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AMENDMENT NO. 3 TO PINE CREEK DRAINAGE BASIN PLANNING STUDY AND MASTER DEVELOPMENT DRAINAGE PLAN FOR THE PINE CREEK AND CORDERA NEIGHBORHOODS (PORTIONS CONTRIBUTING TO PINE CREEK)

October 2002
Minor Text Revisions
February 2003

Prepared For:

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DRAINAGE REPORT STATEMENT

ENGINEER'S STATEMENT:
The attached amendment to the approved drainage basin planning study was prepared under my direction and supervision and is correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the City for drainage reports. I accept responsibility for any liability caused by any negligent acts, errors, or omissions on my part in preparing this report.

COLORADO REGISTERED VANCEL S FOSSINGER 31972 PROFESSIONAL ENGINEER

Vancel Fossinger

Vancel S. Fossinger, Colorado P.E. #31972
For and On Behalf of JR Engineering, Ltd.

3-10-03
Date

DEVELOPER'S STATEMENT:
I, the developer, have read and will comply with all of the requirements specified in this amendment to the approved Pine Creek Drainage Basin Planning Study.

Business Name: LP47, LLC
dba La Plata Investments

By: Thomas Taylor
Thomas Taylor

Title: Director of Development Services

Address: 2315 Briargate Parkway, Suite 100
Colorado Springs, CO 80920

CITY OF COLORADO SPRINGS ONLY:
Filed in accordance with Section 15-3-906 of the Code of the City of Colorado Springs, 1980, as amended.

Timothy R. Mitros

City Engineer
Conditions

Mch 18, 2003
Date

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EXECUTIVE SUMMARY

The "Pine Creek Drainage Basin Planning Study" by Obering, Wurth and Associates, approved June 20, 1989, implemented a stormwater management concept that included use of both private and public detention facilities to limit the fully developed condition peak 100-year flow rate in Pine Creek at Highway 83 to a maximum of 2536 cfs. The study identified the historic peak 100-year flow rate for this location as 1210 cfs and required the Developer of the Briargate area to make improvements to the reach of channel downstream of Highway 83 before the historic rate was exceeded.

"Amendment No. 2 to the Pine Creek Drainage Basin Planning Study and Master Development Drainage Plan for Pine Creek Subdivision," by JR Engineering, approved October 9, 1998, revised the storm water management plan upstream of Highway 83. The revised storm water management plan increased detention in the watershed to limit the peak 100-year discharge from the watershed upstream of Highway 83 to 1210 cfs, the historic peak rate as previously defined in the original Drainage Basin Planning Study (D.B.P.S.). This was consistent with the goals of the original D.B.P.S. as set forth in the section titled "Implementation". This change was driven primarily by heightened environmental concerns regarding construction of extensive improvements in historic watercourses as well as changes in drainage criteria and drainage management philosophy by government agencies and the major landowner in the basin. In addition the revised plan proposed to accomplish more detention within regional detention facilities and thus eliminated the requirement for on-site detention except in areas where downstream conveyance capacity is inadequate consistent with City policy.

Since approval of "Amendment No. 2", a significant amount of land has been developed in the portion of the watershed located between Highway 83 and proposed Powers Boulevard and more detailed land plans have been prepared for the portion of the watershed located upstream of proposed Powers Boulevard. The presence of habitat of the Prebles Meadow Jumping Mouse, (a species listed as "threatened" under the Federal ESA), within the watershed has led to a decrease in the amount of land that can be developed within the watershed. The Storm Water Management Plan contained within this "Amendment 3" to the Drainage Basin Planning Study addresses these changes to bring the plan up to date while maintaining consistency with the goals and concepts provided in the previous approved plans for the watershed. Consistent with the previous plans, the "Amendment 3" plan limits the peak 100-year peak discharge from the watershed upstream of Highway 83 to 1210 cfs, the historic peak rate as defined in the original Pine Creek D.B.P.S.

The City of Colorado Springs adopted "Drainage Criteria Manual Volume 2 – Storm Water Quality Policies, Procedures and Best Management Practices (BMPs)" (DCMV2) and it became effective on November 1, 2002. The current City policy is that development currently planned for the build out of the portion of the Pine Creek Watershed located within the Briargate Master Plan area and downstream of proposed Powers Boulevard is exempt from the new criteria imposed in DCMV2 as this development has been planned for a number of years and it is very difficult to implement the new standards within the planned development. Development located upstream of proposed Powers Boulevard as well as future redevelopment of sites located downstream of Powers Boulevard will be subject to the DCV2.

As reported in the original "Pine Creek Drainage Basin Planning Study" the Pine Creek Drainage Basin has been approved by the City and County as a "No Fee " basin as it relates to City and County Drainage resolutions. This Amendment is intended to serve as the storm water management guideline for the portion of the Pine Creek drainage Basin located upstream of Highway 83.

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I. INTRODUCTION

A. Contract Authorization

This document and associated analysis was prepared with private funds for LP47, LLC dba La Plata Investments by JR Engineering, LLC. La Plata Investments is the major landowner and developer within the study area.

B. Purpose and Scope

This document is to serve as an update and third amendment to the Pine Creek Drainage Basin Planning Study (D.B.P.S.) prepared by Obering, Wurth and Associates as approved June 20, 1989, by the City of Colorado Springs. This document will also serve as the Master Development Drainage Plan for the portions of the Pine Creek and Cordera Neighborhoods located within the Pine Creek drainage basin. This document will replace "Amendment No. 2 to the Pine Creek Drainage Basin Planning Study and Master Development Drainage Plan for the Pine Creek Subdivision" as the current drainage master plan for the Pine Creek Drainage Basin located upstream of State Highway 83.

1. The drainage management plan presented in this document is generally consistent with the plan presented in Amendment No. 2. Key items that the current document will provide include the following:

a) An updated hydrological analysis of the portion of the Pine Creek Basin located east of State Highway 83 (the study area).

b) Updated identification of the drainage facilities that have been constructed within the portion of the basin located east of State Highway 83 (the study area).

c) Updated identification of the existing and current proposed land uses within the easterly portion of the Pine Creek Basin.

d) Revised proposed drainage treatment within the portion of the Pine Creek drainage basin located east of Highway 83. The major treatment revisions from the Amendment 2 plan consist primarily of:

- Eliminating the requirement for adding additional storage volume to existing Regional Detention Facility No. 1. (The outlet modifications proposed in Amendment 2 are still required and have been constructed).
- Proposed modifications to the now existing Regional Detention Facility "B" to reduce the outflow rate.
- Expansion of the storage capacity of Regional Detention Facility "C" to accommodate an expansion of its watershed.
- Combining previously proposed regional Detention Facilities "F" and "G" into one facility to be identified as Regional Detention Facility "F".
- Adding a requirement for detention of developed condition flows to the portion of the study area that is located upstream of the Briargate Master Plan area.
- Division of previously proposed Regional Detention Facility "D" into two smaller regional detention facilities to be identified as Regional Detention Facilities "D1" and "D2".
- Preservation of the portion of Pine Creek North Fork as a natural channel through the Powers Boulevard corridor and approximately 1500 linear feet upstream of the Powers Corridor. (The previous proposed culvert under Powers Boulevard on the north fork will be replaced by four bridges according to the current C.D.O.T. plans for Powers Boulevard.)
- Elimination of the storm sewer outfall from the upper portion of Sub-basin PN13 though utilization of an existing Golf Course Pond as an extended detention facility.
- More detailed planning in the Pine Creek, Cordera (former Johnson Ranch) and Briargate Crossing (formerly a part of the Gatehouse Neighborhood) Neighborhoods. Some changes in proposed drainage patterns also occur in these neighborhoods.
- Minor revisions to the Major Basin Boundary due to updated topography and additional portions of the Pine Creek Neighborhood being diverted to the Kettle Creek Basin. (These diversions have been addressed in the drainage Master planning in the Kettle Creek Basin.)

- A change of drainage patterns in the portion of the Briargate Business Campus located upstream of Regional Detention Facility No. 1 (current Sub-Basins PM6A, PM6B and a portion of PM5).
- The City of Colorado Springs adopted "Drainage Criteria Manual Volume 2 – Storm Water Quality Policies, Procedures and Best Management Practices (BMPs)" (DCMV2) and it became effective on November 1, 2002. The current City policy is that development currently planned for the build out of the portion of the Pine Creek Watershed located within the Briargate Master Plan area and downstream of proposed Powers Boulevard is exempt from the new criteria imposed in DCMV2 as this development has been planned for a number of years and it is very difficult to implement the new standards within the planned development. Development located upstream of proposed Powers Boulevard as well as future redevelopment of sites located downstream of Powers Boulevard will be subject to the DCV2.

2. The current revisions are proposed primarily as a result of changes in land use in the basin (existing and planned). These land use changes have occurred through more detailed planning by the major landowner, the requirement of larger setbacks from the natural stream beds due to the presence of Preble's Meadow Jumping Mouse Habitat, and by less impervious development being constructed than was planned in portions of the watershed. The proposed current changes are consistent with Amendment No. 2 overall concepts.

3. In regards to the Pine Creek and Cordera Neighborhoods, this document will estimate the peak flow rates of storm water runoff and identify the overall concept for treatment of the runoff within the portion of these neighborhoods that will contribute runoff to Pine Creek when they are developed. The identified treatment consists of:

a. The general proposed direction of flow for developed condition drainage.

b. The major components of proposed storm sewer systems including outfall points, proposed detention basin locations and estimated sizes.

c. General guidelines for the proposed treatment of the portion of Pine Creek Channel that is contained within the neighborhoods.

More specific and detailed analysis and drainage treatment plans has or will be provided with the submittal of individual drainage reports for each neighborhood filing and or major drainage facilities within the Pine Creek and Cordera Neighborhoods.

C. Past Studies

A number of previous studies and reports were reviewed during the preparation of the current study. The most relevant studies are listed below along with a brief synopsis. Additional, reports that were reviewed are noted in the reference section of this study.

"**Pine Creek Drainage Basin Planning Study**," June 1988 revised October 1988, by Obering, Wurth and Associates. This study included all of the Pine Creek drainage basin above Academy Boulevard. Key items of this study included the following:

- Major drainage conveyances were primarily to be open channels.
- Required onsite detention to achieve a 35 percent reduction in the peak flow rate resulting from development (the difference between the historic and developed peak rates) on all office, research and development, commercial, and school properties.
- Free discharge from all other properties was proposed.
- The 100-year historic peak flow rate in Pine creek as it crosses under Highway 83 was estimated at 1210 cfs.
- Improvements were to be made to the portion of Pine Creek between Highway 83 and Academy Boulevard to allow it to convey a proposed 100-year peak flow rate from above Highway 83 of 2536 cfs. These improvements were to be made to the channel before the historic flow rate from the area above Highway 83 was exceeded.
- Five regional detention ponds were to be constructed above Highway 83 to regulate the peak 100-year discharge rate in the fully developed condition to 2536 cfs.
- Detention Facility No. 1 was to be constructed on the Pine Creek Main Channel near the intersection of Briargate Parkway and Highway 83 and fitted with a restricter plate to temporarily reduce the planned outflow. The purpose of the reduced outflow was to regulate the down stream 100-year flow in Pine Creek to less than the historic 100-year peak rate. This was to be done to allow development to begin in the watershed before the portion of channel between Academy Boulevard and Highway 83 was improved.

“**Amendment No. 1 to the Pine Creek Drainage Basin Planning Study,**” July 17, 1992, revised July 29, 1992, by Obering, Wurth and Associates.

- This amendment proposed the addition of a sixth regional detention pond. The proposed fully developed condition 100-year peak flow rate from the area above Highway 83 was to remain at 2536 cfs.

“**Amendment No. 2 to Pine Creek Drainage Basin Planning Study and Master Development Drainage Plan for Pine Creek Subdivision,**” October 1998, by JR Engineering. This amendment included reanalysis and revision of the drainage master plan for the area of the basin above State Highway 83. Key items included in the study included the following:

- An updated hydrological analysis of the portion of the Pine Creek Basin located east of State Highway 83 (the study area)
- Identification of the drainage facilities that had been constructed within the portion of the basin located east of Highway 83
- Identification of the then current proposed land uses within the portion of the Pine Creek Basin located east of Highway 83
- Revised proposed drainage treatment within the portion of the Pine Creek drainage basin located east of Highway 83. The treatment revisions consisted primarily of:
  - Eliminating the requirement for on-site detention except in areas where existing outfall lines do not have sufficient capacity to convey free discharge.
  - Increasing the overall detention storage volume to be provided in the proposed regional detention ponds, thus reducing the design storm flow in several locations of Pine Creek including the point that it flows under Highway 83 and onto the grounds of the Air Force Academy (100-year peak discharge rate to be limited to a maximum of 1210 cfs).
  - Replacing proposed lined open channel conveyances with underground storm sewers in several locations.
  - Relocation and reconfiguration of previously proposed regional detention facilities and adding additional regional detention facilities.

D. Agency Jurisdictions

The proposed drainage improvements as well as the majority of the included watershed in the current study are located within the Colorado Springs City limits. The extreme upper portions of the watershed included in this study are unincorporated areas of El-Paso County. Runoff from the unincorporated areas of the watershed has been accounted for in the current study.

The portion of Pine Creek that is located immediately downstream of the study area is located on the grounds of the United States Air Force Academy (USAFA). The original Pine Creek Drainage Basin Planning Study (D.B.P.S.) was reviewed by and contains a letter of approval from the USAFA.

Section VIII of the original Pine Creek D.B.P.S. is titled "Implementation." The second paragraph of this section states that "the primary basin management goal for this particular drainage basin is one of limiting a peak discharge from the study area at State Highway 83 to historic or below for as long a period as possible." Later in the text the "historic peak discharge" is mentioned as the 100-year historic rate of 1210 cfs.

The drainage plan contained in this current proposed Amendment No. 3 to the Pine Creek D.B.P.S. proposes to restrict the peak 100-year flow rate in Pine Creek at Highway 83 to a maximum of 1210 cfs with the upstream watershed in the interim and the future fully developed condition consistent with Amendment No. 2, and with the stated goal of the original Pine Creek D.B.P.S.

The improvements required to accomplish this will be constructed at the expense of and on land owned by La Plata Investments, the major landowner in the study area, except for proposed detention facilities and related storm sewer located in areas upstream of the Briargate Master Plan area.

It is anticipated that the City of Colorado Springs will be the sole agency for review and approval of this Amendment to the D.B.P.S.

It is understood that other agencies such as FEMA, the U.S Army Corps of Engineers, and the U.S. Fish and Wildlife Service will have involvement in review and approval of more detailed plans for individual projects proposed in this study at the time that they are designed.

E. Drainage Criteria

Storm drainage design and management within the study area must conform to the current City Colorado Springs Criteria and the current approved drainage master plans for the area. Some of the areas within the study area have existing drainage systems that were designed assuming that office, research and development, commercial and school sites would be developed with on-site detention. These sites are subject to the on-site detention requirement unless adequate capacity in the downstream storm conveyance system is demonstrated. Sites with this restriction are identified on the Basin Map and Master Plan contained in the appendix of this report. Additional discussion of this topic is contained in Section V. Paragraph E. of this report.

II. PROJECT DESCRIPTION, LOCATION AND DRAINAGE

A. Basin Location and Size

The study area is a portion of the Briargate Community located in the northeast portion of Colorado Springs. As shown on the vicinity map the study area is bounded by the Kettle Creek Drainage Basin on the north and the Cottonwood Creek Drainage Basin on the south. The lower or western limit of the study area of Amendment No. 3 is defined by the crown of Highway 83. The upper limit of the study area is located approximately 22,000 feet to the east of Highway 83 and coincides with the upper limit of the Pine Creek Drainage Basin. The study area is approximately 2,917-acres or 4.56 square miles in size.

B. Major Drainage Ways and Facilities

An updated existing drainage facility map was prepared as a part of this study. A copy of this map is contained in the appendix of this report. As shown on the map a considerable amount of drainage improvements have been constructed within the study area to support the existing development.

1. Storm Sewers Systems

Three significant storm sewer systems were constructed in the Amendment No. 2 study area prior to preparation of that study. They were referenced to as the Focus on the Family storm sewer system, the South Chapel Hills Drive storm sewer system and the North Chapel Hills Drive storm sewer system. The North Chapel Hills Drive storm sewer system has been extended and a system draining Regional Detention Facilities 'B' and 'E' have been constructed since that time. Several smaller storm sewer systems have been constructed as well.

The initial phase of the Focus on the Family storm sewer system was constructed to serve as an outfall from the Focus on the Family Site. The system begins in Summer Field Subdivision Filings No. 5 and 6, is routed through the existing Summer Field Detention Pond, then south in Summerset Drive, west in Research Parkway, west across the Focus on the Family site, then north in Explorer Drive and finally west in Briargate Parkway to outfall into Detention Facility No. 1.

The South Chapel Hills Drive storm sewer begins in Dynamic Drive east of Chapel Hills Drive. It is then routed north in Chapel Hills Drive to outfall into Pine Creek on the west side of Chapel Hills Drive.

The North Chapel Hills Drive storm sewer begins in Sagehill Drive just east of Chapel Hills Drive. The upper portion of the system is routed through existing Regional Detention Facility 'A' at Lexington Drive then is routed southwest in Chapel Hills Drive to outfall into Pine Creek on the west side of Chapel Hills Drive.

2. Pine Creek

Pine Creek is an unimproved natural channel throughout most of the study area. At the downstream end of the study area a concrete box culvert with three (3) 14-foot span by 10-foot rise cells carries the creek under Highway 83. Upstream, a single cell 12-foot span by 10-foot rise concrete box culvert carries the outflow from Detention Facility No. 1 under Briargate Parkway and back to the Pine Creek Channel. On the upstream (north) side of Briargate Parkway, existing Detention Facility No. 1 accepts and detains all of the flow from the upstream Pine Creek Channel. Further upstream a new bridge has been constructed to carry Pine Creek under Chapel Hills Drive. Upstream of Chapel Hills Drive the lower portions of the north and south forks of the creek have been replaced by Regional Detention Facilities 'B' and 'E' and a storm sewer system.

The portion of Pine Creek that begins at Highway 83 and extends approximately 8,500 feet upstream to the historic confluence of the north and south fork of Pine Creek is for the most part heavily vegetated with willows and cattails and appears to be quite stable. This portion of channel is identified as Reaches 1, 2 and 3 on the drainage maps contained in this document. This portion of channel has existed in a unique environment for several years in that it has been sheltered from significant frequent flows and has a minor base flow that provides the moisture required to support the vegetation.

Aerial photography of the study area indicates that considerable water conservation treatment was constructed in the watershed prior to 1955. The treatment consists of small ditch/dikes constructed on the contour in many of the steeper portions of the watershed and several small online retention ponds constructed at frequent intervals along both the north and south forks of Pine Creek upstream of the confluence. There are also several small retention basins spread throughout the watershed to intercept small concentrated flows upstream of the defined Pine Creek Channel. While a detailed analysis of this treatment has not been performed with the current study it is speculated that the treatment has sheltered the downstream channel from all but large infrequent flows. This environment has allowed the vegetation in the channel to become well established.

Upstream of Regional Detention Facilities 'B' and 'E' on the north and south forks the character of the Pine Creek Channel changes as the presence of perennial water in the channel is greatly reduced. Several areas of the channel bottom are dry in all but large rainfall events. Small springs and water impounded in the online retention basins mentioned above keep other areas moist. With the reduction of the available water in the channel the quantity and quality of the vegetation in the channel is also less in the reaches upstream of the confluence than found in the lower reaches of the channel.

C. Existing and Proposed Land Use

Approximately 1400-acres of the 2,910-acre study area are currently developed. The remainder of the area is currently undeveloped rangeland. Much of the remaining undeveloped area is expected to develop at a relatively fast pace in the coming years.

Most of the study area has been master planned for land use. The master plan land uses were utilized for this study. The exhibit contained in the appendix entitled "Subdivision and Land Use Identification Map" indicates the current land use assumption. The following table is a summary of these land uses.

PROJECTED LAND USE
Fully Developed Condition

Land Use	Assumed Percent Impervious	Area (acres)	Percent of Study Area
Golf Course		204	7%
Park		135	4%
Open Space		240	8%
Residential			
1-2 DU/AC	20-25	240	8%
2.5-3 DU/AC	27-30	540	19%
3.5-4 DU/AC	34-37	240	8%
4.5-5 DU/AC	40-44	30	1%
6-18 DU/AC	56-70	130	5%
Light Industrial/Office	83	201	7%
Commercial	95	384	13%
Church	80	20	1%
School	50	110	4%
Misc. Other	50-68	26	1%
Unknown	45	127	4%
Arterial Street	85	285	10%
TOTAL		**2912**	**100%**

D. Existing and Proposed Utilities

Several underground utility lines are in place within the study area. Many more will be constructed to support future development. Consideration was given to the fact that there will be several locations where storm sewer facilities and other utilities must cross. The major anticipated crossings were investigated and no problems that are insurmountable were found. All future storm sewers as well as other underground utilities should be designed and constructed with consideration for existing and future adjacent facilities.

E. Soils/Erosion Potential

A Hydrologic Soils Group Map was provided in the original Pine Creek D.B.P.S. This map shows the hydrologic soil group limits and the soil mapping units as identified in the "Soil Survey of El Paso County Area, Colorado," published by the U.S.D.A. Soil Conservation Service (SCS) in 1975. The map indicates that the majority of the soils in the study area belong to Hydrologic Soil Groups "A" and "B". A portion of the Briargate Business Campus contains soils in the Hydrologic Group "C". A small portion of Sub-basins PN7, PN9, and PN13 contain soils identified as belonging to Hydrologic Soil Group "D".

The erosion potential as reported in the SCS "Soils Survey for El Paso County Area," varies from slight to high in the study area.

III. FIELD INVESTIGATIONS

A. Topographic Mapping

Topographic data utilized in this study was compiled from a variety of sources including City of Colorado Springs FIMS program, aerial mapping done by Aero-Metrics between 1999 and 2001 and topographic mapping done by JR Engineering.

B. Subsurface Investigation

No subsurface investigation was performed specifically for this project. Subsurface investigations will be required for individual projects as appropriate.

C. Environmental Considerations

LP47, LLC dba La Plata Investments, the majority landowner in the study area contracted with an environmental consultant to perform surveys, identify, and map environmentally sensitive areas within the study area. Potential areas of concern are areas that meet the qualifications of wetlands and areas that contain the habitat of the Prebles Meadow Jumping Mouse (PMJM). The PMJM has been listed as a threatened species protected under the Federal Endangered Species Act.

The U.S Army Corps of Engineers issued a 404 permit in January 2001for identified improvements to Pine Creek and adjacent development for the portion of Pine Creek located between Highway 83 and Chapel Hills Drive. The permitting process included consultation with the U.S. Fish and Wildlife Service due to the presence of the PMJM habitat in the area.

A habitat conservation plan (HCP) for the portion of the study area located upstream of Chapel Hills Drive has been prepared by La Plata's consultants and is currently under review by the U.S. Fish and Wildlife Service. Upon obtaining approval of the HCP from U.S. Fish and Wildlife Service an application for a 404 permit to work within waters of the United States will be submitted to the U.S Army Corps of Engineers.

The current plan for wetland disturbance mitigation includes constructing wetlands in the bottoms of several of the regional detention facilities located within the study area. .

In general, one of the goals of the overall plan, consistent with Amendment No. 2, is to minimize the peak flow rates contributed to Pine Creek in order to minimize impacts to the channel.

IV. HYDROLOGIC AND HYDRAULIC DESIGN EVALUATION

A. Basin Hydrology

1. Analysis Purpose

The following items were the goals of the hydrologic analysis performed for Amendment No. 2 and this study:

a. Estimate peak runoff rates for sub-basins to be developed in the future

b. Provide peak flow rates to be used in the design of proposed major conveyances and the evaluation of the ability Pine Creek to convey developed condition flows.

c. Provide inflow and outflow hydrographs and required storage volumes to be used in the design of proposed regional detention facilities and the evaluation of existing regional detention facilities.

d. Demonstrate the adequacy of the proposed plan to control the 100-year peak in Pine Creek as it crosses under Highway 83 to a maximum of 1210 cfs (the historic 100-year peak flow rate established by the original Pine Creek D.B.P.S.).

e. Estimate peak rates that are somewhat conservative so that some flexibility may be available for changes in land use planning. A conservative approach is prudent when working with a drainage system that relies on detention basins and closed conduit conveyance systems with finite capacities.

2. Methodology

The hydrologic analysis performed for this study was based on the Soil Conservation Service (SCS) Dimensionless Unit Hydrograph utilizing the U.S. Army Corps Of Engineers HEC–1 computer program as modified by Haestad Methods Inc., May 1991 version. The HEC-1 model developed in Amendment No. 2 was updated to reflect changed conditions for this study.

As with the previous amendment a new basin map was created along with revised sub-basin boundaries, lag times, and estimated curve numbers. The HEC-1 model created for the fully developed condition with the previous study was updated with the new data. A second model was then created from the first with the upper part of the watershed, east of proposed future Powers Boulevard, evaluated in the "existing condition" in order to evaluate a partially developed or "interim" condition.

a. Times of Concentration

Times of Concentration (TC) were estimated based on actual flow paths in existing developed areas and undeveloped areas for the existing condition model only. Times of concentration for the fully developed condition model were based on estimated flow paths in areas where development has not occurred. Estimated flow paths were patterned after average flow paths for similar existing development located in the Briargate area. Summary sheets containing the data utilized in the TC calculations are included in the appendix of this study. Lag time as utilized in the methodology was calculated as 0.6 TC (in hours).

b. Curve Numbers

A problem that has been encountered in the past has been matching peak flow rates calculated in detailed analyses done for drainage reports to allowable flow rates calculated in non-detailed analyses based on general assumptions for drainage basin planning studies. A goal of the current analysis was to produce peak flow rates for individual sub-basins with the HEC-1 Model that are similar to peak flow rates that would be calculated by the rational method. In an effort to achieve this goal Curve Numbers (CN) utilized in the model were first estimated for individual sub-basins based on the anticipated land uses within the individual sub-basins assuming antecedent moisture condition II. These estimated CN's were then entered into the model and peak 100-year flow rates were generated by the HEC-1 program for individual sub-basins. The peak 100-year flow rates were then entered into a spreadsheet and compared to 100-year peak flow rates generated by a rational method calculation for corresponding sub-basins. The CNs were then adjusted and the process was repeated until a reasonable agreement existed between the peak rates generated by the HEC-1 Model output and the peak rates generated by the rational method calculation. This adjustment caused an increase in the overall predicted peak rates and volumes generated in the study area. No effort was made to adjust Curve Numbers for the undeveloped basins in the Interim condition model, as future design calculations by rational method for the condition are unlikely. Copies of the spreadsheets utilized to calculate and adjust the curve numbers are contained in the appendix of this study.

c. Design Storm

The Type IIA 24 hour storm distribution was utilized in the HEC-I model. Rainfall depths of 4.4" for the 100-year storm and 2.6" for the 5-year storm were used in the simulations. A calculation time interval of 3 minutes was used in order to satisfy the program recommendation that the time interval be less than or equal to .29 lag. A limitation of the Version of HEC-1 program that was used is that it can only generate 300 hydrograph points. At three-minute intervals output is only generated for the first 15-hours of the 24-hour storm. The peak inflow and outflow rates associated with all of the facilities included in the model occur well before 15-hours of the storm has passed so this is considered insignificant for the purpose of this study.

d. Analysis Approach for Areas of Existing Development

The primary importance of including the existing developed areas in the current analysis was to generate hydrographs from these areas that were produced with the same methodology as used in the remainder of the study area. In the current analysis hydrographs from the areas of existing development were added to hydrographs from the areas of future development to produce hydrographs at points of interest to the current proposed plan.

The somewhat conservative methodology used for both the current analysis and Amendment No. 2 has produced hydrographs in some of these areas of existing development that are larger than predicted by the existing approved MDDPs and final drainage reports for these areas. This is not necessarily indicative of problems with the previous analyses but rather is the result of utilizing a different and potentially more conservative approach of analysis that was chosen to allow some tolerance for the unknowns that exist at the D.B.P.S. level of analysis.

One approach that was considered in Amendment No. 2 for modeling the existing areas was to revise the "curve numbers" and "lag times" used in the areas of existing development to produce peak flow rates similar to those produced by previous analyses. This approach was not used, as the resulting hydrographs would be skewed in volume and or in time in comparison with the remainder of the model. Both time and volume are very important when modeling detention facilities so it was determined that it was more appropriate to universally apply the chosen methods of calculating lag times and applying curve numbers than it was to match the output of several previous analyses performed by several individuals using varying methodologies and criteria.

The Amendment No. 2 and the current analysis do not include a detailed analysis of the existing storm sewer systems constructed prior to the implementation of Amendment No. 2. At points in the watershed where runoff rates in excess of the existing downstream storm sewer capacity would result in the excess flow being diverted out of the watershed or conveyed to a substantially different outfall into Pine Creek, a simplistic evaluation of the capacity of the existing storm sewer was made. The downstream capacity was assumed to be equal to the full pipe conveyance capacity of the most restricted segment of the downstream storm sewer of interest. Where storm sewer capacity was found to be less than the 100 year peak flow rates predicted by the current analysis, the HEC I model was revised to divert excess flow from the storm sewer system and route, it to Pine Creek via an approximate surface route or out of the watershed as appropriate for the location. This serves to provide a conservative estimate of the total flows that will be conveyed in Pine Creek through and out of the study area.

B. Major Drainage Way Hydraulics

1. Floodplain Delineation Maps

The Federal Emergency Management Agency, Flood Insurance Study (FIS) for El-Paso County and Incorporated Areas was revised and reissued on March 17, 1997. Six Panels of the Flood Insurance Rate Maps (FIRMs) produced as a part of the FIS include potions of the Pine Creek study area. JR Engineering on behalf of La Plata requested a Letter of Map Revision (LOMR), after La Plata had completed several of the improvements proposed in Amendment 2 to the Pine Creek D.B.P.S. This LOMR subsequently modified three of the six above-mentioned FIRMs. The FEMA case number for the approved LOMR is 00-08-088P. The LOMR was approved by FEMA and became effective on July 28, 2000.

A Map entitled "Pine Creek FEMA 100-Year Flood Zone Limits" is included in the appendix of this report. The map contains the FEMA 100-year flood zone limits for all of the Pine Creek Study area as well as references to the individual FIRM panels and or LOMR that the information was obtained from. The floodzone limits were digitized into the map from the FIRM panels. It should be noted that some adjustments were made to the alignment of some segments of the boundaries in order to get them to generally line up with the Pine Creek Channel Topography because a direct overlay indicates that the overall accuracy of the FIRMS is not good. Due to this, the map should not be used to determine the specific location of the FEMA 100-year floodplain. Specific location of the FEMA floodplain should be determined from the effective FIRMs or LOMR as appropriate.

A request for a conditional letter of map revision (CLOMR) was prepared by JR Engineering on behalf of La Plata Investments for the elimination of the FEMA 100 year flood zone on the South Fork of Pine Creek above Regional Detention Facility 'B'. This CLOMR is based on the construction of proposed storm sewer systems in upstream Briargate Parkway and Union Boulevard. This CLOMR (FEMA Case No. 01-08-202R) was issued by FEMA on August 23, 2001. The area that this CLOMR will impact is noted on the above mentioned map contained in the appendix of this report.

2. Flood Profiles

A detailed hydraulic analysis for Pine Creek or major proposed storm sewers was not included in the scope of this study. A detailed hydraulic analysis of Pine Creek between Chapel Hills Drive and Highway 83 is presented in the draft "Channel Stability Analysis for Pine Creek Chapel Hills Drive to State Highway 83" by JR Engineering, dated May 2002. . The preparation of another similar report is in progress for the portion of Pine Creek located upstream of Chapel Hills Drive that is proposed to remain as an open channel. Hydraulic grade lines for proposed closed conduit conveyances will be prepared with and presented on the construction drawings for the same.

Detailed channel analyses were completed where applicable in preparation of the application submittals for the above referenced CLOMR and LOMR. Results and details of the analyses are contained in these application submittals.

V. PROPOSED DRAINAGE PLAN

A. General Description

Proposed updated plans for the fully developed condition and an interim, partially developed condition have been prepared as a part of this study. Both plans are presented graphically on maps contained in the appendix of this study and are described in the following text.

Consistent with the previous amendment the fully developed condition plan proposes that there be eight regional detention facilities distributed throughout the Briargate Master Plan portion of the study area. Four of these facilities ("No.1", "A", "B", and "E") have been constructed and or modified as proposed in Amendment 2. Detention Facility "C" has been excavated and currently function as a retention pond per the "Amendment No. 2" "Interim Plan". Detention Facilities "D1", "D2", and "F" are proposed to be constructed as future development occurs in the watershed.

Detention is also proposed for the portion of the study area located upstream of the Briargate Master Plan. If the detention requirements for the majority of this area are met in a single detention pond, the facility should be considered the ninth regional detention pond in the study area.

The current analysis indicates that the proposed detention facilities will limit the 100-year peak outflow in Pine Creek from the study area to less than 1210 cfs. Proposed major conveyance facilities throughout the watershed consist of closed conduits and portions of the Pine Creek Natural Channel. The proposed detention facilities are distributed to mitigate high peak flow rates throughout the conveyance system in order to limit the size of the required storm sewers and to mitigate the erosion potential in the natural channels. The Interim Plan indicates the portion of the proposed facilities that are required to support a certain level of development in the study area.

B. Fully Developed Condition Plan

1. Pine Creek North Fork (Sub-basins PNE1 through PNE14)

The watershed begins east of future Powers Boulevard. It is proposed that the peak 100-year discharge from Sub-basins PNE1 and PNE2 located at the upper end of the major basin be limited to not more than 1.17 cfs per acre (the approximate average peak 100-year discharge per acre in the existing condition). For the purpose of modeling the runoff from this area, future land uses were assumed to be consistent with the Bradley Ranch Master Plan within the area covered by that plan. A CN value of 77 was assumed for the unknown developed condition for areas outside of the Bradley ranch plan. Conceptual detention facilities were sized to limit the peak release rate from these Sub–basins to the above mentioned flow rate.

The actual size of the detention facilities for Sub-basins PNE1 and PNE2 shall be adjusted based on the actual density and basin characteristics of proposed future development. However, the 100-year peak outflow rates shall be as established in this study. Sub-basin PNE1 shall be limited to a peak 100-year discharge of 103 cfs and Sub-basin PNE2 shall be limited to a peak 100-year discharge of 127 cfs. If possible at the time of development it is recommended that the detention requirement for both of these sub-basins be met in a single regional facility with a 100 year peak outflow less than or equal to 230cfs. If a single facility is not practical, smaller privately owned and maintained ponds will be required to meet the discharge requirement.

It is proposed that the detained flows from Sub-basins PNE1 and PNE2 as well as the runoff from Sub-basin PNE3 be collected and conveyed in proposed storm sewers to a proposed diversion box at Analysis Point E1. For the purpose of the current analysis it was assumed that the proposed diversion box will be designed such that peak flows less than the approximate 5-year peak rate will be directed to a proposed storm sewer parallel to Pine Creek North Fork in Sub-basin PNE5. A trickle flow shall be discharged to the downstream natural channel in all runoff events to promote the growth of vegetation. Flow rates in excess of the approximate 5-year peak rate shall be allowed to overflow into existing Pine Creek North Fork located in Sub-basin PNE5. The HEC-1 models developed for the report estimates a 100-year peak flow of 239 cfs at Analysis Point E1. Of this 156 cfs will be directed to the proposed downstream storm sewer and 131cfs will overflow to the North Fork Channel. The purpose of this division of flow is to utilize the conveyance capacity of the natural channel in large runoff events and mitigate the potential for erosion in the channel due to increased peak rates, volume, and frequency of runoff events that will occur with development. Some improvements to the natural channel as discussed in Section V. Paragraph D., Sub-paragraph 3. of this report will be required in order to assure capacity and mitigate the potential for erosion. At the time that detailed planning is done for this area this concept should be further analyzed to assure that it fits with the proposed land use and the conveyance system is optimized. A more detailed analysis of the natural channel with perhaps some select erosion control measures added may reveal that the natural channel can carry more frequent flows and the proposed downstream storm sewer can be downsized.

The flow diverted to the proposed storm sewer will be routed from Analysis Point E1 to Analysis Point E2, (Q_5 = 164 cfs, Q_{100} = 310 cfs) along with runoff collected from Sub-basin PNE4. The flow that is allowed to overflow into the natural channel will be routed from Analysis Point E1 to Analysis Point E3 along with runoff from Sub-basin PNE5. At Analysis Point E3 the flow in the natural channel (Q_5 = 5 cfs, Q_{100} = 95 cfs) should be intercepted by a proposed storm sewer and routed to Analysis Point E4 along with the flow from Analysis Point E2.

The peak 100-year runoff from Sub-basin PNE6 should be limited to 15 cfs if further development occurs in the sub-basin (similar to Sub-basins PNE1, and PNE2 discussed above). The runoff from Sub-basin PNE7 will be collected in future streets and storm sewers and conveyed to Analysis Point E4 along with the flow from Sub-basin PNE6. A diversion box is proposed to accept the runoff conveyed to Analysis Point E4 (Q_5 = 238 cfs, Q_{100} = 543 cfs) and divide it between a proposed downstream storm sewer and the downstream natural channel. The diversion box should be designed such that the modeled peak 100-year inflow is divided such that approximately 173 cfs is routed to the proposed downstream storm sewer and approximately 370 cfs is routed to the downstream portion North Fork Channel. During more frequent runoff events all of the inflow up to approximately 130 cfs should be diverted to the proposed downstream storm sewer excepting the trickle flow discussed below. The 130 cfs rate corresponds to the expected 2-year peak rate at Analysis Point E4. A trickle flow shall be directed to the channel in all runoff events to promote the growth of vegetation in the channel. The purpose of this division of flow is to utilize the conveyance capacity of the downstream natural channel in large runoff events and mitigate the potential for erosion due to increased peak rates, volume, and frequency of runoff events that will occur with development of the upstream watershed.

A concept detail drawing of the proposed diversion boxes for Analysis Points E1 and E4 is included in the appendix of this report. It may be possible to combine the proposed diversion boxes with future curb inlets to reduce costs.

The storm sewer portion of the flow from Analysis Point E4 will be routed to Analysis Point E5 where runoff from Sub-basin PNE8 will be added to the proposed storm sewer for combined peak rates of Q_5 = 185 cfs, Q_{100} = 269 cfs. This flow will be routed to Analysis Point E6 along with flow collected from Sub-basin PNE9. The flow at Analysis Point E6 (Q_5 = 194 cfs, Q_{100} = 299 cfs) will be routed to Analysis Point 3 at the proposed rundown channel to proposed Regional Detention Facility "F". The flow allowed to overflow to the natural channel will be conveyed in the natural channel along with runoff from Sub-basin PNE10 to Analysis Point 3. The HEC-1 model indicates peak rates in the natural channel of Q_5 = 180 cfs, Q_{100} = 437 cfs (AP3a) at Analysis Point 3.

Runoff from Sub-basins PNE11 is to be collected on-site and conveyed to Analysis Point 1 in a proposed storm sewer. Runoff from Sub-basin PNE12 is to be collected onsite and added to the storm sewer at Analysis Point 1 (Q_5 = 198 cfs, Q_{100} = 345 cfs). The flow will then be routed to Analysis Point 2 in a proposed storm sewer. Runoff from Sub-basins PNE13 and PNE14 will be collected in the Union and Powers Boulevard right of ways and conveyed to Analysis Point 2 in a storm sewer system to be constructed by CDOT with Powers Boulevard. It is expected that the above-described systems will be joined at Analysis Point 2 for combined peak rate of Q_5 = 255 cfs, Q_{100} = 493 cfs. The flow will be routed in a proposed storm sewer to Analysis Point 3. The peak flow rates expected to enter proposed Detention Facility "F" at Analysis Point 3 are Q_5 = 470 cfs, Q_{100} = 1178 cfs.

2. Pine Creek North Fork (Sub-basins PN7 through PN15)

The runoff from Sub-basins PN7 and PN8 is to be routed through proposed Regional Detention Facility "F". The majority of runoff from developed portions of these Sub-basins should be routed directly to Detention Facility "F" in proposed storm sewers in order to limit the potential for erosion on the banks of the proposed detention facility.

Proposed Regional Detention Facility "F" is planned to have a 100-year peak inflow of 1401cfs, a 100-year peak outflow of 220cfs, and a 100-year storage volume requirement of 56-acre feet. Outflow from Regional Detention Facility "F" will be passed under proposed Royal Pine Drive in a proposed storm sewer and released into the Pine Creek North Fork Channel where it will be conveyed downstream to Existing Regional Detention Facility "E".

Note that portions of Sub-basins PN9 and PN10 were combined and other sub-basin boundaries adjusted from the previous study of this area. The Sub-basin descriptor PN10 is not used in this current study. It is proposed that to the extent practical, runoff from the developed portion of Sub-basin PN9 be collected and conveyed within the proposed development and conveyed in a proposed storm sewer and released to the Pine Creek North Fork Channel upstream of Analysis Point 4. Runoff from the undeveloped portion of the sub-basin will enter the channel at or upstream of Analysis Point 4 located between Regional Facilities "F" and "E".

Existing Regional Detention Facility "E" will receive the routed outflow from Regional Detention Facility "F" as well as all of the runoff from Sub-basins PN9, PN11, and PN12. Regional Detention Facility "E" has a planned 100-year peak inflow of 593 cfs, a 100-year peak outflow of 224 cfs, and a 100-year storage volume requirement of 17-acre feet. Outflow from Regional Detention Facility "E" will be conveyed in an existing storm sewer to Analysis Point 5 located at the western limit of Sub-basin PN15. At Analysis Point 5, the runoff from Sub-basin PN15 will enter the existing storm sewer. A 100-year peak rate of 276 cfs is estimated for the flow in the existing storm sewer at Analysis Point 5.

Runoff from current Sub-basin PN13 a portion of Pine Creek Filing Numbers 26, 32, 33 has been routed to an existing golf-course irrigation pond as defined in the Drainage Report for Pine Creek Filing No. 26. The existing irrigation pond has enough capacity above it's normal water surface to store the entire 100-year runoff volume from Sub-basin PN13 and will release it to Pine Creek at a very small discharge rate via. an existing pipeline that connects the pond in Sub-basin PN13 to the irrigation pond located in Sub-basin PM2. Runoff from Sub-basin PN13 was to be routed to Regional Detention Facility "E" in Amendment No. 2.

Sub-basin PN14 of "Amendment 2" was combined with Sub-basin RM2 in the current study and is not shown on the current map as a separate area.

3. Pine Creek South Fork (Basins PSE1 through PSE11)

The watershed begins east of future Powers Boulevard. It was assumed for the purpose of this plan that in the fully developed condition the area east of and including future Powers Boulevard will be divided into Sub-basins PSE1 through PSE11.

The runoff from Sub-basins PSE1 through PSE6 will be collected in future streets and storm sewers and conveyed to and routed through proposed Regional Detention Facility "D1". Regional Detention Facility "D1" is planned to have a 100-year peak inflow of approximately 611 cfs, a 100-year peak outflow of 89 cfs, and a 100-year storage volume requirement of 19 acre feet. Outflow from Regional Detention Facility "D1" will be routed west, down a proposed storm sewer in the Briargate Parkway right-of-way to Analysis Point 6 at the east side of future Powers Boulevard.

Runoff from Sub-basin PSE7 will be collected in a storm sewer system within the sub-basin and routed in the storm sewer to the proposed Briargate Parkway storm sewer at Analysis Point 6.

Runoff from Sub-basins PSE8 and PSE9 will be routed in proposed parking areas, streets and storm sewers to Proposed Regional Detention Facility "D2". Regional Detention Facility "D2" is planned to have a 100-year peak inflow of approximately 269 cfs, a 100-year peak outflow of 61 cfs, and a 100-year storage volume requirement of 8 acre feet. The outflow from Regional Detention Facility "D2" will be routed in a proposed storm sewer to the proposed Briargate Parkway storm sewer at Analysis Point 6 along with runoff from Sub-basin PSE10. Analysis Point E10 (Q_{100} = 144 cfs) represents the combined flow from Detention Facility D2 and Sub-Basin PSE10. At Analysis Point 6 the flow from Detention Facilities "D1" and "D2" will be combined with the flow from Sub-basins PSE7 and PSE10. The HEC-1 model indicated the 100-year peak flow rate at this location will be 413 cfs. This flow will be conveyed to Analysis Point 6A located on the west side of the Powers Boulevard right of way in the proposed Briargate Parkway storm sewer.

Runoff from Sub-basin PSE11 will be added to the proposed Briargate Parkway storm sewer at or above Analysis Point 6A for an estimated peak 100 year flow of 478 cfs. This flow will be conveyed in the proposed Briargate Parkway storm sewer to Analysis Point 6B.

4. Pine Creek South Fork (Basins PS2 through PS13)

Runoff from Sub-basin PS2 is planned to enter the proposed Briargate Parkway storm sewer at or upstream of Analysis Point 6B. The estimated 100 year peak flow rate at Analysis Point 6B is 547cfs. This flow will be routed in the proposed Briargate Parkway storm sewer to Analysis Point 7 at proposed Austin Bluffs Parkway.

On the east side of Austin Bluffs Parkway, the proposed Briargate Parkway storm system changes from pipe to box culvert. Runoff from Sub-basins PS3 and PS4 is planned to enter the proposed Briargate Parkway box culvert at or upstream of Analysis Point 7. The estimated 100-year peak flow rate at Analysis Point 7 is 908cfs. This flow will be routed in the proposed Briargate Parkway box culvert to Analysis Point 7A.

Runoff from Sub-basins PS5 and PS6 is planned to enter the proposed Briargate Parkway box culvert at or upstream of Analysis Point 7A. The estimated 100 year peak flow rate at Analysis Point 7A is 1190cfs. This flow will be routed in the proposed Briargate Parkway box culvert to Analysis Point 8 at proposed Union Boulevard.

Runoff from Sub-basins PS7 and PS8 is planned to enter the proposed Briargate Parkway box culvert at or upstream of Analysis Point 8. The estimated 100-year peak flow rate at Analysis Point 8 is 1569 cfs. This flow will be routed in the proposed Briargate Parkway box culvert to Regional Detention Facility "C".

Runoff from Sub-basins PS9 is planned to be collected and conveyed in a proposed storm sewer in the future Union Boulevard right-of-way to discharge in the north east corner of existing Detention Facility "C".

Runoff from Sub-basins PS10 is planned to be collected and conveyed within the proposed Pine Creek Filing No. 16 subdivision to a proposed storm sewer outfall that will discharge to the north side of Regional Detention Facility "C". Regional Detention Facility "C" lies in the southern portion of Sub-basins PS10.

Analysis Point 9 represents the combined flow from the Proposed Briargate Parkway storm sewer and Sub-basin PS9 (the combined flow from the Union Boulevard. and Briargate Parkway storm sewer systems). The HEC-1 model indicates a 100-year peak flow rate of 1735cfs at Analysis Point 9. Regional Detention Facility "C" has a planned 100-year peak inflow rate of 1,825 cfs, a 100-year peak outflow rate of 228 cfs, and a 100-year peak storage volume requirement of 72-acre feet. Outflow from Regional Detention Facility "C" will be routed to existing Regional Detention Facility "B" in existing and proposed storm sewer to be located in and adjacent to the Briargate Parkway right-of-way.

The runoff from Sub-basin PS11 will be conveyed with the outflow from Detention Facility "C" in the Briargate Parkway storm sewer to Regional Detention Facility "B". Analysis Point 10 (Q_{100}= 304cfs) represents this combined flow. The runoff from Sub-basin PS12 will be collected and conveyed to Detention Facility "B" in the South Fork of Pine Creek. Regional Detention Facility "B" has a planned 100-year peak inflow rate of 493 cfs, a 100-year peak outflow rate of 219 cfs, and a 100-year peak storage volume requirement of 21 acre feet.

A proposed restrictor plate is to be installed to block the top 4.2 square feet of the existing 54" diameter outlet of Detention Facility "B" in order to achieve the required peak discharge. The addition of the restrictor plate will make it possible to better utilize available storage volume in the pond under the current development plan and is one of the things that makes it possible to eliminate the expansion of the storage volume in downstream Regional Detention Facility No. 1.

Outflow from Regional Detention Facility "B" will be routed in the existing storm sewer to the existing storm sewer junction located near Analysis Point 11. At Analysis Point 11 runoff from Sub-basin PS13 will be combined with the outflow from Existing Regional Detention Facility "B". The flow at Analysis Point 11 (Q_{100} = 263cfs) will then be routed in an existing storm sewer to a storm sewer junction at Analysis Point 5A. At Analysis Point 5A this flow is combined with the flow in the storm sewer from the North Fork of Pine Creek (Analysis Point 5). The combined flow (Q_{100} = 531cfs) will be routed in the existing storm sewer to the outfall in the existing Pine Creek Channel then down the natural channel to Analysis Point 12.

5. Pine Creek Main Channel (Basins PM1 through PM4)

As indicated in the approved "Drainage Letter to Amend the Master Development Drainage Plan for Village Center at Pine Creek and Preliminary/Final Drainage Report for Village Center at Pine Creek and Preliminary/Final Drainage Report for Village Center at Pine Creek Filing No. 2 and Pine Creek Village Center Filing No. 1," by JR Engineering, February 2000, the runoff from Sub-basins PM1 and the portion of PM3 that is planned to be developed as multi-family residential property will enter the Pine Creek Channel via a proposed storm sewer upstream of Analysis Point 12. The runoff from Sub-basin PM2 and the reminder of Sub-basin PM3 will enter the Pine Creek channel at or upstream of Analysis Point 12. The estimated peak flow rates at Analysis

Point 12 are Q_5 = 372 cfs and Q_{100} = 899 cfs. The combined flow will be routed in the Pine Creek channel from Analysis Point 12 to Analysis Point 13 at the east side of Chapel Hills Drive.

Runoff from Sub-basin PM4 will outfall into Pine Creek at two locations between Analysis Points 12 and 13. Runoff from Sub-basin PM4 is included in the peak flow rates estimated at Analysis Point 13 of Q_5 = 399 cfs and Q_{100} = 1017 cfs. The combined flow will be routed under Chapel Hills Drive to Analysis Point 19.

6. Chapel Hills Drive South (Sub-basins CS1 through CS4)

Analysis Point 16 represents the flow collected in Chapel Hills Drive and the existing South Chapel Hills Drive Storm Sewer System located south of the Pine Creek Channel. All or portions of the drainage area contributing to Analysis Point 16 has been included in the "MDDP for Briargate Business Campus," the "MDDP for Village Center at Pine Creek and Preliminary/Final Drainage Report for Village Center at Pine Creek Filing No. 2 and Pine Creek Village Center Filing No. 1," the "Final Drainage Report for Chapel Hills Drive," and or the "Final Drainage Report for Briargate Parkway." This flow enters the Pine Creek Channel at Analysis Point 19 on the west side of Chapel Hills Drive.

7. Chapel Hills Drive North (Sub-basins CN1 through CN3)

Runoff from Sub-basin CN1 will be routed through existing Regional Detention Facility "A". Regional Detention Facility "A" has a planned 100-year peak inflow rate of 222 cfs, a 100-year peak outflow rate of 9 cfs, and a 100-year peak storage volume requirement of 9 acre feet. Outflow from Regional Detention Facility "A" will be routed to Pine Creek Channel in the existing North Chapel Hills Drive Storm Sewer System located in the Chapel Hills Drive right-of-way. Regional Detention Facility "A" has been designed and constructed to facilitate park uses as well as serving as a drainage facility. Regional Detention Facility "A" represents a revision to the "MDDP for Charter Greens," dated January 1993, as well as the "Final Drainage Report for Chapel Hills Drive," dated January 1997. Detailed analysis of existing Regional Detention Facility "A" was provided in the "Preliminary/Final Drainage Report for Park Site at Chapel Hills Drive and Amendment to Final Drainage Report for Chapel Hills Drive", dated December 1997.

Analysis Point 18 represents the flow collected in Chapel Hills Drive and the North Chapel Hills Drive Storm Sewer System north of the Pine Creek Channel. This flow includes the outflow from Regional Detention Facility "A" as discussed above. All or portions of the drainage area contributing to Analysis Point 18 have been included in the "MDDP for Charter Greens," dated January 1993, the "Final Drainage Report for Chapel Hills Drive," dated January 1993 and/or the "Preliminary/Final Drainage Report for Park Site at Chapel Hills Drive and Amendment to Final Drainage Report for Chapel Hills Drive," dated December 1997. This flow will enter the Pine Creek Channel just upstream of Analysis Point 19 on the west side of Chapel Hills Drive.

8. Pine Creek Main Channel (Basins PM5 through PM7)

Analysis Point 19 represents the total estimated flow from the upstream Pine Creek Channel as well as the flow from Chapel Hills Drive and associated storm sewer systems. The peak flow rates in the Pine Creek Channel at Analysis Point 19 are estimated at of Q_5 = 609 cfs and Q_{100} = 1655 cfs. This flow will be routed down the natural Pine Creek Channel to Analysis Point 19A. Runoff from Sub-basin PM6A will outfall to Pine Creek via. an existing storm sewer at 19A. The combined runoff at Analysis Point 19A (Q_5 = 641 cfs, Q_{100} = 1721) cfs will be routed down Pine Creek to Analysis Point 20 near existing Regional Detention Facility No. 1. Runoff from Sub-basin PM5 enters the Pine Creek Channel between Analysis Point 19 and Detention Facility No. 1.

The flow from Sub-basin PM5 is included with the flow in the Pine Creek Channel at Analysis Point 20. The peak flow rates in the Pine Creek Channel at Analysis Point 20 as indicated by the HEC-1 model will be Q5 = 712 cfs and Q_{100} = 1943 cfs.

Runoff from Sub-basin PM6B is planned to be collected in existing storm sewers that outfall to the Pine Creek Channel near Regional Detention Facility No. 1 or outfall directly to Detention Facility No. 1. The area included in Sub-basin PM6B was included in the approved "MDDP for Briargate Business Campus," dated October 1996. As discussed elsewhere in this study, contrary to the approved MDDP the analysis done for the Amendment 2 and the current study assumed free discharge from this sub-basin. Because some development in the sub-basin has preceded this study at least some of the constructed outfall lines from the sub-basin may not be adequate to convey free discharge from developing properties. Discharge from future development in the sub-basin should be limited only by fitting within the land use assumptions made for this current study and the availability of an adequate outfall to Pine Creek. Runoff from Sub-basin PM6 is assumed to be included in the flow in Pine Creek Channel at Analysis Point 21. The peak flow rates at Analysis Point 21 are estimated at $Q_5 = 735$ cfs and $Q_{100} = 2007$ cfs. This is the total estimated flow to Regional Detention Facility No. 1 from Pine Creek Channel.

Runoff from Sub-basin PM7 is planned to be collected and conveyed to Regional Detention Facility No.1 in a proposed storm sewer and open channel system that will originate at the intersection of Highway 83 and Springcrest Road. Free discharge was assumed from the sub-basin. Discharge from future development within the sub-basin should be limited only by fitting within the land use assumptions made for this current study and the availability of an adequate outfall to Regional Detention Facility No. 1.

For the purpose of this analysis it is assumed that all of the runoff from Sub-basin PM8, a portion of the Briargate Parkway right-of-way will be routed through Detention Facility No. 1.

9. Focus on the Family Storm Sewer System (Sub-basins F1 through F7)

The current study does not propose changes to the drainage criteria implemented with previous plans for this area. Due to the capacity limitations of the outfall line from this area onsite detention as called for in the "MDDP for Briargate Business Campus," dated October 1996, will remain a requirement for this area. As discussed in Section IV, Part A, this area was included in the current study so that hydrographs for this area could be produced with methodology consistent with the methodology applied to the remainder of the study area. These hydrographs were needed for addition to hydrographs from the remainder of the study area to evaluate the capacity of Regional Detention Facility No. 1 and the total outflow from the study area.

The more conservative hydrology methodology utilized for the current study generated 100-year storm hydrographs from portions of this area that were in excess of the existing downstream storm sewer capacity. At Analysis Point 22 the excess flow was assumed to flow out of the Pine Creek Drainage Basin into Cottonwood Creek Drainage Basin. At Analysis Point 24 the excess flow was assumed to be routed on the surface to enter Pine Creek Channel near Analysis Point 27. At Analysis Point 25 the excess flow was assumed to be routed on the surface down Briargate Parkway to enter Pine Creek Channel near Analysis Point 26. Flow within the full pipe capacity of the storm sewer system was routed within the HEC 1 model to Regional Detention Facility No. 1. The flow from the Focus on the Family storm sewer combined with the flow from Pine Creek (Analysis Point 21) and flow from Sub-basins PM7 and PM8 represents the total planned inflow to Regional Detention Facility No. 1. The existing Regional Detention No. 1 has been fitted with a modified outlet structure per the recommendations of Amendment 2 to the Pine Creek D.B.P.S. The current analysis demonstrates that the expansion of storage volume that was called for in Amendment 2 is not required due to a decrease in the intensity of development in the watershed from the previous land plan and the shifting of some detention to upstream facilities.

Regional Detention Facility No. 1 is planned to have a 100-year peak inflow rate of 2671 cfs, a 100-year peak outflow rate of 1156 cfs, and a 100-year peak storage volume requirement of 86-acre feet. Outflow from Regional Detention Facility No. 1 will be routed under existing Briargate Parkway to Analysis Point 26 in Pine Creek Channel via an existing 12' span by 10' rise concrete box culvert. At Analysis Point 26 the excess flow that was assumed to be routed in the street from Analysis Point 25 will enter Pine Creek Channel. This flow combined with the outflow from Regional Detention Facility No. 1 will result in peak flow rates estimated at $Q_5 = 463$ cfs and $Q_{100} = 1156$ cfs. The combined flow will be routed down the Pine Creek natural channel to Analysis Point 27 on the east side of Highway 83.

10. Pine Creek Main Channel (Sub-basins PM9, PM10, and PM11)

Sub-basins PM9 and PM11 will be allowed free discharge of the 100-year peak rate to Pine Creek through appropriate conveyance and outlet facilities. Free discharge of the 100-year peak rate from these areas is conducive to limiting the 100-year peak discharge in Pine Creek at Highway 83 to less than 1,210 cfs. Free discharge of the 100-year runoff will allow the bulk of the runoff from these areas to pass downstream ahead of significant discharge from upstream Detention Facility No. 1. The outlet from Detention Facility No. 1 was modified per the Amendment 2 plan to facilitate greater lag of the discharge from the facility than was provided by the facility as it was originally constructed. Due to the proximity of Sub-basins PM9 and PM11 to the discharge point of the D.B.P.S. area, limited detention of storm water from these sub-basins will be required by the City in order to mitigate local peak flows from frequent events and improve storm water quality. The detention requirements will be determined at the time of Final Drainage Report as each sub-basin develops. The facilities to accomplish the above should be designed to not significantly lag the discharge of the larger storms.

Runoff from Sub-basin PM10 is to be controlled to a maximum 100-year peak flow rate of 140 cfs as required by the Final Drainage Report for "Briargate Business Campus Filing No. 13," approved October 31, 1996.

Runoff from Sub-basin PM9 is planned to enter Pine Creek Channel upstream of Analysis Point 27 (Q_5 = 466 cfs, Q_{100} = 1170 cfs). The HEC-I Model for this study assumes that runoff from Sub-basin PM10 and PM11 will enter Pine Creek below Analysis Point 27. Analysis Point 28, at the east side of Highway 83 includes the flow from Analysis Point 27 and Sub-basins PM10 and PM11. The model predicts peak flow rates in the Pine Creek Channel at Analysis Point 28 will be Q_5 = 563 cfs, Q_{100} = 1199 cfs. This is the total planned discharge to Pine Creek from the study area.

C. Amendment No. 3 Interim Condition Drainage Plan

As shown on the map titled "Amendment No. 3 Interim Condition Basin Map and Master Plan," contained in the appendix of this report, the upstream limit of the land assumed to be fully developed in the interim plan coincides with the northeast side of proposed Powers Boulevard. It is assumed that all drainage facilities located downstream of the upper limit of development will be constructed to support the interim plan. Land located upstream of the indicated limit is considered to remain in the existing condition in the interim condition plan. Interim condition sub-basins were delineated for the interim condition analysis. The labels for these sub-basins begin with the letter "I". Assumed development in the interim condition basins was limited to the following:

- The Powers Boulevard Corridor per the current plans for the roadway
- 30-acres north of Old Ranch Road at 1 DU/ AC
- 1.5 acres of temporary gravel road to the CSU Tank site

1. Pine Creek North Fork (Sub-basins IPN1 through IPN3)

Runoff patterns in Sub-basins IPN1 through IPN3 are assumed to remain unchanged from the existing condition except for development of Powers Boulevard. The 100-peak flow rate from these sub-basins will be concentrated at Analysis Point I2 (Q_5 = 47 cfs, Q_{100} = 287 cfs) in the Pine Creek Channel. Storm sewer systems that will be required to serve future development upstream of Powers Boulevard will be constructed under Powers Boulevard both north and south of Pine Creek. These systems are not planned to convey flow in the interim condition.

2. Pine Creek North Fork (Sub-basins IPN4 through IPN7)

Runoff from Sub-basins IPN4 through IPN7 will be collected and conveyed in a storm sewer system to be constructed by CDOT with Powers Boulevard. The proposed storm sewer system will outfall to proposed Regional Detention Facility "F". Initially this system will outfall to a small water quality pond (to be constructed by CDOT) upstream of Detention Facility "F". As Detention Facility "F" is completed it is expected that the water quality pond will be eliminated and the storm water from the CDOT storm sewer system will discharge directly to Regional Detention Facility "F" at Analysis Point 3. Assuming the later condition, the combined flow rates from the CDOT system and the flow in the Pine Creek Channel at Analysis Point 3 are estimated to be Q_5 = 76 cfs and

Q_{100} = 426cfs. This is the total planned flow to the Detention Facility "F" rundown channel in the interim condition.

Downstream the plan is unchanged from the plan presented for the fully developed condition with the exception that peak flow rates in the major facilities are slightly less than for the fully developed condition. Estimated peak flow rates are shown on the "Interim Condition Basin Map and Master Plan."

3. Pine Creek South Fork (Sub-basins IPS1 through IPS7)

Runoff patterns in Sub-basins IPS1 through IPS4 are assumed to remain unchanged from the existing condition. Runoff from Basins IPS1 and IPS2 is expected to be collected in the storm sewer system to be constructed in Briargate Parkway with Powers Boulevard at Analysis Point I6 (Q_5 = 21 cfs, Q_{100} = 129 cfs). The runoff from Sub-basins IPS3 through IPS5 will be collected and conveyed in a roadside ditch to be constructed along the eastern edge of the Powers Boulevard improvements to Analysis Point I8 (Q_5 = 36 cfs, Q_{100} = 220 cfs). This flow will be then be conveyed from Analysis Point I8 to Analysis Point 6 on the proposed Briargate Parkway storm sewer system in a storm sewer to be constructed with Powers Boulevard.

Runoff from Sub-basin IPS6 will also be collected and conveyed to Analysis Point 6 via. a roadside ditch to be constructed along the eastern side of the proposed Powers Boulevard improvements and a storm sewer system to be constructed with proposed Powers Boulevard. The HEC 1 model indicates peak flow rates in the Briargate Parkway storm sewer at Analysis Point 6 will be Q_5 = 70 cfs and Q_{100}= 429cfs. This flow will be routed in the Briargate Parkway storm sewer to Analysis Point 6A where it will be combined with the runoff from Sub-basin IPS7 then routed on downstream in the storm sewer system to Analysis Point 6B. The peak flow rates associated with Analysis Point 6A are estimated at Q_5 = 77 cfs and Q_{100} = 466 cfs.

Downstream the plan is unchanged from the plan presented for the fully developed condition with the exception that peak flow rates in the major facilities are slightly less than for the fully developed condition. Estimated peak flow rates are shown on the "Interim Condition Basin Map and Master Plan."

D. Major Proposed Facilities

1. Storm Sewers

Estimated required storm sewer sizes are indicated on the Maps titled "Basin Map and Master Plan," contained in the appendix of this study. Design of these storm sewers should include a detailed hydraulic analysis and sizes should be adjusted as required. Special attention should be given to the hydraulic grade line near the outlets of detention facilities to assure that backwater in the outfall lines will not interfere with the planned stage/discharge relationship.

2. Detention Facilities

a. General Design Criteria

Design and construction of detention facilities proposed by this plan shall conform to the requirements of the City of Colorado Springs and the State Engineer. To the extent practical the detention facilities shall be recessed into the ground rather than created behind large unarmored embankments. To the extent practical the detention facilities shall be located on the upstream side of street crossings and shall utilize the roadway embankments as dams. The general design criteria for the detention facilities shall include the following:

The 100-year maximum water surface design elevation shall not exceed the height of the emergency spillway with the normal outlet operating normally.

- Each detention facility shall be fitted with an armored emergency spillway capable of passing the full 100-year peak inflow rate. In the case of Regional Detention Facility "E" located downstream of Regional Detention Facility "F" the emergency spillways shall be capable of passing the highest inflow rate associated with the proposed detention facilities located upstream.

- The emergency spillways shall be oriented to direct flow in a manner that will minimize the potential for property damage and threat to human safety downstream should a spill occur. In the case of Detention Facilities "E", and "F" the emergency spillways should be oriented to pass overflow to downstream Pine Creek Channel. Sufficient capacity should be maintained in the Pine Creek Channel to allow the design overflow to pass without damage to structures. In the case of Detention Facilities "B", "C", "D1" and "D2" the emergency spillways should be oriented to pass overflow to the adjacent Briargate Parkway right-of-way. The potential for a large flow to occur down Briargate Parkway should be considered in the design of the roadway and adjacent development. Future detention facilities in Sub-basins PNE1 and PNE2 should be designed to overflow to Reach 9 of the Pine Creek channel.

- At least 2 feet of freeboard shall be provided above the water surface associated with the normal outlet clogged and the emergency spillway passing the full 100-year peak inflow rate.

b. Plan Assumptions for Individual Detention Facilities

The following assumptions were utilized in the hydrologic modeling performed in the preparation of the plan. If the final design of these detention facilities deviates from these assumptions the changes should be modeled in the overall study done for this plan to verify that the changes do not negatively impact downstream facilities or planned peak flow rates downstream.

- **Regional Detention Facility No. 1**

The modeled volume was based on a 2001 aerial survey by Aero-Metrics. The modeled outflow was based on the existing outlet that was recently modified per the recommendations contained in Amendment 2 to the Pine Creek D.B.P.S. The existing outlet is staged. The lowest opening consists of the bottom 2.5' of the box culvert. The remainder of the upstream end of the existing box culvert is blocked. The upper opening of the outlet is contained in a reinforced concrete structure that is constructed on top of and discharges through the top of the box culvert. The upper outlet was modeled as a sharp crested weir with a crest elevation of 6567.2. The upper opening effective length was assumed to be equal to 12.8' (the weir length adjusted for edge contractions). At elevation 6573.0 the back and sidewalls of the upper outlet structure terminate. Discharge will flow over all walls of the structure above the terminal elevation. The tops of the upper walls were as modeled a sharp-crested weirs.

The current HEC-1 Model predicts a maximum 100-year water surface elevation of 6573.3 in the 100-year design storm. This maximum water surface is 1.7 +/- feet lower than the existing emergency spillway crest for the facility.

MODIFIED DETENTION FACILITY NO.1

Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Outflow (cfs)
54	0	0
56	0.02	184
58	2.73	261
60	11.10	326
62	20.65	380
64	30.85	427
66	41.65	470
68	53.04	532
70	65.06	718
72	77.78	969
74	91.03	1,264
75	98.41	1,750

Normal Outlet Staged

Low Stage: 12.1' Wide X 2.50' High Vertical Orifice, Invert = 6553.00
High Stage: 12.8' Wide Weir, Invert = 6567.20

- **Regional Detention Facility "A"**

The stage storage discharge curve is based on the design drawings for the proposed facility. The bottom of the pond is staged in order to maintain certain portions of the pond bottom dry in frequent rainfall events in order to facilitate park uses.

DETENTION FACILITY "A"

Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Outflow (cfs)
796.6	0	2.35
797.0	0.01	2.54
798.0	0.22	3.00
800.00	0.99	3.73
802.0	1.95	4.35
803.5	2.80	4.75
803.51	4.25	5.36
804.0	5.31	5.50
804.1	6.51	8.39
805.5	11.64	9.01
806.5	15.36	9.41

Normal Outlet: 12" Diameter Storm Sewer

Normal Outlet Invert Elevation: 95.0

- **Regional Detention Facility "B"**

The modeled volume is based on an as-built survey dated November 1999 that was prepared for the facility. This facility was originally constructed with more volume than required in the fully developed condition in order to facilitate to obtaining a flood plain development permit for construction of the facility before a CLOMR was approved for the area. With the current plan, the existing outlet will be restricted by installing a steel plate over the top 4.2 square feet of the 54" diameter outlet opening. This will take advantage of the additional capacity in the existing pond and is one of the things that eliminate the need to expand the storage volume of Regional Detention Facility No. 1 under the current drainage master plan.

DETENTION FACILITY "B"

Stage Storage Discharge Data

As-built Volume

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
71.2	0	0
72.0	0.06	20
74	0.66	86
76	2.51	117
78	5.08	142
80	8.05	163
82	11.42	181
84	15.22	198
86	19.49	213
87.6	23.24	225
88	24.23	289
90	29.50	1133

Normal Outlet: 54" dia storm sewer with top 4.2 s.f. blocked

Normal Outlet Invert Elevation: 69.9

- **Regional Detention Facility "C"**
  The modeled volume is based on an updated plan prepared to provide additional storage volume for flow from the increased area of Sub-Basin PS-10, dated 10/30/01.

DETENTION FACILITY "C"
Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
62	0	0
63	0	6
64	1.1	23
66	7.7	70
68	16.9	110
70	26.9	140
72	37.7	168
74	49.2	190
76	61.5	215
78	74.5	232
80	88.4	245

Normal Outlet: 48" dia storm sewer
Normal Outlet Invert Elevation: 62.0

- **Regional Detention Facility "D1"**

DETENTION FACILITY "D1"
Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
100	0	0
102	1.3	37
104	2.9	53
106	5.2	65
108	8.9	75
110	14.1	84
112	20.9	92
114	29.5	100

Normal Outlet: 32" dia. Orifice
Normal Outlet Invert Elevation: 98.7

- **Regional Detention Facility "D2"**

DETENTION FACILITY "D2"
Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
100	0	0
102	0.6	26
104	1.9	38
106	3.5	46
108	5.4	54
110	7.6	60
112	10.1	66

Normal Outlet: 27" dia. Orifice
Normal Outlet Invert Elevation: 99.0

- **Regional Detention Facility "E"**

DETENTION FACILITY "E"
Stage Storage Discharge Data
As-built Volume

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
784	0	0
786	0.3	26
788	2.0	80
790	4.9	133
792	8.3	173
794	12.0	208
796	16.1	238
798	20.6	260
800	25.5	278
802	30.9	1441

Normal Outlet: 54" dia storm sewer
Normal Outlet Invert Elevation: 84.0

- **Regional Detention Facility "F"**

DETENTION FACILITY "F"
Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
13	0	5
14	0.3	30
16	3.7	93
18	10.0	122
20	17.3	146
22	25.2	166
24	33.8	184
26	43.2	201
28	53.6	216
30	65.0	231

Normal Outlet: 54" dia. Orifice with top 4.2 s.f. of opening blocked
Normal Outlet Invert Elevation: 11.5

- **Detention Facility "NE1"**

DETENTION FACILITY "NE1"
Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
49	0	0
50	0	4
52	0.7	35
54	1.5	60
56	2.7	81
58	4.0	92
60	5.6	103
62	7.5	114

Normal Outlet: 36" dia. storm sewer
Normal Outlet Invert Elevation: 49.0

- **Detention Facility “NE2”**

DETENTION FACILITY “NE2”
Stage Storage Discharge Data

Water Surface Elevation (Feet)	Cumulative Storage Volume (AC/FT)	Normal Outlet Discharge (cfs)
21	0	0
22	0	5
24	1.1	41
26	2.5	85
28	4.2	104
30	6.1	121
32	8.4	138
34	11.1	155

Normal Outlet: 42” dia. storm sewer
Normal Outlet Invert Elevation: 21.0

c. Regional Detention Facility Maintenance

The eight Regional Detention Facilities that exist or are proposed in this document are all proposed to be publicly owned and publicly maintained for functional purposes. Any aesthetic maintenance beyond the City’s maintenance will be by and totally at the expense of others and will require an agreement with the City.

3. Pine Creek Channel

a. General

As discussed elsewhere in this report the character of the Pine Creek Channel varies considerably throughout the study area. Portions of the channel are well defined, well vegetated, and are aligned in a manner that allows reasonable development of adjacent land. Other portions are not well defined, lack significant vegetation, lack adequate conveyance capacity, and or are not aligned in a manner that allows reasonable development of adjacent properties. The plan for the majority of Pine Creek Channel between Powers Boulevard and Highway 83 is to preserve it at as a natural channel or a natural channel with some man made stabilization that will serve as the major drainage conveyance. In other portions of the study area storm sewers have been constructed to serve as the major drainage conveyances and the natural channel has been graded to only convey drainage in runoff events that are larger than the 100 year design event.

b. Individual Reach Discussion

The following is a brief discussion of Pine Creek Channel reaches PC1 through PC9 as shown on the maps titled "Basin Map and Master Plan," contained in the appendix of this report.

- **Reach PC1**

Due to concerns about the presence of habitat of the Prebles Meadow Jumping Mouse the channel in this reach will be preserved in its natural condition except for one drop/grade control structure that has been constructed just upstream of Highway 83 and energy dissipaters at the ends of future storm sewer outfalls. JR Engineering has prepared a separate study that assesses the stability of the natural channel as additional development occurs upstream. The current plan is for this reach of channel to be publicly owned and maintained by the City of Colorado Springs.

- **Reach PC2**

This reach is well vegetated and appears to be quite stable at the current time. The only treatment currently proposed for this reach is one minor grade control structure in the low flow channel and some minor modifications to at the riprap inlet channel at Detention Facility No. 1. The minor grade control structure has been constructed. As mentioned in the discussion for Reach PC1, JR Engineering has prepared a separate study that assesses the stability of the natural channel as additional development occurs upstream. The current plan is for this reach of channel to be publicly owned and maintained by the City of Colorado Springs.

- **Reach PC3**

It is expected that treatment in this reach will be limited to energy dissipaters at the outfalls of storm sewers contributing to the channel and potentially minor bank and channel stabilization. The current plan is for this reach of channel to be publicly owned and maintained by the City of Colorado Springs. A detailed Hydraulic Analysis of Reach

PC3 is currently underway by JR Engineering. The results of this analysis will be published in a separate report.

- **Reach PC4**

This reach was historically contained in a valley floor alluvial fan and was characterized by multiple, ill-defined flow paths lacking significant vegetation, natural stability, and adequate conveyance capacity. Due to this an underground storm sewer was constructed to convey up to the 100-year planned discharge through this reach per the recommendations of "Amendment 2". In addition the surface corridor above the storm sewer is graded into a broad swale recessed below the adjacent development. This swale provides an emergency relief channel for the storm sewer and the detention facilities that exist or will be constructed upstream. In keeping with the proposed design criteria for the proposed detention facilities, the swale was designed to allow passage of the largest peak 100-year inflow rate of the facilities to be located upstream. The City of Colorado Springs will be responsible for the maintenance of the proposed storm sewer.

- **Reaches PC5 through PC6**

These reaches are generally well defined and contain some natural vegetation to aid in their stability. However, given the relatively steep natural slopes of these reaches and the lack of heavy vegetation it is anticipated that these reaches will require construction of grade control in order to allow them to convey developed condition flows. The current plan proposes 100-year peak flow rates in these reaches that are similar or lower than historic 100-year flow rates. However, peak flow rates in smaller more frequent events will be increased and the frequency of flows in the channel will be much greater that in the existing condition when the contributing watershed is developed. Development of the watershed will also reduce the sediment inflow into the channel. These factors will increase the potential for erosion of the channel. A detailed Hydraulic Analysis of Reaches PC5 through 6 is currently underway by JR Engineering. The purpose of the analysis is to define areas that will require treatment and to recommend improvements. The results of this analysis will be published in a separate report.

- **Reaches PC7 through PC8**

These reaches are generally well defined and contain some natural vegetation to aid in their stability. However, given the relatively steep natural slopes of these reaches and the lack of heavy vegetation it is anticipated that these reaches would not remain stable if exposed to the increased frequency, volumes and low sediment content of developed condition flows. Given this and the Federal requirement to minimize impacts to the natural channel due to the presence of Prebles Meadow Jumping Mouse Habitat, these reaches will only convey significant developed condition storm water flows in storms larger than the 2 year frequency storm per the current proposed plan. Runoff from more frequent storms and significant portions of the runoff from larger storms will be conveyed in a storm sewer to be constructed parallel to the Creek and outside of the Prebles habit to the extent possible. A trickle flow will be directed through the reach in all runoff events to encourage the growth of vegetation in the reach. This diversion of flows will be facilitated by a diversion structure to be constructed at the upstream end of reach PC8.

It is believed that the channel can convey the infrequent flows as proposed by the current plan in its current natural state without significant risk of uncontrolled erosion. These reaches are included in the detailed hydraulic analysis discussed in previous text for the downstream reaches. The stability of the reaches will be examined in the analysis.

- **Reach PC9**

This majority of this reach can be better described as a wide shallow swale than a well-defined incised channel. Only the upper portion of the channel is incised and exhibits signs of head cutting in the existing condition. The current land plan for the area proposes to preserve the reach as a natural drainage conveyance. Given the relatively steep gradients and the lack of a defined channel in the reach, it is not likely that the reach can serve as the primary conveyance for developed condition flows without significant reshaping and the addition of significant grade control measures.

As both of the above mentioned treatments are contrary to maintaining the area in a relatively natural state and costly, the current plan proposes to shelter the reach from frequent flows and only utilize it to convey a portion of the flows in larger event. This is the same concept presented for downstream Reaches 7 and 8. The current plan recommends that flows below the expected 5-year peak rate be conveyed in a storm sewer constructed parallel to the reach. A portion of the flow from larger runoff events will be conveyed as shallow flow through the reach. It is recommended that the incised upper portion of the reach be regraded to a wide shallow swale and vegetated with native grasses and shrubs. The current plan will shelter the reach from the normal increase in frequency and volume of runoff that accompanies development of the upstream watershed. It is believed that the existing stability of the reach can be maintained under this plan. Planning and design associated with this reach should be done with consideration that this reach will also serve as the emergency relief channel for the upstream watershed.

E. Proposed Drainage Discharge Constraints

The following discharge constraints are proposed for the study area:

a. The requirement for onsite detention to achieve a 35 percent reduction in the peak flow rate resulting from development (the difference between the historic and developed peak rates) on all office, research and development, commercial, and school properties as implemented with the original DB.P.S. will remain in effect for all existing developed properties and for future developing properties within Basins CS2, CS3, F1, F4, F5, F6, F7, PM6B and PM10 as shown on the Fully Developed Condition Drainage Map included in this study unless the following conditions are met.

- A separate detailed drainage analysis or the analysis done for this study demonstrates that the downstream existing or proposed drainage conveyance facilities are adequate to allow a greater discharge rate from the property.
- A detailed drainage analysis or the analysis performed for this study demonstrates that the greater discharge rate will not negatively impact downstream detention facilities or the overall discharge peak discharge goals of this study.

b. Runoff from Basin CS4 as shown on the Fully Developed Condition Drainage Map included in this study shall be routed through the pond labeled as "DFVC" a private detention pond as proposed in the approved "Master Development Drainage Plan for Village Center at Pine Creek and Preliminary /Final Drainage Report for Village Center at Pine Creek Filing No. 2 and Pine Creek Village Center Filing No. 1," by JR Engineering, Ltd., dated February 11,1998 unless the following conditions are met:

- A detailed drainage analysis demonstrates that the downstream existing or proposed drainage conveyance facilities are adequate to allow a greater discharge rate from the drainage basin.
- A separate detailed drainage analysis or the analysis performed for this study demonstrates that the greater discharge rate will not negatively impact downstream detention facilities or the overall discharge peak discharge goals of this study.

c. Free discharge of the 100-year runoff from Sub-basins PM9 and PM11 will be allowed provided that the following criteria is followed:

- Adequate downstream conveyance facilities exist or will be provided in accordance with City of Colorado Springs policy and criteria.
- Land uses must be similar or less intensive than the land uses assumed for the purpose of this study unless a detailed drainage analysis indicates that free discharge from the more intensive land use will not have an adverse affect on the downstream drainage facilities.

Due to the proximity of Sub-basins PM9 and PM11 to the discharge point of the D.B.P.S. area, limited detention of storm water from these sub-basins will be required by the City in order to mitigate local peak rates from frequent runoff events and or improve the storm water quality. The detention requirement will be determined at the time of Final Drainage Report as each sub-basin develops. Facilities to accomplish the above should be designed to not significantly lag the discharge of the larger storms.

d. The 100-year peak discharge from Sub-basins PNE1, PNE2 AND PNE6 will be limited approximately 1.17 cfs/acre. To the extent practical, this should be accomplished in a single regional detention facility.

e. Free discharge of drainage from the remainder of the study area will be allowed provided that the following criteria is followed:

- Adequate down stream conveyance facilities must exist or be provided in accordance with City of Colorado Springs policy and criteria.
- Runoff must be routed through the regional detention facilities as proposed in this study unless a detailed drainage study demonstrates the adequacy of alternative routing to achieve the discharge goals of this study.
- Land uses must be similar or less intensive than the land uses assumed for the purpose of this study unless a detailed drainage analysis indicates that free discharge from the more intensive land use will not have an adverse affect on the downstream drainage facilities.

F. Recommendations for Implementation

The portion of the Pine Creek drainage basin located east of Highway 83 is considered a closed basin thus; the developer of the properties within the basin is responsible for constructing the drainage improvements related to development within the basin. Construction of required drainage improvements should be timed to coincide with or precede construction of the development that the improvement will support. Several major proposed facilities are identified on the Interim Drainage plan included in this study. A summary of these major proposed facilities and the development that the improvements will be required for follows:

- Completion of Regional Detention Facility "C" as a detention pond and the associated inflow collection system will be required to support future development in Sub-basins PS2 through PS10 as well as the portions of Powers Boulevard located in Sub-basins IPS7 and IPS6. The pond as it currently exists (retention) is adequate to support the development of the first phase of Pine Creek Filing 16 now under construction.

- The Briargate Parkway storm sewer system will replace the South Fork of Pine Creek and thus will need to be in place downstream prior the filling of the existing channel that is to be eliminated. A FEMA CLOMR has been issued for the elimination of the FEMA 100 year Flood zone associated with the Pine Creek South Fork located upstream of Detention Facility "B". The CLOMR and the associated application submittal identify the facilities required to be in place prior to the construction of the Union Boulevard and Briargate Parkway embankments across the Pine Creek South Fork. The CLOMR may need to be updated dependent upon the phasing of improvements in the area.

- Regional Detention Facility "F" and the associated inflow collection system and outfall storm sewer will be required to support development in PN7 and PN8.

- Pine Creek Channel Stabilization in Reaches PC5 and PC6 as determined by the analysis currently underway by JR Engineering should be constructed prior to or concurrent with the initial future development within Sub-basins PN7, PN8 and PN9 that contributes to the reach.

- Pine Creek Channel Stabilization in Reach PC3 as determined by the analysis currently underway by JR Engineering should be constructed concurrent with or within one year of the start of significant upstream development that contributes to the reach.

- Additional storm sewers shall be constructed as needed to provide collection systems and outfalls for individual development or prior to pavement construction in the roadways in which they are to be constructed.

- The proposed restriction of the Regional Detention Facility "B" outlet should be constructed concurrent with the connection of Detention Facility "C" to Detention Facility "B".

- The interim plan assumes Powers Boulevard and associated storm sewer improvements will be in place at the time that properties between Union Boulevard and Powers Boulevard are developed. If this is not the case, temporary diversions and conveyances may be needed to protect the area from off-site flows. This should be analyzed on a case-by-case basis as development occurs.

G. Requirements of Governmental Agencies Outside of the City of Colorado Springs

Several governmental agencies external to the City of Colorado Springs will have involvement in the review and approval process for individual construction projects proposed for the study area.

- The Federal Emergency Management Agency has jurisdiction over development within the regulatory 100-year floodplain. The developer will be required to obtain Letter of Map Revisions for modifications that the proposed development will make to the floodplain within the study area.

- The Prebles Meadow Jumping Mouse is currently listed as a threatened species by the U.S. Fish and Wildlife Service. Portions of the study area contain habitat for the mouse. A Habitat Conservation Plan (HCP) for the Briargate Development located upstream of Chapel Hills Drive is currently under review by the Service. Approval of the HCP is expected in early 2003.

- The U.S. Army Corps of Engineers has jurisdiction over development within or modifications to features defined as "waters of the United States." Some or potentially all of the modifications proposed to the Pine Creek Channel may require permitting by the U.S. Army Corps of Engineers. A permit was issued in January 2001 for improvements to Pine Creek downstream of Chapel Hills Drive. Upon the approval of the above mentioned HCP an application will be submitted to the Corps for the proposed disturbances to waters located upstream of Chapel Hills Drive.

- The office of the State Engineer has jurisdiction over many of the dams in the State. Depending upon final design, configurations of the proposed Regional Detention Facilities some may be "Jurisdictional Dams," and may be "exempt" or "nonexempt" from the rules of the State Engineer. All of the facilities constructed to date have been determined to be non-jurisdictional. A representative of the State Engineer has reviewed the draft plans for proposed Detention Faculty "F" and has indicated that the facility will be non-jurisdictional. Future proposed Facilities should be evaluated on an individual basis at the time of design.

\hw\8716-11\Ammended DBPS Oct 19-98

REFERENCES

1. "City of Colorado Springs/County of El Paso Drainage Criteria Manual," dated November 1991.

2. Soils Survey of El Paso County Area, Colorado Soil Conservation Service.

3. "Pine Creek Drainage Basin Planning Study," by Obering, Wurth & Associated, dated June 1988, revised October 1988.

4. "Amendment No. 1 to Pine Creek Drainage Basin Planning Study," by Obering, Wurth & Associated, dated July 29,1992.

5. "Amendment No. 2 to Pine Creek Drainage Basin Planning Study and Master Development Drainage Plan for Pine Creek Subdivision", by JR Engineering, October 1998.

6. "Master Development Drainage Plan for Briargate Business Campus in Pine Creek Basin," by JR Engineering, Ltd., dated October 1996.

7. "Master Development Drainage Plan for Summerfield at Briargate," by JR Engineering, Ltd., dated March 1993.

8. "Master Development Drainage Plan for Charter Greens," by JR Engineering, Ltd., dated November 1992, revised January 1993.

9. "Master Development Drainage Plan for Village Center at Pine Creek and Preliminary/Final Drainage Report for Village Center at Pine Creek Filing No.2 and Pine Creek Village Center Filing No.1," by JR Engineering, Ltd., dated February 1998

10. "Preliminary/Final Drainage Report for Park Site at Chapel Hills Drive and Amendment to Final Drainage Report for Chapel Hills Drive," by JR Engineering, Ltd., dated December 1997

11. "Flood Insurance Rate Study for El Paso County, Colorado and Incorporated Areas," Federal Emergency Management Agency, revised March 17, 1997.

12. "Master Plan, Pine Creek at Briargate," by Downing Thorpe James, drafts dated August 1, 1997.

13. "Gatehouse Neighborhood Plan," by N.E.S. Inc., dated April 1997.

14. "Summerfield Neighborhood Plan," by N.E.S. Inc., dated April 1997.

15. "Johnson Ranch at Briargate, Conceptual Land Use Plan", by LVA., dated August 2001.

16. "Briargate Business Campus," a Land Use Plan, by N.E.S. Inc., dated April 1997.

17. "Final Drainage Report for Pine Creek Channel-Phase I, (from Pond No. 1 to Chapel Hills Drive)," by JR Engineering, Ltd., dated February 1997 (not yet approved).

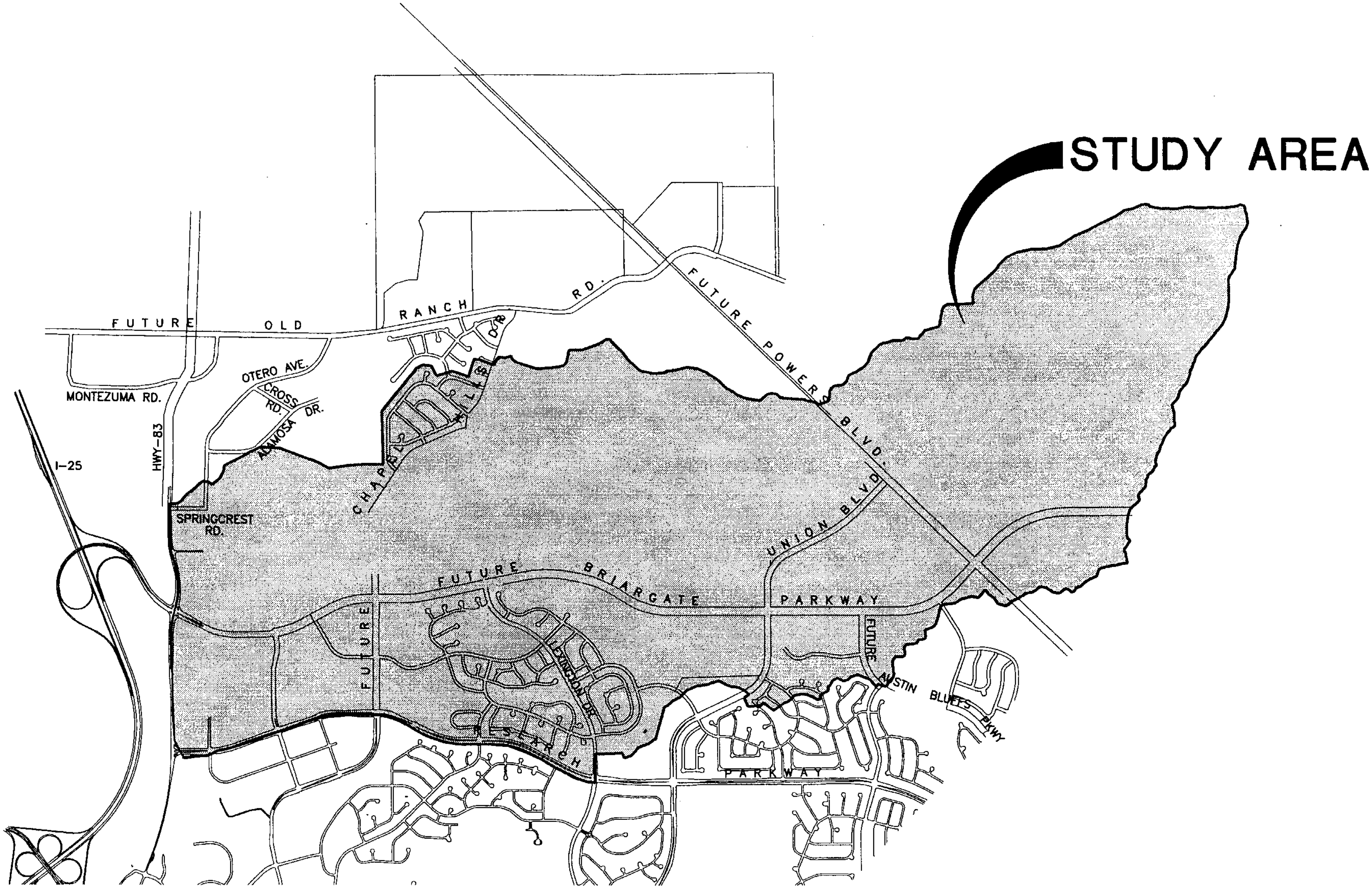
18. "Final Drainage Reports for Pine Creek Filings 3, 4A, B &C, 5, 7A & B, 8, 9, 11, 12, 13, 15, 16, 17, 25, 26, 30, 32, 33, 35 and 36", by JR Engineering, various dates between 1997 and 2002.

19. "HEC-1 Flood Hydrographic Package Users Manual," U.S. Army Corps of Engineers, dated September 1990.

APPENDIX

A.

VICINITY MAP



VICINITY MAP
1" = 3000'

B.

HYDROLOGIC MODEL INPUT CALCULATIONS

- CURVE NUMBERS
- CURVE NUMBER ADJUSTMENT
- LAG TIME

AMMENDMENT No. 3 TO
PINE CREEK DRAINAGE BASIN PLANNING STUDY

FULLY DEVELOPED CONDITION CURVE NUMBERS

9/17/2002

	SUB AREA ONE				SUB AREA TWO				SUB AREA THREE				SUB AREA FOUR				SUB AREA FIVE							
SUB-BASIN LABEL	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	TOTAL AREA AC.	TOTAL AREA S.M.	WEIGHTED CN	WEIGHTED PERCENT INPERVIOUS
CN1	SCHOOL	50.0	84.0	27.0	3 DU/AC	30.0	72.0	12.7	4 DU/AC	40.0	76.0	22.0	M.A. STREET	85.0	93.0	4.0	DET/PARK		64.0	5.0	70.7	0.110	78.4	41.7
CN2	3 DU/AC	30.0	72.0	47.0	M.A. STREET	85.0	93.0	3.0													50.0	0.078	73.3	33.3
CN3	3 DU/AC	30.0	72.0	13.0	5 DU/AC	44.0	77.0	9.5	M.A. STREET	85.0	93.0	5.0									27.5	0.043	77.5	44.8
CS1	SCHL/LNDSCP	20.0	68.0	11.0	3 DU/AC	30.0	72.0	19.6	PARK	10.0	65.0	3.3									33.9	0.053	70.0	24.8
CS2	OPEN SPC.		69.0	1.5	3-4 DU/AC	33.0	73.0	2.0	LI/O	83.0	92.0	39.0	M.A. STREET	85.0	93.0	2.5					45.0	0.070	90.4	78.1
CS3	OPEN SPC.		69.0	1.4	3 DU/AC	30.0	72.0	8.2	LI/O	83.0	92.0	9.0	M.A. STREET	85.0	93.0	12.7	COM	95.0	96.0	1.2	32.5	0.051	86.5	67.3
CS4	COM.	86.0	94.0	12.0	6 DU/AC	56.0	82.0	8.6	OPEN SPC		69.0	2.0	10-16 DU/AC	60.0	83.0	19.7					42.3	0.066	85.3	63.7
F1	COM	62.0	84.0	16.0	SCHOOL	40.0	76.0	4.5	4 DU/AC	37.0	76.0	50.5	WATER TNK	68.0	86.0	5.0					76.0	0.119	78.3	44.5
F2	3 DU/AC	30.0	72.0	20.0	OPEN SPC		69.0	5.0													25.0	0.039	71.4	24.0
F3	3 DU/AC	30.0	72.0	60.0	M.A. STREET	85.0	93.0	8.5	CHURCH	80.0	91.0	4.5									73.0	0.114	75.6	39.5
F4	LI/O	65.0	85.0	11.5	OPEN SPC		69.0	3.5	M.A. STREET	85.0	93.0	7.5	3 DU/AC	30.0	72.0	2.0					24.5	0.038	84.1	59.0
F5	LI/O	65.0	85.0	35.0	M.A. STREET	85.0	93.0	6.0													41.0	0.064	86.2	67.9
F6	LI/O	70.0	87.0	21.5	M.A. STREET	85.0	93.0	3.0													24.5	0.038	87.7	71.8
F7	LI/O	75.0	89.0	29.0	M.A. STREET	85.0	93.0	4.5													33.5	0.052	89.5	76.3
PM1	SCHOOL	50.0	84.0	25.0	3 DU/AC	30.0	72.0	9.5													34.5	0.054	80.7	44.5
PM2	GOLF CRS	2.0	63.0	90.9	3DU/AC	30.0	72.0	9.0	2DU/AC	25.0	70.0	19.6									119.5	0.187	64.8	7.9
PM3	MULTI FAM.	70.0	87.0	8.5	3 DU/AC	30.0	72.0	1.4	OPEN SPC		69.0	27.3									37.2	0.058	73.2	17.1
PM4	GOLF CRS		61.0	23.9	3 DU/AC	30.0	72.0	47.0													70.9	0.111	68.3	19.9
PM5	GOLF CRS		61.0	49.0	2 DU/AC	25.0	70.0	59.0	OPEN SPC		69.0	9.5	LI/O	83.0	92.0	5.8					123.3	0.193	67.4	15.9
PM6A	LI/O	70.0	87.0	27.0																	27.0	0.042	87.0	70.0
PM6B	HOTEL/O	80.0	91.0	23.2																	23.2	0.036	91.0	80.0
PM7	CHURCH	80.0	91.0	15.9	SCHOOL	50.0	84.0	9.9	1 DU/AC	20.0	68.0	53.7	LI/O	83.0	92.0	8.5					88.0	0.138	76.3	40.3
PM8	M.A. STREET	85.0	93.0	8.7																	8.7	0.014	93.0	85.0
PM9	OPEN SPC.		69.0	13.5	COM	80.0	91.0	30.0													43.5	0.068	84.2	55.2
PM10	LI/O	83.0	92.0	31.0																	31.0	0.048	92.0	83.0
PM11	LI/O	83.0	92.0	17.0	M.A. STREET	85.0	93.0	10.0													27.0	0.042	92.4	83.7
PN7	3 DU/AC	30.0	72.0	31.5	OPEN SPC.		69.0	11.4	STREET	85.0	93.0	2.7									45.6	0.071	72.5	25.8
PN8	STREET	85.0	93.0	1.0	COM.	90.0	94.0	16.5	OPEN SPC.		69.0	5.6									23.1	0.036	87.9	68.0
PN9	STREET	85.0	93.0	1.8	2.5 DU/AC	27.3	70.0	16.8	OPEN SPC.		69.0	44.2	6DU/AC	56.0	82.0	7.6					70.4	0.110	71.3	14.7
PN11	3 DU/AC	30.0	72.0	33.0	M.A. STREET	85.0	93.0	9.6	SCHOOL	50.0	84.0	10.8									53.4	0.083	78.2	43.9
PN12	2.5 DU/AC	27.5	70.0	39.5	OPEN SPC.		69.0	25.1													64.6	0.101	69.6	16.8
PN13	3 DU/AC	30.0	72.0	8.8	2 DU/AC	25.0	68.0	4.0	GOLF CRS.		61.0	16.3									29.1	0.045	65.3	12.5
PN15	3 DU/AC	30.0	72.0	33.2	OPEN SPC.		69.0	10.8													44.0	0.069	71.3	22.6
PS2	M.A. STREET	85.0	93.0	3.5	MULTI FAM	70.0	87.0	11.7													15.2	0.024	88.4	73.5
PS3	STREET	80.0	91.0	5.2	COM	90.0	94.0	38.0	OPEN SPC.		69.0	1.9									45.1	0.070	92.6	85.1
PS4	M.A. STREET	85.0	93.0	7.4	4 DU/AC	37.0	76.0	27.3	OPEN SPC.		69.0	3.5									38.2	0.060	78.7	42.9
PS5	M.A. STREET	85.0	93.0	3.4	COM.	90.0	94.0	15.3	OPEN SPC.		69.0	0.8									19.5	0.030	92.8	85.4
PS6	STREET	80.0	91.0	2.2	COM.	90.0	94.0	31.8													34.0	0.053	93.8	89.4
PS7	M.A. STREET	85.0	93.0	6.7	COM.	90.0	94.0	12.7	OPEN SPC.		69.0	0.7									20.1	0.031	92.8	85.2
PS8	M.A. STREET	85.0	93.0	9.1	COM.	90.0	94.0	26.7	OPEN SPC.		69.0	8.0	4 DU/AC	37.0	76.0	27.6					71.4	0.112	84.1	58.8
PS9	M.A. STREET	85.0	93.0	5.8	COM.	90.0	94.0	17.0	4DU/AC	37.0	76.0	12.0									34.8	0.054	87.6	70.9
PS10	OPEN SPC.		69.0	13.5	4.5 DU/AC	40.0	76.0	20.1													33.6	0.053	73.2	23.9
PS11	4 DU/AC	30.0	72.0	23.9	M.A. STREET	85.0	93.0	10.8													34.7	0.054	78.5	47.1
PS12	PARK/O.S.		67.0	82.2	4 DU / AC	37.0	76.0	3.0	SCHOOL	50.0	84.0	6.0	LI/O	83.0	92.0	6.8					98.0	0.153	70.0	9.9
PS13	3 DU/ AC	30.0	72.0	7.1	PARK/ OS		67.0	25.0	M.A. STREET	85.0	93.0	9.8									41.9	0.065	73.9	25.0
																			TOTAL		2049.9	3.203		

AMMENDMENT No. 3 TO
PINE CREEK DRAINAGE BASIN PLANNING STUDY

EAST OF POWERS BLVD
FULLY DEVELOPED CONDITION CURVE NUMBERS

9/17/2002

	SUB AREA ONE				SUB AREA TWO				SUB AREA THREE				SUB AREA FOUR				SUB AREA FIVE							
SUB-BASIN LABEL	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	TOTAL AREA AC.	TOTAL AREA S.M.	WEIGHTED CN	WEIGHTED PERCENT INPERVIOUS
PNE1	UNKNOWN	45.0	77.0	63.5	2 DU/AC	25.0	68.0	19.3	COM.	95	96	1.8	7 DU/AC	60.5	83.5	0.6	PARK		67.0	0.9	86.1	0.135	75.3	41.2
PNE2	UNKNOWN	45.0	77.0	51.1	4 DU/AC	37.0	76.0	25.6	2 DU/AC	25.0	68.0	13.8	7 DU/AC COM.	69.0	86.0	12.7	PARK		67.0	6.1	109.3	0.171	76.1	40.9
PNE3	M.A. STREET	85.0	93.0	5.3	2.5 DU/AC	27.3	70.0	2.8													8.1	0.013	85.0	65.1
PNE4	M.A. STREET	85.0	93.0	6.1	4 DU/AC	37.0	76.0	13.2	2.5 DU/AC	27.5	70.0	12.8	2 DU/AC	25.0	68.0	16.8					48.9	0.076	73.8	36.4
PNE5	OPEN SPC.		69.0	11.6																	11.6	0.018	69.0	0.0
PNE6	UNKNOWN	45.0	77.0	12.6																	12.6	0.020	77.0	45.0
PNE7	M. STREET	85.0	93.0	6.3	2 DU/AC	25.0	68.0	25.9	2.5 DU/AC	27.5	70.0	26.6	3 DU/AC	30.0	72.0	7.0					65.8	0.103	71.6	32.3
PNE8	M. STREET	85.0	93.0	6.0	3 DU/AC	30.0	72.0	6.5	2.5 DU/AC	27.5	70.0	7.5	SCHOOL	50.0	84.0	6.5					26.5	0.041	79.1	46.7
PNE9	COM.	90.0	94.0	3.1	OPEN SPC		69.0	5.4													8.5	0.013	78.1	32.8
PNE10	POWERS	65.0	85.0	5.9	OPEN SPC.		69.0	30.8													36.7	0.057	71.6	10.4
PNE11	COM.	90.0	94.0	45.4																	45.4	0.071	94.0	90.0
PNE12	COM.	90.0	94.0	16.7																	16.7	0.026	94.0	90.0
PNE13	M.A. STREET	85.0	93.0	5.6	POWERS	50.0	80.0	23.6	OPEN SPC		69.0	2.1									31.3	0.049	81.6	52.9
PNE14	POWERS	38.0	75.0	13.1																	13.1	0.020	75.0	38.0
PSE1	M. STREET	85.0	93.0	1.8	2 DU/AC	25.0	68.0	15.0	2.5 DU/AC	27.5	70.0	4.8									21.6	0.034	70.5	30.6
PSE2	M. STREET	85.0	93.0	2.1	3 DU/AC	30.0	72.0	14.5	SCH.	50.0	84.0	1.0	PARK		67.0	0.7					18.3	0.029	74.9	36.3
PSE3	M. STREET	85.0	93.0	12.1	SCH.	50.0	84.0	9.2	PARK/OS		67.0	3.1	WATER TK	19.0	67.0	5.0	3.5 DU/AC	34.0	74.0	20.5	49.9	0.078	79.3	45.7
PSE4	M. STREET	85.0	93.0	5.0	2 DU/AC	25.0	68.0	12.2	2.5 DU/AC	27.5	70.0	29.3	OPEN SPC.		69.0	1.2					47.7	0.075	71.9	32.2
PSE5	M. STREET	85.0	93.0	2.9	3 DU/AC	30.0	72.0	20.2	3.5 DU/AC	34.0	74.0	6.0	PARK		67.0	1.1					30.2	0.047	74.2	35.0
PSE6	M.A. STREET	85.0	93.0	9.8	3.5 DU/AC	34.0	74.0	18.3	PARK/POND		69.0	6.5									34.6	0.054	78.4	42.1
PSE7	COM.	90.0	96.0	37.0																	37.0	0.058	96.0	90.0
PSE8	M. STREET	85.0	93.0	4.4	3.5 DU/AC	34.0	74.0	9.7	4 DU/AC	37.0	76.0	12.4	6 DU/AC	56.0	82.0	9.2	OPEN SPC.	35.0	73.0	1.6	37.3	0.058	78.8	46.5
PSE9	COM.	90.0	94.0	23.0	DET POND		69.0	3.5													26.5	0.041	90.7	78.1
PSE10	M.A. STREET	85.0	93.0	9.0	POWERS	50.0	80.0	7.8	4 DU/AC	37.0	76.0	3.0	OPEN SPC.	35.0	73.0	3.0					22.8	0.036	83.7	60.1
PSE 11	POWERS	50.0	80.0	20.6																	20.6	0.032	80.0	50.0
																			TOTAL		867.1	1.355		

PINE CREEK DRAINAGE BASIN PLANNING STUDY

FULLY DEVELOPED CONDITION OUTPUT SUMMARY AND CURVE NUMBER ADJUSTMENT

9/24/2002

TYPE IIa 24HR STRM @3 MIN. TIME STEP

									HEC1 MODEL W/ COMPUTED CN		HEC1 MODEL W/ ADJUSTED CN					
SUB BASIN I.D.	AREA (sq miles)	AREA (acres)	IMPERVIOUS PERCENT	COMPUTED CN	ADJUSTED CN	COMPUTED C100	TC (min)	LAG (hours)	Q100 (cfs)	Q100/ACRE (cfs)	Q100 (cfs)	Q100/ACRE (cfs)	I100 (in/hr)	RATIONAL METHOD Q100 (cfs/AC)	COMPUTED HEC1 VS. RATIONAL PERCENT	ADJUSTED HEC1 VS. RATIONAL PERCENT
CN1	0.110	70.7	41.7	78.4	78.4	0.60	19.0	0.190	222	3.14	222	3.14	5.23	3.14	0	0
CN2	0.078	50.0	33.3	73.3	75.5	0.55	21.4	0.214	124	2.48	136	2.72	4.91	2.70	-9	1
CN3	0.043	27.5	44.8	77.5	80.0	0.62	15.7	0.157	90	3.27	98	3.56	5.76	3.57	-9	0
CS1	0.053	33.9	24.8	70.0	73.8	0.50	18.1	0.181	77	2.27	91	2.68	5.36	2.67	-18	0
CS2	0.070	45.0	78.1	90.4	98.0	0.82	10.1	0.101	229	5.09	254	5.64	7.04	5.77	-13	-2
CS3	0.051	32.5	67.3	86.5	85.5	0.75	17.7	0.177	138	4.25	134	4.12	5.42	4.09	4	1
CS4	0.066	42.3	63.7	85.3	86.0	0.73	12.8	0.128	185	4.37	188	4.44	6.35	4.65	-6	-5
F1	0.119	76.0	44.5	78.3	78.3	0.62	20.8	0.208	233	3.07	233	3.07	4.98	3.07	0	0
F2	0.039	25.0	24.0	71.4	74.0	0.49	17.1	0.171	62	2.48	69	2.76	5.52	2.73	-10	1
F3	0.114	73.0	39.5	75.6	77.0	0.59	21.5	0.215	199	2.73	210	2.88	4.89	2.87	-5	0
F4	0.038	24.5	59.0	84.1	83.0	0.70	19.7	0.197	92	3.76	89	3.63	5.13	3.61	4	1
F5	0.064	41.0	67.9	86.2	89.0	0.76	12.1	0.121	185	4.51	199	4.85	6.51	4.93	-9	-2
F6	0.038	24.5	71.8	87.7	93.5	0.78	10.6	0.106	116	4.73	131	5.35	6.90	5.39	-14	-1
F7	0.052	33.5	76.3	89.5	90.5	0.81	13.7	0.137	160	4.78	164	4.90	6.15	4.97	-4	-2
PM1	0.054	34.5	44.5	80.7	78.5	0.62	20.3	0.203	116	3.36	107	3.10	5.05	3.11	7	0
PM2	0.187	119.5	7.9	64.8	68.5	0.40	31.0	0.310	157	1.31	193	1.62	3.97	1.58	-20	2
PM3	0.058	37.2	17.1	73.2	71.0	0.45	24.8	0.248	85	2.28	77	2.07	4.52	2.05	10	1
PM4	0.111	70.9	19.9	68.3	71.9	0.47	17.0	0.170	152	2.14	180	2.54	5.54	2.60	-21	-2
PM5	0.193	123.3	15.9	67.4	70.5	0.45	18.5	0.185	247	2.00	286	2.32	5.30	2.36	-18	-2
PM6A	0.042	27.0	70.0	87.0	90.0	0.77	13.1	0.131	122	4.52	132	4.89	6.28	4.84	-7	1
PM6B	0.036	23.2	80.0	91.0	98.0	0.83	11.5	0.115	107	4.61	130	5.60	6.66	5.53	-20	1
PM7	0.138	88.0	40.3	76.3	76.3	0.59	35.3	0.353	191	2.17	191	2.17	3.67	2.17	0	0
PM8	0.014	8.7	85.0	93.0	98.0	0.86	10.0	0.100	48	5.52	51	5.86	7.07	6.08	-10	-4
PM9	0.068	43.5	55.2	84.2	83.5	0.68	14.6	0.146	186	4.28	176	4.05	5.97	4.07	5	0
PM10	0.048	31.0	83.0	92.0	98.0	0.85	11.8	0.118	163	5.26	173	5.58	6.59	5.59	-6	0
PM11	0.042	27.0	83.7	92.4	98.0	0.85	12.1	0.121	139	5.15	152	5.63	6.51	5.55	-8	1
PN7	0.071	45.6	25.8	72.5	74.0	0.50	20.0	0.200	112	2.46	119	2.61	5.09	2.57	-5	2
PN8	0.036	23.1	68.0	87.9	88.5	0.76	12.5	0.125	108	4.68	110	4.76	6.42	4.87	-4	-2
PN9	0.110	70.4	14.7	71.3	70.5	0.44	21.9	0.219	158	2.24	152	2.16	4.85	2.12	5	2
PN11	0.083	53.4	43.9	78.2	79.0	0.61	19.4	0.194	165	3.09	170	3.18	5.17	3.17	-3	0
PN12	0.101	64.6	16.8	69.6	71.0	0.45	22.2	0.222	133	2.06	142	2.20	4.81	2.17	-5	1
PN13	0.045	29.1	12.5	65.3	64.0	0.31	24.1	0.241	48	1.65	42	1.44	4.60	1.43	13	1
PN15	0.069	44.0	22.6	71.3	72.7	0.49	18.6	0.186	106	2.41	112	2.55	5.29	2.57	-7	-1
PS2	0.024	15.2	73.5	88.4	88.4	0.79	15.0	0.150	71	4.67	71	4.67	5.89	4.66	0	0
PS3	0.070	45.1	85.1	92.6	97.5	0.86	11.7	0.117	235	5.21	252	5.59	6.61	5.69	-9	-2
PS4	0.060	38.2	42.9	78.7	78.5	0.61	17.8	0.178	126	3.30	125	3.27	5.41	3.29	0	0
PS5	0.030	19.5	85.4	92.8	96.0	0.86	13.0	0.130	100	5.13	105	5.38	6.30	5.44	-6	-1
PS6	0.053	34.0	89.4	93.8	97.5	0.89	12.6	0.126	181	5.32	190	5.59	6.40	5.67	-6	-1
PS7	0.031	20.1	85.2	92.8	97.5	0.86	11.8	0.118	105	5.22	112	5.57	6.59	5.67	-9	-2
PS8	0.112	71.4	58.8	84.1	83.0	0.70	17.4	0.174	284	3.98	274	3.84	5.47	3.85	3	0
PS9	0.054	34.8	70.9	87.6	90.0	0.78	12.5	0.125	159	4.57	171	4.91	6.42	4.98	-9	-1
PS10	0.053	33.6	23.9	73.2	73.4	0.49	17.7	0.177	89	2.65	90	2.68	5.42	2.68	-1	0
PS11	0.054	34.7	47.1	78.5	80.3	0.63	17.2	0.172	114	3.29	121	3.49	5.50	3.48	-6	0
PS12	0.153	98.0	9.9	70.0	69.0	0.41	23.3	0.233	199	2.03	189	1.93	4.68	1.92	6	1
PS13	0.065	41.9	25.0	73.9	74.3	0.50	14.9	0.149	121	2.89	123	2.94	5.91	2.96	-2	-1
TOTAL	3.203	2049.9														

AMMENDMENT No. 3 TO
PINE CREEK DRAINAGE BASIN PLANNING STUDY

FULLY DEVELOPED CONDITION OUTPUT SUMMARY AND CURVE NUMBER ADJUSTMENT

9/24/2002

TYPE IIa 24HR STRM @3 MIN. TIME STEP

SUB BASIN I.D.	AREA (sq miles)	AREA (acres)	IMPERVIOUS PERCENT	COMPUTED CN	ADJUSTED CN	COMPUTED C100	TC (min)	LAG (hours)	HEC1 MODEL W/ COMPUTED CN Q100 (cfs)	HEC1 MODEL W/ COMPUTED CN Q100/ACRE (cfs)	HEC1 MODEL W/ ADJUSTED CN Q100 (cfs)	HEC1 MODEL W/ ADJUSTED CN Q100/ACRE (cfs)	I100 (in/hr)	RATIONAL METHOD Q100 (cfs/AC)	COMPUTED HEC1 VS. RATIONAL PERCENT	ADJUSTED HEC1 VS. RATIONAL PERCENT
PNE1	0.135	86.1	41.20	75.3	78.0	0.60	18.6	0.186	243	2.82	271	3.15	5.29	3.16	-12	0
PNE2	0.171	109.3	40.88	76.1	77.5	0.60	21.0	0.210	307	2.81	324	2.96	4.96	2.95	-5	0
PNE3	0.013	8.1	65.05	85.0	87.0	0.74	14.3	0.143	35	4.32	37	4.57	6.03	4.46	-3	2
PNE4	0.076	48.9	36.38	73.8	77.5	0.57	15.8	0.158	138	2.82	159	3.25	5.74	3.26	-16	0
PNE5	0.018	11.6	0.00	69.0	66.5	0.35	19.8	0.198	24	2.07	22	1.90	5.12	1.79	13	6
PNE6	0.020	12.6	45.00	77.0	79.5	0.62	15.4	0.154	41	3.25	45	3.57	5.82	3.61	-11	-1
PNE7	0.103	65.8	32.29	71.6	76.0	0.54	17.5	0.175	163	2.48	196	2.98	5.46	2.97	-20	0
PNE8	0.041	26.5	46.65	79.1	81.0	0.63	15.7	0.157	91	3.43	97	3.66	5.76	3.63	-6	1
PNE9	0.013	8.5	32.80	78.1	80.0	0.55	9.7	0.097	31	3.65	33	3.88	7.16	3.92	-7	-1
PNE10	0.057	36.7	10.40	71.6	69.3	0.41	22.8	0.228	81	2.21	73	1.99	4.74	1.95	11	2
PNE11	0.071	45.4	90.00	94.0	96.5	0.89	13.0	0.130	242	5.33	251	5.53	6.31	5.61	-5	-1
PNE12	0.026	16.7	90.00	94.0	97.5	0.89	12.5	0.116	90	5.39	94	5.63	6.42	5.71	-6	-2
PNE13	0.049	31.3	52.90	81.6	81.0	0.67	16.7	0.146	120	3.83	115	3.67	5.58	3.73	3	-1
PNE14	0.020	13.1	38.00	75.0	76.5	0.58	16.8	0.134	40	3.05	42	3.21	5.58	3.22	-6	-1
PSE1	0.034	21.6	30.56	70.5	74.5	0.53	19.7	0.197	49	2.27	59	2.73	5.13	2.74	-21	0
PSE2	0.029	18.3	36.26	74.9	77.0	0.57	16.9	0.169	54	2.95	58	3.17	5.55	3.15	-7	1
PSE3	0.078	49.9	45.70	79.3	79.6	0.62	17.1	0.171	170	3.41	171	3.43	5.52	3.45	-1	-1
PSE4	0.075	47.7	32.20	71.9	75.6	0.54	19.2	0.192	118	2.47	136	2.85	5.20	2.82	-14	1
PSE5	0.047	30.2	34.98	74.2	76.5	0.56	18.1	0.181	81	2.68	90	2.98	5.36	3.00	-12	-1
PSE6	0.054	34.6	42.10	78.4	78.4	0.60	18.9	0.189	109	3.15	109	3.15	5.24	3.16	0	0
PSE7	0.058	37.0	90.00	96.0	96.5	0.89	12.5	0.125	205	5.54	206	5.57	6.42	5.71	-3	-3
PSE8	0.058	37.3	46.50	78.8	80.0	0.63	16.5	0.165	126	3.38	131	3.51	5.62	3.54	-5	-1
PSE9	0.041	26.5	78.10	90.7	97.5	0.82	10.7	0.107	134	5.06	148	5.58	6.87	5.63	-11	-1
PSE10	0.036	22.8	59.90	83.7	83.2	0.71	17.5	0.175	90	3.95	89	3.90	5.46	3.87	2	1
PSE11	0.032	20.6	50.00	80.0	80.0	0.65	21.0	0.210	66	3.20	66	3.20	4.96	3.22	-1	-1
TOTAL	1.355	867.1														

AMMENDMENT No. 3
TO
PINE CREEK DRAINAGE BASIN PLANNING STUDY

INTERIM CONDITION CURVE NUMBERS

9/27/2002

SUB-BASIN LABEL	SUB AREA ONE				SUB AREA TWO				SUB AREA THREE				TOTAL AREA AC.	TOTAL AREA S.M.	WEIGHTED CN
	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.	ASSUMED LAND USE	ESTIMATED PERCENT IMPERVIOUS	ESTIMATED CN	AREA AC.			
IPN1	1DU/AC	20.0	68.0	30.0	PASTURE	0.0	62.0	69.9					99.9	0.156	63.8
IPN2	PASTURE	0.0	62.0	146.5									146.5	0.229	62.0
IPN3	PASTURE	0.0	62.0	50.5	PAVEMENT	100.0	98.0	3.0	OPN SPC		69.0	13.0	66.5	0.104	65.0
IPN4	PASTURE	0.0	62.0	64.8									64.8	0.101	62.0
IPN5	PASTURE	0.0	62.0	29.2									29.2	0.046	62.0
IPN6	OPN SPC	0.0	69.0	13.7	PAVEMENT	100.0	98.0	8.3					22.0	0.034	79.9
IPN7	PASTURE	0.0	62.0	6.5	PAVEMENT.	100.0	98.0	3.7	OPN SPC		69.0	8.2	18.4	0.029	72.4
IPS1	PASTURE	0.0	62.0	80.5									80.5	0.126	62.0
IPS2	PASTURE	0.0	62.0	45.2	GRAVEL RD.		85.0	1.5	OPN SPC		69.0	4.0	50.7	0.079	63.2
IPS3	PASTURE	0.0	62.0	69.6									69.6	0.109	62.0
IPS4	PASTURE	0.0	62.0	107.4									107.4	0.168	62.0
IPS5	PASTURE	0.0	62.0	26.2									26.2	0.041	62.0
IPS6	PASTURE	0.0	62.0	70.3	PAVEMENT	100.0	98.0	3.4					73.7	0.115	63.7
IPS7	OPN SPC	0.0	69.0	8.9	PAVEMENT	100.0	98.0	10.0					18.9	0.030	84.3
												TOTAL	874.3	1.366	

AMMENDMENT No. 3 TO
PINE CREEK DRAINAGE BASIN PLANNING STUDY
FULLY DEVELOPED CONDITION LAG TIME ESTIMATE
9/17/2002

BASIN ID.	OVERLAND FLOW				SWALE OR STREET FLOW					CHANNEL OR S.D. FLOW					TOTAL	TOTAL	TOTAL
	L (ft)	C(10YR	S (%)	T1(min)	TYPE	L (ft)	S (%)	V (fps)	T2(min)	TYPE	L (ft)	S(%)	V (fps)	T3(min)	TC(min)	LAG(min.	LAG(hrs)
CN1	100	0.25	2	12.65	ST	2000	4	5.5	6.06	SD	300	3	19	0.26	18.97	11.38	0.190
CN2	100	0.25	2	12.65	ST	2000	2.5	4.5	7.41	SD	900	1	11	1.36	21.42	12.85	0.214
CN3	100	0.25	2	12.65	ST	1100	5	6	3.06					0.00	15.70	9.42	0.157
CS1	100	0.25	2	12.65	ST	1650	2.5	5	5.50					0.00	18.15	10.89	0.181
CS2	200	0.75	3	6.44	SW	600	6	5	2.00	SD	1400	1.7	14	1.67	10.11	6.06	0.101
CS3	100	0.25	2	12.65	ST	1400	3.2	5.5	4.24	SD	800	4	17	0.78	17.67	10.60	0.177
CS4	250	0.75	2	8.23	ST	1100	3	5	3.67	SD	1050	4	18	0.97	12.87	7.72	0.129
F1	100	0.25	2	12.65	ST	2200	2.3	4.5	8.15					0.00	20.79	12.48	0.208
F2	100	0.25	2	12.65	ST	1000	3	5	3.33	SD	800	4.5	12	1.11	17.09	10.25	0.171
F3	100	0.25	2	12.65	ST	2650	3	5	8.83					0.00	21.48	12.89	0.215
F4	100	0.25	2	12.65	ST	1700	2	4	7.08					0.00	19.73	11.84	0.197
F5	200	0.75	2.8	6.59	SW	1000	3.3	3	5.56					0.00	12.15	7.29	0.121
F6	200	0.75	3	6.44	SW	1000	3.9	4	4.17					0.00	10.61	6.36	0.106
F7	200	0.75	3	6.44	SW	1300	3	3	7.22					0.00	13.66	8.20	0.137
PM1	100	0.25	2	12.65	ST	1700	1.5	4	7.08	SD	650	3.5	19	0.57	20.30	12.18	0.203
PM2	300	0.25	6	15.24	SW	3300	5	3.5	15.71						30.96	18.57	0.310
PM3	300	0.25	3.0	19.16	SW	650	6.0	3.5	3.10	CH	900	2	6.00	2.50	24.75	14.85	0.248
PM4	100	0.25	2	12.65	ST	800	6	6.5	2.05	SD	2600	5	19	2.28	16.98	10.19	0.170
PM5	100	0.25	2	12.65	ST	1800	5	6	5.00	SD	600	3.5	12	0.83	18.48	11.09	0.185
PM6A	100	0.25	4.4	9.75	SW	150	5	6	0.42	SD	2800	4	16	2.92	13.08	7.85	0.131
PM6B	60	0.25	3	8.57	SW	250	3	5	0.83	SD	2000	4	16	2.08	11.48	6.89	0.115
PM7	300	0.25	9	13.33	SW	4600	2.5	3.5	21.90	SD	200	14	30	0.11	35.35	21.21	0.353
PM8	300	0.75	5	6.67	ST	1200	5	6	3.33					0.00	10.00	6.00	0.100
PM9	300	0.75	2	9.02	ST	2000	4	6	5.56					0.00	14.57	8.74	0.146
PM10	300	0.75	2	9.02	SW	500	4	4.5	1.85	SD	800	2.5	15	0.89	11.76	7.06	0.118
PM11	300	0.75	2	9.02	ST	450	4	6	1.25	SD	1350	2.5	12	1.88	12.14	7.29	0.121
PN7	120	0.25	2.0	13.85	ST	1500	3.0	5.0	5.00	SD	630	3	9	1.17	20.02	12.01	0.200
PN8	200	0.75	2.0	7.36	ST	1600	2.5	5.5	4.85	SD	350	6	18	0.32	12.54	7.52	0.125
PN9	300	0.25	6.0	15.24						CH	2400	4	6	6.67	21.91	13.14	0.219
PN11	150	0.25	2.0	15.49	ST	600	2.0	5.0	2.00	SD	1700	4	15	1.89	19.38	11.63	0.194
PN12	150	0.25	2.0	15.49	ST	1400	4.0	5.5	4.24	SD	2200	4	15	2.44	22.17	13.30	0.222
PN13	450	0.25	4.0	21.34	ST/SW	900	4.0	5.5	2.73					0.00	24.07	14.44	0.241
PN15	100	0.25	2	12.65	ST	1800	3	5	6.00						18.65	11.19	0.186
PS2	100	0.25	2.0	12.65	ST	700	4.0	5.5	2.12	SD	150	1	12	0.21	14.97	8.98	0.150
PS3	200	0.75	2.0	7.36	ST	600	2.0	4.5	2.22	SD	1500	1	12	2.08	11.67	7.00	0.117
PS4	125	0.25	2.0	14.14	ST	900	3.0	6.0	2.50	SD	680	2	10	1.13	17.77	10.66	0.178
PS5	60	0.25	2.0	9.79					0.00	SD	960	2	5	3.20	12.99	7.80	0.130
PS6	200	0.75	2.0	7.36	ST	1000	2.5	4.5	3.70	SD	900	2	10	1.50	12.57	7.54	0.126
PS7	200	0.75	2.0	7.36	ST	1400	4.0	5.5	4.24	SD	125	2	10	0.21	11.81	7.09	0.118
PS8	50	0.25	2.0	8.94	ST	1880	3.0	6.0	5.22	SD	1350	1	7	3.21	17.38	10.43	0.174
PS9	200	0.75	2.0	7.36	ST	800	2.0	5.0	2.67	SD	1500	2	10	2.50	12.53	7.52	0.125
PS10	100	0.25	2.0	12.65	ST	1700	2.6	6.0	4.72	SD	350	6	18	0.32	17.69	10.61	0.177
PS11	100	0.25	2.0	12.65	ST	1400	5.0	6.0	3.89	SD	900	4	21.0	0.71	17.25	10.35	0.172
PS12	300	0.25	9.0	13.33	SW	300	3.0	3.0	1.67	CH	3000	2.5	6.0	8.33	23.33	14.00	0.233
PS13	300	0.25	12.0	12.13	SW	200	6.0	5.0	0.67	ST	500	2	4.00	2.08	14.88	8.93	0.149

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AMMENDMENT No. 3 TO
PINE CREEK DRAINAGE BASIN PLANNING STUDY
EAST OF POWERS BLVD.
FULLY DEVELOPED CONDITION LAG TIME ESTIMATE
9/17/2002

BASIN ID.	OVERLAND FLOW				SWALE OR STREET FLOW					CHANNEL OR S.D. FLOW					TOTAL	TOTAL	TOTAL
	L (ft)	C(10YR	S (%)	T1(min)	TYPE	L (ft)	S (%)	V (fps)	T2(min)	TYPE	L (ft)	S(%)	V (fps)	T3(min)	TC(min)	LAG(min.	LAG(hrs)
PNE1	100	0.25	2.0	12.65	ST	1600	7.0	7.5	3.56	SD	2600	6.0	18	2.41	18.61	11.16	0.186
PNE2	100	0.25	2.0	12.65	ST	1600	5.0	6.0	4.44	SD	3500	4.0	15	3.89	20.98	12.59	0.210
PNE3	100	0.25	2.0	12.65	ST	500	2.0	5.0	1.67					0.00	14.31	8.59	0.143
PNE4	100	0.25	2.0	12.65	ST	1200	5.0	8.0	2.50	SD	600	2.0	16	0.63	15.77	9.46	0.158
PNE5	300	0.25	7.0	14.49	SW	1900	4.0	6.0	5.28					0.00	19.76	11.86	0.198
PNE6	100	0.25	2.0	12.65	ST	600	4.0	5.5	1.82	SD	1000	6.0	18	0.93	15.39	9.23	0.154
PNE7	100	0.25	2.0	12.65	ST	1600	5.0	8.0	3.33	SD	1600	3.0	18	1.48	17.46	10.48	0.175
PNE8	100	0.25	2.0	12.65	ST	1400	5.0	8.0	2.92	SD	150	2.0	14	0.18	15.74	9.44	0.157
PNE9	200	0.75	2.0	7.36	ST	700	2.0	5.0	2.33					0.00	9.70	5.82	0.097
PNE10	300	0.25	4.0	17.42	SW	200	4.0	5.0	0.67	CH	1700	3.5	6	4.72	22.81	13.69	0.228
PNE11	200	0.75	2.0	7.36	ST	1600	3.0	6.0	4.44	SD	1000	2.0	14	1.19	13.00	7.80	0.130
PNE12	200	0.75	2.0	7.36	ST	1500	3.0	6.0	4.17	SD	100	2.0	16	0.10	11.63	6.98	0.116
PNE13	300	0.50	6.0	10.76	SW	1200	2.6	7.0	2.86	SD	800	2.0	14	0.95	14.57	8.74	0.146
PNE14	300	0.53	6.0	10.22	SW	700	1.3	5.0	2.33	SD	450	1.5	9	0.83	13.39	8.03	0.134
PSE1	150	0.25	2.0	15.49	ST	1400	3.0	6.0	3.89	SD	200	3.0	9	0.37	19.75	11.85	0.197
PSE2	120	0.25	2.0	13.85	ST	1300	4.0	7.0	3.10					0.00	16.95	10.17	0.169
PSE3	150	0.25	4.0	12.32	ST	1200	2.5	5.5	3.64	SD	700	2.5	10	1.17	17.12	10.27	0.171
PSE4	150	0.25	3.0	13.55	ST	1600	3.7	6.0	4.44	SD	900	3.0	12	1.25	19.24	11.55	0.192
PSE5	120	0.25	2.0	13.85	ST	1600	4.0	6.3	4.23					0.00	18.08	10.85	0.181
PSE6	100	0.25	2.0	12.65	ST	900.0	4.0	5.5	2.73	SD	1700	1.0	8	3.54	18.91	11.35	0.189
PSE7	200	0.75	2.0	7.36	ST	1600.0	3.0	6.0	4.44	SD	300	1.0	7	0.71	12.52	7.51	0.125
PSE8	100	0.25	2.0	12.65	ST	1000	3.0	6.0	2.78	SD	600	3.0	9.0	1.11	16.53	9.92	0.165
PSE9	200	0.75	2.0	7.36	ST	1000	3.0	5.0	3.33					0.00	10.70	6.42	0.107
PSE10	300	0.46	3.0	14.43	SW	1100	1.6	6.0	3.06					0.00	17.48	10.49	0.175
PSE11	300	0.46	2.0	16.49	SW	1750	2.0	7.0	4.17	SD	250	2.0	12.0	0.35	21.00	12.60	0.210

OVERLAND FLOW (TC=1.87*(1.1-C10)(*L^.5)*S^-.33)

STREET AND SWALE VELOCITY PER MANNINGS BASED ON AN ESTIMATED AVERAGE FLOW RATE

CHANNEL VELOCITY PER MANNINGS BASED ON APPROXMATE SECTION AND FLOW RATE
STORM DRAIN VELOCITY PER MANNINGS BASED ON AN ESTIMATED STORM DRAIN SIZE

AMMENDMENT No. 3
TO
PINE CREEK DRAINAGE BASIN PLANNING STUDY

INTERIM CONDITION LAG TIME ESTIMATE

9/27/2002

BASIN ID.	OVERLAND FLOW				SHALLOW CONCENTRATED FLOW					CHANNEL FLOW					TOTAL TC(min)	TOTAL LAG(min.)	TOTAL LAG(hrs)
	L (ft)	C(10YR)	S (%)	TC(min)	TYPE	L (ft)	S (%)	V (fps)	TC(min)	TYPE	L (ft)	S(%)	V (fps)	TC(min)			
IPN1	300	0.25	10.0	12.88	GRASS CHAN.	5000	5.0	3.6	23.15					0.00	36.03	21.62	0.360
IPN2	300	0.25	4.7	16.52	GRASS CHAN.	4200	4.2	3.3	21.21					0.00	37.73	22.64	0.377
IPN3	300	0.25	6.6	14.77	GRASS CHAN.	1600	4.5	3.4	7.84	NAT. CHANNEL	950	3.4	7.0	2.26	24.87	14.92	0.249
IPN4	300	0.25	7.0	14.49	GRASS CHAN.	2350	3.0	2.8	13.99	TRAP DITCH	1000	3.0	6.0	2.78	31.25	18.75	0.313
IPN5	300	0.25	6.0	15.24	GRASS CHAN.	1600	3.0	2.8	9.52					0.00	24.77	14.86	0.248
IPN6	300	0.5	6.0	10.76	STREET	1200	2.6	7.0	2.86	STORM SEWER DITCH / PIPE	800	2.0	14.0	0.95	14.57	8.74	0.146
IPN7	300	0.25	2.0	21.90	GRASS CHAN.	450	2.2	2.6	2.88					0.00	24.79	14.87	0.248
IPS1	300	0.25	5.3	15.88	GRASS CHAN.	3600	3.7	3.1	19.35					0.00	35.23	21.14	0.352
IPS2	300	0.25	3.3	18.57	GRASS CHAN.	2600	5.7	3.8	11.40					0.00	29.97	17.98	0.300
IPS3	300	0.25	13.6	11.63	GRASS CHAN.	3050	5.7	3.8	13.38					0.00	25.01	15.01	0.250
IPS4	300	0.25	4.7	16.52	GRASS CHAN.	1650	4.2	3.3	8.33	NAT. CHANNEL	1700	3.1	5.0	5.67	30.52	18.31	0.305
IPS5	300	0.25	3.3	18.57	GRASS CHAN.	1900	2.6	2.6	12.18					0.00	30.75	18.45	0.307
IPS6	300	0.25	6.0	15.24	GRASS CHAN.	3600	3.7	3.1	19.35	TRAP DITCH	500	2.0	5.0	1.67	36.26	21.76	0.363
IPS7	300	0.46	2.0	16.49	TRAP DITCH	1750	2.0	5.0	5.83	STORM SEWER	250	2.0	12.0	0.35	22.67	13.60	0.227

NOTE: LAG TIMES IN SUB-BASINS NOT INCLUDED IN THE TABLE ABOVE ARE THE SAME AS THE FULLY DEVELOPED CONDITION.

C.

HYDROLOGIC MODEL (HEC-1) OUTPUT

C-1

HEC-1 MODEL OUTPUT

5-YEAR STORM, FULLY DEVELOPED CONDITION

```
HEC1 S/N: 1343000062        HMVersion: 6.33     Data File: PCDEV5.DAT

****************************************
**************************************
*                                      *
*                                                                                  *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                                           *   U.S. ARMY CORPS OF ENGINEERS
*                                                                                  *   HYDROLOGIC ENGINEERING CENTER
*               MAY    1991            *                                           *          609 SECOND STREET
*                                                                                  *      DAVIS, CALIFORNIA 95616
*             VERSION 4.0.1E           *                                           *          (916) 756-1104
*                                                                                  *
*                                      *
*                                      
*  RUN DATE  09/24/2002  TIME  15:58:29 *
*                                      
*                                      *
*                                      
****************************************
**************************************
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X      X  XXXXXXX   XXXXX           X
X      X  X        X     X         XX
X      X  X        X                X
XXXXXXX   XXXX     X        XXXXX   X
X      X  X        X                X
X      X  X        X     X          X
X      X  XXXXXXX   XXXXX          XXX
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::::::::::::::::::::::::::::::::::::::::::
::::::::::::::::::::::::::::::::::::::::::
:::                                    :::
:::  Full Microcomputer Implementation :::
:::                by                  :::
:::        Haestad Methods, Inc.       :::
:::                                    :::
::::::::::::::::::::::::::::::::::::::::::
::::::::::::::::::::::::::::::::::::::::::

37 Brookside Road * Waterbury, Connecticut  06708 * (203) 755-1666
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```
THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL    LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
```

```
                                    HEC-1 INPUT                                           PAGE  1

LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID   PINE CREEK DRAINAGE BASIN - 24HR, FULL DEVELOPED CONDITION (TYPE IIa5 YEAR)
  2           ID   FULLY DEVELOPED CONDITION MODEL ASSUMING EXISTING AND PLANNED DEVELOPMENT
  3           ID   WEST OF POWERS AND LAND USE AND MAJOR STREETS PER DECEMBER 2001 CONCEPT PLAN
  4           ID   FOR CORDERA (JOHNSON RANCH) EAST OF POWERS
  5           ID   THIS IS A MODIFIED VERSION OF THE DBPS AMENDMENT 2 MODEL. THE MODEL HAS BEEN
  6           ID   REVISED IN AREAS THAT HAVE CHANGED SIGNIFICANTLY FROM THE AMENDMENT 2
  7           ID   ASSUMPTIONS. OTHER AREAS HAVE NOT BEEN CHANGED
  8           ID   CN VALUES HAVE BEEN ADJUSTED TO PRODUCE PEAK 100 YEAR FLOW RATES SIMILAR TO
  9           ID   100 YEAR FLOW RATES PRODUCED BY RATIONAL METHOD.
 10           ID   **********************************************************************
 11           ID   BEGIN CALCULATIONS IN THE PINE CREEK NORTH FORK WATERSHED
```

```
*** FREE ***
                ID  *******************************************************************************
                *DIAGRAM
                IT      3       0       0     300
                IO      5

                KK SB-PNE1
                KM    COMPUTE HYDROGRAPH FOR BASIN PNE1
                BA    .135
                IN      15
                PB     2.6
                PC    0000   .0005   .0015   .0030   .0045   .0060   .0080   .0100   .0120   .0143
                PC   .0165   .0188   .0210   .0233   .0255   .0278   .0320   .0390   .0460   .0530
                PC   .0600   .0750   .1000   .4000   .7000   .7250   .7500   .7650   .7800   .7900
                PC   .8000   .8100   .8200   .8250   .8300   .8350   .8400   .8450   .8500   .8550
                PC   .8600   .8638   .8675   .8713   .8750   .8788   .8825   .8863   .8900   .8938
                PC   .8975   .9013   .9050   .9083   .9115   .9148   .9180   .9210   .9240   .9270
                PC   .9300   .9325   .9350   .9375   .9400   .9425   .9450   .9475   .9500   .9525
                PC   .9550   .9575   .9600   .9625   .9650   .9675   .9700   .9725   .9750   .9775
                PC   .9800   .9813   .9825   .9838   .9850   .9863   .9875   .9888   .9900   .9913
                PC   .9925   .9938   .9950   .9963   .9975   .9988   1.000
                LS       0      78
                UD    .186

                KKRR-DFNE1
                KM    ROUTE FLOW FROM BASIN PNE1 THROUGH A CONCEPTUAL DETENTION FACILITY. ASSUME A
                KM    36" DIA OUTLET WITH INVERT AT EL. 49.00  OUTLET Q ESTIMATED WITH BUREAU OF
                KM    PUBLIC ROADS NOMOGRAPH FOR INLET CONTROL OF CULVERTS. VOLUME BASED ON
                KM    CONCEPTUAL TRAPEZOID POND WITH A 160'X80' BOTTOM AND  4:1 SIDE SLOPES.
                KO       3       1
                RS       1    STOR       0
                SV       0       0     0.7     1.5     2.7     4.0     5.6     7.5
                SE      49      50      52      54      56      58      60      62
                SQ       0       4      35      60      81      92     103     114

                KKRT-APNE1
                KM    ROUTE DISCHARGE FROM DFNE1 TO APE1
                RD     700    .043    .013                    CIRC     3.5
                                            HEC-1 INPUT
LINE            ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10          PAGE  2

                KK SB-PNE2
                KM    COMPUTE HYDROGRAPH FOR BASIN PNE2
                BA    .171
                LS       0    77.5
                UD    .210

                KKRR-DFNE2
                KM    ROUTE FLOW FROM BASIN PNE2 THROUGH A CONCEPTUAL DETENTION FACILITY. ASSUME A
                KM    42" DIA OUTLET WITH INVERT AT EL. 21.   OUTLET Q ESTIMATED WITH BUREAU OF
                KM    PUBLIC ROADS NOMOGRAPH FOR INLET CONTROL OF CULVERTS. VOLUME BASED ON
                KM    CONCEPTUAL TRAPEZOID POND WITH A 105'X210' BOTTOM WITH 4:1 SIDE SLOPES.
                KO       3       1
                RS       1    STOR       0
                SV       0       0     1.1     2.5     4.2     6.1     8.4    11.1
                SE      21      22      24      26      28      30      32      34
                SQ       0       5      41      85     104     121     138     155

                KKRT-DFNE2
                KM    ROUTE DISCHARGE FROM DFNE2 TO APE1
                RD     150     .03    .013                    CIRC     3.5

                KK SB-PNE3
                KM    COMPUTE HYDROGRAPH FOR BASIN PNE3
                BA    .013
                LS       0      87
                UD   0.143

                KK    APE1
                KM    COMBINE ROUTED FLOW FROM DFNE1 AND DFNE2 WITH THE FLOW FROM BASIN PNE3
                KO       0       3
                HC       3

                KK    APE1
                KM    A DIVERSION BOX IS PROPOSED AT APE1 TO SPLIT THE FLOW. OUTFLOW
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KM    LESS THAN THE 5-YEAR PEAK FLOW +/- SHALL BE CONVEYED DOWNSTREAM IN A
KM    PROPOSED STORM DRAIN. FLOWS GREATER THAN THE 5-YEAR PEAK FLOW SHALL
KM    OVERFLOW AND BE CONVEYED IN THE NATURAL CHANNEL. THE DIVERTED FLOW RATIO
KM    IS BASED ON A CONCEPT DIVERSION BOX WITH A 48" DIA. OUTLET TO THE DOWNSTREAM
KM    STORM SEWER WITH ITS INVERT SET 7 FEET LOWER THAN A 10 FOOT LONG WEIR OUTLET
KM    TO THE DOWNSTREAM CHANNEL C=3.3.
DT    APE1a
DI      130     152     178     211     251     293
DQ        0      12      33      61      93     130

KK RT-APE1
KM    ROUTE THE FLOW TO BE CONVEYED IN THE STORM DRAIN FROM APE1 TO APE2
RD     2600    .035    .013             CIRC       4

KK SB-PNE4
KM    COMPUTE HYDROGRAPH FOR BASIN PNE4
BA     .076
LS        0    77.5
UD     .158
```



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LINE      ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

KK    APE2
KM    COMBINE THE ROUTED FLOW FROM BASIN PNE4 WITH THE ROUTED FLOW IN THE PROPOSED
KM    STORM SEWER FROM THE PROPOSED DIVERSION BOW AT APE2
HC        2

KK RT-APE2
KM    ROUTE FLOW FROM APE2 IN THE PROPOSED BLUE ROAD STORM SEWER TO A POINT JUST
KM    DOWNSTREAM OF APE3
RD      600    .016    .013             CIRC     5.0

KKDR-APE1a
KM    RETRIEVE THE FLOW DIVERTED TO THE NATURAL CHANNEL AT APE1
DR    APE1a

KKRT-APE1a
KM    ROUTE THE RETRIEVED FLOW FROM APE1 TO APE3 IN THE NATURAL CHANNEL.
KM    USE GENERALIZED CHANNEL SECTION AND AVERAGE SLOPE
RD     2700    .037    0.04             TRAP      20       5

KK SB-PNE5
KM    COMPUTE HYDROGRAPH FOR BASIN PNE5
BA     .018
LS        0    66.8
UD     .198

KK    APE3
KM    COMBINE ROUTED FLOW FROM BASIN PNE5 WITH THE ROUTED FLOW IN THE NATURAL
KM    CHANNEL AT APE3
HC        2

KK    APE3a
KM    COMBINE ROUTED FLOW FROM APE3 WITH THE ROUTED FLOW IN THE PROPOSED BLUE RD.
KM    STORM SEWER FROM APE2 JUST DOWNSTREAM OF APE3
HC        2

KKRT-APE3a
KM    ROUTE FLOW FROM APE3a TO APE4
RD      300    .016    .013             CIRC     6.5

KK SB-PNE6
KM    COMPUTE HYDROGRAPH FOR BASIN PNE6
BA      .02
LS        0    79.5
UD     .154

KKRR-DFNE6
KM    ROUTE FLOW FROM BASIN PNE6 THROUGH A CONCEPTUAL DETENTION FACILITY. ASSUME A
KM    18" DIA OUTLET WITH INVERT AT EL. 0.  OUTLET Q ESTIMATED WITH BUREAU OF
KM    PUBLIC ROADS NOMOGRAPH FOR INLET CONTROL OF CULVERTS. VOLUME BASED ON
KM    CONCEPTUAL TRAPEZOID POND WITH A 111' SQUARE BOTTOM WITH 4:1 SIDE SLOPES.
KO        3       1
RS        1    STOR       0
SV        0    0.01    0.21    0.45    0.83    1.37
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 135        SE       0       1       2       4       6       8
                                        HEC-1 INPUT                                      PAGE  4

LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 136        SQ       0     2.8     6.5    11.0    14.0    16.0

 137        KKRT-DFNE6
 138        KM    ROUTE DETAINED FLOW FROM BASIN PNE6 TO APE4
 139        RD    2700    .034    .013            CIRC       2

 140        KK SB-PNE7
 141        KM    COMPUTE HYDROGRAPH FOR BASIN PNE7
 142        BA    .103
 143        LS       0      76
 144        UD    .175

 145        KK    APE4a
 146        KM    COMBINE ROUTED FLOW FROM BASIN PNE6 WITH THE FLOW FROM BASIN PNE7 JUST
 147        KM    UPSTREAM OF APE4
 148        HC       2

 149        KK     APE4
 150        KM    COMBINE ROUTED FLOW FROM APE3a AND BASIN AP4a
 151        HC       2

 152        KK     APE4
 153        KM    A DIVERSION BOX IS PROPOSED AT APE4 TO SPLIT THE FLOW. FLOW LESS THAN
 154        KM    THE 2-YEAR PEAK FLOW +/- SHALL BE CONVEYED BY THE PROPOSED STORM DRAIN.
 155        KM    PORTIONS OF FLOWS GREATER THAN THE 2-YEAR PEAK FLOW  SHALL OVERFLOW TO
 156        KM    THE NATURAL CHANNEL. THE DIVERTED FLOW RATIO IS BASED A CONCEPT DIVERSION
 157        KM    BOX WITH A 48" DIA. OUTLET TO THE DOWNSTREAM STORM SEWER WITH ITS INVERT
 158        KM    SET 7 FEET LOWER THAN A 20 FOOT LONG WEIR OUTLET TO THE DOWNSTREAM CHANNEL C=
 159        DT    APE4a
 160        DI     130     211     347     514     708
 161        DQ       0      66     187     342     528

 162        KK RT-APE4
 163        KM    ROUTE THE FLOW IN THE PROPOSED STORM SEWER FROM APE4 TO APE5
 164        RD    1700    .013    .013            CIRC       4

 165        KK SB-PNE8
 166        KM    COMPUTE HYDROGRAPH FOR BASIN PNE8
 167        BA    .041
 168        LS       0    81.0
 169        UD    .157

 170        KK     APE5
 171        KM    COMBINE ROUTED FLOW FROM BASIN PNE8 WITH THE ROUTED FLOW IN THE STORM SEWER
 172        KM    FROM APE4
 173        HC       2

 174        KK RT-APE5
 175        KM    ROUTE FLOW FROM APE5 TO APE6
 176        RD    1000    .015    .013            CIRC     5.5
                                        HEC-1 INPUT                                      PAGE  5

LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


 177        KK SB-PNE9
 178        KM    COMPUTE HYDROGRAPH FOR BASIN PNE9
 179        BA    .013
 180        LS       0    80.0
 181        UD    .097

 182        KK     APE6
 183        KM    COMBINE ROUTED FLOW FROM BASIN PNE9 WITH THE ROUTED FLOW FROM APE5
 184        HC       2

 185        KK RT-APE6
 186        KM    ROUTE FLOW FROM APE6 TO AP3 AT THE CHANNEL RUNDOWN TO DETENTION FACILITY F
 187        RD     800    .033    .013            CIRC     5.5

 188        KK    APE4a
 189        KM    RETRIEVE THE FLOW DIVERTED TO THE NATURAL CHANNEL AT APE4
 190        DR    APE4a
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KKRT-APE4a
KM   ROUTE FLOW FROM APE4 IN THE NATURAL CHANNEL TO AP3 AT THE CHANNEL RUNDOWN
KM   TO DETENTION FACILITY F. USE GENERALIZED CHANNEL SECTION AND AVERAGE SLOPE
RD    2100    .033    0.05             TRAP      10       3

KKSB-PNE10
KM   COMPUTE HYDROGRAPH FOR BASIN PNE10
BA    .057
LS       0    69.3
UD    .228

KK    AP3a
KM   COMBINE ROUTED FLOW IN THE CHANNEL WITH THE FLOW FROM BASIN PNE10
KM   THIS IS THE TOTAL FLOW IN THE NATURAL CHANNEL ABOVE DETENTION
KM   FACILITY F
HC       2

KKSB-PNE11
KM   COMPUTE HYDROGRAPH FOR BASIN PNE11
BA    .071
LS       0    96.5
UD    .130

KKRT-PNE11
KM   ROUTE THE FLOW FROM BASIN PNE11 TO AP1 AT THE OUTFALL FROM BASIN PNE12
RD     400     .03    .013             CIRC       5

KKSB-PNE12
KM   COMPUTE HYDROGRAPH FOR BASIN PNE12
BA    .026
LS       0    97.5
UD    .116
                                  HEC-1 INPUT                                    PAGE  6

LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

KK    AP1
KM   COMBINE THE ROUTED FLOW FROM BASIN PNE11 WITH THE FLOW FROM BASIN PNE12
HC       2

KK  RT-AP1
KM   ROUTE THE FLOW FROM AP1 TO AP2
RD    1150     .01    .013             CIRC     6.0

KKSB-PNE13
KM   COMPUTE HYDROGRAPH FOR BASIN PNE13
BA    .049
LS       0    81.0
UD    .146

KKRT-PNE13
KM   ROUTE THE FLOW FROM BASIN PNE13 TO AP2 IN THE PROPOSED POWERS RAMP B
KM   STORM SEWER
RD     950    .002    .013             CIRC     6.0

KKSB-PNE14
KM   COMPUTE HYDROGRAPH FOR BASIN PNE14
BA    .020
LS       0    76.5
UD    .134

KK    AP2a
KM   COMBINE THE FLOW FROM BASINS PNE13 AND PNE14
HC       2

KK    AP2
KM   COMBINE THE FLOW FROM BASINS PNE13 AND PNE14 WITH THE ROUTED FLOW FROM AP1
HC       2

KK  RT-AP2
KM   ROUTE THE FLOW FROM AP2 TO AP3 AT THE RUNDOWN CHANNEL TO DF-F
RD     400     .06    .013             CIRC     6.0

KK    AP3
KM   COMBINE ROUTED FLOW FROM AP2 WITH ROUTED FLOW FROM APE6 AND THE FLOW IN THE
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KM   NATURAL CHANNEL (AP3a). THIS IS THE TOTAL FLOW TO THE DF-F RUNDOWN CHANNEL
HC        3

KK  SB-PN7
KM   COMPUTE HYDROGRAPH FOR BASIN PN7
BA     .071
LS        0     74.0
UD     .200

KK  SB-PN8
KM   COMPUTE HYDROGRAPH FOR BASIN PN8
BA     .036
LS        0     88.5
UD     .125
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KK   APDFF
KM   COMBINE THE FLOW FROM BASINS PN7 AND PN8 AND AP3. THIS IS THE TOTAL
KM   INFLOW TO DETENTION FACILITY F
HC        3

KK  RR-DFF
KM   ROUTE FLOW THRU A PROPOSED REGIONAL DETENTION FACILITY.
KM   VOLUME REFLECTS CURRENT DRAFT DESIGN
KM   DISCHARGE ASSUMES THE 54" DIA OUTLET SET AT INVERT ELEV. 11.5 IS RESTRICTED
KM   TO A 11.7 SF OPENING BY A STEEL PLATE COVERING THE TOP 1.4' OF THE PIPE.
KM   DISCHARGE CALCULATED WITH THE ORIFICE EQUATION WITH HEAD CALCULATED TO
KM   THE CENTER OF THE OPENING AREA @ ELEVATION 13.28
KO        3       1
RS        1    STOR       0
SV        0     .18     2.6     8.1    15.4   23.70    32.6    42.4    53.1    64.8
SE       13      14      16      18      20      22      24      26      28      30
SQ        5      30      93     122     146     166     184     201     216     230

KK  RT-DFF
KM   ROUTE THE OUTFLOW FROM DETENTION FACILITY F DOWN PINE CREEK NORTH FORK FROM
KM   ROYAL PINE DRIVE TO AP-4
RD     2400     .02    .060            TRAP      20       3

KK  SB-PN9
KM   COMPUTE HYDROGRAPH FOR BASIN PN9
BA     .110
LS        0     70.5
UD     .219

KK     AP4
KM   COMBINE ROUTED FLOW RT-DFF WITH FLOW FROM BASIN PN9 AT AP-4
HC        2

KK  RT-AP4
KM   ROUTE THE FLOW IN PINE CREEK NORTH FORK CHANNEL FROM AP4
KM   TO DETENTION FACILITY "E" ABOVE STONEGLEN DR.
RD     1400    .032    .060            TRAP      20       3
KM   PN10 DESCRIPTOR NOT USED

KK SB-PN11
KM   COMPUTE HYDROGRAPH FOR BASIN PN11
BA     .083
LS        0     79.0
UD     .194

KK SB-PN12
KM   COMPUTE HYDROGRAPH FOR BASIN PN12
BA   0.101
LS        0     71.0
UD     .222
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KK   APDFE
KM   COMBINE ROUTED FLOW FROM AP4 WITH FLOW FROM BASINS PN11 AND PN12
KM   THIS IS THE TOTAL INFLOW TO DETENTION FACILITY E
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307      HC        3

308      KK  RR-DFE
309      KM   NOTE: THE INPUT POND VOLUME REFLECTS THE AS-BUILT SURVEY FOR THE PC 200 LOMR
310      KM   ROUTE FLOW THRU THE THE EXISTING DETENTION FACILITY. ASSUME
311      KM   THE EXISTING 54" DIA IS UN-RESTRICTED INVERT AT ELEVATION 84.
312      KM   OUTLET Q ESTIMATED WITH BUREAU OF PUBLIC ROADS NOMOGRAPH FOR
313      KM   INLET CONTROL OF CULVERTS. DISCHARGE ABOVE EL 800 INCLUDES FLOW
314      KM   OVER EMERGENCY SPILLWAY
315      KO        3       1
316      RS        1    STOR       0
317      SV        0    0.29    1.95    4.92    8.27   11.99   16.09   20.60   25.51   30.89
318      SE      784     786     788     790     792     794     796     798     800     802
319      SQ        0      26      80     133     173     208     238     260     278    1441

320      KK  RT-DFE
321      KM   ROUTE THE OUTFLOW FROM DETENTION FACILITY "E" IN A STORM DRAIN TO AP-5
322      RD     1500    .025    .013            CIRC     4.5

323      KK SB-PN15
324      KM   COMPUTE HYDROGRAPH FOR BASIN PN15
325      BA     .069
326      LS        0    72.7
327      UD     .186

328      KK      AP5
329      KM   COMBINE ROUTED FLOW FROM DFE WITH FLOW FROM BASIN PN15
330      HC        2

331      KK  RT-AP5
332      KM   ROUTE THE FLOW AT AP5 TO AP5A AT THE CONFLUENCE OF THE FLOWS FROM THE
333      KM   NORTH AND SOUTH FORKS OF PINE CREEK
334      RD      150    .025    .013            CIRC     5.5
335      KM   ***********************************************************************
336      KM   ***** BEGIN CALCULATIONS FOR THE SOUTH FORK OF PINE CREEK WATERSHED ****
337      KM   ***********************************************************************

338      KK SB-PSE1
339      KM   COMPUTE HYDROGRAPH FOR BASIN PSE1
340      BA     .034
341      LS        0    74.5
342      UD     .197

343      KK RT-PSE1
344      KM   ROUTE FLOW FROM PSE1 THROUGH PSE2 TO APE7
345      RD     1100    .036    .013            CIRC       3
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LINE     ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

346      KK SB-PSE2
347      KM   COMPUTE HYDROGRAPH FOR BASIN PSE2
348      BA   0.029
349      LS        0      77
350      UD     .169

351      KK     APE7
352      KM   COMBINE ROUTED FLOW FROM PSE1 WITH THE FLOW FROM BASIN PSE2 AT APE7
353      HC        2

354      KK RT-APE7
355      KM   ROUTE FLOW FROM APE7 TO APE8
356      RD     1800    .025    .013            CIRC       4

357      KK SB-PSE3
358      KM   COMPUTE HYDROGRAPH FOR BASIN PSE3
359      BA     .078
360      LS        0    79.6
361      UD     .171

362      KK     APE8
363      KM   COMBINE ROUTED FLOW FROM APE7 WITH THE FLOW FROM BASIN PSE3 AT APE8
364      HC        2

365      KK SB-PSE4
366      KM   COMPUTE HYDROGRAPH FOR BASIN PSE4
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 367          BA     .075
 368          LS        0     75.6
 369          UD     .192

 370          KK RT-PSE4
 371          KM    ROUTE FLOW FROM PSE4 THROUGH PSE5 TO APE9
 372          RD     1350     .036     .013              CIRC      3.5

 373          KK SB-PSE5
 374          KM    COMPUTE HYDROGRAPH FOR BASIN PSE5
 375          BA     .047
 376          LS        0     76.5
 377          UD     .181

 378          KK      APE9
 379          KM    COMBINE ROUTED FLOW FROM PSE4 TO FLOW FROM BASIN PSE5 AT APE9
 380          HC        2

 381          KK RT-APE9
 382          KM    ROUTE FLOW FROM APE9 TO DF D1.
 383          RD      900      .02     .013              CIRC      4.5

 384          KK SB-PSE6
 385          KM    COMPUTE HYDROGRAPH FOR BASIN PSE6
 386          BA     .054
 387          LS        0     78.4
 388          UD     .189
                                   HEC-1 INPUT                                     PAGE 10
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 389          KK     DFD1a
 390          KM    COMBINE THE FLOW FROM BASIN PSE6 TO THE ROUTED FLOW FROM APE9
 391          HC        2

 392          KK      DFD1
 393          KM    COMBINE THE FLOW FROM PSE6 AND THE ROUTED FLOW FROM APE9 TO THE FLOW AT APE8
 394          KM    THIS IS THE TOTAL INFLOW TO PROPOSED DETENTION FACILITY D1
 395          HC        2

 396          KK RR-DFD1
 397          KM    ROUTE FLOW THRU DETENTION FACILITY DFD1
 398          KM    ASSUME BOTTOM TO BE 202' WIDE X 128' LONG AT EL 100
 399          KM    W 4:1 SIDE SLOPES, END SLOPES VARY
 400          KM    ASSUME A 32" DIA OUTLET WITH INVERT AT 98.67.
 401          KM    OUTLET Q ESTIMATED WITH ORIFICE EQUATION ASSUMING c=0.60
 402          KM    AND DOWNSTEAM STORM DRAIN IN NON PRESSURE FLOW
 403          RS        1     STOR        0
 404          KO        3        1
 405          SV        0      1.3      2.9      5.2      8.9     14.1     20.9     29.5
 406          SE      100      102      104      106      108      110      112      114
 407          SQ        0       37       53       65       75       84       92      100

 408          KK RT-DFD1
 409          KM    ROUTE FLOW FROM DFD1 TO AP6 IN BRIARGATE PARKWAY ON THE EAST SIDE OF POWERS
 410          RD     1550     .025     .013              CIRC        4

 411          KK SB-PSE7
 412          KM    COMPUTE HYDROGRAPH FOR BASIN PSE7
 413          BA     .058
 414          LS        0     96.5
 415          UD     .125

 416          KK SB-PSE8
 417          KM    COMPUTE HYDROGRAPH FOR BASIN PSE8
 418          BA     .058
 419          LS        0     80.0
 420          UD     .165

 421          KK RT-PSE8
 422          KM    ROUTE FLOW FROM PSE8 TO DETENTION FACILITY D2 ON THE EAST SIDE
 423          KM    OF POWERS BLVD
 424          RD     1200     .027     .013              CIRC        4

 425          KK SB-PSE9
 426          KM    COMPUTE HYDROGRAPH FOR BASIN PSE9
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BA    .041
LS       0    97.5
UD    .107
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KK    DFD2
KM   COMBINE ROUTED FLOW FROM PSE8 AND PSE9 AT DETENTION FACILITY D2
HC       2

KK RR-DFD2
KM   ROUTE FLOW THRU DETENTION FACILITY D2
KM   ASSUME BOTTOM TO BE 130'WIDE X 200' LONG W 4:1 SIDE SLOPES
KM   ASSUME A 27" DIA OUTLET WITH INVERT 99.00.
KM   OUTLET Q ESTIMATED WITH ORIFICE EQUATION ASSUMING c=0.60
KM   AND DOWNSTEAM STORM DRAIN IN NON PRESSURE FLOW
RS       1    STOR       0
KO       3       1
SV       0      .6     1.9     3.5     5.4     7.6    10.1
SE     100     102     104     106     108     110     112
SQ       0      26      38      46      54      60      66

KK RT-DFD2
KM   ROUTE FLOW FROM DFD2 TO APE10
RD     250     .01    .013            CIRC       3

KK SBPSE10
KM   COMPUTE HYDROGRAPH FOR BASIN PSE10
BA   0.036
LS       0    83.2
UD    .175

KK   APE10
KM   COMBINE ROUTED FLOW FROM DETENTION FACILITY D2 WITH THE FLOW FROM
KM   BASIN PSE10
HC       2

KK     AP6
KM   COMBINE THE ROUTED FLOW AT AP10 TO THE ROUTED FLOW FROM DETENTION FACILITY
KM   D1 AND BASIN PSE7
HC       3

KK  RR-AP6
KM   ROUTE FLOW FROM AP6 TO AP6A ON THE WEST SIDE OF POWERS BLVD.
RD     600     .02    .013            CIRC       6

KKSB-PSE11
KM   COMPUTE HYDROGRAPH FOR BASIN PSE11
BA   0.032
LS       0    80.0
UD    .210

KK    AP6A
KM   COMBINE FLOW FROM PSE11 TO ROUTED FLOW FROM AP6 AT AP6A
HC       2
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KK RT-AP6A
KM   ROUTE FLOW FROM AP6A AT THE WEST SIDE OF POWERS BLVD TO AP6B.
RD     600     .02    .013            CIRC     6.0

KK  SB-PS2
KM   COMPUTE HYDROGRAPH FOR BASIN PS2
BA    .024
LS       0    88.4
UD    .150

KK    AP6B
KM   COMBINE FLOW FROM PS2 TO THE ROUTED FLOW AT AP6B
HC       2

KK RT-AP6B
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483            KM   ROUTE FLOW FROM AP6B TO AP7 AT THE BRIARGATE
484            KM   PKWY./ AUSTIN BLUFFS PKWY. INTERSECTION
485            RD      780      .02     .013              CIRC      6.5

486            KK  SB-PS3
487            KM   COMPUTE HYDROGRAPH FOR BASIN PS3
488            BA    .070
489            LS       0     97.5
490            UD    .117

491            KK  SB-PS4
492            KM   COMPUTE HYDROGRAPH FOR BASIN PS4
493            BA    .060
494            LS       0     78.5
495            UD    .178

496            KK     AP7
497            KM   COMBINE ROUTED FLOW AT AP7 WITH FLOW FROM BASINS PS3 AND PS4
498            HC       3

499            KK  RT-AP7
500            KM   ROUTE THE COMBINED FLOW AT AP7 TO AP7A
501            RD     1050     .022     .013              TRAP        9

502            KK  SB-PS5
503            KM   COMPUTE HYDROGRAPH FOR BASIN PS5
504            BA    .030
505            LS       0     96.0
506            UD     .13

507            KK  SB-PS6
508            KM   COMPUTE HYDROGRAPH FOR BASIN PS6
509            BA    .053
510            LS       0     97.5
511            UD    .126
                                         HEC-1 INPUT                                     PAGE 13

LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


512            KK    AP7A
513            KM   COMBINE ROUTED FLOW AT AP7A WITH FLOW FROM BASINS PS5 AND PS6
514            HC       3

515            KK RT-AP7A
516            KM   ROUTE THE COMBINED FLOW AT AP7A TO AP8
517            RD      800     .022     .013              TRAP       11

518            KK  SB-PS7
519            KM   COMPUTE HYDROGRAPH FOR BASIN PS7
520            BA    .031
521            LS       0     97.5
522            UD    .118

523            KK  SB-PS8
524            KM   COMPUTE HYDROGRAPH FOR BASIN PS8
525            BA    .112
526            LS       0     83.0
527            UD    .174

528            KK     AP8
529            KM   COMBINE ROUTED FLOW AT AP8 WITH FLOW FROM BASINS PS7 AND PS8
530            HC       3

531            KK  RT-AP8
532            KM   ROUTE THE COMBINED FLOW AT AP8 TO AP9, AT DF C
533            RD      250     .022     .013              TRAP       16

534            KK  SB-PS9
535            KM   COMPUTE HYDROGRAPH FOR BASIN PS9
536            BA    .054
537            LS       0     90.0
538            UD    .125

539            KK  RT-PS9
540            KM   ROUTE THE FLOW FROM BASIN PS9 TO AP9, AT DF C
541            RD      880     .025     .013              CIRC      4.0
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