Amended
Master Drainage Basin Planning Study

for

NORTH SHOOK'S RUN
TEMPLETON GAP BASIN A
Sub-basin 2

Prepared for

The
City of
Colorado Springs

Submitted by:

ENGINEERING PROFESSIONALS INC.
Fort Collins • Colorado Springs

FOR CHECK-OUT ONLY

December 26, 1989

Mr. Gary Haynes, P. E.
City of Colorado Springs
Department of Public Works
Engineering Division
P. O. Box 1575
Mail Code 435
Colorado Springs, Colorado 80901

RE: NORTH SHOOK'S RUN, TEMPLETON GAP BASIN A, SUB-BASIN 2 MASTER DRAINAGE STUDY

Dear Mr. Haynes:

Attached is a copy of the Amended Master Drainage Basin Panning Study for North Shook's Run, Templeton gap Basin A, Sub-basin 2. The study and report is the culmination of our engineering efforts pertaining to this sub-basin located in the vicinity of Old Farm. The report was authorized by the Colorado Springs City Council and Department of Public Works.

The study should be used as a guide for future drainage improvements within this sub-basin. Numerous alternatives were studied before recommending the alternative which is outlined in the report.

If you have any questions concerning this report please do not hesitate to call.

Respectfully submitted,

ENGINEERING PROFESSIONALS, INC.

John P. McGinn, P.E.

Andrew W. McCord, P.E.

JPM:AWMc/cms

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Offices also located in Fort Collins, CO and Newport Beach, CA

ENGINEER'S CERTIFICATION:

The attached master drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the City of Colorado Springs for drainage reports and said report is in conformity with the master plan of the drainage basin. I accept responsibility for any liability caused by negligent acts, errors or omissions on my part in preparing this report.

COLORADO REGISTERED JOHN P. McGINN 19534 PROFESSIONAL ENGINEER

John P McGinn 19534 ____________ 12/26/89

Date

Master Drainage Basin Planning Study
North Shook's Run, Templeton Gap Basin A, Sub-basin 2

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Master Drainage Basin Planning Study
North Shook's Run, Templeton Gap Basin A, Sub-basin 2

INTRODUCTION

Authorization

This Amended Master Drainage Basin Planning Study was authorized by the City of Colorado Springs. The study is an update of a portion of the Master Drainage Basin Planning Study titled, *Engineering Study and Revision of the North Shook's Run -- Templeton Gap Drainage Basin*, prepared by Lincoln DeVore in 1977.

Scope and Purpose of Report

The purpose of this study is to analyze the existing and future drainage conditions within this subbasin, determine the potential drainage problems, and develop a drainage improvement scheme to reduce future flooding impact on the area.

The specific scope of work for the entire project includes the following:

Task 1: Prepare base mapping for the study area using photogrammetric mapping methods combined with modern field surveying techniques.

Task 2: Prepare an existing facilities inventory for the entire planning area.

Task 3: Perform a hydrologic analysis of the study area utilizing the *City of Colorado Springs/El Paso County Drainage Criteria Manual.*

Task 4: Perform hydraulic calculations and analysis of existing structures within the study area.

Task 5: Develop drainage system alternatives which will alleviate problems due to inadequate drainage facilities.

Task 6: Prepare a preliminary design report for alternatives based on the previously completed work and input from the City Engineering Division staff.

Task 7: Prepare an Amended Master Drainage Plan for North Shook's Run, Templeton Gap Basin A, Sub-basin 2, which includes the selected drainage system alternative and incorporates review comments from the City Engineering Division and other agencies affected by the study.

Summary of Data Obtained

The following is a list of technical information used in the preparation of this report:

1. "City of Colorado Springs/El Paso County Drainage Criteria Manual", prepared jointly by City of Colorado Springs Department of Public Works Engineering Division, HDR Infrastructure, Inc., El Paso County Department of Public Works Engineering Division, dated October 1987.

2. "City of Colorado Springs Engineering Division Standard Specifications", prepared by City Engineering Division, dated March 1989.

3. "Engineering Study and Revision of the North Shook's Run -- Templeton Gap Drainage Basin", prepared by Lincoln DeVore, dated September 1977.

4. Soil Survey of El Paso County Area, Colorado, prepared by U. S. Department of Agriculture, Soil Conservation Service.

5. "Drainage Report The Ridge Subdivision", prepared by R. Keith Hook and Associates, Inc., dated February 1973.

6. "Drainage Report Turquoise Subdivision", prepared by Leigh Whitehead and Associates, dated August 15, 1973.

7. "Turquoise Subdivision Filing No. 2 Drainage Report and Plan", prepared by Leigh Whitehead and Associates, dated March 25, 1978.

8. "Drainage Report for Old Farm Subdivision No. 1", prepared by H. J. Kraettli and Sons, dated June 5, 1978.

9. "Drainage Report for Old Farm Subdivision No. 2", prepared by H. J. Kraettli and Sons, dated September 26, 1978.

10. "Drainage Report for Old Farm Subdivision No. 4", prepared by H. J. Kraettli and Sons, Inc., dated November 15, 1978.

11. "Drainage Report and Plan Phelan Subdivision", prepared by Leigh Whitehead and Associates, dated August 22, 1979.

12. "Drainage Report for Old Farm Subdivision No. 7", prepared by Berge-Brewer and Associates, Inc. dba H. J. Kraettli and Sons, dated August 3, 1982.

13. "Drainage Report for Old Farm Subdivision No. 9", prepared by Berge-Brewer and Associates, Inc. dba H. J. Kraettli and Sons, dated November 18, 1982.

14. "Drainage Report for Old Farm Business Park", prepared by Berge-Brewer and Associates, Inc., dated October 5, 1983.

15. "Drainage Report for Starwatch Subdivision", prepared by Berge-Brewer and Associates, Inc., dated December 9, 1983.

16. "Drainage Report for Old Farm Center", prepared by Berge-Brewer and Associates, Inc., dated November 1, 1984.

17. "Drainage Report for Templeton Heights Filing No. 1 and Master Drainage Study", prepared by Weiss Consulting Engineers, Inc., dated September 24, 1987.

18. "Preliminary Design Report North Shook's Run, Templeton Gap Basin A, Sub-basin 2", prepared by Engineering Professionals, Inc., dated May 3, 1989.

The following is a list of planning and mapping information used in the preparation of this report:

1. Drawings for Old Farm Center Preliminary Grading and Landscaping Plan, prepared by Yergensen, Obering and Whittaker, dated November 10, 1987, City Planning Department No. PD DP 84-378-A1(87) approved December 1, 1987.

2. Plat map of Old Farm Subdivision Filing No. 1, prepared by H. J. Kraettli and Sons, dated April 17, 1978.

3. Plat map of Old Farm Business Park, prepared by Berge-Brewer and Associates, Inc., dated October 12, 1983.

4. Plat map of Starwatch Filing No. 1, prepared by Berge-Brewer and Associates, Inc.

5. Plat map of Starwatch Filing No. 2, prepared by Berge-Brewer and Associates, Inc., dated February 1, 1985.

6. Water utility location maps O11, O12, P11, and P12, prepared by City of Colorado Springs, Department of Utilities, Water Division.

7. Wastewater utility location maps O11, O12, P11, and P12, prepared by City of Colorado Springs, Department of Utilities, Wastewater Division.

8. Gas Utility location maps O11, O12, P11, and P12, prepared by City of Colorado Springs, Department of Utilities, Gas Division.

Mapping and Surveying

The sources of topographic mapping used in conjunction with this project are listed below.

1. Two-foot contour interval, 1-inch to 200-feet scale topographic map, prepared by Landmark Mapping, Ltd., Lakewood, Colorado, Dated May 1989.

2. 7.5 Minute quadrangle maps prepared by the U.S. Geological Survey.

3. One-foot contour interval, 1-inch to 10-feet scale topographic mapping, prepared by Engineering Professionals, Inc., Colorado Springs, Colorado, Dated May 1989.

BASIN CHARACTERISTICS

Basin Description

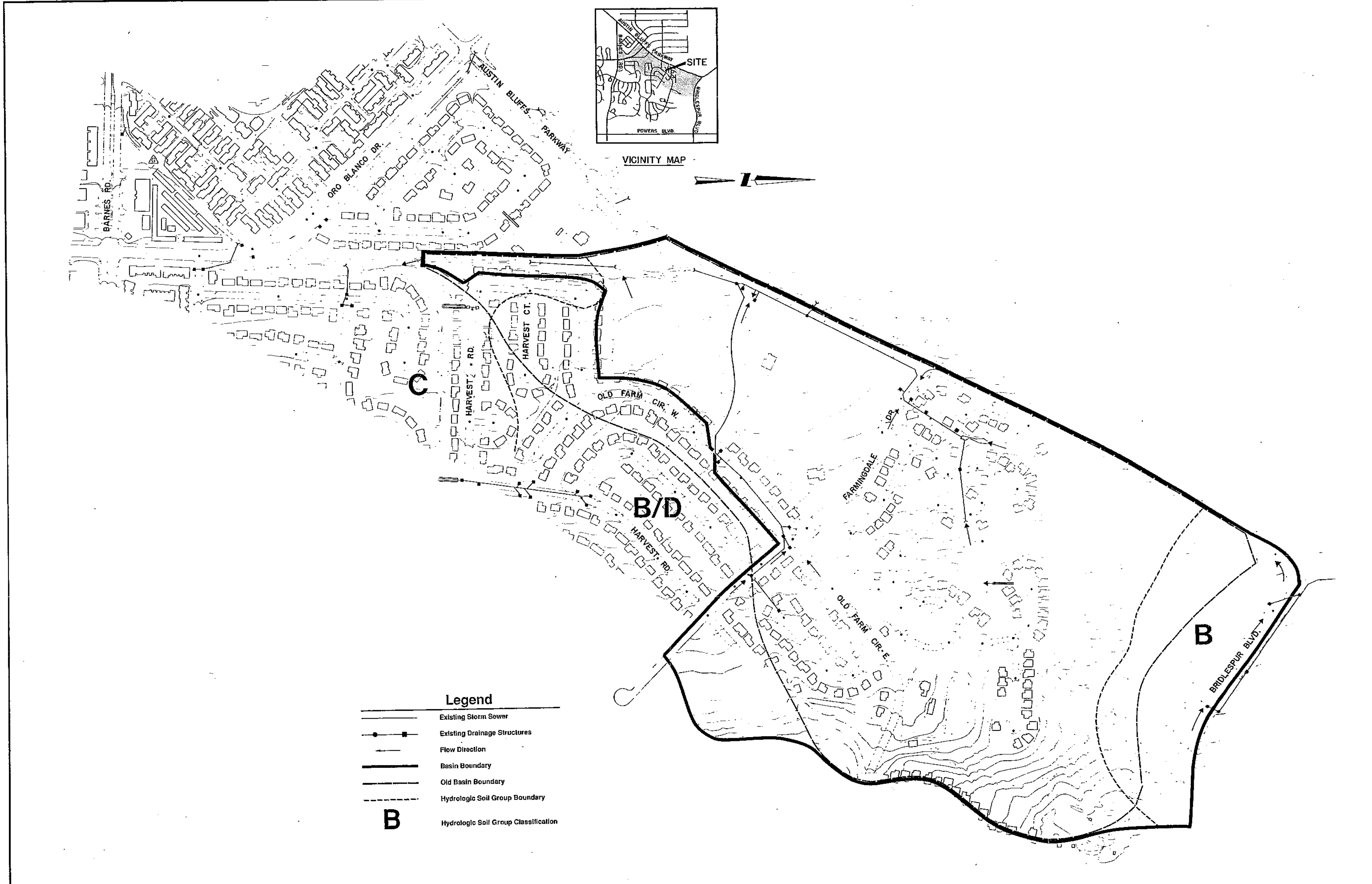
The basin being studied in this report is a subbasin of the North Shook's Run Templeton Gap Drainage Basin. The overall master drainage basin encompasses over eleven square miles of area in northeastern Colorado Springs. Some portions of the overall master drainage basin are under the jurisdiction of El Paso County. The basin being evaluated in this report is designated as Basin A Subbasin 2 in the Templeton Gap Master Drainage Basin Planning Study. The subbasin is generally in an area commonly known as Old Farm.

The basin is of an elongated fan shape with the axis of the fan running northeast-southwest. The northwestern basin boundary is Austin Bluffs Parkway, previously known as Templeton Gap Road. The northern boundary is the newly constructed Bridal Spur Boulevard. The eastern boundary runs through the existing 2 million gallon Templeton Gap water storage tank and continues in a southerly direction adjacent to Silo Ridge across Old Farm Circle East and along the ridge in the park land to Farmingdale Drive. The boundary then follows Farmingdale Drive to the northwest to approximately Old Farm Circle West. The boundary then follows Old Farm Circle West to Harvest Road where it turns to the west. The boundary then follows the existing concrete-lined channel to the south along the eastern side to the confluence with the main drainage channel. The boundary then turns to the north and runs along the west side of the concrete-lined channel to Austin Bluffs Parkway. The basin boundary is depicted on Figure 1 and shows the relationship to the old basin boundary.

The basin in the original configuration shown in the Master Drainage Basin Planning Study contained approximately 128 acres. The basin currently contains about 145 acres. The change in acreage has been caused by the shift in the northerly and easterly basin boundaries due to the urbanization of the area. Street patterns and grading have changed the drainage patterns of the area. With the construction of Bridal Spur Boulevard some runoff that would have previously drained to the Cottonwood Creek Basin is now draining into the North Shook's Run Templeton Gap Basin.

The topography of the basin is varied. The southern end of the basin is generally flat with slopes from about three to eight percent. To the north the basin becomes quite steep in areas. This area is generally between Bridal Spur Boulevard and the Starwatch Subdivisions. Fall across the entire basin is about 315 feet with over 200 feet of the fall occurring between Farmingdale Drive and Bridal Spur Boulevard.

Before development of the basin there was a well defined channel that ran generally to the south and somewhat parallel to and east of Austin Bluffs Parkway. The channel has been filled in and replaced with a storm sewer network throughout most of the basin. The remains of the original channel is visible in areas within and upstream of the Starwatch Subdivisions as well as in the undeveloped area south of Old Farm Drive.



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DRAWN dlb	DATE May 1989	

CLIENT: City of Colorado Springs

TITLE: Templeton Gap Basin A Sub-basin 2 Basin Boundary -- Figure 1

JOB NO. 743.3

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Climate

The Colorado Springs area can be described, in general as semi-arid high plains. Precipitation amounts in the region range from 14 to 16 inches per year. The majority of the precipitation occurs as rainfall during the spring and summer months. Thunderstorms are quite common during the summer, and are typified by quick-moving low pressure cells which draw moisture from the Gulf of Mexico into the region.

Soils and Geology

Soils within the study area vary from hydrologic soil type B through D, as identified by the United States Department of Agriculture, Soil Conservation Service. The hydrologic soil group classifications are shown on Figure 2. The predominant soil types are Bresser, Nunn, and Stapleton-Bernal. In general, the relatively flatter sloped areas have moderate infiltration rates and the steeper areas have low infiltration rates. The runoff from this basin is relatively high due in part to the poor infiltration of the soil and the steep slopes. Another factor contributing to the high runoff rate is the urbanization of the basin. Development has left more areas with impermeable surfaces such as roads and roof tops.

Relationship to Other Subbasins

Basin A Subbasin 2 is generally a self-contained drainage basin. With the construction of Austin Bluffs Parkway, formerly Templeton Gap Road, runoff from the northwest has been diverted away from the basin. Portions of Basin A Subbasin 1, northwest of Subbasin 2, historically flowed into Subbasin 2. Austin Bluffs Parkway has effectively diverted this water to the southwest. Some of the runoff from Subbasin 1 can cross Austin Bluffs Parkway through a 30" CMP and during larger storm events by overtopping the roadway. The Master Drainage Basin Planning Study shows a proposed 27" pipe crossing Austin Bluffs at approximately Old Farm Drive. This pipe would carry runoff picked up by two eight-foot inlets and one twelve-foot inlet. This is the only runoff water which is external to subbasin 2. Presently, a 30" CMP is located perpendicular to Austin Bluffs Parkway between Farmingdale Drive and Old Farm Drive. Drainage patterns indicate that very little water enters this pipe and crosses into Subbasin 2.

Present Development

The basin is approximately fifty percent developed. Current land use within the basin is predominantly single-family residential. Other land uses include a convenience store located at the corner of Old Farm Drive and Austin Bluffs Parkway, and a church located south of Farmingdale Drive between Old Farm Circle West and Austin Bluffs Parkway. The City of Colorado Springs operates a water storage tank in the northeast corner at the high point of the basin.

The Old Farm residential subdivisions are completely built-out while the predominant residential development in the basin, Starwatch Subdivision, is about fifty percent complete. Starwatch Subdivision is a private subdivision which was built without strict compliance to the City of Colorado Springs drainage criteria. A significant amount of this subdivision is platted as utility and drainage parcels. Most of this land will likely remain in its natural state because of steep slopes in the area and the existence of utility services. The present subdivisions in the vicinity of Basin A Subbasin 2 are shown on Figure 3. Figure 4 is a land use map of the same area.

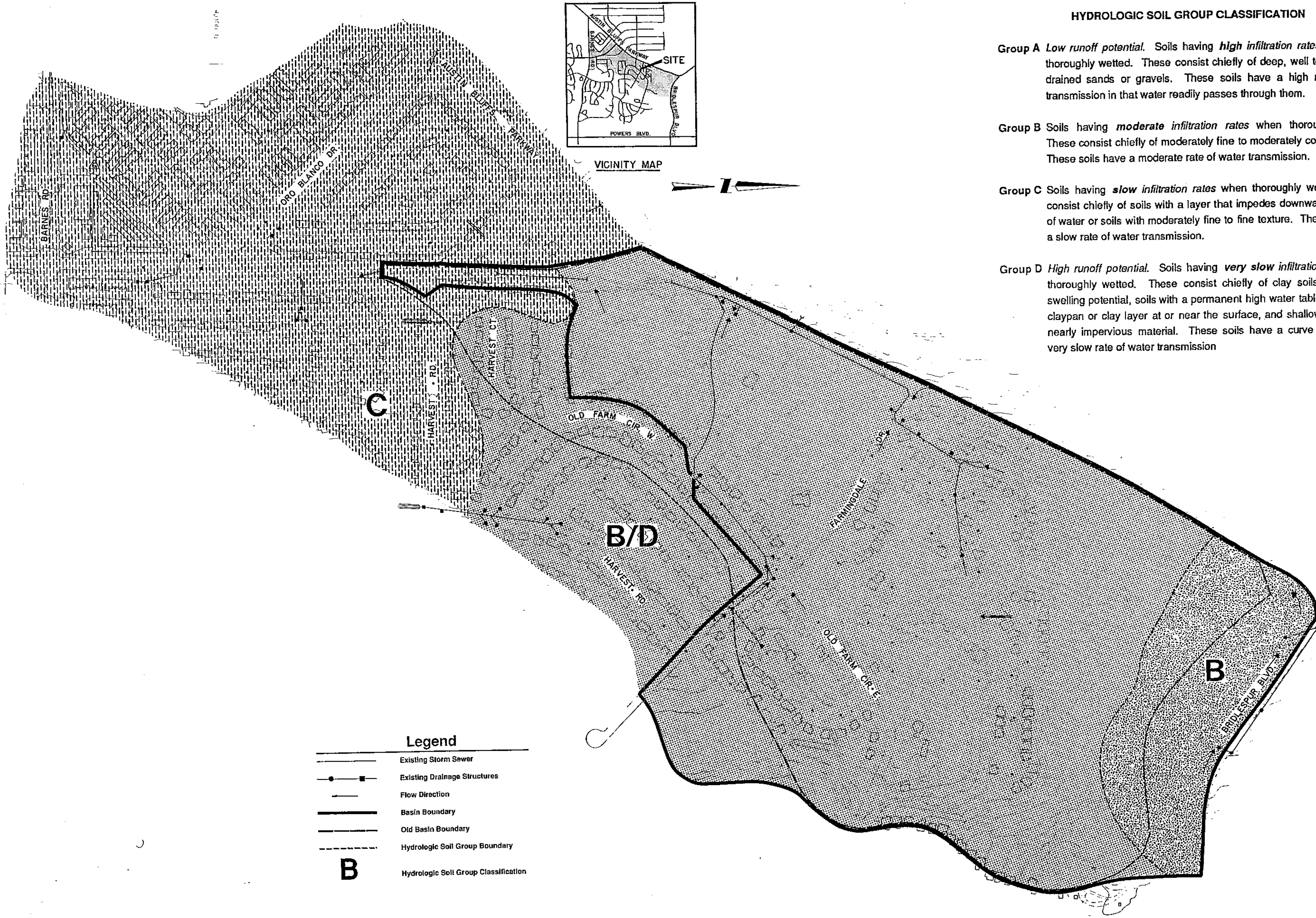
Future Development

Future development in the basin will occur in three general areas. These areas are Templeton Heights, Old Farm Center, and the unplatted land east of Old Farm Business Park. The Templeton Heights area located between Bridal Spur Boulevard and Starwatch subdivision has been master planned to include medium density residential, park land, and a business park.

The Old Farm Center area, bounded by Austin Bluffs Parkway, Farmingdale Drive, Old Farm Circle West, and Old Farm Drive, is planned as a Business Center. This area had previously been approved for development as a strip type commercial center.

The undeveloped area southeast of Old Farm Drive and Austin Bluffs Parkway is zoned for office development. Our research indicates there are no approved plans for this site.

The future development proposed for the study area includes land uses which will generate relatively high runoff amounts. With the completion of these developments the runoff potential from the basin will be dramatically increased over the present potential.



HYDROLOGIC SOIL GROUP CLASSIFICATION

Group A *Low runoff potential.* Soils having ***high infiltration rates*** even when thoroughly wetted. These consist chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission in that water readily passes through them.

Group B Soils having ***moderate*** *infiltration rates* when thoroughly wetted. These consist chiefly of moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

Group C Soils having ***slow*** *infiltration rates* when thoroughly wetted. These consist chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

Group D *High runoff potential.* Soils having ***very slow*** *infiltration rates* when thoroughly wetted. These consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over a nearly impervious material. These soils have a curve number for a very slow rate of water transmission

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TITLE: Templeton Gap Basin A Sub-basin 2 Soils Map -- Figure 2

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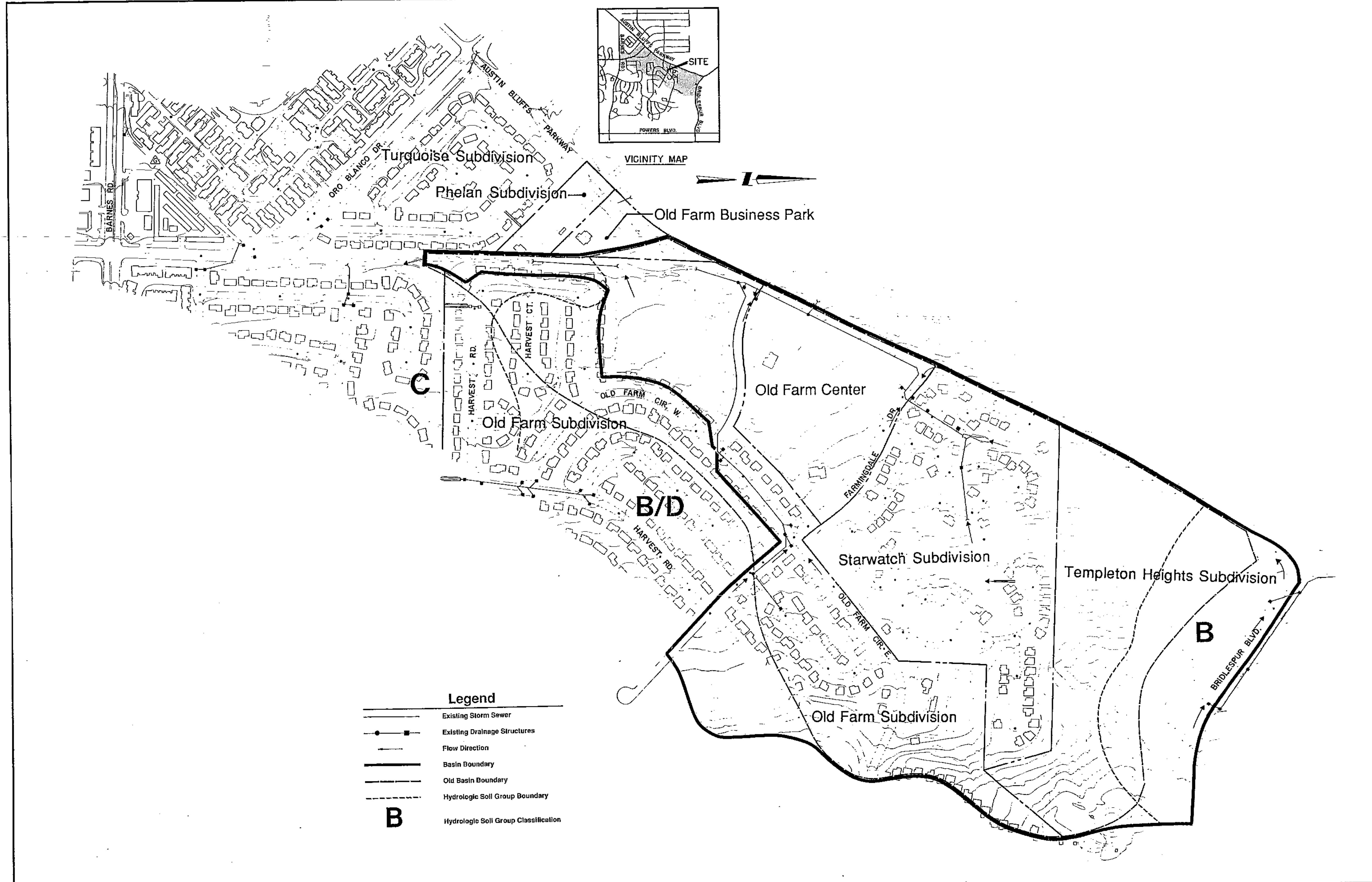

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CLIENT: City of Colorado Springs

TITLE: Templeton Gap Basin A Sub-basin 2 Subdivision Map -- Figure 3

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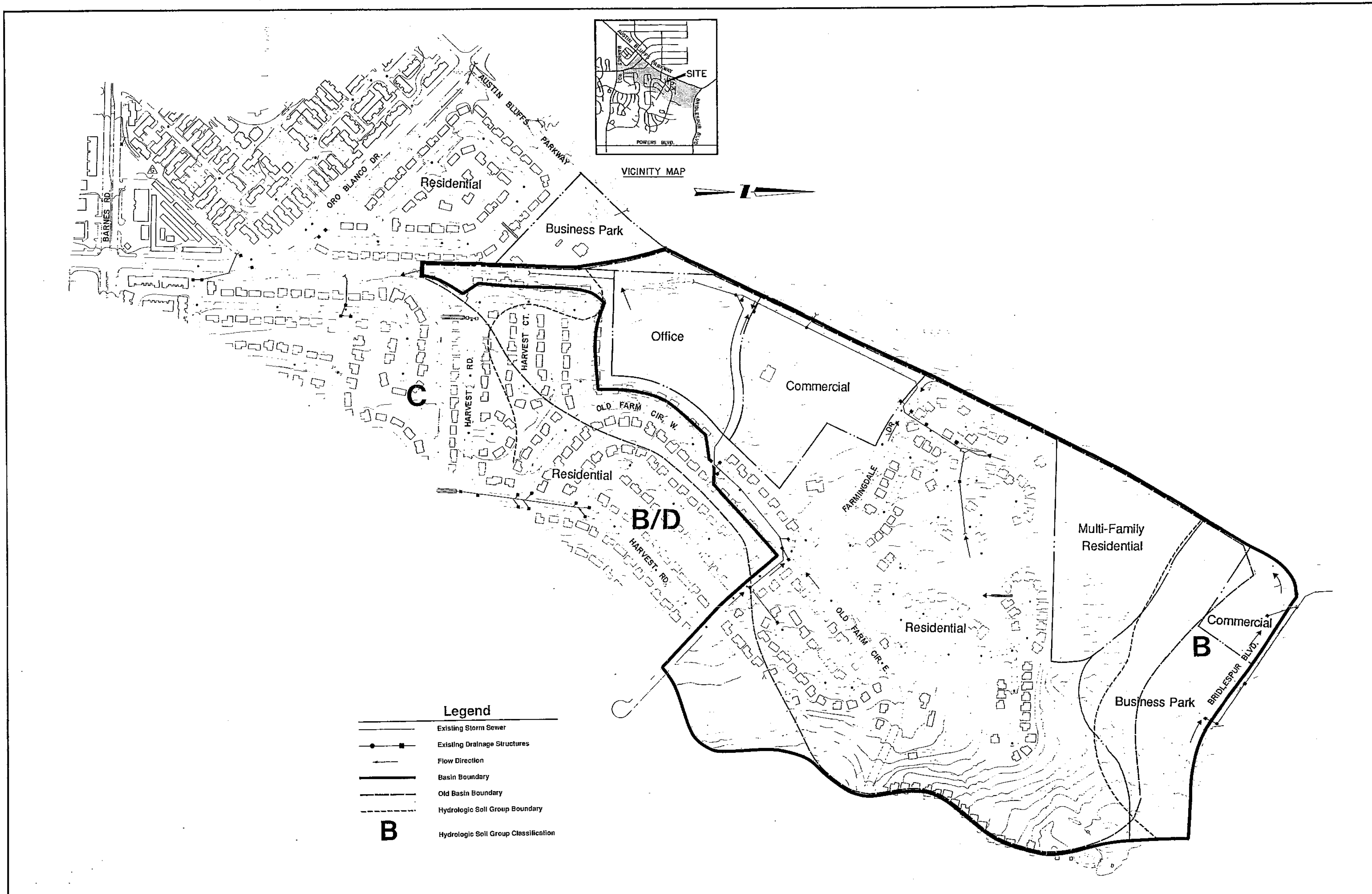

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CLIENT: City of Colorado Springs

TITLE: Templeton Gap Basin A Sub-basin 2 Land Use Map -- Figure 4

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PAST STUDIES

Master Basin Studies

The current Master Drainage Basin Planning Study governing the study area is titled *Engineering Study and Revision of the North Shook's Run -- Templeton Gap Drainage Basin*, prepared by Lincoln DeVore and dated September 1977. This study was an update of the original North Shook's Run -- Templeton Gap Master Basin Study prepared in September 1963 by United Western Engineers.

The adopted Master Drainage Basin Planning Study was prepared using criteria which is no longer accepted by the City of Colorado Springs or El Paso County. A new joint City/County criteria manual was adopted by both entities in late 1987. This new criteria is considerably different from the criteria used to complete the present Master Drainage Basin Planning Study. Most notably the design storms have changed. The previous study used two design storms; the five-year, six-hour storm with a 2.1 inch rainfall amount; and the 100-year, six-hour storm with a 3.5 inch rainfall amount. The rainfall distribution of both storms was patterned after the Soil Conservation Service's Type IIA storm.

The current criteria calls for four storms to be analyzed. They are the 100 year, 24 hour storm; the 10 year, 24 hour storm; the 100 year, 2 hour storm; and the 10 year, 2 hour storm. The amount of rainfall for these storms varies throughout the county. From the isopluvial maps in the Drainage Criteria Manual, the rainfall amounts used in this study are 4.4 inches, 3.1 inches, 3.05 inches, and 2.06 inches for the respective storms. The effect of the criteria change was to increase the initial storm's (5 and 10 year) runoff amount. The runoff from the 100 year storm from the new criteria is higher than that from the old criteria but the increase is not of the same relative magnitude as the increase in runoff from the initial storm. In fact, the peak discharge for the 100-year six-hour storm, 100-year 24-hour storm, and the 100-year two-hour storm are fairly comparable.

The other notable change in the criteria is the capacity of inlets. In the previous criteria a table of allowable inlet capacities was used. The current criteria calls for calculation of the inlet capacity based upon slope, flow to the inlet, shape of the inlet, and cross section of the road. The effect of this change has been to more closely model the actual capacity of curb inlets based on the actual flow characteristics of the inlet. The capacity of the inlet varies, as it does in the field, allowing for more inlet capacity as the flow to the inlet increases.

Drainage Reports

The approved individual drainage reports within the basin have been reviewed during the preparation of this report although little emphasis was placed on these reports other than to determine general drainage patterns and possible drainage restrictions placed upon the developer. Hydrology was performed using the new criteria and the recently flown topographic map. The drainage reports were used to supplement information compiled from the existing drainage conditions.

HYDROLOGIC ANALYSIS

The hydrologic analysis of North Shook's Run, Templeton Gap Basin A, Subbasin 2 has focused primarily on determining the future condition peak flow rates for the 100-year and 10-year storms with durations of both 24-hours and 2-hours. Consideration of the existing conditions was used during the hydrologic analysis. Hydrologic calculations established during the analysis of this basin have been included in the Technical Addendum to the "Preliminary Design Report, North Shook's Run, Templeton Gap Basin A, Sub-basin 2," prepared by Engineering Professionals, Inc., dated May 3, 1989. The hydrologic calculations are summarized within this report.

Rainfall

Precipitation amounts used in this report were established using isopluvial maps from *NOAA Atlas, Volume III* contained in the City of Colorado Springs & El Paso County Drainage Criteria Manual. Rainfall amounts used in this report were 4.4 inches, 3.1 inches, 3.05 inches, and 2.06 inches for the 100-year, 24-hour storm; 10-year, 24-hour storm; 100-year, 2-hour storm; and the 10-year, 2-hour storm, respectively. The rainfall pattern for both storm durations were modeled after the Soil Conservation Service's Type IIA storm. Antecedent moisture condition II was used for determining actual runoff amounts for the 24-hour duration storms. Runoff amounts for the 2-hour duration storms were determined using antecedent moisture condition III in order to model runoff potential from short duration "thunderstorms" occurring on successive days, a condition which often occurs in this region.

The 24-hour duration storm models a general type storm of high volume, high intensity and long duration. The 2-hour storm models the more frequent "cloud burst." This storm is typified by low volume, short duration, and high intensity.

Soils and Land Use

In order to determine runoff amounts from this basin, soil types and land use information are needed along with the amount of precipitation. All of this data, including precipitation amounts, antecedent moisture condition, land use, topography, and soil conditions, is fed into the hydrologic model to generate peak flow information for each given condition.

As shown on Figure 2, the basin is made up of Type B, C, and D soil groups, as described in the *Soil Survey of El Paso County Area, Colorado*, prepared by U. S. Department of Agriculture. Descriptions of the different Hydrologic Soil Groups are outlined on Figure 2.

Figure 3 and 9 Figure 4 show the subdivisions and the land use, respectively, associated with this basin. The subdivision map shows areas which have previously been platted in and near the basin. The land use map was compiled using platting and zoning information. These maps provided information which determined the curve number designation for each particular subbasin within the study. The curve number was used in the hydrologic model to determine peak runoff and volume of runoff.

Hydrologic Results

The computer model used to generate the hydrologic output is *Hydrosim*, developed by Ronald K. Christensen, P.E. for Hydrosoft Microcomputer Software. The software is a complete hydrograph analysis package which has the capability to model complex drainage basins.

A description of the capabilities of *Hydrosim* is provided in the Technical Addendum to the "Preliminary Design Report, North Shook's Run, Templeton Gap Basin A, Sub-basin 2."

Hydrosim was used to develop the 100-year and 10-year peak flows for both 24-hour and 2-hour storms. Peak flow amounts are tabulated and presented in the table on Exhibit II, Hydrology. It should be noted that some of the peak flows listed in this table have been truncated by the computer program due to the existing pipe at a particular design point. The computer model uses the existing pipe to route the upstream hydrograph. When the pipe size is inadequate to handle the entire flow, the model assumes that the pipe has an infinite amount of head available above the pipe. This is inconsistent with actual conditions in the basin. Most pipes do not have the headroom required to develop sufficient head for the pipe. Instead, the water will overtop the existing pipe and flow overland. This inconsistency actually allows for easy determination of inadequate pipe sizing.

Exhibit I, Recommended Alternative, shows actual peak flow amounts to the various drainage structures and points of interest within the basin. These flows were determined from routing the runoff through the ultimate storm sewer system proposed for the basin.

HYDRAULIC ANALYSIS

General

The hydraulic analysis associated with this study focused on determining the adequacy of the existing drainage structures, as well as determining future facilities to aleviate present problems. Information used in the analysis included the topographic map specifically made for this study, additional topography in areas of concern, design drawings of structures, and field measurements of the structures.

Flows used to evaluate the drainage structures were calculated utilizing hydrologic methods explained previously in this report. Future condition flows were used. Methods used to evaluate each drainage structure are in conformance with the new City/County criteria manual.

Existing Conditions

We have divided the storm sewer system into four distinct reaches for the purpose of this report. Each reach is distinctive due to location, storm sewer deficiencies, and possible drainage solutions. Reaches are shown on Exhibit III.

Reach A is the storm sewer system located in the Starwatch Subdivisions. All of these facilities are presently private systems maintained by the Starwatch Homeowners Association. The system consists of a series of grated inlets which direct runoff to a 24" RCP. The 24" RCP discharges flows into a small detention pond. The pond outlet flow is carried by a 36" RCP which also carries runoff collected by two grated inlets within the Starwatch Subdivisions and by two 6' foot curb inlets located in a sump condition on Farmingdale Drive just east of Austin Bluffs Parkway. The two sump inlets are public facilities.

√The second reach, Reach B, is the storm sewer system located between Farmingdale Drive and Old Farm Drive. The present system consists of a single 36" RCP running parallel to Austin Bluffs Parkway. One 30" CMP lateral exists at a point about one-third of the distance from Old Farm Drive to Farmingdale Drive. This lateral allows runoff from Basin A Subbasin 1 to cross Austin Bluffs Parkway into the study basin. Two inlets located at the intersection of Austin Bluffs Parkway and Old Farm Drive direct runoff into the 36" RCP.

Reach C is the storm sewer network in Old Farm Drive, Old Farm Circle West, and Farmingdale Drive. The network consists of six curb inlets and a main storm sewer line which is a 30" RCP. A 12" drain line from the 2 MG Templeton Gap water storage tank at the top of the basin is connected into the 10' curb inlet located at the southeast corner of Farmingdale Drive and Old Farm Circle West. Runoff is collected into the series of inlets and then transported by the 30" RCP to the junction of the 36" RCP in Reach B. This confluence occurs at the southeast corner of the intersection of Austin Bluffs Parkway and Old Farm Drive.

At this confluence point, Reach D begins. This reach extends to the outfall point of the basin. The outfall is located at the confluence of two concrete-lined channels located east of Turquoise Subdivision and just southwest of Old Farm Subdivision Filing No. 1. The reach presently consists of a 36" RCP which discharges flows into the old "natural" channel. From this unimproved channel, the water is directed into a 42" RCP located behind Old Farm Subdivision Filing No. 1 and Old Farm Business Park. The 42" RCP then discharges flows into a four foot deep concrete-lined channel. This channel then carries the runoff to the outfall point.

Inventory of Structures

All significant drainage structures within the basin and adjacent to the basin, were reviewed as a part of this study. This was done to help establish the existing operation of each structure and its ability to convey future peak flows. This information was used in the development of alternative drainage schemes for the basin. A listing of the stuctures is shown on Exhibit IV.

EVALUATION OF DRAINAGE ALTERNATIVES

General

The following is a description of the analysis used in the formulation of the recommended alternative. The discussion is based around the four reaches described in the previous section.

Reach A

Reach A is presently undersized from, and including, the detention pond downstream. The detention pond storage capacity is presently about 0.35 acre-feet. The original plans called for a pond with a storage capacity of over 1.5 acre-feet. With the pond being approximately a quarter of the proposed size, it acts more as a mechanism to provide head to the inlet of the 36" RCP than as a detention pond. Because the pond is not functioning as shown originally in the Starwatch Subdivision Drainage Report, the 36" RCP is inadequate to convey flows through the subdivision. During larger storm events the possibility of overtopping the road is very possible. If this was to occur, flooding would be a certain problem at the cul-de-sac end of Twinkle Way. At this point there is a drainage easement for the 36" RCP as it runs between Twinkle Way and Farmingdale Drive. According to the drainage plan there was to be a small swale which would direct overflow water from Twinkle Way between the houses to Farmingdale Drive. This overflow swale does not appear to exist and the lack of a swale could potentially pose as a flooding danger to the adjacent houses.

Three alternatives for this area were analyzed. Alternative 1 is to improve the existing detention pond so that the downstream facilities will function properly. Alternative No. 2 is to join the 24" RCP with the 36" RCP and then extend the 36" RCP upstream through the existing channel to the north end of Starwatch Subdivision. The third alternative is to install a storm sewer on the east side of Austin Bluffs Parkway adjacent to Starwatch Subdivision.

In order to expand the existing detention pond so that the storage volume of the pond is nearer to the designed size, two undeveloped lots would need to be acquired. After obtaining these lots, extensive regrading of the pond would be needed to provide extra storage capacity. A rough grading of the pond, with the assumption that it could include the two lots, was performed. A steep side slope of 2:1 (Horizontal:Vertical) was used to provide extra storage capacity. The volume of this potential pond was approximately one acre-foot. In order to achieve more storage, the pond and associated piping would need to be deepened.

Using the regraded pond, a routing of the 100 year, 24 hour storm was performed through the reservoir. The results indicate that runoff would fill the pond to capacity. The water surface elevation comes to within a fraction of a foot of overtopping the roadway dam. Due to the necessary acquisition of two adjacent lots and the distinct possibility of overtopping, especially when the outlet becomes partially clogged, we feel that this is not a viable alternative to alleviate the drainage concerns in the area.

The extension of the 36" RCP to the north edge of Starwatch Subdivision, will only serve to move the drainage problems upstream. The existing pipe needs significant head water to create sufficient capacity in the pipe. This can be solved by reducing the flow to the inlet or enlarging the pipe and inlet. If the pipe was enlarged, it would be necessary to replace most, if not all of the pipe in Reaches A, B, and D, to avoid surcharging of manholes and/or inlets.

Extension of the 36" RCP will only relocate the existing problem upstream; therefore, we do not recommend this alternative for Reach A.

Installing a storm sewer on the east side of Austin Bluffs Parkway will serve to pick up two sources of runoff. The first source of runoff will be the water that is in the eastern curb and gutter of Austin Bluffs Parkway. The second source of runoff will be that which is generated from Templeton Heights Subdivision. A 24" RCP would tie-in to the existing 36" RCP at the intersection of Farmingdale Drive and Austin Bluffs Parkway using a manhole or junction box structure. The pipe would then be extended northeastward in the Austin Bluffs Parkway Right-of-Way. The pipe would be extended past the Starwatch Subdivision and a temporary flared-end section installed.

The Templeton Heights Subdivision internal drainage system could then be connected to two outfall systems. Most of the storm water would be directed to the existing channel upstream of the detention pond, with a portion of the runoff directed toward the 24" RCP in Austin Bluffs Parkway. By directing a portion of this water away from the detention pond and Starwatch Subdivision, the existing drainage system in this area will not receive as much runoff and thus will function within its capacity. The runoff volume from the 10 year storm that is directed to the 24" RCP can enter the pipe under a head condition equal to the top of the pipe. In other words, no ponding above the top of the pipe would occur during the initial storm. This flow rate is approximately 13 cfs. In the major storm condition, ponding would be allowed. It is calculated that approximately 23 cfs will enter the 24" RCP under these conditions.

In order to allow the 24" RCP to carry more water and also to alleviate ponding problems at the sump inlets located just east of Austin Bluffs Parkway in Farmingdale Drive, an inlet can be installed in Austin Bluffs Parkway adjacent to Starwatch Subdivision. This inlet will be a 10' D-10R inlet with a capacity of 13.98 cfs and 10.97 cfs for the 100 year and 10 year storms respectively. By removing some of the runoff water from Austin Bluffs Parkway, two purposes are served. First, less water is routed to the sump inlets in Farmingdale Drive, and more water-free driving room is provided on Austin Bluffs Parkway. The width of flow in Austin Bluffs Parkway is about 17 feet for the 10 year storm and about 20 feet for the 100 year storm. By removing some water from the roadway more driving room is provided and the City's criteria concerning driving width during storm events will be satisfied.

Reach B

Reach B is undersized in its entire length. Presently a 36" RCP transports water from the sump inlets in Farmingdale Drive to the junction of the 30" RCP near the intersection of Austin Bluffs Parkway and Old Farm Drive. The flows at the upstream portion of this reach are approximately 160 cfs and 95 cfs for the 24 hour, 100 year and 10 year storms respectively. The flows at the downstream end of the reach are approximately 235 cfs and 145 cfs for the 24 hour, 100 year and 10 year storms respectively. The capacity of the existing 36" RCP based upon Mannings Equation is about 105 cfs. The pipe is about 40 cfs under capacity for the 10 year storm event. This does not include account storm water entering the basin through the existing 30" CMP crossing Austin Bluffs Parkway north of Old Farm Drive.

Two alternatives were studied for this reach. Both alternatives suggest installation of a relief storm sewer between Old Farm Drive and Farmingdale Drive. The first alternative would not allow storm water from Subbasin 1 to cross Austin Bluffs Parkway. The second alternative would allow runoff from Subbasin 1 to cross Austin Bluffs Parkway into Subbasin 2.

In order to obtain non-pressure flow conditions, a relief sewer is required in Reach B. This relief sewer could either be a separate storm sewer line or a a replacement of the existing pipe with a larger line. The cost of up-sizing the existing line would be greater than providing a parallel line due to the increased pipe size and work involved in removing the existing line. Therefore, we have looked at a system that would incorporate a parallel relief sewer.

The relief sewer, a 36" RCP, is routed around the proposed shopping center as shown on the approved development plan for Old Farm Center. The 36" RCP was selected to provide continuity of pipe size throughout the system. Consideration was given to connecting the proposed 24" RCP from Reach A with a 24" relief sewer through Reach B. However, it was felt that the cost savings provided by using a smaller pipe size would be partially lost in the design and construction of this system. Another potential problem with this connection is the space required. Crossing the 24" RCP and 36" RCP will require as much as eight feet of clear passage. The benefits received by using a 36" RCP through Reach B would out weigh the cost. Additional flow can be carried by the 36" RCP thus minimizing problems associated with excess flow at a point downstream.

The real decision to be made in Reach B actually concerns Reach D. The decision must be made as to whether storm water from Subbasin 1 will be allowed to cross Austin Bluffs Parkway into Subbasin 2. The reason this decision concerns Reach D is because of the capacity associated with the concrete-lined channel in the lower section of Reach D. The capacity of this channel is approximately 600 cfs. The ultimate 100 year flow from the basin is about 420 cfs. This is a difference of about 180 cfs.

If Reach D is rebuilt to accommodate the 600 cfs capacity of the downstream channel, then it makes sense to allow storm water from Subbasin 1 into Subbasin 2. If this capacity is not carried upstream, then runoff should not be allowed to cross basin lines.

√ The optimum location for runoff from Subbasin 1 to cross Austin Bluffs Parkway is near the intersection of Old Farm Drive and Austin Bluffs Parkway. A 48" diameter RCP would be installed diagonally across the intersection to allow storm water into Subbasin 2. The existing 30" CMP crossing Austin Bluffs Parkway is not in an efficient location for collection of storm water runoff. The pipe could remain in place, although it does not provide any real benefit to the storm sewer system.

Reach C

If Reaches A, B, and D are considered the main storm sewer line in this basin, then Reach C is the eastern branch. This reach is also undersized from approximately Old Farm Circle West to Austin Bluffs Parkway. The inlets on this branch are not sufficient to provide the necessary flow to the main storm sewer. This problem is accentuated in this reach because of the lack of sump inlets. The only sump inlets on the system are located on the upstream end.

Runoff that is in Old Farm Circle West at Old Farm Drive could possibly leave the basin and continue downstream and eventually discharge to the concrete channel system from Harvest Road. If the runoff is not picked up by the two inlets located at the intersection of Old Farm Drive and Old Farm Circle West, then some water will continue to the southwest. Storm water that is in the western curb and gutter will tend to turn the corner at Old Farm Drive and continue in Old Farm Drive to the inlets located at Austin Bluffs Parkway. However, a portion of the runoff will not be able to turn the corner due to its momentum. The portion of water that does not turn the corner, could potentially cause flooding problems downstream. This water will eventually be directed to the outfall channel by way of two small channels located between houses on Harvest Road. The more storm water that stays in Subbasin 2, the less impact there is on the channels off of Harvest Road.

Three alternatives were analyzed for this reach. The first alternative would be to remove and replace the existing 30" RCP in Old Farm Drive with a 36" RCP. The second alternative would be to provide a relief sewer along the rear boundary of the undeveloped parcel adjacent to Old Farm Drive. The third alternative is a combination of the previous two. It would include a relief sewer like the second alternative, but would parallel the existing 30" RCP in Old Farm Drive.

For all three alternatives, a better collection system needs to be built. In analyzing the flow patterns in Old Farm Circle West, it appears that most of the flow is carried in the western curb and gutter. The placement of a 10' D-10R inlet approximately 100 feet upstream of the existing inlet on the western side of Old Farm Drive at Old Farm Circle West, will allow for the collection of considerably more runoff. By adding this inlet another 15.51 cfs will be collected and placed into the system during the 100 year storm. An inlet on the eastern side of the road is not required.

The first alternative would include the removal and replacement of approximately 1,495 feet of 30" RCP with 36" and 48" RCP. Asphalt patching and replacement would involve about 1,600 square yards of area. The second alternative is the provision of a relief sewer through the undeveloped land adjacent to Old Farm Drive. This would include the removal and replacement of a short section of 30" RCP near the intersection of Old Farm Drive and Old Farm Circle West. A junction box would be constructed which would split the flow. Some flow would continue in the existing 30" RCP and the remaining flow would outfall through a new 36" RCP constructed through the undeveloped parcel. Approximately 1,340 feet of 36" RCP along with 80 feet of 48" RCP would be needed to construct this alternative. Asphalt removal and replacement would be reduced to about 910 square yards.

The third alternative would involve the removal and replacement of the same section of pipe near the intersection of Old Farm Drive and Old Farm Circle West as indicated in the second alternative. Also, the same type of flow splitting device would be used to split flows into the existing 30" RCP and the new 36" RCP. This new 36" RCP would be constructed adjacent and parallel to Old Farm Drive in the Right-of-Way and/or a drainage easement. This alternative would include approximately 1,235 feet of 36" RCP, 50 feet of 48" RCP, and 910 square yards of asphalt removal and replacement.

To compare the alternatives for Reach C cost estimates were prepared. The cost estimates included only the items that were different for each alternative. From these estimates, the third alternative was the least expensive solution.

Reach D

In Reach D the storm water is collected and channeled to the outfall of the subbasin. Presently this reach consists of section of 36" RCP which discharges into a section of the old "natural" channel. From this channel the runoff enters a 42" RCP located between Old Farm Subdivision Filing No. 1 and Old Farm Business Center. The 42" RCP then discharges into the concrete-lined channel located at the downstream end of the subbasin. This channel then outfalls the subbasin's runoff into the main concrete-lined channel upstream of Oro Blanco Drive.

Three alternatives were also analyzed for this reach. The first alternative was continuing the existing concrete-lined channel section upstream to approximately the intersection of Old Farm Drive and Austin Bluffs Parkway. The second alternative was to use a 72" RCP from the concrete-lined channel to the intersection of Old Farm Drive and Austin Bluffs Parkway. Both of these alternatives would be used to convey approximately the capacity of the existing concrete-lined channel. The third alternative would be the same as the second except that a 60" RCP would be used instead of a 72" RCP.

From the alternatives listed it is obvious that this reach is inadequate. Both the 36" and 42" RCPs in the reach are constraints to the system. In order to convey the storm runoff from Subbasin 2 only, a 60" RCP is needed. A 72" RCP or equivalent concrete-lined channel is required if the full capacity of the existing concrete-lined channel is to be utilized.

The existing 42" and 36" RCP will be removed and replaced and new storm sewer will be added through the "natural" channel section. The total length of this section is approximately 1,130 feet.

In order to compare these three alternatives, cost estimates were developed. The difference between the first two alternatives was almost fifty percent. The concrete-lined channel alternative was considerably less expensive than the 72" RCP alternative. The difference between using a concrete-lined channel and using the 60" RCP alternative is about 15 percent, with the 60" RCP being less expensive.

The decision that must be made is whether to spend the extra 15 percent to allow the existing channel to operate at or near its capacity during a 100 year storm event. If the channel was extended, then a portion of the flow from Subbasin 1 would be allowed to enter Subbasin 2 and discharge through the existing concrete-lined channel.

We feel that the extra expenditure needed to construct the concrete-lined channel versus constructing the 60" RCP is far outweighed by the benefits. The existing channel could then be used at or near its physical capacity, with due consideration of freeboard requirements. By allowing runoff from Subbasin 1 to discharge to this channel, less drainage will concentrate at the intersection of Austin Bluffs Parkway and Oro Blanco Drive. This will mean that a smaller structure can be constructed across this intersection than would be needed if all of the water from Subbasin 1 was directed to this point.

Basis of Design

Basic design criteria contained in the City of Colorado Springs / El Paso County Drainage Criteria Manual were used during the design of drainage alternatives. Major facilities were designed to convey the 100-year discharge. If a structure is not able to handle the 100-year flow, provisions were made for containing the flow considered. The alternatives evaluated in this study all convey the 100-year storm without significant property damage.

Alternative Evaluation

The evaluation process for the alternatives outlined included a recommendation of a drainage scheme for the basin. Input from the City of Colorado Springs was included, along with input from El Paso County concerning areas where the basin impacted their future plans. Using this input and review, a recommended alternative for the basin was established.

RECOMMENDED ALTERNATIVE

After analyzing alternatives for each reach separately, and then considering the interaction that each alternative has on the other reaches, one alternative for the entire subbasin came to the forefront. This is the alternative which has been reviewed and agreed upon as the wisest course of action for this Subbasin. The recommended alternative is shown graphically on Exhibit I.

From the lowest portion of the basin, the recommended alternative includes extending the existing concrete-lined channel to approximately Austin Bluffs Parkway and Old Farm Drive. At this point, three pipes will discharge into the channel. The existing 36" RCP will discharge along with a 48" RCP transporting runoff from Subbasin 1. Another 48" RCP, which combines Reaches B and C discharges at this same point.

In Reach C, a 36" RCP will be installed parallel to the existing 30" RCP in Old Farm Drive. A 36" RCP will also replace the existing storm sewer in Old Farm Circle West up to approximately the intersection with Farmingdale Drive. In addition a new 10' D-10R will be installed approximately 100 feet upstream of the existing inlet located on the northwest corner of Old Farm Drive and Old Farm Circle West. The existing 30" RCP should be replaced with a 36" RCP to provide more efficient flow through this section of pipe. At the Northeast intersection of Old Farm Circle East and Farmindale Drive, the 10' inlet should be altered (lowered) so that flow enters the inlet in a more efficient manner.

Reach B will include the 36" RCP relief sewer through Old Farm Center. In addition, the previously mentioned 48" RCP will be installed to transport runoff from Subbasin 1. On the upstream end of the new 36" RCP in Farmingdale Drive, the sump inlet located on the north side of the road will remain connected to the 36" RCP. The inlet on the south side of the road will remain connected to the existing 36" RCP and the section of 36" RCP between this inlet and the new 36" RCP will be abandoned.

At the intersection of Austin Bluffs Parkway and Farmingdale Drive a new manhole will be installed which will connect a new 24" RCP to the existing 36" RCP. The 24" RCP will parallel Austin Bluffs Parkway to the north end of Starwatch Subdivision. A new 10' D-10R will be installed in Austin Bluffs Parkway to pick up some of the curb flow.

The enclosed plan shows the location and general relationship of the recommended alternative to the existing storm sewer system.

In addition to providing the storm sewer components mentioned above, other items should be considered. The sump inlet located at the northeast corner of the intersection of Old Farm Circle West and Farmingdale Drive is a critical inlet in this system. Work could be accomplished to improve the efficiency and thus the capacity of this inlet. A very significant portion of the upstream runoff is directed to this inlet. The more runoff that can be collected by this inlet, the less storm water that has the potential of bypassing the downstream inlets and crossing basin boundaries. Attention to this inlet could alleviate downstream street drainage problems.

The sump inlets located at the intersection of Austin Bluffs Parkway and Old Farm Drive could be improved in efficiency should these streets be improved. The main consideration at this intersection is to improve the efficiency of the inlet located on the southwest corner.

Prior to the development of the vacant parcel of land between Farmingdale Drive and Old Farm Drive (Formerly Old Farm Center) a revised and updated drainage report and plan will be required for this Site. The update must incorporate the proposed 36 inch relief sewer as well as overflow provisions for potential clogging of the sump inlets in Farmingdale Drive. Berming shall be utilized along Farmingdale Drive such that an overflow elevation is established equal to the design depth of ponding (1.26 Ft) for referenced inlets No. 8 and 9.

ECONOMIC ANALYSIS

General

During the analysis of alternatives for this basin, cost estimates were devloped for each alternative. This information was used for comparative purposes only during the evaluation and analysis of alternatives. Detailed information on these estimates are included with the *Preliminary Design Report North Shook's Run, Templeton Gap Basin A, Sub-basin 2.*

Cost of Improvements

Construction costs for the recommended alternative was based on 1989 prices and includes engineering, contingencies, legal, and administrative costs. Land acquisition costs, if any, have not been included in this economic analysis. Table 1 outlines the costs for the recommended alternative.

TABLE 1
ESTIMATE OF COSTS
for
North Shook's Run, Templeton Gap Basin A, Subbasin 2

Item	*Quantity*	*Unit*	*Unit Price*	*Amount*
24" RCP	880	lf	$33.00	$29,040.00
36" RCP	3,170	lf	$50.00	$158,500.00
48" RCP	130	lf	$77.00	$10,010.00
24" Flared End Section	1	ea	$500.00	$500.00
24"--45 Degree Bend	2	ea	$400.00	$800.00
36"--45 Degree Bend	4	ea	$440.00	$1,760.00
Concrete Lined Channel	1,130	lf	$160.00	$180,800.00
60" Manhole	3	ea	$2,100.00	$6,300.00
72" Manhole	1	ea	$2,400.00	$2,400.00
Junction Box	1	ea	$2,750.00	$2,750.00
10' D-10R Curb Inlet	2	ea	$3,150.00	$6,300.00
Revegetation	1.2	ac	$3,500.00	$4,200.00
Asphalt Patching	985	sy	$24.00	$23,640.00
Fence Removal and Reconstruction	690	lf	$25.00	$17,250.00
Miscellaneous Drainage Upgrades	1	ls	$5,000.00	$5,000.00
Construction Subtotal				**$449,250.00**
Contingency @ 10%				$44,925.00
Construction Total				**$494,175.00**
Administration, Engineering, Legal				$49,417.50
TOTAL				**$543,592.50**

TABLE 2
COST BREAKDOWN
for
North Shook's Run, Templeton Gap Basin A, Subbasin 2

	Capital Improvement (Reimbursible)		City (Reimbursible)		Developer	
Item	*Quantity*	*Amount*	*Quantity*	*Amount*	*Quantity*	*Amount*
24" RCP	---	---	---	---	880	$29,040.00
36" RCP	2,170	$108,500.00	---	---	1,000	$50,000.00
48" RCP	50	$3,850.00	80	$6,160.00	---	---
24" Flared End	---	---	---	---	1	$500.00
24"--45 Deg Bend	1	$400.00	---	---	1	$400.00
36"--45 Deg Bend	2	$880.00	---	---	2	$880.00
Concrete Channel	---	---	1,130	$180,800.00	---	---
60" Manhole	---	---	---	---	3	$6,300.00
72" Manhole	---	---	---	---	1	$2,400.00
Junction Box	1	$2,750.00	---	---	---	---
10' D-10R Inlet	1	$3,150.00	---	---	1	$3,150.00
Revegetation	0.4	$1,400.00	0.3	$1,050.00	0.5	$1,750.00
Asphalt Patch	910	$21,840.00	---	---	75	$1,800.00
Fence	---	---	---	---	690	$17,250.00
Misc Upgrades	1	$5,000.00	---	---	---	---
Construction Subtotal		**$147,770.00**		**$188,010.00**		**$113,470.00**
Contingency @ 10%		$14,777.00		$18,801.00		$11,347.00
Construction Total		**$162,547.00**		**$206,811.00**		**$124,817.00**
Admin, Eng, Legal		$16,254.70		$20,681.10		$12,481.70
TOTAL		**$178,801.70**		**$227,492.10**		**$137,298.70**

TABLE 3
DRAINAGE FEES
For
Unplatted Land Within Subbasin 2

AREA NAME	ACRES
Templeton Heights	30.6
Old Farm (south of Old Farm Drive)	6.9
TOTAL	37.5

BASIN FEE: $2,822.00 per Acre

Total Drainage fees of unplatted areas: $105,825.00

6.6 Acres of platted land which is undeveloped (Old Farm Center); Basin fees have been paid.

Total Drainage fees of undeveloped areas: $124,450.20

POSSIBLE PHASING AND FUNDING MECHANISMS

The phasing of the recommended alternative is dictated by downstream capacity. Improvements on the downstream end of the basin should be completed first. The first phase of the basin improvements would be to construct the concrete-lined channel between Old Farm Subdivision Filing No. 1 and Old Farm Business Center. The construction would include tying into the existing channel and continuing upstream to approximately Austin Bluffs Parkway and Old Farm Drive. This phase could include the construction of the 48" RCP across Austin Bluffs Parkway or it could wait until the next phase. This would depend upon funding and drainage considerations at the intersection of Austin Bluffs Parkway and Oro Blanco Drive. The conditions of the downstream facilities, particularly through the Park Vista Area, need to be considered prior to adding the 48" crossing to the system. It is possible that modifications to these downstream facilities need to be completed prior to completion of facilities in this Subbasin. This determination should be made on a case by case basis. This phase of construction would be completed using capital improvement funds. Assuming that the construction of the channel will be prior to development of the adjoining parcels. The City would then be reimbursed out of the basin fund for work completed through the undeveloped land portion of the phase.

The second phase would consist of adding an inlet on Old Farm Circle West upstream of the Old Farm Drive intersection. The storm sewer proposed for Old Farm Drive from Farmingdale Drive to the new channel would also be included in this phase. This phase would include 24", 36", and 48" RCP. The entire cost of this phase would be born by the City through capital improvement funds.

The next two phases would be triggered by development in the adjacent areas. The first of these two phases is the construction of the 36" RCP relief sewer through Old Farm Center. The abandonment of a section of the existing 36" RCP would also be included. Financing of this option would be by the developer assuming that the approved development plan is resubmitted. Our understanding is that resubmittal is a very real possibility for this parcel of land. If development occurred as shown on the existing development plan, the City would need to obtain the necessary easements and provide for the construction of the 36" RCP relief sewer.

The last phase is tied to development of Templeton Heights Subdivision. When this parcel is developed, the 24" RCP adjacent to Austin Bluffs Parkway along with the inlet could be constructed. Funding for this phase would be born by the Templeton Heights development. It can be argued that the Starwatch Subdivision has some responsibility for building this phase due to the lack of storage provided in the existing detention pond.

DESIGN AND CONSTRUCTION PROBLEMS

A problem area which needs to be pointed out is the Park Vista subdivision area. The area was designed without anticipation of the upstream drainage basin being developed in the urban manner in which it has been developed. Since its construction most of the tributary area has been developed. This area is approximately 1,440 acres which is over two square miles. All of this area drains through the Park Vista subdivision. The problem is specifically addressed in the *Engineering Study and Revision of the North Shook's Run -- Templeton Gap Drainage Basin* by Lincoln-DeVore in September 1977. The area was also addressed in the original Master Drainage Basin Study prepared by United Western Engineers in September 1963. Specific solutions to this problem are addressed in the Lincoln-DeVore report. The Lincoln-DeVore report governs over the two reports for the basin. This current report is a revision to a small sub-basin which contributes flow to the Park Vista area. The recommendations included within this report will convey runoff from Sub-basins 1 and 2 through the study area in a much safer manner. The effect of this will be to decrease the travel time of the runoff to the existing channel just downstream of Sub-basin 2. The runoff then travels approximately one mile before entering the Park Vista subdivision. The effect of the one mile of travel will be to attenuate the the peak flow generated by the decrease in travel time through Sub-basin 2. It should be noted that the recommendations do not increase the volume of flow to the Park Vista subdivision. The proposed improvements will have a positive impact on the drainage situation in Sub-basin 2 and a minimal impact upon the Park Vista subdivision, however, consideration should be given to the Park Vista situation during the phasing of some of the recommended improvements, so that additional adverse impacts are not realized by the Park Vista Subdivision.

There are a few possible problem areas associated with the design and construction of the recommended alternative. The first problem area is located along the suggested alignment for the concrete-lined channel. There is an existing sanitary sewer line in this area which needs to be considered during design and construction. Easements could also pose a problem. Easements exist in Old Farm Subdivision Filing No. 1 and Turquoise Subdivision. There are no drainage easements for Old Farm Business Park, however there is a statement on the plat that allows additional easements to be obtained at the building permit stage. The design of the channel will include some drop structures so that the property adjacent to the channel will be useable. Consideration needs to be given to the relationship of Austin Bluffs Parkway to the channel. A short stretch of pipe may be needed so as not to have a steep and deep channel embankment adjacent to the roadway.

Easements will need to be obtained from the owner of the undeveloped land south of Old Farm Drive. An easement will be needed for the construction of the 24" relief sewer parallel to Old Farm Drive, and possibly for the construction of the concrete-lined channel.

As stated earlier, it may be necessary to obtain an easement from Old Farm Center in order to place the 30" RCP relief sewer through that parcel. The design of this relief sewer would need to take into account the proposed development plans for Old Farm Center.

The channel along Austin Bluffs Parkway will have to be raised with the use of drop structures to avoid excessive depth in the channel. It may be necessary to extend a large RCP for a short distance to eliminate the problem of a steep embankment adjacent to Austin Bluffs Parkway.

Another design and construction problem is located at the intersection of Austin Bluffs Parkway and Farmingdale Drive. The area also includes the sump inlets located on Farmingdale Drive. There are various utilities located in this vicinity and careful consideration must be given to placement of the new storm sewer in this area.

Hydrologic Calculations

SUMMARY TABLE

UNITS

Flow: cubic feet per second
Time: minutes
Volume: acre-feet
Area: acres
Stage: feet
Pipe size: inches

Hyd. No.	Brch No.	Peak Flow	Time of Peak	Volume	Basin Area	Maximum Stage	Pipe size
1	1	1.9	350.00	0.10	0.38		
2	2	29.7	350.00	1.58	6.90		
3	2	17.5	350.00	0.79	6.90		
4	2	52.6	350.00	2.64	14.88		
5	3	6.2	350.00	0.33	2.21		
6	3	11.0	350.00	0.60	3.94		24
7	3	27.1	350.00	1.40	9.83		24
8	3	72.0	350.00	3.81	27.65		36
9	3	79.0	350.00	4.18	29.99		36
10	3	100.4	350.00	5.31	37.16		
11	2	153.0	350.00	7.95	52.04		36
12	3	3.9	350.00	0.19	0.78		18
13	3	4.8	350.00	0.24	0.96		
14	2	157.8	350.00	8.19	53.00		36
15	3	20.8	350.00	1.02	4.32		
16	2	178.6	350.00	9.21	57.32		
17	4	4.8	350.00	0.24	1.32		18
18	4	29.7	350.00	1.45	10.16		
19	4	38.0	350.00	1.86	12.91		
20	4	51.7	350.00	2.55	17.38		24
21	4	43.3	350.00	4.37	18.01		24
22	4	129.5	350.00	8.98	46.35		30
23	4	90.1	350.00	8.81	47.24		30
24	4	90.2	350.00	20.64	48.14		30
25	4	81.0	350.00	20.04	49.31		
26	5	49.6	350.00	2.65	10.89		
27	2	309.2	350.00	31.90	117.52		
28	2	176.4	280.00	43.02	117.52		36
29	2	176.4	280.00	43.02	117.52		
30	3	41.8	350.00	2.18	8.92		
31	2	218.2	350.00	45.20	126.44		
32	2	167.8	280.00	41.24	126.44		42
33	2	176.1	350.00	41.66	128.70		

SUMMARY TABLE

UNITS

Flow: cubic feet per second
Time: minutes
Volume: acre-feet
Area: acres
Stage: feet
Pipe size: inches

Hyd. No.	Brch No.	Peak Flow	Time of Peak	Volume	Basin Area	Maximum Stage	Pipe size
1	1	1.4	350.00	0.07	0.38		
2	2	19.1	350.00	1.01	6.90		
3	2	8.8	350.00	0.43	6.90		
4	2	32.5	350.00	1.62	14.88		
5	3	3.7	350.00	0.20	2.21		
6	3	6.5	350.00	0.36	3.94		24
7	3	15.6	350.00	0.83	9.83		24
8	3	41.0	350.00	2.23	27.65		36
9	3	45.2	350.00	2.45	29.99		36
10	3	58.5	350.00	3.14	37.16		
11	2	90.9	350.00	4.76	52.04		36
12	3	2.7	350.00	0.13	0.78		18
13	3	3.3	350.00	0.16	0.96		
14	2	94.2	350.00	4.93	53.00		36
15	3	13.8	350.00	0.66	4.32		
16	2	108.0	350.00	5.59	57.32		
17	4	3.0	350.00	0.15	1.32		18
18	4	17.0	350.00	0.85	10.16		
19	4	21.9	350.00	1.10	12.91		
20	4	30.0	350.00	1.52	17.38		24
21	4	32.2	350.00	1.63	18.01		24
22	4	84.7	350.00	4.47	46.35		30
23	4	86.3	350.00	4.55	47.24		30
24	4	87.9	350.00	4.64	48.14		30
25	4	79.3	350.00	4.74	49.31		
26	5	33.7	350.00	1.78	10.89		
27	2	221.1	350.00	12.11	117.52		
28	2	176.4	350.00	12.86	117.52		36
29	2	176.4	350.00	12.86	117.52		
30	3	28.4	350.00	1.47	8.92		
31	2	204.9	350.00	14.33	126.44		
32	2	167.8	350.00	13.90	126.44		42
33	2	172.9	350.00	14.15	128.70		

Hydrology
2 Hour – 100 Year Storm

SUMMARY TABLE

UNITS

Flow: cubic feet per second
Time: minutes
Volume: acre-feet
Area: acres
Stage: feet
Pipe size: inches

Hyd. No.	Brch No.	Peak Flow	Time of Peak	Volume	Basin Area	Maximum Stage	Pipe size
1	1	2.0	40.00	0.09	0.38		
2	2	27.7	50.00	1.60	6.90		
3	2	16.3	50.00	0.78	6.90		
4	2	47.3	40.00	2.64	14.88		
5	3	6.8	50.00	0.40	2.21		
6	3	12.1	50.00	0.72	3.94		24
7	3	29.8	50.00	1.76	9.83		24
8	3	82.5	50.00	4.90	27.65		36
9	3	85.9	50.00	5.29	29.99		36
10	3	101.1	50.00	6.63	37.16		
11	2	147.1	40.00	9.27	52.04		36
12	3	4.0	40.00	0.18	0.78		18
13	3	4.9	40.00	0.22	0.96		
14	2	152.0	40.00	9.49	53.00		36
15	3	20.0	40.00	0.99	4.32		
16	2	172.0	40.00	10.48	57.32		
17	4	5.2	40.00	0.26	1.32		18
18	4	34.1	40.00	1.84	10.16		
19	4	43.0	40.00	2.34	12.91		
20	4	58.0	40.00	3.17	17.38		24
21	4	43.4	40.00	6.33	18.01		24
22	4	132.2	50.00	11.66	46.35		30
23	4	90.2	40.00	11.62	47.24		30
24	4	90.5	40.00	12.62	48.14		30
25	4	80.4	40.00	12.44	49.31		
26	5	44.1	40.00	2.44	10.89		
27	2	296.4	40.00	25.36	117.52		
28	2	176.4	40.00	29.00	117.52		36
29	2	176.4	40.00	29.00	117.52		
30	3	39.9	40.00	2.01	8.92		
31	2	216.4	40.00	31.01	126.44		
32	2	167.8	40.00	31.14	126.44		42
33	2	175.5	50.00	31.59	128.70		

SUMMARY TABLE

UNITS

Flow:	cubic feet per second
Time:	minutes
Volume:	acre-feet
Area:	acres
Stage:	feet
Pipe size:	inches

Hyd. No.	Brch No.	Peak Flow	Time of Peak	Volume	Basin Area	Maximum Stage	Pipe size
1	1	1.3	40.00	0.06	0.38		
2	2	17.1	40.00	1.03	6.90		
3	2	7.8	40.00	0.41	6.90		
4	2	30.5	40.00	1.60	14.88		
5	3	4.0	40.00	0.24	2.21		
6	3	7.1	40.00	0.43	3.94		24
7	3	18.5	40.00	1.04	9.83		24
8	3	48.3	40.00	2.86	27.65		36
9	3	52.8	40.00	3.12	29.99		36
10	3	67.7	40.00	3.92	37.16		
11	2	98.2	40.00	5.52	52.04		36
12	3	2.6	40.00	0.11	0.78		18
13	3	3.2	40.00	0.14	0.96		
14	2	101.5	40.00	5.66	53.00		36
15	3	13.5	40.00	0.64	4.32		
16	2	114.9	40.00	6.29	57.32		
17	4	3.4	40.00	0.16	1.32		18
18	4	21.7	40.00	1.08	10.16		
19	4	27.4	40.00	1.38	12.91		
20	4	37.1	40.00	1.87	17.38		24
21	4	39.2	40.00	1.96	18.01		24
22	4	94.5	40.00	5.16	46.35		30
23	4	89.2	40.00	5.19	47.24		30
24	4	89.4	40.00	13.12	48.14		30
25	4	78.9	40.00	13.14	49.31		
26	5	30.3	40.00	1.56	10.89		
27	2	224.2	40.00	20.99	117.52		
28	2	176.4	40.00	34.00	117.52		36
29	2	176.4	40.00	34.00	117.52		
30	3	26.9	40.00	1.28	8.92		
31	2	203.3	40.00	35.29	126.44		
32	2	167.8	30.00	34.98	126.44		42
33	2	172.9	40.00	35.25	128.70		

TIME OF CONCENTRATION CALCULATIONS

Design Point	C10	L (ft)	S (%)	Overland Tc (min)	G=Gutter Flow G	L (ft)	ΔH (ft)	V (ft/sec)	Tc (min)	C=Channel C	L (ft)	ΔH (ft)	V (ft/sec)	Tc (min)	P=Pipe P	L (ft)	ΔH (ft)	V (ft/sec)	Tc (min)	Total Tc (min)	Area (acres)
1					X	625	28	7.73	1.35											1.35	0.38
2	0.55	340	5	11.09	X	1175	58	8.11	2.41											13.50	6.90
3					X	3525	202	8.74	6.72											6.72	7.98
4	0.4	410	15.12	10.72	X	290	19	9.35	0.52	X	110	4	7.55	0.24						11.48	2.21
5	0.4	350	10	11.37						X	300	44	15.16	0.33						11.70	1.73
6	0.4	340	30	7.77	X	250	12	8.00	0.52	X	325	50	15.52	0.35						8.64	5.89
7	0.4	300	12	9.90						X	1250	146.1	13.53	1.54						11.44	17.82
8	0.4	220	9.09	9.30	X	675	32	7.95	1.41											10.72	2.34
9	0.4	70	4.28	6.75	X	1250	68	8.52	2.45											9.19	7.17
10					X	625	23	7.01	1.49											1.49	0.78
11	0.55	70	3	5.97	X	105	1	3.56	0.49											6.46	0.18
12	0.55	300	19	6.68	X	240	7	6.24	0.64	X	430	21	8.75	0.82						8.14	4.32
13	0.4	65	7	5.52	X	450	30	9.43	0.80											6.31	1.32
14	0.4	280	58	5.66	X	170	9	8.40	0.34	X	600	52	11.65	0.86						6.85	8.84
15	0.4	300	31	7.22	X	110	1	3.48	0.53	X	425	24	9.40	0.75						8.50	2.75
16	0.4	300	55	5.96	X	1075	78	9.84	1.82											7.78	4.47
17					X	860	17	5.13	2.79											2.79	0.63
18	0.4	300	31	7.22	X	1900	114	8.95	3.54	X	350	60	16.39	0.36						11.11	28.34
19	0.4	120	5	8.39	X	400	27	9.49	0.70											9.09	0.89
20	0.4	90	6	6.83	X	460	31	9.48	0.81											7.64	0.90
21	0.4	200	13	7.87	X	525	13	5.75	1.52	X	450	32	10.55	0.71						10.11	10.89
22	0.4	40	1	8.28	X	750	20	5.96	2.10											10.37	1.17
23	0.55	240	13	6.78						X	650	32	8.78	1.23						8.01	8.92
24	0.4	70	3	7.59	X	1700	57	6.69	4.24											11.83	3.52
25	0.4	70	3	7.59	X	950	29	6.38	2.48											10.08	4.66
26	0.4	60	1	10.14	X	975	25	5.85	2.78											12.92	5.11
27	0.4	50	1	9.26	X	850	24	6.14	2.31											11.56	1.75
28																					
29	0.4	140	7	8.10	X	350	9	26.52	0.22	X	600	12	5.60	1.79						10.10	2.26
30	0.4	250	10	9.61																9.61	0.59
31																					

Hydraulic Calculations

SUMP INLET CALCULATIONS

$Q_i = 1.7\ (L_i + 1.8\ W)\ (d_{max} + W/12)^{1.85}$

Where Q_i = Inlet Capacity in cfs
L_i = Length of inlet
W = 2 feet
d_{max} = Maximum ponding depth

Structure Number 8

$Q_i = 1.7\ (L_i + 1.8\ W)\ (d_{max} + W/12)^{1.85}$

$Q_i = 1.7\ (6 + 1.8\ (2))\ (1.26 + 2/12)^{1.85}$
$Q_i = 31.5$ cfs

Structure Number 9

$Q_i = 1.7\ (L_i + 1.8\ W)\ (d_{max} + W/12)^{1.85}$

$Q_i = 1.7\ (6 + 1.8\ (2))\ (1.26 + 2/12)^{1.85}$
$Q_i = 31.5$ cfs

Structure Number 10

$Q_i = 1.7\ (L_i + 1.8\ W)\ (d_{max} + W/12)^{1.85}$

$Q_i = 1.7\ (6 + 1.8\ (2))\ (1.59 + 2/12)^{1.85}$
$Q_i = 46.3$ cfs

Structure Number 16

$Q_i = 1.7\ (L_i + 1.8\ W)\ (d_{max} + W/12)^{1.85}$

$Q_i = 1.7\ (10 + 1.8\ (2))\ (1.0 + 2/12)^{1.85}$
$Q_i = 30.7$ cfs

CURB OPENING INLETS COMPUTATIONS

W = 2 feet

n = 0.016

Inlet No.	Cross slope (Sx) ft/ft	Slope (S) ft/ft	Q (cfs)	Q/S^1/2	T	Fw	FwT	L1 0.77	L2 0.462	L3 1.65	Qi/Q	Qi	Q2/Q 0.60	Li Qi<Q2 L1<L2	Q1>Q2 L1>L2	Use L1	Actual Q1	Q-Qi Qc	Additional Flow Ratio	Additional Inlet Flow Due to Compound Section	Actual Q1	Q-Qi Qc
100 Year 24 Hour Storm																						
2	0.02	0.044	29.70	141.59	19.48	2.89	56.23	43.30	26.00	92.79	1.00	29.70	17.82			10	12.18	17.52	0.04	1.19	13.37	16.33
4	0.02	0.0364	1.90	9.96	7.20	2.15	15.44	11.89	7.14	25.48	1.00	1.90	1.14			8	1.28	0.62	0.75	1.43	2.70	-0.80
11	0.02	0.028	178.80	1068.53	41.56	2.64	109.68	84.46	50.72	180.98	1.00	178.80	107.28			6	45.77	133.03	0.00	0.00	45.77	133.03
12	0.02	0.05	51.70	231.21	23.41	3.18	74.53	57.39	34.46	122.97	1.00	51.70	31.02			10	18.95	32.75	0.02	1.03	19.98	31.72
13	0.02	0.05	17.90	80.05	15.73	2.96	46.49	35.80	21.50	76.72	1.00	17.90	10.74			10	7.92	9.98	0.02	0.29	8.21	9.69
14	0.02	0.0333	86.10	471.83	30.59	2.73	83.40	64.22	38.57	137.60	1.00	86.10	51.66			10	30.17	55.93	0.01	0.86	31.03	55.07
15	0.02	0.0333	28.80	157.82	20.29	2.53	51.34	39.53	23.74	84.71	1.00	28.80	17.28			10	12.25	16.55	0.05	1.44	13.69	15.11
100 Year 2 Hour Storm																						
2	0.02	0.044	20.30	96.78	16.89	2.81	47.47	36.55	21.95	78.32	1.00	20.30	12.18			10	8.91	11.39	0.10	2.03	10.94	9.36
4	0.02	0.0364	2.00	10.48	7.34	2.16	15.81	12.18	7.31	26.09	1.00	2.00	1.20			8	1.31	0.69	0.75	1.50	2.81	-0.81
11	0.02	0.028	178.80	1068.53	41.56	2.64	109.68	84.46	50.72	180.98	1.00	178.80	107.28			6	45.77	133.03	0.00	0.00	45.77	133.03
12	0.02	0.05	51.70	231.21	23.41	3.18	74.53	57.39	34.46	122.97	1.00	51.70	31.02			10	18.95	32.75	0.02	1.03	19.98	31.72
13	0.02	0.05	17.90	80.05	15.73	2.96	46.49	35.80	21.50	76.72	1.00	17.90	10.74			10	7.92	9.98	0.02	0.29	8.21	9.69
14	0.02	0.0333	89.70	491.55	31.06	2.73	84.92	65.39	39.27	140.12	1.00	89.70	53.82			10	31.20	58.50	0.01	0.90	32.10	57.60
15	0.02	0.0333	28.80	157.82	20.29	2.53	51.34	39.53	23.74	84.71	1.00	28.80	17.28			10	12.25	16.55	0.05	1.44	13.69	15.11
10 Year 24 Hour Storm																						
2	0.02	0.044	19.10	91.06	16.50	2.80	46.20	35.57	21.36	76.22	1.00	19.10	11.46			10	8.48	10.62	0.10	1.91	10.39	8.71
4	0.02	0.0364	1.40	7.34	6.42	2.09	13.40	10.32	6.20	22.12	1.00	1.40	0.84			8	1.09	0.31	0.80	1.12	2.21	-0.81
11	0.02	0.028	85.10	508.57	31.46	2.51	79.05	60.87	36.56	130.43	1.00	85.10	51.06			6	24.83	60.27	0.01	0.85	25.68	59.42
12	0.02	0.05	24.00	107.33	17.55	3.02	52.99	40.81	24.51	87.44	1.00	24.00	14.40			10	10.08	13.92	0.02	0.48	10.56	13.44
13	0.02	0.05	6.00	26.83	10.44	2.73	28.46	21.91	13.16	46.95	1.00	6.00	3.60			10	3.23	2.77	0.30	1.80	5.03	0.97
14	0.02	0.0333	52.50	287.70	25.41	2.64	67.01	51.60	30.99	110.56	1.00	52.50	31.50			10	20.08	32.42	0.02	1.05	21.13	31.37
15	0.02	0.0333	11.90	65.21	14.56	2.38	34.62	26.66	16.01	57.12	1.00	11.90	7.14			10	5.93	5.97	0.15	1.79	7.71	4.19
10 Year 2 Hour Storm																						
2	0.02	0.044	11.60	55.30	13.69	2.70	36.96	28.46	17.09	60.99	1.00	11.60	6.96			10	5.63	5.97	0.16	1.86	7.48	4.12
4	0.02	0.0364	1.30	6.81	6.24	2.07	12.95	9.97	5.99	21.37	1.00	1.30	0.78			8	1.04	0.26	0.80	1.04	2.08	-0.78
11	0.02	0.028	85.10	508.57	31.46	2.51	79.05	60.87	36.56	130.43	1.00	85.10	51.06			6	24.83	60.27	0.01	0.85	25.68	59.42
12	0.02	0.05	24.00	107.33	17.55	3.02	52.99	40.81	24.51	87.44	1.00	24.00	14.40			10	10.08	13.92	0.20	4.80	14.88	9.12
13	0.02	0.05	6.00	26.83	10.44	2.73	28.46	21.91	13.16	46.95	1.00	6.00	3.60			10	3.23	2.77	0.30	1.80	5.03	0.97
14	0.02	0.0333	55.20	302.49	25.89	2.65	68.51	52.75	31.68	113.04	1.00	55.20	33.12			10	20.92	34.28	0.02	1.10	22.03	33.17
15	0.02	0.0333	11.90	65.21	14.56	2.38	34.62	26.66	16.01	57.12	1.00	11.90	7.14			10	5.93	5.97	0.15	1.79	7.71	4.19

CURB INLET COMPUTATIONS

W = 2 feet

n = 0.016

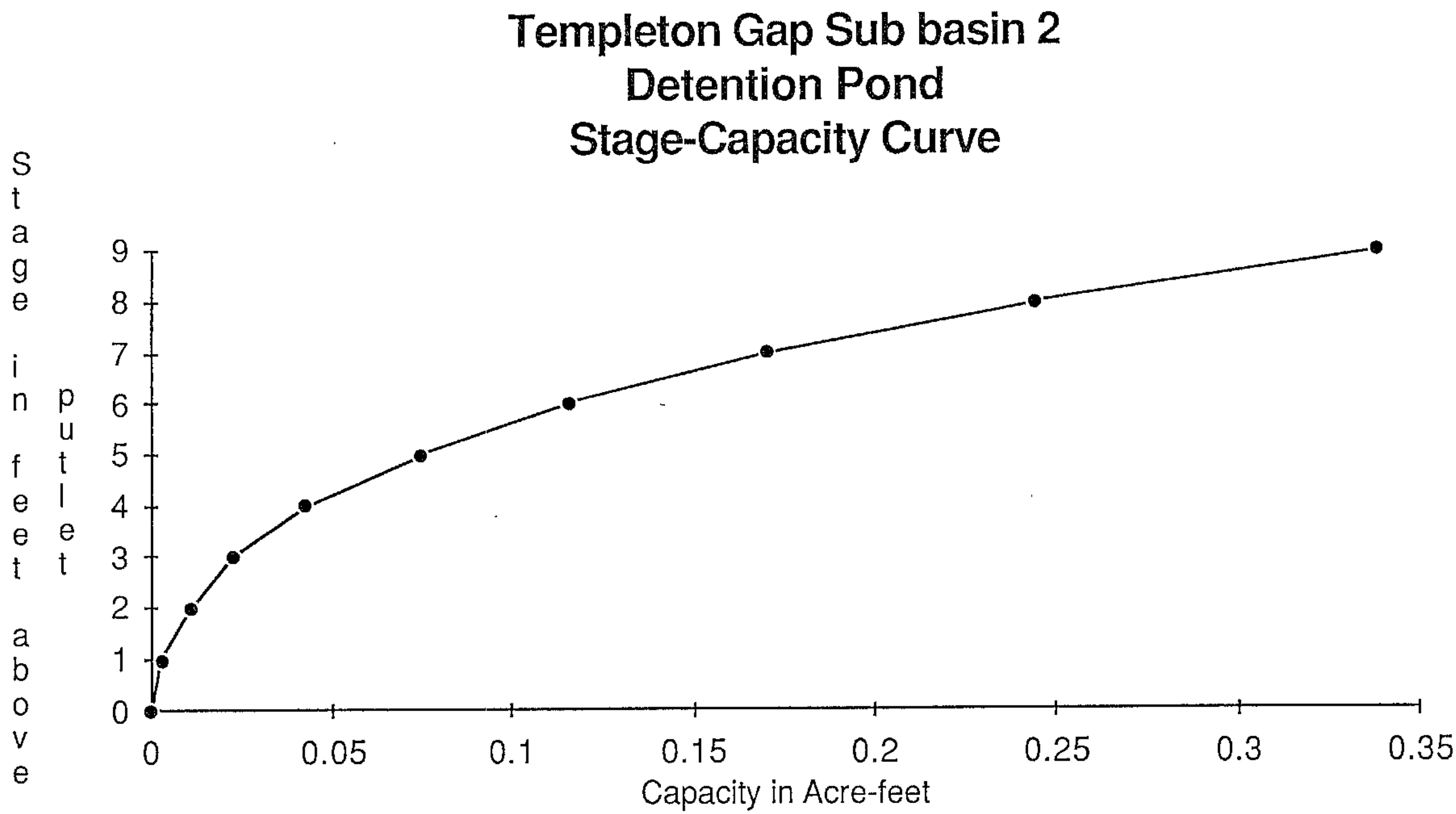
Inlet No.	Cross slope (Sx) ft/ft	Slope (S) ft/ft	Q (cfs)	Q/S^1/2	T	Fw	FwT	L1 0.77	L2 0.462	L3 1.65	Qi/Q	Qi	Q2/Q 0.60	Li Qi<Q2 L1<L2	Q1>Q2 L1>L2	Use L1	Actual Q1	Q-Qi Qc	Additional Flow Ratio	Additional Inlet Flow Due to Compound Section	Actual Q1	Q-Qi Qc
100 Year 24 Hour Storm																						
2	0.02	0.044	29.70	141.59	19.48	2.89	56.23	43.30	26.00	92.79	1.00	29.70	17.82			10	12.18	17.52	0.04	1.19	13.37	16.33
4	0.02	0.0364	1.90	9.96	7.20	2.15	15.44	11.89	7.14	25.48	1.00	1.90	1.14			8	1.28	0.62	0.75	1.43	2.70	-0.80
11	0.02	0.028	178.80	1068.53	41.56	2.64	109.68	84.46	50.72	180.98	1.00	178.80	107.28			6	45.77	133.03	0.00	0.00	45.77	133.03
11	0.02	0.028	178.80	1068.53	41.56	2.64	109.68	84.46	50.72	180.98	1.00	178.80	107.28			10	56.15	122.65	0.00	0.00	56.15	122.65
11	0.02	0.028	178.80	1068.53	41.56	2.64	109.68	84.46	50.72	180.98	1.00	178.80	107.28			15	66.03	112.77	0.00	0.00	66.03	112.77
12	0.02	0.05	51.70	231.21	23.41	3.18	74.53	57.39	34.46	122.97	1.00	51.70	31.02			5	14.36	37.34	0.02	1.03	15.39	36.31
12	0.02	0.05	51.70	231.21	23.41	3.18	74.53	57.39	34.46	122.97	1.00	51.70	31.02			10	18.95	32.75	0.02	1.03	19.98	31.72
12	0.02	0.05	51.70	231.21	23.41	3.18	74.53	57.39	34.46	122.97	1.00	51.70	31.02			15	22.28	29.42	0.02	1.03	23.32	28.38
13	0.02	0.05	17.90	80.05	15.73	2.96	46.49	35.80	21.50	76.72	1.00	17.90	10.74			5	6.00	11.90	0.02	0.29	6.29	11.61
13	0.02	0.05	17.90	80.05	15.73	2.96	46.49	35.80	21.50	76.72	1.00	17.90	10.74			10	7.92	9.98	0.02	0.29	8.21	9.69
13	0.02	0.05	17.90	80.05	15.73	2.96	46.49	35.80	21.50	76.72	1.00	17.90	10.74			15	9.32	8.58	0.02	0.29	9.60	8.30
14	0.02	0.0333	86.10	471.83	30.59	2.73	83.40	64.22	38.57	137.60	1.00	86.10	51.66			5	22.86	63.24	0.01	0.86	23.72	62.38
14	0.02	0.0333	86.10	471.83	30.59	2.73	83.40	64.22	38.57	137.60	1.00	86.10	51.66			10	30.17	55.93	0.01	0.86	31.03	55.07
14	0.02	0.0333	86.10	471.83	30.59	2.73	83.40	64.22	38.57	137.60	1.00	86.10	51.66			15	35.48	50.62	0.01	0.86	36.34	49.76
15	0.02	0.0333	28.80	157.82	20.29	2.53	51.34	39.53	23.74	84.71	1.00	28.80	17.28			5	9.29	19.51	0.05	1.44	10.73	18.07
15	0.02	0.0333	28.80	157.82	20.29	2.53	51.34	39.53	23.74	84.71	1.00	28.80	17.28			10	12.25	16.55	0.05	1.44	13.69	15.11
15	0.02	0.0333	28.80	157.82	20.29	2.53	51.34	39.53	23.74	84.71	1.00	28.80	17.28			15	14.41	14.39	0.05	1.44	15.85	12.95
10 Year 24 Hour Storm																						
2	0.02	0.044	19.10	91.06	16.50	2.80	46.20	35.57	21.36	76.22	1.00	19.10	11.46			10	8.48	10.62	0.10	1.91	10.39	8.71
4	0.02	0.0364	1.40	7.34	6.42	2.09	13.40	10.32	6.20	22.12	1.00	1.40	0.84			8	1.09	0.31	0.80	1.12	2.21	-0.81
11	0.02	0.028	85.10	508.57	31.46	2.51	79.05	60.87	36.56	130.43	1.00	85.10	51.06			6	24.83	60.27	0.01	0.85	25.68	59.42
11	0.02	0.028	85.10	508.57	31.46	2.51	79.05	60.87	36.56	130.43	1.00	85.10	51.06			10	30.46	54.64	0.01	0.85	31.31	53.79
11	0.02	0.028	85.10	508.57	31.46	2.51	79.05	60.87	36.56	130.43	1.00	85.10	51.06			15	35.83	49.27	0.01	0.85	36.68	48.42
12	0.02	0.05	24.00	107.33	17.55	3.02	52.99	40.81	24.51	87.44	1.00	24.00	14.40			5	7.64	16.36	0.02	0.48	8.12	15.88
12	0.02	0.05	24.00	107.33	17.55	3.02	52.99	40.81	24.51	87.44	1.00	24.00	14.40			10	10.08	13.92	0.02	0.48	10.56	13.44
12	0.02	0.05	24.00	107.33	17.55	3.02	52.99	40.81	24.51	87.44	1.00	24.00	14.40			15	11.86	12.14	0.02	0.48	12.34	11.66
13	0.02	0.05	6.00	26.83	10.44	2.73	28.46	21.91	13.16	46.95	1.00	6.00	3.60			5	2.45	3.55	0.30	1.80	4.25	1.75
13	0.02	0.05	6.00	26.83	10.44	2.73	28.46	21.91	13.16	46.95	1.00	6.00	3.60			10	3.23	2.77	0.30	1.80	5.03	0.97
13	0.02	0.05	6.00	26.83	10.44	2.73	28.46	21.91	13.16	46.95	1.00	6.00	3.60			15	3.80	2.20	0.30	1.80	5.60	0.40
14	0.02	0.0333	52.50	287.70	25.41	2.64	67.01	51.60	30.99	110.56	1.00	52.50	31.50			5	15.22	37.28	0.02	1.05	16.27	36.23
14	0.02	0.0333	52.50	287.70	25.41	2.64	67.01	51.60	30.99	110.56	1.00	52.50	31.50			10	20.08	32.42	0.02	1.05	21.13	31.37
14	0.02	0.0333	52.50	287.70	25.41	2.64	67.01	51.60	30.99	110.56	1.00	52.50	31.50			15	23.61	28.89	0.02	1.05	24.66	27.84
15	0.02	0.0333	11.90	65.21	14.56	2.38	34.62	26.66	16.01	57.12	1.00	11.90	7.14			5	4.49	7.41	0.15	1.79	6.28	5.62
15	0.02	0.0333	11.90	65.21	14.56	2.38	34.62	26.66	16.01	57.12	1.00	11.90	7.14			10	5.93	5.97	0.15	1.79	7.71	4.19
15	0.02	0.0333	11.90	65.21	14.56	2.38	34.62	26.66	16.01	57.12	1.00	11.90	7.14			15	6.97	4.93	0.15	1.79	8.76	3.14

CURB INLET COMPUTATIONS

W = 2 feet

n = 0.016

Inlet No.	Cross slope (Sx) ft/ft	Slope (S) ft/ft	Q (cfs)	Q/S^1/2	T	Fw	FwT	L1 0.77	L2 0.46	L3 1.65	Ql/Q	Qi	Q2/Q 0.60	Li Ql<Q2 L1<L2	Q1>Q2 L1>L2	Use L1	Actual Q1	Q-Qi Qc	Additional Flow Ratio	Additional Inlet Flow Due to Compound Section	Actual Q1	Q-Qi Qc
100 Year 24 Hour Storm																						
14	0.02	0.0333	89.73	491.72	31.06	2.73	84.93	65.40	39.28	140.14	1.00	89.73	53.84			10	31.21	58.52	0.01	0.90	32.11	57.62
15	0.02	0.0333	28.76	157.60	20.28	2.53	51.31	39.51	23.73	84.66	1.00	28.76	17.26			10	12.24	16.52	0.05	1.44	13.68	15.08
10 Year 24 Hour Storm																						
14	0.02	0.0333	55.24	302.71	25.90	2.65	68.53	52.77	31.69	113.08	1.00	55.24	33.14			10	20.94	34.30	0.02	1.10	22.04	33.20
15	0.02	0.0333	6.73	36.88	11.76	2.28	26.81	20.64	12.40	44.23	1.00	6.73	4.04			10	3.71	3.02	0.15	1.01	4.72	2.01
x	0.02	0.058	31.51	130.84	18.91	3.30	62.33	48.00	28.83	102.85	1.00	31.51	18.91			10	12.40	19.11	0.05	1.58	13.98	17.53
x	0.02	0.058	20.96	87.03	16.23	3.20	51.98	40.03	24.04	85.77	1.00	20.96	12.58			10	8.87	12.09	0.10	2.10	10.97	9.99
x	0.02	0.0667	57.62	223.11	23.10	3.67	84.73	65.24	39.18	139.80	1.00	57.62	34.57			10	20.06	37.56	0.04	2.30	22.37	35.25
12	0.02	0.0667	35.25	136.50	19.21	3.55	68.12	52.45	31.50	112.39	1.00	35.25	21.15			10	13.39	21.86	0.06	2.12	15.51	19.74
x	0.02	0.0667	15.08	58.39	13.97	3.34	46.63	35.91	21.57	76.94	1.00	15.08	9.05			10	6.67	8.41	0.10	1.51	8.18	6.90
12	0.02	0.0667	6.90	26.74	10.42	3.15	32.81	25.27	15.17	54.14	1.00	6.90	4.14			10	3.51	3.39	0.29	2.00	5.52	1.39
x	0.02	0.0667	33.20	128.55	18.78	3.53	66.32	51.07	30.67	109.43	1.00	33.20	19.92			10	12.75	20.45	0.14	4.65	17.40	15.80
13	0.02	0.0667	15.80	61.19	14.22	3.35	47.62	36.67	22.02	78.57	1.00	15.80	9.48			10	6.93	8.87	0.13	2.05	8.98	6.82
x	0.02	0.0667	2.01	7.78	6.56	2.84	18.65	14.36	8.62	30.77	1.00	2.01	1.21			10	1.28	0.73	0.31	0.62	1.91	0.10
13	0.02	0.0667	0.10	0.41	2.17	1.64	3.55	2.73	1.64	5.86	1.00	0.10	0.06			10	0.13	-0.02	2.00	0.21	0.34	-0.23



Templeton Gap Sub basin 2
Detention Pond
Stage-Capacity Curve
Stage in feet above outlet
0
1
2
3
4
5
6
7
8
9
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
Capacity in Acre-feet

T-GAP DETENTION POND

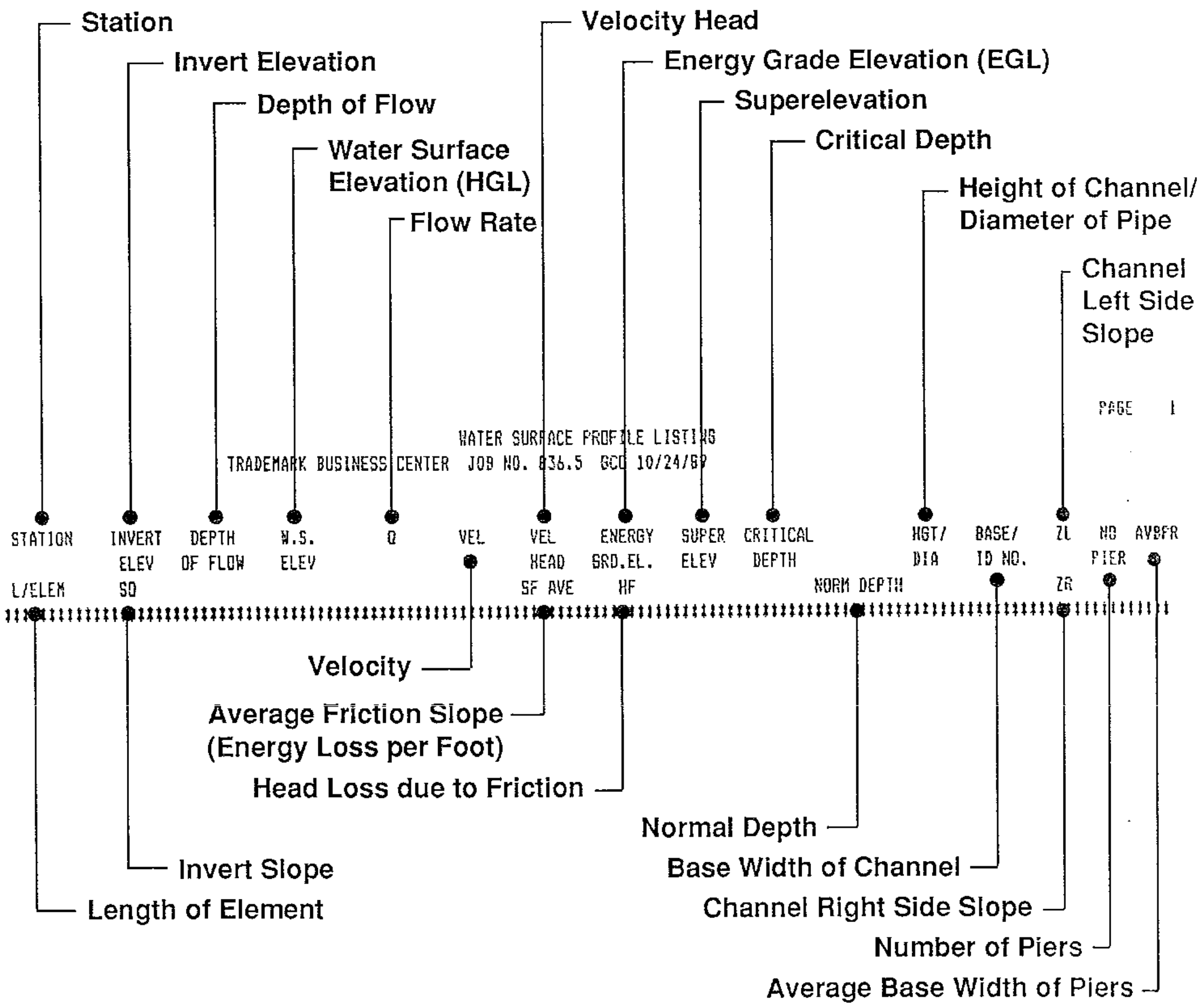
Proposed Grading

Stage	Area (sq in)	Area (ac)	Δ Volume (ac-ft)	Accum Volume (ac-ft)
60.9	0			
			0.035985	0.035985
62	1.14	0.07		
			0.176768	0.212753
64	1.94	0.11		
			0.269169	0.481921
66	2.75	0.16		
			0.365014	0.846935
68	3.61	0.21		
			0.088499	0.935434
68.4	4.10	0.24		

Existing

60.9	0			
			0.005682	0.005682
62	0.18	0.01		
			0.043618	0.0493
64	0.58	0.03		
			0.097567	0.146866
66	1.12	0.06		
			0.162994	0.30986
68	1.72	0.10		

WSPG
Hydraulic Computer Model
Output Legend



SYMBOL DEFINITIONS

Symbol	Definition
A	Cross sectional area of flow
Ap	Cross sectional area of pier
b	Base width of channel
bnet	Net base width of channel
bp	Average base width of piers
D	Depth of flow
Dc	Critical depth
Δ L	Length between two stations
Δ H	Drop in invert between two points
DH	Maximum open flow depth in a section: Ten feet above channel in closed section, Height of channel in closed section
DN	Normal depth
E	Specific Energy
EGL	Energy Grade Line elevation
e	Invert cross fall in inches
F	Force
g	Gravitational constant (32.2 ft/sec/sec)
HAPT	Head loss due to angle point
HB	Head loss due to bend or curve
HF	Head loss due to friction
HJ	Head loss in a junction
HMH	Head loss in a manhole
Ht	Head loss in a transition
HV	Velocity head
INV	Invert elevation in a channel section
M	Momentum
n	Manning`s n, coefficient of roughness
No P	Number of piers in a section (max. of 10)
P	Hydrostatic Pressure
Q	Flow Rate
R	Radius of pipe
RH	Hydraulic radius
r	Radius of curve on horizontal alignment
SF	Friction slope (energy loss per foot)
Sc	Critical slope (slope at critical depth)
So	Invert slope
SE	Super elevation
STA	Station
V	Velocity
WP	Wetted perimeter
WS/HGL	Water Surface or Hydraulic Grade Line Elevation
ZL	Left side slope
ZR	Right side slope

These variables may have one of two suffixes

1. Identifies the variable at the upstream end of an element.
2. Identifies the variable at the downstream end of an element.

Example: V1: is velocity at the upstream end
V2: is velocity at the downstream end.

Throughout this program: U/S is the upstream end
D/S is the downstream end

DEPTH OF FLOW (D)

Depth computation when area of flow is known.

1. CHAN. TYPE 1:

$$D = \frac{1}{Z_L + Z_R}\{-bnet+\sqrt{(bnet)^2 +2(ZL + ZR)\ [A+(e \cdot bnet/24)]}\}$$

2. CHAN. TYPE 2:

$$D = (1/bnet)\ [A +(e \cdot bnet/24)]$$

3. CHAN. TYPE 3:

Compute AH (The Full Area of a closed section).

If A is less than AH see Process 1. Otherwise:

D = HGL - Inv.

4. CHAN. TYPE 4:

Compute AH.

If A is not less than AH

D = HGL - Inv.

Otherwise

Find D by trial and error from Area of part full pipe.

5. CHAN. TYPE 5:

Find D by trial and error from Area of part full section.

6. CHAN. TYPE 6:

Compute AH

If A is not less than AH then

D = HGL - Inv.

Otherwise:

Find D by iteration from Area of part full irregular section.

COMPUTATIONAL PROCEDURES

Assumptions are: Steady one dimentional flow and incompressible fluids.

BASIC EQUATIONS OF STEADY FLOW

a) Equation of Continuity

$A1.V1 = A2.V2 = Q$

b) Manning's Formula (friction slope)

$Sf = \{Qn/[1.486\ A\ (RH)^{2/3}\ ]\}^2$

c) Bernoulli's Equation (open flow)

$D2 + HV2 + \Delta L\ Sfav + D1 + HV1 + \Delta L\ So$ where $HV = V /2g$

d) Bernoulli's Equation (pressure flow)

$D2 + HV2 + \Delta L\ Sfav + Hm = D1 + HV1 + \Delta L\ So$

where Hm is miscel.losses.

e) Angle Point Loss

$Hapt = 0.0033\ \theta\ HV$

Where θ is deflection angle in degrees. The District recommends not to exceed 6 .

f) Bend Loss

$HB = 0.2\ HV\sqrt{\Delta/90}$

where Δ is central angle of bend in degrees.

g) Manhole Loss

$Hmh = 0.05\ HV$ (No. MN) where No. MH is number of manhole in a reach

h) Specific Energy

$E = D + HV$

i) Pressure - Momentum

$P2 + M2 = P1 + M1 = F$

where $M = (Q)^2 / (Ag)$

j) Critical Depth Dc

Dc is the depth of flow at minimum energy, to find Dc by parabolic method see References 12.6.4 otherwise iterate for Dc in the specific energy equat.

$Ec = f\ (Dc) = Dc + HVC$

k) Normal Depth Dn

Dn is the depth of uniform and is found by iteration from Manning's formula

$A(RH)^{2/3} = f\ (Dn) = [Qn] / [1.486\ So^{1/2}]$

REACH ANALYSIS

a) OPEN FLOW

Intermediate points are computed on the W.S. profile in a reach using the standard step method. The difference in velocity head between two adjacent points is held to a maximum of ten percent.

$\Delta L = (E2 - E1) / (So - Sfav)$

b) Pressure Flow

$EGL\ 1 = EGL\ 2 + Hf + Hm$

$D1 = EGL\ 1 - HV1 - INV.\ 1$

If W.S. profile rises to the soffit of a conduit before the end of the reach or if the H.G.L. breaks seal before the end of the reach, minor losses are adjusted to reflect only the portion of the reach under pressure.

Super Elevation (S.E.)

Super elevation is computed in curving channels as follows:

CHAN. TYPE 1: (Trap. Sect.)

Subcritical flow: $S.E. = 1.15\ [HV/r]\ [b + D\ (ZL + ZR)]$

Supercritical flow: $S.E. = 2.6\ [HV/r]\ [b + D\ (ZL + ZR)]$

CHAN. TYPE 2: (Rect. Sect.)

Subcritical flow: $S.E. = HV\ b/r$

Supercritical flow: $S.E. = 2\ HV\ b/r$

TRANSITION ANALYSIS

If V2 is greater than V1 then

$Ht = 0.1\ [HV2 - HV1]$

otherwise:

$Ht = 0.2\ [HV1 - HV2]$

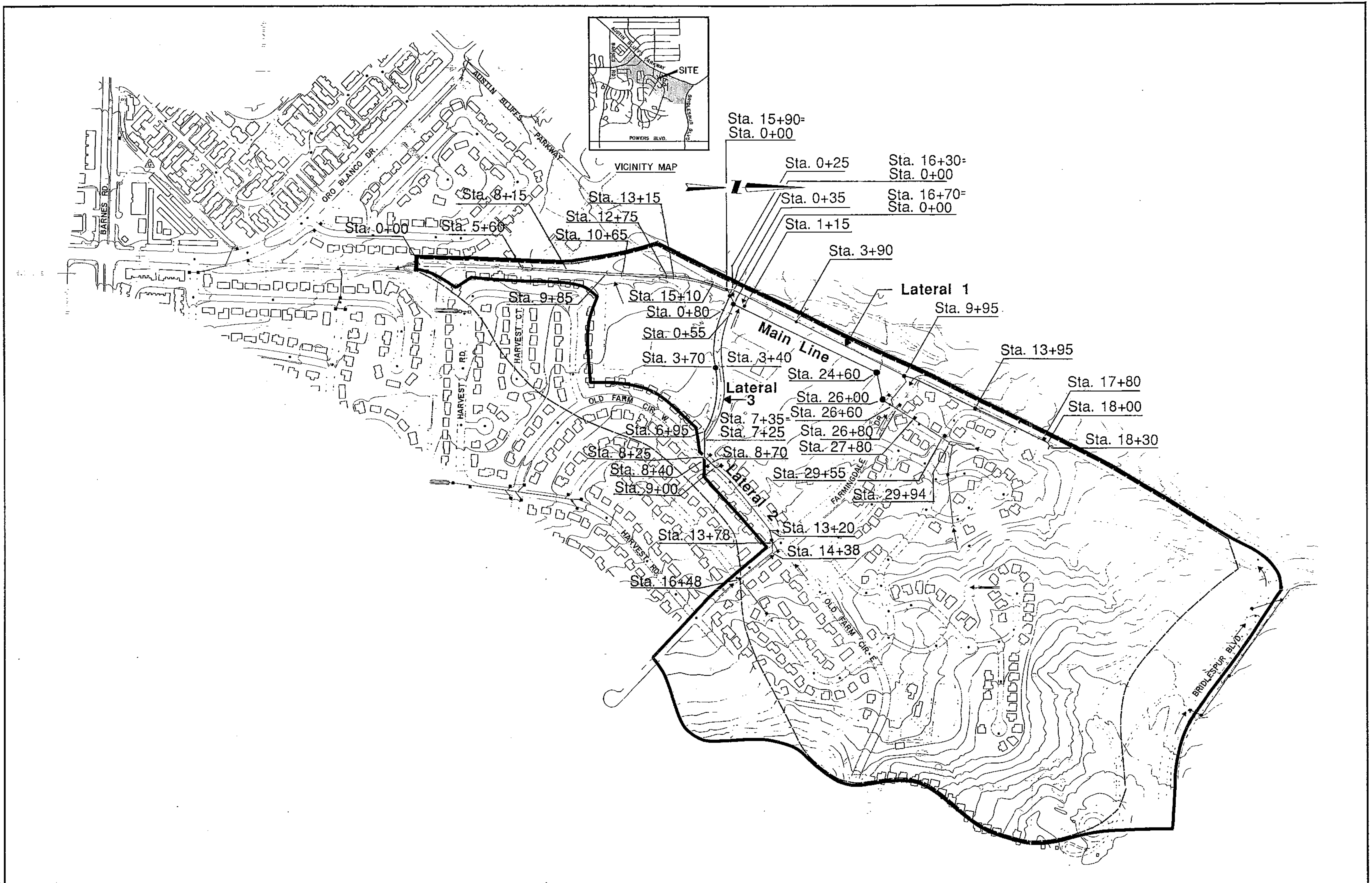
JUNCTION ANALYSIS

$$\Delta Y = [(Q2.V2) - (Q1.V1) - (Q3.V3.COSO3)\ (1/g)\ (1/A\ ave)] + \Delta L\ Sf\ av$$

where $A\ ave = [(A1 + A2)/2]$

and $\Delta Y = D1 + \Delta H - D2$

$HJ = \Delta Y + HV1 - HV2$



engineering professionals inc.	Pikes Peak Research Park 5475 Mark Dabling Boulevard Suite 111 Colorado Springs, Colorado 80918 (719) 590-8866	NO. / REVISIONS / BY / DATE	PREPARED UNDER THE SUPERVISION OF JPM DESIGNED AWMc · CHECKED AWMc · SCALE NTS DRAWN dlb · DATE May 1989	CLIENT: City of Colorado Springs	TITLE: Templeton Gap Basin A Sub-basin 2 Hydraulic Calculations -- Figure 5	JOB NO. 743.3 SHEET NO. / OF SHEETS

Hydraulic Computer Analysis
Main Line

Main Line
10 Year Flows

INPUT FILE LISTING

```
T1 TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 3/2/89 MBP
T2 MAIN STORM SEWER SYSTEM
T3 10 YEAR FLOWS
SD     0.00 560.00  1       .013
R    560.00 571.50  1       .013
R    561.00 571.50  2       .013
R    815.00 577.87  2       .013
R    816.00 579.87  2       .013
JX   955.00 584.60  2  2    .013   25.5        584.00       90.00
R   1065.00 586.15  2       .013
R   1066.00 588.12  2       .013
R   1275.00 593.37  2       .013
R   1315.00 594.37  2       .013                             2.59
R   1316.00 596.37  2       .013
R   1510.00 601.24  2       .013                            12.62
JX  1570.00 603.24  2  6   5.013   90.0   78.3 603.24 604.34 30.00 45.00
JX  1630.00 604.56  6  7    .013   33.7        606.56       45.00       1
JX  1670.00 606.58  9  8    .013   30.4        607.36       45.00 45.00 2
R   2460.00 647.60  5       .013                                  45.00 1
R   2600.00 654.82  5       .013                                  45.00 1
R   2660.00 657.92  5       .013
JX  2680.00 658.16  5  9    .013   30.1        659.66       90.00
JX  2780.00 659.38  5  9    .013    4.5        660.88       90.00
SH  2994.00 662.00  3       .013
```

CARD CODE	SECT NO	CHN TYPE	NO OF PIERS	AVE PIER WIDTH	HEIGHT 1 DIAMETER	BASE WIDTH	ZL	ZR	INV DROP	Y(1)	Y(2)	Y(3)	Y(4)	Y(5)	Y(6)	Y(7)	Y(8)	Y(9)	Y(10)
CD	1	1	0	.00	3.97	4.50	.88	.58	.00										
CD	2	1	0	.00	4.00	4.70	1.00	1.00	.00										
CD	3	4			3.00														
CD	6	4			4.00														
CD	7	4			2.00														
CD	8	4			2.50														
CD	9	4			1.50														
CD	10	4			1.00														

WATER SURFACE PROFILE - TITLE CARD LISTING

HEADING LINE NO 1 IS -

TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 8/2/89 NSP

HEADING LINE NO 2 IS -

MAIN STORM SEWER SYSTEM

HEADING LINE NO 3 IS -

10 YEAR FLOWS

WATER SURFACE PROFILE - ELEMENT CARD LISTING

```
ELEMENT NO   1 IS A SYSTEM OUTLET     *          *     *
                   U/S DATA   STATION    INVERT  SECT                                      W S ELEV
                                  .00  560.00    1                                              .00

ELEMENT NO   2 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                               560.00  571.50    1             .013                                         .00    .00     .00     0
WARNING - ADJACENT SECTIONS ARE NOT IDENTICAL - SEE SECTION NUMBERS AND CHANNEL DEFINITIONS

ELEMENT NO   3 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                               561.00  571.50    2             .013                                         .00    .00     .00     0

ELEMENT NO   4 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                               815.00  577.87    2             .013                                         .00    .00     .00     0

ELEMENT NO   5 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                               816.00  579.87    2             .013                                         .00    .00     .00     0

ELEMENT NO   6 IS A JUNCTION          *          *     *     *               *                *               *
                   U/S DATA   STATION    INVERT  SECT LAT-1 LAT-2   N      Q3        Q4    INVERT-3 INVERT-4  PHI 3   PHI 4
                               985.00  584.00    2     2     0   .013     26.9        .0   584.00      .00  90.00     .00

ELEMENT NO   7 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                              1065.00  586.12    2             .013                                         .00    .00     .00     0

ELEMENT NO   8 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                              1066.00  588.12    2             .013                                         .00    .00     .00     0

ELEMENT NO   9 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                              1275.00  593.37    2             .013                                         .00    .00     .00     0

ELEMENT NO  10 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                              1315.00  594.37    2             .013                                         .00   2.59     .00     0

ELEMENT NO  11 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                              1316.00  596.37    2             .013                                         .00    .00     .00     0

ELEMENT NO  12 IS A REACH             *          *     *
                   U/S DATA   STATION    INVERT  SECT             N                                       RADIUS  ANGLE  ANG PT  MAN H
                              1510.00  601.24    2             .013                                         .00  12.62     .00     0
```

WATER SURFACE PROFILE - ELEMENT CARD LISTING

```
ELEMENT NO  13 IS A JUNCTION         *          *     *     *                 *                    *               *
                  U/S DATA   STATION    INVERT  SECT LAT-1 LAT-2   N      Q3        Q4    INVERT-3 INVERT-4  PHI 3   PHI 4
                               1590.00  603.24    2     6     5  .013     90.0      78.5  603.24   604.24  30.00   45.00

ELEMENT NO  14 IS A JUNCTION         *          *     *     *                 *                    *               *
                  U/S DATA   STATION    INVERT  SECT LAT-1 LAT-2   N      Q3        Q4    INVERT-3 INVERT-4  PHI 3   PHI 4
                               1620.00  604.56    6     7     0  .013     35.7        .0  606.56      .00  45.00     .00

ELEMENT NO  15 IS A JUNCTION         *          *     *     *                 *                    *               *
                  U/S DATA   STATION    INVERT  SECT LAT-1 LAT-2   N      Q3        Q4    INVERT-3 INVERT-4  PHI 3   PHI 4
                               1670.00  606.88    9     8     0  .013     50.4        .0  607.36      .00  45.00   45.00
WARNING - ADJACENT SECTIONS ARE NOT IDENTICAL - SEE SECTION NUMBERS AND CHANNEL DEFINITIONS

ELEMENT NO  16 IS A REACH            *          *     *
                  U/S DATA   STATION    INVERT  SECT                   N                                     RADIUS  ANGLE  ANG PT  MAN H
                               2460.00  647.60    5                  .013                                       .00    .00   45.00     1

ELEMENT NO  17 IS A REACH            *          *     *
                  U/S DATA   STATION    INVERT  SECT                   N                                     RADIUS  ANGLE  ANG PT  MAN H
                               2600.00  654.82    5                  .013                                       .00    .00   45.00     1

ELEMENT NO  18 IS A REACH            *          *     *
                  U/S DATA   STATION    INVERT  SECT                   N                                     RADIUS  ANGLE  ANG PT  MAN H
                               2660.00  657.92    5                  .013                                       .00    .00     .00     0

ELEMENT NO  19 IS A JUNCTION         *          *     *     *                 *                    *               *
                  U/S DATA   STATION    INVERT  SECT LAT-1 LAT-2   N      Q3        Q4    INVERT-3 INVERT-4  PHI 3   PHI 4
                               2680.00  658.16    5     9     0  .013     30.1        .0  659.66      .00  90.00     .00

ELEMENT NO  20 IS A JUNCTION         *          *     *     *                 *                    *               *
                  U/S DATA   STATION    INVERT  SECT LAT-1 LAT-2   N      Q3        Q4    INVERT-3 INVERT-4  PHI 3   PHI 4
                               2780.00  659.38    5     9     0  .013      4.5        .0  660.88      .00  90.00     .00
```

WATER SURFACE PROFILE - ELEMENT CARD LISTING

ELEMENT NO 21 IS A SYSTEM HEADWORKS * *

U/S DATA	STATION	INVERT	SECT	W S ELEV
	2994.00	662.00	5	.00

NO EDIT ERRORS ENCOUNTERED-COMPUTATION IS NOW BEGINNING

ERROR MESSAGE NO. 32 - CRITICAL DEPTH MAY BE INACCURATE IN ELEMENT 19 INCREMENT = .000010
ERROR MESSAGE NO. 32 - CRITICAL DEPTH MAY BE INACCURATE IN ELEMENT 18 INCREMENT = .000010
ERROR MESSAGE NO. 32 - CRITICAL DEPTH MAY BE INACCURATE IN ELEMENT 17 INCREMENT = .000010
ERROR MESSAGE NO. 32 - CRITICAL DEPTH MAY BE INACCURATE IN ELEMENT 16 INCREMENT = .000010
** WARNING NO. 2 ** - WATER SURFACE ELEVATION GIVEN IS LESS THAN OR EQUALS INVERT ELEVATION IN HDWKDS, W.S.ELEV = INV + DC

WATER SURFACE PROFILE LISTING

TEMPLETON GAT SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 8/2/89 MSP

MAIN STORM SEWER SYSTEM

10 YEAR FLOWS

STATION / L/ELEM	INVERT ELEV / SO	DEPTH OF FLOW	W.S. ELEV	Q	VEL	VEL HEAD / SF AVE	ENERGY GRD.EL. / HF	SUPER ELEV	CRITICAL DEPTH	NORM DEPTH	HGT/ DIA	BASE/ ID NO.	ZL / ZR	NO PIER	AVBPR
.00	560.00	2.45	562.45	351.4	21.52	7.20	569.65	.00	4.33		3.97	4.50	.88	0	.00
231.92	.02054					.02167	5.46			2.47			.88		
231.92	565.17	2.41	567.58	351.4	22.01	7.53	575.11	.00	4.33		3.97	4.50	.88	0	.00
197.28	.02054					.02392	4.72			2.47			.88		
449.20	569.22	2.33	571.55	351.4	23.09	8.28	579.83	.00	4.33		3.97	4.50	.88	0	.00
110.80	.02054					.02726	3.02			2.47			.88		
560.00	571.50	2.24	573.74	351.4	23.28	8.40	582.14	.00	4.22		4.00	4.50	1.00	0	.00
1.00	.00000					.02662	.03			.00			1.00		
561.00	571.50	2.24	573.74	351.4	23.29	8.43	582.17	.00	4.22		4.00	4.50	1.00	0	.00
67.18	.02508					.02701	1.81			2.28			1.00		
628.18	573.18	2.22	575.41	351.4	23.49	8.58	583.99	.00	4.22		4.00	4.50	1.00	0	.00
186.82	.02508					.02925	5.46			2.28			1.00		
815.00	577.87	2.15	580.02	351.4	24.64	9.44	589.46	.00	4.22		4.00	4.50	1.00	0	.00
.31	2.00000					.02965	.01			.65			1.00		
815.31	578.49	2.21	580.70	351.4	23.74	8.75	589.46	.00	4.22		4.00	4.50	1.00	0	.00
.37	2.00000					.02641	.01			.65			1.00		
815.68	579.22	2.29	581.51	351.4	22.64	7.96	589.47	.00	4.22		4.00	4.50	1.00	0	.00
.32	2.00000					.02317	.01			.65			1.00		
816.00	579.87	2.37	582.24	351.4	21.58	7.24	589.48	.00	4.22		4.00	4.50	1.00	0	.00
JUNCT STR	.02444					.02725	4.61						1.00		
985.00	584.00	2.02	586.02	324.5	24.87	9.38	595.41	.00	4.04		4.00	4.50	1.00	0	.00
90.00	.02650					.03455	2.77			2.15			1.00		

WATER SURFACE PROFILE LISTING
TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 3/2/97 MSP
MAIN STORM SEWER SYSTEM
10 YEAR FLOWS

STATION	INVERT ELEV	DEPTH OF FLOW	W.S. ELEV	Q	VEL	VEL HEAD	ENERGY GRD.EL.	SUPER ELEV	CRITICAL DEPTH		HGT/ DIA	BASE/ ID NO.	ZL	NO PIER	AVBPR
L/ELEM	SO					SF AVE	HF			NORM DEPTH			ZR		
1065.00	586.12	1.97	588.09	324.5	25.47	10.09	598.18	.00	4.04		4.00	4.50	1.00	0	.00
.23	2.00000					.03509	.01			.62			1.00		
1065.23	586.58	2.01	588.59	324.5	24.84	9.59	598.18	.00	4.04		4.00	4.50	1.00	0	.00
.41	2.00000					.03172	.01			.62			1.00		
1065.64	587.39	2.08	589.47	324.5	23.69	8.72	598.20	.00	4.04		4.00	4.50	1.00	0	.00
.36	2.00000					.02787	.01			.62			1.00		
1066.00	588.12	2.11	590.23	324.5	22.59	7.93	598.21	.01	4.04		4.00	4.50	1.00	0	.00
209.00	.02512					.02712	5.67			2.18			1.00		
1275.00	593.37	2.11	595.48	324.5	23.25	8.40	603.88	.00	4.04		4.00	4.50	1.00	0	.00
40.00	.02500					.02859	1.14			2.18			1.00		
1315.00	594.37	2.10	596.46	324.5	23.48	8.57	605.03	.00	4.04		4.00	4.50	1.00	0	.00
.06	2.00000					.02868	.00			.62			1.00		
1315.06	594.49	2.11	596.59	324.5	23.31	8.44	605.04	.00	4.04		4.00	4.50	1.00	0	.00
.35	2.00000					.02664	.01			.62			1.00		
1315.41	595.19	2.18	597.37	324.5	22.22	7.67	605.05	.00	4.04		4.00	4.50	1.00	0	.00
.31	2.00000					.02337	.01			.62			1.00		
1315.72	595.81	2.26	598.08	324.5	21.19	6.98	605.05	.00	4.04		4.00	4.50	1.00	0	.00
.28	2.00000					.02050	.01			.62			1.00		
1316.00	596.37	2.35	598.72	324.5	20.20	6.34	605.06	.00	4.04		4.00	4.50	1.00	0	.00
57.39	.02510					.01822	1.05			2.18			1.00		
1373.39	597.81	2.41	600.22	324.5	19.45	5.88	606.11	.00	4.04		4.00	4.50	1.00	0	.00
50.48	.02510					.01622	.82			2.18			1.00		

WATER SURFACE PROFILE LISTING
TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #741.3 9/2/99 MEP
MAIN STORM SEWER SYSTEM
10 YEAR FLOWS

STATION	INVERT ELEV	DEPTH OF FLOW	W.S. ELEV	Q	VEL	VEL HEAD	ENERGY GRD.EL.	SUPER ELEV	CRITICAL DEPTH		HGT/ DIA	BASE/ ID NO.	ZL	NO PIER	AVBPR
L/ELEM	SO					SF AVE	HF			NORM DEPTH			ZR		
1423.87	599.08	2.50	601.58	324.5	18.55	5.35	606.92	.00	4.04		4.00	4.50	1.00	0	.00
36.56	.02510					.01424	.52			2.19			1.00		
1460.43	600.00	2.59	602.59	324.5	17.68	4.86	607.45	.00	4.04		4.00	4.50	1.00	0	.00
27.80	.02510					.01251	.35			2.18			1.00		
1488.23	600.69	2.68	603.37	324.5	16.86	4.42	607.79	.00	4.04		4.00	4.50	1.00	0	.00
21.77	.02510					.01098	.24			2.18			1.00		
1510.00	601.24	2.78	604.02	324.5	16.09	4.02	608.03	.00	4.04		4.00	4.50	1.00	0	.00
JUNCT STR	.03300					.01575	1.26						1.00		
1570.00	603.24	1.52	604.76	156.0	17.08	4.54	609.29	.00	2.71		4.00	4.50	1.00	0	.00
JUNCT STR	.03300					.02900	1.16						1.00		
1630.00	604.56	1.85	606.41	120.3	21.18	6.97	613.38	.00	3.30		4.00	.00	.00	0	.00
JUNCT STR	.05800					.21050	8.42						.00		
1670.00	606.88	1.43	608.31	69.9	21.01	6.86	615.17	.00	2.66		3.00	.00	.00	0	.00
599.71	.05154					.05163	30.96			1.43			.00		
2269.71	637.79	1.43	639.22	69.9	21.01	6.86	646.08	.00	2.66		3.00	.00	.00	0	.00
190.29	.05154					.04915	9.35			1.43			.00		
2460.00	647.60	1.47	649.07	69.9	20.23	6.36	655.44	.00	2.66		3.00	.00	.00	0	.00
47.22	.05157					.04501	2.13			1.43			.00		
2507.22	650.04	1.50	651.54	69.9	19.68	6.02	657.56	.00	2.66		3.00	.00	.00	0	.00
45.30	.05157					.04077	1.85			1.43			.00		
2552.52	652.37	1.56	653.93	69.9	18.77	5.47	659.41	.00	2.66		3.00	.00	.00	0	.00
27.93	.05157					.03594	1.00			1.43			.00		

WATER SURFACE PROFILE LISTING
TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 8/2/89 HSP
MAIN STORM SEWER SYSTEM
10 YEAR FLOWS

STATION	INVERT ELEV	DEPTH OF FLOW	W.S. ELEV	Q	VEL	VEL HEAD	ENERGY GRD.EL.	SUPER ELEV	CRITICAL DEPTH		HGT/ DIA	BASE/ ID NO.	ZL	NO PIER	AVBPR
L/ELEM	SO					SF AVE	HF			NORM DEPTH			ZR		
2580.45	653.81	1.62	655.44	69.9	17.89	4.97	660.41	.00	2.66		3.00	.00	.00	0	.00
15.55	.05167					.03171	.62			1.43			.00		
2610.00	654.82	1.69	656.51	69.9	17.06	4.52	661.03	.00	2.66		3.00	.00	.00	0	.00
.74	.05167					.02953	.02			1.43			.00		
2600.74	654.86	1.69	656.55	69.9	17.01	4.50	661.05	.00	2.66		3.00	.00	.00	0	.00
14.29	.05167					.01780	.40			1.43			.00		
2615.03	655.60	1.76	657.36	69.9	16.22	4.09	661.45	.00	2.66		3.00	.00	.00	0	.00
11.07	.05167					.02459	.27			1.43			.00		
2626.10	656.17	1.83	658.00	69.9	15.46	3.72	661.72	.00	2.66		3.00	.00	.00	0	.00
8.77	.05167					.02178	.19			1.43			.00		
2634.87	656.62	1.91	658.53	69.9	14.75	3.38	661.91	.00	2.66		3.00	.00	.00	0	.00
7.03	.05167					.01932	.14			1.43			.00		
2641.90	656.98	1.99	658.97	69.9	14.06	3.07	662.04	.00	2.66		3.00	.00	.00	0	.00
5.58	.05167					.01718	.10			1.43			.00		
2647.48	657.27	2.07	659.35	69.9	13.41	2.79	662.14	.00	2.66		3.00	.00	.00	0	.00
4.42	.05167					.01532	.07			1.43			.00		
2651.90	657.50	2.17	659.67	69.9	12.78	2.54	662.21	.00	2.66		3.00	.00	.00	0	.00
3.42	.05167					.01370	.05			1.43			.00		
2655.32	657.68	2.27	659.95	69.9	12.19	2.31	662.25	.00	2.66		3.00	.00	.00	0	.00
2.49	.05167					.01231	.03			1.43			.00		
2657.81	657.81	2.38	660.19	69.9	11.62	2.10	662.29	.00	2.66		3.00	.00	.00	0	.00
1.60	.05167					.01115	.02			1.43			.00		

WATER SURFACE PROFILE LISTING
TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 8/2/89 MSP
MAIN STORM SEWER SYSTEM
10 YEAR FLOWS

STATION	INVERT ELEV	DEPTH OF FLOW	W.S. ELEV	Q	VEL	VEL HEAD	ENERGY GRD.EL.	SUPER ELEV	CRITICAL DEPTH		HGT/ DIA	BASE/ ID NO.	ZL	NO PIER	AVBPR
L/ELEM	SO					SF AVE	HF			NORM DEPTH			ZR		
2659.41	657.89	2.51	660.39	65.9	11.09	1.91	662.30	.00	2.55		3.00	.00	.00	0	.00
.59	.05157					.01022	.01			1.43			.00		
2660.00	657.92	2.60	660.52	65.9	10.55	1.73	662.31	.00	2.55		3.00	.00	.00	0	.00
JUNCT STR	.01200					.00569	.13						.00		
2680.00	658.16	4.89	663.04	39.2	5.63	.49	663.54	.00	2.05		3.00	.00	.00	0	.00
JUNCT STR	.01220					.00318	.32						.00		
2780.00	659.38	4.19	663.57	35.3	4.99	.39	663.96	.00	1.93		3.00	.00	.00	0	.00
JUNCT STR	.01220					.00553	-.55						.00		
2994.00	662.00	4.19	666.19	35.3	4.99	.39	666.58	.00	1.93		3.00	.00	.00	0	.00

Main Line
100 Year Flows

INPUT FILE LISTING

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T1 TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 5/2/89 MSP
T2 MAIN STORM SEWER SYSTEM
T3 100 YEAR FLOWS
SO      0.00 569.10  1      .013
R     560.00 571.50  1      .013
R     561.00 571.50  2      .013
R     815.00 577.87  2      .013
R     911.00 579.87  2      .013
JX    985.00 584.00  2  2   .013   39.9        584.00       90.00
R    1065.00 586.12  2      .013
R    1066.00 588.12  2      .013
R    1275.00 593.37  2      .013
R    1315.00 594.37  2      .013                             2.59
R    1316.00 596.37  2      .013
R    1510.00 601.24  2      .013                            12.47
JX   1590.00 603.24  2  6  5.013  160.0   142.4 603.24 604.24 30.00 45.00
JX   1650.00 604.56  6  7   .013   35.7        606.56       45.00       1
JX   1670.00 606.88  9  8   .013   92.5        607.35       45.00 45.00 2
R    2460.00 647.60  5      .013                                  45.00 1
R    2500.00 654.82  5      .013                                  45.00 1
R    2560.00 657.92  5      .013
JX   2680.00 658.16  5  9   .013   11.5        659.65       70.00
JX   2720.00 659.38  5  9   .013    7.3        660.88       90.00
SH   2791.00 662.00  5      .013
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WSPG

WATER SURFACE PROFILE - CHANNEL DEFINITION LISTING

CARD CODE	SECT NO	CHN TYPE	NO OF PIERS	AVE PIER WIDTH	HEIGHT 1 DIAMETER	BASE WIDTH	ZL	ZR	INV DROP	Y(1)	Y(2)	Y(3)	Y(4)	Y(5)	Y(6)	Y(7)	Y(8)	Y(9)	Y(10)
CD	1	1	0	.00	3.97	4.50	.88	.88	.00										
CD	2	1	0	.00	4.00	4.50	1.00	1.00	.00										
CD	5	4			3.00														
CD	6	4			4.00														
CD	7	4			2.00														
CD	8	4			2.50														
CD	9	4			1.50														
CD	10	4			1.00														

WATER SURFACE PROFILE - TITLE CARD LISTING

HEADING LINE NO 1 IS -

TEMPLETON GAP SUGGESTED ALTERNATIVE MAIN RUN JOB #743.3 8/2/89 MSP

HEADING LINE NO 2 IS -

MAIN STORM SEWER SYSTEM

HEADING LINE NO 3 IS -

100 YEAR FLOWS

WATER SURFACE PROFILE - ELEMENT CARD LISTING

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ELEMENT NO   1 IS A SYSTEM OUTLET    *          *      *
                   U/S DATA   STATION    INVERT  SECT                                         W S ELEV
                                  .00  560.00    1                                                .00

ELEMENT NO   2 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                               560.00  571.50    1               .013                                            .00    .00     .00     0
WARNING - ADJACENT SECTIONS ARE NOT IDENTICAL - SEE SECTION NUMBERS AND CHANNEL DEFINITIONS

ELEMENT NO   3 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                               561.00  571.50    2               .013                                            .00    .00     .00     0

ELEMENT NO   4 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                               815.00  577.87    2               .013                                            .00    .00     .00     0

ELEMENT NO   5 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                               816.00  579.87    2               .013                                            .00    .00     .00     0

ELEMENT NO   6 IS A JUNCTION         *          *     *     *                 *                  *             *
                   U/S DATA   STATION    INVERT  SECT LAT-1 LAT-2   N      Q3        Q4   INVERT-3 INVERT-4  PHI 3   PHI 4
                               985.00  584.00    2     2     0   .013     37.9        .0   584.00      .00  90.00     .00

ELEMENT NO   7 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                              1065.00  586.12    2               .013                                            .00    .00     .00     0

ELEMENT NO   8 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                              1066.00  588.12    2               .013                                            .00    .00     .00     0

ELEMENT NO   9 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                              1275.00  593.77    2               .013                                            .00    .00     .00     0

ELEMENT NO  10 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                              1315.00  594.37    2               .013                                            .00   2.59     .00     0

ELEMENT NO  11 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                              1316.00  596.37    2               .013                                            .00    .00     .00     0

ELEMENT NO  12 IS A REACH            *          *      *
                   U/S DATA   STATION    INVERT  SECT              N                                          RADIUS  ANGLE  ANG PT  MAN H
                              1511.00  601.24    2               .013                                            .00  11.62     .00     0
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