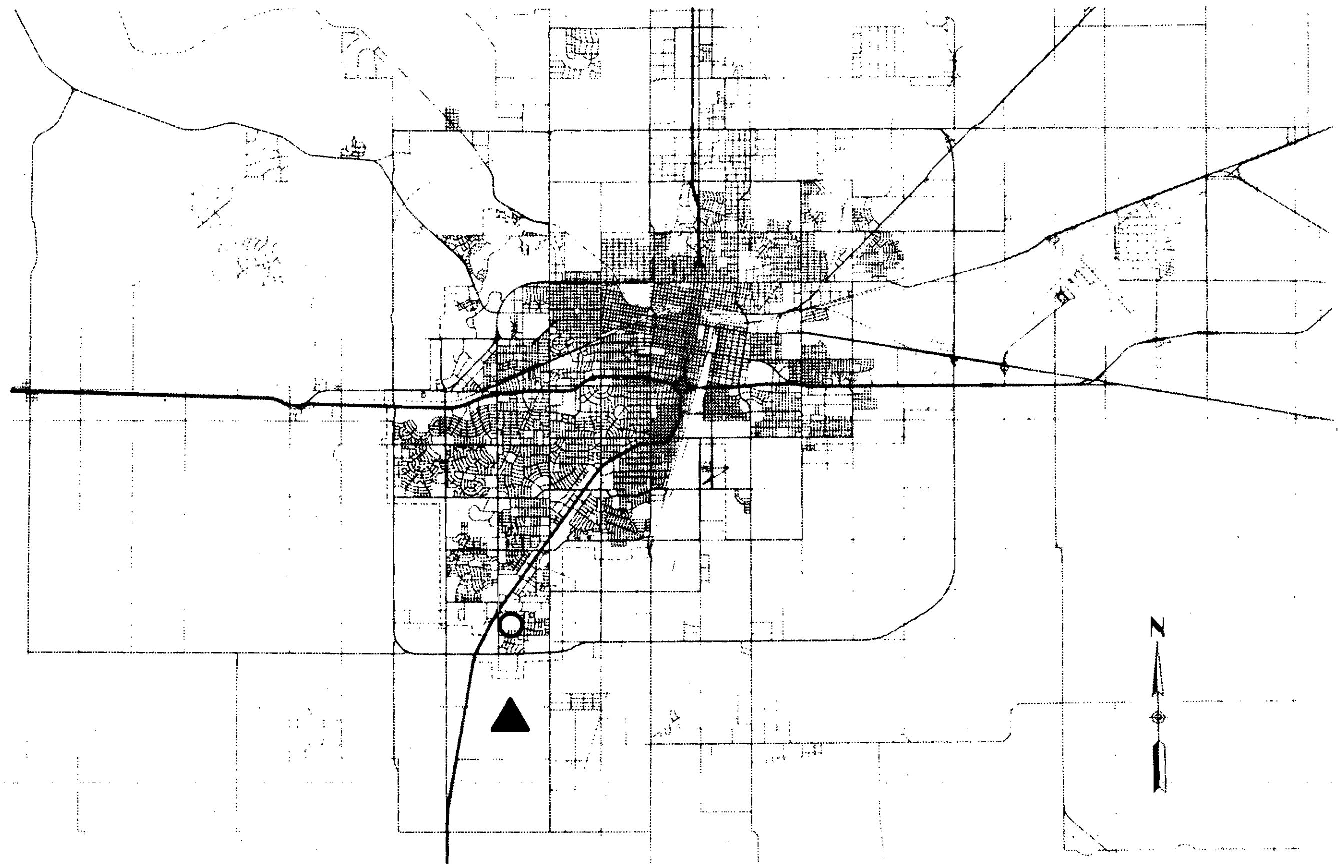
3.13 Playa No. 15 Study Area (P15)

3.13.1 Existing Problems

Playa No. 15 (McCarty Lake)

Playa No. 15 is located southeast of the intersection of Western Street and S.W. 58th Avenue. The playa drainage basin, which totals 3,651 acres, is primarily low-density residential (LDR) with a significant portion undeveloped. Flooding problems have been reported at McCarty Lake. A survey of the playa perimeter revealed a primary damage elevation of 3,623.1 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will be reached by the 17-year flood event.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.



STUDY AREA: PLAYA NO. 14 "P14"

PROJECT NAME: DIAMOND HORSESHOE LAKE

DESCRIPTION: NEW PUMP STATION AND FORCE MAIN

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P14)

FIGURE
3.12-1

Hillside/Hampton Storm Sewer Problem Area (HH)

The existing Hillside and Hampton storm sewer system starts near the intersection of Hillside Avenue and Hampton Drive in southwest Amarillo. This lateral extends along Hampton to Wentworth Drive and connects with another lateral coming from Hurst Road down Wentworth Drive. At Hampton and Wentworth, the storm sewer continues along Hampton to Norwich, then east to Bell Street. At Bell Street, storm sewer joins another lateral from the south along Bell Street. The Bell Street south lateral collects inlets along Bell Street. After joining the Bell Street south lateral, the storm sewer continues north along Bell Street to Hillside. At Hillside, another lateral coming from the north down Bell Street connects to the storm sewer. The Bell Street, north lateral starts at Sandie Drive near Amarillo High School north of the Bell Street and S.W. 45th Avenue intersection, and collects inlets from the north along Bell Street.

At Bell Street and Hillside, the storm sewer turns into a 72-inch RCP and runs east along Hillside Avenue to Star Lane. At Star Lane, a 54-inch lateral connects to the storm sewer. The Star Lane lateral picks up inlets along Star Lane. The storm sewer continues along S.W. 58th Avenue under Interstate 27 to Western Street. At Western Street, another lateral draining areas north of Hillside Avenue connects to the storm sewer. The Western Street north lateral starts near 47th Avenue and Western, continuing south along Western. At West 51st and 53rd Avenues, major laterals join the storm sewer as it runs along Western Street. The Western Street north lateral finally connects at Hillside Avenue before the outfall at a ditch just east of Western Street.

The Hillside/Hampton storm sewer system intercepts about 1,635 acres before the outfall to McCarty Lake (Playa No. 15). Development in the watershed is residential for the most part. Some commercial and light industrial areas exist along Interstate 27. One characteristic of the drainage area intercepted by this system is that for larger storms, some of the storm water normally intercepted by the Hillside and Hampton system runs off to the Lawrence Lake playa drainage area. This "conditional" drainage area is approximately 197 acres. Watershed slopes are relatively flat at less than 0.5 percent slope.

Assumptions used in the analysis of the Hillside/Hampton system included use of flow curves developed for the Storm Water Criteria manual for the two-year storm event. Curve numbers varied according to land use. Street capacities were computed for 37-foot wide residential streets, 45-foot collector streets, and 65-foot wide arterial street at slopes of 0.0030 ft/ft. The topographic maps for this area were compiled in 1978, and some additional streets have been completed since that time. Parameters used for analyzing the storm sewers included use of a minimum storm sewer slope of 0.001, minimum diameter of 18-inch, minimum allowable velocity of two feet per second, and minimum cover of one foot. It was assumed that what the street does not carry, the storm sewer will.

Reported flooding problems exist at the outfall to McCarty Lake for the larger storms. Surcharging of the storm sewer system occurs at several points as storm water is intercepted by inlets and major laterals connected to the system. As the storm sewer system capacity is exceeded, shallow street flooding results at some locations in the system.

S.W. 58th Avenue Storm Sewer Problem Area (FE)

The S.W. 58th Avenue study area is located in southwest Amarillo, north of McCarty Lake. Presently, the existing storm sewer starts at Georgia Street and S.W. 58th Avenue and runs along S.W. 58th Avenue, picking up nine inlets before the outfall to McCarty Lake. These inlets collect runoff from residential areas north of McCarty Lake and along S.W. 58th Avenue.

The contributing area to the outfall at McCarty Lake is about 293 acres. Development in the watershed is primarily residential with some undeveloped areas north of S.W. 58th. Watershed slopes vary from two to three percent.

The S.W. 58th Avenue Storm Sewer System was analyzed using a two-year/top-of-curb standard. Flow curves developed for Amarillo were used to compute the two-year flows. Curve numbers varied according to land use. S.W. 58th Avenue was assumed to be 65-feet wide and to slope at 0.0030 ft/ft to the inlets. The topographic maps for this area were compiled in 1978. S.W. 58th Avenue was completed in about 1980 and the connector was completed about 1985, linking S.W. 58th Avenue to Hillside Avenue. Parameters used for analyzing the storm sewers included minimum storm sewer slope of 0.001, minimum diameter of 18-inch, minimum allowable velocity of two feet per second, and minimum cover of one foot. Some surcharging of inlets was considered in the analysis. This surcharging could occur during the two-year and larger storm events, particularly in the lower reaches of the storm sewer near inlets FE-7, 8 and 9 as McCarty Lake backs up.

Shallow street flooding along S.W. 58th Avenue occurs as levels in McCarty Lake rise, causing the storm sewer to back up. Also, upstream runoff to the uppermost inlet at Georgia and S.W. 58th Avenue overloads the storm sewer system.

Catalpa Storm Sewer Problem Area (CA)

The Catalpa storm sewer system starts at two area inlets along Interstate 27 (I27) at Farmers Avenue and Catalpa Lane. One lateral starts at Farmers and I27, extending southeast along Farmers to Western Street. This lateral picks up one other lateral at Greenhaven Road. At Western, the Farmers lateral joins with another lateral draining to the north along Western Street. The storm sewer then continues along Western Street north to Catalpa Lane, where it joins the Catalpa Lane lateral coming from I27. The storm sewers then discharge to a drainage ditch paralleling Catalpa Lane which eventually discharges to McCarty Lake (Playa No. 15).

The contributing drainage area of the storm sewer system at its discharge point into the drainage ditch east of Catalpa and Western is about 535 acres. Development in the watershed varies from commercial and multi-family along I27 to residential between I27 and Western Street. Watershed slopes vary from less than 0.5 percent in the upper watershed, to 0.5 to 1.0 percent in the middle portion, to three to four percent in the lower portion west of Western Street.

Flooding problems have been experienced at the intersection of Catalpa and Western and the upper watershed along the I27 frontage roads. As water levels rise in McCarty Lake, a tailwater condition is created on the Catalpa storm sewer outfall.

3.13.2 Proposed Improvements

Playa No. 15 (McCarty Lake)

Under ultimate basin development conditions, the primary damage elevation will be reached at the 13-year flood event. Under the 100-year event, the flood level in McCarty Lake would reach 3,625 feet, 1.9 feet above the primary damage elevation. A summary of ultimate levels for Playa No. 15 is found in Table A-2 in Appendix A.

In order to provide 100-year protection at McCarty Lake, it is proposed to excavate 900,000 cubic yards of material from the playa and install a new 4,000 gpm playa pump station. In addition, Georgia Street, which bisects the playa north-south, should be raised to elevation 3,622 feet and culverts installed to provide protection from the 25-year event. Pumped discharges from Playa No. 15 would be directed to the proposed Spring Draw

Outfall in southwest Amarillo. Proposed improvements for Playa No. 15 are summarized in Table 3.13-1 and their location shown in Figure 3.13-1.

TABLE 3.13-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McCARTY LAKE STUDY AREA (P15)
McCARTY LAKE (PLAYA NO. 15)

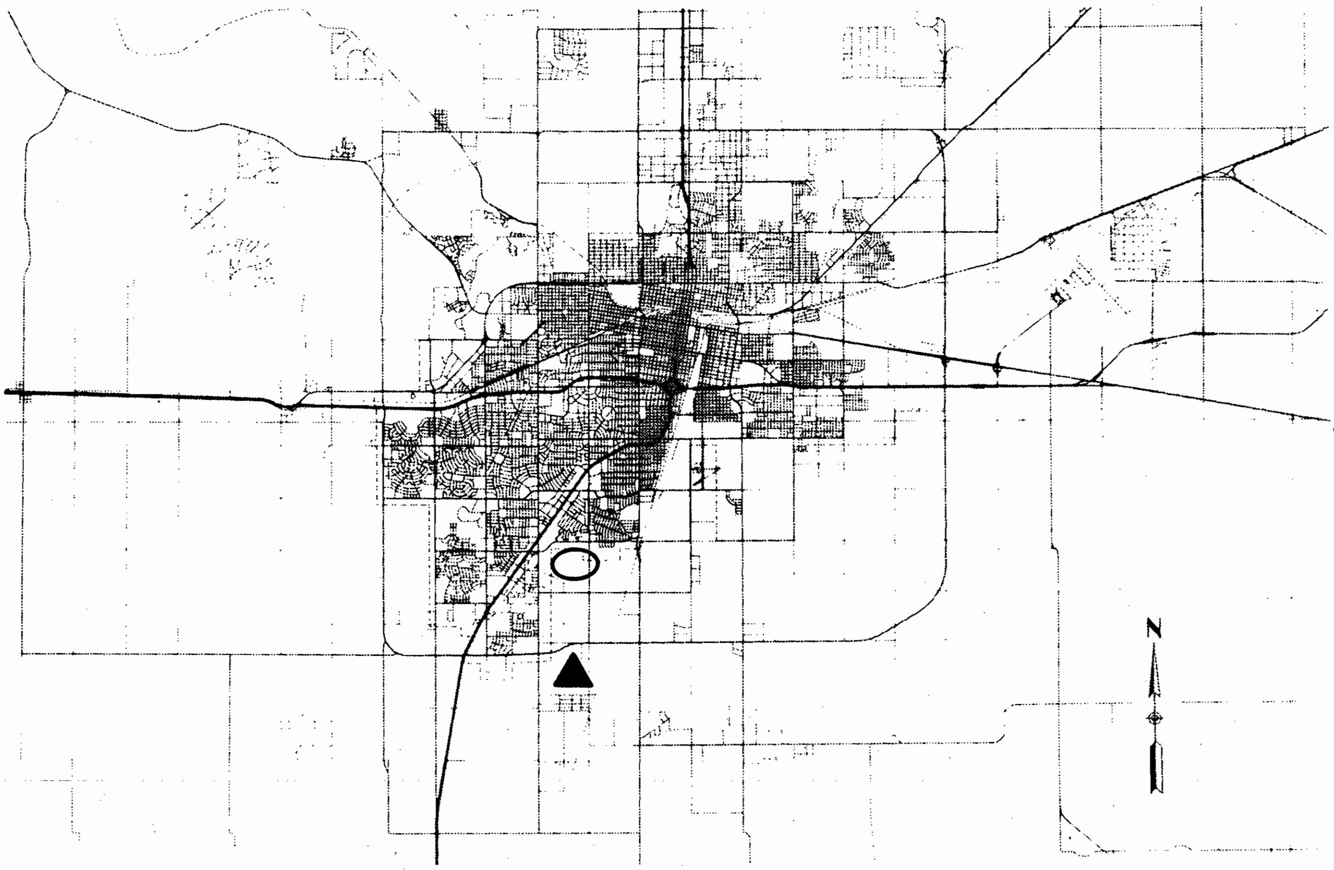
INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P15-A		Excavate playa Raise Georgia Street	900,000 3,200	CY LF	$4.50 $60.00	$4,050,000 $192,000
P15-B	P15-C	New 4,000 gpm pumping station with 20" suction line		LS		$200,000
P15-C	P17-E	New 18" force main	2,900	LF	$55.00	$159,500
					TOTAL	$4,601,500

(See Drawing Set Sheet No. P15-2)

A summary of ultimate levels under the proposed improvement scenario is found in Table A-3 in Appendix A.

Hillside/Hampton Storm Sewer Problem Area (HH)

Proposed improvements to the Hillside/Hampton system include connecting part of the Fleetwood system, Fulton Lateral, to Hillside/Hampton system and, diverting part of the system with a relief interceptor described in the Catalpa storm sewer improvements. This relief interceptor would take the existing Hampton/Norwich lateral, Bell Street south lateral, and Star Lane laterals and combine them with the proposed upgrade to the Catalpa Lane storm sewer. This interception would relieve the present overloading of the Hillside/Hampton storm sewer system, requiring construction of fewer parallel or replacement lines.



STUDY AREA: PLAYA NO. 15 "P15"

PROJECT NAME: McCARTY LAKE

DESCRIPTION: NEW PUMP STATION AND FORCE MAIN/EXCAVATE PLAYA/ RAISE ROAD

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P15)

FIGURE 3.13-1

Other improvements consist of storm sewer replacement in 51st Avenue, Western Street and Bell Street, as well as, inlet additions/replacements at various locations in the system.

Costs for new or replacement inlets, and parallel or replacement lines, are shown in Table 3.13-2. These costs do not reflect improvements for the Catalpa relief interceptor described in the Catalpa problem area discussion (Table 3.13-4). Boring and jacking under I-27 is anticipated for any future storm sewer work. Proposed improvements for the Hillside/Hampton Storm Sewer System are summarized in Table 3.13-2 and their location shown in Figure 3.13-2.

S.W. 58th Avenue Storm Sewer Problem Area (FE)

Proposed improvements for the S.W. 58th Avenue system consist of diverting runoff from Drainage Area FE-9 (see Sheet P15-2) into a new 25-foot inlet at the northeast corner of S.W. 58th Avenue and Georgia Street. Flows from the new inlet would be carried by a new 36-inch storm sewer to a new outfall channel beginning at the southwest corner of S.W. 58th Avenue and Georgia Street and running south along Georgia Street for approximately 500 feet before discharging into Playa No. 15. Another new inlet would be constructed on the north curb of S.W. 58th Avenue approximately 400 feet west of the intersection of Georgia Street and connected to the existing 24-inch diameter storm sewer. The existing 24-inch sewer would be plugged east of this new inlet and flows would be discharged into a new outfall channel located approximately 1,800 feet west of Georgia Street. Finally, the existing 54-inch sewer would be plugged to isolate flows which would be discharged into the existing outfall located approximately 2,700 feet west of Georgia Street. Proposed improvements for the S.W. 58th Avenue Storm Sewer System are summarized in Table 3.13-3 and their location shown in Figure 3.13-3.

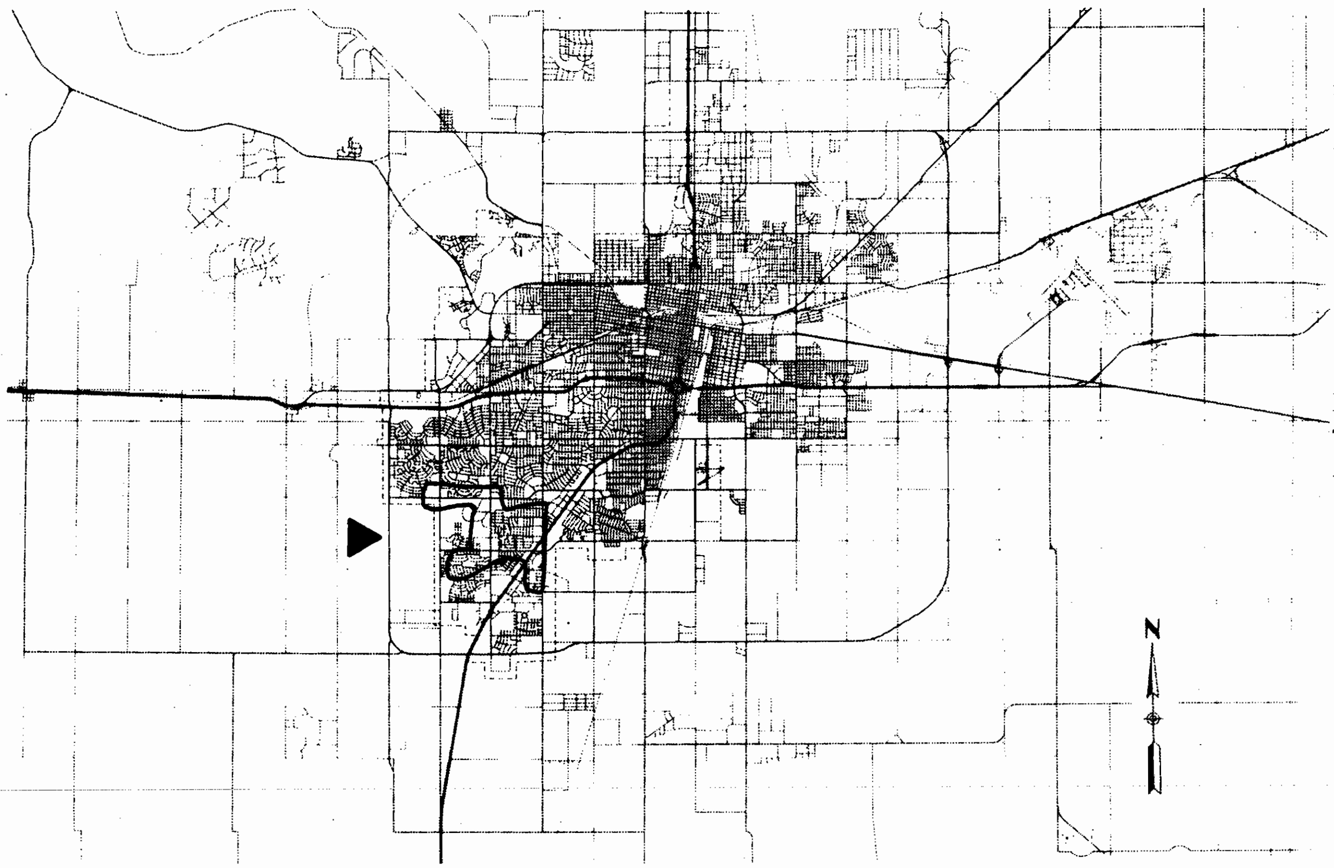
TABLE 3.13-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McCARTY LAKE STUDY AREA (P15)
HILLSIDE/HAMPTON STORM SEWER (HH)

INLET/NODE FROM	INLET/NODE TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
FW-A	FW-B	Parallel w/30" RCP	1,672	LF	$58	$96,976
FW-B	FW-C	Parallel w/30" RCP	1,058	LF	$58	$61,364
FW-C	FW-D	Parallel w/30" RCP	2,137	LF	$58	$123,946
FW-D	FW-E	Parallel w/30" RCP	1,283	LF	$58	$74,414
FW-E	FW-F	Parallel w/30" RCP	374	LF	$58	$21,692
FW-F	FW-G	Parallel w/30" RCP	368	LF	$58	$21,344
FW-G	FW-H	Parallel w/36" RCP	284	LF	$79	$22,436
FW-H	FW-I	Parallel w/36" RCP	331	LF	$79	$26,149
FW-I	FW-J	Plug Existing 30" RCP		LS		$ 2,000
FW-I	HH-O	New 48" RCP	950	LF	$111	$105,450
HH-O	HH-P1	Parallel w/36" RCP	1,700	LF	$79	$134,300
HH-P1	HH-P2	Parallel w/48" RCP	245	LF	$111	$27,195
HH-P2	HH-Q1	Parallel w/48" RCP	710	LF	$111	$78,810
HH-Q1	HH-Q2	Parallel w/48" RCP	510	LF	$111	$56,610
HH-Q2	HH-R	Parallel w/48" RCP	797	LF	$111	$88,467
HH-R	HH-S	----	1,230	LF	$0	$0
HH-S	HH-T1	----	615	LF	$0	$0
HH-T1	HH-T2	----	1,046	LF	$0	$0
HH-T2	HH-N	----	391	LF	$0	$0
HH-N	HH-U1	----	800	LF	$0	$0
HH-U1	HH-U2	----	800	LF	$0	$0
HH-U2	HH-V1	----	800	LF	$0	$0
HH-V1	HH-V2	----	800	LF	$0	$0
HH-V2	HH-V3	----	482	LF	$0	$0
HH-Y1	HH-Y2	----	753	LF	$0	$0
HH-Y2	HH-V3	----	457	LF	$0	$0
HH-V3	HH-Z	----	766	LF	$0	$0
HH-Z	HH-AA	----	1,033	LF	$0	$0
HH-FF	HH-EE	----	935	LF	$0	$0
HH-EE	HH-DD1	----	765	LF	$0	$0

TABLE 3.13-2 (CONTINUED)
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McCARTY LAKE STUDY AREA (P15)
HILLSIDE/HAMPTON STORM SEWER (HH)

INLET/NODE FROM	INLET/NODE TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
HH-DD2	HH-DD3	----	373	LF	$0	$0
HH-DD3	HH-DD4	Replace w/48" RCP	311	LF	$139	$43,229
HH-DD4	HH-DD5	Replace w/48" RCP	317	LF	$139	$44,063
HH-DD5	HH-DD6	Replace w/48" RCP	324	LF	$139	$45,036
HH-DD6	HH-DD7	----	329	LF	$0	$0
HH-DD7	HH-DD8	----	317	LF	$0	$0
HH-DD8	HH-DD1	----	996	LF	$0	$0
HH-DD1	HH-CC	Replace w/60" RCP	1,120	LF	$204	$228,480
HH-CC	HH-BB	----	931	LF	$0	$0
HH-BB	HH-AA	Replace w/72" RCP	529	LF	$266	$140,714
HH-AA	OUT-FALL	----	396	LF	$0	$0
					SUBTOTAL	$1,442,675
INLET IMPROVEMENTS						
		10' Inlets	2	EA	$2,300	$4,600
		15' Inlets	3	EA	$2,700	$8,100
		20' Inlets	1	EA	$3,000	$3,000
		30' Inlets	3	EA	$4,000	$12,000
					SUBTOTAL	$27,700
MANHOLES						
		Manholes	2	EA	$5,000	$10,000
					SUBTOTAL	$10,000
					TOTAL	$1,480,375

(See Drawing Set Sheet No. P15-1)



STUDY AREA: McCARTY LAKE "P15"

PROJECT NAME: HILLSIDE/HAMPTON "HH"

DESCRIPTION: INTERCEPT FULTON LATERAL
FROM FLEETWOOD SYSTEM
DIVERT PORTION TO CATALPA/
PARALLEL EXISTING HILLSIDE TRUNK

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P15-HH)

FIGURE 3.13-2

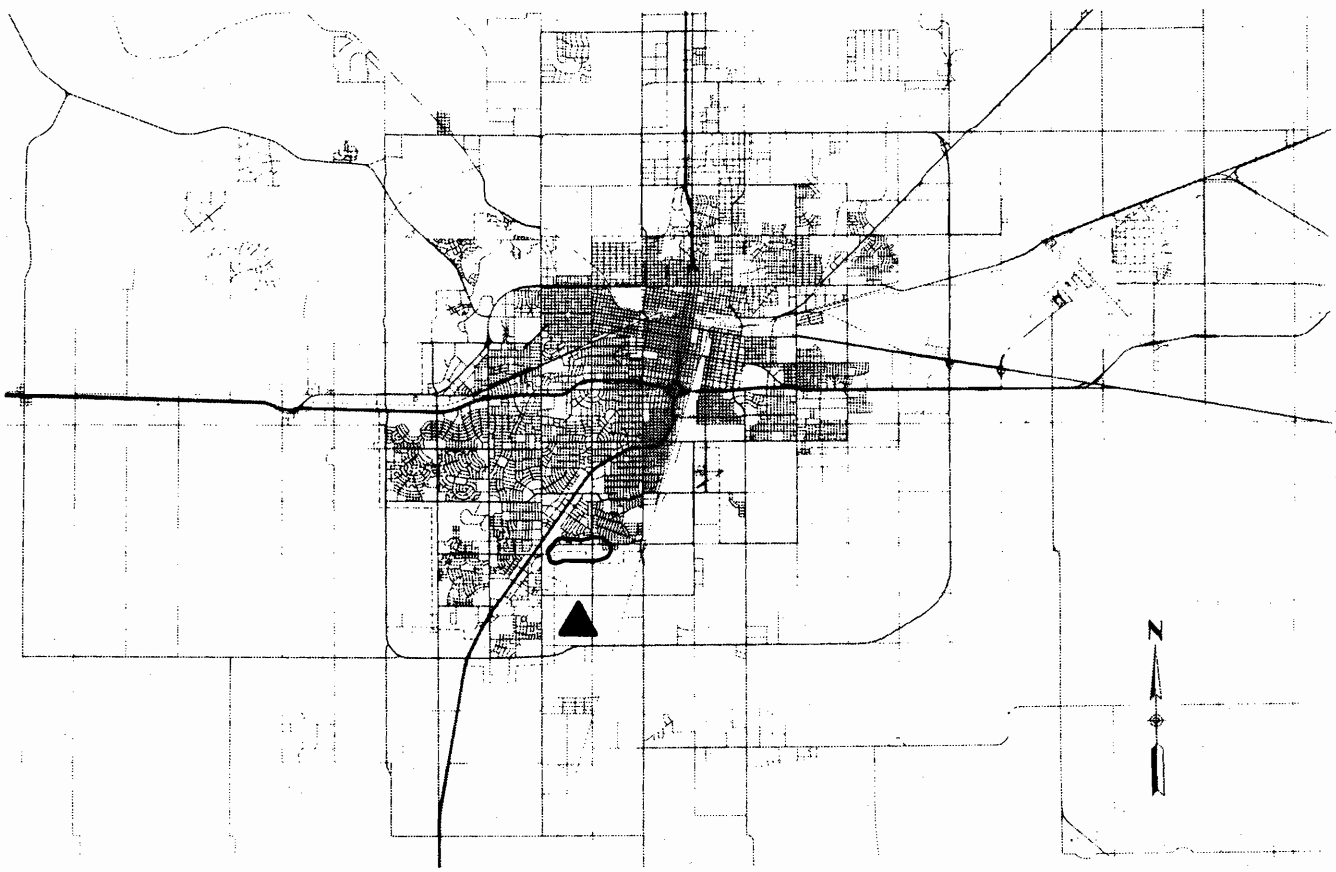
TABLE 3.13-3
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McCARTY LAKE STUDY AREA (P15)
S.W. 58TH AVENUE STORM SEWER (FE)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
FE-A	OUT-FALL	New 36" RCP Headwall & Channel	120	LF LS	$79	$9,480 $50,000
FE-A	FE-B1	Plug Existing RCP		LS		$5,000
FE-B1	FE-B	---	650	LF	$0	$0
FE-B	FE-C	---	550	LF	$0	$0
FE-C	OUT-FALL	New 42" RCP Headwall & Channel	300	LF LS	$94	$28,200 $50,000
FE-C	FE-D	Plug Existing RCP		LS		$5,000
FE-D	FE-E	---	600	LF	$0	$0
FE-E	FE-F	---	350	LF	$0	$0
					SUBTOTAL	$147,680
INLET IMPROVEMENTS						
FE-A		25' Inlet	1	EA	$4,100	$4,100
FE-B1		10' Inlet	1	EA	$2,300	$2,300
					SUBTOTAL	$6,700
					TOTAL	$154,080

(See Drawing Set Sheet No. P15-2)

Catalpa Storm Sewer (CA)

Proposed improvements to the Catalpa storm sewer consist of constructing a relief interceptor to replace the existing trunkline and lateral along Catalpa Lane to I27. This relief interceptor would extend from I27, along Catalpa to Star Lane, following Star Lane to Dartmouth Street, down Dartmouth to Campus Drive, west along Campus Avenue to Bell Street, and intercept the Norwich/Hampton Lateral from the Hillside system. This relief interceptor would be sized to take storm flows away from the Hillside/Hampton system, upgrade the I-27 outfall to a greater level of flood protection, and convey storm flows to McCarty Lake.



STUDY AREA: McCARTY LAKE "P15"

PROJECT NAME: S.W. 58TH AVE. "FE"

DESCRIPTION: DIVERT TO McCARTY VIA NEW OUTFALLS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P15-FE)

FIGURE 3.13-3

Inlets would be added along the proposed interceptor route at Star Lane, Campus, and Bell Streets. Storm sewers would be replaced along Greenhaven Road, Western Street, and Catalpa Lane. Storm sewer replacement, storm sewer extensions, inlet improvements, and channel improvements are summarized in Table 3.13-4 and their location shown in Figure 3.13-4.

TABLE 3.13-4
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McCARTY LAKE STUDY AREA (P15)
CATALPA STORM SEWER (CA)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
CA-N	CA-A	Replace w/42" RCP	505	LF	$118	$59,590
CA-A	CA-B	Replace w/42" RCP	1,128	LF	$118	$133,104
CA-C	CA-B	----	484	LF	$0	$0
CA-B	CA-D	----	391	LF	$0	$0
CA-D	CA-E	----	410	LF	$0	$0
CA-E	CA-F	Replace w/42" RCP	666	LF	$118	$78,588
CA-G	CA-G1	Replace w/42" RCP	561	LF	$118	$66,198
CA-G1	CA-F	Replace w/42" RCP	462	LF	$118	$54,516
CA-F	CA-F1	Replace w/48" RCP	340	LF	$139	$47,260
CA-F1	CA-H	Replace w/2-6'x3' RCB	505	LF	$500	$252,500
CA-H	CA-I	Replace w/2-6'x3' RCB	756	LF	$500	$378,000
CA-I	CA-J	Replace w/2-6'x3' RCB	599	LF	$500	$299,500
HH-A	HH-B	Replace w/30" RCP	1,040	LF	$73	$75,920
HH-C	HH-D	----	290	LF	$0	$0
HH-D	HH-E	----	280	LF	$0	$0
HH-E	HH-B	----	780	LF	$0	$0
HH-B	HH-F	----	810	LF	$0	$0
HH-F	HH-G	----	720	LF	$0	$0
HH-G	HH-H	----	750	LF	$0	$0
HH-M	HH-H	Plug Existing 48" RCP		LS		$2,000
HH-H	HC-M	Replace w/60" RCP	500	LF	$204	$102,000
HC-M	HC-N	New 72" RCP	1,200	LF	$213	$255,600

TABLE 3.13-4 (CONTINUED)
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
McCARTY LAKE STUDY AREA (P15)
CATALPA STORM SEWER (CA)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
HC-N	HC-O	New 72" RCP	600	LF	$213	$127,800
HC-O	HC-P	New 72" RCP	1,050	LF	$213	$223,650
HC-P	HC-Q	Replace w/72" RCP	500	LF	$266	$133,000
HH-Y1	HC-Q	Plug Existing 54" RCP		LS		$2,000
HC-Q	CA-K	New 72" RCP	950	LF	$213	$202,350
CA-K	CA-L	Replace w/72" RCP	565	LF	$266	$150,290
CA-L	CA-J	Replace w/72" RCP	1,400	LF	$266	$372,400
CA-J	OUT-FALL	Replace w/2-6'x 4' RCB	143	LF	$500	$71,500
					SUBTOTAL	$3,087,766
INLET IMPROVEMENTS						
		4' Inlets w/ Lat.	2	EA	$1,900	$3,800
		5' Inlets w/ Lat.	2	EA	$2,200	$4,400
		10' Inlets w/ Lat.	2	EA	$2,700	$5,400
		15' Inlets no Lat.	1	EA	$2,700	$2,700
		20' Inlets no Lat.	1	EA	$3,000	$3,000
					SUBTOTAL	$19,300
MANHOLES						
		Manholes	11	EA	$3,500	$38,500
					SUBTOTAL	$38,500
					TOTAL	$3,145,566

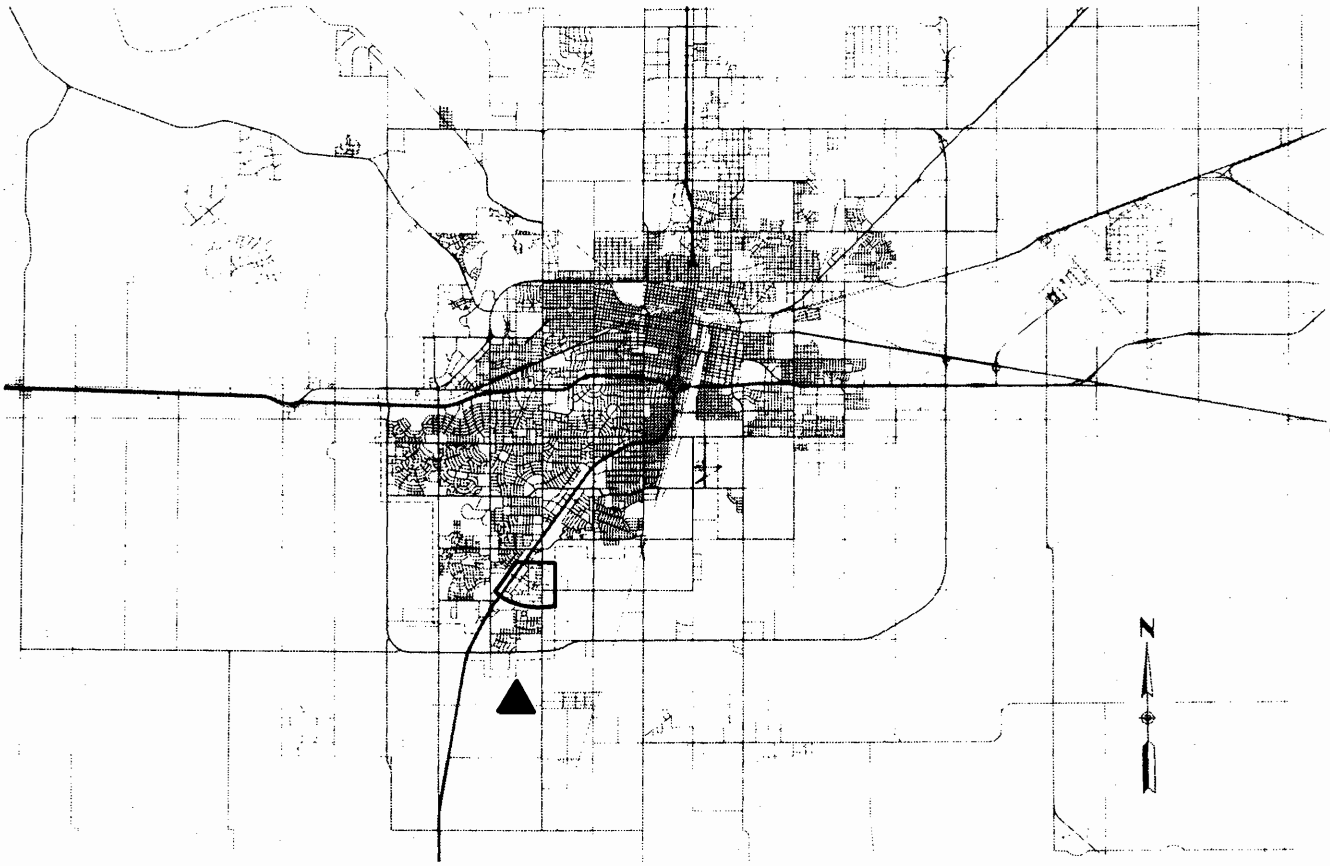
(See Drawing Set Sheet No. P15-1)

3.14 Playa No. 16 Study Area (P16)

3.14.1 Existing Problems

<u>Playa No. 16 (Willow Grove Lake)</u>

Playa No. 16 (Willow Grove Lake, also known as South Washington Lake) is located northeast of the intersection of S.W. 58th Avenue and Washington Street. The playa



STUDY AREA: McCARTY LAKE "P15"

PROJECT NAME: CATALPA LANE "CA"

DESCRIPTION: REPLACE TRUNK/REPLACE SOME LATERALS AND INLETS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P15-CA)

FIGURE **3.13-4**

drainage basin, which totals 1,245 acres, is approximately half low-density residential (LDR) and half undeveloped. Flooding problems have been reported at Willow Grove Lake. A survey of the playa perimeter revealed a primary damage elevation of 3,630.4 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will be reached by the 29-year flood event.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

Rushmore/Hayden Storm Sewer Problem Area (RH)

The Rushmore/Hayden storm sewer systems are located in south Amarillo near South Washington Street and 48th Avenue. Two storm sewer systems convey flows to Willow Grove Lake (Playa No. 16). The north system captures runoff into inlets along S.W. 46th Avenue and conveys them down Hayden Street to S.W. 48th Avenue to an outfall between Hayden and Hughes Street. The south system captures runoff into inlets along Everest Street and conveys them to an outfall at the end of Everest Street. The lower portion of this system is in place; however, the streets have not been constructed.

The contributing drainage area for the north system is about 205 acres to the outfall, and development is primarily residential. The contributing drainage area for the south system is about 307 acres to the outfall, and development is primarily residential, with some undeveloped areas south of S.W. 58th Avenue. Watershed slopes vary from one to two percent.

Flooding problems occur as lake levels in Willow Grove Lake rise, causing a tailwater condition to occur at the storm sewer outfalls. The streets experience shallow flooding as the lake level rises, backing up the storm sewer systems.

3.14.2 Proposed Improvements

Playa No. 16 (Willow Grove Lake)

Under ultimate basin development conditions, the primary damage elevation will be reached at the 20-year flood event. Also, events greater than the five-year event cause water to spread out into the street cuts as well as to the east side of S. Washington Street. A summary of ultimate levels for Playa No. 16 is found in Table A-2 in Appendix A.

Under the 100-year event, the flood level in Willow Grove Lake would reach 3,631.5 feet, 1.1 feet above the primary damage elevation and approximately 2.5 feet above spill/spread elevation of the playa. In order to provide 100-year flood event protection at Willow Grove Lake, a new 500 gpm playa pump station is proposed. Pumped discharges from Playa No. 16 would be directed to the proposed Spring Draw outfall in southwest Amarillo. In addition, approximately 5,000 cubic yards of material should be excavated from the playa in order to connect the two storage chambers to the pump station. Proposed improvements and associated construction costs for Playa No. 16 are summarized in Table 3.14-1 and their location shown in Figure 3.14-1.

A summary of ultimate levels under the proposed improvement scenario is found in Table A-3 in Appendix A.

Rushmore/Hayden Storm Sewer Problem Area (RH)

The storm sewers in this area were analyzed using the "HYDRA" storm sewer modelling program to meet certain conditions. Pipe sizes required to pass a two-year/top-of-curb standard were determined. These pipe sizes and inlet capacities were analyzed using flow curves developed for the City of Amarillo based on drainage area, curve numbers, and watershed slope. The estimated top-of-curb hydraulic capacity of streets as compiled by the

TABLE 3.14-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WILLOW GROVE LAKE STUDY AREA (P16)
WILLOW GROVE LAKE (PLAYA NO. 16)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P16-A		Excavate playa to connect chambers	5,000	CY	$6.00	$30,000
P16-B	P16-C	New 500 gpm pump station with 8" suction line		LS		$110,000
P16-C	P17-D	New 6" force main	7,900	LF	$35.00	$276,500
					TOTAL	$416,500

(See Drawing Set Sheet No. P16-1)

City of Amarillo was computed assuming 37-foot wide streets, with slopes at 0.003 ft/ft.

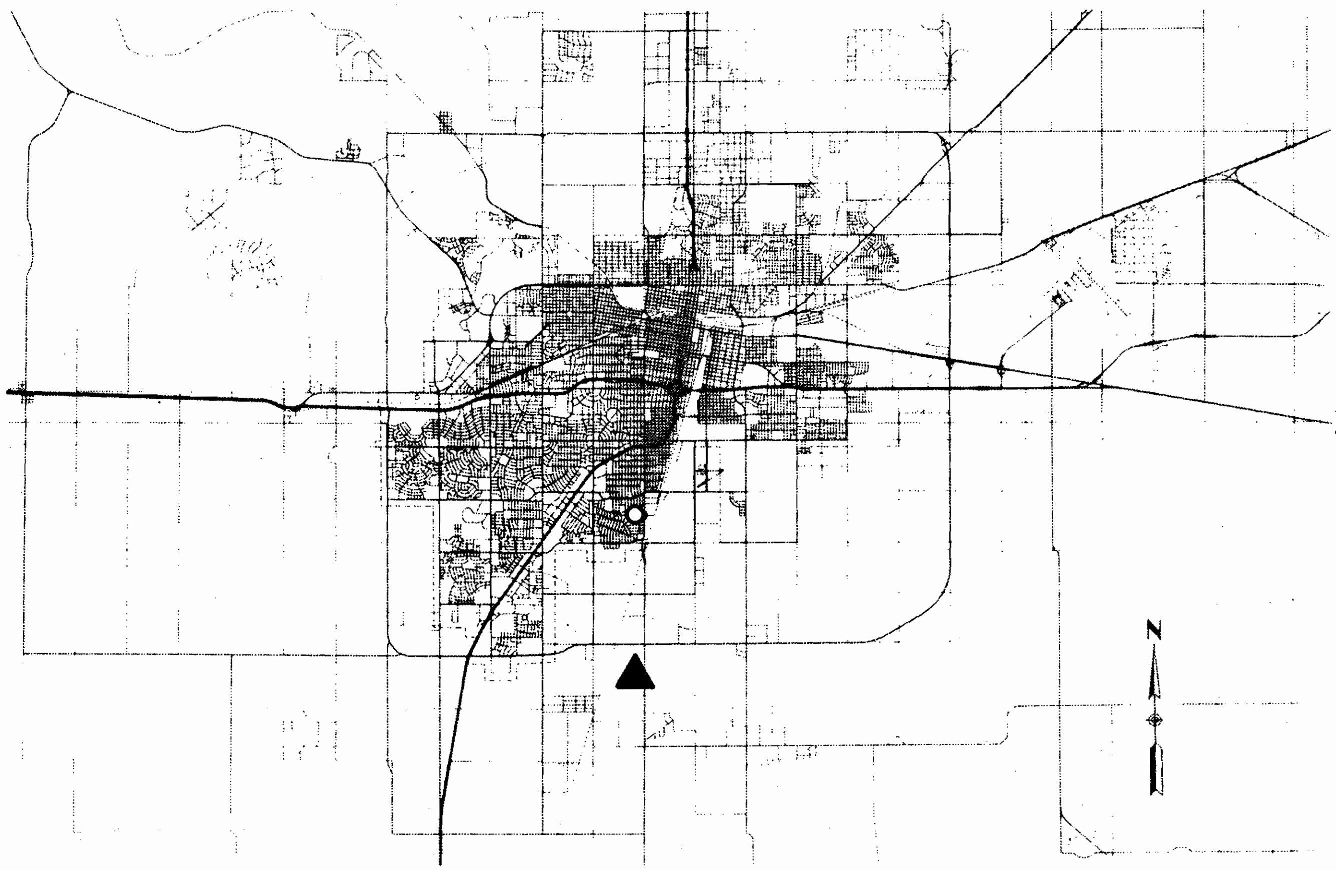
Some of the streets had not been completed based on the originally approved plat. Other assumptions included a minimum storm sewer slope of 0.001, minimum diameter of 18 inches, minimum allowable velocity of two feet per second, and a minimum cover of two feet. It was assumed that what the street does not carry, the storm sewer will. Inlet capacities were increased in some cases at the upstream end of the storm sewer system to capture runoff and put it into the system. Proposed improvements to the north and south systems are summarized in Table 3.14-2 and their location shown in Figure 3.14-2.

3.15 Playa No. 17 Study Area (P17)

3.15.1 Existing Problems

Playa No. 17 (Bennett Lake)

Playa No. 17 is located northeast of the intersection of Georgia Street and S.W. 45th Avenue. The playa drainage basin, which totals 891 acres, is composed primarily of low-density residential and commercial development. A survey of the playa perimeter revealed



STUDY AREA: PLAYA NO. 16 "P16"

PROJECT NAME: WILLOW GROVE LAKE

DESCRIPTION: NEW PUMP STATION AND FORCE MAIN

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P16)

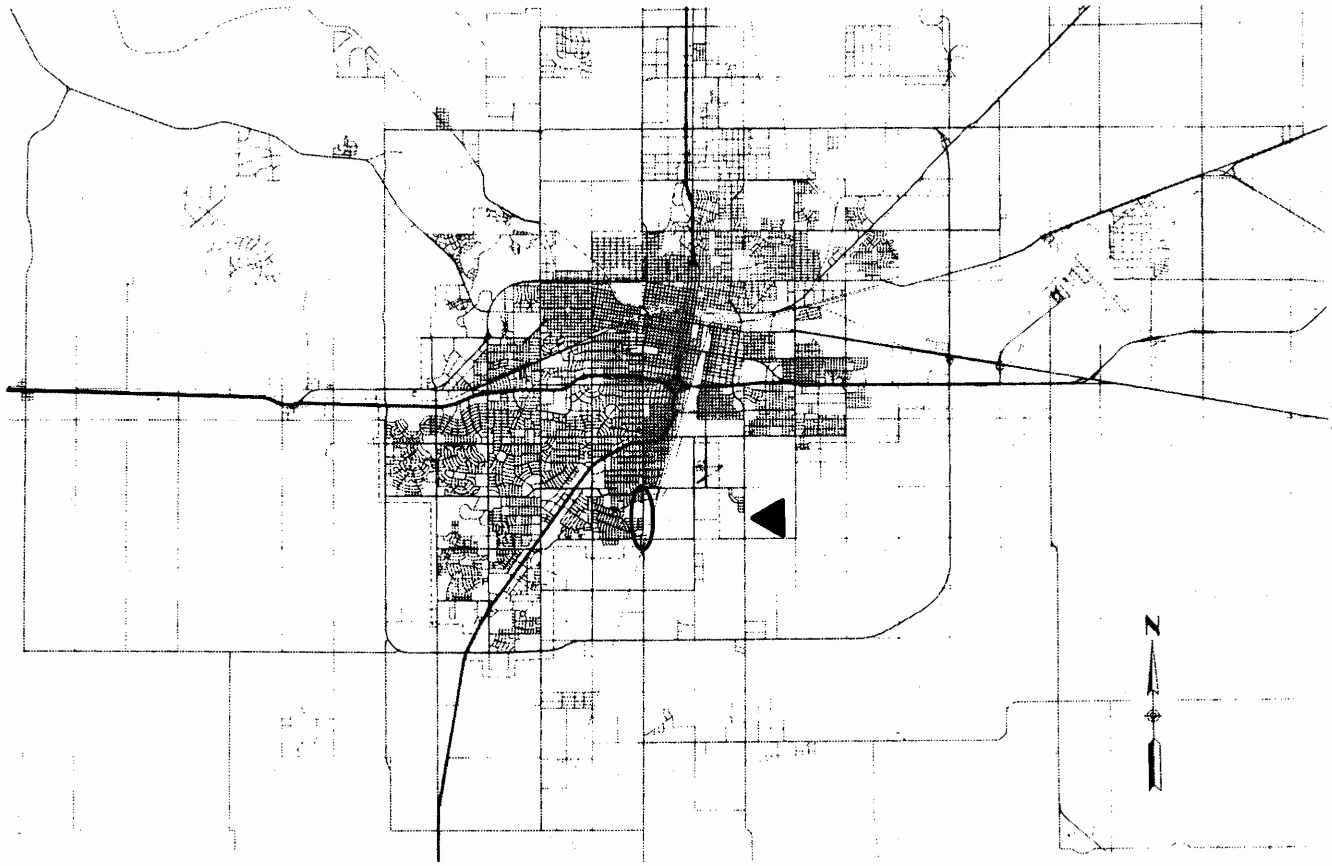
FIGURE
3.14-1

TABLE 3.14-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WILLOW GROVE LAKE STUDY AREA (P16)
RUSHMORE/HAYDEN STORM SEWER (RH)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
RH-I	RH-J	Replace w/60" RCP	625	LF	$204	$127,500
RH-J	RH-K	Replace w/60" RCP	605	LF	$204	$123,420
RH-K	RH-L	Replace w/66" RCP	580	LF	$235	$136,300
RH-L	OUTFALL	Replace w/66" RCP	62	LF	$235	$14,570
RH-A	RH-B	---	577	LF	$0	$0
RH-B	RH-C	---	256	LF	$0	$0
RH-C	RH-D	---	311	LF	$0	$0
RH-F	RH-E	---	255	LF	$0	$0
RH-E	RH-D	---	269	LF	$0	$0
RH-D	RH-G	---	1017	LF	$0	$0
RH-G	RH-H	---	149	LF	$0	$0
RH-H	OUTFALL	---	199	LF	$0	$0
					SUBTOTAL	$401,790
INLET IMPROVEMENTS						
		25' INLET	1	EA	$4,100	$4,100
					SUBTOTAL	$4,100
MANHOLES						
		MANHOLES	3	EA	$3,500	$10,500
					SUBTOTAL	$10,500
					TOTAL	$416,390

(See Drawing Set Sheet No. P16-1)

a primary damage elevation of 3,639.7 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will be reached by the 17-year flood event.



STUDY AREA: WILLOW GROVE "P16"

PROJECT NAME: RUSHMORE/HAY "RH"

DESCRIPTION: REPLACE EXISTING STORM SEWER

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P16-RH)

FIGURE 3.14-2

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

3.15.2 Proposed Improvements

Playa No. 17 (Bennett Lake)

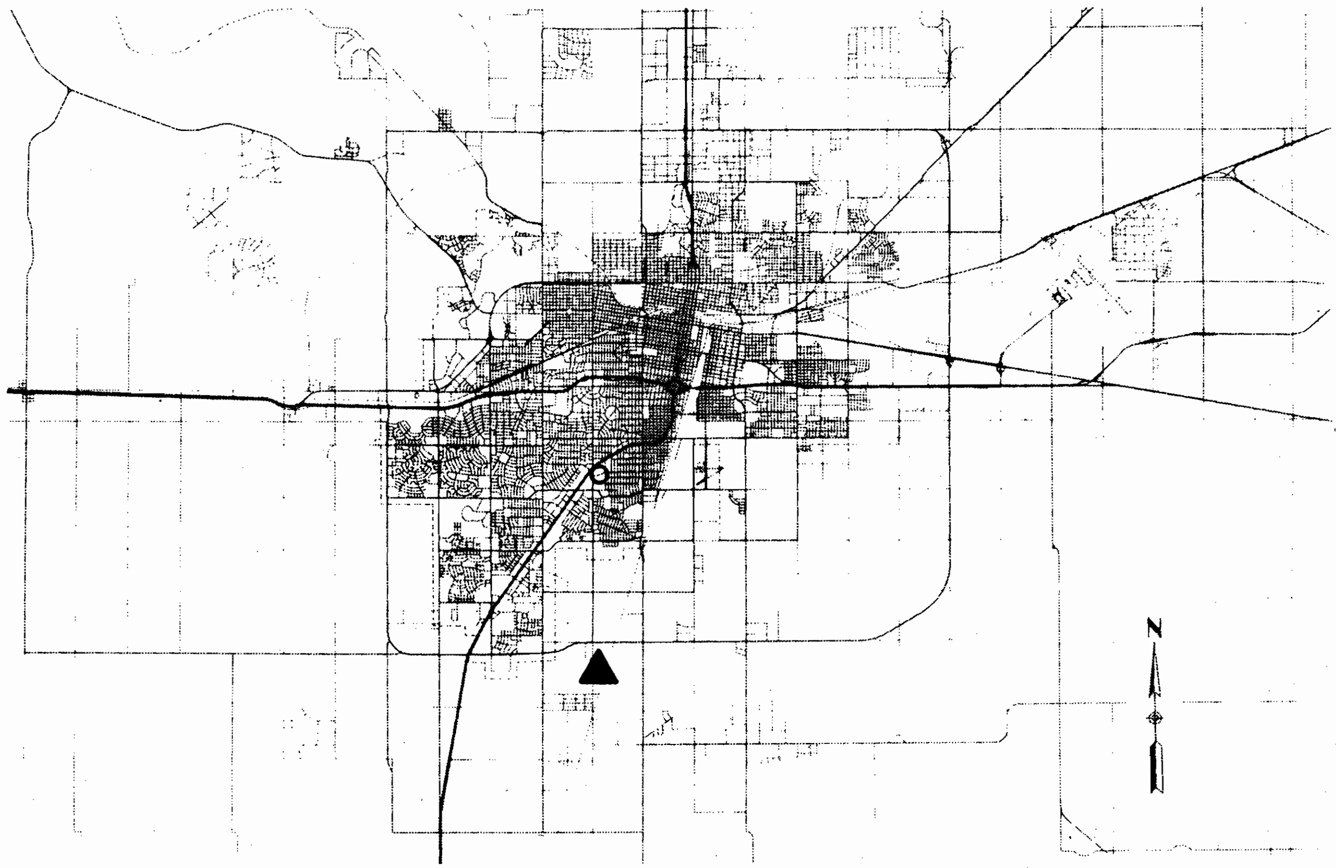
Under ultimate basin development conditions, the primary damage elevation will be reached at the 17-year flood event. Under the 100-year event, the flood level in Bennett Lake would reach 3,642.5 feet, 2.8 feet above the primary damage elevation. A summary of ultimate levels for Playa No. 17 is found in Table A-2 in Appendix A.

In order to provide 100-year protection at Bennett Lake, the existing playa pump would be mothballed and a new 5,000 gpm pump station constructed. In addition, approximately 140,000 cubic yards of earth would need to be excavated from the playa. Pumped discharges from Playa No. 17 would be directed to the Spring Draw outfall in southwest Amarillo. The existing pump and force main to Lawrence Lake will be left intact for emergency or occasional use. Proposed improvements and associated construction costs for Playa No. 17 are summarized in Table 3.15-1 and their location shown in Figure 3.15-1.

TABLE 3.15-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 17 STUDY AREA (P17)
BENNETT LAKE (PLAYA NO. 17)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P17-A		Excavate playa	140,000	CY	$5.00	$700,000
P17-B	P17-C	New 5,000 gpm pump station		LS		$225,000
P17-C	P17-D	New 20" force main	3,900	LF	$60.00	$234,000
P17-D	P17-E	New 20" force main	5,500	LF	$60.00	$330,000
P17-E	P17-F	New 27" force main	7,500	LF	$70.00	$525,000
P17-F	P17-G	New 30" force main	13,300	LF	$80.00	$1,040,000
P17-G	P17-H	New 33" force main	19,400	LF	$85.00	$1,649,000
					TOTAL	$4,703,000

(See Drawing Set Sheet No. P17-1)



STUDY AREA: PLAYA NO. 17 "P17"

PROJECT NAME: BENNETT LAKE

DESCRIPTION: UPGRADE PUMP STATION/REROUTE FORCE MAIN/ EXCAVATE PLAYA

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P17)

FIGURE
3.15-1

A summary of ultimate levels under the proposed improvement scenario is found in Table A-3 in Appendix A.

3.16 Playa No. 18 Study Area (P18)

3.16.1 Existing Problems

Playa No. 18

Playa No. 18 is located just south of the intersection of Farmers Road and Burlington Road.

The playa drainage basin, which totals 4,235 acres, is essentially undeveloped. A survey of the playa perimeter revealed a primary damage elevation of 3,581.8 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,581.4 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year flood event.

3.16.2 Proposed Improvements

Playa No. 18

Under ultimate basin development conditions, the 100-year level for Playa No. 18 is 3,582 feet, slightly above the existing primary damage elevation of 3,581.8 feet. A summary of ultimate levels for Playa No. 18 is found in Table A-2 in Appendix A.

In order to ensure that the 100-year flood elevation under ultimate conditions remains at or below the primary damage elevation, approximately 60,000 cubic yards of material should be removed from Playa No. 18. A portion of this material should be reused to raise Burlington Road to an elevation at or above the 25-year elevation of 3580.7 feet. Proposed improvements and associated construction costs for Playa No. 18 are summarized in Table 3.16-1 and their location shown in Figure 3.16-1.

TABLE 3.16-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 18 STUDY AREA (P18)
PLAYA NO. 18

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P18-A		Excavate playa	60,000	CY	$4.50	$270,000
		Raise Burlington Road	3,800	LF	$60.00	$228,000
					TOTAL	$498,000

(See Drawing Set Sheet No. P18-4)

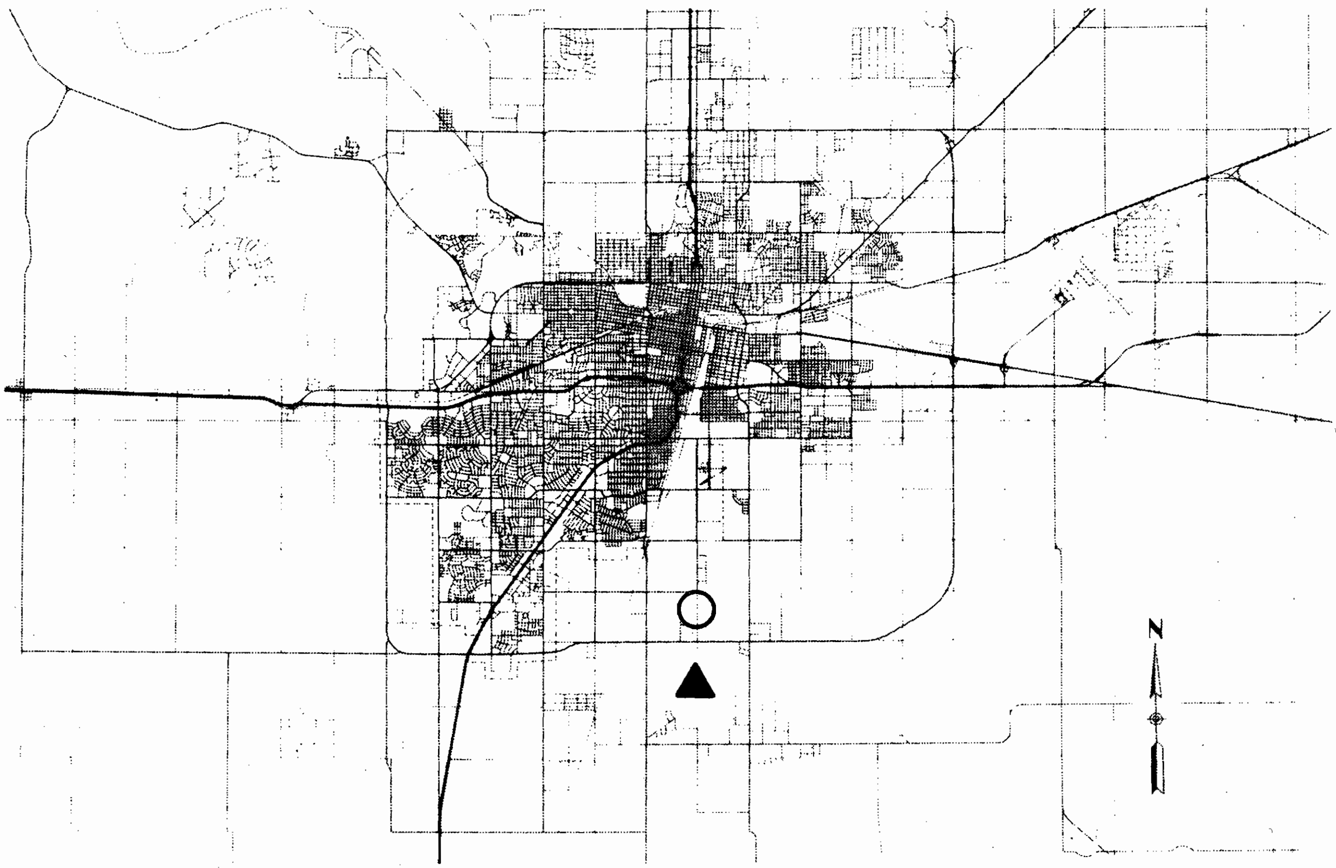
A summary of ultimate levels under the proposed improvements scenario is found in Table A-3 in Appendix A.

3.17 Playa No. 19 Study Area (P19)

3.17.1 Existing Problems

<u>Playa No. 19</u>

Playa No. 19 is located just east of the Santa Fe Railroad tracks between 34th and 46th Avenues. The playa drainage basin, which totals 712 acres, is primarily industrial with some low density residential development. A survey of the Playa No. 19 perimeter revealed a primary damage elevation of 3,639.8 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,635.9 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year flood event.



STUDY AREA: PLAYA NO. 18 "P18"

PROJECT NAME: PLAYA NO. 18

DESCRIPTION: EXCAVATE PLAYA/RAISE ROAD

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P18)

FIGURE 3.16-1

3.17.2 Proposed Improvements

Playa No. 19

Under ultimate basin development conditions, the 100-year flood level for Playa No. 19 is 3,636.1 feet, well below the existing primary damage elevation of 3,639.8 feet. Therefore, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 19 is found in Table A-2 in Appendix A.

3.18 Playa No. 20 Study Area (P20)

3.18.1 Existing Problems

Playa No. 20 (Gooch Lake)

Playa No. 20 is located in Southeast Park just south of 46th Avenue. The playa drainage basin, which totals 7,197 acres, is primarily undeveloped with portions devoted to parkland and low-density residential (LDR) use. A survey of the playa perimeter revealed a primary damage elevation of 3,578.5 feet. Results of the ASAPP model indicate that, under existing watershed conditions, the 100-year flood elevation for the playa is 3,577.2 feet, 1.3 feet below the primary damage elevation.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

27th Avenue/Railroad Storm Sewer Problem Area (R3)

Problems occur in the area of 27th Avenue and ATSF Railroad when runoff accummulates in the 27th Avenue underpass sag beneath the railroad span and prevents vehicle passage due to flood water depth. The underpass is currently drained by four inlets connected to a 36-inch storm sewer. A small sump pump discharges into the underpass from the ramp area of the solid waste transfer station at 27th Avenue and Grant.

Likely causes of flooding at the 27th Avenue/railroad underpass are: 1) inadequate inlet capacity of the inlets located in the sag of the underpass; 2) inadequate storm sewer capacity to discharge at high enough flow rates to keep the underpass from flooding; and 3) additional runoff which is diverted to the underpass which might otherwise go to another storm sewer system.

3.18.2 Proposed Improvements

Playa No. 20 (Gooch Lake)

Under ultimate basin development conditions, the 100-year flood elevation in Playa No. 20 is 3,577.6 feet, 0.9 feet below the primary damage elevation. Therefore, since Gooch Lake has adequate capacity for all flood events up through the 100-year event under ultimate conditions, no improvements are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 20 is found in Table A-2 in Appendix A.

27th Avenue/Railroad Storm Sewer Problem Area (R3)

Proposed improvements for the 27th Avenue/Railroad system consist of additional inlet capacity west of the sag to intercept storm water before it enters the sag under the railroad. The improvements are summarized in Table 3.18-1 and their location shown in Figure 3.18-1.

TABLE 3.18-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
GOOCH LAKE STUDY AREA (P20)
27th AVENUE/RR STORM SEWER (R3)

INLET/NODE FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
R3-CA	R3-CB	New 24" lateral	130	LF	$31	$4,030
					SUBTOTAL	$4,030
INLET IMPROVEMENTS						
		4' Inlets	4	EA	$1,500	$6,000
					SUBTOTAL	$6,000
MANHOLES						
		Manholes	1	EA	$3,500	$3,500
					SUBTOTAL	$3,500
					TOTAL	$13,530

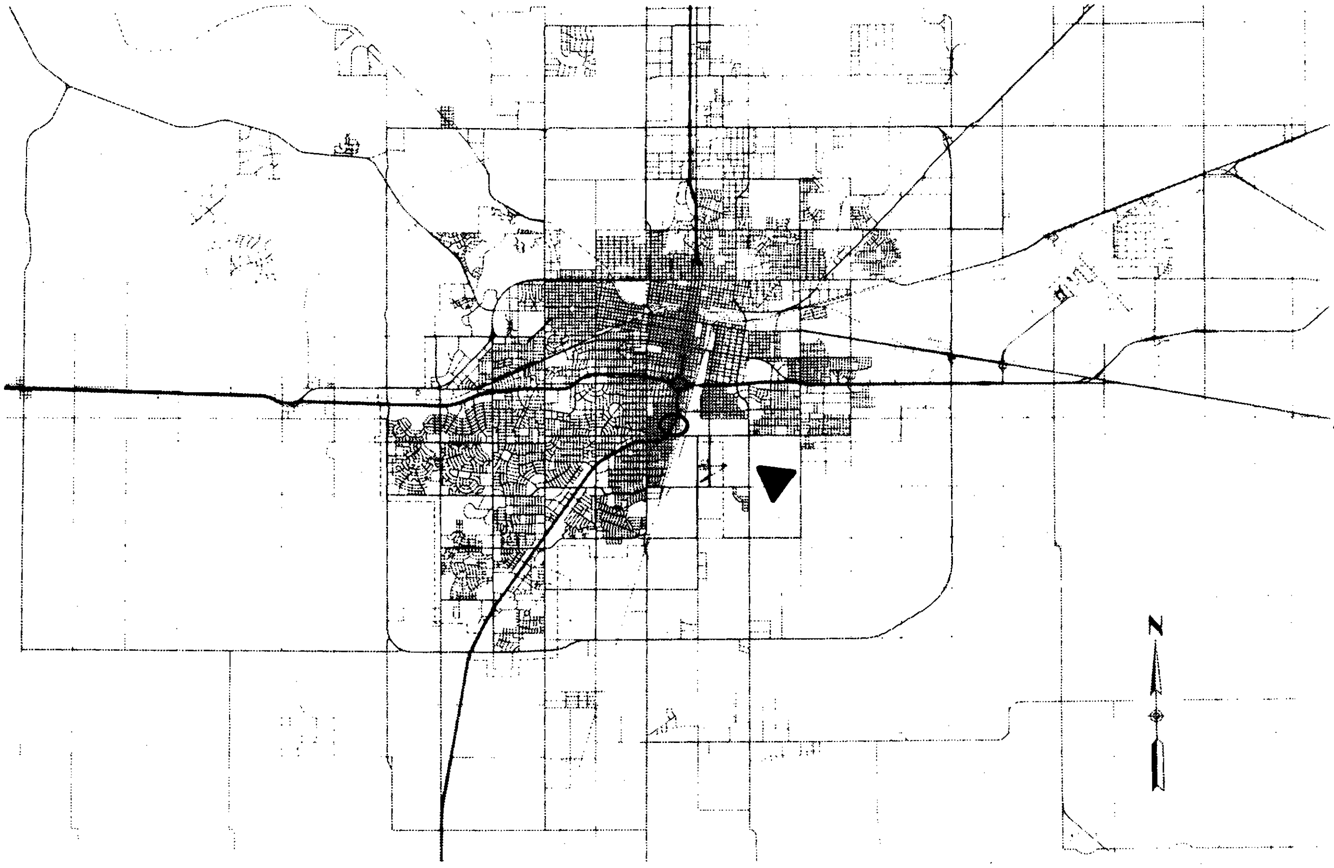
(See Drawing Set Sheet No. P20-3)

3.19 Playa No. 21 Study Area (P21)

3.19.1 Existing Problems

Playa No. 21 (T-Anchor Lake)

Playa No. 21 is located north of Interstate 40 and east of Ross Street. The playa drainage basin, which totals 2,612 acres, is fully developed. Development is mixed, consisting primarily of low-density residential, commercial, and industrial development. Flooding problems have been reported at T-Anchor Lake. A single pump, rated at approximately 3,000 gpm, is used to dewater T-Anchor Lake during dry weather. A survey of the playa perimeter revealed a primary damage elevation of 3,613.6 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will be reached by the 41-year flood event.



STUDY AREA: GOOCH LAKE "P20"

PROJECT NAME: 27TH AVE./RR "R3"

DESCRIPTION: ADD LATERAL/ADD INLET CAPACITY

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P20-R3)

FIGURE
3.18-1

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

10th Avenue/Railroad Storm Sewer Problem Area (R2)

Problems occur in the area of 10th Avenue and the ATSF Railroad when runoff accumulates in the 10th Avenue underpass sag beneath the railroad span and prevents vehicle passage due to flood water depth. The underpass is currently drained by two inlets on the north and south curbs of 10th Avenue below the railroad span. Laterals from the inlets combine at a 24-inch trunk line which runs along 10th Avenue, collecting runoff from intervening areas along the way before discharging near T-Anchor Lake as a 48-inch outfall.

A second storm drain system collects surface runoff from the area northwest of the 10th Avenue/Railroad area, including the intersection of 10th Avenue and Grant, and discharges to T-Anchor Lake through storm sewers running along 11th Avenue. This system includes a 42-inch storm sewer which runs through the underpass, but is not directly connected to the other system.

There are three likely causes of flooding at the 10th Avenue/Railroad underpass: 1) inadequate inlet capacity of the two sag inlets; 2) inadequate capacity in the 10th Avenue storm sewer system; or 3) inadequate capacity in the 11th Avenue storm sewer system which causes runoff to bypass the curb inlets at the corner of 10th Avenue and Grant and flow into the sag.

A review of the hydraulic performance of each storm sewer system indicates that the main problem is excessive surcharging of the 10th Avenue storm sewer system. When this system is analyzed under the two-year storm event, the hydraulic grade line (HGL) in the 24-inch line at the sag is approximately 3,651 feet. The elevation of the bottom of the sag is approximately 3,641 feet, about 10 feet below the HGL. Under this event, water would pond to over 10 feet deep in the sag before drainage would occur.

Interstate 40 Storm Sewer Problem Area (I4)

The I40 problem area is located east of the downtown area of Amarillo. Existing storm sewer systems along I40 drain to T-Anchor Lake. The systems control runoff draining from the south to the north to T-Anchor Lake through drainage ditches and culverts along I40. Existing flooding problems have been experienced along the south frontage road of I40 from Osage Street to Seminole Street. The storm sewer sytems back up as a result of high lake levels in T-Anchor Lake. This effect has caused some shallow street flooding along Osage, Birmingham, Nelson, Manhattan, Highland, and Marrs Streets just south of I40.

3.19.2 Proposed Improvements

Playa No. 21 (T-Anchor Lake)

Under ultimate basin development conditions, the primary damage elevation will be reached at the 41-year flood event. Under the 100-year event, the flood level in T-Anchor Lake would reach 3,614.5 feet, 0.9 feet above the primary damage elevation. A summary of ultimate levels for Playa No. 21 is found in Table A-2 in Appendix A.

In order to provide 100-year protection at T-Anchor Lake, additional pumping capacity of 3,100 gpm is proposed. The existing pumping station at T-Anchor Lake would be upgraded with an additional pump, arranged in parallel, to bring the total pumping capacity to 6,100 gpm when rated at the pump shutoff elevation of 3,582 feet. To accommodate the increased flow rate, the existing 16-inch discharge line would need to be replaced by a 24-inch line and the existing 16-inch suction line replaced with a 24-inch line or paralleled with a 20-inch line. In addition to the increased pump capacity, approximately 30,000 cubic yards of material should be excavated from the chamber openings to ensure that the full playa capacity is available at all times. Proposed improvements and estimated construction costs are summarized in Table 3.19-1 and their location shown in Figure 3.19-1.

A summary of ultimate levels for Playa No. 21 under the proposed improvements scenario is found in Table A-3 in Appendix A.

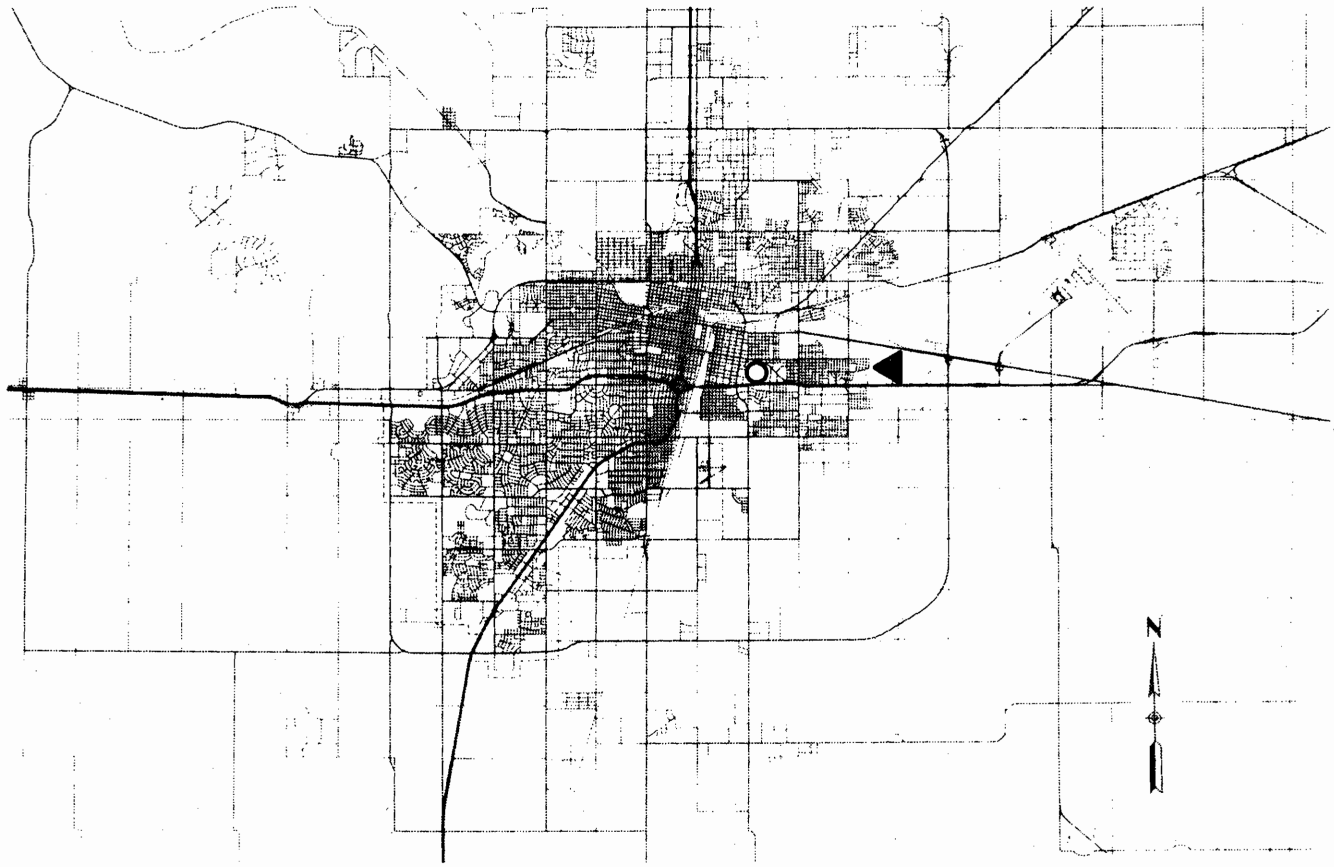
TABLE 3.19-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
T-ANCHOR LAKE STUDY AREA (P21)
T-ANCHOR LAKE (PLAYA NO. 21)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P21-A		Excavate deeper chamber openings	30,000	CY	$6.00	$180,000
P21-B	P21-C	Add new 3,100 gpm pump	1	LS		$140,000
		Replace 16" suction line with 24"	100	LF	$70.00	$7,000
		Raise top of structure to el. 3614.0'		LS		$10,000
P21-C	P21-D	Replace 16" force main with 24"	7,700	LF	$70.00	$539,000
					TOTAL	$876,000

(See Drawing Set Sheet No. P21-2)

10th Avenue/Railroad Storm Sewer Problem Area (R2)

Proposed improvements for the 10th Avenue/railroad system consist of a new storm water pump station located in the 10th Avenue median approximately 70 feet east of the railroad bridge center pier where the two 18-inch laterals meet at the manhole and form the 24-inch trunk sewer. A 4,000 gpm vertical propeller pump is recommended. This rating should be sufficient to handle the peak two-year runoff from a contributing area of approximately 2.4 acres. The wetwell should be sized to provide the minimum pump cycle time recommended by the manufacturer; any additional wetwell oversizing would have the benefit of allowing the pumping system to handle larger flood events. The existing inlets



STUDY AREA: PLAYA NO. 21 "P21"

PROJECT NAME: T-ANCHOR LAKE

DESCRIPTION: UPGRADE PUMP STATION AND FORCE MAIN/EXCAVATE PLAYA

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P21)

FIGURE 3.19-1

and 18-inch laterals should be adequate to feed the wetwell.

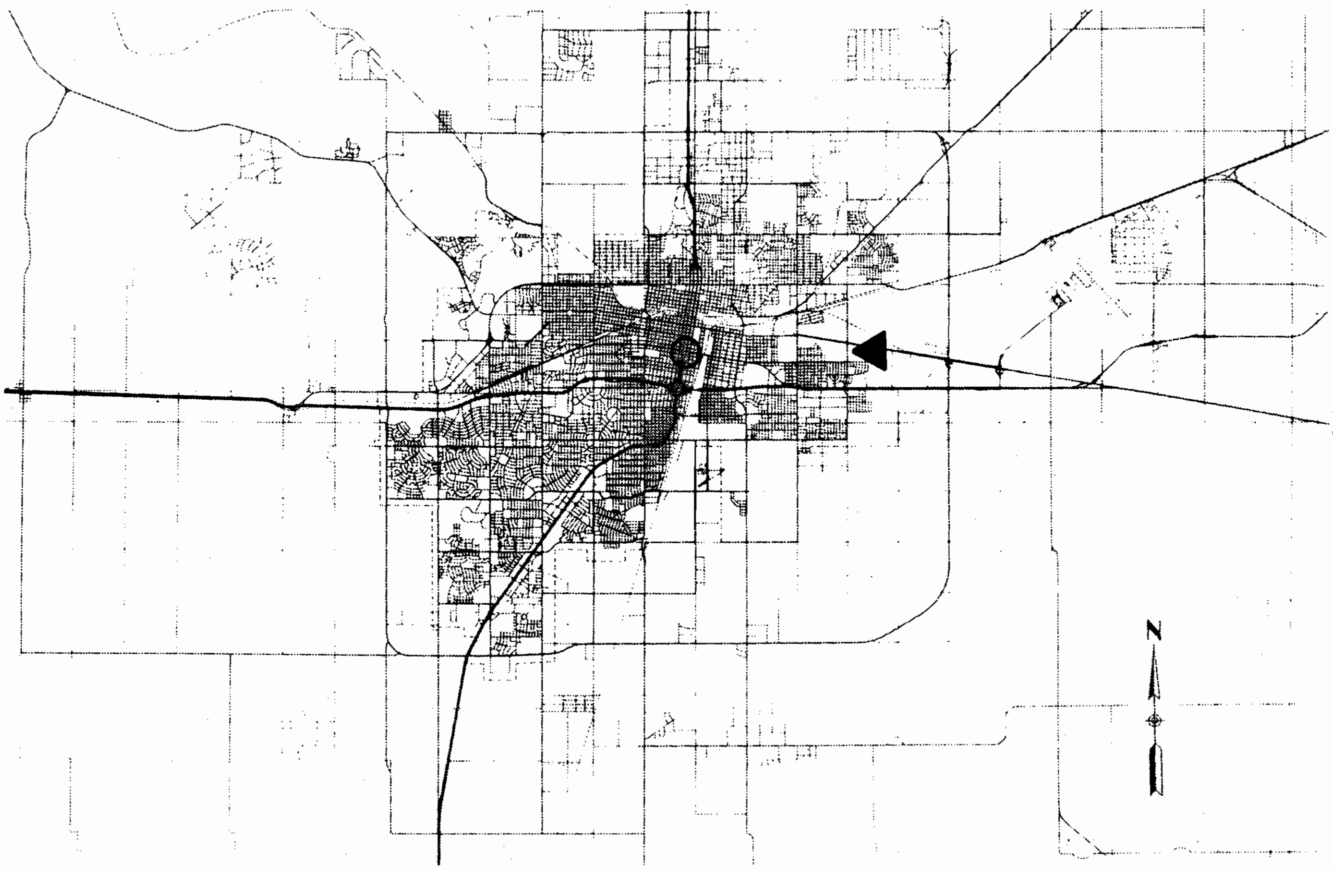
Construction of the wetwell will require the blocking of at least two traffic lanes. The required plan dimensions of the wetwell will require that it extend beneath the left-hand traffic lanes in each direction. The pump motor should be centered on the median, protected with posts or an enclosure, and located with a baseplate elevation no lower than about 3,544 feet msl. The pump discharge would be connected directly to the existing 24-inch reinforced concrete storm sewer.

To ensure that no additional runoff beyond that from the 2.4-acre contributing area enters the sag, the inlets on the corner of Grant and 10th Avenue should be cleared of any obstructions and routinely maintained. Also, the curb inlet on the northwest corner of 10th Avenue and Garfield (node R2-B) should be modified as necessary to collect surface runoff from the 3.6-acre area north of the proposed pump station. This work may include minor grading of the area. Improvements are summarized in Table 3.19-2 and their location shown in Figure 3.19-2.

TABLE 3.19-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
T-ANCHOR LAKE STUDY AREA (P21)
10TH AVENUE/RAILROAD STORM SEWER (R2)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
PUMP STATION IMPROVEMENTS						
R2-AB		New 4,000 gpm pump station		LS		$75,000
R2-AB	R2-B	Upgrade 24" RCP	1	EA	$5,000	$5,000
					SUBTOTAL	$80,000
INLET IMPROVEMENTS						
R2-B		Modify to 10' curb and area inlet	1	EA	$10,000	$10,000
					SUBTOTAL	$10,000
					TOTAL	$90,000

(See Drawing Set Sheet No. P21-1)



STUDY AREA: T-ANCHOR "P21"

PROJECT NAME: 10TH AVE./RR "R2"

DESCRIPTION: DIVERT PORTION OF AREA/ADD NEW PUMP STATION

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P21-R2)

FIGURE
3.19-2

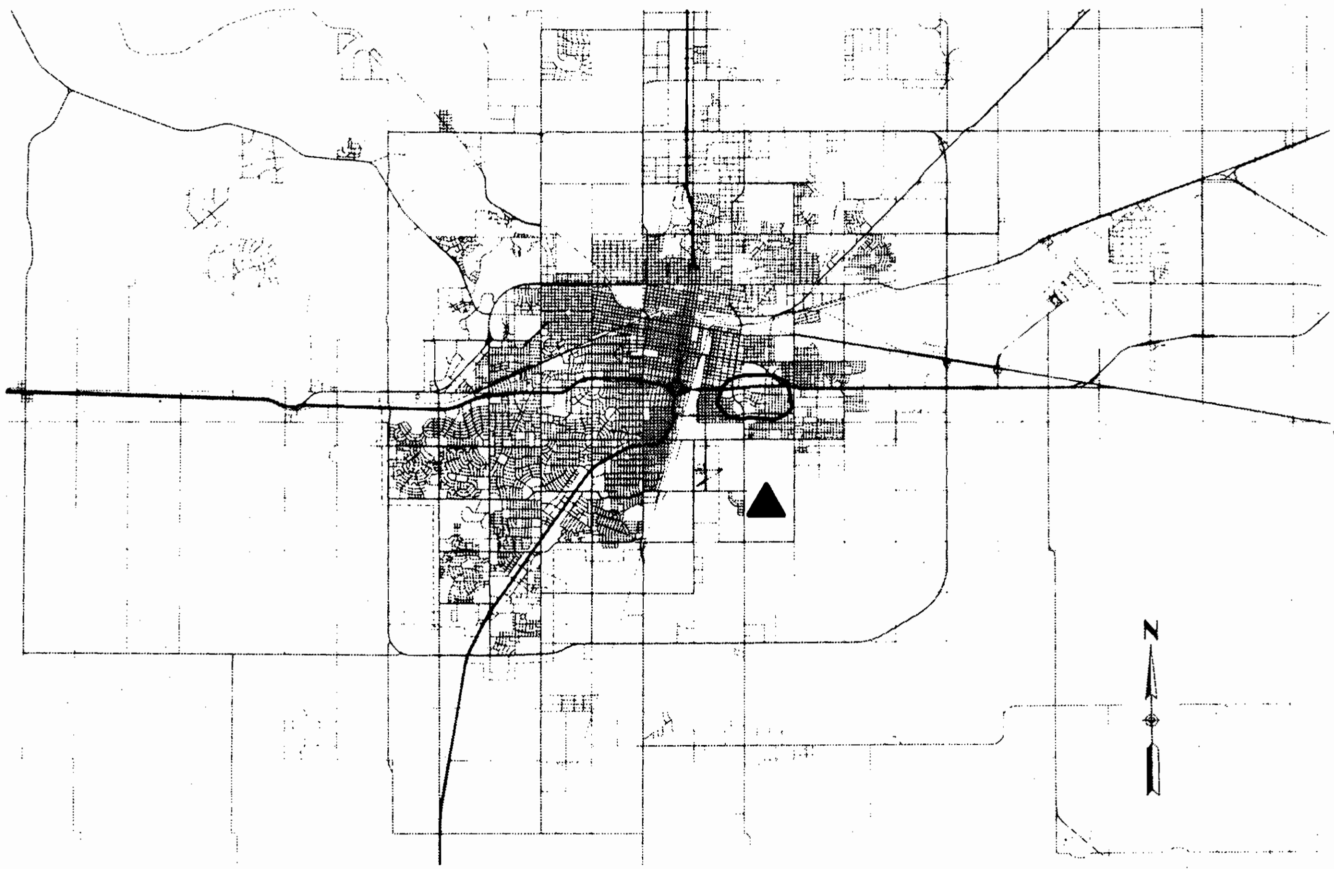
Interstate 40 Storm Sewer Problem Area (I4)

Proposed drainage improvements to the Interstate 40 (I4) problem area are to extend laterals south from the I40 storm sewers along the south frontage road at selected intervals to intercept runoff in residential areas before it reaches the frontage road. A lateral extension with inlets proposed east of Osage Street along 27th Avenue will tie into the existing Osage system to intercept an area south of 27th Avenue. The existing Osage system should be upgraded from the intersection of Osage Street and 27th Avenue along Osage Street and 21st Avenue to I40. A proposed extension with inlets at the intersection of Birmingham and 19th Avenue will run north to the I40 system. Another proposed lateral extension will run along Nelson Street and 19th Avenue with inlets proposed at the intersections of Nelson Street and 22nd Avenue, 19th Avenue and Nelson Street, and Pittsburg Street and Nelson Street. From the intersection of Pittsburg Street and Nelson Street, it will extend north to connect with the I40 system. The last proposed extension south of I40 begins at the intersection of 19th Avenue and Seminole Street and extends west to Marrs Street. From Marrs Street it will extend north to the I40 system. Inlets are proposed at the intersections of 19th Avenue and Seminole Street, 19th Avenue and Roosevelt Street, 19th Avenue and Marrs Street, and Marrs Street and I40 service road. Improvements are summarized in Table 3.19-3 and their location shown in Figure 3.19-3.

TABLE 3.19-3
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
T-ANCHOR LAKE STUDY AREA (P21)
INTERSTATE 40 STORM SEWER (I4)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
I4-L	I4-M	New 36" RCP	400	LF	$99	$39,600
I4-M	I4-M.1	Replace w/36" RCP	515	LF	$99	$50,985
I4-M.1	I4-N	Replace w/48" RCP	825	LF	$139	$114,675
I4-N	I4-O	Replace w/48" RCP	855	LF	$139	$118,845
I4-O	I4-R	Replace w/2-6'x3' RCB	600	LF	$250	$150,000
I4-P	I4-R	New 1-6'x3' RCB	600	LF	$125	$75,000
I4-R	OUTFALL	Replace w/2-6'x3' RCB	1141	LF	$250	$285,250
I4-K	OUTFALL	New 36" RCP	460	LF	$79	$36,340
I4-G	I4-H	New 24" RCP	950	LF	$31	$29,450
I4-H	I4-I	New 30" RCP	350	LF	$58	$20,300
I4-I	I4-J	New 36" RCP	320	LF	$79	$25,280
I4-J	OUTFALL	New 36" RCP	570	LF	$79	$45,030
I4-A	I4-B	New 30" RCP	330	LF	$58	$19,140
I4-B	I4-C	New 48" RCP	330	LF	$111	$36,630
I4-C	OUTFALL	New 48" RCP	750	LF	$111	$83,250
					SUBTOTAL	$1,129,775
INLET IMPROVEMENTS						
		4' Inlet	19	EA	$1,900	$36,100
		5' Inlet	7	EA	$2,200	$15,400
		10' Inlet	5	EA	$2,700	$13,500
		15' Inlet	4	EA	$3,100	$12,400
		20' Inlet	3	EA	$3,600	$10,800
		25' Inlet	2	EA	$4,100	$8,200
		30' Inlet	2	EA	$4,600	$9,200
					SUBTOTAL	$105,600
MANHOLES						
		Manholes	9	EA	$3,500	$31,500
					SUBTOTAL	$31,500
					TOTAL	$1,266,875

(See Drawing Set Sheet No. P21-4)



STUDY AREA: T-ANCHOR "P21"

PROJECT NAME: INTERSTATE 40 "I4"

DESCRIPTION: ADD NEW LATERALS/ADD INLET CAPACITY

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P21-I4)

FIGURE
3.19-3

3.20 Playa No. 22 Study Area (P22)

3.20.1 Existing Problems

Playa No. 22

Playa No. 22 is located north of S.E. 3rd Avenue and east of Eastern Street. The playa drainage basin, which totals 5,135 acres, is primarily industrial with some low-density residential (LDR) development. A survey of the playa perimeter revealed a primary damage elevation of 3,614.2 feet. Results of the ASAPP model indicate that, under existing watershed conditions, the 100-year water surface elevation reaches 3,590.9 feet, well below the primary damage elevation.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

Unnamed Tributary Problem Area (UT)

An unnamed tributary located in northeast Amarillo drains to Playa No. 22. This tributary orginates north of Panhandle Boulevard (Fritch Hwy.) and drains southeasterly across the abandoned Chicago, Rock Island & Pacific (CRI&P) Railroad at Eastern Street. From there, it drains under Northeast 13th and Northeast 11th Avenues to Amarillo Boulevard. At Amarillo Boulevard, a 54-inch diameter storm sewer connects to the culvert carrying the unnamed tributary under the roadway. The tributary continues as a well-defined ditch to a bridge at the ATSF Railroad crossing. Another abandoned CRI&P right-of-way crosses just below the ATSF bridge.

The contributing drainage area intercepts over 1,300 acres prior to entering Playa No. 22. The watershed is primarily undeveloped in the upper portion, with some residential areas east of Eastern Street and commercial areas along Amarillo Boulevard. Watershed slopes vary from 0.5 to one percent.

Flooding problems occur even from very light showers at the abandoned CRI&P railroad underpass at Eastern Street. Flows detour around the old CRI&P embankment, at times flooding the CRI&P railroad crossing. Other problems have occurred with siltation of the culverts at N.E. 13th and N.E. 11th Avenue A tailwater situation exists at the outfall from the culvert at Amarillo Boulevard.

3.20.2 Proposed Improvements

Playa No. 22

Under ultimate basin development conditions, the 100-year water surface elevation will reach 3,591 feet, some 23 feet below the primary damage elevation. Since Playa No. 22 has more than adequate capacity to store runoff for all events through the 100-year event, no improvements are proposed in the Master Plan.

A summary of ultimate levels for Playa No. 22 is found in Table A-2 in Appendix A.

Unnamed Tributary Problem Area (UT)

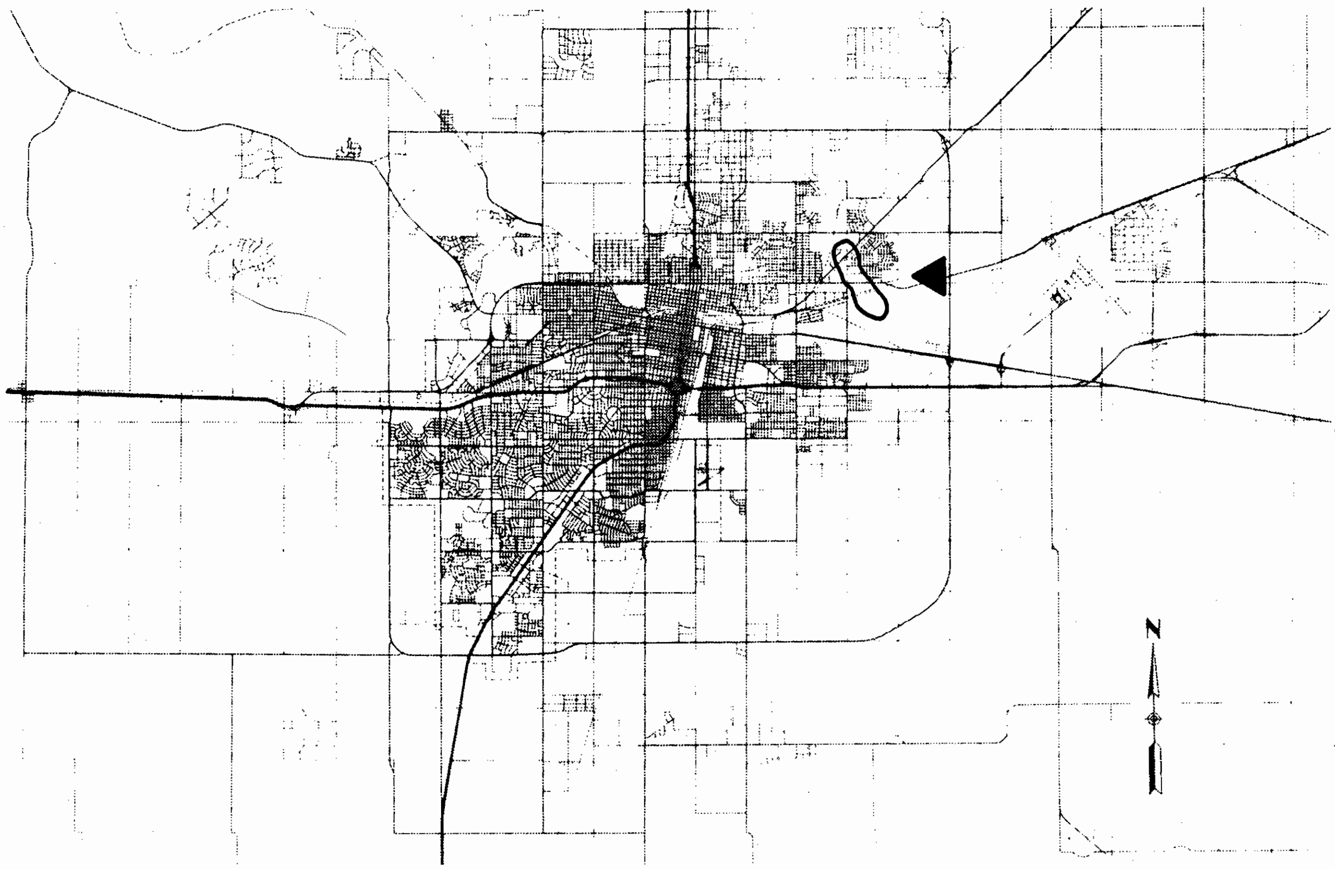
The unnamed tributary was analyzed using the HEC-2 Water Surface Profiles program. An existing condition stream model was constructed from field survey information, bridge plans from the Texas Department of Transportation, as-built construction plans from the City of Amarillo, and cross sections taken from 1 inch=200 feet topographic maps. Storm flows at selected discharge points were computed from peak runoff curves developed for Amarillo, Texas. For purposes of this problem area, the ultimate two-year, five-year, 10-year, 25-year, 50-year, and 100-year flows were computed and used in the model study.

Proposed improvements consist of realigning the unnamed tributary through the abandoned CRI&P Railroad ROW, avoiding the Eastern Street crossing, making culvert and headwall improvements at N.E. 13th, N.E. 11th, and Amarillo Boulevard, and channel improvements below and above the ATSF railroad crossing as the tributary outfalls to Playa 22. Improvements are summarized in Table 3.20-1 and their location shown in Figure 3.20-1.

TABLE 3.20-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 22 STUDY AREA (P22)
UNNAMED TRIBUTARY CHANNEL REACH (UT)

CHANNEL STATION						
FROM	**TO**	**PROPOSED IMPROVEMENTS**	**NO. UNITS**	**UNIT**	**UNIT COST**	**TOTAL COST**
UT 30+41	UT 31+40	Replace w/2 - 8'x 5' RCB at Amarillo Boulevard	99	LF	$405	$40,095
UT 31+40	UT 56+40	Improve channel from Amarillo Boulevard through N.E. 13th Avenue	2500	CY	$8.00	$20,000
UT 45+34	UT 45+66	Replace w/3 - 5'x 4' RCB at N.E. 11th Avenue	32	LF	$940	$30,080
UT 53+80	UT 54+20	Replace w/3 - 8'x 3' RCB at N.E. 13th Avenue	40	LF	$750	$30,000
UT 66+73	UT 66+87	New crossing through abandoned RR embankment east of N. Eastern Street		LS		$30,000
UT 73+96	UT 77+84	Replace culverts at Panhandle Boulevard		LS		$30,000
					TOTAL	$180,175

(See Drawing Set Sheet No. P22-1)



STUDY AREA: "P22"

PROJECT NAME: UNNAMED TRIBUTARY "UT"

DESCRIPTION: CULVERT AND ROAD IMPROVEMENTS/ CHANNELIZE UPPER REACHES

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P22-UT)

FIGURE 3.20-1

3.21 Playa No. 23 Study Area (P23)

3.21.1 Existing Problems

Playa No. 23 (Wild Horse Lake)

Playa No. 23 is located just south of Amarillo Boulevard and west of Hughes Street. The playa drainage basin, which totals 2,330 acres, is primarily industrial with some low-density residential (LDR), high-density residential (HDR), and commercial development. The basin is essentially completely developed. Approximately 800 acres of the 2,330 total acreage is served by storm sewers which divert some flow out of the Wild Horse basin and into East Amarillo Creek. A survey of the playa perimeter revealed a primary damage elevation of 3,619.2 feet. Results of the ASAPP model indicate that, under existing watershed conditions, the 100-year water surface elevation reaches 3,616.7 feet, 2.5 feet below the primary damage elevation.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

Ong/Lipscomb Problem Area (OL)

The existing Ong/Lipscomb storm sewer system is the western extension of the downtown storm sewer system. It originates near W. 15th Avenue and continues north along Parker to W. 11th Avenue. At W. 11th Avenue, the storm sewer turns east for half a block and continues north along Lipscomb Street to Line Avenue. At Line Avenue, the storm sewer bends north to S. 8th Avenue, where a lateral connects from the west. The storm sewer continues east down S. 8th Avenue for one block and turns north down Ong Street to the Burlington-Northern (BNRR) railroad tracks, just north of S. 5th Avenue. Near the railroad tracks, the storm sewer joins another lateral draining from the west. The storm sewer continues along the railroad tracks and eventually ties into the downtown storm sewer system.

Shallow street flooding near the BNRR railroad tracks has been experienced in the past. Shallow street flooding has also been experienced along Ong and Lipscomb Streets.

3.21.2 Proposed Improvements

Playa No. 23 (Wild Horse Lake)

Under ultimate basin development conditions, the 100-year flood event results in a water surface elevation in Wild Horse Lake of 3,616.7 feet, 2.5 feet below the primary damage elevation. A summary of ultimate levels for Playa No. 23 is found in Table A-2 in Appendix A.

The proposed improvements to the Ong/Lipscomb ("OL") storm sewer system will result in the diversion of runoff from almost 800 acres to Wild Horse which, under frequent storm events, was previously diverted around the playa to East Amarillo Creek. With the proposed Ong/Lipscomb improvements in place, the 100-year flood level rises to 3,620.9 feet, about 1.7 feet above the primary damage elevation. In order to reduce this resulting 100-year flood level to the primary damage elevation level, a 30-inch gravity relief culvert is proposed at the northeast corner of Wild Horse Lake. The relief culvert would terminate in a new junction box constructed on the existing 96-inch downtown storm sewer outfall at a point near N.W. 6th Avenue between Washington and Hughes. The relief culvert would be fitted with a flap gate where it discharges into the junction box to prevent backflow into Wild Horse Lake when the 96-inch storm sewer is surcharged. Proposed improvements and estimated construction costs for Wild Horse Lake are summarized in Table 3.21-1 and their location shown in Figure 3.21-1.

TABLE 3.21-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WILD HORSE LAKE STUDY AREA (P23)
WILD HORSE LAKE (PLAYA NO. 23)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P23-A	P23-B	Install new 30" relief culvert	850	LF	$55	$46,750
P23-B		New junction box and flap valve		LS		$25,000
					TOTAL	$72,000

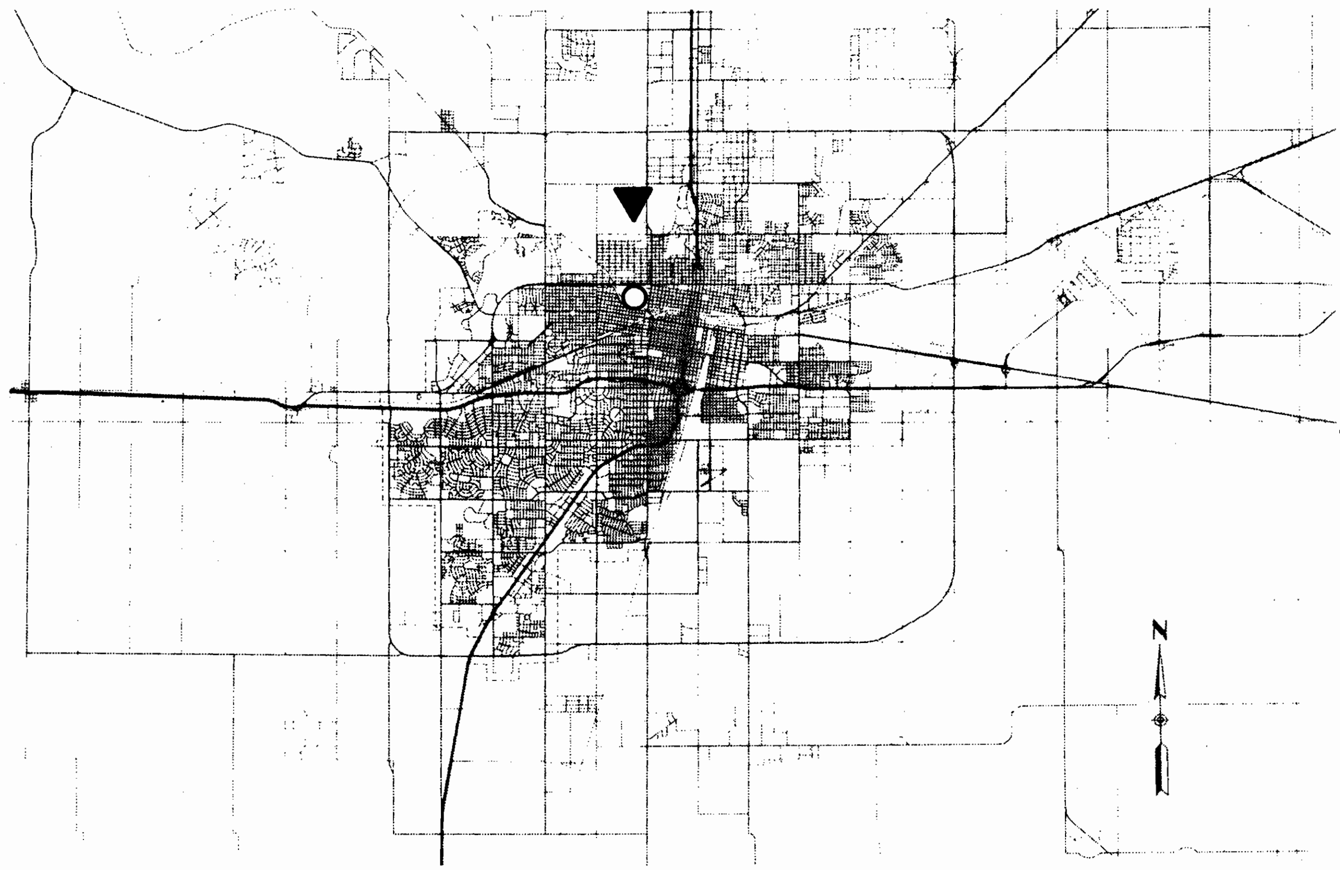
(See Drawing Set Sheet No. P23-1)

A summary of ultimate levels under the proposed improvement scenario is found in Table A-3 in Appendix A.

Ong/Lipscomb Problem Area (OL)

Proposed improvements for the Ong/Lipscomb system consist of diverting a portion of the drainage area away from the downtown storm sewer system and replacing inlets in the upper watershed of the storm sewer system. A proposed storm sewer replacement begins at the intersection of W. 15th Avenue and Parker Street and extends north to the BNRR. Along this system, inlet improvements are proposed along Parker Street at the intersections of W. 15th Avenue and Clover Drive, along W. 11th Avenue from Parker Street to Lipscomb Street, the intersection of Line Avenue and Lipscomb Street, and along Ong Street at the intersections of W. 8th Avenue, S. 6th Avenue, S. 5th Avenue, and the railroad.

The diversion occurs at the intersection of the railroad and Ong Street and at S. 3rd Avenue and Ong Street. These proposed new storm sewers will extend along Ong Street from the railroad to Wild Horse Lake. In order for pumped discharges which enter the existing 72-inch sewer west of Ong Street from the Royal Inn Pump Station at Lawrence



STUDY AREA: PLAYA NO. 23 "P23"

PROJECT NAME: WILD HORSE LAKE

DESCRIPTION: NEW GRAVITY RELIEF CULVERT

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P23)

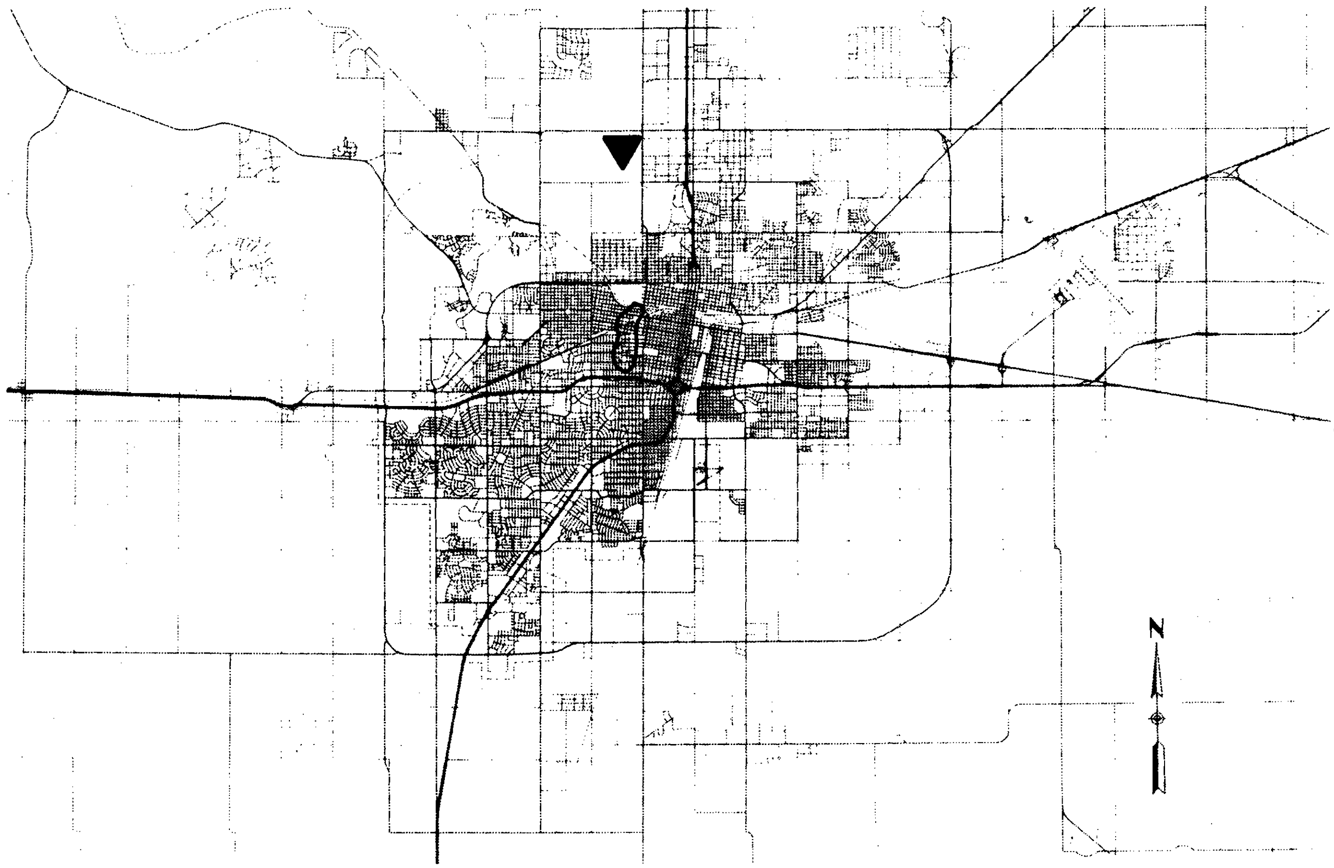
FIGURE 3.21-1

Lake to continue to reach the downtown storm sewer system and thereafter Thompson Park, a diversion structure is proposed at the intersection of Ong Street and S. 3rd Avenue. This diversion structure would have a short overflow wall in front of the entrance to the proposed 84-inch storm sewer to Wild Horse Lake and an underflow wall in front of the entrance to the existing 72-inch sewer leading to Thompson Park. This arrangement would allow dry-weather pumped discharges to continue to reach Thompson Park but cause most storm water discharges to be diverted to Wild Horse Lake as intended. The proposed storm sewer improvements are described in Table 3.21-2 and their location shown in Figure 3.21-2.

TABLE 3.21-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WILD HORSE LAKE STUDY AREA (P23)
ONG/LIPSCOMB STORM SEWER (OL)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
OL-A	OL-B	Replace w/36" RCP	1750	LF	$99	$173,250
OL-B	OL-BA	Replace w/48" RCP	1230	LF	$139	$170,970
OL-BA	OL-C	Replace w/60" RCP	280	LF	$204	$57,120
OL-C	OL-D	Replace w/60" RCP	330	LF	$204	$67,320
OL-D	OL-E	Replace w/66" RCP	1440	LF	$235	$338,400
OL-E	OL-F	New 78" RCP	470	LF	$270	$126,900
OL-F		New Diversion Structure		LS		$15,000
OL-F	OUTFALL	New 84" RCP	1890	LF	$315	$595,350
				SUBTOTAL		$1,544,310
INLET IMPROVEMENTS						
		10' Inlets w/ Lat.	20	Each	$2,700	$54,000
		Area Inlet	1	Each	$5,000	$5,000
				SUBTOTAL		$59,000
MANHOLES						
		Manholes	3	Each	$3,500	$10,500
				SUBTOTAL		$10,500
				TOTAL		$1,613,810

(See Drawing Set Sheet No. P23-1)



STUDY AREA: WILD HORSE LAKE

PROJECT NAME: ONG/LIPSCOMB "OL"

DESCRIPTION: DIVERT PORTION OF DOWNTOWN STORM SEWERS TO WILD HORSE LAKE/ ADD STORM SEWER

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P23-OL)

FIGURE 3.21-2

3.22 Playa No. 24 Study Area (P24)

3.22.1 Existing Problems

Playa No. 24 (Martin Lake)

Playa No. 24 is located east of Mirror Street just south of N.E. 15th Avenue. The playa drainage basin, which totals 1,658 acres, is composed of low-density residential (LDR), high-density residential (HDR), commercial, and industrial development with a significant undeveloped portion. A survey of the playa perimeter revealed a primary damage elevation of 3,624.6 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will be reached by the five-year flood event.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.

3.22.2 Proposed Improvements

Playa No. 24 (Martin Lake)

Under ultimate basin development conditions, the primary damage elevation of 3,624.6 will be reached at just under the five-year flood event. Under the 100-year event, the flood level in Martin Lake would reach 3,629.7 feet, 5.1 feet above the primary damage elevation. A summary of ultimate levels for Playa No. 24 is found in Table A-2 in Appendix A.

In order to provide 100-year protection at Martin Lake, it is proposed to replace the existing pump with a new 5,000 gpm pump, upsize the suction and discharge lines, and excavate approximately 450,000 cubic yards of earth from the playa. In order for the new pump to be effective in evacuating the playa, the shutoff elevation should be lowered to 3,600 feet. The proposed improvements and associated construction costs for Martin Lake are summarized in Table 3.22-1 and their location shown in Figure 3.22-1.

TABLE 3.22-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 24 STUDY AREA (P24)
MARTIN LAKE (PLAYA NO. 24)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P24-A		Excavate playa	450,000	CY	$5	$2,250,000
P24-B	P24-C	Replace existing pump w/ 5000 gpm		LS		$90,000
		Replace existing suction line w/ 20"	100	LF	$60	$6,000
P24-C	P24-D	Replace existing force main w/ 20"	4,800	LF	$60	$288,000
					TOTAL	$2,634,000

(See Drawing Set Sheet No. P24-1)

A summary of ultimate levels under the proposed improvements scenario is found in Table A-3 in Appendix A.

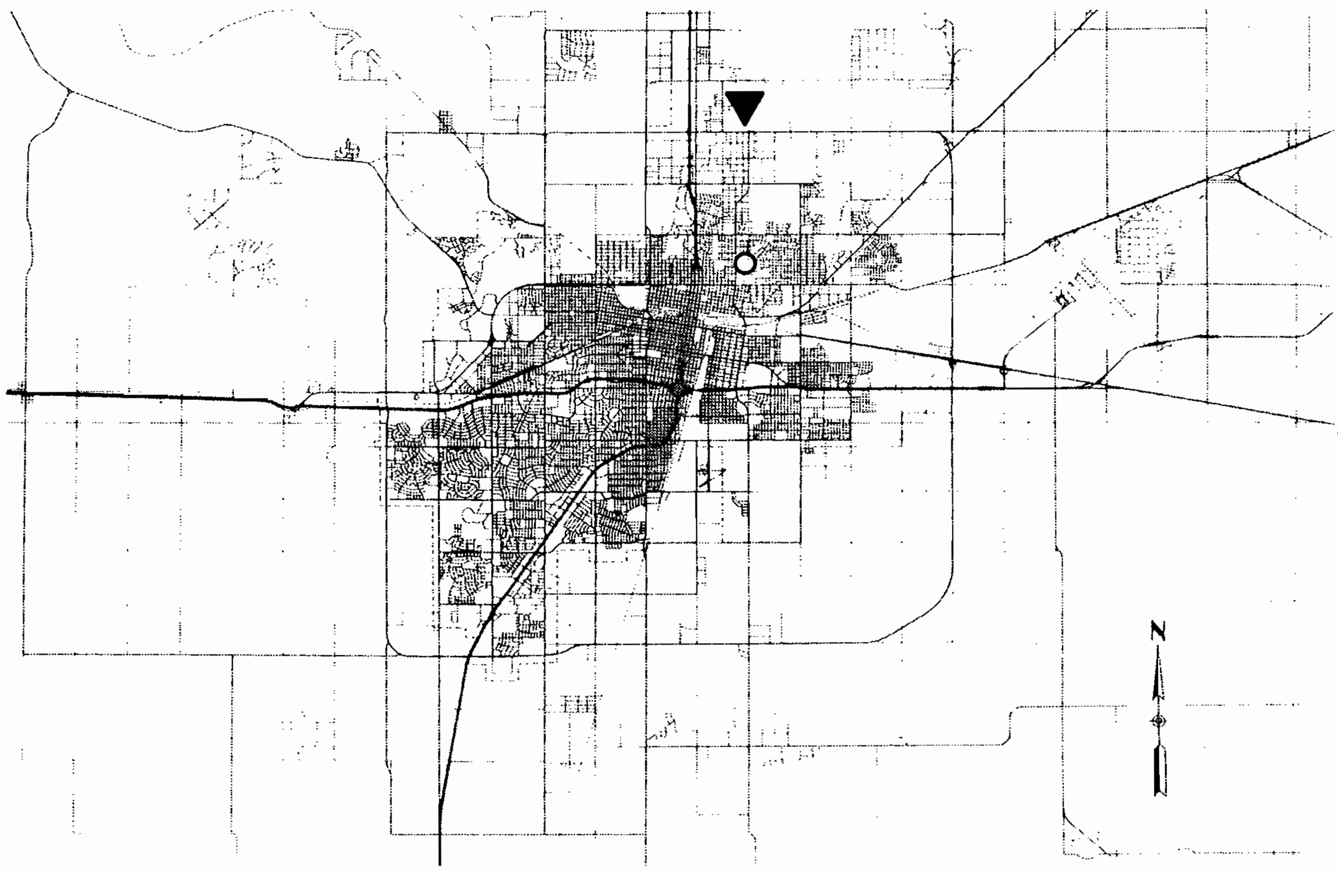
3.23 Playa No. 26 Study Area (P26)

3.23.1 Existing Problems

Playa No. 26 (Juett Lake)

Playa No. 26 is located southeast of the intersection of Interstate 40 and Lakeside Drive. The playa drainage basin, which totals 3,246 acres, is primarily undeveloped but with some commercial development. A survey of the playa perimeter revealed a primary damage elevation of 3,568.2 feet. Results of the ASAPP model indicate that, under existing watershed conditions, this damage elevation will be reached by the five-year flood event.

A summary of playa levels for existing conditions is found in Table A-1 in Appendix A.



STUDY AREA: PLAYA NO. 24 "P24"

PROJECT NAME: MARTIN LAKE

DESCRIPTION: UPGRADE PUMP STATION AND FORCE MAIN/EXCAVATE PLAYA

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P24)

FIGURE 3.22-1

3.23.2 Proposed Improvements

Playa No. 26 (Juett Lake)

Under ultimate basin development conditions, the primary damage elevation will be reached at the four-year flood event. Under the 100-year event, the flood level in Juett Lake would reach 3,571.4 feet, 3.2 feet above the primary damage elevation. A summary of ultimate levels for Playa No. 26 is found in Table A-2 in Appendix A.

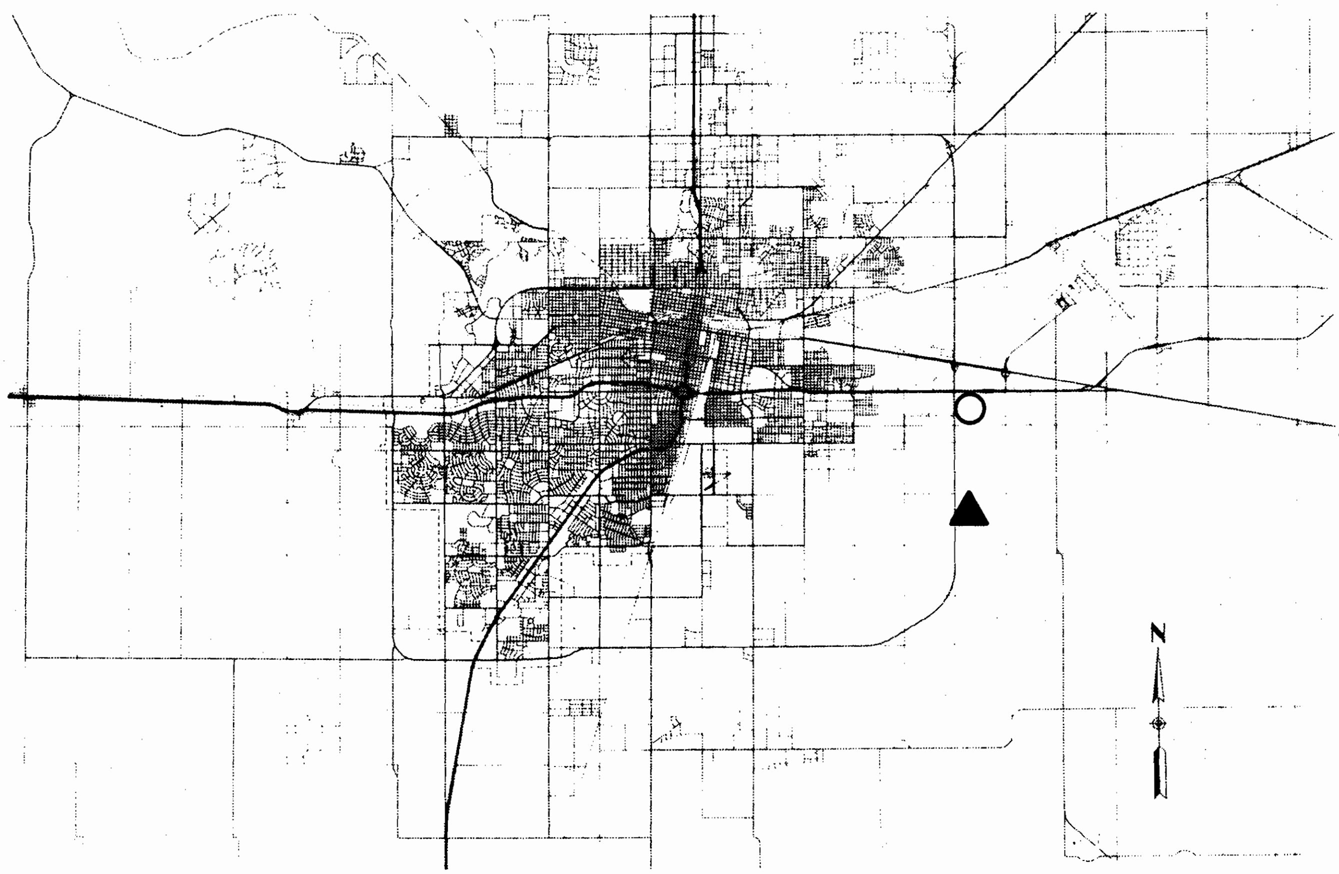
Due to the relatively low elevations of the primary and secondary damage points with respect to the calculated flood levels as well as the remote location and large size of Juett Lake, no pumping or excavation projects were judged to be economical for protection against the 100-year event. Therefore, it is proposed that the City negotiate to relocate each property to an elevation above 3,571.5 or undertake a floodproofing project to protect the properties. Proposed improvements for Playa No. 26 are summarized in Table 3.23-1 and their location shown in Figure 3.23-1.

TABLE 3.23-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 26 STUDY AREA (P26)
JUETT LAKE (PLAYA NO. 26)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P26-A		Relocate or floodproof salvage business		LS		$30,000
P26-B		Relocate or floodproof boat business		LS		$20,000
					TOTAL	$50,000

(See Drawing Set Sheet No. P26-1)

A summary of ultimate levels for Playa No. 26 under the proposed improvements scenario is found in Table A-3 in Appendix A.



STUDY AREA: PLAYA NO. 26 "P26"

PROJECT NAME: JUETT LAKE

DESCRIPTION: RELOCATE OR FLOODPROOF PROPERTIES/RAISE ROAD

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P26)

FIGURE
3.23-1

3.24 Playa No. 27 Study Area (P27)

3.24.1 Existing Problems

Playa No. 27 (TSTC Lake)

Playa No. 27 is located just west of FM 1912 on the northern edge of the Amarillo Golf Club. The playa drainage basin, which totals 2,773 acres, is primarily parkland and industrial development. Playa No. 27 occasionally collects overflows from Playa No. 60 located approximately one mile south. The Playa No. 60 basin area of 964 acres is not part of the Playa No. 27 area of 2,773 acres. A survey of the Playa No. 27 perimeter revealed a primary damage elevation of 3,547.1 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,546.4 feet. Therefore, no significant damages result for any events up through the 100-year event, including events in excess of the five-year event which result in overflows from Playa No. 60.

3.24.2 Proposed Improvements

Playa No. 27 (TSTC Lake)

Under ultimate basin development conditions, the 100-year level for Playa No. 27 is 3,546.6 feet, or 0.5 feet below the existing primary damage elevation of 3,547.1 feet. Therefore, no improvements for the basin are recommended in this Master Plan. Should development pressure occur, a hydrologic model of the Playa No. 60/Playa No. 27 system should be prepared which accounts for the hydraulic behavior of the Playa No. 60 overflow. A backwater profile model should then be prepared to estimate the water surface elevations of the overflows from Playa No. 60 to Playa No. 27 for a full range of flood events.

A summary of ultimate levels for Playa No. 27 is found in Table A-2 in Appendix A.

3.25 Playa No. 28 Study Area (P28)

3.25.1 Existing Problems

Playa No. 28 (Airport Lake)

Playa No. 28 is located east of Lakeside Drive alongside the main runway of Amarillo International Airport. The playa drainage basin, which totals 2,264 acres, is primarily airport with some related industrial development. A survey of the Playa No. 28 perimeter revealed the only significant damage location to be the airport runway at approximate elevation 3,604 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,587.9 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

3.25.2 Proposed Improvements

Playa No. 28 (Airport Lake)

Under ultimate basin development conditions, the 100-year level for Playa No. 28 is 3,588.1 feet, well below the runway elevation of 3,604.0 feet. Therefore, no playa improvements are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 28 is found in Table A-2 in Appendix A.

3.26 Playa No. 29 Study Area (P29)

3.26.1 Existing Problems

Playa No. 29

Playa No. 29 is located south of the intersection of Amarillo Boulevard East and Folsom Road. The playa drainage basin, which totals 371 acres, has some industrial development. Playa No. 29 occasionally overflows to Playa No. 33, located approximately two miles to the northeast. The Playa No. 29 overflow elevation is estimated at 3,591.2 feet. Under existing basin development conditions, the ASAPP model indicates that this elevation is reached under the 50-year storm event. Since the ASAPP model is not equipped to perform storage routing of a storm hydrograph, precise flood elevations for events in excess of the 50-year have not been determined. However, it appears that no significant damages would result for any event up through the 100-year event.

A summary of existing levels for Playa No. 29 is found in Table A-1 in Appendix A.

3.26.2 Proposed Improvements

Playa No. 29

Under ultimate basin development conditions, the ASAPP model indicates that the overflow elevation of 3,591.2 feet would be reached under the 50-year event. Because Playa No. 29 basin is essentially fully developed, no increased flood levels are anticipated under ultimate conditions compared to existing conditions. Therefore, no improvements for the basin are recommended in this Master Plan. Should development pressure occur, a hydrologic model of the basin should be prepared which accounts for the hydraulic behavior of the playa overflow. Accurate playa flood elevations can then be established by storage routing a full series of storm events through the playa.

A summary of ultimate levels for Playa No. 29 is found in Table A-2 in Appendix A.

3.27 Playa No. 30 Study Area (P30)

3.27.1 Existing Problems

Playa No. 30

Playa No. 30 is located east of Western Street and south of Sundown Lane. The playa drainage basin, which totals 310 acres, is essentially undeveloped. Playa No. 30 occasionally overflows to Playa No. 12 located approximately one mile southwest on the opposite side of Western Street. The Playa No. 30 overflow elevation is estimated at 3,631 feet. A survey of the playa perimeter revealed a primary damage elevation of 3,634.7 feet. Under existing basin development conditions, the ASAPP model indicates that the overflow elevation is reached under the 50-year storm event. Since the ASAPP model is not equipped to perform storage routing of a storm hydrograph, precise flood elevations for events in excess of the 50-year event have not been determined. However, it can be stated that, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

A summary of existing levels for Playa No. 30 is found in Table A-1 in Appendix A.

3.27.2 Proposed Improvements

<u>Playa No. 30</u>

Under ultimate basin development conditions, the ASAPP model indicates that the overflow elevation of 3,631 feet would be reached under the 50-year event. Due to the lack of existing or proposed development in the Playa No. 30 area, no improvements for the basin are recommended in this Master Plan. Should development pressure occur, a hydrologic model of the basin should be prepared which accounts for the hydraulic behavior of the playa overflow. Accurate playa flood elevations and channel reach profiles can then be established by storage routing a full series of storm events through the playa.

A summary of ultimate levels for Playa No. 30 is found in Table A-2 in Appendix A.

3.28 Playa No. 33 Study Area (P33)

3.28.1 Existing Problems

Playa No. 33

Playa No. 33 is located between Amarilo Boulevard East and St. Francis approximately one mile west of Parsley. The playa drainage basin, which totals 6,539 acres, is essentially undeveloped. Playa No. 33 occasionally collects overflows from Playa No. 29 located approximately two miles to the southwest. The Playa No. 29 basin area of 371 acres is not part of the Playa No. 33 area of 6,539 acres. A survey of the Playa No. 33 perimeter revealed a primary damage elevation of 3,563 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,563 feet. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year event, including events in excess of the 50-year event which result in overflows from Playa No. 29.

3.28.2 Proposed Improvements

Playa No. 33

Under ultimate basin development conditions, the 100-year level for Playa No. 33 is 3,563.7 feet, slightly above the existing primary damage elevation of 3,563.5 feet. The primary damage elevation at this site is a barn which is not likely to suffer significant damage under 0.2 feet of inundation. For this reason, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 33 is found in Table A-2 in Appendix A.

3.29 Playa No. 34 Study Area (P34)

3.29.1 Existing Problems

Playa No. 34

Playa No. 34 is located south of Interstate 40 and east of Pullman Road. The playa drainage basin, which totals 2,711 acres, is essentially undeveloped. A survey of the Playa No. 34 perimeter revealed a primary damage elevation of 3,550.5 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,550.9 feet, 0.4 feet above the primary damage elevation.

A summary of existing levels for Playa No. 34 is found in Table A-1 in Appendix A.

3.29.2 Proposed Improvements

Playa No. 34

Under ultimate basin development conditions, the 100-year level for Playa No. 34 is 3,551.4 feet, 0.9 feet above the existing primary damage elevation of 3,550.5 feet. A summary of ultimate levels for Playa No. 34 is found in Table A-2 in Appendix A.

In order to reduce the 100-year water surface in the playa to an elevation at or below the primary damage elevation, construction of a gravity outfall channel from Playa No. 34 to the unnamed playa to the southeast along a natural overflow route is proposed. A summary of the proposed improvements is given in Table 3.29-1 and their location shown in Figure 3.29-1.

TABLE 3.29-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 34 STUDY AREA (P34)
PLAYA NO. 34

INLET/NODE						
FROM	**TO**	**PROPOSED IMPROVEMENTS**	**NO. UNITS**	**UNIT**	**UNIT COST**	**TOTAL COST**
P34-A	P34-B	New outfall channel and box culverts		LS		$110,000
					TOTAL	$110,000

(See Drawing Set Sheet No. P34-1)

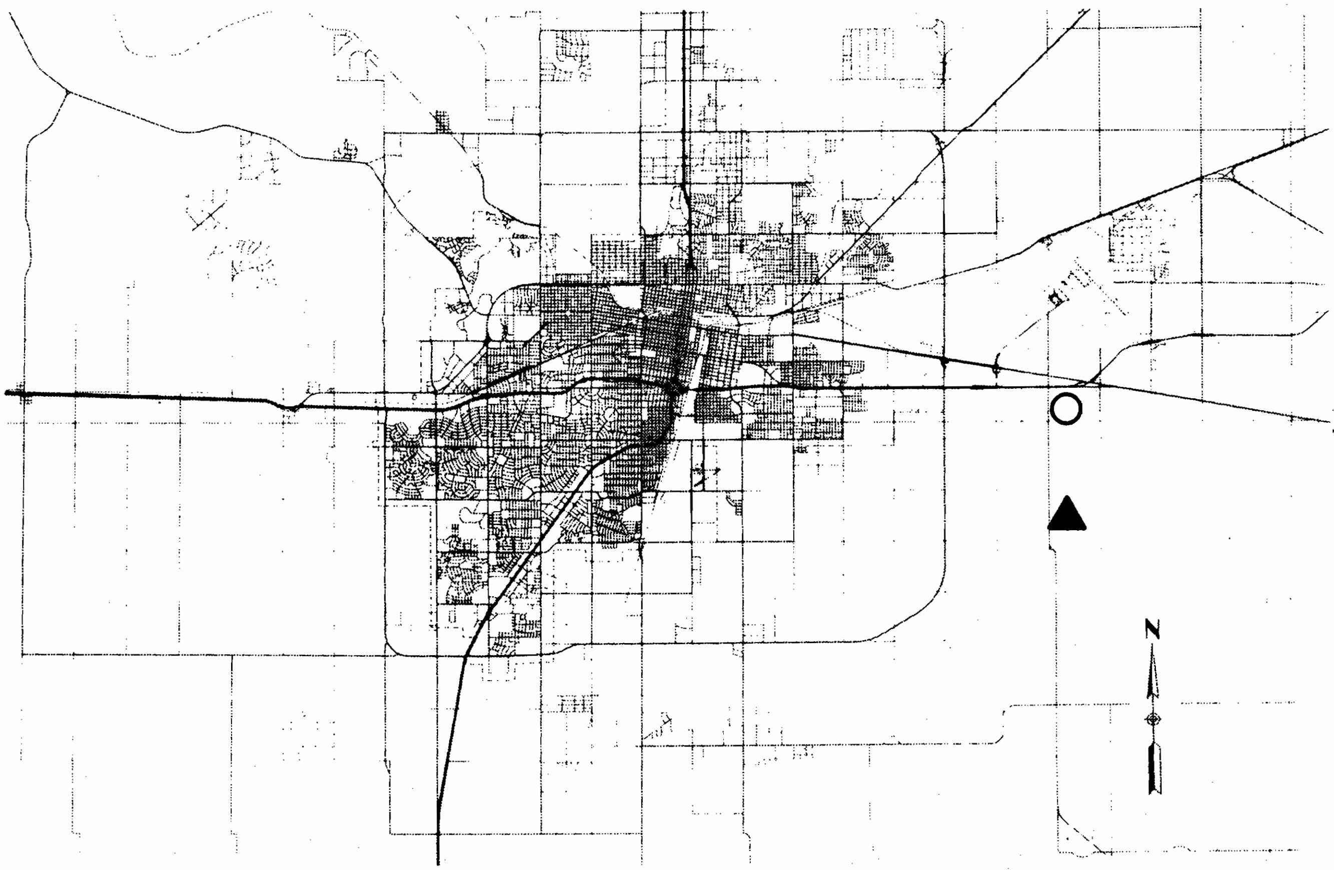
A summary of ultimate levels for Playa No. 34 under the proposed improvements scenario is found in Table A-3 in Appendix A.

3.30 Playa No. 35 Study Area (P35)

3.30.1 Existing Problems

Playa No. 35

Playa No. 35 is located just south of Interstate 40 and east of Jackrabbit Road. The playa drainage basin, which totals 1,658 acres, is primarily undeveloped. Playa No. 35 occasionally collects overflows from Playa No. 61 located approximately 1.5 miles west northwest. The Playa No. 61 basin area of 753 acres is not part of the Playa No. 35 area of 1,658 acres. A survey of the Playa No. 35 perimeter revealed a primary damage elevation of 3,549 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,549.4 feet. Therefore, under existing basin development conditions, the primary damage elevation is protected against the 67-year event. These results account for overflows from Playa No. 61, which occur for events in excess of the 10-year event.



STUDY AREA: PLAYA NO. 34 "P34"

PROJECT NAME: PLAYA NO. 34

DESCRIPTION: NEW GRAVITY RELIEF CHANNEL

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P34)

FIGURE
3.29-1

3.30.2 Proposed Improvements

Playa No. 35

Under ultimate basin development conditions, the 100-year level for Playa No. 35 is 3,550 feet, approximately one foot above the existing primary damage elevation of 3,549 feet, which would be protected against the 42-year flood event. A summary of ultimate levels for Playa No. 35 is found in Table A-2 in Appendix A.

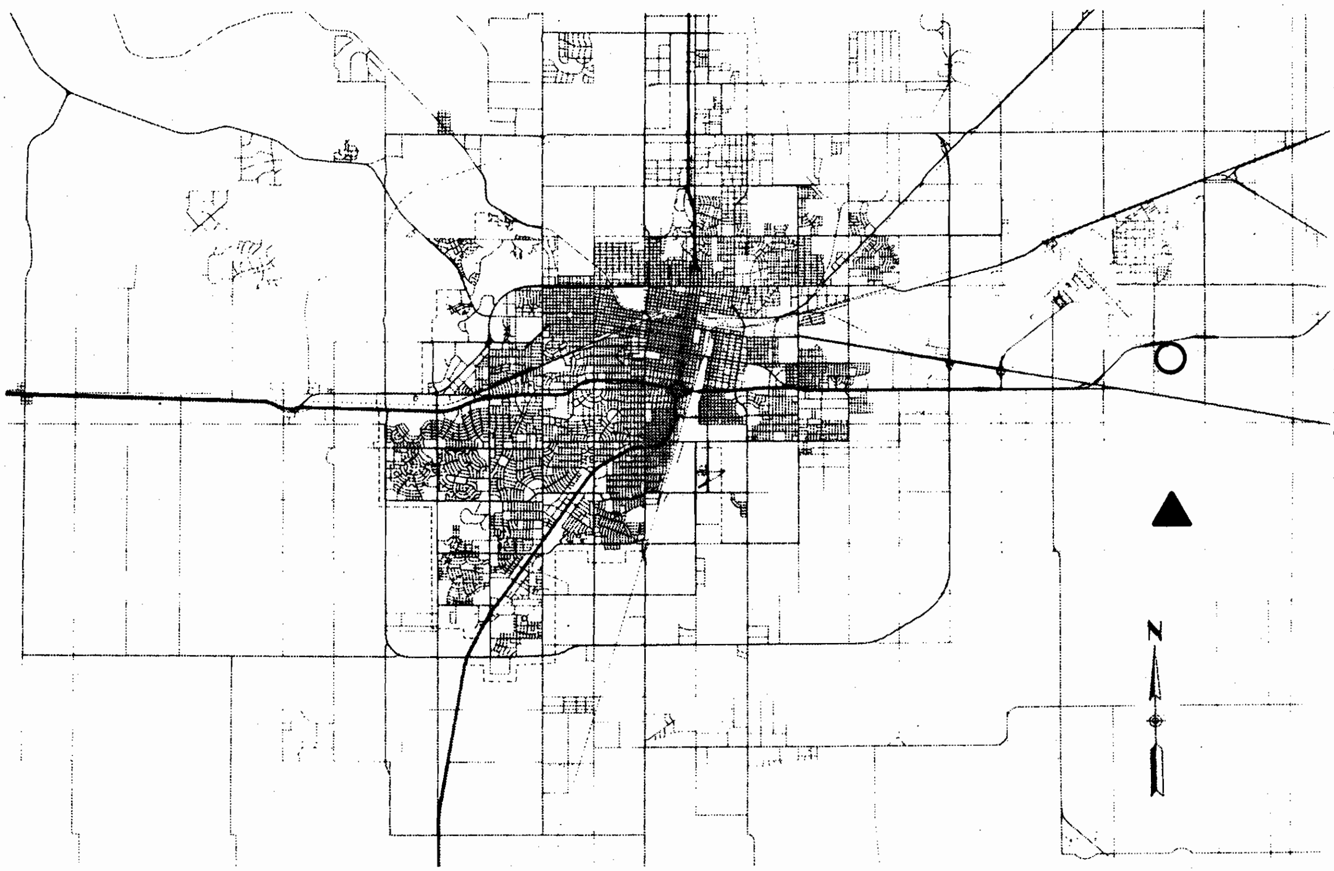
In order to provide protection against the 100-year event under ultimate basin development conditions, it is proposed to relocate or floodproof the existing business up to an elevation of 3,550 feet.

Should development pressure arise near Playa No. 35, a hydrologic model of the Playa No. 61/Playa No. 35 system should be prepared which accounts for the hydraulic behavior of the Playa No. 61 overflow. A backwater profile model should then be prepared to estimate the water surface elevations of the overflows from Playa No. 61 to Playa No. 35 for a full range of flood events. Planned development should not be allowed to encroach upon the 100-year playa flood plain nor the higher 100-year flood plain resulting from overflows from Playa No. 61. Proposed improvements for Playa No. 35 are summarized in Table 3.30-1 and their location shown in Figure 3.30-1.

TABLE 3.30-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
PLAYA NO. 35 STUDY AREA (P35)
PLAYA NO. 35

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
P35-A		Relocate/floodproof existing business		LS		$ 50,000
					TOTAL	$ 50,000

(See Drawing Set Sheet No. P35-1)



STUDY AREA: PLAYA NO. 35 "P35"

PROJECT NAME: PLAYA NO. 35

DESCRIPTION: RELOCATE OR FLOODPROOF PROPERTY/RAISE ROAD

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (P35)

FIGURE
3.30-1

A summary of ultimate levels under the proposed playa improvement scenario is found in Table A-3 in Appendix A.

3.31 Playa No. 36 Study Area (P36)

3.31.1 Existing Problems

Playa No. 36

Playa No. 36 is located on S.E. 46th Avenue just east of Whitaker Road. The playa drainage basin, which totals 766 acres, is essentially undeveloped. A survey of the Playa No. 36 perimeter revealed a primary damage elevation of 3,597.8 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,592.6 feet, 5.2 feet below the primary damage elevation. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

3.31.2 Proposed Improvements

Playa No. 36

Under ultimate basin development conditions, the 100-year level for Playa No. 36 is 3,593.1 feet, well below the existing primary damage elevation of 3,597.8 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 36 area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 36 is found in Table A-2 in Appendix A.

3.32 Playa No. 58 Study Area (P58)

3.32.1 Existing Problems

Playa No. 58

Playa No. 58 is located north of Amarillo Boulevard East near Amarillo International Airport. The playa drainage basin, which totals 733 acres, is essentially undeveloped but has some industrial development. A survey of the Playa No. 58 perimeter revealed the only potential damage location to be the railroad and Amarillo Boulevard East embankments with top elevations of approximately 3,576 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,569.7 feet, well below the potential damage elevation. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

3.32.2 Proposed Improvements

Playa No. 58

Under ultimate basin development conditions, the 100-year level for Playa No. 58 is 3,570.2 feet, well below the potential damage elevation of 3,576 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 58 area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 58 is found in Table A-2 in Appendix A.

3.33 Playa No. 59 Study Area (P59)

3.33.1 Existing Problems

Playa No. 59

Playa No. 59 is located south of St. Francis Avenue and west of Parsley Road. The playa drainage basin, which totals 970 acres, is essentially undeveloped. A survey of the Playa No. 59 perimeter revealed a primary damage elevation of 3,591 feet. Under existing basin development conditions, the 100-year playa level was found to be 3,561.2 feet, well below the primary damage elevation. Therefore, due to the lack of development in the area, no significant damages result for any events up through the 100-year event.

3.33.2 Proposed Improvements

Playa No. 59

Under ultimate basin development conditions, the 100-year level for Playa No. 59 is 3,561.8 feet, well below the existing primary damage elevation of 3,591 feet. Therefore, due to the lack of existing or proposed development in the Playa No. 59 area, no improvements for the basin are recommended in this Master Plan.

A summary of ultimate levels for Playa No. 59 is found in Table A-2 in Appendix A.

3.34 Playa No. 60 Study Area (P60)

3.34.1 Existing Problems

Playa No. 60

Playa No. 60 is located south of the TSTC campus and west of FM 1912. The playa drainage basin, which totals 964 acres, is half developed under parkland and industrial uses. Playa No. 60 occasionally overflows to Playa No. 27 located approximately one mile to the north. The Playa No. 60 overflow elevation is estimated at 3,556.6 feet. Under existing basin development conditions, the ASAPP model indicates that this elevation is reached under the five-year storm event. Since the ASAPP model is not equipped to perform storage routing of a storm hydrograph, precise flood elevations for events in excess of the five-year event have not been determined. However, since the primary damage elevation for Playa No. 60 is 3,567.7 feet, it can be stated that no significant damages result for any events up through the 100-year event.

A summary of existing levels for Playa No. 60 is found in Table A-1 in Appendix A.

3.34.2 Proposed Improvements

Playa No. 60

Under ultimate basin development conditions, the ASAPP model indicates that the overflow elevation of 3,556.6 feet would be reached under the five-year event. Due to the lack of development in the Playa No. 60 overflow area, no improvements for the basin are recommended in this Master Plan. Should development pressure arise, a hydrologic model of the basin should be prepared which accounts for the hydraulic behavior of the playa overflow. Accurate playa flood elevations can then be established by storage routing a full series of storm events through the playa.

A summary of ultimate levels for Playa No. 60 is found in Table A-2 in Appendix A.

3.35 Playa No. 61 Study Area (P61)

3.35.1 Existing Problems

Playa No. 61

Playa No. 61 is located south of Amarillo International Airport and one mile east of Pullman Road. The playa drainage basin, which totals 753 acres, is primarily undeveloped with some industrial use. Playa No. 61 occasionally overflows to Playa No. 35 located approximately 1.5 miles east southeast. The Playa No. 61 overflow elevation is estimated at 3,594 feet. Under existing basin development conditions, the ASAPP model indicates that this elevation is reached under the 10-year storm event. Since the ASAPP model is not equipped to perform storage routing of a storm hydrograph, precise flood elevations for events in excess of the 10-year event have not been determined. However, since no damageable facilities were identified in the playa vicinity, it can be stated that no significant damages result for any events up through the 100-year event.

A summary of existing levels for Playa No. 61 is found in Table A-1 in Appendix A.

3.35.2 Proposed Improvements

Playa No. 61

Under ultimate basin development conditions, the ASAPP model indicates that the overflow elevation of 3,594 feet would be reached under the 10-year event. Due to the lack of damageable facilities in the Playa No. 61 overflow area, no improvements for the basin are recommended in this Master Plan. Should development pressure arise, a hydrologic model of the basin should be prepared which accounts for the hydraulic behavior of the playa overflow. Accurate playa flood elevations can then be established by storage routing a full series of storm events through the playa.

A summary of ultimate levels for Playa No. 61 is found in Table A-2 in Appendix A.

3.36 West Amarillo Creek Study Area (CW)

3.36.1 Existing Problems

Westgate Mall/Puckett West Problem Area (WM)

The Westgate Mall/Puckett West problem area is located in west Amarillo south of Interstate 40 (I40) near Westgate Mall. The area surrounds the Westgate Mall detention pond and lies between Soncy Road on the west, Coulter Street on the east, and S.W. 34th

Avenue to the south. Existing flooding problems have occurred as runoff from upstream areas enters the detention pond. Street capacities are exceeded for significant storms, and shallow street flooding results along Gerald, Farrell, and Holyoke Streets.

Analyzing the existing street and natural drainage patterns south of the Westgate Mall area revealed two large areas which split the flow entering the detention pond south of the mall. The west area drains about 227 acres before it enters the detention pond. The east area drains about 175 acres before it enters the detention pond. Development for the most part is single family residential, with some commercial and small business development along S.W. 34th Avenue and Coulter. Watershed slopes vary from 0.5 to two percent.

Flood routings were performed on the south and north ponds draining the areas around Westgate Mall to establish a two-year ponded developed condition with which to start the design of the proposed storm sewer systems using the "HYDRA" computer program.

San Jacinto Heights East Problem Area (SJ)

The East San Jacinto Creek is located west of the downtown area of Amarillo and flows through the San Jacinto Heights East area. The creek begins in San Jacinto Park at the outfall to a small storm sewer system draining the intersection of West 2nd Avenue and Louisiana Street. The creek is essentially a small four-foot concrete-lined channel through San Jacinto Park which empties to an 18-inch RCP under Northwest 2nd Avenue and exits on Mississippi Street. Mississippi Street acts as the main conveyance for East San Jacinto Creek to Amarillo Boulevard West. A bowl-shaped inlet at Amarillo Boulevard West collects flows to two seven-foot by nine-foot concrete box culverts under Amarillo Boulevard West. A natural channel drains to a culvert under N.W. 9th Avenue, and thence to a culvert under N.W. 11th Avenue. Presently, N.W. 11th Avenue is the northerly city limit of Amarillo.

The contributing drainage area is about 289 acres at Amarillo Boulevard West and about 355 acres at N.W. 11th Avenue. Development in the watershed is primarily residential with some commercial development along Amarillo Boulevard West. Two schools and a small park area comprise other land uses in the watershed. Watershed slopes vary from one to two percent.

Runoff problems occur at the downstream end of San Jacinto Park at Mississippi Street, causing shallow flooding along Mississippi Street to Amarillo Boulevard West. The culverts at N.W. 9th and N.W. 11th Avenues have silted up to some extent from erosion and runoff.

Westcliff Channel Reach (WC)

The Westcliff Channel Reach includes a tributary to the main channel of West Amarillo Creek. The study reach is approximately 3.7 miles in total length, beginning at the upstream end at W. 9th Avenue near Amarillo Boulevard. The total watershed area for this reach is 2.4 square miles.

Flooding problems are primarily concentrated near the upstream end of the study reach. There is limited development along the channel and only a small number of roadway crossings relative to its overall length. Most of these roadway crossings have been recently constructed and have drainage structures equipped to pass major flood events. The main flooding problem identified in this reach is located near W. 9th Avenue extending downstream to Kouba Drive (private entrance to Range Riders). The existing channel is poorly defined and was found to have inadequate flow capacity. In addition, the drainage structure which exists at Kouba Drive was found to have insufficient capacity to pass the design flood event.

Amarillo Country Club Channel Reach (AC)

The Amarillo Country Club channel reach extends from Amarillo Boulevard to Bushland Boulevard. Its total length is approximately one mile and includes several roadway crossings. The channel reach extends through the Amarillo Country Club Golf Course between Bushland Boulevard and Gem Lake Road. The watershed for this reach is presently developed with the exception of the golf course area.

Flooding problems in this reach are primarily due to inadequate drainage structures under roadways at several locations and a limited channel capacity at a few isolated locations. Design flood flows were found to cause significant overtopping at Gem Lake Road, W. 3rd Avenue, W. 2nd Avenue, N.W. 4th Avenue, and N.W. 5th Avenue. In addition, the channel capacity between Gem Lake Drive and W. 3rd Avenue was found to be insufficient and resulted in flooding of West Hills Boulevard.

Tascosa Country Club Channel Reach (TC)

The Tascosa Country Club channel reach is located on a tributary to the main channel of West Amarillo Creek which extends from Amarillo Blvd to the downstream end of the study reach. The channel reach is approximately 2.5 miles in total length. Although portions of the watershed area for this reach are developing, most of the area remains largely undeveloped. The Tascosa Country Club development lies at the downstream end of the study reach. Minor flooding problems were found in this area which is primarily limited to one residential roadway crossing at Trevino Avenue. The roadway and drainage structures at this location were found to be inadequate to pass major storm flows, which resulted in significant overtopping of the roadway.

Wolflin Avenue Channel Reach (WA)

The Wolflin Avenue Channel Reach extends from Amarillo Boulevard upstream to the Burlington Northern Railroad (BNRR). The channel reach is approximately 0.55 miles in total length, located in a heavily urbanized watershed.

Flooding problems exist at numerous locations in this reach. They are primarily due to insufficient capacity of various drainage structures and storm sewer facilities. Problem areas include the crossings at Amarillo Boulevard, Wolflin Avenue, and the BNRR. In addition, poorly-defined channels produce flooding problems between Amarillo Blvd and Wolflin Avenue and between Wolflin Avenue and the BNRR. In addition, the intersection of Coulter Drive and BNRR experiences some flooding due to overflow from a channel located west of Coulter Drive. The channel delivers storm flows to the Westgate Mall North Detention Pond located north of the BNRR.

Medical Park Channel Reach (MP)

The Medical Park channel reach extends from W. 9th Avenue to west of Halstead Drive. The total length of this reach is approximately 1.6 miles. Medical Park Lake No. 1 and Medical Park Lake No. 2 are located in this reach, just upstream of W. 9th Avenue. Several roadway crossings in this reach present the major flooding problems.

Flooding problems in this reach exist at W. 9th Avenue, Wallace Boulevard, Wallace Lane, Coulter Drive, Fleming Drive, and Halstead Drive. Problems at W. 9th Avenue, Wallace Boulevard, and South Wallace Boulevard are due to inadequate capacity of the existing culverts. Problems at Coulter Drive, Fleming Drive, and Halstead Drive are primarily due to inadequate storm sewer facilities and a lack of adequate drainage structures to pass flood flows beneath these roadways.

3.36.2 Proposed Improvements

Westgate Mall/Puckett West Problem Area (WM)

Proposed improvements to the Westgate Mall/Puckett West problem area consist of the installation of new storm sewer systems. This would alleviate some of the shallow street flooding on Gerald, Farrell, and Holyoke Streets. One storm sewer would drain the west area starting at S.W. 34th Avenue and Irving Lane, continuing north along Clabern, turning and following Canode Drive to Gerald Drive, and then to an outfall at the mall detention pond. The other storm sewer would drain the east area starting south of S.W. 34th Street near Timber and Rigdon Streets. The storm sewer would continue north along Rigdon to S.W. 34th Street and on to Canode Street. The storm sewer would turn east down Canode to Holyoke, then at Holyoke it would turn turn north and eventually outfall to the detention pond behind Westgate Mall.

The proposed drainage improvements are summarized in Table 3.36-1 and their location shown in Figure 3.36-1.

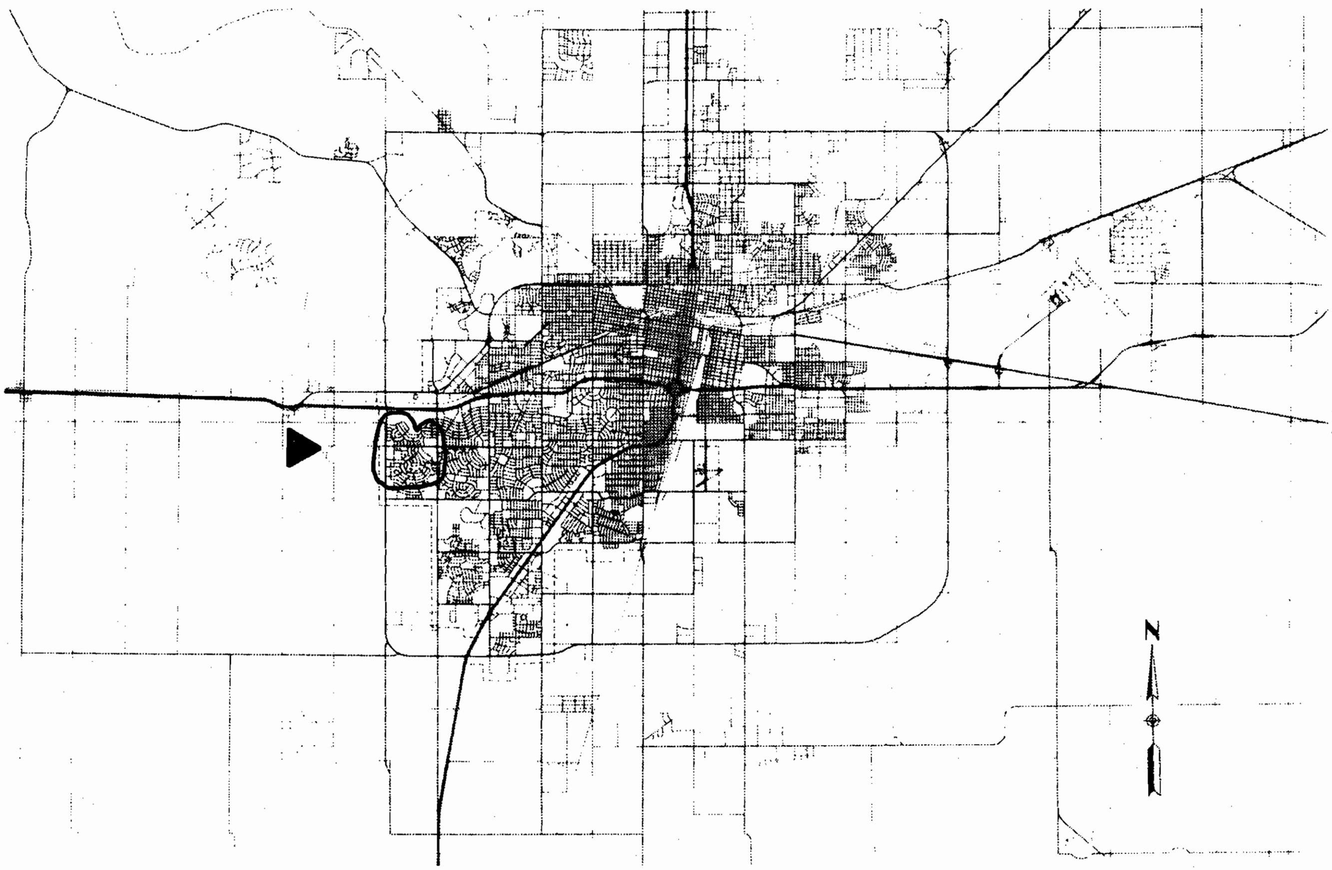
San Jacinto Heights East Problem Area (SJ)

The existing channel was analyzed using the "HEC-2 Water Surface Profiles" program by the U.S. Army Corps of Engineers. Runoff to the existing channel was computed using runoff curves developed for the City of Amarillo. Proposed improvements will consist of upgrading culvert capacities at N.W. 9th, N.W. 11th, and Amarillo Boulevard. Also, a storm sewer system is proposed for Mississippi Street from just north of San Jacinto Park to the frontage road of Amarillo Boulevard. The proposed improvements are summarized in Table 3.36-2 and their location shown in Figure 3.36-2.

TABLE 3.36-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WEST AMARILLO CREEK STUDY AREA (CW)
WESTGATE MALL STORM SEWER (WM)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
WM-A	WM-B	New 30" RCP	320.00	LF	$58.00	$18,560
WM-B	WM-C	New 36" RCP	560.00	LF	$79.00	$44,240
WM-C	WM-D	New 42" RCP	540.00	LF	$94.00	$50,760
WM-D	WM-E	New 42" RCP	300.00	LF	$94.00	$28,200
WM-E	WM-F	New 42" RCP	600.00	LF	$94.00	$56,400
WM-F	WM-G	New 42" RCP	480.00	LF	$94.00	$45,120
WM-G	WM-H	New 42" RCP	480.00	LF	$94.00	$45,120
WM-H	OUTFALL	New 48" RCP	120.00	LF	$111.00	$13,320
WM-I	WM-J	New 36" RCP	640.00	LF	$79.00	$50,560
WM-J	WM-K	New 42" RCP	360.00	LF	$94.00	$33,840
WM-K	WM-L	New 42" RCP	320.00	LF	$94.00	$30,080
WM-L	WM-M	New 42" RCP	320.00	LF	$94.00	$30,080
WM-M	WM-N	New 48" RCP	320.00	LF	$111.00	$35,520
WM-N	WM-O	New 54" RCP	320.00	LF	$128.00	$40,960
WM-O	OUTFALL	New 54" RCP	120.00	LF	$128.00	$15,360
					SUBTOTAL	$538,120
INLET IMPROVEMENTS						
		4' Inlets	4	EA	$1,900	$7,600
		5' Inlets	8	EA	$2,200	$17,600
		10' Inlets	7	EA	$2,700	$18,900
		15' Inlets	3	EA	$3,100	$9,300
		25' Inlets	1	EA	$4,100	$4,100
					SUBTOTAL	$57,500
STREET RECONSTRUCTION						
WM-A	WM-O	Regrading - 34th Street		LS		$50,000
					SUBTOTAL	$50,000
					TOTAL	$645,620

(See Drawing Set Sheet No. CW-8



STUDY AREA: WEST AMARILLO CREEK "CW"

PROJECT NAME: WESTGATE MALL "WM"

DESCRIPTION: ADD NEW STORM SEWER SYSTEMS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CW-WM)

FIGURE
3.36-1

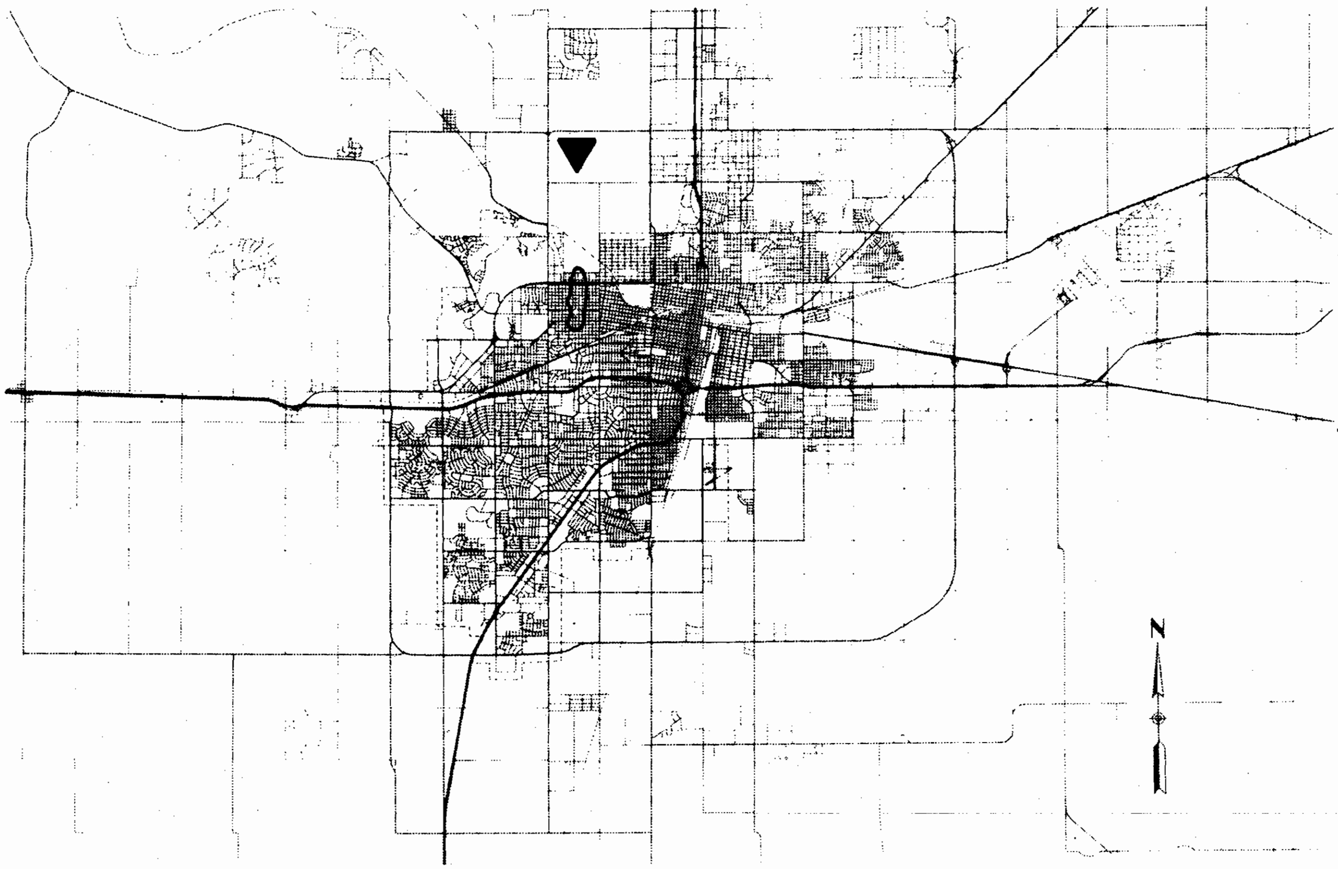
**TABLE 3.36-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WEST AMARILLO CREEK STUDY AREA (CW)
SAN JACINTO HEIGHTS EAST CHANNEL REACH (SJ)**

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
SJ 9+83	SJ 10+20	Replace w/two-barrel, 8'x 4' RCB at N.W 11th Avenue	37	LF	$810	$29,970
SJ 17+14	SJ 17+87	Replace w/three-barrel, 7'x 3' RCB at N.W. 9th Avenue	73	LF	$550	$40,150
SJ 20+30	SJ 21+35	Install one-barrel, 7'x 3' RCB at Amarillo Boulevard	105	LF	$570	$59,850
SJ 21+35	SJ 30+70	New 54" RCP w/8-5' inlets and 2-10' inlets	350	LF	$146	$51,100
SJ 30+70	SJ 34+20	New 48" RCP w/1-5' inlet, 1-10' inlet, and 2-20' inlets	910	LF	$160	$145,600
					TOTAL	$326,670

(See Drawing Set Sheet No. CW-2)

Westcliff Channel Reach (WC)

Proposed improvements in the Westcliff channel reach consist of increasing the capacity of the drainage structure at Kouba Drive by installing additional culverts. These additional culverts will prevent overtopping of the roadway for the 25-year flood event and limit the depth of overtopping to less than 1.5 feet for the 100-year event. In addition, channel improvements between Kouba Drive and W. 9th Avenue are included in order to provide additional channel capacity to convey storm runoff. A summary of the proposed improvements for the Westcliff Channel Reach is presented in Table 3.36-3 and their location shown in Figure 3.36-3.



STUDY AREA: WEST AMARILLO CREEK "CW"

PROJECT NAME: SAN JACINTO HEIGHTS EAST "SJ"

DESCRIPTION: CULVERT AND ROAD IMPROVEMENTS/ CHANNELIZE LOWER REACHES

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CW-SJ)

FIGURE **3.36-2**

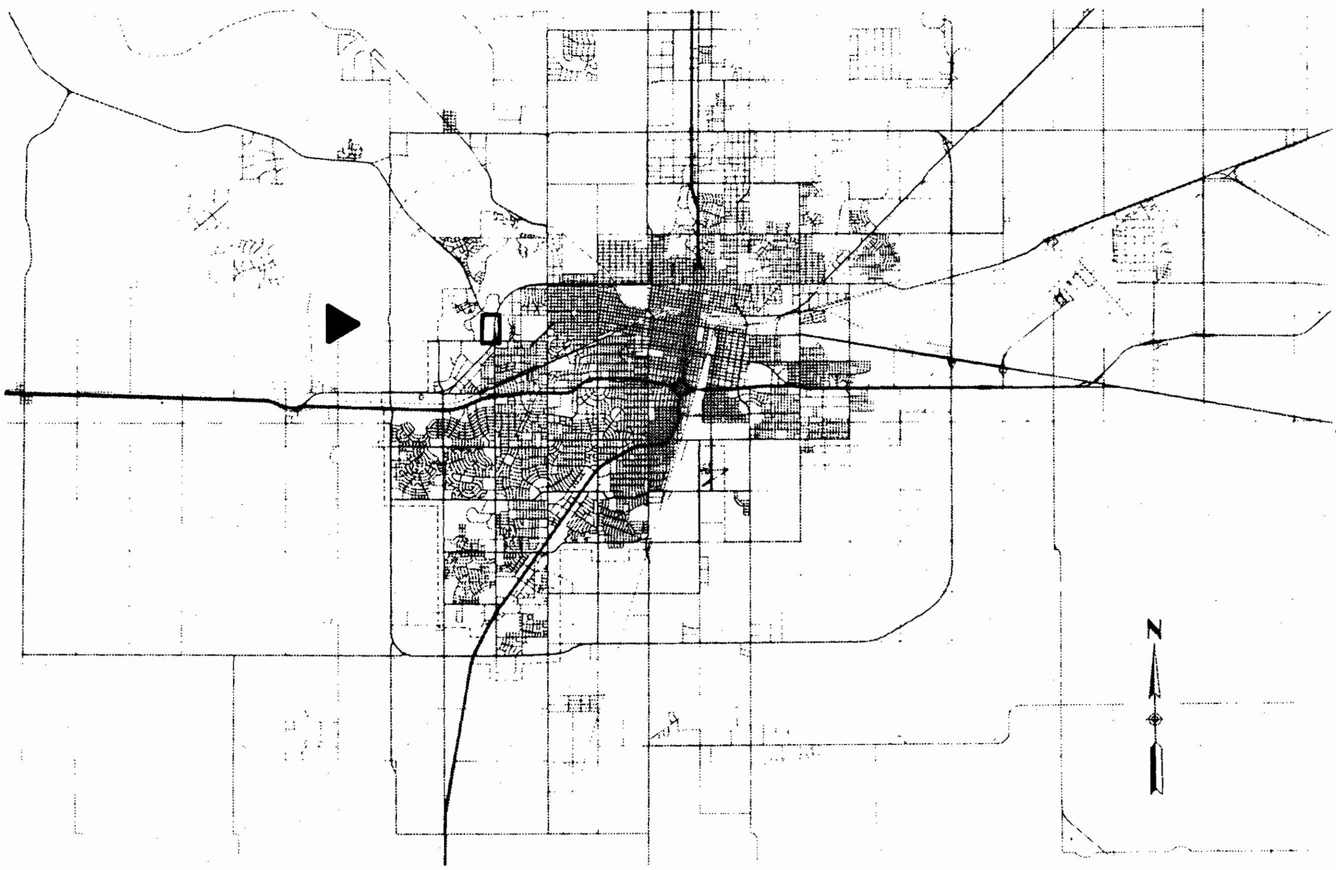
TABLE 3.36-3
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WEST AMARILLO CREEK STUDY AREA (CW)
WESTCLIFF CHANNEL REACH (WC)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
T1 183+36	T1 183+81	Install additional 2-36" CMP at Kouba Drive	45	LF	$180.00	$8,100
T1 183+81	T1 193+86	Excavate channel from W. 9th Ave to Kouba Drive	800	CY	$8.75	$7,000
					TOTAL	$15,000

(See Drawing Set Sheet No. CW-5)

Amarillo Country Club Channel Reach (AC)

Proposed improvements in the Amarillo Country Club channel reach consist of increasing the hydraulic capacity of the drainage structures at the south frontage road of Amarillo Boulevard West, N.W. 4th Avenue, W. 2nd Avenue, W. 3rd Avenue, and Gem Lake Road. Proposed improvements include installing additional culvert barrels or completely replacing the drainage structure. In addition, channel improvements between Gem Lake Road and W. 3rd Avenue are proposed in order to convey the storm runoff for major storm events while preventing flooding of West Hills Boulevard. The proposed improvements for the Amarillo Country Club Channel Reach are summarized in Table 3.36-4 and their location shown in Figure 3.36-4.



STUDY AREA: WEST AMARILLO CREEK "CW"

PROJECT NAME: WESTCLIFF TRIBUTARY "WC"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS/
CHANNEL IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CW-WC)

FIGURE
3.36-3

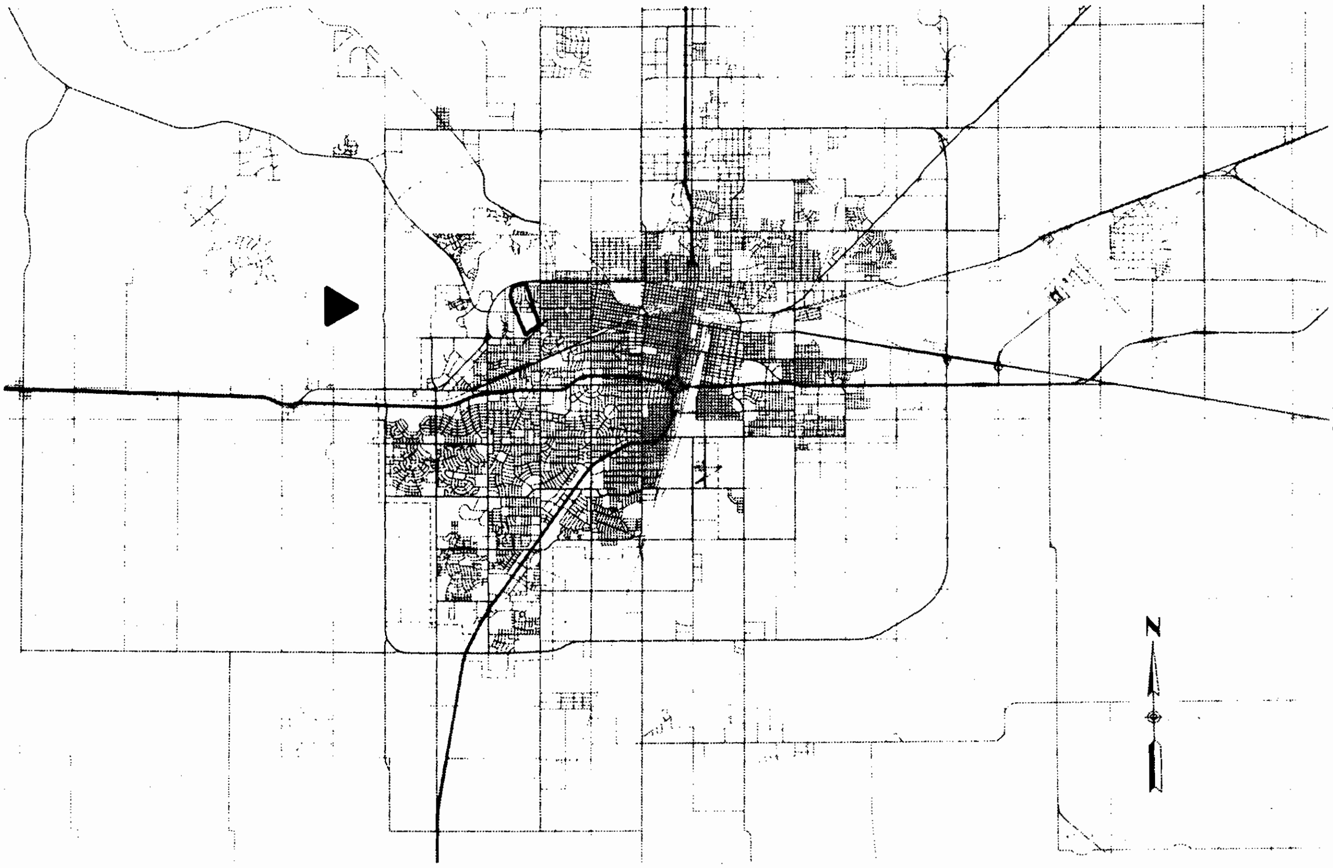
TABLE 3.36-4
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WEST AMARILLO CREEK STUDY AREA (CW)
AMARILLO COUNTRY CLUB CHANNEL REACH (AC)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
T2 134+06	T2 134+29	Install additional 2-5'x3' concrete boxes at south frontage road of Amarillo Boulevard West	23	LF	$390.00	$9,000
T2 138+09	T2 138+37	Replace existing RCP at N.W. 4th Ave with 2-8'x6' concrete boxes	28	LF	$570.00	$16,000
T2 157+87	T2 158+48	Replace existing RCP's at W. 2nd Ave with 4-7'x3' concrete boxes	61	LF	$720.00	$43,900
T2 161+19	T2 161+69	Install 2-8'x5' concrete boxes at W. 3rd Ave and raise road	50	LF	$680.00	$34,000
T2 166+83	T2 167+13	Replace existing concrete box at Gem Lake Road with 2-6'x5' concrete boxes	30	LF	$500.00	$15,000
T2 161+69	T2 166+83	Improve channel from Gem Lake Road to W. 3rd Ave; remove and replace 2 private driveway structures, remove existing concrete channel lining	500	CY	$8.00	$4,000
				LS		$17,000
					TOTAL	$139,000

(See Drawing Set Sheet No. CW-6)

Tascosa Country Club Channel Reach (TC)

Proposed improvements in the Tascosa Country Club channel reach consist of replacing the existing concrete pipe culvert at Trevino Avenue with a reinforced concrete box culvert structure. These improvements will allow Trevino Avenue to pass the 25-year flood event without overtopping the roadway and pass the 100-year flood event with less than 1.5 feet of overtopping. The proposed improvements for the Tascosa Country Club Channel Reach are summarized in Table 3.36-5 and their location shown in Figure 3.36-5.



STUDY AREA: WEST AMARILLO CREEK "CW"

PROJECT NAME: AMARILLO COUNTRY CLUB CHANNEL "AC"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS/ CHANNEL IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CW-AC)

FIGURE
3.36-4

TABLE 3.36-5
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WEST AMARILLO CREEK STUDY AREA (CW)
TASCOSA COUNTRY CLUB CHANNEL REACH (TC)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
T2-86+18	T2-86+71	Replace existing RCP at Trevino Ave with 2-4'x3' concrete boxes	53	LF	$225.00	$11,900
					TOTAL	$12,000

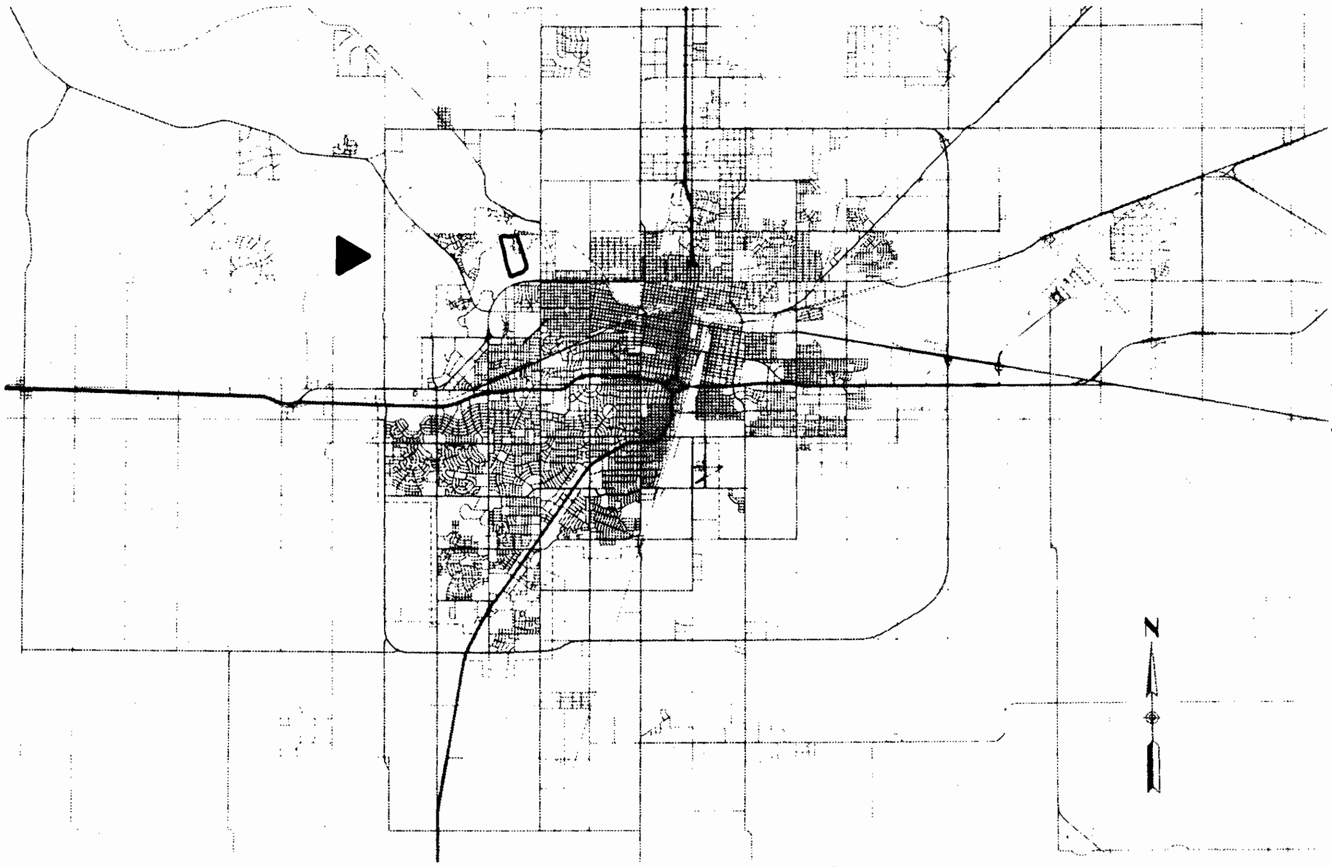
(See Drawing Set Sheet No. CW-2)

Wolflin Avenue Channel Reach (WA)

Proposed improvements in the Wolflin Avenue channel reach consist of modifications to existing drainage structures, channel improvements, and the installation of storm sewer systems. Proposed improvements include increasing the capacity of the Amarillo Boulevard culverts by installing additional barrels and installing a new culvert at the Amarillo Boulevard service road. Channel improvements are proposed between the Amarillo Boulevard service road and Wolflin Avenue in order to convey storm runoff away from the Wolflin Avenue area. An inlet structure and installation of a 54-inch reinforced concrete pipe from the inlet structure to Wolflin Avenue are included. In addition, replacement of the existing culvert under the BNRR at the entrance to the Westgate Mall Detention Pond is recommended in order to prevent flow upstream of the BNRR from overtopping Coulter Drive. The proposed improvments for the Wolflin Avenue Channel Reach are presented in Table 3.36-6 and their location shown in Figure 3.36-6.

Medical Park Channel Reach (MP)

Proposed improvements in the Medical Park channel reach consist of replacing the



STUDY AREA: WEST AMARILLO CREEK "CW"

PROJECT NAME: TASCOSA COUNTRY CLUB CHANNEL "TC"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CW-TC)

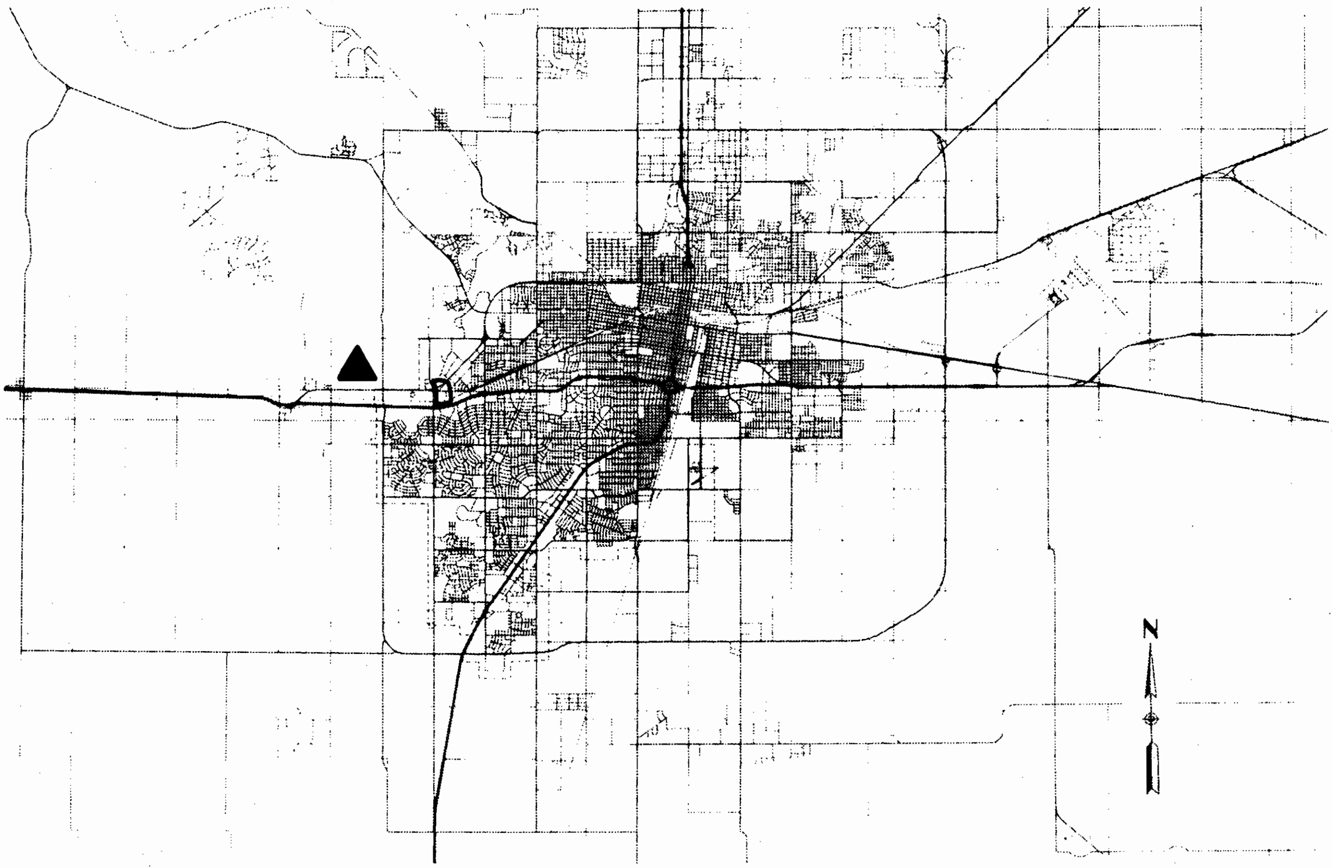
FIGURE **3.36-5**

existing drainage structures at W. 9th Avenue and S. Wallace Boulevard with reinforced concrete box culverts. Concrete box culverts are also proposed at Wallace Boulevard in order to supplement the existing three-barrel, 42-inch RCP culvert. A 54-inch RCP is proposed to be installed from the Halstead Drive/Fleming Avenue intersection to Coulter Street. This would allow storm runoff which now enters Fleming Avenue at Halstead to be conveyed beneath the roadway to the downstream side of Coulter Street. A summary of the proposed improvements for the Medical Park Channel Reach are listed in Table 3.36-7 and their location shown in Figure 3.36-7.

TABLE 3.36-6
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WEST AMARILLO CREEK STUDY AREA (CW)
WOLFLIN AVENUE CHANNEL REACH (WA)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
T3 2+10	T3 5+60	Install 3-42-inch RCP's at Amarillo Blvd. W.	350	LF	$369	$129,000
T3A 0+30	T3A 0+65	Install 3-8'x4' concrete boxes at Amarillo Blvd. W. service road	35	LF	$1,140	$40,000
T3A 0+65	T3A 5+80	Improve channel between Wolflin Avenue and Amarillo Blvd. W service road	3,650	LF	$820	$30,000
T3A 5+80	T3A 16+09	Install 54-inch RCP from BNRR to Wolflin Avenue including inlet structure at BNRR outlet structure	550	LF	$193	$106,000
BNRR at entrance to Westgate Mall North Detention Pond		Replace existing CM arch pipe with 4-10' x 8' concrete boxes	550	LF	$1,150	$115,000
					TOTAL	$420,000

(See Drawing Set Sheet No. CW-9)



STUDY AREA: WEST AMARILLO CREEK "CW"

PROJECT NAME: WOLFLIN AVENUE CHANNEL "WA"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS/
CHANNEL IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CW-WA)

FIGURE
3.36-6

TABLE 3.36-7
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
WEST AMARILLO CREEK STUDY AREA (CW)
MEDICAL PARK CHANNEL REACH (MP)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
77+90	79+50	Replace existing CMP at W. 9th Ave with 2-10'x10' concrete boxes	160	LF	$1,160	$185,600
92+90	95+00	Install 4-10'x8' concrete boxes at Wallace Blvd to supplement existing 3-42" RCP;	210	LF	$1,090	$228,900
		Construct concrete channel bottom lining	200	CY	$290.00	$58,000
116+99	118+09	Replace existing RCP's at S. Wallace Blvd with 3-8'x7' concrete boxes, remove/replace d/s apron	110	LF	$910.00	$100,100
139+05	146+05	Install 54" RCP with 2 inlets at Fleming Avenue, improve channel u/s of Halstead and d/s of Coulter	700	LF	$150.00	$105,000
					TOTAL	$678,000

(See Drawing Set Sheet No. CW-5)

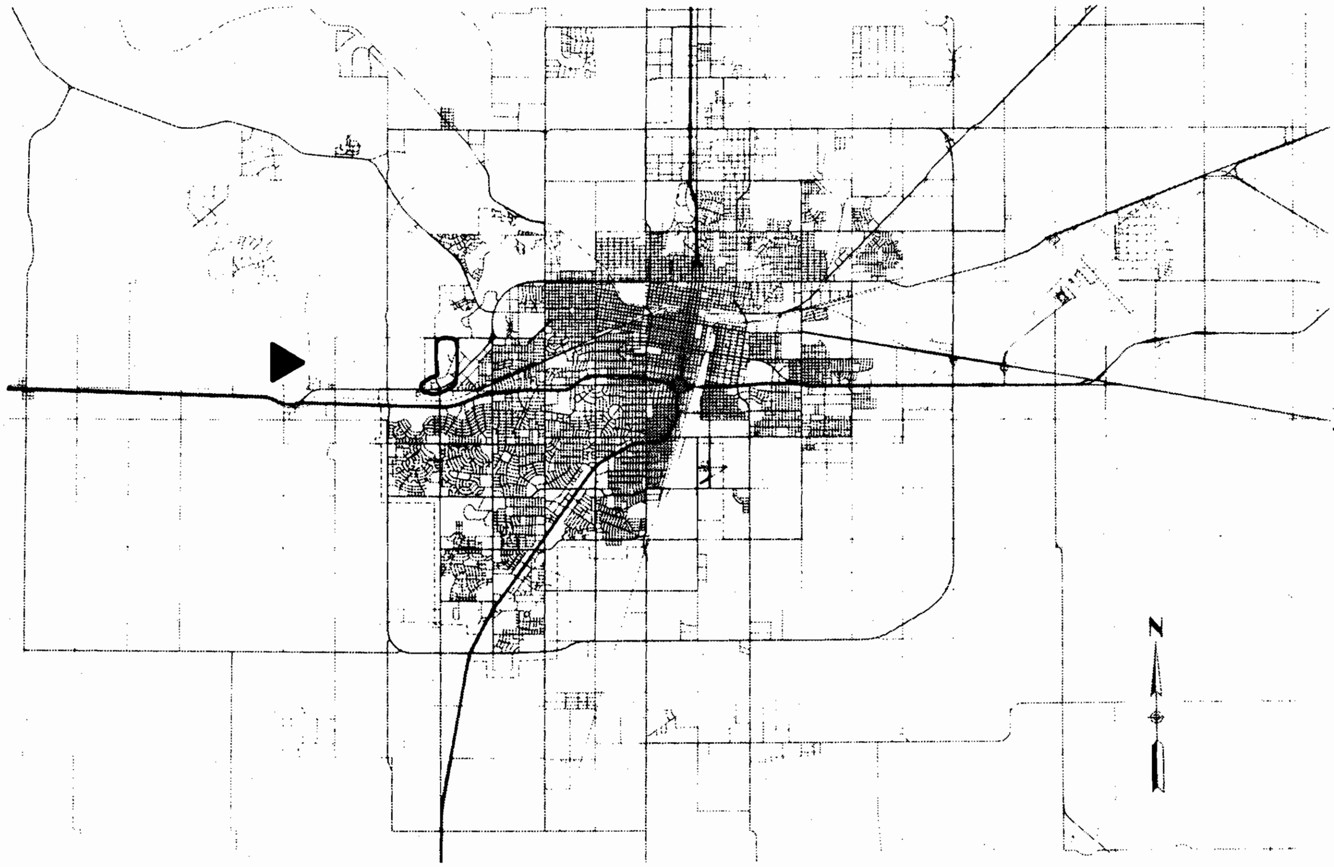
3.37 East Amarillo Creek Study Area (CE)

3.37.1 Existing Problems

S.E. 3rd Avenue/Railroad Storm Sewer Problem Area (R1)

Problems occur in the area of S.E. 3rd Avenue and ATSF Railroad when runoff accummulates in the S.E. 3rd Avenue underpass sag beneath the railroad span and prevents vehicle passage due to flood water depth. The underpass is currently drained by inlets connected to an 8' x 6' wet well. A small pump station with two 7.5 HP pumps rated at 550 gpm discharges through two four-inch discharge lines to a manhole which gravity drains to the City of Amarillo's downtown storm sewer system.

Likely causes of flooding at the S.E. 3rd Avenue/Railroad underpass are: 1) inadequate inlet capacity of the inlets located in the sag of the underpass; 2) inadequate



STUDY AREA: WEST AMARILLO CREEK "CW"

PROJECT NAME: MEDICAL PARK CHANNEL "MP"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CW-MP)

FIGURE 3.36-7

pump capacity to discharge at high enough flow rates to keep the underpass from flooding; and 3) additional runoff to underpass which might otherwise go to the downtown storm sewer system.

North Bolton Street Problem Area (BO)

An unnamed tributary identified as North Bolton Creek drains a residential and mobile home development north of N.E. 24th Avenue and Grand Street in the vicinity of Bolton Street and N.E. 32nd Avenue. The tributary drains to a 6' x 6' reinforced box culvert under the Panhandle and Santa Fe (PSF) Railway and onto East Amarillo Creek. The contributing drainage area intercepts about 183 acres to the culvert at the PSF railroad. Development is primarily residential and mobile homes, with some parts still undeveloped. Watershed slopes vary from one to two percent.

Flooding problems occur when runoff from areas south of NE 32nd Avenue drain to and down Mesa Verde Street to Bolton Street and exceed the capacity of the overflow channel. The channel drains to an 18-inch concrete pipe running under an alley behind mobile homes along Bolton Street before entering the box culvert under the railroad.

Lower East Amarillo Creek Channel Reaches (M1)

Approximately 2.5 miles of the main stem of the Lower East Amarillo Creek were studied. The area includes the northern edge of the East Amarillo Creek study area, approximately 1.4 miles downstream of Willow Creek Drive, to the River Road crossing. The total watershed area in this reach ranges from 5.4 square miles at the upstream edge to 16.3 square miles at the downstream edge.

Flooding problems occur when flows exceed the capacity of the Pavillard Drive, Willow Creek Drive, Yucca Avenue, and Irwin Road culverts and result in overtopping of each of these roadways. In addition, the 100-year flood plain was found to inundate several residential structures in a reach extending from approximately 1,400 feet downstream of Yucca Avenue to 600 feet upstream of Irwin Road.

Hastings to River Road Channel Reaches (M2)

Approximately 1.3 miles of the main stem of East Amarillo Creek extending from River Road to Hastings Avenue were studied. The reach includes several road crossings over East Amarillo Creek and a number of residential structures located near the channel. The total watershed area in this reach ranges from 2.1 square miles at the upstream end to 5.4 square miles at River Road. Three Thompson Park dams control most of the watershed area upstream of Hastings Avenue; however, these structures provide only minimal flood protection benefits for the larger flood events.

Flooding problems occur when flows exceed the capacity of the drainage structures at River Road, Dumas Drive, Cliffside Drive, Central Avenue, and Colorado Avenue. For the larger flood events, the drainage structure at Dumas Drive, a 570-ft corrugated steel arch pipe, causes significant backwater effects in the upstream channel.

N.W. 24th Avenue Channel Reach (M3)

Approximately 1.8 miles of the main stem of East Amarillo Creek extending from Hastings Avenue to N.W. 13th Avenue were studied. The reach includes one mile of channel through the Thompson Park area. The total watershed area in this reach ranges from 0.5 square miles at the upstream end to 2.1 square miles at the downtream end.

Most of the flooding problems in this reach are concentrated in the area upstream of and including N.W. 24th Avenue. A 96-inch diameter storm sewer, which is the outfall for the downtown storm sewer system, currently exists in what was once a natural drainage channel and terminates at N.W. 24th Avenue. The channel upstream of N.W. 24th Avenue was filled to a point above the crown of the pipe. Four street crossings, including N.W. 24th Avenue, N.W. 15th Avenue, N.W. 14th Avenue, and N.W. 13th Avenue, do not include drainage structures which will pass runoff under the roadways. Storm sewer inlets do exist at each of the streets which will direct runoff into the 96-inch diameter storm sewer pipe located below the street. The storm sewer inlets have a limited capacity, and once this capacity is exceeded, each of the streets are subjected to road overflow.

Echo Street Tributary Channel Reach (T1)

Approximately 2.3 miles of a tributary to the main channel of East Amarillo Creek near Echo Street were studied. The confluence of this tributary with the main channel of East Amarillo Creek is located downstream of Willow Creek Drive and extends upstream to Hastings, eventually parallel to Echo Street. The tributary watershed encompasses approximately 2.6 square miles.

Flooding problems in this reach occur when the capacity of the culverts at Willow Creek Drive, near the tributary's confluence with the main channel of East Amarillo Creek, are exceeded, resulting in overtopping of the roadway. In addition, the tributary crosses Central Avenue near Echo Street where there is no drainage structure to pass runoff beneath the roadway. This results in frequent overtopping of Central Avenue.

St. Francis Avenue Tributary Channel Reach (T2)

Approximately 0.8 miles of a tributary to the main channel of East Amarillo Creek near St. Francis Avenue were studied. The confluence of this tributary with the main channel of East Amarillo Creek is located approximately 780 feet downstream of River Road. The total watershed area of this tributary at the confluence with the main channel of East Amarillo Creek is approximately 0.5 square miles.

Flooding problems in this reach are primarily located near its crossing of Valley Avenue. No drainage structure is located at Valley Avenue, which results in frequent overtopping of the roadway. In addition, the channel is poorly defined in this reach.

Valley Park Tributary Channel Reach (T3)

Approximately 1.4 miles of a tributary to the main channel of East Amarillo Creek, which joins the main channel just upstream of Central Avenue, were studied. The total watershed area of this tributary at the confluence with the main channel is approximately 1.2 square miles.

Flooding problems in this reach occur at the Dumas Drive crossing of Tributary 3. At this location, a 1,200-foot, eight-foot by five-foot reinforced concrete box conveys runoff beneath Dumas Drive into the main channel of East Amarillo Creek. When the capacity of this drainage structure is exceeded, flow begins to overtop the underpass of Central Avenue at Dumas Drive and a backwater effect results upstream at other roadway crossings. Flooding problems were also found at Park Avenue, Hastings Avenue, and Angelus Drive upstream of Dumas Drive. These problems are associated with the absence of a drainage structure or a drainage structure with insufficient capacity at the crossing.

Ross Rogers Tributary Channel Reaches (T4)

Approximately 1.3 miles of a tributary to the main channel of East Amarillo Creek, which joins the main channel near Central Avenue, were studied. The reach includes three residential street crossings and conveys runoff from a watershed area of approximately 1.2 square miles.

Flooding problems generally exist at each of the residential street crossings, including Colorado Avenue, Buck Drive, Studebaker Avenue, and Hastings Avenue. The problems are the result of the insufficient capacity of drainage structures at Colorado Avenue, Studebaker Avenue, and Hastings Avenue and the absence of a drainage structure at Buck Drive to pass runoff beneath the street.

3.37.2 Proposed Improvements

S.E. 3rd Avenue/Railroad Storm Sewer Problem Area (R1)

Proposed improvements will consist of upgrading the existing lift station, adding storage capacity, and diverting some storm water runoff away from the sag under the railroad underpass. Replacement of the existing pumps with one 6,000 gpm propeller pump and rebuilding the wetwell are proposed. Also, diversion of about 6.8 acres to curb inlets toward the north and diversion of about 3.8 acres to curb inlets toward the south will relieve the railroad sag of storm water runoff. Proposed improvements are summarized in Table 3.37-1 and their location shown in Figure 3.37-1.

North Bolton Street Problem Area (BO)

Proposed improvements at Bolton Street include adding a storm sewer system to intercept offsite flows coming down N. Bolton Creek to N.E. 32nd Avenue in an area inlet, and adding inlets along Mesa Verde Street at N.E. 32nd Avenue, Beaver Avenue, and Bolton Street. The outfall would occur just beyond the alley running parallel to Bolton

TABLE 3.37-1
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
3RD AVENUE/RR STORM SEWER (R1)

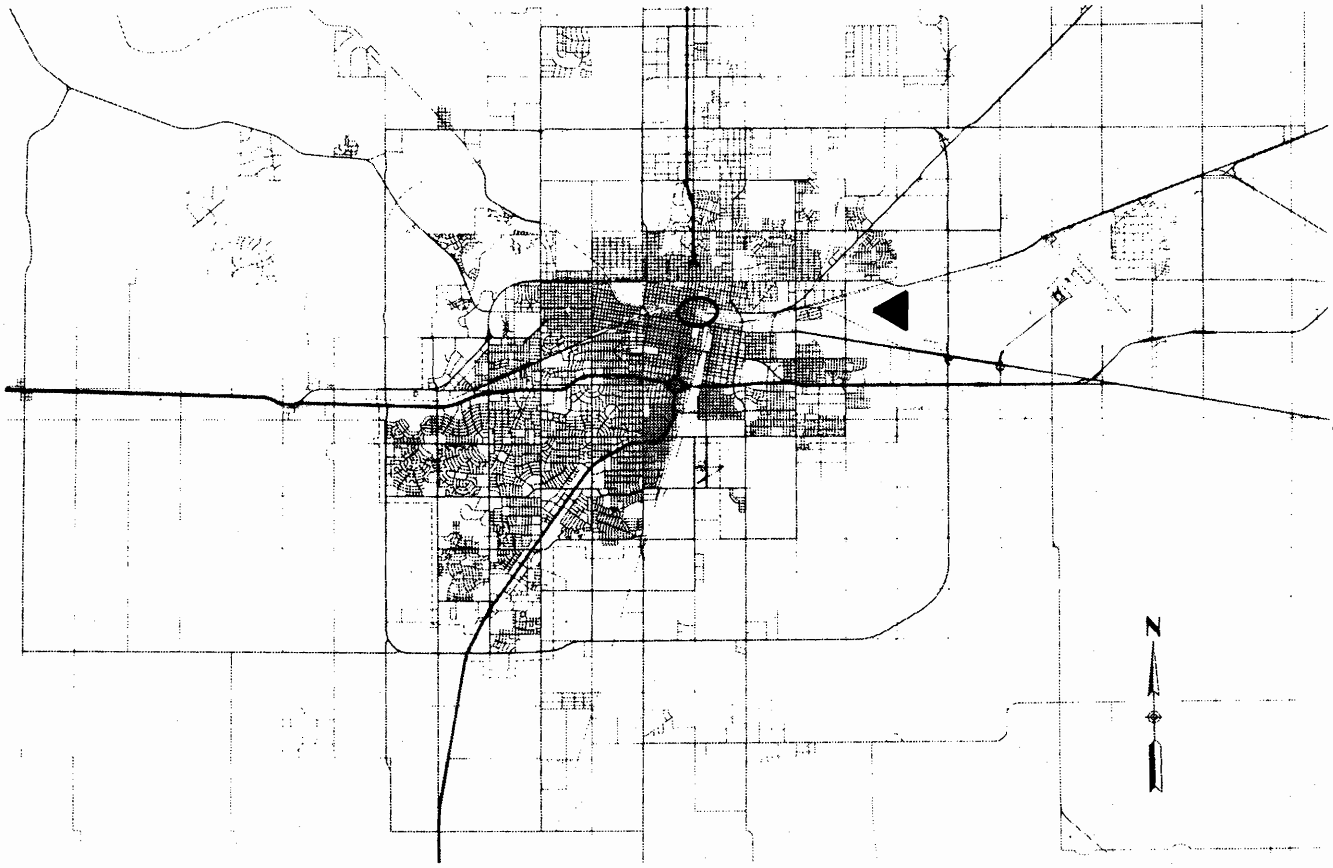
INLET/NODE						
FROM	**TO**	**PROPOSED IMPROVEMENTS**	**NO. UNITS**	**UNIT**	**UNIT COST**	**TOTAL COST**
PUMP STATION IMPROVEMENTS						
R1-A		Pump Station		LS		$75,000
R1-B	R1-C	Force Main (20")	500	LF	$70	$70
					SUBTOTAL	$110,000
STORM SEWER IMPROVEMENTS						
R1-D	R1-E	New 24" Lateral	300	LF	$31.00	$9,300
					SUBTOTAL	$9,300
INLET IMPROVEMENTS						
		10' Inlets	3	EA	$2,300	$6,900
					SUBTOTAL	$6,900
MANHOLES						
		Manholes	1	EA	$3,500	$3,500
					SUBTOTAL	$3,500
					TOTAL	$129,700

(See Drawing Set Sheet No. CE-8)

Street. The existing six-foot by six-foot box culvert can pass the 100-year storm event. Drainage improvements are summarized in Table 3.37-2 and their location shown in Figure 3.37-2.

Lower East Amarillo Creek Channel Reach (M1)

Proposed improvements to the Lower East Amarillo Creek channel reach consist of replacing the existing corrugated steel arch culverts at Willow Creek Drive and Pavillard Drive with 50-foot span reinforced concrete bridges. These new structures, along with minor channel improvements between the two structures, will eliminate overtopping of both



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: 3RD AVE./RR "R1"

DESCRIPTION: DIVERT PORTION OF AREA/UPGRADE PUMP STATION

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-R1)

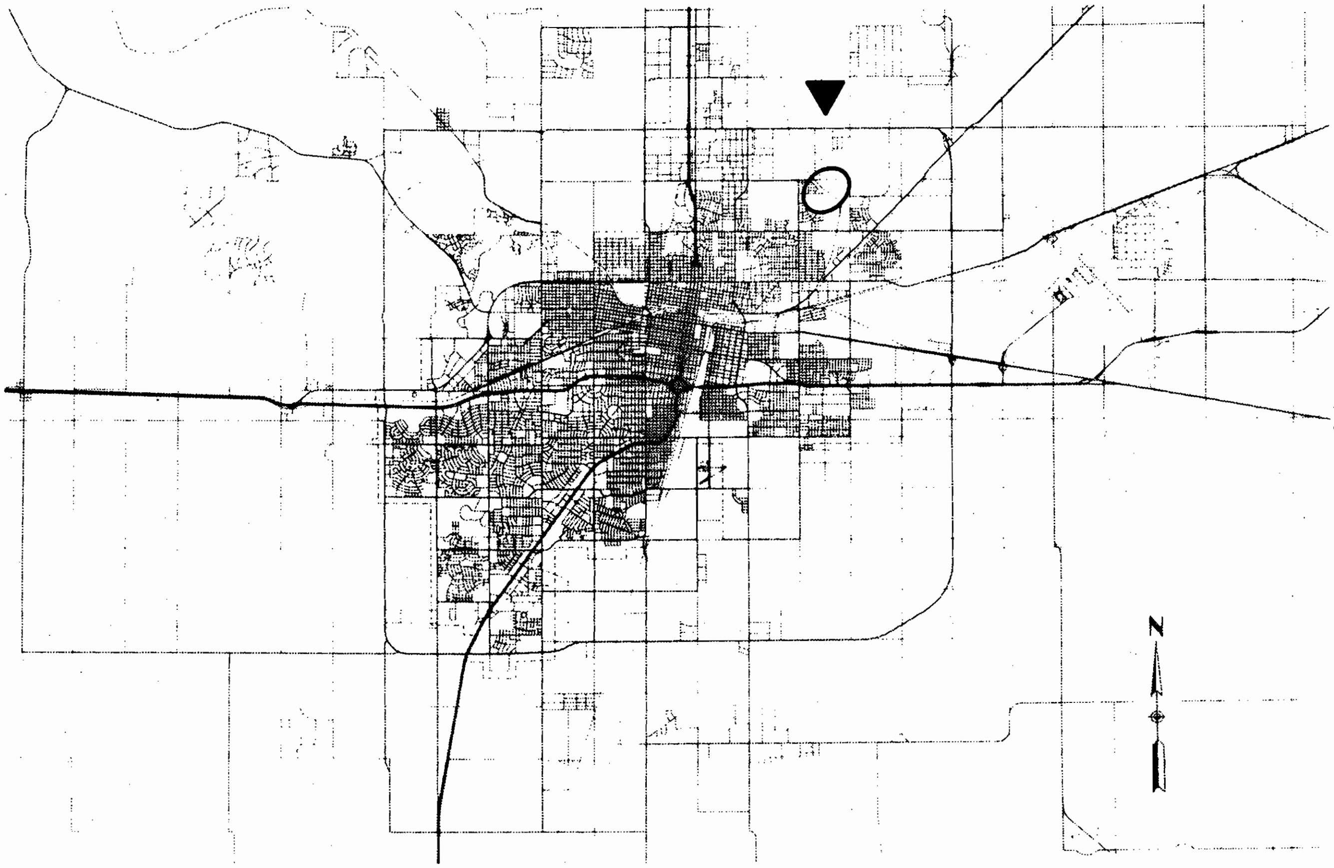
FIGURE 3.37-1

TABLE 3.37-2
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
NORTH BOLTON STORM SEWER (BO)

INLET/NODE						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
STORM SEWER IMPROVEMENTS						
BO-A	BO-B	New 42" RCP	280	LF	$94	$26,320
BO-B	BO-C	New 48" RCP	420	LF	$110	$46,200
BO-C	OUTFALL	New 48" RCP	150	LF	$110	$16,500
					SUBTOTAL	$89,020
INLET IMPROVEMENTS						
		4' Inlets	14	EA	$1,900	$26,600
		30' Inlets	1	EA	$4,600	$4,600
					SUBTOTAL	$31,200
STREET RECONSTRUCTION						
BO-A	BO-C	Regrading - Bolton Street		LS		$75,000
					SUBTOTAL	$75,000
					TOTAL	$195,220

(See Drawing Set Sheet No. CE-6)

roadways for the 25-year flood event and limit the depth of overtopping to less than 1.5 feet for the 100-year flood event. Replacement of the existing corrugated steel arch culverts at Yucca Avenue and Irwin Road is also recommended. Larger reinforced concrete box culverts will pass the 25-year flood event below the top of each roadway and minimize the depth of overtopping at the 100-year flood event. In addition, channel improvements are recommended in the vicinity of Yucca Avenue and Irwin Road to prevent inundation of adjacent residential structures under 100-year flood conditions. The channel improvements would require excavation of the existing channel, which would result in a loss of several trees which currently exist along both channel banks. Proposed improvements for the Lower East Amarillo Creek Channel Reach (M1) are summarized in Table 3.37-3 and their location shown in Figure 3.37-3.



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: N. BOLTON ST. "BO"

DESCRIPTION: ADD NEW STORM SEWER SYSTEM

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-BO)

FIGURE
3.37-2

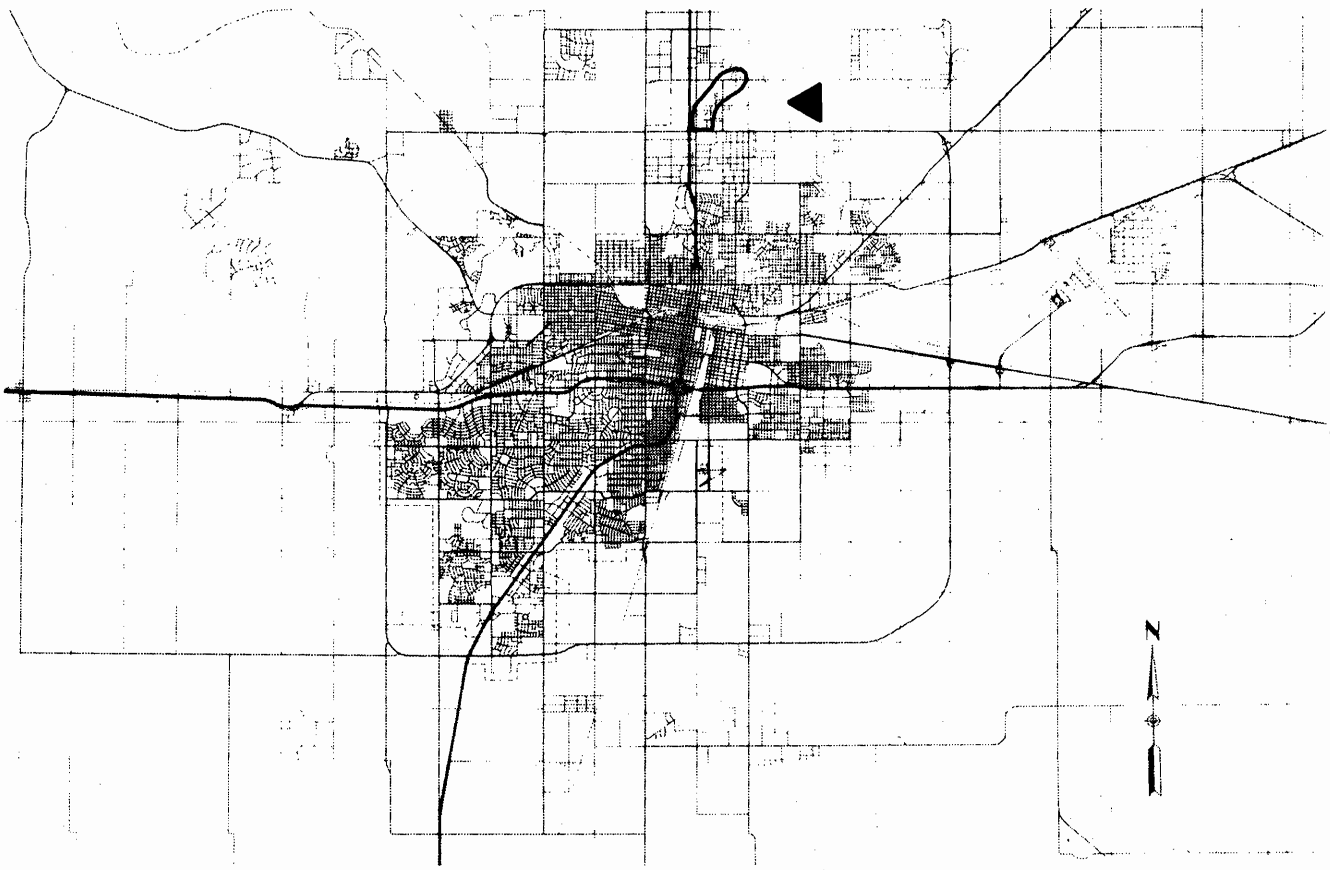
TABLE 3.37-3
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
LOWER EAST AMARILLO CREEK CHANNEL REACH (M1)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
72+90	73+27	Replace existing arch culverts at Willow Creek Drive with 50-foot long concrete bridge	1,700	SF	$50	$85,000
74+82	75+08	Replace existing arch culverts at Pavillard Drive with 50-foot long concrete bridge	1,700	SF	$50	$85,000
73+27	74+82	Improve channel between Willow Creek and Pavillard	2,700	CY	$5.50	$14,900
108+68	109+03	Replace existing arch culverts at Yucca Ave with 4-10'x10' concrete boxes	40	LF	$1,600	$64,000
114+48	114+88	Replace existing arch culverts at Irwin Ave with 4-10'x8' concrete boxes	40	LF	$1,275	$51,000
95+83	122+33	Improve channel between Yucca and Irwin	37,500	CY	$4.75	$178,100
					TOTAL	$478,000

(See Drawing Set Sheet No. CE-2)

Hastings Avenue to River Road Channel Reach (M2)

Proposed improvements to the Hastings Avenue to River Road channel reach consist of replacing or modifying the drainage structures at several roadway crossings and channel improvements to the main channel of East Amarillo Creek. Modifications to the River Road reinforced concrete box culverts are recommended in the form of adding two additional nine-foot by seven-foot barrels. A considerable amount of backwater is caused by the St. Francis/Dumas Drive crossing of East Amarillo Creek for larger flood events. The existing 570-foot long, corrugated steel arch pipe does have sufficient capacity to pass the 25-year flood peak discharge without overtopping St. Francis Avenue and causing



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: LOWER EAST AMARILLO CREEK CHANNEL "M1"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-M1)

FIGURE
3.37-3

flooding of the underpass at Dumas Drive. A second two-barrel, ten-foot by seven-foot reinforced concrete box is recommended to be added at this site in order to prevent overtopping of the roadways. In addition, modifications to the St. Francis Avenue/Dumas Drive structure also provide benefits upstream at the Cliffside Drive bridge by lowering the water surface levels at this crossing. Channel improvements are recommended between St. Francis Avenue and Cliffside Drive in order to increase the conveyance capacity of this reach, as well as reducing flood levels at the Cliffside Drive bridge crossing. Central Avenue is currently equipped with a two sixteen-foot by seven-foot corrugated steel arch culverts which have inadequate capacity to meet the design criteria. Replacement of these culverts with a 60-foot span, reinforced concrete bridge is recommended in order to meet the criteria. The Colorado Avenue crossing of the main channel of East Amarillo Creek consists of a single eighteen-foot by eight-foot corrugated steel arch culvert. Replacement of this structure with a two-barrel, nine-foot by eight-foot reinforced concrete box culvert is recommended in order to eliminate overtopping of the roadway at the 25-year flood event and minimize overtopping to less than 1.5 feet at the 100-year flood event. Proposed improvements for the Hastings Avenue to River Road Channel Reach (M2) are summarized in Table 3.37-4 and their location shown in Figure 3.37-4.

<u>N.W. 24th Avenue Channel Reach (M3)</u>

Proposed improvements to the N.W. 24th Avenue channel reach consist of upgrading each of the street crossings from N.W. 24th Avenue to N.W. 13th Avenue. Each of the four street crossings in this reach have only one storm sewer inlet to convey runoff beneath the street, which is inadequate for the 25-year flood event. A two-barrel, eight-foot by six-foot reinforced concrete box culvert is recommended at N.W. 24th Avenue to pass runoff

TABLE 3.37-4
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
HASTINGS AVENUE TO RIVER ROAD CHANNEL REACH (M2)

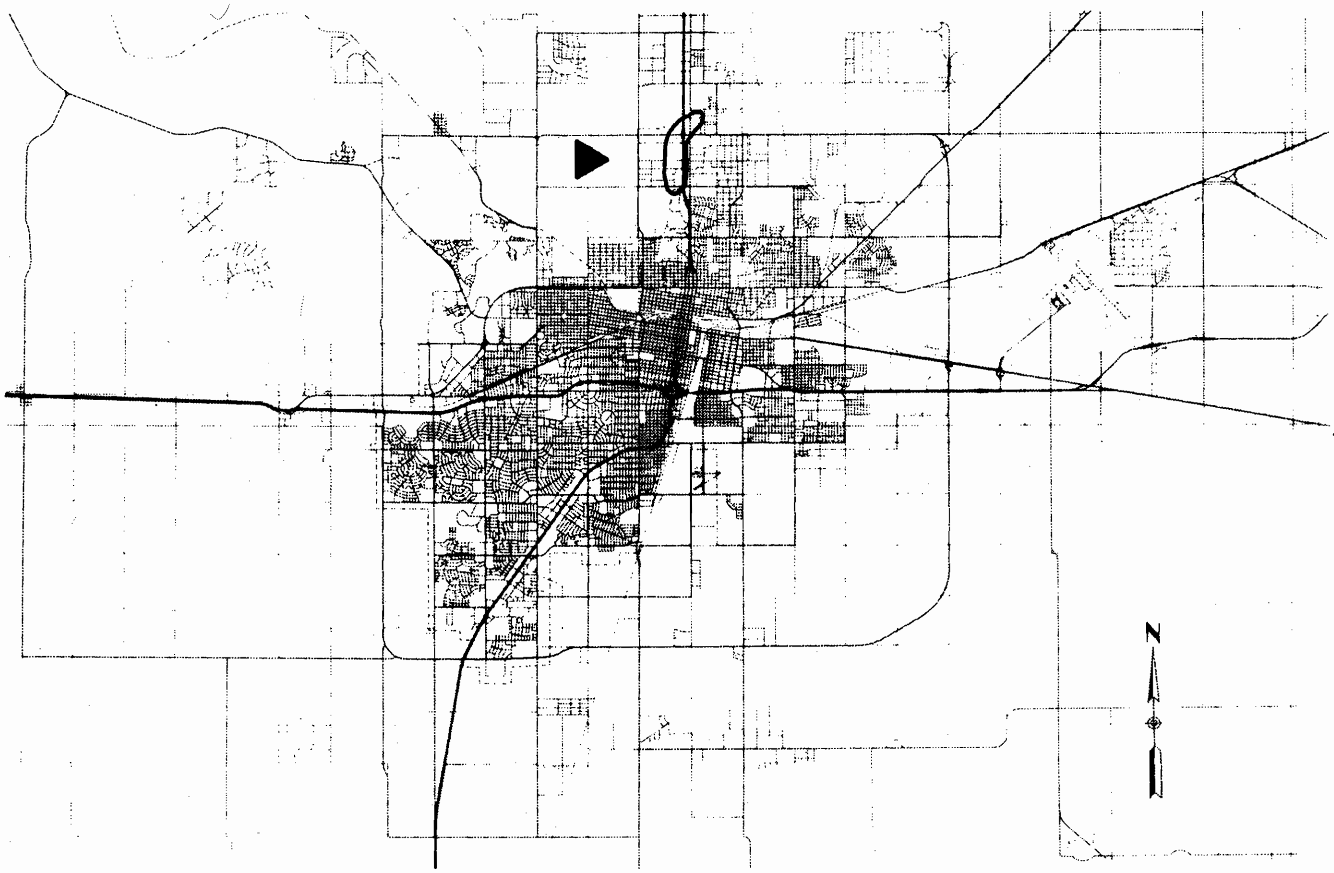
CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
134+03	134+53	Add 2-9'x7' concrete boxes to existing boxes at River Road	50	LF	$600.00	$30,000
140+93	146+63	Add 2-10'x7' concrete boxes to parallel existing arch under overpass at St. Francis Ave	570	LF	$600.00	$342,000
146+63	152+33	Improve channel between Cliffside and Dumas Dr./St. Francis Ave	3,000	CY	$5.00	$15,000
174+61	175+09	Replace existing arch culverts at Central Ave with 60' concrete bridge and raise road	2,040	SF	$50.00	$102,000
184+00	184+50	Replace existing arch culvert at Colorado Ave with 2-9'x8' concrete boxes	50	LF	$760.00	$38,000
					TOTAL	$527,000

(See Drawing Set Sheet No. CE-4)

beneath this roadway. Three-barrel, five-foot by three-foot reinforced concrete box culverts are recommended at N.W. 15th Avenue, N.W. 14th Avenue, and N.W. 13th Avenue, along with raising the roadway profiles in order to meet the current drainage criteria standards. Proposed improvements for the N.W. 24th Avenue Channel Reach (M3) are summarized in Table 3.37-5 and their location shown in Figure 3.37-5.

Echo Street Tributary Channel Reach (T1)

Proposed improvements to the Echo Street Tributary channel reach consist of modifications to the existing drainage structure at Willow Creek Drive and installation of a culvert at Central Avenue near Echo Street along with minor channel improvements. The Willow Creek Drive crossing of the Echo Street Tributary is equipped with two 72-inch



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: HASTINGS AVE. TO RIVER RD. CHANNEL "M2"

DESCRIPTION: ROAD AND CULVERT/BRIDGE IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-M2)

FIGURE
3.37-4

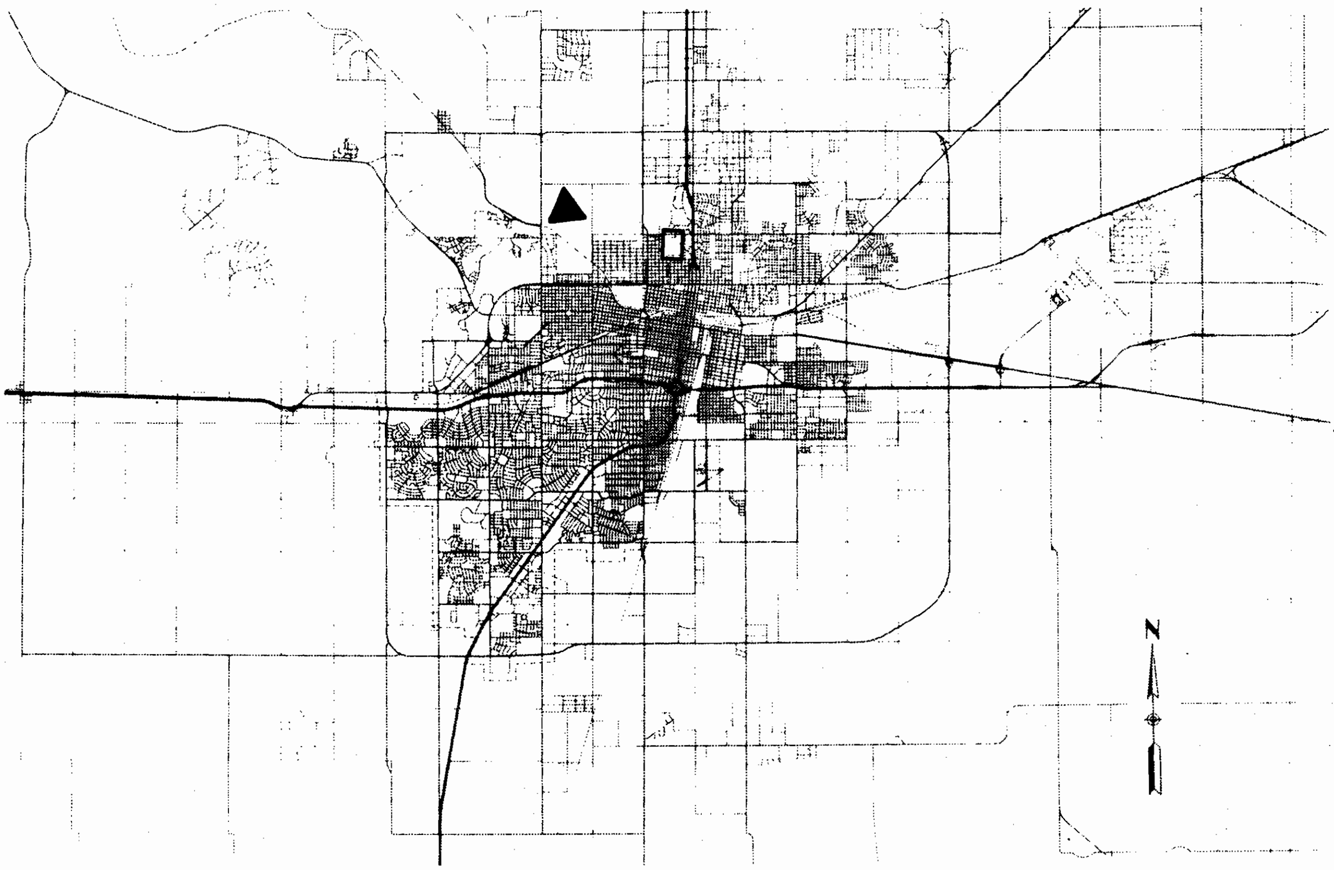
TABLE 3.37-5
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
N.W. 24TH AVENUE CHANNEL REACH (M3)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
258+00	258+80	Install 2-8'x6' concrete boxes to parallel existing 96" at N.W. 24th;	80	LF	$420.00	$33,600
		Construct new drop structure and modify existing culvert outlet headwall/apron		LS		$23,000
284+84	285+24	Install 3-5'x3' concrete boxes at N.W. 15th Ave	40	LF	$450.00	$18,000
288+84	289+24	Install 3-5'x3' concrete boxes at N.W. 14th Ave and raise road;	40	LF	$600.00	$24,000
		Excavate channel u/s and d/s	900	CY	$5.50	$5,000
292+84	293+24	Install 3-5'x3' concrete boxes at N.W. 13th Ave and raise road;	40	LF	$600.00	$24,000
		Excavate channel u/s and d/s	900	CY	$5.50	$5,000
					TOTAL	$133,000

(See Drawing Set Sheet No. CE-7)

diameter corrugated steel culverts which have inadequate capacity to meet the current criteria. These culverts are recommended to be replaced by a two-barrel, ten-foot by seven-foot reinforced concrete box culvert which will pass the 25-year flood peak discharge without overtopping and limit overtopping under 100-year flood conditions to less than 1.5 feet.

Central Avenue crosses this tributary near the intersection with Echo Street. No drainage structure exists at Central Avenue and the channel is poorly defined at this site. Under current conditions, runoff flows over Central Avenue and inundates part of Echo Street for the larger flood events. A new drainage structure, consisting of a five-barrel, eight-foot by four-foot reinforced concrete box culvert, is recommended at Central Avenue along with a raise in the roadway profile at the crossing location. In addition, an improved channel is recommended upstream and downstream of Central Avenue and parallel to Echo Street in order to convey runoff to the culvert and direct it downstream of Central Avenue



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: N.W. 24TH STREET CHANNEL "M3"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-M3)

FIGURE 3.37-5

in order to prevent flooding along Echo Street. Proposed improvements for the Echo Street Tributary Channel Reach (T1) are summarized in Table 3.37-6 and their location shown in Figure 3.37-6.

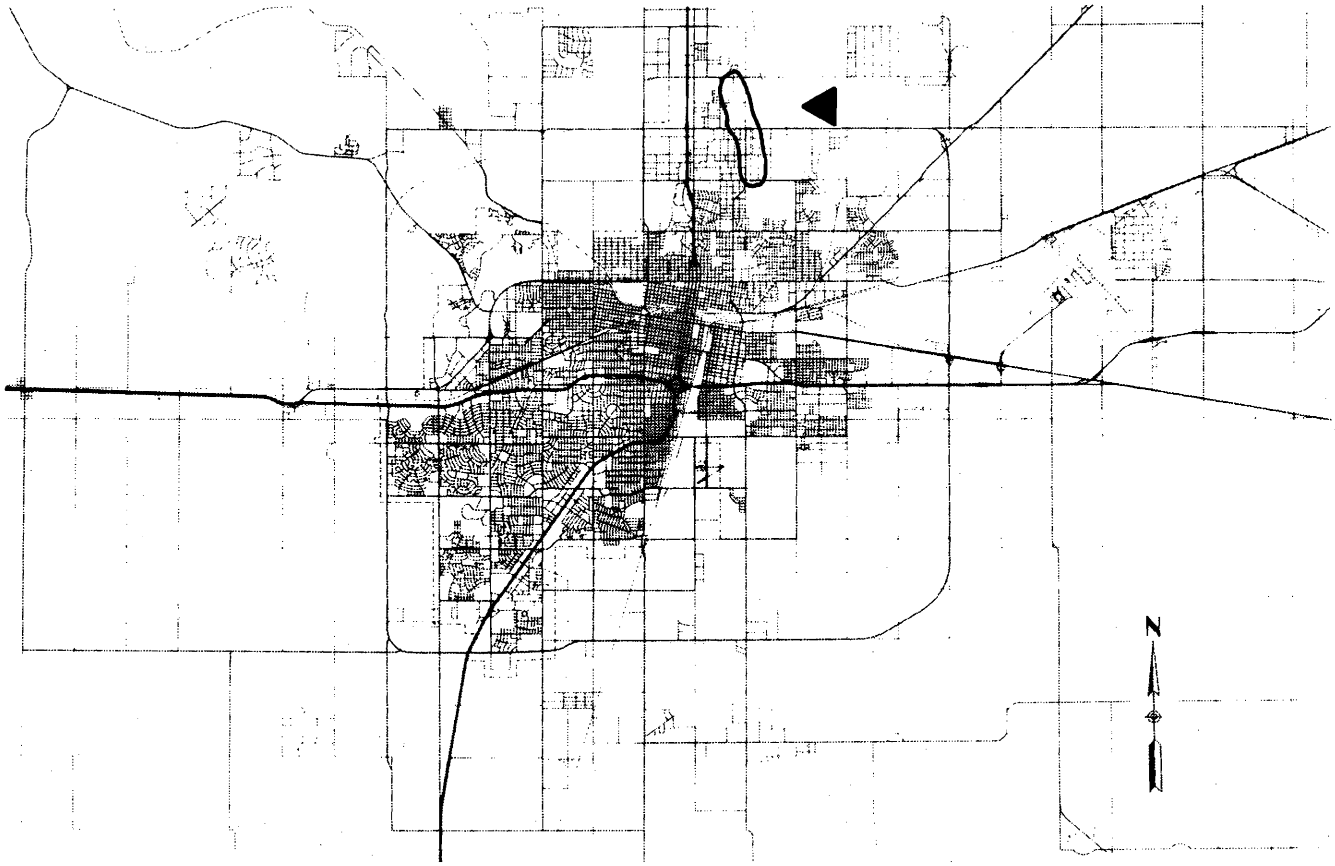
TABLE 3.37-6
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
ECHO STREET TRIBUTARY CHANNEL REACH (T1)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
T1 5+50	T1 5+80	Replace existing CMPs at Willow Creek Drive with 2-10'x7' concrete boxes	30	LF	$970.00	$29,100
T1 96+07	T1 96+37	Install 5-8'x4' concrete boxes at Central Avenue and raise road;	30	LF	$2,000	$60,000
T1 92+22	T1 100+22	Improve channel u/s and d/s of Central Ave parallel to Echo St	2,600	CY	$5.75	$15,000
					TOTAL	$104,000

(See Drawing Set Sheet No. CE-5)

St. Francis Avenue Tributary Channel Reach (T2)

Minor improvements are recommended for the St. Francis Avenue Tributary channel reach. Valley Avenue, a residential street that crosses the tributary, does not include a drainage structure to pass flow beneath the roadway. Under existing conditions, flow passes between residential structures located upstream, flows across the top of the roadway surface, and continues downstream between other residential structures through a poorly-defined channel. Improvement recommendations include installing a five-barrel, five-foot by three-foot reinforced concrete box culvert at Valley Avenue and minor improvements to the channel immediately upstream and downstream of the structure. Proposed improvements for the St. Francis Avenue Tributary Channel Reach (T2) are summarized in Table 3.37-7 and their location shown in Figure 3.37-7.



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: ECHO STREET TRIBUTARY "T1"

DESCRIPTION: ROAD AND CULVERT/BRIDGE IMPROVEMENTS/ CHANNEL IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-T1)

FIGURE
3.37-6

TABLE 3.37-7
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
ST. FRANCIS AVENUE TRIBUTARY CHANNEL REACH (T2)

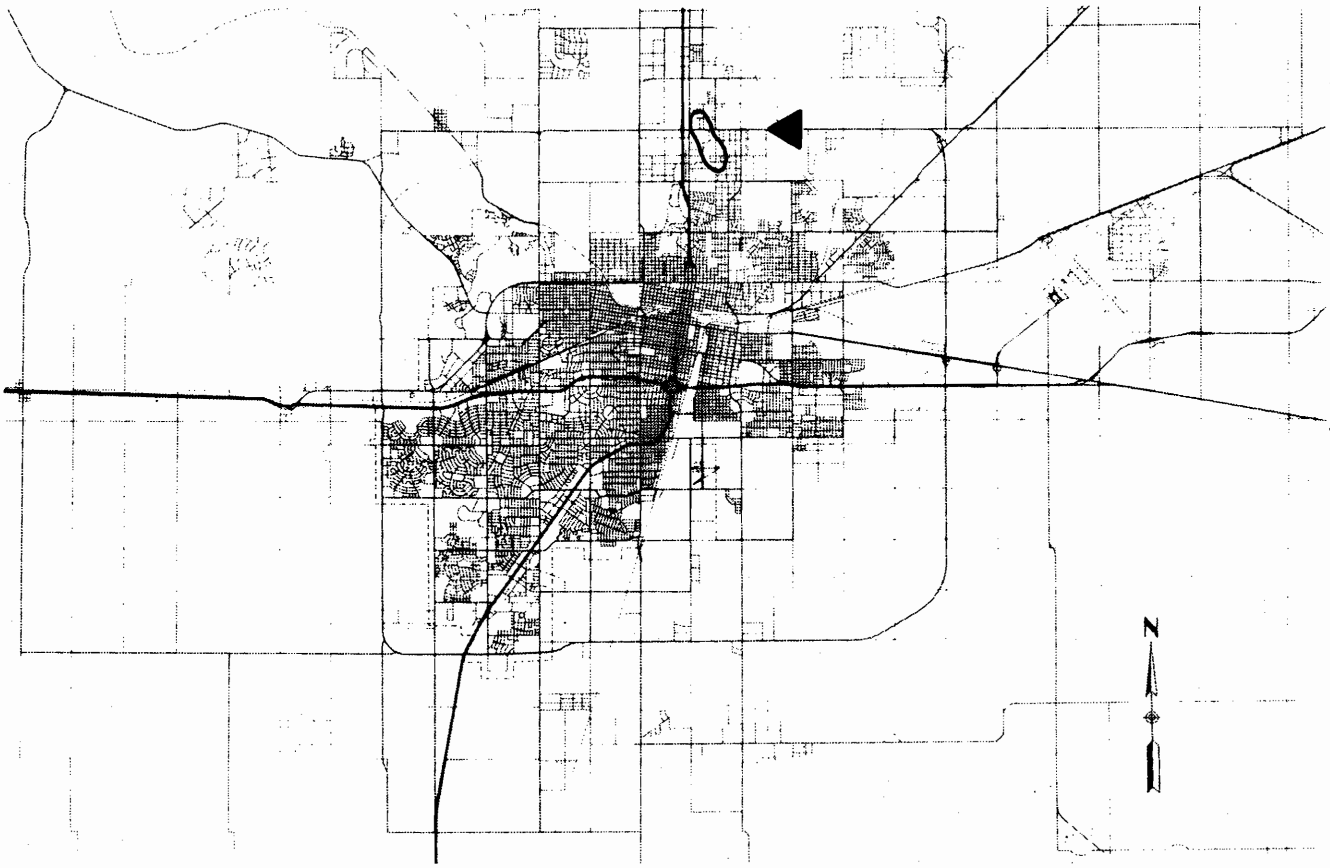
CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
T2 28+90	T2 29+20	Install 5-5'x3' concrete boxes at Valley Ave and raise road;	30	LF	$770.00	$23,100
T2 25+05	T2 33+05	Excavate channel 400' u/s and d/s of Valley Ave	800	CY	$7.50	$6,000
					TOTAL	$29,000

(See Drawing Set Sheet No. CE-5)

Valley Park Tributary Channel Reach (T3)

Proposed improvements to the Valley Park Tributary channel reach include adding additional capacity at River Road to pass runoff from the east side of Dumas Drive to the main channel of East Amarillo Creek west of the highway. An existing single-barrel, eight-foot by five-foot reinforced concrete box is utilized to pass runoff from River Road to the main channel of East Amarillo Creek. The structure is 1,200 feet in total length and extends under Dumas Drive. Once the capacity of the existing structure is exceeded, flow will begin to overtop River Road and then flow over Rose Drive and the Central Avenue underpass at Dumas Drive. Recommended improvements include modifications to the existing structure at River Road, channel improvements from River Road to Dumas Drive, installation of new culverts at Rose Drive and the Central Avenue underpass, and channel improvements in the ditch parallel to Central Avenue from Dumas Drive to the main channel of East Amarillo Creek. In addition, improvements are recommended upstream of River Road at three street crossings; Park Avenue, Hastings Avenue, and Angelus Drive.

Recommendations at Park Avenue, which currently has no drainage structure to pass runoff beneath the street, include installing a new culvert and raising the road profile.



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: ST. FRANCIS AVE. TRIBUTARY "T2"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS/ CHANNEL IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-T2)

FIGURE
3.37-7

Replacement of the existing corrugated steel arch culverts with a three-barrel, seven-foot by five-foot reinforced box culvert at Hastings Avenue, and installation of a three-barrel, five-foot by three-foot reinforced concrete box culvert at Angelus Drive is recommended in order to meet the current drainage criteria. Proposed improvements for the Valley Park Tributary Channel Reach (T3) are summarized in Table 3.37-8 and their location shown in Figure 3.37-8.

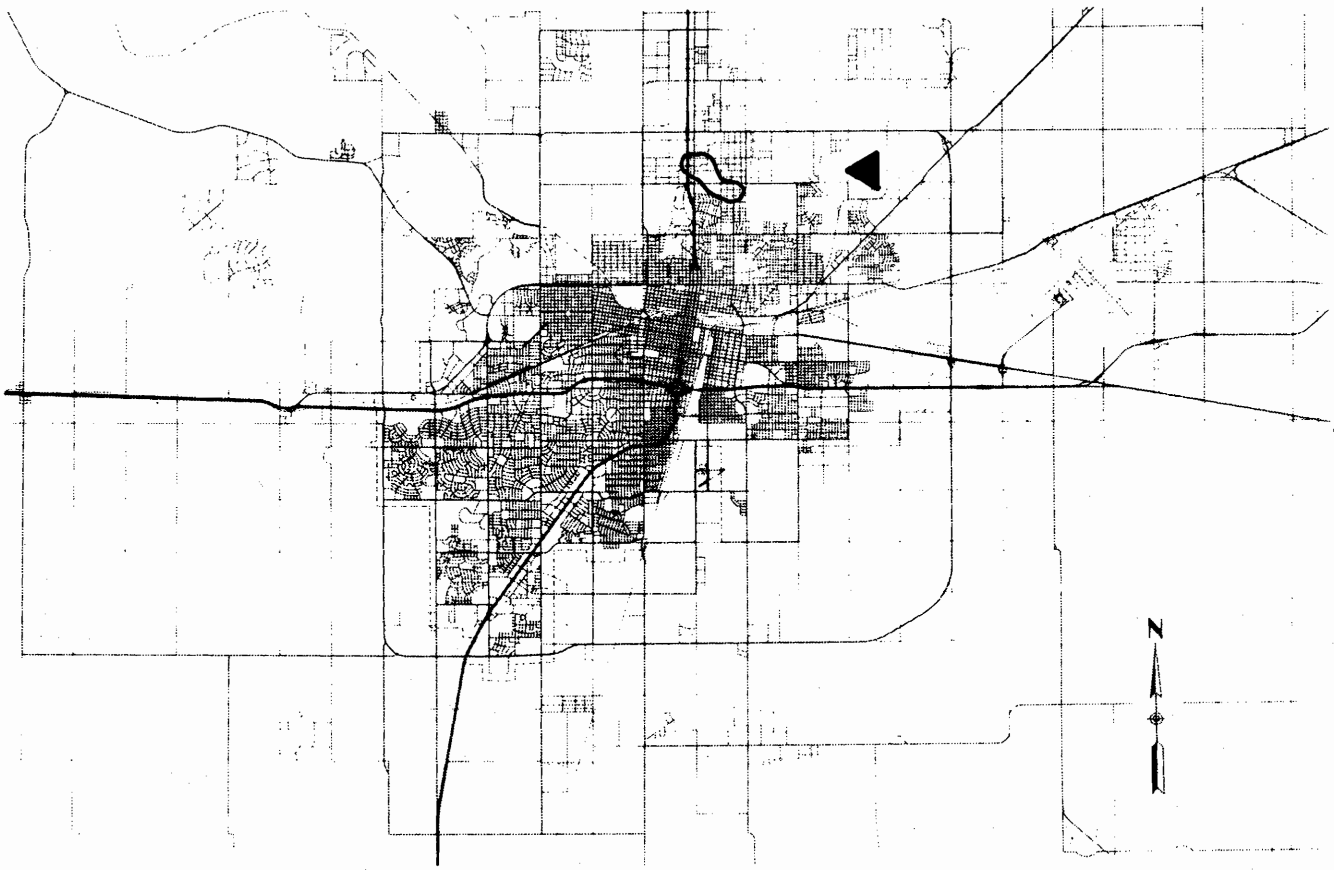
TABLE 3.37-8
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
VALLEY PARK TRIBUTARY CHANNEL REACH (T3)

CHANNEL STATION		PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
FROM	TO					
T3 16+30	T3 16+60	Modify existing 3-6'x6' boxes to divert portion to new channel		LS		$5,000
T3 11+35	T3 11+65	Install 2-6'x5' concrete boxes at Rose Drive and raise road;	30	LF	$500.00	$15,000
T3 10+00	T3 16+30	Excavate channel from River Road to Dumas Dr.	700	CY	$7.00	$4,900
T3 7+00	T3 10+00	Install 2-6'x5' concrete boxes at Dumas Dr. overpass/Central Ave	300	LF	$390.00	$117,000
T3 0+00	T3 7+00	Excavate channel from Dumas Dr. to East Amarillo Creek	550	CY	$7.25	$4,000
T3 22+15	T3 22+45	Install 4-8'x4' concrete boxes at Park Ave and raise road	30	LF	$1,600	$48,000
T3 41+80	T3 42+20	Replace existing arch culverts at Hastings Ave with 3-7'x5' concrete boxes	40	LF	$675.00	$27,000
T3 61+90	T3 62+30	Install 3-5'x3' concrete boxes at Angelus Drive	40	LF	$425.00	$17,000
					TOTAL	$238,000

(See Drawing Set Sheet No. CE-5)

Ross Rogers Tributary Channel Reach (T4)

Proposed improvements to the Ross Rogers Tributary channel reach are limited to four street crossings. These crossings include Colorado Avenue, Buck Drive, Studebaker



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: VALLEY PARK TRIBUTARY "T3"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS/ CHANNEL IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-T3)

FIGURE 3.37-8

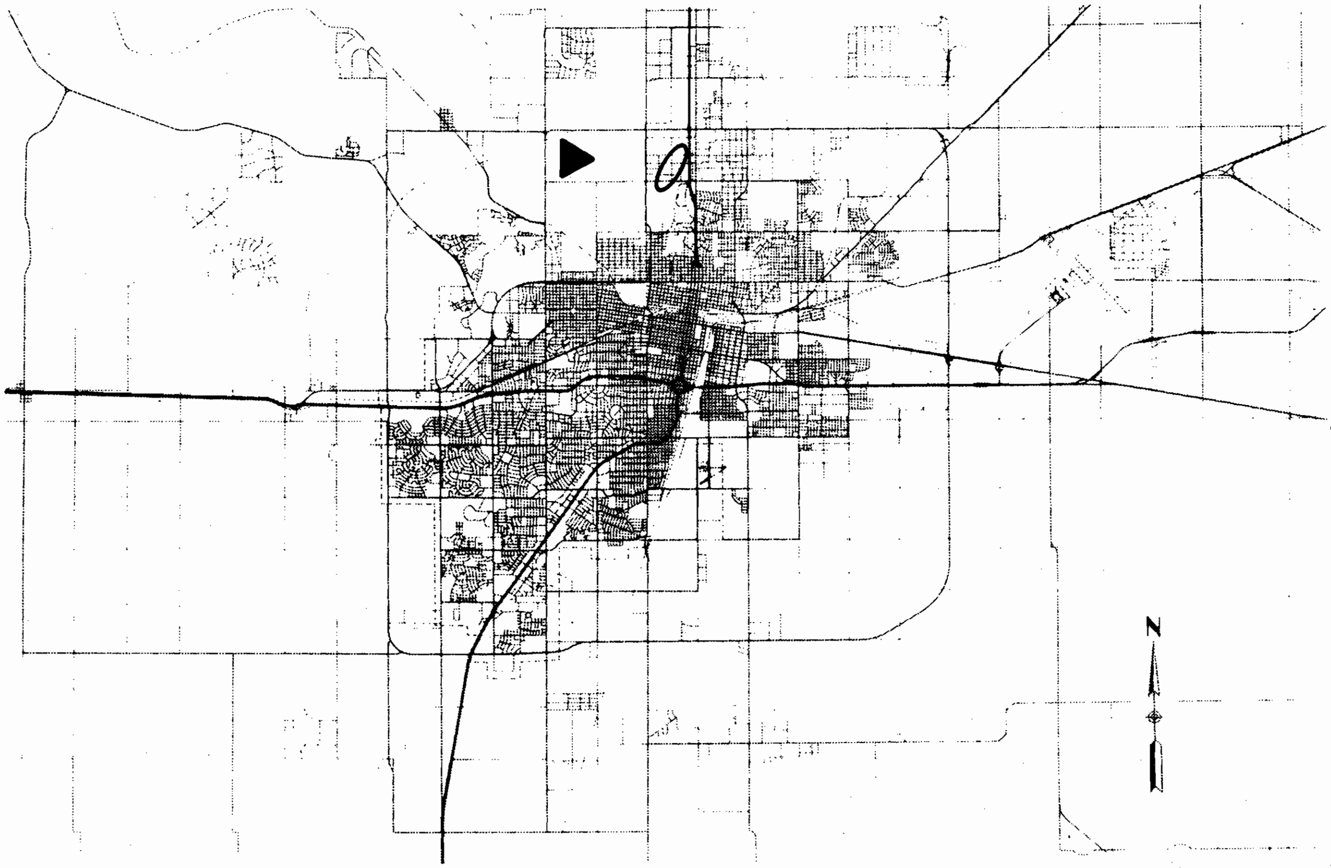
Avenue, and Hastings Avenue. Each of the roadways and drainage structures at these locations fail to meet the current drainage criteria. Recommended improvements at Colorado Avenue include replacing the existing corrugated steel arch culvert with a three-barrel, ten-foot by five-foot reinforced concrete box culvert and raising the roadway profile at the stream crossing. Buck Drive currently has no drainage structure installed to pass runoff under the roadway; therefore, all runoff flows on top of the pavement across the width of the street. Recommended improvements at Buck Drive include installing a three-barrel, ten-foot by five-foot reinforced concrete box culvert and raising the profile of the roadway to accomodate this structure.

Studebaker Avenue currently has an 18-inch corrugated steel pipe to pass runoff under the street. The capacity of this pipe is minimal compared to the design peak flow; therefore, modifications to this structure are recommended. These modifications include replacing the existing corrugate steel pipe with a two-barrel, nine-foot by six-foot reinforced concrete box culvert and raising the profile of the roadway at the stream crossing. Hastings Avenue currently includes a corrugated steel arch culvert to pass runoff under the roadway. The capacity of this culvert was inadequate to pass the 25-year peak flow under the opening; therefore, replacement of this structure is recommended. The replacement structure includes a single-barrel, ten-foot by seven-foot reinforced concrete box culvert. Proposed improvements for the Ross Rogers Tributary Channel Reach (T4) are summarized in Table 3.37-9 and their location shown in Figure 3.37-9.

TABLE 3.37-9
PROPOSED IMPROVEMENTS AND ESTIMATED CONSTRUCTION COSTS
EAST AMARILLO CREEK STUDY AREA (CE)
ROSS ROGERS TRIBUTARY CHANNEL REACH (T4)

CHANNEL STATION						
FROM	TO	PROPOSED IMPROVEMENTS	NO. UNITS	UNIT	UNIT COST	TOTAL COST
T4 10+76	T4 11+00	Replace existing arch culverts at Colorado Ave with 3-10'x5' concrete boxes and raise road	30	LF	$1,200	$36,000
T4 15+57	T4 15+87	Install 3-10'x5' concrete boxes at Buck Ave and raise road	30	LF	$1,270	$38,100
T4 30+25	T4 30+62	Replace existing RCP at Studebaker Ave with 2-9'x6' concrete boxes and raise road	37	LF	$600	$22,200
T4 36+72	T4 37+04	Replace existing arch at Hastings Ave with 10'x7' concrete box	32	LF	$470	$15,000
					TOTAL	$111,000

(See Drawing Set Sheet No. CE-4)



STUDY AREA: EAST AMARILLO CREEK "CE"

PROJECT NAME: ROSS ROGERS TRIBUTARY "T4"

DESCRIPTION: ROAD AND CULVERT IMPROVEMENTS

CITY OF AMARILLO
STORM WATER MANAGEMENT MASTER PLAN

PROJECT IDENTIFICATION MAP (CE-T4)

FIGURE
3.37-9

4.0 CAPITAL IMPROVEMENTS PROGRAM

The Capital Improvements Program for the City of Amarillo is a specific long-range plan to address current and future flooding problems in the area. For the purposes of this Master Plan, the Capital Improvements Program takes the form of a project list which addresses the City's most severe drainage problems. In order to establish the complete Program, identified capital projects must be prioritized according to acceptable rating criteria and the resulting project list scheduled for implementation in a manner which meets the City's ability to fund the projects.

The Capital Improvements Program Project List developed through the Master Plan process consists of projects in three general categories: 1) playa improvement projects; 2) storm sewer projects; and 3) channel improvement projects. Table 4.1 lists 14 playa projects, 17 storm sewer projects, and 14 channel projects, along with the estimated construction cost for each. The costs given are construction only and do not include right-of-way, easement acquisition, major utility relocations, engineering, or project administration.

Although numerous criteria can be used to establish priorities for implementation of the proposed Capital Improvements Program projects, the following are proposed criteria arranged in order of decreasing importance:

- Severity of existing problem;
- Safety;
- Capital cost;
- Preserving/enhancing existing property values;
- Development potential;
- Social/environmental impacts; and
- Maintenance/operating costs.

In order to prioritize the Capital Improvements Program Project List, it is recommended the City first assign relative weighting factors to each of the above rating criteria. Next, a rating scale is established where a high number might indicate that the criterion under consideration is important/critical and a low number not important/trivial. Once the weighting factors and rating scale are established, each project can be scored by summing the product of score and weighting factor for each of the rating criteria established. The project with the highest score would be considered as the top priority.

Once the City has prioritized the Capital Improvements Program Projects, an implementation schedule would be developed. A key consideration in developing the implementation schedule is the City's ability to finance the various projects. Consideration of the City's financing abilities and means is not addressed in this Master Plan. However, several suggestions are offered which the City should consider when developing the implementation schedule:

- In general, a downstream project is recommended to be undertaken before an upstream project. This applies primarily to the channel projects and less so to playa and contributing storm sewer projects. Should a storm sewer project emerge as a higher priority than the downstream playa, the short-term flooding relief provided by first undertaking the storm sewer project might well outweigh the benefits derived from tailwater reduction associated with the playa project.

- It is recommended that the City consider projects which are interdependent when establishing the implementation schedule. The three primary examples of interdependent projects follow:

    1) Fleetwood/Terrace (FW) - Hillside/Hampton (HH) - Catalpa (CA). A portion of the existing FW system is to be diverted to HH at Fulton and Bell. Similarly, a portion of the HH system is to be diverted to CA at Norwich and Bell. This interdependency suggests that all three projects be treated as one large storm sewer project and constructed simultaneously. However, it is possible to construct the CA project first and postpone FW and HH.

2) South Amarillo Playa Pumping Project.
The Master Plan recommends that Playas 7, 14, 15, 16, and 17 be linked to a common force main which discharges to Spring Draw near Helium Road two miles south of FM 2186. Operation constraints suggest that these projects be constructed in a downstream to upstream sequence beginning with P7 or that the entire outfall system be installed from the upstream playa, P17, to the discharge point. For purposes of this Master Plan, these five playa projects are structured according to the second option. Therefore, the four downstream playa projects are shown to tie in only to the outfall force main. The City may elect, however, to construct portions of the outfall in phases according to the first option.

3) Ong/Lipscomb Diversion (OL) and Wild Horse Lake (P23).
Project OL involves the diversion of runoff from several hundred acres south and west of Wild Horse Lake. Because this diversion to Wild Horse Lake will increase levels in the playa, the OL project is recommended to be constructed after, or in conjunction with, the P23 project.

TABLE 4.1
CAPITAL IMPROVEMENTS PROGRAM PROJECT LIST

DESIGNATION	PROJECT	DESCRIPTION	CONST. COST
		PLAYA PROJECTS	
P5	McDonald Lake	New pump station and force main; excavate playa	$1,938,000
P6	Lawrence Lake	Add a new pump and replace force main at each pump station	$1,183,000
P7	Playa No. 7	New pump station and force mains to outfall at Spring Draw; excavate playa	$650,000
P14	Diamond Horseshoe	New pump station and force mains	$162,000
P15	McCarty Lake	New pump station and force mains; excavate playa; raise Georgia St.	$4,602,000
P16	Willow Grove Lake	New pump station and force mains; minor playa excavation	$417,000
P17	Bennett Lake	New pump station and force main; abandon force main to Lawrence Lake; excavate playa	$4,703,000
P18	Playa No. 18	Excavate playa; raise Burlington Road	$498,000
P21	T-Anchor Lake	Add new pump/replace existing suction and force mains; raise or waterproof pump station; excavate playa	$876,000
P23	Wild Horse Lake	New gravity relief culvert and flap gate, new junction box at connection to existing 96-inch	$72,000
P24	Martin Lake	Replace existing pump/upgrade suction and force mains; excavate playa	$2,634,000
P26	Juett Lake	Floodproof businesses	$50,000
P34	Playa No. 34	New gravity outfall channel and box culverts	$110,000
P35	Playa No. 35	Floodproof business	$50,000
		STORM SEWER PROJECTS	
P5-CO	Coulter Street	Replace storm sewer/add inlets along Coulter from S.W. 45th to Hillside; new storm sewer/inlets along Van Winkle and S.W. 45th	$940,000
P6-OE	Olsen/Emil	New storm sewer/inlets along Olsen and Emil to Western; replace storm sewer and inlets along Western and Olsen to outfall	$1,104,000

TABLE 4.1 (CONTINUED)
CAPITAL IMPROVEMENTS PROGRAM PROJECT LIST

DESIGNATION	PROJECT	DESCRIPTION	CONST. COST
P6-DD	Dilday Draw	Replace storm sewer/inlets along Western from Crouch to Wolflin; parallel I-40 storm sewer from Western to outfall	$662,000
P6-TS	S.W. 26th Ave.	New storm sewer/inlets along Bowie to Elmwood, along W. 28th to Elmwood, along Hawthorne to Wimberly, S. Austin, and Georgia, along Austin to Georgia; parallel storm sewer along S.W. 26th from Elmwood to outfall	$1,993,000
P6-JN	Julian Blvd.	New storm sewer/inlets along Julian from W. 15th to I-40 system	$338,000
P6-FW	Fleetwood Drive	Construct interconnections along Terrace and Fleetwood; replace storm sewer in Ridgecrest, Teckla, and Bell; replace storm sewer in Western and extend new segment to tie-in at Ridgecrest/Western; divert Fulton storm sewer (360 acres) to Hillside/Hampton system at Bell	$1,195,000
P15-HH	Hillside/Hampton	Add Fulton lateral from Fleetwood system. Prallel Bell Street lateral with new storm sewer to increase capacity; replace storm sewer along Hampton from Hillside to Wentworth; divert Hampton/Norwich lateral to Catalpa system w/new storm sewer/inlets; replace storm sewer along W. 51st from Shawnee to Goodnight; replace storm sewer along Western from W. 53rd to S.W. 58th	$1,480,000
P15-CA	Catalpa Lane	Add Hampton/Norwich lateral from Hillside/Hampton System; construct relief interceptor from Bell Street to I27; add Star Lane lateral to system; replace storm sewer along Farmers from I-27 to Greenhaven; replace storm sewer/add inlets along Western from Arden to outfall	$3,146,000
P15-FE	S.W. 58th Ave.	Replace storm/inlets along S.W. 58th Street - add two new outfalls to McCarty Lake	$154,000
P16-RH	Rushmore/Hayden	Replace storm sewer/inlets along Everest from Trinchera to outfall	$416,000
P20-R3	27th Ave./RR	Divert runoff from sag with new storm sewer lateral/inlets	$14,000
P21-I4	Interstate 40	New storm sewer/inlets along 19th and Marrs to I-40, along 19th and Pittsburg to I-40, along Birmingham to I-40; replace storm sewer along Osage from 27th to I-40	$1,267,000

TABLE 4.1 (CONTINUED)
CAPITAL IMPROVEMENTS PROGRAM PROJECT LIST

DESIGNATION	PROJECT	DESCRIPTION	CONST. COST
P21-R2	10th Ave./RR	New storm water pumping station; divert runoff from sag with new lateral/inlet; modify existing inlet	$90,000
P23-OL	Ong/Lipscomb	Divert to Wild Horse Lake by replacing storm sewer/inlets from Parker/15th to Ong/RR and add new storm sewer/inlets from Ong/RR to Wild Horse outfall	$1,614,000
CW-WM	Westgate Mall	New storm sewer/inlets from Sleepy Hollow/Tripp to outfall at Gainsborough/Elmhurst; new storm sewer/inlets from Irving/S.W. 34th to outfall at Simpson/Gerald	$646,000
CE-R1	3rd Ave./RR	Upgrade existing storm water pumping station/add new force main; divert runoff from sag with new lateral/inlets	$130,000
CE-BO	N. Bolton St.	New storm sewer/inlets along Mesa Verde from N.E. 32nd to N. Bolton	$195,000
		CHANNEL PROJECTS	
P22-UT	Unnamed Tributary	Replace culverts at Panhandle Blvd; improve crossing at RR/N. Eastern; add culvert and raise road at N.E. 13th Ave; replace culverts and raise road at N.E. 11th Ave; replace culvert at Amarillo Blvd; improve channel from Amarillo Blvd through N.E. 13th Ave	$180,000
CW-SJ	San Jacinto Hts. E.	Add culvert at Amarillo Blvd; replace culverts at N.W. 9th Ave; replace culverts at N.W. 11th Ave; add new storm sewer under Mississippi from N.W. 2nd Ave to Amarillo Blvd	$327,000
CW-WC	Westcliff	Add culverts at Kouba Drive; improve channel from Kouba to 9th Ave	$15,000
CW-AC	Amarillo CC	Add culverts at Amarillo Blvd; replace culverts at N.W. 4th Ave and W. 2nd Ave; install culverts and raise road at W. 3rd Ave; replace culvert at Gem Lake Road; improve channel from Gem Lake Road to W. 3rd Ave	$139,000
CW-TC	Tascosa CC	Replace existing culvert at Trevino Ave with two 4' x 3' concrete boxes	$12,000
CW-WA	Wolflin Ave.	New storm sewer from BNRR to Wolfin; channel improvements from Wolfin to Amarillo Boulevard West; new culverts at Amarillo Boulevard West and BNRR	$420,000

TABLE 4.1 (CONTINUED)
CAPITAL IMPROVEMENTS PROGRAM PROJECT LIST

DESIGNATION	PROJECT	DESCRIPTION	CONST. COST
CW-MP	Medical Park	Replace culvert at W. 9th Avenue; new and replaced culverts at Wallace Blvd.; replace culverts at South Wallace Blvd.; storm sewer at Fleming Drive; channel improvements upstream of Halstead Drive	$678,000
CE-M1	Lower E. Amar. Cr.	Replace culverts at Willow Creek and Pavillard Drives/improve channel; replace culverts at Yucca and Irwin Aves/improve channel	$478,000
CE-M2	Hastings to River	Upgrade River Rd and Dumas Hwy crossings; improve channel between Cliffside and Dumas Hwy; replace culverts at Central and Colorado Aves	$527,000
CE-M3	N.W. 24th Avenue	Upgrade crossing at N.W. 24th; new culverts at N.W. 15th; new culverts, channel excav, and road raises at N.W. 14th and 13th	$133,000
CE-T1	Echo St. Tributary	Replace culverts at Willow Creek Drive; new culverts and road raise at Central Ave; improve channel between Central and Echo	$104,000
CE-T2	St. Francis Ave. Trib.	New culverts, road raise, and channel improvements at Valley Ave	$29,000
CE-T3	Valley Park Trib.	Modify/upgrade drainage system from River Road to E Amarillo Creek with culverts and channel improvements at Rose Drive and Dumas Hwy; new culverts and road raise at Park Ave; replace culverts at Hastings Ave; new culverts at Angelus Drive	$238,000
CE-T4	Ross Rogers Trib.	Replace culverts and raise road at Colorado and Studebaker Aves; new culverts and road raise at Buick Dr; replace culvert at Hastings Ave	$111,000

Reports

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Preliminary Engineering Report, T-Anchor Lake Drainage Improvements, Freese and Nichols, 1975
Playa Lake Flood Study, Freese & Nichols, Inc., August 1985
Report on Surface Drainage, Amarillo, Texas, Freese, Nichols, and Endress, 1968
Hydrology-Westgate Mall Shopping Center, Reports No. 1 and 2, Frank T. Wheby, August 14, 1978 and March 8, 1979
Hydrologic Investigation of the Lawrence Lake Playa, Powell & Powell, 1983
Operating Procedures for the Lake Pumps for the City of Amarillo, Texas, 1991
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Report on Storm Sewer System, Amarillo, Texas, Freese & Nichols, 1946
Proposed Storm Sewer Locations, Dept. of Engr., May, 1963
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High Flood Hazard Area Studies, Amarillo, Texas, USCOE, September, 1986

Maps and Plans

Flood Insurance Rate Maps, FEMA, 1982 (photocopied)
Flood Insurance Rate Maps, FEMA, 1982 (folded)
Aerial Photo Key Maps and Topographic Key Maps
USGS Quad Maps (Cliffside, Pleasant Valley, Mayer, Amarillo West, Amarillo East, Pullman, Buffalo Stadium, The Palisades, Thomas Ranch)
Wastewater Collection Study Maps (III-3, III-4, VII-1)
City of Amarillo Drainage Area Map, 1986
City of Amarillo Storm Sewer Map, 1987
Quad map photocopies (oversized) with playa watershed delineations
Zoning Maps, City of Amarillo, Dept. of Planning
Site Development Plans-Westgate Mall
Paper and Mylar Topographic Maps for Amarillo (1"=200' scale) - about 155 sheets
City of Amarillo Storm Sewer Map (mylar) - Engineering Dept.
Plans of Proposed Street Improvements for Amarillo, Tx. (Hatfield Engineering)
City of Amarillo Street Improvements Engineering Department - Fulton Dr. from Alley East of Bell to Alley West of Justin & Intersection of Bell & Fulton
Department of the Air Force - 500 Unit Armed Services Housing Project for Amarillo Air Force Base, Tx.
Proposed Storm Sewer - North of 26th Ave. (Hatfield Eng.)
26th Avenue Storm Sewer - Georgia to Paramount Blvd. (Hatfield Eng.)

City of Amarillo Street Improvements - South Georgia from 26th to Civic Circle
City of Amarillo Storm Sewer Improvements - Western St.
City of Amarillo Storm Sewer Improvements - Bell Street Sewer Facilities
City of Amarillo Storm Sewer Improvements - Wolfin at Georgia
City of Amarillo Storm Sewer Improvements - 40th & Western
Proposed Sewer Extension - Ridgecrest Addition (Hatfield Eng.)
General Layout - Storm Sewer Drains - Lawrence Park Addition (Hatfield Eng.)
26th Street Storm Sewer Problem Area - Plans - no profile
Olsen/Emil Storm Sewer Problem Area
Fleetwood/Terrace Storm Sewer Problem Area
Bell Street Storm Sewer Problem Area
Rushmore/Hayden - Storm Sewer & Drainage Area (8 1/2x14)
A.T.& S.F. Railroad from West of Grand St. to +/- .5 mile North of Cherry
Modifications to Thompson Park Dam No. 3 (HDR)
Westcliff Park Unit No. 13 - Proposed Street Elevations (Plum Creek Dr.)
Channing St. from S.E. 8th to S.E. 13th
Storm Sewer Extension - S. Ong St.
Storm Sewer Improvements - Extension of Existing Line - S.E. 12th Ave.
Storm Sewer Improvements - Dilday - Draw Storm Sewer
Storm Sewer Improvements - South Lincoln St. Project
Amarillo Texas Storm Sewers 1927
Mirror - Echo Connector from La Mesa to St. Francis
72" Storm Sewer along Chicago, Rock Island, & Gulf Railroad Right-of-Way
Storm Sewer Improvements - First Ave. (Hughes-Ong)
San Jacinto Park
West Hills Park - South
West Hills Park - North
Drain Sewer - Eight Ave. (from C.R.I.&G. RR to Ong St.)
Northwest Second Avenue
RM 1061 - (Currently Under Construction)
S.W. 2nd Ave. (San Jacinto Park) - (8 1/2x14)
Georgia Street North of 26th Avenue - Elmwood Dr.

Storm Sewer As-Built Plans

South Washington St. Lake
Amarillo Creek Storm Sewer from N.W. 12th Ave. to N.W. 24th Ave. (2 sets)
S.W. 46th Ave from Fannin St. to Washington St.
Storm Sewer in Green Haven - Farmers to Gary Ln.
Farmers Ave. from Canyon E-Way to Western (2 sets)
Storm Sewer for Farmers Ave. from I-27 to Star Ln., Star Ln. from Farmers Ave.to - Ex. Paving
Arden Rd. from City Limits West of Bell to Canyon E-Way
Bell St. from I-27 to Hollywood Rd.
Hillside - S.W. 58th Connector from Western to Royce
Hatton from Tulip to Farmers, Catalpa from Canyon E-Way to Western, & Cross - from Canyon E-Way to Ward, Ward from Cross to Hillside & Hillside from Ward - to Western (2 sets)

Reconstruction of S.E. 27th Ave. from Grant St. to Buchanan St.
Bell St. Storm Sewer from Hillside Ave. South 1080'
Storm Sewer Improvement at Terrace & Julie Drives
Teckla Blvd. - 45th to Ridgecrest with Storm Sewer
Terrace Dr. from Overlook to S.W. 40th
Lawrence Lake West Storm Drainage System in Erik Ave. to Bell St.
Bell Street Storm Sewer from Norwich 530'(+/-) South of Estacado Lane
Storm Sewer from Duniven Lake to Overlook - Terrace Dr. Intersection
E. Side Georgia from Alley N. of Mockingbird to S.W. 26th
Coulter Drive from S.W. 34th to S.W. 45th
Extension of Storm Sewer from S.W. 40th & Terrace to Western & Ridgecrest
Structure Details - Ridgecrest Addition Storm Sewer (Hatfield Engineering)
Reroute T-Anchor Lake Storm Sewer Pump Line w/ Pump Line Improvements
Avondale St. from S.W. 16th to S.W. 9th (sheets 1-10) & (sheets 6-10)
S.W. 58th Ave. from Western St. to Washington St. and Georgia St. from S.W. 58th - to Wilbur Dr.
T-Anchor Lake - Storm Sewer & Excavation
S.E. Sixth Ave. from Grand to Bolton
Proposed location of 16" Dia. storm sewer force main - Julian Blvd.
Coulter St. from 660' South of Arden, to S.W. 45th Ave.; and Arden Rd. from Bell - to Coulter
Lawrence Lake: Site Plan, Grading Plan, Storm Sewer Outlet Profiles, & Pump - Relocation Details - Royal Inn Pump
S.W. 15th from Madison - Jefferson Alley to Bryan Street
Lipscomb St. storm sewer from S. 5th to the Alley South of S. 9th
N.W. 1st Ave. from Louisiana St. to Georgia St.
Southeast Third Avenue
Coulter Street from S.W. 9th Avenue to U.S. 66
Star Lane from Hillside Rd. to Approx. 300' N.E. of Farmers
Storm Sewer Improvements - S.W. 57th Avenue & Western Street
East Side of Georgia St. from Wolfin Ave. to Austin St.

Texas Department of Transportation Plans

IH-40 East & West of Loop 335 (Lakeside) - Storm Sewer & Drainage Area
IH-40 Near T-Anchor Lake - Storm Sewer & Drainage Area
IH-40 East & West of T-Anchor Lake - Storm Sewer & Drainage Area
Loop 335 (Lakeside) North & South of IH-40 - Storm Sewer Layout Sheets
10th St. Underpass - Storm Sewer & Drainage Area
Highway 66 & IH-40 - from Helium to Coulter
Highway 66 - Wild Horse Lake - Louisiana, Georgia, & Fannin Sts.
Highway 66 & Avondale
U.S. Hwy 60/66 - Hwy 136 to Eastern St.
U.S. Hwy 60/66 - 9th St. to Western St.
U.S. Hwy 60/66 - Area of Wild Horse Lake
IH-27 & Catalpa - McCarty Lake
IH-27/Hillside - McCarty Lake
IH-40 - Coulter to Bell - Lawrence Lake

IH-40 - Bell to Julian - Lawrence Lake
IH-40 - Julian to Van Buren
IH-40 - T-Anchor Blvd. - T-Anchor Lake
IH-40 - Westgate Mall - culvert, channel, & drainage area info.
Bushland Blvd/Hwy 29 - Amarillo Country Club
Filmore/34th System - Gooch Lake

Analysis and Design Techniques

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U.S. Dept. of Agriculture, Soil Conservation Service, *Urban Hydrology for Small Watersheds*, Technical Release 55, June, 1986.

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1.0 GENERAL

The Amarillo Simulation Analysis of Playa Performance (ASAPP) model runs on a an IBM compatible PC running DOS Version 3.2 or higher. Execution time varies widely depending on the type and speed of the computer's CPU, the disk access speed, and the complexity of the playa simulation. Execution time for a typical playa simulation on computers of comparable hard disk access speeds of less than 12 MS is in excess of 10 minutes on a 386/387-25 MHz based machine and approximately one minute on a 486-33 MHz machine. Execution from a floppy diskette is not recommended due to the excess execution time required. The model is executed from the DOS prompt by entering the command ASAPP. The ASAPP menu screen is divided into three windows. All input takes place in the bottom or green window. The middle or red window contains various options referenced to entries in the green window. The top or blue window never changes.

The program makes use of many data files. These files must be located in the same directory as the executable program. The following is a list of data files and their descriptions. All data files are in text format.

NAMES OF DATA FILES	
File Name	**Description**
ASAPPREC.DAT	Required daily rainfall data file
ASAPEVAP.DAT	Required monthly evaporation data file
PLAYA##.DAT	File containing descriptive data of playa ##
PUMP#.DAT	File containing descriptive data on pump #
PUMP#.CRV	File containing rating curve for pump #
TRANS##.DAT	Default name for a trace file of pump flows
SPILL##.DAT	Default name for a trace file of spill flows
BYPASS##.DAT	Default name for a trace file of bypass flows
TRACE##.DAT	Default name for a trace file of playa levels
REVISED.EAC	Default name for revised playa geometry after capacity modifications
STAT##.out	Default name for file containing statistical summary of simulations

2.0 OVERVIEW OF USER INPUTS

When running a simulation, the user is prompted for a number of input items. Most prompts are supplied with a default response so that the user can progress quickly through the program. The following list describes the input responses in the order that they occur:

1. Playa to simulate by number. A list of playas for which data is available is displayed in the red window.
2. Confirmation of playa by description. User presses ENTER to continue.
3. Choice of existing or ultimate runoff condition. User enters E for existing or U (default) for ultimate.
4. Select control method by number to modify playa characteristics if desired. The default is none. A list of methods is displayed in the red window. These methods are described in Section 3.1. When finished or if no modifications are desired, enter 0 (default) to progress to the simulation prompts.
5. Provide name of file containing import flows. Unless the playa receives pumped, spill, or bypass flows from another playa, the response to this prompt is NONE. Otherwise, the name of the file containing the inflow file is to be supplied here. Use of this feature is described in Section 3.2.
6. Enter the start and stop date to define the simulation period. The default is period of record. If the user requests a period shorter than the period of record, the statistical analysis may not be significant. A warning of this condition will occur in the output.
7. Select option to output a trace during simulation. The default is no trace. If a trace is desired, the user is prompted for the start and stop years. If the trace is to be used as an inflow file for another playa, the full period of record must be used. There are four types of traces: spill, level, pumped flow and bypass flow. All but level traces can be used as inflow files for other playas.
8. Enter the starting water surface elevation for the simulation. The default is the bottom of the playa.

   The program then proceeds with the simulation. The year being simulated is displayed at the bottom of the green window as the simulation progresses.

9. After simulation is complete, the user is prompted to provide the name of the file to contain the simulation results summary. A default name is provided. If a previous file of the same name exists, it will be written over.

Upon completion of the simulation, a summary of playa levels for selected return periods is displayed. This information is also stored in the summary output file. After the results are displayed, the user is returned to the playa selection menu and the program is reset for another simulation. To terminate execution of ASAPP, enter END in lieu of a playa number.

3.0 SPECIAL OPERATING INSTRUCTIONS

3.1 Playa Modifications Options

The data files associated with ASAPP contain default playa characteristics and playa operation data based on site-specific information. Several methods are provided to allow the user to temporarily change these defauts. The user can change the playa capacity, runoff characteristics, pumping operation, imported flows, and drainage bypasses. Each of these changes are accessed from the *Select Control Method* prompt.

Option 1: Change Playa Capacity. This option allows the user to quickly and simply evaluate the effect of excavation from a playa. To use the option, the user must furnish a *no change* elevation. The no change elevation is defined as the level above which there will be no excavation. This level must be chosen high enough to provide some area from which to excavate. The user also must provide the deepest excavation elevation that is to be permitted. The program will not allow excavation below this elevation. This option can be used to avoid excavation below the groundwater table or below some future planned drain elevation. The user next furnishes the desired percent increase in playa storage capacity. This percentage is based on the area between the playa bottom and the no-change elevation. Finally, the user is prompted for the maximum allowable side slope in feet horizontal to one foot vertical. The program will then proceed to recompute the playa area and capacity at multiple levels to achieve the desired percent increase with the imposed constraints. The algorithm attempts to remove material uniformly from the full height of the excavation area until the maximum allowable side slope is reached. If more excavation is needed, the algorithm then begins excavating at the specified side slope until the minimum excavation elevation or the desired capacity is reached. The program will compute the actual percent increase achieved. If the original area and capacity relationships are not in good agreement, the increases achieved vary erratically. Also, if the constraints were too restrictive, the full increase may not be achieved. The method employed is an approximation and the simulation can be sensitive to exactly where materials are removed from a playa; therefore, specific excavation plans should be evaluated by developing elevation and area relationships based on the excavation plan, creating a new PLAYA##.EAC file containing this information, and simulating.

The results of a revision to a playa's capacity are stored in an output file. This file is named REVISED.EAC by default or the user can specify another name. If a previous file exists with the same name, it will be written over. The revised playa capacity remains in effect until the user is returned to the initial playa selection prompt, at which time the capacity is reset to the original data.

<u>Option 2</u>: Change Runoff Characteristics. This option lets the user quickly and temporarily change the runoff curve number for the current playa or the size of the drainage basin without permanently modifying the data files. Original values are reset after the simulation.

<u>Option 3</u>: Change Pumping Operations. This option lets the user operate up to two pumps by specifying the pump capacity and the shutoff elevation. Pump capacity is expressed in gallons per minute at the shutoff elevation. The program will then develop a synthetic rating curve for other elevations based on the flow at the shutoff. The shutoff elevation is

the trigger elevation below which pumping ceases. Note that existing pumps are modeled with pump-specific information. Entering this routine when modeling playas with existing pumps will cause the existing rating curves to be recomputed based on a synthetic rating curve. Results may then differ slightly from runs based on the existing data.

Option 4: Change Imported Flows. This option allows the user to select the name of a file to be used for imported flows. These flows can be spills, pumped flows, or bypass flows from other playas. The flows are added to the playa contents during the simulation. The user is responsible for keeping track of which inflow contributes to which playa and under what development conditions. The user may also specify this file name at a later prompt before execution.

Option 5: Change Drainage Area Bypass. This option allows the user to select runoff from a portion of a playa's contributing basin to be intercepted and diverted away from the playa up to a threshold amount. This is primarily used for modeling playas which have storm sewers in the drainage basin that divert runoff out of the basin. When these sewers become surcharged, the diversion peaks at a threshold flow rate. In order to perform this complex function, the model makes use of a synthetic hydrograph to estimate peak flows and assign time variate volumes.

Option 6: Shell to DOS. This is option is provided for convenience only.

Option 7: Return to Previous Menu. At any time that the *Select Control Method* prompt appears in the green window, this option may be chosen to return to the *Select Playa* prompt. When option 7 is chosen, all values are returned to their default values.

3.2 Linking Playas

Many of the playas in the Amarillo area do not operate independently but rather are linked to other playas. The ASAPP model has provisions to handle such playa systems whether existing or proposed. The ASAPP model can accommodate one input link and one output link per playa. Since the model simulates only one playa at a time, linked systems must be simulated in the order of upstream or contributing playa to downstream or receiving playa. Outflows may consist of pumpage, spills, or storm sewer bypass flows. These flows are written to trace files which can then be read as inputs to receiving playas. See Paragraph 1.0 for a listing of the default names for these files. The user must manage the spill files so that the correct playa and the correct conditions (existing, ultimate, pumps) are read as inputs. For many of the playas for which linked conditions are known to exist, default inflow file names are included in the playas's data file and the user is prompted with them before execution. This alerts the user to a known linked condition; it does not indicate that the file actually exists or was created for the appropriate conditions.

4.0 PLAYA SIMULATION RESULTS

Table A-1, A-2, and A-3, and Figures A-1 and A-2 show the results obtained in running the ASAPP model during the course of the Master Plan work. Table A-1 gives results under existing (1992) watershed and playa conditions. Table A-2 gives results under ultimate watershed but existing playa conditions. Table A-3 gives results for ultimate watershed conditions considering the playa improvements proposed in this Master Plan. Figures A-1 and A-2 present the results of Tables A-1 and A-2 graphically.

TABLE A-1 -- SUMMARY OF PLAYA LEVELS UNDER EXISTING CONDITIONS

Playa Number	Playa Description	Primary Damage El.	Secondary Damage El.	Bottom Elev.	Base Flood El.	Max Level Elev.	100 Year Elev.	50 Year Elev.	25 Year Elev.	10 Year Elev.	5 Year Elev.	2 Year Elev.
1	S.W. 34TH & HELIUM	NONE	NONE	3729.9	3738.5	3738.5	3738.5	3738.5	3738.5	3738.2	3737.4	3735.5
3	S.W. 45TH & SONCY	3723.5	3731.6	3700.0	3710.7	3710.9	3706.9	3708.0	3706.8	3705.3	3704.2	3702.6
4	SOUTH OF HILLSIDE, EAST OF HELIUM	NONE	NONE	3690.0	3699.5	3699.5	3698.3	3697.6	3697.0	3696.0	3695.0	3693.5
5	McDONALD LAKE	3696.8	NONE	3656.0	3690.6	3690.3	3689.1	3688.4	3687.5	3686.2	3685.1	3683.1
6	LAWRENCE LAKE	3624.5	3624.5	3562.0	3629.5	3628.3	3626.7	3625.3	3618.7	3607.5	3598.2	3583.9
7	ARDEN & SONCY	3672.7	3672.9	3667.4	3674.3	3674.4	3673.6	3673.2	3672.7	3672.1	3671.4	3670.2
8	HOLLYWOOD & SONCY	3681.4	3682.1	3670.0	3680.1	3679.8	3678.7	3678.1	3677.4	3676.4	3675.6	3674.1
10	SUNDOWN & COULTER	3667.7	3669.0	3655.0	3666.0	3666.0	3664.7	3664.0	3663.2	3662.1	3661.2	3659.5
11	SUNDOWN & BELL	3661.5	NONE	3636.5	3645.3	3645.1	3644.1	3643.6	3643.0	3642.2	3641.5	3640.3
12	McCORMICK & WESTERN	3629.0	3629.2	3617.0	3625.9	3625.9	3624.8	3624.2	3623.6	3622.6	3621.7	3620.3
13	HOLLYWOOD & WESTERN	3624.2	3625.3	3616.5	3624.7	3624.9	3624.1	3623.6	3623.2	3622.4	3621.6	3620.2
14	DIAMOND HORSESHOE LAKE	3658.1	3658.6	3635.0	3659.9	3660.0	3659.1	3658.2	3657.3	3655.7	3654.3	3652.1
15	McCARTY LAKE	3623.1	3623.3	3615.0	3625.4	3625.9	3624.9	3624.3	3623.6	3622.7	3621.8	3620.0
16	WILLOW GROVE LAKE	3630.4	3630.8	3560.0	3630.7	3632.0	3631.4	3630.9	3630.3	3629.3	3628.0	3621.4
17	BENNETT LAKE	3639.7	3640.0	3605.0	3644.3	3643.2	3642.5	3641.9	3641.1	3638.5	3635.6	3630.7
18	FARMERS & BURLINGTON	3581.8	3582.0	3568.4	3582.6	3582.5	3581.4	3580.8	3580.0	3578.7	3577.5	3575.2
19	SANTA FE LAKE	3639.8	3640.4	3615.0	3636.2	3636.9	3635.9	3635.1	3634.2	3632.6	3631.1	3628.6
20	GOOCH LAKE	3578.5	NONE	3562.4	3578.7	3578.4	3577.2	3576.5	3575.8	3574.6	3573.6	3571.5
21	T-ANCHOR LAKE	3613.6	3613.7	3580.0	3615.6	3615.5	3614.5	3613.9	3613.1	3611.5	3609.3	3602.8
22	S.E. 3RD & EASTERN	3614.2	3615.2	3579.0	3591.0	3591.8	3590.9	3590.2	3589.5	3588.4	3587.4	3585.5
23	WILD HORSE LAKE	3619.2	3619.4	3609.2	3623.0	3617.8	3616.7	3616.0	3615.4	3614.5	3613.8	3612.5
24	MARTIN LAKE	3624.6	3625.0	3588.0	3631.4	3630.7	3629.5	3628.7	3628.0	3626.5	3624.4	3617.7
26	JUETT LAKE	3568.2	3570.3	3539.0	3571.8	3572.0	3571.2	3570.6	3570.0	3569.1	3568.1	3565.9
27	F.M. 1912 & T.S.T.I.	3547.1	3548.0	3530.0	3547.6	3547.6	3546.4	3545.6	3544.7	3543.3	3541.9	3539.4
28	AIRPORT LAKE	3604.0	NONE	3575.6	3589.6	3589.1	3587.9	3587.2	3586.4	3585.2	3584.0	3581.7
29	TRIANGLE & FOLSOM	3591.9	3593.2	3580.0	3591.2	3591.2	3591.2	3591.2	3591.0	3590.2	3589.5	3587.9
30	SOUTH OF SUNDOWN & S.E. OF WESTERN	3634.7	3634.8	3625.5	3631.0	3631.0	3631.0	3631.0	3630.7	3630.2	3629.7	3628.7
33	SOUTH OF ST. FRANCIS & EAST OF GRAND	3563.5	3581.0	3553.5	3564.5	3564.1	3563.0	3562.4	3561.7	3560.7	3559.7	3557.9
34	S.E. 34TH & EAST OF PULLMAN	3550.5	3550.8	3510.0	3552.1	3552.0	3550.9	3550.2	3549.5	3548.3	3547.1	3545.1
35	I-40 & SPUR 228	3549.0	3550.2	3522.0	3550.2	3550.5	3549.4	3548.6	3547.6	3546.0	3544.6	3542.1
36	S.E. 46TH & WHITAKER	3597.8	3599.1	3585.8	3593.5	3593.8	3592.6	3592.0	3591.4	3590.5	3589.7	3588.4
58	NORTH OF U.S. 66 & EAST OF PARSLEY	3576.0	NONE	3562.6	3570.2	3571.0	3569.7	3569.1	3568.4	3567.4	3566.6	3565.3
59	SOUTH OF ST. FRANCIS & WEST OF PARSLEY	3591.0	3591.0	3552.8	3562.2	3562.4	3561.2	3560.5	3559.8	3558.6	3557.6	3556.0
60	SOUTH OF F.M. 2575 & WEST OF F.M. 1812	3567.7	3569.2	3551.5	3556.6	3556.6	3556.6	3556.6	3556.6	3556.6	3556.6	3555.9
61	SOUTH OF AIRPORT BLVD. & EAST OF PULLMAN	NONE	NONE	3584.3	3594.0	3594.0	3594.0	3594.0	3594.0	3594.0	3593.0	3592.2

TABLE A-2 -- SUMMARY OF PLAYA LEVELS UNDER ULTIMATE CONDITIONS

Playa Number	Playa Description	Primary Damage Elev.	Secondary Damage Elev.	Bottom Elev.	Top Elev.	Base Flood Elev.	Max Level Elev.	100 Year Elev.	50 Year Elev.	25 Year Elev.	10 Year Elev.	5 Year Elev.	2 Year Elev.
1	S.W. 34TH & HELIUM	NONE	NONE	3729.9	3738.5	3738.5	3738.5	3738.5	3738.5	3738.5	3738.5	3738.0	3736.5
3	S.W. 45TH & SONCY	3723.5	3731.6	3700.0	3719.0	3710.7	3711.8	3710.0	3709.0	3707.9	3706.2	3704.9	3703.1
4	SOUTH OF HILLSIDE, EAST OF HELIUM	NONE	NONE	3690.0	3702.0	3699.5	3700.0	3698.9	3698.3	3697.6	3696.5	3695.6	3694.0
5	McDONALD LAKE	3696.8	NONE	3656.0	3694.0	3690.6	3690.7	3689.6	3688.9	3688.1	3686.7	3685.6	3683.6
6	LAWRENCE LAKE	3624.5	3624.5	3562.0	3630.0	3629.5	3628.3	3626.7	3625.3	3618.7	3607.5	3598.2	3583.9
7	ARDEN & SONCY	3672.7	3672.9	3667.4	3675.0	3674.3	3674.7	3674.0	3673.5	3673.1	3672.4	3671.8	3670.6
8	HOLLYWOOD & SONCY	3681.4	3682.1	3670.0	3680.0	3680.1	3680.0	3679.1	3678.5	3677.9	3676.9	3676.1	3674.6
10	SUNDOWN & COULTER	3667.7	3669.0	3655.0	3666.0	3666.0	3666.0	3665.2	3664.5	3663.8	3662.7	3661.8	3660.1
11	SUNDOWN & BELL	3661.5	NONE	3636.5	3648.0	3645.3	3645.5	3644.5	3644.0	3643.4	3642.6	3641.9	3640.7
12	McCORMICK & WESTERN	3629.0	3629.2	3617.0	3627.0	3625.9	3626.3	3625.3	3624.7	3624.1	3623.2	3622.3	3620.8
13	HOLLYWOOD & WESTERN	3624.2	3625.3	3616.5	3628.0	3624.7	3625.2	3624.3	3623.9	3623.5	3622.7	3622.0	3620.6
14	DIAMOND HORSESHOE LAKE	3658.1	3658.6	3635.0	3660.0	3659.9	3660.0	3659.4	3658.5	3657.6	3656.1	3654.7	3652.4
15	McCARTY LAKE	3623.1	3623.3	3615.0	3626.0	3625.4	3626.0	3625.0	3624.5	3623.8	3622.9	3622.0	3620.3
16	WILLOW GROVE LAKE	3630.4	3630.8	3560.0	3632.0	3630.7	3632.0	3631.5	3631.2	3630.7	3629.8	3628.7	3624.4
17	BENNETT LAKE	3639.7	3640.0	3605.0	3646.0	3644.3	3643.2	3642.5	3641.9	3641.1	3638.5	3635.6	3630.7
18	FARMERS & BURLINGTON	3581.8	3582.0	3568.4	3584.0	3582.6	3583.2	3582.0	3581.4	3580.7	3579.5	3578.3	3576.1
19	SANTA FE LAKE	3639.8	3640.4	3615.0	3638.0	3636.2	3637.0	3636.1	3635.4	3634.4	3632.9	3631.4	3628.9
20	GOOCH LAKE	3578.5	NONE	3562.4	3580.0	3578.7	3578.8	3577.6	3576.9	3576.2	3575.1	3574.0	3572.0
21	T-ANCHOR LAKE	3613.6	3613.7	3580.0	3616.0	3615.6	3615.5	3614.5	3613.9	3613.1	3611.5	3609.3	3602.8
22	S.E. 3RD & EASTERN	3614.2	3615.2	3579.0	3592.0	3591.0	3591.9	3591.0	3590.3	3589.6	3588.5	3587.5	3585.6
23	WILD HORSE LAKE	3619.2	3619.4	3609.2	3626.0	3623.0	3617.8	3616.7	3616.0	3615.4	3614.5	3613.8	3612.5
24	MARTIN LAKE	3624.6	3625.0	3588.0	3632.0	3631.4	3631.0	3629.7	3629.0	3628.2	3626.8	3625.0	3618.6
26	JUETT LAKE	3568.2	3570.3	3539.0	3572.0	3571.8	3572.0	3571.4	3571.0	3570.4	3569.5	3568.6	3566.7
27	F.M. 1912 & T.S.T.I.	3547.1	3548.0	3530.0	3548.0	3547.6	3547.8	3546.6	3545.9	3545.0	3543.6	3542.2	3539.6
28	AIRPORT LAKE	3604.0	NONE	3575.6	3590.0	3589.6	3589.2	3588.1	3587.4	3586.6	3585.4	3584.2	3582.0
29	TRIANGLE & FOLSOM	3591.9	3593.2	3580.0	3591.2	3591.2	3591.2	3591.2	3591.2	3591.0	3590.2	3589.4	3587.9
30	SOUTH OF SUNDOWN & S.E. OF WESTERN	3634.7	3634.8	3625.5	3631.0	3631.0	3631.0	3631.0	3631.0	3631.0	3630.5	3630.1	3629.1
33	SOUTH OF ST. FRANCIS & EAST OF GRAND	3563.5	3581.0	3553.5	3565.0	3564.5	3564.4	3563.7	3563.0	3562.3	3561.3	3560.3	3558.5
34	S.E. 34TH & EAST OF PULLMAN	3550.5	3550.8	3510.0	3552.0	3552.1	3552.0	3551.4	3550.8	3550.1	3549.1	3548.0	3545.9
35	I-40 & SPUR 228	3549.0	3550.2	3522.0	3552.0	3550.2	3550.9	3550.0	3549.3	3548.4	3547.0	3545.6	3543.0
36	S.E. 46TH & WHITAKER	3597.8	3599.1	3585.8	3596.0	3593.5	3594.3	3593.1	3592.6	3592.0	3591.1	3590.3	3588.9
58	NORTH OF U.S. 66 & EAST OF PARSLEY	3576.0	NONE	3562.6	3572.0	3570.2	3571.4	3570.2	3569.5	3568.9	3567.9	3567.0	3565.6
59	SOUTH OF ST. FRANCIS & WEST OF PARSLEY	3591.0	3591.0	3552.8	3564.0	3562.2	3562.9	3561.8	3561.2	3560.4	3559.3	3558.3	3556.6
60	SOUTH OF F.M. 2575 & WEST OF F.M. 1812	3567.7	3569.2	3551.5	3556.6	3556.6	3556.6	3556.6	3556.6	3556.6	3556.6	3556.6	3556.1
61	SOUTH OF AIRPORT BLVD. & EAST OF PULLMAN	NONE	NONE	3584.3	3594.0	3594.0	3594.0	3594.0	3594.0	3594.0	3594.0	3593.6	3592.5

TABLE A-3 – SUMMARY OF PLAYA LEVELS UNDER ULTIMATE CONDITIONS WITH PROPOSED MASTER PLAN IMPROVEMENTS

Playa Number	Playa Description	Primary Damage Elev.	Secondary Damage Elev.	Bottom Elev.	Base Fld Elev.	ASAAP MODEL RESULTS						
						Max Level Elev.	100 Year Elev.	50 Year Elev.	25 Year Elev.	10 Year Elev.	5 Year Elev.	2 Year Elev.
5	McDONALD LAKE	3696.8	NONE	3656.0	3690.6	3688.3	3687.0	3685.9	3684.6	3682.1	3679.0	3672.5
6	LAWRENCE LAKE	3624.5	3624.5	3562.0	3629.5	3626.3	3624.5	3619.0	3612.8	3603.5	3595.8	3583.5
7	ARDEN & SONCY	3672.7	3672.9	3664.0	3674.3	3673.4	3672.9	3672.5	3672.1	3671.3	3670.7	3669.5
14	DIAMOND HORSESHOE LAKE	3658.1	3658.6	3635.0	3659.9	3659.7	3658.1	3657.1	3655.9	3654.2	3652.8	3650.5
15	McCARTY LAKE	3623.1	3623.3	3614.0	3625.4	3624.1	3623.1	3622.5	3621.7	3620.3	3618.8	3616.7
16	WILLOW GROVE LAKE	3630.4	3630.8	3560.0	3630.7	3630.4	3628.8	3627.4	3624.1	3616.4	3609.2	3596.3
17	BENNETT LAKE	3639.7	3640.0	3605.0	3644.3	3641.5	3639.7	3637.1	3634.3	3630.3	3627.0	3621.2
18	FARMERS & BURLINGTON	3581.8	3582.0	3568.4	3582.6	3582.8	3581.8	3581.2	3580.5	3579.5	3578.4	3576.0
21	T-ANCHOR LAKE	3613.6	3613.7	3580.0	3615.6	3614.2	3613.6	3613.0	3612.1	3610.1	3607.8	3602.1
23	WILD HORSE LAKE	3619.2	3619.4	3609.2	3623.0	3620.0	3619.2	3618.6	3618.0	3617.0	3615.9	3614.2
24	MARTIN LAKE	3624.6	3625.0	3588.0	3631.4	3626.8	3624.5	3621.7	3618.7	3614.4	3610.9	3605.0
26	JUETT LAKE	3571.4	3517.4	3539.0	3571.8	3572.0	3571.4	3571.0	3570.4	3569.5	3568.6	3566.7
34	S.E. 34TH & EAST OF PULLMAN	3550.5	3550.8	3510.0	3552.1	3551.1	3550.5	3550.0	3549.4	3548.3	3547.4	3545.6
35	I-40 AND SPUR 228	3549.0	3550.2	3522.0	3550.2	3550.9	3550.0	3549.3	3548.4	3547.0	3545.6	3543.0

AMARILLO STORM WATER MANAGEMENT MASTER PLAN
ESTIMATED DAMAGE RETURN PERIODS
FOR SPECIFIC PLAYAS UNDER EXISTING CONDITIONS

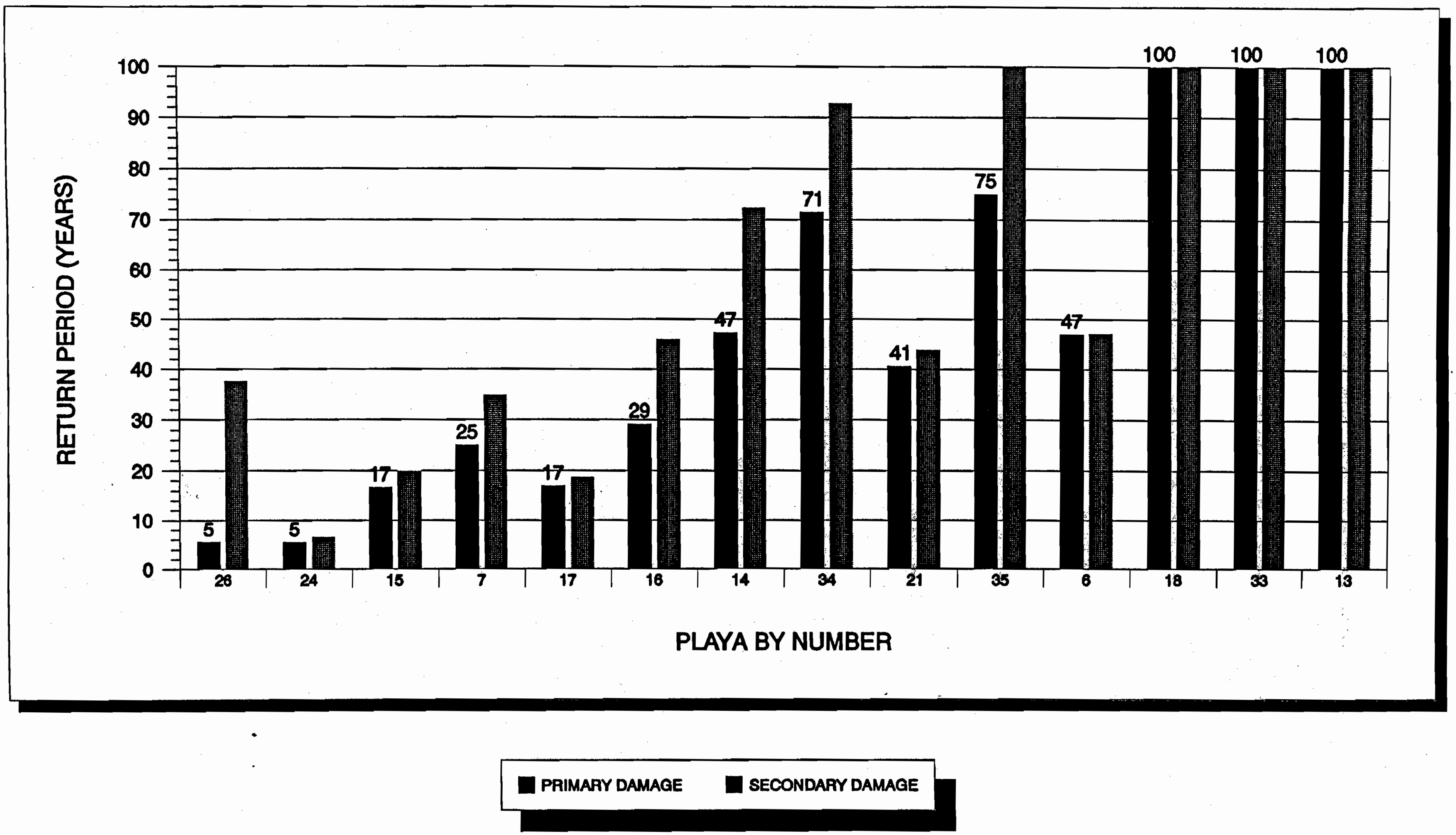

FIGURE A-1

AMARILLO STORM WATER MANAGEMENT MASTER PLAN
ESTIMATED DAMAGE RETURN PERIODS
FOR SPECIFIC PLAYAS UNDER ULTIMATE CONDITIONS

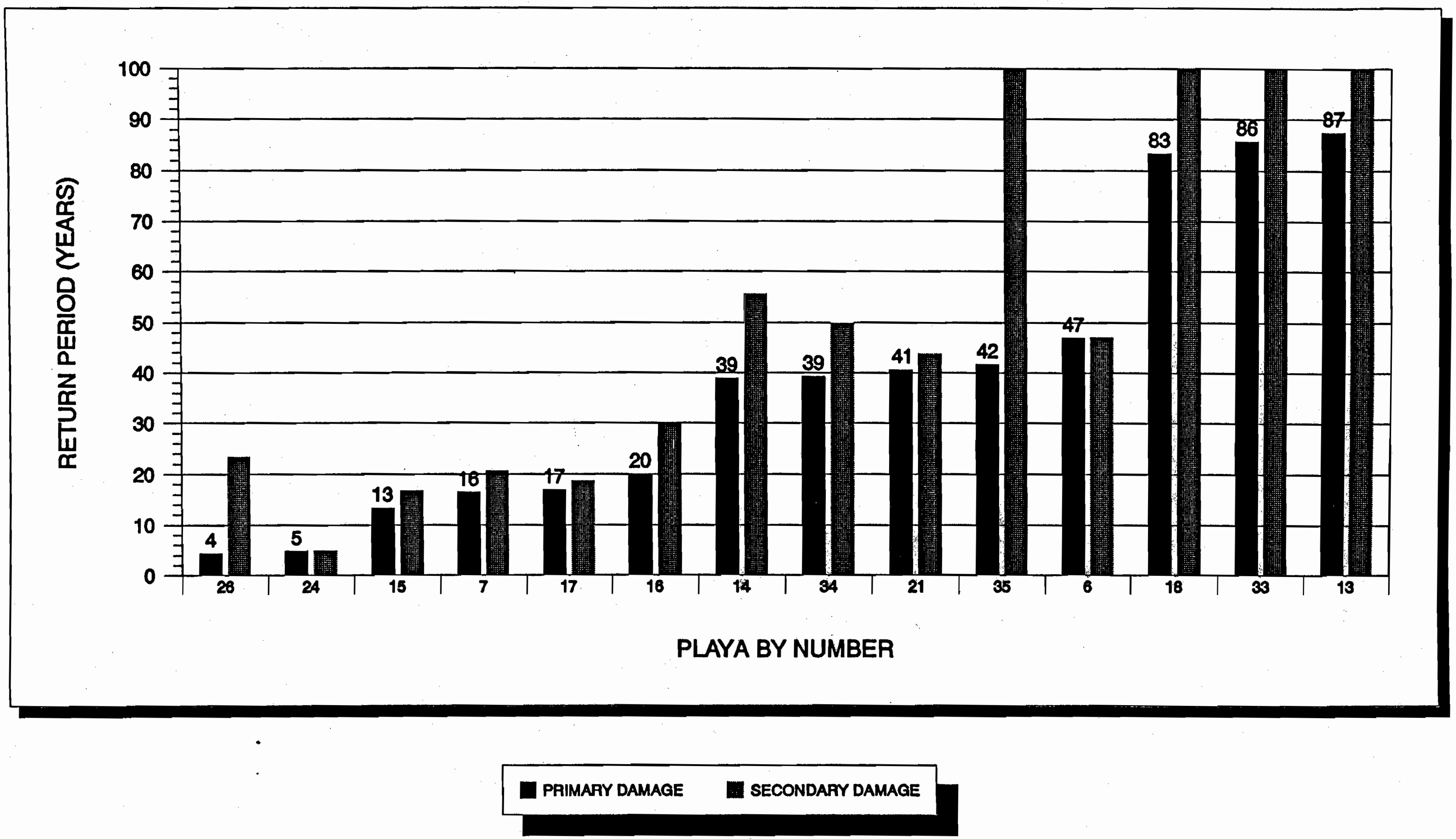

FIGURE A-2

The SCS curve numbers for each playa drainage basin were developed from land use information provided in the *Amarillo Comprehensive Plan.* Seven specific land uses are identified in the plan. An eighth land use, major roadways, was also identifiable. Although the roadways land use was not of interest to the developers of the plan, it is useful for the hydrologic purpose of developing curve numbers. Each of the eight land uses was assigned a curve number corresponding to its runoff characteristics. These curve numbers are shown in Table B-1.

Composite curve numbers for individual playa drainage basins were developed for existing conditions by taking a weighted average of the contributing land use curve numbers. Curve numbers for ultimate conditions were estimated based on full development of the areas identified as undeveloped under existing conditions. The development was assumed to result in the same land uses as projected in the *Amarillo Comprehensive Plan.* The plan suggests that these new land uses will be distributed among six uses as described in Table B-3. These new uses result in a weighted average curve number of 87.3. Ultimate curve numbers were determined assuming that existing land uses, except for undeveloped areas, will not change. The new development land use curve number was substituted for the undeveloped curve number in generating composite ultimate curve numbers. Table B-4 details the breakdown of land uses with each playa basin and the composite curve numbers for existing and ultimate conditions.

While the method described above is appropriate for large scale modeling of run-off volumes, it is not appropriate for the design of hydraulic structures where determination of time variate flows is required. For these analyses, curve numbers should be determined according to the methods described in the *Amarillo Storm Water Criteria Manual.*

Table B-1
Curve Numbers Assigned to Specific Land Uses

Abbreviation	Description	Curve Number
LDR	Low Density Residential	85.0
HDR	High Density Residential	87.0
OFF	Office - Retail	95.0
COM	Commercial	95.0
IND	Industrial	93.0
PSP	Public and Semi-Public	80.0
RD	Roads	98.0
UND	Undeveloped	84.0
DEV	New Development Composite	87.3

Table B-2
New Development Composite Curve Estimate

New Land Use	Percent of New Development	Curve Number
Low Density Residential	27%	85
High Density Residential	3%	87
Commercial	13.3%	95
Industrial	5%	93
Public and Semi-Public	23.3%	80
Right-of-Way	28.3%	91
New Development Composite	100%	87.3

Table B-3
Land Uses and Playa Basin Curve Numbers

Playa #	Land Use								Drainage Basin (acres)	Curve Numbers	
	LDR	HDR	OFF	COM	IND	PSP	UND/ DEV	RD		Existing	Ultimate
1							100%		918	84.0	87.3
3							100%		1058	84.0	87.3
4							100%		1801	84.0	87.3
5	16%	2%	5%			14%	64%		1539	84.2	86.3
6	61%	4%	13%	14%		8%			5838	87.3	87.3
7	4%					2%	94%		1631	84.0	87.1
8						10%	90%		1224	83.6	86.6
10							100%		1567	84.0	87.3
11						6%	94%		1875	83.8	86.9
12							100%		1156	84.0	87.3
13	11%				9%	3%	78%		1833	84.8	87.3
14	35%	6%	2%	19%			29%	9%	1102	88.0	89.0
15	46%	1%	3%	7%		4%	40%		3651	85.3	86.6
16	44%		1%		9%	1%	44%		1245	85.3	86.8
17	44%	5%	2%	47%		2%			891	90.0	90.0
18					9%		91%		4235	84.8	87.8
19	27%			1%	54%	1%	17%		712	89.2	89.8
20	13%	5%	2%	3%	4%	14%	58%	1%	7197	84.7	86.6
21	27%	7%	4%	26%	25%	5%		5%	2612	90.6	90.6
22	21%	1%	0%	2%	63%	3%	9%		5135	90.1	90.4
23	19%	23%		21%	34%	3%			2330	90.1	90.1
24	23%	17%	1%	15%	13%	5%	25%		1658	87.4	88.3
26				33%	4%		62%		3246	88.1	90.1
27		10%			40%	33%	17%		2773	86.6	87.2
28					14%	60%	26%		2264	82.8	83.7
29					100%				371	93.0	93.0
30							100%		310	84.0	87.3
33	2%				4%	1%	94%		6539	84.3	87.4
34				3%			97%		2711	84.3	87.5
35					14%		86%		1658	85.3	88.1
36							100%		766	84.0	87.3
58					20%		80%		733	85.8	88.4
59							100%		970	84.0	87.3
60					34%	17%	50%		964	86.4	88.0
61					38%		62%		753	87.4	89.5

TABLE B-4
LAND USES AND PLAYA BASIN CURVE NUMBERS

PLAYA	DESCRIPTION	EXISTING CURVE #	ULTIMATE CURVE #
1	PLAYA AT S.W. 34TH & HELIUM	84.0	87.3
3	PLAYA AT S.W. 45TH & SONCY	84.0	87.3
4	PLAYA SOUTH OF HILLSIDE AND EAST OF HELIUM	84.0	87.3
5	McDONALD LAKE PLAYA	84.2	86.3
6	LAWRENCE LAKE PLAYA	87.3	87.3
7	PLAYA AT ARDEN & SONCY	84.0	87.1
8	PLAYA AT HOLLYWOOD & SONCY	83.6	86.6
10	PLAYA AT SUNDOWN & COULTER	84.0	87.3
11	PLAYA AT SUNDOWN & BELL	83.8	86.9
12	PLAYA AT McCORMICK & WESTERN	84.0	87.3
13	PLAYA AT HOLLYWOOD & WESTERN	84.8	87.3
14	DIAMOND HORSESHOE LAKE PLAYA	88.0	89.0
15	McCARTY LAKE PLAYA	85.3	86.6
16	WILLOW GROVE LAKE PLAYA	85.3	86.8
17	BENNETT LAKE PLAYA	90.0	90.0
18	PLAYA AT FARMERS & BURLINGTON	84.8	87.8
19	SANTA FE LAKE PLAYA	89.2	89.8
20	GOOCH LAKE PLAYA	84.7	86.6
21	T-ANCHOR LAKE PLAYA	90.6	90.6
22	PLAYA AT S.E. 3RD & EASTERN	90.1	90.4
23	WILD HORSE LAKE PLAYA	90.1	90.1
24	MARTIN LAKE PLAYA	87.4	88.3
26	JUETT LAKE PLAYA	88.1	90.1
27	PLAYA AT F.M. 1912 & T.S.T.I.	86.6	87.2
28	AIRPORT LAKE PLAYA	82.8	83.7
29	PLAYA AT TRIANGLE & FOLSOM	93.0	93.0
30	PLAYA SOUTH OF SUNDOWN & S.E. OF WESTERN	84.0	87.3
33	PLAYA 1 MILE SOUTH OF ST. FRANCIS & EAST OF GRAND	84.3	87.4
34	PLAYA AT S.E. 34TH & EAST OF PULLMAN	84.3	87.5
35	PLAYA AT I-40 & SPUR 228	85.3	88.1
36	PLAYA AT S.E. 46TH & WHITAKER	84.0	87.3
58	PLAYA NORTH OF U.S. 66 & EAST OF PARSLEY	85.8	88.4
59	PLAYA SOUTH OF ST. FRANCIS & WEST OF PARSLEY	84.0	87.3
60	PLAYA SOUTH OF F.M. 2575 & WEST OF F.M. 1812	86.4	88.0
61	PLAYA 0.7 MILES SOUTH OF AIRPORT BLVD. & 1 MILE EAST OF PULLMAN	87.4	89.5