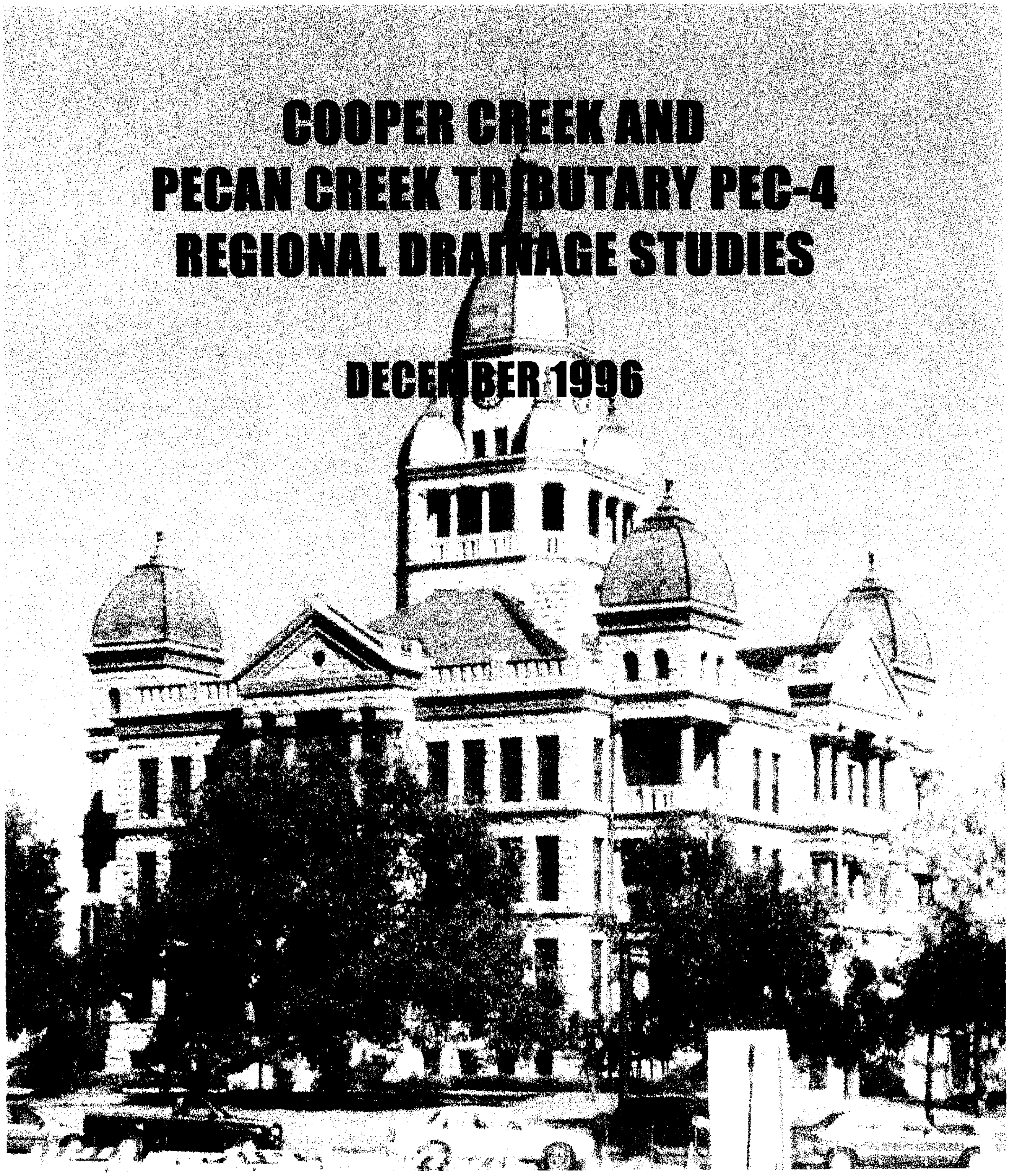
COOPER CREEK AND PECAN CREEK TRIBUTARY PEC-4 REGIONAL DRAINAGE STUDIES

DECEMBER 1996



CITY of DENTON, TEXAS

FREESE • NICHOLS

COOPER CREEK AND PECAN CREEK TRIBUTARY PEC-4 REGIONAL DRAINAGE STUDIES

DECEMBER 1996

STATE OF TEXAS
MICHAEL J. DELLIES
65938
REGISTERED
PROFESSIONAL ENGINEER

12/17/96

CITY of DENTON, TEXAS

FREESE • NICHOLS

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Executive Summary

The purpose of the Cooper Creek Regional Detention and Tributary PEC-4 Channel Improvement Studies is to determine an effective approach to managing the 100-year floodplain. As a result of continued development within the Cooper and Pecan Creek watersheds, the 100-year floodplain has exceed the stream banks along Cooper Creek and Tributary PEC-4, both of which flow through the City of Denton.

In the case of the Cooper Creek Regional Detention Study, several possible sites for detention ponds were evaluated for their effectiveness in reducing the 100-year floodplain. Thirteen scenarios of individual pond construction and combination of ponds were evaluated to determine an effective combination of pond options. Approximate floodplain width reductions were used to estimate the benefit of implementing the regional detention program. The benefit was compared to the cost of each scenario and a recommendation was made regarding the scenario that should be implemented by the City. As a result of the evaluation, detention ponds are recommended at two sites west of Locust Street (FM 2164), one on the main stem of Cooper Creek and the other on Tributary C-6. The anticipated construction cost of the recommended detention pond construction is approximately $3,296,900. The resulting reduction in the 100-year floodplain width is expected to provide a benefit of approximately $926,500 in increase property value by reducing the Cooper Creek floodplain by nearly 65 acres. Since the recommended regional detention ponds evaluated in this study do not reduce the floodplain to within the channel banks, an additional study is recommended to determine the extent of improvements to the Cooper Creek channel and culverts to maintain the floodplain within the channel area.

In the case of the Tributary PEC-4 Channel Improvement Study, several iterations were made to determine the combination of channel and culvert improvements that are needed to reduce the 100-year floodplain such that flow diversions are eliminated and flow is maintained within the channel banks. The recommended improvements consist primarily of a concrete channel with a 20-foot bottom width and 2:1 sideslopes. Improvements to each street crossing were made to eliminate backwater conditions caused by the existing culverts. The cost of the recommended channel and culvert improvements is $3,735,500. Future design of the improvement should consider alternatives to concrete lining, such as gabions, segmental retaining walls and interlocking slope pavers to enhance the aesthetics of the channel. Since improvements to the culvert at the M.K.T. Railroad will require a parallel track to prevent disruption in rail service, a bypass option was evaluated at Robertson Street to investigate the possibility of reducing the cost of culvert improvements at the M.K.T. Railroad. The bypass option is expected to cost approximately $597,200, which is nearly $440,000 less than replacing the existing culverts beneath the railroad.

PART I - INTRODUCTION

Purpose of the Regional Drainage Studies

The City of Denton is like many cities across the country that have shared the common experience of flooding. As the Denton continues to grow, the uncontrolled runoff generated by the additional development places a burden on the existing streams within the city, resulting in a serious threat to the health and well-being of the community. It is the goal of the two studies summarized herein to provide the City of Denton with an overall approach to floodplain management along Cooper Creek and Pecan Creek Tributary PEC-4. It is through floodplain management that the City of Denton can plan to mitigate existing and future flood losses and implement an aggressive capital improvements program to achieve the flood protection it desires.

Floodplain Definition

Most of the time, the streams within the City of Denton are tranquil and the flow of water is confined to the stream channel. Occasionally the right combination of rainfall and antecedent soil moisture results in more stormwater than the stream channel can carry. When the stream channel is too small to contain all of the stormwater flow, the water overflows the stream banks onto the adjacent floodplain area. The floodplain is as much a part of the stream as its channel and management practices must be in place to protect the floodplain and provide adequate conveyance of the stormwater without causing flood damage.

It is not economically justifiable to contain the largest flood that could occur within the banks of the stream channel. Therefore, the City of Denton has adopted the criteria of containing the 100-year flood as its acceptable level of flood protection. By definition, the 100-year flood has a one percent chance of being equaled or exceeded in any given year. The 100-year floodplain has become the standard level of floodplain management throughout the country and is used by the Federal Emergency Management Agency (FEMA) to establish flood insurance rates.

Effect of Development on Floodplain

As development occurs, the watershed typically undergoes a transformation from grassed and/or wooded areas that intercept rainfall through infiltration to areas covered with buildings and pavement that shed the rainfall into drainage collection systems that discharge into the city's streams. The impervious cover that accompanies development produces greater volumes of runoff and places a burden on the existing streams.

To accommodate the additional stormwater flow, the stream channel will naturally tend to widen through erosion. However, due to the attractiveness of the original stream channel, development generally occurs along the channel, especially residential development. In these areas, the stream is often prohibited from the natural expansion process by some form of channel bank lining. Therefore, since the stream cannot expand to accommodate the additional stormwater flow, the floodplain width and or elevation increases.

Floodplain Management Plan Objective

By keeping the flood waters within the stream channel areas and reclaiming the floodplain area for development, the risk of potential property loss is reduced significantly.

There is a common misconception among the general public that floodplain management is designed to protect the floodplains.[1] On the contrary, floodplain management is designed to protect people by reducing the pain, suffering and economic loss which accompanies flood disasters.

There are several areas within the Cooper Creek and Tributary PEC-4 watersheds where the current 100-year floodplain covers residential and commercial property and the owners of these properties are required to obtain flood insurance. In some cases, vacant property cannot be developed because it is within the floodplain limits. The objective of the regional drainage studies is to determine the impact of future development within the watershed and the improvements needed to reclaim some of these floodplain areas. Therefore, alternatives will be presented that reduce the width of the 100-year floodplain in an attempt to maintain the 100-year flood waters within the stream channel. In the case of the Cooper Creek detention pond evaluation, this may not be possible without additional improvements to the Cooper Creek channel.

A risk-based evaluation of the flood control benefits provided by the mitigation alternatives was performed. The benefits of a flood control alternative were estimated as the difference in damage at the site with and without the flood control improvements in place. The damage is a function of the hydrologic response of the watershed to rainfall input and the value of property subject to resulting flooding.[2]

The floodplain management scenarios presented in this study are not intended for direct application to detailed design, but are intended to represent a conceptual design solution. The conceptual designs are intended to provide guidance to the City of Denton in making decisions regarding future implementation of flood control improvements in these areas. Prior to preparing construction documents for these improvements, detailed designs of the improvements should be prepared to include conditions or concerns not considered in this study but which may influence the final design.

Report Organization

The evaluation of Cooper Creek and Tributary PEC-4 were performed independently and therefore, are discussed separately in this report. Part I describes the analysis of a possible Cooper Creek detention pond system and Part II discusses the evaluation of the Tributary PEC-4 channel and culvert improvements.

PART II - COOPER CREEK REGIONAL DETENTION STUDY

1.0 BACKGROUND

The focus of the Cooper Creek detention pond study was the reduction of the 100-year floodplain width upstream of Mingo Road through implementation of a system of regional detention ponds within the watershed. The purpose of the detention pond study is two-fold: (1) evaluate possible combination of constructing detention ponds within the Cooper Creek drainage area in order to reduce the Cooper Creek floodplain width, and (2) develop a comparison of detention pond construction cost data and the value of property reclaimed from the 100-year floodplain.

This Cooper Creek watershed upstream of Mingo Road is currently developed residential (mostly single-family) with a trapezoidal, unlined earthen channel. There are several culvert crossings that have limited capacity and cause backwater conditions within the stream channel. The 100-year floodplain generally extends beyond the stream banks and into the residential yards. Furthermore, there are significant areas in the upstream reaches of Cooper Creek and its tributaries that are presently undeveloped and future development of these areas may worsen the backwater problems, causing additional flooding along Cooper Creek.

2.0 RECONNAISSANCE AND DATA COLLECTION

Coordinate with City Staff

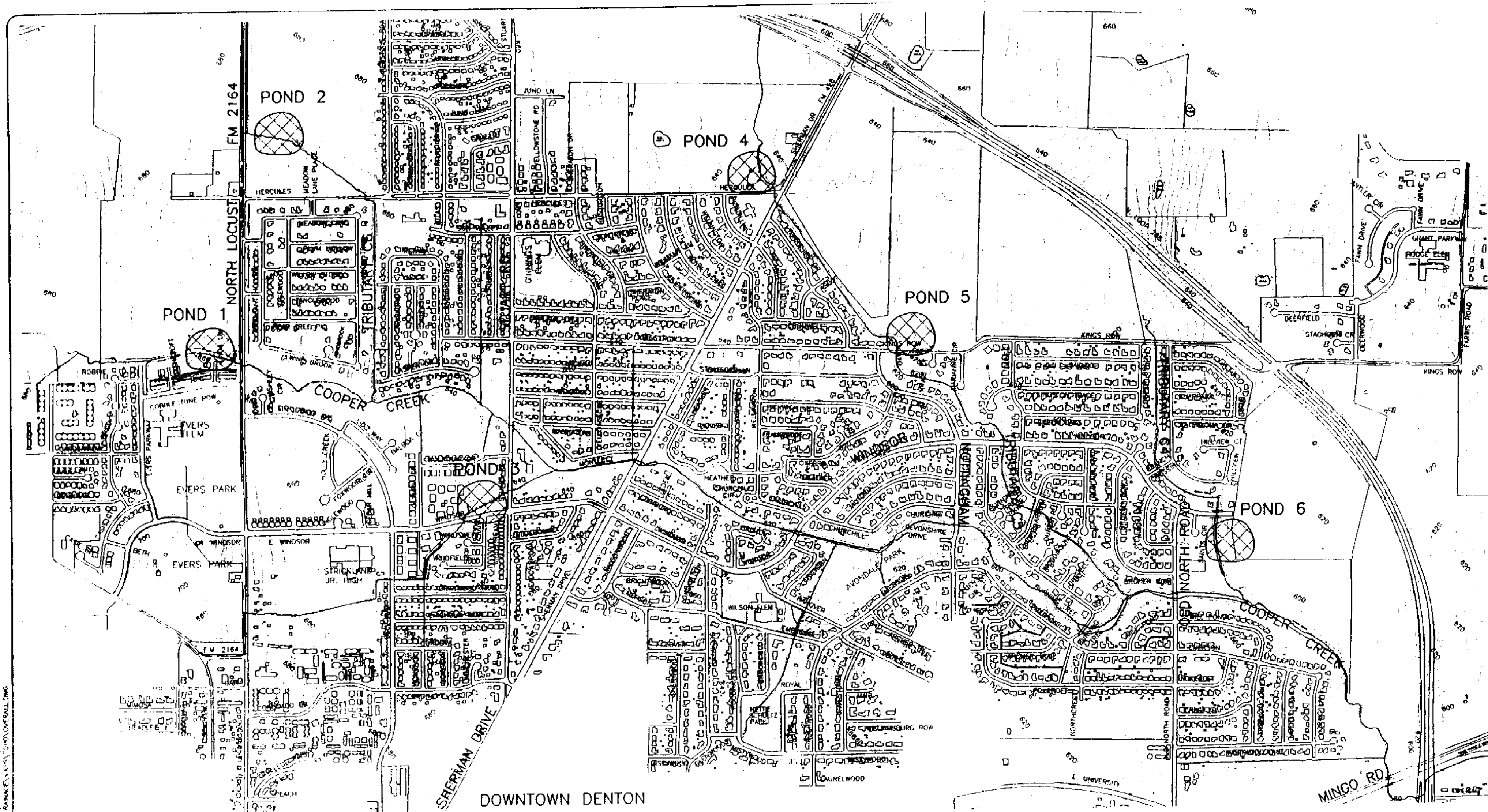
At the beginning of the project, a coordination meeting was held at the City of Denton engineering office to discuss the scope of the detention pond evaluation. The project team discussed specific design issues with the City staff, including the possible location of the detention pond sites. In addition to the four pond sites listed in the engineering services agreement, two additional sites were agreed upon. Information required for the detention pond evaluation was reviewed. The locations of the detention pond sites are listed in Table II-1 and are shown in Figure II-1.

Table II-1 Detention Pond Locations

Pond	Location
#1	West of F.M. 2164, just north of the Cobblestone Townhomes
#2	North of Hercules Street and east of F.M. 2164, near the confluence of two Cooper Creek tributaries
#3	North of Windsor Street, east of Stuart Street, south of Wolftrap Drive, and east of the Windsor Village Apartments
#4	East of Sherman Drive and north of Hercules Street
#5	North of Kings Row opposite Marianne Circle
#6	South of Windsor Street, east of Old North Road, north of the Mormon Church and west of the vacant EDS property

Site Investigation

At a coordination meeting with city staff, six sites were selected for evaluation to determine if construction of detention ponds would provide sufficient storage to reduce the



CITY OF DENTON

COOPER CREEK

REGIONAL DETENTON STUDY

DETENTION POND LOCATIONS

FREESE-NICHOLS

FIGURE II–1

Cooper Creek 100-year floodplain. Figure II-2 contains photographs of the proposed detention pond sites. The sites were selected in currently undeveloped areas along tributaries of Cooper Creek. One site was selected on the main stem of Cooper Creek in a location that has a contributing drainage area comparable to the remaining five sites.

The investigation phase of the project began with a visit to the detention pond sites to observe the existing conditions at each site, which are located in undeveloped, or pasture land areas. In general, all of the detention pond sites were found to be in relatively good shape and no obvious causes for concern were found.

Computer Models

Since an existing HEC-1 model of the Cooper Creek watershed was unavailable, the hydrologic model obtained for Cooper Creek was in the SWFHYD format. SWFHYD, or Southwest Fort Worth Hydrology[3], is a PC-based hydrograph development program developed by the Fort Worth District of the Corps. For this study, the SWFHYD model was converted to the HEC-1[4] format, since HEC-1 is more widely used and accepted by the Federal Emergency Management Agency.

The results of the Corps' SWFHYD model and the new HEC-1 model were compared. The only major difference between the two input data sets is that the SWFHYD model included 5 basins in the study area, whereas there were 17 subbasins in the new HEC-1 model. There were two major differences in the results between the SWFHYD results were observed: (a) the peak discharge in the updated HEC-1 model occurs sooner and (b) the peak discharges were usually larger. Table II-2 contains a comparison of the peak discharges generated with the models.

Table II-2 Comparison of the SWFHYD and 1 Models

Location	1985 Development Conditions				1996 Development	
	SWFHYD Model		HEC-1 Conversion of SWFHYD		HEC-1 Used in Current Study	
	Peak Discharge (cfs)	Time (hrs)	Peak Discharge (cfs)	Time (hrs)	Peak Discharge (cfs)	Time (hrs)
Below Trib C-6 (J-5)	3,211	16.50	3,292	12.67	3,126	12.50
Sherman Drive	5,083	16.25	5,222	12.33	N/A	N/A
Below Trib C-5 (J-2)	8,247	16.50	8,454	12.58	8,732	12.50
Below Trib C-4 (J-1)	10,534	16.50	10,929	12.58	10,842	12.58

The computations methods of SWFHYD and HEC-1 were reviewed. A comparison of the routing parameters between the SWFHYD model and the HEC-1 model used in this study revealed that the SWFHYD model assumes more storage in the stream reaches than the HEC-1 model. Replacing the storage/discharge tables in the HEC-1 conversion of the SWFHYD model with the ones used for this study gave peak discharge values at common design points within 2% of the model used for this study.

The hydraulic computer model was obtained from the Corps of Engineers for Cooper Creek in the HEC-2[5] format and was used to develop storage-discharge relationships for use in hydrograph routing and to evaluate the 100-year floodplain profile.

Existing Utilities

The City staff was contacted regarding utilities at the six detention pond sites. Maps were obtained from the City showing the approximate locations of water and sanitary sewer lines in the vicinity of the pond sites. In general, the city-owned utilities are within

Figure II - 2a -- Detention Pond Site #1

Figure II - 2b -- Detention Pond Site #2

Figure II - 2c -- Detention Pond Site #3

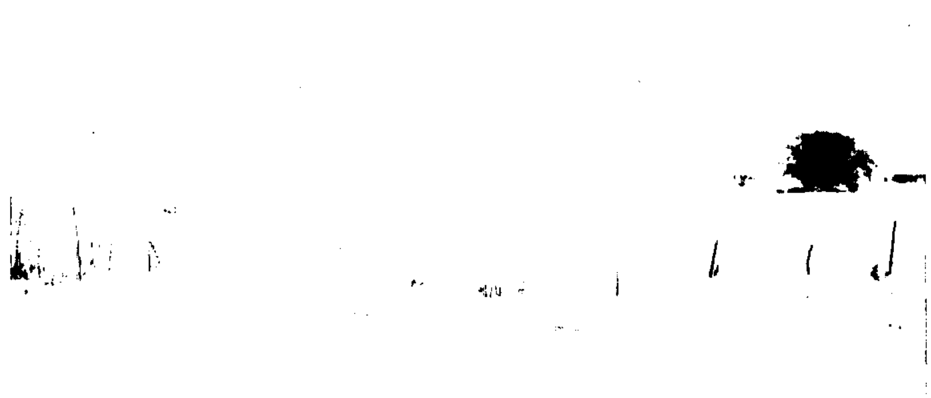


Figure II - 2d -- Detention Pond Site #4

Figure II - 2e -- Detention Pond Site #5

Figure II - 2f -- Detention Pond Site #6 the existing street rights-of-way. Existing utilities that should be considered during future detailed design of the detention ponds include:

- There is an existing 10-inch sanitary sewer along the north side of the creek at pond site #1 - future pond construction may require relocating the sewer line to the other side of the creek or constructing the detention embankment north of the sewer line.
- There is an existing 10-inch sanitary sewer crossing pond site #3 - future pond construction may require relocating the sewer line such that it parallels Windsor and Stuart Streets.

Available Base Mapping

A base map was obtained from the City staff for the Cooper Creek watershed upstream of Mingo Road. The approximate drainage area boundaries were delineated on the base map by the City staff using the City's GIS software. These boundaries were used to help delineate the contributing drainage areas for the detention ponds. The extent of existing development is also shown on the base map.

Field Surveys

Field surveys of the project sites were performed by Walker and Associates Surveying, Inc. to obtain the data necessary for preliminary sizing of the Cooper Creek detention ponds. The field data were collected using the global positioning system (GPS). The topography covered by the field surveys included an area approximately 800-feet square, which is approximately equal to fifteen acres. The fifteen acre limit was arbitrarily chosen at the beginning of the project as a typical detention pond size for an urban setting.

The field data were used to develop base maps for each of the project areas. One-foot contours were generated and included on the topographic maps. Permanent objects, such as buildings, trees, fences, streets, etc. were also included in the field surveys and located on the base maps.

Geotechnical Investigation

A geotechnical investigation was performed by Fugro-McClelland, Inc. to determine the general subsurface conditions at each detention pond site. Based upon geologic maps and observations made in the field, soils and rock encountered were consistent with the Grayson Marl, Main Street Limestone and Woodbine geologic formations. The engineering characteristics of the subsurface materials encountered were also evaluated, and recommendations were made for retaining structure foundations, lateral earth pressures, backfill, and related earthwork, including slope ratios for slopes of various heights.

Soil borings were drilled to depths ranging from fifteen to thirty feet and soil samples were collected for laboratory testing. One soil boring was taken at each detention pond site. The locations of the soil borings are shown in Figure II-3. The soil samples were tested for moisture content, liquid and plastic limits, percent passing the No. 200 sieve, unit dry weight, and Torvane shear. Copies of the boring logs are included in Appendix A. Additional information on the subsurface soils at the project sites can be found in the *Geotechnical Engineering Study*[6] prepared by Fugro-McClelland.

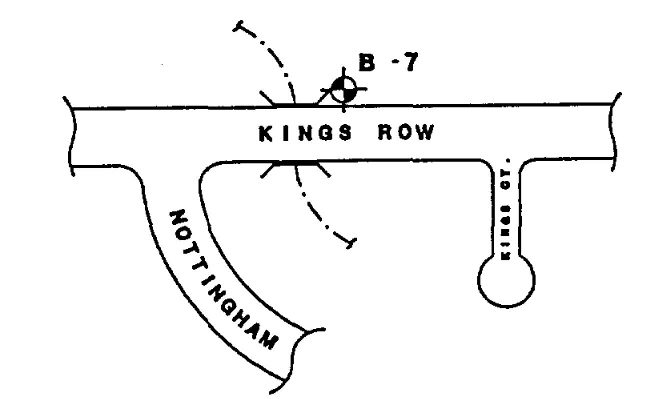


Figure II - 3e -- Soil Boring At Pond Site #5

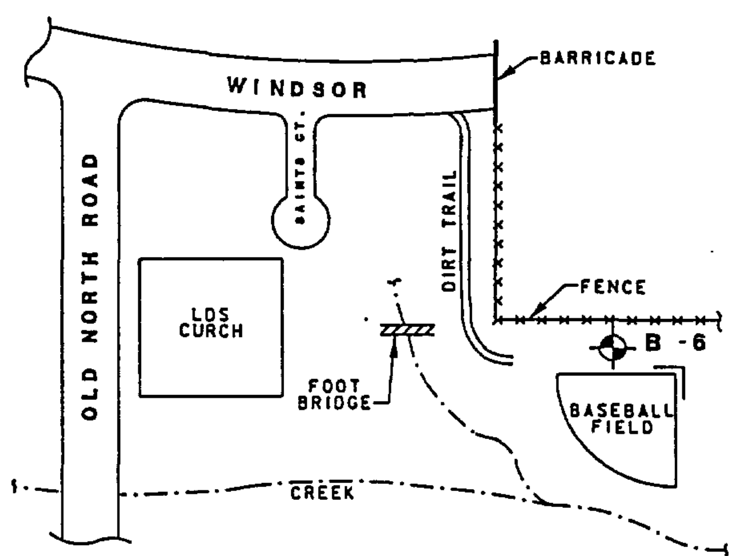


Figure II - 3f -- Soil Boring At Pond Site #6

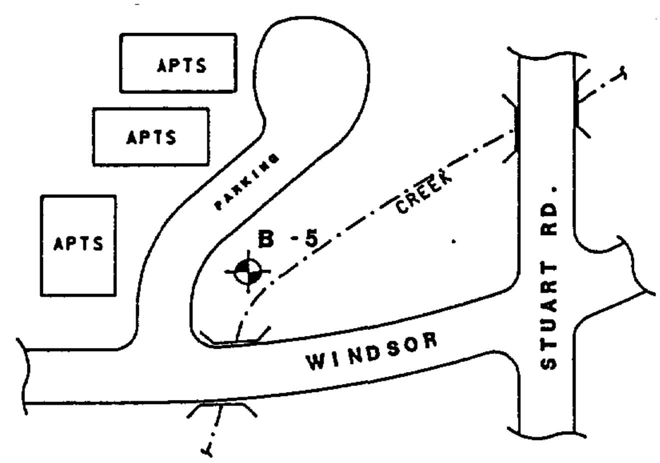


Figure II - 3c -- Soil Boring At Pond Site #3

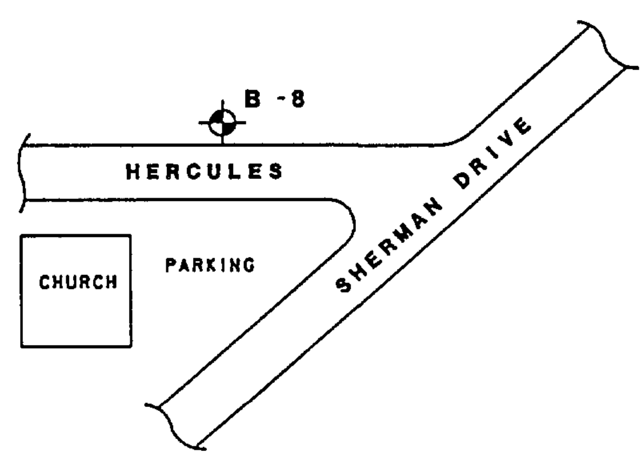


Figure II - 3d -- Soil Boring At Pond Site #4

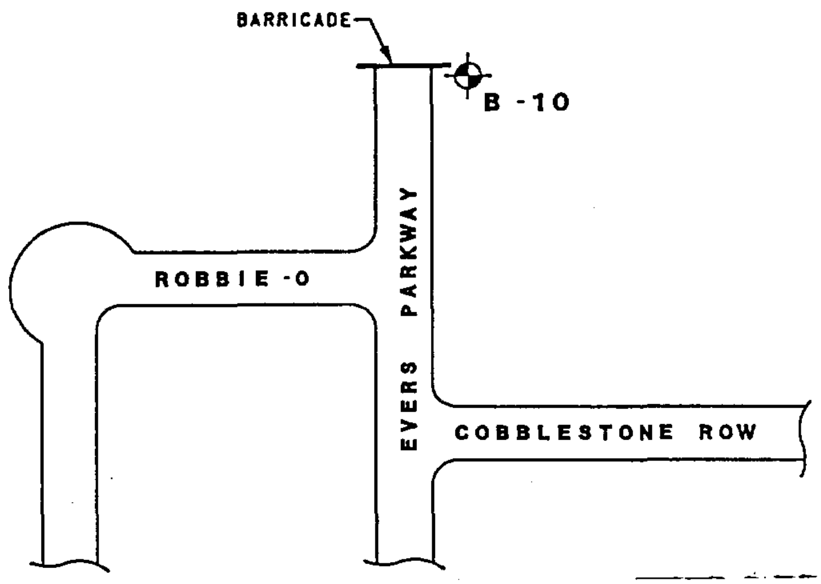


Figure II - 3a -- Soil Boring At Pond Site #1

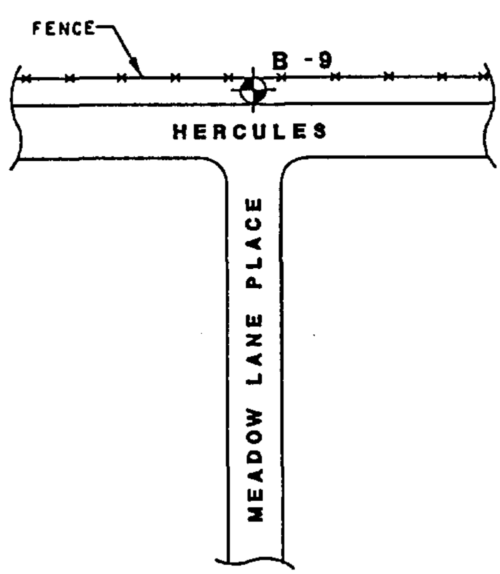


Figure II - 3b -- Soil Boring At Pond Site #2

3.0 HYDROLOGIC ANALYSIS

Contributing Drainage Areas

Cooper Creek is located on the north side of Denton. The drainage area upstream of Mingo Road includes approximately five square miles. For the purposes of this study, the Cooper Creek watershed above Mingo Road was divided into 17 subbasins. Figure II-4 shows the locations of the subbasins and their relation to the proposed detention facilities.

At present, the majority of the development within the contributing drainage area is residential, with a few schools and parks scattered within the watershed. There are a few pasture areas within the watershed that have the potential for development. These areas, located near F.M. 2164, Sherman Drive, Kings Row and Loop 288, are likely to be developed as single family residential with commercial and/or light industrial. The property with the highest probability for development is adjacent to Loop 288.

Pond Evaluation Criteria

Conceptual designs for the ponds were developed based on the *Denton Drainage Criteria Manual*[7], including the following criteria:

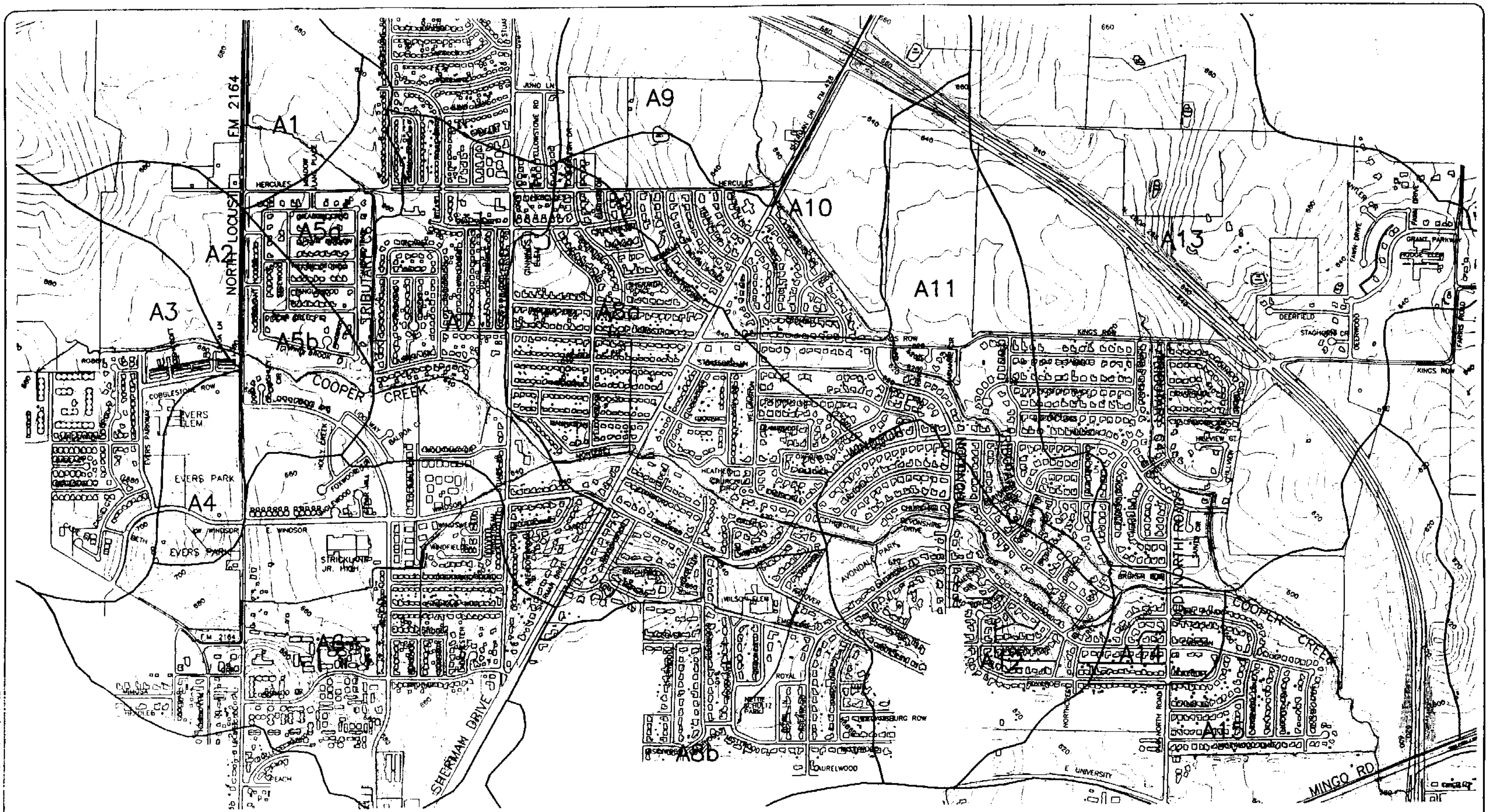
- The pond outlet structures should be sized such to reduce the downstream 100-year peak discharge enough to significantly reduce the 100-year floodplain width. The maximum available pond size for the conceptual evaluation is fifteen acres.
- Existing stream banks and natural topography should be used as much as possible for the proposed detention facility. Existing roadway culverts

should be used as outlet structures for the detention ponds whenever feasible.

- Detention would be obtained through a combination of excavation for additional depth and fill for embankments, and the amount of water detained should be six to eight feet deep.
- The ponds should be set back 100 to 200 feet from existing roadways to allow future development of property adjacent to the roadways.
- The pond embankments, either excavated or placed by fill, should have 4:1 side slopes, and the embankment crowns should be at least 12-feet wide. The interior of the ponds should be grass lined to facilitate alternative uses for the pond areas and keep construction costs down.
- Concrete-lined pilot channels at a 0.5% grade may be needed to accommodate low flows through the detention ponds.
- The ponds should hold water only during a storm event and for a short period afterwards; no water would be permanently impounded. Bottom slopes should not be less than 1 percent.

Computer Modeling

The evaluation of the detention ponds was performed by simultaneously evaluating hydrologic and hydraulic models of the Cooper Creek watershed. Hydrologic models were used to compute runoff hydrographs at selection design points. The hydraulic models were used to determine storage-discharge relationships to route flood hydrographs in the hydrologic models and to predict water surface profiles after the peak discharges were



A1 0.376 SQ. MI.	A7 0.259 SQ. MI.	A13 0.884 SQ. MI.
A2 0.072 SQ. MI.	A8a 0.401 SQ. MI.	A14 0.057 SQ. MI.
A3 0.559 SQ. MI.	A8b 0.372 SQ. MI.	A15 0.148 SQ. MI.
A4 0.062 SQ. MI.	A9 0.302 SQ. MI.	
A5a 0.109 SQ. MI.	A10 0.287 SQ. MI.	
A5b 0.097 SQ. MI.	A11 0.297 SQ. MI.	
A6 0.467 SQ. MI.	A12 0.288 SQ. MI.	

N

0 1000' 2000'
1" = 1000'

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COOPER CREEK
REGIONAL DETENTION STUDY
DRAINAGE AREA MAP

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FIGURE II–4

determined.

Synthetic unit hydrographs were developed by the HEC-1 model for each subbasin using the Snyder method. By definition, a unit hydrograph is a plot of discharge versus time for a storm producing one inch of rainfall over the entire drainage basin. In order to compute the unit hydrograph, the HEC-1 program requires rainfall data and the values of the Snyder method lag time and watershed coefficient.

Rainfall Data

Rainfall depths for storms are applied to the unit hydrograph to determine the resulting peak stormwater discharges produced by those storms. Rainfall data for the 5-, 10-, 25-, 50-, and 100-year frequency storms were derived from intensity-duration-frequency curves from the *Drainage Design Criteria Manual.* The 500-year storm was derived from rainfall data contained in the *Detailed Project Report*[8]. Table II-3 contains a listing of the rainfall data used in the hydrologic models.

Table II-3 Rainfall Depths (Inches)

Storm	Rainfall Duration							
Frequency	5-min	15-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
5-yr	0.57	1.23	2.33	2.92	3.24	3.84	4.56	5.40
10-yr	0.63	1.38	2.67	3.40	3.84	4.62	5.40	6.24
25-yr	0.72	1.58	3.15	3.98	4.50	5.40	6.36	7.44
50-yr	0.80	1.78	3.55	4.58	5.16	6.30	7.32	8.28
100-yr	0.86	1.90	3.85	5.00	5.70	6.96	8.16	9.48
500-yr	0.90	2.00	4.24	6.20	6.90	8.30	9.80	11.8

Precipitation Losses

Interception, depression storage and infiltration within each basin are combined and handled as precipitation losses in the hydrologic models. Initial and hourly rainfall loss rates vary with storm frequency and soil type. Typically, storms with a lower return interval (i.e. more frequent storms) will have higher initial and hourly loss rates. Clay soils have lower loss rates than sandy soils due to the lower permeability of the clay soils. The initial and hourly loss rates used in this project were derived from Table 6 of the *Detailed Project Report* and are summarized in Table II-4.

Table II-4 Initial and Hourly Rainfall Loss Rates

	Initial Loss Rate (inches per hour)					Hourly Loss Rate (inches per hour)				
Subbasin	5-yr	10-yr	25-yr	50-yr	100-yr	5-yr	10-yr	25-yr	50-yr	100-yr
A-1	1.70	1.40	1.13	0.97	0.83	0.23	0.19	0.14	0.12	0.09
A-2	1.78	1.45	1.17	1.01	0.85	0.24	0.20	0.14	0.12	0.09
A-3	1.69	1.39	1.12	0.96	0.82	0.23	0.19	0.13	0.11	0.08
A-4	1.98	1.58	1.29	1.09	0.89	0.26	0.22	0.15	0.13	0.10
A-5a	1.41	1.20	0.95	0.84	0.75	0.20	0.16	0.12	0.10	0.07
A-5b	1.41	1.20	0.95	0.84	0.75	0.20	0.16	0.12	0.10	0.07
A-6	1.78	1.45	1.17	1.00	0.84	0.24	0.20	0.14	0.12	0.09
A-7	1.43	1.22	0.97	0.85	0.76	0.20	0.16	0.12	0.10	0.07
A-8a	1.50	1.27	1.01	0.88	0.78	0.21	0.17	0.13	0.11	0.08
A-8b	1.90	1.54	1.24	1.06	0.88	0.25	0.21	0.15	0.13	0.10
A-9	1.40	1.20	0.95	0.84	0.75	0.20	0.16	0.12	0.10	0.07
A-10	1.40	1.20	0.95	0.84	0.75	0.20	0.16	0.12	0.10	0.07
A-11	1.41	1.21	0.96	0.85	0.75	0.20	0.16	0.12	0.10	0.07
A-12	1.63	1.35	1.08	0.94	0.81	0.22	0.18	0.13	0.11	0.08
A-13	1.72	1.42	1.14	0.98	0.83	0.23	0.19	0.14	0.12	0.09
A-14	1.54	1.29	1.03	0.90	0.78	0.21	0.17	0.13	0.11	0.08
A-15	1.50	1.27	1.01	0.88	0.77	0.21	0.17	0.12	0.10	0.07

Watershed Soil Types

There are approximately 33 different soil classifications within the Cooper Creek watershed, which were classified as either clay or sand based on the Soil Conservation Service soil maps for Denton County[9] and Table 7 of the *Detailed Project Report*. The resulting approximate areas of sand and clay soils are shown Figure II-5. The percentage of sand and clay in each subbasin was determined by overlaying the soil classification map in the SCS soil maps onto the drainage basin map and area summarized in Table II-5.

Table II-5 Watershed Subbasin Soil Composition

Basin No.	% Sand	% Clay	Basin No.	% Sand	% Clay
1	50.7	49.3	8b	83.8	16.2
2	63.7	36.3	9	0	100
3	47.5	52.5	10	0	100
4	95.9	4.1	11	2.0	98.0
5a	1.2	98.8	12	37.9	62.1
5b	1.0	99.0	13	53.9	46.1
6	62.9	37.1	14	22.6	77.4
7	5.7	94.3	15	16.5	83.5
8a	17.0	83.0			

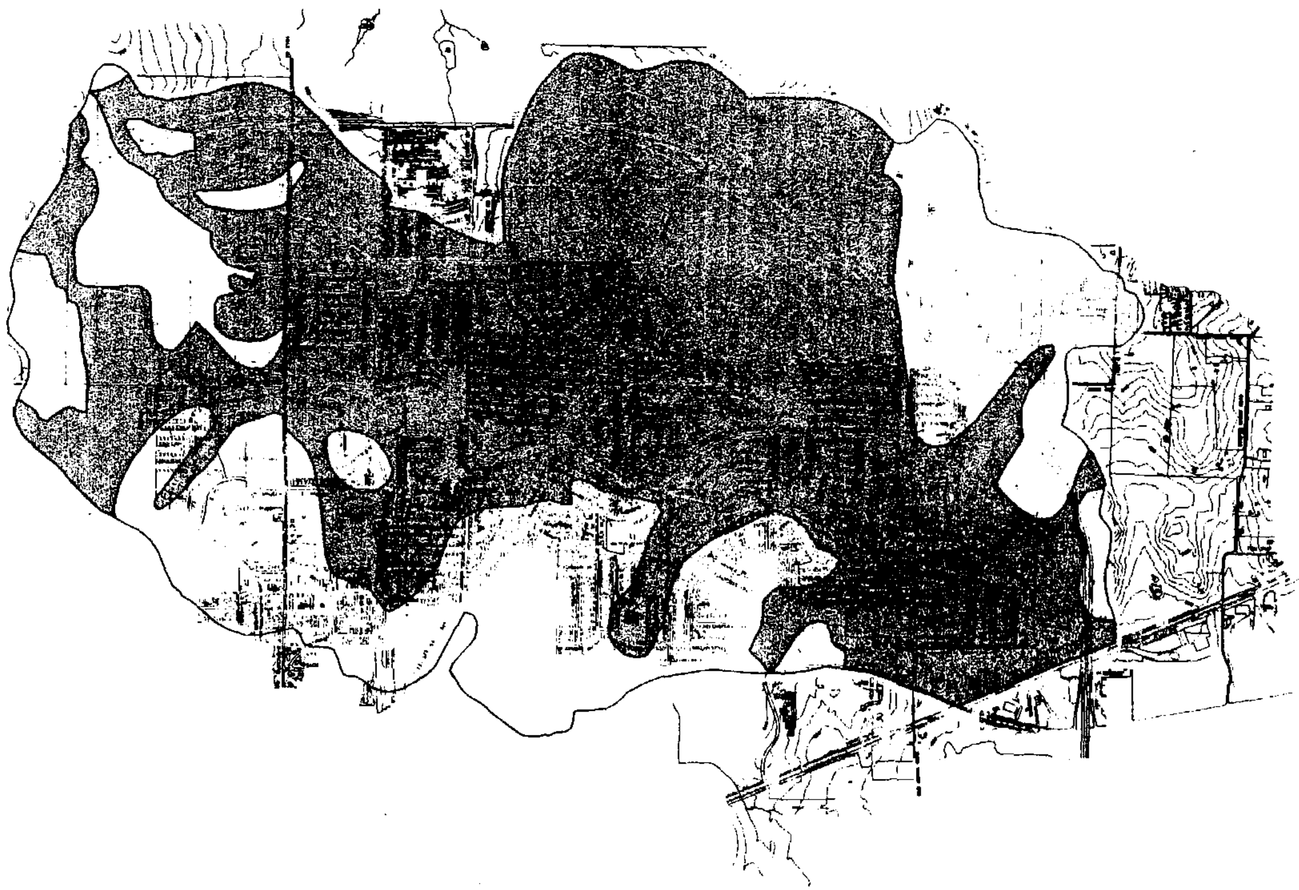
The percentages of basin urbanization present conditions and probable future conditions for each watershed were estimated by inspection of maps provided by the City and comparison with values reported in the *Detailed Project Report*.

Previous studies by the Corps suggest that the percentage of the watershed that is impervious at 100% urbanization is approximately 50%. Therefore, for this study, the percentage of the basin that is impervious for both existing and ultimate watershed development was assumed to be half of the percentage urbanized.

Lag Time

The lag time is time interval between the center of the rainfall duration and the peak discharge. The Corps developed relationships between the watershed lag time and the amount of watershed urbanization. These relationships, often referred to as the "urbanization curves", relate the basin lag time to the stream length, stream length to the basin centroid, and the average basin slope. There are two sets of "urbanization curves" that were developed by the Corps. One set of curves was developed for primarily sandy soils, while the other was developed for soils that are primarily clay. Copies of these curves can be found in Appendix B.

The lag times used in the development of runoff hydrographs were estimated from the "urbanization curves" after computing the watershed characteristics - stream length, stream length to the watershed centroid, and the basin slope. Lag times reflecting both current and ultimate watershed development conditions were developed. The stream length was measured from the discharge point in question to the upstream limits of the drainage basin. The length to the basin centroid is measured from the discharge point in question to a point on the stream nearest the basin centroid. The average stream slope was computed as the elevation difference between points along the stream at 15% and 85% of the stream length above the discharge point in question. A more detailed discussion of this method may be found in the *Detailed Project Report*. Table II-6 contains a summary of the parameters used in computing the lag time for each subbasin.



LEGEND

CLAY SOILS

SANDY SOILS

N

0 2000' 4000'
1"=2000'

CITY OF DENTON

COOPER CREEK
REGIONAL DETENTION STUDY

SOILS MAP

FREESE • NICHOLS

FIGURE II-5

Table II-6 Peak Lag Time Parameters

Subbasin	Area (sq. mi.)	Stream Length (miles)	Length to Centroid (miles)	Average Basin Slope (ft/mi)	Current % Urb.	Current Lag Time (hours)	Ultimate % Urb.	Ultimate Lag Time (hours)
A-1	0.376	1.22	0.66	44.06	10	0.57	80	0.37
A-2	0.072	0.60	0.40	56.03	15	0.36	90	0.23
A-3	0.559	1.35	0.45	37.54	35	0.44	90	0.32
A-4	0.062	0.41	0.23	40.56	10	0.32	50	0.25
A-5a	0.109	0.58	0.43	29.22	80	0.18	90	0.16
A-5b	0.097	0.33	0.19	25.30	80	0.11	90	0.10
A-6	0.467	1.08	0.54	42.26	85	0.35	90	0.33
A-7	0.259	0.45	0.29	41.30	85	0.13	90	0.13
A-8a	0.401	0.63	0.42	36.01	85	0.20	90	0.19
A-8b	0.372	0.94	0.55	75.50	90	0.32	90	0.32
A-9	0.302	0.65	0.40	28.17	10	0.28	70	0.19
A-10	0.287	0.97	0.40	56.95	20	0.27	70	0.20
A-11	0.297	0.96	0.76	28.87	90	0.25	90	0.25
A-12	0.288	0.79	0.34	21.28	85	0.25	90	0.24
A-13	0.884	1.17	0.89	49.24	10	0.62	70	0.42
A-14	0.057	0.27	0.16	32.35	85	0.11	90	0.10
A-15	0.148	0.63	0.26	11.88	30	0.28	70	0.22

The watershed coefficient, C_p, used in the Snyder method accounts for flood wave and storage conditions and is a function of the lag time, duration of runoff producing rain, effective area contributing to peak flow and drainage area. A typical value of 0.72, reported by the Corps for the Dallas-Fort Worth area in *Detailed Project Report*, was used in this analysis.

Hydrograph Routing

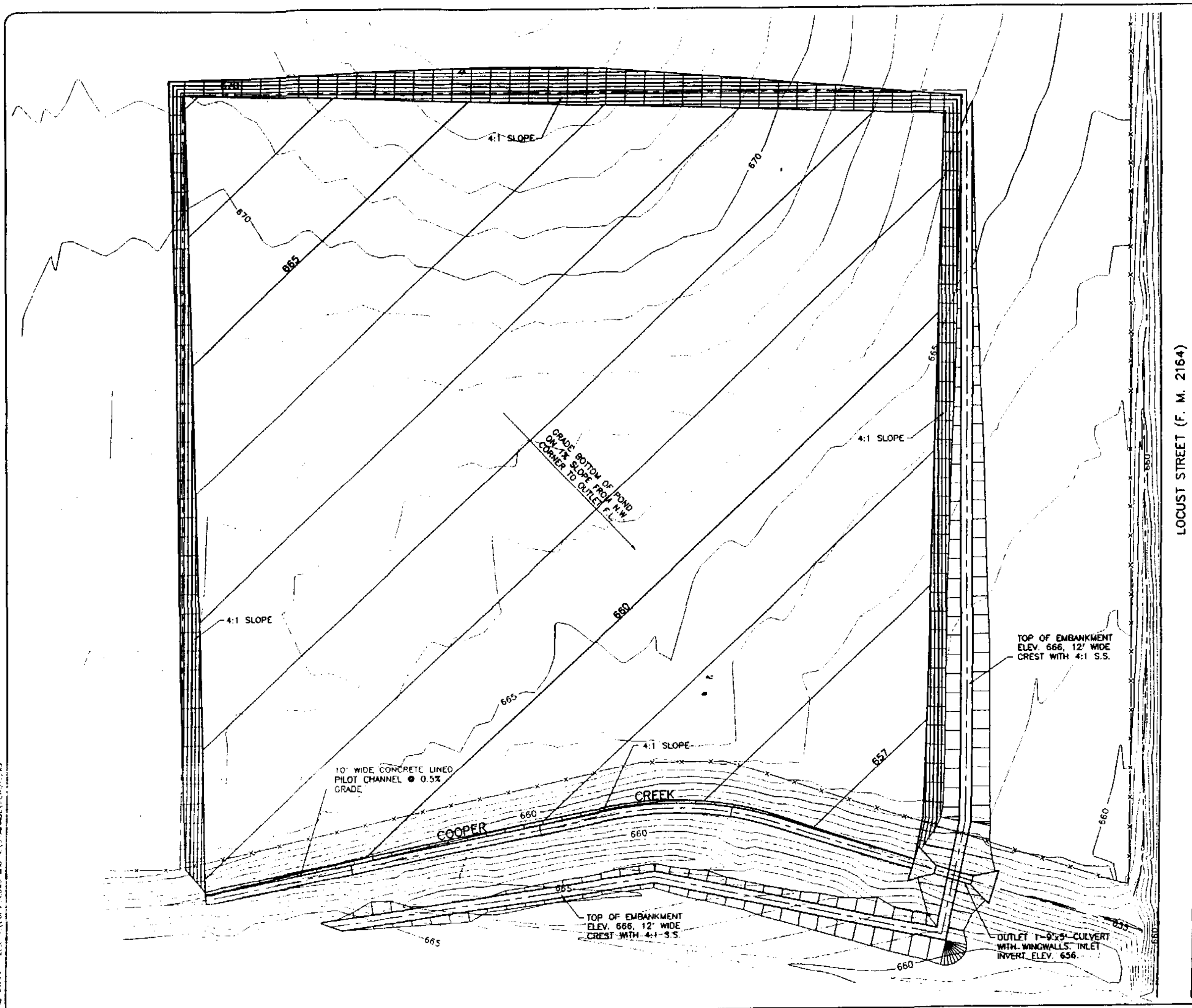
The modified Puls method was used to route runoff hydrographs between design points. Storage-discharge relationships for the six main stem reaches and the two tributary C-5 reaches were developed using the existing Cooper Creek HEC-2 stream models.

Similar data for tributary C-6 below detention pond 2 were estimated from topographic maps and information obtained during a site visit.

Conceptual Detention Pond Layouts

Conceptual layouts of the six detention ponds were developed using the topographic maps developed with the field survey data. The maximum size of the detention facility was limited to approximately fifteen acres at each site (refer to field survey discussion). The proposed detention ponds at sites #1, #2, #4 and #5 are located on large, undeveloped properties. Detention pond sites #3 and #6 are located on undeveloped properties, but the available area for detention storage is significantly smaller than the other ponds. The storage capacity of each pond was assumed to be provided by a combination of excavation and embankment construction. Conceptual designs for the ponds are found in Figures II-6 through II-11.

Several iterations of outflow hydrograph development were performed for each of the detention pond locations to determine the outlet structure needed to prevent the ponds from exceeding their maximum capacity for the assumed 15-acre pond configurations. The outlet structures required for ponds at sites #3 and #6 are quite large because the storage capacity at these locations is very limited. Only pond 5 uses an existing outlet structure. A new outlet structure will be required for the other ponds since they are set back from the existing roadway culverts and require an embankment to achieve sufficient volume. Table II-7 contains a summary of the detention pond conceptual designs.



NOTE: DETENTION POND LAYOUT IS CONCEPTUAL. THE FINAL LOCATION AND LAYOUT MAY BE DIFFERENT THAN SHOWN, DEPENDING ON FINAL DESIGN CONSIDERATIONS.

N

0 50' 100' 200'

1" = 100'

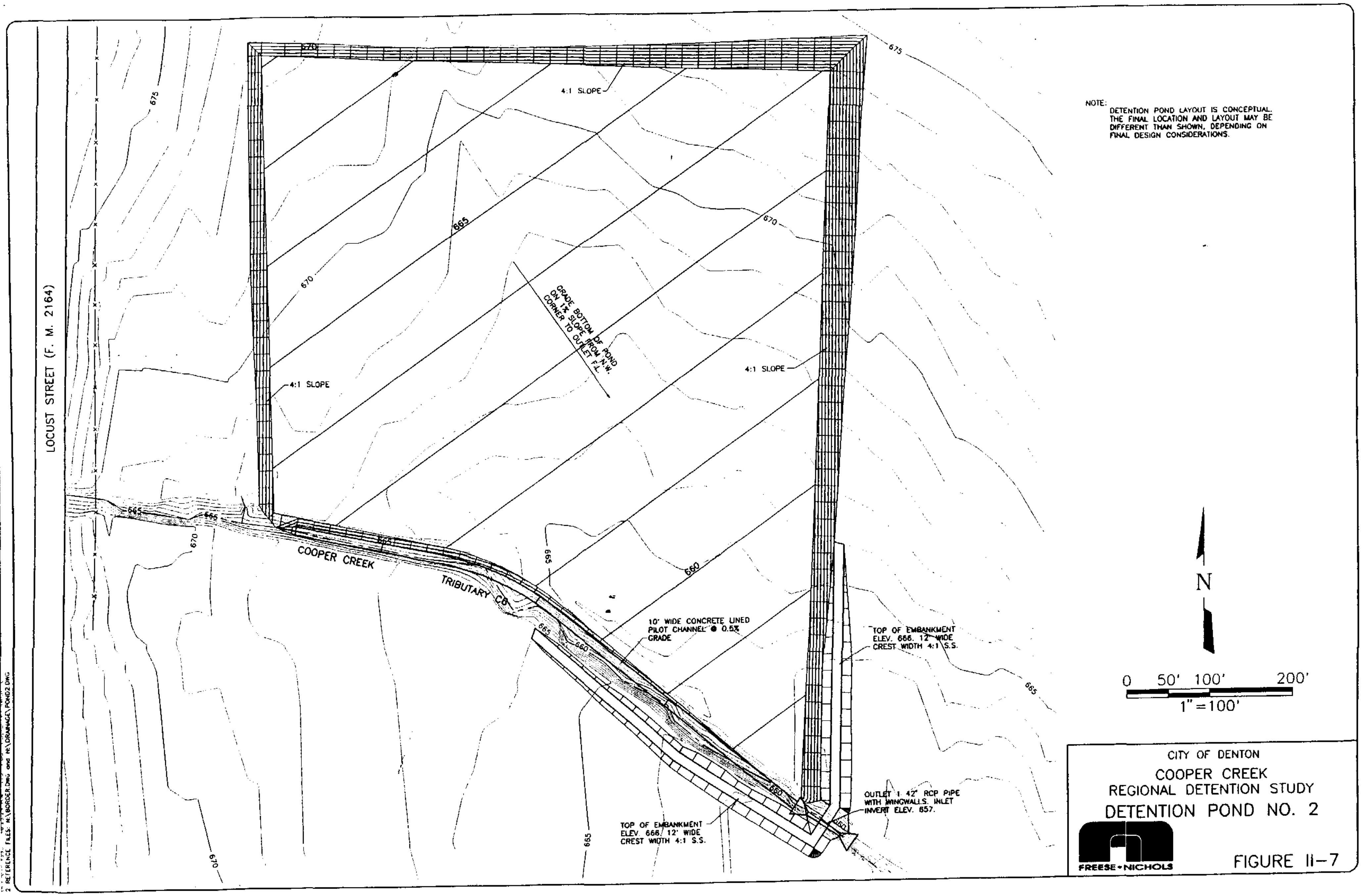
CITY OF DENTON

COOPER CREEK
REGIONAL DETENTION STUDY

DETENTION POND NO. 1

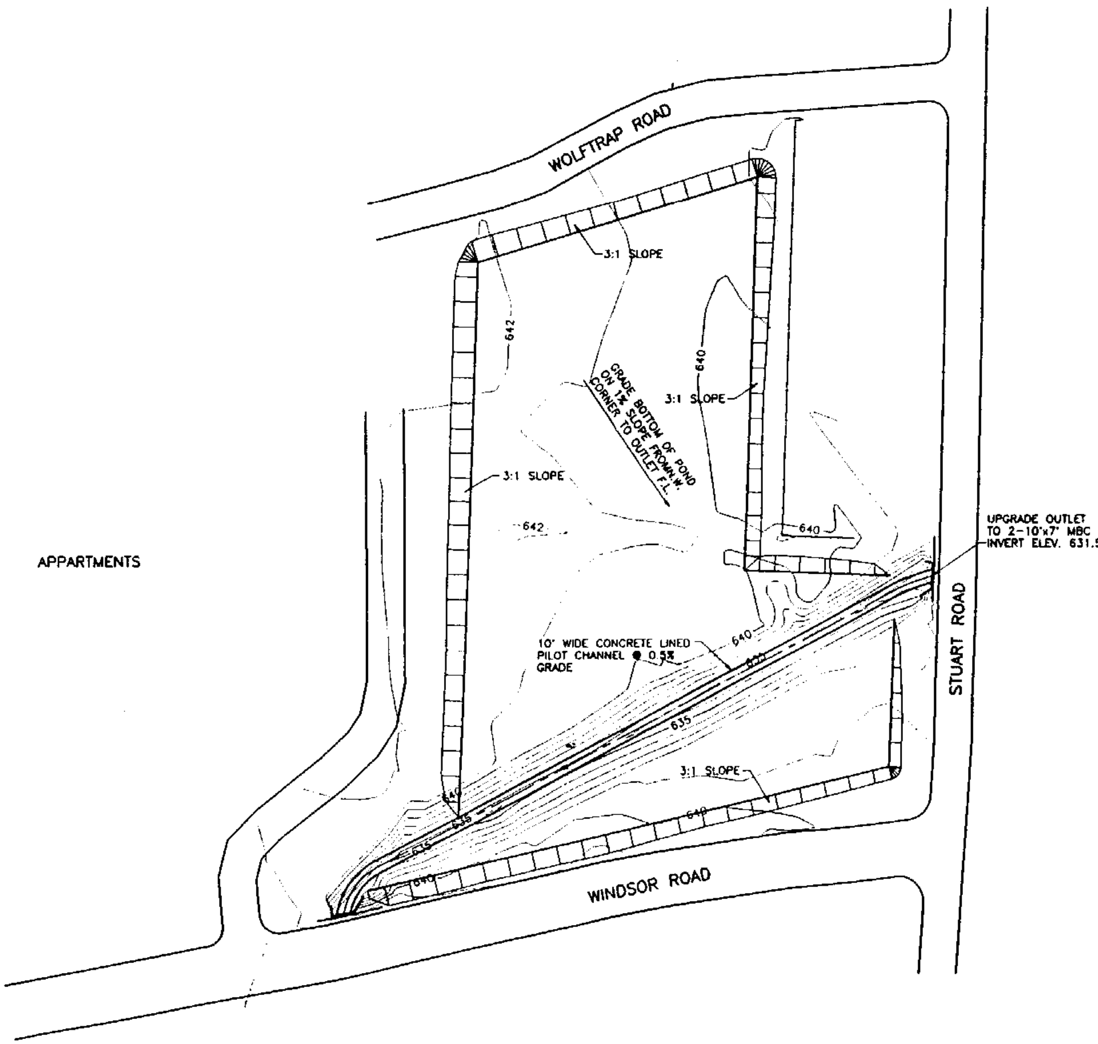
FREESE • NICHOLS

FIGURE II-6



LOCUST STREET (F. M. 2164)
NOTE: DETENTION POND LAYOUT IS CONCEPTUAL. THE FINAL LOCATION AND LAYOUT MAY BE DIFFERENT THAN SHOWN, DEPENDING ON FINAL DESIGN CONSIDERATIONS.
4:1 SLOPE
4:1 SLOPE
4:1 SLOPE
GRADE BOTTOM OF POND ON 1% SLOPE FROM N.W. CORNER TO OUTLET F.L.
COOPER CREEK
TRIBUTARY CB
10' WIDE CONCRETE LINED PILOT CHANNEL @ 0.5% GRADE
TOP OF EMBANKMENT ELEV. 666. 12' WIDE CREST WIDTH 4:1 S.S.
TOP OF EMBANKMENT ELEV. 666. 12' WIDE CREST WIDTH 4:1 S.S.
OUTLET 1 42" RCP PIPE WITH WINGWALLS. INLET INVERT ELEV. 657.
675
670
665
660
N
0 50' 100' 200'
1"=100'
CITY OF DENTON
COOPER CREEK
REGIONAL DETENTION STUDY
DETENTION POND NO. 2
FREESE • NICHOLS
FIGURE II-7
2 REFERENCE FILES: N:\BORDER.DWG and N:\DRAINAGE\POND2.DWG

NOTE:
DETENTION POND LAYOUT IS CONCEPTUAL. THE FINAL LOCATION AND LAYOUT MAY BE DIFFERENT THAN SHOWN, DEPENDING ON FINAL DESIGN CONSIDERATIONS.



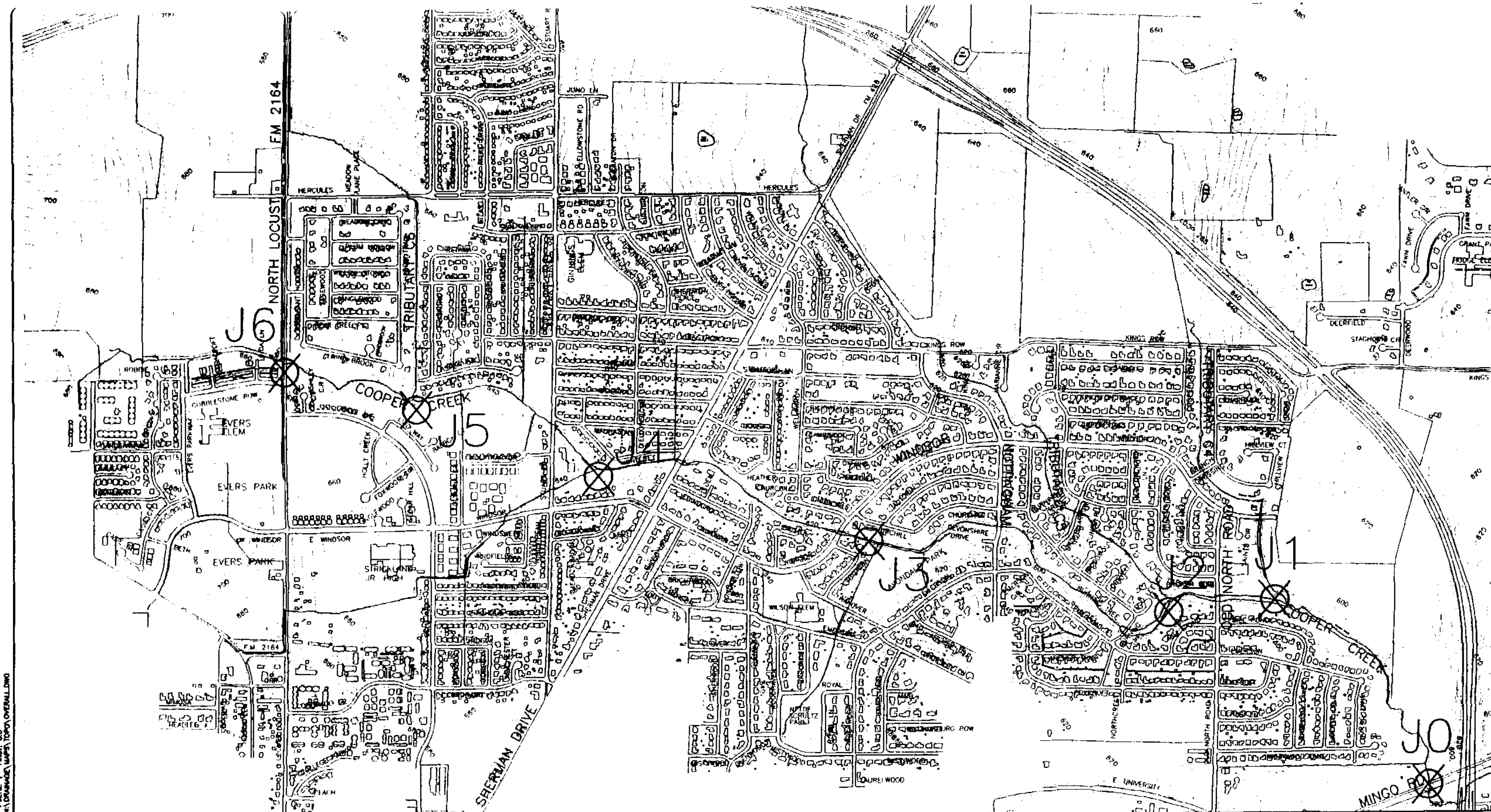
N

0 50' 100' 200'
1"=100'

CITY OF DENTON
COOPER CREEK
REGIONAL DETENTION STUDY
DETENTION POND NO. 3

FREESE • NICHOLS

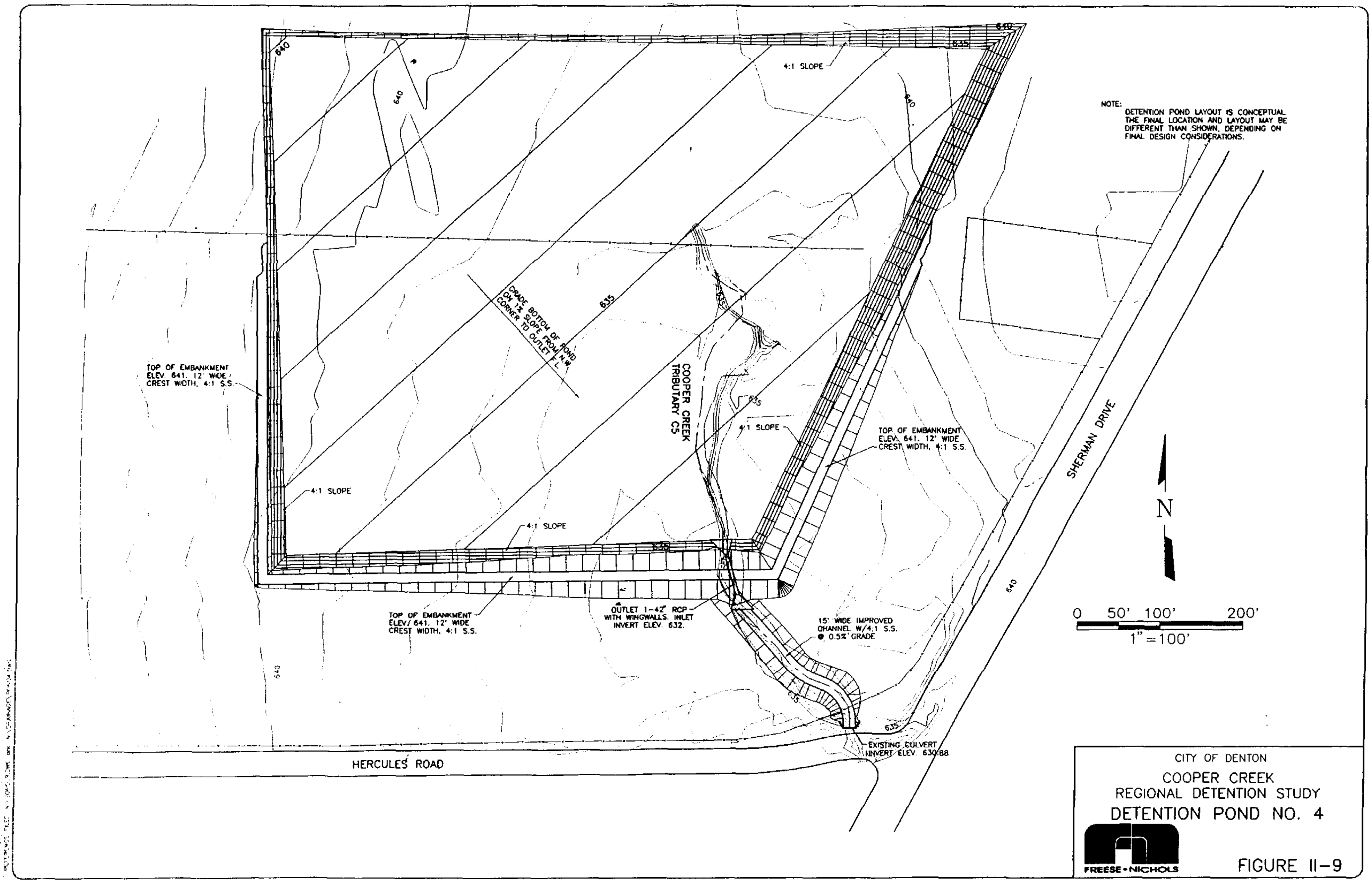
FIGURE II–8



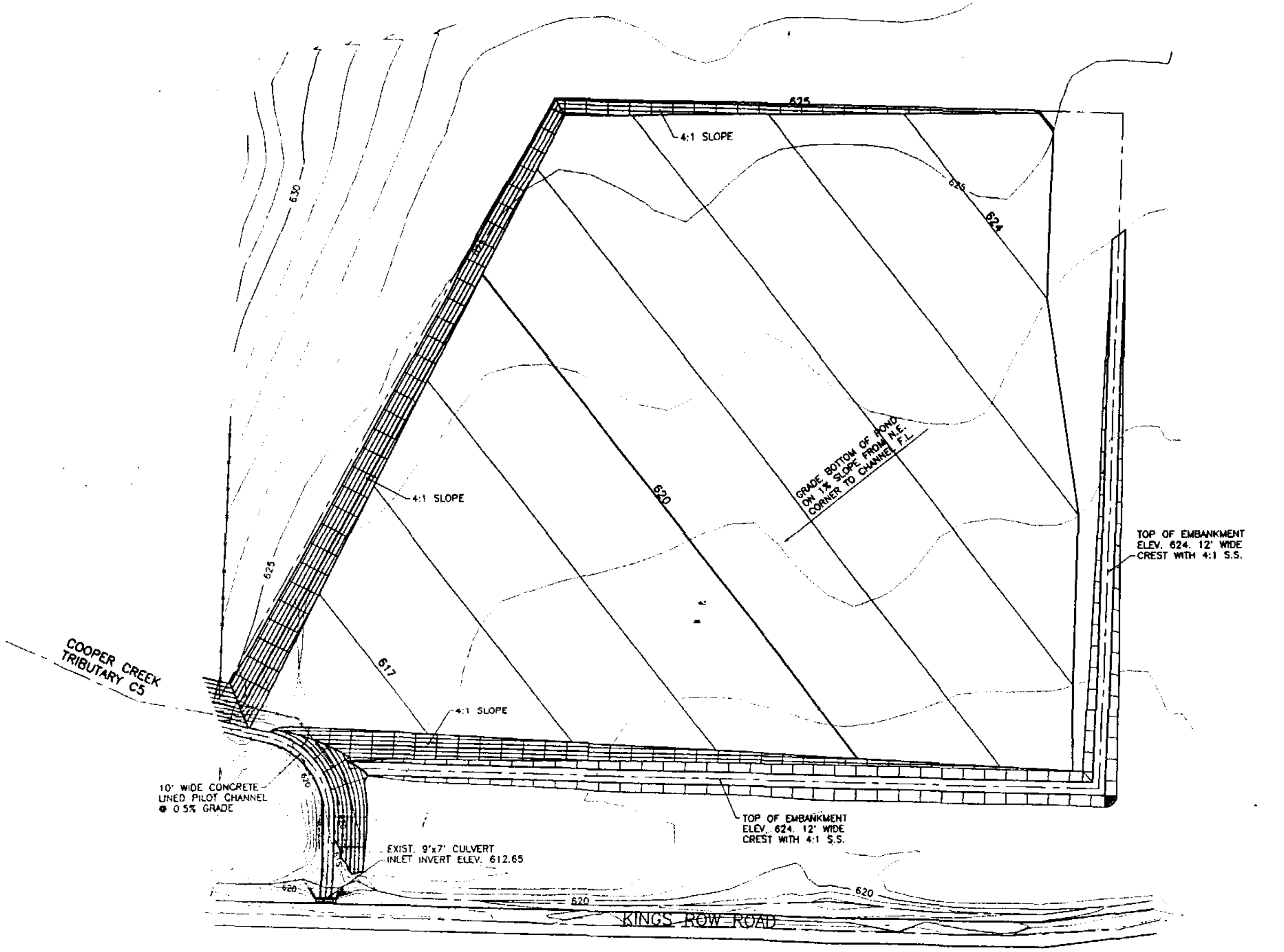
CITY OF DENTON
COOPER CREEK
REGIONAL DETENTION STUDY
HEC-1 DESIGN POINTS

FREESE-NICHOLS

FIGURE II-13


NOTE:
DETENTION POND LAYOUT IS CONCEPTUAL. THE FINAL LOCATION AND LAYOUT MAY BE DIFFERENT THAN SHOWN, DEPENDING ON FINAL DESIGN CONSIDERATIONS.
4:1 SLOPE
640
635
GRADE BOTTOM OF POND ON 1% SLOPE FROM N.W. CORNER TO OUTLET F.L.
COOPER CREEK TRIBUTARY C5
TOP OF EMBANKMENT ELEV. 641. 12' WIDE CREST WIDTH, 4:1 S.S.
TOP OF EMBANKMENT ELEV. 641. 12' WIDE CREST WIDTH, 4:1 S.S.
TOP OF EMBANKMENT ELEV. 641. 12' WIDE CREST WIDTH, 4:1 S.S.
OUTLET 1-42" RCP WITH WINGWALLS. INLET INVERT ELEV. 632.
15' WIDE IMPROVED CHANNEL W/4:1 S.S. @ 0.5% GRADE
EXISTING CULVERT INVERT ELEV. 630.88
SHERMAN DRIVE
HERCULES ROAD
N
0 50' 100' 200'
1" = 100'
CITY OF DENTON
COOPER CREEK
REGIONAL DETENTION STUDY
DETENTION POND NO. 4
FREESE • NICHOLS
FIGURE II-9

NOTE:
DETENTION POND LAYOUT IS CONCEPTUAL. THE FINAL LOCATION AND LAYOUT MAY BE DIFFERENT THAN SHOWN, DEPENDING ON FINAL DESIGN CONSIDERATIONS.



CITY OF DENTON

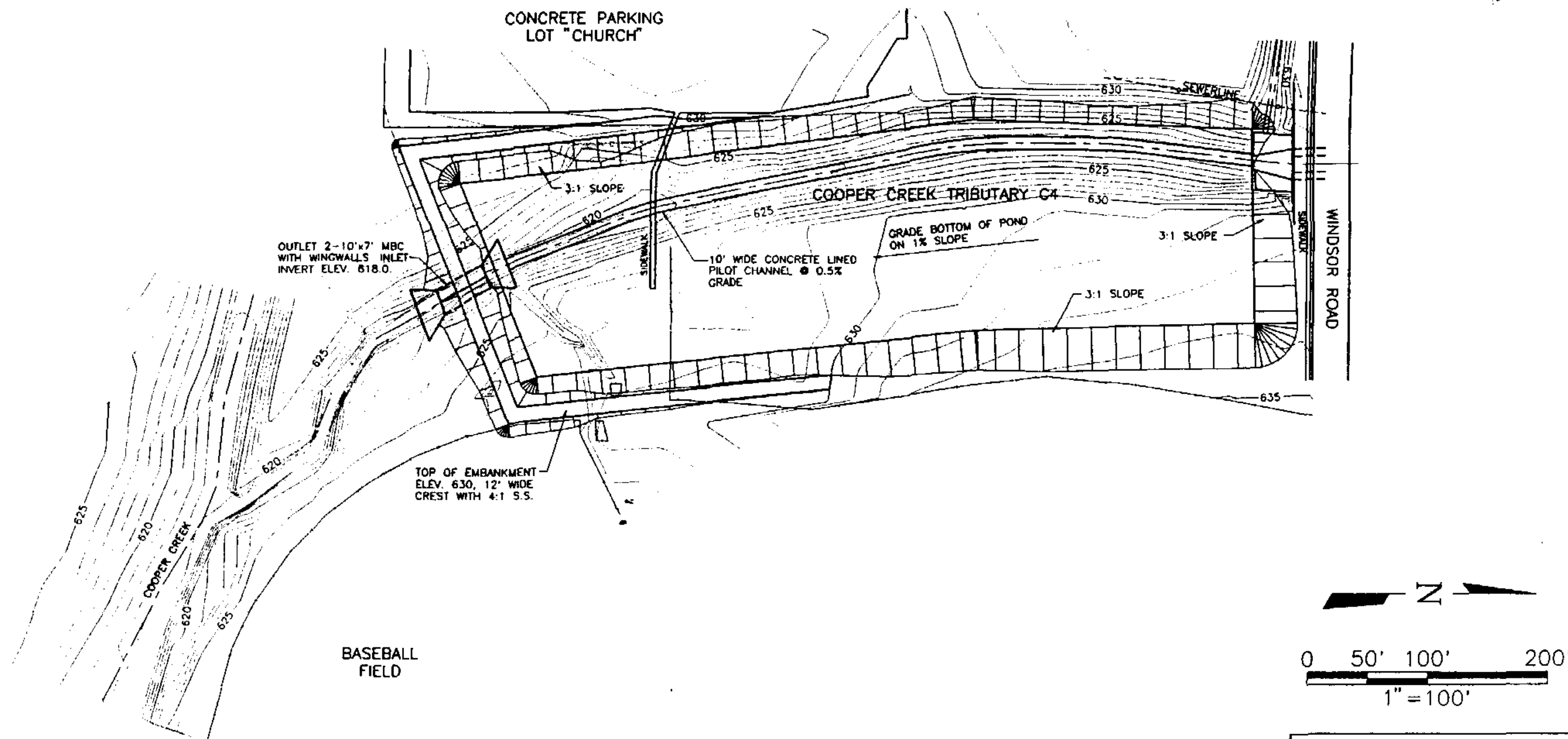
COOPER CREEK
REGIONAL DETENTION STUDY

DETENTION POND NO. 5

FREESE-NICHOLS

FIGURE II-10

NOTE:
DETENTION POND LAYOUT IS CONCEPTUAL. THE FINAL LOCATION AND LAYOUT MAY BE DIFFERENT THAN SHOWN, DEPENDING ON FINAL DESIGN CONSIDERATIONS.



CITY OF DENTON

COOPER CREEK
REGIONAL DETENTION STUDY

DETENTION POND. NO. 6

FREESE • NICHOLS

FIGURE II–11

FIGURE II-12 - DETENTION POND EFFECTIVENESS

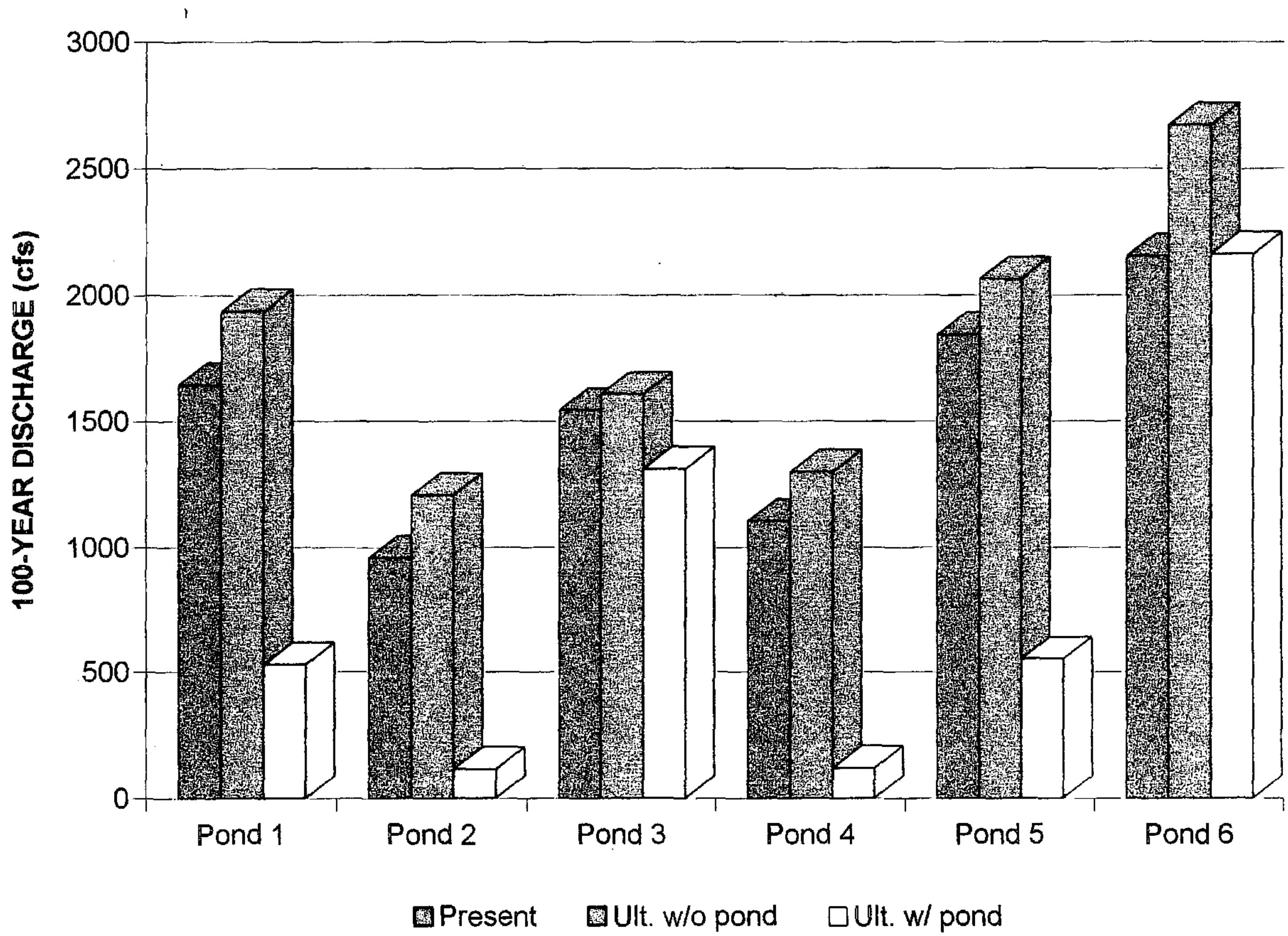


Table II-7 Summary of Detention Pond Conceptual Designs

Pond	Location	Controlled Subbasin	Pond Depth[a] (feet)	Storage Volume[b] (ac. ft.)	Outlet Structure
#1	Main stem of Cooper Creek west of Locust and north of the Snider Addition	A-3	10	121	5'x9' Box Culvert
#2	Tributary C-6 northeast of the intersection of Locust and Hercules	A-1	9	112	42" Pipe
#3	Northwest of the intersection of Windsor and Stuart Road	A-6	8	24	2-7'x10' Box Culverts
#4	Tributary C-5 northwest of the intersection of Hercules and Sherman Drive	A-9	10	89	42" Pipe
#5	Tributary C-5 northeast of the intersection of Yorkshire and Kings Row	A-9, A-10	8	50	2-5'x6' Box Culverts
#6	Tributary C-4 just north of the main stem of Cooper Creek east of Old North Road	A-6	14	42	2-7'x10' Box Culverts

a. Pond depth includes one foot of freeboard above the 100-year flood level.
b. Volumes do not include freeboard.

Peak Discharge Reductions

Upon selection of the outlet structures for the ponds, hydrographs for the 100-year storms were routed through the proposed detention ponds to determine the effectiveness of the ponds in reduce downstream peak flows. With the conceptual designs discussed earlier, the 100-year peak flows immediately downstream of the ponds were reduced by up to 91 percent. The reductions at sites #3 and #6 were relatively small (slightly less than 20 percent) due to the low availability of storage at these sites. A summary of the detention pond peak flow reduction performances is provided in Table II-8 and illustrated in Figure II-12.

Table II-8 Detention Pond Effectiveness in Reducing Peak Flows

		Peak Discharges at Pond Locations (cfs)					
Frequency	Development Condition	Pond #1	Pond #2	Pond #3	Pond #4	Pond #5	Pond #6
100-yr	present	1,645	959	1,545	1,105	1,844	2,156
	ult. w/o pond	1,937	1,208	1,608	1,300	2,063	2,672
	ult. with pond	531	115	1,311	120	552	2,162
50-yr	present	1,512	878	1,427	1,021	1,698	1,972
	ult. w/o pond	1,791	1,115	1,487	1,207	1,907	2,462
	ult. with pond	491	104	n/a	111	503	n/a
25-yr	present	1,331	769	1,260	901	1,494	1,727
	ult. w/o pond	1,583	984	1,313	1,069	1,685	2,169
	ult. with pond	425	93	n/a	99	432	n/a
10-yr	present	1,116	639	1,068	766	1,264	1,436
	ult. w/o pond	1,346	835	1,116	920	1,437	1,832
	ult. with pond	346	77	n/a	84	345	n/a
5-yr	present	936	518	912	665	1,086	1,167
	ult. w/o pond	1,160	713	952	809	1.252	1,543
	ult. with pond	268	56	n/a	67	274	n/a

The reductions in peak discharges along the Cooper Creek stream were evaluated at seven design points. The locations of these design points are listed below and illustrated in Figure II-13:

Jct. #0	Upstream of Mingo Road
Jct. #1	Downstream of Tributary C4
Jct. #2	Downstream of Tributary C5
Jct. #3	Downstream of unnamed tributary downstream of Windsor Street
Jct. #4	Downstream of unnamed tributary downstream of Stuart Road
Jct. #5	Downstream of Tributary C6
Jct. #6	Downstream of Locust Street (FM2164)

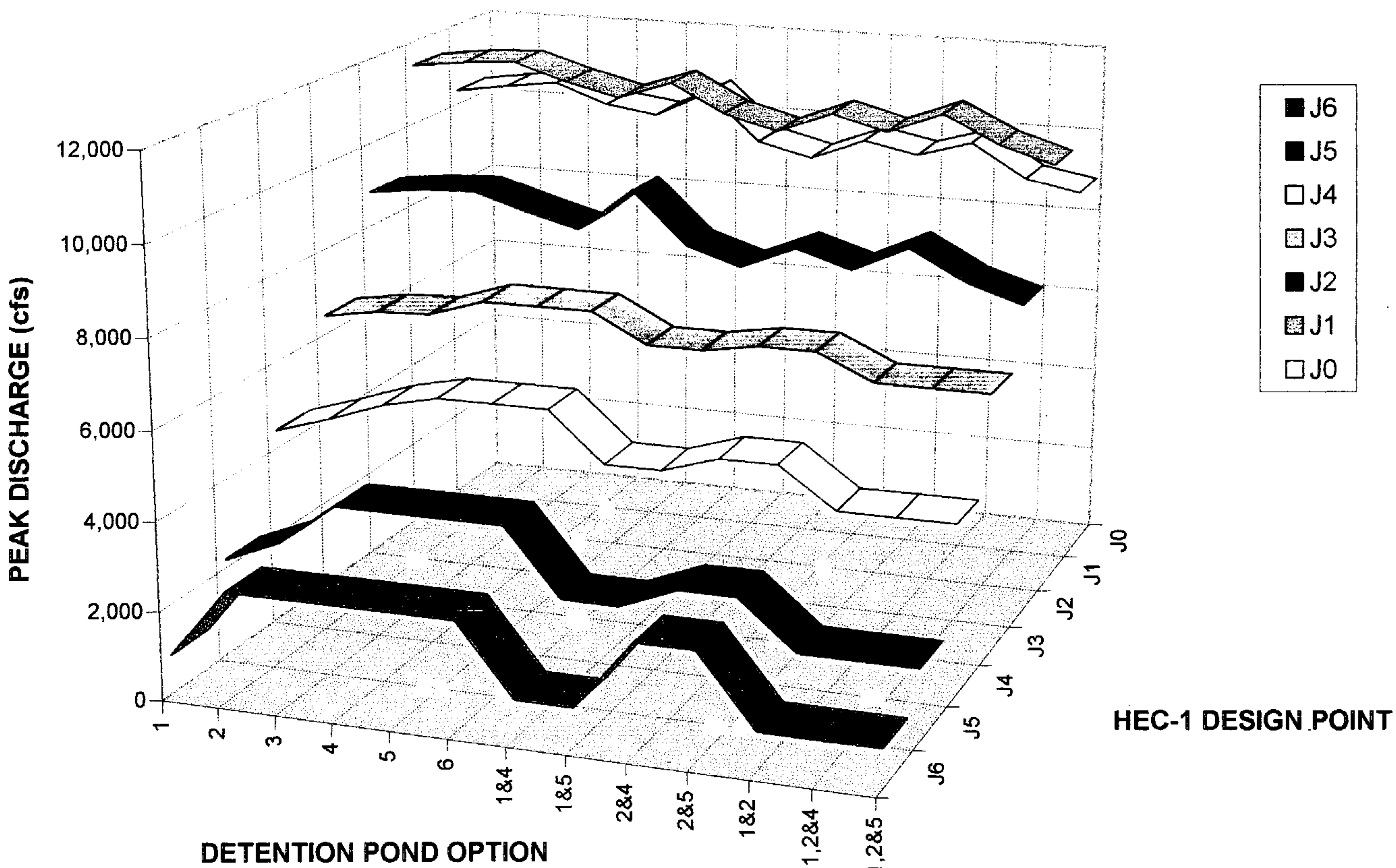
Thirteen scenarios of detention pond construction were evaluated to determine an effective approach to reducing the Cooper Creek 100-year discharges. In the first six scenarios, the effect of each pond on the Cooper Creek floodplain was evaluated. The second six scenarios included combinations of ponds at sites #1, #2, #4 and #5. Pond sites #3 and #6 were eliminated from consideration due to the lack of available detention at these sites and their minimal effect on reducing the Cooper Creek discharges. The options evaluated did not consider simultaneous construction of ponds at sites #4 and #5, since these sites are located on the same tributary to Cooper Creek.

Construction of a single detention pond at any of the six sites evaluated in this study will reduce the peak discharge at Mingo Road by less than ten percent. The greatest reduction in flow (8%) at Mingo Road results from construction of the pond at Site #5. A smaller reduction is obtained by construction of a pond at Sites #1 or #2, but due to their distance from Mingo Road, attenuation of the flood hydrograph lessens the overall reduction at Mingo Road. Various combinations of constructing ponds at Sites #1, #2, #4 and #5 can be expected to increase the overall reduction at Mingo Road to as much as 19%. A summary of the peak flow reductions along Cooper Creek is listed in Table II-9. Figure II-14 graphically illustrates the peak flow reductions for the twelve scenarios.

<u>Table II-9 Summary of Cooper Creek Peak Flows</u>

Detention Pond Option	Peak Discharges (cfs)						
	Jct. #6	Jct. #5	Jct. #4	Jct. #3	Jct. #2	Jct. #1	Jct. #0
No Ponds	2448	3,700	5,545	7,110	9,338	11,949	10,917
1 Only	876	2,193	4,432	6,448	8,832	11,428	10,351
2 Only	2,448	2,708	4,828	6,654	8,979	11,585	10,542
3 Only	2,448	2,708	5,262	6,708	9,039	11,660	10,703
4 Only	2,448	3,700	5,545	7,110	8,651	11,283	10,255
5 Only	2,448	3,700	5,545	7,110	8,320	10,973	10,072
6 Only	2,448	3,700	5,545	7,110	9,338	11,406	10,701
1 & 4	876	2,193	4,432	6,448	8,144	10,757	9,582
1 & 5	876	2,193	4,432	6,448	7,749	10,422	9,274
2 & 4	2,448	2,708	4,828	6,654	8,291	10,926	9,785
2 & 5	2,448	2,708	4,828	6,654	7,896	10,589	9,527
1 & 2	876	1,629	3,927	6,063	8,500	11,090	9,904
1, 2 & 4	876	1,629	3,927	6,063	7,812	10,468	9,121
1, 2 & 5	876	1,629	3,927	6,063	7,417	10,080	8,882

FIGURE II-14 - EFFECT OF DETENTION PONDS ON PEAK DISCHARGES



4.0 HYDRAULIC ANALYSIS

The existing HEC-2 model of Cooper Creek was used to estimate stormwater flow floodplain profiles and channel flow velocities for 100-year frequency storm. Peak discharge data from the HEC-1 hydrologic model for conditions with the proposed detention ponds in place were input into the HEC-2 model. No evaluations were performed to determine the channel and bridge/culvert improvements that may needed along Cooper Creek. Peak flows from the 100-year storm without the proposed detention facilities were also used in the HEC-2 model to generate floodplain profile and flow velocity data for comparison purposes.

Floodplain Profiles

The 100-year floodplain elevations along Cooper Creek are expected to increase less than 0.6 feet due to the anticipated future development. Construction of the detention ponds will reduce the 100-year ultimate development floodplain by as much as 1.9 feet. However, the reductions in floodplain elevations for the various detention pond scenarios does not necessarily reduce the floodplain to within the channel limits. The results of the floodplain elevation analysis are summarized in Table II-10. Floodplain profiles as also illustrated in Figure II-15.

Floodplain profiles were computed for Cooper Creek upstream of Mingo road for existing and ultimate development watershed conditions. Floodplain profiles were computed for each detention pond scenario for the 100-year ultimate development water conditions. Although the detention ponds are expected to reduce the peak discharges along Cooper Creek, backwater conditions are still expected at each street crossing for each detention pond scenario, suggesting the need for additional culvert capacity.

Table II-10 Comparison of 100-Year Floodplain Elevations

Bridge Location	Top of Bridge	Present Development	Ultimate Development	
		No Ponds	No Ponds	W/ Ponds 1 & 2
Locust St.	661.2	662.3	662.4	661.6
Stuart St.	638.6	642.0	642.6	640.9
Sherman Dr.	630.3	633.1	633.4	632.7
Windsor St.	620.9	622.2	622.5	621.8
Nottingham	610.4	613.4	613.5	613.2
Burning Tree	600.8	605.0	605.2	604.8
Old North Ave.	598.4	598.9	599.1	597.8
Mingo Rd.	585.5	592.6	593.1	592.1
M.K.T. Railroad	592.7	591.9	593.0	591.8

The M.K.T. Railroad bridge immediately downstream of Mingo Road causes a significant backwater condition to occur. The water surface profile increases approximately three feet at the railroad bridge, although the railroad is not expected to be overtopped by the 100-year flood. However, since Mingo Road is significantly lower than the railroad, it is expected to be overtopped by approximately seven feet. The Mingo Road culvert increases the floodplain elevation by an additional one to two feet. Improvements to the Cooper Creek channel should include some downstream channel improvements, which are outside the scope of this study, to lower the floodplain at this location.

The 100-year floodplain at Burning Tree Lane is expected to be approximately five feet over the roadway. This is due to the relatively low elevation of Burning Tree Lane

FIGURE II-15a. FLOODPLAIN COMPARISON
(MK&T RR TO BURNING TREE)

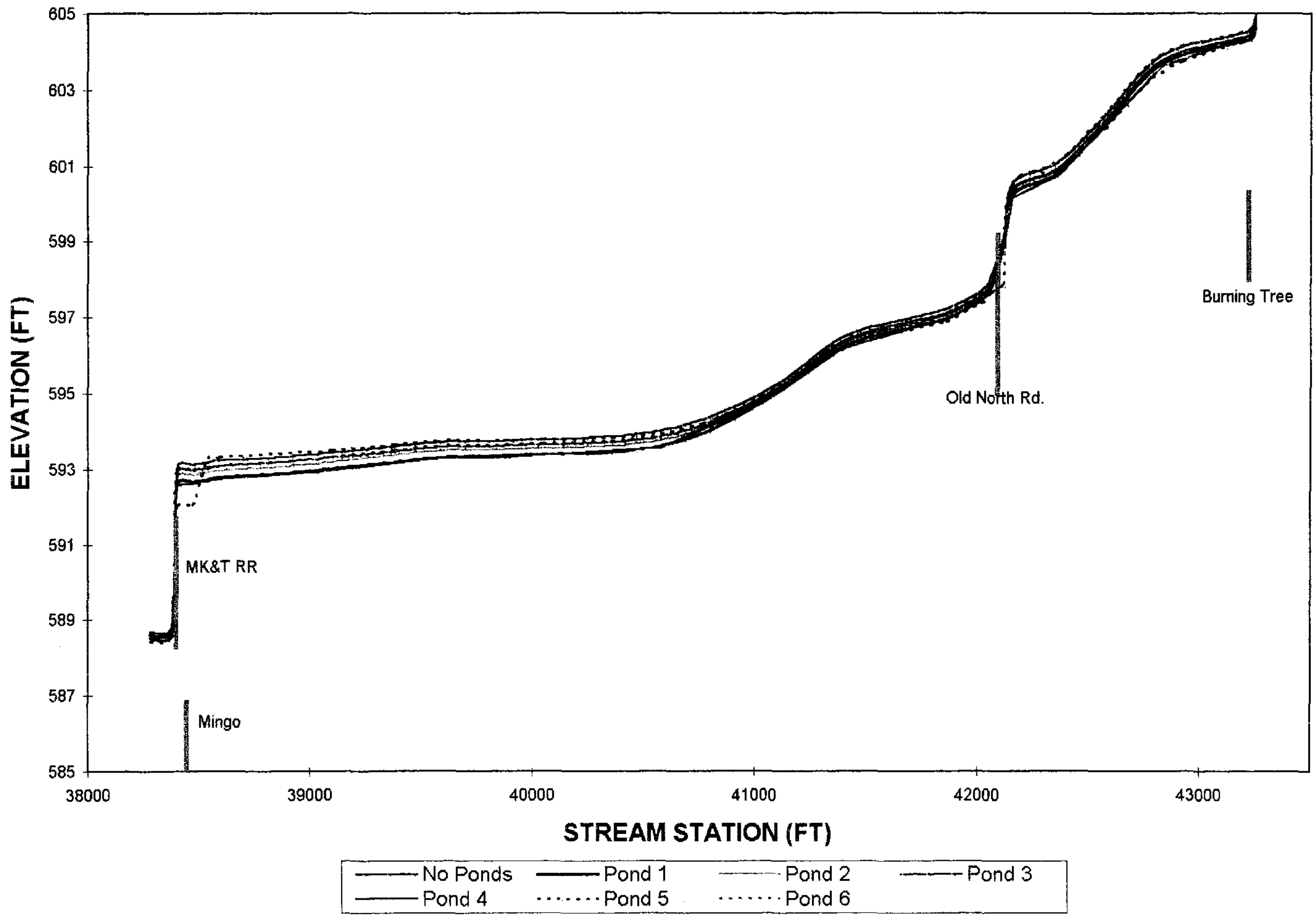
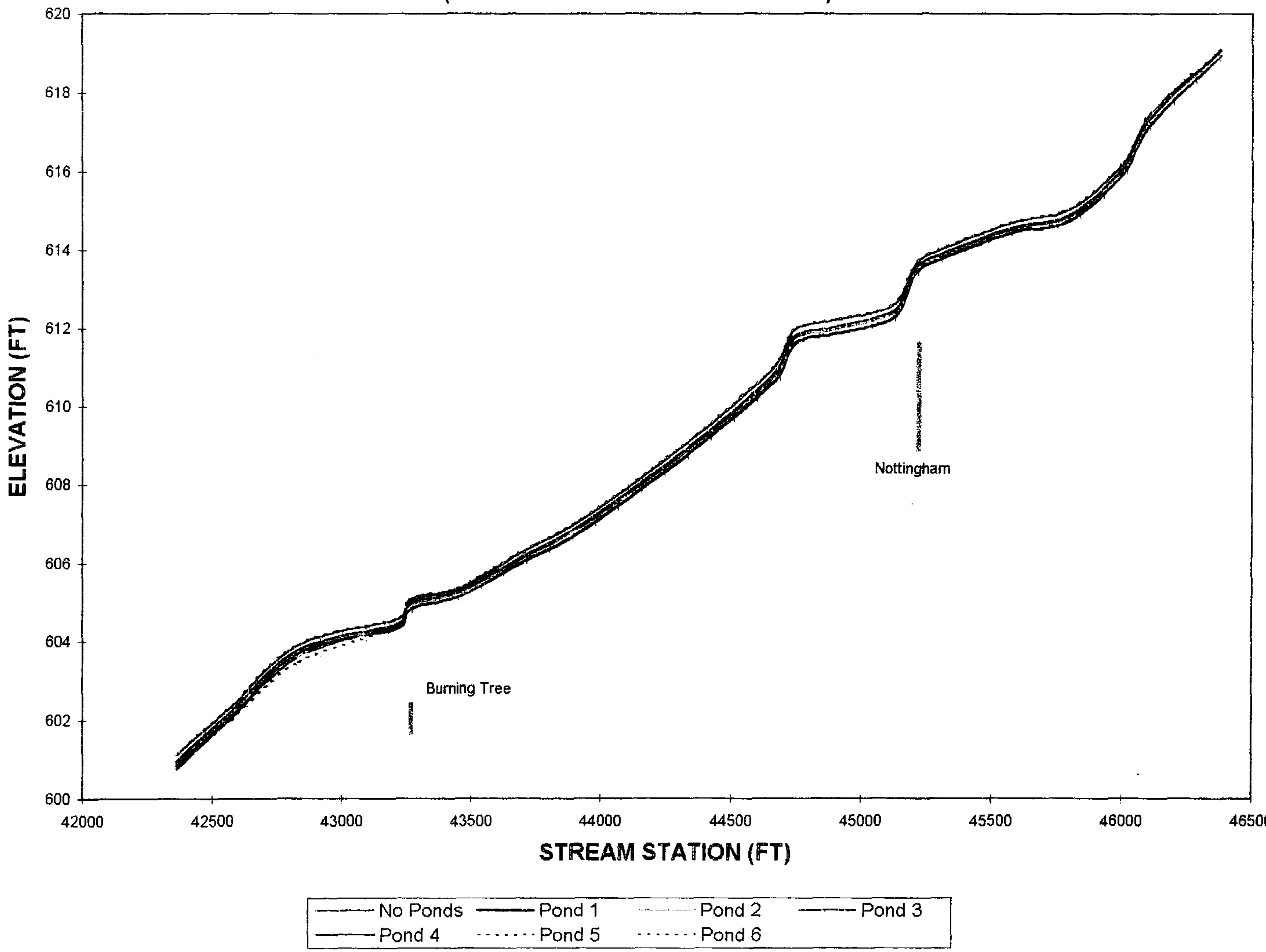


FIGURE II-15b. FLOODPLAIN COMPARISON
(BURNING TREE TO NOTTINGHAM)



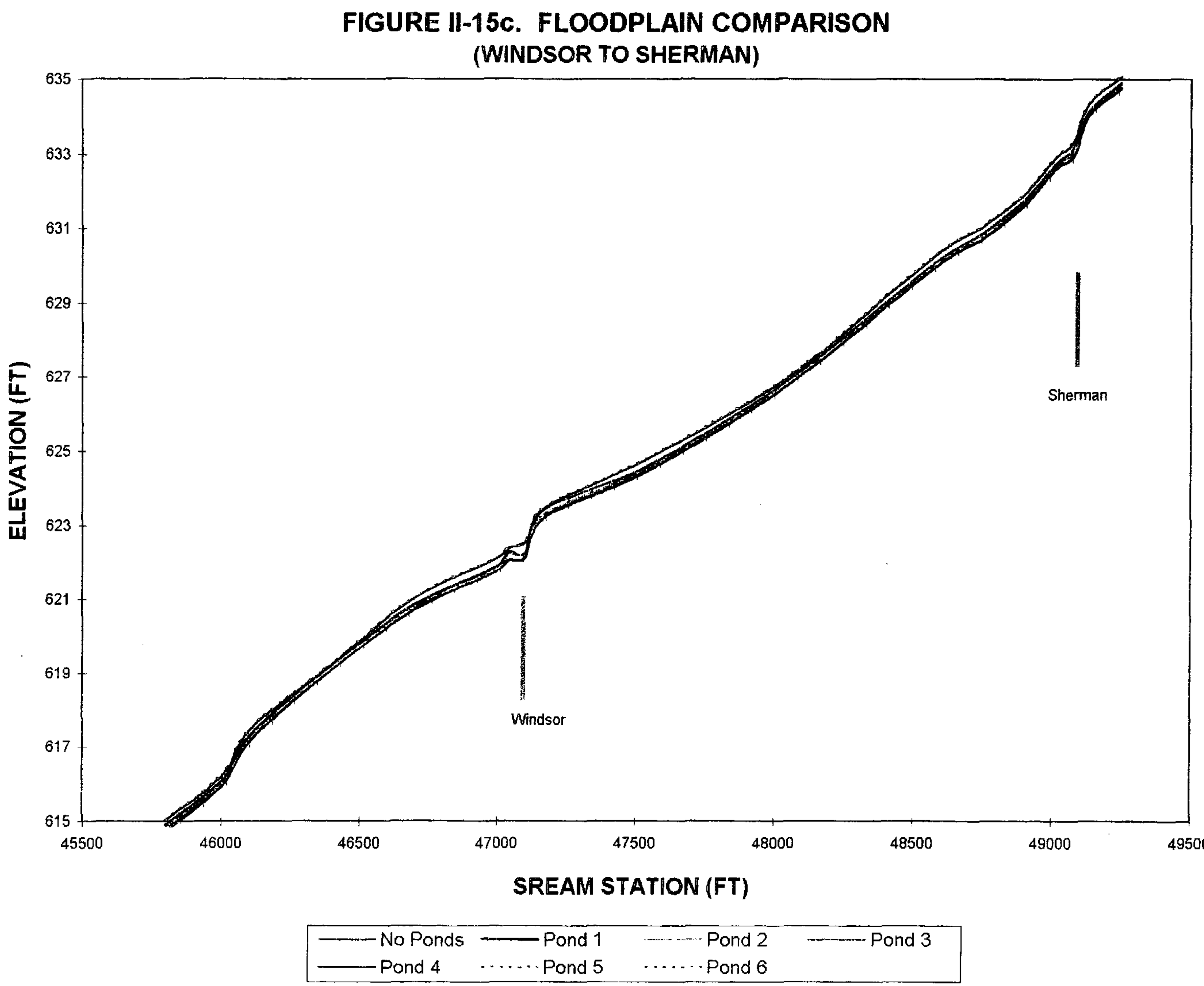


FIGURE II-15c. FLOODPLAIN COMPARISON
(WINDSOR TO SHERMAN)
635
633
631
629
627
625
623
621
619
617
615
ELEVATION (FT)
Sherman
Windsor
45500
46000
46500
47000
47500
48000
48500
49000
49500
SREAM STATION (FT)
No Ponds
Pond 1
Pond 2
Pond 3
Pond 4
Pond 5
Pond 6

FIGURE II-15d. FLOODPLAIN COMPARISON
(STUART ST.)

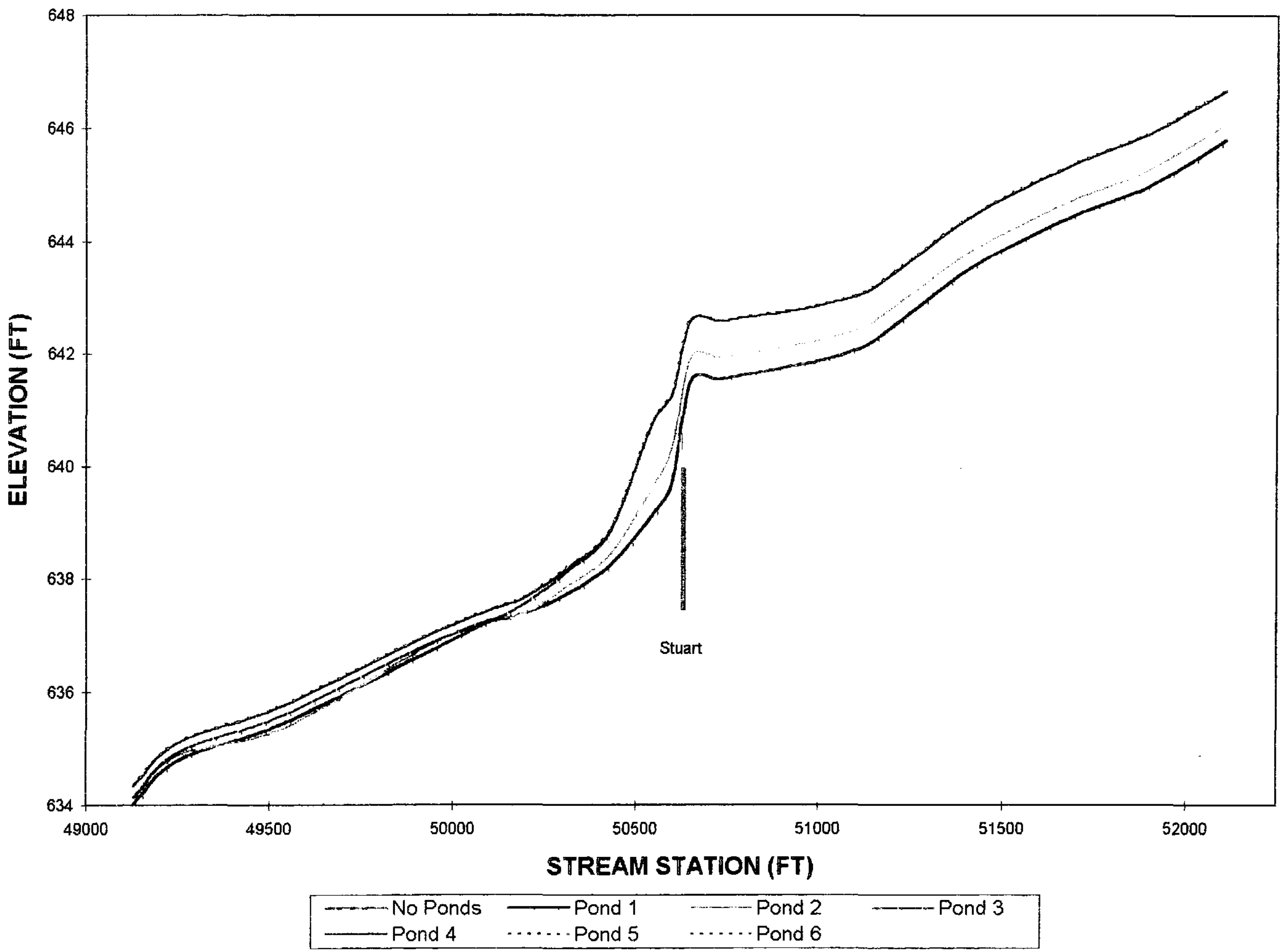
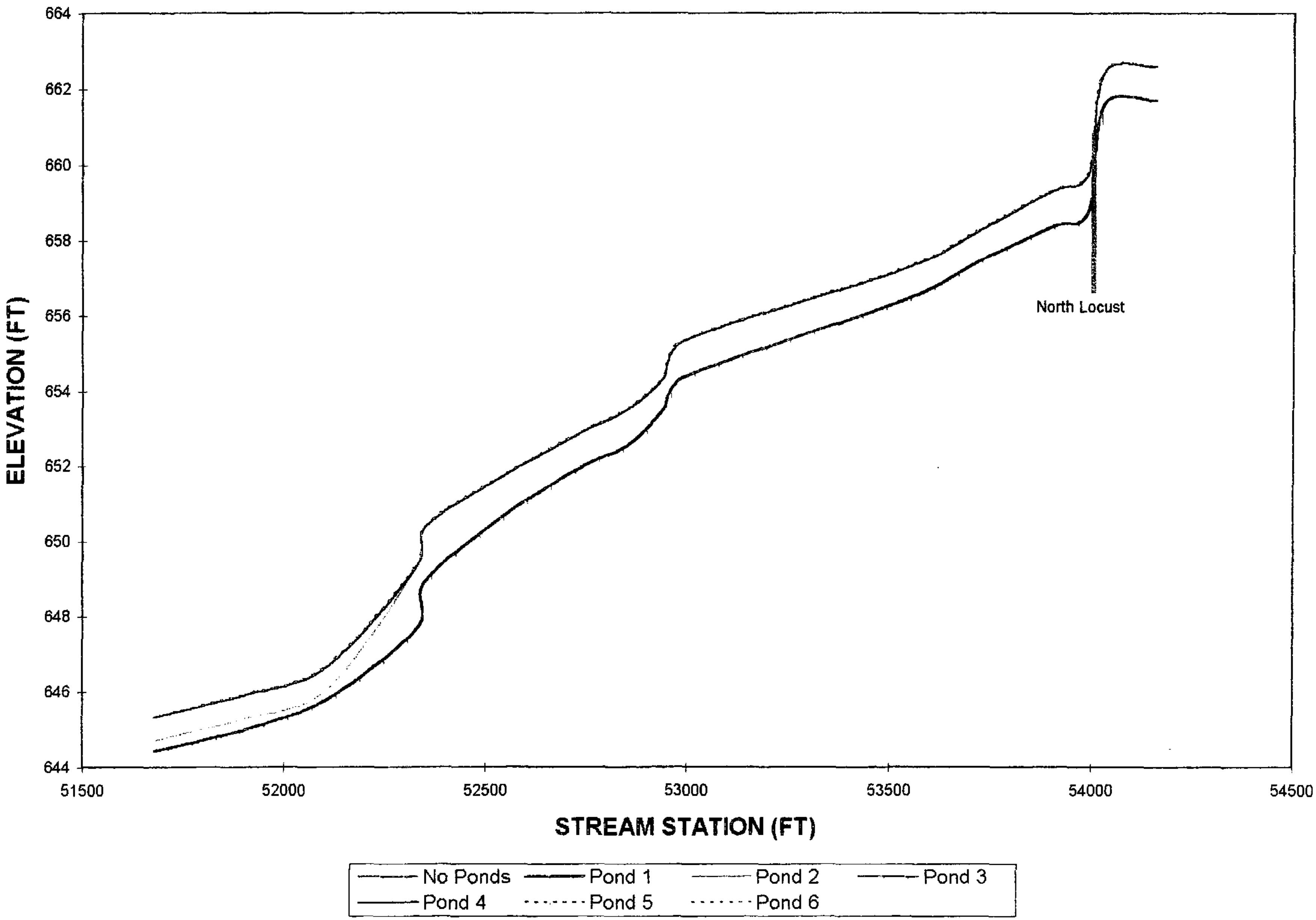


FIGURE II-15e. FLOODPLAIN COMPARISON
(NORTH LOCUST)



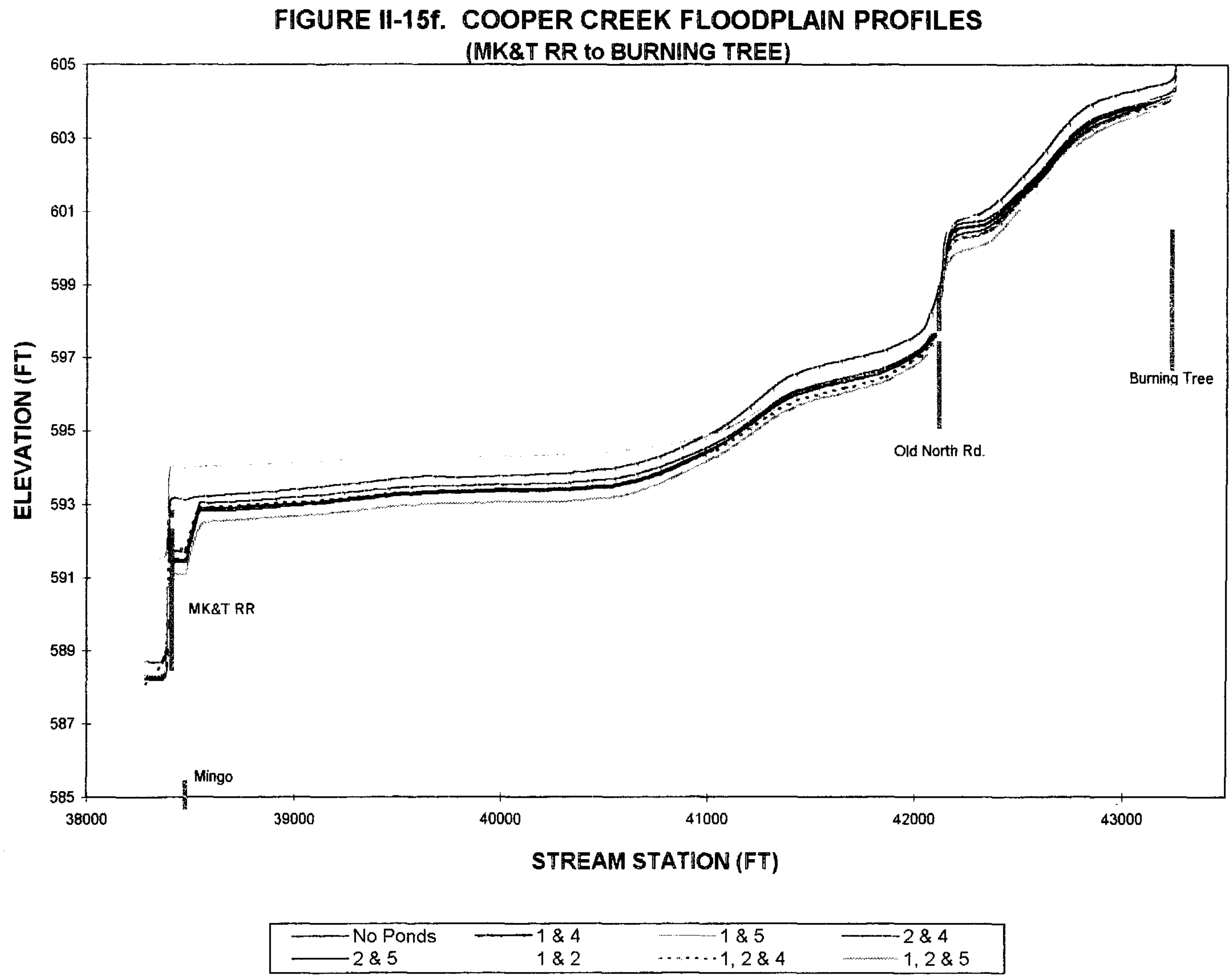


FIGURE II-15f. COOPER CREEK FLOODPLAIN PROFILES
(MK&T RR to BURNING TREE)
605
603
601
599
597
595
593
591
589
587
585
ELEVATION (FT)
Burning Tree
Old North Rd.
MK&T RR
Mingo
38000
39000
40000
41000
42000
43000
STREAM STATION (FT)
No Ponds
1 & 4
1 & 5
2 & 4
2 & 5
1 & 2
1, 2 & 4
1, 2 & 5

FIGURE II-15g. COOPER CREEK FLOODPLAIN PROFILES
(BURNING TREE TO NOTTINGHAM)

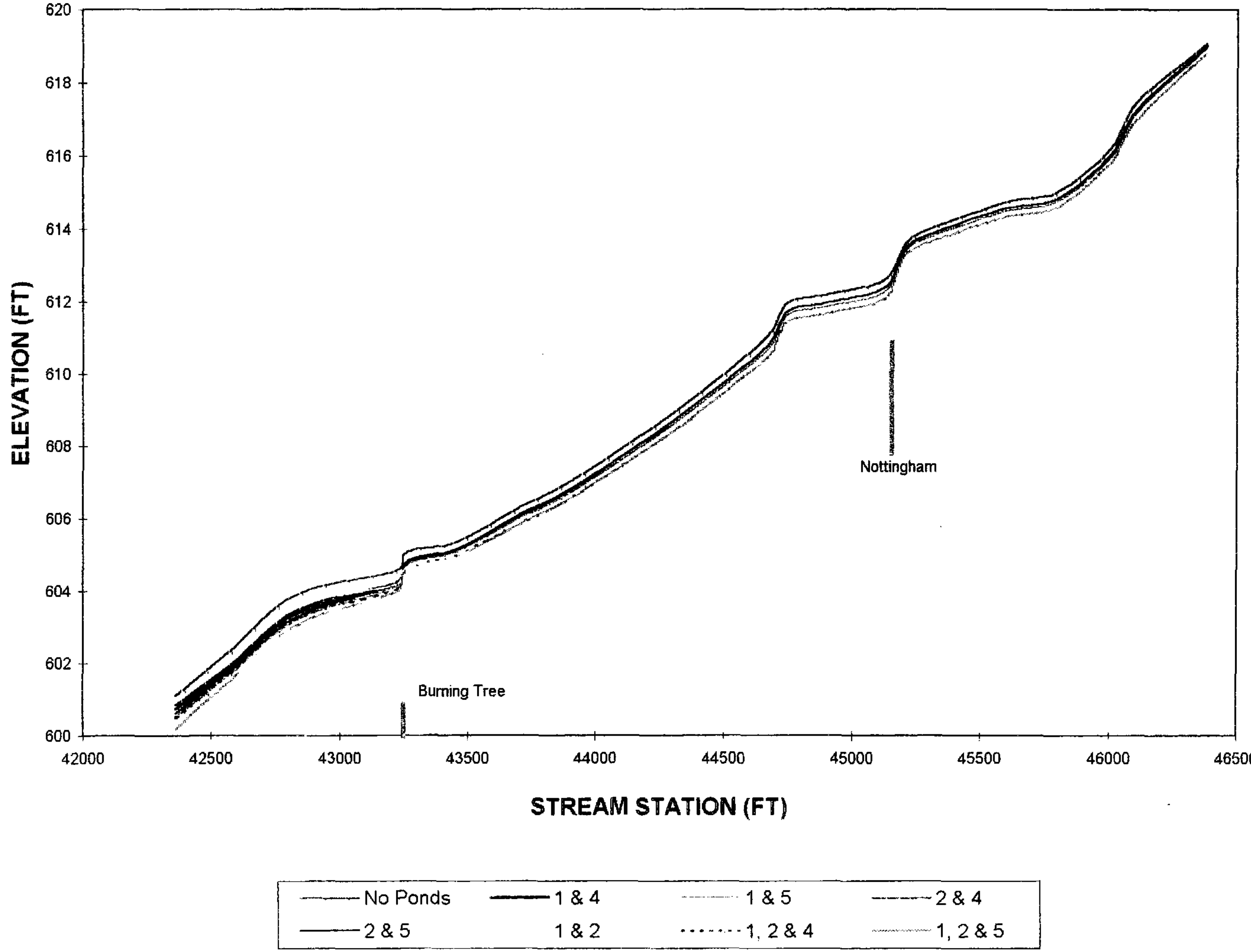


FIGURE II-15h. COOPER CREEK FLOODPLAIN PROFILES
(WINDSOR TO SHERMAN)

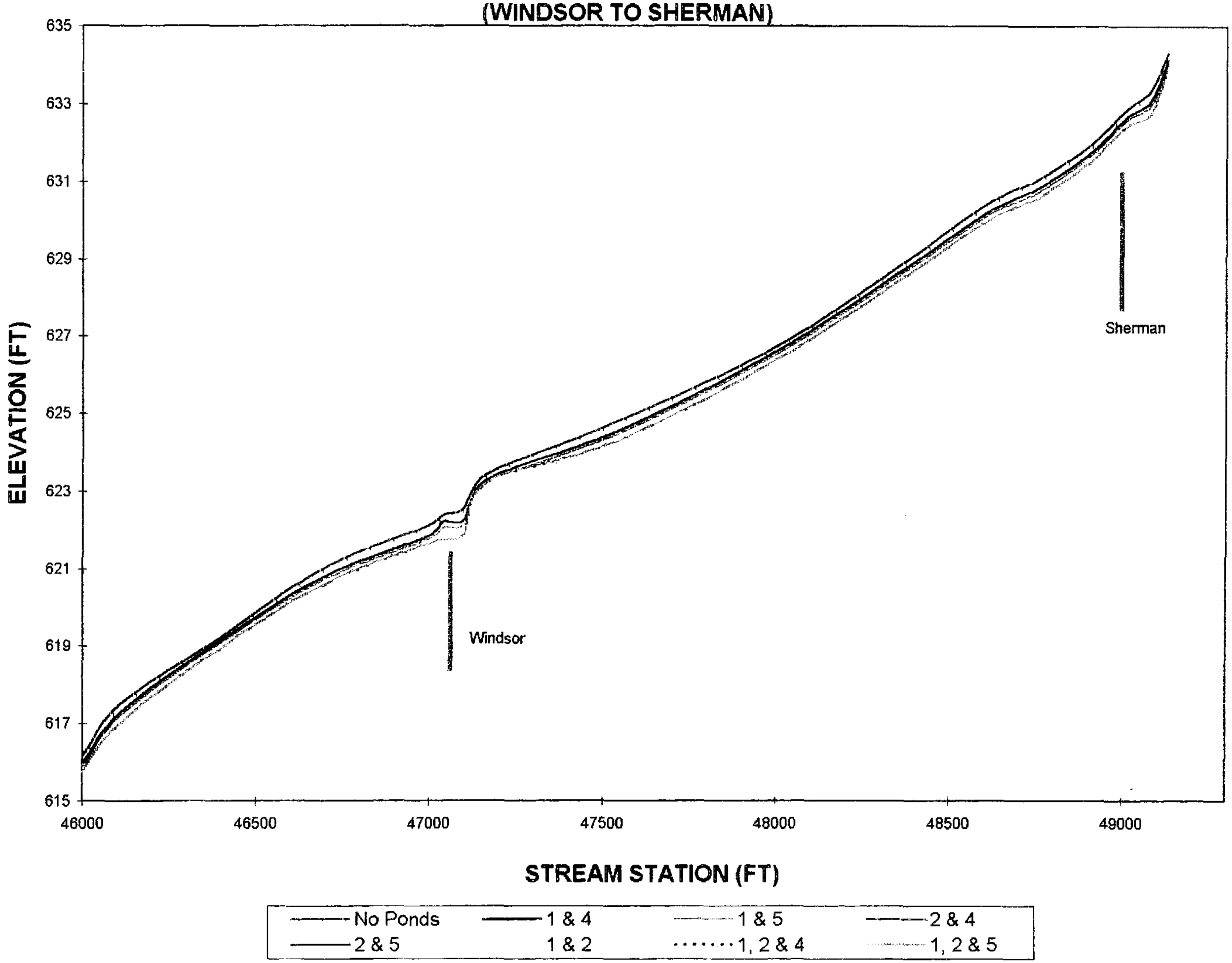


FIGURE II-15i. COOPER CREEK FLOODPLAIN PROFILES (STUART)

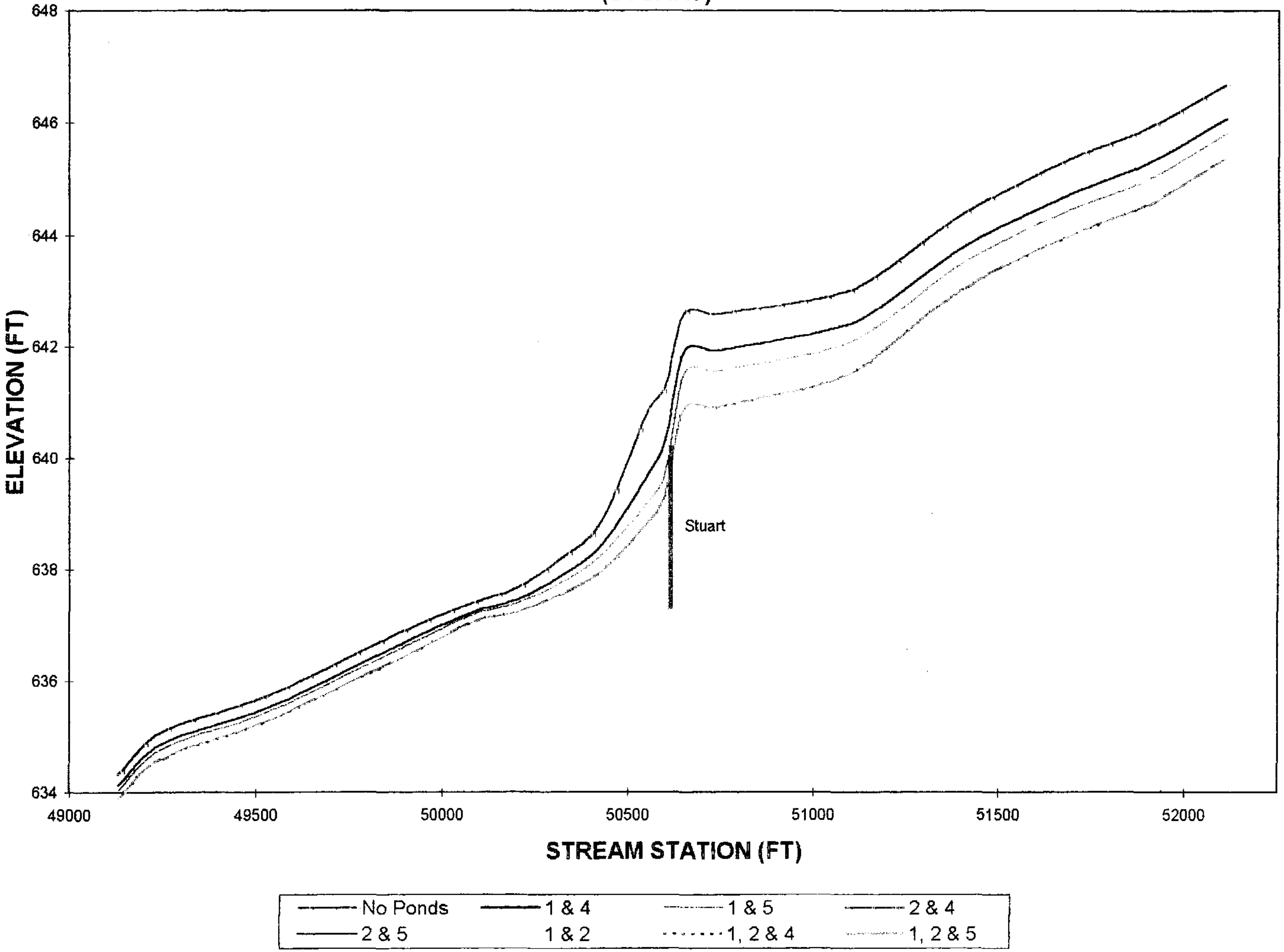
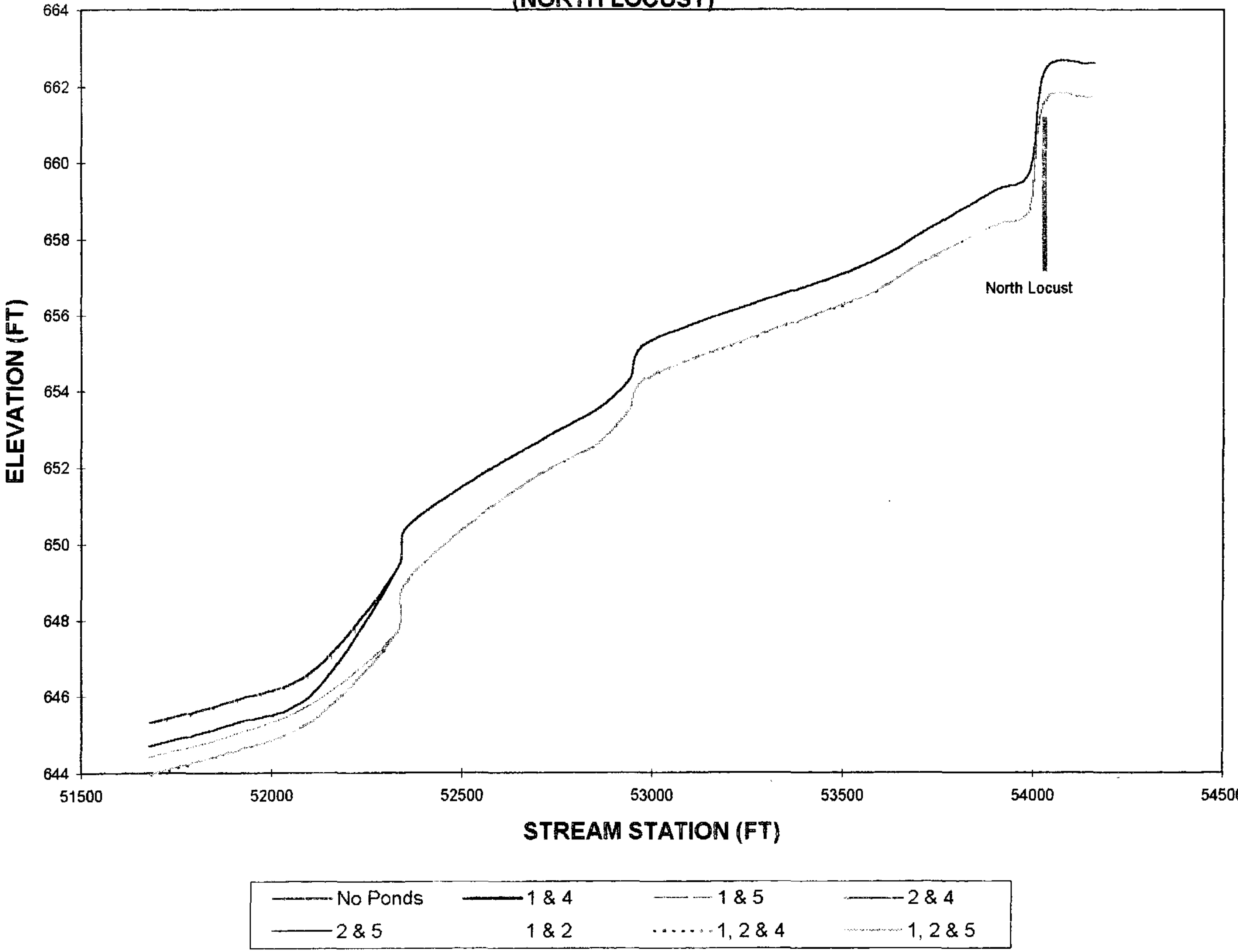
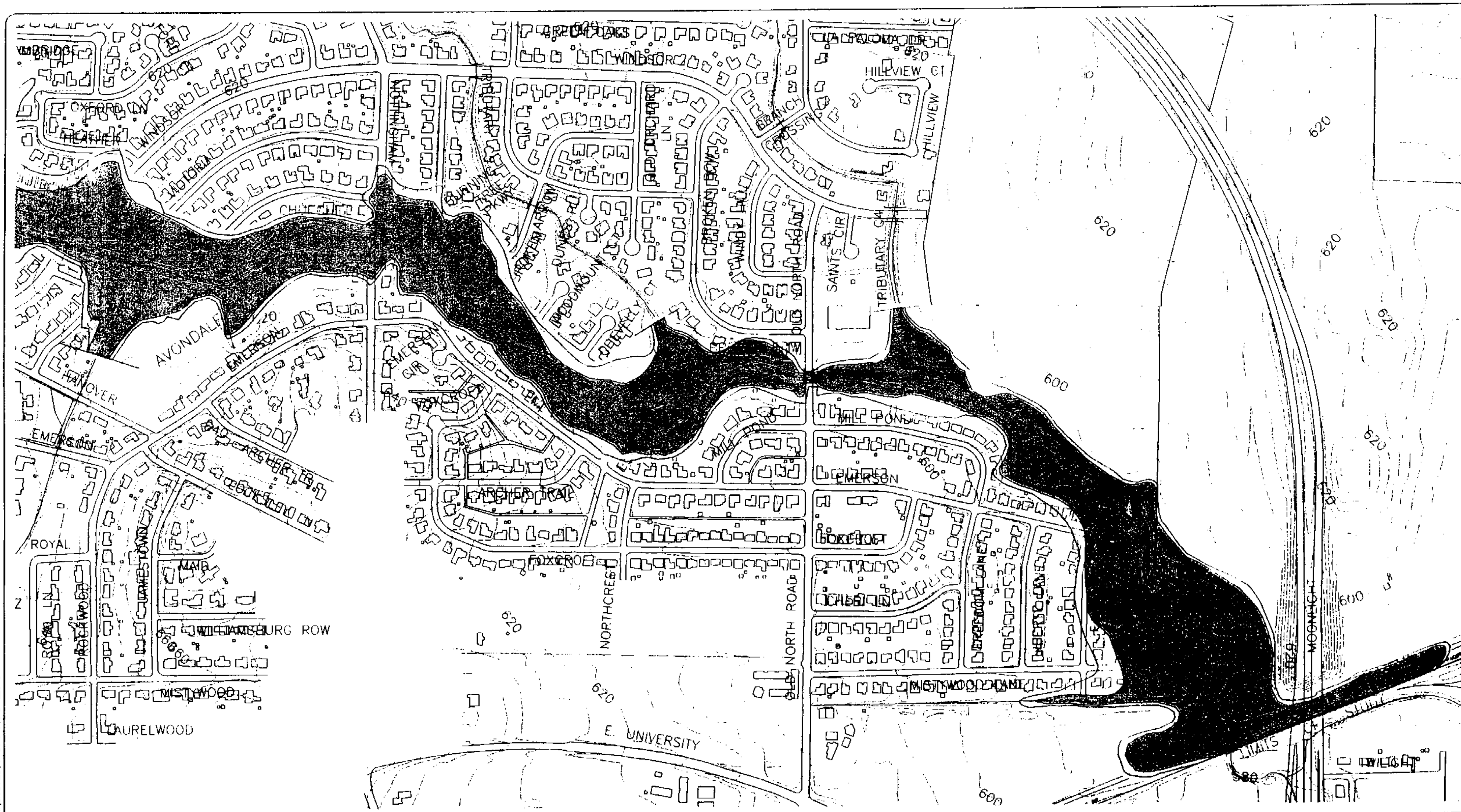


FIGURE II-15j. COOPER CREEK FLOODPLAIN PROFILES
(NORTH LOCUST)





LEGEND

ULT. WATERSHED DEVELOPMENT WITHOUT DETENTION PONDS

ULT. WATERSHED DEVELOPMENT WITH PONDS AT SITES NO 1 & 2

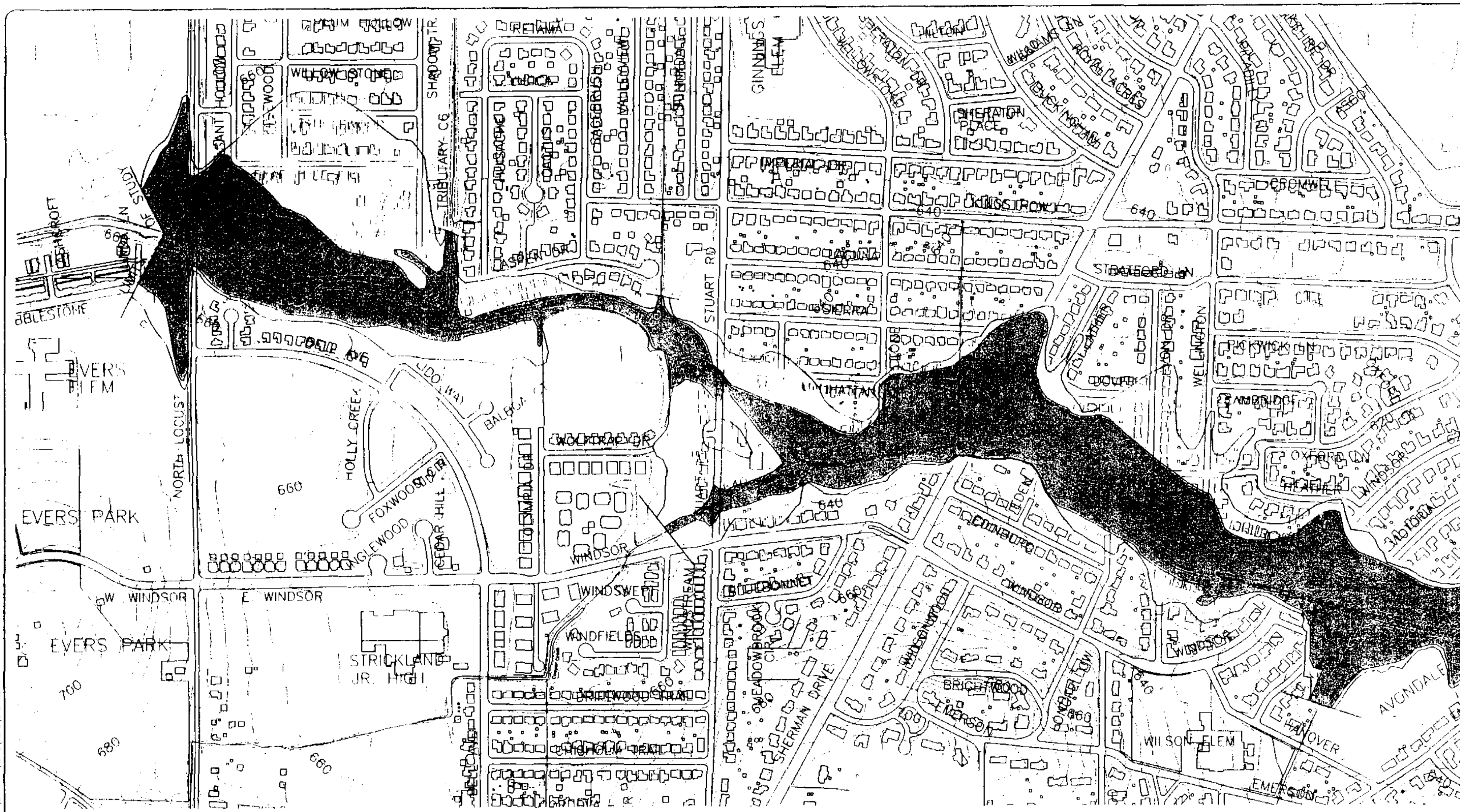
N

0 500' 1000'
1" = 500'

CITY OF DENTON
COOPER CREEK
REGIONAL DETENTON STUDY
COOPER CREEK 100-YEAR
FLOOD PLAIN COMPARISON

FREESE • NICHOLS

FIGURE II-16A



LEGEND

ULT. WATERSHED DEVELOPMENT WITHOUT DETENTION PONDS

ULT. WATERSHED DEVELOPMENT WITH PONDS AT SITES NO. 1 & 2

N

0 500' 1000'
1" = 500'

CITY OF DENTON
COOPER CREEK
REGIONAL DETENTON STUDY
COOPER CREEK 100-YEAR
FLOOD PLAIN COMPARISON

FREESE • NICHOLS

FIGURE II-16B

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combination and insufficient channel capacity between Old North Road and Burning Tree Lane. Future improvements to the Cooper Creek channel in this area should include increasing the channel capacity.

Each of the culverts at Nottingham, Windsor and Sherman are expected to be overtopped by the 100-year floodplain. The profiles suggest that each culvert causes an increase in the floodplain due to insufficient capacity. However, the overall floodplain profile from Old North to Sherman suggests that there is insufficient channel capacity and culvert improvements alone would not keep the floodplain from overtopping the roadway at these crossings.

The floodplain profile from Sherman Drive to Locust Street suggest that there is sufficient capacity in the channel not to cause submergence at Stuart and Locust. However, the profiles also suggest that the culvert capacity should be increased at these locations to eliminate backwater conditions that exist upstream of the crossings.

Channel Flow Velocities

Typical, soils will experience erosion when flow velocities reach six feet per second. The stormwater flow velocities predicted by the HEC-2 model suggest that the flow velocities within the Cooper Creek channel are above eight feet per second in several locations, particularly near the roadway culverts.

As shown in Table II-11, the anticipated channel flow velocities are not expected to be reduced significantly with the addition of the proposed detention facilities. An additional simulation was made using the HEC-2 model assuming 100% retention at the detention pond sites (i.e. no contribution to the Cooper Creek flows from the subbasins contributing to the detention ponds). The results indicate that there will be erosive velocities in the Cooper Creek channel whether or not the detention ponds are constructed, the channel flow velocities would remain erosive even if the detention ponds were design for zero release. The evaluation of erosion protection along the Cooper Creek stream is outside the scope of this study and further analysis should be performed as part of a comprehensive master plan for the Cooper Creek watershed.

Table II-11 Average Cooper Creek Channel Flow Velocities

	Present Development		Future Development		
Channel Reach	w/o Ponds	w/ Ponds	w/o Ponds	w/ Ponds	w/ Ponds and 100% Detention
Locust to Stuart	5.8	5.1	6.0	5.2	5.3
Stuart to Sherman	7.3	6.5	7.6	6.6	6.7
Sherman to Windsor	8.3	7.8	8.5	8.1	7.6
Windsor to Nottingham	8.7	8.4	8.8	8.5	8.0
Nottingham to Burning Tree	8.1	7.8	8.2	7.9	7.5
Burning Tree to Old North	8.6	8.7	8.6	8.7	9.0
Old North to Mingo	6.5	6.5	6.9	6.4	6.1

5.0 ANTICIPATED PROJECT COSTS

To assist in selecting an economical approach to floodplain management for the Cooper Creek watershed, probable construction costs were estimated for the conceptual detention pond configurations present in this study. The projected construction costs were then compared to the approximate value of property removed from the floodplain. A cost-benefit relationship was then developed for each of the twelve alternatives discussed.

Probable Construction Costs

Items considered in the conceptual construction estimates include site preparation, excavation, fill placement and compaction, sodding, outlet structures, and land purchases. The estimates also include provisions for mobilization, contractor overhead and profit, and contingencies. There may be additional items, such as athletic field equipment, to consider upon the final design of the detention ponds that are not included in these estimates.

Table II-12 contains a summary of the estimated costs associated with construction of the detention facilities. Detailed breakdowns of the estimated construction costs are listed in Appendix C.

Cost-Benefit Analysis

The cost of constructing the detention ponds and the various options were compared to the approximate benefit of the reduced 100-year floodplain. The expected benefit was assumed to be the recovery of property value if the property is removed from within the Cooper Creek 100-year floodplain limits. The area recovered was determined by averaging the change in floodplain width at each section in the hydraulic model and multiplying by the channel reach length between cross sections.

Table II-12 Probable Construction Costs for Regional Detention

Detention Pond Option	Estimated Construction Costs[a]	Estimated Value of Property Removed From the Floodplain[b]	Overall "Cost" of Detention Pond Option
#1 Only	$3,012,000	$785,700	$2,226,300
#2 Only	$2,256,400	$361,800	$1,894,600
#3 Only	$701,500	$211,800	$489,700
#4 Only	$1,684,400	$127,300	$1,557,100
#5 Only	$1,378,100	$65,200	$1,312,900
#6 Only	$1,040,500	$32,400	$1,008,100
#1 & #4	$4,696,400	$831,600	$3,864,600
#1 & #5	$4,390,100	$796,100	$3,594,000
#2 & #4	$3,940,800	$419,300	$3,521,500
#2 & #5	$3,634,500	$477,100	$3,157,400
#1 & #2	$5,268,400	$926,500	$4,341,900
#1, #2 & #4	$6,952,800	$1,037,800	$5,915,000
#1, #2 & #5	$6,646,500	$1,113,100	$5,533,400

a. Include cost of purchasing property for pond construction based on $1.00 per square foot.

b. Values of property removed from floodplain are based on $0.85 per square foot.

Based on information obtained from the City staff, typical property values along Cooper Creek are roughly $1.00 per square foot. The approximate value of the property within the floodplain was assumed to be valued at 15% of the property outside the floodplain limits, or $0.15 per square foot.

In general, as the benefit of the detention pond scenario goes up, so does the anticipated construction cost. The cost-benefit relationship for the detention pond scenarios is illustrated in Figure II-17. Of the individual pond scenarios, construction of

January 7, 1997

ERRATA SHEET

The anticipated construction cost listed in the second paragraph of the Executive Summary for the recommended detention pond option should be $5,268,400.

The following table replaces the table on Page II-24:

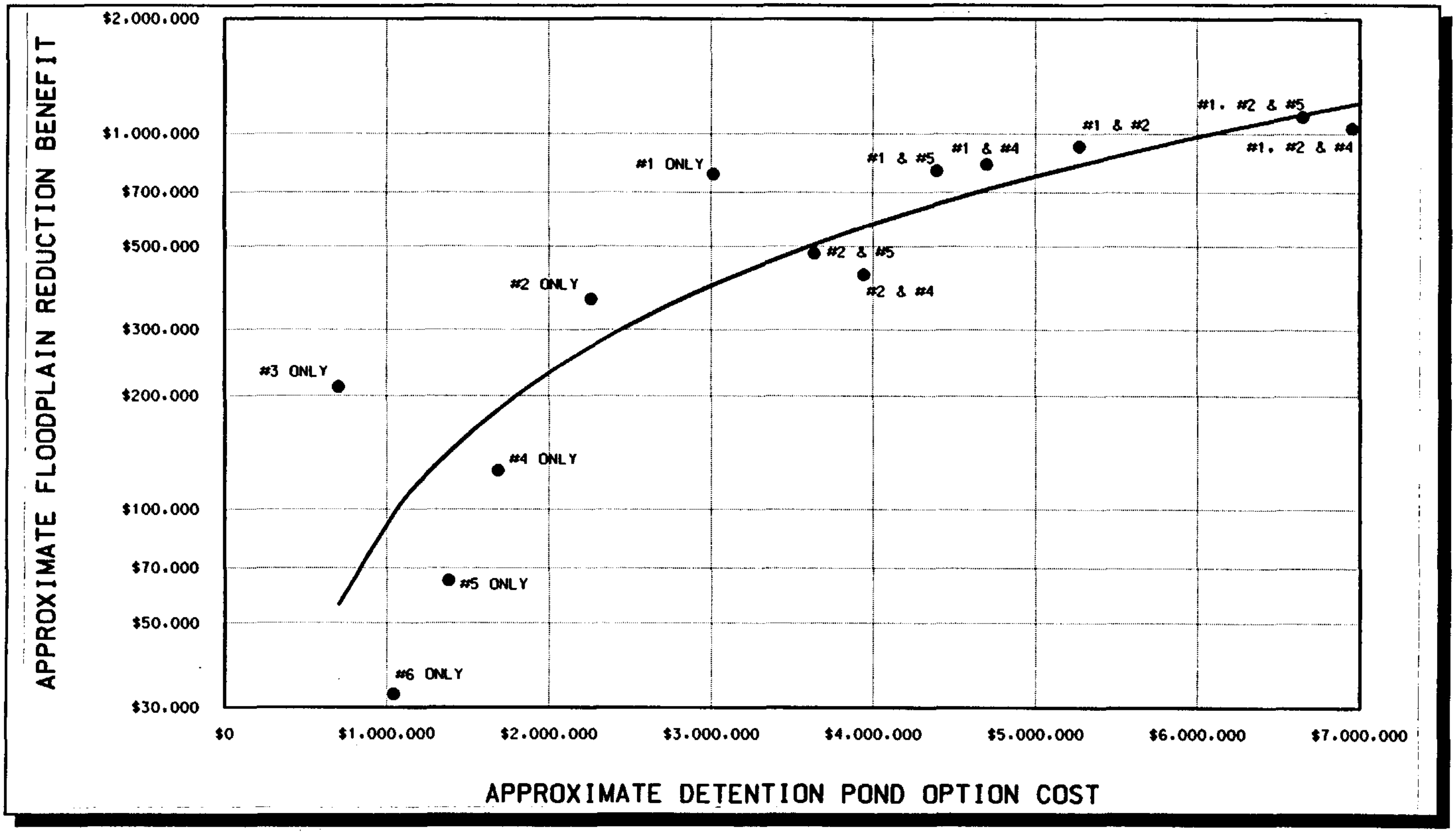
Table II-12 Probable Construction Costs for Regional Detention

Detention Pond Option	Estimated Construction Costs[a]	Estimated Value of Property Removed From the Floodplain[b]	Overall "Cost" of Detention Pond Option
#1 Only	$3,012,000	$785,700	$2,226,300
#2 Only	$2,256,400	$361,800	$1,894,600
#3 Only	$701,500	$211,800	$489,700
#4 Only	$1,684,400	$127,300	$1,557,100
#5 Only	$1,378,100	$65,200	$1,312,900
#6 Only	$1,040,500	$32,400	$1,008,100
#1 & #4	$4,696,400	$831,600	$3,864,600
#1 & #5	$4,390,100	$796,100	$3,594,000
#2 & #4	$3,940,800	$419,300	$3,521,500
#2 & #5	$3,634,500	$477,100	$3,157,400
#1 & #2	$5,268,400	$926,500	$4,341,900
#1, #2 & #4	$6,952,800	$1,037,800	$5,915,000
#1, #2 & #5	$6,646,500	$1,113,100	$5,533,400

a. Include cost of purchasing property for pond construction based on $1.00 per square foot.

b. Values of property removed from floodplain are based on $0.85 per square foot.

Figure II-17 - COST-BENEFIT RELATIONSHIP FOR REGIONAL DETENTION POND OPTIONS



the pond at Site #1 provides the greatest benefit, but it also has the highest cost. The combinations of ponds that include Site #1 vary slightly in benefit (within $317,000) but vary significantly in construction cost (over a range of $2,563,000). The detention pond combination scenarios without the pond at Site #1 provide a lower overall benefit.

6.0 TNRCC CONSIDERATIONS

The Texas Natural Resource Conservation Commission regulations governing detention facilities were reviewed for their applicability to the proposed detention facilities. The TNRCC typically requires any dam designed to impound floodwater that is six feet or greater in height to be permitted. Although the detention facilities proposed in this report are higher than six feet, no permitting will be required as long as there is no permanent storage of water within the detention facilities.

Therefore, since the proposed detention facilities are not going to permanently store water, no TNRCC permits will be required. However, should the final design of a detention pond incorporate permanently ponded water to enhance the aesthetic quality of the detention facility (i.e. such as in a park), an application must be made to the TNRCC for a permit.

7.0 REGIONAL DETENTION RECOMMENDATIONS

Based on the results of the detention pond analysis, floodplain management for the Cooper Creek watershed should include detention ponds at Sites #1 and #2. In addition, further investigation is needed to determine channel and culvert improvements along Cooper Creek.

The regional detention pond recommendation is based on the prioritization of the thirteen detention pond scenarios evaluated. For each scenario, a priority ranking was developed by assigning values based on the anticipated construction costs, estimated floodplain reclamation benefit and overall reduction of the 100-year floodplain at the culvert crossings. Table II-13 provides a listing of the ranking categories and associated values, and Table II-14 contains the ranking of the detention pond scenarios.

Table II-13 Detention Pond Priority Ranking Schedule

Ranking Category	Assigned Value						
	7	6	5	4	3	2	1
A. Est. Const.. Cost (Millions of Dollars)	< $1	$1-2	$2-3	$3-4	$4-5	$5-6	> $6
B. Benefit (Millions of Dollars	> $1	$0.8-1.0	$0.6-0.8	$0.4-0.6	$0.25-0.4	$0.1-0.25	< $0.1
C. Benefit as % of Cost	> 30%	25-30%	20-25%	15-20%	10-15%	5-10%	< 5%
D. Floodplain Reduction at Culverts	> 40%	35-40%	25-30%	20-25%	15-20%	10-15%	< 10%

Table II-14 Detention Pond Option Ranking

Detention Pond Option	A	B	C	D	Overall Score
#1 & #2	4	6	5	6	21
#1, #2 & #5	1	7	3	7	18
#1 Only	4	5	5	3	17
#1 & #4	3	6	3	5	17
#1, #2, & #4	1	7	2	7	17
#3 Only	7	2	6	1	16
#1 & #5	3	5	3	5	16
#2 Only	5	6	3	1	15
#2 & #4	4	4	2	3	13
#2 & #5	4	4	2	3	13
#4 Only	6	2	1	1	10
#5 Only	6	1	1	2	10
#6 Only	6	1	1	1	9

The options with pond at Sites #3 and #6 are not recommended since the available storage at these sites is not large enough to significantly reduce the 100-year Cooper Creek floodplain. Scenarios with single ponds at Sites #4 and #5 provide the minimal reductions in the 100-year floodplain. The three-pond combinations are relatively expensive and do not provide a significant amount of benefit beyond the two-pond combinations.

Therefore, with these considerations in mind and the priority ranking, the recommended option is to construct regional detention ponds at Sites #1 and #2. If a phased approach to implementing the recommended detention pond construction is desired, it is recommended to construct the pond at Site #1 prior to the pond at Site #2.

The detention pond layouts presented in this report are conceptual in nature, and the final configuration and location of the ponds should be determined during the final design. Channel velocities within the Cooper Creek channel are expected to be erosive whether or not the regional detention ponds are constructed. Therefore, further investigations should be made to determine the appropriate measures needed to protect the maintain the floodplain within the channel banks, protect the channel and banks from erosion, and evaluate the capacity requirements for the culvert crossings along Cooper Creek.

PART III - PECAN CREEK TRIBUTARY PEC-4 IMPROVEMENTS

1.0 BACKGROUND

The objective of the Pecan Creek Tributary PEC-4 evaluation is to investigate and present a conceptual channel improvement design that will reduce the 100-year floodplain width for the portion of Tributary PEC-4 between Locust Street to the main stem of Pecan Creek. The conceptual channel improvement developed in this study was based on hydrologic and hydraulic analyses of the PEC-4 stream.

The existing PEC-4 100-year floodplain exceeds the limits of the channel banks for most of the stream downstream of Locust Street. The railroad culvert on PEC-4 near the Bell Avenue and Robertson street intersection is expected to cause stormwater to be diverted north along Bell Avenue.

In keeping with the 1990 City of Denton Drainage Design Criteria Manual, the goal of the PEC-4 channel improvement study is to evaluate the channel improvements required to reduce the 100-year floodplain an keep it within the channel banks. To accomplish this objective, the capacity of all the bridges and culverts downstream of Locust Street were evaluated to determine the need for improving each crossing to be in conformance with the City's 1990 criteria (100-year frequency).

2.0 RECONNAISSANCE AND DATA COLLECTION

Coordinate with City Staff

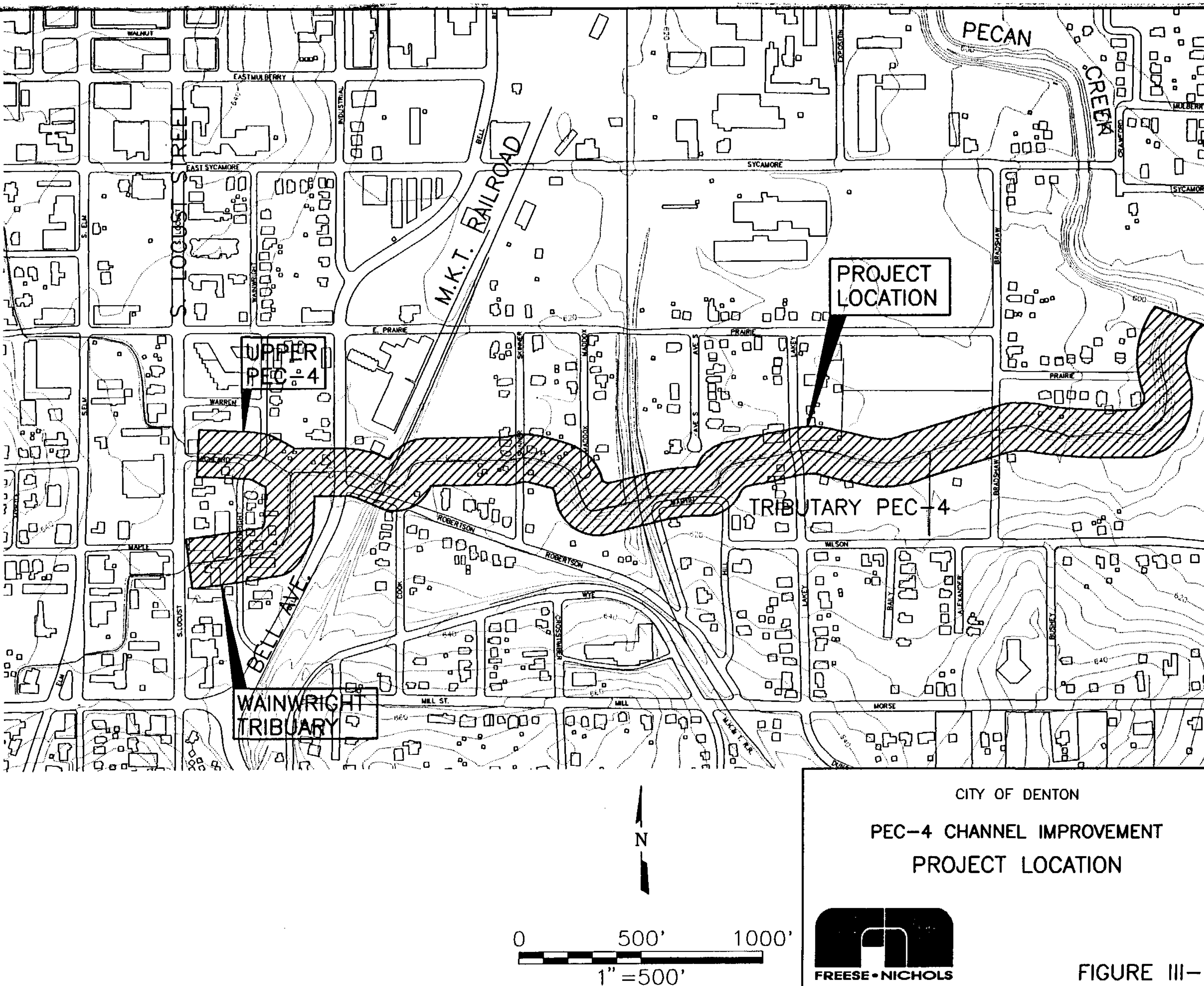
At the beginning of the project, a coordination meeting was held with the City staff to discuss the scope of the channel improvement evaluation. The project team reviewed the past flooding problems along the PEC-4 tributary. The City is particularly concerned with the backwater conditions that exist upstream of the M.K.T. Railroad near the intersection of Bell Avenue and Robertson Street. The project limits include Tributary PEC-4 and the small tributary to PEC-4 east of Locust street to the confluence with the main stem of Pecan Creek (see Figure III-1).

Site Investigation

Following the coordination meeting with the City staff, the design team performed a visit to the project site. In general, the existing concrete-lined channel appears to be in relatively good shape, given the age of the concrete lining. There were several areas, however, where the concrete channel side had heaved and/or slide downward as a result of deterioration of the toe of the sideslope paving..

Figure III-2 shows a problem that was fairly typical along the PEC-4 channel. A shown in the photographs, the original connection between the channel bottom and the sideslope paving has deteriorated, leaving the wire mesh reinforcement exposed and susceptible to corrosion. In many cases, there is no longer a connection between the slope paving and the channel bottom. This creates a situation where the soil behind the slope paving can wash away.

The deterioration of the connection between the slope paving and the channel



CITY OF DENTON

PEC-4 CHANNEL IMPROVEMENT

PROJECT LOCATION

FREESE • NICHOLS

FIGURE III-1

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Figure III-2a - Typcial Channel Deterioration

Figure III-2b - Gap Between Sideslope and Channel Bottom

bottom can lead to erosion of the soil behind the slope paving and eventual failure of the channel section. Figure III-3 shows a typical sideslope failure in which the bottom of the slope paving section slid toward the center of the channel. As shown in Figure III-3b, the bottom of the slope pavement section at this location slid approximately five inches.

Continued loss of soil behind the slope pavement can lead to more serious slope paving failure. Figure III-4 illustrates contains photographs of a section of channel near Robertson Street just downstream of the M.K.T. Railroad. A section of the concrete sideslope paving has buckled and slid downward, leaving a scour hole approximately seventeen inches deep behind the slope pavement (Figure III-4b).

There are also signs of movement at a few of the culverts along Tributary PEC-4. Figure III-5 shows the movement of one of the culvert wingwalls at the Lakey Street crossing. Further movement can lead to complete loss of soil behind the wingwall and failure of the utility pole guy wires. The existing culvert wingwall has moved approximately seven inches toward the channel.

Computer Models

Hydrologic and hydraulic stream computer models of PEC-4 were obtained from the COE used as reference models for this study. These models were developed by the Corps as part of the City of Denton Flood Insurance Rate Study released in 1987. The model developed by the Corps' included a split flow near Bell Avenue, upstream of the M.K.T. Railroad crossing.

A new HEC-1 model was developed based on the existing Corps model. Discharges were computed at each of the culvert crossings between Locust Street and the main stem of Pecan Creek. The discharges computed by the HEC-1 model were input in to a new hydraulic stream model of PEC-4. The new hydraulic model was developed in the HEC-RAS format. For the purposes of channel improvement development, all discharges were assumed to be contained within the PEC-4 watershed (i.e. no split flow is present).

Field Surveys

Field surveys of Tributary PEC-4 were performed to obtain the stream geometry information needed for the hydraulic modeling. Field data were collected along the channel reach and in the overbank areas within twenty-five feet of the channel. A topographic map of the area surveyed was prepared showing the visible features within the study area, which was used as the base map for the study.

Existing Utilities

Maps were obtained from the City staff showing the locations of water and sanitary sewer lines along the PEC-4 study area. In general, there is a water and sanitary sewer line crossing the PEC-4 stream at each street crossing. In some instances, an existing gas line crossing was noted in addition to the water and sanitary sewer. Also, there is an existing six-inch water line crossing the channel just east of the abandoned railroad track near Avenue S, and an existing twelve-inch sanitary sewer crossing just upstream of Bradshaw Street.

The design of future improvements to the PEC-4 channel should include the possible relocation of these utilities. In some cases, the existing sanitary sewer line may already be at the minimum slope allowable, preventing any relocation of the sewer line and

Figure III-4a - Sideslope Failure (Heaving and Sliding)

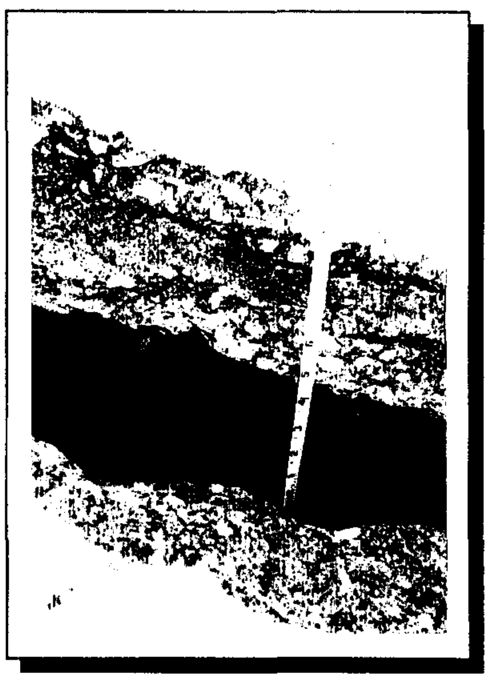


Figure III-4b - Soil Loss Behind Sideslope Pavement Slab

Figure III-5a - Beginning of Soil Loss Failure at Lakey Street

Figure III-5b - Wingwall Movement

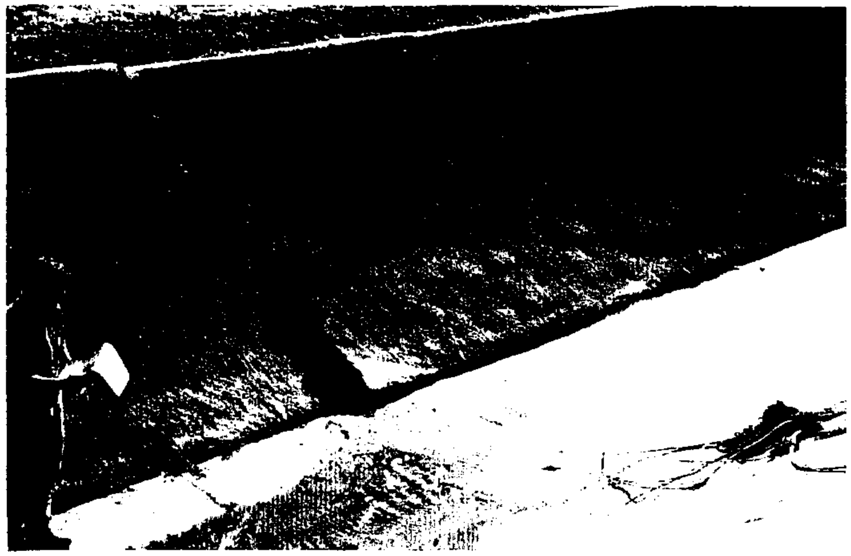


Figure III-3a - Sideslope Failure (Sliding)

Figure III-3b - Movement of Sideslope Pavement Slab requiring an aerial crossing. The other utilities, including gas, water, electric, etc., may be relocated to accommodate future PEC-4 channel improvements, since they do not require gravity flow conditions.

3.0 WATERSHED HYDROLOGY

The previous floodplain delineation performed Corps of Engineers was based on discharges computed using the NUDALLAS computer program. The Corps' discharges used in their floodplain analysis included diversion of flow from the PEC-4 watershed to the main stem of Pecan Creek. The flow diversion was developed due to insufficient capacity of the culvert at the M.K.T. Railroad crossing.

The purpose of the hydrologic analysis in this study is to develop new discharges that do not reflect a diversion of storm water at the M.K.T. Railroad culvert. The PEC-4 hydraulic analysis would then be based on the new discharges and the floodplain limits would be re-delineated.

Watershed Characteristics

The PEC-4 watershed is located in the southeastern portion of the City of Denton and drains into Pecan Creek. The upper reach of the watershed is located within the University of North Texas campus near Avenue C. There are into two branches of Tributary PEC-4 in the upper portion of the watershed that converge between Bell Avenue and Wainwright Street, approximately 500 feet downstream of Locust Street. For this study, the northern branch is referred to as "Upper PEC-4" and the southern branch is referred to as "Wainwright Tributary". Below the confluence of these two branches, the existing channel, hereinafter referred to as "Lower PEC-4" flows east approximately 4,380 feet before discharging into Pecan Creek.

The PEC-4 watershed includes approximately 1.56 square miles, most of which is presently developed commercial or light industrial. There are some residential areas located in the portion of the PEC-4 watershed east of the M.K.T. Railroad.

The elevation of the terrain ranges from approximately 726 feet National Geodetic Vertical Datum at the upstream end of the watershed to approximately 589 feet NGVD at Pecan Creek.

The PEC-4 watershed is located in a region of mean temperate climatological conditions, experiencing occasional extremes of rainfall of relatively short duration. Climatological data available from the Denton 2 SE weather station at the city's wastewater treatment plan are considered indicative of the conditions prevailing in the region. There are no official stream gaging stations located in the Pecan Creek watershed.

Peak Discharge Development

Since the Corps' hydrologic model included split flow at the M.K.T. Railroad, new peak discharges were developed at key locations along the PEC-4 reach. Hydrographs were developed using HEC-1 to determine the peak 100-year discharges for use in the hydraulic stream model. The locations of the HEC-1 design points are illustrated in Figure III-6.

The PEC-4 watershed was divided into 7 subbasins for determination of the design flows along PEC-4. Dividing the basin into smaller subbasins provides more detailed information at the major channel crossings which will be used as design points for the channel improvements. Figure III-7 is shows the location of the subbasins at each of the HEC-1 design points.

Synthetic unit hydrographs were developed for the 10-, 25-, 50-, 100-, and 500-year frequency storms using the Snyder method within HEC-1. Precipitation and precipitation loss data used to develop the hydrograph was obtained from the *Denton Drainage Criteria Manual* (refer to discussion In Section 3.0 of Part I).

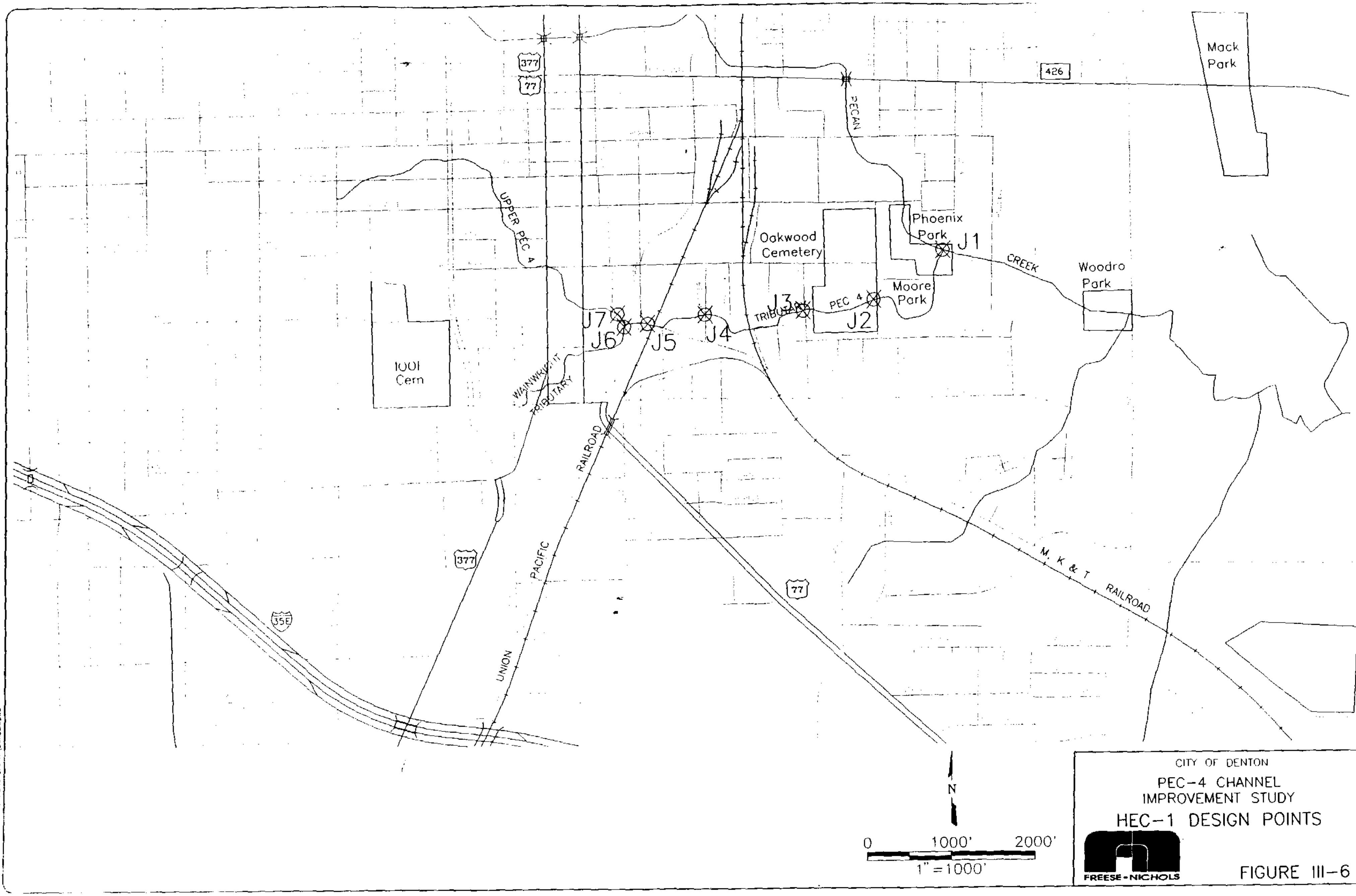
The soil maps for the County of Denton suggest that there are eight classifications of soil types within the PEC-4 watershed. Each of these soil types were divided into sandy or clay soils (Figure III-8). Table III-1 contains a summary of the percentages of sand and clay within the PEC-4 watershed.

Table III-1 PEC-4 Watershed Subbasin Soil Composition

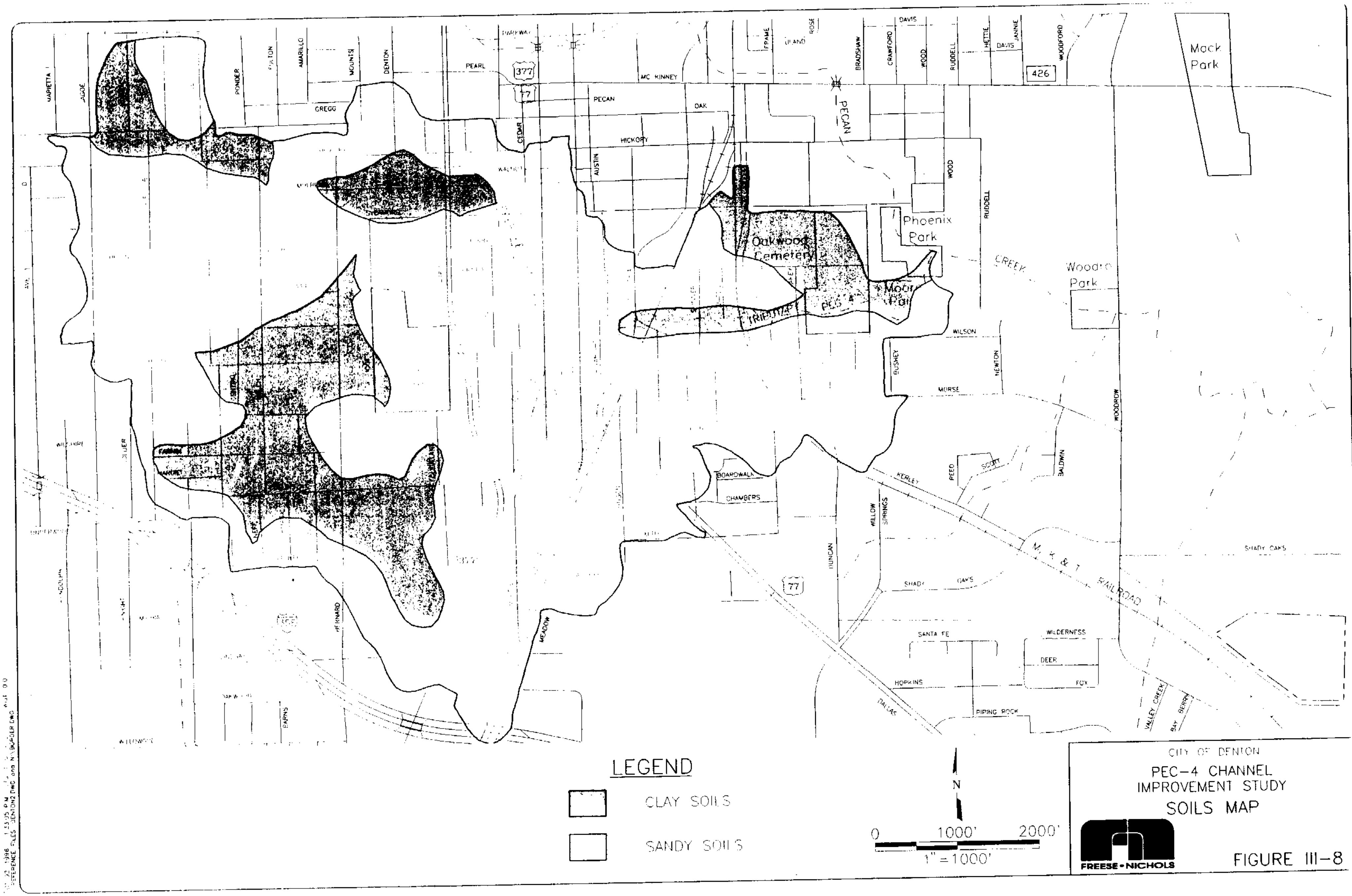
Basin No.	% Sand	% Clay	Basin No.	% Sand	% Clay
1	81.9	18.1	5	85.5	14.5
2	73.8	26.2	6	62.0	38.0
3	90.3	9.7	7	63.6	36.4
4	71.1	28.9			

Lag times for each subbasin were developed in a similar manner as discussed in Part II, Cooper Creek Regional Detention Study, using the "urbanization curves" for the Dallas/Fort Worth area. Table III-2 contains a summary of the lag times for the PEC-4 subbasins and the parameters used to compute the lag times.

Flood hydrographs were routed using Muskingum Method and the routing reaches correspond to the subbasin boundaries. Flood hydrographs were developed for the 10-, 25-, 50-, 100-, and 500-year flood events. A comparison of the peak discharges for the seven HEC-1 design points is provided in Table III-3.



CITY OF DENTON
PEC-4 CHANNEL IMPROVEMENT STUDY
HEC-1 DESIGN POINTS
FIGURE III-6



Mack Park
Phoenix Park
Oakwood Cemetery
Woodrow Park
PEC-4
TRIBUTARY
CREEK
M. K. & T. RAILROAD
PEARL
MC KINNEY
PECAN
HICKORY
OAK
GREGG
WILSON
MORSE
KERLEY
BOARDWALK
CHAMBERS
SHADY OAKS
SANTA FE
WILDERNESS
DEER
FOX
HOPKINS
PIPING ROCK
DALLAS
MEADOW
BERNARD
DUNCAN
WILLOW SPRINGS
BUSHEY
NEWTON
WOODROW
BALDWIN
REED
SCOTT
RUDDELL
WOOD
BRADSHAW
CRAWFORD
DAVIS
WOODFORD
AUSTIN
CEDAR
PONDER
FULTON
AMARILLO
MOUNTS
DENTON
377
77
426
LEGEND
CLAY SOILS
SANDY SOILS
N
0
1000'
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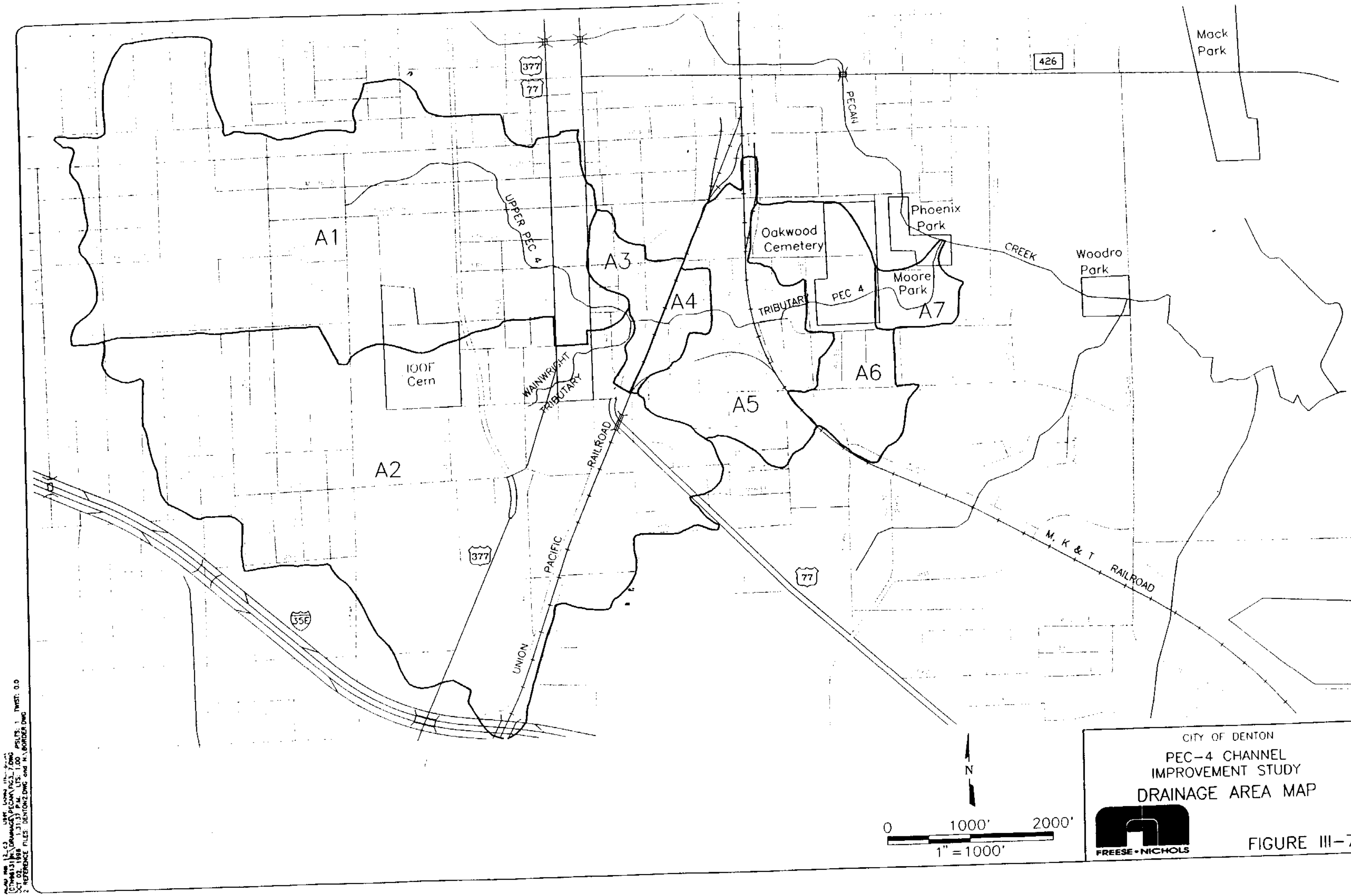
1" = 1000'
CITY OF DENTON
PEC-4 CHANNEL
IMPROVEMENT STUDY
SOILS MAP
FREESE • NICHOLS
FIGURE III-8



CITY OF DENTON
PEC-4 CHANNEL IMPROVEMENT STUDY
DRAINAGE AREA MAP

FIGURE III-7

Table III-2 PEC-4 Subbasin Peak Lag Time Parameters

Subbasin	Area (sq. mi.)	Stream Length (miles)	Length to Centroid (miles)	Average Basin Slope (ft/mi)	Current % Urb.	Current Lag Time (hours)	Ultimate % Urb.	Ultimate Lag Time (hours)
A-1	0.560	1.41	0.85	48.60	60	0.56	90	0.47
A-2	0.680	1.06	0.58	78.12	60	0.37	90	0.32
A-3	0.044	0.30	0.10	33.00	60	0.19	90	0.16
A-4	0.022	0.27	0.09	91.59	60	0.17	90	0.14
A-5	0.128	0.49	0.13	14.05	60	0.25	90	0.21
A-6	0.100	0.38	0.08	90.51	60	0.17	90	0.14
A-7	0.020	0.34	0.13	67.05	60	0.17	90	0.14

Table III-3 PEC-4 100-Year Peak Discharge Comparison

Location of Design Point		Corps of Engineers 1985 FIS	Freese and Nichols	
Junction	Description		Existing (1996) Development	Ultimate Watershed Development
J7	Upper PEC-4	1,650	1,408	1,603
J6	Wainwright	n/a	2,130	2,261
J5	MKT Railroad	3,600	3,547	4,002
J4	Skinner St.	2,100[a]	3,612	4,072
J3	Lakey St.	2,700[a]	4,019	4,477
J2	Bradshaw St.	2,700[a]	4,326	4,794
J1	Above Confl. with Pecan Cr.	2,700[a]	4,387	4,868

a. Includes diversion of 1,500 cfs.

4.0 HYDRAULIC ANALYSIS

The objective of the hydraulic analysis was to evaluate the impact of the proposed channel improvements on the 100-year water surface elevations of Tributary PEC-4. Additionally, the intent of the channel improvements is to maintain the 100-year floodplain within the channel banks.

Channel Improvement Criteria

The *Denton Drainage Criteria Manual* requires channels to contain the runoff from a 25-year event plus one foot of freeboard or the runoff from a 100-year event, which ever is greater. To be consistent with the City's current criteria, the split flow condition at Bell Avenue cannot occur, and improvements are needed to eliminate the flow diversion. Since the existing hydrologic model does not reflect conditions without the diversion, it was necessary to determine the discharge-frequency relationships at specific design points along PEC-4.

Hydraulic Computer Model

Evaluation of the hydraulic characteristics of PEC-4 was performed using the HEC-RAS backwater computer model developed by the COE. Selected parameters such as floodplain width, flood elevation, and flow velocities were analyzed to determine the effects of channel and culvert improvements in maintaining the 100-year floodplain within the channel banks.

The existing hydraulic stream model of PEC-4 obtained from the COE were used as reference models for the analysis. The Corps' model was revised to include the information obtained in the 1996 field surveys performed by Walker and Associates.

Cross sections were generated using a topographic map compiled from the 1996 survey and were imported into the HEC-RAS program. Channel and overbank roughness factors used by the COE were verified during the field investigation. Discharges used for computing water surface profiles were obtained from the HEC-1 models as previously discussed.

Existing Channel Geometry

Upper PEC-4

Upper PEC-4, below Locust Street, is approximately 430 feet long and has an average fall of about 0.008 feet per foot. The first segment of Upper PEC-4 from Locust Street downstream to Wainwright Street consists of a 20-foot wide dressed masonry wall channel with a concrete bottom (Figure III-9a). From Wainwright Street downstream to the junction with Wainwright Tributary, the remaining portion of Upper PEC-4 is a concrete lined channel with a bottom width of approximately eight feet (Figure III-9b). Scattered large trees and patches of dense brush surround this portion of Upper PEC-4.

Wainwright Tributary

The Wainwright Tributary, from downstream of Locust Street, is approximately 700 feet long and has an average fall of about 0.0082 feet per foot. The segment of this reach from downstream of Locust street to the junction with Upper PEC-4 is a concrete lined channel with a bottom with of approximately six feet (Figure III-10). There are metal buildings are located on either side of the channel. Small dense grouping of trees, short to medium grasses, and small brush surround the channel through this reach.

Lower PEC-4

Lower PEC-4 is approximately 4,380 feet long and has an average fall of nearly 0.0068 feet per foot. From the confluence of Upper PEC-4 with the Wainwright Tributary to a point approximately 40 downstream of Bradshaw Street, Lower PEC-4 is an eight-foot wide concrete lined channel (Figure III-11a). Below that point, Lower PEC-4 is an undeveloped earth channel (Figure III-11b). The overbank and channel conditions vary greatly along short lengths of this reach. Scattered trees line the banks from the junction to Bell Avenue. A single family residence is located approximately ten feet from the left bank of the channel on the upstream side of Bell Avenue.

Between Bell Avenue and Skinner Street the channel is surrounded by open fields with short to medium grass, large scattered trees, and small underbrush. Two single family residences are located on either side of the channel on the upstream side of Skinner Street.

Below Skinner Street, downstream to Lakey Street, dense trees, tall grass, and patches of heavy underbrush line the channel. There is a metal footbridge approximately 580 feet downstream of Skinner Street, just downstream of an abandoned railroad crossing. The abandoned railroad crossing was removed in an effort to eliminate local flooding due to backwater at the culvert. Just upstream of Lakey Street, the entrance to a church parking lot has been constructed along the left bank of the channel.

Below Lakey Street PEC-4 winds through Fred Moore Park. Along this section the overbanks are characterized by wide open fields with short grass and scattered trees. Lighted baseball fields, a concrete sidewalk, and chain link fence parallel the channels left

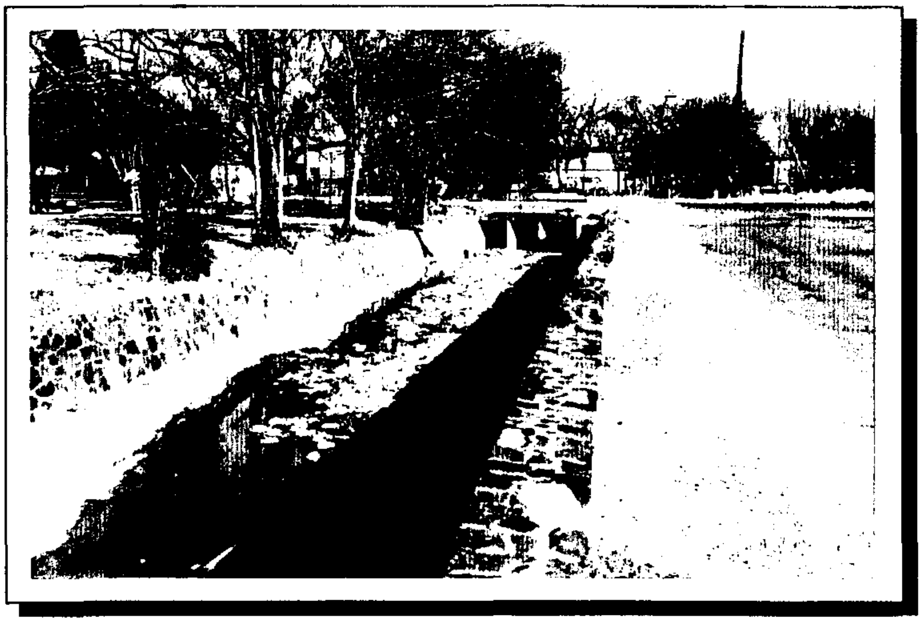


Figure III-9a - Upper PEC-4 Upstream of Wainwright Street

Figure III-9b - Upper PEC-4 Downstream of Wainwright Street

Figure III-10a - Wainwright Tributary Upstream of Wainwright Street

Figure III-10b - Wainwright Tributary Downstream of Wainwright Street

Figure III-11a - Typical Concrete Section in Lower PEC-4 Channel

Figure III-11b - Unlined PEC-4 Channel Downstream of Bradshaw Street bank.

Approximately 40 feet downstream of Bradshaw Street PEC-4 returns to a natural creek bed. The low banks from this point downstream to Pecan Creek consists of a combination of bare ground and short grasses, scattered large trees and intermittent patches of heavy underbrush. The high banks are relatively flat and are covered with short to medium grass.

Evaluation of Channel Improvements

Because the channel is already concrete lined, the flood control method considered included widening the existing channel and increasing the culvert/bridge capacities at the street crossings. Channel improvements were assumed to extend the entire length of PEC-4 from the downstream side of the Locust Street crossing to the confluence of Tributary PEC-4 with the main stem of Pecan Creek. A trapezoidal channel geometry was assumed in the evaluation with 2:1 (horizontal to vertical) sideslopes as required by *Denton Drainage Criteria Manual*. The typical section used in the hydraulic model to represent the proposed channel improvements is shown in Figure III-12.

As mentioned earlier, the policy established by the City of Denton requires lined channels to convey the greater of the 25-year storm plus one foot of freeboard. Therefore, channel widths from ten to twenty five feet widths were evaluated to determine an effective method for containing the 25-year storm water within the channel and reducing the area inundated by the 100-year floodplain.

In general, excavation required to obtain the increased channel widths was assumed to be centered about the existing creek centerline were possible. However, the channel template was sifted to either side of the channel along some reaches to avoid existing structures along the bank.

In several areas of the channel, the HEC-RAS computer model predicted supercritical flow velocities. Supercritical flow is not desirable in channel design since the velocity is very fast and the flow is considered to be unstable. Changes in channel slope, shape or roughness can create a hydraulic jump (an abrupt transition from supercritical to subcritical flow). Since water surface elevations are higher downstream of a hydraulic jump, floodwaters can inundate areas outside the channel area if the channel is designed for supercritical flow.

In order to maintain channel flow velocities within the subcritical range, the slope of the improved channel was "stepped" in several locations. Providing small drops along the channel flow line effectively flattens the channel slope along each section of PEC-4 sufficient to prevent supercritical flow from developing.

Based on the evaluation of the various channel widths, a 20-foot wide trapezoidal section with 2:1 sideslopes was selected as the base channel design. The conceptual layout of the base channel design is illustrated in Figure III-13.

Culvert and Bridge Improvements

Included in the base design were improvements to the culvert and bridge crossings along PEC-4. The evaluation of the type and size of new culverts at the street crossings were based on conceptual sizes developed in 1975 for the *Comprehensive Master Drainage Plan*[10]. Since the culvert improvements recommended in the 1975 master plan were based on the 25-year design frequency, adjustments were made in the recommended

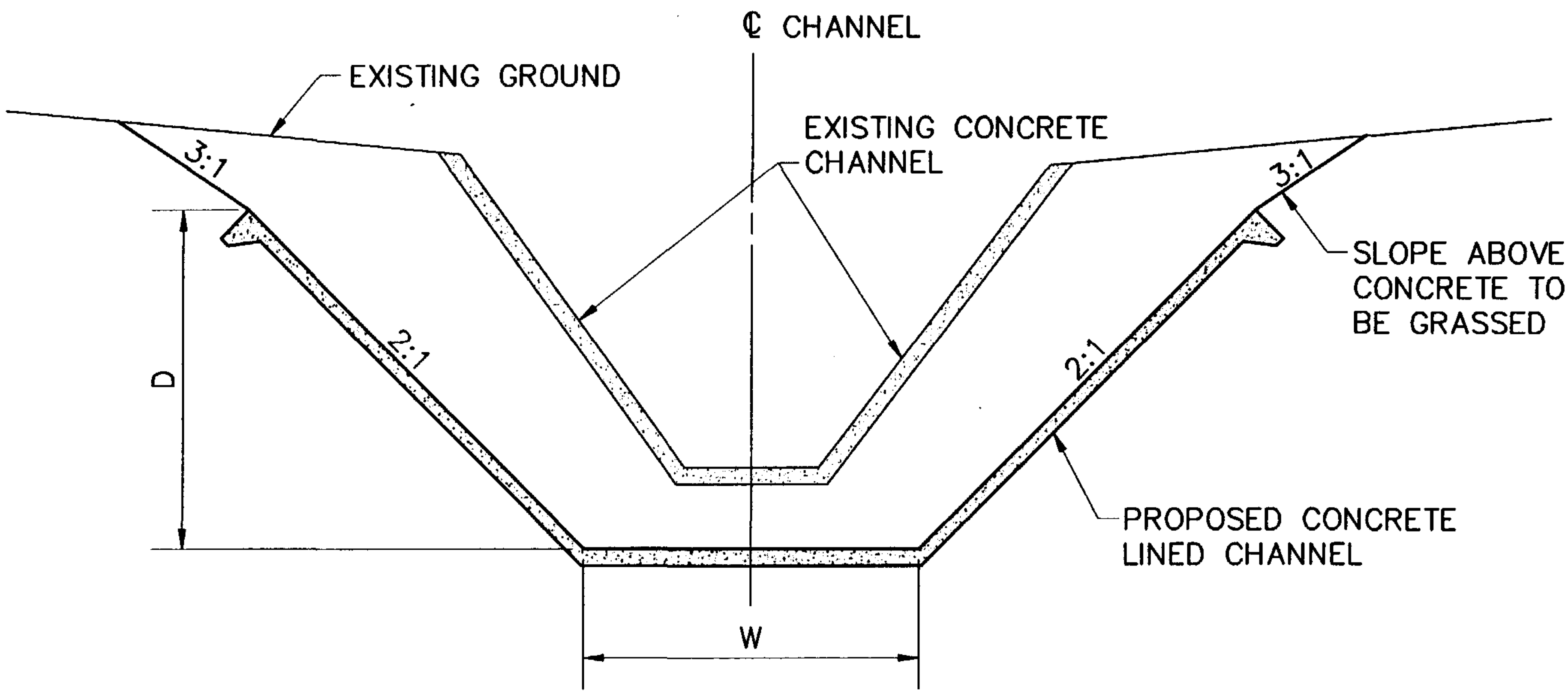
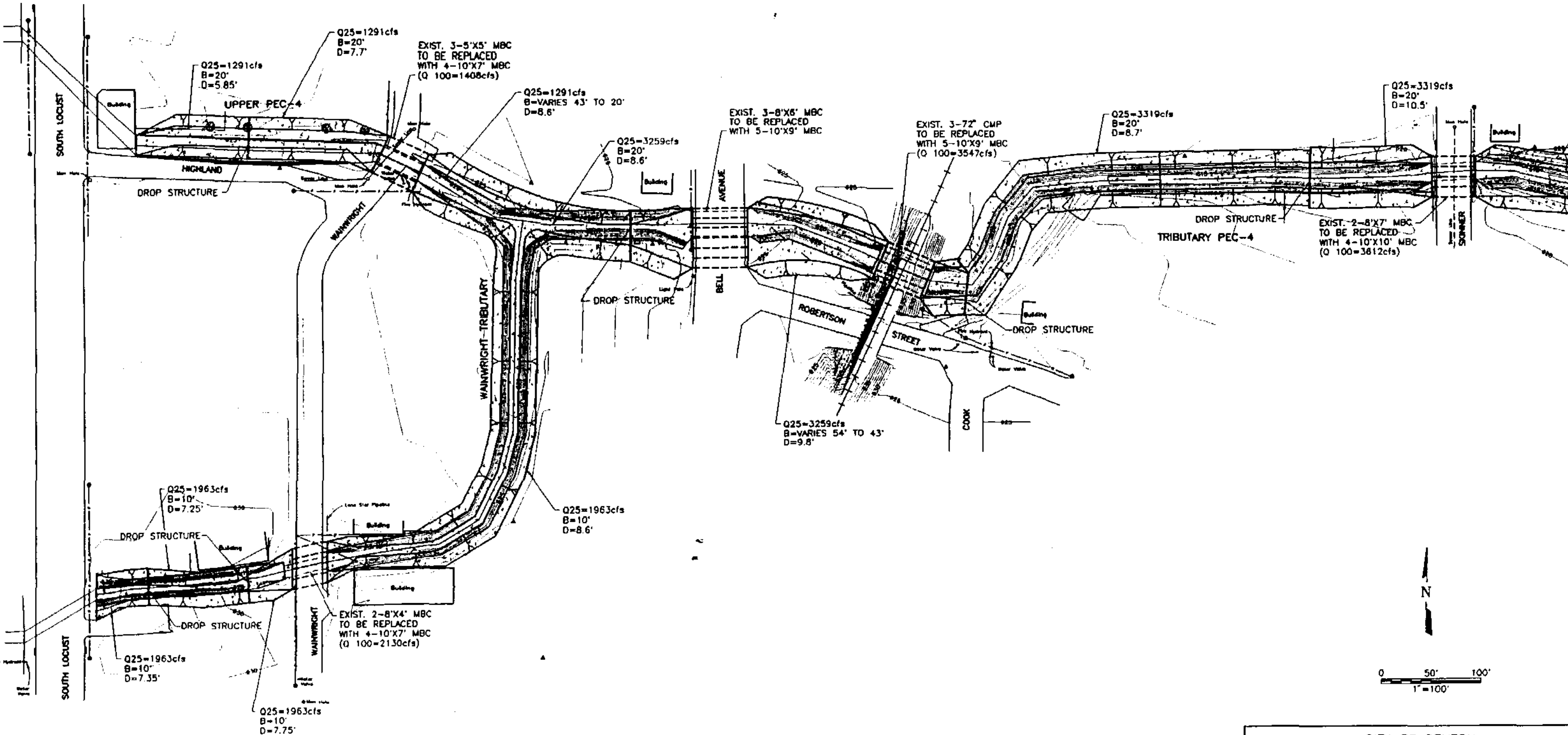


FIGURE III–12
TYPICAL CHANNEL SECTION

W = CHANNEL BOTTOM WIDTH – VARIES FROM 8' TO 20'.

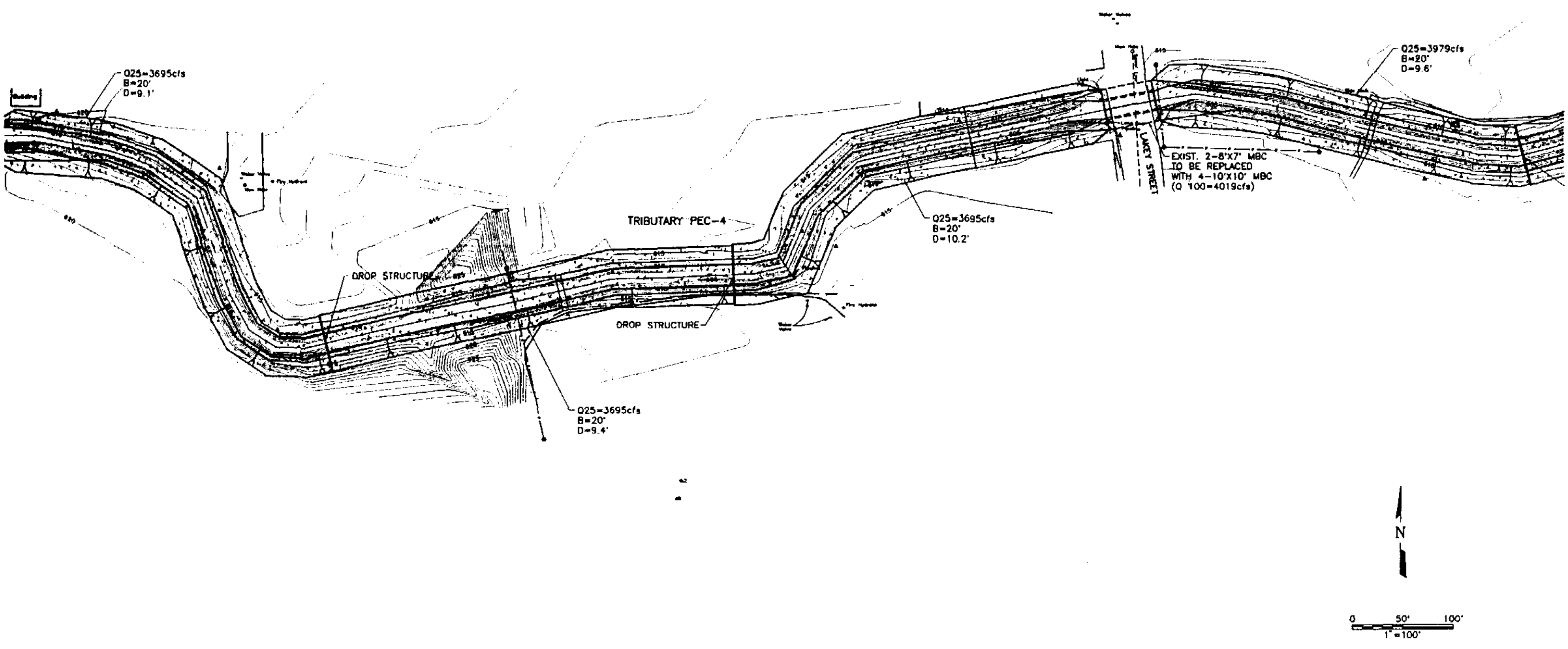
D = CHANNEL DEPTH (LIMIT OFSLOPE PAVING) – VARIES, BUT GENERALLY CORRESPONDS TO THE 25–YEAR FLOW DEPTH PLUS ONE FOOT OF FREEBOARD.



CITY OF DENTON
REGIONAL DRAINAGE STUDY
PEC-4 CREEK
CHANNEL IMPROVEMENTS

FREESE-NICHOLS

FIGURE III-13a

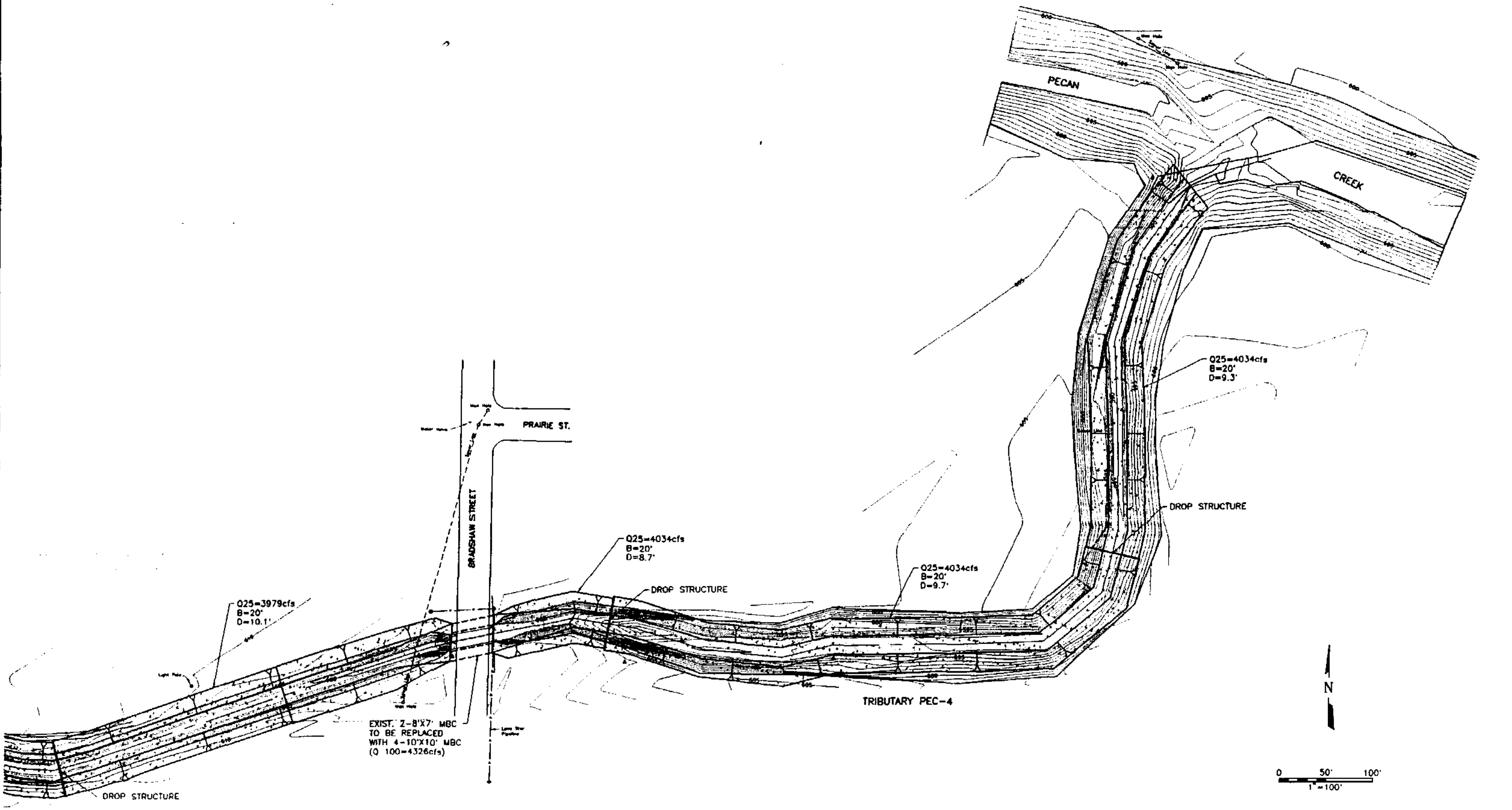


CITY OF DENTON
REGIONAL DRAINAGE STUDY

PEC-4 CREEK
CHANNEL IMPROVEMENTS

FREESE•NICHOLS

FIGURE III-13b



CITY OF DENTON
REGIONAL DRAINAGE STUDY

PEC-4 CREEK
CHANNEL IMPROVEMENTS

FREESE•NICHOLS

FIGURE III-13c

culvert improvements to accommodate the 100-year frequency flow as required in the *Drainage Criteria Manual*. Table III-4 contains a summary of the culvert improvements developed in this study.

Table III-4 PEC-4 Culvert Improvements

Culvert Location	Existing Culvert	Improvements Recommended in 1975 Master Plan	Improvements Recommended in This Study
Wainwright Street (Wainwright Tributary)	2-8'x4'	Replace with four 7'x5' boxes	Replace with four 10'x7' boxes
Wainwright Street (Upper PEC-4)	3-5'x5'	Add one 3'x5' box culvert	Replace with four 10'x7' boxes
Bell Avenue	3-8'x6'	Add one 10'x6' box culvert	Replace with five 10'x9' boxes
M.K.T. Railroad	3-72"	Replace with nine 5'x5' boxes	Replace with five 10'x9' boxes
Skinner Street	2-8'x7'	Add two 6'x7' box culverts	Replace with four 10'x10' boxes
M.K.T. Railroad[a]	2-108"	Replace with three 10'x7' boxes	n/a
Lakey Street	2-8'x7'	Add two 8'x7' box culverts	Replace with four 10'x10' boxes
Bradshaw Street	2-8'x7'	Add two 8'x7' box culverts	Replace with four 10'x10' boxes

a. The railroad at this location was abandoned and the culvert removed.

Floodplain Characteristics

The 100-year floodplain along PEC-4 for existing conditions includes roughly 120 acres, with an average flow depth of thirteen feet. Approximately fifteen percent of the flow is outside of the existing channel banks. There are currently 194 structures inundated by the 100-year floodplain (1996 watershed development).

The proposed channel improvements discussed above were input into the hydraulic stream model to analyze their effect on the 100-year floodplain. Analysis of the PEC-4 floodplain indicates that the channel improvements will result in a lowering of the water surface elevation an average of 5.5 feet between Locust Street and Pecan Creek. In doing so, the 100-year floodplain will be confined within the channel banks. Table III-5 contains a summary of the 100-year floodplain elevations at the culverts along PEC-4 within the study area. A graphic representation of the floodplain elevation comparison at the culvert crossings is shown in Figure III-14. A more comprehensive floodplain profile comparison is shown in Figure III-15.

Table III-5 Comparison of 100-Year Floodplain Elevations

Location	Top of Bridge	Existing Development	Ultimate Watershed Development	
		Without Imrovements	Without Imrovements	With Improvements
Wainwright Street (Wainwright Trib.)	628.3	632.7	633.8	627.7
Wainwright Street (Upper PEC-4)	626.5	632.5	633.4	625.6
Bell Avenue	624.7	630.0	630.7	624.3
M.K.T. Railroad	636.1	630.0	630.8	622.3
Skinner Street	621.0	624.9	626.1	619.0
Lakey Street	614.1	618.5	618.7	611.5
Bradshaw	608.1	614.9	615.7	607.5

FIGURE III-14 - FLOODPLAIN ELEVATIONS AT CULVERTS

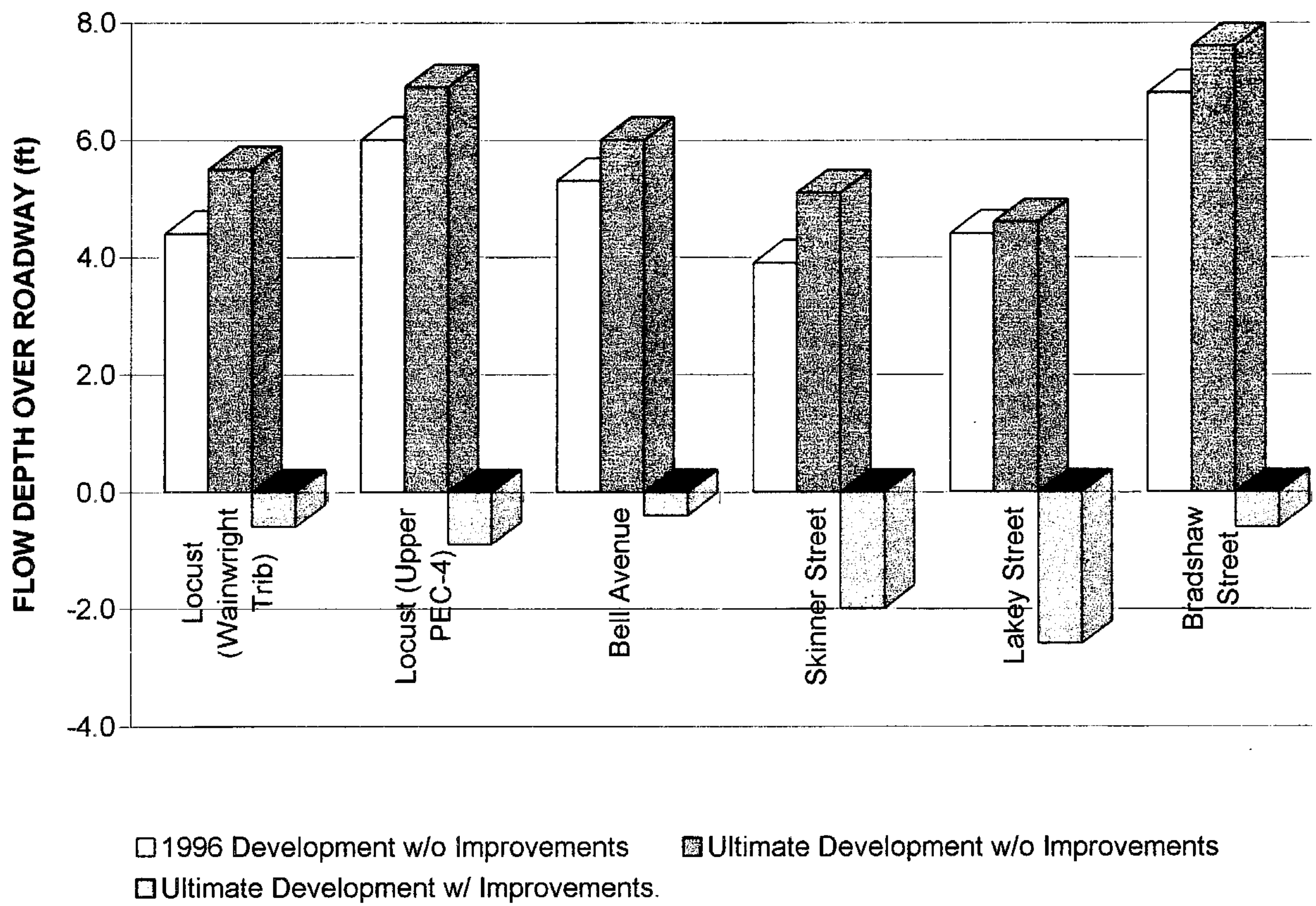


FIGURE III-15. 100-Year Floodplain Profile Comparison (PEC-4)

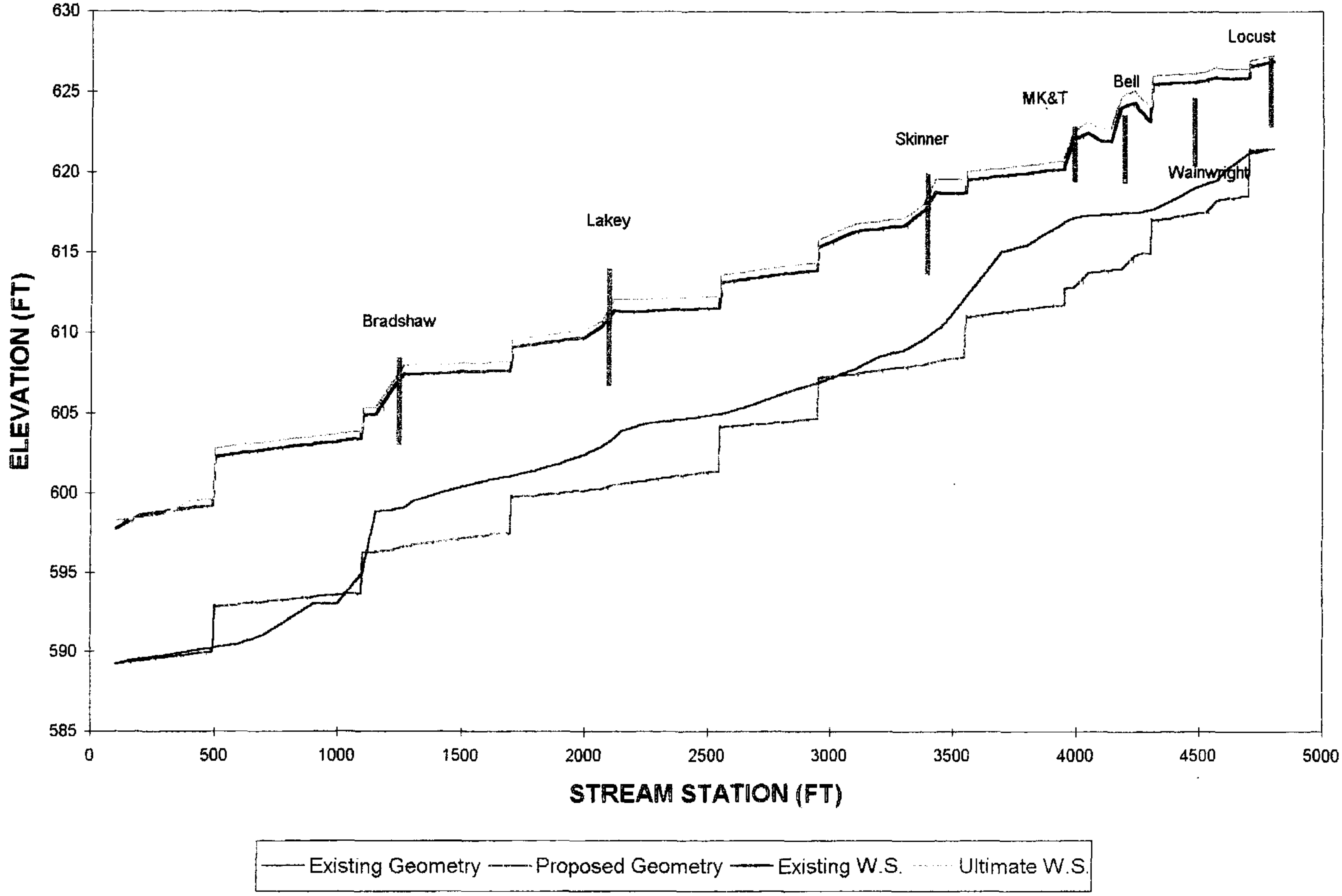
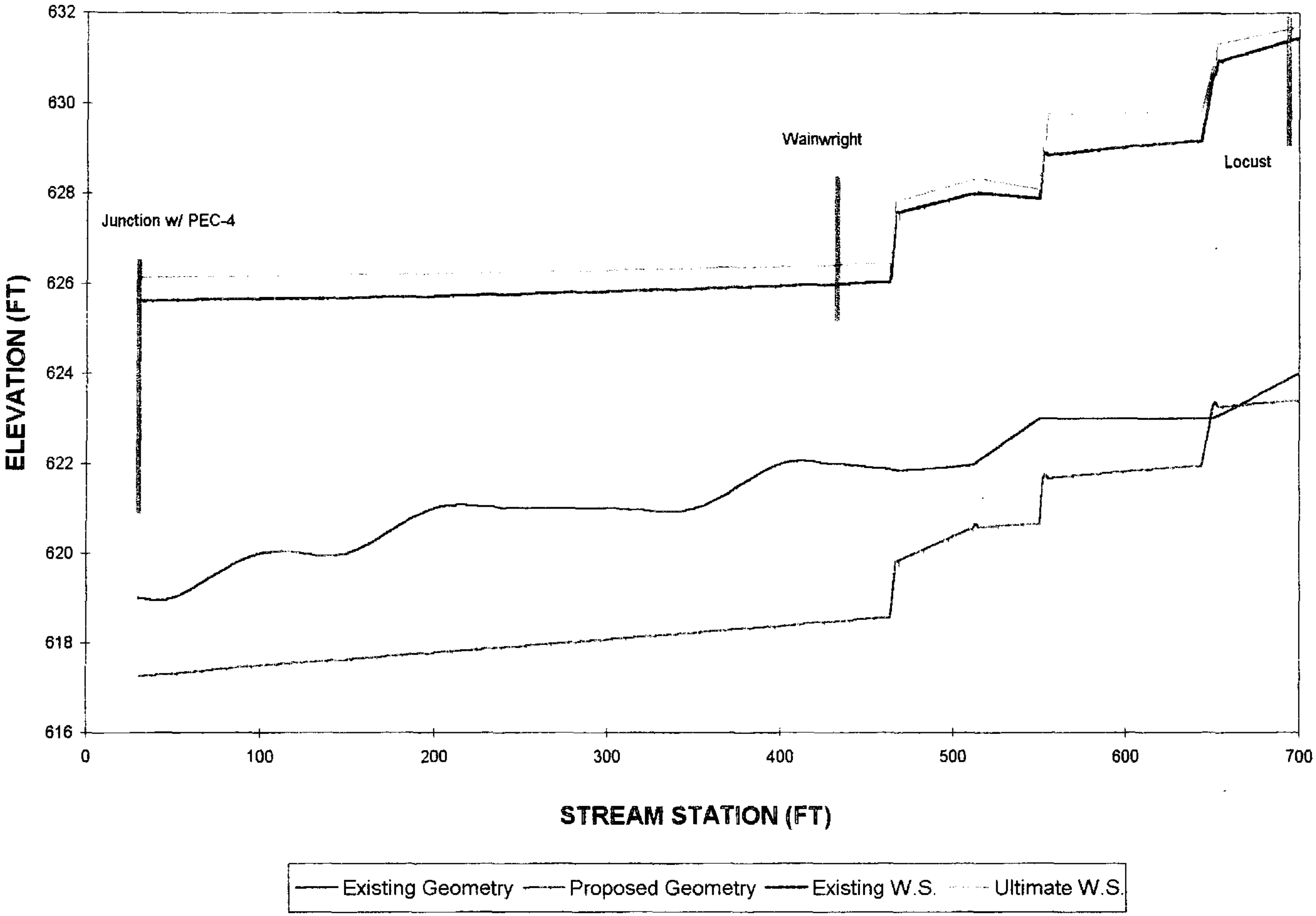
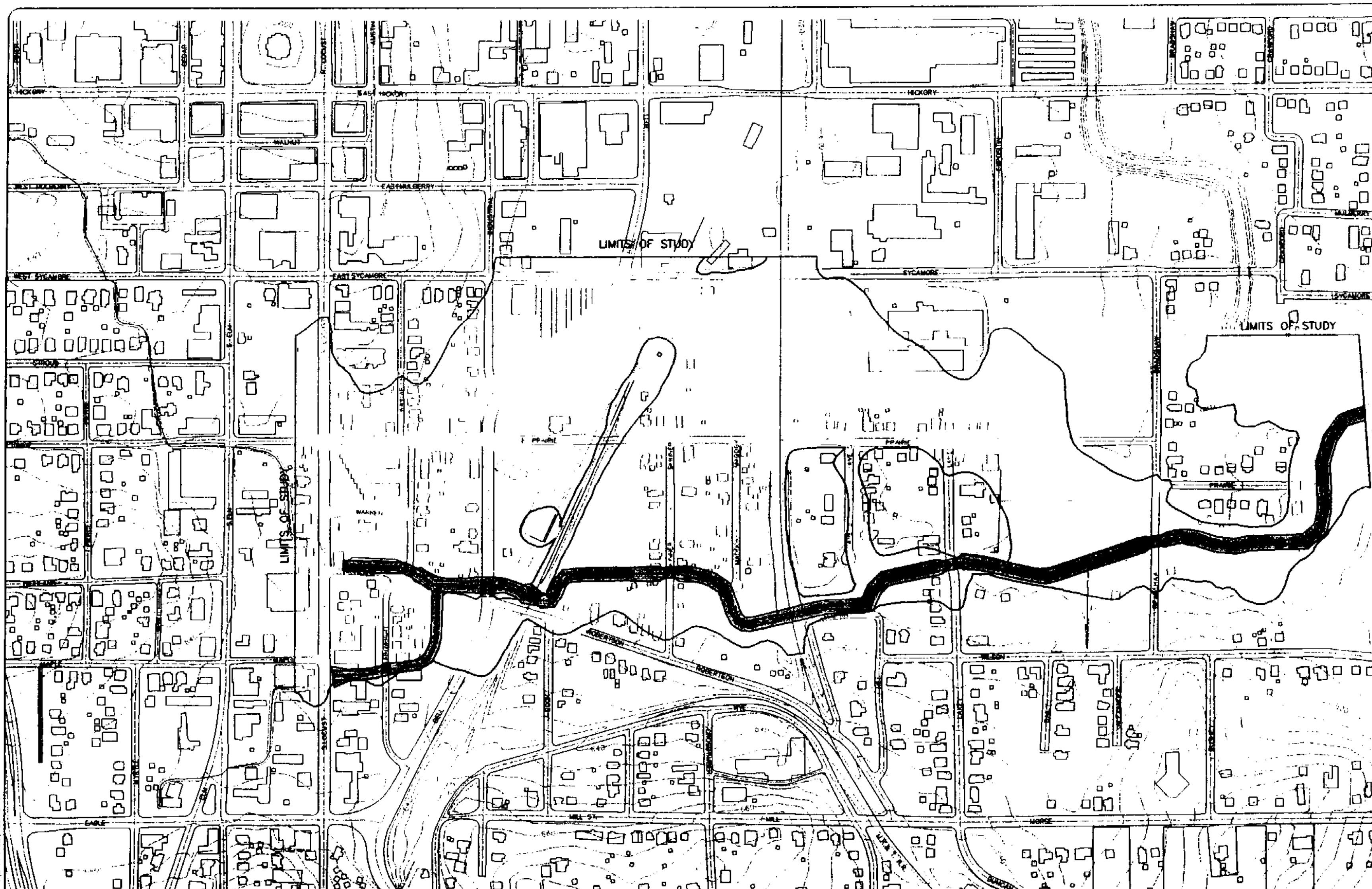


FIGURE III-15. 100-Year Floodplain
Profile Comparison (Wainwright Tributary)





LEGEND

EXIST. 100-YEAR FLOODPLAIN

100-YEAR FLOODPLAIN WITH IMPROVEMENTS IN PLACE

N

0 100' 200' 400'
1"=200'

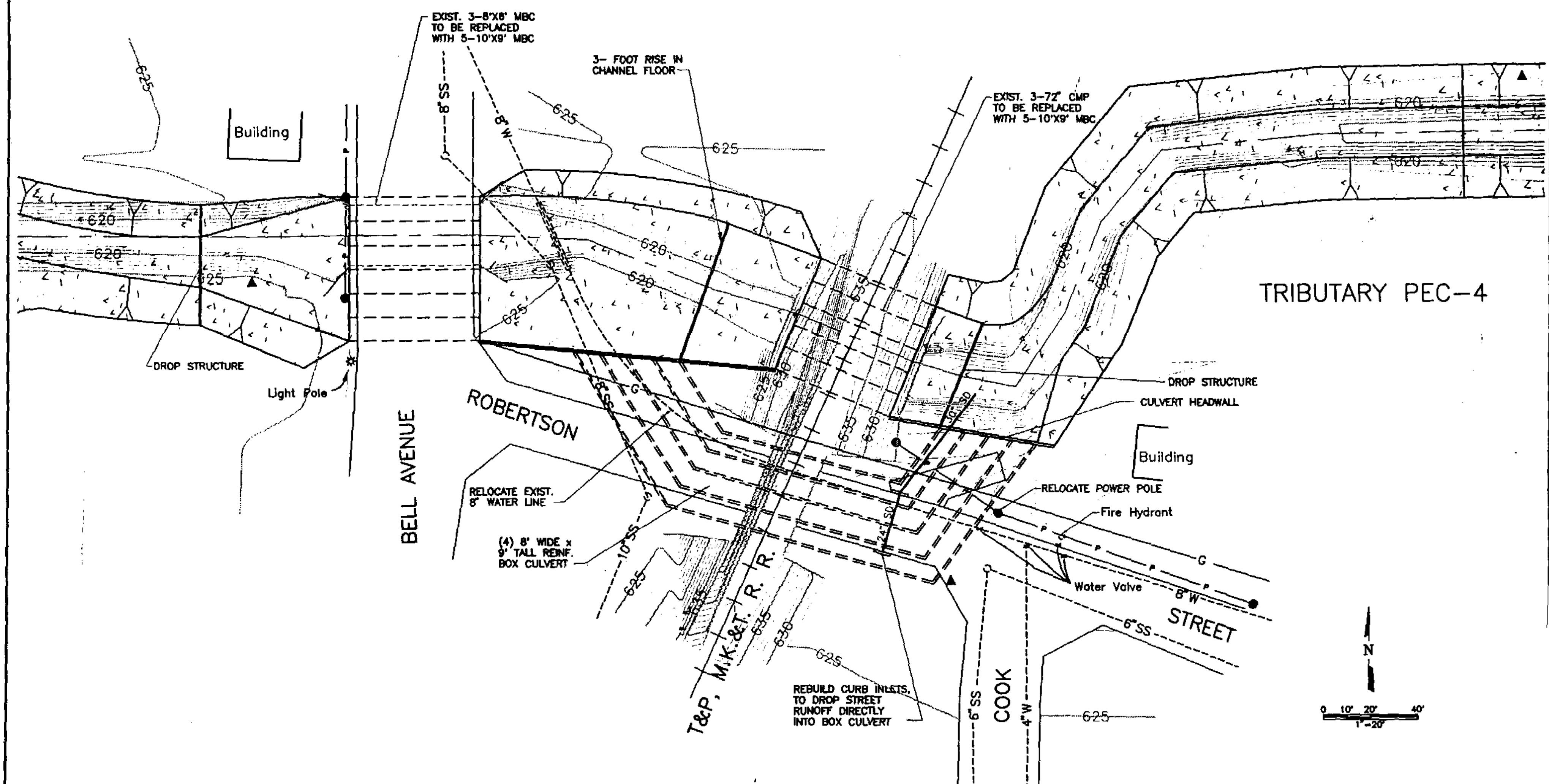
CITY OF DENTON

PEC-4 CHANNEL IMPROVEMENT STUDY

100-YEAR FLOODPLAIN COMPARISON

FREESE • NICHOLS

FIGURE III-16



CITY OF DENTON

PEC-4 CHANNEL IMPROVEMENT STUDY

ROBERTSON STREET BYPASS

FREESE • NICHOLS

FIGURE III-17

5.0 ADDITIONAL CONSIDERATIONS

Robertson Street Bypass Tunnel

The culvert improvements developed in the base design at the M.K.T. Railroad assumed the existing culvert beneath the railroad will be replaced with a new culvert in the same location. Since construction of a new culvert beneath the railroad would be fairly expensive, an alternative was developed for providing the additional culvert capacity needed by constructing a bypass tunnel.

The bypass tunnel would be constructed such that the upstream invert is located between the Bell Avenue and the existing railroad culvert and the downstream invert is located approximately fifty feet downstream of the railroad culvert outlet. The tunnel will turn south from the main channel and will be constructed under the railroad overpass, beneath Robertson Street. The tunnel will daylight on the other side of the railroad and join the main channel.

The downstream floodplain elevation of 622.5± causes the existing culvert under the railroad to function in under outlet control conditions (i.e. culvert capacity is limited by the depth of water at the culvert outlet). With a maximum headwater elevation upstream of the railroad of 625±, the capacity of the existing culvert beneath the railroad is approximately 600 cfs. Since the 100-year frequency peak discharge expected at the railroad culvert is 4,000 cfs, the additional capacity required by the bypass tunnel is approximately 3,400 cfs.

The bypass system will have to operate under the same constraint at the existing culvert under the railroad. That is, the tailwater elevation of 622.5 and the maximum headwater elevation of 625. With a maximum headloss of 2.5 feet, five 10'x8' box culverts would be required to convey 3,400 cfs through the bypass.

The capacity of the existing railroad culvert can be increased and the size of the bypass system reduced if the tailwater elevation downstream of the railroad is reduced. The existing railroad culvert capacity will be limited at the culvert inlet when the tailwater elevation drops below 622.2±. At that point, reducing the tailwater elevation will have no effect on the capacity of the existing railroad culvert.

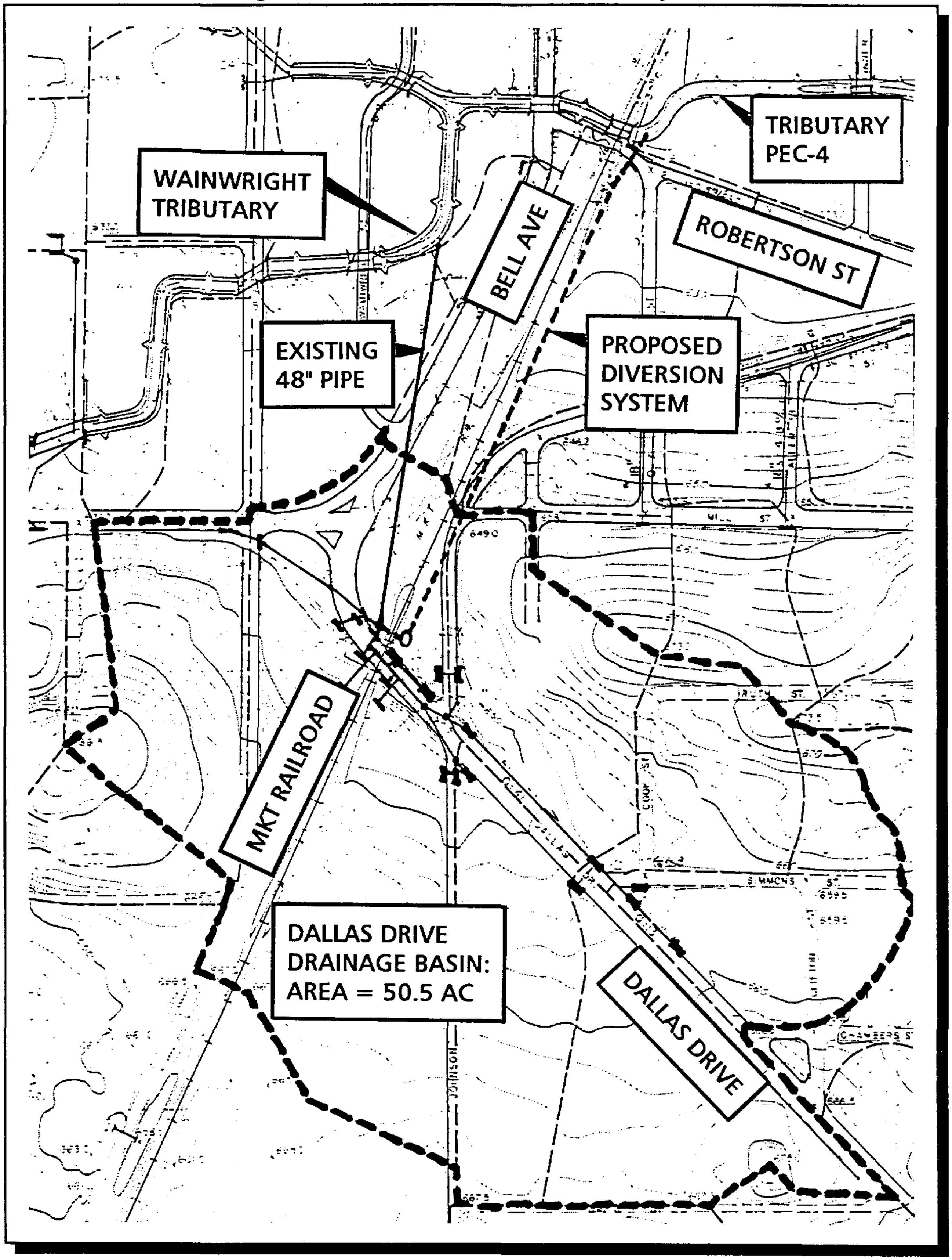
If the channel bottom downstream of the railroad in the base design is lowered one foot, the existing railroad capacity increases to approximately 780 cfs, and four 9'x8' box culverts would be required to convey the remaining 3,220 cfs in the bypass system. The proposed location of the bypass system is shown in Figure III-17. The differential costs of constructing the bypass tunnel are addressed later in this report.

Dallas Drive Diversion

One option available to reduce the peak flow in Tributary PEC-4 near the M.K.T. Railroad is to divert the drainage from Dallas Drive. Dallas Drive crosses underneath the M.K.T. Railroad approximately 1,300 feet southwest of the Robertson Street crossing. Approximately 50.5 acres drain to the Dallas Drive underpass, where storm water is collected by a series of curb inlets and conveyed by an existing 48-inch storm drain pipe to the Wainwright Tributary 300 feet upstream of the confluence with Tributary PEC-4.

Diversion of the storm water collected by the Dallas Drive system to the downstream side of the M.K.T. Railroad near Robertson Street (Figure III-18) will reduce the 100-year peak flow in Tributary PEC-4 by approximately 265 cfs, or slightly less that 7%. As a result

Figure III-18 - Dallas Drive Diversion System



of the lower peak discharge, the width of the channel improvements between the M.K.T. Railroad and the confluence of Upper PEC-4 and the Wainwright Tributary can be reduced. The diversion will include crossing the railroad near Dallas Drive and installing approximately 1,300 feet of 66-inch diameter pipe from Dallas Drive to Robertson Street.

Alternative Channel Lining Materials

In areas where the appearance of the channel is as important as its capacity for flood water conveyance, alternatives to concrete channel lining should be explored. Typical alternatives to concrete slope paving in channel improvements include gabions, segmental retaining walls, interlocking pavers and place stone.

Gabions are rock-filled wire baskets that are typically design as gravity walls. The concept of a gravity wall is that the weight of the wall itself is significant to resist potential movement of the soil behind the wall. Gabions can also be designed as a facade for channel bank erosion protection in shallower channels where slope stability is not a concern. An example of how gabions can improve the channel aesthetics is shown in Figure III-19.

Segmental retaining walls are typically constructed of concrete modular blocks and are available from a number of manufacturers in a wide variety of shapes and colors. These walls are particularly effective when the wall height is no greater than four feet. For walls taller than four feet, geogrid soil reinforcement is required (depending on the existing soil conditions) and space outside the channel may be limited by existing structures.

Interlocking pavers are typically made of concrete and have spaces within the paving unit to allow vegetation to grow. This type of channel lining material is used for protection against erosion and provide no stability to resist slope movements.

Placed stone offers a more natural appearance than the other alternatives discussed above. However, the construction costs of a placed stone wall are typically higher than the other alternatives because of the amount of hand labor needed to construct the stone wall.

If alternative channel lining materials are appropriate and concrete paving is needed, there are several options available for altering the appearance of the concrete paving. In channel bottom applications, a pattern can be stamped into the concrete prior to setting to give the appearance of a natural rock bottom. Form liners may also be used in vertical wall application to make the concrete look like a number of materials, including wood, brick, stone, etc. Dyes can be added to the concrete mix to change the color of the concrete to more closely match the existing site conditions.

Each of the above alternatives provide a more aesthetic appearance than a flat concrete finish. The segmental retaining walls and gabions give the appearance of hand placed rock, although the wire baskets are readily apparent with the gabions. The gabions and interlocking pavers will facilitate the planting of grasses and vines as an integral part of the lining material. Examples of segmental retaining wall, placed stone and modified concrete construction are shown in Figure III-20.

One area along Tributary PEC-4 that is particularly suited to one of these alternative channel lining materials is the segment through Fred Moore Park. For the purpose of the hydraulic evaluation, it was assumed that a segmental retaining wall will be constructed through the park area. There are few objects adjacent to the channel within the park area

FIGURE III-19 - CONCRETE VS. GABION CHANNEL LINING EXAMPLES

Figure III-19a - Concrete Channel
Upstream of M.K.T. Railroad

Figure III-19b - Gabion Channel
Upstream of M.K.T. Railroad

FIGURE III-20 - ALTERNATIVE CHANNEL LINING EXAMPLES

Figure III-20a - Segmental Retaining Wall With
Interlocking Pavers in Channel Bottom

Figure III-20b - Placed Stone Retaining Wall With
Stamped and Stained Concrete Bottom

to prevent installation of geogrid soil reinforcement behind the segmental retaining wall. Following construction, the area disturbed by construction activities can be sodded and the sod will grow quickly, minimizing the overall impact on the park appearance.

6.0 ANTICIPATED PROJECT COSTS

Probable construction costs were estimated for the conceptual channel improvements presented in this study. The projected construction costs developed on a reach-by-reach basis and were compared to the approximate value of property removed from the floodplain in each reach. A cost-benefit relationship was developed for each reach.

Probable Construction Costs

Items considered in the conceptual construction estimates include site preparation, excavation, concrete channel lining, culvert improvements, sodding, and land purchases. The estimates also include provisions for mobilization, contractor overhead and profit, and contingencies. Alternative channel lining options were not included in the estimation of probable construction costs, but comparative costs for the materials are presented herein.

Table III-6 contains a summary of the estimated costs associated with construction of the channel improvements. Detailed breakdowns of the estimated construction costs are listed in Appendix D.

Cost-Benefit Analysis

The anticipated construction cost of the recommended channel and culvert improvements were compared to the expected benefit of the improvements. The expected benefit was assumed to be the increase in property value from the value of land within the floodplain to the value of land outside the floodplain. The area recovered from the floodplain was estimated by averaging the floodplain width at the hydraulic model cross sections.

Table III-6 Probable Construction Costs for PEC-4 Improvements

Tributary PEC-4 Channel Improvement Reach	Estimated Construction Costs[b]	Estimated Value of Property Removed From the Floodplain[c]	Overall "Cost" of Improvements
Pecan Cr. to Bradshaw St.	$953,900	$340,600	$613,300
Bradshaw St. to Lakey St.	$535,300	$399,900	$135,400
Lakey St. to Skinner St.	$724,300	$1,036,700	- $312,400
Skinner St. to Bell Ave.	$1,328,700	$571,700	$757,000
Bell Ave. to Locust St.[a]	$816,600	$782,300	$34,300

a. Includes both Upper PEC-4 and Wainwright Tributary.

b. Include cost of purchasing property for pond construction based on $0.80 per square foot.

c. Increase in property value after being removed from floodplain is based on $0.68 per square foot.

The assumed value of property along Tributary PEC-4 is $0.80 per square foot. The reduced value of the same property if located within the 100-year floodplain was assumed to be $0.12 per square foot, or roughly 15% of the non-flooded value.

7.0 404 PERMIT CONSIDERATIONS

Channel improvements projects often fall under the jurisdiction of the Corps of Engineers 404 permit program. In the case of the Tributary PEC-4 improvements, the work will most likely fall under one of two nationwide permits: (1) Nationwide 26, and (2) Nationwide 3.

The Nationwide 26 permit is required for all fill (improvements) within channel areas below the headwaters of waters of the United States. The point at which a stream is below the headwaters approaches a contributing drainage area on the order of 25 square miles. The permit allows up to ten acres below the normal high water mark upstream of the headwaters to be filled (modified). The normal high water mark is usually observed as the distinct pont at which vegetation along the stream ends. If the improvements below the normal high water mark are less than one acre, no formal notification to the Corps is necessary.

The other permit that covers the majority of the Tributary PEC-4 channel is the Nationwide 3 Permit. Under this permit, repairs are authorized for the maintenance of existing channels. This permit covers the portion of the PEC-4 channel that is concrete lined. Under this permit, repairs are authorized for the maintenance of existing channels.

Since most of the PEC-4 channel is presently concrete lined, the recommended improvements should fall under the Nationwide 3 Permit and no notification of the proposed improvements to the Corps is required. The portion of the proposed PEC-4 improvements immediately upstream of the main stem of Pecan Creek should fall under the Nationwide 26 Permit, since the area of the proposed improvements below the normal high water mark is less than ten acres. Additionally, since the area below the high water mark is expected to be approximately 0.40 acres, no notification should be required. However, we recommend that a letter be submitted to the Corps of Engineers during the design of the proposed improvement to requesting the Corps concurrence.

It should be noted that all of the current nationwide permits expire on January 21, 1997. At that time, the Corp may renew or modify the present requirements of the nationwide permits. It is our understanding that the Corps is currently considering modifying the Nationwide 26 permit requirements. The modification being considered is to reduce the limit of notification from one acre to 0.50 acres, or possibly 0.30 acres. Should the limit be dropped to 0.30 acres, the Corps will require notification of the proposed PEC-4 improvements, although the work would still be below the 10-acre high water mark limit.

8.0 CHANNEL IMPROVEMENT RECOMMENDATIONS

Channel improvements and culvert improvements were evaluated for effectiveness in returning the 100-year floodplain to within the channel banks. Based on the evaluation, the bottom width of the PEC-4 channel should be increased to 20 feet and the overall channel slope should be reduced. Additional culvert capacity is needed for each of the culverts between Locust Street and Pecan Creek to meet the City's design criteria and to eliminate local channel overtopping due to excessive backwater conditions.

A significant result of the recommended channel and culvert improvements for the City is the elimination of the split flow conditions at the railroad crossing. Eliminating the split flow conditions will effectively recover approximately 105 acres of existing floodplain area. The recovery of this area is particularly important because of the large number of businesses and homes located adjacent to the stream within the existing floodplain.

Both the Railroad Crossing Culvert and the Robertson Street Bypass were effective in passing the 100-year flows and maintaining the water surface within the channel banks. Replacement of the existing culverts beneath the railroad will necessarily include a shoo-fly connection to prevent down-time in rail service. However, construction of the Robertson Street Bypass will be difficult given the limited working space beneath the railroad overpass and the existing water, sanitary sewer and gas lines may need relocating to accommodate the bypass culverts. Both of these options will be expensive and will require coordination with the M.K.T. Railroad. Additional evaluations may be made to determine possible downstream channel improvements that may decrease the tailwater at the railroad culvert, thereby reducing the required culvert capacity.

Alternative channel lining materials should be considered, especially through Fred Moore Park, since there is little additional cost associated with the alternative lining materials. The channel lining alternatives presented herein enhance the aesthetic appearance of the stream and are likely to be more favorably accepted when presented at public meetings.

The design of the channel improvements should be performed in close coordination with the Corps of Engineers and possible changes in the 404 Permit program should be anticipated. At the present time, little coordination is needed, a formal request should be made to the Corps for concurrence on compliance with the terms of the existing nationwide permits.

APPENDIX A

SOIL BORING LOGS

FUGRO

START DATE: 2-22-96
END DATE: 2-22-96
COMPLETION DEPTH (FT): 30.00

WATER LEVEL OBSERVED AT 10.00 FT UPON COMPLETION OF DRILLING

LOG OF BORING NO. B-1

COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS
Denton, Texas

TYPE: AUGER

LOCATION: See Plate A.1

EXISTING GRADE ELEVATION: 620.00

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SAMPLES	SYMBOL	SOIL / ROCK CLASSIFICATION	H2O	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	8				SANDY LEAN CLAY (CL), brown, with organic material to 3 feet, firm to very stiff			13							
	11							15							
5	22							11	36	20	16	58			
10	4.5+				SANDY LEAN CLAY (CL), light brown, light gray, hard		610.0 10.0	16	27	14	13	55	122	1.1	2.467
15	2.0				CLAYEY SAND (SC), brown to light brown, very stiff to hard		605.0 15.0	17	24	16	8	39		0.3	
20	4.5+							17						0.4	
25	50+				CLAYEY SAND (SC), limestone gravel, weathered limestone, tan, hard		595.0 25.0	14							
					LIMESTONE, gray, hard		593.0 27.0								
	100/1"														
30							590.0 30.0	20							

Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE, AUGER, ROCK BIT, SPLIT BARREL, THD CONE, ROCK CORE, SEEPAGE LEVEL, WATER LEVEL

PLATE A. 4

REPORT NO.: 1401-0026

FUGRO

START DATE: 2-22-96
END DATE: 2-22-96
COMPLETION DEPTH (FT): 30.00

WATER LEVEL OBSERVED AT 10.00 FT UPON COMPLETION OF DRILLING

LOG OF BORING NO. B-2

COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS
Denton, Texas

TYPE: AUGER

LOCATION: See Plate A.1

EXISTING GRADE ELEVATION: 622.00

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SAMPLES	SYMBOL	SOIL / ROCK CLASSIFICATION	H2O	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	4.0				SANDY LEAN CLAY (CL), brown, trace of gravel and organic material, very stiff			15						1.5	
	4.0				- free swell at 2 feet = 0.5 percent			14	26	17	9	51	107	1.1	
5	4.5+				SANDY LEAN CLAY (CL), tan to reddish-brown and light gray, very stiff to hard		617.0 5.0	10						2.5+	
	46							9	28	13	15	51			
10	3.5							18					115	1.4	0.716
15	3.5				FAT CLAY (CH), tan, with some sand, very stiff to hard		607.0 15.0	22	62	24	38	87		1.6	
20	4.5+							20						2.5+	
25	4.5+							21						2.5+	
					LIMESTONE, gray, argillaceous, very hard		595.0 27.0								
30	100/0.75						592.0 30.0								

Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE, AUGER, ROCK BIT, SPLIT BARREL, THD CONE, ROCK CORE, SEEPAGE LEVEL, WATER LEVEL

PLATE A. 5

REPORT NO.: 1401-0026

START DATE: 2-22-96
END DATE: 2-22-96
COMPLETION DEPTH (FT): 30.00

WATER LEVEL OBSERVED AT 16.00 FT
UPON COMPLETION OF DRILLING

LOG OF BORING NO. B-3

COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS
Denton, Texas

TYPE:

LOCATION: See Plate A.1

EXISTING GRADE ELEVATION: 616.00

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SOIL / ROCK CLASSIFICATION	H2O	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
0	3.5		SANDY LEAN CLAY (CL), brown, with organics, very stiff			12						0.5	
	2.5					15	23	13	10	53		0.5	
5	4.5+		SANDY LEAN CLAY (CL), tan to gray, hard		611.0 / 5.0	9							
	4.5+					11	35	18	17	58	121		10.63
10	4.5+											2.5+	
15	3.5		CLAYEY SAND (SC), reddish brown, very stiff to hard		601.0 / 15.0	15	31	15	16	42		1.2	
16				▼									
20	35					19							
	100/1.5"				591.0	17							
25			SHALEY CLAY, tan to gray, hard		25.0								
			SHALE, gray, hard		590.0 / 26.0								
	100/1.0"				586.0	13							
30					30.0								

Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE, AUGER, ROCK BIT, SPLIT BARREL, THD CONE, ROCK CORE, SEEPAGE LEVEL, WATER LEVEL

PLATE A. 6

START DATE: 2-22-96
END DATE: 2-22-96
COMPLETION DEPTH (FT): 25.00

NO WATER OBSERVED IN BOREHOLE

LOG OF BORING NO. B-4

COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS
Denton, Texas

TYPE: AUGER

LOCATION: See Plate A.1

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SAMPLES	SYMBOL	SOIL / ROCK CLASSIFICATION EXISTING GRADE ELEVATION: 608.00	H2O	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	4.5+				CLAYEY SAND (SC), brown, hard			11						0.4	
	4.5+				- soil non-plastic at 2 feet			17				17		0.3	
							603.0								
5	4.5+				CLAYEY SAND (SC), gray, iron nodules, hard - free swell at 6' = 0.3 percent		5.0	10	21	17	4	33	104	0.3	
	4.5+							12						1.9	
							598.0								
10	4.0				LEAN CLAY WITH SAND (CL), tan to light brown, hard		10.0	18						2.5+	
15	4.5+							18	49	23	26	84	115	2.5+	4.219
							589.0								
	100/1.75"				SHALE, gray, calcareous, with traces of clay, hard to very hard		19.0								
20								16							
	100/0.0"						583.0	12							
25							25.0								

Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE, AUGER, ROCK BIT, SPLIT BARREL, THD CONE, ROCK CORE, SEEPAGE LEVEL, WATER LEVEL

PLATE A. 7

START DATE: 2-22-96
END DATE: 2-22-96
COMPLETION DEPTH (FT): 18.00

NO WATER OBSERVED IN BOREHOLE

LOG OF BORING NO. B-5

COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS
Denton, Texas

TYPE: AUGER

LOCATION: See Plate A.1

EXISTING GRADE ELEVATION: 641.00

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	SOIL / ROCK CLASSIFICATION	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	3.0	CLAYEY SAND (SC), brown to light gray, very stiff to hard		20						1.3	
	4.5+			12						2.5+	
5	4.5+		635.0	15	40	16	24	43		1.3	4.22
		LEAN CLAY WITH SAND (CL), tan, very stiff	6.0								
	4.0			19						1.6	
10	3.0			20	38	16	22	69		1.1	
	100/0.5"		628.0								
		LIMESTONE, gray with shale seams, very hard	13.0								
				11							
15											
	100/0.0"		623.0								
			18.0								

Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE; AUGER; ROCK BIT; SPLIT BARREL; THD CONE; ROCK CORE; SEEPAGE LEVEL; WATER LEVEL

START DATE: 2-22-96 END DATE: 2-22-96 COMPLETION DEPTH (FT): 30.00	SEEPAGE OBSERVED AT 11.00 FT UPON COMPLETION OF DRILLING	**LOG OF BORING NO. B-6** COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS Denton, Texas

TYPE: AUGER — LOCATION: See Plate A.1

EXISTING GRADE ELEVATION: 630.00

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	SOIL / ROCK CLASSIFICATION	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	4.0	SANDY LEAN CLAY (CL), brown, with calcareous deposits, very stiff		20						1.6	
	4.0	- free swell at 3 feet = 0.8 percent		15	41	19	22	58	109	1.9	
5	4.0			17	41	17	24	61	115	1.8	2.846
	4.0			17						1.3	
10	1.5	CLAYEY SAND (SC), reddish-brown, tan, light gray, stiff	620.0 10.0	21	28	14	14	47	114	1.5	1.901
15	4.5+		614.0	16							
		SHALE, brown, calcareous, hard to very hard	16.0								
	100/1.5"										
20				10							
	100/0.0"		605.0								
25		SHALE, gray, with limestone seams, very hard	25.0	10							
	100/0.0"		600.0	9							
30		Note: Surface elevation is estimated from topographical data.	30.0								

KEY: THIN WALLED TUBE; AUGER; ROCK BIT; SPLIT BARREL; THD CONE; ROCK CORE; SEEPAGE LEVEL; WATER LEVEL

PLATE A. 9

START DATE: 3-5-96
END DATE: 3-5-96
COMPLETION DEPTH (FT): 15.00

NO WATER OBSERVED IN BOREHOLE

LOG OF BORING NO. B-7

COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS
Denton, Texas

TYPE: AUGER

LOCATION: See Plate A.1

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SAMPLES	SYMBOL	SOIL / ROCK CLASSIFICATION EXISTING GRADE ELEVATION: 623.00	H2O	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	3.0				FAT CLAY WITH SAND (CH), brown, with calcareous nodules, very stiff			25						1.6	
	3.0							22	52	22	30	80		1.5	
							618.0								
5	4.5+				LEAN CLAY WITH SAND (CL), tan to light gray, with calcareous nodules, hard		5.0	17	39	17	22	71		1.3	
	4.5+							19					111		2.211
							614.0								
	100/1.5"				LIMESTONE, tan to gray argillaceous, hard to very hard		9.0	10							
10															
	100/0.0"						608.0	7							
15							15.0								

Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE · AUGER · ROCK BIT · SPLIT BARREL · THD CONE · ROCK CORE · SEEPAGE LEVEL · WATER LEVEL

PLATE A. 10

FUGRO

START DATE: 3-5-96
END DATE: 3-5-96
COMPLETION DEPTH (FT): 30.00

NO WATER OBSERVED IN BOREHOLE

LOG OF BORING NO. B-8

COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS
Denton, Texas

TYPE: AUGER

LOCATION: See Plate A.1

EXISTING GRADE ELEVATION: 637.00

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SOIL / ROCK CLASSIFICATION	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	2.5		FAT CLAY (CH), brown, with organic material, trace of calcareous nodules, very stiff		25						1.5	
	2.5				28	57	24	33	90		1.1	
5	2.0		- free swell at 5 feet = 1.4 percent		33	62	24	38	84	95	1.1	1.736
	2.5				30						1.0	
			- sand seam	627.0								
10	4.5+		SANDY LEAN CLAY (CL), tan, with calcareous nodules, hard	10.0	16	40	19	21	74	121	1.1	4.6
				622.0								
15	4.5+		FAT CLAY (CH), light gray to olive gray, hard	15.0	18						2.5+	
20	4.5+				23	57	26	31	99		2.5+	
				614.0								
			SHALE, gray, hard	23.0								
	100/2.0"				18							
25												
	100/1.0"			607.0	17							
30				30.0								

Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE, AUGER, ROCK BIT, SPLIT BARREL, THD CONE, ROCK CORE, SEEPAGE LEVEL, WATER LEVEL

REPORT NO.: 1401-0026

FUGRO

START DATE: 3-5-96 END DATE: 3-5-96 COMPLETION DEPTH (FT): 30.00	WATER LEVEL OBSERVED AT 15.00 FT UPON COMPLETION OF DRILLING	LOG OF BORING NO. B-9 COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS Denton, Texas

TYPE: AUGER — LOCATION: See Plate A.1

LOG OF BORING NO. B-9

DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SAMPLES	SYMBOL	SOIL / ROCK CLASSIFICATION EXISTING GRADE ELEVATION: 661.00	H2O	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	2.5				LEAN CLAY WITH SAND (CL), tan, brown to gray, trace of organics, calcareous nodules, very stiff			14						0.3	
	2.0							25	47	23	24	80		1.1	
5	3.0							24						1.1	
					- limestone seams at 6 feet		654.0								
	4.5+				SANDY LEAN CLAY (CL), tan, traces of gravel, hard		7.0	10	29	16	13	56		1.1	
							651.0								
10	4.0				SANDY FAT CLAY (CH), tan to olive green		10.0	14	50	27	23	64	111	1.4	3.739
15	4.5+					▼	645.0	19						2.5+	
					LEAN CLAY (CL), gray, hard		16.0								
20	4.5+				- weathered limestone		640.0	18	49	23	26	99		2.5+	
					SHALE, gray, hard to very hard		21.0								
25	100/1.5"							19							
30	100/0.75"						631.0	17							
							30.0								

Note: Elevation is estimated from topographical data.

KEY: THIN WALLED TUBE; AUGER; ROCK BIT; SPLIT BARREL; THD CONE; ROCK CORE; SEEPAGE LEVEL; WATER LEVEL

PLATE A. 12

START DATE: 3-5-96 END DATE: 3-5-96 COMPLETION DEPTH (FT): 20.00	NO WATER OBSERVED IN BOREHOLE	**LOG OF BORING NO. B-10** COOPER & PECAN CREEK DRAINAGE IMPROVEMENTS Denton, Texas

TYPE: AUGER — LOCATION: See Plate A.1

EXISTING GRADE ELEVATION: 665.00

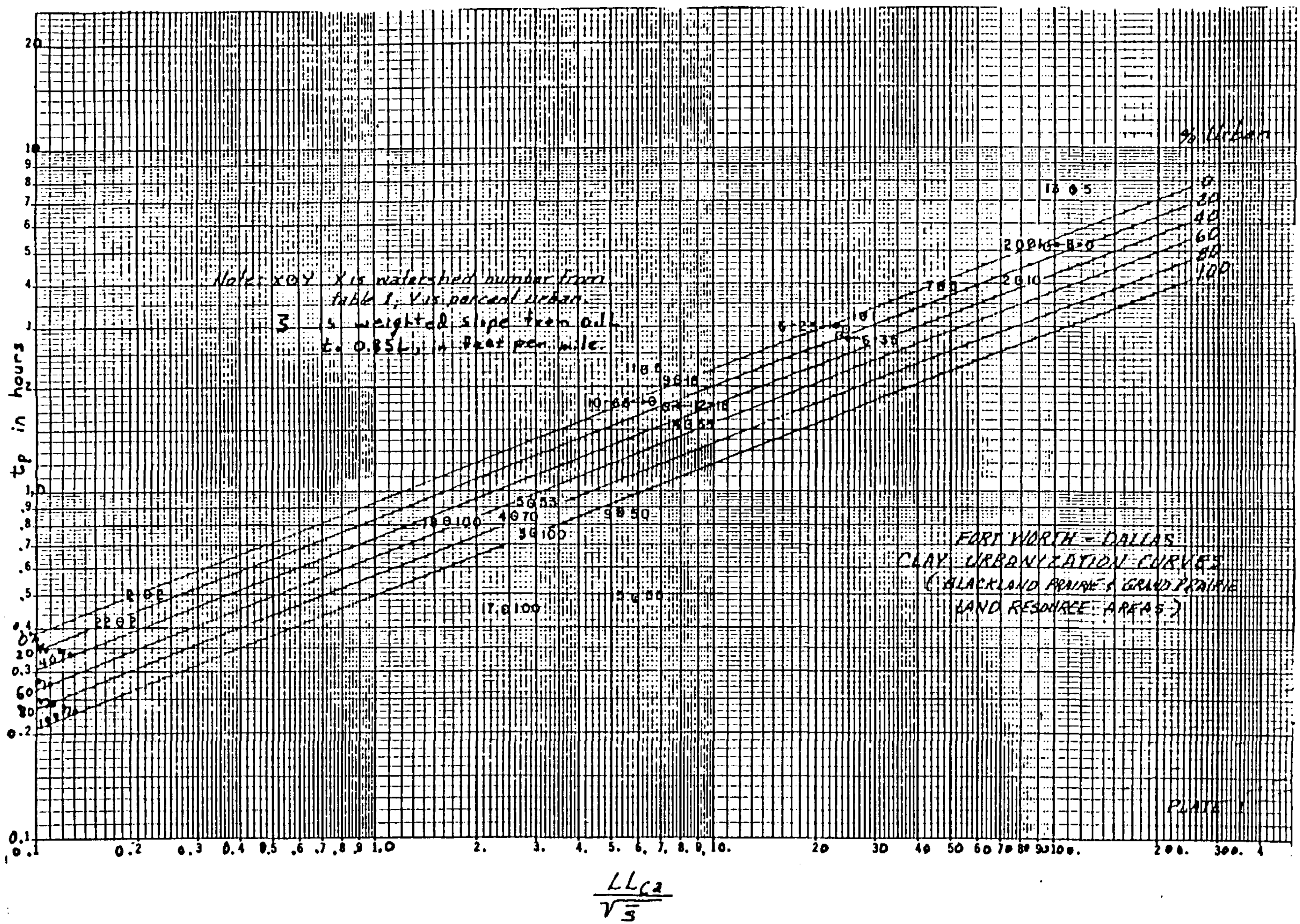
DEPTH, (FT)	POCKET PEN. (TSF) BLOWS N PER FOOT	REC. / RQD, %	SAMPLES	SYMBOL	SOIL / ROCK CLASSIFICATION	H2O	STRATA ELEVATION DEPTH (FT)	WATER CONTENT, %	LIQUID LIMIT (LL), %	PLASTIC LIMIT (PL), %	PLASTICITY INDEX (PI)	PASSING NO. 200 SIEVE, %	UNIT DRY WEIGHT, PCF	TORVANE (TSF)	UC (KSF)
	2.0				SANDY FAT CLAY (CH), brown, trace of gravel, calcareous nodules, very stiff			21						1.4	
	2.0							18	50	21	29	59		1.2	
5	4.0				SANDY FAT CLAY (CH), gray, tan, reddish-brown, hard to very hard		660.0 5.0	30	61	27	34	58	106	1.2	2.002
	100/0.75"				- argillaceous limestone seams										
10	4.5+				LEAN CLAY (CL), gray, weathered shale, hard		655.0 10.0	19	49	23	26	99		2.5+	
					SHALE, gray, hard, very hard		654.0 11.0								
15	100/1.0"							11							
20	100/0.5"						645.0 20.0	16							

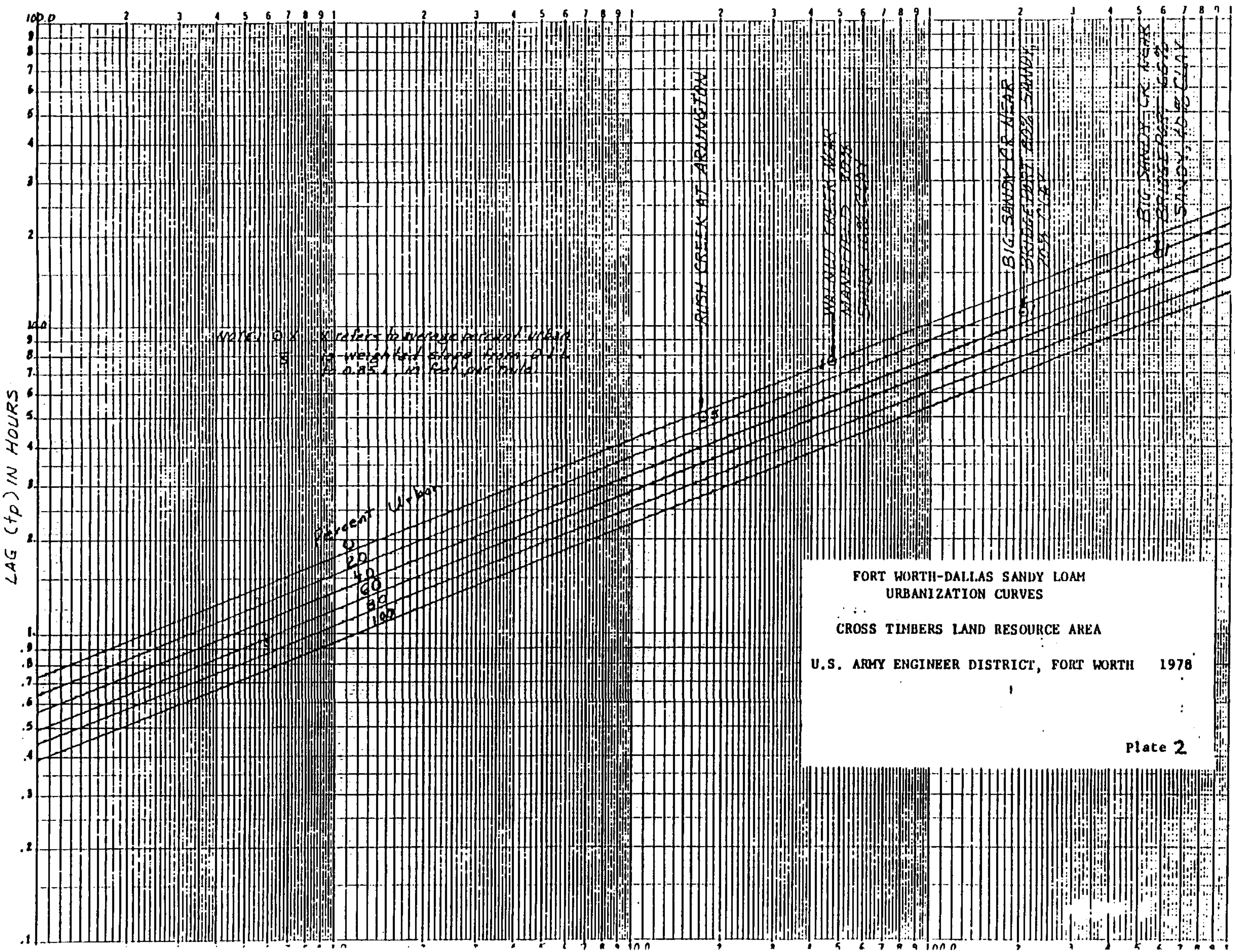
Note: Surface elevation is estimated from topographical data.

KEY: THIN WALLED TUBE; AUGER; ROCK BIT; SPLIT BARREL; THD CONE; ROCK CORE; SEEPAGE LEVEL; WATER LEVEL

APPENDIX B

DALLAS/FT. WORTH URBANIZATION CURVES


% Urban
0
20
40
60
80
100
Note: X Ø Y X is watershed number from table I, Y is percent urban.
S is weighted slope from 0.1L to 0.85L, in feet per mile.
t_p in hours
$\frac{LL_{ca}}{\sqrt{S}}$
FORT WORTH - DALLAS
CLAY URBANIZATION CURVES
(BLACKLAND PRAIRIE & GRAND PRAIRIE LAND RESOURCE AREAS)
PLATE 1



LAG (tp) IN HOURS
Percent Urban
0
20
40
60
80
100
RUSH CREEK AT ARLINGTON
WALNUT CREEK NEAR MANSFIELD
BIG SANDY CR NEAR BRIDGEPORT 80% SANDY
BIG SANDY CR NEAR
FORT WORTH-DALLAS SANDY LOAM
URBANIZATION CURVES
CROSS TIMBERS LAND RESOURCE AREA
U.S. ARMY ENGINEER DISTRICT, FORT WORTH 1978
Plate 2

APPENDIX C

COOPER CREEK REGIONAL DETENTION CONSTRUCTION COSTS

OPINION OF PROBABLE CONSTRUCTION COST

COOPER CREEK REGIONAL DETENTION PONDS

POND #1

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Clearing and grubbing	19	AC	$3,000.00	$55,786
2	Unclassified excavation	124,700	CY	$8.00	$997,600
3	Placement of select fill	1,190	CY	$15.00	$17,850
4	Furnish 9'x5' box culvert	40	LF	$350.00	$14,000
5	Headwalls for 9'x5' box culvert	2	EA	$3,500.00	$7,000
6	Hydromulch seeding	90,000	SY	$1.25	$112,500
7	Purchase property for construction	810,000	SF	$1.00	$810,000
				SUBTOTAL :	$2,014,736
				Mobilization (5%):	$100,737
				Overhead & Profit (10%):	$201,474
				SUBTOTAL :	$2,316,947
				Contingencies (30%):	$695,085
				TOTAL - POND #1:	**$3,012,032**

POND #2

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Clearing and grubbing	13	AC	$3,000.00	$39,945
2	Unclassified excavation	92,550	CY	$8.00	$740,400
3	Placement of select fill	3,600	CY	$15.00	$54,000
4	42" reinforced concrete pipe	50	LF	$200.00	$10,000
5	Headwalls for 42" concrete pipe	2	EA	$2,200.00	$4,400
6	Hydromulch seeding	64,444	SY	$1.25	$80,556
7	Purchase property for construction	580,000	SF	$1.00	$580,000
				SUBTOTAL :	$1,509,301
				Mobilization (5%):	$75,466
				Overhead & Profit (10%):	$150,931
				SUBTOTAL :	$1,735,698
				Contingencies (30%):	$520,710
				TOTAL - POND #2:	**$2,256,408**

OPINION OF PROBABLE CONSTRUCTION COST

COOPER CREEK REGIONAL DETENTION PONDS

POND #3

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Clearing and grubbing	4	AC	$3,000.00	$11,123
2	Unclassified excavation	26,400	CY	$8.00	$211,200
3	10'x7' Reinforced box culvert	100	LF	$550.00	$55,000
4	Headwalls for twin 10'x7' box culvert	2	EA	$4,000.00	$8,000
5	Hydromulch seeding	17,944	SY	$1.25	$22,431
6	Purchase property for construction	161,500	SF	$1.00	$161,500
			SUBTOTAL :		$469,254
			Mobilization (5%):		$23,463
			Overhead & Profit (10%):		$46,926
			SUBTOTAL :		$539,643
			Contingencies (30%):		$161,893
			TOTAL - POND #3:		$701,536

POND #4

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Clearing and grubbing	12	AC	$3,000.00	$35,813
2	Unclassified excavation	49,280	CY	$8.00	$394,240
3	Placement of select fill	6,000	CY	$15.00	$90,000
4	42" reinforced concrete pipe	50	LF	$200.00	$10,000
5	Headwalls for 42" concrete pipe	2	EA	$2,200.00	$4,400
6	Hydromulch seeding	57,778	SY	$1.25	$72,223
7	Purchase property for construction	520,000	SF	$1.00	$520,000
			SUBTOTAL :		$1,126,676
			Mobilization (5%):		$56,334
			Overhead & Profit (10%):		$112,668
			SUBTOTAL :		$1,295,678
			Contingencies (30%):		$388,704
			TOTAL - POND #4:		$1,684,382

OPINION OF PROBABLE CONSTRUCTION COST

COOPER CREEK REGIONAL DETENTION PONDS

POND #5

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Clearing and grubbing	11	AC	$3,000.00	$33,575
2	Unclassified excavation	37,500	CY	$8.00	$300,000
3	Placement of select fill	2,200	CY	$15.00	$33,000
4	Hydromulch seeding	54,167	SY	$1.25	$67,709
5	Purchase property for construction	487,500	SF	$1.00	$487,500
			SUBTOTAL :		$921,784
			Mobilization (5%):		$46,090
			Overhead & Profit (10%):		$92,179
			SUBTOTAL :		$1,060,053
			Contingencies (30%):		$318,016
			TOTAL - POND #5:		**$1,378,069**

POND #6

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Clearing and grubbing	4	AC	$3,000.00	$12,053
2	Unclassified excavation	56,140	CY	$8.00	$449,120
3	10'x7' Reinforced box culvert	50	LF	$550.00	$27,500
4	Headwalls for twin 10'x7' box culvert	2	EA	$4,000.00	$8,000
5	Hydromulch seeding	19,444	SY	$1.25	$24,306
6	Purchase property for construction	175,000	SF	$1.00	$175,000
			SUBTOTAL :		$695,979
			Mobilization (5%):		$34,799
			Overhead & Profit (10%):		$69,598
			SUBTOTAL :		$800,376
			Contingencies (30%):		$240,113
			TOTAL - POND #6:		**$1,040,489**

APPENDIX D

PEC-4 CHANNEL IMPROVEMENT COSTS

OPINION OF PROBABLE CONSTRUCTION COST

TRIBUTARY PEC-4 CHANNEL IMPROVEMENTS

REACH #1 - PECAN CREEK TO BRADSHAW ST.

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Site preparation	1	LS	$7,000.00	$7,000
2	Fill placement	16,440	CY	$15.00	$246,600
3	Remove existing culvert	1	LS	$1,500.00	$1,500
4	Concrete channel lining	888	CY	$250.00	$222,000
5	Furnish 10'x10' box culvert	200	LF	$700.00	$140,000
6	Headwalls for 10'x10' box culvert	2	EA	$3,500.00	$7,000
7	Asplalt street pavement at culvert	417	SY	$20.00	$8,340
8	Hydromulch seeding	2,556	SY	$1.25	$3,195
9	Purchase property for construction	3,050	SF	$0.80	$2,440
			SUBTOTAL :		$638,075
			Mobilization (5%):		$31,904
			Overhead & Profit (10%):		$63,808
			SUBTOTAL :		$733,787
			Contingencies (30%):		$220,137
			TOTAL - REACH #1:		**$953,924**

REACH #2 - BRADSHAW ST. TO LAKEY ST.

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Site preparation	1	LS	$4,000.00	$4,000
2	Unclassified channel excavation	1,071	CY	$8.00	$8,568
3	Remove existing culvert	1	LS	$1,500.00	$1,500
4	Concrete channel lining	670	CY	$250.00	$167,500
5	Furnish 10'x10' box culvert	200	LF	$700.00	$140,000
6	Headwalls for 10'x10' box culvert	2	EA	$3,500.00	$7,000
7	Asplalt street pavement at culvert	417	SY	$20.00	$8,340
8	Hydromulch seeding	1,889	SY	$1.25	$2,362
9	Purchase property for construction	23,523	SF	$0.80	$18,819
			SUBTOTAL :		$358,089
			Mobilization (5%):		$17,905
			Overhead & Profit (10%):		$35,809
			SUBTOTAL :		$411,803
			Contingencies (30%):		$123,541
			TOTAL - REACH #2:		**$535,344**

OPINION OF PROBABLE CONSTRUCTION COST

TRIBUTARY PEC-4 CHANNEL IMPROVEMENTS

REACH #3 - LAKEY ST. TO SKINNER ST.

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Site preparation	1	LS	$5,000.00	$5,000
2	Unclassified channel excavation	2,300	CY	$8.00	$18,400
3	Remove existing culvert	1	LS	$1,500.00	$1,500
4	Concrete channel lining	1,080	CY	$250.00	$270,000
5	Furnish 10'x10' box culvert	200	LF	$700.00	$140,000
6	Headwalls for 10'x10' box culvert	2	EA	$3,500.00	$7,000
7	Asplalt street pavement at culvert	417	SY	$20.00	$8,340
8	Hydromulch seeding	3,112	SY	$1.25	$3,890
9	Purchase property for construction	37,898	SF	$0.80	$30,319
				SUBTOTAL :	$484,449
				Mobilization (5%):	$24,223
				Overhead & Profit (10%):	$48,445
				SUBTOTAL :	$557,117
				Contingencies (30%):	$167,136
				TOTAL - REACH #3:	**$724,253**

REACH #4 - SKINNER ST. TO BELL AVE.

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Site preparation	1	LS	$5,000.00	$5,000
2	Unclassified channel excavation	1,837	CY	$8.00	$14,696
3	Remove existing culverts	1	LS	$3,000.00	$3,000
4	Railroad shoo-fly	2,000	LF	$200.00	$400,000
5	Concrete channel lining	607	CY	$250.00	$151,750
6	Furnish 10'x9" box culvert	440	LF	$630.00	$277,200
7	Headwalls for 10'x9" box culvert	4	EA	$3,500.00	$14,000
8	Asplalt street pavement at culvert	140	SY	$20.00	$2,800
9	Hydromulch seeding	1,778	SY	$1.25	$2,223
10	Purchase property for construction	22,652	SF	$0.80	$18,122
				SUBTOTAL :	$888,791
				Mobilization (5%):	$44,440
				Overhead & Profit (10%):	$88,880
				SUBTOTAL :	$1,022,111
				Contingencies (30%):	$306,634
				TOTAL - REACH #4:	**$1,328,745**

OPINION OF PROBABLE CONSTRUCTION COST

TRIBUTARY PEC-4 CHANNEL IMPROVEMENTS

REACH #5 - BELL AVE. TO LOCUST ST.

	Description	Quantity	Unit	Engineer's Estimate Unit Price	Total
1	Site preparation	1	LS	$6,000.00	$6,000
2	Unclassified channel excavation	4,254	CY	$8.00	$34,032
3	Remove existing culvert	1	LS	$3,000.00	$3,000
4	Concrete channel lining	1,102	CY	$250.00	$275,500
5	Furnish 10'x7' box culvert	360	LF	$490.00	$176,400
6	Headwalls for 10'x7' box culvert	2	EA	$3,500.00	$7,000
7	Asplalt street pavement at culvert	834	SY	$20.00	$16,680
8	Hydromulch seeding	5,340	SY	$1.25	$6,675
9	Purchase property for construction	26,136	SF	$0.80	$20,909
			SUBTOTAL :		$546,196
			Mobilization (5%):		$27,310
			Overhead & Profit (10%):		$54,620
			SUBTOTAL :		$628,126
			Contingencies (30%):		$188,438
			TOTAL - REACH #5:		$816,564

OPINION OF PROBABLE CONSTRUCTION COST

TRIBUTARY PEC-4 CHANNEL IMPROVEMENTS

ROBERTSON STREET BYPASS

				Engineer's Estimate	
	Description	Quantity	Unit	Unit Price	Total
1	Site preparation	1	LS	$4,000.00	$4,000
2	Remove existing pavement	270	SY	$7.50	$2,025
3	Relocate utilities	1	LS	$7,500.00	$7,500
4	Concrete channel lining	607	CY	$250.00	$151,750
5	Furnish 9'x8' box culvert	440	LF	$504.00	$221,760
6	Headwalls for 9'x8' box culvert	2	EA	$3,500.00	$7,000
7	Asplalt street pavement at culvert	270	SY	$20.00	$5,400
8	Hydromulch seeding	1,780	SY	$1.25	$2,225
			SUBTOTAL :		$399,435
			Mobilization (5%):		$19,972
			Overhead & Profit (10%):		$39,944
			SUBTOTAL :		$459,351
			Contingencies (30%):		$137,806
			TOTAL - REACH #6:		**$597,157**

DALLAS DRIVE DIVERSION

				Engineer's Estimate	
	Description	Quantity	Unit	Unit Price	Total
1	66" reinforced concrete pipe	1,300	LF	$270.00	$351,000
2	8' square manhole	2	EA	$5,000.00	$10,000
3	Hydromulch seeding	4,340	SY	$1.25	$5,425
			SUBTOTAL :		$366,425
			Mobilization (5%):		$18,322
			Overhead & Profit (10%):		$36,643
			SUBTOTAL :		$421,390
			Contingencies (30%):		$126,417
			TOTAL - REACH #6:		**$547,807**

APPENDIX E

REFERENCES

LIST OF REFERENCES

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