FLOOD PROTECTION STUDY

CITY OF ORANGE, TEXAS

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Carter :: Burgess

Consultants in Engineering, Architecture,
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April 18, 1994

Mr. James Foyle
City of Orange
402 South 10th Street
Orange, TX 77630

Reference: Flood Protection Study Final Report Submittal

Dear Mr. Foyle:

We are hereby submitting to the City of Orange 24 copies of the Final Report of the Flood Protection Study as outlined in Section II, Task IV, Part C, of our Professional Services Agreement dated November 2, 1992.

Today's final report submittal includes 24 copies of the report including the executive summary and the proposed Drainage Design Manual for the City of Orange, and four copies of the Appendix bound under separate cover. The Appendix is categorized by watershed and contains detailed computer printouts of all the hydraulic analysis and plots of the cross sections analyzed.

It has been a pleasure to serve you in the preparation of this report. Please call me if you have any questions regarding this submittal.

Sincerely,

CARTER & BURGESS, INC.

Albert C. Petrasek, Jr., Ph.D., P.E.
Associate

cc: Mr. Bob Wear - TWDB
ACP/cdl
Enclosure

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EXECUTIVE SUMMARY

This flood prevention study is the result of an agreement signed on March 12, 1992, between the City of Orange, Texas and the Texas Water Development Board. The agreement provided funding for a flood study that would incorporate major portions of Orange County. On November 2, 1992, the City of Orange entered into an agreement with Carter & Burgess, Inc., to obtain professional engineering services for the study. The following report is the culmination of that effort.

This report consists of seven major sections, namely; the Introduction, Drainage Criteria and Methodology Review, Watershed Analysis, Conclusions, Recommendations, Local Project Ranking System and Capital Improvements Program, and Additional Concerns. The Introduction section gives a brief review of the history of the City of Orange, its flooding, and previous flood studies. The Drainage Criteria and Methodology Review section discusses the methodology used for hydrologic and hydraulic analysis in this report. The Watershed Analysis section provides a description of each watershed and details the steps taken for each analysis. Note that in this section and in those following, the approach is from large to small. The large watersheds susceptible to wide-scale flooding are analyzed first then the smaller local watersheds. The Conclusions section discusses the results of each analysis. The Recommendations section presents proposed solutions with estimated costs for various flood protection projects. The Local Project Ranking System and Capital Improvements Program section presents a project priority ranking system and a yearly budgeting program to maintain steady progress for implementing the recommendations in the study. The last section, Additional Concerns, discusses other various pertinent topics as outlined in the contract.

SUMMARY OF CONCLUSIONS

Flooding in the City of Orange can originate from a wide variety of sources. The sources can be divided into two main categories. The first category includes sources that can result in wide-scale flooding. These sources include the Sabine River, the hurricane surge and the large bayous that flow through the city. The second category includes sources that can produce flooding in more localized or smaller sub-watershed areas. These sources include undersized storm drain pipes, drainage swales and inlets in various areas throughout the city. The following summary of conclusions reached in this study discusses these two categories independently.

Wide-Scale Flooding

The City of Orange is susceptible to wide-scale flooding from the Sabine River and from hurricane surge that originates in the Gulf of Mexico. Either of these conditions can inundate a majority of the central city with several feet of water. Certain areas of the city are also susceptible to inundation from the four large bayous that flow through the area. Flood flows conveyed in these bayous, Adams Bayou, Coopers Gully, Hudson Gully and Little Cypress Bayou, can exceed the banks of the bayou and cause flood damage to urbanized areas of the city. This flooding can occur somewhat independently of peak flood flows from either the Sabine River or the hurricane surge.

The analyses conducted in the course of this study evaluated measures to prevent flooding from occurring as a result of either Sabine River flood flows, hurricane surge or bayou flooding from the four previously-named bayous.

In analyzing flooding from the Sabine River and the hurricane surge, the engineers concluded that levee protection systems would be required to protect the vulnerable areas of the city from inundation. The Sabine River and the hurricane surge both can flood the city from the east and inundate large urbanized areas. The engineers investigated seven levee protection alternatives that could be constructed to protect various areas of the city. The primary criteria for evaluating the levee protection alternatives was the protection afforded by the alternative and the cost of construction. The seven alternatives were narrowed to three alternatives that will be presented in this report.

In analyzing flooding from the four bayous mentioned earlier, the engineers identified various combinations of channel improvements, bridge improvements and diversions that could be constructed to prevent flooding from these sources. These evaluations only considered flooding from the bayous themselves, and did not superimpose flooding effects from the Sabine River or the hurricane surge. This means that even if the improvements were constructed on the bayous, the same areas of the city susceptible to flooding from the Sabine River and the hurricane surge would still be vulnerable.

Localized Flooding

Localized flooding concerns in the city were most often caused by drainage structures that were not able to convey runoff from more frequently occurring rainfall events. The result of this being that local sub-watershed areas experience street flooding, yard flooding and possibly water inside homes and businesses fairly frequently.

In the course of the analysis of these local flooding concerns, the engineers determined that most often the drainage structure, whether it be a storm water pump station, storm drain pipe, small drainage swale or set of curb inlets, could convey runoff from less than the one or two year return frequency storm. Therefore the drainage problem was experienced on a rather frequent basis.

The City of Orange indicated that there were seven areas in the city that experienced this type of localized flooding on a regular basis. These seven areas are the Cherry Ave. and 13th St. sub-watershed area, the Coopers Gully pump station area, the upper end of the Dayton Street ditch at Bluebonnet Drive, a sub-watershed of Hudson Gully, the North Simmons Drive area, the Old Town area, and an area near Sunset Drive.

To remedy drainage concerns from most of these areas, larger drainage pipes should be installed along with more curb inlets to convey more runoff away from the street. An intensity-duration-frequency curve was recommended for the City of Orange so that the Rational Method could be used to determine the peak design flow for the proposed pipes. In determining the required pipe sizes, the engineers used a storm frequency of five years. This design procedure increased the runoff carrying capacity of the pipes from their current one to two year design level to a five year design level.

The engineers proposed pipe size enlargements for five of the seven localized flooding study areas mentioned above. These included the Cherry Ave. area, the Dayton Street ditch area, Hudson Gully, the Old Town area, and the Sunset Drive area.

Drainage swale/ditch improvements were proposed for Hudson and Coopers Gully and the upstream end of Dayton Street ditch. Both of these proposed improvements will allow the proposed larger storm drain pipes to drain the street areas more efficiently during a five year storm.

Finally, the engineers concluded that pumping capacity improvements at the Coopers Gully pump station are required to bring the station up to a 100-year capacity. Also, the installation of flap gates along north Simmons Drive at Little Cypress Bayou will prevent flood flows from the Bayou from backing up through culverts under Simmons Drive and into residential areas on the west side of Simmons. The installation of these flap gates will provide flood protection only until downstream flood levels on either Little Cypress or the Sabine River exceed the top of road elevation of Simmons Drive. Then the flood waters will over top Simmons Drive from the east and begin to inundate larger areas.

SUMMARY OF RECOMMENDATIONS

The following paragraphs present the items recommended in this study to prevent flooding in the Orange area. The recommendations will be presented for the wide-scale flooding concerns first and then the localized-area flooding concerns.

Wide-Scale Flooding

The items recommended in this in this report to prevent wide-scale flooding in the Orange area are listed below along with an estimate of probable cost to implement the improvement.

Levee Alternative No. 1	$ 42,225,000
Levee Alternative No. 2	$ 62,015,000
Levee Alternative No. 3	$ 95,040,000
Adams Bayou Dredging	$ 9,700,000
Adams Bayou Diversion	$ 7,600,000

Localized Flooding

The items recommended in this report to prevent localized-area flooding as described earlier are listed below along with an estimate of probable cost to construct each improvement.

Bluebonnet Drive Improvements	$ 74,000
Channel Improvements @ Hwy. 87	$ 82,000
Flap Gates @ Simmons Drive	$ 68,000
Upgrade Coopers Pump Station	$ 630,000
Additional Bluebonnet Line	$ 40,000
Line Segment CH2	$ 225,000
Dayton Ditch Downstream Culverts	$ 24,000
Line Segment SD1	$ 18,000
Line Segment SD3	$ 16,000
Line Segment HG6	$ 55,000
Line Segment HG5	$ 40,000
Line Segment HG7	$ 13,000
Line Segment OT1	$ 49,000
Line Segment HG1	$ 195,000
Line Segment HG8	$ 319,000
Line Segement CH1	$ 531,000
Line Segment OT2	$ 142,000
Line Segment OT3	$ 133,000
Line Segment OT4	$ 124,000
Line Segment HG4	$ 286,000
Line Segment SS1	$ 553,000
Hudson Gully Channel Improvement	$1,200,000
Coopers Gully channel Improvement	$ 975,000
SUBTOTAL	**$ 5,792,000**
ENGINEERING AND CONTINGENCIES (30%)	**$ 1,738,000**
TOTAL ESTIMATED CONSTRUCTION COST	**$ 7,530,000**

CONSTRUCTION PRIORITY AND CAPITAL IMPROVEMENTS PROGRAM

The items recommended for improvement in the localized flooding section of this report are items that the City could begin constructing immediately under the current capital improvement program. The projects in this section have been prioritized according to a system that considered how the flooding situation affected traffic, number of citizens, public safety and social need. The system also considered construction time and whether the construction could be accomplished as a stand-alone project or as part of a multi-phase project.

Once the projects were prioritized, the projects were grouped according to a proposed capital improvements project budget of approximately $200,000 per year.

A section on funding outlines how the city may be able to allocate or provide the required $200,000 annually for the implementation of the localized flood protection projects.

ADDITIONAL ITEMS

To prevent future urbanization from causing localized flooding problems in the Orange area, the City should implement the use of a drainage design manual. The manual would provide guidelines for designing drainage structures such as pipes, ditches, and storm sewer inlets. A proposed drainage design manual is included as an appendix to this study to aid the City in this endeavor.

The City should also approve and require the use of the drainage design manual by enacting an ordinance stating that purpose. The text for a proposed ordinance to accomplish the manual's approval is included.

In summary, if the City requires the use of the drainage design manual for future drainage projects and implements localized-flooding improvements on a consistent annual basis, the City will see positive results in reducing and preventing localized flooding problems.

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I. INTRODUCTION

A. CITY HISTORY

The City of Orange is located on the west bank of the Sabine River in the extreme southeast corner of Texas. The city has a long and colorful history that reaches back to its first American-Indian inhabitants, and includes the Spanish, the French, and the early Texas pioneers. The area's first permanent settlers were drawn to the vast natural resources of the land that include timber, fertile soil, a mild climate, and the navigable Sabine River. The area's permanent population began to grow and eventually the City of Orange was incorporated in 1858. By the end of the 19th century, Orange had become established as a port for the cotton trade and, with the railroad, as a valuable link between the eastern and western portions of the United States. In 1914, the Army Corps of Engineers dredged the harbor of Orange so that shipyards could be built to aid the nation's efforts during World War I. The operation was a great improvement to the existing water transportation facilities and resulted in bringing prosperity and a population increase to the city. At the onset of World War II, the United States Navy built a base at Orange which again resulted in a great increase of the city's population. A local industrial boom accompanied the naval base and brought with it rapid growth and development throughout the city. Much of the industrial growth centered itself just south of the city limits along a stretch of the river now known as "chemical row." Ship building, petroleum refining and paint manufacturing became the area's dominant industries. After World War II the city's growth leveled off and the Navy's need for a base declined. Eventually, the naval station was moth-balled in 1965. The base closing resulted in a population decrease and removed an important part of the area's economy.

The City of Orange has sought to preserve its rich heritage through the dedication of many historic sites and homes throughout the city. Today, the City of Orange is still an economically and aesthetically attractive city that offers an established industrial base, many natural resources, a mild climate, and an extensive transportation network that includes rail lines, an interstate highway, and a deep water port. The City of Orange's 1990 population was 19,381 according to the latest Bureau of the Census report. The County of Orange's 1990 population was 80,509 according to the same report. (See Reference 1)

B. FLOODING HISTORY

Ironically, water, which is one of Orange County's greatest assets by providing transportation and irrigation, is also one of its greatest liabilities. The City of Orange lies on relatively flat, low-lying ground adjacent to the Sabine River and is located only a few miles inland from the Gulf of Mexico. Large bayous and several gullies also pass through the City on their way to the Sabine River. This proximity to so many water ways makes the City prone to flooding. The City is susceptible to flooding from both wide-scale storm events such as a rising Sabine River or a hurricane in the Gulf, and from localized rain storm runoff. Localized flooding occurs quite frequently as the region's average annual rainfall is fifty-seven inches.

Several floods of significant magnitude have occurred and been documented in Orange. Documentation is based on eye-witness accounts and on river measurements made from the Sabine River Authority staff gauge in Orange and the U.S. Geological Survey's stream gaging station at Rutliff, about 30 miles upstream of Orange. The study released by the Army Corps of Engineers in 1968 lists the ten highest gauge heights on the Sabine River at Orange to that date. (See Reference 2) The gauge heights are included as Table I.1 below.

TABLE I.1 Ten Highest Sabine River Gauge Readings at Orange Through 1968

Rank	Date of Crest	Gauge Height *(MSL)
1	April 25, 1913	6.6
2	September 12, 1961	6.6
3	August 24, 1915	6.1
4	May 24, 1953	6.0
5	August 18, 1915	5.9
6	May 3, 1914	5.8
7	April 17, 1923	5.8
8	December 25, 1923	5.7
9	June 10, 1950	5.4
10	June 1, 1914	5.2

* USC&GS MSL Datum of 1929

Table I.1's highest gauge reading of 6.6 in 1961 was the result of tides caused by Hurricane Carla. The maximum discharge recorded on the Sabine to date is 121,000 cfs and occurred on May 24, 1953. Both of these storms caused extensive flooding throughout Orange. In September of 1963, another hurricane, Hurricane Cindy, struck the Texas coast. This storm was accompanied by very heavy rainfall in the Beaumont - Port Arthur - Orange area, and resulted in extensive local flooding. Total rainfall accumulations caused by Cindy were 22.8 inches at Orange. There is little definite information on past flood flows of Adams Bayou. However, one large flood of Adams Bayou was recorded in September 1958. Rainfall for this flood averaged about 10.5 inches over the watershed and produced a peak discharge of about 7,500 cfs.

C. APPLICABLE FLOOD STUDIES

Several existing flood studies were consulted for this report. A brief description of each and its relation to the City of Orange follows.

1. *Report on a Comprehensive Drainage Plan for the City of Orange, Texas and Metropolitan Area* - by George J. Schaumburg Consulting Engineers, November 1958

This study was commissioned by the City of Orange to help solve its drainage problems. The study set forth design criteria and presented preliminary designs and cost data for improvements in the Adams Bayou, Coopers Gully, Little Cypress Bayou and Sabine River watersheds. The study was very thorough and serves as the model for this report.

2. *Drainage Master Plan Northwest Area City of Orange, Texas* - by Gary Grahm of Bob Shaw Consulting Engineers, Port Arthur, Texas, March 1980

This study was commissioned by the City of Orange to serve as a flexible guide to direct construction of and improvements to drainage systems within the northwest portion of the city, especially as development occurs. The study was written to be an addendum to and compatible with the Schaumburg report of 1958.

3. *Flood Plain Information Sabine River and Adams Bayou Orange, Texas Area* - by U.S. Army Corps of Engineers, Galveston District, July 1968

This study was requested by the City of Orange, commissioned by the Army Corps of Engineers, and prepared by Turner, Collie & Braden, Inc. Consulting Engineers of Houston, Texas. The study brought together a record of the largest known floods on the Sabine River and calculated and mapped the probable extent of future flooding in the vicinity of Orange due to the Standard Project Flood.

4. *Stage 2 Documentation Report Lower Sabine River Basin, Texas and Louisiana* - by U.S. Army Corps of Engineers, Galveston and Fort Worth Districts, September 1979

This study was commissioned in 1974 by two resolutions of Congress and was directed by the Corps of Engineers. The purpose was to present findings of investigations concerning water resource problems and needs in the lower Sabine River Basin. The lower basin was defined as the area from Toledo Bend Dam to Sabine Lake. One of the study's findings was that all of the existing flood control works in the study area are located in Orange County. Those existing works consist of a locally owned levee along Little Cypress Bayou and a small levee and floodwall which protect the former U.S. Naval Base at Orange. The study concluded that the levees "provide only minimal protection from hurricane flooding." The study proposed a combination of larger earthen levees and concrete floodwalls to provide adequate protection from hurricanes.

5. *Flood Insurance Study - City of Orange, Texas* - by Federal Emergency Management Agency, July 6, 1982

This study was authorized by the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. It was performed by Tetra Tech, Inc. in 1980 and released by FEMA in 1982. The purpose of the study was to convert the City of Orange to the regular program of flood insurance administered by FEMA and to assist local and regional planners in sound flood plain management. The report illustrates flood profiles for the 10, 50, 100, and 500 year storms and maps the flood plain throughout the city.

6. *Sabine River Flood Study* - by Brown & Root, Inc., January 1993

This study was commissioned by the Texas Water Development Board and was performed by Brown and Root, Inc. The purpose was to present findings of institutional and hydraulic issues associated with flooding of the Sabine River. Peak flood flows were predicted for the lower Sabine River north of Orange County. The predicted flood elevations were lower than those of the previous FEMA studies for the City of Orange. The lower predicted flood elevations result from the lower flow values used in the study. The lower flow values were due to a change in statistical probability methods used to analyze coincident hurricane surge values and Sabine River flooding.

II. DRAINAGE CRITERIA AND METHODOLOGY REVIEW

Drainage criteria are those design factors that influence the level of flood protection a particular community will possess. Methodology is the means by which drainage criteria and pertinent data are analyzed to derive meaningful answers to flooding questions. This section of the report reviews the methods and criteria used for the flood control analysis performed in this study. (See References 3,4, and 5) The section is divided into three sub-sections; hydrology, hydraulics, and storm drain analysis.

A. HYDROLOGY

The planning, design and construction of drainage facilities are based on the study of hydrology and its use to determine accurate predictions of storm runoff over a particular watershed. The best data source from which to base the design of storm drainage and flood control systems is, of course, continuous long-term records of rainfall and resulting storm runoff . Unfortunately though, it is not often possible to obtain such records in sufficient quantities as weather records do not often date back very far and land development alters the runoff volumes produced by similar storms. Therefore, the accepted practice that is used most often today is to relate storm runoff to the amount of rainfall over a particular watershed along with different parameters of the watershed. This relation provides a means of estimating the rates, timing and volume of runoff expected from watersheds at various rainfall recurrence intervals.

This sub-section discusses the two methods of hydrology used for analysis in this study. The first is the Rational Equation which applies to smaller drainage areas of usually less than 200 acres and the second is the U.S. Army Corps of Engineers' HEC-1 Flood Hydrograph Package computer program which applies to larger drainage areas of usually greater than 200 acres.

1. The Rational Equation

Most communities today use the widely known and accepted Rational Equation to calculate the amount of storm water runoff a watershed of 200 acres or less will generate. The Rational Equation is stated as follows:

$$Q = C\,I\,A$$

where **Q** = the amount of runoff in cubic feet per second (CFS)
C = the runoff coefficient or "C-factor"
I = the rainfall intensity in inches per hour
A = the watershed area in acres.

Each community determines its own level of flood protection based on the values it adopts for both **C** and **I** in the equation. The area, **A**, is a constant for each watershed and is therefore not subject to adaptation.

The "C-factor" is a runoff coefficient that varies according to soil type and land use. It is generally accepted that altering land use through urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization alters the hydrology of a watershed by improving its hydraulic efficiency, reducing its surface infiltration and reducing its storage capacity. The more impervious and densely developed an area, the higher the value of the C-factor attributed to it and thus the higher the calculated runoff. For example, undeveloped farm land may use a C of 0.3 while a paved parking lot may use a C of 0.9. The C-factor can therefore have a great impact on the quantity of runoff calculated. For this reason, the C-factor for design of urbanized storm drain systems should always be chosen assuming a fully developed watershed. This helps to prevent future flooding due to increased development. Zoning ordinances and zoning maps can aid in determining what the fully developed C-factor will be.

C-factors are generally derived through experimentation and, over time, fairly standard values have emerged. Most values are widely accepted and generally correspond well, especially among adjacent communities as the topography and soil conditions are often quite similar. Three southeast Texas counties located near Orange County, namely; Brazoria, Fort Bend, and Montgomery, have all adopted the same C-factor values. These values are presented in Table II.1 entitled "Rational Method Runoff Coefficients for 5-10 Year Frequency Storms in Brazoria, Fort Bend, and Montgomery Counties, Texas." The table is very thorough and was used as a resource in this study. It is recommended that the City of Orange adopt and use these values. A complete drainage design manual has been recommended and is discussed later in this report in Part F of Section VII entitled "Additional Concerns".

TABLE II.1 Rational Method Runoff Coefficients for 5-10 Year Frequency Storms in Brazoria, Fort Bend, and Montgomery Counties, Texas

Description of Area	**Basin Slope < 1%**	**Basin Slope 1-3.5%**	**Basin Slope 3.5-5%**
Single Family Residential Districts			
Lots greater than 1/2 acre	0.30	0.35	0.40
Lots 1/4 - 1/2 acre	0.40	0.45	0.50
Lots less than 1/4 acre	0.50	0.55	0.60
Multi-Family Residential Districts	0.60	0.65	0.70
Apartment Dwelling Areas	0.75	0.80	0.85
Business Districts			
Downtown	0.85	0.87	0.90
Neighborhood	0.75	0.80	0.85
Industrial Districts			
Light	0.50	0.65	0.80
Heavy	0.60	0.75	0.90
Railroad Yard Areas	0.20	0.30	0.40
Cemeteries	0.10	0.18	0.25
Playgrounds	0.20	0.28	0.35
Streets			
Asphalt	0.80	0.80	0.80
Concrete	0.85	0.85	0.85
Concrete Drives and Walks	0.85	0.85	0.85
Roofs	0.85	0.85	0.85
Lawn Areas			
Sandy Soil	0.05	0.08	0.12
Clay Soil	0.15	0.18	0.22
Woodlands			
Sandy Soil	0.15	0.18	0.25
Clay Soil	0.18	0.20	0.30
Pasture			
Sandy Soil	0.25	0.35	0.40
Clay Soil	0.30	0.40	0.50
Cultivated			
Sandy Soil	0.30	0.55	0.70
Clay Soil	0.35	0.60	0.80

As a comparison, the values that Schaumburg assigned to **C** in the 1958 report on a comprehensive drainage plan for the City of Orange (See Reference 6) are as follows:

C = 0.6 for Commercial Areas
C = 0.4 for Residential Areas

These values correspond well with those listed for similar categories in Table II.1. The "Basin Slope < 1%" column applies to the City of Orange in most cases because of its low sloped terrain.

The "rainfall intensity", **I**, is the average rainfall rate in inches-per-hour over a watershed. The value is based on the chosen storm frequency of occurrence and a rainfall duration equal to the "time of concentration." The storm frequency is a statistical variable based on the probable return interval in years of a particular size storm. For instance, a 100-year storm frequency has a 1/100th (or one percent) chance of occurring in any given year while a 5-year storm has a 1/5th (or 20 percent) chance of occurring in any given year, based on past records of rainfall. The "time of concentration", **Tc**, is the time required for runoff to travel from the most distant part, hydraulically, of the watershed to any point of interest along a drainage route. Runoff reaches its maximum value when the time of concentration has been reached at a particular point since at this time all portions of the watershed are contributing runoff to that point.

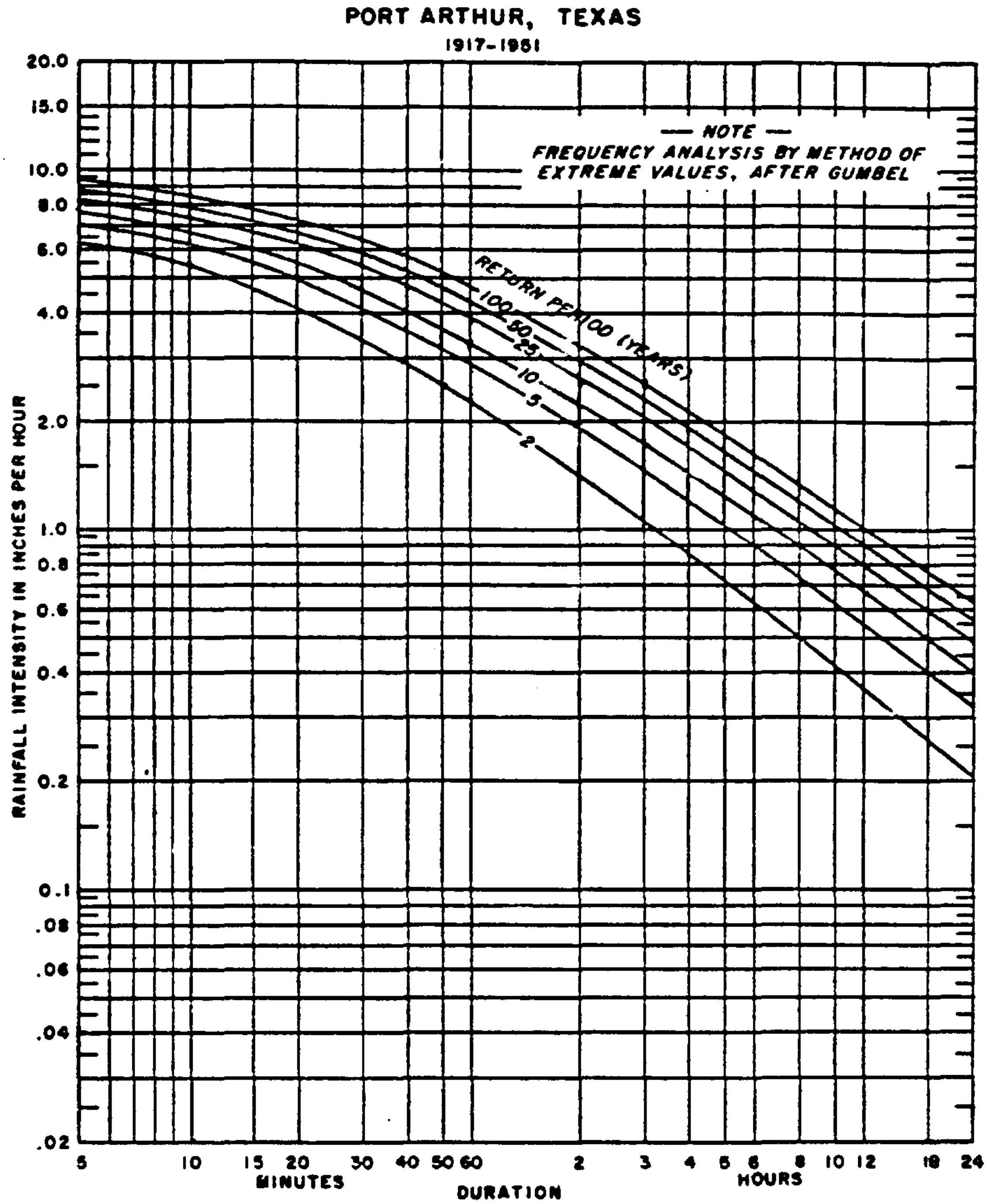
The time of concentration **Tc** and intensity value **I** have an inverse relationship for any given watershed. As the time of concentration decreases, or as runoff reaches an inlet faster, rainfall intensities increase. Rainfall intensities increase due to the natural phenomenon that very intense rainfalls last only a short amount of time while less intense rainfalls can last much longer. For instance, a heavy down pour of rain may last only about 15 minutes while a light steady drizzle may last for hours or even days. To drain effectively, storm drains need to carry just enough capacity at each point to drain the rainfall intensity that corresponds to the time of concentration for that drain at that point. For example, if it takes 10 minutes for runoff to reach an inlet, then that portion of the drain needs to be designed for a rainfall intensity that corresponds to a 10 minute time of concentration. A one-hour trip needs to be designed for a smaller intensity that corresponds to a one-hour time of concentration.

Once the time of concentration is known, the corresponding rainfall intensity, I, may be determined from rainfall intensity-duration-frequency curves. These curves graphically relate rainfall durations (the time of concentration) to rainfall intensities in inches-per-hour over a particular region. The rainfall intensity-duration-frequency curves presented in the Schaumburg report are based on the U.S. Department of Commerce Weather Bureau bulletin released in 1955 entitled "Technical Paper No. 25." Comparing these curves to a more recent study released by the National Oceanic and Atmospheric Administration in 1977 shows very little change. Therefore the Schaumburg report's curves were used for the analysis in this study. The curves have been included as Exhibit II.1 entitled "Rainfall Intensity Curves."

As previously stated, the City of Orange does not have a written drainage design manual for determining proper values for I but has adopted the values recommended by the Schaumburg report of 1958. The Schaumburg report recommended using an I value of 2.9 inches-per-hour throughout the city. This value was based on a five-year frequency storm with a sixty minute duration or time of concentration. This intensity value tends to be low as most smaller watersheds, especially those with storm drain systems, have times of concentration much lower than sixty minutes and therefore higher intensity values. An average initial time of concentration, or that time required for runoff to reach the most upstream inlet, for a developed portion of a city is usually between 5 and 10 minutes. As a matter of fact, many cities have set maximum initial times of concentration allowed in their design criteria for certain types of land use. For example, the City of Dallas limits its residential initial time of concentration to 15 minutes or less and its commercial time to 10 minutes or less. Beyond the initial time of concentration, the value may be incremented up to 5 minutes or more depending on the velocity and distance traveled along a drainage route. As most typical city storm drains are less than 2000 feet in length and have velocities around 3 feet per second, it is rare that the total accumulated time of concentration exceeds 30 minutes. Thus, the Schaumburg report's value of a 60 minute time of concentration for all drainage design projects is too high for most of the smaller watersheds in Orange. The corresponding 5-year rainfall intensity value of 2.9 inches-per-hour is therefore too low. This lower intensity value leads to lower calculated runoffs and thus to undersized storm drains.

For the analysis in this report, each watershed's time of concentration was determined by first assigning a minimum value of 10 minutes for the initial time of concentration and then incrementing that time by the amount of time required for the runoff to reach the next point of analysis.

EXHIBIT II.1 Rainfall Intensity Curves



RAINFALL INTENSITY--DURATION--FREQUENCY CURVES

for the

Port Arthur, Texas Area

Data 1917 - 1951

Reference: U. S. Dept. of Commerce Technical Paper #25

2. HEC-1 Flood Hydrograph Package Computer Program

HEC-1 is a computer program created by the United States Army Corps of Engineers to calculate runoff amounts for watersheds that are generally larger than 200 acres. (See Reference 7) Because of its versatility and accuracy, the program has become widely used and is the accepted standard for most runoff analysis performed today.

The program works by creating a stream network model which simulates the runoff response of a river basin to rainfall over that basin. The program combines hydrography and routing computations in its analysis. The following paragraphs describe the elements required to develop a HEC-1 computer model.

One process of the HEC-1 program is to the determine the design storm rainfall. Design storm rainfall can be described in terms of frequency, duration, areal extent and distribution of intensity with time. A design storm's rainfall distribution in time is handled by the HEC-1 program by assuming a symmetrical, single-peaked design hyetograph, or design storm. The engineer's choice for frequency and duration is dependent upon the physical characteristics, location and study objectives. In most cases, design will be based on a 24-hour duration storm event. The HEC-1 program has the capability to modify runoff hydrographs to account for progressively smaller design storm volumes as areal coverage increases. The HEC-1 users manual suggests how to model storm rainfall depth versus drainage area relationships.

Another process of the HEC-1 program is to determine the "excess" rainfall. Only a portion of the rainfall volume which falls on a watershed during a storm event actually ends up as stream runoff. The remainder is intercepted by infiltration, depression storage, evaporation and other mechanisms. The volume of rainfall which becomes runoff is termed the excess rainfall. The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed as abstractions, or losses.

Having determined the design storm excess rainfall, the next process of the program is to determine the storm runoff hydrograph at particular points of interest. As a flood wave passes downstream through a channel or detention facility, its shape is altered due to the effects of storage. The procedure for determining how the shape of the flood hydrograph changes is termed "flood routing". Flood routing can be used to determine the effects of storage on a flood's runoff pattern, or its hydrograph.

HEC-1 uses all of these parameters to calculate the amount of runoff that will be produced by certain storm frequencies. The most common storm frequencies analyzed are the 10, 50, 100, and 500 year intervals.

B. HYDRAULICS

The planning, design and construction of drainage facilities is not only based on the study of hydrology but also on the study of hydraulics. Hydraulics is used to determine what quantity of runoff a drainage system will convey and the resulting water surface elevation. This information is useful and necessary to design the most efficient sections for channels and storm drains and to predict flood elevations. This sub-section discusses basic hydraulics and two methods of hydraulic analysis.

The state of flow in a channel is at all times either uniform, gradually varied, or rapidly varied. Different methods for determining water surface profiles are applicable to each of these conditions of flow. A brief description of each type of flow is provided below.

Uniform Flow

When a section of channel is sufficiently long and unchanging such that the flow depth is not changing (i.e. the force of gravity and channel resistance can be considered balanced), then the flow profile can be analyzed assuming uniform flow. Under these circumstances the depth remains constant and can be determined with Manning's equation. Manning's equation will be discussed in detail below.

Gradually Varied Flow

In the majority of channel flow situations, the state of flow is gradually varied. In other words, the depth is gradually changing with longitudinal distance along the channel due to an imbalance between the forces of gravity and channel resistance. Under these conditions, the recommended means for determining flow profiles is with the standard step method. The standard step method is an iterative process in which the one-dimensional energy equation is solved to find the water surface elevation at a cross-section. Manning's equation is utilized to determine channel losses due to friction. Losses due to channel non-uniformities are usually calculated with empirical coefficients. A widely accepted computer model for calculating gradually varied flow profiles is the U.S Army Corps of Engineers' HEC-2 Water Surface Profiles computer program. (See Reference 8) The HEC-2 program will also be discussed below.

Rapidly Varied Flow

Rapidly varied flow involves extreme conditions such as waterfalls and is not considered in this discussion.

1. Mannings Equation

Manning's equation is an empirical equation which relates friction slope, flow depth, channel roughness, and channel cross-sectional shape to flow rate. The friction slope is a measure of the rate at which energy is being lost in the flow to channel resistance. When the channel slope and the friction slope are equal ($S_f = S_o$) the flow is uniform and Manning's equation may be used to determine the depth of the uniform flow. Uniform flow is also known as normal depth.

Manning's equation is stated as follows:

$$V = \frac{1.49}{n} R^{2/3} S_f^{1/2}$$

or

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2}$$

where
- Q = total discharge cubic feet per second (cfs)
- V = velocity of flow (ft/sec)
- n = Manning's coefficient of roughness
- A = cross-sectional area of the flow (ft^2)
- R = hydraulic radius of the channel (ft) (flow area/wetted perimeter)
- S_f = friction slope, the rate at which energy is lost due to channel resistance

Manning's "n" value is an experimentally derived constant which represents the effect of channel roughness in the Manning's equation. Considerable care must be given to the selection of an appropriate "n" value for a given channel due to its significant effect on the character of the flow. A list of "n" values used in this study is provided in Table II.2. Much more extensive lists of n-values are available in most hydraulic text books.

TABLE II.2 "N" Values

Conveyance	"n" value
Natural Channel	
Rock bottom	.03
Light vegetation	.03
Moderate vegetation	.05
Heavy vegetation	.08
Concrete-lined Channel	.015
Flood Plains	
Wooded areas	.15
Residential	.15
Marsh	.08
Reinforced Concrete Pipe	.013

2. HEC-2 Flood Hydrograph Package Computer Program

HEC-2 is a widely accepted computer model for calculating gradually varied flow profiles. (See Reference 8) The program uses the standard step method for open channel flow and can readily accommodate modifications in channel design and losses at bridges, culverts, drop structures and transitions. Program input includes the flow, cross section geometry, cross section characteristics, and slope along each reach to be analyzed. The program begins computation at a cross section of known or estimated water surface elevation and proceeds upstream for subcritical flow, and downstream for supercritical flow. Program output includes flow velocity, flow widths, and water surface elevations.

The HEC-2 program requires accurate cross sectional data to be effective. Cross sectional data includes point elevations and stations, "n" values for the length of the cross section, and the distance between cross sections. Cross-sections should be placed such that the channel configuration between them is largely uniform. In areas where channel properties are rapidly changing, the distance between cross-sections should be appropriately less. The HEC-2 cross sectional information used for the analysis in this report came mostly from flood studies published by the Federal Emergency Management Agency (FEMA). Some modifications to FEMA's data were made based on channel improvements and visual surveys.

The HEC-2 program also requires an accurate determination of the starting water surface elevation, especially in the vicinity of the first cross-section. The best method of determining a starting water surface elevation is with a known rating curve or from past backwater studies. The least favorable is the slope-area method which determines normal depth given the friction slope and discharge. It is important to begin water surface profile analyses a significant distance downstream of the point(s) of interest for subcritical flow and upstream of the point(s) of interest for supercritical flow. The starting water surface elevations used in the analysis of this study were taken from past backwater studies of the receiving stream at the proper points of confluence. The receiving streams in this study were the Sabine River and Adams Bayou.

Special care is required in handling energy losses due to bridges. The HEC-2 users manual presents several methods for determining bridge losses and may be consulted for more detail. The method used most often in this study was the Special Bridge Method.

C. STORM DRAIN ANALYSIS

Storm drains are usually constructed of reinforced concrete pipe and are aligned in streets or other public right-of-ways. Storm drains are designed to carry a desired frequency storm peak flow with the most efficient pipe size possible to minimize cost. Exhibit II.2 illustrates three different levels of storm drain design that are possible. Storm drains designed to carry the 100-year storm are usually cost prohibitive due to greater material and construction costs. Individual communities must decide what level of flood protection they desire and what price they are willing to pay. Once a specific level of protection is approved, peak flows may be determined using the Rational Equation. Flow velocities and water surface elevations are determined using Manning's Equation assuming uniform flow conditions. A general outline of the storm drain design process is described below.

Usually the first step for a storm drain design is to outline the area to be drained. The outlined area is then divided into sub-drainage areas that are determined by inlet locations. Each inlet drains a sub-area. Inlets are located in natural low-lying areas and along curbs at spacing intervals that prevent flows from becoming excessive in the streets. The storm drain alignment is usually set at this time. Once the sub-drainage areas are known the C-factor value for each is estimated. Preliminary pipe sizes are then chosen. The adequacy of the preliminary pipe sizes is then analyzed using the Rational Equation to determine a flow value and Manning's Equation to determine the flow velocity and water surface elevation. The beginning or downstream water surface elevation must be known before analysis can begin. Based on the results of the analysis, the pipe size and or slope is either increased or decreased accordingly.

Head losses, or changes in water surface elevation due to changes in flow conditions, must also be calculated. The equation for the head loss (feet) at an inlet or manhole is stated as follows:

$$head\ loss = \frac{(V_2^2 - KV_1^2)}{2g}$$

where

V_1 = velocity in the upstream pipe (fps).
V_2 = velocity in the downstream pipe (fps).
K = junction or structure coefficient of loss.

A special case of sudden contraction is the entrance loss for pipes. The equation for head loss at the entrance to a pipe is given as follows:

$$head\ loss = K \frac{V^2}{2g}$$

where

K = entrance loss coefficient. (See Table 5.4)
V = flow velocity in pipe (fps).

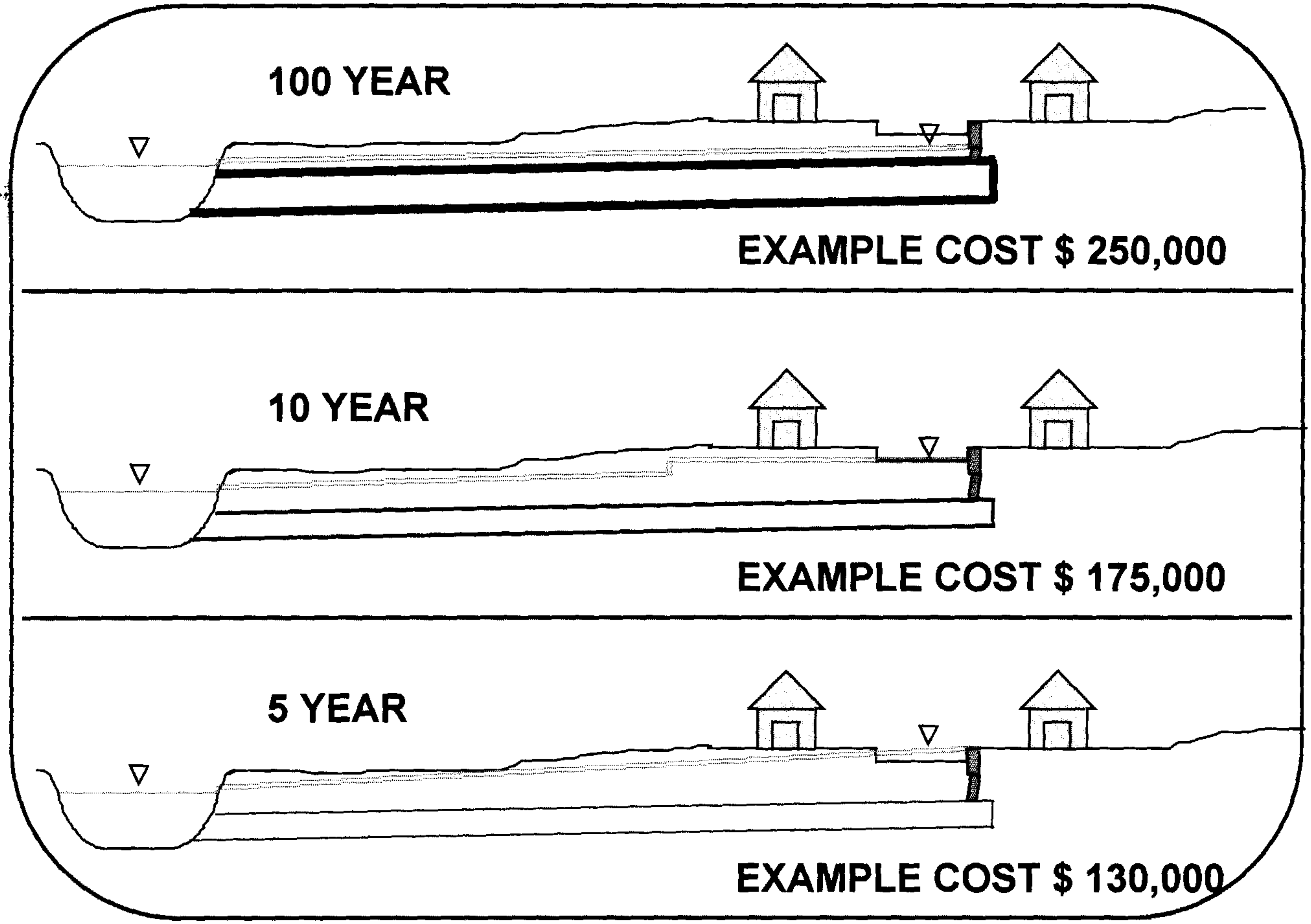
The analysis for pipe size adequacy and head losses continues for the length of the drain.

The storm drain analysis performed in this study made use of a computer spreadsheet developed in-house by Carter & Burgess, Inc. The spreadsheet is named HYDRADAL and incorporates both the Rational and Manning's equations.

For purposes of this study, storm drains were designed to carry the 5-year rainfall runoff. Intensities, for use in the Rational Equation, were taken off of the intensity-duration-frequency curve discussed earlier. The rainfall intensity for a particular design was based on the time of concentration to that particular point of interest. The minimum pipe sizes used were 18" as smaller pipes are difficult to maintain. The "n" value used for concrete pipe was 0.013. Sub-drainage areas were determined using existing storm drain maps provided by the City of Orange.

EXHIBIT II.2

LEVELS OF FLOOD PROTECTION



CITY OF ORANGE

Carter-Burgess

III. WATERSHED ANALYSIS

This section describes the physical characteristics of each watershed studied and how it was analyzed. The section is divided into three sub-sections; data acquisition, watersheds susceptible to wide-scale flooding, and watersheds susceptible to localized flooding.

A. DATA ACQUISITION

The data used for this study came from many sources. The Federal Emergency Management Association (FEMA) provided copies of existing Flood Insurance Studies (FIS) for the City of Orange and its surrounding communities. FEMA also provided micro-filmed copies of pertinent HEC-1 and HEC-2 inputs and outputs. The City of Orange provided blue-line aerial maps of the city at 1" = 100' scale, an extensive computer planimetric and topographic database, and schematic plans of existing storm drains at 1" = 100' with delineated surface flow directions. The City of Orange also provided copies of previous applicable flood studies. The United States Army Corps of Engineers (COE) provided copies of previous applicable flood studies and provided useful information on neighboring Port Arthur's levee construction. Engineers from Carter & Burgess, Inc. also made several trips to Orange to visually survey the study areas. Photographs and video recordings were made for additional reference.

Carter & Burgess hired Klinkhammer & Associates to provide actual survey measurements of several areas. Specific information included cross sections along Dayton Street Ditch and spot elevation checks to verify contour elevations and elevations along the existing levee by Coopers Gully pump station.

B. WATERSHEDS SUSCEPTIBLE TO WIDE-SCALE FLOODING

Wide-scale flooding in Orange may be caused by the Sabine River and by hurricanes originating in the Gulf of Mexico. The Sabine River is about 300 miles long and has a watershed encompassing over 9,700 square miles. For such a large river there are very few flood control measures in place. (See Reference 9) Large amounts of rainfall in the upper basin can significantly raise the amount of flow as well as the elevation of the river downstream. The Gulf of Mexico, only ten miles downstream from Orange, is capable of producing hurricanes with enough rainfall and high enough tidal surges to flood large portions of the City. Wide-scale flooding may have a duration of several days or even weeks if due to the Sabine River. The Navy built a small levee and floodwall to help protect its base in Orange from wide scale flooding. The existing levee will be discussed in greater length in the Levee sub-section.

For the purposes of this study the "wide-scale" flooding definition incorporated Adams Bayou and Little Cypress Bayou because their watersheds are much larger than those in the "localized flooding" category. This sub-section describes the analysis of Adams Bayou, Little Cypress Bayou and the Levee protection alternatives.

1. Adams Bayou

Two alternatives were studied regarding flooding due to Adams Bayou. The first was diverting flow out of and away from Adams Bayou and the second was widening the Adams Bayou channel by dredging.

Diversion of Flow

One way to reduce flooding along Adams Bayou would be to decrease the amount of flow in Adams by diverting it directly to the Sabine River. Several diversion routes were analyzed. The most plausible route was determined to be diverting flow from Adams Bayou to Little Cypress Lake through a gravity flow channel just north of and parallel to I-10. A schematic layout of the diversion channel is shown in Exhibit III.1 Note that this alignment benefits only that portion of the floodplain downstream of I-10. The length of the diversion channel would be approximately 10,500 feet.

The first step of the analysis was to size the gravity flow channel. The difference in elevation between the 100-year water surface in Adams Bayou just upstream of I-10 and the 5-year water surface in Little Cypress Lake is only 5 feet. Such a small difference in elevation over such a long distance, means the proposed channel invert gradient must be very flat. The slope was set at 0.05%. Such a flat channel requires a large cross sectional area to carry the diversion flow. A channel bottom width of 100' was used along with a depth of 8 feet. As the channel would be grass lined, the side slopes were set at 3:1. Hydraulic analysis of the channel showed it would divert up to 3,300 cubic feet per second (cfs) away from Adams Bayou to the Sabine River.

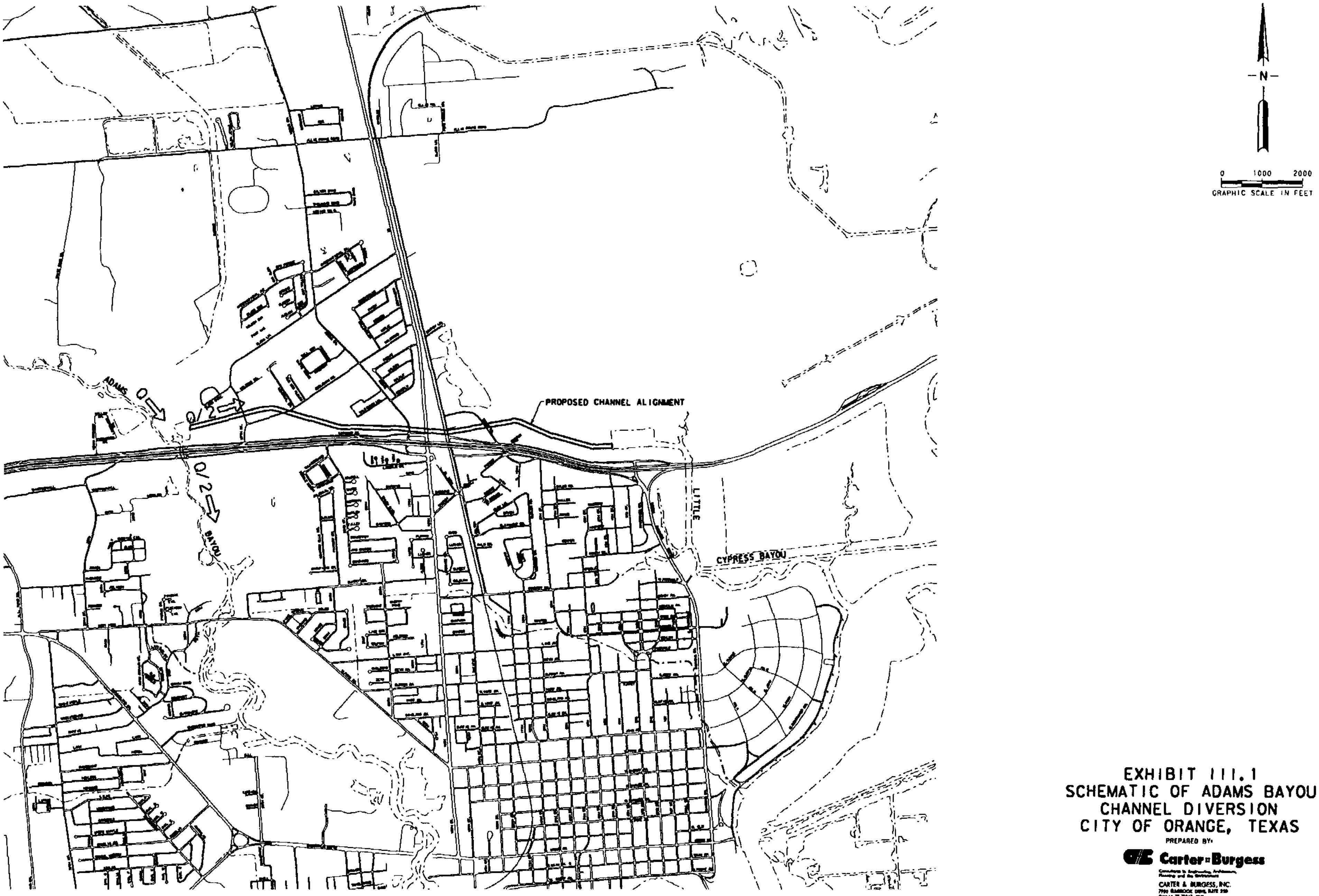


EXHIBIT III.1
SCHEMATIC OF ADAMS BAYOU
CHANNEL DIVERSION
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.
APRIL, 1994

The next step of the analysis was to determine the downstream effects of the diversion on the Adams Bayou floodplain. The existing HEC-2 analysis was run and the resulting floodplain was plotted on a planimetric map. See Exhibit III.2 for a plan view of the HEC-2 cross section locations. Then the HEC-2 input was altered to reflect the diversion of 3,300 cfs and the analysis was re-run. The resulting floodplain of the diversion alternate was then plotted on the same planimetric map.

The final step of the analysis was to determine the benefit gained by the reduced floodplain and to estimate the construction costs of the diversion channel. The results are discussed in the Conclusions section of this report.

Channel Widening

Another alternative analyzed to reduce flooding along Adams Bayou was to widen the existing channel by dredging. One benefit of this alternate was that the floodplain reduction could be extended north of I-10 as far as the channel was dredged unlike the diversion channel alternate whose benefits were limited to south of I-10.

The first step of this analysis was to plot the Adams Bayou floodplain on a planimetric map. Next, the proposed channel widening width was set at 50'. The HEC-2 input was then altered using Channel Improvement cards to reflect a 50' widening. The HEC-2 analysis was re-run and the resulting floodplain was plotted on the same planimetric map.

The final step of the analysis was to determine the benefit gained by the reduced floodplain and to estimate the dredging costs of widening the channel. The results are discussed in the Conclusions section of the report.

VIEWS OF ADAMS BAYOU

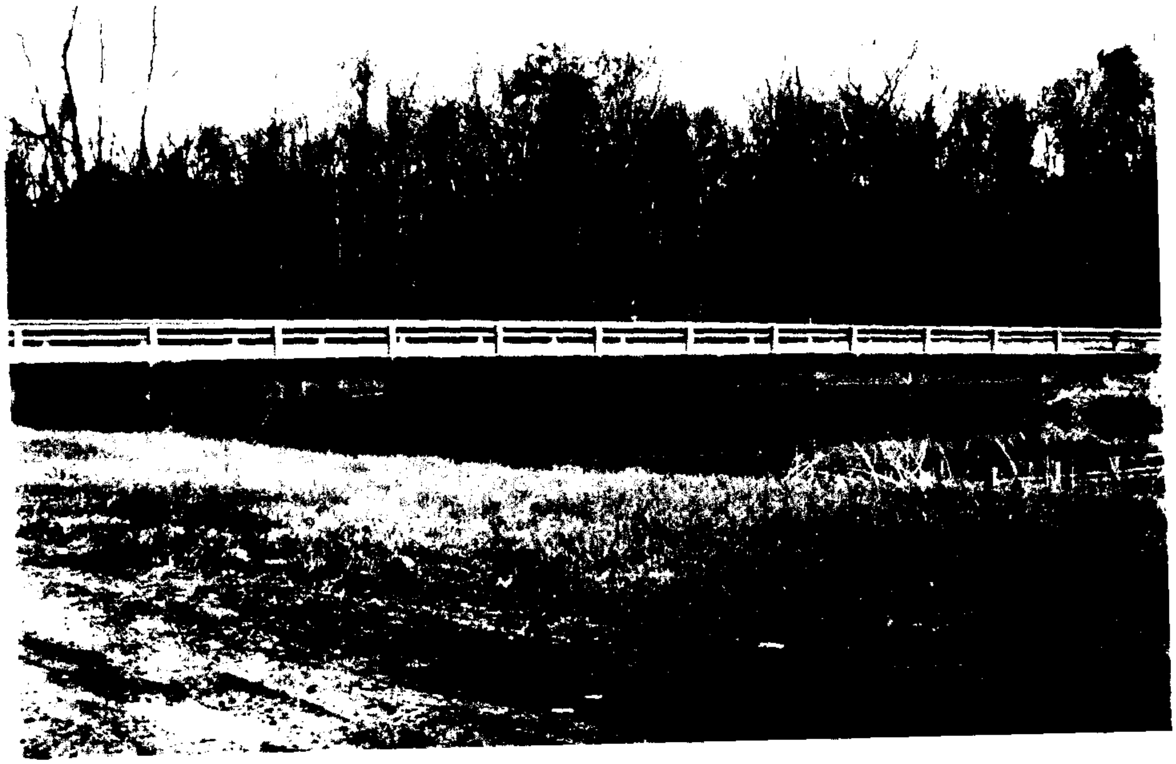
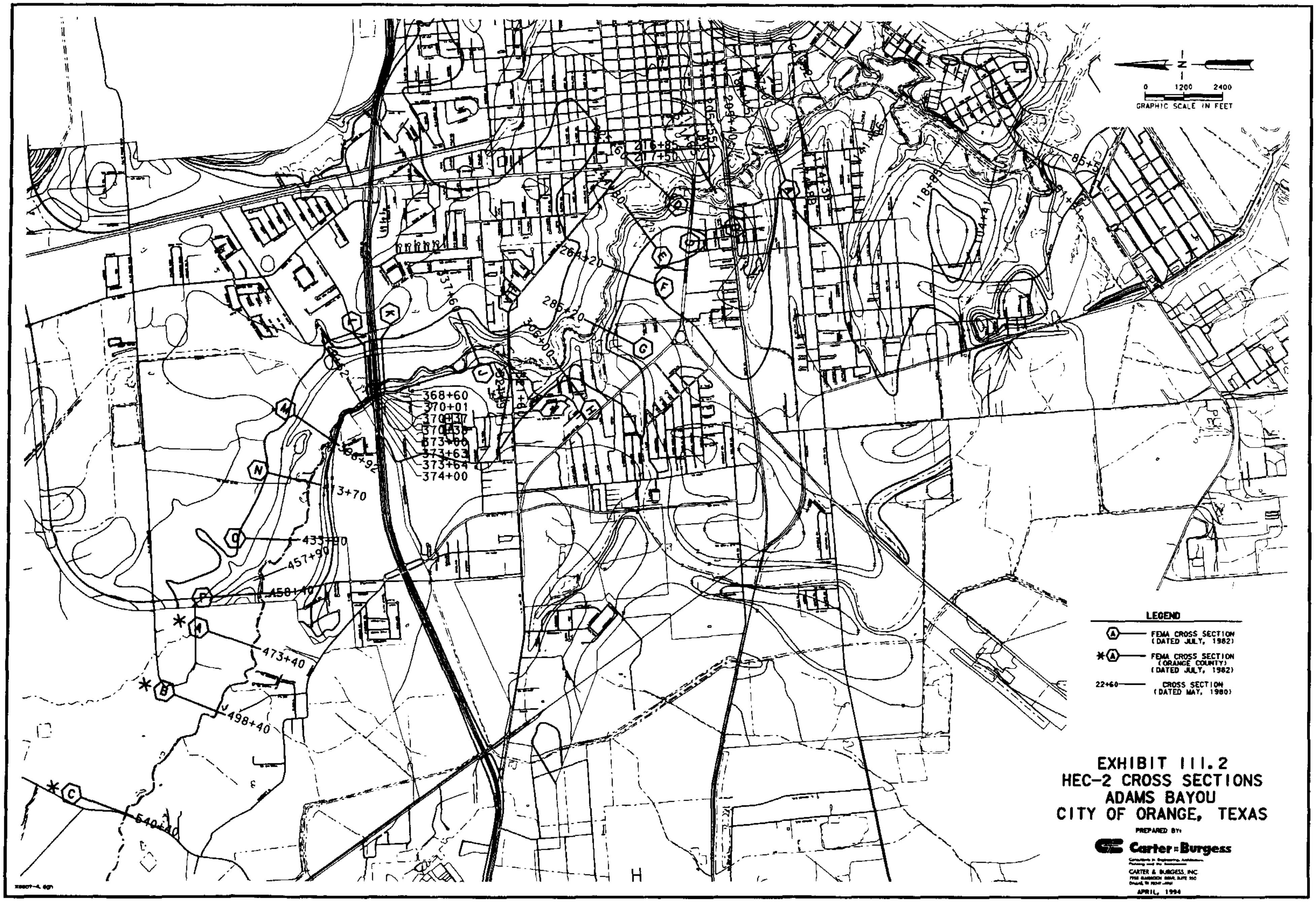


PHOTO III.1 Western Ave. Bridge

PHOTO III.2 Interstate-10 Bridge

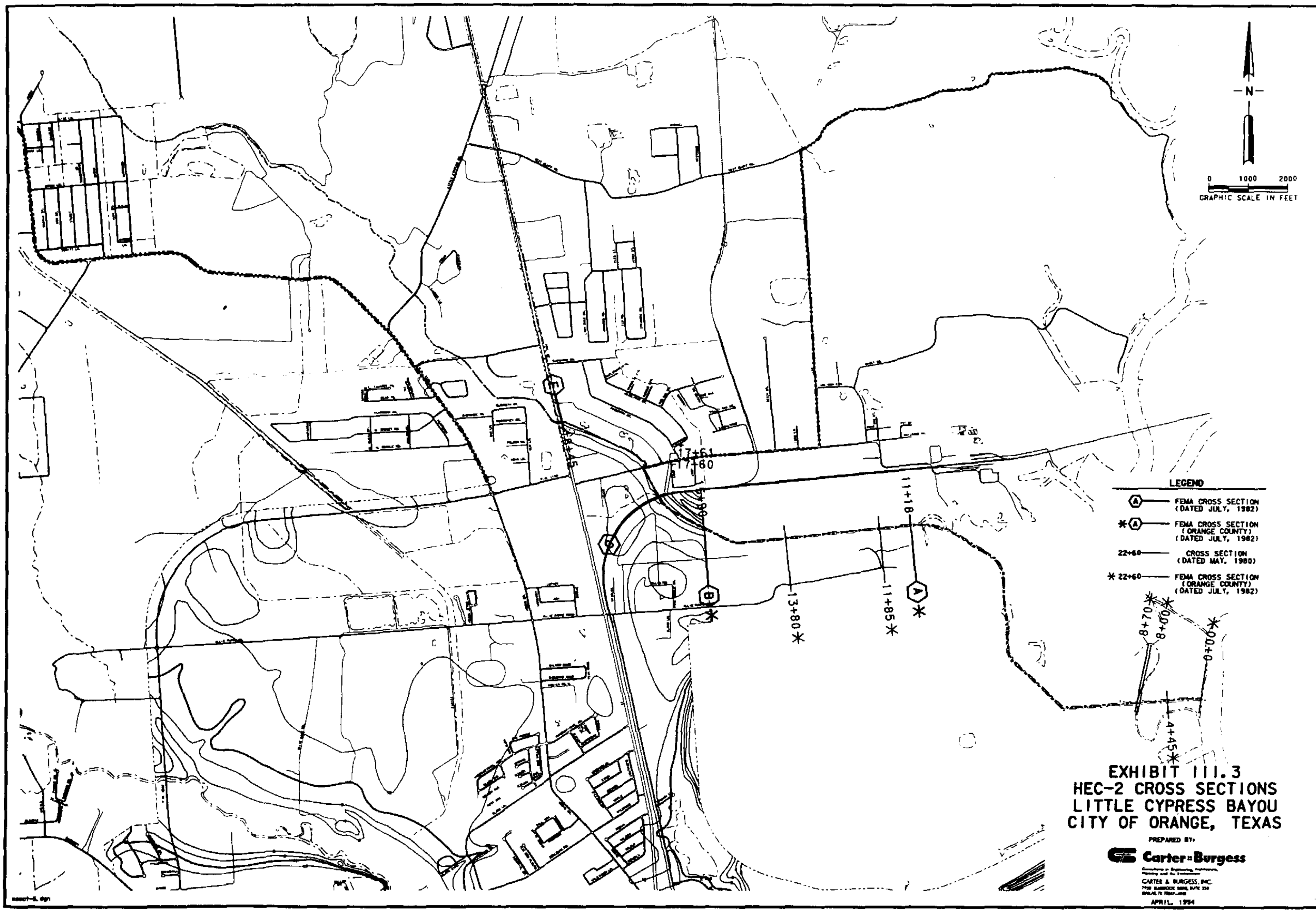

N
0 1200 2400
GRAPHIC SCALE IN FEET
LEGEND
FEMA CROSS SECTION (DATED JULY, 1982)
FEMA CROSS SECTION (ORANGE COUNTY) (DATED JULY, 1982)
22+60 CROSS SECTION (DATED MAY, 1980)
368+60
370+01
373+63
373+64
374+00
413+70
433+90
457+90
458+40
473+40
498+40
540+40
264+20
285+20
216+85
217+50
EXHIBIT III.2
HEC-2 CROSS SECTIONS
ADAMS BAYOU
CITY OF ORANGE, TEXAS
PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.
APRIL, 1994

2. Little Cypress Bayou

Little Cypress Bayou is a watershed located north of the City of Orange that drains approximately 18,000 acres. The watershed was included in the FEMA study performed for the City of Orange. However, in the analysis during this study discrepancies were noted between the HEC-2 input and output files provided by FEMA. One discrepancy noted is a difference in the lengths of the two studies. The micro-filmed output files and the published Flood Insurance Study both correspond and show the HEC-2 analysis all the way to cross section number 17.000 for a total study length of over 43,000 feet. The micro-filmed input file, though, ends at cross section number 23.00 for a total study length of only 23,000 feet. Cross section 23.00 is located about 1,800 feet upstream of the Highway 87 bridge. The shorter input file therefore limited the extent of this study's analysis. See Exhibit III.3 for a plan view of the input file's cross sections.

Another discrepancy noted between the input and output files was a difference in flow values. The 100-year flow in the micro-filmed output file, as well as the published FIS report, was 3,750 cfs at Jack's landing. The 100-year flow, though, in the input file at the same location was 5,208 cfs. For the purposes of this study those flows from the published FIS report were assumed to be correct and were used in the analysis.

The first step of the analysis was to alter the HEC-2 input file to match the assumptions previously stated. The resulting 100-year floodplain was then plotted on a planimetric map. Within the limited reach of this analysis there were a total of 19 homes observed within the floodplain boundary.


N
0 1000 2000
GRAPHIC SCALE IN FEET
17+61
17+60
11+18
13+80
11+85
8+70
8+00
0+00
4+45
A
B
LEGEND
FEMA CROSS SECTION (DATED JULY, 1982)
FEMA CROSS SECTION (ORANGE COUNTY) (DATED JULY, 1982)
22+60 CROSS SECTION (DATED MAY, 1980)
22+60 FEMA CROSS SECTION (ORANGE COUNTY) (DATED JULY, 1982)
EXHIBIT III.3
HEC-2 CROSS SECTIONS
LITTLE CYPRESS BAYOU
CITY OF ORANGE, TEXAS
PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.
APRIL, 1994

The next step of the analysis was to determine how the floodplain might be reduced. Only three bridges were included in the analysis, namely; the Southern Pacific Railroad, the F.M. 1130, and the Highway 87 bridges. Both the Southern Pacific Railroad and F.M. 1130 bridge's low chords were above the 100-year water surface and therefore no improvements are recommended. The Highway 87 bridge experienced pressure flow and appears to be somewhat of a constraint. However, the top of road is not flooded and therefore no improvements are recommended at this time to Highway 87. No further analysis was performed as there are so few houses in the floodplain that any full scale improvements made to Little Cypress Bayou would not likely warrant the cost. Other recommendations to reduce the impact of floods in Little Cypress Bayou are included in the Recommendations section of this report.

3. Levee Protection Alternatives

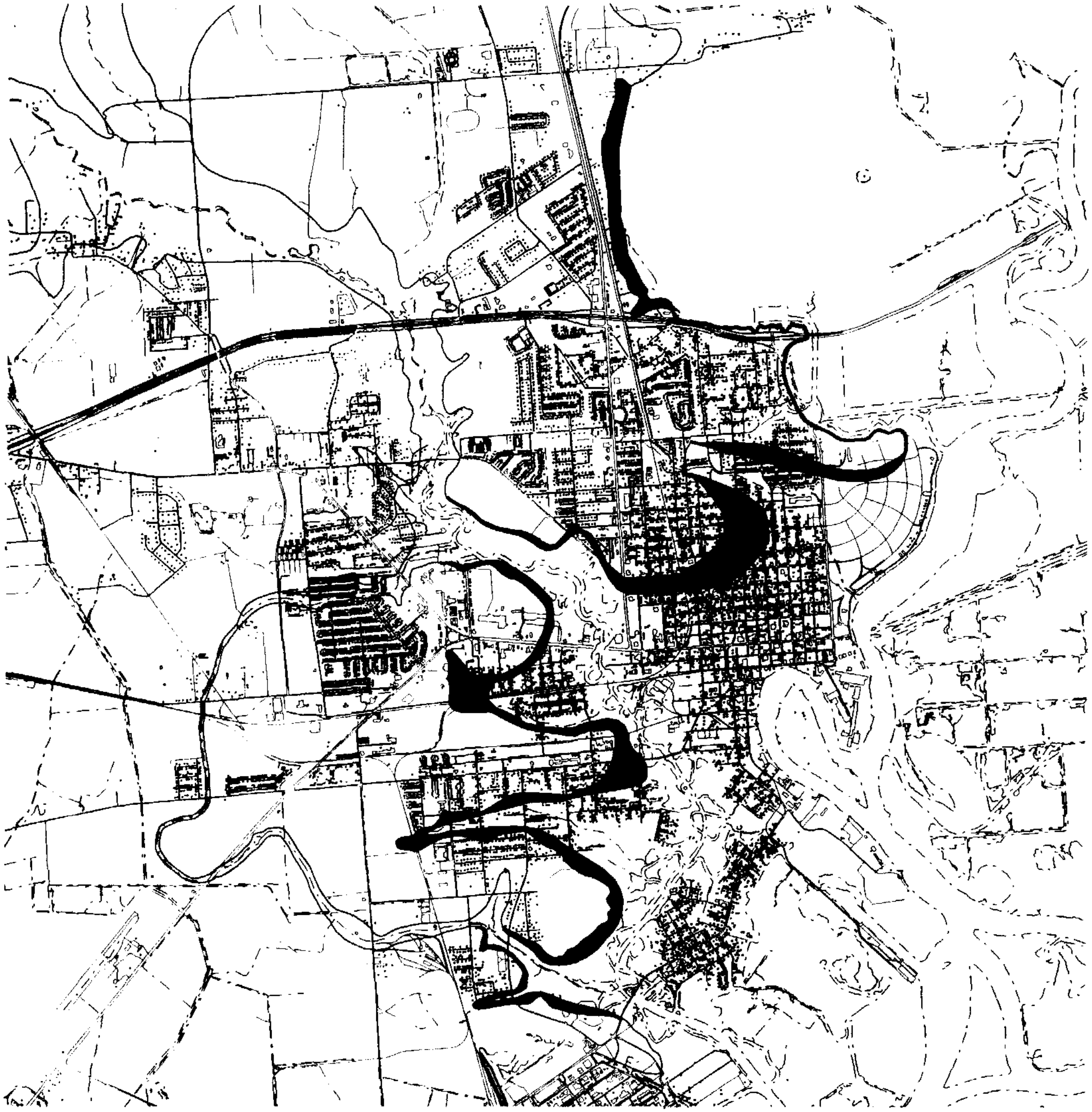
The biggest threat of wide scale flooding in Orange comes not from Adams Bayou but from the Sabine River. The river can rise due to flooding upstream and due to tidal surges from hurricanes in the Gulf of Mexico. Recognizing this threat, the U.S. Navy built a small levee to protect its base at Orange. (See Reference 1) The levee begins near the Simmons and Green intersection then heads north parallelling the Sabine River. The levee then runs west parallelling Dewey Avenue. The levee ends near the Simmons and Dewey intersection. A pump station was built where the levee crossed Coopers Gully. The pump station is discussed in the Coopers Gully sub-section of Localized Flooding. The existing levee provides only minimal protection from hurricane flooding. (See PHOTOS III.3&4)

FEMA published a flood insurance study for the City of Orange in July of 1982. (See Reference 1) This study presented water surface profiles for different year storms and mapped the 100-year floodplain. The results of this Orange study, however, conflict with the adjacent FEMA study of Calcasieu Parish across the Sabine River in Louisiana. (See Reference 10) The Calcasieu Parish study found lower flood elevations and a slightly smaller floodplain. The accuracy of the Calcasieu Parish study was confirmed by the Brown and Root, Inc. study of the Sabine River recently released in January of 1993. (See Reference 9) Exhibit III.4 entitled "Comparison of 100-year Floodplains" and Exhibit III.5 entitled "Comparison of 100-year Flood Profiles" illustrate the differences between the two studies.

VIEWS OF EXISTING LEVEE

PHOTO III.3 Existing Levee Near Coopers Pump Station

PHOTO III.4 Existing Levee Near Coopers Pump Station



LEGEND

- 100-YEAR FLOOD PLAIN BOUNDARY IN ORANGE COUNTY F.I.S. STUDY
- 100-YEAR FLOOD PLAIN BOUNDARY IN SABINE RIVER AUTHORITY STUDY

EXHIBIT III.4
COMPARISON OF
100 YEAR FLOODPLAINS
CITY OF ORANGE, TEXAS

PREPARED BY:

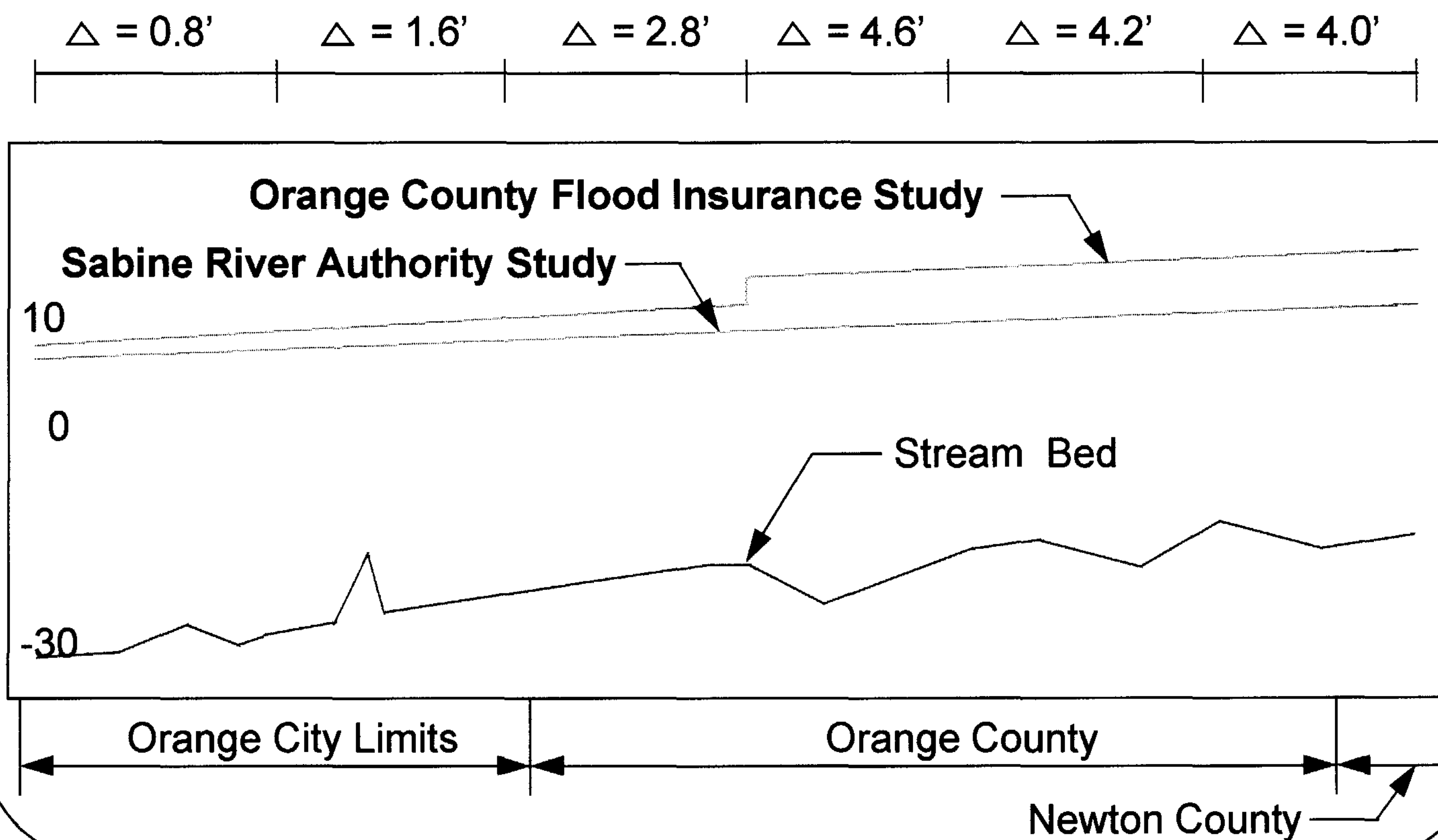
Carter & Burgess

CARTER & BURGESS, INC

APRIL, 1994

EXHIBIT III.5 Comparison of 100-Year Flood Profiles

SABINE RIVER



The only reasonable way to protect the entire City of Orange from wide-scale flooding from the Sabine River is by constructing a levee. This report assumed a levee design height of 14' MSL to provide protection against the Standard Project Flood (SPF) of elevation 10' MSL plus an additional 4 feet of freeboard. For the purposes of this report, three levee options of differing alignments were analyzed. Each alignment is a combination of earthen levee and concrete floodwall. Floodwall sections are intended to be used where earthen levees are not practical due to right-of-way constraints, aesthetics, or environmental impact. Floodwall sections are recommended along Front Street and along portions of West Park Ave. The three levee alignment options studied are as follows:

No.1 - This proposed alignment protects primarily the City of Orange only. The levee begins near the Simmons and I-10 intersection and follows the existing levee alignment past the Coopers Gully pump station. It then follows the riverbank to Dupont Drive and encompasses the Cove area then turns inland and follows the east bank of Adams Bayou all the way to the intersection of West Park and Link Ave. Exhibit III.6 gives a schematic alignment of the levee alternative.

No.2 - This proposed alignment protects primarily the cities of Orange, West Orange and Pinehurst. The levee has the same layout as No.1 on the east side of Adams Bayou but also includes additional levee along the west side of Adams Bayou to protect the cities of West Orange and Pinehurst. The additional levee begins near the hospital on Strickland Drive and parallels Adams Bayou to Smith Street in West Orange where it turns southwest and bends around Courtland Ave. and heads northwest to its end near the intersection of Western and Hwy 87. Exhibit III.7 shows a schematic layout of the levee alternative.

No.3 - This levee alternative also has the same layout as No. 1 on the east side of Adams Bayou and includes additional levee to the west of Adams. However, the western levee in this option is extended to include the industrial area known as "chemical row." The additional levee begins near the hospital on Strickland Drive and parallels Adams Bayou to just beyond Dupont Drive where it turns westerly to incorporate the industries then heads north to its end near the Orange Airport. A schematic layout of this levee alternative is shown on Exhibit III.8.

Pump stations will be required to remove storm runoff that accumulates within the protected area of these levee alternatives. Pump stations will be located in naturally low lying areas and have runoff channeled to them. Levee Option No.1 of this report includes five pump stations. Option No. 2 includes a total of seven pump stations and Option No. 3 also includes a total of nine pump stations.

A typical proposed levee cross section is shown as Exhibit III.9. Preliminary cost estimates and recommendations regarding these levee alternatives are discussed in the Conclusions and Recommendations section of this report.

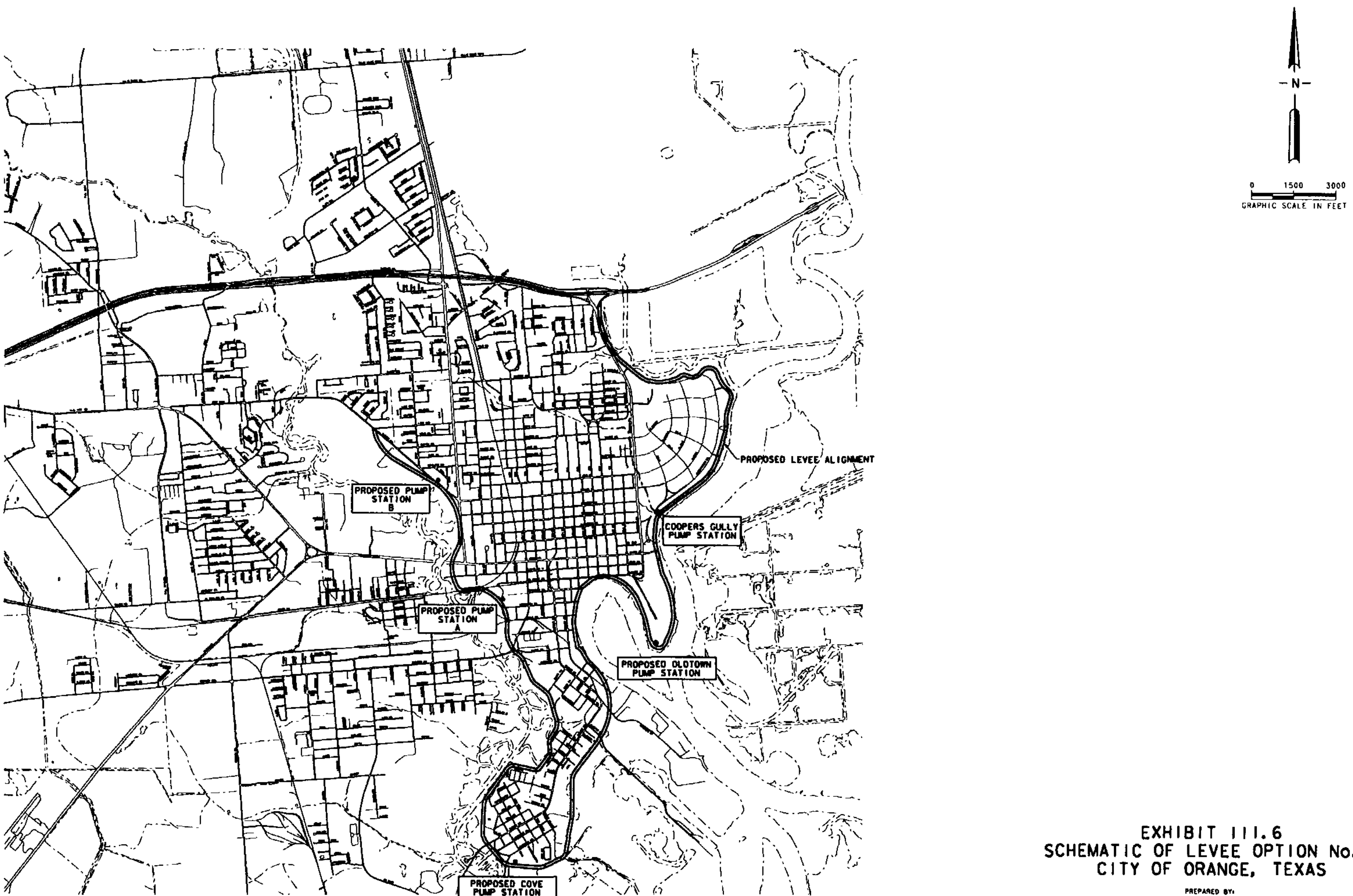
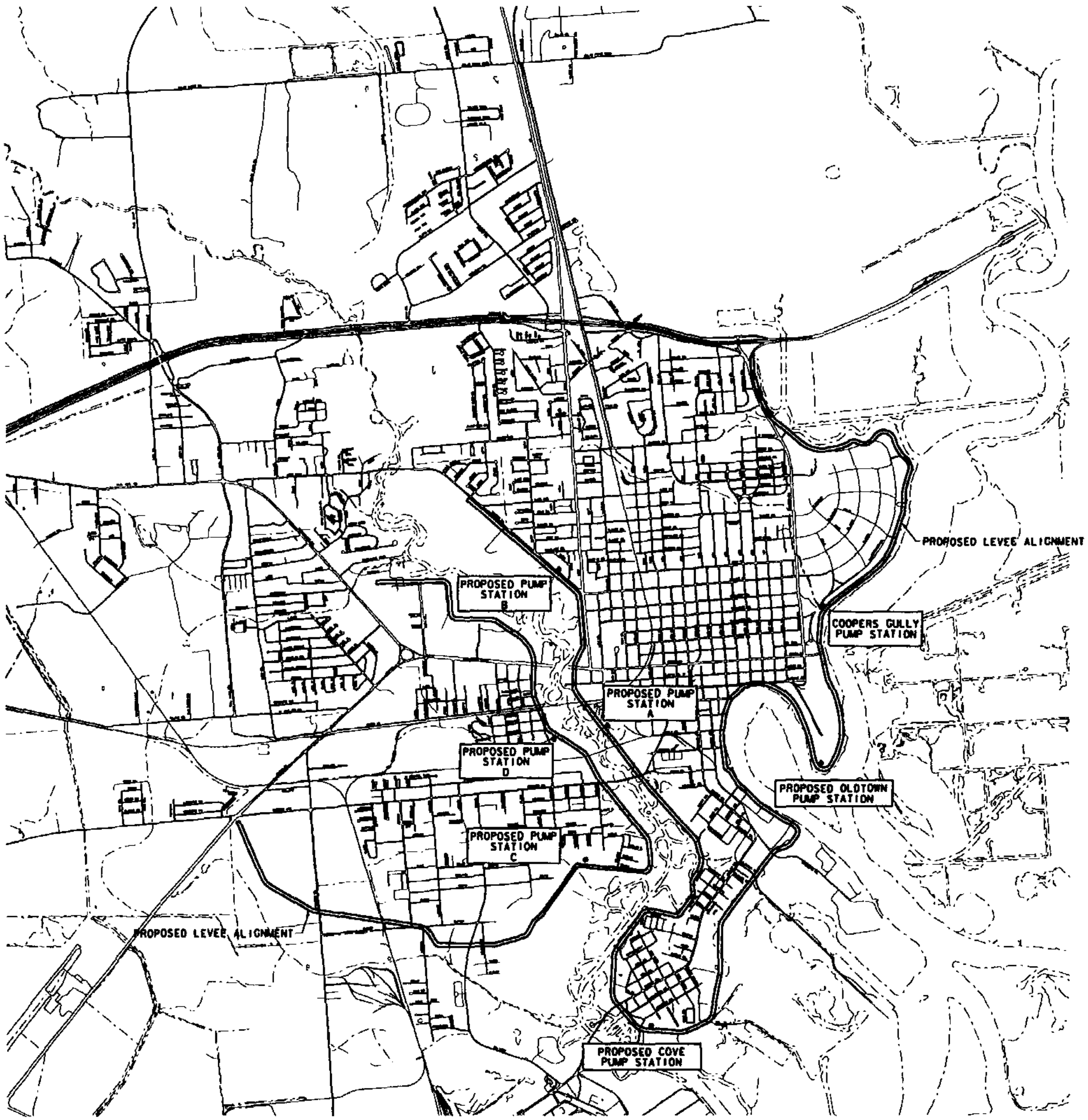


EXHIBIT III.6
SCHEMATIC OF LEVEE OPTION No. 1
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter::Burgess
CARTER & BURGESS, INC.
APRIL, 1994

EX1.DGN



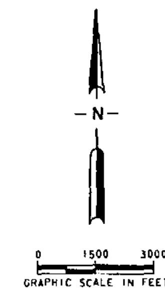


EXHIBIT III.7
SCHEMATIC OF LEVEE OPTION No. 2
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.

APRIL, 1994

EX4.DGN

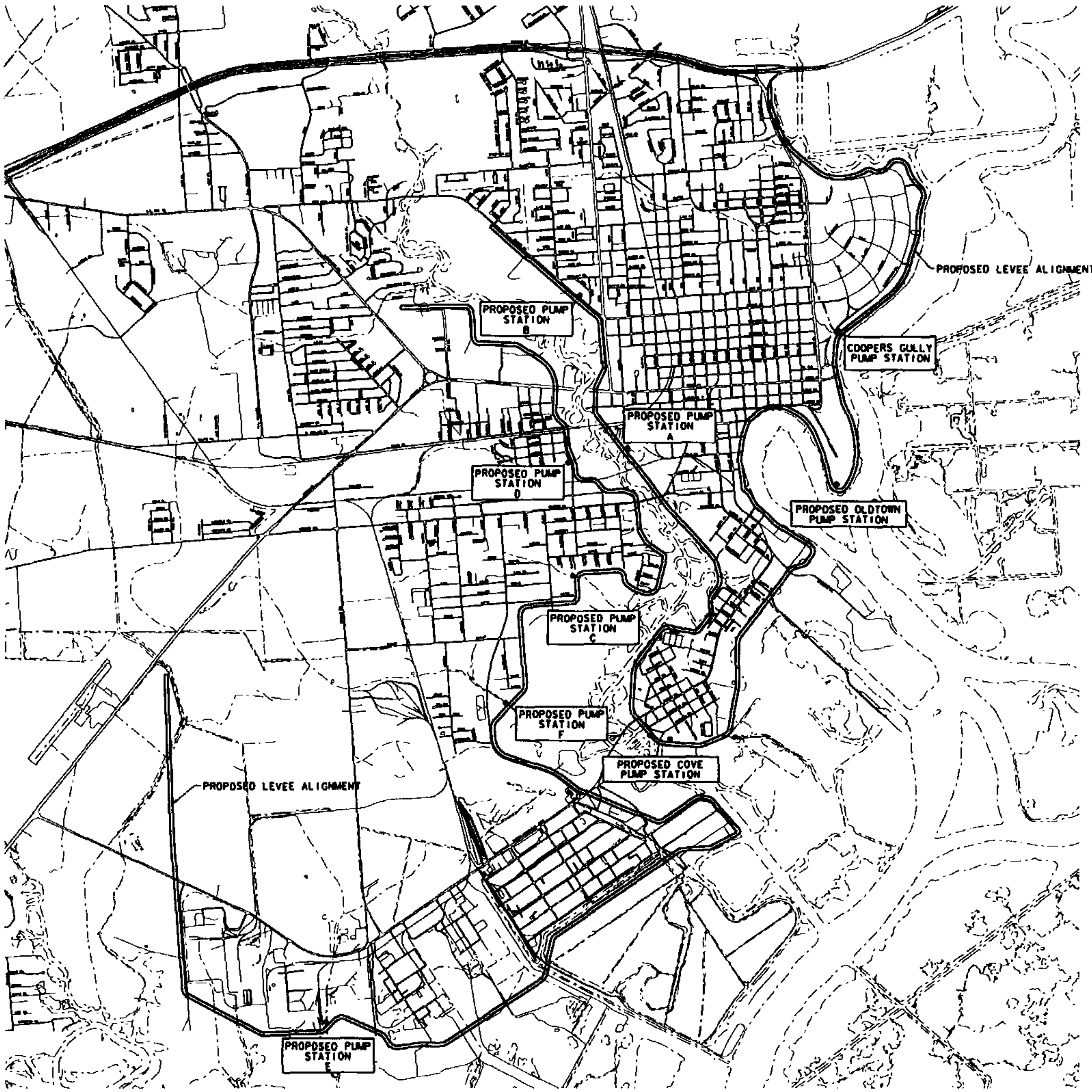
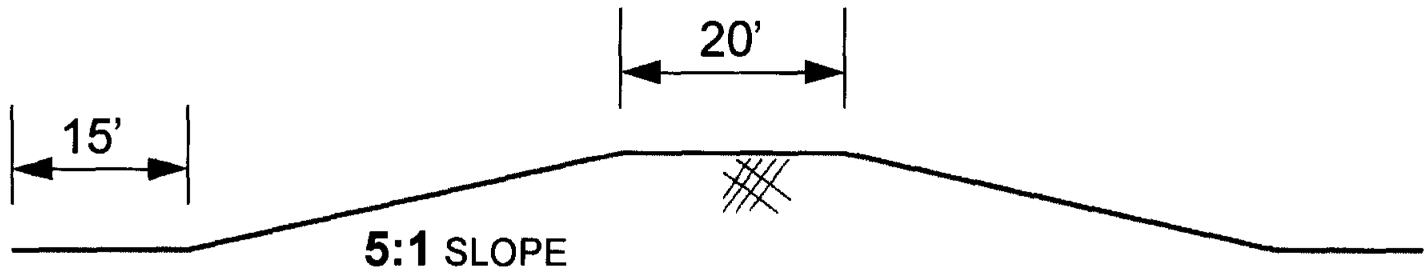


EXHIBIT III.8
SCHEMATIC OF LEVEE OPTION No. 3
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.
DECEMBER, 1993

EX5.DGN

EXHIBIT III.9 TYPICAL LEVEE CROSS SECTION



LEVEE DIMENSIONS	RANGE
HEIGHT :	1’ - 8’
SIDE SLOPES :	3:1 - 5:1*
WIDTH OF FILL :	30’ - 100’
WIDTH OF RIGHT-OF-WAY:	60’ - 130’

* Depending on soil stability

C. WATERSHEDS SUSCEPTIBLE TO LOCALIZED FLOODING

On a local level, flooding is most often the result of increased development. As the natural land is, altered the drainage characteristics change and generally increase the rate of storm water runoff. Increased storm runoff increases the risk of flooding. Much of the City of Orange was developed in spurts without provision for adequate storm water drainage systems to convey the increased runoffs. The combination of high rainfall amounts, high runoff rates, and inadequate storm drain and channel capacities has created conditions conducive to localized flooding in and around Orange. Seven local areas known to contain flooding concerns were presented to the Consultants by the City of Orange for analysis. The seven areas are the Cherry Ave. and 13th St. watershed, Coopers Gully pump station, Dayton Street Ditch (especially along Hwy. 87 near Bluebonnet Drive), Hudson Gully, North Simmons Drive, Old Town, and Sunset Drive. Exhibit III.10 entitled "Localized Areas Prone to Flooding" shows each of these areas. Exhibit III.11 entitled "Watersheds" provides a boundary map of the City's watersheds. Each localized area will be discussed in greater detail in the following sections.

LEGEND

1. CHERRY AVENUE
2. COOPERS GULLY
3. DAYTON STREET DITCH
4. HUDSON GULLY (ROSELAWN)
5. NORTH SIMMONS DRIVE
6. OLDTOWN
7. SUNSET DRIVE

EXHIBIT III.10
LOCALIZED AREAS
PRONE TO FLOODING
CITY OF ORANGE, TEXAS

PREPARED BY:

Carter & Burgess

CARTER & BURGESS, INC.

APRIL, 1994

EXT.DGN

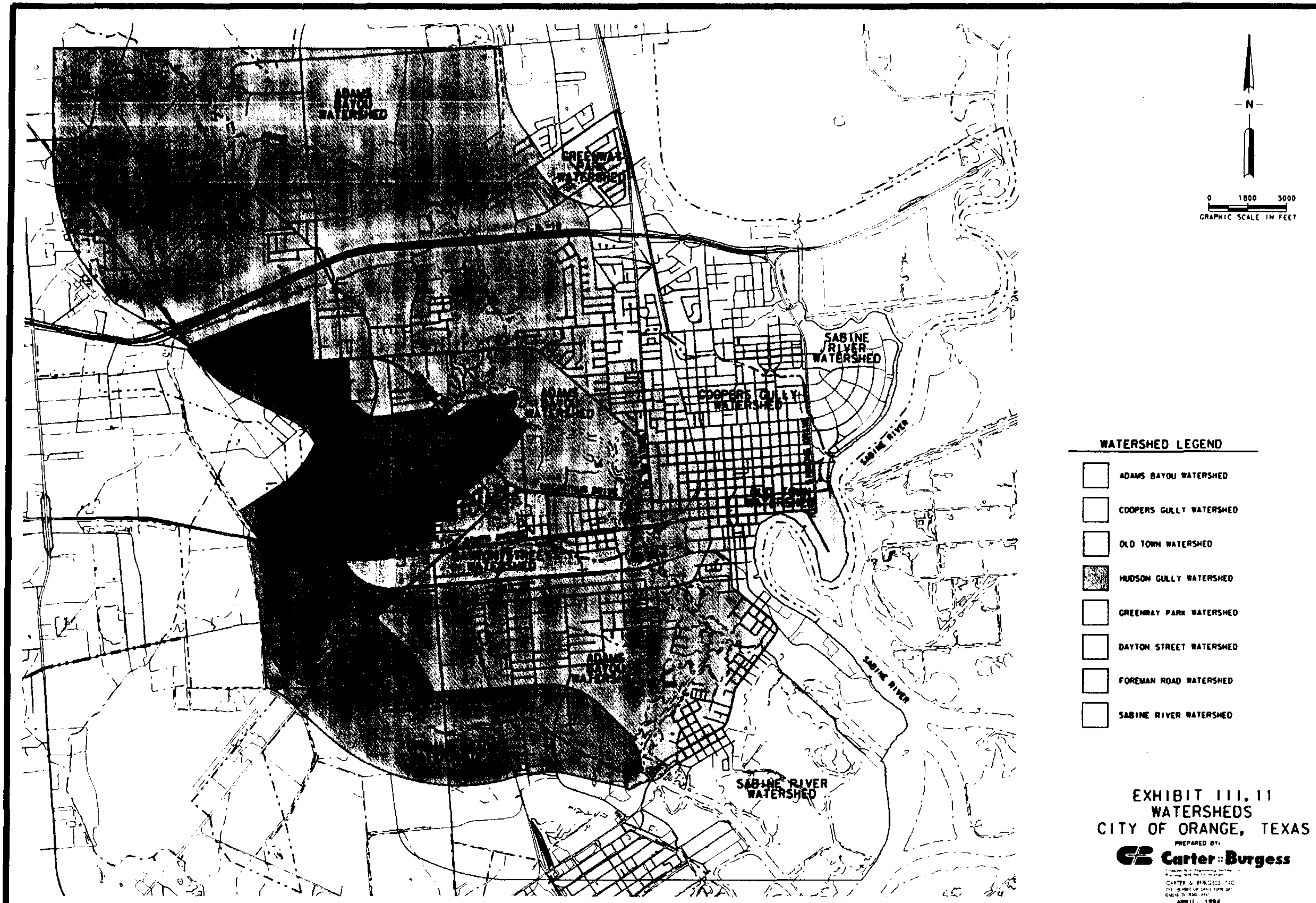


EXHIBIT III.11
WATERSHEDS
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter::Burgess
CARTER & BURGESS, INC.
APRIL, 1994

1. Cherry Ave. & 13th St.

The area in the vicinity of Cherry Ave. & 13th St. was designated by the City of Orange as an area prone to flood. The Cherry Ave. & 13th St. watershed is roughly bounded by Elm on the south, 14th St. on the west, Curtis Ave. on the north, and 10th St. on the east. The watershed encompasses approximately 122 acres. The natural ground slope, or drainage pattern, is from east to west. The receiving water is Adams Bayou. The watershed high point is approximately elevation 10' near Curtis Ave. & 13th St. The watershed is fully developed as residential. An existing storm drain system is in place. A discussion of the existing system's hydrologic and hydraulic analysis follows.

Refer to Section II, entitled "Drainage Criteria and Methodology Review", for an in-depth discussion of drainage criteria derivation and the methodology used for the hydrologic and hydraulic analysis of this local watershed.

In analyzing the drainage problems in this watershed area, the initial goal was to determine the capacity of the existing storm drain pipes. The first step of the analysis was to create a hydraulic model of the existing system using the computer program HYDRADAL prepared by Carter & Burgess, Inc. Pipe sizes, reach lengths, and invert elevations were input using data from drainage maps furnished by The City of Orange. The existing system has pipe sizes ranging from a double 3'x5' box at the outfall to 15" diameter reinforced concrete pipes (RCP) at the most upstream end. See Exhibit III.12 entitled "Cherry Ave. Watershed" for a plan view of the existing system. Sub-drainage areas were delineated using existing inlet locations, flow arrows from the City's storm sewer maps, and contour maps. Each sub-area was measured and entered into the program. The C-factor assigned was 0.5 to reflect the residential development with moderate density.

The initial runoff "Time of Concentration" used was 10 minutes. The residential terrain is rather flat, is well vegetated, and runoff must travel several hundred feet to reach the first inlet, therefore, ten minutes was considered adequate.

During analysis, the program increments the initial time of concentration by the flow time in the pipe for each reach. Flow time equals length of pipe divided by flow velocity. The summation of time is then used to determine the rainfall intensity for the next reach of pipe. As discussed previously, as the time increases, rainfall intensity decreases. The input data was then checked for accuracy to complete the first step of analysis.

The second step of the analysis was to run the completed hydraulic model using different year storm frequencies that correspond to different rainfall intensities in an attempt to determine the capacity of the pipes. Larger storm frequencies result in greater rainfall intensities. The storm frequencies evaluated were the 5-year, 2-year, and 1-year events. Starting water surface elevations were also determined and input for each year storm frequency. The starting water surface for Cherry Ave. & 13th St. is the corresponding water surface in Adams Bayou just north of W. Green Ave. After running HYDRADAL, the resulting water surface profile for each evaluation was then compared to the existing street surface elevations to determine if flooding would occur. Flooding was defined for this analysis as "a water surface greater than one foot above the street gutter or inlet elevation." If flooding conditions existed for the 5-year storm then the 2-year storm was analyzed. Similarly, if flooding conditions existed for the 2-year storm then the 1-year storm was analyzed. The conclusions and recommendations of the analysis are presented in the following sections entitled "Conclusions" and "Recommendations." The outputs from HYDRADAL are included in the Appendix.

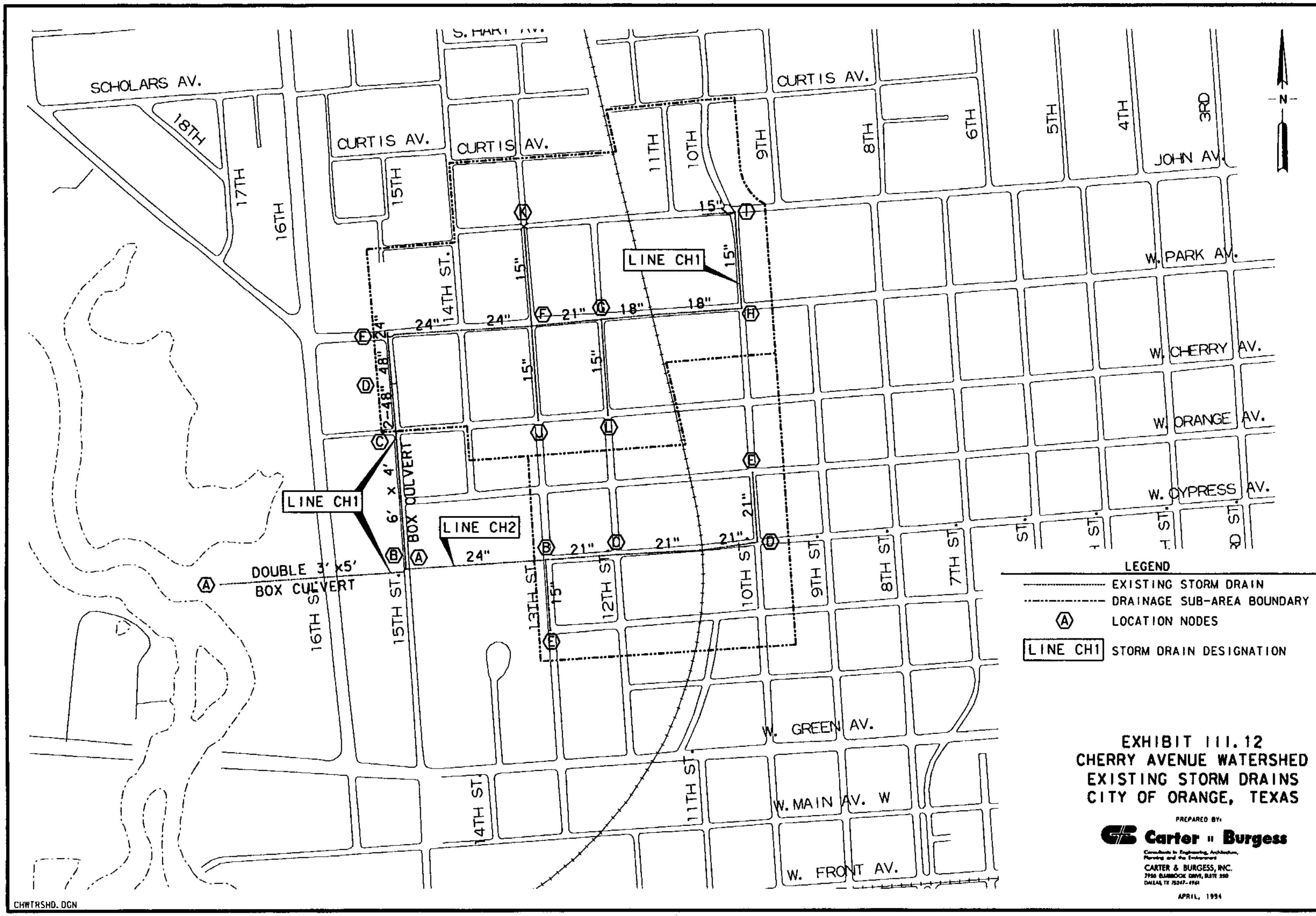


EXHIBIT III.12
CHERRY AVENUE WATERSHED
EXISTING STORM DRAINS
CITY OF ORANGE, TEXAS

2. Coopers Gully

Coopers Gully is another watershed that was designated as prone to flooding. Coopers Gully is a natural drainage way that extends through the center of the City of Orange. A channel consisting of both grassed and concrete lined sections drains the watershed and outfalls through a pump station at the Sabine River. The watershed is roughly bounded by 20th Street on the west between Melwood Ave. and Barkins Ave. and extends southeasterly to the Sabine River in eastern Orange. See Exhibit III.13 for a plan view of the Coopers Gully watershed. The watershed is fully developed with over 90% in single family homes and the remainder in commercial, multi-family, and park land. The watershed encompasses approximately 1,024 acres and slopes from west to east at roughly four feet per mile. The watershed high point is approximately elevation 12' near 16th St. & Barkins Ave.

The City of Orange currently maintains one storm water pump station to aid in the removal of runoff from urbanized areas. This pump station is located at the outfall of Coopers Gully into the Sabine River on the east side of the city. The pump station was improved in 1963. The pump station currently contains four 62,000 gallon per minute (gpm) pumps, two 30,000 gpm pumps and one 15,000 gpm sump pump. Therefore, the total nominal capacity of the station is 323,000 gpm.

The area draining to the pump station is approximately 1,024 acres (1.6 square miles). The drainage area is primarily fully-developed, and has been since the publication of the 1982 Flood Insurance Study for the City of Orange.

Currently, the gravity flow sluice gates at the pump station are lodged in the closed position. Consequently, the station is activated whenever there is runoff in the watershed, regardless of the observed flood levels on the Sabine River.

The peak discharges computed for the Flood Insurance Study at the Coopers Gully pump station are shown in Table III.1.

VIEWS OF COOPERS PUMP STATION

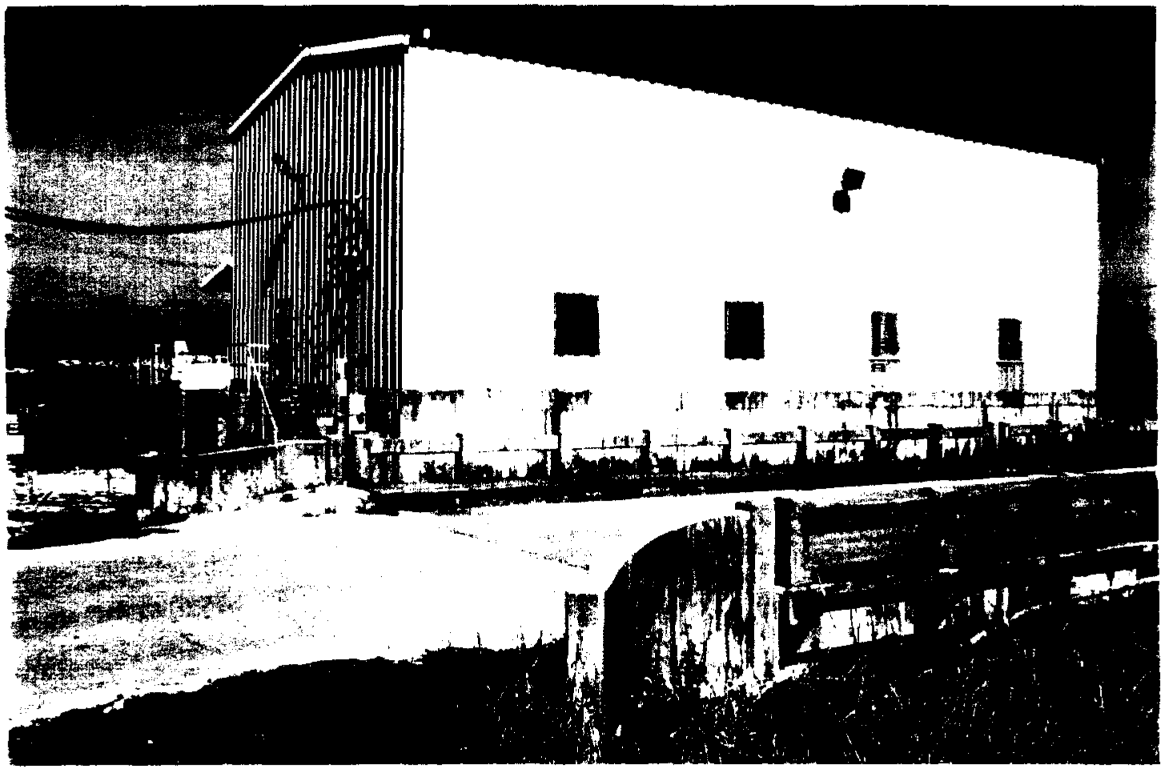


PHOTO III.5 Coopers Pump Station

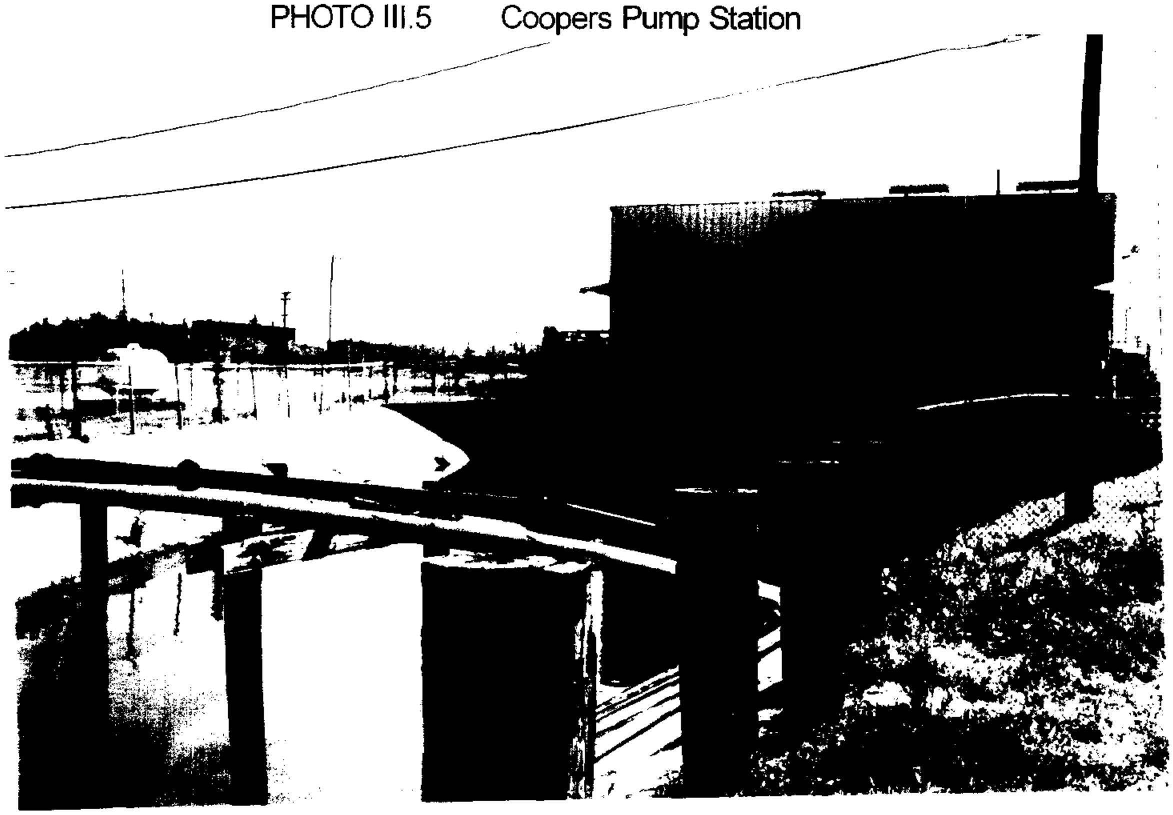


PHOTO III.6 Coopers Pump Station

TABLE III.1 PEAK DISCHARGES AT COOPERS GULLY PUMP STATION

FREQUENCY	DISCHARGE (cfs)
10-Year	1,250
50-year	1,470
100-year	1,640
500-year	1,780

These discharges were verified through the use of the Corps of Engineers' HEC-1 Flood Hydrograph Package computer program. Copies of the computer printouts can be found in the Appendix.

Two-foot contour interval topographic maps of the Coopers Gully watershed were used to determine the available storage volume for use in the analysis of the existing pumping capacity of the station. In conducting the pumping analysis, the assumption was made that the maximum flood elevation that could be allowed was elevation 5.5 ft. The one inch to 200 feet scale base maps of the city showed several structures inside the six feet contour elevation along Coopers Gully. Any elevation below approximately 5.0 ft. appeared to be contained within the channel of Coopers Gully. The finished floor elevations of the individual houses inside the elevation six contour should be verified to determine a more accurate approximation of the maximum ponding elevation that could be allowed along the creek.

The Coopers Gully channel is trapezoidal-shaped from the pump station at the Sabine River all the way to the Southern Pacific railroad crossing at the headwaters. The channel is concrete lined along several reaches including from the pump station to cross section 10+01, from just downstream of East John Avenue to the downstream side of Curtis Avenue, and from the upstream side of 11th Street to the downstream side of the railroad crossing. The City of Orange currently has plans to construct additional concrete lining in the channel from Curtis Avenue to Turret Avenue.

Hydraulic Analysis

Flood protection alternatives for Coopers Gully were analyzed using the U.S. Army Corps of Engineers' Water Surface Profiles HEC-2 computer program. The official version of the Coopers Gully computer model was obtained from the Federal Emergency Management Agency (FEMA). The model was created in March of 1980 for use in the City of Orange Flood Insurance Study that was completed in July of 1982. The model contained water surface elevations in Coopers Gully from the Sabine River to the Limit of Study at the Southern Pacific railroad for the 10-, 50-, 100-, and 500-year storm events. Upon receipt of the model from FEMA, Carter & Burgess revised the cross sections to reflect current conditions and corrected certain errors in the model. The revised version of the existing conditions model is presented in the Appendix. Revisions made include changes in n-value to reflect additional concrete-lined sections, removal of bridges that are no longer in place, and corrections to some of the existing cross sections. See Exhibit III.14 for location of the HEC-2 cross sections. Cross section plots of the revised existing channel are also presented in the Appendix.

For purposes of the Coopers Gully flood protection analysis, the 100-year storm flows were used to evaluate flood protection alternatives. The starting water surface was assumed to originate directly from the Sabine River, thus ignoring any influence of the pump station. The starting water surface elevations for FEMA's analysis as well as the current analysis assumed that the Sabine River was not at flood stage. This is because the one-hundred year water surface elevation of the Sabine River is too high to allow any drainage from Coopers Gully. Flood protection alternatives that are able to protect the City from large floods on the Sabine River are discussed in the sub-section entitled "Watersheds Susceptible to Wide-Scale Flooding." The starting water surface elevation used for FEMA's analysis and this 100-year flow analysis was elevation 1.2'.

Fully-urbanized peak discharges were computed in the Coopers Gully watershed using the U.S. Army Corps of Engineers' HEC-1 Flood Hydrograph Package computer program. The methodology applied to compute these discharges is discussed in Section II, Drainage Criteria and Methodology Review. The discharges computed by FEMA and this study compare favorably. Therefore, the assumption was made that the watershed was essentially fully-developed at the time of the Flood Insurance Study.

Flood Protection Analysis

The primary consideration of the analysis along Coopers Gully was to prevent the flooding of existing structures. To identify the areas where potential structure flooding could occur, the approximate 100-year floodplain of Coopers Gully, without Sabine River effects, was plotted on topographic maps that contained a 2-foot contour interval. Areas where the floodplain boundary encompassed existing structures were considered to be possible structural flooding areas. The chart below shows the published FIS elevations and top widths for a 100-year flood on Coopers Gully. The elevations do not include the effects of the Sabine River flood elevations.

CROSS SECTION	100-YEAR WATER SURFACE ELEV. (Ft)	TOP WIDTH OF FLOODPLAIN (Ft)
A	2.0	100
B	2.0	29
C	3.8	112
D	7.1	1324
E	7.3	1542
F	7.6	820
G	7.9	1266

HEC-2 models of Coopers Gully were obtained from FEMA. Upon investigation, several discrepancies were discovered that have not been resolved with FEMA. In particular, several models were obtained that had widely varying 100-year discharges for the Gully. The printouts that contained the flows published in the FIS report did not produce the computed water surface elevations published in the report. Therefore, there is either a discrepancy in correct flows or the model that contains the correct cross section information for the existing gully. Consequently, the HEC-2 model was not used in the analysis of proposed improvements. Instead, a normal depth analysis using Manning's formula was used to determine proposed channel size improvements. The Federal Highway Administration's culvert analysis program HY8 was used to determine the size of the required proposed culverts.

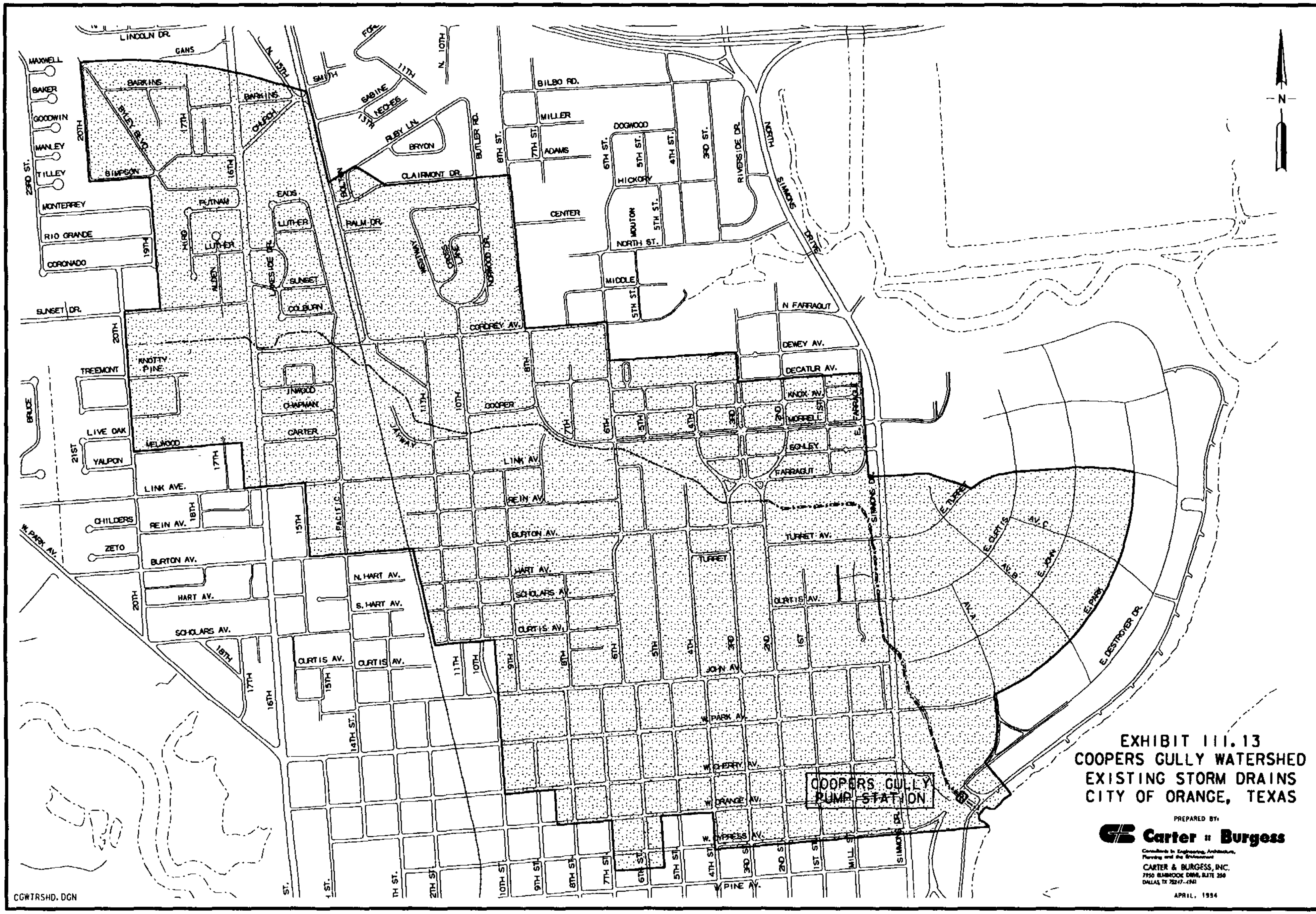


EXHIBIT III.13
COOPERS GULLY WATERSHED
EXISTING STORM DRAINS
CITY OF ORANGE, TEXAS
PREPARED BY:
Carter :: Burgess
CARTER & BURGESS, INC.
APRIL, 1994
COOPERS GULLY PUMP STATION
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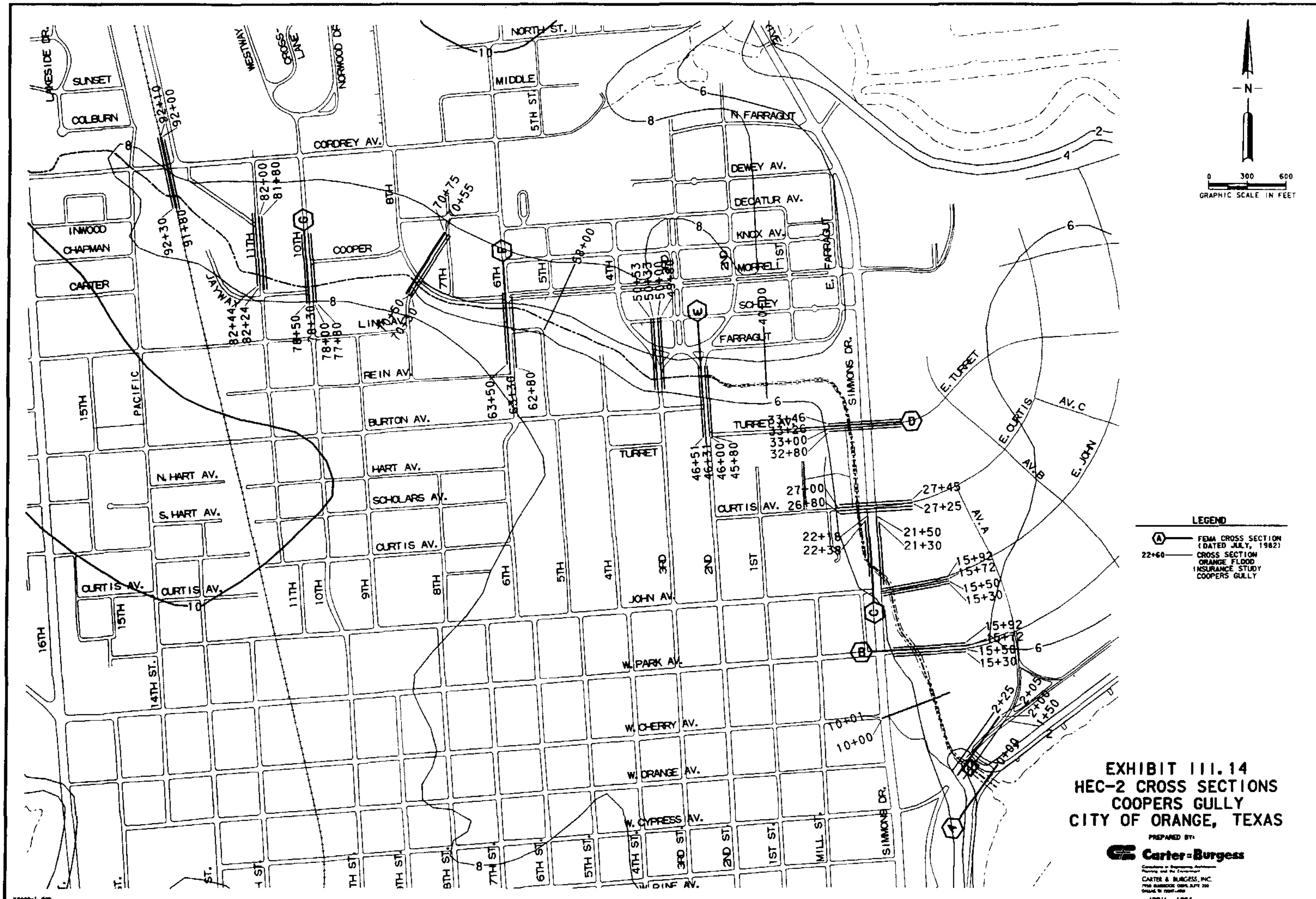


EXHIBIT III.14
HEC-2 CROSS SECTIONS
COOPERS GULLY
CITY OF ORANGE, TEXAS
PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.
APRIL, 1994
LEGEND
FEMA CROSS SECTION (DATED JULY, 1982)
CROSS SECTION ORANGE FLOOD INSURANCE STUDY COOPERS GULLY
GRAPHIC SCALE IN FEET
0
300
600
N
NORTH ST.
MIDDLE
SUNSET
COLBURN
LAKESIDE DR.
WESTWAY
CROSS-LANE
NORWOOD DR.
CORDREY AV.
N. FARRAGUT
DEWEY AV.
DECATUR AV.
KNOX AV.
MORRELL
SCHLEY
FARRAGUT
INWOOD
CHAPMAN
CARTER
COOPER
JAYWAX
LINK AV.
REIN AV.
BURTON AV.
HART AV.
SCHOLARS AV.
CURTIS AV.
N. HART AV.
S. HART AV.
TURRET
JOHN AV.
W. PARK AV.
W. CHERRY AV.
W. ORANGE AV.
W. CYPRESS AV.
PINE AV.
E. TURRET
E. CURTIS
E. JOHN
AV. A
AV. B
AV. C
SIMMONS DR.
PACIFIC
16TH
15TH
14TH ST.
11TH
10TH
9TH
8TH
7TH
6TH
5TH
4TH
3RD
2ND
1ST
E. FARRAGUT
MILL ST.
92+10
92+00
92+30
91+80
82+00
81+80
82+44
82+24
78+50
78+30
78+00
77+80
70+75
70+55
70+50
70+30
63+50
63+30
62+80
58+00
50+53
50+33
50+00
49+80
46+51
46+31
46+00
45+80
40+00
33+46
33+26
33+00
32+80
27+00
26+80
27+45
27+25
22+18
22+38
21+50
21+30
15+92
15+72
15+50
15+30
10+01
10+00
2+25
2+05
2+00
1+50
0+00

3. Dayton Street Ditch

The Dayton Street Ditch watershed encompasses portions of both the City of Orange and the City of West Orange. The watershed is named for Dayton Street in West Orange, the first street crossing encountered upstream from the ditch's outfall. The watershed area encompasses approximately 562 acres and is roughly bounded by Brown Dr. (Hwy. 87) on the west, MacArthur Dr. (Hwy. 87) on the north, Adams Bayou on the east, and the Missouri Pacific Railroad on the south. See Exhibit III.15 for a plan view of the watershed.

The natural ground slope, or drainage pattern, is from west to east. The receiving water is Adams Bayou. The watershed high point is approximately elevation 16' near the railroad intersection at the western-most part of the watershed. The watershed is zoned approximately 70% residential and 30% commercial. Two-thirds of the watershed have been developed. A small storm drain is in place at the upstream end along Bluebonnet Dr. The drain outfalls to a grass swale, or ditch, that flows east along Hwy. 87. The ditch crosses Hwy. 87 via a double 5'x2' RCB and one 36"RCP relief drain. The ditch then flows east and south around the Walmart property and then continues southeasterly to Adams Bayou.

The primary flooding concern in this watershed is the intersection of Bluebonnet and Hwy. 87 at the watershed's upstream end. This intersection floods during even moderate rains and disrupts the flow of traffic, blocks business entrances, and threatens several homes. Parking lots are frequently inundated. Home owners downstream in West Orange along the creek route also attest to water flowing on their properties.

The hydraulic analysis of this watershed and intersection required a two part process. First, a HEC-2 model was created to analyze the open channel, or ditch, flow. Second, an improved storm drain was modeled along Bluebonnet Dr. and Hwy 87. Refer to section II., entitled "Drainage Criteria and Methodology Review" , for an in-depth discussion of drainage criteria derivation and the methodology used for hydrologic and hydraulic analysis.

To create an accurate HEC-2 model it was first necessary to obtain cross section data along the stream. Carter and Burgess contacted Klinkhammer and Associates Surveying of Orange to provide the cross sectional data. Additional information included in the model was obtained from visual survey during site visits, from aerial photographs, and from the Walmart property grading plans. The combined information was then input into the HEC-2 program to create a hydraulic model of existing conditions. The assumed "n" values were 0.04 for maintained natural channel, 0.05 for overgrown natural channel, 0.15 for overbank area, and 0.018 for concrete-lined channel. Because the HEC-2 program is limited to analyzing only one type of culvert at one time, the Hwy. 87 culvert was entered in as a double 2'x7' RCB to model the existing double 2'x5' RCB and 36" RCP relief drain. Discharges were calculated using the Rational Equation and the 5-year storm intensity values. The beginning water surface used was elevation 3.3 ft., which is the 5-year flow elevation in Adams Bayou at the Dayton Street confluence. The model of existing conditions was then run and reviewed. See Exhibit III.16, for a plan view of HEC-2 cross sections. The existing condition HEC-2 output is included in the Appendix.

The model of existing conditions resulted in a water surface elevation of 11.3 ft. at the Bluebonnet and Hwy. 87 intersection. This water surface is clearly too high to allow drainage of the intersection since the approximate pavement elevation is 11.0'. Under these conditions, the existing inlets cannot accept any additional water and the intersection floods.

To alleviate this condition, several improvements were modeled in to the existing HEC-2 model to lower the water surface. The first set of improvements considered were changes in n-values to reflect a properly maintained (primarily mowed) channel and to include additional portions of concrete-lining. An n-value change from 0.05 to 0.04 to reflect proper maintenance upstream of the Davis Street crossing resulted in a final water surface of 11.1 feet at Bluebonnet Drive. All further analysis assumed proper channel maintenance with an n-value of 0.04. This change had little impact on draining the intersection. Next, a change in n-value from .04 to .018 to reflect concrete lining from the Walmart culverts to Bluebonnet resulted in a final water surface of 10.6 feet. Again, this change had little impact on allowing the intersection to drain.

The second set of improvements considered were changes to the culverts at Hwy. 87. In the existing model the water surface jumped 0.5 feet at this crossing which indicates a "bottleneck" or constriction of flow. The culvert sizes were increased from double 2'x7' RCBs to double 4'x7' RCBs. The difference in flowline elevation between the upstream and downstream sides of Highway 87 will allow the larger culverts. The resulting water surface was 10.9 feet at Bluebonnet Drive. Again, this was not a significant improvement toward draining the intersection. Next, the ditch was regraded holding a constant slope of 0.2% from Walmart's upstream set of 5'x5' boxes at elevation 4.0' to the Bluebonnet intersection. The resulting invert elevation change was from 7.2 feet to 5.75 feet at the intersection. This change was necessary to allow any improvements to the storm drain in Bluebonnet. The resulting water surface elevation at Bluebonnet was 10.3 feet. To attempt to lower the water surface even further, concrete lining of the channel was modeled in from the Walmart boxes to Bluebonnet. The resulting water surface was 10.0 feet.

The third set of improvements considered in were changes to the culverts in West Orange at Shell Dr. and Tanglewood Dr. (Note: The rest of the model remained as existing conditions.) In the existing model the water surface jumped 1.5 feet at the Tanglewood culverts indicating a constriction of flow. Shell Drive has only two 48" concrete pipes for conveyance and Tanglewood has only three 36" pipes. Both of these "bottlenecks" were eliminated by increasing the culvert sizes at each intersection to dual 6'x4' boxes and regrading from Cross Section 4250 to Cross Section 5900 to increase channel capacity and to create a consistent slope. The slope was held at 0.13%. The resulting water surface was 10.8 feet which indicates the downstream improvements have little impact on the Bluebonnet intersection. However, the water surface was lowered greater than a foot between Cross Sections 4845 and 5967 in West Orange. This decreased water surface will result in lower flooding potential for homes along those reaches in West Orange. Next, concrete lining of the channel from the Walmart boxes to Bluebonnet was modeled in. The resulting water surface was 10.1 feet.

The final set of improvements modeled in was the combination of the first three. The channel was regraded from Cross Section 4250 to Cross Section 5900 and regraded and lined with concrete from the Walmart boxes to Bluebonnet and culverts were improved at Tanglewood, Shell, and Hwy. 87. The resulting water surface at Bluebonnet was 9.4 feet, for a total decrease of 2.0 feet from original conditions.

The second step of analysis was to model an improved storm drain along Bluebonnet Drive. The water flows south down Bluebonnet and concentrates in the sump at the Hwy. 87 intersection. The existing drain consists of only two 4 ft. long low point inlets connected by a 15" lateral pipe with a 24" RCP outfall to a grass lined ditch. See Exhibit III.17 for a plan view of the existing system.

The 5-year flow for this subarea was calculated to be 106 cfs using the Rational Equation. However, since the existing water surface elevation at the upstream end of the ditch along Highway 87 was 11.2', the pipe will actually carry even less flow.

An improved storm drain system was considered along Bluebonnet to more efficiently convey water away from the low point at the intersection. The improved system's starting water surface was 9.4 ft. to reflect an improved Dayton Street Ditch as described above. Double 5'x3' RCPs were used for the outfall and majority of the mainline. Boxes were chosen since there was not enough cover for an equivalent pipe. Six 10' inlets were considered, four in the sump area and two located approximately 200' upstream to intercept some of the flow down Bluebonnet.

In order to determine the downstream effects of enlarging the culverts in the Dayton St. ditch, an analysis was performed to calculate the impact that larger culverts would have on time of concentration or ponded storage along the ditch. Theoretically, enlarging the culverts could negate the ponding effects that the existing culverts provide and increase the flow downstream of the enlarged culvert.

The ponding analysis performed for the Dayton St. ditch centered on the culverts at Tanglewood Drive. According to the hydraulic analysis performed on the existing Dayton St. ditch culverts, the culverts at Tanglewood Dr. appeared to be causing the largest restriction to flow in the ditch. When a culvert causes a restriction in ditch flow, storm runoff tends to pond, or be stored, on the upstream side of the culvert. If the ponding effects are large enough, the flow released to the downstream side of the culvert will be smaller than the incoming flow in the ditch on the upstream side of the culvert. The degree to which the culvert will affect downstream flows in the ditch depends on the amount of water that can be stored in the ditch and surrounding areas on the upstream side of the culvert. As flow approaches the culverts, it begins to be restricted due to the size of the culvert opening. As it is restricted, it begins to back up in the upstream storage areas. If the storage area were large enough, and the culvert opening small enough, theoretically all of the incoming runoff could be held in the storage area and no flow would be released downstream. If the ponding area upstream of the culvert is small, the water will fill up the ponding area quickly and over top the culvert. If the over topping occurs relatively soon, the flow continues downstream as if there were no culvert or restriction. In this case the ability of the culvert to act as a flow restriction would be lessened.

A storage analysis was performed for the Tanglewood Dr. culverts to determine if the existing culverts were having much effect on the flows in the ditch downstream. Without the culverts in the ditch at Tanglewood Dr., the five-year flow in the ditch is 176 cfs. With the existing three 36-inch diameter RCP's in the ditch and allowing for the storage that could be provided in the upstream areas, the resulting flow that would be released through the culverts is 163 cfs. Therefore, the existing culverts appear to be lowering the five-year flow from an unrestricted 176 cfs down to 163 cfs. Using the 163 cfs as the ditch flow in the hydraulic analysis, and comparing the resulting water surface elevations with those resulting from a flow of 176 cfs, the water surface elevation in the ditch dropped 0.1 ft from the full unrestricted flow of 176 cfs. Consequently, the smaller culverts do not appear to be having much effect on lowering the flows in the ditch. The flow lowering that does occur does not result in much lowering of the water surface elevation in the ditch.

The results of this analysis apply to all of the existing culverts in the Dayton St. ditch. Although the culverts do pond water on the upstream side, the ponding area is small enough and over topping of the culverts occurs soon enough that the culverts are not acting as enough of a flow restriction to affect water surface elevations in the ditch. Consequently, enlarging the culverts in the ditch as recommended in this report will not have a significant effect on the flows in the ditch.

The results of the analysis are discussed in the Conclusions and Recommendations sections.

DAYTON STREET DITCH PHOTOS

PHOTO III.7 View Along Hwy. 87 Toward Bluebonnet

PHOTO III.8 View of Culverts at Hwy. 87

DAYTON STREET DITCH PHOTOS

PHOTO III.9 View of Tanglewood Culverts

PHOTO III.10 View of South St. Culvert

DAYTON STREET DITCH PHOTOS

PHOTO III.11 View of Holly Rd. Culverts

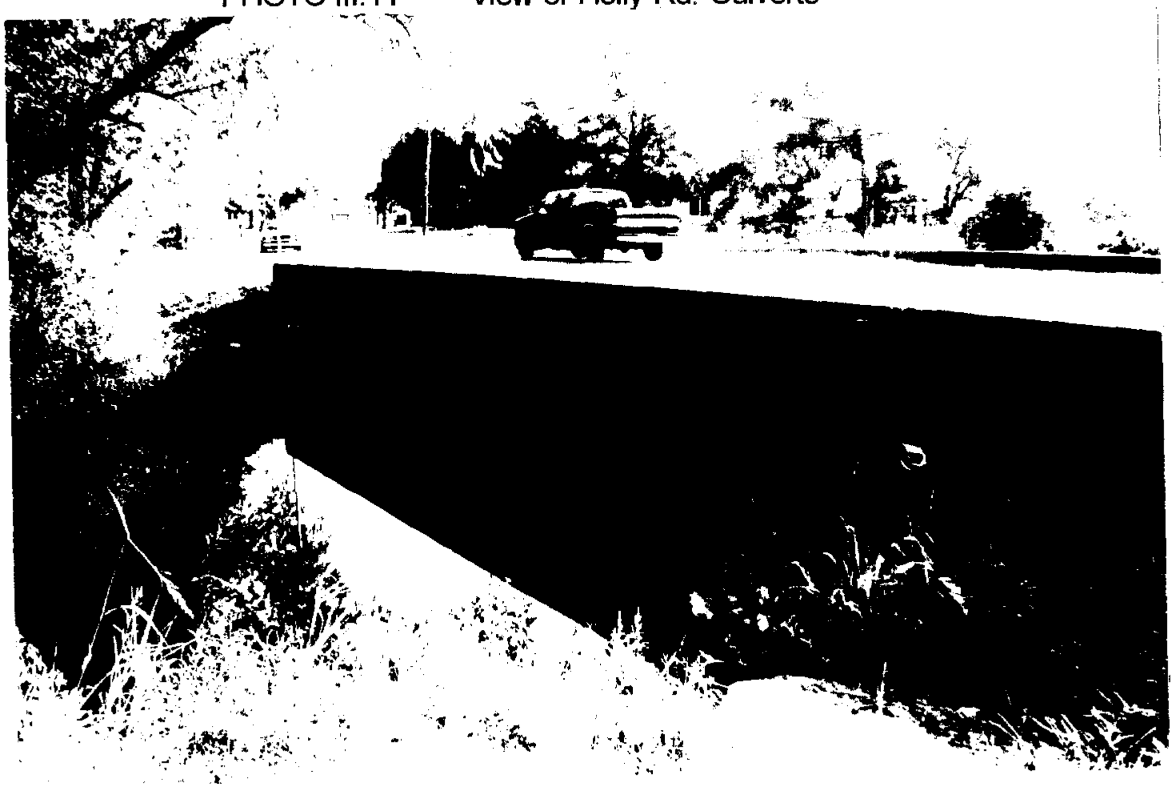


PHOTO III.12 View of Dayton St. Culverts

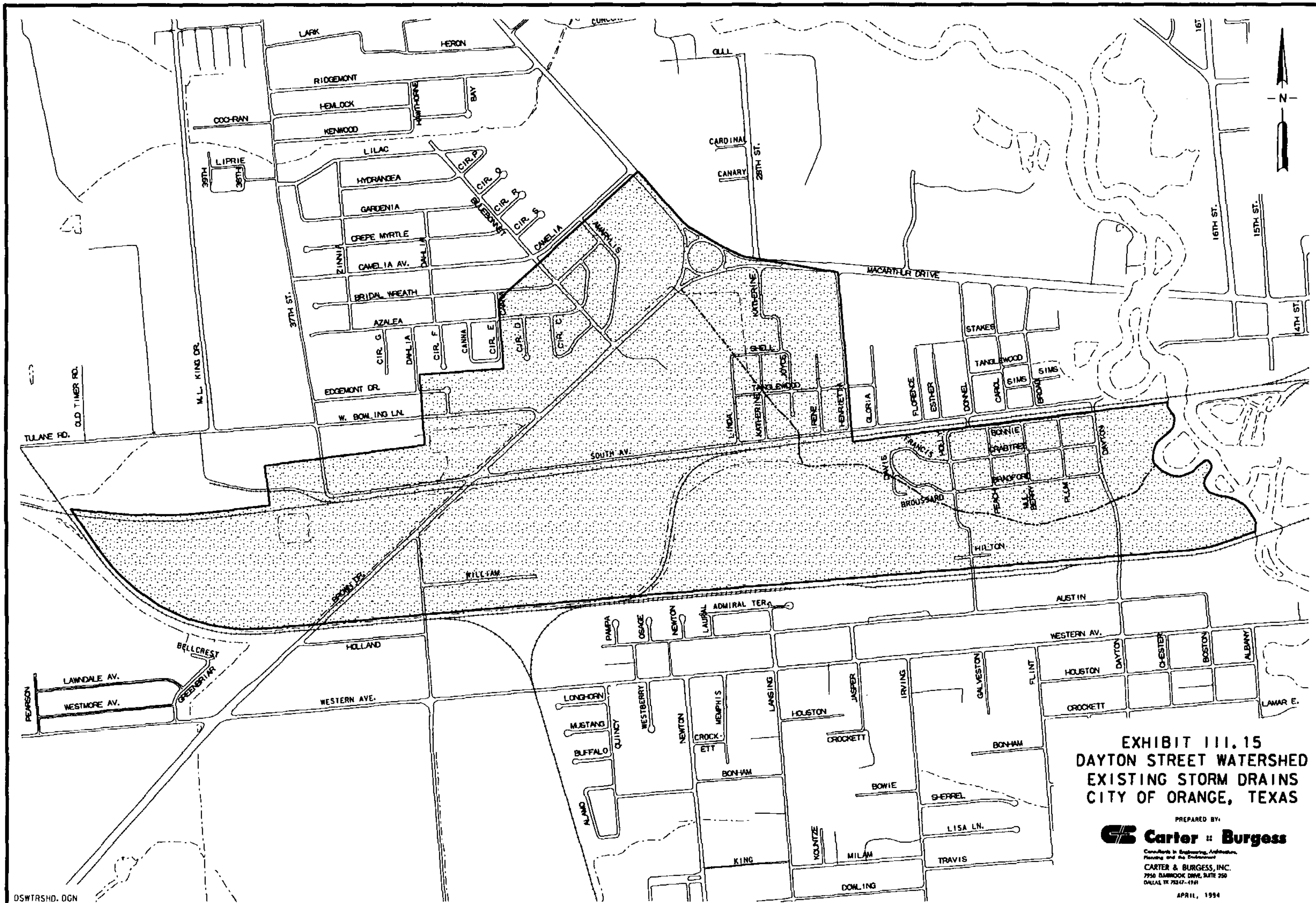


EXHIBIT III.15
DAYTON STREET WATERSHED
EXISTING STORM DRAINS
CITY OF ORANGE, TEXAS

PREPARED BY:

Carter :: Burgess

Consultants in Engineering, Architecture, Planning and the Environment

CARTER & BURGESS, INC.
7950 ELMBROOK DRIVE, SUITE 250
DALLAS, TX 75247-4961

APRIL, 1994

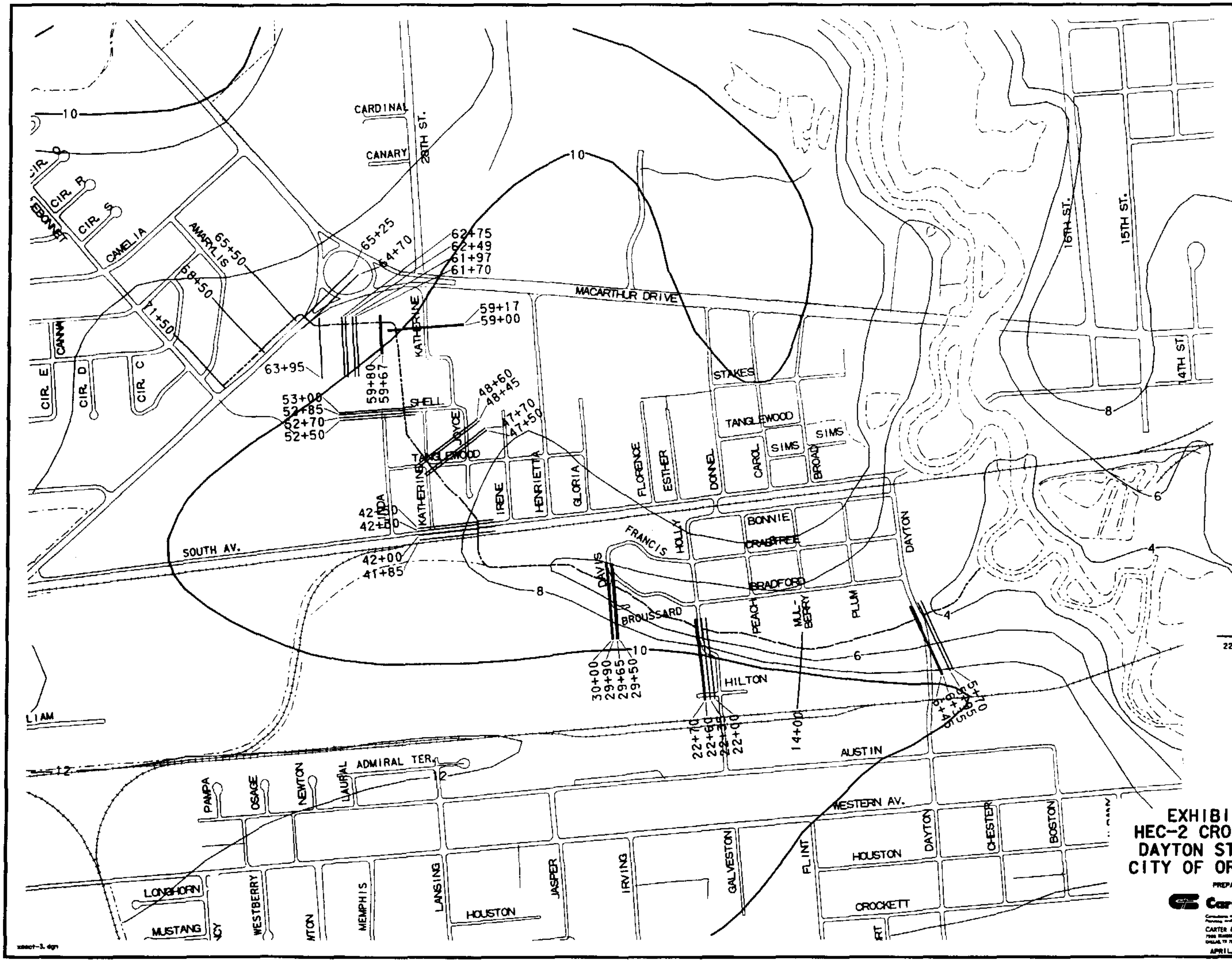


EXHIBIT III.16
HEC-2 CROSS SECTIONS
DAYTON STREET DITCH
CITY OF ORANGE, TEXAS

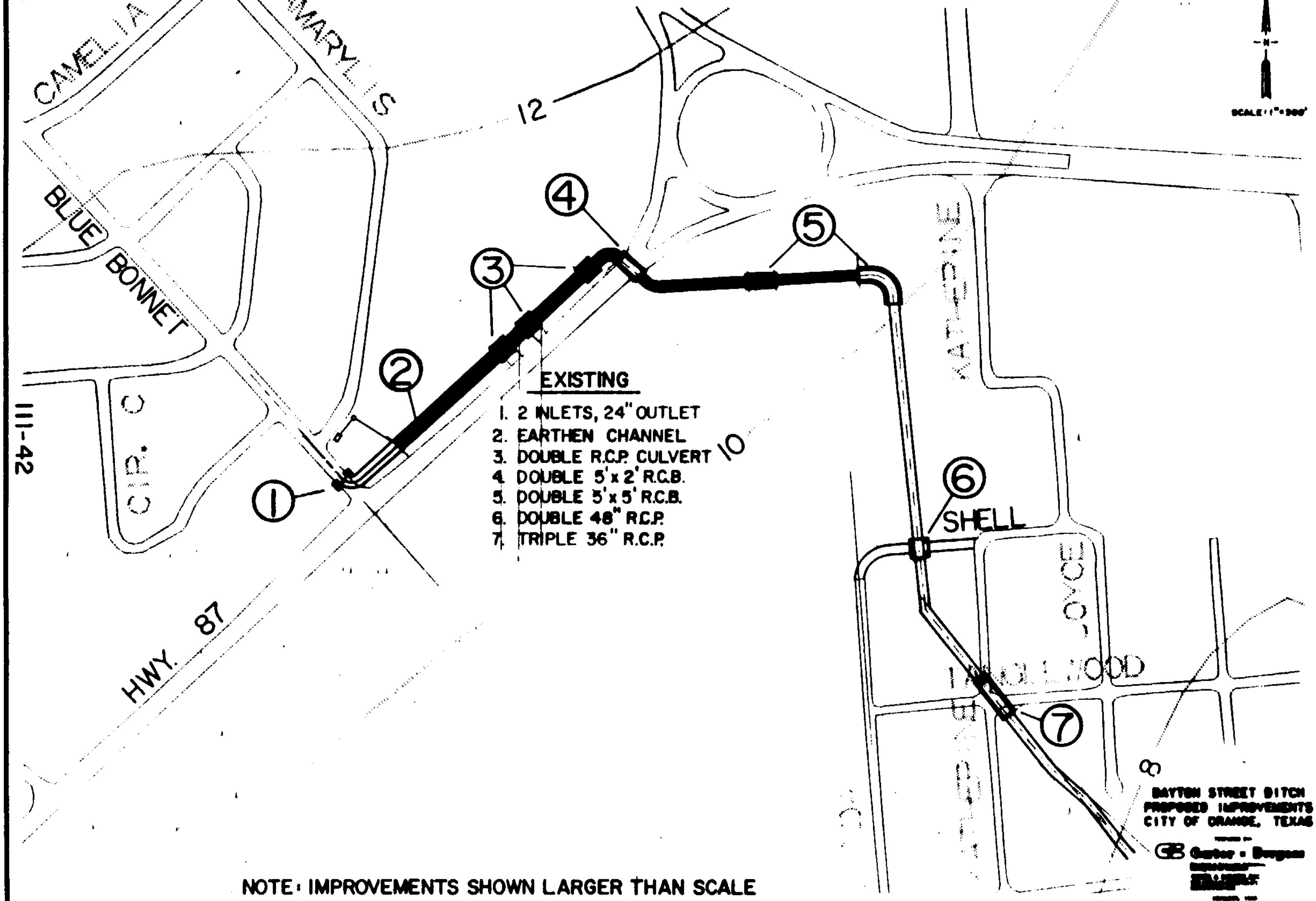
PREPARED BY:

Carter & Burgess

CARTER & BURGESS, INC.

APRIL, 1994

EXHIBIT III.17 Bluebonnet Existing Storm Drain Schematic



4. Hudson Gully

Hudson Gully is another watershed that was designated as prone to flooding. Hudson Gully is a large natural drainage way that encompasses the Roselawn Addition west of Adams Bayou. A channel consisting of both natural and concrete lined sections drains the watershed and outfalls directly to Adams Bayou just behind Baptist Hospital of Orange. The watershed is roughly bounded by the Missouri Pacific Railroad on the west, Mockingbird St. on the north, Adams Bayou on the east, and Hwy. 87 on the south. Approximately one-third of the watershed is developed as residential and the remaining two-thirds is undeveloped land. The watershed encompasses approximately 910 acres and slopes from west to east at roughly four feet per mile. The watershed high point is approximately elevation 17' near the intersection of the Missouri Pacific Railroad and Strickland Dr.

The Hudson Gully channel is trapezoidal-shaped along its whole reach. The channel is concrete lined from Cross Section 2261, just west of Strickland at Bay, to Cross Section 5532, at 37th St. (also known as Old Airport Rd).

Hydraulic Analysis
Flood protection alternatives for Hudson Gully were analyzed using the U.S. Army Corps of Engineers' Water Surface Profiles HEC-2 computer program. The official version of the Hudson Gully computer model was obtained from the Federal Emergency Management Agency (FEMA). The model was created in May of 1980 for use in the City of Orange Flood Insurance Study that was completed in July of 1982. The model simulated water surface elevations in Hudson Gully from Adams Bayou to the 37th St. bridge, the Limit of Study, for the 10-, 50-, 100-, and 500-year storm events. Upon receipt of the model from FEMA, Carter & Burgess revised the cross sections to correct certain data entry errors in the model. The revised version of the existing conditions model is presented in the Appendix. Cross section plots of the revised existing channel are also presented in the Appendix.

For purposes of the Hudson Gully flood protection analysis, the 100-year storm flows were used to evaluate flood protection alternatives. The starting water surface was obtained from Adams Bayou at the confluence with Hudson Gully.

Flood Protection Analysis
The primary consideration of the analysis along Hudson Gully was to prevent the direct flooding of existing structures. The secondary consideration was to lower the hydraulic grade line in the channel to allow better drainage of the existing storm sewer pipes that drain into Hudson Gully. To identify the areas where potential structure flooding could occur, the approximate 100-year floodplain of Hudson Gully was plotted on topographic maps that contained a 2-foot contour interval. Areas where the floodplain boundary encompassed existing structures were considered to be possible structural flooding areas. In the analysis, only one such reach of the gully was identified. The 100-year floodplain of Hudson Gully was contained within the channel banks for the remainder of the reaches along the gully. See Exhibit III.18 for the HEC-2 cross section locations.

Storm Drain Analysis
As mentioned in the watershed description, an existing storm drain system is in place. A discussion of the existing system's hydrologic and hydraulic analysis follows.

Refer to section II entitled "Drainage Criteria and Methodology Review" , for an in-depth discussion of drainage criteria derivation and the methodology used for hydrologic and hydraulic analysis.

An analysis was conducted of the storm drain pipes on the southern bank of Hudson Gully to determine their capacity for conveying runoff. The first step of the analysis was to create a hydraulic model of the existing drains using the computer program HYDRADAL. Pipe sizes, reach lengths, and invert elevations were input using data from drainage maps furnished by The City of Orange. See Exhibit III.19 entitled "Hudson Gully Watershed." The existing drains all outfall directly to Hudson Gully. Sub-drainage areas were delineated using existing inlet locations, flow arrows from the City's storm sewer maps, and contour maps. Each sub-area was measured and input into HYDRADAL as acres. The C-factor used was 0.5 to reflect the residential development. The initial Time of Concentration used was 10 minutes. The residential terrain is rather flat, is well vegetated, and runoff must travel several hundred feet to reach the first inlet. Therefore, ten minutes was considered adequate. The initial time of concentration can be calculated as described in the "Drainage Criteria and Methodology Review", however, the maximum value of 10 minutes was used for the reasons discussed earlier in the section on Cherry Ave. and 13th Street.

The second step of the analysis was to run the completed hydraulic model using different year storm frequencies that correspond to different rainfall intensities. Larger storm frequencies result in greater rainfall intensities. The storm frequencies evaluated were 5-year, 2-year, and 1-year events. Starting water surface elevations were also determined and input for each storm frequency. The starting water surface elevations were obtained from the HEC-2 analysis discussed above. After running HYDRADAL, the resulting water surface profile for each evaluation was then compared to the existing street surface elevations to determine if flooding would occur. Flooding was defined for this analysis as "a water surface greater than one foot above the street gutter or inlet elevation." If flooding conditions existed for the 5-year storm then the 2-year storm was analyzed. Similarly, if flooding conditions existed for the 2-year storm then the 1-year storm was analyzed. The conclusions and recommendations of the analysis are presented in the following sections entitled "Conclusions" and "Recommendations." The outputs from HYDRADAL are included in the Appendix.

HUDSON GULLY PHOTOS

PHOTO III.13 View of Concrete Lined Channel D/S of 37th St.

PHOTO III.14 View of Natural Channel U/S of 37th St.

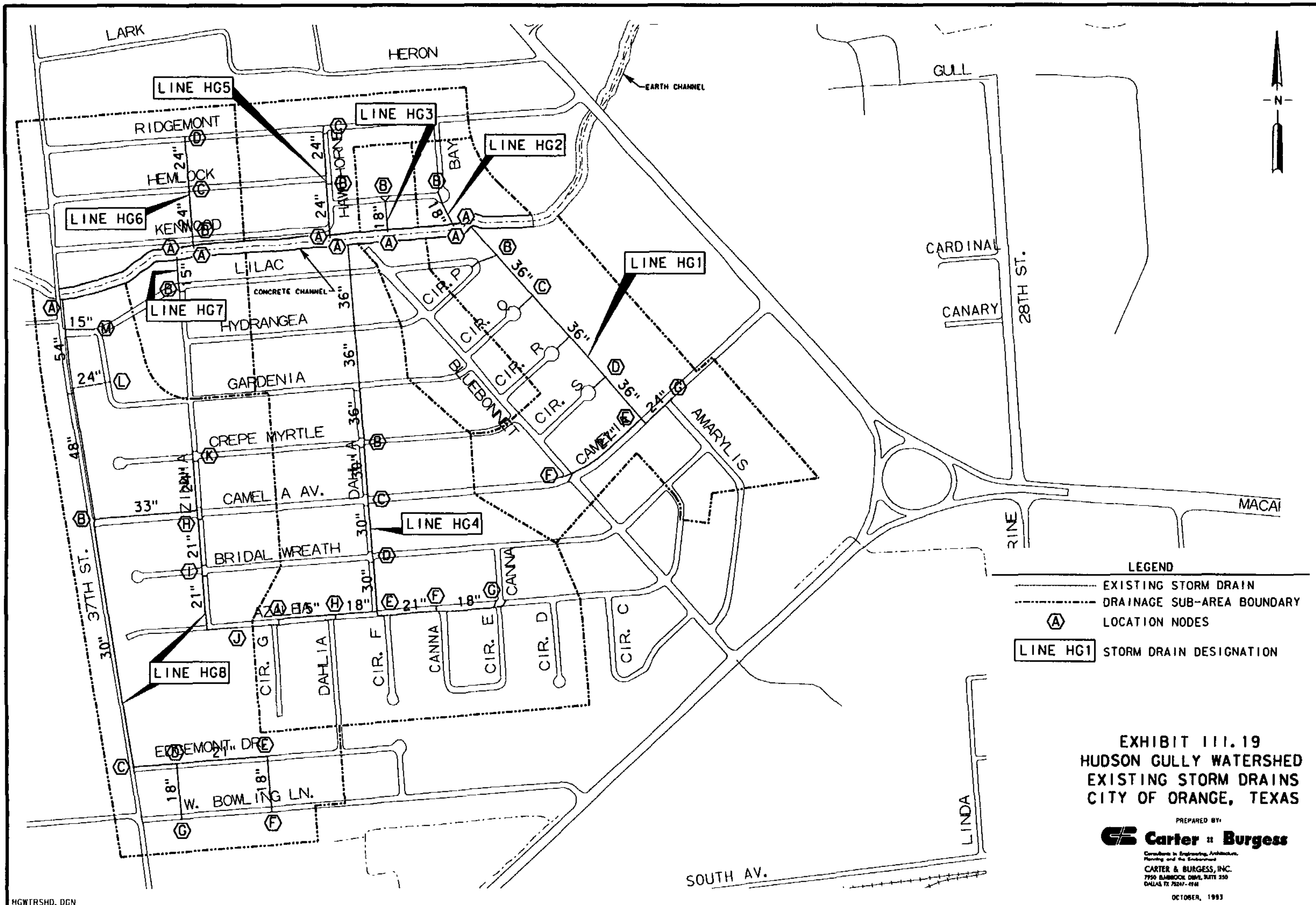


EXHIBIT III.19
HUDSON GULLY WATERSHED
EXISTING STORM DRAINS
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter :: Burgess
CARTER & BURGESS, INC.

OCTOBER, 1993

5. North Simmons Drive

The residential area known as Brownwood located southwest of Interstate 10 and Simmons Drive intersection is another area designated by the City of Orange as prone to flood. The watershed was named for its major thoroughfare and eastern boundary, the northern portion of Simmons Drive.

The North Simmons Drive watershed is roughly bounded by Interstate 10 on the north, 6th St. on the west, Dewey Ave. on the south, and Simmons Drive on the east. The natural ground slope, or drainage pattern is from northwest to southeast. The receiving water is Little Cypress Bayou. The watershed high point is approximately elevation 10' near 6th St. and Dogwood St. The watershed is fully developed as residential. An existing storm drain system is in place. A discussion of the existing system's hydrologic and hydraulic analysis follows.

Refer to Section II, entitled "Drainage Criteria and Methodology Review," for an in-depth discussion of drainage criteria derivation and the methodology used for hydrologic and hydraulic analysis in this study.

An analysis was conducted on the storm drain pipes in the vicinity of North Simmons Drive to determine their hydraulic capacity. The first step of the analysis was to create a hydraulic model of the existing drains using the computer program HYDRADAL. Pipe sizes, reach lengths, and invert elevations were input using data from drainage maps furnished by The City of Orange. The existing drains all outfall to a small tributary of Little Cypress Bayou that crosses beneath Simmons Drive. See Exhibit III.20 entitled "North Simmons Drive Watershed" for a plan view of the existing system. Sub-drainage areas were delineated using existing inlet locations, flow arrows, and contour maps. Each sub-area was measured and input into HYDRADAL as acres. A C-factor of 0.5 was used to reflect the residential development in the sub-watershed. The initial Time of Concentration used was 10 minutes.

The second step of the analysis was to run the completed hydraulic model using different year storm frequencies that correspond to different rainfall intensities. The storm frequencies evaluated were the 5-year, 2-year, and 1-year events. Starting water surface elevations were also determined and input for each year storm frequency. The starting water surface for the North Simmons area is the corresponding water surface in the Sabine River at the mouth of Little Cypress Bayou.

After running HYDRADAL, the resulting water surface profile for each evaluation was then compared to the existing street surface elevations to determine if flooding would occur. If flooding conditions existed for the 5-year storm then the 2-year storm was analyzed. Similarly, if flooding conditions existed for the 2-year storm then the 1-year storm was analyzed. The conclusions and recommendations of the analysis are presented in the following sections entitled "Conclusions" and "Recommendations." The outputs from HYDRADAL are included in the Appendix.

In addition to the existing storm drains, another cause of potential flooding was also analyzed. It was found that some flooding can be due to backwater conditions that occur along Little Cypress Bayou. A backwater condition occurs whenever the Sabine River rises high enough to cause Little Cypress Bayou to back up and flow through the two existing box culverts beneath Simmons Drive. The two existing box culverts run beneath Simmons Drive and outfall into Little Cypress Bayou. The first culvert is located between N. Farragut Ave. and North Street and measures 7' x 7'. The second culvert is located just north of Hickory St. and measures 3' X 3'. To alleviate the backwater condition in this area, it is recommended that sluice and flap gates be installed at the two existing box culvert locations. The installation of sluice and flap gates will allow water to flow in one direction only, out toward Little Cypress Bayou and not in from it. The flap gate works automatically and the sluice gate operates manually to provide positive closure in case the flap gate ever jams in the open position.

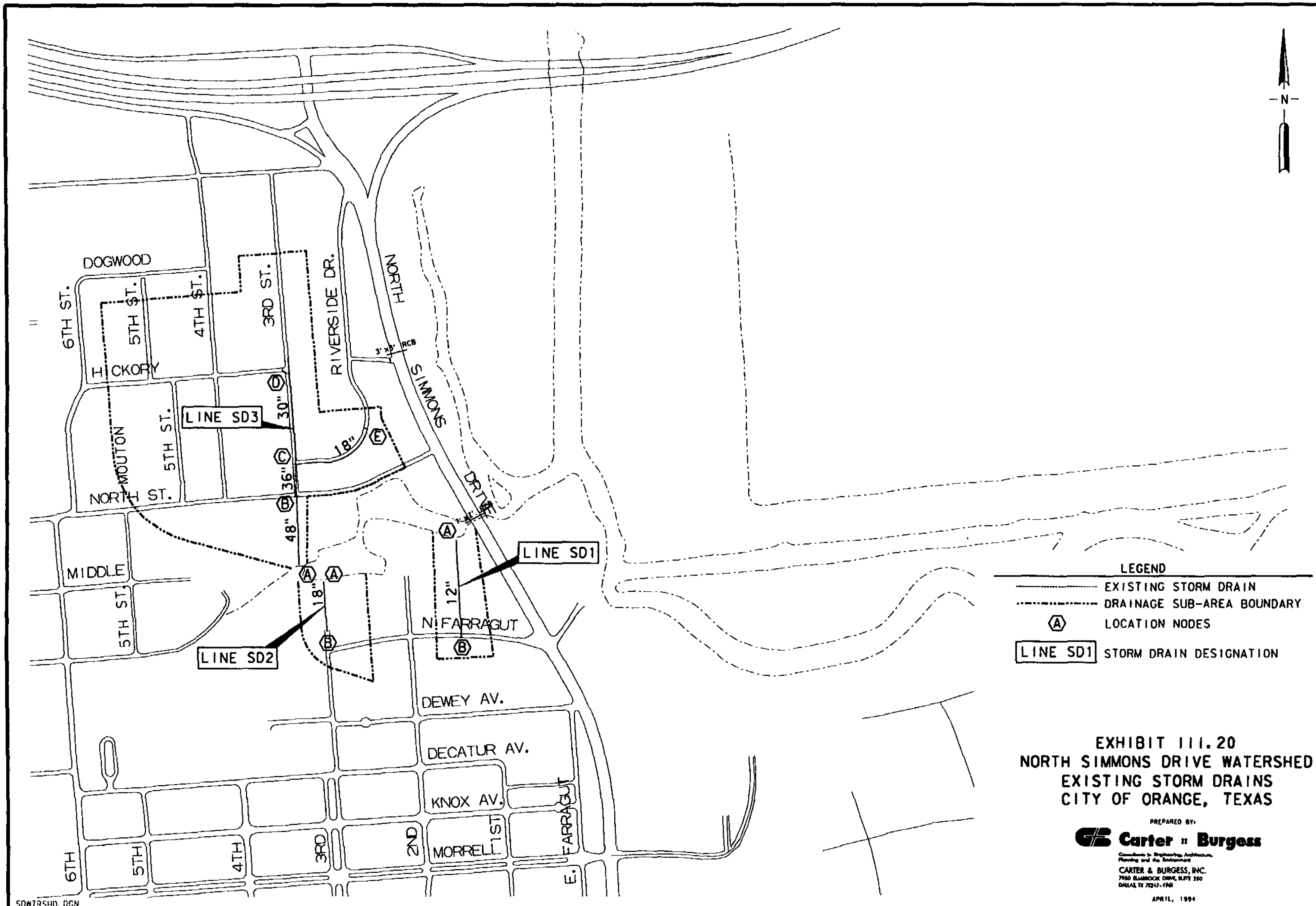


EXHIBIT III.20
NORTH SIMMONS DRIVE WATERSHED
EXISTING STORM DRAINS
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.

APRIL, 1994

6. Old Town Area

An area of downtown Orange was also designated by the City of Orange as an area prone to flood. The watershed is located in an area referred to as Old Town.

The Old Town watershed is roughly bounded by Cypress Ave. on the north, 9th St. on the west, Polk Ave. on the south, and the Sabine River on the east. The natural ground slope, or drainage pattern, is from the northwest to the southeast. The receiving water is the Sabine River. The watershed high point is approximately elevation 9' near the intersection of Pine Ave. and 7th St. The watershed is fully developed as approximately 80% commercial and 20% residential. An existing network of storm drains is in place.

The first step of the analysis was to create a hydraulic model of the existing drains using the computer program HYDRADAL. Pipe sizes, reach lengths, and invert elevations were input using data from drainage maps furnished by The City of Orange. The existing drains all outfall to the Sabine River via 36" or smaller RCP's. See Exhibit III.21 entitled "Old Town Watershed" for a plan view of the existing system. Sub-drainage areas were delineated using existing inlet locations, flow arrows, and contour maps. Each sub-area was measured and input into HYDRADAL as acres. The initial runoff Time of Concentration used was 10 minutes. The residential terrain is rather flat, is well vegetated, and runoff must travel several hundred feet to reach the first inlet. Therefore, ten minutes was considered adequate. The initial time of concentration can be calculated as described in the "Drainage Criteria and Methodology Review", however, many cities assign maximum values for the initial Tc. Most designers use the maximum value for Tc and forego the calculations. The maximum values tend to be slightly conservative for larger sub-drainage areas. In the analysis, the program increments the initial time of concentration by the flow time in the pipe for each reach. Flow time equals flow velocity times length of pipe. The summation of time is then used to determine the rainfall intensity for the next reach of pipe. As discussed previously, as the time increases, rainfall intensity decreases. The input data was then checked for accuracy to complete the first step of analysis.

The second step of the analysis was to run the completed hydraulic model using different year storm frequencies that correspond to different rainfall intensities. The storm frequencies evaluated were the 5-year, 2-year, and 1-year events. Starting water surface elevations were also determined and input for each year storm frequency. The starting water surface for the Old Town area is the corresponding water surface in the Sabine River near the north end of Orange Harbor Island. After running HYDRADAL, the resulting water surface profile for each evaluation was then compared to the existing street surface elevations to determine if flooding would occur. If flooding conditions existed for the 5-year storm then the 2-year storm was analyzed. Similarly, if flooding conditions existed for the 2-year storm then the 1-year storm was analyzed. The conclusions and recommendations of the analysis are presented in the following sections entitled "Conclusions" and "Recommendations." The outputs from HYDRADAL are included in the Appendix.

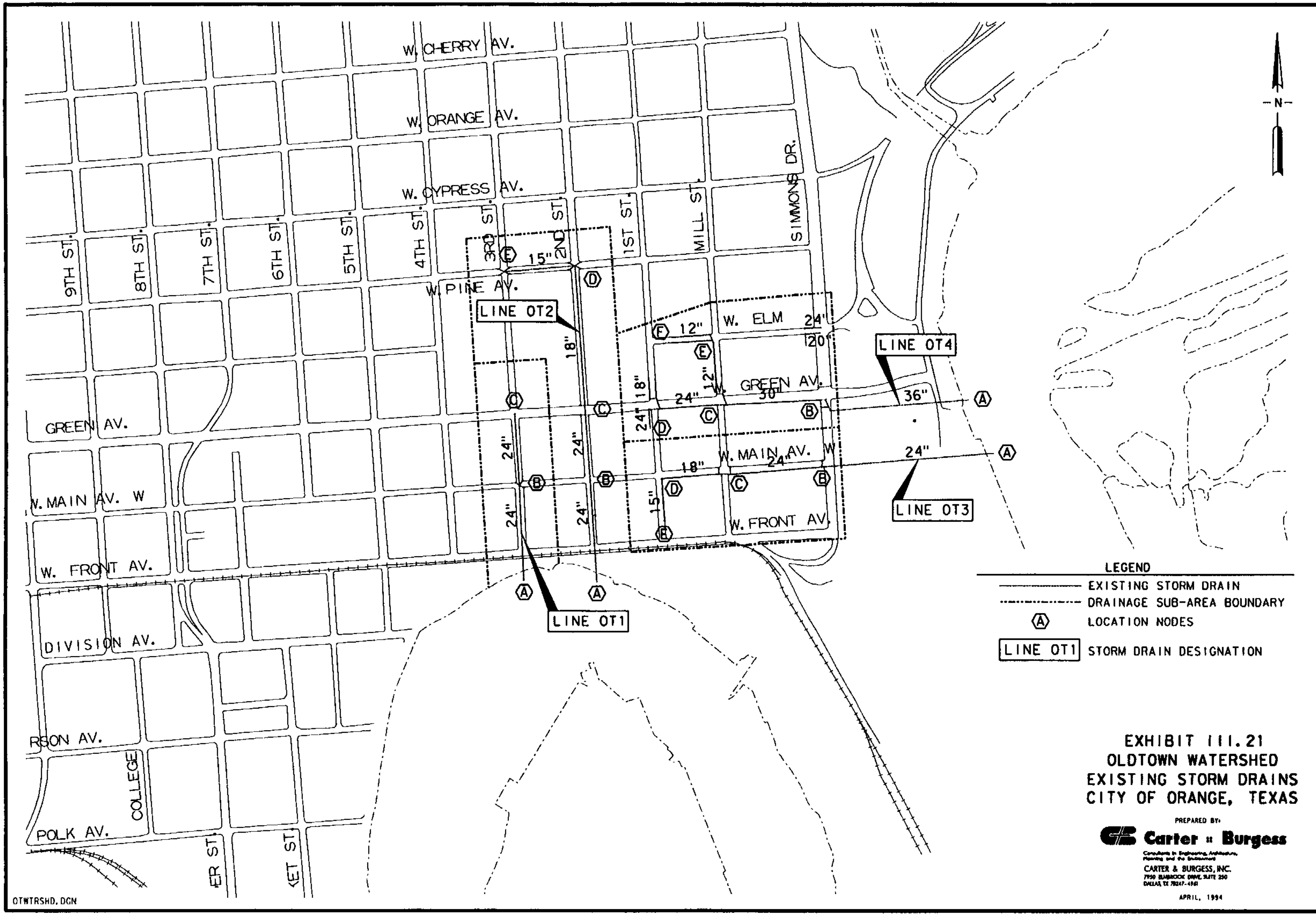


EXHIBIT III.21
OLDTOWN WATERSHED
EXISTING STORM DRAINS
CITY OF ORANGE, TEXAS

7. Sunset Drive

Areas in the Charlemont Addition near Sunset Drive and 23rd St. were also designated as prone to flood. The watershed was named Sunset Drive as the longest reach of the drain is on that street.

The Sunset Drive watershed is roughly bounded on the west by the eastern edge of the Sunset Grove Country Club golf course, on the north by Maxwell Drive, on the east by 19th and 20th Streets, and on the south by Sunset Drive. The watershed encompasses approximately 62 acres. The natural ground slope, or drainage pattern, is from the northeast to the southwest. The drain outfalls directly to Adams Bayou. The watershed high point is approximately elevation 12' near 23rd St. and Tilley Circle. The area is developed as purely residential. An existing network of storm drains is in place. A discussion of the existing system's hydrologic and hydraulic analysis follows.

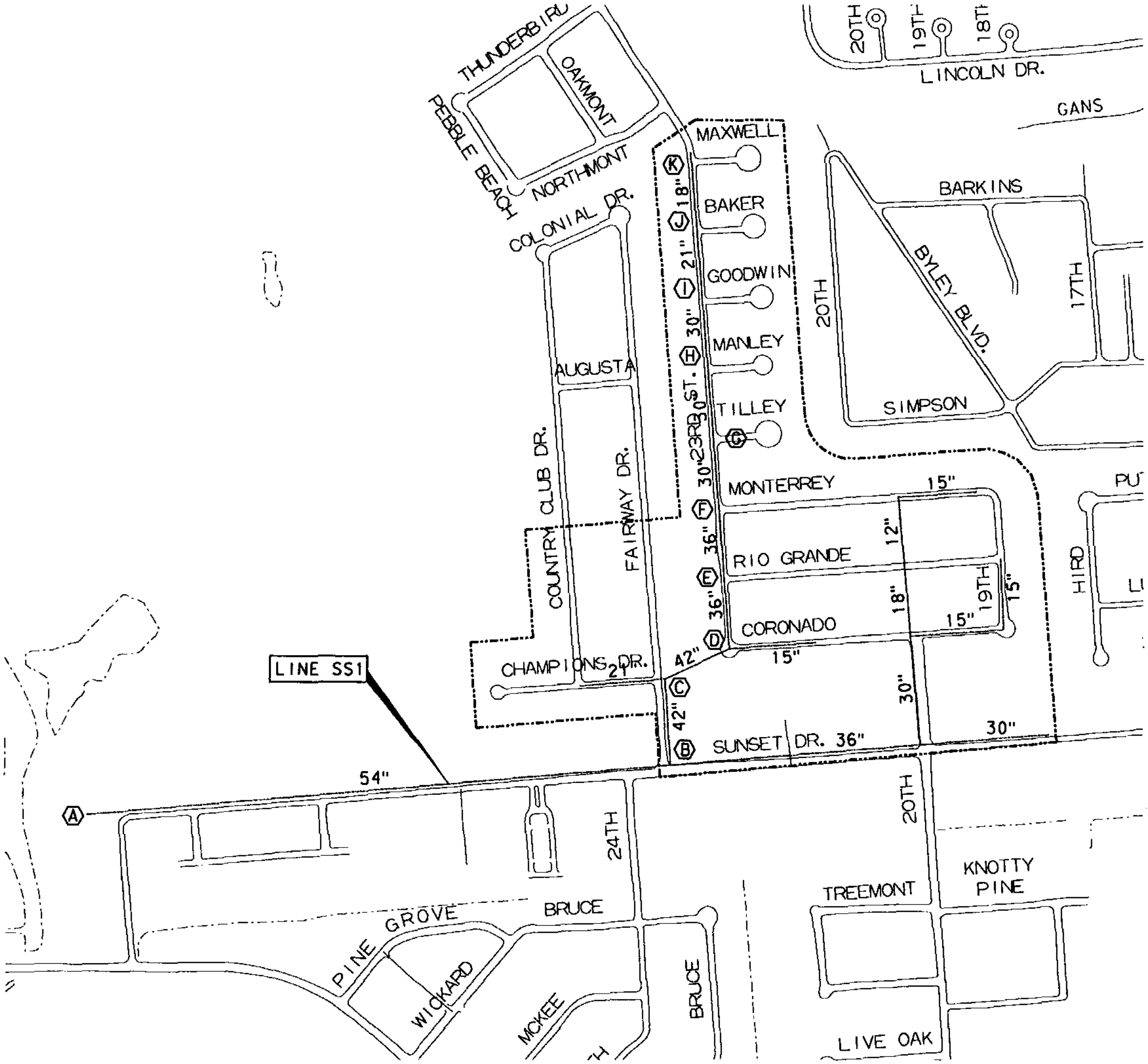
Section II entitled "Drainage Criteria and Methodology Review", provides an in-depth discussion of drainage criteria derivation and the methodology used for hydrologic and hydraulic analysis of this sub-watershed area.

Each of the existing drains within the study area of this sub-watershed were analyzed to determine their hydraulic capacity. First, a hydraulic model of the existing drains using the computer program HYDRADAL was created. Pipe sizes, reach lengths, and invert elevations were input using data from drainage maps furnished by The City of Orange. The existing system has pipe sizes ranging from 54" RCP at the outfall to 15" RCP in some lateral reaches. See Exhibit III.22 entitled "Sunset Drive Watershed" for a plan view of the existing system under consideration. Sub-drainage areas were delineated using existing inlet locations, flow arrows, and contour maps. Each sub-area was measured and input into HYDRADAL as acres. The C-factor used was 0.5 to reflect the residential development. The initial runoff "Time of Concentration" used was 10 minutes. As in the other storm sewer analyses, the residential terrain is rather flat, is well vegetated, and runoff must travel several hundred feet to reach the first inlet. Therefore, ten minutes was considered adequate.

As in the other analyses, the second step was to run the completed hydraulic model using different year storm frequencies. The storm frequencies evaluated were the 5-year, 2-year, and 1-year events. Starting water surface elevations were also determined and input for each year storm frequency. The starting water surface for the

Sunset Drive area is the corresponding water surface in Adams Bayou just upstream of the Park Ave. bridge. After running HYDRADAL, the resulting water surface profile for each evaluation was then compared to the existing street surface elevations to determine if flooding would occur. If flooding conditions existed for the 5-year storm then subsequent smaller frequencies were analyzed to determine the capacity of the pipe. The conclusions and recommendations of the analysis are presented in the following sections entitled "Conclusions" and "Recommendations." The outputs from HYDRADAL are included in the Appendix.

Costs for these improvements may be minimized by keeping the last 1000' as a natural drainage ditch.



LEGEND

- EXISTING STORM DRAIN
- DRAINAGE SUB-AREA BOUNDARY
- (A) LOCATION NODES
- LINE SS1 STORM DRAIN DESIGNATION

EXHIBIT III.22
SUNSET DRIVE WATERSHED
EXISTING STORM DRAINS
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.

APRIL, 1994

SUWTRSHD.DGN

IV. CONCLUSIONS

A. WIDE-SCALE FLOODING

The following sections present the conclusions reached during the analysis of the areas in the City of Orange that are prone to wide-scale flooding.

1. Adams Bayou

Diversion of Flow

The result of diverting 3300 cfs, or nearly one-half of the flow away from Adams Bayou to the Sabine River, was only approximately a 2 ft. reduction of the 100-year floodplain elevation. For example, the 100-year water surface elevation decreased by 1.9 feet at Western Ave. The decrease in water surface also only minimally decreases the width of the floodplain south of I-10. The number of houses and buildings removed from the floodplain was only 18. The complete HEC-2 analysis outputs are included in the Appendix.

A cost estimate was prepared for the preferred diversion route. The estimate is presented in Table IV.1.

TABLE IV.1 Adams Bayou Diversion Preliminary Cost Estimate

ITEM	AMOUNT	UNIT	COST/UNIT	COST
Bridges	35,000	sf	$40.00	1,400,000
Excavation	692,000	cy	$5.75	3,979,000
Right-of-way	48	acre	$10,000.00	480,000
Houses	14	each	$60,000.00	840,000
Buildings	4	each	$200,000.00	800,000
Seeding	232,000	sy	$0.28	65,000
Total Cost				**$7,564,000**

Based on the high cost of construction, the minimal reduction of flood elevations, the small number of properties in the Adams Bayou flood plain downstream of I-10, and the possible adverse impact on the wildlife preserve, it was concluded that no action be taken regarding diversion of flow from Adams Bayou.

Channel Widening by Dredging

The result of widening the Adams Bayou channel by 50' was approximately a 2' reduction of the 100-year floodplain elevation. The reduction of the water surface also reduced the width of the floodplain. That section of the Adams Bayou floodplain north of I-10 contains much more development than the section south of I-10 as most of the southern section is dedicated as a nature preservation area. The number of houses and structures removed from the floodplain was 83.

A preliminary cost estimate for the Adams Bayou dredging is presented in Table IV.2.

TABLE IV.2 Adams Bayou Dredging Preliminary Cost Estimate

ITEM	AMOUNT	UNIT	COST/UNIT	COST
Dredging	1,700,000	cy	$5.00	8,500,000
Bridges/ Adjustments	30,000	sf	$40.00	1,200,000
Total Cost				**$9,700,000**

The benefit of removing 83 houses from the floodplain at $60,000 per house was calculated to be approximately $5,000,000. Based on the resulting benefit/cost ratio of less than one, ($5,000,000 / $9,700,000) the minimal reduction of flood elevations, and the possible adverse impact on the wildlife preserve, it was concluded that no action be taken regarding widening the channel of Adams Bayou.

2. Little Cypress Bayou

Based on the Little Cypress Bayou analysis discussed in Section III of this report, it is concluded that no structural improvements are needed at this time. Several recommendations are made in the Recommendations section to help minimize the effects of flooding in Little Cypress Bayou.

3. Levee Protection Alternatives

A benefit-cost analysis was performed for each of the three levee protection alternatives presented in Section III. To calculate the benefit-cost ratios, it was necessary to have an estimated dollar value for both benefits and costs on an annual basis.

The estimated costs were determined first for each levee alignment. The cost estimates were based on a levee height of 14 feet above MSL, with fill quantities determined following the existing ground profile along the alignment using the average end area method. Pump station costs were based on current pump station construction costs of similar capacity. A large contingency (35%) was added to cover engineering fees and costs unaccounted for due to the preliminary nature of the estimate. Tables IV.3 to IV.5 present the preliminary cost estimates.

The second step of the benefit-cost ratio analysis was to determine the benefits gained by having a levee in place. Obviously, this is not as tangible as determining the estimated costs. Nevertheless, the Army Corps of Engineers has developed stage-damage loss curves to estimate flood damage values that can in turn be termed as benefits assuming the damages are prevented. The Corp's curves present a specific percent value loss due to different levels of flooding for different types of structures. To relate the curves to the City of Orange it was necessary to compile a list of structure types and to assign a value weight factor to each based on its representative value to the city as a whole. The percent values from the curves were then multiplied by their value weight factor and summed to arrive at an estimated total percent structural damage value loss to the city. Tables IV.6 to IV.8 present the estimated percent value losses to the city. The percent damage value losses were then translated into dollar value losses by multiplying the percent value loss by the appraised dollar value of the portion of the city susceptible to flooding. The Orange County Appraisal District provided the 1993 appraised values for Orange County and its cities. The raw land component was subtracted and the result was multiplied by 0.5 assuming one-half of the citys' and county's value was susceptible to flooding. The resulting appraised value of property protected by levee alternative No. 1 was calculated to be $178 million, levee alternative No. 2 was $239 million, and levee alternative No. 3 was $1,402 million. These values were then multiplied by the percent value loss factors for different storm frequencies, or levels of flooding, to obtain the estimated total dollar value loss for each storm frequency. Table IV.9 presents these losses for Levee alternative No. 3.

The final step in determining the cost-benefit ratio was to convert both costs and benefits to an annual basis for comparison. The total dollar value loss for each storm was converted to an expected annual dollar loss based on each storm's probability of occurrence. The expected annual damages, or benefit assuming a levee is in place, was $2.8 million for alternate No. 1, $3.8 million for alternate No. 2, and $22.4 million for alternate No. 3. The total construction cost for each levee alternative was converted to an expected annual cost by amortizing the project over a 100-year period at 6% interest and including an annual maintenance cost. The expected annual cost for alternative No. 1 was $3.5 million, for alternative No. 2 was $5.5 million and for alternative No. 3 was $7.7 million. These expected annual benefits and costs give resulting benefit-cost ratios of 0.8 for alternative No.1, 0.7 for alternative No. 2, and 2.9 for alternative No. 3. Table IV.10 presents the figures for Alternative No. 3.

Final comments regarding the levee alternatives will be discussed in the Recommendations section.

TABLE IV.3
ESTIMATE OF PROBABLE COST
LEVEE ALTERNATIVE No. 1
(East bank of Adams only)

ITEM	COST
Levee - 55,000 L.F.	**$15,700,000**
Pump Station - Coopers	**$4,700,000**
(600,000 GPM NET REQ'D)	
Pump Station - Old Town	**$2,300,000**
(180,000 GPM)	
Pump Station - Cove	**$3,300,000**
(330,000 GPM)	
Pump Station - A	**$3,000,000**
(290,000 GPM)	
Pump Station - B	**$2,300,000**
(180,000 GPM)	
Subtotal Construction Cost	**$31,300,000**
Engineering and Contingencies (35%)	**$10,955,000**
Total Estimated Construction Cost	**$42,255,000**

TABLE IV.4
ESTIMATE OF PROBABLE COST
LEVEE ALTERNATIVE No. 2
(Both banks of Adams, excluding Chemical Row)

ITEM	COST
Levee - 81,000 L.F.	**$23,000,000**
Pump Station - Coopers	**$4,700,000**
(600,000 GPM NET REQ'D)	
Pump Station - Old Town	**$2,300,000**
(180,000 GPM)	
Pump Station - Cove	**$3,300,000**
(330,000 GPM)	
Pump Station - A	**$3,000,000**
(290,000 GPM)	
Pump Station - B	**$2,300,000**
(180,000 GPM)	
Pump Station - C	**$5,800,000**
(850,000 GPM)	
Pump Station - D	**$4,500,000**
(550,000 GPM)	
Subtotal Construction Cost	**$48,900,000**
Engineering and Contingencies (35%)	**$17,115,000**
Total Estimated Construction Cost	**$66,015,000**

TABLE IV.5
ESTIMATE OF PROBABLE COST
LEVEE ALTERNATIVE No. 3
(Both banks of Adams, including Chemical Row)

ITEM	COST
Levee - 108,000 L.F.	**$30,800,000**
Pump Station - Coopers	**$4,700,000**
(600,000 GPM NET REQ'D)	
Pump Station - Old Town	**$2,300,000**
(180,000 GPM)	
Pump Station - Cove	**$3,300,000**
(330,000 GPM)	
Pump Station - A	**$3,000,000**
(290,000 GPM)	
Pump Station - B	**$2,300,000**
(180,000 GPM)	
Pump Station - C	**$5,800,000**
(850,000 GPM)	
Pump Station - D	**$4,500,000**
(550,000 GPM)	
Pump Station - E	**$7,900,000**
(1,420,000 GPM)	
Pump Station - F	**$5,800,000**
(860,000 GPM)	
Subtotal Construction Cost	**$70,400,000**
Engineering and Contingencies (35%)	**$24,640,000**
Total Estimated Construction Cost	**$95,040,000**

CITY OF ORANGE, TEXAS

TABLE IV.6 STRUCTURAL STAGE-DAMAGE LOSSES

AS PERCENT OF STRUCTURE VALUE LOST DUE TO DIFFERENT DEPTHS OF FLOODING

TYPE OF STRUCTURE	USACE CURVE#	VALUE WEIGHT FACTOR	DEPTH OF FLOODING (FEET)									
			0	1	2	3	4	5	6	7	8	9
HOUSE-ONE STORY	RS1	0.200	10	21	27	32	37	43	46	50	54	58
HOUSE-TWO STORY	RS3	0.150	5	21	27	31	34	37	39	40	40	42
APARTMENT	RS6	0.085	5	18	25	30	34	38	41	43	46	48
AUTO SERVICE	34	0.005	0	3	3	3	4	5	8	12	17	23
BANK	46	0.005	0	11	11	12	13	15	17	19	22	24
BUSINESS	88	0.100	0	1	2	3	5	8	11	13	16	18
CHEMICAL PLANT	115	0.100	0	15	15	17	20	25	30	35	40	40
CHEMICAL REFINERY	109	0.100	0	9	9	9	9	10	11	12	13	13
CHURCH	118	0.005	0	10	11	11	12	12	13	14	14	15
CITY HALL	121	0.005	0	1	1	1	2	2	3	4	6	8
DEPT STORE-MEDIUM	181	0.010	0	3	7	7	7	9	11	14	17	20
FABRICATION SHOP	391	0.005	0	2	5	10	15	20	25	30	35	40
FIRE STATION	232	0.005	0	1	5	5	5	6	7	9	11	14
GROCERY-MEDIUM	283	0.005	0	3	4	5	6	7	10	14	20	29
HARDWARE	307	0.005	0	12	12	12	12	12	12	14	15	18
HOSPITAL	331	0.010	0	0	0	20	25	30	35	40	43	47
HOTEL	334	0.010	0	1	2	2	2	3	5	6	9	11
LABORATORY-CHEMICAL	349	0.005	1	1	3	5	8	12	16	21	26	32
LIQUOUR STORE	364	0.010	0	1	1	2	2	3	5	6	8	11
LOADING DOCK-INDUST.	367	0.010	0	1	1	1	3	3	5	8	12	16
MACHINE SHOP-HEAVY	382	0.005	0	1	1	1	3	5	8	12	16	21
OFFICE-SMALL	451	0.050	0	12	14	17	19	23	27	31	35	40
OIL STORAGE TANKS	448	0.020	0	2	2	2	2	2	2	2	2	2
POLICE STATION	481	0.010	0	12	14	17	19	23	27	31	35	40
POST OFFICE	505	0.010	0	8	15	24	25	26	27	29	32	36
RESTAURANT-REGULAR	550	0.010	0	15	18	20	23	25	27	28	30	33
SCHOOL-PUBLIC	571	0.010	0	8	12	15	15	16	17	19	22	25
SERVICE STATION	583	0.010	0	0	1	3	5	7	10	13	16	19
SEWAGE TREATMENT	586	0.010	0	2	4	4	4	5	6	8	12	16
TELEPHONE EXCHANGE	616	0.005	0	12	14	17	19	23	27	31	35	40
THEATER-INDOOR	619	0.005	0	2	3	4	4	4	5	7	10	13
UTILITY CO	655	0.010	0	0	0	10	14	18	22	26	30	34
WAREHOUSE	709	0.005	0	0	1	1	1	3	5	8	12	16
WATER SUPPLY	999	0.010	0	0	0	0	0	0	0	0	0	0
	TOTAL	1.000										
SUMMATION OF PERCENT LOSS TIMES VALUE WEIGHT FACTOR			3.2	12.8	16.0	19.1	21.7	25.2	27.9	30.5	33.3	35.7
Which may also be described as...												
ESTIMATED TOTAL PERCENT STRUCTURAL DAMAGE VALUE LOSS TO CITY *			3.2	12.8	16.0	19.1	21.7	25.2	27.9	30.5	33.3	35.7

* These values are estimates only. It was assumed the city is level at elevation 5 'MSL and the value weight factors are representative for the City of Orange.

CITY OF ORANGE, TEXAS

TABLE IV.7 CONTENTS AND INVENTORY STAGE-DAMAGE LOSSES

AS PERCENT OF VALUE LOST DUE TO DIFFERENT DEPTHS OF FLOODING

TYPE OF STRUCTURE	USACE CURVE#	VALUE WEIGHT FACTOR	DEPTH OF FLOODING (FEET) 0	1	2	3	4	5	6	7	8	9
HOUSE-ONE STORY	RC1	0.200	8	42	60	71	77	82	85	86	87	88
HOUSE-TWO STORY	RC3	0.150	4	24	34	40	47	53	56	58	58	58
APARTMENT	RC6	0.085	6	34	44	55	67	77	87	97	100	100
AUTO SERVICE	32	0.005	10	40	60	85	100	100	100	100	100	100
BANK	47	0.005	0	50	87	95	100	100	100	100	100	100
BUSINESS	89	0.100	0	2	6	10	15	19	24	28	33	38
CHEMICAL PLANT	116	0.100	0	25	50	75	100	100	100	100	100	100
CHEMICAL REFINERY	110	0.100	0	30	100	100	100	100	100	100	100	100
CHURCH	119	0.005	10	28	54	70	84	90	95	97	99	100
CITY HALL	999	0.005	0	0	0	0	0	0	0	0	0	0
DEPT STORE-MEDIUM	182	0.010	0	18	33	65	88	95	100	100	100	100
FABRICATION SHOP	392	0.005	0	10	20	30	40	50	60	70	75	80
FIRE STATION	233	0.005	0	10	25	50	75	91	100	100	100	100
GROCERY-MEDIUM	284	0.005	4	22	44	74	96	100	100	100	100	100
HARDWARE	308	0.005	8	33	52	70	75	88	100	100	100	100
HOSPITAL	332	0.010	0	0	0	0	10	20	80	83	86	89
HOTEL	999	0.010	0	0	0	0	0	0	0	0	0	0
LABORATORY-CHEMICAL	350	0.005	0	43	60	60	60	60	70	70	80	80
LIQUOUR STORE	365	0.010	0	20	40	60	81	100	100	100	100	100
LOADING DOCK-INDUST.	368	0.010	0	10	10	10	10	10	10	10	20	20
MACHINE SHOP-HEAVY	386	0.005	0	2	5	10	15	20	25	30	40	50
OFFICE-SMALL	999	0.050	0	0	0	0	0	0	0	0	0	0
OIL STORAGE TANKS	999	0.020	0	0	0	0	0	0	0	0	0	0
POLICE STATION	999	0.010	0	0	0	0	0	0	0	0	0	0
POST OFFICE	999	0.010	0	0	0	0	0	0	0	0	0	0
RESTAURANT-REGULAR	551	0.010	0	73	88	100	100	100	100	100	100	100
SCHOOL-PUBLIC	999	0.010	0	0	0	0	0	0	0	0	0	0
SERVICE STATION	584	0.010	0	25	42	62	90	100	100	100	100	100
SEWAGE TREATMENT	999	0.010	0	0	0	0	0	0	0	0	0	0
TELEPHONE EXCHANGE	999	0.005	0	0	0	0	0	0	0	0	0	0
THEATER-INDOOR	999	0.005	0	0	0	0	0	0	0	0	0	0
UTILITY CO	656	0.010	0	1	1	5	7	10	11	12	13	14
WAREHOUSE	710	0.005	0	11	16	19	21	23	28	35	47	67
WATER SUPPLY	999	0.010	0	0	0	0	0	0	0	0	0	0
	TOTAL	1.000										
SUMMATION OF PERCENT LOSS TIMES VALUE WEIGHT FACTOR			2.9	23.3	40.7	49.2	56.8	60.8	64.1	66.0	67.3	68.2
Which may also be described as...												
ESTIMATED TOTAL PERCENT CONTENTS DAMAGE VALUE LOSS TO CITY *			2.9	23.3	40.7	49.2	56.8	60.8	64.1	66.0	67.3	68.2

* These values are estimates only. It was assumed the city is level at elevation 5 'MSL and the value weight factors are representative for the City of Orange.

CITY OF ORANGE, TEXAS

TABLE IV.9 SUMMATION AND VALUATION OF STAGE-DAMAGE LOSSES

SUMMARY OF PERCENT DAMAGES

DAMAGE CLASSIFICATION	DEPTH OF FLOODING (FEET)									
	0	1	2	3	4	5	6	7	8	9
Estimated Total Percent STRUCTURAL Damage Loss to City	3.2	12.8	16.0	19.1	21.7	25.2	27.9	30.5	33.3	35.7
Estimated Total Percent CONTENTS Damage Loss to City	2.9	23.3	40.7	49.2	56.8	60.8	64.1	66.0	67.3	68.2
Estimated Total Percent EQUIPMENT Damage Loss to City	0.1	8.0	14.4	18.4	25.2	28.3	29.8	31.2	32.6	33.3

DEPTH OF FLOODING DETERMINATION FOR STORM FREQUENCIES

FREQUENCY STORM	FLOOD ELEVATION		CITY ELEVATION		FLOOD DEPTH
Standard Project Flood	10.0	-	5	=	**5**
100-year Flood	7.8	-	5	=	**2.8**
50-year Flood	6.9	-	5	=	**1.9**
25-year Flood	6.0	-	5	=	**1**
10-year Flood	4.8	-	5	=	**0**
5-year Flood	4.2	-	5	=	**0**
2-year Flood	3.6	-	5	=	**0**
0-year Flood	0.0	-	5	=	**0**

LEVEE ALTERNATIVE NO. 3

ESTIMATED DOLLAR VALUE OF DAMAGE LOSS FOR STORM FREQUENCIES

TYPE OF DAMAGE	VALUE WEIGHT FACTOR	PERCENT DAMAGE LOSS FOR EACH FREQUENCY							
		SPF	**100**	**50**	**25**	**10**	**5**	**2**	**0**
STRUCTURAL	0.5	25.2	18.5	15.7	12.8	3.2	0.0	0.0	0.0
CONTENTS/INVENTORY	0.3	60.8	47.5	39.0	23.3	2.9	0.0	0.0	0.0
EQUIPMENT	0.2	28.3	17.6	13.8	8.0	0.1	0.0	0.0	0.0
	TOTAL 1.000								
SUMMATION OF PERCENT LOSS TIMES VALUE WEIGHT FACTOR		37	27	22	15	2.5	0	0	0
MULTIPLY BY 1993 APPRAISED VALUE OF CITY IMPROVEMENTS (MILLIONS)		1402	1402	1402	1402	1402	1402	1402	1402
ESTIMATED TOTAL DOLLAR VALUE LOSS FOR EACH STORM FREQUENCY (MILLIONS) *		$512	$379	$313	$210	$35	$0	$0	$0

* These values are estimates only. It was assumed the city is level at elevation 5 'MSL and the value weight factors are representative for the City of Orange.

CITY OF ORANGE, TEXAS

TABLE IV.10 BENEFIT-COST RATIO

FOR LEVEE ALTERNATIVE NO. 3

BENEFIT DETERMINATION

FREQUENCY STORM	ANNUAL PROBABILITY OF OCCURANCE	DAMAGES (MILLIONS)*	DAMAGE INTERVAL (MILLIONS)	OCCURANCE INTERVAL	ANNUAL DAMAGES (MILLIONS)
SPF	0.000	512			
			512	0.002	$1.0
SPF	0.002	512			
			445	0.008	$3.6
100-YR	0.010	379			
			346	0.010	$3.5
50-YR	0.020	313			
			261	0.020	$5.2
25-YR	0.040	210			
			123	0.060	$7.4
10-YR	0.100	35			
			17	0.100	$1.7
5-YR	0.200	0			
			0	0.300	$0.0
2-YR	0.500	0			
			0	0.500	$0.0
0-YR	1.000	0			
EXPECTED ANNUAL DAMAGES or, ANNUAL BENEFIT (MILLIONS)					**$22.4**

COST DETERMINATION

LEVEE PROJECT COST OF $95,040,000 AMORTIZED OVER 100 YEAR PERIOD AT 6%	**$5.7**
EXPECTED ANNUAL MAINTENANCE COST (MILLIONS)	**$2.0**
EXPECTED ANNUAL COST (MILLIONS)	**$7.7**

BENEFIT-COST RATIO

22.4 / 7.7 = 2.9

* These values are estimates only. It was assumed the city is level at elevation 5 'MSL and the value weight factors are representative for the City of Orange.

B. LOCALIZED FLOODING

Several small drainage areas were studied within the limits of the study area to determine if drainage could be improved on a more local, as opposed to wide-scale, flooding level. These local flooding issues were generally in areas too small to be considered in either the Flood Insurance Study for the City of Orange, or the previously-released Sabine River Study commissioned by the Sabine River Authority. Therefore, the source of flooding in these local areas is generally not from the nearby creek, bayou, or river, but more commonly from a storm sewer system that is undersized.

In an effort to examine these local flooding concerns, the engineers asked the City staff for suggestions on areas that might warrant further study. In response to this request the engineers examined seven localized areas for flooding problems. These seven areas, as discussed in Section III.D, were the area around the intersection of Cherry Street and 13th Street, the Coopers Gully watershed area, Dayton Street Ditch, the Hudson Gully watershed area, the North Simmons Drive area, the Old Town watershed, and an area in the vicinity of Sunset Drive. Exhibit III.10, "Localized Areas Prone to Flooding", shows the location of each area. The conclusions of the analyses are presented in two parts, storm sewer improvements and other improvements.

1. Storm Sewer Improvements

Existing storm sewer pipe capacities were analyzed for six of the areas mentioned above. These areas were the area around the intersection of Cherry Street and 13th Street, Dayton Street Ditch, the Hudson Gully watershed area, the North Simmons Drive area, the Old Town watershed, and the area in the vicinity of Sunset Drive. The mainline segments in each of these areas were analyzed to determine their current storm water carrying capacity. Mainlines were designated by the larger pipe size and longer reach of pipe progressing upstream. Laterals were incorporated as changes in flow only. The 1-, 2-, and 5-year hydraulic grade line elevations from the peak flows were calculated for each mainline segment. The 1-, 2-, and 5-year peak flows were computed according to the methodology discussed in the drainage criteria section, whereby the storm intensity in inches per hour was taken off of the intensity-duration-frequency chart for the City of Orange. If the computed hydraulic gradient elevation for a particular pipe segment exceeded the street elevation for a certain storm frequency (i.e., 1-, 2-, 5-year) then the capacity of the pipe segment was assumed to be exceeded. Generally, when the hydraulic gradient elevation exceeds the street elevation, storm runoff will begin to back up in the street gutters. For the six areas studied, the hydraulic gradient elevation exceeded the street elevation in all of the pipe segments for the 2-year storm. In some cases, the hydraulic gradient elevations exceeded the street elevations for a flow that would be less than the 1-year storm. The conclusion that can be drawn from this analysis is that the existing storm sewers are undersized for the six areas analyzed. Exhibits III.12, III.19, III.20, III.21, and III.22 illustrate existing pipe sizes, drainage areas, and location nodes for the six storm sewer areas analyzed.

2. Other Improvements

In the course of analyzing storm sewer system capacities for the six drainage areas mentioned above, the capacities of several outfall channels and the capacity of the Coopers Gully pump station were also analyzed.

a. Coopers Gully

Pumping Capacity
Various pumping scenarios were evaluated to determine the approximate height that water would pond behind the Coopers Gully pump station for differing discharges. These pumping scenarios were conducted using the existing station capacity of 323,000 gpm. The pumping scenarios were conducted to determine the storm frequency that the current pump station is capable of pumping to keep the flood levels on Coopers Gully below elevation 5.5 ft. This evaluation is not valid for floods that originate on the Sabine River. The results of this evaluation are given in Table IV.11.

TABLE IV.11 Peak Storm Elevations at Coopers Gully Pump Station

Storm Frequency	Peak Discharge (cfs)	Peak Elevation (msl)
10-year	1,250	5.3
50-year	1,470	5.7
100-year	1,640	6.0

As Table IV.11 indicates, the current pump station can pump the 10-year storm and keep the water surface below elevation 5.5 ft. A 50-year storm would exceed the station's capacity to maintain flood levels below elevation 5.5 ft.

Channel Analysis

As stated in Section III, the HEC-2 models provided by FEMA for Coopers Gully did not produce results that compared with the published information in the FIS report for the City of Orange. Based on the official information that was obtained from FEMA regarding floodplain top widths and water surface elevations in Coopers Gully (without the effects of the Sabine River), some qualitative judgements could be made regarding the flooding potential of Coopers Gully.

As shown in the table in Section III, the floodplain top widths for a 100-year flood on Coopers Gully can be as wide as 1500 feet. With a corresponding water surface elevation of over seven feet, the result would indicate over 100 structures located within the 100-year floodplain of the gully. Finished-floor elevations would need to be determined on the houses within the floodplain to ascertain whether or not the structure actually would be inundated during the 100-year event.

In order to reduce the level of flooding, ditch and culvert improvements would be necessary to lower water surface elevations along Coopers Gully. The improvements would need to be constructed from Curtis Avenue to 10th Street. The goal of the improvements would be to lower the water surface elevation below the finished floor elevations of the structures. Several structures could still be located within the floodplain. To determine if they would still be inundated by the lower flood elevations, finished-floor elevations would need to be determined on those structures.

The cost of the recommended ditch and culvert improvements are discussed in the Recommendations section of this report.

b. Dayton Street Ditch

Dayton Street Ditch was also analyzed for local flooding issues. The area of primary interest was the intersection of Bluebonnet Drive and Highway 87. Ponding water along Bluebonnet on the north side of the intersection impedes traffic entering Highway

87 and has also risen high enough to enter some homes along Bluebonnet. This problem seems to occur after only minor rainfall. It was determined that only one set of storm sewer inlets exist near the intersection to carry water away during a storm. Since the area draining to these inlets is nearly 35 acres, the inlets can be overwhelmed with runoff very easily. In addition to studying the inlet capacity, the capacity of Dayton Street Ditch was also studied. In order to construct a system that would adequately remove storm water away from the Bluebonnet-Highway 87 intersection, improvements would need to be made at the culverts under Highway 87, along the current ditch from the culverts up to Bluebonnet, at each of the existing driveways along the ditch adjacent to Highway 87 and to the inlet and storm sewer system approximately 100 feet upstream of the intersection along Bluebonnet. The results of the Dayton Street Ditch analysis also indicated that culvert improvements could be constructed at Shell Drive and Tanglewood Drive in West Orange. These culvert improvements would not affect the Bluebonnet Drive intersection problems, but they would lower water surface elevations approximately 1.5 feet where the ditch crosses Shell Drive and Tanglewood Drive.

c. Hudson Gully

Hudson Gully was also one of the areas studied for localized flooding. The ditch was studied to determine if lower water surface elevations in the ditch would improve the storm sewer capacity of the pipes that outfall into the ditch. Although the flood carrying capacity of the ditch appears to be adequate, the five-year water surface elevations are high enough to impede conveyance from the storm sewers that outfall into the ditch. Therefore, during a five-year storm, the storm sewer pipes cannot discharge to their design capacity because the water surface in the ditch is too high. In order to improve storm sewer outfall capacity for the five-year storm, the size of the ditch would need to be enlarged and an extra culvert would need to be installed under Highway 90. Also, in the course of studying storm sewers in the Hudson Gully watershed, the engineers concluded that an additional storm sewer line could be installed along Bluebonnet to improve drainage on the northern end of the street. The additional line would begin near Circle R, extend along Bluebonnet and outfall into Hudson Gully.

d. North Simmons Drive

In addition to analyzing the storm sewers in the vicinity of North Simmons Drive, the addition of flap gates to the 7'x7' box culvert under Simmons on the tributary to Little Cypress Bayou was analyzed. It was determined that adding flap gates on the downstream side of the culverts at the tributary would prevent rising water from either the Sabine River or Little Cypress Bayou from backing up into the neighboring subdivisions until the water surface was high enough to overtop Simmons Drive. A smaller set of flap gates would also need to be installed at the 3'x3' culvert under Simmons that outfalls into Little Cypress Bayou.

V. RECOMMENDATIONS

A. WIDE-SCALE FLOODING

1. Adams Bayou

The two alternatives considered in this study to lower water surface elevations in Adams Bayou were the construction of a diversion channel and widening of the existing channel by dredging. As stated in the Conclusions section, neither alternative appears to be economically practical based on the analysis. Therefore, neither of these plans are recommended. However, other recommendations are provided below to help minimize the impact of future floods in Adams Bayou.

The first recommendation is that the City strictly prohibit any further development within the floodplain. Another recommendation is that the City determine the finished floor elevations of all structures in question within the floodplain boundary. Affected homeowners should then be informed of their situation and encouraged to purchase flood insurance if they are not presently covered.

One final recommendation is that the City consider purchasing property within the floodplain as it becomes available and thereby eventually reducing the number of structures subject to flooding. The FEMA Section 1362 Buyout Program, shown in Table VII.1, could be a source of funds for the City to purchase structures subject to flooding.

2. Little Cypress Bayou

Based on the Little Cypress Bayou analysis discussed in Section III of this report, no structural improvements are recommended at this time. However, other recommendations are provided below to help minimize the impact of any future floods in Little Cypress Bayou.

The first recommendation is that the City strictly prohibit any further development within the floodplain. Another recommendation is that the City determine the finished floor elevations of all structures in question within the floodplain boundary. Affected homeowners should then be informed of their situation and encouraged to purchase flood insurance if they are not presently covered. One final recommendation is that the City consider purchasing property within the floodplain as it becomes available and thereby eventually reducing the number of structures subject to flooding.

3. Levee Protection Alternatives

Benefit-cost ratios were determined in the Conclusions section for the three levee alternatives. Levee alternative No. 1 had a benefit-cost ratio of 0.8, alternative No. 2 of 0.7, and alternative No. 3 of 2.9. Based on the lower benefit-cost ratios for alternative No. 1 and alternative No. 2, it is recommended that alternative No. 3 be considered the most viable. As can be seen from the total costs of these alternatives, each one is beyond the scope of a local capital improvements program. Therefore, should the City desire to provide levee protection, it is recommended that funding assistance be sought from the Corps of Engineers or the Texas Water Development Board.

B. LOCALIZED FLOODING

The following recommendations are made to relieve local flooding concerns for the areas studied in the City of Orange. The recommendations are presented in two parts, storm sewer improvements and other improvements.

1. Storm Sewer Improvements

Six areas were studied to determine the pipe sizes required to convey the five-year storm in an underground storm sewer system. These areas, as discussed earlier, were the area around the intersection of Cherry Street and 13th Street, Dayton Street Ditch, the Hudson Gully watershed area, the North Simmons Drive area, the Old Town watershed, and the area in the vicinity of Sunset Drive. The recommended pipe size changes along with an estimate of the cost to construct the improvements are shown in Table V.1 through Table V.5. The tables show the recommended pipe sizes for each line segment for the areas studied. The storm sewer improvements in the upper area of Dayton Street Ditch are discussed with the other improvements to the ditch in Section V subsection B2b. Section VI, entitled "Watershed and Local Drainage Project Priority Ranking System," presents a recommended priority for completing construction of the storm sewer line segments.

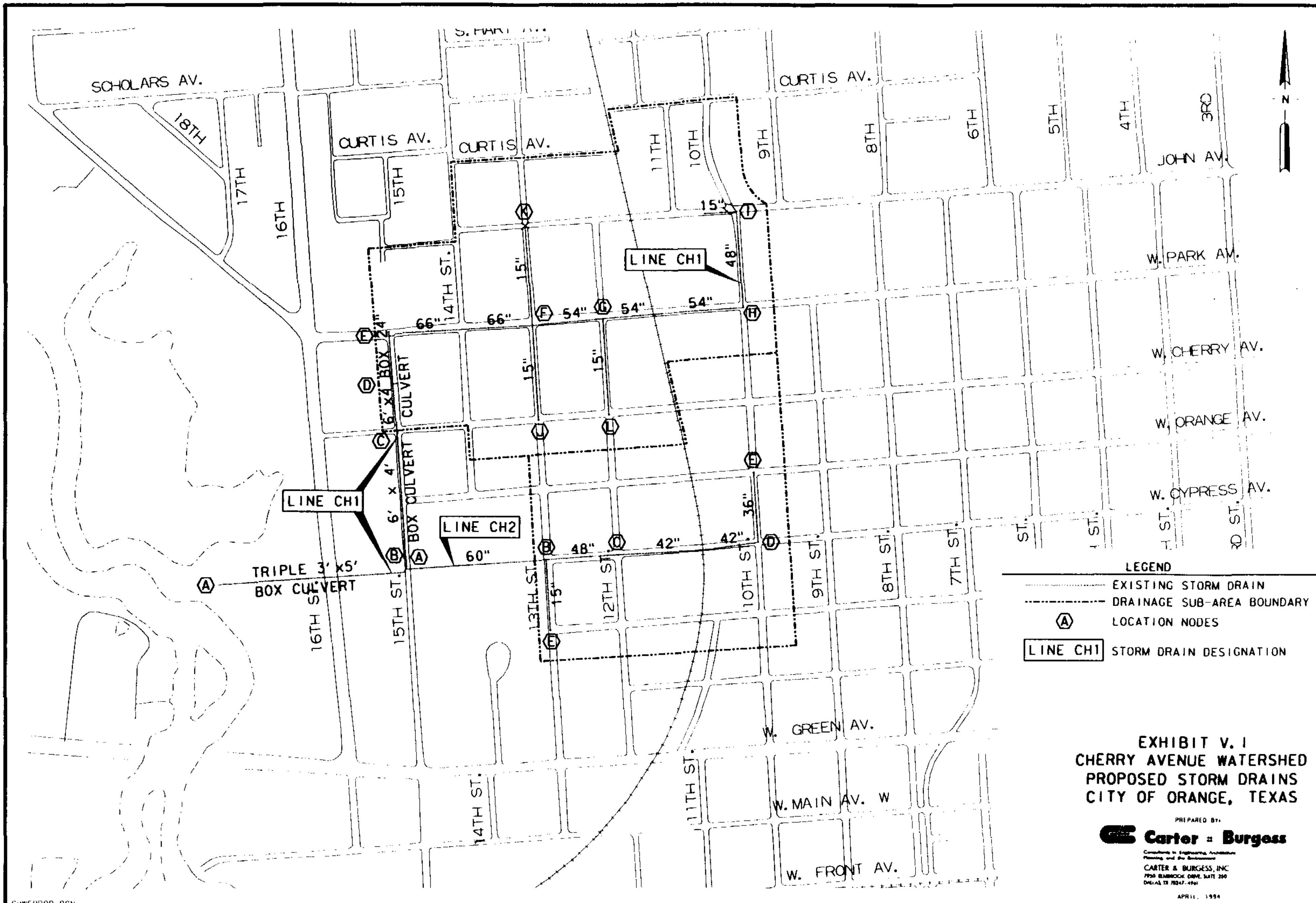


EXHIBIT V.1
CHERRY AVENUE WATERSHED
PROPOSED STORM DRAINS
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.

APRIL 1994

TABLE V.1
COST ESTIMATES FOR STORM DRAIN UPGRADES
CHERRY AVE. WATERSHED
CITY OF ORANGE,TEXAS

Line Name	Section From:	To:	Existing Pipe Size	Proposed Pipe Size	Unit	$/Unit	Quanity	Estimated Cost
CH1				**Inlets**	ea.	$1,500	14	**$21,000**
	I	H	15"	**48"**	LF	$95	440	**$41,800**
	H	F	18"/21"	**54"**	LF	$106	1100	**$116,600**
	F	E	24"	**66"**	LF	$123	750	**$92,250**
	E	D	48"	**6X4 RCB**	LF	$110	210	**$23,100**
	D	C	2 - 48"	**6X4 RCB**	LF	$110	250	**$27,500**
	C	B	6X4 RCB	**6X4 RCB**	LF	$110	720	**$79,200**
	B	A	2-3'X5' RCB	**3-3X5 RCB**	LF	$130	1000	**$130,000**
							Total Line Cost	**$531,450**
CH2				**Inlets**	ea.	$1,500	10	**$15,000**
	E	D	21"	**36"**	LF	$72	350	**$25,200**
	D	C	21"	**42"**	LF	$84	740	**$62,160**
	C	B	21"	**48"**	LF	$95	380	**$36,100**
	B	A	24"	**60"**	LF	$115	750	**$86,250**
							Total Line Cost	**$224,710**
							Sub-Total Construction Cost	**$756,160**
							Engineering and Contingencies (30%)	**$226,848**
							Total Construction Cost	**$983,008**

Note: Utility relocation costs are difficult to estimate without detailed maps and are not included.

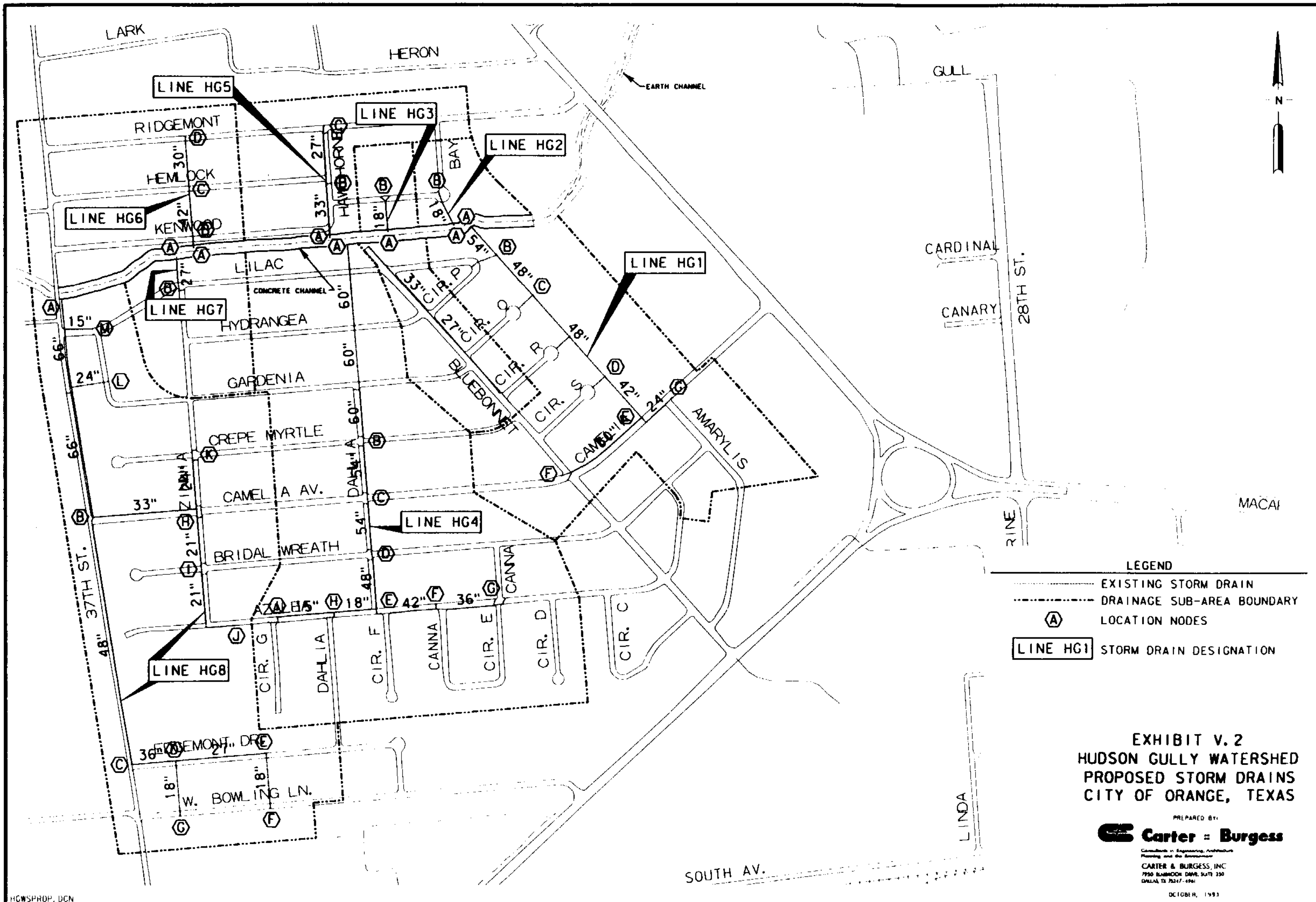


EXHIBIT V.2
HUDSON GULLY WATERSHED
PROPOSED STORM DRAINS
CITY OF ORANGE, TEXAS

TABLE V.2
COST ESTIMATES FOR STORM DRAIN UPGRADES
HUDSON GULLY WATERSHED
CITY OF ORANGE,TEXAS

Line Name	Section From:	To:	Existing Pipe Size	Proposed Pipe Size	Unit	$/Unit	Quanity	Estimated Cost
HG1				Inlets	ea.	$1,500	10	$15,000
	F	E	27"	30"	LF	$63	500	$31,500
	E	D	36"	42"	LF	$84	275	$23,100
	D	C	36"	48"	LF	$95	575	$54,625
	C	B	36"	48"	LF	$95	300	$28,500
	B	A	36"	54"	LF	$106	400	$42,400
							Total Line Cost	$195,125
HG2			18"	Existing system is adequate.				
HG3			18"	Existing system is adequate.				
HG4				Inlets	ea.	$1,500	18	$27,000
	G	F	18"	36"	LF	$72	300	$21,600
	F	E	21"	42"	LF	$84	300	$25,200
	E	D	30"	48"	LF	$95	300	$28,500
	D	C	30"	54"	LF	$106	300	$31,800
	C	B	30"	54"	LF	$106	320	$33,920
	B	A	36"	60"	LF	$115	1025	$117,875
							Total Line Cost	$285,895
HG5				Inlets	ea.	$1,500	6	$9,000
	C	B	24"	27"	LF	$58	300	$17,400
	B	A	24"	33"	LF	$68	200	$13,600
							Total Line Cost	$40,000
HG6				Inlets	ea.	$1,500	8	$12,000
	D	C	21"	30"	LF	$63	275	$17,325
	C	B	24"	42"	LF	$84	250	$21,000
	B	A	24"	42"	LF	$84	60	$5,040
							Total Line Cost	$55,365
HG7				Inlets	ea.	$1,500	3	$4,500
	B	A	15"	27"	LF	$58	150	$8,700
							Total Line Cost	$13,200
HG8				Inlets	ea.	$1,500	12	$18,000
	E	D	21"	27"	LF	$58	200	$11,600
	D	C	21"	36"	LF	$72	250	$18,000
	C	B	30"	48"	LF	$95	1275	$121,125
	B	A	48"/54"	66"	LF	$123	1225	$150,675
							Total Line Cost	$319,400

TABLE V.2

COST ESTIMATES FOR STORM DRAIN UPGRADES

HUDSON GULLY WATERSHED

CITY OF ORANGE,TEXAS

Line Name	Section From:	To:	Existing Pipe Size	Proposed Pipe Size	Unit	$/Unit	Quanity	Estimated Cost
Proposed Line				**Inlets**	ea.	$1,500	6	**$9,000**
on Bluebonnet	**C**	**B**	27"	**27"**	LF	$58	300	**$17,400**
	B	**A**	33"	**33"**	LF	$68	200	**$13,600**
							Total Line Cost	**$40,000**
							Sub-Total Construction Cost	**$948,985**
							Engineering and Contingencies (30%)	**$284,696**
							Total Construction Costs	**$1,233,681**

Note: Utility relocation costs are difficult to estimate without detailed maps and are not included.

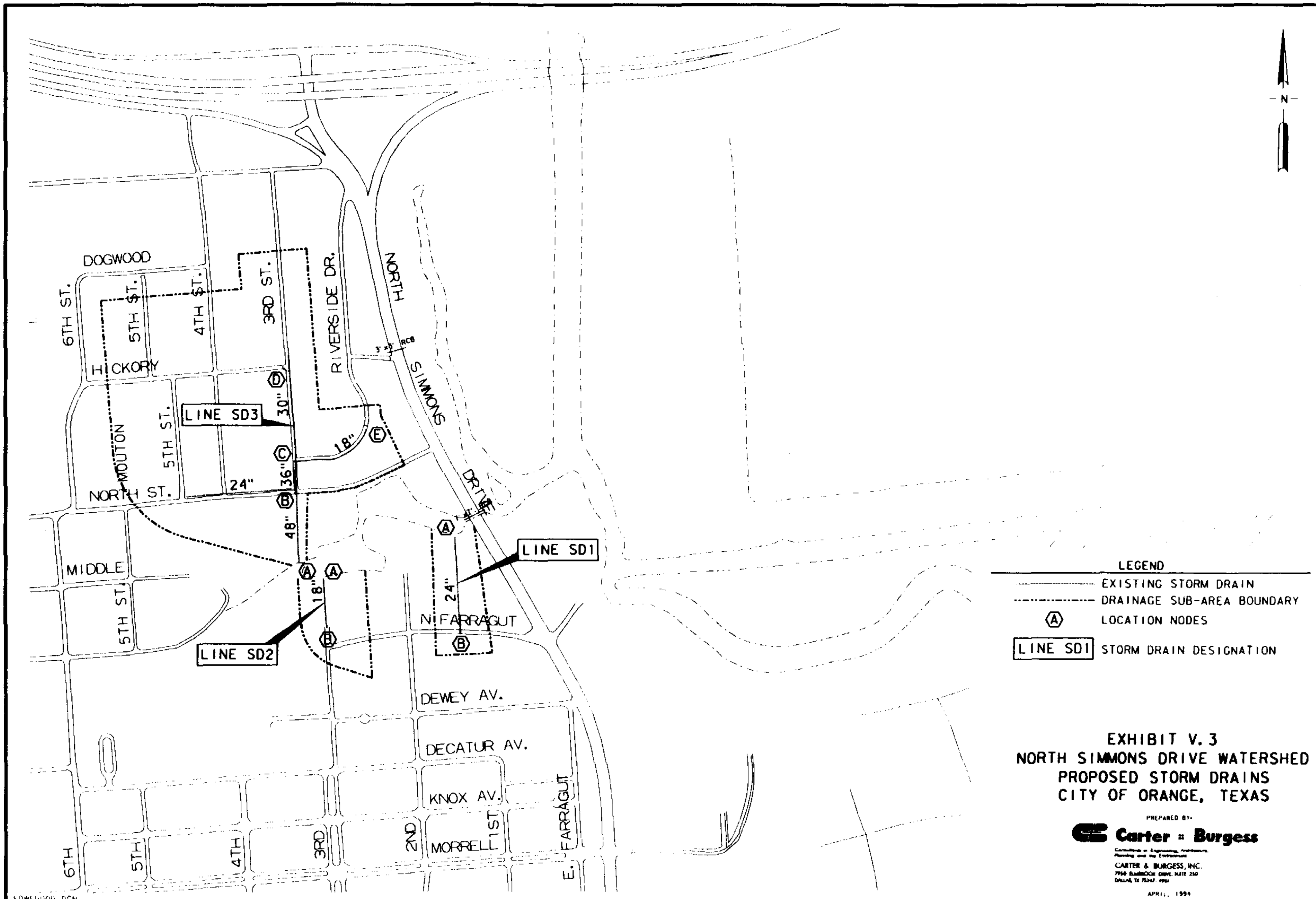


EXHIBIT V.3
NORTH SIMMONS DRIVE WATERSHED
PROPOSED STORM DRAINS
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC.
APRIL, 1994

TABLE V.3

COST ESTIMATES FOR STORM DRAIN UPGRADES

NORTH SIMMONS WATERSHED

CITY OF ORANGE,TEXAS

Line Name	Section From:	To:	Existing Pipe Size	Proposed Pipe Size	Unit	$/Unit	Quanity	Estimated Cost
SD1				**Inlets**	ea.	$1,500	2	**$3,000**
	A	**B**	12"	**24"**	LF	$50	300	**$15,000**
							Total Line Cost	**$18,000**
SD2		18"	Existing system is adequate.					
SD3		Existing mainline is adequate, however, a lateral is needed on North St. between 3rd and 5th.						
Lateral				**Inlets**	ea.	$1,500	4	**$6,000**
	A	**B**	none	**24"**	LF	$50	200	**$10,000**
							Total Line Cost	**$16,000**
							Sub-Total Construction Cost	**$34,000**
							Engineering and Contingencies (30%)	**$10,200**
							Total Construction Cost	**$44,200**

Note: Utility relocation costs are difficult to estimate without detailed maps and are not included.

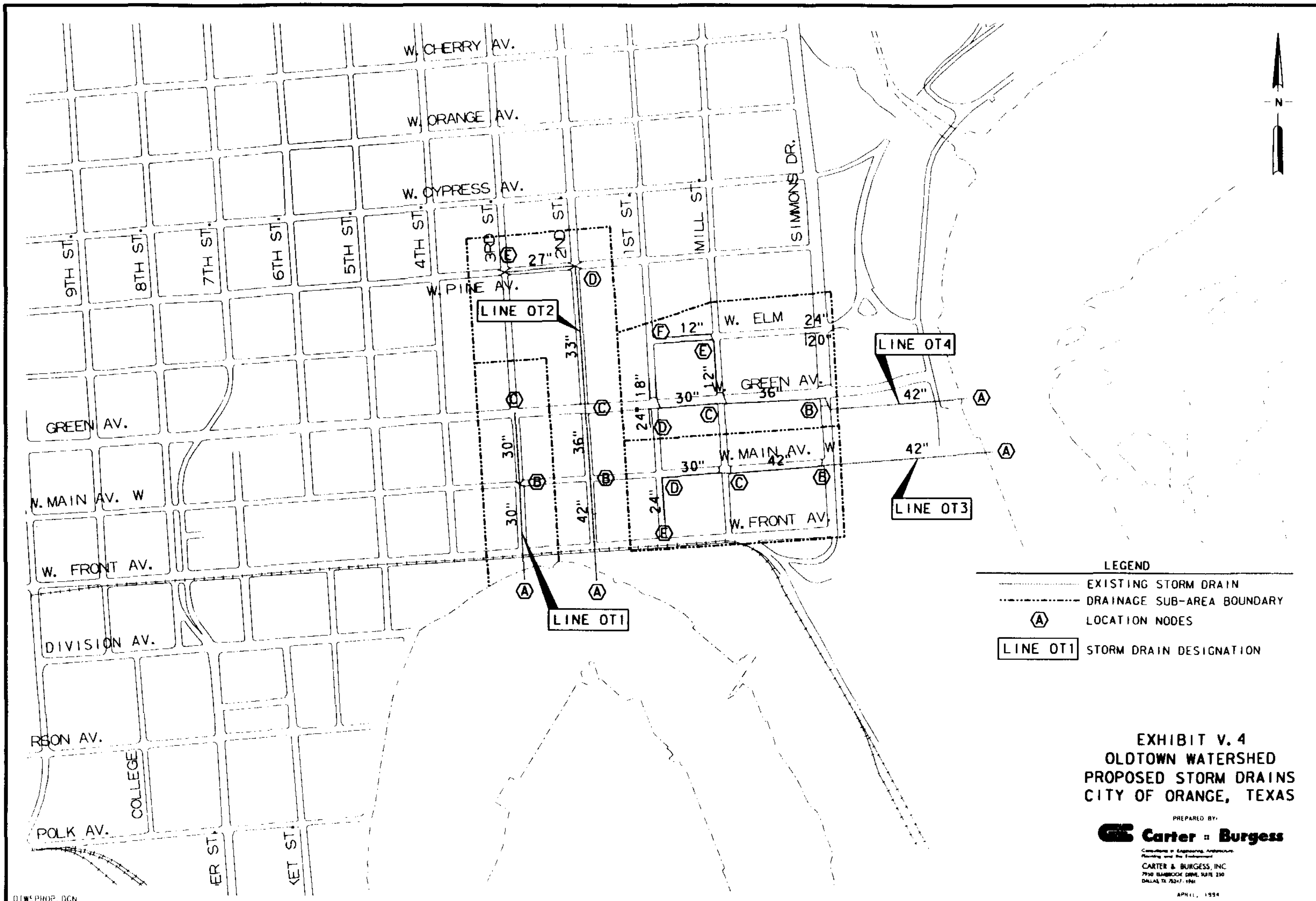


EXHIBIT V.4
OLDTOWN WATERSHED
PROPOSED STORM DRAINS
CITY OF ORANGE, TEXAS

TABLE V.4
COST ESTIMATES FOR STORM DRAIN UPGRADES
OLD TOWN WATERSHED
CITY OF ORANGE,TEXAS

Line Name	Section From:	To:	Existing Pipe Size	Proposed Pipe Size	Unit	$/Unit	Quanity	Estimated Cost
OT1				**Inlets**	ea.	$1,500	8	**$12,000**
	B	**C**	24"	**30"**	LF	$50	360	**$18,000**
	A	**B**	24"	**30"**	LF	$63	490	**$30,870**
						Total Line Cost		**$48,870**
OT2				**Inlets**	ea.	$1,500	12	**$18,000**
	E	**D**	15"	**27"**	LF	$58	350	**$20,300**
	D	**C**	18"	**33"**	LF	$68	800	**$54,400**
	C	**B**	24"	**36"**	LF	$72	360	**$25,920**
	B	**A**	24"	**42"**	LF	$84	490	**$41,160**
						Total Line Cost		**$141,780**
OT3				**Inlets**	ea.	$1,500	12	**$18,000**
	E	**D**	15"	**24"**	LF	$50	220	**$11,000**
	D	**C**	18"	**30"**	LF	$63	290	**$18,270**
	C	**B**	24"	**42"**	LF	$78	525	**$40,950**
	B	**A**	24"	**42"**	LF	$78	800	**$62,400**
						Total Line Cost		**$132,620**
OT4				**Inlets**	ea.	$1,500	8	**$12,000**
	D	**C**	24"	**30"**	LF	$63	340	**$21,420**
	C	**B**	30"	**36"**	LF	$72	500	**$36,000**
	B	**A**	36"	**42"**	LF	$78	700	**$54,600**
						Total Line Cost		**$124,020**
						Sub-Total Construction Cost		**$447,290**
						Engineering and Contingencies (30%)		**$134,187**
						Total Construction Cost		**$581,477**

Note: Utility relocation costs are to difficult to estimate without detailed maps and are not included.

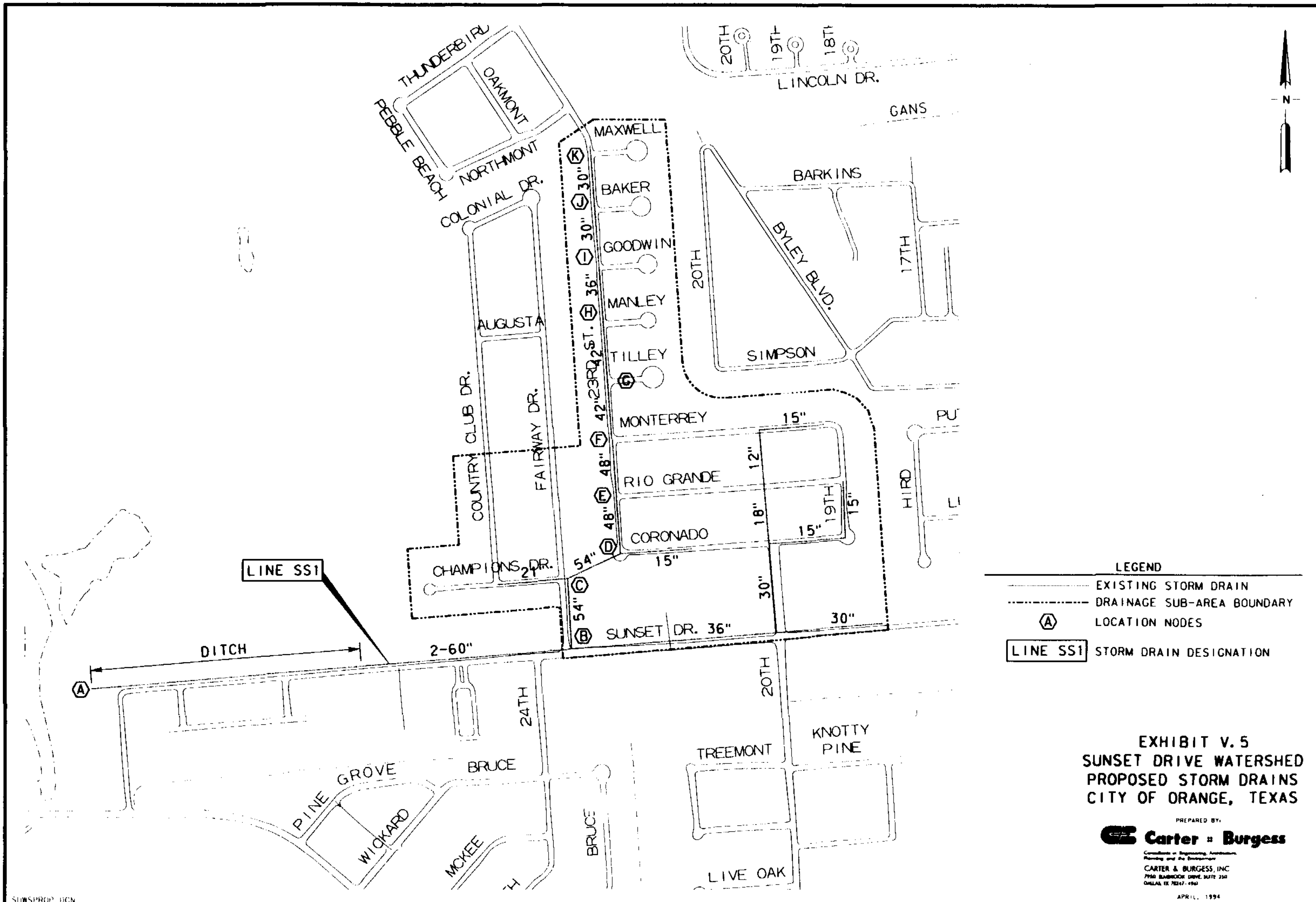


EXHIBIT V.5
SUNSET DRIVE WATERSHED
PROPOSED STORM DRAINS
CITY OF ORANGE, TEXAS

PREPARED BY:
Carter & Burgess
CARTER & BURGESS, INC
APRIL, 1994

TABLE V.5

COST ESTIMATES FOR STORM DRAIN UPGRADES

SUNSET DRIVE WATERSHED

CITY OF ORANGE,TEXAS

Line Name	Section From:	To:	Existing Pipe Size	Proposed Pipe Size	Unit	$/Unit	Quanity	Estimated Cost
SS1				Inlets	ea.	$1,500	18	$27,000
	K	J	18"	30"	LF	$63	300	$18,900
	J	I	21"	30"	LF	$63	300	$18,900
	I	H	30"	36"	LF	$72	300	$21,600
	H	G	30"	42"	LF	$84	300	$25,200
	G	F	30"	42"	LF	$84	300	$25,200
	F	E	36"	48"	LF	$95	300	$28,500
	E	D	36"	48"	LF	$95	300	$28,500
	D	C	42"	54"	LF	$106	400	$42,400
	C	B	42"	54"	LF	$106	330	$34,980
	B	A	54"	2-60"	LF	$230	1100	$253,000
		Adams	54"	DITCH	LF	$18	1600	$28,800
Sub-Total Construction Cost								$552,980
Engineering and Contingencies (30%)								$165,894
Total Construction Cost								$718,874

Note: Utility relocations are difficult to estimate without detailed maps and are not included.

2. Other Improvements

As a result of studying storm sewer capacities for the six drainage areas mentioned above, the capacities of several other drainage structures were also analyzed.

a. Coopers Gully

Pumping Capacity
The Coopers Gully pump station can currently pump the discharge resulting from a 10-year storm along Coopers Gully and maintain flood levels below elevation 5.5 ft. A pumping evaluation was conducted to determine the pumping capacity needed to provide 100-year protection for the structures behind the Coopers Gully levee.

Assuming the levee could prevent flood waters from the Sabine River from entering the interior levee area, 448,000 gpm of pumping capacity would be needed to maintain a peak flood level below 5.5 ft within the levee. Since the current station has a capacity of 323,000 gpm, an additional 125,000 gpm capability would be needed.

Pump manufacturers have indicated that it may be possible to change out a 62,000 gpm pump and replace it with a 104,000 gpm pump and still maintain the hydraulic integrity of the station. A more detailed evaluation of the pump station wet well is warranted by both the engineers and a pump manufacturer to determine if this is possible. If the possibility is good that a 104,000 gpm pump can be substituted for one of the existing 62,000 gpm pumps, a model test may still be required to verify that the larger pump will not cause vortexing or other hydraulic problems.

If a 104,000 gpm pump would function in an existing 62,000 gpm pump slot, three new 104,000 gpm pumps and motors would need to be purchased to bring the station to an approximate 448,000 gpm capacity. The cost of a 104,000 gpm pump and motor is approximately $210,000. The total cost for three 104,000 gpm pumps and motors would be approximately $756,000 including a 20 percent contingency.

Channel Analysis

In order to lower flood elevations in the ditch along Coopers Gully, ditch and culvert improvements are recommended. The ditch should be widened to a 40-ft bottom width from downstream of Curtis Avenue to 10th Street. Improvements could be carried beyond 10th Street, however the FIS study ended at tenth street. The slope of the ditch should be 0.06% and have 3 (horizontal) to 1 (vertical) side slopes. In order to construct the ditch improvements in the narrow right of way between Curtis Avenue and Turret Road, concrete lining is recommended to lessen the top width required. The recommended culvert improvement sizes are shown in Table V.6. Table V.7 shows an estimated cost for these improvements.

TABLE V.6 Recommended Culvert Improvements in Coopers Gully

LOCATION	PROPOSED CULVERT SIZE	PROPOSED FLOW AREA (SQ FT)	EXISTING FLOW AREA (SQ FT)
Curtis Avenue	4 - 8' X 10' RCB	320	171
Turret Road	4 - 8' X 10' RCB	320	160
2nd Street	4 - 8' X 10' RCB	320	168
3rd Street	4 - 8' X 10' RCB	320	169
6th Street	4 - 8' X 10' RCB	320	143
10th Street	5 - 7' X 7' RCB	245	119

TABLE V.7

COST ESTIMATE FOR DITCH IMPROVEMENTS

COOPERS GULLY WATERSHED

CITY OF ORANGE, TEXAS

Item	Quantity	Units	Cost Per Unit ($)	Estimated Cost
Channel Excavation	63,500	CY	$5.75	**$365,125**
Concrete Slope Paving (Curtis to Turret)	4,400	SY	$30.00	**$132,000**
Seeding	14	AC	$1,400.00	**$19,600**
ROW Preparation	13	AC	$700.00	**$9,100**
ROW Purchase	7	AC	$6,000.00	**$42,000**
Culverts at Curtis (4-8x10 RCB)	181	CY	$350.00	**$63,350**
Culverts at Turret (4-8x10 RCB)	181	CY	$350.00	**$63,350**
Culverts at 2nd (4-8x10 RCB)	181	CY	$350.00	**$63,350**
Culverts at 3rd (4-8x10 RCB)	181	CY	$350.00	**$63,350**
Culverts at 6th (4-8x10 RCB)	181	CY	$350.00	**$63,350**
Culverts at 10th (5-7x7 RCB)	144	CY	$350.00	**$50,400**
Street Replacement (6)	1,320	SY	$30.00	**$39,600**
			Subtotal Construction Cost	**$974,575**
			Engineering and Contingencies (30%)	**$292,373**
			Total Estimated Construction Cost	**$1,266,948**

Note: Utility relocation costs are difficult to estimate without detailed maps and are not included.

b. Dayton Street Ditch

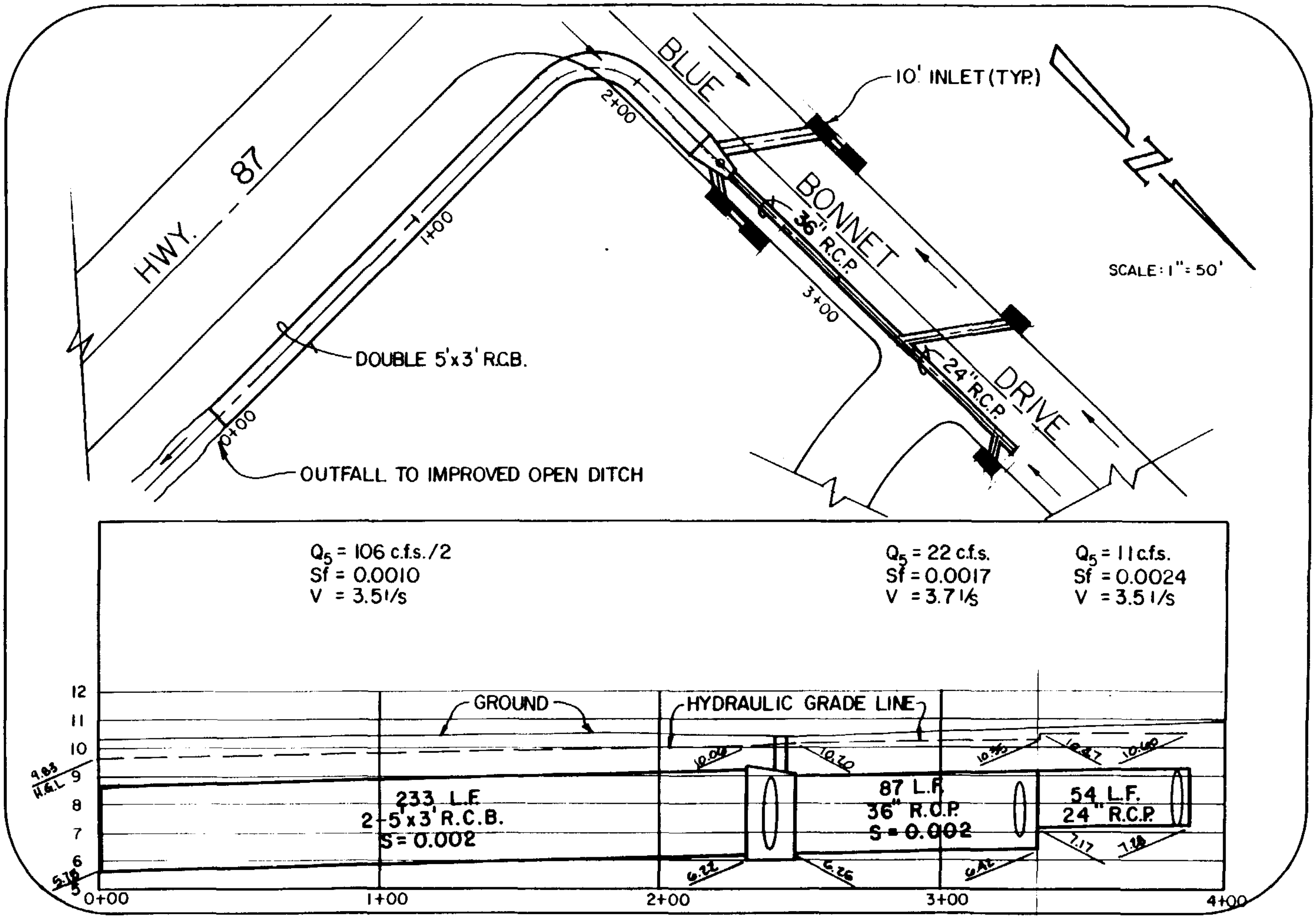
Recommendations for improving the capacity of the Dayton Street Ditch to improve the ponding situation at the intersection of Bluebonnet Drive and Highway 87 include adding a double 6-ft. by 4-ft. box culvert under Highway 87, increasing the culverts under the three driveways to double 5-ft. by 4-ft. boxes, lowering and concrete lining the ditch from Highway 87 to Bluebonnet and adding an inlet and increased storm sewer system along Bluebonnet Drive. See Exhibit V.6 for a plan and profile view of the proposed improvements. Exhibit V.7 shows a plan view of the proposed storm drain improvements along the ditch. Table V.8 shows the approximate costs for installing the improvements. As illustrated in the next section's project priority ranking, each item in the project can be phased to lessen the burden on a capital improvement budget. If the Texas Department of Transportation decides to upgrade Highway 87 in the vicinity of Bluebonnet Drive, the department should be contacted and asked to share in the cost of constructing the ditch and culvert improvements.

Construction on this reach of Hwy 87 is presently being planned by the Texas Highway Department. The proposed culvert improvements should be scheduled to coincide with the highway construction.

The recommended culvert improvements for the street crossings in West Orange include constructing a double 6-ft. by 4-ft box culvert at both Shell Drive and Tanglewood Drive. These projects will mainly benefit West Orange and do not need to be constructed to alleviate the ponding situation at Bluebonnet Drive and Highway 87.

EXHIBIT V.6

PROPOSED STORM DRAIN AT BLUEBONNET AND HWY. 87



CITY OF ORANGE

Carter-Burgess

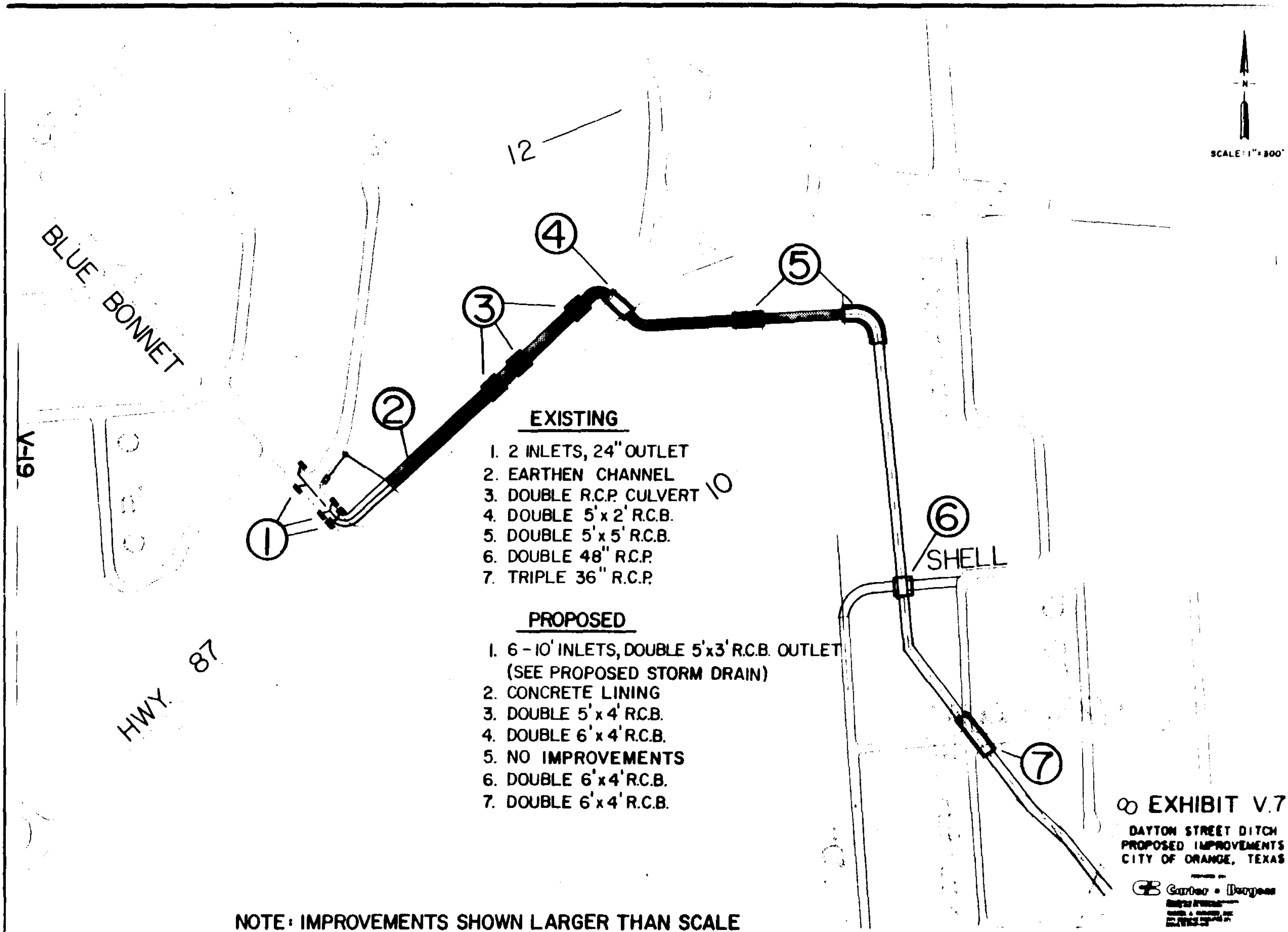

N
SCALE: 1" = 300'
12
10
BLUE BONNET
HWY. 87
SHELL
1
2
3
4
5
6
7
EXISTING
1. 2 INLETS, 24" OUTLET
2. EARTHEN CHANNEL
3. DOUBLE R.C.P. CULVERT
4. DOUBLE 5'x 2' R.C.B.
5. DOUBLE 5'x 5' R.C.B.
6. DOUBLE 48" R.C.P.
7. TRIPLE 36" R.C.P.
PROPOSED
1. 6 - 10' INLETS, DOUBLE 5'x3' R.C.B. OUTLET (SEE PROPOSED STORM DRAIN)
2. CONCRETE LINING
3. DOUBLE 5'x 4' R.C.B.
4. DOUBLE 6'x 4' R.C.B.
5. NO IMPROVEMENTS
6. DOUBLE 6'x4' R.C.B.
7. DOUBLE 6'x 4' R.C.B.
EXHIBIT V.7
DAYTON STREET DITCH
PROPOSED IMPROVEMENTS
CITY OF ORANGE, TEXAS
Carter • Burgess
NOTE: IMPROVEMENTS SHOWN LARGER THAN SCALE

TABLE V.8
COST ESTIMATES FOR
DAYTON STREET DITCH WATERSHED
CITY OF ORANGE,TEXAS

Item	Unit	$/Unit	Quanity	Estimated Cost
Bluebonnet Storm Drain System				
24" RCP	LF	$50	85	**$4,250**
36" RCP	LF	$72	70	**$5,040**
36" RCP Laterals	LF	$72	50	**$3,600**
Double 5'x3' RCB (outfall to ditch)	LF	$180	240	**$43,200**
Inlets	ea.	$1,500	6	**$9,000**
Paving	SY	$17	493	**$8,381**
			Total Storm Drain Cost	**$73,471**
Channel Improvements from Bluebonnet Outfall to 2nd Exist. 5'x5' RCB				
Grading/Excavation	CY	$5	580	**$2,900**
Concrete Lining	SY	$18	2233	**$40,194**
Double 5'x4' RCB (3 driveways)	LF	$190	90	**$17,100**
Double 6'x4' RCB (Hwy. 87)	LF	$200	70	**$14,000**
Paving (for RCB)	SY	$17	460	**$7,820**
			Total Channel Cost	**$82,014**
Downstream Improvements				
Grading/Excavation	CY	$5	380	**$1,900**
Double 6'x4' RCB (Shell Dr.)	LF	$200	20	**$4,000**
Double 6'x4' RCB (Tanglewood)	LF	$200	75	**$15,000**
Paving	SY	$17	160	**$2,720**
			Total Downstream Cost	**$23,620**
			Subtotal Construction Cost	**$179,105**
			Engineering and Contingencies (30%)	**$53,732**
			Total Estimated Construction Cost	**$232,837**

Note: Utility relocation costs are difficult to estimate without detailed maps and are not included.

c. Hudson Gully

As a result of the analysis of the Hudson Gully ditch capacity, the recommendation is made to increase the ditch capacity from Highway 90 upstream to the 37th Street bridge crossing and to add one culvert under Highway 90. The increase in ditch capacity would lower water surface elevations and thus allow the adjacent existing and proposed storm sewer systems to function better during the design 5-year storm. The recommended improvements include adding an additional 9-ft. by 9-ft. reinforced box culvert at Highway 90, increasing the bottom width of the existing ditch by 20 feet from Highway 90 to 37th Street and replacing the concrete slope paving in the area of the ditch where the paving currently exists. A diagram of the proposed culvert upgrade is provided as Exhibit V.8 A cost estimate for accomplishing this construction is included as Table V.9.

EXHIBIT V.8

HUDSON GULLY
PROPOSED CULVERT UPGRADE

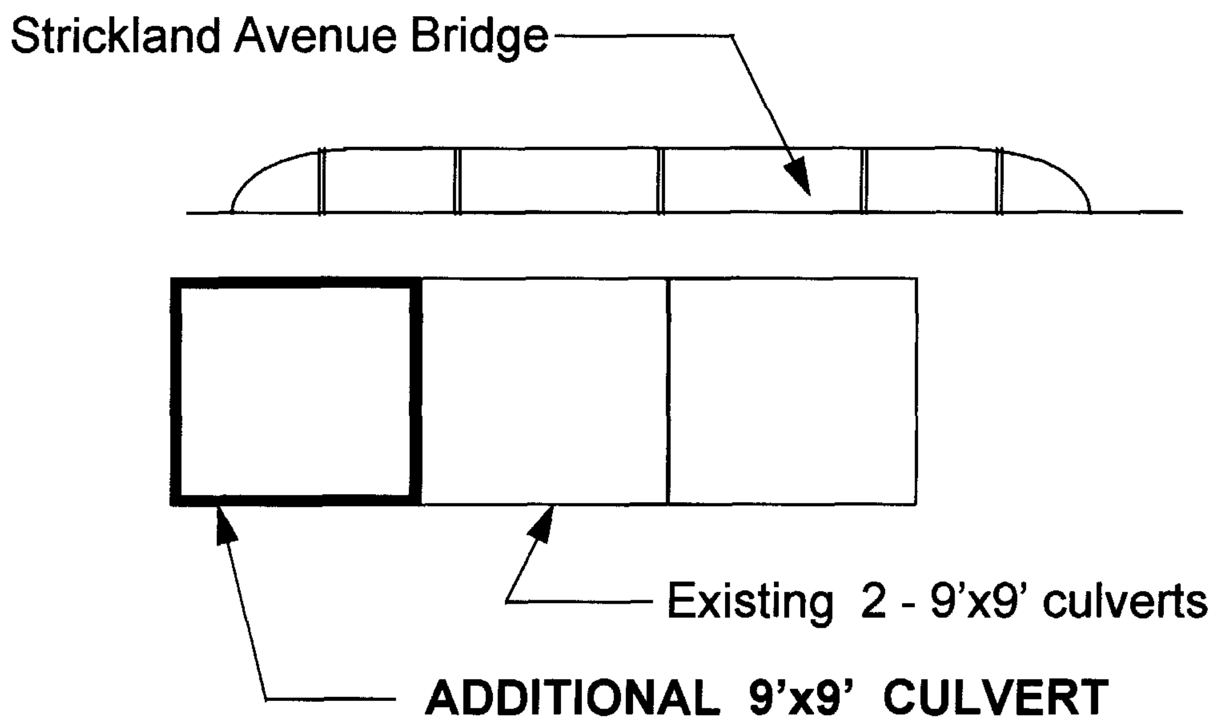


TABLE V.9
COST ESTIMATES FOR CHANNEL IMPROVEMENTS
HUDSON GULLY WATERSHED
CITY OF ORANGE, TEXAS

Item	Unit	$/Unit	Quanity	Estimated Cost
9'x9' Culvert beneath Strickland Ave.	LF	$520.00	80	**$41,600**
Channel Excavation	CY	$5.75	36,000	**$207,000**
Concrete Slope Paving	SY	$30.00	32,000	**$960,000**
Subtotal Construction Cost				**$1,208,600**
Engineering and Contingencies (30%)				**$362,580**
Total Estimated Construction Cost				**$1,571,180**

Note: Utility relocation costs are difficult to estimate without detailed maps and are not included.

d. North Simmons Drive

To prevent backwater from entering the subdivisions on the west side of Simmons Drive from high water in either the Sabine River or Little Cypress Bayou, two sets of flap gates are recommended at culverts crossing Simmons Drive. The recommendation is for one 7-ft. by 7-ft. flap gate to be added on the downstream side of the 7-ft. box under Simmons Drive at the tributary to Little Cypress Bayou. An additional 3-ft. by 3-ft. flap gate should be added to a 3-ft box culvert that also crosses Simmons Drive to the north of the 7-ft. box culvert. Sluice gates should also be added at each location to allow the culverts to be closed off should the flap gates be lodged in the open position. A schematic of the flap gate and sluice gate workings is shown in Exhibit V.9. The costs associated with the addition of these flap gates is shown in Table V.10.

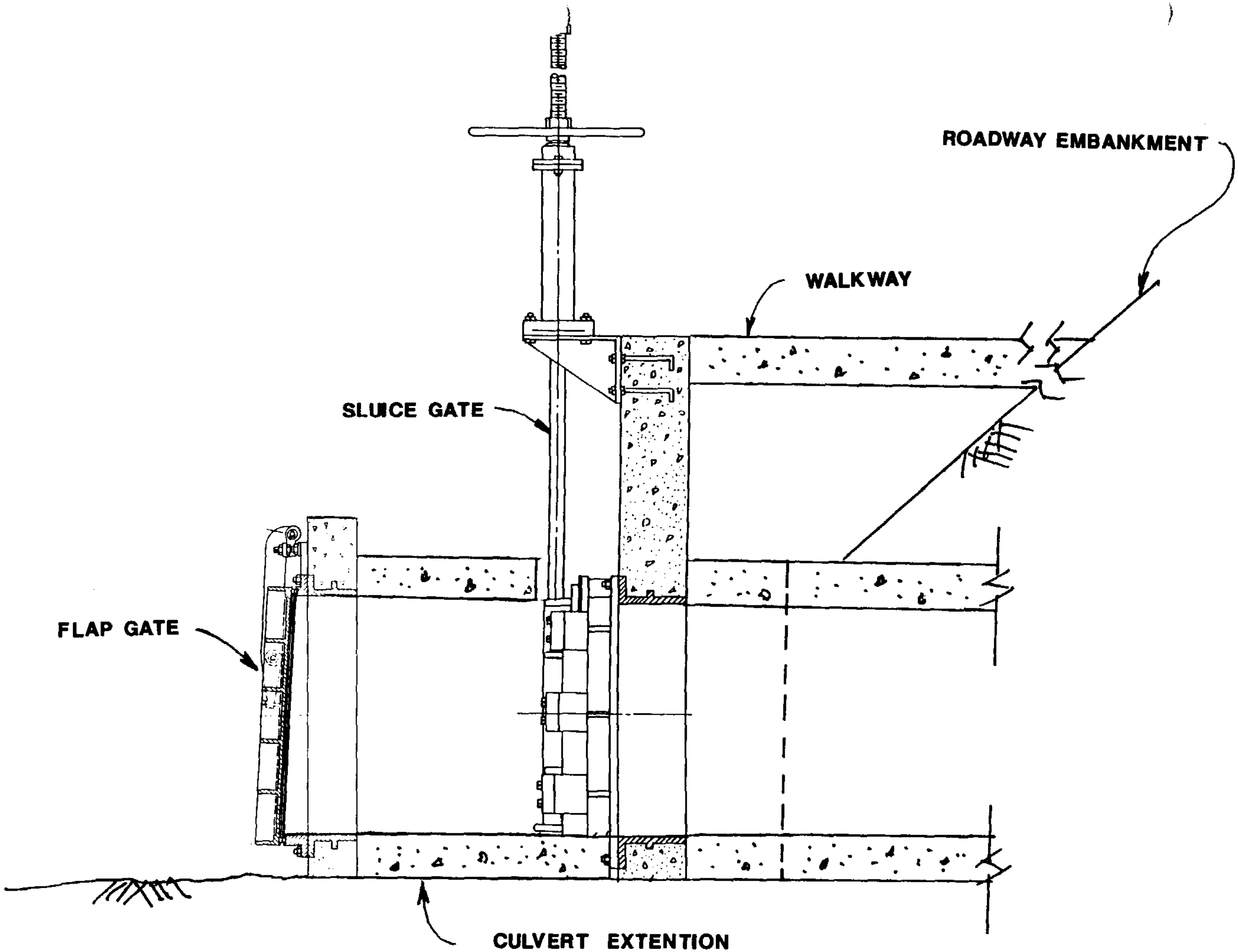


EXHIBIT V.9

PRELIMINARY GATE LAYOUT

TABLE V.10
COST ESTIMATES FOR
NORTH SIMMONS DRIVE FLAPGATES
CITY OF ORANGE,TEXAS

Item	Unit	$/Unit	Quantity	Estimated Cost
84"x84" Flap Gate				
Sluice Gate and Thimble	EA	$23,800	1	**$23,800**
Flap Gate and Thimble	EA	$20,000	1	**$20,000**
Concrete	CY	$300	11	**$3,300**
Excavation	CY	$11	20	**$220**
Catwalk - 36" Wide	EA	$500	1	**$500**
Resodding	EA	$200	1	**$200**
			Total Flapgate Cost	**$48,020**
36"x36" Flap Gate				
Sluice Gate and Thimble	EA	$9,300	1	**$9,300**
Flap Gate and Thimble	EA	$8,500	1	**$8,500**
Concrete	CY	$300	4	**$1,200**
Excavation	CY	$11	20	**$220**
Catwalk - 36" Wide	EA	$500	1	**$500**
Resodding	EA	$200	1	**$200**
			Total Flapgate Cost	**$19,920**
			Subtotal Construction Costs	**$67,940**
			Engineering and Contingency (30%)	**$20,382**
			Total Estimated Construction Cost	**$88,322**

VI. LOCAL DRAINAGE PROJECT PRIORITY RANKING SYSTEM AND CAPITAL IMPROVEMENTS PROGRAM

A. LOCAL DRAINAGE PROJECT PRIORITY RANKING SYSTEM

In order to determine which watershed or local drainage project should receive top priority for construction of improvements, a priority ranking system was developed. The purpose of the priority ranking system is to remove some of the subjectivity from trying to decide which of the recommended projects should be completed first. The system attempts to integrate several objective factors with a scoring system to arrive at a total weighted score that can be used to rank the projects.

The objective factors used in the priority ranking system include cost, traffic affected, people affected, public safety impact, implementation time, social need, and time impact. Each factor is given a raw score according to the values in Table VI.1, with a score of five being the highest desirable, and a score of one being the lowest desirable.

TABLE VI.1 Priority Ranking System Objective Factor

Objective Factor	Raw Score	Description	
Cost			
	5	Estimated Construction Cost	< $100,000
	4	Estimated Construction Cost	< $200,000
	3	Estimated Construction Cost	< $400,000
	2	Estimated Construction Cost	< $700,000
	1	Estimated Construction Cost	>$1,000,000
Traffic Affected			
	5	Present conditions impact interstate highways	
	4	Present conditions impact state highways	
	3	Present conditions impact major thoroughfares	
	2	Present conditions impact local thoroughfares	
	1	Present conditions impact neighborhood streets	

Objective Factor	Raw Score	Description
People Affected		
	5	Present conditions impact 1,000 or less
	4	Present conditions impact 500 or less
	3	Present conditions impact 250 or less
	2	Present conditions impact 100 or less
	1	Present conditions impact 50 or less
Public Safety Impact		
	5	Present conditions impact access by emergency services in light rains
	4	during light to moderate rains
	3	during moderate rains
	2	during thunderstorms
	1	during the ten-year rain event
Implementation Time		
	5	This project can be designed and constructed within 9 months
	4	within 12 months
	3	within 18 months
	2	within 24 months
	1	will require more than 2 years for project design and construction
Social Need		
	5	This project will benefit all citizens equally
	4	This project will mainly benefit economically depressed areas in our community
	3	more benefits to economically depressed areas than non-economically depressed areas
	2	more benefits to non-economically depressed areas than economically depressed areas
	1	mainly benefits non-economically depressed areas

Objective Factor	**Raw Score**	**Description**
		TABLE VI.1 (Con't)
Impact		
	5	Stand alone project will eliminate a significant problem
	4	project is part of a multi-project program requiring less than 3 years to complete
	3	part of a multi-project program requiring less than 5 years to complete
	2	part of a multi-project program requiring less than 10 years to complete
	1	project is part of a multi-project program requiring more than 15 years to complete

After each project is given a raw score for each objective factor, the raw scores are multiplied by a weighting factor. The weighting factor allows the ranking system to give more importance to a particular factor. For instance, public safety impacts can be considered to be of more importance than implementation time, and thus get a higher weighted score. The weighting factors for each of the seven objective factors are shown below in Table VI.2.

TABLE VI.2 Priority Ranking System Weighting Factor

Objective Factor	Weighting Factor
Cost	5
Traffic Affected	7
People Affected	4
Public Safety Impact	10
Implementation Time	3
Social Need	5
Time Impact	6

To arrive at the total score for a particular project or group of projects, simply multiply the raw score of the objective factor by the weighting factor. Then add all seven of the weighted values for each project.

Table VI.3 presents a ranking of the local drainage projects by watershed. According to the table, the highest ranked projects by watershed are those along Dayton Street Ditch. These projects received high weighted scores for cost, public safety impact and the amount of traffic affected. The next highest ranked projects were those in the Coopers Gully watershed. This project consists primarily of the improvements to the pump station. The project received high scores for cost, the number of people affected and public safety impact.

The tables that follow Table VI.3 rank the individual projects within each watershed by the same priority ranking system. Thus, the City can begin implementing projects in the highest ranking watershed, and can implement the projects according to their individual rank within the watershed.

For comparison purposes, all of the projects are ranked together and sorted in order from highest rank to lowest rank in Table VI.II.

B. CAPITAL IMPROVEMENTS PROGRAM

Based upon the results of the priority ranking system described in the previous section, the City of Orange should begin an annual capital improvements program to accomplish the recommended localized flooding improvements. The 23 localized flooding improvement projects recommended for implementation were prioritized according to the ranking system described above. The results of this ranking are shown in Table VI.11.

If the City could use $100,000 in entitlement funds from the Community Block Grant program and about $100,000 from the general fund each year, then a fairly aggressive capital improvements program could be implemented. The ranking in Table VI.11 shows which projects should be undertaken first. In order to assist the City in developing this capital improvements program for the projects recommended in this

TABLE VI.3
LOCAL DRAINAGE PROJECTS RANKED BY WATERSHED

PROJECT	FACTORS								
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
OLD TOWN	3	3	5	1	1	5	1		RAW SCORE
	15	21	20	10	3	25	6	100	WEIGTHTED SCORE
NORTH SIMMONS	4	3	5	2	4	4	5		
	20	21	20	20	12	20	30	143	
HUDSON GULLY	1	2	4	3	2	2	2		
	5	14	16	30	6	10	12	93	
COOPERS GULLY	2	3	5	3	4	4	3		
	10	21	20	30	12	20	18	131	
DAYTON ST. DITCH	4	4	5	5	4	1	4		
	20	28	20	50	12	5	24	159	
SUNSET DRIVE	2	2	4	3	2	1	4		
	10	14	16	30	6	5	24	105	
CHERRY & 13TH	2	2	3	3	2	4	3		
	10	14	12	30	6	20	18	110	

TABLE VI.4

RANKING OF OLD TOWN DR. LOCAL DRAINAGE PROJECTS

PROJECT	FACTORS								
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
LINE SEGMENT OT1	5	3	3	1	4	5	1		RAW SCORE
	25	21	12	10	12	25	6	111	WEIGTHTED SCORE
LINE SEGMENT OT2	4	3	4	1	3	5	1		
	20	21	16	10	9	25	6	107	
LINE SEGMENT OT3	4	3	4	1	3	5	1		
	20	21	16	10	9	25	6	107	
LINE SEGMENT OT4	4	3	4	1	3	5	1		
	20	21	16	10	9	25	6	107	

TABLE VI.6
RANKING OF HUDSON GULLY LOCAL DRAINAGE PROJECTS

PROJECT		COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS
WEIGHT		5	7	4	10	3	5	6	
LINE SEGMENT HG1	RAW SCORE	4	1	2	3	4	2	4	
	WEIGTHTED SCORE	20	7	8	30	12	10	24	111
LINE SEGMENT HG4		3	1	3	3	3	2	4	
		15	7	12	30	9	10	24	107
LINE SEGMENT HG5		5	1	1	3	5	2	4	
		25	7	4	30	15	10	24	115
LINE SEGMENT HG6		5	1	2	3	5	2	4	
		25	7	8	30	15	10	24	119
LINE SEGMENT HG7		5	1	1	3	5	2	4	
		25	7	4	30	15	10	24	115
LINE SEGMENT HG8		3	2	3	3	2	2	4	
		15	14	12	30	6	10	24	111
ADD'L BLUEBON LINE		5	2	2	3	4	2	4	
		25	14	8	30	12	10	24	123
CHANNEL IMPROVMT		1	2	4	3	1	2	4	
		5	14	16	30	3	10	24	102

TABLE VI.4
RANKING OF OLD TOWN DR. LOCAL DRAINAGE PROJECTS

PROJECT	FACTORS								
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
LINE SEGMENT OT1	5	3	3	1	4	5	1		RAW SCORE
	25	21	12	10	12	25	6	111	WEIGTHTED SCORE
LINE SEGMENT OT2	4	3	4	1	3	5	1		
	20	21	16	10	9	25	6	107	
LINE SEGMENT OT3	4	3	4	1	3	5	1		
	20	21	16	10	9	25	6	107	
LINE SEGMENT OT4	4	3	4	1	3	5	1		
	20	21	16	10	9	25	6	107	

TABLE VI.5
RANKING OF NORTH SIMMONS DR. LOCAL DRAINAGE PROJECTS

PROJECT	FACTORS								
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
LINE SEGMENT SD1	5	1	1	3	5	3	4		RAW SCORE
	25	7	4	30	15	15	24	120	WEIGTHTED SCORE
LINE SEGMENT SD3	5	1	1	3	5	3	4		
	25	7	4	30	15	15	24	120	
FLAP GATES	5	3	5	2	5	4	5		
	25	21	20	20	15	20	30	151	

TABLE VI.6
RANKING OF HUDSON GULLY LOCAL DRAINAGE PROJECTS

PROJECT	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
LINE SEGMENT HG1	4	1	2	3	4	2	4		RAW SCORE
	20	7	8	30	12	10	24	111	WEIGTHTED SCORE
LINE SEGMENT HG4	3	1	3	3	3	2	4		
	15	7	12	30	9	10	24	107	
LINE SEGMENT HG5	5	1	1	3	5	2	4		
	25	7	4	30	15	10	24	115	
LINE SEGMENT HG6	5	1	2	3	5	2	4		
	25	7	8	30	15	10	24	119	
LINE SEGMENT HG7	5	1	1	3	5	2	4		
	25	7	4	30	15	10	24	115	
LINE SEGMENT HG8	3	2	3	3	2	2	4		
	15	14	12	30	6	10	24	111	
ADD'L BLUEBON LINE	5	2	2	3	4	2	4		
	25	14	8	30	12	10	24	123	
CHANNEL IMPROVMT	1	2	4	3	1	2	4		
	5	14	16	30	3	10	24	102	

TABLE VI.7
RANKING OF COOPERS GULLY LOCAL DRAINAGE PROJECTS

PROJECT	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
	FACTORS								
WEIGHT	5	7	4	10	3	5	6		
UPGRADE PUMP STA.	2	3	5	3	4	4	3		RAW SCORE
	10	21	20	30	12	20	18	131	WEIGTHTED SCORE
IMPROVE CHANNEL	1	2	4	3	2	3	2		
	5	14	16	30	6	15	12	98	

TABLE VI.8
RANKING OF DAYTON ST. DITCH LOCAL DRAINAGE PROJECTS

PROJECT	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
BLUEBONNET SYSTM	5	4	5	5	4	5	3		RAW SCORE
	25	28	20	50	12	25	18	178	WEIGTHTED SCORE
CHANNEL IMPRVMTS	5	4	5	4	4	5	3		
	25	28	20	40	12	25	18	168	
DOWNSTRM IMPR	5	1	3	3	4	2	4		
	25	7	12	30	12	10	24	120	

TABLE VI.9
RANKING OF SUNSET DRIVE DR. LOCAL DRAINAGE PROJECTS

PROJECT	FACTORS								
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
LINE SEGMENT SS1	2	2	4	3	2	1	4		RAW SCORE
	10	14	16	30	6	5	24	105	WEIGTHTED SCORE

TABLE VI.10
RANKING OF CHERRY ST. LOCAL DRAINAGE PROJECTS

PROJECT	FACTORS								
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
LINE SEGMENT CH1	3	1	3	3	3	4	3		RAW SCORE
	15	7	12	30	9	20	18	111	WEIGTHTED SCORE
LINE SEGMENT CH2	4	1	2	3	4	4	4		
	20	7	8	30	12	20	24	121	

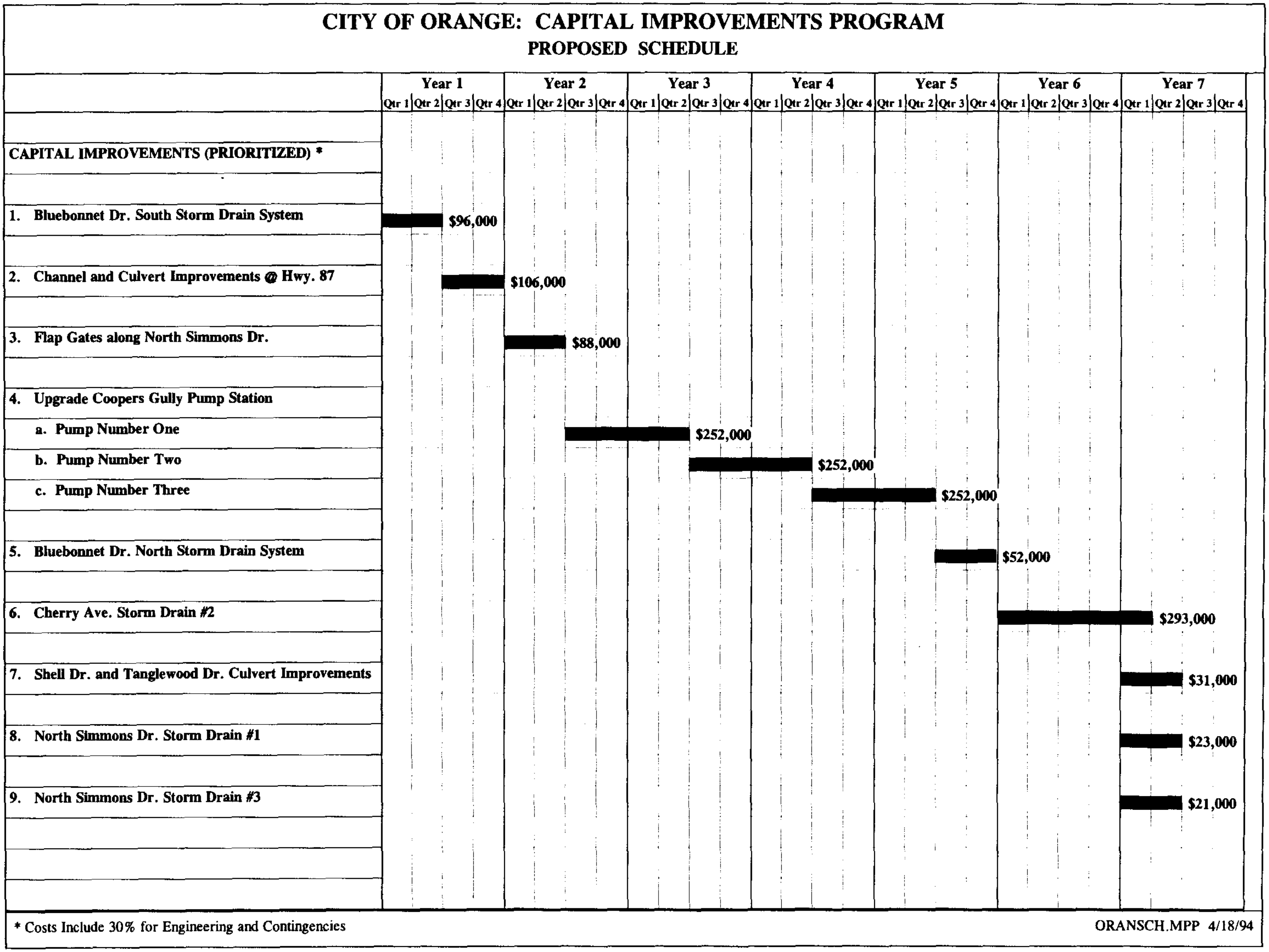
TABLE VI.11
CONSTRUCTION PRIORITY OF LOCAL DRAINAGE PROJECTS

PROJECT	FACTORS								
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS	
WEIGHT	5	7	4	10	3	5	6		
BLUEBONNET SYSTM	5	4	5	5	4	5	3		RAW SCORE
(@ Dayton Ditch)	25	28	20	50	12	25	18	178	WEIGTHTED SCORE
CHANNEL IMPRVMTS	5	4	5	4	4	5	3		
(@ Highway 87)	25	28	20	40	12	25	18	168	
FLAP GATES	5	3	5	2	5	4	5		
(@ N. Simmons Dr.)	25	21	20	20	15	20	30	151	
UPGRADE PUMP STA.	2	3	5	3	4	4	3		
(@ Coopers Gully)	10	21	20	30	12	20	18	131	
ADD'L BLUEBON LINE	5	2	2	3	4	2	4		
(To Hudson Gully)	25	14	8	30	12	10	24	123	
LINE SEGMENT CH2	4	1	2	3	4	4	4		
	20	7	8	30	12	20	24	121	
DOWNSTRM IMPR	5	1	3	3	4	2	4		
(@ Dayton St.)	25	7	12	30	12	10	24	120	
LINE SEGMENT SD1	5	1	1	3	5	3	4		
	25	7	4	30	15	15	24	120	
LINE SEGMENT SD3	5	1	1	3	5	3	4		
	25	7	4	30	15	15	24	120	
LINE SEGMENT HG6	5	1	2	3	5	2	4		
	25	7	8	30	15	10	24	119	
LINE SEGMENT HG5	5	1	1	3	5	2	4		
	25	7	4	30	15	10	24	115	
LINE SEGMENT HG7	5	1	1	3	5	2	4		
	25	7	4	30	15	10	24	115	

TABLE VI.11
CONSTRUCTION PRIORITY OF LOCAL DRAINAGE PROJECTS

PROJECT	FACTORS							
	COST	TRAFFIC AFFECTED	PEOPLE AFFECTED	PUBLIC SAFETY IMPACT	IMPLEMENTATION TIME	SOCIAL NEED	IMPACT	TOTALS
WEIGHT	5	7	4	10	3	5	6	
LINE SEGMENT OT1	5	3	3	1	4	5	1	
	25	21	12	10	12	25	6	111
LINE SEGMENT HG1	4	1	2	3	4	2	4	
	20	7	8	30	12	10	24	111
LINE SEGMENT HG8	3	2	3	3	2	2	4	
	15	14	12	30	6	10	24	111
LINE SEGMENT CH1	3	1	3	3	3	4	3	
	15	7	12	30	9	20	18	111
LINE SEGMENT OT2	4	3	4	1	3	5	1	
	20	21	16	10	9	25	6	107
LINE SEGMENT OT3	4	3	4	1	3	5	1	
	20	21	16	10	9	25	6	107
LINE SEGMENT OT4	4	3	4	1	3	5	1	
	20	21	16	10	9	25	6	107
LINE SEGMENT HG4	3	1	3	3	3	2	4	
	15	7	12	30	9	10	24	107
LINE SEGMENT SS1	2	2	4	3	2	1	4	
	10	14	16	30	6	5	24	105
HUDSON GULLY	1	2	4	3	1	2	4	
Channel Improvement	5	14	16	30	3	10	24	102
COOPERS GULLY	1	2	4	3	2	3	2	
Channel Improvement	5	14	16	30	6	15	12	98

EXHIBIT VI.1

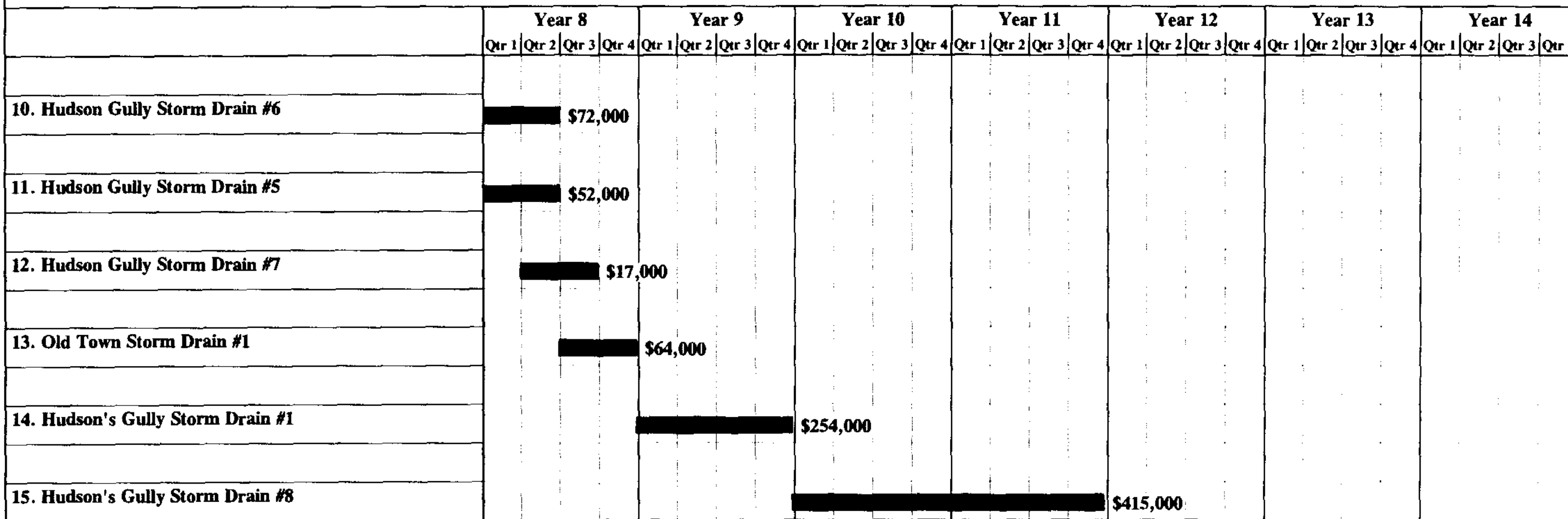


CITY OF ORANGE: CAPITAL IMPROVEMENTS PROGRAM
PROPOSED SCHEDULE
Year 1
Year 2
Year 3
Year 4
Year 5
Year 6
Year 7
Qtr 1 Qtr 2 Qtr 3 Qtr 4
CAPITAL IMPROVEMENTS (PRIORITIZED) *
1. Bluebonnet Dr. South Storm Drain System
$96,000
2. Channel and Culvert Improvements @ Hwy. 87
$106,000
3. Flap Gates along North Simmons Dr.
$88,000
4. Upgrade Coopers Gully Pump Station
a. Pump Number One
$252,000
b. Pump Number Two
$252,000
c. Pump Number Three
$252,000
5. Bluebonnet Dr. North Storm Drain System
$52,000
6. Cherry Ave. Storm Drain #2
$293,000
7. Shell Dr. and Tanglewood Dr. Culvert Improvements
$31,000
8. North Simmons Dr. Storm Drain #1
$23,000
9. North Simmons Dr. Storm Drain #3
$21,000
* Costs Include 30% for Engineering and Contingencies
ORANSCH.MPP 4/18/94

EXHIBIT VI.1 (Con't.)

CITY OF ORANGE: CAPITAL IMPROVEMENTS PROGRAM

PROPOSED SCHEDULE



* Costs Include 30% for Engineering and Contingencies

ORANSCH.MPP 4/18/94

VII. ADDITIONAL CONCERNS

This section discusses various additional topics as outlined in the City's contract.

A. BASE MAP PREPARATION

Base maps have been created and presented to the city. The base maps cover the entire city limits and are scaled to 1"=200'. The two-feet contour interval topographic information was based on the U.S.G.S. Orange, Texas quadrangle compiled in 1932. Planimetric information originated from digital computer files provided by the City. This information included streets, railroads, creeks, lakes, rivers, houses, and buildings.

The base map information has been compiled in Microstation format. One of the benefits of having this data in Microstation format is the maps may be updated, added to, and printed at any time. A preliminary effort to map existing storm drains in the database was begun. This effort should be continued. In addition to storm drains, all other utilities such as water lines, phone lines, power lines, and sanitary sewers should be included in this mapping system for future reference. A complete utility mapping system would have many benefits to the city for new construction, maintenance and repair, billing, and future GIS needs.

B. STORM DRAIN CROSS-REFERENCING SYSTEM

A storm drain cross-referencing system was developed to facilitate location and maintenance scheduling of existing storm drains. The system is based on designation by watershed, outfall sequence, and location node. Watersheds are designated by the following symbols:

AB = Adams Bayou
CG = Coopers Gully
HG = Hudson Gully
LC = Little Cypress
OT = Old Town

The outfall sequence is designated by number in the order of appearance from a watershed's confluence to its headwater. The location node is designated by a letter at each intersection or pipe size change. The location nodes start with A at the outfall and progress through the alphabet upstream along the mainline. Lateral nodes may be designated by any letter not already used. For extensive drain networks, letters may be doubled such as AA or BB to designate additional nodes beyond Z. An example of the cross referencing system is the storm drain reach designation **CG2BC**. CG refers to Coopers Gully, 2 refers to the 2nd outfall (progressing upstream) from the Sabine River, and BC refers to the storm drain reach between the second and third street intersections. All of the existing storm drains that were entered in to this study's database have been cross referenced. They are shown in Exhibits III.12, III.19, III.20, III.21, and III.22.

C. IMPACT OF FULLY-DEVELOPED WATERSHEDS

As a watershed develops from a natural or rural setting to urbanized land uses, the rate and volume of runoff from a given rainfall will be increased. Urbanization alters the hydrology of a watershed by improving its hydraulic efficiency, reducing its surface infiltration and reducing its storage capacity. The more impervious and densely developed an area, the higher the rate of runoff. As the runoff increases, the flow draining to rivers and creeks increases also. If the rivers and creeks remain in their natural condition, the increased flow will spill out of the banks more frequently than it did before the watershed experienced development. The result will be higher water surface elevations in the river for the same rainfall amount. Several watersheds in, or adjacent to, the City of Orange have or may undergo this process of urbanization.

The larger watersheds in the Orange area that were analyzed in this report were Adams Bayou, Coopers Gully, Old Town, Hudson Gully and Dayton Street. The effects that urbanization or development may have on these watersheds is discussed in the following paragraphs.

The watershed that contributes runoff to Adams Bayou is the largest watershed specifically analyzed in this study. Currently, the 83-square mile watershed is perhaps 20 to 30 percent developed. Due to the relatively large size of this watershed, and current state of development, full urbanization could greatly increase the flows in Adams Bayou as it passes through the City of Orange. If the watershed did reach full development, the floodplain width and water surface elevations in the Bayou, through the City, would be increased. The result of this increase would be that more land and structures would be inundated or surrounded by flood waters during larger floods. However, given the recent growth rate in the county, the Adams Bayou watershed will not reach full development in the near future. As the watershed approaches full development, some of the projects analyzed in this report, but not found financially feasible at this time, may need to be implemented. These include diverting flow around Orange to the Sabine River and dredging Adams Bayou to improve the conveyance of the channel.

The area within the Coopers Gully watershed is primarily urbanized. As a result the flows in Coopers Gully are not expected to increase to a large degree. However, the existing drainage system is inadequate to convey this fully-urbanized flow in some areas. The inadequacy may be due in part to development that exceeded improvements to the drainage system.

The Old Town watershed is also primarily fully developed. Consequently, the amount of runoff for a given rainfall in the watershed is not expected to increase to a large degree. Several improvements to the storm drain system are recommended in this report to adequately convey the amount of runoff that reaches the system.

Hudson Gully on the west side of Orange drains a watershed that is approximately 40 percent developed. As the watershed develops, flows and water surface elevations can be expected to increase. Presently, not much development extends beyond the current upstream limit of the improved channel on Hudson Gully. This undeveloped area is not experiencing much current development. If development increases, improvements to the Gully similar to those recommended in this report may become economically feasible.

The area on the west side of Orange and south of the Hudson Gully watershed has been called the Dayton Street watershed in this study. This watershed is approximately 50 percent developed. Further development will increase the flows in the ditch that drains the watershed. The improvements that are recommended in this report to improve the conveyance of the drainage system on the upper end of the watershed (along Highway 87 and Bluebonnet Drive) drain an area that is already developed. The areas in the watershed that may experience an increase in flows along the drainage ditch are those areas in West Orange south of South Avenue and along the ditch.

D. ENVIRONMENTAL RESOURCES AND PROTECTION

As has been documented in numerous studies and reports, many areas in the Orange vicinity are rich in environmental areas such as mature woodlands and wetlands that provide wildlife habitat and enhance the quality of life. As development increases in a previously-undeveloped area or as the construction of flood control projects are considered, these sensitive areas deserve consideration to insure that impacts due to construction and urbanization are minimized.

Areas within the City of Orange that contain mature woodlands are primarily along Adams Bayou, along the Sabine River and along Little Cypress Bayou south of Interstate 10. These woodland areas are shown in the Master Plan for the City.

Due to the relatively flat terrain, abundance of moisture and proximity to a major river, the City of Orange and surrounding areas contain numerous areas that could contain wetland plants and wildlife. The National Wetland Inventory maps compiled by the federal government show many areas in the Orange vicinity that are classified as wetlands. These maps are available for review at the Corps of Engineers District office in Galveston.

Development, in general, has often been cited as being detrimental to environmental areas such as woodlands and wetlands. Not only does the development compete for space with these areas, but the process of development can lead to long-term degradation of existing areas.

The urbanization of a previously-undeveloped area has the potential to not only increase the amount of surface storm water runoff, but also to substantially increase the amount of pollutants discharged into the water bodies that feed these areas. The source of some of these pollutants is rainfall precipitation that runs across impervious surfaces and collects soil particles, heavy metals, trace organic compounds, hydrocarbons, nutrients, pesticides and other contaminants during the journey to the receiving stream. Once in the receiving stream, these pollutants may settle out and accumulate along the stream bed, or they may stay suspended or both. Suspended solids increase turbidity, convey nutrients, organic toxic compounds and bacteria adsorbed on their surface, and compete with the aquatic life for dissolved oxygen.

As residential neighborhoods mature, they tend to become more impervious as decks, patios, driveways, infill developments and road improvements are constructed. Older neighborhoods have increased litter generation and pet dropping rates, while the general level of urban housekeeping declines.

Typically, undeveloped land is between 0 and 10 percent impervious cover, depending on soil types and vegetation coverage. As land is developed, a lot for an average single-family residence becomes 25 to 30 percent impervious while some large commercial developments such as shopping centers become 95 to 100 percent impervious. Pollutants rapidly accumulate on these impervious areas and are easily washed away by the next storm event. It has been estimated that, once deposited, up to 90 percent of the deposited atmospheric pollutants are delivered to receiving waters by storm water runoff.

Numerous alternatives are available to mitigate the impacts that construction or urbanization has on the environment. The alternatives could comprise an entire volume. Those that are particularly applicable to the Orange area are discussed briefly in the following paragraphs.

One method for preventing impacts of development on existing trees is to enact a tree preservation ordinance within the City. Tree preservation ordinances can either prevent the removal of trees or require the replacement of trees removed during development. Many ordinances require a one-to-one or greater replacement ratio for removed trees above a prescribed size. Tree preservation ordinances are gaining acceptability in developing cities such as Dallas, Austin and Houston.

Another method for preserving woodlands and wildlife habitat is to set aside certain areas as nature preserves. This has been done along Adams Bayou in the City of Orange. It is recommended that this area remain a preserve and that the restrictions to development in this area remain in place.

Wetlands protection regulations are currently in place at the federal level and have been getting stricter in recent years. Section 404 of the Clean Water Act contains regulations that prevent certain impacts to wetlands or waters of the United States.

Currently the Corps of Engineers is charged with the responsibility for enforcing and implementing the regulations. Any development or construction that occurs within the Orange area is susceptible to the regulations. Before construction is initiated on any flood control project, a wetlands determination should be made on the project area. Impacts to wetlands within the project area determine the level of permitting that is required by the Section 404 process.

After any project is constructed that affects 100-year floodplain elevations within the City of Orange, surrounding cities or Orange County, the governing local entity should request from the Federal Emergency Management Agency a Letter of Map Revision. This will insure that the floodplains within the area are accurately depicted on the Flood Insurance Rate Maps.

There are also numerous methods available to lessen the amount of pollutants that reach receiving streams as a result of the urbanization process described above. The City could encourage or require the construction of water quality ponds to trap sediment that washes off of commercial, industrial or residential areas. The ponds can also be designed to remove other pollutants. The use of ponds is applicable in existing developed areas. However, the construction of the ponds is more easily required for areas undergoing development. Other programs such as litter control and prevention, used oil collection facilities and educational programs aimed at preventing pollution in the urban area have been effective throughout the state.

As for the projects discussed in this report, none of the projects considered for implementation, with the exception of the levee alternatives, would impact the large stands of mature woodlands around Orange. Levee construction along Adams Bayou and the Sabine River, as studied herein, could impact mature woodlands. This potential impact somewhat decreases the feasibility of such alternatives even further. Construction of a levee along Adams Bayou would be difficult without impacting either the existing nature preserve or existing established neighborhoods. If impacts to the existing woodlands are unavoidable, mitigation measures such as tree replacement should be considered.

Construction of the channel alternatives recommended or the levee alternatives would impact certain existing wetland areas. Impacts to wetland areas are regulated by the Corps of Engineers and Section 404 of the Clean Water Act. Before either of these types of projects are constructed, a wetlands determination should be made in the project area and mitigation of the potential impacts considered in the plan of development. When impacts are unavoidable, a Section 404 permit should be sought from the Corps of Engineers.

E. EROSION AND MAINTENANCE PROGRAM

The natural channels in Orange and surrounding communities need to be regularly maintained. Regular maintenance should include mowing, debris and shrub removal, and dredging when necessary. Drainage capacities can be severely restricted due to overgrown natural channels. Photo VII.1 is an example of maintained conditions. The photo was taken in February of 1993. Contrast this picture with Photo VII.2 which shows the same channel in October of 1993. The channel is located in Orange by the Walmart store and is part of Dayton Street Ditch.

Erosion protection is necessary to insure that channels maintain their capacity and stability and to avoid excessive transport and deposition of eroded material. The three main parameters which affect erosion are vegetation, soil type and the magnitude of flow velocities and turbulence. In general, silty and sandy soils are the most vulnerable to erosion.

The necessity for erosion protection should be anticipated in the following settings:

1. Areas of channel curvature, especially where the radius of the curve is less than three times the design flow top width.
2. Around bridges where channel transitions create increased flow velocities.
3. When the channel invert is steep enough to cause excessive flow velocities.
4. Along grassed channel side slopes where significant sheet flow enters the channel laterally.
5. At channel confluences.
6. In areas where the soil is particularly prone to erosion.

Sound engineering judgement and experience should be used in locating areas requiring erosion protection. It is often prudent to analyze potential erosion sites following a significant flow event to pinpoint areas of concern.

Minimum Erosion Protection Requirements recommended for Orange County are as follows:

Confluences - A healthy cover of grass must also be established above the top edge of the lining extending to the top of the bank. The top edge of the lining shall extend to the 25-year water surface elevation.

Bends - When required, erosion protection must extend along the outside bank of the bend and at least 20 feet downstream of it. Additional protection on the channel bottom and inside bank, or beyond 20 feet downstream, will be required if high allowable velocities are exceed.
Culverts - In areas where outlet velocities exceed six feet per second on to a grass-lined channel, channel lining or an energy dissipation structure will be required.
Outfalls - Erosion protection will be necessary in areas of high turbulence or velocity as typically found at the outfall of backslope drains, roadside ditches, and storm sewers into the main channel.

The use of rip rap is encouraged because of its proven past performance, its flexibility, and its high Manning's "n" value (approximately 0.04). Rip rap is defined as broken concrete rubble or well-rounded stone. A discussion of rip rap design can be found in Hydraulic Design of Flood Control Channels, EM 110-2-1601, U.S. Department of the Army, Corps of Engineers, July 1970.

Rip rap used for channel lining should conform to the following general characteristics.

1. Minimum mat thickness is 18 inches.
2. 80-pound to 150-pound blocks, evenly graded.
3. Minimum 6-inch thickness per block.
4. No exposed steel in broken concrete rubble.
5. Thickness of layer at toe of slope should be increased below the anticipated scour depth.
6. Maximum steepness of the side slope, 2 (horizontal) to 1 (vertical).
7. Gravel bedding or filter fabric required for extensive installations or where warranted by soil conditions.

The use of backslope drains and swales should be investigated. These systems collect overland flow from channel overbanks and other areas not draining to the storm sewer collection system. Their purpose is to prevent excessive overland flow from eroding grass-lined channel side slopes as it enters the channel. Subject to City approval, backslope drains may not be required in undeveloped or sparsely developed areas.

VISUAL CONTRAST OF MAINTAINED AND OVERGROWN CHANNELS

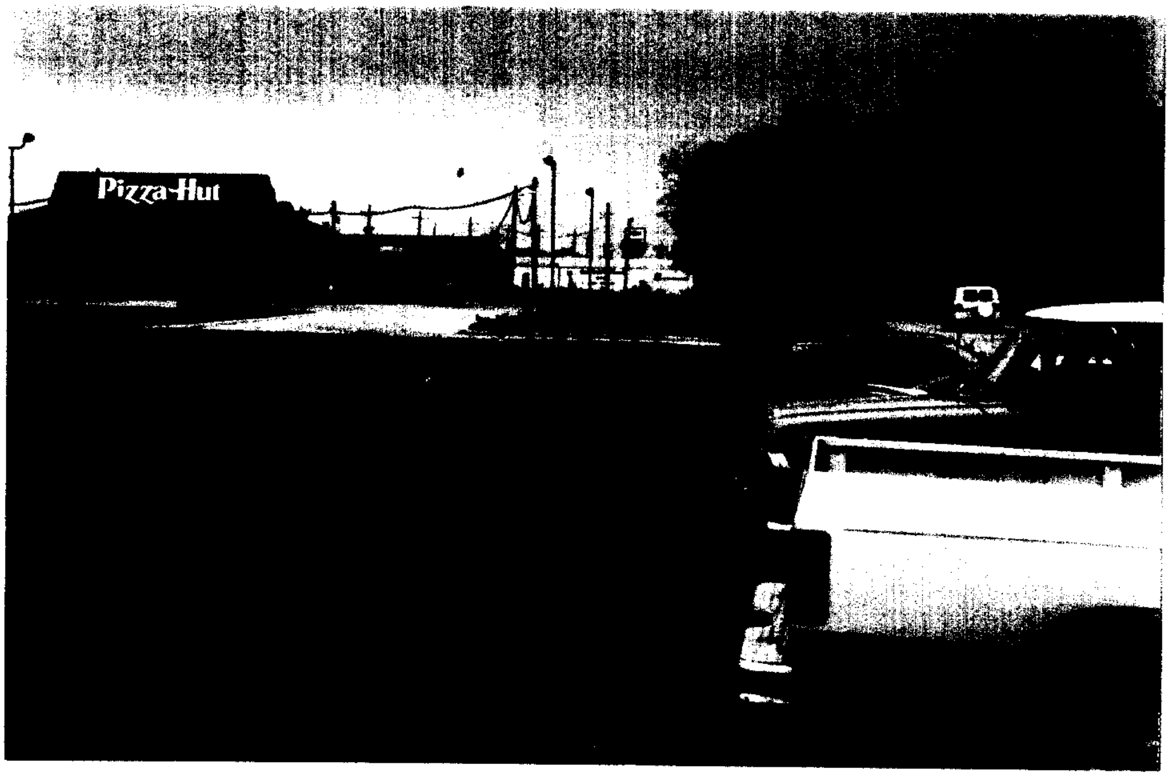


PHOTO VII.1 Maintained Channel

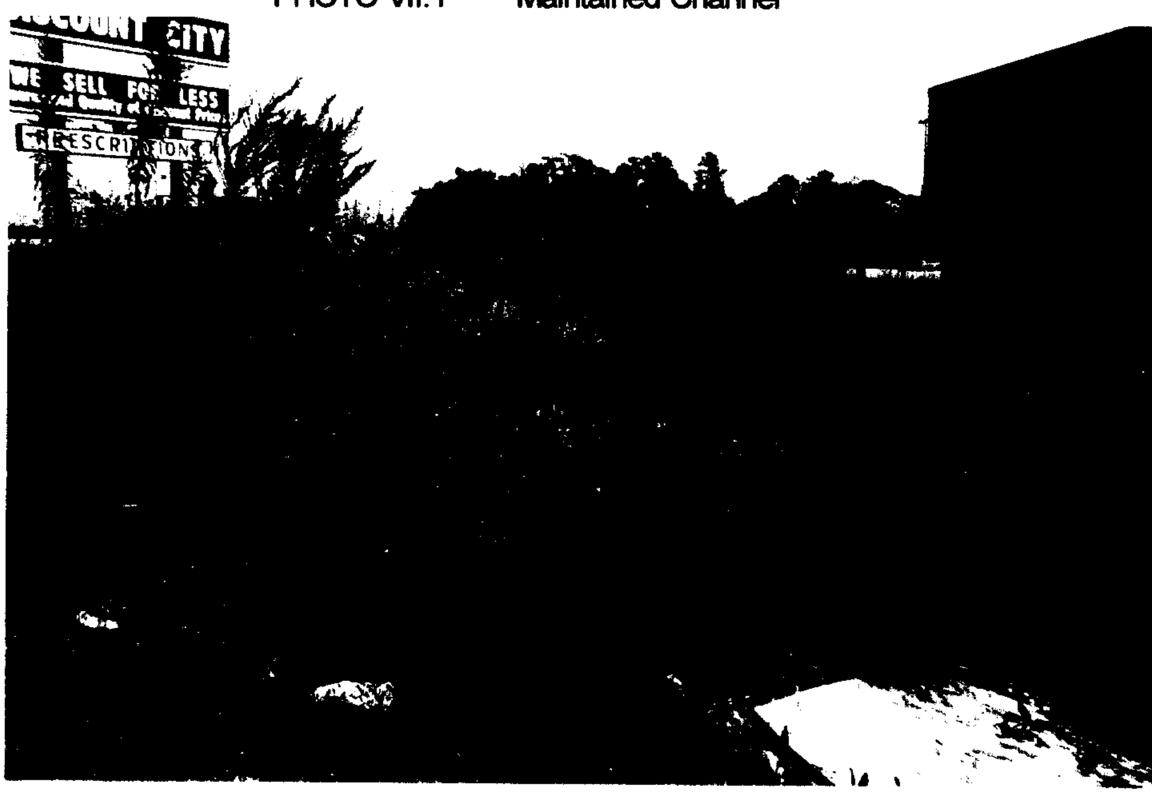


PHOTO VII.2 Overgrown Channel

F. NEW EPA STORM WATER REGULATIONS

Recently, the EPA passed regulations concerning storm water runoff releases to waters of the U.S. The legislative authority falls under the Water Quality Act of 1987. The specific storm water program is referred to as the National Pollutant Discharge Elimination System, or NPDES.

NPDES legislation requires municipalities and certain industries to obtain storm water permits from their regional EPA authority. The first tier of the program affected only larger cities with greater than 100,000 population and industries within specific standard industrial code (SIC) groups. Eventually, all cities will be regulated, including Orange.

To comply with the regulations, larger cities had to submit a two-part application. The application addressed many areas including the city's legal authority to enforce initiatives, identification of pollutant sources, characterizing and analyzing discharges, establishing a storm water management program, and a thorough fiscal analysis for the program.

The NPDES regulation for smaller cities of less than 100,000 population is scheduled to be released in October of 1994. Most likely their scope will be smaller than that for the larger cities. Until the new regulations are released, Orange is not currently affected. The City should monitor the progress of the regulations to be released in October of 1994. When the regulations are released, a dead line of from one to three years will probably be provided to comply with the regulations.

Since the regulations have not been released, the cost of compliance for cities such as Orange is difficult to estimate.

G. FLOODWAY DEVELOPMENT

The floodplain areas along major bayous and drainage-ways such as Adams Bayou and Little Cypress Bayou provide unique open space corridors, which when preserved, form a useful network of recreational linkages. These areas are typically rich in mature tree cover and offer passive settings for recreational opportunities.

The natural, wooded floodplain areas should be preserved and utilized for recreational purposes. The 1990 Texas Outdoor Recreation Plan states that most streams in the region are under-utilized and recommends utilization of these areas.

Types of recreation which are suitable in these areas include hiking, bicycling, jogging, nature study areas, fishing, picnicking and camping. Other recreational opportunities such as soccer, football, baseball and basketball may be considered in areas where constant high water is not a problem. Facilities may be constructed with flood considerations in mind, thus allowing recreation during times of low rainfall.

These natural floodplain corridors and wetland areas provide good opportunities for obtaining matching grant money from the Texas Parks and Wildlife Department and should be pursued by the City of Orange. These funds could match donated land or money targeted for development of recreational facilities and could aid in developing the corridors as park spaces.

Planning for the open space and floodplain corridors should be coordinated with overall, comprehensive planning for the City.

H. PROPOSED DRAINAGE MANUAL AND DRAINAGE ORDINANCE

It is recommended that Orange prepare or authorize the preparation of a comprehensive Drainage Design Manual. The manual would discuss methodology and set guidelines for all future drainage design and improvements. A proposed draft manual is presented at the conclusion of this report.

I. COMPUTER HARDWARE AND SOFTWARE ANALYSIS

It is also recommended that the city obtain an Intergraph Microstation work station to make use of the base maps prepared for the City. A plotter would also be useful to create work maps for the City or other concerned individuals.

J. POTENTIAL SOURCES OF FUNDING

FUNDING OPTIONS

Several options are available to the City of Orange to generate the funds needed to finance the capital improvements recommended in this study. Table VII.1 presents a summary of the entities that have funds available for certain flood control improvements. Other options for generating funds are discussed in the following paragraphs.

Storm Water Utility

The City of Orange should investigate the possibility of establishing a storm water utility to fund improvements related to storm water runoff. A storm water utility sets up a funding program to finance and maintain capital improvements for flood control, drainage, erosion control and water quality projects. A user fee system, similar to those for water and wastewater utilities, can provide funding for storm water management. The funding program establishes drainage fees for each parcel within the City. The drainage fee for each parcel of land within the City's jurisdiction is based on the site runoff characteristics.

The fees charged for each parcel are determined according to the parcel size and the percent of impervious surface contained on the parcel.

Development of a storm water utility requires working with the residents, developers, businesses, churches, schools and other entities within the community to ensure public acceptance and awareness of the proposed fee structure. The utility organization and fee structure is generally established through a city ordinance. The ordinance is the basis for the utility financing and operation. Two types of fees may be established in the ordinance which includes user fees and new development fees.

User fees are established in the ordinance based on the share of runoff for each property. Often, the average runoff potential is related to the average impervious area of a single family residence. The equivalent residential unit is used to set the fee with larger parcels with greater impervious areas paying higher fees.

Typical residential fees for cities in Texas that have established storm water utilities have ranged from one to four dollars per residence per month. Commercial parcel fees have ranged from two cents to five cents per 100 square feet of impervious cover on the parcel. Adjustments to the fees can be allowed for on-site detention or the use of other storm detention or water quality features.

Texas Water Development Board Funding

According to the Texas Water Development Board, the Board is allowed to provide loans from the Flood Control Account of the Texas Water Development Fund to Political subdivisions for both structural and nonstructural projects. To date it has not been Board policy to fund projects that provide less than 100-year frequency flood protection. Their enabling legislation (Texas Water Code chapter 17.771-17.776) and board rules (TAC 363.401-363.404) regarding loans from the Flood control Account require that basin-wide planning and demonstrations of significant reductions in water surface elevations accompany applications for funding. Consequently, it does not appear that recommended storm sewer and drainage system improvements would be eligible for funding, nor would maintenance activities on City area channels. Purchase of floodplain land by the City for use as public open space would, however, be eligible for funding. The larger, more expensive levee projects discussed, but not recommended, in this report appear to have potential as projects eligible for funding. Should the City decide to explore Board funding of these projects, it is suggested they discuss the projects with appropriate Board staff and obtain copies of the Board rules referenced above before preparing an engineering feasibility study to support an application.

TABLE VII.1 Potential Sources for Financial Assistance

ORGANIZATION OR AGENCY	ASSISTANCE OFFERED	CONTACT NAME AND PHONE
Texas Water Department	Water Development Fund Loans for Flood Control Projects; Loan Rate: Approx. 6%	Charlotte Brigham 512-463-7926
Texas Water Development Board	Water Assistance Fund Flood Control Planning Grants; Grant Amount: 50% up to $100,000	Charlotte Brigham 512-463-7926
Federal Emergency Management Agency	Section 1362 Buyout Program; Assistance Amount: Full Property Value Minus Insurance Claims	Jim Legrotte 817-898-5162
Housing and Urban Development	Community Development Block; Grant City Currently Receives $400,000 per year	Jerome Bassett City of Orange
Department of Agriculture Farmers Home Administration	Loan Assistance for rural Storm Drainage Projects; Loan Rate: As Low As 5%	Harold Carter 512-774-1301
U.S. Army Corps of Engineers	Section 205 Small Flood Control Project Assistance; Maximum Project Amount: $5 Million, 25 to 50% Local Sponsor Amount	Michael Kieslich 409-766-3059
U.S. Army Corps of Engineers	Section 14 Stream Bank Erosion; Project Assistance Maximum Project Grant: $500,000	Michael Kieslich 409-766-3059
U.S. Army Corps of Engineers	Section 208 Assistance For Channel Snagging and Clearing for Flood Control Projects	Michael Kieslich 409-766-3059

GLOSSARY

Conduit: Any closed device for conveying flowing water.

Control: The hydraulic characteristic which determined the stage-discharge relationship in a conduit.

Critical Flow: The state of flow for a given discharge at which the specific energy is a minimum with respect to the bottom of the conduit.

Entrance Loss: Head lost in eddies or friction at the inlet to a conduit, headwall or structure.

Flood: An overflow of lands not normally covered by water and that are used or usable by man. Floods have two essential characteristics: The inundation of land is temporary; and the land is adjacent to and inundated by overflow from a river or stream or an ocean, lake, or other body of standing water.

Normally, a "flood" is considered as any temporary rise in stream flow or stage, but not the ponding of surface water, that results in significant adverse effects in the vicinity. Adverse effects may include damages from overflow and land areas, temporary backwater effects in sewers and local drainage channels, creation of unsanitary conditions or other unfavorable situations by deposition of materials in stream channels during flood recessions, rise of ground water coincident with increased stream flow and other problems.

Flood Peak: The maximum instantaneous discharge of a flood at a given location. It usually occurs at or near the time of the flood crest.

Flood Plain: The relatively flat area or low lands adjoining the channel of a river, stream or watercourse or ocean, lake or other body of standing water, which has been or may be covered by flood water.

Flood Profile: A graph showing the relationship of water surface elevation to location, the latter generally expressed as distance above mouth for a stream of water flowing in an open channel. It is generally drawn to show surface elevation for the crest of a specific flood, but may be prepared for conditions at a given time or stage.

Flood Stage: The stage or elevation at which overflow of the natural banks of a stream or body of water begins in the reach or area in which the elevation is measured.

Freeboard: The distance between the normal operating level and the top of the side of an open channel left to allow for wave action, floating debris, or any other condition or emergency without overflowing structure.

Head Loss: The effect of obstructions, such as narrow bridge openings or building that limit the area through which water must flow, raising the surface of the water upstream from the obstruction.

Headwater: Depth of water in the stream channel measured from the invert of culvert.

HEC-1: Computer program to analyze a Flood Hydrograph. This program is available from the U.S. Army Corps of Engineers.

HEC-2: Computer Program to analyze a Water Surface Profile. This program is available from the U.S. Army Corps of Engineers.

Hydraulic Gradient: A line representing the friction head available at any given point within the system.

Intermediate Regional Flood
(100-YEAR FLOOD): A flood having an average frequency of occurrence in the order of once in 100 years although the flood may occur in any year. It is based on statistical analyses of stream flow records available for the watershed and analyses of rainfall and runoff characteristics in the "general region of the watershed".

Invert The flowline of pipe or box (inside bottom).

Left Bank The bank on the left side of a river, stream, or watercourse, looking downstream.

Low Steel (or Low Chord): The lowest point of a bridge or other structure over or across a river, stream, or watercourse that limits the opening through which water flows. This is referred to as "low chord" in some applications.

Manning's Equation: The uniform flow equation used to relate velocity, hydraulic radius and energy gradient slope.

Open Channel: A channel in which water flows with a free surface.

Rational Formula: The means of relating runoff with the area being drained and the intensity of the storm rainfall.

Right Bank The bank on the right side of a river, stream or watercourse, looking downstream.

Standard Project Flood: The flood that may be expected from the most severe combination of meteorological and hydrological conditions that is considered reasonably characteristic of the geographical area in which the drainage basin is located, excluding extremely rare combinations. Peak discharges for these floods are generally about 40% to 60% of the Probably Maximum Floods for the same basins. Such floods, as used by the Corps of Engineers, are intended as practicable expressions of the degree of protection that should be sought in the design of flood control works, the failure of which might be disastrous.

Soffit: The inside top of pipe or box.

Surcharge: Height of water surface above the crown of a closed conduit at the upstream end.

Tailwater: Total depth of flow in the downstream channel measured from the invert at the culvert outlet.

Time of Concentration: The estimated time in minutes required for runoff to flow from the most remote section of the drainage area to the point at which the flow is to be determined.

Total Head Line (Energy Line): A line representing the energy in flowing water. It is plotted a distance above the profiles of the flow line of the conduit equal to the normal depth plus the normal velocity head plus the friction head for conduits flowing under pressure.

Uniform Flow: A condition of flow in which the discharge, or quantity of water flowing per unit of time, and the velocity are constant. Flows will be at normal depth and can be computed by the Manning Equation.

Watershed: The area drained by a stream or drainage system.

REFERENCES

1. "Flood Insurance Study," City of Orange, Texas, Orange County, July 6, 1982; Federal Emergency Management Agency.

2. "Flood Plain Information Sabine River and Adams Bayou Orange, Texas Area," U.S. Army Corps of Engineers, Galveston District, July 1968.

3. "Brazoria County Drainage Criteria Manual Draft," Brazoria County, Texas, June 1990.

4. "Drainage Criteria Manual for Fort Bend County, Texas," Fort Bend County Drainage District, Texas, November 1987.

5. "Criteria Manual for the Design of Flood Control and Drainage Facilities in Harris County, Texas," Harris County Flood Control District, Houston, Texas, February, 1984.

6. "Report on a Comprehensive Drainage Plan for the City of Orange, Texas and Metropolitan Area," George J. Schaumburg Consulting Engineers, November 1958.

7. "HEC-1 Flood Hydrograph Package, Users Manual," U.S. Army Corps of Engineers, Davis, California, September 1981.

8. "HEC-2 Water Surface Profiles, Users Manual," U.S. Army Corps of Engineers, Davis, California, September 1982.

9. "Sabine River Flood Study," Brown & Root, INC., January 1993.

10. "Flood Insurance Study," Calcasieu Parish, Louisiana Unincorporated Areas, January 15, 1988; FEMA.

DRAFT

DRAINAGE DESIGN MANUAL FOR THE CITY OF ORANGE, TEXAS

Compiled by:

Carter & Burgess, Inc.
7950 Elmbrook Drive
Suite 250
Dallas, TX 75247

April 1994

ORDINANCE NO. ________________

AN ORDINANCE OF THE CITY OF ORANGE, TEXAS, ADOPTING THE STORM DRAINAGE DESIGN MANUAL OF 1994, PREPARED BY CARTER & BURGESS, INC., CONSULTING ENGINEERS, AS THE STORM DRAINAGE POLICY FOR THE CITY OF ORANGE, TO BE USED IN CONNECTION WITH DRAINAGE DESIGN FOR ALL CONSTRUCTION AND/OR DEVELOPMENT WITHIN THE CITY AND ALL SUBDIVISIONS WITHIN THE CITY AND ITS EXTRATERRITORIAL JURISDICTION; PROVIDING FOR REPEAL OF ALL PRIOR STORM DRAINAGE DESIGN CRITERIA IN CONFLICT WITH THIS MANUAL; PROVIDING A SEVERABILITY CLAUSE; AND PROVIDING AN EFFECTIVE DATE.

WHEREAS, upon review of the Storm Drainage Design Manual of 1994, prepared by Carter & Burgess, Inc., Consulting Engineers, the City Council finds that the provisions thereof are proper, and are further necessary in order to protect and promote the health, safety and general welfare of the City of Orange and its citizens, and the said Manual should be adopted and applied to all construction and/or development within the City of Orange and all subdivisions within the City and within the extraterritorial jurisdiction of the City of Orange; now, therefore,

BE IT ORDAINED BY THE CITY COUNCIL OF THE CITY OF ORANGE, TEXAS;

SECTION 1. That the City Council, having reviewed the provisions of the Storm Drainage Design Manual of 1994, prepared by Carter & Burgess, Inc., Consulting Engineers, attached hereto, finds the provisions of such Storm Drainage Design Manual to be proper and necessary in order to promote and protect the health, safety, and general welfare of the City of Orange and its citizens, and the same is hereby approved and adopted by the City Council. A current copy of said Storm Drainage Design Manual shall be kept on file in the office of the City Engineer and in the office of the City Secretary.

SECTION 2. That where a conflict exists between the said Storm Drainage Design Manual and the City of Orange Subdivision Ordinance now in force and effect and as may be hereafter amended, the said Storm Drainage Design Manual shall control.

SECTION 3. That said Storm Drainage Design Manual shall take precedence over and be controlling over all City ordinances, resolutions and policies which pertain to storm drainage design.

SECTION 4. That said Storm Drainage Design Manual shall apply to all subdivisions of land within the limits of the City of Orange and within its extraterritorial jurisdiction, and shall also apply to all developments and all proposed drainage improvements within the City and its extraterritorial jurisdiction.

SECTION 5. That all prior storm drainage criteria heretofore adopted by the City of Orange, including but not limited to the "Report on a Comprehensive Drainage Plan for the City of Orange, Texas and Metropolitan Area" prepared by George J. Schaumburg Consulting Engineers in November, 1958 are hereby repealed.

SECTION 6. That should any word, section, phrase, or portion of this ordinance be held to be void or invalid for any purpose, the remaining provisions of said ordinance shall continue in full force and effect and such invalidity shall not affect the validity of any other portion of said ordinance.

SECTION 7. This ordinance shall take effect immediately from and after its passage, as the law in such case provides.

DULY ADOPTED by the City Council of the City of Orange, Texas, on the ______ day of ____________, 1994.

APPROVED:

____
MAYOR
PRO TEM

DULY RECORDED:

____
CITY SECRETARY

APPROVED AS TO FORM:

____
CITY ATTORNEY

TABLE OF CONTENTS

SECTION 1

1.0 INTRODUCTION

1.1 PURPOSE

The purpose of this drainage manual is to establish standard principles and practices for the design and construction of drainage systems within the City of Orange. The design factors, formulas, graphs and procedures are intended for use as engineering guides in the solution of drainage problems involving determination of the quantity, rate of flow and conveyance of storm water.

Methods of design other than those indicated herein may be considered in difficult cases where experience clearly indicates they are preferable. However, there should be no extensive variations from the practices established herein without the express approval of the City of Orange.

1.2 SCOPE

The manual presents various applications of accepted principles of surface drainage engineering and is a working supplement to basic information obtainable from standard drainage handbooks and other publications on drainage. It is presented in a format that gives logical development of solutions to the problems of storm drainage.

The past procedures and practices that have been used to design drainage facilities in the City of Orange, along with numerous drainage criteria manuals for other areas were reviewed to determine the most appropriate techniques and criteria for drainage design for use in the City of Orange. This was especially true of Fort Bend County's Drainage Criteria Manual prepared for Fort Bend County Drainage District, which was used as the primary guide in selecting drainage criteria and in preparing this Criteria Manual for the City of Orange.

1.3 DRAINAGE POLICY

The basic objective of the City of Orange is to construct and maintain facilities intended to minimize the threat of flooding to all areas of the City and comply with the requirements of the National Flood Insurance Program. The ultimate goal is intended to be accomplished by the construction and maintenance of 100-year design drainage facilities and flood control measures to provide 100-year flood protection in all areas of the City of Orange. The 100-year design drainage facilities are defined as all public channels within dedicated rights-of-way approved and accepted by the City and all other public flood control structures and facilities dedicated to, approved and accepted by the City. Additionally, it is the City's intent to insure that adequate facilities are constructed to accommodate new development such that existing property will not be subjected to additional flooding and so as not to increase the limits of the flood plains as shown on the flood insurance rate maps for the City of Orange and other entities (County, Levee Improvement Districts, and Municipal Utility Districts).

It is not economically feasible to construct storm sewer facilities which are large enough to keep the street systems from becoming inundated during severe storm events. The topographic relief of the coastal prairie is too flat to allow for quick runoff during severe storm events. The net effect of the City's policies will be to insure that for new developments the ponding in the street systems will be of minimum depth and duration, and most importantly, that minimum new house slab elevations are set at least 12 inches above the maximum anticipated ponding levels. The intent of this policy is that there should be no street ponding for minor storm events, minor street ponding for larger events, and major ponding for the 100-year event storms but without water in structures. Every attempt will be made to design major thoroughfares so that they are passable during severe storm events.

To accomplish the goal of eliminating existing flooding conditions and to insure that future drainage problems do not develop, additional drainage improvement measures shall be taken. The measures considered appropriate by the City include further channel improvements to existing watercourses, pump stations and the construction of storm water detention facilities.

The City has included in this manual criteria covering the design of storm water systems to serve both existing and new developments. The City of Orange has quantified the needed improvements for existing development in most of the watersheds in the City of Orange and is responsible for the approval. Upon the completion of all new 100-year design drainage facilities, the City will accept, maintain, and operate said facilities for flood control purposes as an extension of the City's existing system if the facilities are constructed in accordance with plans approved by the City of Orange. However, those drainage facilities, including detention facilities, which are planned and accepted for maintenance by some other perpetual special purpose district (such as a

Levee Improvement District) will not be accepted by the City. The criteria in this manual is considered a minimum for the City of Orange. Approval from other applicable agencies may be required. Ultimate approval for any variance of the criteria contained in this manual must be given by the City of Orange.

SECTION 2

2.0 HYDROLOGY

2.1 GENERAL

The planning, design and construction of drainage facilities are based on the determination of one of more aspects of storm runoff.

Continuous long-term records of rainfall and resulting storm runoff in an area provide the best data source from which to base the design of storm drainage and flood control systems in that area. However, it is not possible to obtain such records in sufficient quantities for all locations requiring storm runoff computations. Therefore, the accepted practice is to relate storm runoff to rainfall, thereby providing a means of estimating the rates, timing and volume of runoff expected within local watersheds at various recurrence intervals.

It is generally accepted that urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization generally alters the hydrology of a watershed by improving its hydraulic efficiency, reducing its surface infiltration and reducing its storage capacity.

Because of its versatility and accuracy, the widely used computer program HEC-1 is recommended as the primary tool for modeling storm runoff hydrographs in the City of Orange.

For certain small drainage areas (generally less than 200 acres in size), the widely used Rational Method provides a useful means of determining peak discharges. If the engineer wishes to use an alternative design technique, it is recommended that the City of Orange Engineer be consulted prior to design.

2.2 RAINFALL-RUNOFF COMPUTATIONS USING HEC-1

A stream network model which simulates the runoff response of a river basin to rainfall over that basin can be developed utilizing the HEC-1 computer program by the appropriate combination of hydrography and routing computations. The following sections describe the elements required to develop a HEC-1 computer model.

2.2.1 Design Storm Rainfall

Design storm rainfall can be described in terms of frequency, duration, areal extent and distribution of intensity with time. A design storm's rainfall distribution in time is handled by the HEC-1 program by assuming a symmetrical, single-peaked design hyetograph (design storm). The engineer's choice for frequency and duration is dependent upon the physical characteristics, location and study objectives. In most cases, design will be based on a 24-hour duration storm event. The HEC-1 program has the capability to modify runoff hydrographs to account for progressively smaller design storm volumes as areal coverage increases. The HEC-1 users manual suggests how to model storm rainfall depth versus drainage area relationships, based on Figure 15 in the National Weather Service's Technical Paper No. 40 which presents a means of reducing point rainfall totals as drainage area size increases.

2.2.2 Design Storm Losses

Only a portion of the rainfall volume which falls on a watershed during a storm event actually ends up as stream runoff. The remainder is intercepted by infiltration, depression storage, evaporation and other mechanisms. The volume of rainfall which becomes runoff is termed the "excess" rainfall. The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed abstractions or losses.

2.2.3 Design Storm Runoff

2.2.3.1 General

Given the design storm excess rainfall, it is necessary to determine the storm runoff hydrograph at the point of interest utilizing the HEC-1 program. The Clark unit hydrograph for a drainage area is described by three parameters: TC, R and a time-area curve. TC represents the time of concentration and R is a storage coefficient for the area. The time-area curve defines the cumulative area of the watershed contributing runoff to the design point as a function of time.

A thorough statistical analysis of historical rainfall and runoff data taken from selected watersheds in the Fort Bend County vicinity was performed to correlate TC and R to drainage area physiographic characteristics. These characteristics include the length, slope and roughness of the basin's longest watercourse, the average basin slope and the effective imperviousness of the basin. From this analysis, the following equations were derived:

$$TC + R = 128\,\frac{(L/\sqrt{S})^{.57}\,(N)^{.8}}{(S_o)^{.11}\,(10)^{I}}$$

(2-1)

and

$$TC = (TC + R) \times 0.38\,(\log S_o)$$

(2-2)

$$R = (TC + R) - TC$$

(2-3)

where	TC	=	Clark's time of concentration
	R	=	Clark's storage coefficient
	L	=	length of the longest watercourse within the drainage area (miles)
	S	=	average slope along the area's longest watercourse (ft/mile)
	N	=	Manning's weighted roughness coefficient along the longest watercourse (see Step 4 of Section 2.2.4)
	S_o	=	average basin slope of land draining overland into the longest watercourse (ft/mile)
	I	=	effective impervious ratio

The effective impervious ratio (I) used in equation (2-1) is determined by:

$$I = CD \times 10^{-4}$$

(2-4)

where:	C	=	the average percent of impervious cover of the developed area (in percent)
	D	=	% of the subarea that is developed

Determination of TC and R is carried out by the solution of Equations 2-1, 2-2 and 2-3. These parameters may then be input into the HEC-1 program to model the runoff process. Input of the time-area curve is handled internally by HEC-1 unless the engineer specifies a particular time-area relationship. An example of the step-by-step procedure for the development of a design runoff hydrograph is presented in Section 9.

For a detailed discussion of unit hydrograph theory and application, the engineer is referred to the Handbook of Hydrology, by David R. Maidment, 1993.

2.2.4 Flood Routing

As a flood wave passes downstream through a channel or detention facility, its shape is altered due to the effects of storage. The procedure for determining how the shape of the flood hydrograph changes is termed flood routing. Flood routing can be used to determine the effects of this storage on a flood's runoff pattern (i.e., its hydrograph).

2.3 RAINFALL-RUNOFF COMPUTATIONS USING THE RATIONAL EQUATION

Most communities today use the Rational Equation to calculate the amount of stormwater runoff a particular area will generate. The equation is stated as follows: **Q = C I A** where **Q** represents the amount of runoff in CFS, **C** is the runoff coefficient or "C-factor" based on soil type and land use, **I** is the rainfall intensity in inches per hour, and **A** is the watershed area in acres. Each community determines its level of flood protection based on the values it adopts for both **C** and **I**. Area is, of course, a constant for each watershed.

2.3.1 The Runoff Coefficient

The runoff coefficient C, or C-factor, varies according to soil type and land use. The more impervious and densely developed an area, the higher the value. For example, undeveloped farm land may use C = 0.3 while a paved parking lot may use C = 0.9. The factor should always be chosen assuming a fully developed watershed. This reasoning helps to prevent future flooding due to increased development. Zoning ordinances can aid in choosing a fully developed C-factor.

C-factors are widely accepted and generally correspond well among adjacent communities. For example, the South Texas counties of Brazoria, Fort Bend, and Montgomery have all adopted the same C values. The values are listed in Table II.1.

The City of Orange currently does not have a written drainage design manual that states its values for **C**. However, the city has adopted and recommends the standards presented by the Schaumburg report of 1958. The values that Schaumburg assigned to **C** are as follows:

C = **0.6** for Commercial Areas

C = **0.4** for Residential Areas

These values correspond well with those listed for similar categories in Table II.1. The "Basin Slope < 1%" column applies to Orange because of its low sloped terrain.

TABLE II.1 Rational Method Runoff Coefficients for 5-10 Year Frequency Storms in Brazoria, Fort Bend, and Montgomery Counties, Texas

Description of Area	Basin Slope < 1%	Basin Slope 1-3.5%	Basin Slope 3.5-5%
Single Family Residential Districts			
Lots greater than 1/2 acre	0.30	0.35	0.40
Lots 1/4 - 1/2 acre	0.40	0.45	0.50
Lots less than 1/4 acre	0.50	0.55	0.60
Multi-Family Residential Districts	0.60	0.65	0.70
Apartment Dwelling Areas	0.75	0.80	0.85
Business Districts			
Downtown	0.85	0.87	0.90
Neighborhood	0.75	0.80	0.85
Industrial Districts			
Light	0.50	0.65	0.80
Heavy	0.60	0.75	0.90
Railroad Yard Areas	0.20	0.30	0.40
Cemeteries	0.10	0.18	0.25
Playgrounds	0.20	0.28	0.35
Streets			
Asphalt	0.80	0.80	0.80
Concrete	0.85	0.85	0.85
Concrete Drives and Walks	0.85	0.85	0.85
Roofs	0.85	0.85	0.85
Lawn Areas			
Sandy Soil	0.05	0.08	0.12
Clay Soil	0.15	0.18	0.22
Woodlands			

Sandy Soil	0.15	0.18	0.25
Clay Soil	0.18	0.20	0.30
Pasture			
Sandy Soil	0.25	0.35	0.40
Clay Soil	0.30	0.40	0.50
Cultivated			
Sandy Soil	0.30	0.55	0.70
Clay Soil	0.35	0.60	0.80

2.3.2 Rainfall Intensity (i)

Rainfall intensity (i) is the average rainfall rate in inches per hour which is considered for a particular basin or sub-basin and is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the critical time of concentration for all portions of the drainage area under consideration that contribute flow to the point of interest. The frequency of occurrence is a statistical variable which is established by design standards or chosen by the engineer as a design parameter.

The time of concentration used in the rational equation is the critical time of concentration for the point of interest. The critical time of concentration is the time associated with the peak runoff from all or part of the upstream drainage area to the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire drainage area is contributing, in which case, the time of concentration is the time for water to flow from the most remote point in the watershed to the point of interest. However, the runoff rate may reach a peak prior to the time the entire upstream drainage area is contributing. In this instance, only the portions of the drainage area able to contribute flow to the point of interest during the critical time of concentration should be used in determining the peak discharge. A trial and error procedure can be used to determine the critical time of concentration.

The time of concentration to any point in a storm drainage system is a combination of the "inlet time" and the "time to flow in the conduit".

The inlet time is the time for water to flow over the surface to the storm sewer level. Inlet time decreases as the slope and the imperviousness of the surface increases, and it increases as the distance over which the water has to travel increases and as retention by the contact surfaces increases. Average velocities for estimating travel time for overland flow can be calculated using Figure 2-6.

The inlet time shall be determined by direct computation using the following formula:

$$T = \frac{D_F}{60V} \tag{2-7}$$

where

T	=	overland flow time (minutes).
D_F	=	flow distance (feet).
V	=	average velocity of runoff flow (ft/sec).

If the overland flow time is calculated to be in excess of 20 minutes, the designer should verify that the time is reasonable considering the projected ultimate development of the area.

The time of flow in the conduit is the quotient of the length of the conduit and the velocity of flow as computed using the hydraulic characteristics of the conduit. The time of concentration within a conduit is usually less than the actual time for the flood crest to reach a given point by an amount equal to the time required to fill the conduit. The time required to fill the conduit is defined as the time of storage. The time of storage shall be neglected in the design of storm runoff conduits even though it may represent an appreciable percentage to the total time of concentration in some instances. This procedure will not substantially affect he precision of the calculations and will contribute to a conservative design.

2.3.3 <u>Drainage Area (A)</u>

The size and shape of the drainage area must be determined. The area may be determined through the use of topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. A drainage area map shall be provided for each project. The drainage area contributing to the system being designed and the drainage subarea contributing to each inlet point shall be identified. The outlines of the drainage divides must follow actual lines rather than the artificial land divisions as used in the design of sanitary sewers. The drainage divide lines are determined by the pavement slopes, locations of downspouts, paved and unpaved yards, grading of lawns and many other features that are introduced by the urbanization process.

As mentioned previously, the drainage area used in determining peak discharges is the portion of the area that contributes flow to the point of interest within the critical time of concentration.

SECTION 3

3.0 CRITERIA & DESIGN OF OPEN CHANNEL FLOW

3.1 GENERAL

In a major drainage system, open channels offer significant advantages over closed conduits in regard to cost, flow capacity, flood storage, recreation, and aesthetics. However, open channels require considerable right-of-way and maintenance. Careful consideration must be given in the design process to insure that disadvantages are minimized and the benefits maximized. When a design approach not covered in this manual is to be used, it should be reviewed and discussed with the City of Orange Engineer prior to commencing significant portions of the design effort.

3.2 OPEN CHANNEL HYDRAULICS

3.2.1 Manning's Equation

Manning's equation is an empirical equation which relates friction slope, flow depth, channel roughness, and channel cross-sectional shape to flow rate. The friction slope is a measure of the rate at which energy is being lost in the flow to channel resistance. When the channel slope and the friction slope are equal (S_f = S_o) the flow is uniform and Manning's equation may be used to determine the depth for uniform flow (normal depth).

Manning's equation is as follows:

$$9V = \frac{1.49}{n} R^{2/3} S_f^{1/2} \tag{3-2}$$

or

$$Q = \frac{1.49}{n} AR^2/3\ S_f^{1/2} \tag{3-3}$$

where Q = total discharge (cfs)

V = velocity of flow (ft/sec)

n = Manning's coefficient of roughness

A = cross-sectional area of the flow (ft^2)

R = hydraulic radius of the channel (ft) (flow area/wetted perimeter)

S_f= friction slope, the rate at which energy is lost due to channel resistance

3.2.1.1 **Manning's "n" Value**

Manning's "n" value is an experimentally derived constant which represents the effect of channel roughness in the Manning's equation. Considerable care must be given to the selection of an appropriate "n" value for a given channel due to its significant effect on the character of the flow. Table 5 provides a listing of "n" values for various channel conditions.

3.3 CHANNEL DESIGN

The proper hydraulic design of a channel is of primary importance to insure that nuisance drainage conditions, flooding, sedimentation and erosion problems do not occur. The following general criteria should be utilized in the design of open channels.

3.3.1 **Design Frequency**

All open channels in the City of Orange shall be designed to contain the runoff from the 100-year frequency 24-hour duration storm within the right-of-way while providing one foot of freeboard. In those cases where channel modifications are necessary to control increased flows from proposed development, proposed water surface profiles are restricted such that the 100-year flood profile under existing conditions shall not be increased. In addition, the channel must be designed to have sufficient freeboard to provide for adequate drainage of lateral storm sewers during the 25-year storm. If the capacity of the existing channel downstream of the project is less than the 100-year design discharge, consideration shall be given for more frequent events to ensure that the frequency of downstream flooding is not increased.

3.3.2 **Required Analyses**

The following information must be submitted to the City Engineer for the design of open channels.

(1) A vicinity map of the site and subject reach. The subject reach is defined as the stretch of channel necessary for any altered flow profile to match the upstream and downstream existing profiles.

(2) A detailed map of the area and subject reach with all pertinent physiographic information.

(3) A watershed map showing the existing and proposed drainage area boundary along with all subarea delineations and all areas of existing or proposed development.

(4) Discharge calculations specifying methodology and key assumptions used including discharges at key locations.

(5) Hydraulic calculations specifying methodology and key assumptions used including discharges at key locations.

(6) A profile of the subject reach which includes the following:

 (a) All pertinent water surface profiles. This will minimally include the 25- and 100-year frequency floods for both existing and proposed channel conditions.

 (b) All existing and proposed bridge, culvert and pipeline crossings.

 (c) The location of all tributary and drainage confluences.

 (d) The location of all hydraulic structures (e.g. dams, weirs, drop structures, etc.).

(7) A map delineating existing and proposed rights-of-way.

(8) Benchmark, elevation, datum and year of adjustment.

(9) Typical existing and proposed cross-sections.

(10) A soils report which addresses erosion and slope stability.

3.3.3 <u>Design Considerations</u>

The path taken by an existing, naturally-carved channel often represents the most logical general pathway of flow. For runoff rates associated with undeveloped conditions, the natural channel is largely stable against erosion and is topographically efficient in draining adjacent land. In light of this, it is logical that the engineer should consider taking advantage of naturally carved drainageways when locating and designing open channels.

Although there are numerous channel designs available to the engineer, a judicious design must conform to certain hydraulic, aesthetic, and safety-related standards. In situations where the use of a natural drainage course is infeasible, the engineer must choose between an earthen channel or a lined channel. Grassed channels generally produce lower flow velocities and greater channel storage. They are, in most cases, aesthetically and economically superior to concrete or riprap-lined waterways. However, grass-lined channels require more right-of-way, are vulnerable to erosion, and must be continually maintained. They can also have problems with side slope stability and/or sediment deposition.

In areas where land values are extremely high, or right-of-way is limited, concrete or riprap-lined channels may be the design of choice. However concrete channels can be significantly more expensive. In addition, they tend to move water faster and store less water possibly resulting in higher peak discharges downstream.

3.3.3.1 Optimal Channel Configuration Characteristics

Side Slope - In grass-lined channels, normal maximum slope is 3 (horizontal): 1 (vertical), which is also the practical limit for mowing equipment. In some areas, sideslopes flatter than 3:1 may be necessary due to local soil conditions.

Bottom Width - In grass-lined channels the minimum channel bottom width should be six feet. In concrete-lined channels the minimum bottom width should be eight feet.

Curvature - In general, centerline curves should be as gradual as possible and not have a radius of less than three times the design flow top width unless erosion protection is provided, and not less than 100 feet. The maximum curvature for any man-made channel should be 90°.

Manning's "n" Value - Table 5 provides Manning's roughness coefficient to be used in man-made channels. Alternative values should be discussed with the City Engineer.

3.3.4 Minimum Requirements for Channel Design

3.3.4.1 Grass-Lined Channels

The following are minimum requirements to be used in the design of all grass-lined channels:

(1) Maximum side slopes shall be 3:1. Slopes flatter than 3:1 may be necessary in some areas due to local soil conditions.

(2) Minimum bottom width is six (6) feet.

(3) A minimum maintenance berm is required on both sides of the channel of between 15 and 30 feet depending upon channel size.

(4) Backslope interceptor structures are necessary at a maximum of 800 foot intervals to prevent sheet flow over the ditch side slopes.

(5) Channel slopes must be revegetated immediately after construction to minimize erosion.

(6) Flow from roadside ditches must be conveyed to the channel through a roadside ditch interceptor structure and pipe.

(7) Unless waived by the City of Orange Engineer, a geotechnical investigation and report must be provided.

3.3.4.2 **Concrete-Lined Trapezoidal Channels**

All partially or fully concrete-lined trapezoidal channels must meet or exceed the following minimum design requirements:

(1) All concrete shall be Class A concrete unless noted otherwise.

(2) Fully lined cross-sections shall have a minimum bottom width of eight (8) feet.

(3) Concrete slope protection placed on 3:1 side slopes shall have a minimum thickness of 4 inches and minimum 6 x 6 x W2.9 x W2.9 welded wire fabric or equivalent reinforcing.

(4) Concrete slope protection placed on 2:1 side slopes shall have a minimum thickness of 4-inches and minimum 6 x 6 x W4.0 x W4.0 welded wire fabric or equivalent reinforcing.

(5) Concrete slope protection placed on 1.5:1 slopes should have a minimum thickness of 5-inches and minimum 4 x 4 x W.4.0 x W4.0 reinforcement or equivalent. Cast-in-place concrete sideslopes should not be steeper than 1.5:1.

(6) All slope paving shall include a minimum 18-inch toe wall at the top and sides and a 24-inch toe wall across or along the channel bottom for clay soils. In sandy soils, a 36-inch toe wall is recommended across the channel bottom.

(7) In instances where the channel is fully lined, backslope drainage structures may not be required. Partially lined channels will require backslope drainage structures.

(8) Weep holes shall be used to relieve hydrostatic head behind lined channel sections. The specific type, spacing and construction

method for the weep holes will be based on the recommendations of the geotechnical report.

(9) Where construction is to take place under conditions of mud and/or standing water, a seal slab of Class C concrete shall be placed in channel bottom prior to placement of concrete slope paving.

(10) Control joints shall be provided at approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.3.4.3 **Rectangular Concrete Pilot Channels**

In areas where it is necessary to use a vertical-walled rectangular section the following minimum requirements are to be addressed:

(1) All concrete shall be Class A concrete unless noted otherwise.

(2) The structural steel design should be based on ASTM A 615, Grade 60 steel.

(3) Minimum bottom width shall be eight (8) feet.

(4) For bottom widths twelve (12) feet or greater, the channel bottom shall be graded at 1% toward the channel center line. (Differs from Harris County criteria.)

(5) Minimum height of vertical walls shall be (4) feet. Heights above this shall be in two (2) foot increments. Exceptions shall be on a case by case basis.

(6) Escape stairways shall be located at the upstream side of all street crossings, but not to exceed 1,400 feet intervals.

(7) For rectangular concrete pilot channels with grass side slopes, the top of the vertical wall should be constructed to allow for future placement of concrete slope paving.

(8) Weep holes should be used to relieve hydrostatic pressures. The specific type, spacing and construction method for the weep holes will be based on the recommendations of the geotechnical report.

(9) Where construction is to take place under conditions of mud and/or standing water a seal slab of Class C concrete should be placed in channel bottom prior to placement of concrete slope paving.

(10) Concrete pilot channels may be used in combination with slope paving or a maintenance shelf. Horizontal paving sections should be analyzed as one way paving capable of supporting maintenance equipment having a concentrated wheel load of up to 1,350 lbs.

(11) Control joints shall be provided at approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.4 EROSION

Erosion protection is necessary to insure that channels maintain their capacity and stability and to avoid excessive transport and deposition of eroded material. The three main parameters which affect erosion are vegetation, soil type and the magnitude of flow velocities and turbulence. In general, silty and sandy soils are the most vulnerable to erosion.

The necessity for erosion protection should be anticipated in the following settings:

(1) Areas of channel curvature, especially where the radius of the curve is less than three times the design flow top width.

(2) Around bridges where channel transitions create increased flow velocities.

(3) When the channel invert is steep enough to cause excessive flow velocities.

(4) Along grassed channel side slopes where significant sheet flow enters the channel laterally.

(5) At channel confluences.

(6) In areas where the soil is particularly prone to erosion.

Sound engineering judgement and experience should be used in locating areas requiring erosion protection. It is often prudent to analyze potential erosion sites following a significant flow event to pinpoint areas of concern.

3.4.1 Minimum Erosion Protection Requirements

Minimum Erosion Protection Requirements for the City of Orange are as follows:

Confluences - A healthy cover of grass must also be established above the top edge of the lining extending to the top of the bank. The top edge of the lining shall extend to the 25-year water surface elevation.

Bends - When required, erosion protection must extend along the outside bank of the

3.5 WATER SURFACE PROFILES

The state of flow in a channel is at all times either uniform, gradually varied, or rapidly varied. A different method for determining water surface profiles is applicable to each of these conditions of flow.

3.5.1 Uniform Flow

When a section of channel is sufficiently long and unchanging such that the flow depth is not changing (i.e. the force of gravity and channel resistance can be considered balanced), then the flow profile can be analyzed assuming uniform flow. Under the circumstances, the depth which is constant, can be determined with Manning's equation (see Section 3.2.6).

3.5.2 Gradually Varied Flow

In the majority of channel flow situations, the state of flow is gradually varied. In other words, the depth is gradually changing with longitudinal distance along the channel due to an imbalance between the forces of gravity and channel resistance.

The recommended means for determining flow profiles under these conditions is with the standard step method. The standard step method is an iterative process in which the one-dimensional energy equation is solved to find the water surface elevation at a cross-section. Manning's equation is utilized to determine channel losses due to friction. Losses due to channel non-uniformities are usually calculated with empirical coefficients.

A widely accepted computer model for calculating gradually varied flow profiles in the U.S. Army Corps of Engineers' program HEC-2, Water Surface Profiles. The HEC-2 model can readily accommodate modifications in channel design and losses at bridges, culverts, drop structures and transitions. The program begins computation at a cross section of known or estimated water surface elevation and proceeds upstream for subcritical flow, and downstream for supercritical flow.

The following general guidelines should be followed with the use of the HEC-2 computer program:

(1) Cross-sections should be placed such that the channel configuration between them is largely uniform. In areas where channel properties are rapidly changing, the distance between cross-sections should be appropriately less.

(2) The accuracy of the flow profile is largely dependent on a correct determination of the starting water surface elevation, especially in the vicinity of the first cross-section. The best method of determining starting water surface elevation is with a known rating curve or from past backwater studies. The least favorable is the slope-area method which determines normal depth given the friction slope and discharge. It is important to begin water surface profile analyses a significant distance downstream of the point(s) of interest for subcritical flow and upstream of the point(s) of interest for supercritical flow.

(3) Errors can occur with the improper handling of energy losses, thus loss coefficients should be chosen carefully. The engineer should carefully select a particular bridge routine and understand its operation. If the independent hand calculation of a head loss can be accomplished more accurately, it should be input to the program. Proper care should be taken to ascertain that computed losses are reasonable.

3.5.3 Rapidly Varied Flow

When depth changes abruptly over a short distance the flow profile is rapidly varied. Rapidly varied flow is a local phenomenon which occurs in such areas as the contraction beneath a sluice gate, where the channel slope changes from mild to steep, where the flow passes over a weir, and in a hydraulic jump. Determination of the change of the flow profile at such locations must be carried out on a site specific basis by the engineer.

3.5.4 Energy Losses

Analysis of flow profiles in open channels must include proper consideration of energy losses due to local disturbances such as bridges, drop structures, transitions and confluences. In many cases, such head losses are adequately handled with empirical coefficients. When specific site conditions warrant a more careful analysis, or when a particular program cannot handle local losses, hand calculated losses may be utilized in the flow profile. The following guidelines should be followed for typical sources of nonfrictional energy losses.

3.5.4.1 Expansions and Contractions

Losses at transitions are generally expressed in terms of the absolute change in velocity head between downstream and upstream of the transition. The head loss is given by:

$$h_1 = C\frac{(V_2^2 - V_1^2)}{2_g}$$

(3-4)

where

h_1	=	head loss across the transition (ft)
C	=	empirical expansion or contraction coefficient
V_2, V_1	=	average channel velocity (fps) of the downstream and upstream section, respectively
g	=	acceleration of gravity (32.2 ft.sec^2)

Typical transition loss coefficients for subcritical flow are as follows:

Transition	Coefficient Contraction	Coefficient Expansion
Gradual or warped	0.1	0.3
Bridge Sections, wedge, straight-lined	0.3	0.5
Abrupt or square-edged	0.6	0.8

Source: HEC-2 User's Manual.

The above transition loss coefficients are also adequate for general design and supercritical flow; however, the effects of standing waves and other considerations make exact determination of losses in supercritical flow difficult. Therefore, with important transitions, a more detailed analysis may be necessary (see Section 3.6).

3.5.4.2 <u>Bends</u>

The HEC-2 program does not make allowances for energy losses due to significant bends in the channel. In most cases, losses in channel bends are negligible. However, when the radius of a bend is less than three times the design top width of flow, energy losses due to the bend should be included in the backwater analysis. Such losses are expressed in terms of the velocity head multiplied by a loss coefficient and may be input to a computer run.

3.5.4.3 Bridges

There are numerous methods available to compute losses associated with flow through a bridge. Sources of energy loss in bridges include flow resistance, channel transitions and direct obstructions to the flow such as piers. Each bridge should be examined individually to determine the best approach. The bridge routines found in HEC-2 are recommended for their versatility and flexibility. Brief descriptions of what they do and when they should be used are as follows:

Normal Bridge Method - The normal bridge method computes the water surface profile through the bridge in the same manner as in a natural river section except that the flow area and wetted perimeter are modified. The normal bridge method should be used when friction losses are the predominate consideration. This includes long culverts under low flow conditions and in cases where the bridge and abutments are small obstructions to the flow. Because the special bridge method requires a trapezoidal approximation of the bridge opening for low flow solution, the normal bridge method can be used when the flow area cannot be reasonably approximated by a trapezoid. Also, when deeply submerged weir flow exists over a bridge, the normal bridge method is preferred.

Special Bridge Method - The special bridge method is capable of solving flow problems where losses are due primarily to factors other than friction. It uses different hydraulic formulas to compute losses depending on the existence of low flow, pressure flow, weir flow, or some combination of these at the bridge. Special care must be taken to ensure that the special bridge method is used properly and its results are reasonable. Whenever flow crosses critical depth in a structure, the special bridge method should be used.

The use of alternative means for computing bridge-related losses is encouraged when then engineer is properly aware of how and why such a strategy is appropriate and its results are reasonable.

SECTION 4

4.0 CRITERIA AND DESIGN OF CULVERTS AND BRIDGES

4.1 GENERAL

For small drainage areas the most economical means of moving open channel flow beneath a road or railroad is generally with culverts. Discussion in this section will address procedures for determining the most cost effective culvert size and shape given a design discharge and allowable headwater elevation. The design procedures for the culverts referenced in this section pertain only to those in the main channels and not those in roadside ditches which are covered in Section 5.0 Storm Sewers and Overland Flow. In addition, this section will include a brief discussion of the hydraulic and hydrologic considerations pertinent to bridge design. This section considers all design to be completed for ultimate development. Where appropriate, the actual construction of a crossing may be phased as development occurs. In this case, both the ultimate and the interim phase must be shown on the construction plans. Calculations for each must be submitted for approval. The ultimate right-of-way is required even for an interim phase of construction.

4.2 CULVERTS

4.2.1 Design Frequency

All culverts in the City of Orange shall be designed to handle the 100-year flood flow for fully developed conditions without causing upstream or downstream water surface profiles to exceed maximum levels as defined in Section 3.3.1.

4.2.2 Culvert Alignment

Culverts shall be aligned parallel to the longitudinal axis of the channel to insure maximum hydraulic efficiency and minimum erosion. In areas where a change in alignment is necessary, the turn shall be made upstream in the natural channel and appropriate erosion protection shall be provided.

4.2.3 Headwalls

Headwalls and endwalls shall be utilized to control erosion and scour, to anchor the culvert against lateral pressures, and to insure bank stability. All headwalls shall be constructed of reinforced concrete and may be either straight and parallel to the channel, flared, or warped, with or without aprons, as required by site and hydraulic conditions. Protective guardrails should be included along culvert headwalls.

4.2.4 Manning's "n" Values

The minimum Mannings "n" value to be used in concrete culverts shall be 0.013. For corrugated metal, the "n" value shall be as follows:

Corrugation (Span x Depth)	"n"
2-2/3" x 1/2"	0.024
3" x 1"	0.027
5" x 1"	0.027
6" x 2"	0.030

4.3 CULVERT HYDRAULIC DESIGN

The fundamental objective of hydraulic design of culverts is to determine the most economical diameter at which the design discharge is passed without exceeding the allowable headwater elevation or causing erosion problems. However, there are numerous hydraulic considerations in culvert design which can render the decision-making process somewhat complex.

4.3.1 Culvert Design Procedure

In the hydraulic design of culverts an investigation shall be made of four different operating conditions, all as shown on FORM "3". It is not necessary that the Engineer know prior to the actual calculations which condition of operation (Case I, II, III or IV) exists. The calculations will make this known.

Case I operation is a condition where the capacity of the culvert is controlled at the inlet with the upstream water level at or below the top of the culvert.

Case II operation is also a condition where the capacity of the culvert is controlled at the inlet with the upstream water level above the top of the culvert with the downstream water level below the top of the culvert.

Case III operation is a condition where the capacity of the culvert is controlled at the outlet with the upstream and downstream water levels above the top of the culvert.

Case IV operation is a condition where the capacity of the culvert is controlled at the outlet with the upstream water level above the top of the culvert and the downstream water level equal to one of two levels to be calculated.

4.3.2 <u>Headwater Depth</u>

In all culvert design, headwater, or depth of ponding at the entrance to the culvert, is an important factor in culvert capacity. The headwater depth (HW) is the vertical distance from the culvert entrance invert to the energy line of the approaching flow. Due to low velocities in most entrance pools and the difficulty in determining velocity head in any flow, the energy line can often be assumed coincident with the water surface.

4.3.3 <u>Tailwater Depth</u>

For culverts under outlet control, tailwater depth is an important factor in computing both headwater depth and the hydraulic capacity of the culvert. If flow in the channel downstream of the culvert is subcritical, a computer-aided backwater analysis or calculation of normal depth is warranted to determine the tailwater elevation. If the downstream flow is supercritical, tailwater is inconsequential to the culvert's hydraulic capacity.

4.3.4 <u>Conditions at Entrance</u>

Culvert performance is significantly affected by inlet efficiency, especially for conditions of inlet-controlled flow. Changes in the culvert edge geometry can significantly change discharge capacity. Selection of a particular inlet type is contingent on the relative weightings the engineer assigns to considerations of the effect on peak flows, cost, and topography. In other words, the ideal inlet geometry is not necessarily the most efficient.

The entrance head losses may be determined by the following equation:

$$h_e = K_e \left(\frac{V_2^2 - V_1^2}{2_g} \right)$$

(4-6)

where h_e = entrance head loss (ft)

V_2 = velocity of flow in culvert (fps)

V_1 = velocity of flow approaching culvert (fps)

K_e = entrance loss coefficient.

For calculation of headwater with inlet-controlled culverts, the design nomograph presented in this manual account for various typical kinds of inlet geometry.

4.3.5 Step-By-Step Design Procedure

It is possible by involved hydraulic computations to determine the probable type of flow under which a culvert will operate for a given set of conditions. However, such computations can be avoided by determining the headwater necessary for a given discharge under both inlet and outlet flow conditions. The larger of the two will define the type of control and the corresponding headwater depth. Culvert design forms and instructions are provided in Section 9.

4.4 BRIDGES

4.4.1 Bridge Design Considerations

At a minimum, bridges must be designed to pass the fully developed 100-year design flow without causing backwater problems, structural damage, or erosion.

The low chord of all bridges must be located at least one foot above the 100-year flood elevation, or at or above the level of natural ground, whichever is higher.

Newly constructed bridges must be designed to completely span the existing or proposed channel such that the channel will pass under the bridge without modification. Energy losses due to flow transitions shall be minimized. In addition, provision must be made for future channel enlargements should they become necessary.

4.4.1.1 Bents and Abutments

Bents and abutments must be aligned parallel to the longitudinal axis of the channel so as to minimize obstruction of the flow. Bents shall be placed as far away from the channel centerline as possible and if possible should be eliminated entirely from the channel bottom.

4.4.1.2 Interim Channels

Bridges and bents constructed on existing or interim channels shall be designed to accommodate the ultimate channel section with a minimum of structural modification.

4.4.1.3 Erosion Protection

Increased turbulence and velocities associated with flow in the vicinity of bridges requires the use of erosion protection in affected areas.

4.5 HEC-2

The HEC-2 program is capable of determining flow profiles and energy losses through both bridges and culverts. However, it should be used carefully and with due respect for the assumptions and limitations of the bridge/culvert routines.

SECTION 5

5.0 <u>CRITERIA AND DESIGN OF STORM SEWERS AND OVERLAND FLOW</u>

5.1 GENERAL

The discussion presented in this section will be directed primarily at curb-and-gutter streets with underground storm sewers. Roadside ditch systems are acceptable in certain instances, but are not preferred.

Figure 2 illustrates the effect on the hydraulic grade line of a storm sewer for three outlet conditions. Assuming the outlet channel is at its 25-year water level, it can be seen from Part A of Figure 2 that the hydraulic grade line for the standard design condition remains at or below the gutter level at the furthest inlet. For this condition, there is no street ponding and the storm sewers are functioning at or below their design capacity.

Parts B and C of the Figure show the case where the tailwater condition is above the design level. Street ponding begins to occur throughout the storm sewer drainage system, as the storm sewers are unable to operate at their design capacity. This local flooding situation could also occur when the tailwater is below design conditions if local rainfall is in excess of that used in the design of the storm sewer system. As this widespread street ponding starts to occur, provisions must be made to limit the depth of ponding to a level below that which will cause significant property damage. In general, flood elevations shall be considered unacceptable when they exceed the lowest of the following: 1) one foot over natural ground; 2) one foot over top or curb; or 3) one foot below the lowest slab elevation.

5.1.1 <u>General Design Guidelines</u>

Storm sewers shall be designed to carry the design storm peak flow. A detailed description of these techniques is contained in Section 2.0 of this manual.

For all storm sewer systems or for enclosing an existing open channel, the hydraulic calculations and hydraulic profiles along with the construction plans of the closed-conduit system must be submitted to the City for review.

5.2.3 Head Losses

5.2.3.1 Head Losses at Structures

The equation for the head loss (feet) at an inlet or manhole is as follows:

$$head\ loss = \frac{(V_2^2 - KV_1^2)}{2g} \tag{5-1}$$

where

V_1 = velocity in the upstream pipe (fps).

V_2 = velocity in the downstream pipe (fps).

K = junction or structure coefficient of loss.

5.2.3.2 Entrance Losses

A special case of sudden contraction is the entrance loss for pipes. The equation for head loss at the entrance to a pipe is given as follows:

$$head\ loss = K\frac{V^2}{2g} \tag{5-2}$$

where

K = entrance loss coefficient.

V = flow velocity in pipe (fps).

5.2.4 Manholes

Manholes shall be placed at the location of all pipe size of cross section changes, pipe sewer intersections or P.I.'s, pipe sewer grade changes, street intersections, at maximum intervals of 500 feet measured along the centerline of the pipe sewer; and at all inlet lead intersections with the pipe sewer where precast concrete pipe sewers are designed.

5.2.5 Inlets

5.2.5.1 Inlet Spacing

Curb inlets must be spaced to handle the design storm discharge so that the hydraulic gradient does not exceed the roadway gutter elevation. Inlets shall be spaced so that the maximum travel distance of water in the gutter will not exceed six-hundred feet (600′) one way for residential streets and three-hundred feet (300′) one way on major thoroughfares and streets within commercial developments. Curb inlets shall be located on intersecting side streets to major thoroughfares for all original designs or developments. Special conditions warranting other locations of inlets shall be determined on a case-by-case basis.

5.2 STORM SEWERS

5.2.1 <u>Design Criteria</u>

The following specific criteria and requirements shall apply to the design and construction of storm sewers in the City of Orange.

(1) Calculation of the hydraulic grade line for design conditions in a specific branch of storm sewer shall proceed upstream from the level of the 5-year water surface elevation in the outfall channel.

(2) The minimum diameter of a pipe in a sewer line shall be 24".

(3) The Manning's "n" value to be used in a reinforced concrete pipe storm sewer shall be 0.013.

(4) The minimum velocity of flow to be allowed in a section of storm sewer flowing full shall be 3 fps. The maximum velocity shall be 10 fps.

(5) Provisions must be made for all adjacent undeveloped areas with natural drainage patterns directing overland flow into the across planned development.

(6) Before a particular storm sewer design will be reviewed, the following items must be presented:

   (a) A contour and drainage area map showing all pertinent subareas, including contributing off-site areas.

   (b) A listing of all relevant hydrologic design flow calculations, which shall include all contributing off-site flows.

   (c) Calculations for determining the hydraulic gradient, along with a profile of its location.

   (d) A plan showing the placement of storm sewers and the location of all pipe size changes, grade changes and pipe intersections.

(7) All storm sewers shall be constructed with reinforced concrete pipe or approved equal. Corrugated galvanized metal pipe, or other approved equal, may be used only at the storm sewer outfall into unlined channels. All cast-in-place concrete storm sewers shall follow the alignment of the right-of-way or easement.

(8) All storm sewer inlet leads shall be designed in a straight line alignment.

(9) Storm sewers shall be located in public street rights-of-way or in easements

that will not prohibit future maintenance access.

(10) In most cases where easements are restricted to storm sewers, the pipe should be centered within the limits of the easement.

(11) For all storm sewers having a cross-sectional area equivalent to a forty-two inch (42") inside diameter pipe or larger, soil borings with logs shall be made along the alignment of the storm sewer at intervals not to exceed five-hundred feet (500') and to a depth not less than three feet (3') below the flowline of the sewer. The required bedding of the storm sewer as determined from these soil borings shall be shown in the profile of each respective storm sewer. The design engineer shall inspect the open trench and may authorize changes in the bedding indicated on the plans. Such changes shall be shown on the record drawings and, along with soil boring logs, submitted to the County Drainage District Office. All bedding shall be constructed as specified in the City of Houston Department of Public Works publication, Specifications for Sewer Construction, Form E-14-62 and all subsequent revisions, or approved equal.

(12) All storm sewer outfalls shall conform with the requirements and specifications defined in Section 3.0, Open Channel Flow.

5.2.2 General Design Methodology

It is recommended that design of a storm sewer system proceed as follows:

(1) Determine the 5-year water surface elevation in the channel at the storm sewer outfall using appropriate backwater calculations.

(2) Determine the design flow rates for all sections of storm sewer based on drainage area size.

(3) Assuming storm sewer pipes are full at design flows, determine the appropriate sizes for all sections of storm sewer using Manning's equation and assuming uniform flow conditions.

(4) Begin calculation at the 5-year water surface elevation in the outfall channel and plot the hydraulic gradient for the design storm. Include all relevant energy losses. The hydraulic gradient must not exceed the roadway gutter flowline elevation.

SECTION 6

6.0 CRITERIA AND DESIGN OF LEVIED AREAS

Flood plains cover a significant area within Orange County, Texas. This area may be developed to the limits of the floodway if a levee system is constructed to protect the area from high water levels on the adjacent watercourse (usually the Sabine River). The components of the levee system shall include an internal drainage system, a levee, a pump station or adequate storage capacity and a gravity outlet with an outfall channel to the river. The City of Orange design criteria for each component are defined in the following sections. The City's minimum design standards shall be governed by the rules and regulations as established by the Federal Emergency Agency (FEMA) including any updates as they occur. The engineer is advised to check the current FEMA rules and regulations. Maintenance of these facilities generally will not be the responsibility of the City of Orange.

6.1 INTERNAL DRAINAGE SYSTEM

The internal drainage system for the levied area shall included the network of channels, lakes, and storm sewers which drain the levied area to the outfall structure. Refer to Section 3.0 Open Channel Flow and Section 5.0 Storm Sewers and Overland Flow for City of Orange construction requirements and design criteria.

6.2 LEVEE SYSTEM

6.2.1 Frequency Criteria

The levee system shall include a levee embankment that will protect the development from the 100-year frequency flood event on the adjacent watercourse. Projection from the 100-year frequency event shall include protection from the 100-year water surface elevation on the watercourse, as well as protection from any associated wind and wave action.

6.2.2 Design Criteria

The following specific criteria and requirements shall apply to the design and construction of a levee in the City of Orange, Texas:

(1) A geotechnical investigation shall be required on the levee foundation (the existing natural ground). Soil borings shall be required with a maximum

spacing of 1,000 feet and a minimum depth equal twice the height of the levee embankment.

(2) The foundation area shall be stripped for the full width of the levee. Stripping shall include removal of all grass, trees, and surface root systems.

(3) Embankment material shall be CH or CL as classified under the Unified Soil Classification System and shall have the following properties:

(a) Liquid Limit greater than or equal to 30.

(b) Plasticity Index greater than or equal to 15.

(c) Percent Passing No. 200 Sieve greater than or equal to 50.

A geotechnical investigation shall be required on the embankment material to determine the levee side slopes and methods employed to control subsurface seepage.

The embankment material shall be compacted to a minimum density of 95 percent using the standard proctor compaction test at approximately plus or minus three percent optimum moisture content. The embankment material shall be placed in lifts of not more than 12 inches thick.

(4) The levee top and side slopes shall be adequately protected by grass cover or other suitable material.

(5) The minimum levee top width shall be ten feet.

(6) The levee slope shall be one vertical to a minimum of three horizontal.

(7) The minimum top of levee elevation shall be the 100-year water surface elevation on the adjacent watercourse plus three feet of freeboard.

(8) The levee shall be continuous and shall either completely encompass the development or tie into natural ground located outside of the limits of the adjacent watercourse's 100-year flood plain.

(9) All pipes and conduits passing through the levee shall have anti-seep collars, flap gates and slope protection.

(10) The minimum right-of-way for the levee shall be from toe to toe. In addition, the establishment of an easement for maintenance and access, which may be located within the right-of-way, shall be required. Access shall be provided with either a minimum 10-foot easement adjacent to the levee, a minimum 10-foot levee top width or a minimum 10-foot horizontal

berm on either side of the levee. A minimum 20-foot wide easement should be established in at least two locations to provide access to the levee right-of-way from a nearby public road.

6.3 PUMP STATION

6.3.1 Frequency Criteria

To prevent flooding within levied areas, pumps are recommended (instead of only storage) to remove interior drainage when the exterior river stage reaches a level that prevents gravity outflow. In order to determine the required pump capacity so that the maximum ponding level within the levied area will not be exceeded on the average more than about once in 100 years, the following design criteria have been developed.

The two sets of criteria provided below differ depending on whether the storm that occurs over the levied area during high exterior river stages is an independent or dependent event as compared to the storm that produced the high river stages. If the two events are independent of each other, then a coincidental probability relationship exists and the first set of criteria (Section 6.3.2) should be utilized. Since high exterior flood stages requiring the pumping of interior drainage can exist independent of rainfall occurring over the levied area (e.g. high water levels on the Sabine River versus rainfall in the City of Orange) the probability of these two independent severe storm events occurring at the same time is much smaller than their individual probabilities. As a result, the design rainfall used in determining the required pumping capacity can be reduced below the design 100-year frequency rainfall by an amount related to the frequency that flood stages in the receiving watercourse impede gravity overflow. If the two events are dependent (i.e., they result from the same storm event), the second set of criteria (Section 6.3.2.1) based on the design 100-year frequency rainfall should be utilized.

6.3.2 Design Criteria Assuming Coincidental Events

This criteria presumes that the storm event causing a high flood stage outside of the levied area is independent of the storm event occurring over the levied area (e.g. a levied area draining into the Sabine River in Orange County). The following steps should be taken for determining the required pumping capacity:

(1) Select the maximum ponding level within the levied area that should not be exceeded more than once in 100 years on the average. Normally, this level will be equal to the maximum water surface elevations associated with the 100-year flood event computed in designing the internal drainage system (channels) of the levied area, including the required minimum freeboard of one foot. This will be the level which, when equalled or exceeded by exterior flood stages, will present gravity outflow and require total pumping to remove any runoff that might occur within the levied area.

(2) From a rating or backwater curve applicable to the location on the watercourse where the gravity outflow point of the levied area exists, determine the discharge corresponding to the maximum ponding level.

(3) Determine the percentage of time that the discharge (obtained from Step 2 above) is equalled or exceeded. Given this percentage of time, determine the frequency of the rainfall event corresponding to the coincidental probability of these two events.

(4) Use TP-40 or other appropriate rainfall frequency curve to obtain the rainfall amounts associated with the return period (obtained from Step 3 above) to be used for determining the required pumping capacity.

6.3.2.1 Design Criteria Assuming Same Event

This criteria presumes the storm event causing high flood stages outside of the levied area is the same (dependent) storm event occurring over the levied area. The design rainfall amounts to be used for sizing the required pump capacity will be associated with the 100-year rainfall event.

6.3.3 Design Criteria

All levied areas within the City of Orange that are equipped with a pump station shall be capable of maintaining the design pumping capacity with its largest single pump inoperative. The capacity of a pump station designed under Section 6.3.2 shall be adequate to remove a minimum volume of water from the levied area within 24 hours without exceeding the maximum ponding elevation within the levied area. If a pump station is not provided, adequate storage volume below the maximum ponding level must be provided to contain the entire design storm. The volume of runoff to be pumped shall be the greater of either: The runoff resulting form the appropriate rainfall amount as determined in Step 4 of Section 6.3.2.

A pump station designed under Section 6.3.2.1 shall have a combination of storage volume/pumping capacity adequate to maintain the runoff resulting from the 100-year frequency event below the maximum ponding level. All pump stations in the City of Orange shall be equipped with auxiliary power for emergency usage.

6.4 GRAVITY OUTLET AND OUTFALL CHANNEL

An outlet shall be required to release the gravity flow from the levied area through the outfall channel to the adjacent watercourse during low flow conditions on the receiving channel. The outlet shall be equipped with an automatically functioning gate to prevent any external flow from entering the levied area.

The outlet and outfall channel shall be designed in accordance with Section 3.0 Open Channel Flow. The velocities within the outfall channel at the adjacent river shall not exceed 5.0 feet per second.

SECTION 7

SECTION 7
LIST OF TABLES

TABLE 1

COEFFICIENTS OF RUNOFF AND MAXIMUM INLET TIMES

Zoning District	Runoff Coefficient C	Maximum Inlet Time In Minutes
Conceptual Planned Development	Variable	10 to 20
Duplex District	0.70	15
General Office District	0.90	10
General Retail	0.90	10
Heavy Commercial	0.90	10
Industrial District	0.90	10
Light Commercial	0.90	10
Limited Office District	0.90	10
Mid-range Office District	0.90	10
Multi-family	0.80	10
Multiple Family, High Rise	0.80	10
Neighborhood Office District	0.85	10
Neighborhood Service District	0.90	10
Office	0.90	10
Parking District	0.90	10
Residential 1 Acre	0.45	20
Residential 1/2 Acre	0.45	20
Residential 10,000 SF	0.65	15
Residential 13,000 SF	0.65	15
Residential 16,000 SF	0.65	15
Residential 5,000 SF	0.65	15
Residential 7,500 SF	0.65	15
Shopping Center	0.90	10
Townhouse 6 Units/Acre	0.80	15
Townhouse 9 Units/Acre	0.85	15
Townhouse 12 Units/Acre	0.90	10
Townhouse 15 Units/Acre	0.90	10

Non-Zoned Land Uses

Land Use	Runoff Coefficient C
Church	0.8
School	0.7
Park	0.4
Cemetery	0.4
Agricultural	0.3

TABLE 2

ROUGHNESS COEFFICIENTS FOR CLOSED CONDUITS

Material of New Construction	Recommended Roughness Coefficient "n"
Concrete Pipe Storm Sewer	0.013

Material of Existing Systems	
Concrete Pipe Storm Sewer	
Fair Alignment, Ordinary Joints	0.015
Poor Alignment, Poor Joints	0.017
Concrete Pipe Culverts	0.012
Monolithic Concrete Culverts	0.012

NOTE: Reinforced concrete pipe is the accepted material for construction of storm sewers. The use of other materials for the construction of storm sewers shall have prior approval from the City Engineering Department.

TABLE 3

VELOCITIES IN CLOSED CONDUITS

Type of Conduit	Min. Velocity	Max. Velocity
Culverts	2.5 fps	15 fps
Inlet Laterals	2.5 fps	15 fps
Storm Sewers	2.5 fps	10 fps

Storm sewers shall discharge into open channels at a maximum velocity of 5 feet per second.

TABLE 4

VELOCITY HEAD LOSS COEFFICIENTS FOR CLOSED CONDUITS

MANHOLE AT CHANGE IN PIPE DIRECTION

DESCRIPTION	ANGLE	HEAD LOSS COEFFICIENT Kj
	90	1.00
	60	0.80
	45	0.65
Angle	30	0.50

BEND IN PIPES

DESCRIPTION	ANGLE	HEAD LOSS COEFFICIENT Kj
	* 90°	0.80
	* 60°	0.60
	** 45°	0.50
Angle	** 30°	0.45

ENLARGEMENTS IN PIPE SIZES WITH CONSTANT FLOW

DESCRIPTION	RATIO OF UPSTREAM DIAMETER TO DOWNSTREAM DIAMETER	HEAD LOSS COEFFICIENT Kj
	0.81	1.00
	0.82	0.90
	0.84	0.80
	0.85	0.70
	0.86	0.60
	0.88	0.50
	0.90	0.40
	0.92	0.30

* Only as authorized by City Engineer
** Horizontal curves are the accepted method of construction

TABLE 5

ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Channel Description	Roughness Coefficient Minimum	Normal	Maximum	Maximum Velocity ft/sec
MINOR NATURAL STREAMS - TYPE I CHANNEL				
Moderately Well Defined Channel				
Grass and Weeds, Little Brush	0.025	0.030	0.033	8
Dense Weeds, Little Brush	0.030	0.035	0.040	8
Weeds, Light Brush on Banks	0.030	0.035	0.040	8
Weeds, Heavy Brush on Banks	0.035	0.050	0.060	8
Weeds, Dense Willows on Banks	0.040	0.060	0.080	8
Irregular Channel with Pools and Meanders				
Grass and Weeds, Little Brush	0.030	0.036	0.042	8
Dense Weeds, Little Brush	0.036	0.042	0.048	8
Weeds, Light Brush on Banks	0.036	0.042	0.048	8
Weeds, Heavy Brush on Banks	0.042	0.060	0.072	8
Weeds, Dense Willows on Banks	0.048	0.072	0.096	8
Flood Plain, Pasture				
Short Grass, No Brush	0.025	0.030	0.035	8
Tall Grass, No Brush	0.030	0.035	0.050	8
Flood Plain, Cultivated				
No Crops	0.025	0.030	0.035	8
Mature Crops	0.030	0.040	0.050	8
Flood Plain, Uncleared				
Heavy Weeds, Light Brush	0.035	0.050	0.070	8
Medium to Dense Brush	0.070	0.100	0.160	8
Trees with Flood Stage below Branches	0.080	0.100	0.120	8
MAJOR NATURAL STREAMS - TYPE I CHANNEL				
The roughness coefficient is less than that for minor streams of similar description because banks offer less effective resistance.				
Moderately Well Defined Channel	0.025	---	0.060	8
Irrigular Channel	0.035	---	0.100	8
UNLINED VEGETATED CHANNELS - TYPE II CHANNEL				
Mowed Grass, Clay Soil	0.025	0.030	0.035	8
Mowed Grass, Sandy Soil	0.025	0.030	0.035	6

TABLE 5

ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Channel Description	Roughness Coefficient Minimum	Normal	Maximum	Maximum Velocity ft/sec
UNLINED NON-VEGETATED CHANNELS - TYPE II CHANNEL				
Clean Gravel Section	0.022	0.025	0.030	8
Shale	0.025	0.030	0.035	10
Smooth Rock	0.025	0.030	0.035	15
LINED CHANNELS - TYPE III				
Smooth Finished Concrete	0.013	0.015	0.020	15
Riprap (Rubble)	0.030	0.040	0.050	12

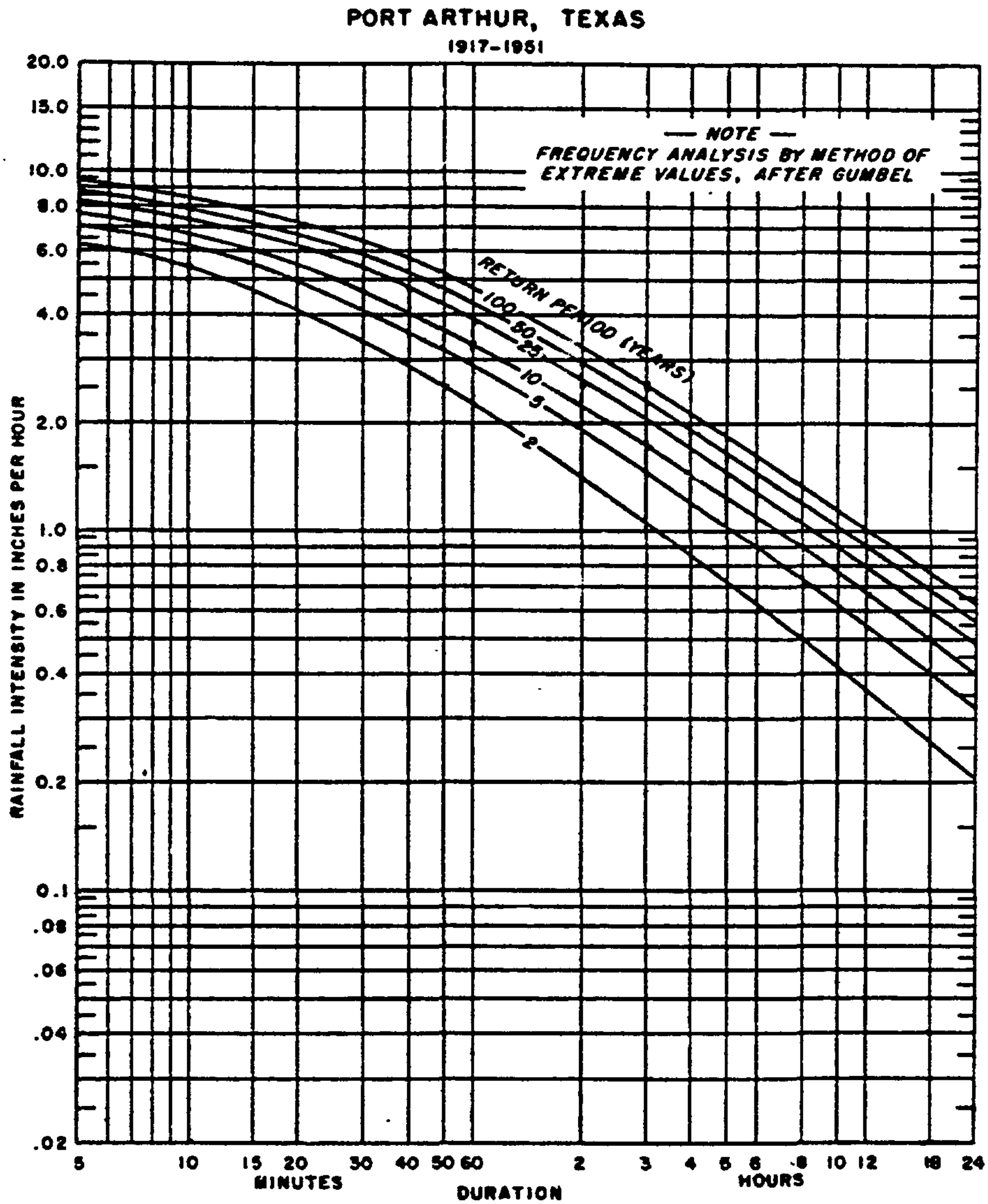
SECTION 8

SECTION 8
LIST OF FIGURES

SECTION 8

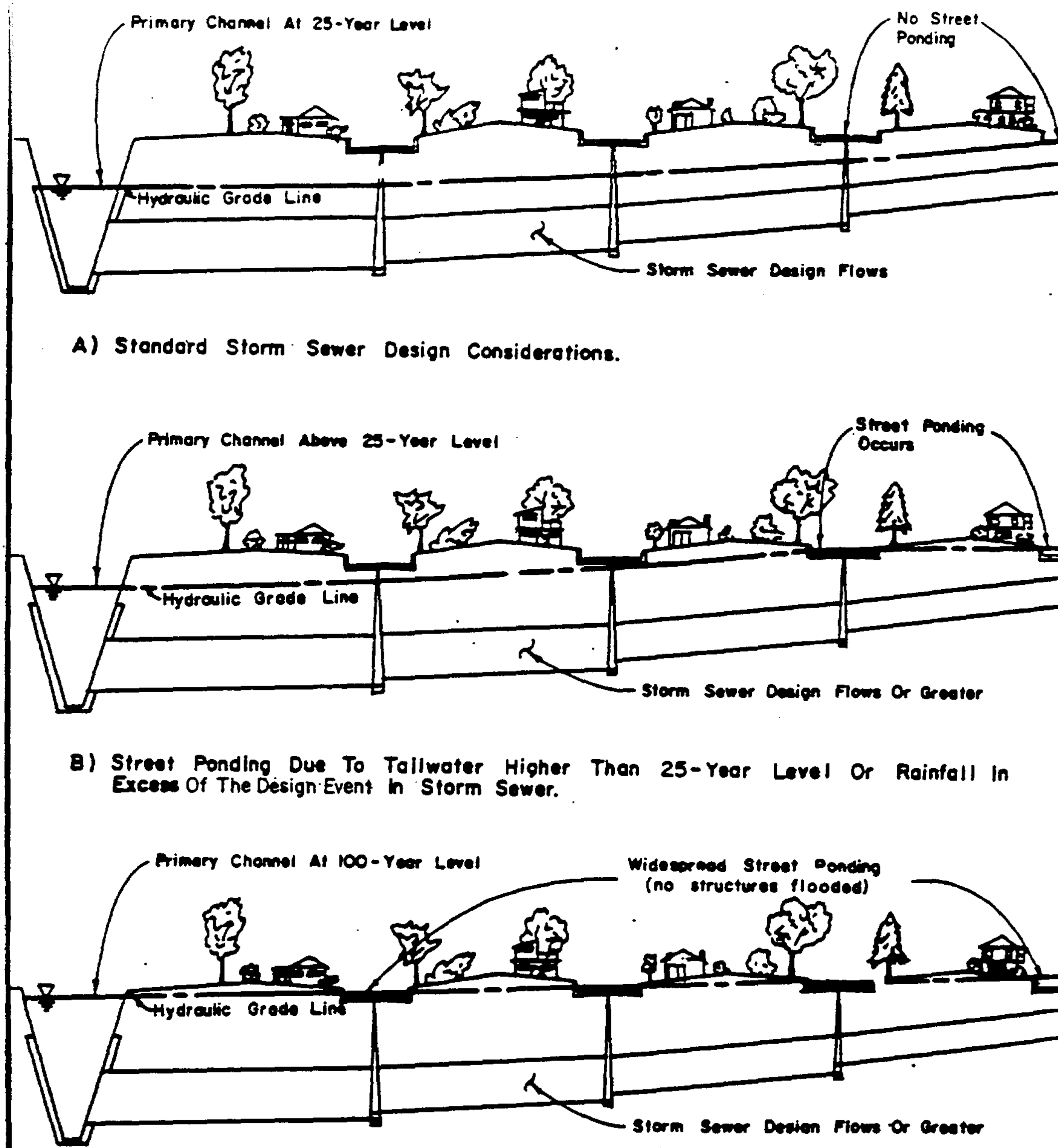
LIST OF FIGURES (Cont.)

FIGURE 1



RAINFALL INTENSITY--DURATION--FREQUENCY CURVES
for the
Port Arthur, Texas Area
Data 1917 - 1951

Reference: U. S. Dept. of Commerce Technical Paper #25



A) Standard Storm Sewer Design Considerations.

B) Street Ponding Due To Tailwater Higher Than 25-Year Level Or Rainfall In Excess Of The Design Event In Storm Sewer.

C) System Operating At Maximum Capacity.

Source:

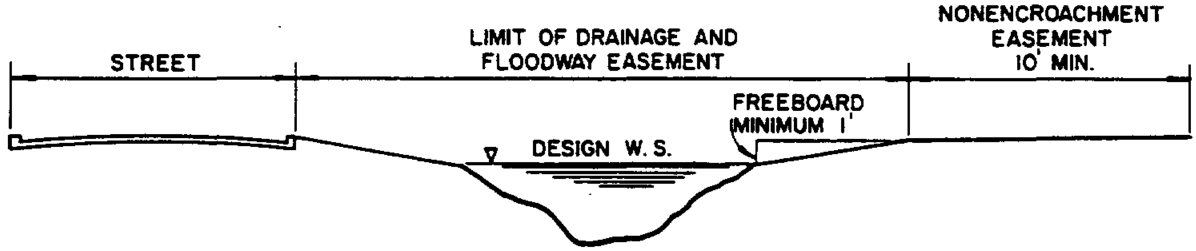
Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, February, 1984.

STORM SEWER-CHANNEL INTERACTION

FIGURE 2

CREEKS MAY REMAIN IN OPEN NATURAL CONDITION IF:

(1) THEY COMPLY WITH THE SUBDIVISION ORDINANCE;
(2) TREE COVERAGE IS ADEQUATE TO BE ACCEPTABLE TO THE CITY;
(3) UNSANITARY OR UNACCEPTABLE DRAINAGE CONDITIONS DO NOT EXIST IN THE CREEK;
(4) APPROVED BY THE CITY ENGINEER.



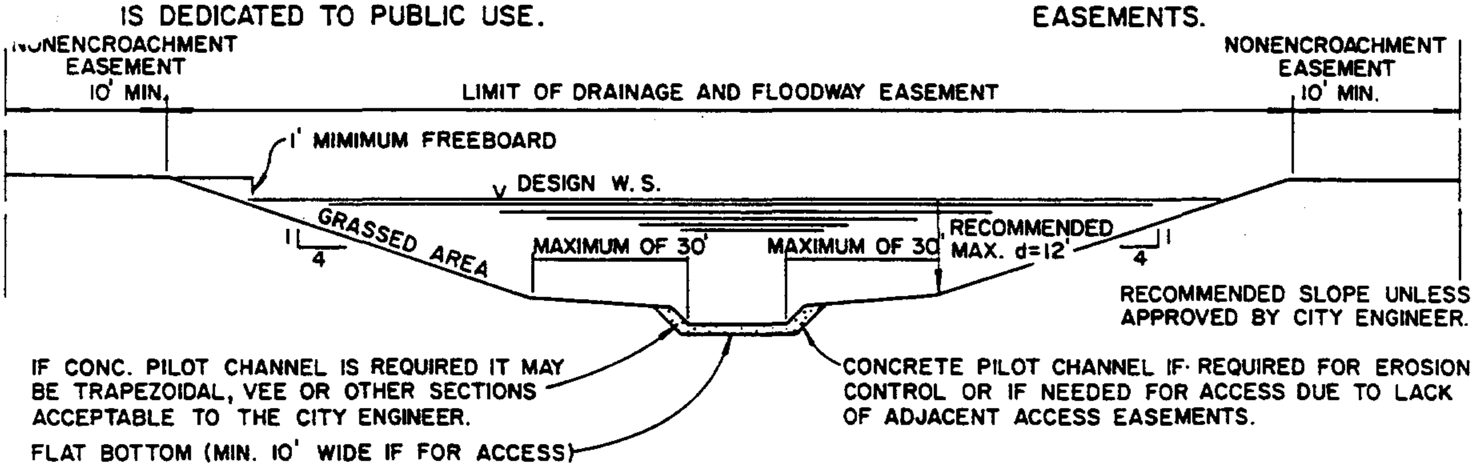
UNIMPROVED CHANNEL

TYPE I – NATURAL

NOTE: TYPE I OR II – IF STEEPER THAN 3:1 SLOPE ABOVE DESIGN W.S., THE NON-ENCROACHMENT ESMT. SHALL BE 15 FEET WIDE TO PROVIDE A STABLE ACCESS ESMT., IF ACCESS HAS NOT OTHERWISE BEEN PROVIDED.

NOTE: A PARALLEL STREET ON AT LEAST ONE SIDE OF TYPE I CHANNELS IF THE DRAINAGE AND FLOODWAY IS DEDICATED TO PUBLIC USE.

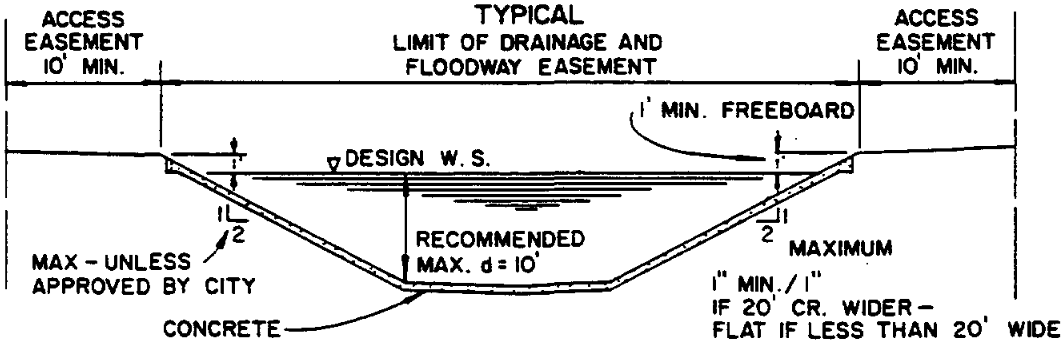
NOTE: NO ENCROACHMENTS SHALL BE BE PERMITTED IN ACCESS EASEMENTS.



UNLINED CHANNELS

TYPE II — UNLINED WITH MAINTENANCE SECTION

TYPICAL



TYPE III - LINED

OPEN CHANNEL TYPES

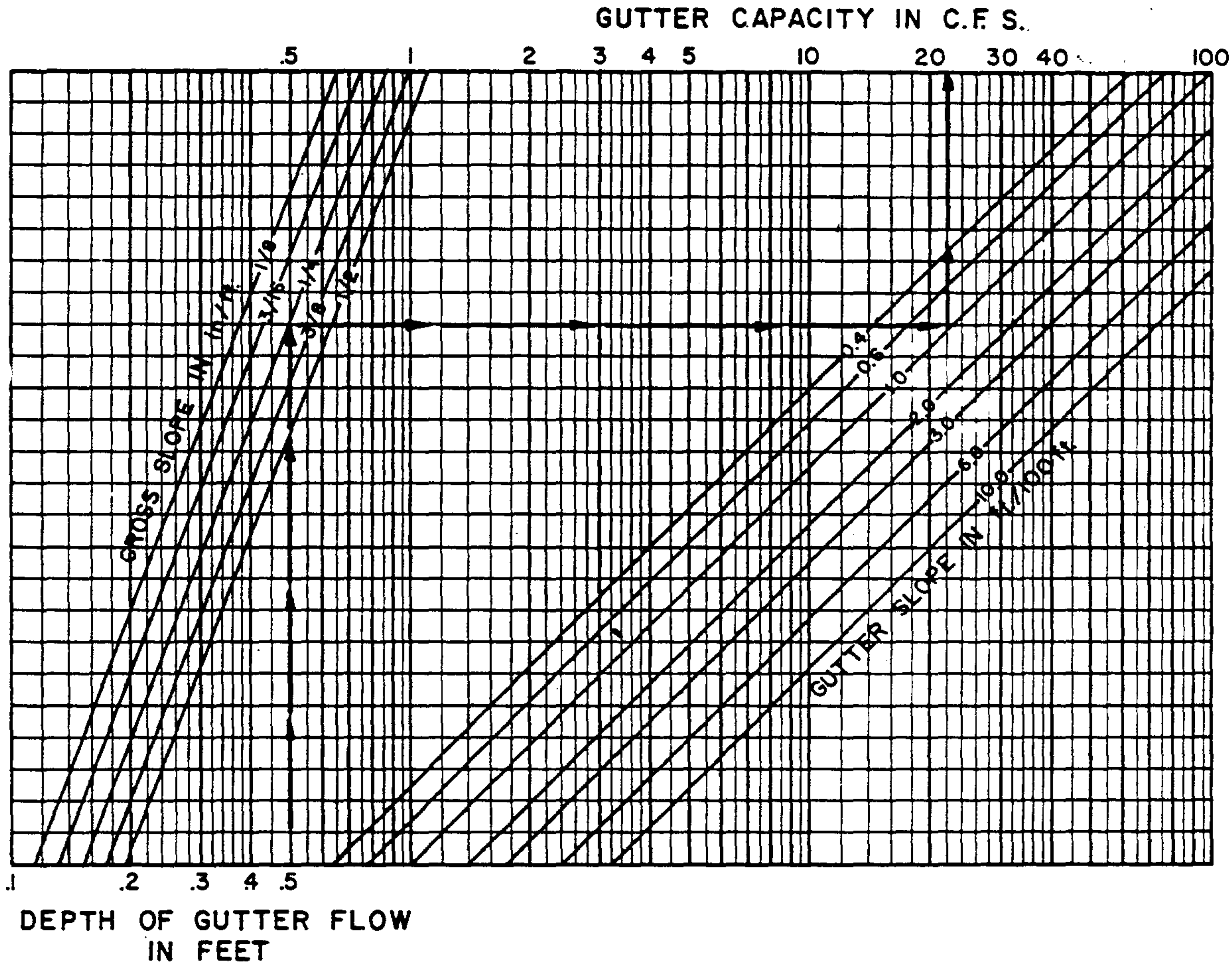
FIGURE 3

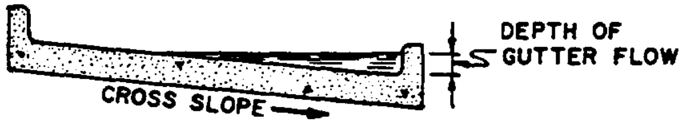
EXAMPLE

Known:
Major Thoroughfare,
Pavement Width = 33'
Gutter Slope = 1.0%
Pavement Cross Slope = 1/4"/1'
Depth of Gutter Flow = .5'

Find:
Gutter Capacity

Solution:
Enter Graph at .5'
Intersect Cross Slope = 1/4"/1'
Intersect Gutter Slope = 1.0%
Read Gutter Capacity = 22 c.f.s.

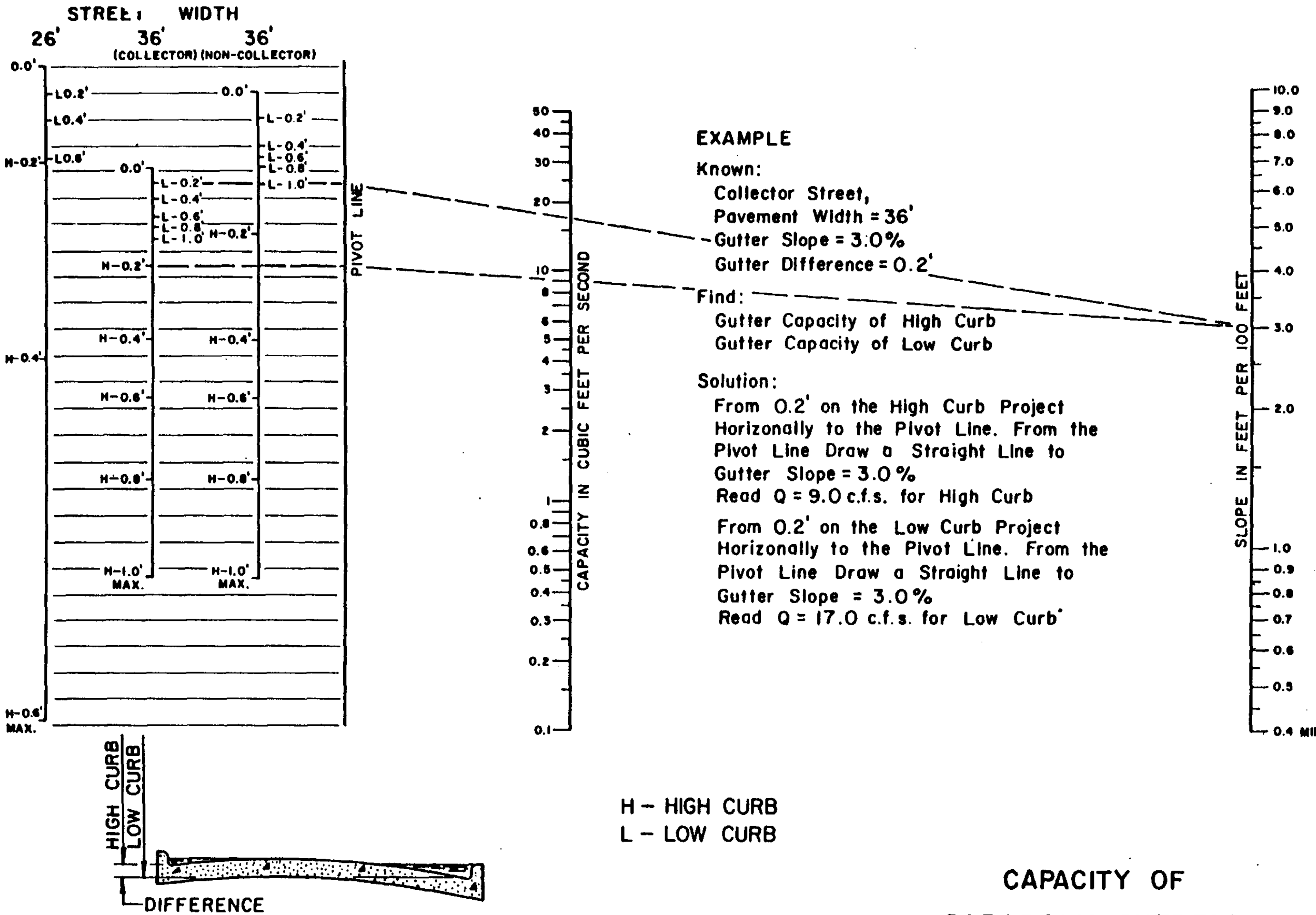




(Roughess Coefficient n = .0175)

CAPACITY OF TRIANGULAR GUTTERS

FIGURE 4



EXAMPLE

Known:

Collector Street,
Pavement Width = 36'
Gutter Slope = 3.0%
Gutter Difference = 0.2'

Find:

Gutter Capacity of High Curb
Gutter Capacity of Low Curb

Solution:

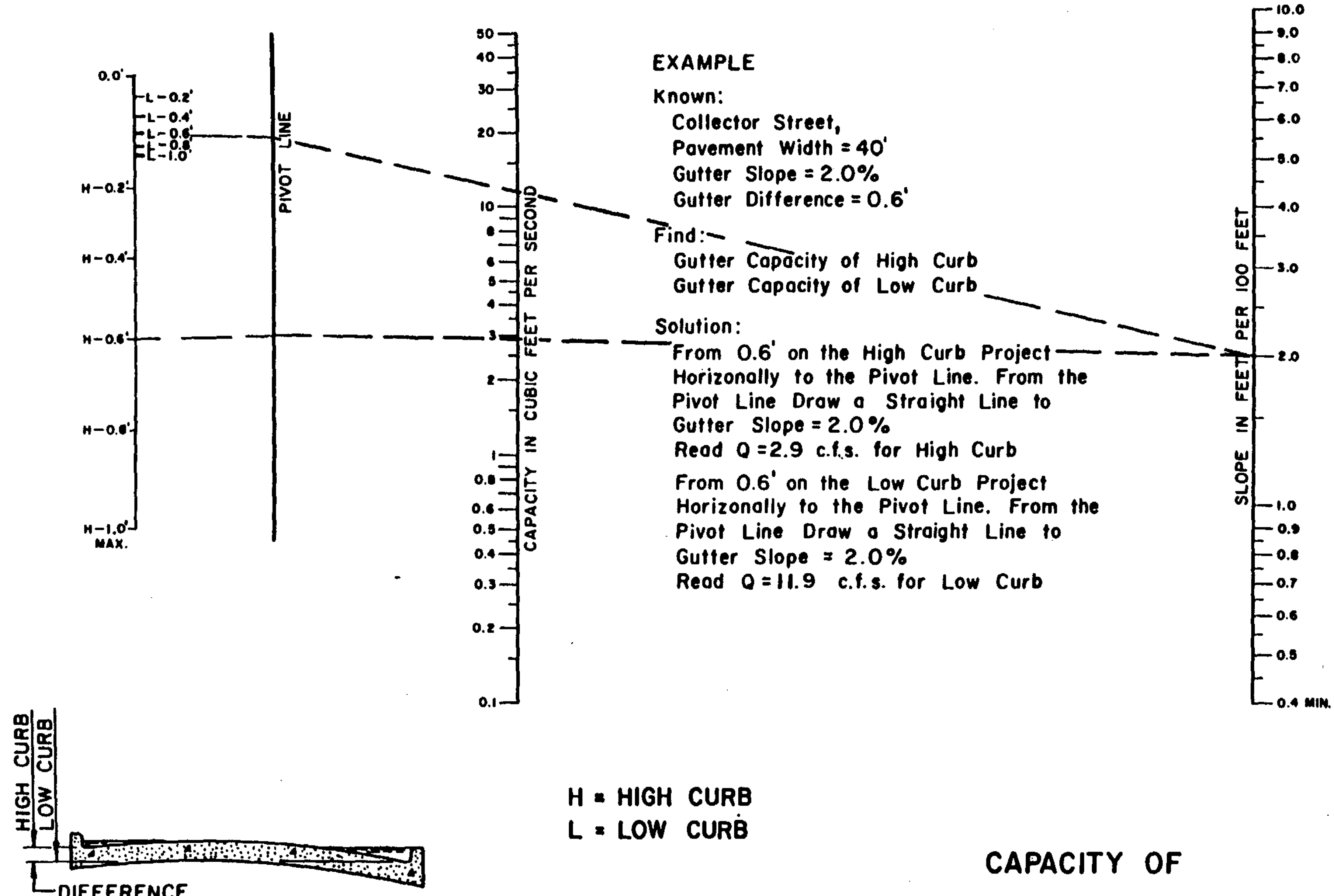
From 0.2' on the High Curb Project Horizonally to the Pivot Line. From the Pivot Line Draw a Straight Line to Gutter Slope = 3.0%
Read Q = 9.0 c.f.s. for High Curb

From 0.2' on the Low Curb Project Horizonally to the Pivot Line. From the Pivot Line Draw a Straight Line to Gutter Slope = 3.0%
Read Q = 17.0 c.f.s. for Low Curb

H – HIGH CURB
L – LOW CURB

CAPACITY OF PARABOLIC GUTTERS (26' & 36' STREET WIDTHS)

FIGURE 5



EXAMPLE

Known:

Collector Street,
Pavement Width = 40'
Gutter Slope = 2.0%
Gutter Difference = 0.6'

Find:

Gutter Capacity of High Curb
Gutter Capacity of Low Curb

Solution:

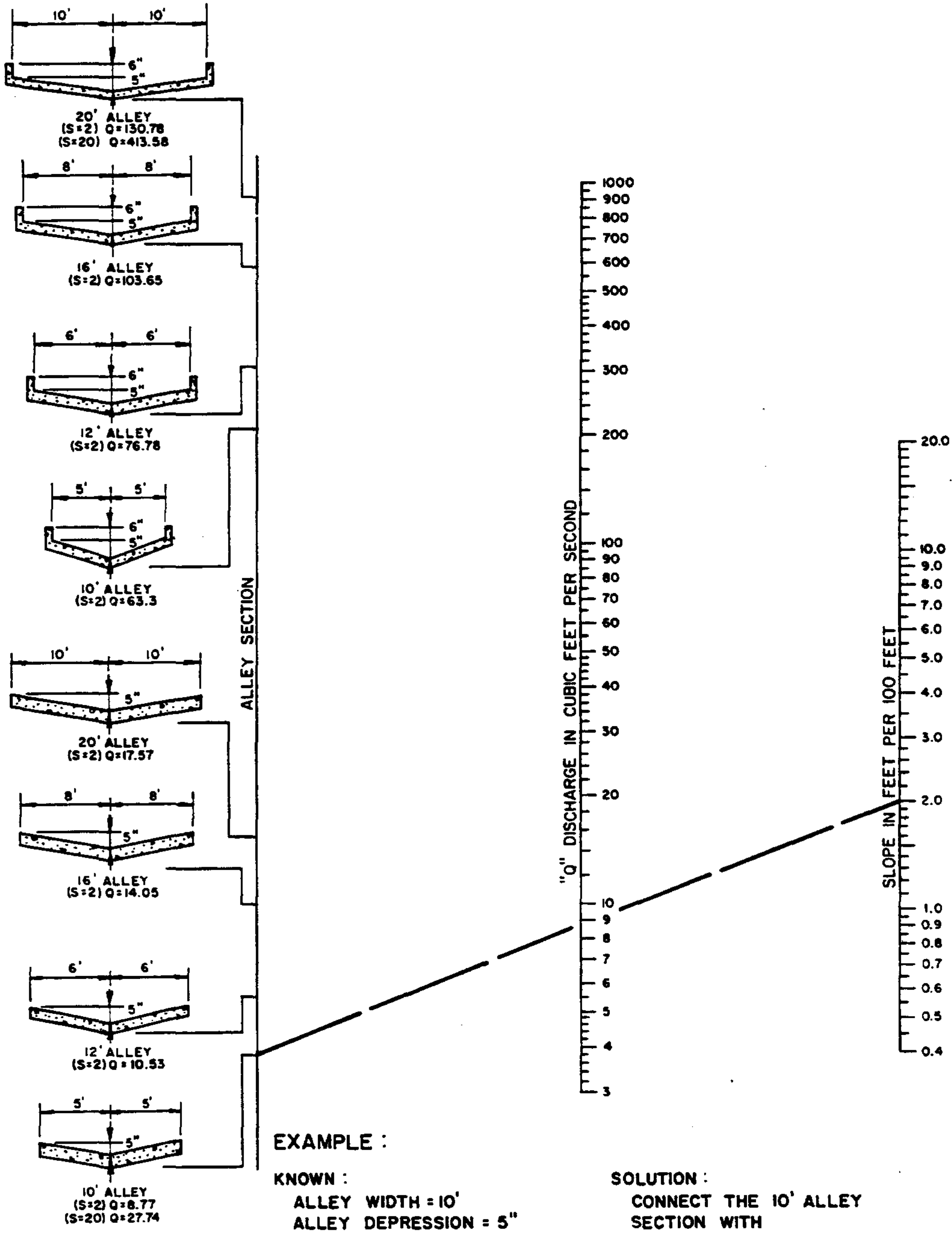
From 0.6' on the High Curb Project Horizonally to the Pivot Line. From the Pivot Line Draw a Straight Line to Gutter Slope = 2.0%
Read Q = 2.9 c.f.s. for High Curb

From 0.6' on the Low Curb Project Horizonally to the Pivot Line. From the Pivot Line Draw a Straight Line to Gutter Slope = 2.0%
Read Q = 11.9 c.f.s. for Low Curb

H = HIGH CURB
L = LOW CURB

CAPACITY OF PARABOLIC GUTTERS (40' F-F STREET WIDTH)

FIGURE 6



EXAMPLE :

KNOWN :
ALLEY WIDTH = 10'
ALLEY DEPRESSION = 5"
INVERT SLOPE = 2.2%

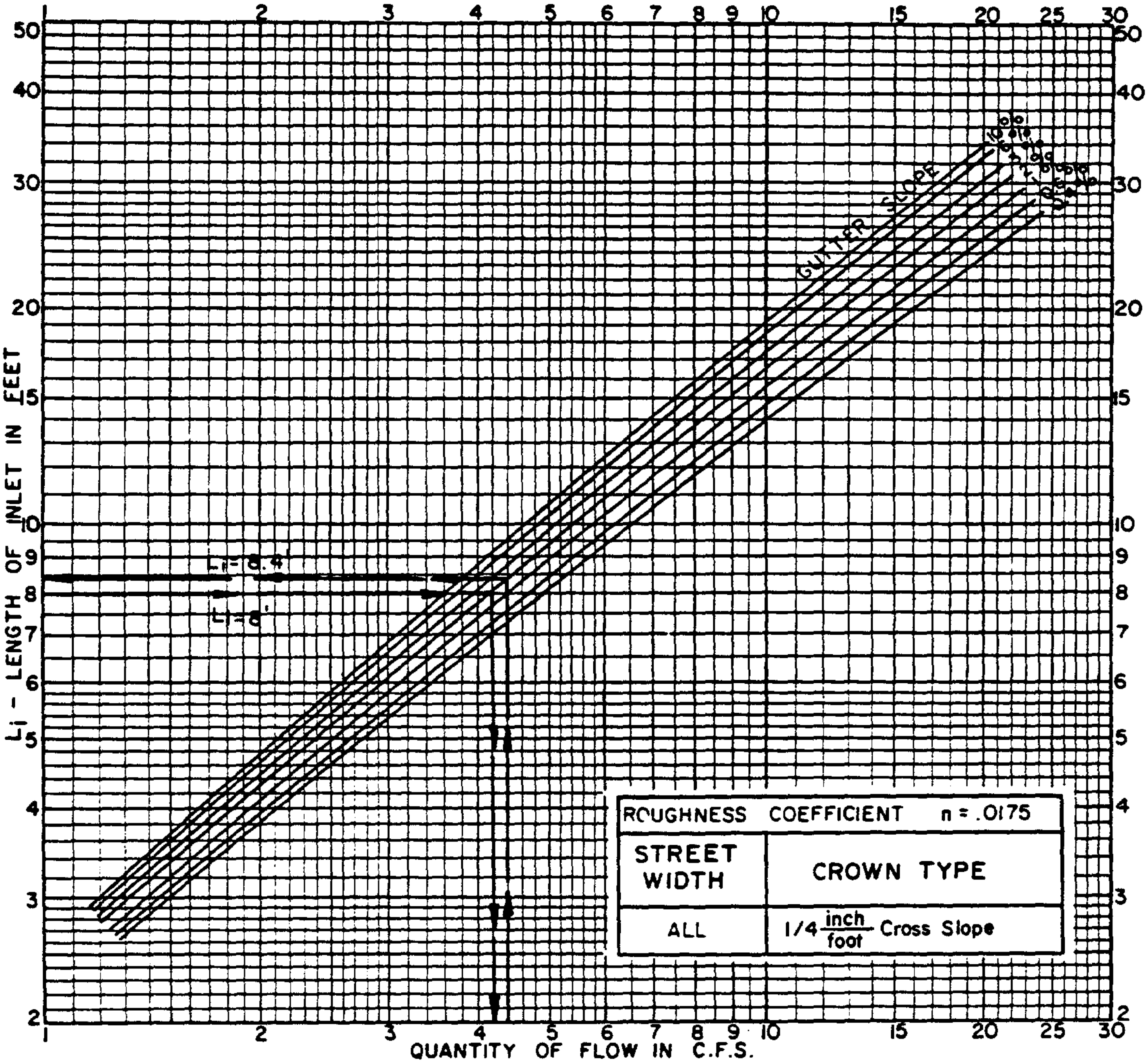
FIND :
INVERT FLOW (Q)

SOLUTION :
CONNECT THE 10' ALLEY SECTION WITH SLOPE = 2.2 %
READ Q = 9.4 c.f.s.

Note :
The Capacities Obtained From This Nomograph are Based on a Straight Horizonal Alignment. Curved Alignments May Result in Reduced Capacity.

CAPACITY OF ALLEY SECTIONS
$n = 0.0175$

FIGURE 7



EXAMPLE

Known:

Pavement Width = 24'
Gutter Slope = 2.0 %
Pavement Cross Slope = 1/4"/1'
Gutter Flow = 4.4 cfs

Find:

Length of Inlet Required (L_i)

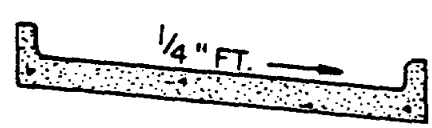
Solution:

Enter Graph at 4.4 cfs
Intersect Slope = 2.0 %
Read L_i = 8.4'

Decision:

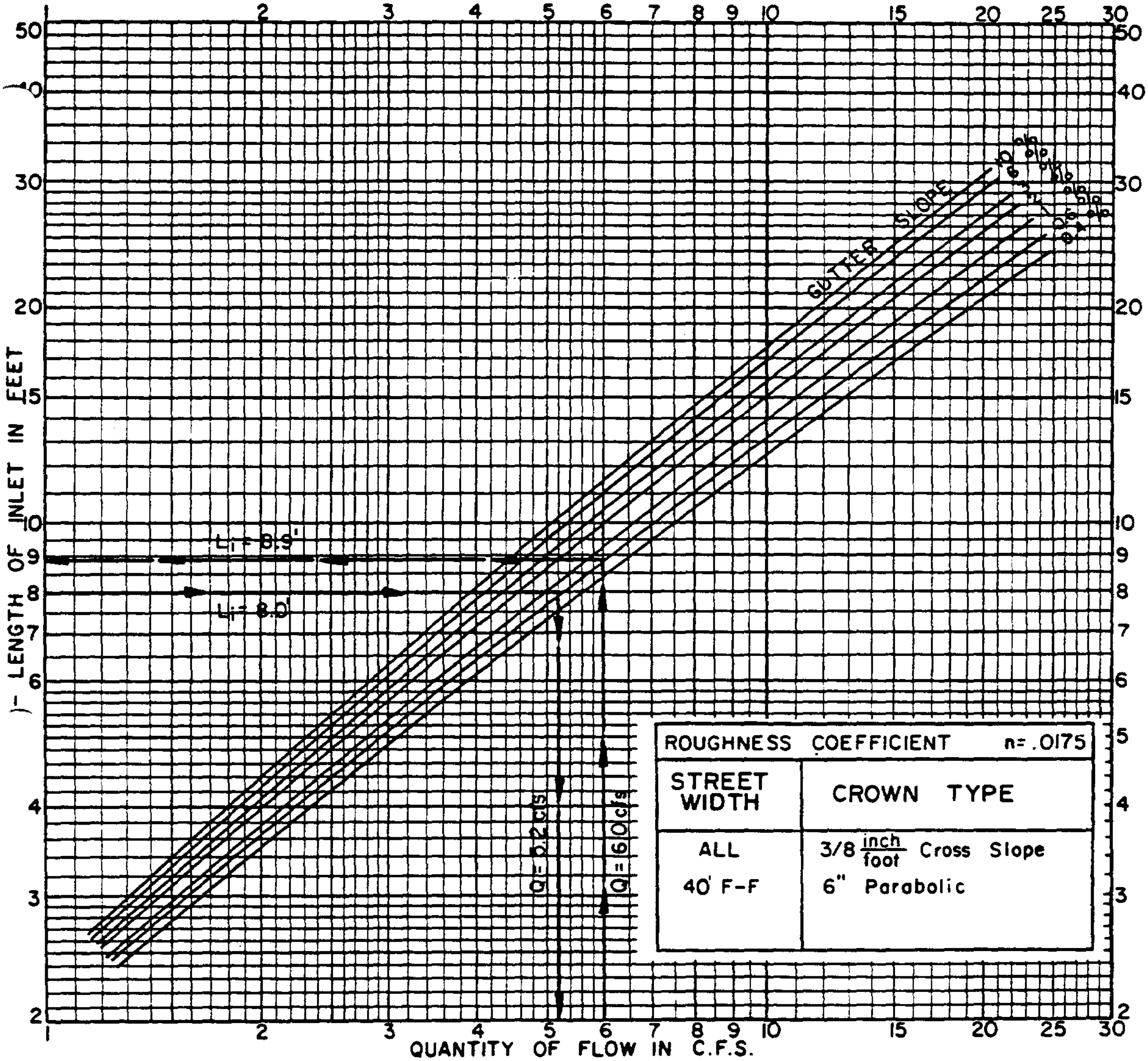
1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 2.0%
Read Q = 4.2 cfs
Remaining Gutter Flow =
4.4 cfs − 4.2 cfs = 0.2 cfs



RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGU[illegible]



ROUGHNESS COEFFICIENT n= .0175	
STREET WIDTH	CROWN TYPE
ALL	3/8 $\frac{\text{inch}}{\text{foot}}$ Cross Slope
40' F-F	6" Parabolic

EXAMPLE

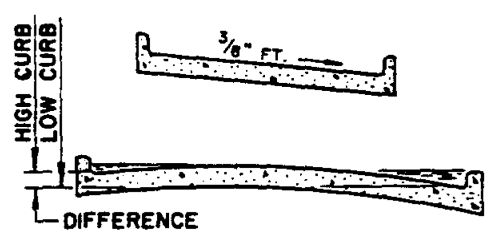
Known:
Pavement Width = 44'
Gutter Slope = 0.6 %
6" Parabolic Crown
Gutter Flow = 6.0 cfs

Find:
Length of Inlet Required (L_i)

Solution:
Enter Graph at 6.0 cfs
Intersect Slope = 0.6 %
Read L_i = 8.9'

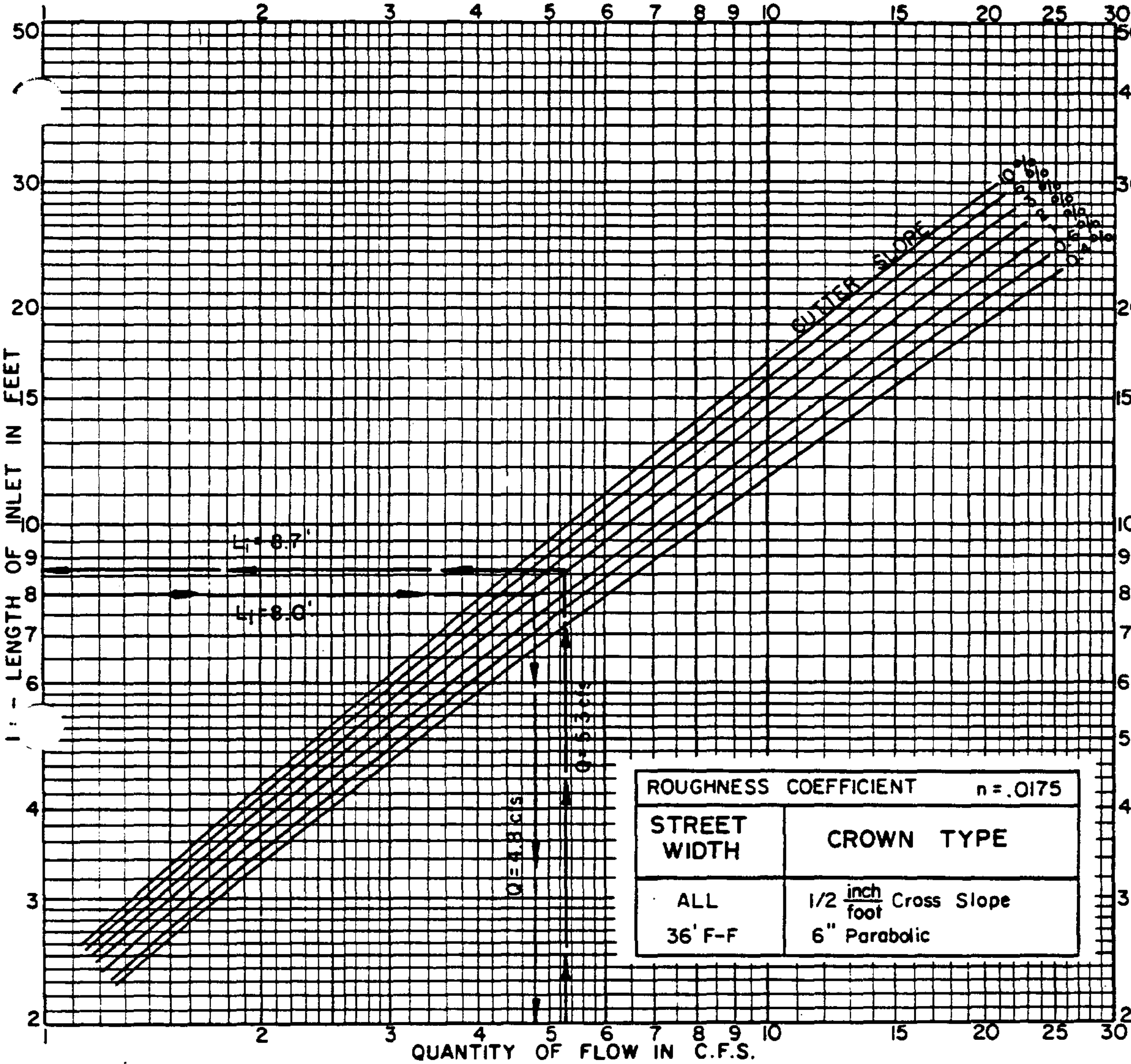
Decision:
1. Use 10' Inlet
No Flow Remains in Gutter
2. Use 8' Inlet
Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 0.6 %
Read Q = 5.2 cfs
Remaining Gutter Flow =
6.0 cfs — 5.2 cfs = 0.8 cfs



RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGURE 9



EXAMPLE

Known:

Pavement Width = 36'
Gutter Slope = 2%
6" Parabolic Crown
Gutter Flow = 5.3 cfs

Find:

- Length of Inlet Required (L_i)

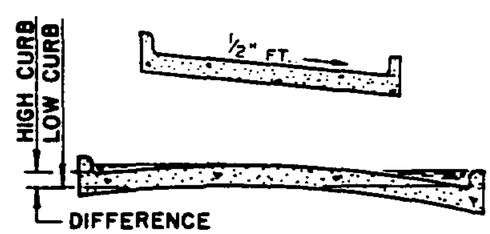
Solution:

Enter Graph at 5.3 cfs
Intersect Slope = 2%
Read L_i = 8.7'

Decision:

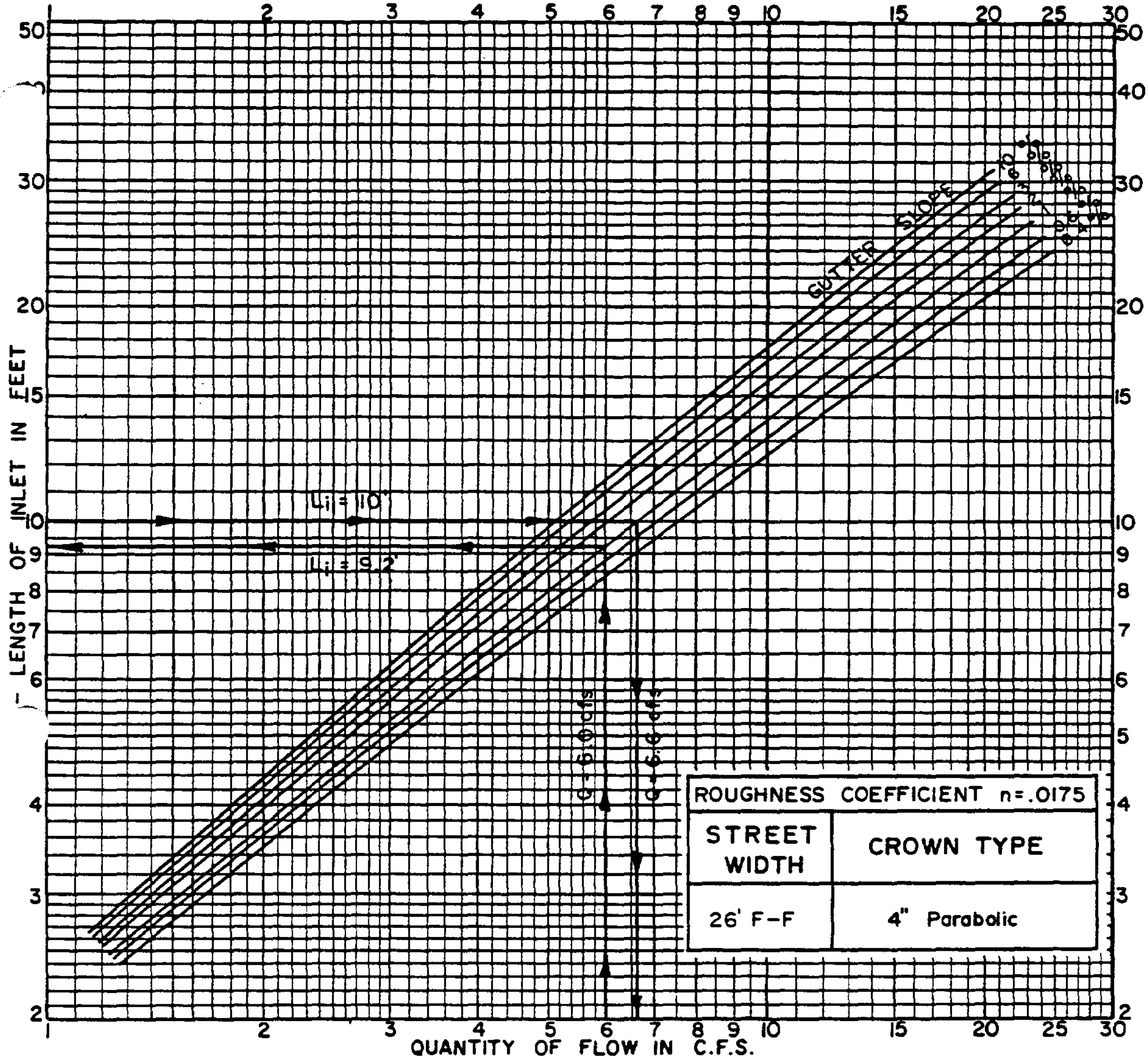
1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 2%
Read Q = 4.8 cfs
Remaining Gutter Flow =
5.3 cfs − 4.8 cfs = 0.5 cfs



RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGURE 10



EXAMPLE

Known:

Pavement Width = 26'
Gutter Slope = 1%
4" Parabolic Crown
Gutter Flow = 6.0 cfs

Find:

Length of Inlet Required (L_i)

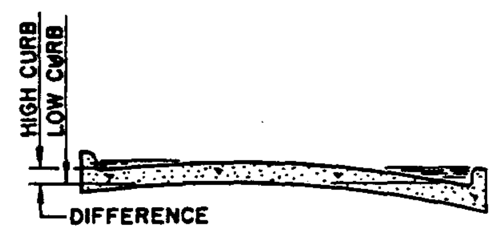
Solution:

Enter Graph at 6.0 cfs
Intersect Slope = 1%
Read L_i = 9.2'

Decision:

1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

Use 10' Inlet
Enter Graph at L_i = 10'
Intersect Slope = 1%
Read Q = 6.6 cfs
No Flow Remains in Gutter



RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGURE 11

EXAMPLE

Known:

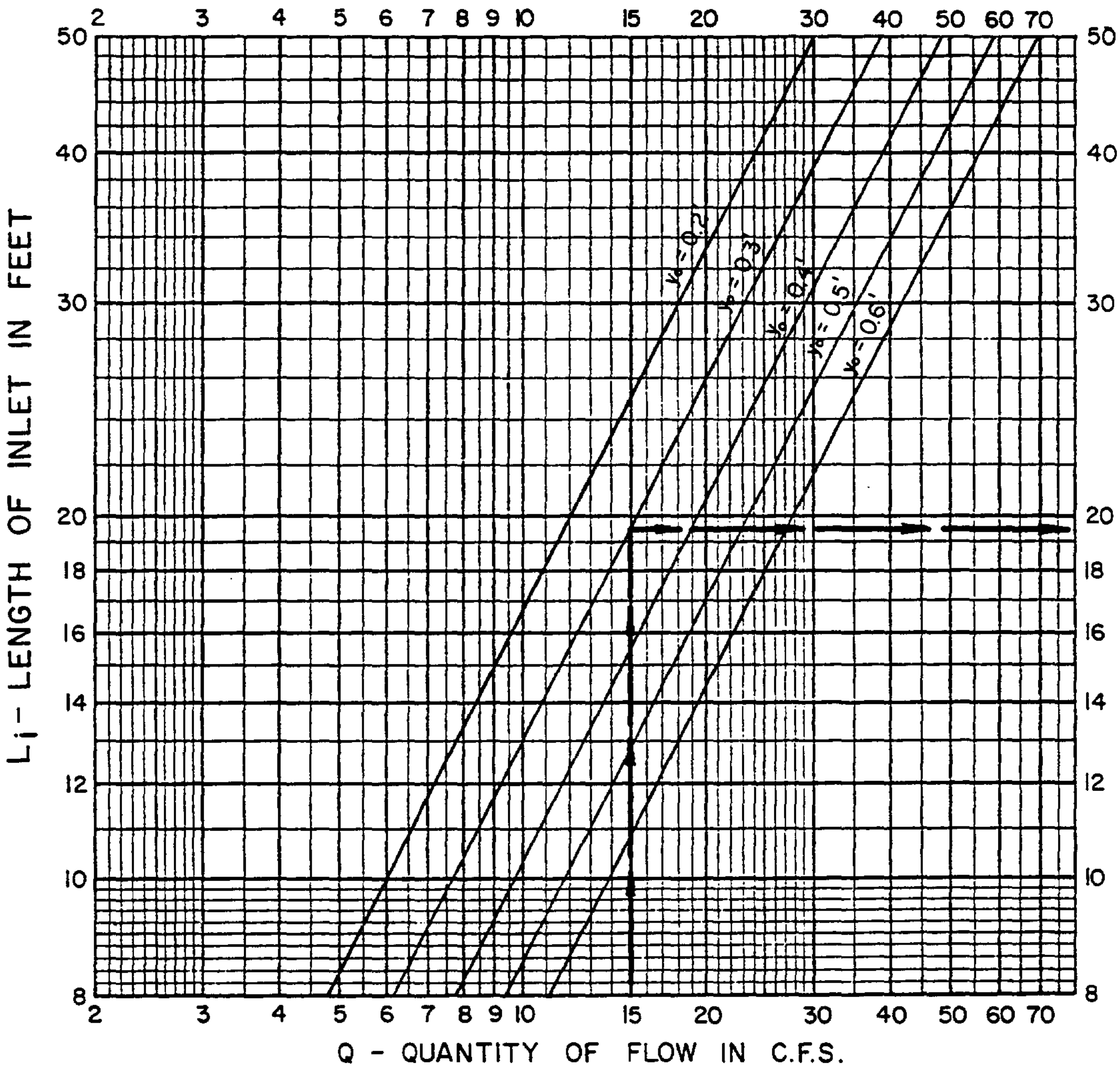
Quantity of Flow = 15.0 c.f.s.
Maximum Depth of Flow Desired in Gutter At Low Point (y_o) = 0.3'

Find:

Length of Inlet Required (L_i)

Solution:

Enter Graph at 15.0 c.f.s.
Intersect y_o = 0.3'
Read L_i = 19.5'
Use 20' Inlet



ROUGHNESS COEFFICIENT n = .0175	
STREET WIDTH	CROWN TYPE
ALL	Straight and Parabolic

RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES AT LOW POINT

FIGURE 12

EXAMPLE:

Known:

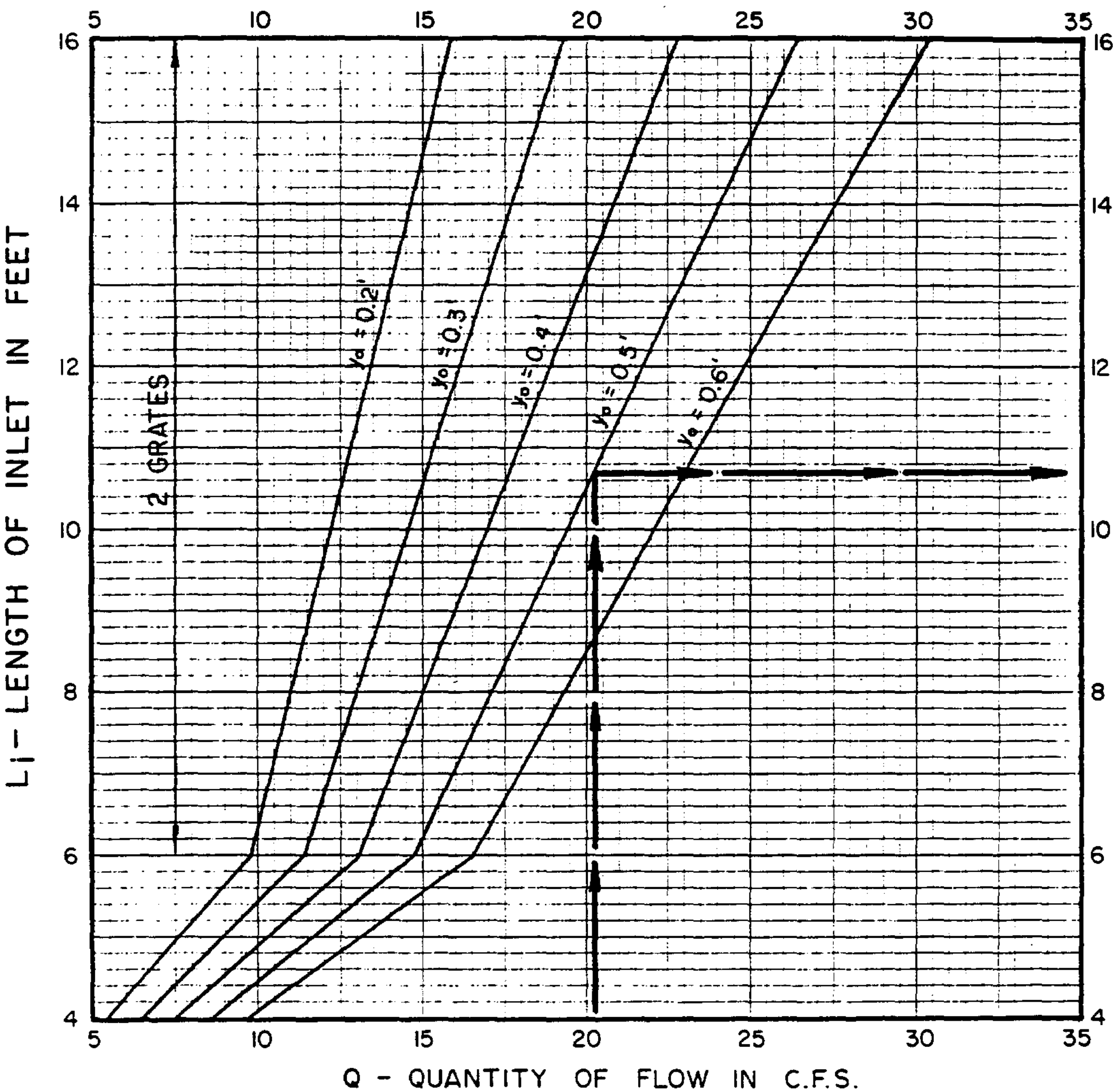
Quantity of Flow = 20.0c.f.s.
Maximum Depth of Flow Desired in Gutter At Low Point (y_o) = 0.5'

Find:

Length of Inlet Required (L_i)

Solution:

Enter Graph at 20.0c.f.s.
Intersect y_o = 0.5'
Read L_i = 10.6'
Use 12' Inlet with 2 grates



ROUGHNESS COEFFICIENT n = .0175	
STREET WIDTH	CROWN TYPE
ALL	Straight and Parabolic

RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES AT LOW POINT

FIGURE 13

EXAMPLE :

Known :

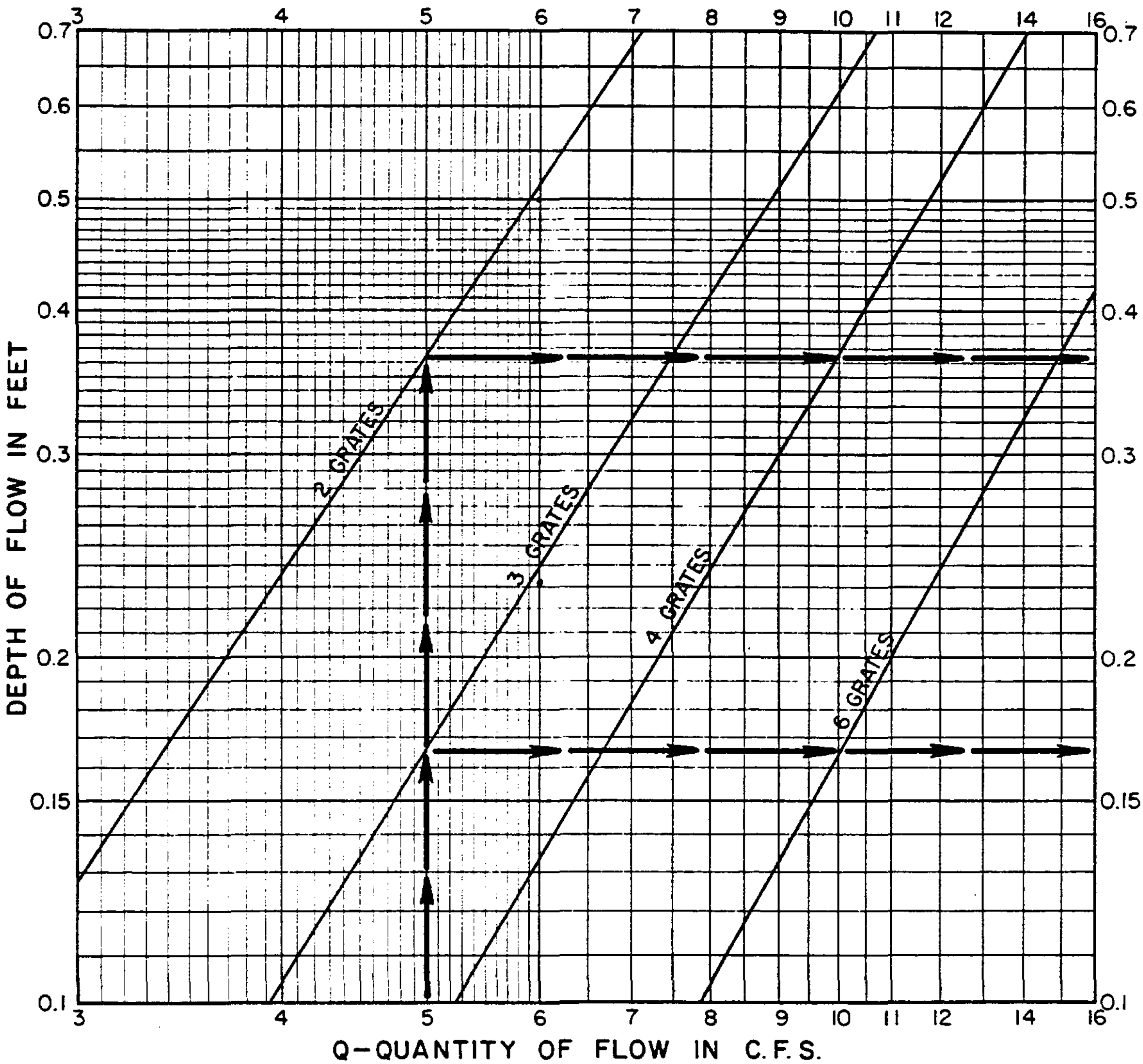
Quantity of Flow = 5.0 c.f.s.
Maximum Depth of Flow Desired at Low Point = 0.3

Find :

Inlet Required

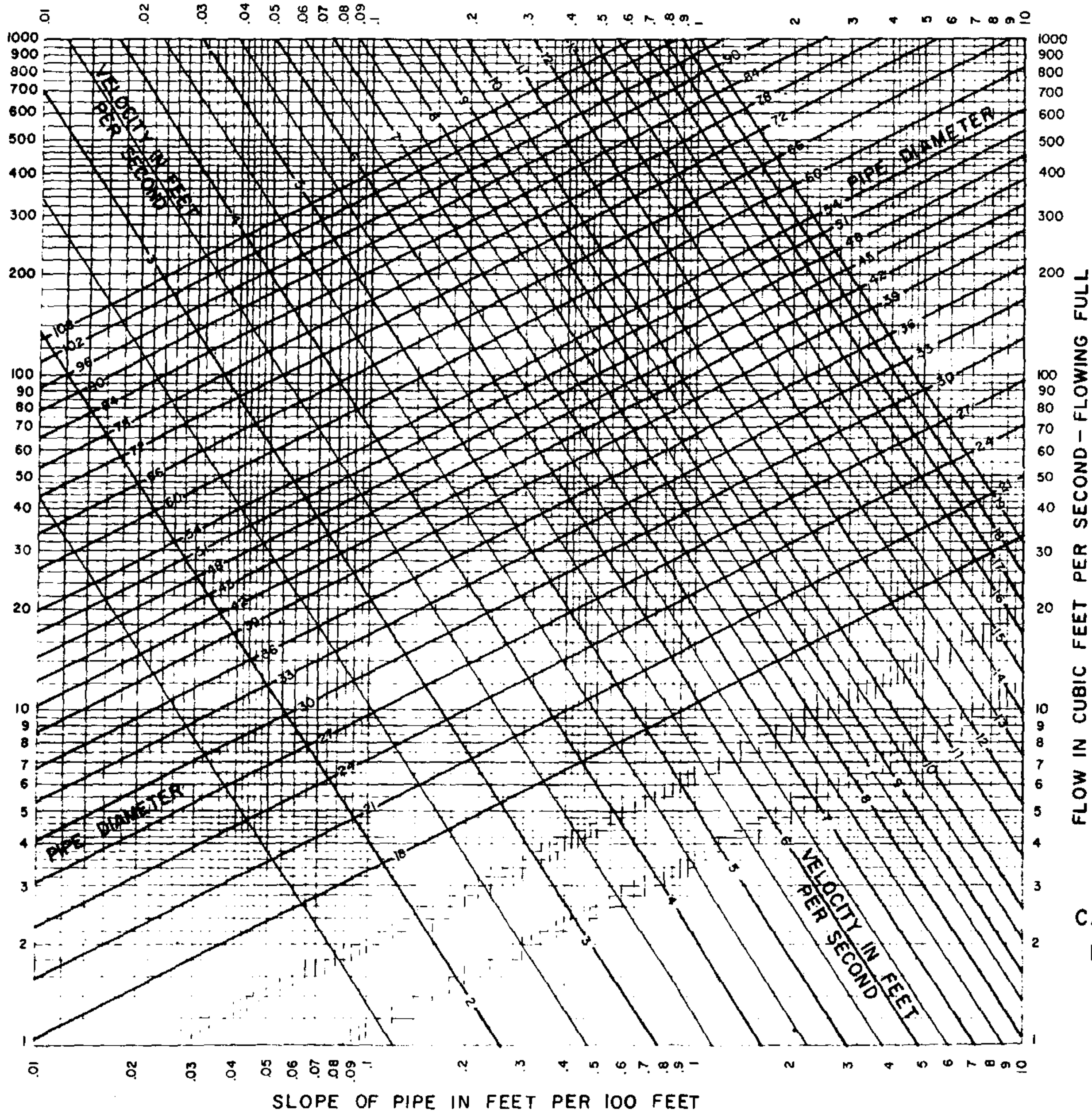
Solution :

Enter Graph at 5.0 c.f.s.
Intersect 3 - Grate at 0.165
Intersect 2 - Grate at 0.365
Use 3 - Grate



GRATE INLET
CAPACITY CURVES
AT LOW POINT

FIGURE 14



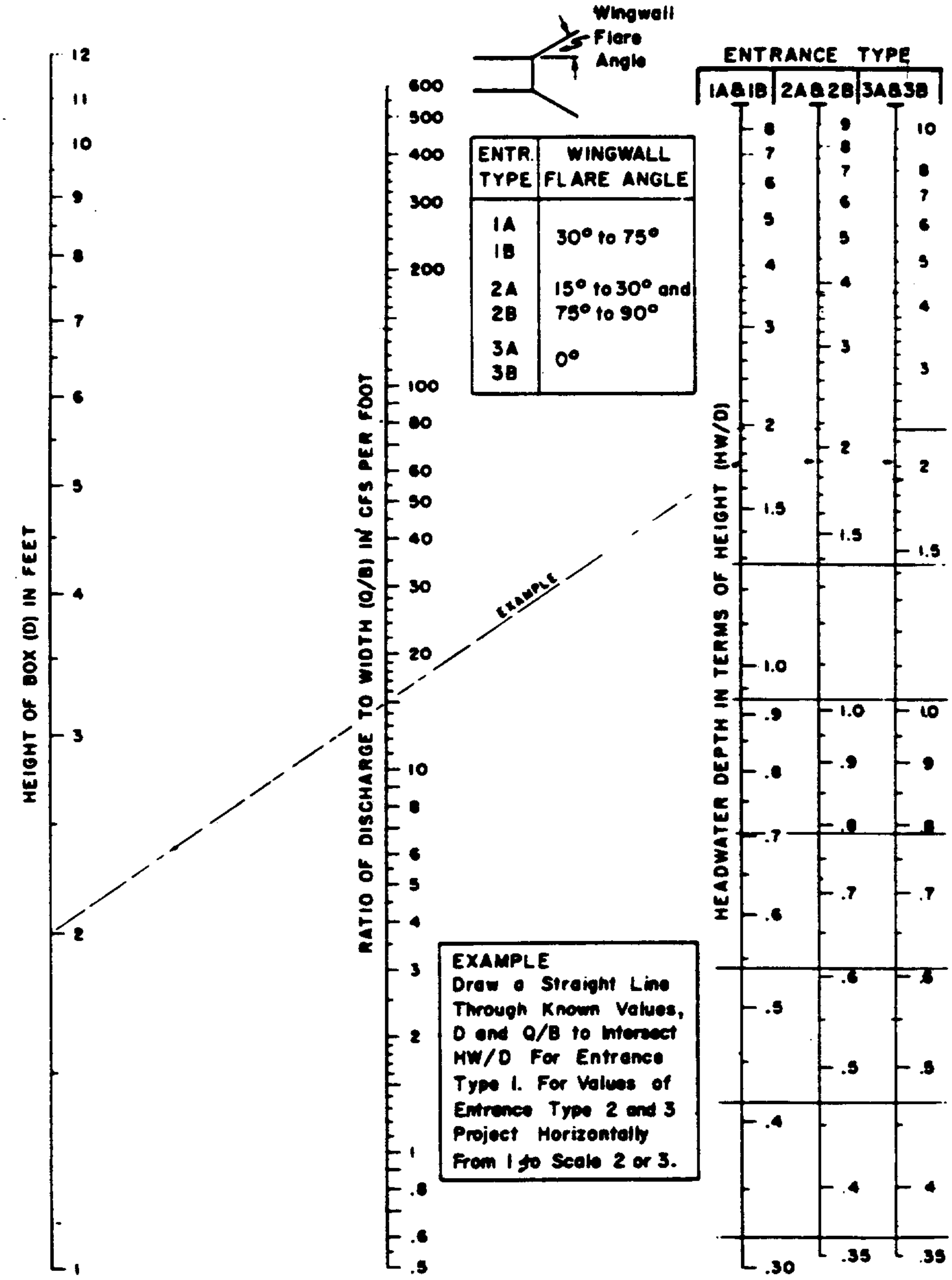
A GRAPHICAL SOLUTION OF MANNING'S EQUATION

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

$n = 0.013$

CAPACITY OF CIRCULAR PIPES FLOWING FULL

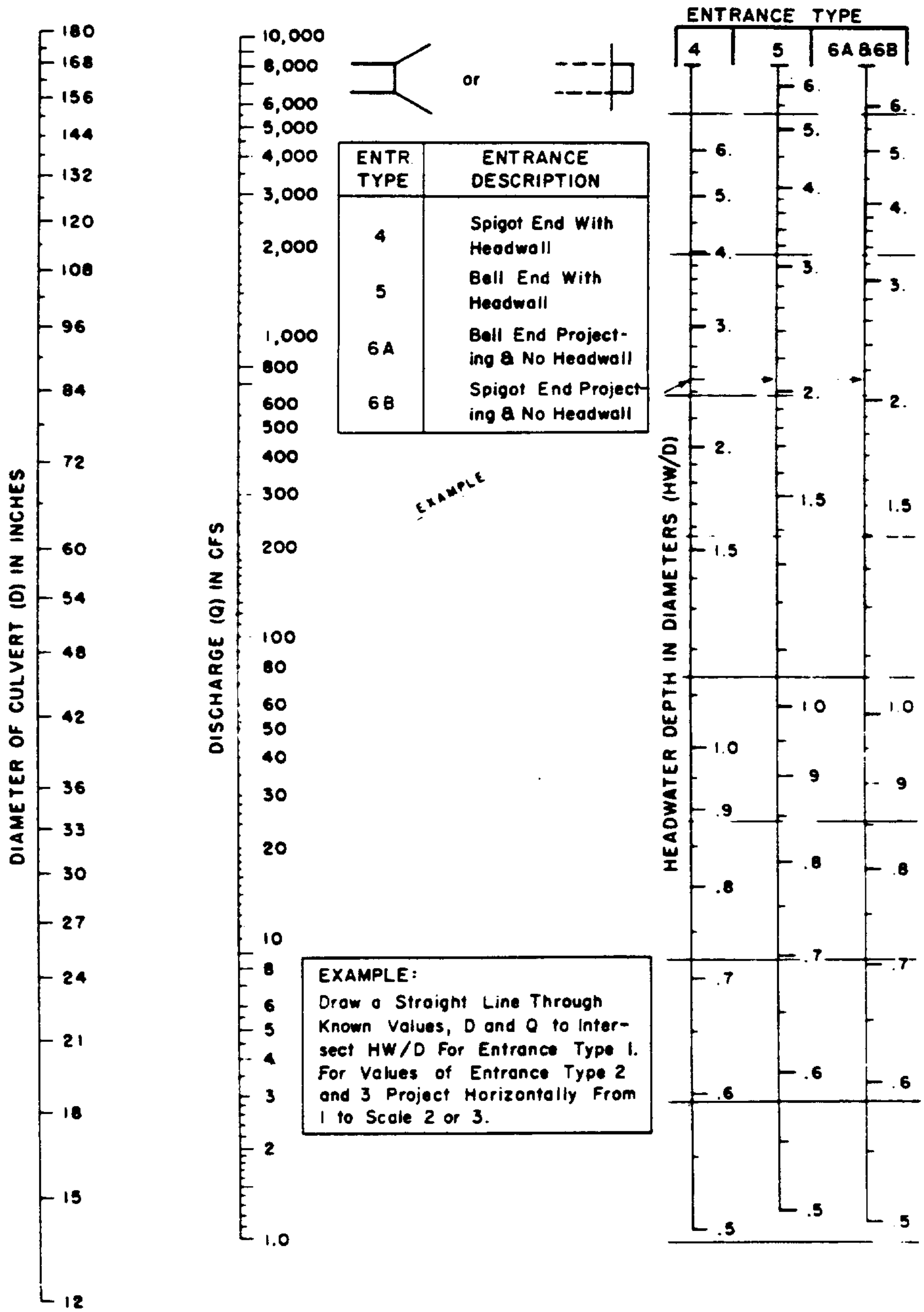
FIGURE 15



BUREAU OF PUBLIC ROADS JAN 1963

HEADWATER DEPTH FOR CONCRETE BOX CULVERT WITH INLET CONTROL

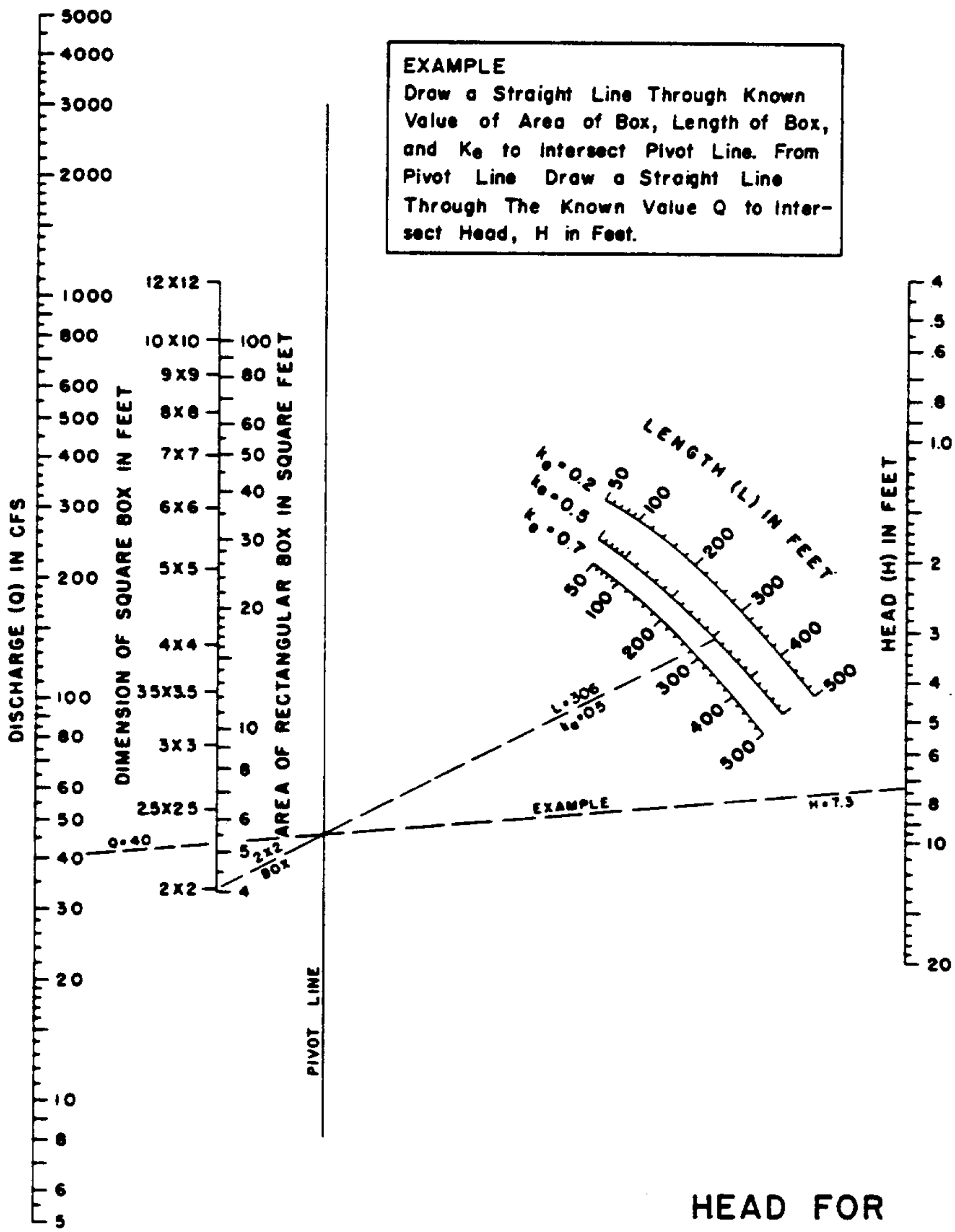
FIGURE 16



BUREAU OF PUBLIC ROADS JAN 1963

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

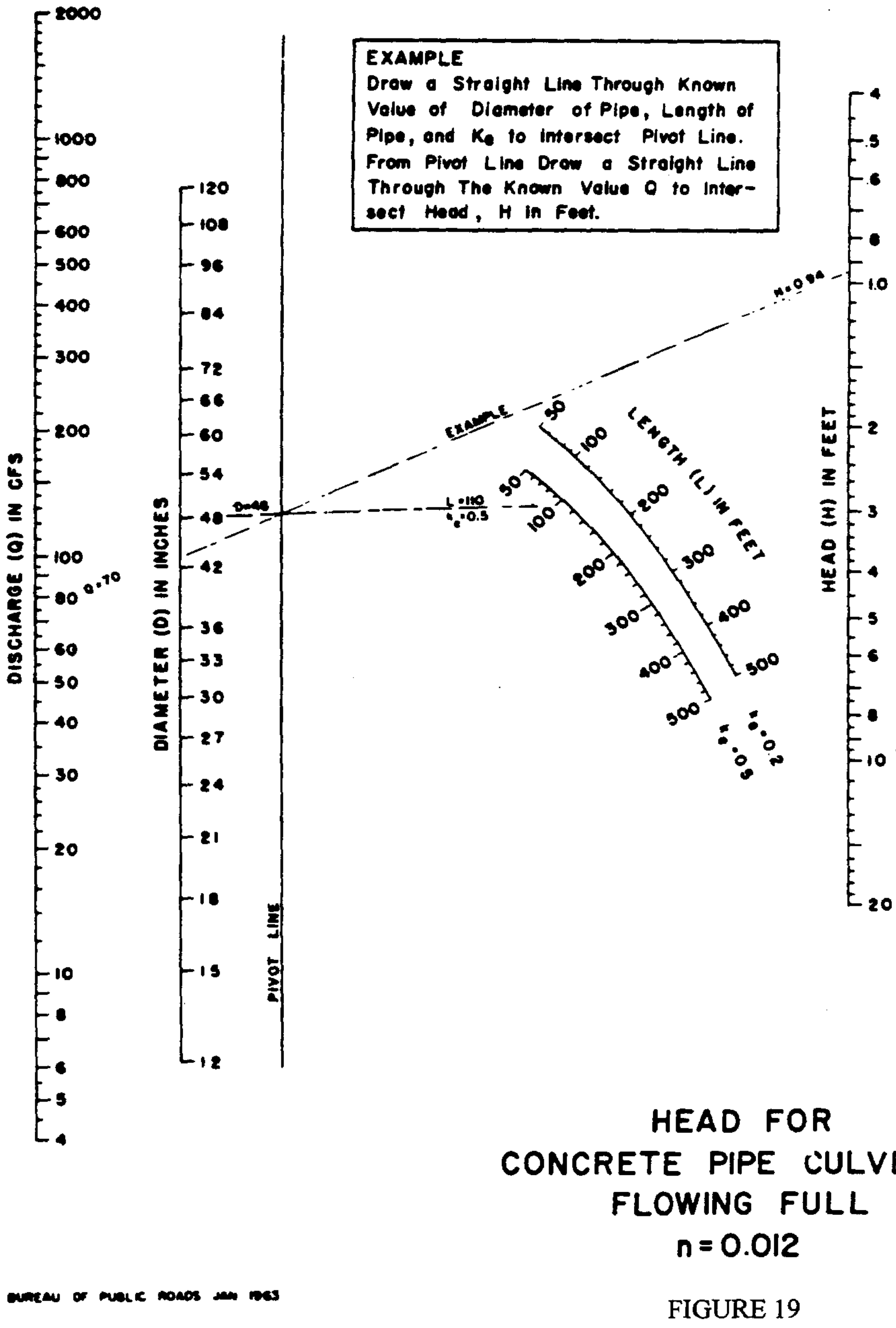
FIGURE 17



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN 1963

FIGURE 18



HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL
n = 0.012

FIGURE 19

EXAMPLE

Known:

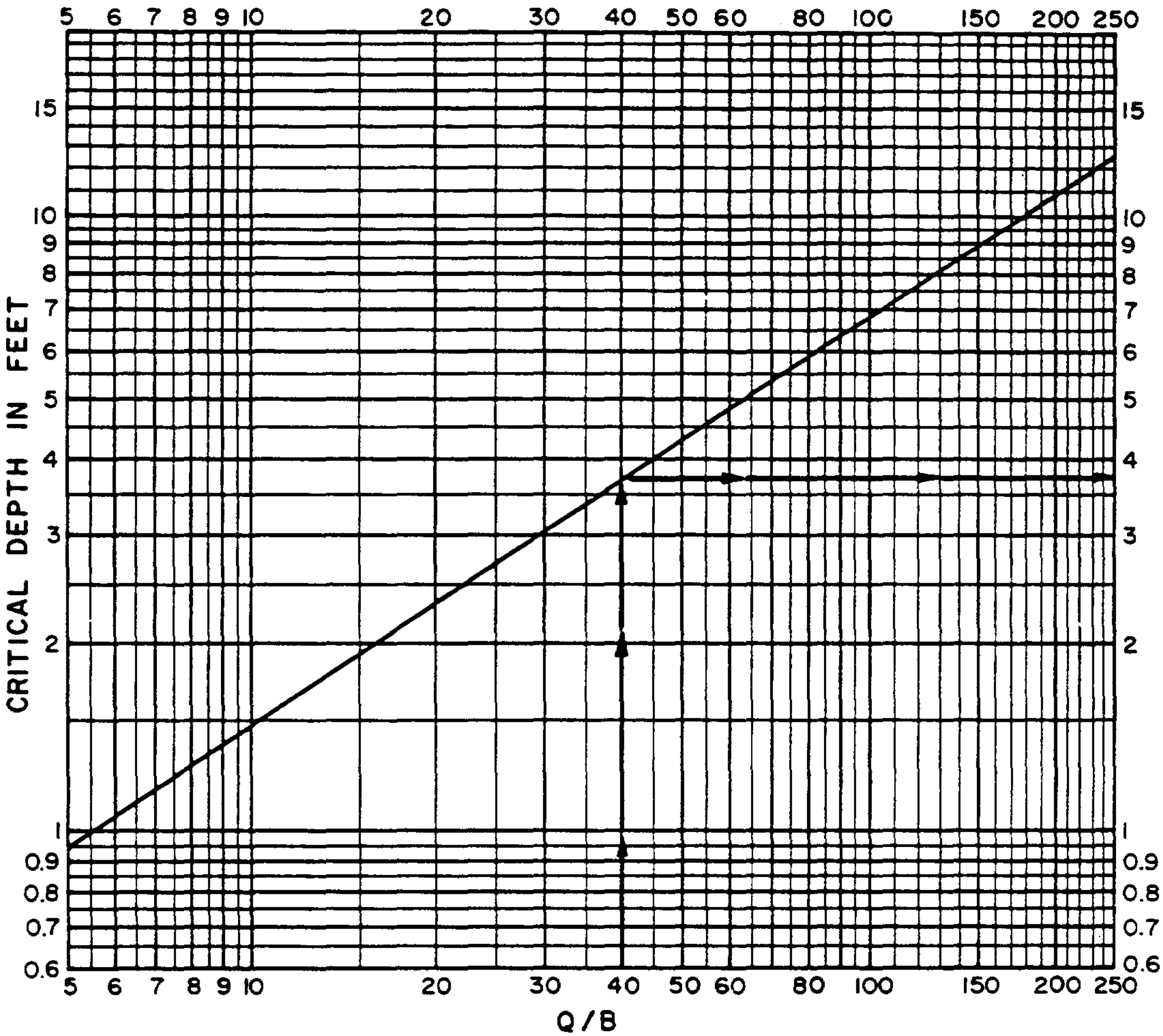
Discharge = 200 c.f.s.
Width of Conduit = 5'
Q/B = 40

Find:

Critical Depth

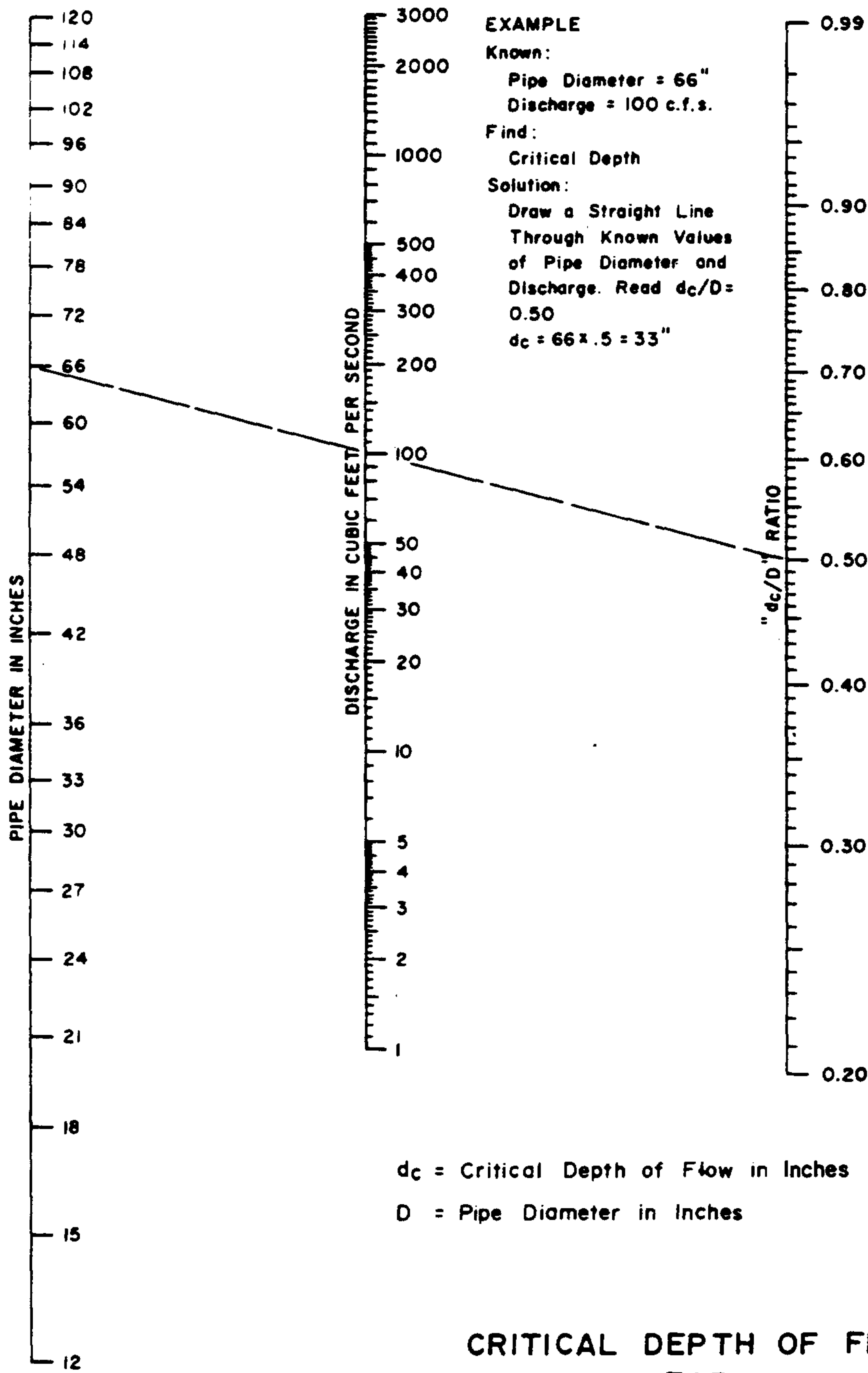
Solution:

Enter Graph at Q/B = 40
Intersect Critical Depth
at 3.7



CRITICAL DEPTH
OF FLOW FOR
RECTANGULAR CONDUITS

FIGURE 20



CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

FIGURE 21

TEXAS HIGHWAY DEPARTMENT

STORM DRAIN INLETS

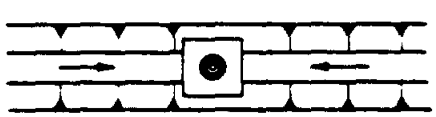
INLET TYPE	INLET DESCRIPTION	AVAIL. INLET SIZES	WHERE USED	DESIGN CURVES
I	 STANDARD CURB OPENING INLET ON GRADE	4' 6 8' 10'	26' LOCAL STREET 36' COLLECTOR STREET 40' COLLECTOR STREET ALLEYS	
IA	 STANDARD CURB OPENING INLET AT LOW POINT	4' 6' 8' 10'	26' LOCAL STREET 36' COLLECTOR STREET ALLEY	
II	 RECESSED CURB OPENING INLET ON GRADE	4' 6' 8' 10'	40' COLLECTOR STREET 2-24' MAJOR STREET 2-33' MAJOR STREET 2-36' MAJOR STREET	
IIA	 RECESSED CURB OPENING INLET AT LOW POINT	4' 6' 8' 10'	40' COLLECTOR STREET 2-24' MAJOR STREET 2-33' MAJOR STREET 2-36' MAJOR STREET	
III	 COMBINATION INLET ON GRADE	4' 6' 8'	COMBINATION INLETS TO BE USED IN ALLEYS ONLY WITH WRITTEN APPROVAL FROM CITY ENGINEER.	
IIIA	 COMBINATION INLET AT LOW POINT	4' 6' 8'	COMBINATION INLETS TO BE USED IN ALLEYS ONLY WITH WRITTEN APPROVAL FROM CITY ENGINEER.	
IV	 GRATE INLETS	2 GRATE 3 GRATE 4 GRATE 6 GRATE	GRATE INLETS TO BE USED WHERE SPACE RESTRICTIONS PROHIBIT OTHER INLET TYPES OR AT LOCATIONS WITH NO CURB. AND WITH WRITTEN APPROVAL FROM THE CITY ENGINEER	
V	 TYPE 'Y' DROP INLET	2 x 2' 3' x 3' 4' x 4'	OPEN CHANNELS	

FIGURE 22

SECTION 9

SECTION 9

DESIGN INSTRUCTIONS AND FORMS

OPEN CHANNEL CALCULATIONS FORM

Column 1	Downstream limit of the section of channel under consideration.
Column 2	Upstream limit of the section of channel under consideration.
Column 3	Type of channel as shown in Figure 3 is entered here.
Column 4	Flow in the section of channel under consideration.
Column 5	Roughness coefficient of the channel cross-section taken from Table 4.
Column 6	Slope of the channel which is most often parallel to the slope of the hydraulic gradient.
Column 7	Square root of Column 6.
Column 8	Calculation is made using the values in Columns 4, 5 and 7.
Column 9	Assumed width of the bottom width of the channel.
Column 10	Assumed depth of flow.
Column 11	Assumed slope of the sides of the channel.
Column 12	Areas of flow which is calculated based on Columns 9, 10 and 11.
Column 13	Wetted perimeter calculated from Columns 9, 10 and 11.
Column 14	Value is calculated from the Columns 12 and 13.
Column 15	Column 14 raised to the 2/3 power.
Column 16	Product of Column 13 times Column 15. When the value of Column 16 equals the value of Column 8, the channel has been adequately sized. When the value of Column 16 exceeds the value of Column 8 by more than 5%, then the channel width or depth should be decreased and another trial section analyzed.
Column 17	Calculation is based on the values of Columns 4 and 12.
Column 18	Calculation is based on Column 17.
Column 19	Remarks concerning the channel section analyzed may be entered.

NOTE: This form should not be used to calculate stream profiles.

OPEN CHANNEL CALCULATIONS

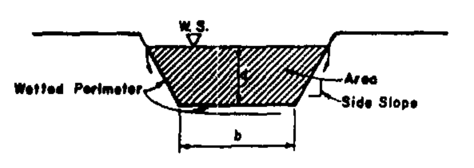
BY ________________

DATE ________________

____

____

OPEN CHANNEL ________________



CHANNEL STATION		Channel Type	Flow "Q" (c.f.s.)	Roughness Coeff. "n"	Slope "S" (ft./ft.)	"$S^{1/2}$"	$\frac{Q \times n}{1.486 \times S^{1/2}}$	Width "b" (feet)	Depth "d" (feet)	Side Slope	Area "A" (sq. ft.)	Wetted Perimeter "WP" (feet)	Hydraulic Radius "R" = $\frac{A}{WP}$ (feet)	$R^{2/3}$	$A \times R^{2/3}$	Velocity $V = \frac{Q}{A}$ (f.p.s.)	Velocity Head $\frac{V^2}{2g}$ (ft.)	REMARKS
From	To																	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

HYDRAULIC DESIGN OF CULVERTS

INFORMATION ON UPPER RIGHT OF FORM:

Culvert Location

This is a word description of the physical location.

Length

The actual length of the culvert.

Total Discharge, QT

This is the flow computed on the Storm Water Runoff Form.

Design Storm Frequency

Obtained from Table 1 and used on the Storm Water Runoff Form.

Roughness Coefficient, n

Obtained from Table 2.

Maximum Velocity

Obtained from Table 3.

Tailwater

This is the design depth of water in the downstream channel and is obtained in connection with the channel design performed on the Open Channel Calculations Form or by water surface profile calculations.

D. S. Channel Width

This is the bottom width of the downstream channel. The culvert should be approximately this width whenever possible.

Entrance Description

This is a listing of the actual condition as shown in the "Culvert Entrance Data" shown on the calculation sheet.

Roadway Elevation

The elevation of the top of curb at the upstream end of culvert.

U. S. Culvert F. L.

The flow line of the culvert at the upstream end.

Difference

The difference in elevations of the roadway and the upstream flow line.

Required Freeboard

The vertical distance required for safety between the upstream design water surface and the roadway elevation or such other requirements which may occur because of particular physical conditions.

Allowable Headwater

This is obtained by subtracting the freeboard from the difference shown immediately above.

D. S. Culvert F. L.

The flow line elevation of the downstream end of the culvert.

Culvert Slope, So

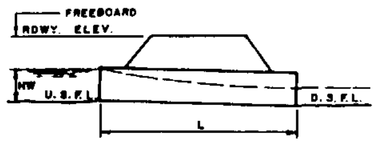
This is the physical slope of the structure calculated as indicated

Columns 1-10	Deal with selection of trial culvert size and are explained as follows:
Column 1	Total design discharge, Q, passing through the culvert divided by the allowable maximum velocity gives trial total area of culvert opening.

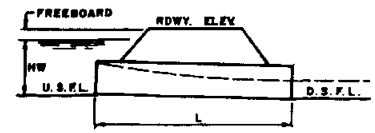
Column 2	Culvert width should be reasonably close to the channel bottom width, W, downstream of the culvert.
Column 3	Lower range for choosing culvert depth is trial area of culvert opening, Column 1, divided by channel width, Column 2.
Column 4	Allowable headwater obtained from upper right of sheet.
Column 5	Trial depth, D, of culvert corresponding to available standard sizes and between the numerical values of Columns 3 and 4.
Columns 6 - 8	Are solved simultaneously based on providing a total area equivalent to the trial area of opening in Column 1.
Column 6	Number of culvert openings.
Column 7	Inside width of one opening.
Column 8	Inside depth of one opening if culvert is box structure or diameter if culvert is pipe.
Column 9	Column 6 multiplied by Column 7 and Column 8.
Columns 11-15	(Inlet Control) and 16 - 27 (Outlet Control) deal with Headwater Calculations which verify hydraulics of trial culvert selected and are explained as follows:
Column 11	Obtained from the upper right of sheet.
Column 12	When the allowable headwater is equal to or less than the value in Column 8, enter Case I. When the allowable headwater is more than the value in Column 8, enter Case II.
Column 13	Column 10 divided by Column 7.
Column 14	Obtained from Figure 16 for box culverts or Figure 17 for pipe culverts.
Column 15	Column 14 multiplied by column 8.
Column 16	Obtained from upper right of sheet.
Column 17	Obtained from Figure 18 for box culverts and Figure 19 for pipe culverts.
Column 18	Tailwater depth from upper right of sheet.
Column 19	So, culvert slope, multiplied by culvert length, both obtained from upper right of sheet.
Column 20	Sum of Columns 17 and 18 minus Column 19.
Column 21	Obtained from Figure 18 for box culverts and Figure 19 for pipe culverts.

Column 22	Critical depth obtained from Figure 20 for box culverts and Figure 21 for pipe culverts.
Column 23	Sum of Columns 22 and 8 divided by two.
Column 24	Tailwater depth from the upper right of sheet.
Column 25	Enter the larger of the two values shown in Column 23 or Column 24.
Column 26	Previously calculated in Column 19 and may be transposed.
Column 27	The sum of Columns 21 and 25 minus Column 26.
Column 28	Enter the larger of the values from either Column 15, 20 or 27. This determines the controlling hydraulic conditions of the particular size culvert investigated.
Column 29	When the Engineer is satisfied with the hydraulic investigations various culverts and has determined which would be the most economical selection. This description should be entered.

INLET CONTROL

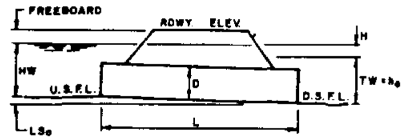


CASE I
INLET NOT SUBMERGED

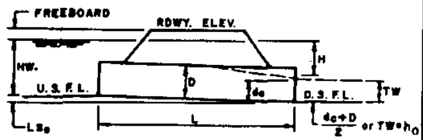


CASE II
INLET SUBMERGED

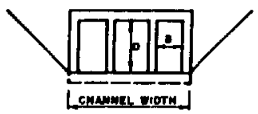
OUTLET CONTROL



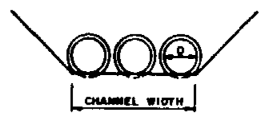
CASE III
OUTLET SUBMERGED



CASE IV
OUTLET NOT SUBMERGED



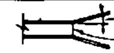
TYPICAL BOX CULVERT



TYPICAL PIPE CULVERT

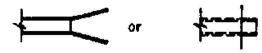
CULVERT ENTRANCE DATA

CONCRETE BOX CULVERT



TYPE	FLARE ANGLE	ENTRANCE EDGE	K_e
1A	30° to 75°	Square	0.4
1B	30° to 75°	Round	0.3
2A	15° to 30° & 75° to 90°	Square	0.5
2B	15° to 30° & 75° to 90°	Round	0.3
3A	0° (Extension of Sides)	Square	0.7
3B	0° (Extension of Sides)	Round	0.5

CONCRETE PIPE



TYPE	ENTRANCE DESCRIPTION	K_e
4	Spigot End With Headwall	0.5
5	Bell End With Headwall	0.2
6A	Bell End Projecting With No Headwall	0.3
6B	Spigot End Projecting With No Headwall	0.6

CULVERT DESIGN CALCULATIONS

____

CULVERT LOCATION ______________________

______________________ LENGTH, L ______________

TOTAL DISCHARGE, Q ________ DESIGN STORM FREQ. ________

TOTAL DISCHARGE + FACTOR OF SAFTEY OF 25% Q_{Design} = ____

ROUGHNESS COEFF., n ____________ MAX. VEL. ________

TAILWATER ______________ D. S. CHANNEL WIDTH ________

ENTRANCE DESCRIPTION ______________________

RDWY. ELEV. ____________ U.S. CULV. F.L. ____________

U.S. CULV. F.L. ____________ D.S. CULV. F.L. ____________

DIFFERENCE ____________ DIFFERENCE ____________

REQ'D. FREEBOARD ________ FT. CULV. SLOPE, $S_o = \frac{\text{DIFF. FT.}}{\text{LENGTH FT.}}$

ALLOW. HEADWATER ________ FT. S_o = ________

TRIAL CULVERT: Trial Area of Opening $T{\cdot}A_c = \frac{Q_{Design}}{V_{max}}$ (sq. ft.)	Channel Width "W" (feet)	DEPTH RANGE D. R.: $\frac{T{\cdot}A_c}{W}$ (feet)	DEPTH RANGE D. R.: AHW (feet)	Try Depth "D" (feet)	POSSIBLE CULVERT SIZES: No. Openings	POSSIBLE CULVERT SIZES: Width of Box "B" (feet)	POSSIBLE CULVERT SIZES: Box Depth or Pipe Dia. "D" (feet)	POSSIBLE CULVERT SIZES: Total Culvert Area "Ac" (sq. ft.)	POSSIBLE CULVERT SIZES: "Q" (Design) Each Opening (c.f.s.)	HEADWATER CALCULATION — INLET CONTROL (See Figure 25 & 26): Entrance Type	Case No.	$\frac{Q_{Design}}{B}$ (c.f.s.)	$\frac{HW}{D}$	HW	OUTLET CONTROL (See Figure 27, 28, 29, & 30): Entrance Coeff. K_e	CASE III $HW = H + TW - L \times S_o$ (feet): "H" (feet)	"TW" (feet)	$L \times S_o$ (feet)	"HW" (feet)	CASE IV $HW = H + h_o - L \times S_o$ (feet): "H" (feet)	$h_o = \frac{d_c+D}{2}$ or h_o = TW (use larger): d_c (feet)	$\frac{d_c+D}{2}$ (feet)	TW (feet)	h_o (feet)	$L \times S_o$ (feet)	"HW" (feet)	The Greater Controlling Head Water (Inlet or Outlet) (feet)	SELECTED CONDUIT SIZE
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29

Q_{Design} = CALCULATED FLOW Q + FACTOR OF SAFTEY OF 25%. $Q_{Design} = Q \times 1.25$

STORM SEWER CALCULATIONS FORM

Column 1	Upstream station of the section of conduit being designed. Normally, this would be the point of a change in quantity of flow, such as inlet or a change in grade.
Column 2	Downstream station of the section of conduit being designed.
Column 3	Distance in feet between the upstream and downstream stations.
Column 4	Drainage sub-area designation from which flow enters the conduit at the upstream station.
Column 5	Area in acres of the drainage sub-area entering the conduit.
Column 6	Runoff coefficient, obtained from Table 1, based on the characteristics of the subdrainage area.
Column 7	Column 5 multiplied by Column 6.
Column 8	Obtained by adding the value shown in Column 7 to the value shown immediately above in Column 8.
Column 9	This time in minutes is transposed from Column 19 on the previous line of calculations. The original time shall be equal to the time of concentration as shown on Table 1.
Column 10	Design Storm Frequency.
Column 11	Using the time at the upstream station shown in Column 10, this value is taken from Figure 1.
Column 12	Column 8 multiplied by Column 11.
Column 13	This slope should be computed from the profile of the ground surface. Normally, the hydraulic gradient will have a slope approximately the same as the proposed conduit and will be located above the inside crown of the conduit.
Column 14	Utilizing the values in Column 12 and 13, a conduit size should be selected. In the case of concrete pipe, Figure 15 may be used.
Column 15	Velocity in the selected conduit based on the values in Columns 12, 13 and 15. Taken from Figure 15 for concrete pipe.
Column 16	Friction head loss is the product of Column 3 times Column 13.

Column 17 Calculation is made utilizing the values of Columns 15 and 16.

Column 18 Calculation is based on the values of Column 3 and 15.

Column 19 Sum of Columns 9 and 18.

Column 20 Special design comments may be entered here.

STORM SEWER CALCULATIONS

BY ____________________

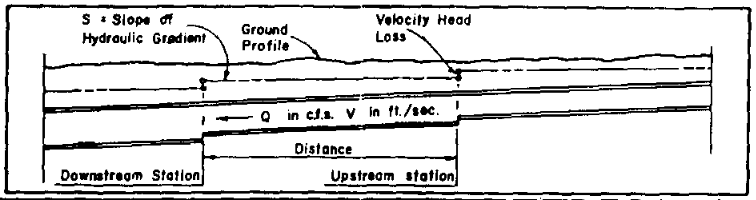
DATE ____________________

____

____

STORM SEWER LINE ____________________

INITIAL INLET TIME _____ MINUTES



RUNOFF COLLECTION POINT (Inlet or Manhole)		Distance Between Collection Points	INCREMENTAL DRAINAGE AREA				Accumulated "CA"	Time at Upstream Station (minutes)	Design Storm Frequency (yrs.)	Intensity "I" (inches/hr.)	Storm Water Runoff "Q" (c.f.s.)	Slope of Hydraulic Gradient "S" (ft./ft.)	Selected Storm Sewer Size	Velocity in Sewer Between Collection Points "V" (f.p.s.)	Head Loss Coeff. K_j	Velocity Head Loss at Upstream Station $K_j \frac{V^2}{2g}$ (feet)	Flow Time in Sewer $\frac{\text{Distance}}{V \times 60}$ (minutes)	Time at Downstream Station (minutes)	REMARKS
UPSTREAM STATION	DOWNSTREAM STATION		Area No.	Drainage Area "A" (Acres)	Runoff Coeff. "C"	Incremental "CA"													
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

INLET DESIGN CALCULATIONS FORM

Column 1	Inlet number or designation . The first inlet shown is the most upstream.
Column 2	Construction plan station of the inlet.
Column 3	Design Storm Frequency is the same as the Design Storm Frequency of the sewer.
Column 4	Time of concentration for each inlet is taken from Table 1.
Column 5	Using the time of concentration and the Design Storm Frequency, rainfall intensity is taken from Figure 1.
Column 6	Runoff Coefficient is taken from Table 1 according to the zoning of the drainage area.
Column 7	Area drained by the specific inlet. Care should be taken to keep the drainage area flow separate into the appropriate street gutters.
Column 8	Product of Column 5 multiplied by Column 6 and 7.
Column 9	If there is any flow which was not fully intercepted by an upstream inlet, it should be entered here.
Column 10	Sum of Columns 8 and 9.
Column 11	Capacity of the street in which the inlet is located, from either Figures 7, 8 or 9. If the total gutter flow shown in Column 10 is in excess of the value in Column 11, the inlet should be moved upstream. If it is substantially less than the value in Column 11, an investigation should be made to see if the inlet can be moved downstream.
Column 12	Street gutter slope to be used in selecting the proper size inlet.
Column 13	Crown type of the street on which the inlet is located.
Column 14	Selected size of the inlet taken from Figures 10-16.
Column 15	Inlet type taken from Figure 22.
Column 16	If the selected inlet does not intercept all of the gutter flow, the difference between the two values should be entered here and in Column 9 of the inlet which will intercept the flow.

INLET DE[illegible]GN CALCULATIONS

BY ____________________

DATE ____________________

INLET		Design Storm Frequency (yrs.)	AREA RUNOFF Q = CIA					Carry-Over From Upstream Inlet (c.f.s.)	Total Gutter Flow (c.f.s.)	Gutter Capacity (c.f.s.)	Gutter Slope (ft./100 ft.)	Crown Type	SELECTED INLET		Carry-Over To Downstream Inlet (c.f.s.)
No.	Location		Time Of Conc. (min.)	Intensity I (In./hr.)	Runoff Coeff. "C"	Area (Ac.)	"Q" (c.f.s.)						Length "Li" (Feet)	Type	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

SECTION 10

SECTION 10

REFERENCES

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