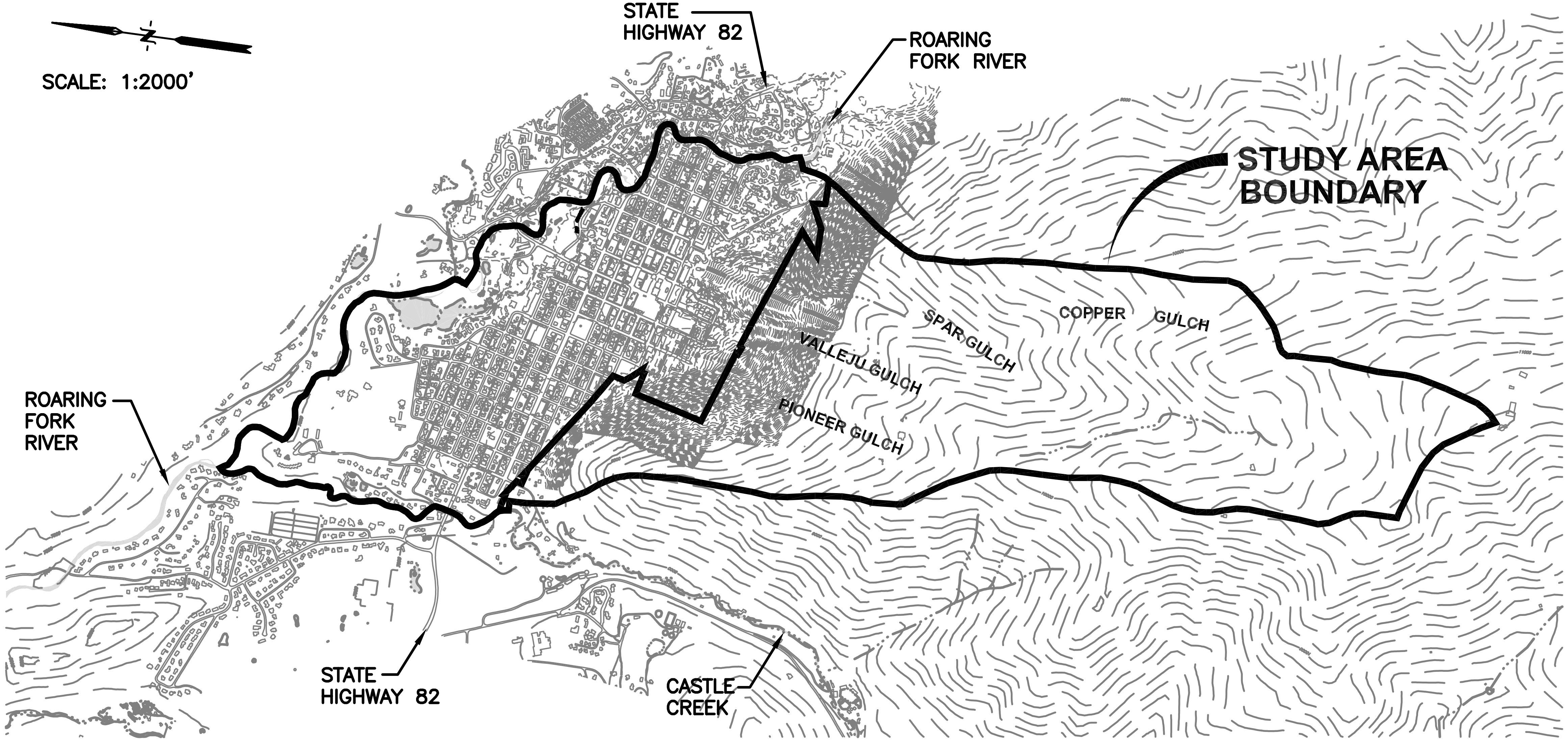
SURFACE DRAINAGE MASTER PLAN FOR THE CITY OF ASPEN



PREPARED FOR:

CITY OF ASPEN

NOVEMBER 2001

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STORM DRAINAGE MASTER PLAN
FOR
THE CITY OF ASPEN, COLORADO

PREPARED FOR
The City of Aspen, Colorado

PREPARED BY
WRC Engineering, Inc.
1963-20
November 2001

SURFACE DRAINAGE MASTER PLAN
FOR
THE CITY OF ASPEN, COLORADO

EXECUTIVE SUMMARY

INTRODUCTION

This report presents the surface drainage master plan for the City of Aspen and the Aspen Mountain watershed. This effort was sponsored by the City of Aspen. WRC Engineering, Inc. is the consultant for this project whom, in cooperation with the project sponsor, has prepared this master plan report.

A Comprehensive Drainage Plan is comprised of three basic components (see Figure ES-1). The Regulatory component of the Drainage Plan includes laws, zoning ordinances, and regulations that the City creates to enforce plans and policy regarding surface drainage. This component ensures that the adopted drainage policy will be followed. The Drainage Criteria Manual component of the Comprehensive Drainage Plan provides the methodology and techniques that developers and engineers must use to design developments and other projects that properly account for surface runoff. The final component of the Comprehensive Drainage Plan is the Surface Drainage Master Plan Report.

The Surface Drainage Master Plan Report analyzes a given area and estimates the amount of surface runoff expected and the direction that this runoff will flow. It analyzes the existing drainage facilities and determines which facilities cannot convey this flow without flooding. The Surface Drainage Master Plan Report evaluates alternatives to upgrade or construct new drainage facilities to reduce or alleviate flooding problems and estimates the costs associated with these alternatives. The Surface Drainage Master Plan Report then recommends the "best" alternative based on an evaluation of the various factors that affect the selection of the alternatives and prepare a conceptual design of the selected alternative. A general description of the Surface Drainage Master Plan report for the City of Aspen follows below.

PURPOSE AND OBJECTIVES

The purpose of this study is to prepare a Surface Drainage Master Plan that identifies the drainage improvements needed to address the snowmelt and storm runoff problems that exist in the developed portion of the City of Aspen and the available alternatives that would address the mudflow potential from Aspen Mountain. The major objectives include the protection or minimization of adverse effects on the City of Aspen due to snowmelt and storm runoff from Aspen Mountain, the protection or minimization of adverse effects on the City of Aspen due to potential mud floods or mudflows off of Aspen Mountain, the regulation of new development or re-development considering surface runoff and mudflow events, and the enhancement of the water quality of snowmelt and stormwater runoff.

PLANNING PROCESS

The planning effort began in August 1997. Since that time a series of progress and technical advisory meetings have taken place to exchange information and discuss ideas and findings of the study. Participants that regularly attended these meetings included the project sponsor staff and representatives of various interested parties. Phase I of the project covered the hydrological, hydraulic, and mudflow analyses for the study area. The initial Phase I report was submitted in July 1998, and the final report was submitted in April 1999. The second phase (Phase II) covers the alternative evaluation and conceptual design of the selected alternative. The preliminary Phase II report was submitted in December 1999. An additional portion of Phase II is the preparation of a Drainage Criteria Manual to include snowmelt, stormwater, and mudflow criteria, which is submitted separate from this report.

DEFINITIONS

Stormwater -	Rain and snow precipitation generated by various meteorological events (storm systems).
Stormwater Runoff –	Runoff generated from both rainfall and snowmelt.
Drainage Facilities –	Systems of natural and manmade facilities that collect and convey surface and sub-surface water.
Flood –	An infrequent stormwater runoff event, which results in runoff quantities that exceed the capacity of available drainage facilities.
Clearwater Flood –	A stormwater runoff event that contains a minimal concentration of soil and debris and acts as a low viscosity fluid. The peak sediment concentration by volume will typically be less than 20%.
Mud flood –	A stormwater runoff event which contains a concentration of soil and debris but whose characteristics (i.e. density, viscosity, and yield stress) act more similar to a clearwater flood than a mudflow. The peak sediment concentration by volume will typically vary from 20% to 40%.
Mudflow –	A stormwater runoff event that contains a high concentration of soil and debris and acts as a high viscosity, hyperconcentrated fluid. The peak sediment concentration by volume will typically vary from 40% to 55%.
Return Frequency –	A statistical designation for a runoff event that will happen on the average once during the designated period of time. For instance, a runoff event with a return

frequency of 100-years will occur or be exceeded, on the average, once every one hundred years, or in other words, it has a 1% chance of being equaled or exceeded in any given year. A 2-year runoff event has a rate of flow that has a 50% chance of being equaled or exceeded in any given year.

SURFACE RUNOFF ANALYSIS

Significant areas of urban development in the City lie at the base of the north side of Aspen Mountain adjacent to the Roaring Fork River. Runoff from the north side of the mountain flows through the City on its way to the Roaring Fork River. The selected study area, shown in Figure ES-2, is bounded on the north by the Roaring Fork River, on the west by the drainage boundary of Castle Creek, and on the east and south by the drainage basins on Aspen Mountain tributary to the City. The 2.5 square mile study area for this project includes the Spar, Pioneer, Vallejo, and Copper Gulches to the south. Surface runoff from the study area generally travels to the north and outfalls into the Roaring Fork River.

There are essentially three drainage systems that collect runoff from Aspen Mountain and convey the flow through the City to the Roaring Fork River (see Figure ES-2). Drainage System 1 encompasses the east side of Aspen and collects and conveys runoff from Spar Gulch and the east side of the City. This system includes existing storm sewers beneath Spring Street, Original Street, and Cooper Street. Drainage System 2 collects and conveys surface runoff from Vallejo Gulch and the central portion of the City. This system consists mainly of an existing storm sewer beneath Mill Street. Drainage System 3 encompasses the City's west side and collects and transmits runoff from Pioneer Gulch, the western portion of Aspen Mountain, and the west side of the City of Aspen. This system includes existing storm sewers beneath Aspen Street, Garmisch Street, Main Street, and Francis Street.

Figure ES-3 shows the location and relative size of the existing storm sewer system in the City of Aspen, and schematically represents a larger pipe diameter with a larger line thickness.

SNOWMELT ANALYSIS

Snowmelt from Aspen Mountain in the spring and early summer causes flow and sediment problems in the City of Aspen. The flow associated with snowmelt is relatively small but of a long duration, and the snowmelt also conveys sediment that is deposited in the streams channels, streets, and storm sewers. The quantity of sediment is relatively small in comparison to the mud floods and mudflows previously discussed. The snowmelt flows and deposits constitute more of a maintenance problem as opposed to the risks of property losses and loss of life that the mudflows and stormwater flows produce. It must be remembered, though, that ignoring regular maintenance will cause irreparable damages due to the cumulative effect of sediment deposits.

There is a possibility that a major rainfall event could occur when there is still snow on the ground. If the snow is ripe (partial melting has already taken place and the snow has little capacity to absorb water), rainfall can produce runoff that is much greater than would be expected by rain alone or snowmelt alone.

Man-made snow should not increase the rate of runoff in the spring and summer, and it should have little direct effect on the potential for mudflows on Aspen Mountain. Manufactured snow will increase the duration of the spring runoff and will increase the probability of occurrence of a rain-on-snow event.

EXISTING FACILITY CAPACITY ANALYSIS

The total drainage system capacity includes runoff that is conveyed by both the existing storm sewer system and flow in the streets. Figure ES-4 supplies a graphical representation of the capacity of the existing drainage facilities in Aspen. The flows corresponding to each return frequency are calculated according to the Rainfall and Runoff Sections of the proposed City of Aspen Drainage Criteria Manual.

Most of Drainage System 1 appears to have the capacity to convey a runoff event with a magnitude of between a 10-year and a 50-year frequency. If a runoff event occurred that had a greater return frequency, flooding would occur. The combined capacity of the street and storm sewer along Cooper Street and Spring Street appears to have sufficient capacity to convey the 100-year runoff event.

The existing drainage facilities of Drainage System 2 have a total capacity to convey the 100-year return frequency flow.

Drainage System 3 is undersized and, depending on the location, most of the facilities only have the capacity to convey the 2-year to 50-year rainfall runoff event. In general, the total system capacity decreases as it proceeds downstream (becomes closer to the Roaring Fork River). The total capacity of the street and storm sewer system north of Hopkins Street is between a 2-year and a 10-year return frequency, and the total drainage system capacity south of Hopkins varies between a 10-year and a 100-year return frequency.

MUDFLOW ANALYSIS

The occurrence of small mudflow events off Aspen Mountain, larger mudflow events on adjacent watersheds, and the results of previous geologic studies suggest that the mountain above Aspen is prone to mud flood and mudflow events. Figure ES-5 presents the potential geologic hazards on Aspen Mountain above the City of Aspen obtained from the U.S. Geological Survey. This map shows that much of the watershed above Aspen is unstable (i.e. alluvial fans, landslide areas, rock fall areas, and other unstable slopes).

FLO-2D is a computer model that estimates the amount of runoff that will occur during a rain event and the depth of flow (water and sediment) that will occur due to this runoff generated from rain or snowmelt.

The purpose of the various FLO-2D model runs is to estimate the effect that a possible mudflow event may have on the City and to determine the best way to model the effect that new development will have on a mud flood and mudflow event. Based on the results of the modeling effort, several conclusions were reached.

The inclusion of sediment in the stormwater flow (making it a mud flood event rather than just a stormwater runoff event) increases the depth of flow. Figure ES-6 shows the maximum depth of flow of the 100-year runoff event considering only water. Figure ES-7 displays the maximum depth of flow of the 100-year runoff event considering water and sediment (a mud flood). Accounting for sediment in the flood event increases the maximum depth of flow by up to 6 feet in some places.

It appears that only revisions in topography in sensitive areas have a significant effect on mudflow depth. Figure ES-8 presents the maximum depth of flow for the area at the south end of Mill Street with the elevation contours as they were in 1997, before the re-grading on Aspen Mountain south of Aspen Street. Figure ES-9 shows the maximum depth of flow for the area at the south end of Mill Street with the topography after the re-grading on Aspen Mountain. The re-grading (movement of about 25,000 cubic yards of material) caused a maximum decrease in the maximum flood depth of about 1 foot. Other flow depth differences at the base of the mountain were minor.

New development should be modeled using FLO-2D to determine the effect that a potential mudflow event will have on the new development and on adjacent developments. The node spacing used in the model should be 50 feet or less. This level of detail will allow each building to be represented in the model and will provide a more accurate estimate of the mudflow event in and around the development. A hypothetical development at the south end of Mill Street caused the maximum mudflow depth to change drastically. In some places, it increased the depth of flow by over 6 feet, and in other places, it decreased the depth of flow by over 6 feet. Figure ES-10 provides the maximum flow depth for this area without the hypothetical development, and Figure ES-11 displays the maximum flow depth after the hypothetical development has been constructed. Figure ES-12 shows the change in the maximum flow depth due to the hypothetical development.

ALTERNATIVES DEVELOPMENT

Three In-City alternatives were developed that would convey runoff (water only) through the City of Aspen from the base of Aspen Mountain to the Roaring Fork River. Two On-Mountain alternatives were also generated that would prevent or reduce the magnitude of a mudflow event on Aspen Mountain, and a third alternative was devised to regulate development in consideration of a mudflow event but not prevent a mudflow event from occurring. The alternatives developed for Aspen Mountain (On-Mountain) are independent of the alternatives developed for the In-City area (i.e. On-Mountain Alternative 1 can be used with In-City Alternative 2).

On-Mountain Alternative 1 consists of a boulder-lined channel and drain system in the bottom of the major gulches on Aspen Mountain. The goal of this alternative is to prevent a mudflow event from occurring. Figure ES-13 shows the location of the proposed channel/drain system. On-Mountain Alternative 2 consists of a series of buried concrete walls in the bottom of the major gulches that would limit the size of a mudflow event. Figure ES-14 displays the location of these walls. On-Mountain Alternative 3 regulates new construction so it will not increase the effect that a mudflow event will have on the City. Alternative 3 will not prevent or reduce the magnitude of a mudflow event. It is proposed that, within the designated mudflow hazard area (see Figure ES-15), development or redevelopment be required to assess and analyze the impact of mudflows on the development as well as impacts of the development on mudflow depths and distribution downstream of the development.

In-City Alternatives 1 and 2 collect and convey the 100-year stormwater runoff event from the base of Aspen Mountain to the Roaring Fork River through a new storm sewer system. The streets are also used to convey runoff. Figure ES-16 and Figure ES-17 show Alternative 1 and Alternative 2, respectively. In-City Alternative 3 collects and conveys runoff from the initial storm event (as defined in the proposed Storm Drainage Criteria Manual) entirely within the existing and proposed storm sewers. The initial storm event is the 2-year, 5-year, or 10-year storm event depending on the building density. Figure ES-18 presents Alternative 3.

A comparison of the various In-City alternatives, based upon current cost estimates, is provided by Table ES-1, and a comparison of the On-Mountain alternatives is supplied by Table ES-2.

CONCLUSIONS

The In-City design alternative selected by the City staff for a more detailed analysis is essentially Alternative 3 with some minor modifications. The total cost of the project would be about $6,204,000 in 1999 dollars. Costs associated with Drainage Systems 1, 2, and 3 would be $2,280,000, $455,000, and $3,469,000, respectively.

Figure ES-19 graphically displays the recommended construction priority for the recommended In-City alternative. The highest priority reaches, or the reaches that should be constructed first, are the proposed storm sewer and water quality extended detention basin east of the intersection of Francis Street and Garmish Street, the storm sewer at the south end of Mill Street, and the storm sewer and collection system on Ute Avenue. Storm sewers either do not exist at these locations or are under-sized. These high priority construction improvements would cost approximately $1,400,000 for the storm sewer and extended detention basin east of Garmisch Street, $260,000 for the storm sewer on the south end of Mill Street, and $70,000 for the storm sewer system improvements on Ute Avenue.

The recommended On-Mountain alternative is Alternative 3. Alternative 1 and Alternative 2 are very expensive and financing is not currently available to spend on these alternatives. The recommended Alternative 3 will provide an interim solution until Alternative 1 and Alternative 2 can be tested and financing can be found to pay for these costly alternatives. The selected alternative will prevent the effect of a mudflow event from becoming worse due to new development.

TABLE ES-1: COMPARISON OF IN-CITY ALTERNATIVES

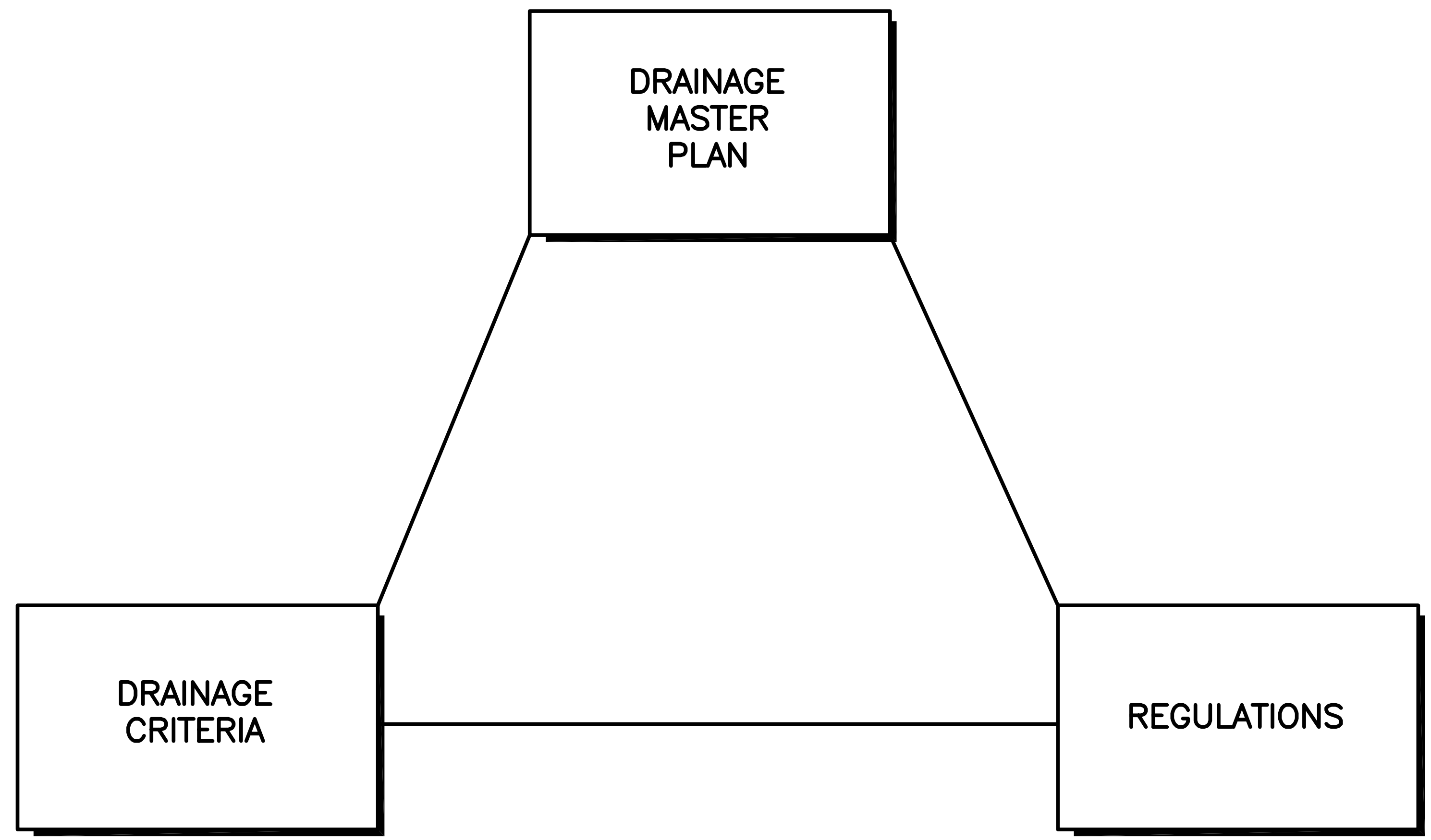
	CONCEPTUAL COST ESTIMATE	LEVEL OF PROTECTION	COMMENTS
ALTERNATIVE 1	$13,297,000		Flow conveyed by street and storm sewer
System 1	$5,870,000	100-Year	
System 2	$443,000	100-Year	
System 3	$6,984,000	100-Year	
ALTERNATIVE 2	$17,501,000		Flow conveyed by street and storm sewer
System 1	$7,170,000	100-Year	
System 2	$443,000	100-Year	
System 3	$9,888,000	100-Year	
ALTERNATIVE 3	$6,204,000		Flow conveyed by storm sewer only
System 1	$2,280,000	10-Year	
System 2	$455,000	10-Year	
System 3	$3,649,000	2- to 10-Year	

NOTES: In general, alternatives for each system are independent from other systems
The estimated costs are based in the 1999 currency value.

TABLE ES-2: COMPARISON OF ON-MOUNTAIN ALTERNATIVES

	CONCEPTUAL COST ESTIMATE	AVERAGE ANNUAL OPERATION AND MAINTENANCE EXPENSE	IMPACT ON MOUNTAIN AESTHETICS	RELATIVE RISK OF FAILURE	COMMENTS
ALTERNATIVE 1-CHANNEL/DRAIN	$10,969,000	Low	Low	Low	Stability analysis will need to be performed
ALTERNATIVE 2-CUTOFF WALL	$7,758,000	Medium	Low	Medium	Potential for erosion to expose walls
ALTERNATIVE 3-REGULATORY CONTROL	$0	High	None	High	Potential for 10's of millions of dollars of damage and loss of life. It will cost new development to implement regulations.

NOTES: In general, alternatives for each system are independent from other systems
The estimated costs are based in the 1999 currency value.



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CITY OF ASPEN
MASTER DRAINAGE PLAN

TOOLS OF COMPREHENSIVE
DRAINAGE PLAN

PROJECT NUMBER
1963

FIGURE
ES-1

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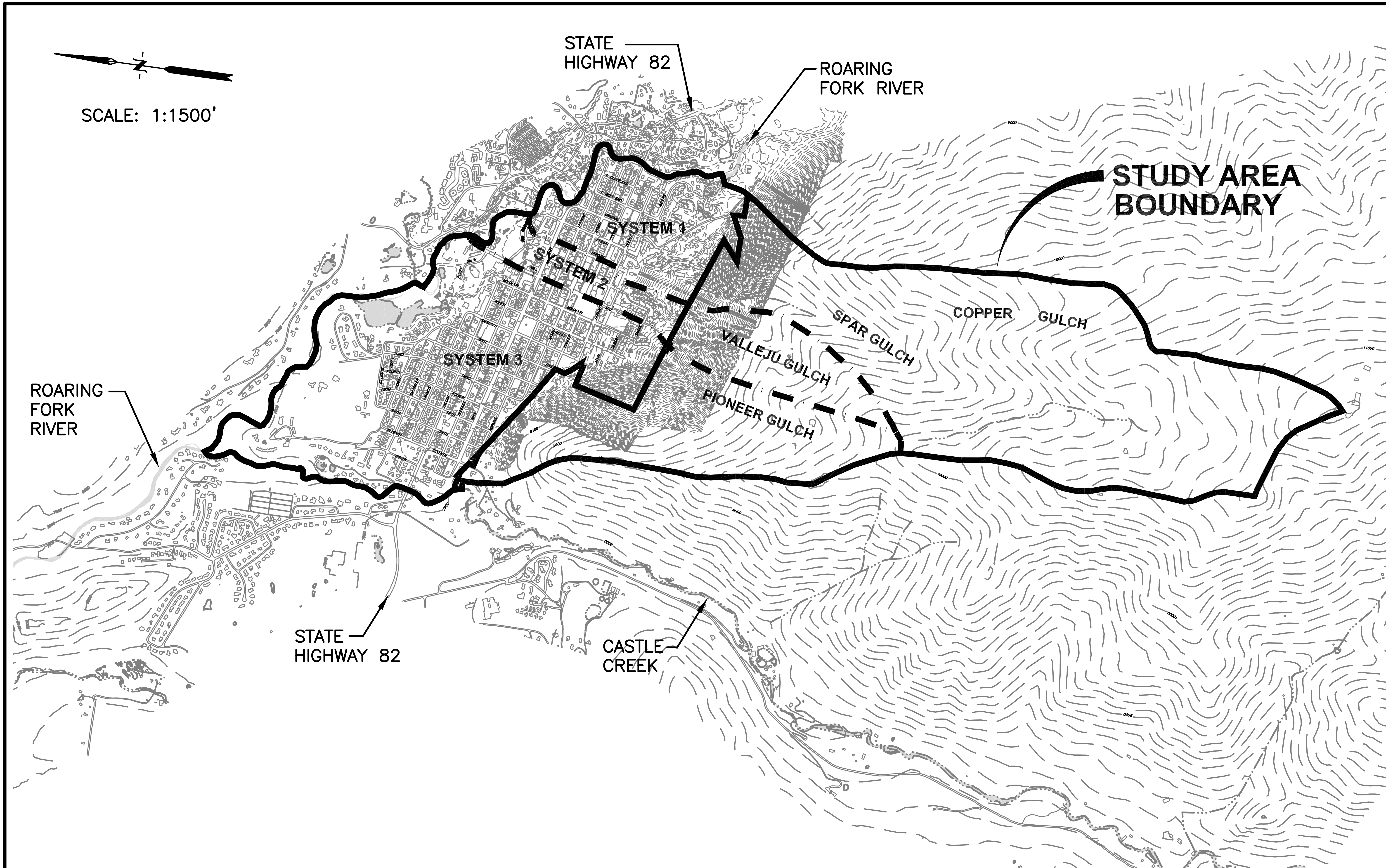

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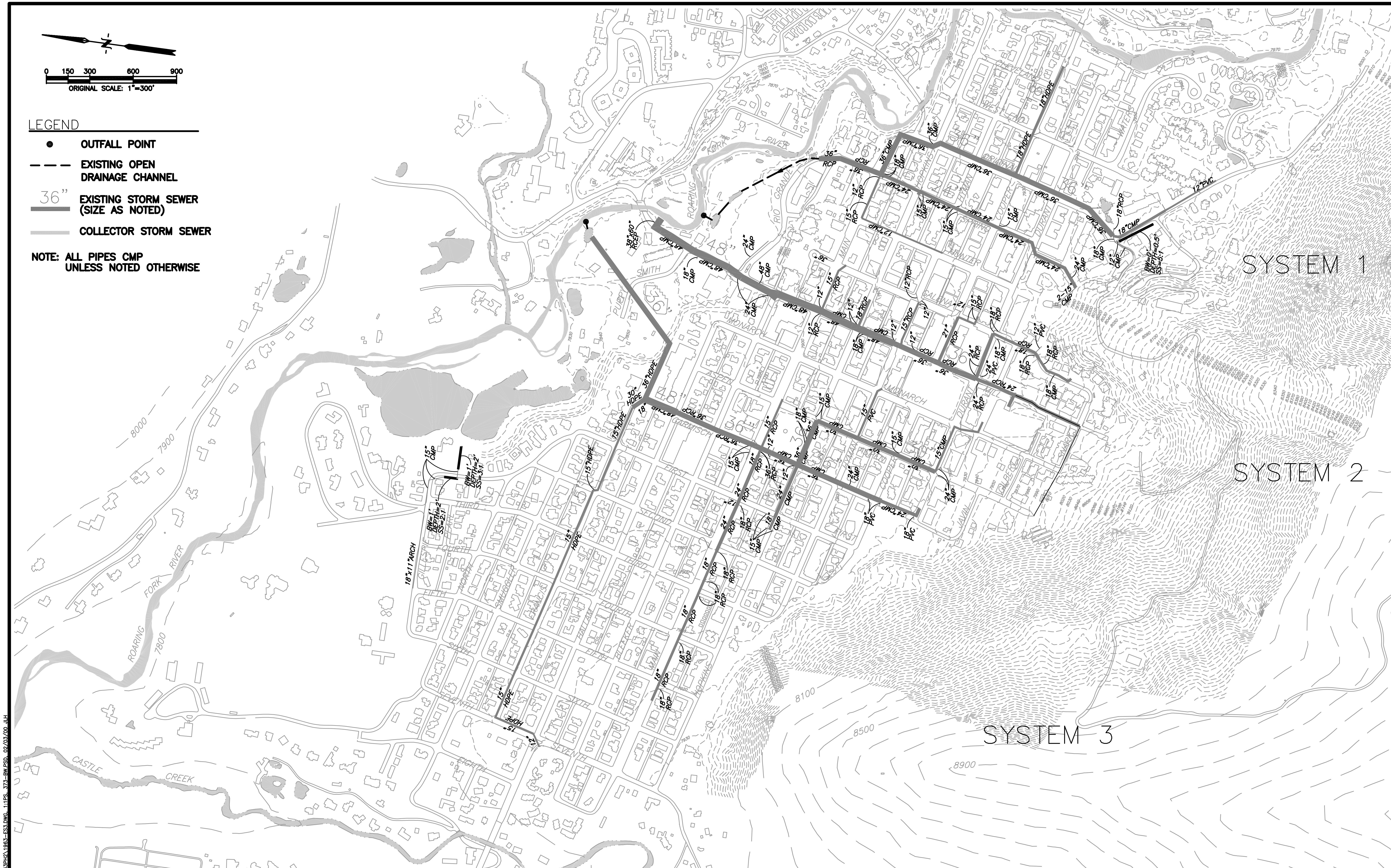

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1963\PH2\1963-ES3.DWG 1:1PS 373-BW.PSD 02/03/00 JLH

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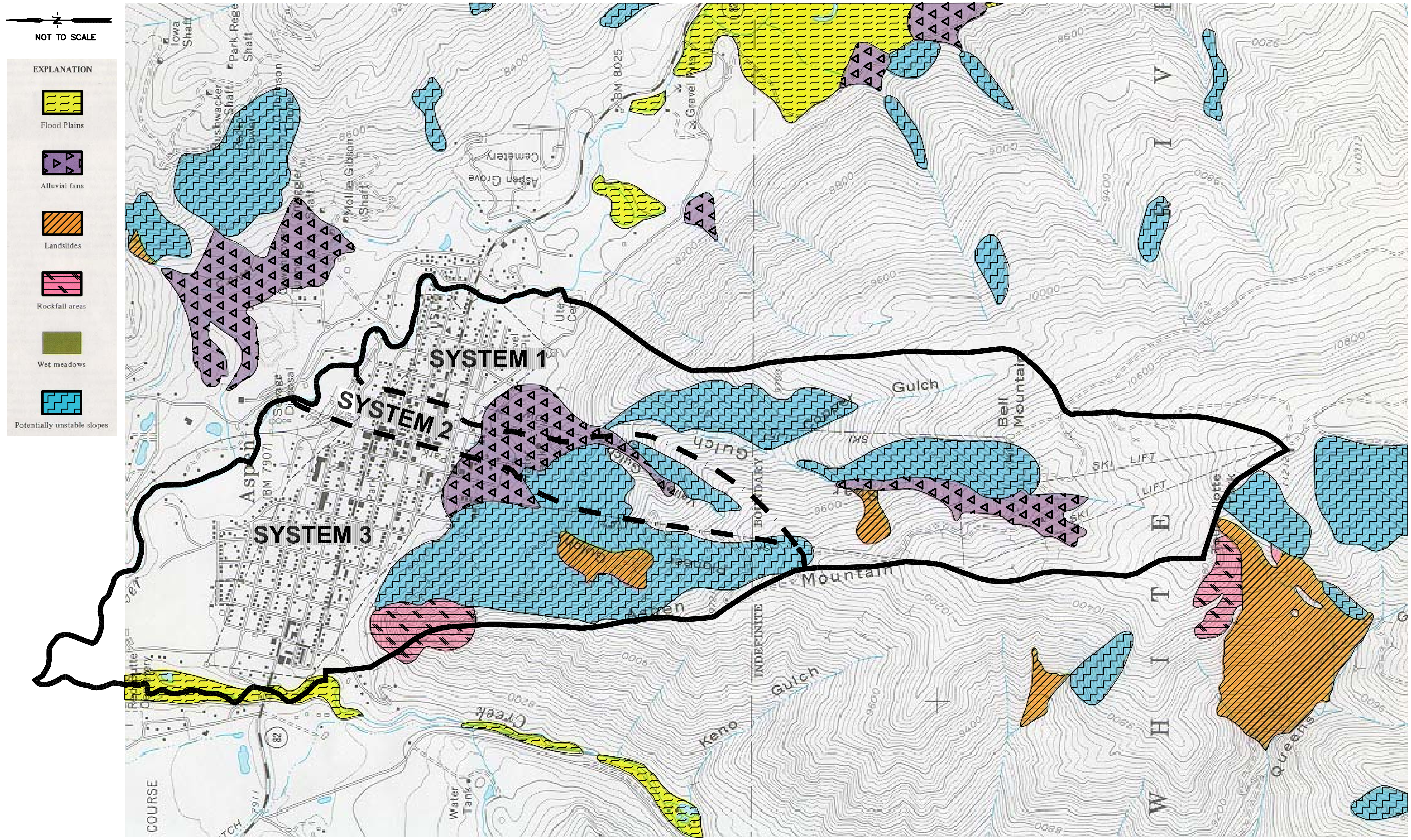
CITY OF ASPEN
MASTER DRAINAGE PLAN

MAGNITUDE OF FLOODING PROBLEMS

PROJECT NUMBER 1963

FIGURE ES-4



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CITY OF ASPEN
MASTER DRAINAGE PLAN

POTENTIAL GEOLOGIC HAZARDS AREA

PROJECT NUMBER
1963
FIGURE
ES-5

1963PH2\1963-ES5.DWG, 1=1PS, 367-2PSD, 02/03/00 JLH

1963PH2\1963-ES6.DWG, 1:2PS 1:300MS, 1050C-BW2.PC2, 350ARFWATER.PSD, 02/02/00 JLH

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CITY OF ASPEN
MASTER DRAINAGE PLAN

MAXIMUM FLOW DEPTH, 100-YEAR EVENT, WATER ONLY THROUGH CITY

PROJECT NUMBER
1963PH2

FIGURE
FS-6

1963PH2 1963-ES7.DWG, 1:1PS 1:300MS, 357-ES7.PSD, 1050C-BW2.PC2, 02/01/00 JLH

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CITY OF ASPEN
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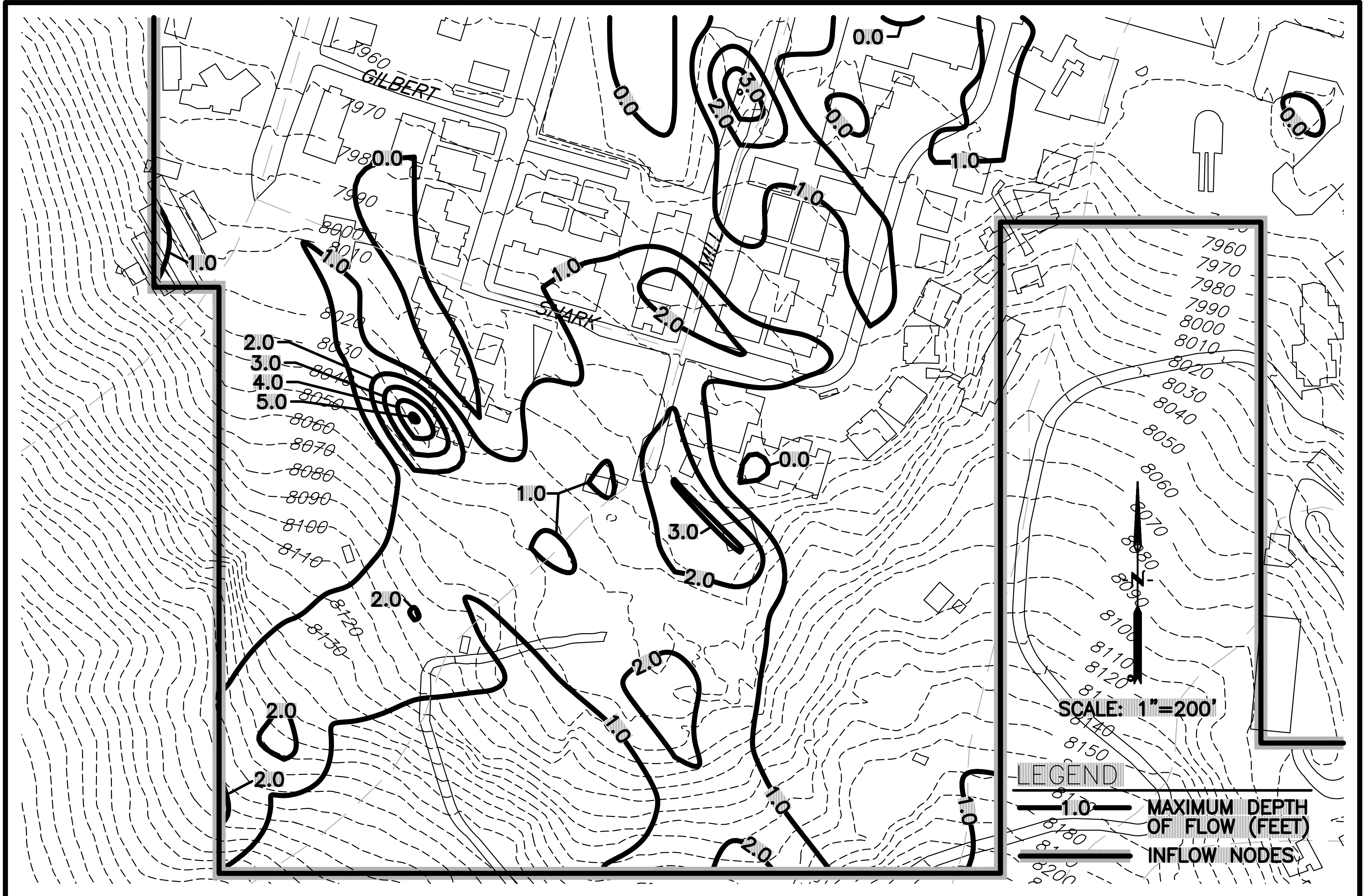
MAXIMUM FLOW DEPTH, 100-YEAR EVENT, MUDFLOW THROUGH CITY

PROJECT NUMBER 1963
FIGURE ES-7

1963PH2\1963-ES8.DWG, 1:1PS 1:200MS, LJET, 350DEPOLDTOPO.PSD, 02/02/00 JLH



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CITY OF ASPEN
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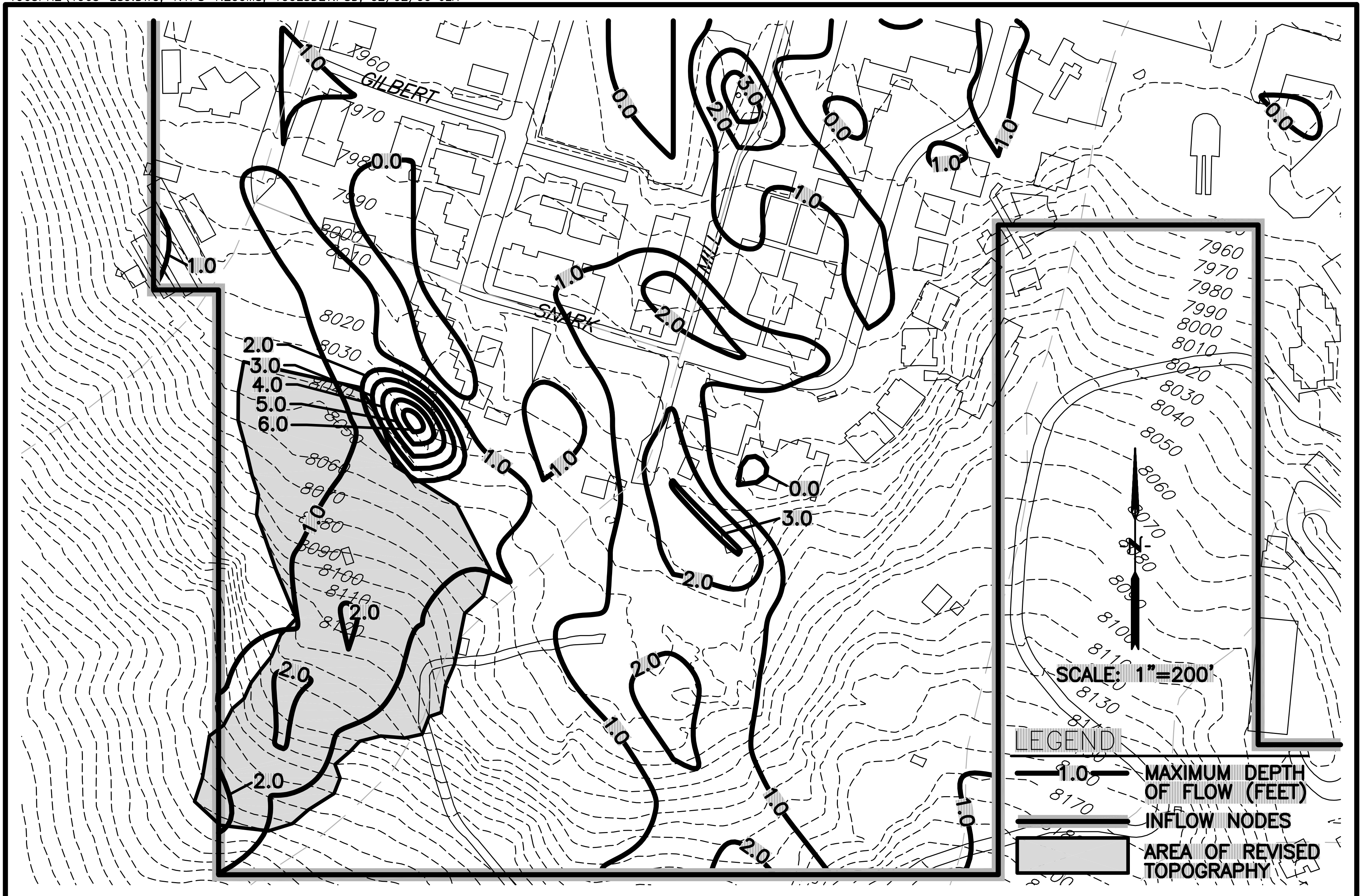
MAXIMUM FLOW DEPTH, 100-YEAR EVENT,
OLD TOPOGRAPHY

PROJECT NUMBER
1963

FIGURE
ES-8

1963PH2\1963-ES9.DWG, 1:1PS 1:200MS, 100ESDEV.PSD, 02/02/00 JLH



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CITY OF ASPEN
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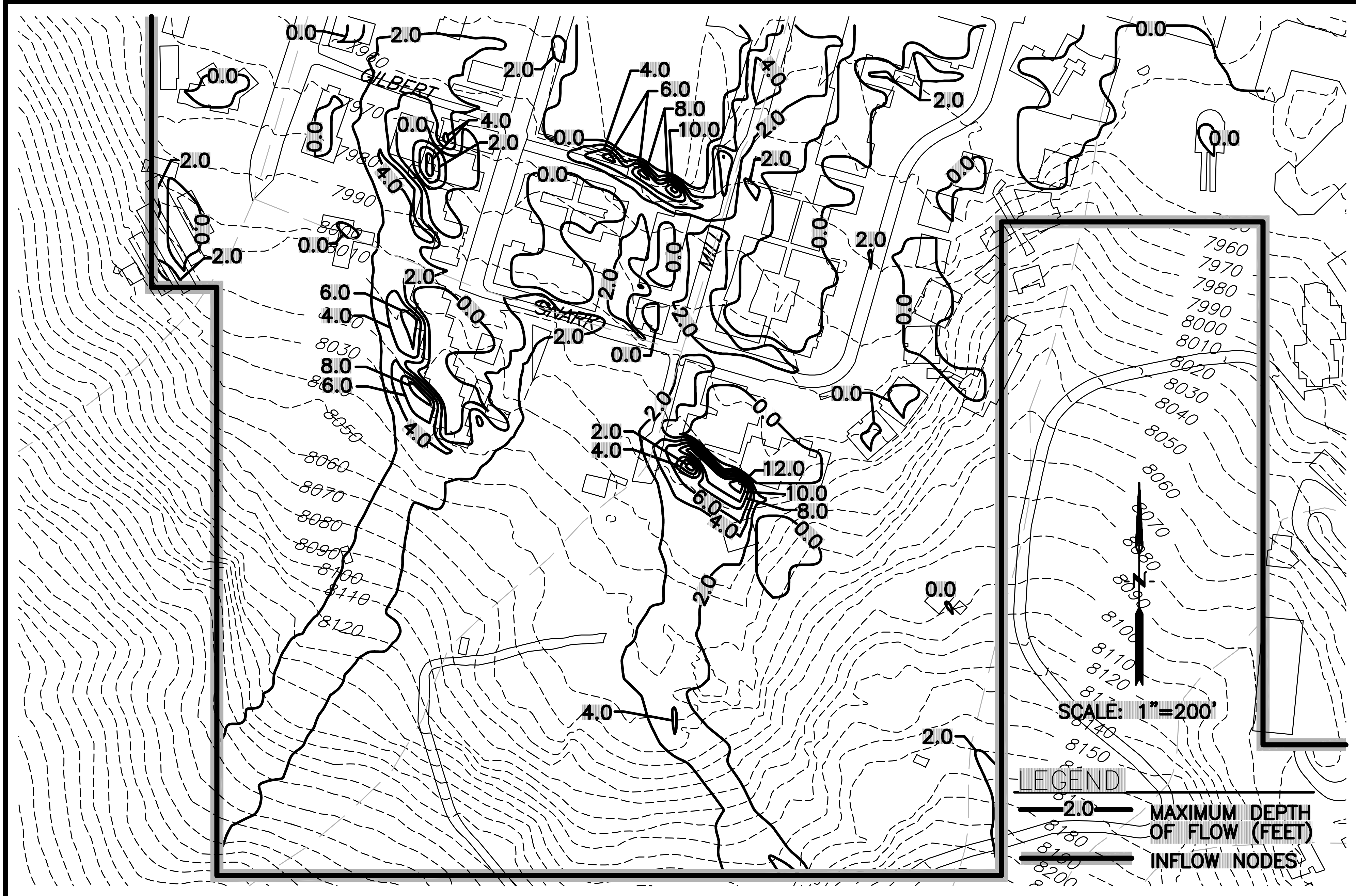
MAXIMUM FLOW DEPTH
100-YEAR EVENT, REVISED TOPOGRAPHY

PROJECT NUMBER
1963
FIGURE
ES-9

1963PH2\1963-ES10.DWG, 1:1PS 1:200MS, 25EXISTDEV.PSD, 02/02/00 JLH



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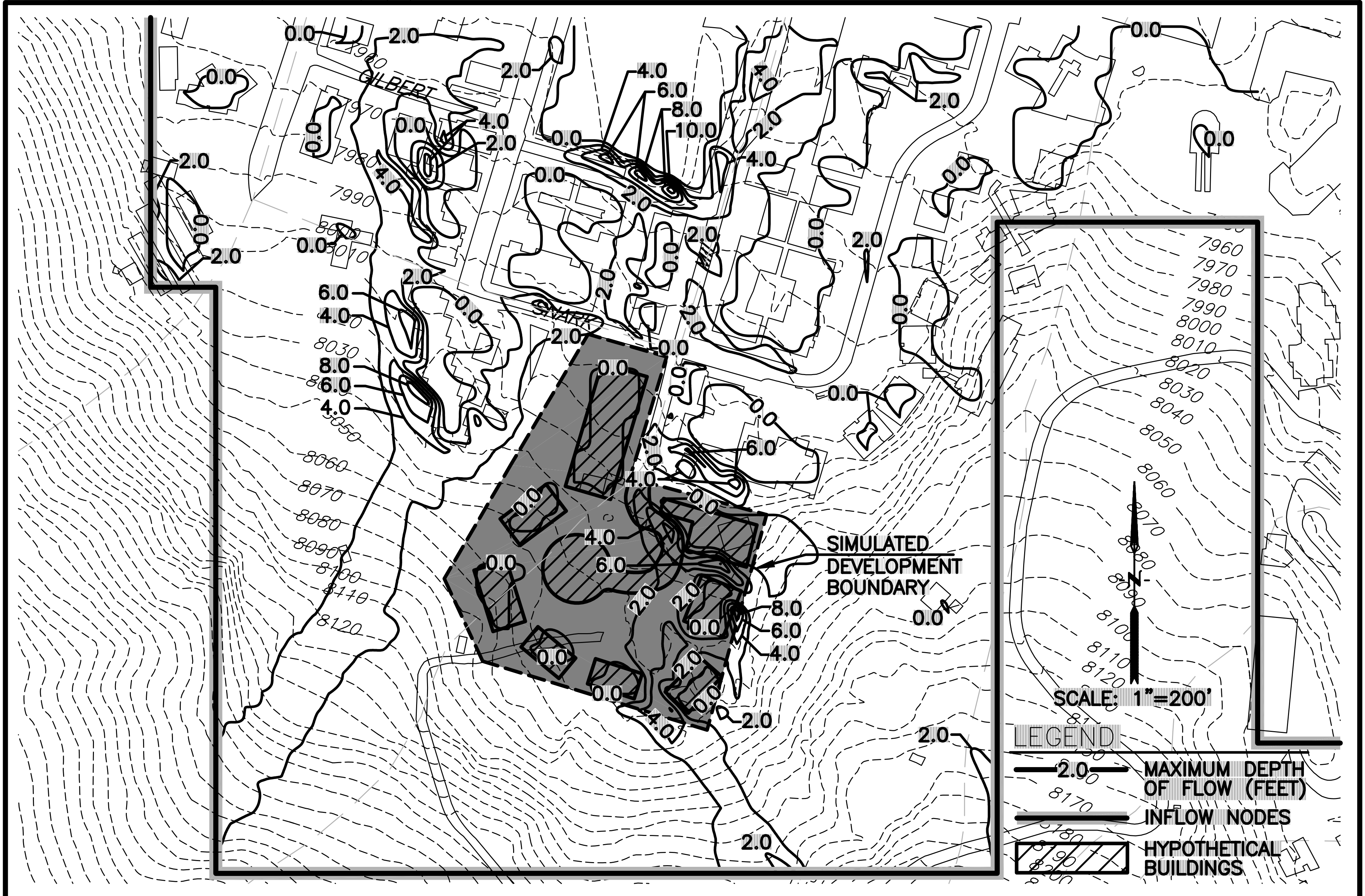
CITY OF ASPEN
MASTER DRAINAGE PLAN

MAXIMUM FLOW DEPTH
25-FOOT SPACING, EXISTING DEVELOPMENT

PROJECT NUMBER 1963

FIGURE ES-10

1963PH2\1963ES11.DWG, 1:1PS 1:200MS, 25NEWDEV.PSD, 02/02/00 JLH



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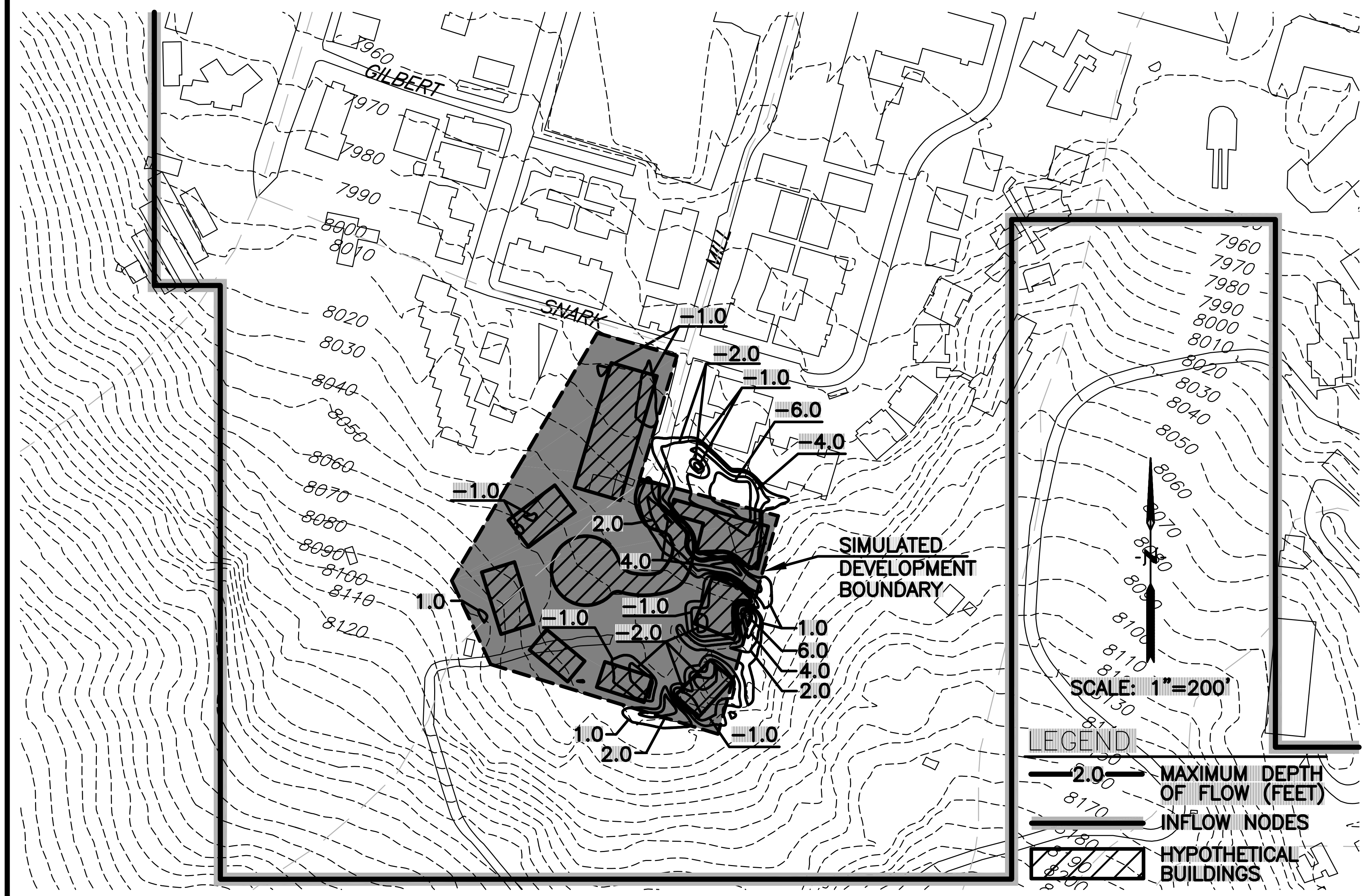
1963PH2\1963ES12.DWG, 1:1PS 1:200MS, CHANGE-25.PSD, 02/02/00 JLH



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CITY OF ASPEN
MASTER DRAINAGE PLAN

CHANGE IN FLOW DEPTH DUE TO
HYPOTHETICAL DEVELOPMENT
25-FOOT SPACING

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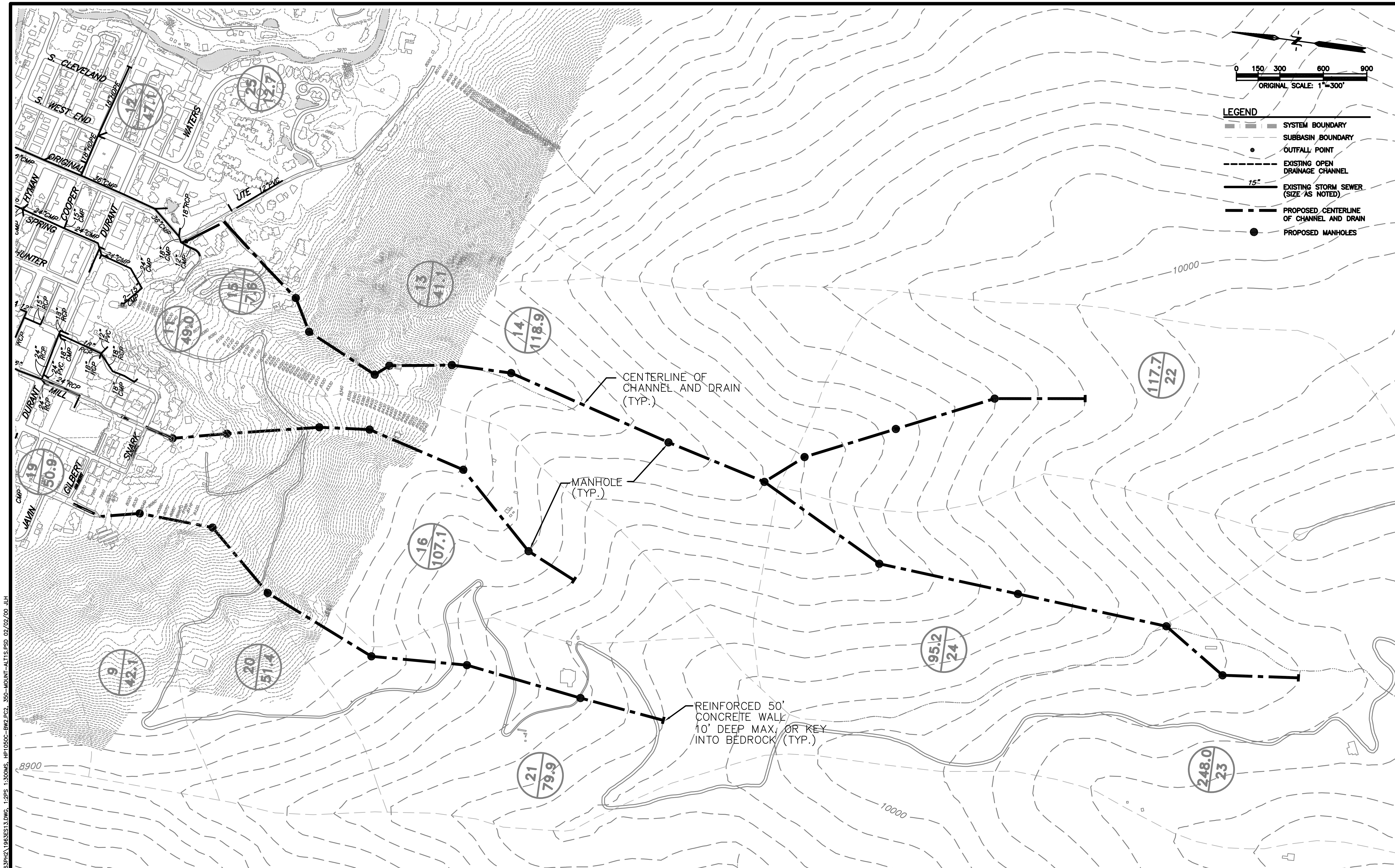
FIGURE
ES-12



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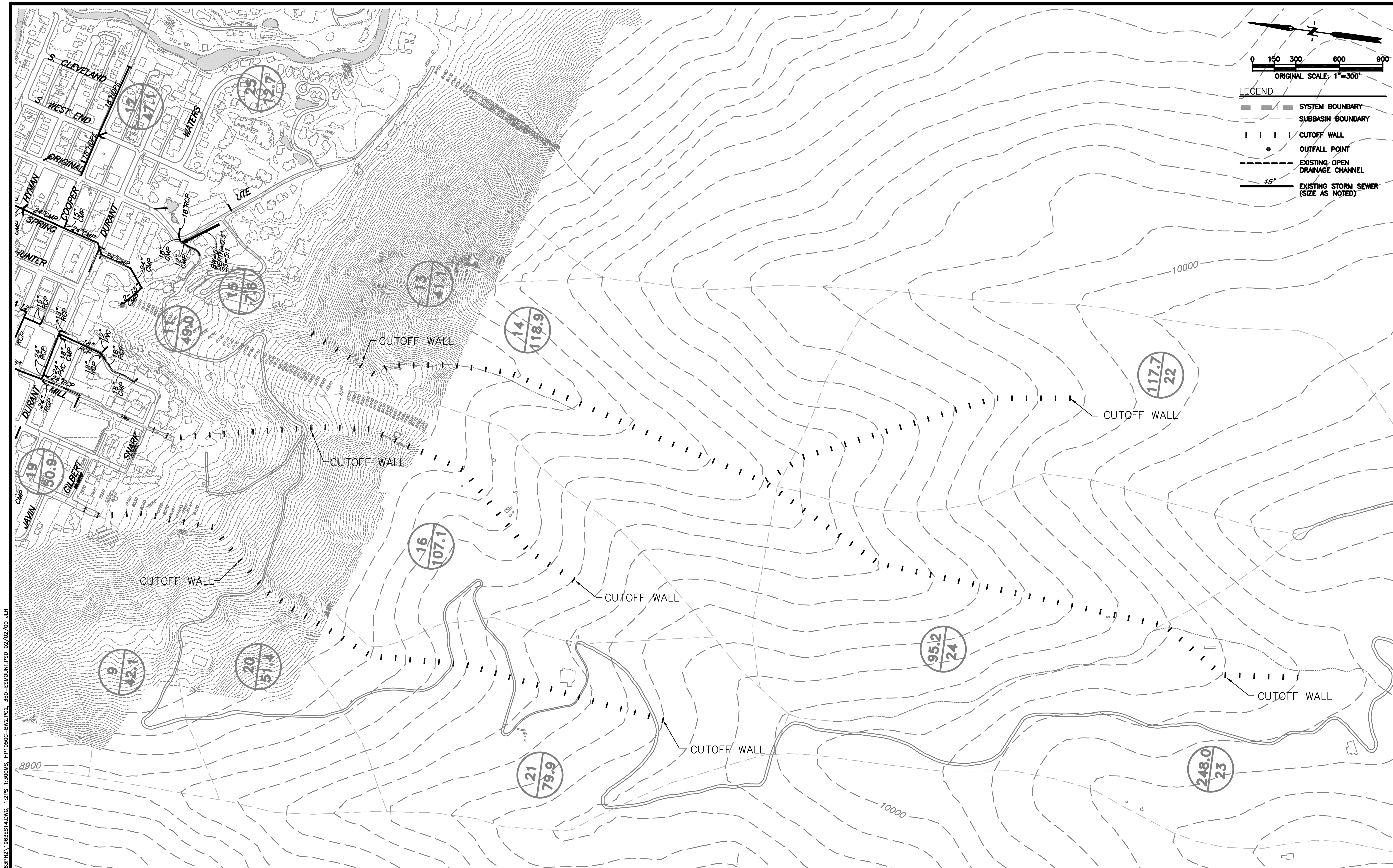
ALTERNATIVE NO. 1 – ON MOUNTAIN (CHANNEL/DRAIN ALTERNATIVE)

PROJECT NUMBER 1963PH2

FIGURE ES-13



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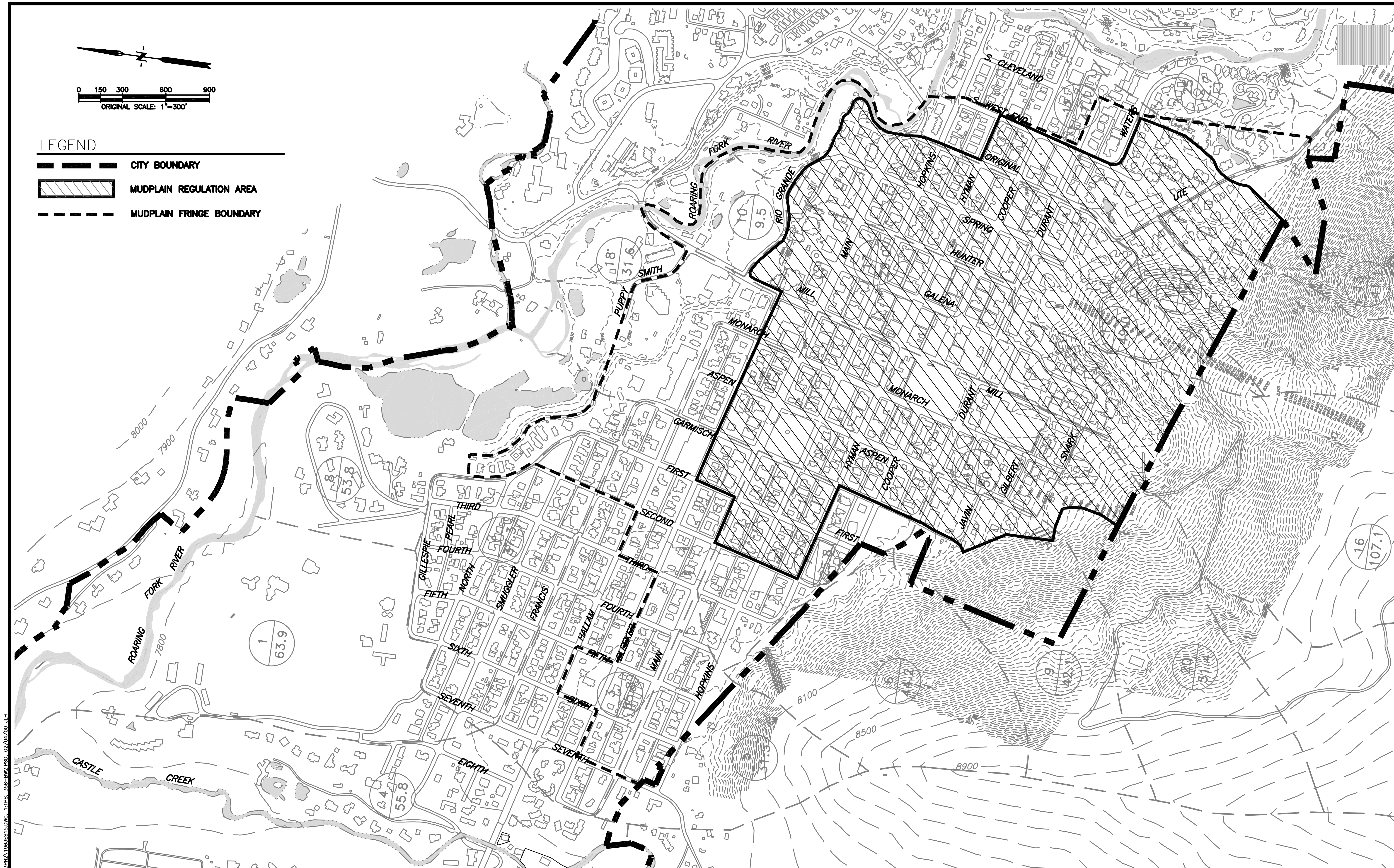
ALTERNATIVE NO. 2 – ON MOUNTAIN
(CUTOFF WALL ALTERNATIVE)

PROJECT NUMBER 1963PH2

FIGURE ES-14



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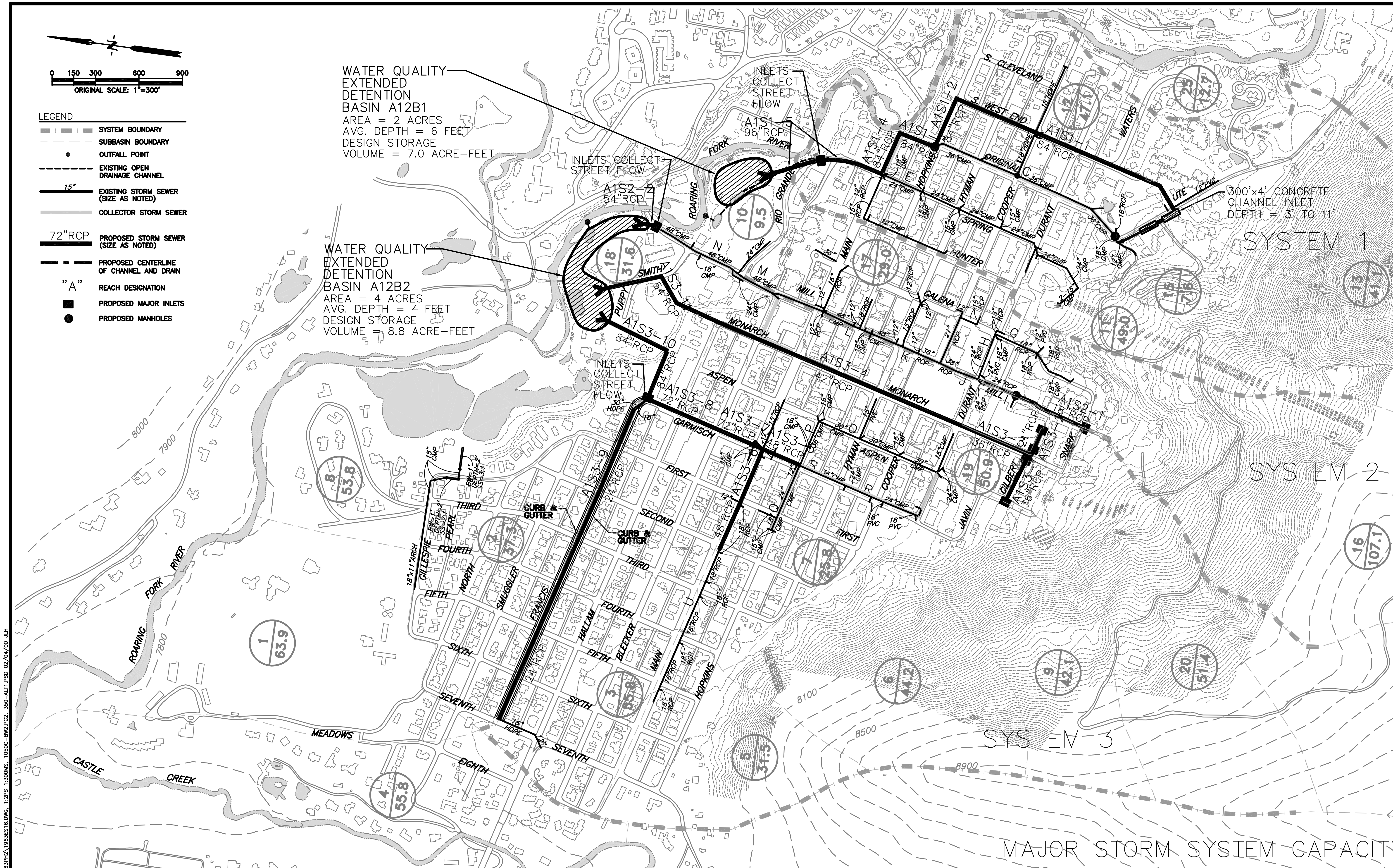

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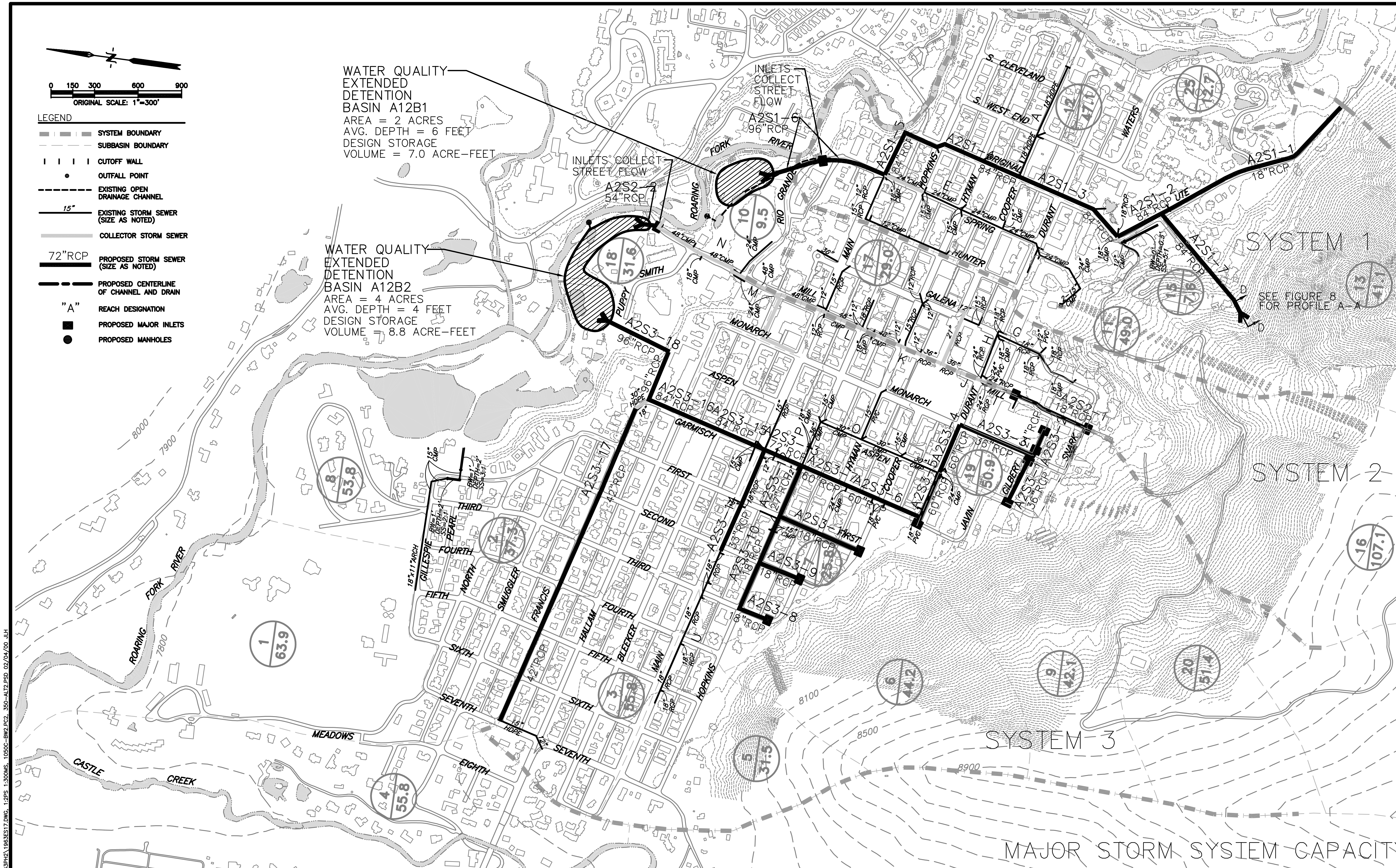
CITY OF ASPEN
MASTER DRAINAGE PLAN

ALTERNATIVE NO. 1 – IN CITY
(ADDITIONAL STORM SEWER ALTERNATIVE)

PROJECT NUMBER
1963PH2

FIGURE
ES-16



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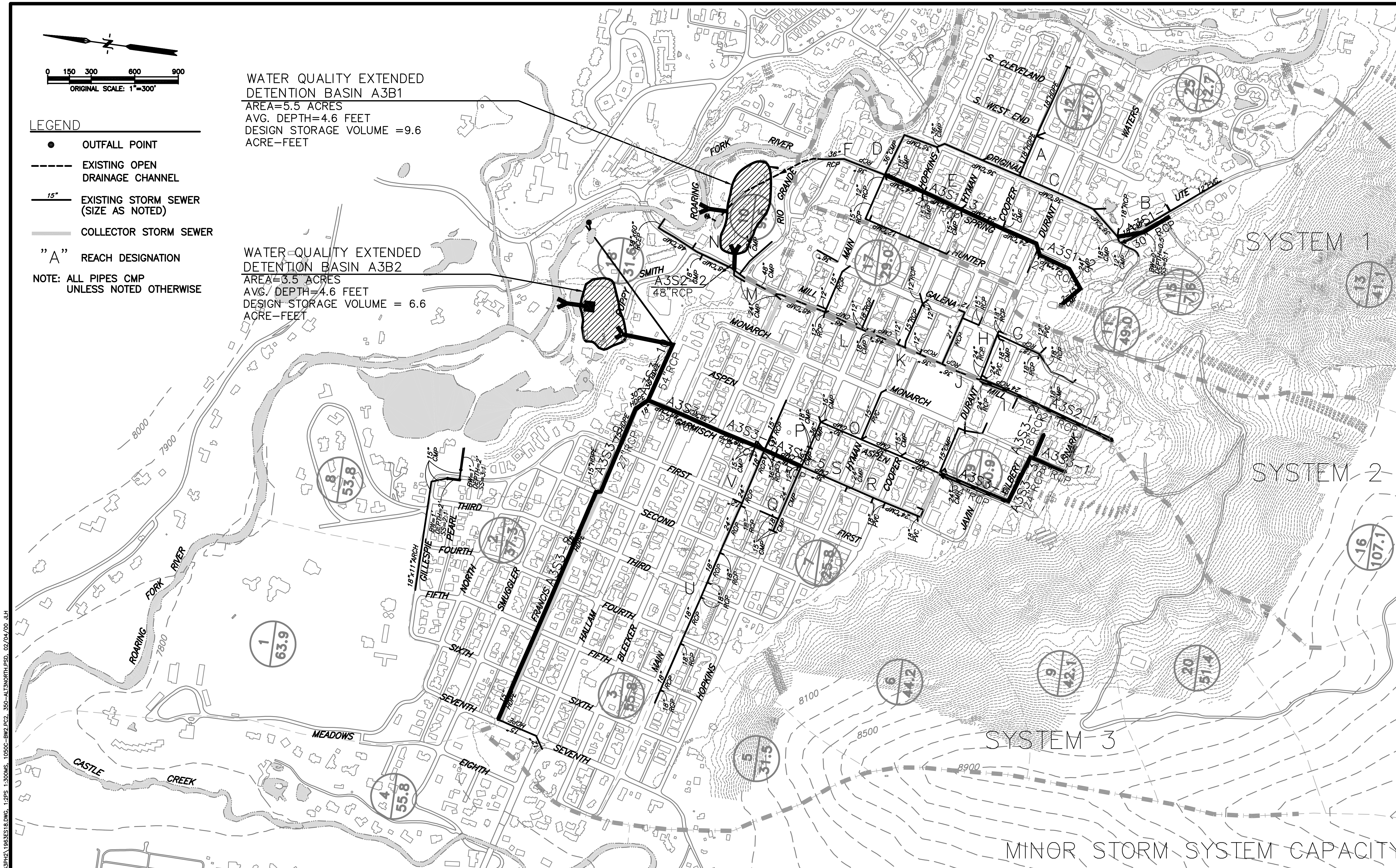
CITY OF ASPEN
MASTER DRAINAGE PLAN

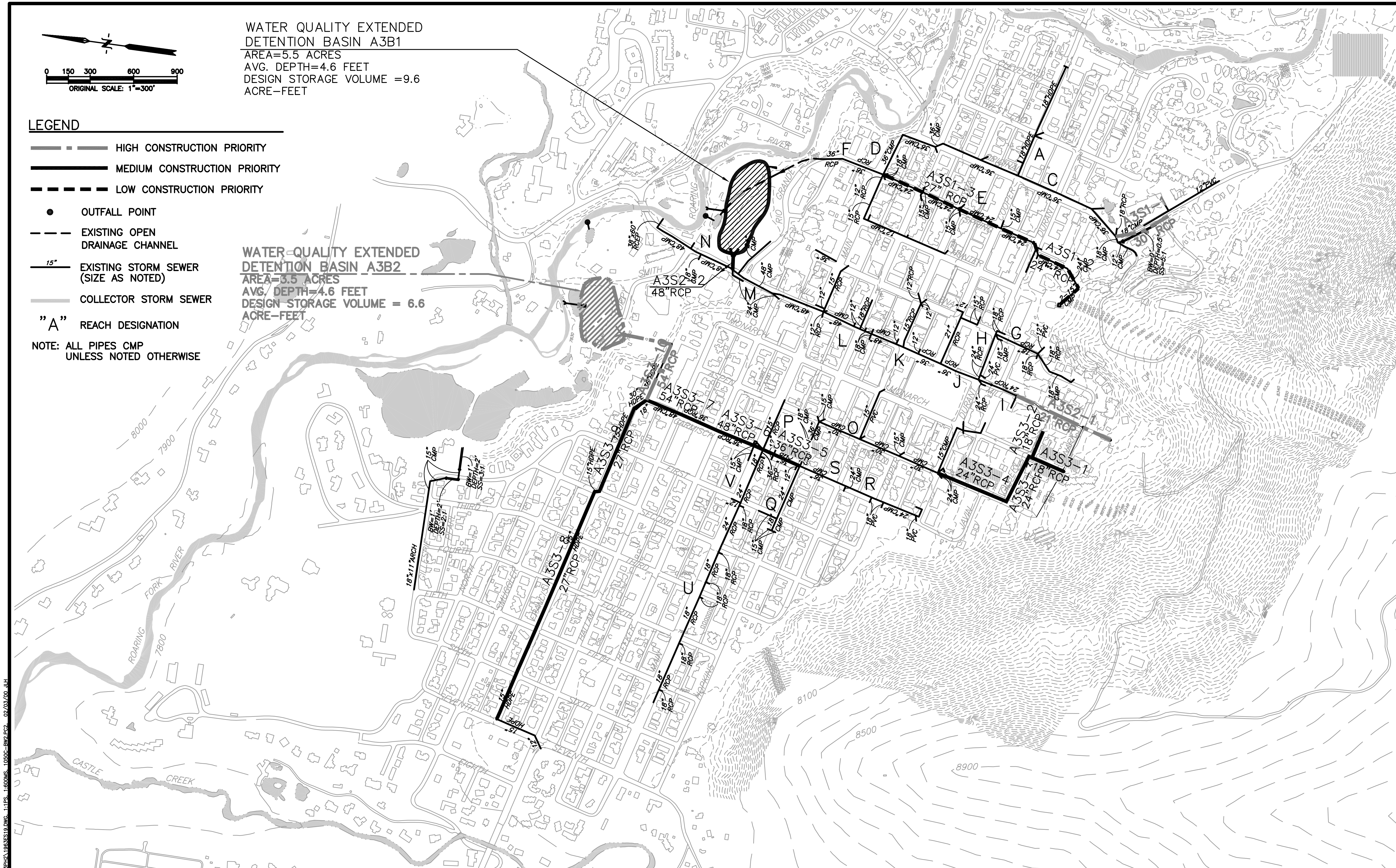
ALTERNATIVE NO. 2 - IN CITY
(REPLACEMENT STORM SEWER ALTERNATIVE)

PROJECT NUMBER 1963PH2

FIGURE ES-17



WRC ENGINEERING, INC. 950 SOUTH CHERRY STREET SUITE 404 DENVER, COLORADO 80246 PHONE NO: (303) 757-8513 FAX NO: (303) 758-3208	DESIGNED BAC DRAWN JLH CHECKED AJL REVISED AS-BUILT	NO.	BY	DATE	REVISION DESCRIPTION	CITY OF ASPEN MASTER DRAINAGE PLAN	ALTERNATIVE NO. 3 – IN CITY	PROJECT NUMBER 1963PH2 FIGURE ES-18



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NO.	BY	DATE	REVISION DESCRIPTION

CITY OF ASPEN
MASTER DRAINAGE PLAN

CONSTRUCTION PRIORITY OF
RECOMMENDED IN-CITY ALTERNATIVE

PROJECT NUMBER 1963
FIGURE ES-19

STORM DRAINAGE MASTER PLAN
FOR
THE CITY OF ASPEN, COLORADO

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I. INTRODUCTION

A. AUTHORIZATION

This report is part of a study authorized by the City of Aspen to provide comprehensive stormwater planning for the City of Aspen. The study area, shown in Figure 1 through 4, consists of the watershed above and through the City of Aspen, Colorado, which encompasses a portion of the City of Aspen and unincorporated parts of Pitkin County.

The study is divided into two phases. The first phase (Phase I) covers the hydrological, hydraulic, and mudflow analyses for the study area. The second phase (Phase II) covers the alternative evaluation and conceptual design of the selected alternative. The results of both of these phases are included in this report. An additional portion of Phase II is the preparation of a Storm Drainage Criteria Manual to include stormwater and mudflow criteria which is submitted separately from this report.

B. PURPOSE AND SCOPE

The purpose of this study is to prepare a drainage master plan that identifies the drainage improvements needed to address the stormwater runoff problems that exist in the developed portion of the City and the available alternatives that would address the mudflow potential from Aspen Mountain.

The general scope of Phase I of this project is as follows:

1. Gather and assemble available information on existing drainage facilities, comprehensive plans, land use plans, zoning and land ownership maps, and other applicable information.

2. Perform a field investigation to identify all major drainage structures and environmentally sensitive areas.

3. Determine the hydrologic aspects of the drainage area including runoff rates and volumes under existing conditions for various return periods of storm events.

4. Evaluate the hydraulic capacity of the existing drainage systems and the relative level of protection provided by these facilities.

5. Identify existing and potential future drainage problems within the study area by comparison of anticipated future runoff rates with the hydraulic capacity of existing and current planned systems.

6. Estimate the magnitude and path of potential debris and mudflow events that could occur for a range of runoff events

The general scope of Phase II of this project is as follows:

1. Develop alternative plans for addressing the identified stormwater drainage problems.

2. Develop alternative plans for addressing the potential mudflow problems.

3. Evaluate stormwater and mudflow alternatives using factors such as cost, public acceptance, cost effectiveness, applicability, stormwater quality benefits, and public health and safety to result in a specific recommendation for further actions and/or improvements.

4. Prepare preliminary plan and profile drawings for the selected alternative.

5. Prepare preliminary cost estimates for the proposed facilities.

6. Prepare a Storm Drainage Criteria Manual specifically for the City of Aspen.

7. Prepare mudflow criteria for the City of Aspen.

8. Prepare a final report presenting the master plan and preliminary design information.

C. MAPPING AND SURVEYS

Two sets of topographic maps were supplied by the City of Aspen for this project. One topographic map covered Aspen Mountain at a contour interval of 100 feet and the other covered the City at a 10-foot contour interval. The maps were submitted in electronic format in an AutoCAD Version 12.0 drawing.

Soil surveys prepared by the U.S. Department of Agriculture Soil Conservation Service were used to determine soil types and hydrologic soil groups. The locations of city parks, electric lines, water lines, and storm sewers were provided by the City.

D. ANALYSIS AND DESIGN CRITERIA

The Master Plan analysis was performed in accordance with the guidelines and criteria set forth in the proposed Aspen Storm Drainage Criteria Manual. This Criteria was modified slightly as the project progressed, so there are slight differences in the criteria used to estimate the capacity of the existing facilities and the criteria used to perform the conceptual design of the chosen alternative.

Subsequent to the initial data gathering and field reconnaissance work to establish watershed boundaries and existing storm drainage features, the study area was divided into separate sub-watersheds within the major watershed. The physical parameters (i.e., area, length, slope, and percent imperviousness of each sub-watershed for future development conditions) were then defined and the design storm hydrographs developed for the various recurrence intervals using the Colorado Urban Hydrograph Procedure. The combining of the hydrographs, channel routing, and routing of the flood hydrographs through the existing ponds were performed using the Storm Water Management Model.

Water quality treatment of non-point source runoff from the City of Aspen are currently not required by the U.S. E.P.A., but since water quality facilities will be required in the future to treat these non-point source discharges, they were included in the proposed alternatives. The proposed water quality facilities will also help maintain the aesthetic beauty of the Roaring Fork River.

E. PROJECT ADVISORY COMMITTEE

This study is sponsored by the City of Aspen Engineering Department. Various community groups and citizens also provided input to the project. The following agencies, companies and citizen groups were notified and invited to participate in the progress meetings:

ORGANIZATION

City of Aspen Planning and Zoning Department
City of Aspen Parks Department
City of Aspen Street Department
Pitkin County
Aspen Ski Company
Savanna Ltd.

F. ACKNOWLEDGMENTS

We would like to acknowledge the participation of Nick Adeh from the City of Aspen Engineering Department who provided insights, information, and guidance that were integral to the analysis and preparation of this Master Plan.

II. STORMWATER RUNOFF AND DRAINAGE FACILITY CAPACITY ANALYSIS

A. INTRODUCTION

The purpose of this section is to analyze the path and the magnitude of the runoff of different frequency storm events through the City of Aspen and the capabilities of the existing drainage facilities to convey this flow to the Roaring Fork River. This section is presented as the initial portion of the Storm Runoff Master Plan Study that WRC Engineering is currently preparing for the City of Aspen. This section will discuss the existing conditions with in the City and identify areas of concern in respect to storm runoff conveyance. Future documents will present alternatives to alleviate possible drainage problems and other aspects of the Master Plan Study.

B. STUDY AREA

The City of Aspen is located in Township 10 South, Range 85 West of the 6th Principal Meridian, in Pitkin County. The City lies at the base of the north side of Aspen Mountain adjacent to the Roaring Fork River. Runoff from the north side of the mountain flows through the City on its way to the Roaring Fork River. The Roaring Fork River empties into the Colorado River at Glenwood Springs.

The selected study area is bounded on the north by the Roaring Fork River, on the west by the drainage boundary of Castle Creek, and on the east and south by the drainage basins on Aspen Mountain tributary to the City. The 2.5 square mile study area for this project includes the Spar, Pioneer, Vallejo, and Copper Gulches to the south. The study area also includes that portion of the City of Aspen that is bounded by the Roaring Fork River to the north and Castle Creek to the west. Surface runoff from the study area generally travels to the north and outfalls into the Roaring Fork River.

WRC used storm sewer information, field data, and topographic mapping to subdivide the study area into 25 sub-watersheds. The largest sub-watershed size is 248 acres (Spar Gulch) and the smallest is 9.5 acres (Sub-watershed 10 which flows directly into the Roaring Fork River). The sub-watershed delineations are presented in Figures 1 through 4.

The mountain sub-watersheds that drain into the City are divided into Pioneer Gulch, Lower Spar Gulch, Upper Spar Gulch, Vallejo Gulch, and Copper Gulch (designated by WRC as Sub-basins 21, 14, 23, 16, and 22, respectively).

The Summer Ditch (originally constructed by miners in the 19th century to divert runoff from mining claims) diverts upstream Spar Gulch runoff to Keno Gulch. The capacity of the Summer Ditch (as limited by inlet control on the downstream 18-inch culvert) has been previously estimated by others at 11 cfs. For this study, it was conservatively assumed that the entire runoff from Sub-basin 23 flows into Sub-basin 24 (no diversion by the Summer Ditch).

Based on the information gathered, WRC has identified nine general areas where runoff enters the City from the mountain watersheds to the south. These areas correspond to low points on Sub-watersheds 13, 14, 15, 16, 20, 9, 6, and 5 (See Figures 2 and 3). Flow from Sub-watershed 13, 14, and 15 is intercepted by Ute Avenue, continues to the northwest, eventually enters a major storm sewer trunk line along Original Street, and passes through a series of water quality ponds before outfalling into the Roaring Fork River.

Runoff from Sub-watershed 16 does not appear to enter the City in a single concentrated channel. However, the street layout, topography, and location of inlets and trunk lines indicate that runoff from this sub-watershed will concentrate at Durant Avenue and Mill Street. Runoff from Sub-watershed 16 enters the storm sewer trunk line along Mill Street prior to outfalling into the Roaring Fork River.

Similarly, runoff from Sub-watersheds 20, 9, 6, and 5 appear to lack a single well-defined point of entry. Runoff from Sub-watershed 6 is partially impeded by the existing bike trail embankment. Runoff from Sub-watershed 5 is intercepted along Hopkins Avenue. Runoff from these sub-watersheds is collected by the streets and storm sewers in the southwest portion of Aspen and routed to the storm sewer trunk line along Garmisch Street. This trunk line then empties into the Roaring Fork River.

Based on the information gathered, there appears to be three existing major outfalls to the Roaring Fork River (Figures 5 and 6). These outfalls occur at the termination of storm sewer lines, which extend along Spring Street, Mill Street, and Garmisch Street.

Existing storm sewer system pipes range in size from 12 inches to 48 inches and vary in type from High-Density Polyethylene (HDPE) plastic to reinforced concrete to corrugated metal. The majority of the existing pipes appear to be corrugated metal.

C. HYDROLOGY

The Colorado Urban Hydrographic Procedure (CUHP) was used to analyze the runoff from each sub-watershed. Table 1 summarizes the hydrologic characteristics of each sub-watershed. The data provided in Table 1 was used as input to the model. The output from the model are the runoff hydrographs from each sub-watershed, which are then used as input to the Storm Water Management Model (SWMM).

Rainstorms that have a duration that exceeds six-hours can produce large amounts of total precipitation; however, these storms are rarely intense and seldom result in urban flooding problems. Very intense rainfall in the Aspen area usually results from convective storms and frontal stimulated convective storms. These types of storms are often less than one-hour or two-hours in duration and can produce brief periods of high rainfall intensities. These short duration intense rainstorms appear to cause most of the urban flooding problems.

The recommended design storm distribution takes into account the observed "leading intensity" nature of the convective storms. In addition, the temporal distributions were designed to be used with CUHP, the published NOAA one-hour precipitation values, and the Horton's infiltration loss equation. They were developed to approximate the recurrence frequency of peak flows and volumes (i.e., 2-Year through 100-year) that were estimated for the watershed for which rainfall/runoff data was collected. The procedure for the design storm distributions and the preliminary results were reported at the 1979 International Symposium on Urban Storm Runoff.

Table 2 presents the rainfall distribution for a two-hour storm event over the City of Aspen for the 2-year, 5-year, 10-year, 50-year, and 100-year frequencies. Table 3 supplies the results of the CUHP modeling and provides the peak flows from each sub-watershed shown in Figures 1 through 4.

D. HYDRAULICS

The Stormwater Management Model (SWMM) program was used to route the hydrographs from one sub-watershed to the next along drainage routes or reaches. These reaches can be modeled as storm sewers, channels, street flow, or a combination of the various types of reach elements. Figure 5 shows a schematic drawing of the SWMM routing, and Figure 6 provides the design points and reach elements used in SWMM. Table 4 supplies the characteristics of each drainage reach.

Table 5 provides the peak flow for each return period along each drainage reach and design point in the City of Aspen. In general, these drainage reaches correspond to the major drainage facilities (storm sewers, streets, and channels) in Aspen. The exceptions are the reach along Garmisch Street from Hopkins to Main and the reach along Mill Street from Main to Bleeker. Flow along these reaches will be discussed in the following section.

E. DRAINAGE SYSTEM CAPACITY

The existing drainage system conveying runoff from Aspen Mountain and the City to the Roaring Fork River is comprised of storm sewers, streets (serving as open channels), and open channels. The SWMM model provides a general flow analysis from Aspen Mountain and through the City of Aspen. To provide a greater level of detail and a better analysis of the existing drainage facilities within the City, the SWMM drainage reaches were further subdivided within the City. The reaches defining the existing City drainage facilities are shown in Figure 7. Each of the major drainage facilities in Aspen is designated by a letter.

Since the flow rate could not be determined directly from the SWMM output for some of the drainage facilities (Garmisch Street from Hopkins to Main and Mill Street from Main to Bleeker), the flow along these "sub-routes" was determined indirectly. The unknown flow at a point in a basin can be determined from the known flow downstream of the location and the drainage areas at the desired location and at the downstream location. This relationship is shown below (USBR, Flood Hydrology Manual, 1989):

$$Q_2 = \frac{Q_1 * A_2^{0.5}}{A_1^{0.5}}$$

where:

Q_1 = Known flow from SWMM downstream from location 2
A_1 = Drainage area input to SWMM at point downstream of location 2
A_2 = Drainage area at point location 2 (planimetered)
Q_2 = Flow at location 2

Table 8 shows the flow at each of the sub-routes or City drainage facilities for the 2-year, 5-year, 10-year, 50-year, and 100-year storm events.

Table 6 provides the capacity of the major storm sewers in the City of Aspen, which correspond to a drainage reach. For the purposes of this study a major storm sewer was defined as having an 18-inch diameter or greater. The exception to this was the 18-inch storm sewer on Hopkins from Hunter to Spring. Due to its short length and small drainage area, this storm sewer was defined as a lateral and was not included in the analysis.

Table 7 supplies the capacity of the street flow along the drainage reaches. It was assumed, for the purpose of this preliminary analysis, that the street flow would be limited to a 6-inch depth at the curb. No flow into the adjacent yards would be permitted. It was further assumed that the cross slope of the streets was 2%. The street width was scaled from Figure 7.

If the street does not have curb and gutters and runoff could flow into garages along the street, there is actually no street flow capacity, and the total system capacity will be the storm sewer capacity.

The only channel conveying runoff is a small swale adjacent to Ute Avenue from Aspen Alps South Road to Original Street. The capacity of this channel is 2.4 cfs. Since there was only one channel a separate table to show the channel capacity for each reach was not included.

F. RESULTS

A comparison of the magnitude of the flow events for a given return period was made to the capacity of the existing drainage facilities. Based upon this comparison, the following observations were made:

- The capacity of the drainage system along Original Street is insufficient to convey the 50-year and the 100-year storm events. The storm sewer, alone, should be able to transport the 5-year flow event.

- The drainage facilities along Spring Street south of Main Street have the capacity to convey the 100-year runoff event, but the storm sewer has only a 2-year capacity. The drainage facilities downstream of Main Street do not have the capacity to convey the 50-year flood event. This storm sewer should be able to transport the 10-year flow event.

- In general, the drainage facilities along Mill Street appear to have sufficient capacity to convey the 100-year flood. The exceptions would be Reach H (Durant Street). The storm sewer system alone has capacity to convey the 5-year flow event.

- The Garmisch Street conveyance system appears to be undersized. Depending on the reach, it will not pass runoff between the 10-year to 50-year event without flooding. The storm sewer, by itself, does not have the capacity to pass even the 2-year flow event.

- Since most of Francis Street does not have curb and gutter, the Francis Street drainage system will not pass the 2-year flow event without flooding. The outlet of the Francis Street and Garmisch Street systems (Reach S) is also greatly undersized and can only convey the 2-year flow event. There is no street flow in Reach Z.

Table 8 provides a comparison and summary of the analysis of the existing flow and the existing drainage system capacity.

G. DISCUSSION

The calculations shown in Tables 6, 7, and 8 provide a general analysis of the drainage facilities in the City of Aspen. This analysis did not consider debris in the runoff. If debris is transported by the runoff, the capacity of the storm sewers and streets will be reduced. The inlets to the storm sewers may also be a limiting factor in some areas for the capacity of the storm sewer system. Our analysis of the storm sewer system assumed that the pipes would be flowing full under the given conditions. In reality, the extremely high velocities in some of the storm sewers will create a high hydraulic head requirement that will limit the amount of flow entering the storm sewer. The storm sewers actual capacity could be much reduced. If the flow in the street is limited to a 6-inch depth, there should not be a problem of people being “washed away” by the flow. However, if the depth is allowed to increase, this may become a problem and a limitation on the flow a street may safely convey will need to be considered.

Due to the large difference in the flow rate and the existing system capacity along Original Street, allowing the street to convey a greater depth of flow (and thus a greater quantity of water) will not solve this area's flooding problems. An additional drainage facility should be constructed or a portion of the flow will need to be diverted to another conveyance system.

The Mill Street conveyance system appears to be sized correctly for most of its length. At the north end of this system, there is some unused capacity.

The flow entering the Garmisch Street and Francis Street systems greatly exceeds the capacity of these drainage facilities. An additional or larger conveyance system will be needed to transport this flow to the river.

H. STORMWATER MODEL VERIFICATION

The City of Aspen has proposed to use the FLO-2D program (discussed in Section III) to model the effects of debris flow on existing and future development in the city. For model verification purposes, the results of the FLO-2D model were compared to the CUHP/SWMM modeling results at select locations. The two models were found to yield reasonably similar results at these locations. Direct comparisons could not be made for all of the watersheds studied. Differences in the runoff results of the two models is related to differences in model input.

The values of the soil parameters input into the FLO-2D model were meant to be representative values for the watershed, as a whole. Within the basin, there are areas with different types of soils that have different infiltration and runoff characteristics. Due to the large number of grid nodes in the FLO-2D model (6508 nodes), it was not possible, for the purposes of this study, to provide individual soil parameter values for each node. Also, there is little soil information available up-gradient from the City.

The accuracy of FLO-2D and its correlation to the results of the CUHP/SWMM output could be improved considerably by providing individual soil parameter values for each node. As a check, the soil parameters input into FLO-2D were varied for two sub-watersheds (Sub-watershed 13 and Sub-watershed 16), so that they reflected the values corresponding to the type of soil identified by the SCS Soil Survey for those specific sub-watersheds. The outflow from these two sub-watersheds using FLO-2D and the more detailed soil parameters was very close to the outflow calculated by CUHP/SWMM.

The accuracy of the FLO-2D model could also be improved by increasing the number of nodes (or decreasing the node spacing) in the model.
The results from the CUHP/SWMM model were further compared using the HEC-1 curve number method in Sub-watershed 16. The output from the two models compared favorably.

In conclusion, the CUHP/SWMM model produced flow values similar to the results determined by FLO-2D and HEC-1 models at comparable locations. Based on these comparisons, we believe that the CUHP/SWMM model produces accurate and valid results and is acceptable for determining the runoff entering and flowing through the City of Aspen.

III. MUDFLOW ANALYSIS

A. INTRODUCTION

This Section discusses and analyses the potential and magnitude of mud floods and mudflows that may develop due to rainfall events, snowmelt, or rain on snow events. Several different types of models were used to perform these analyses. FLO-2D estimates the amount of runoff that will occur during a rain event and the depth of flow, water and sediment, that will occur due to this rain.

Mudflows are very viscous, hyperconcentrated sediment flows, whose fluid properties change dramatically as they flow down alluvial fans or steep channels. The behavior of the mudflow is a function of the fluid matrix properties (i.e. density, viscosity, and yield stress), channel geometry, slope, and roughness. Viscosity is in turn a function of the type of sediment (clay or silt), the sediment concentration, and the water temperature. Mudflows have high sediment concentrations and high yield stresses, which may produce laminar flow. Smaller rain events (i.e. 10-year or 25-year storm event) are more likely to cause mudflows. Usually, the peak concentration of sediment during a mudflow event is about 45%, and the average sediment concentration is between 20% and 35%.

Since mud floods contain a higher proportion of water than mudflows, mud floods are less viscous and are always turbulent. Mud floods will be produced by larger flood events such as the 100-year flood.

B. FLO-2D

1. Introduction

FLO-2D is a two-dimensional, finite difference flood routing model, which uses a kinematic wave or diffusive wave equation to estimate overland flow. In addition to modeling water-only flow, the program will also model hyperconcentrated sediment flow. Hyperconcentrated sediment flow is runoff that contains a very high concentration of sediment.

FLO-2D requires a representation of the topography of the study area. This is accomplished by establishing a network of nodes and assigning x-y coordinates and elevations to each node. The nodes must be placed in a rectangular grid with equal spacing between nodes. A typical grid node spacing is 200 feet to 500 feet. Decreasing the node spacing increases the number of nodes and decreases the length of time step used in the model. Both factors increase the model's run time.

Flow in the model is generated by simulating rainfall on each node in the study area or by inputting a runoff hydrograph at select nodes. Rainfall and inflow hydrographs cannot be used simultaneously. The amount and direction of overland flow is calculated in eight directions – directly forward and backward, to each side, and in the four diagonal directions.

Mud floods and mudflows are modeled using inflow hydrographs. The input data contains the hydrograph data, flow versus time, and the concentration of sediment conveyed by the flow, concentration by volume versus time. FLO-2D routes the hyperconcentrated flows, tracking the sediment volumes through the system. Changing sediment concentration, dilution effects, and the remobilization of deposits are simulated at each node. Mudflow cessation and deposition can be predicted by the model. Sediment concentration governs the movement of the fluid matrix. Quadratic rheological equations are used to predict viscous and yield stresses as a function of sediment concentration.

The model also accounts for the initial rainfall abstractions and infiltration. Infiltration is estimated for each node using the Green-Ampt equation. The flow area and storage volume associated with each node can be reduced to represent buildings. Streets can also be modeled to increase the conveyance through these nodes.

This model has several other applications and capabilities which were not used in this study and will not be presented in this report.

Results generated by the FLO-2D model include outflow hydrographs at designated nodes, maximum flow depths and velocities, and a summary of the total inflow, outflow, storage, and losses within the study area.

2. Theory

The governing equations used in FLO-2D to route water flow and/or hyperconcentrated sediment flow include the numerical integration of the continuity equation and the dynamic wave equation. The continuity equation ensures the conservation of fluid volume, and the form used in this analysis is:

$$i = \frac{dh}{dt} + \frac{dh * V_x}{dx} + \frac{dh * V_y}{dy} \tag{1}$$

Where: i = Excess rainfall intensity

h = flow depth

t = time

V_x, V_y = depth average velocity

The dynamic wave equations estimate the motion of the fluid and are shown as:

$$S_{fx} = S_{ox} - \frac{dh}{dx} - \frac{V_x}{g} * \frac{dV_x}{dx} - \frac{V_y}{g} * \frac{dV_x}{dy} - \frac{1}{g} * \frac{dv_x}{dt} \tag{2}$$

$$S_{fy} = S_{oy} - \frac{dh}{dy} - \frac{V_y}{g} * \frac{dV_y}{dy} - \frac{V_x}{g} * \frac{dV_y}{dx} - \frac{1}{g} * \frac{dV_y}{dt} \tag{3}$$

Where: S_{fx}, S_{fy} = Friction Slope

S_{ox}, S_{oy} = Bed Slope

h = Flow depth

V_x, V_y = Depth averaged velocity

g = Coefficient of gravity

t = time

Approximations of these equations, the kinematic wave equation or the diffusive wave equation, can be used with little loss in accuracy when the drainage slopes are relatively steep (i.e. alluvial fans). In this study the diffusive wave equations were used by FLO-2D:

$$S_{fx} = S_{ox} - \frac{dh}{dx} \tag{4}$$

and

$$S_{fy} = S_{oy} - \frac{dh}{dy} \tag{5}$$

Flow depth, velocities, and discharges between adjacent nodes are calculated every time step. The model makes a single sweep of the grid system, explicitly solving for flow depth one node at a time for each time step, and storage volumes at each node for both water and sediment are computed. The inflow, outflow, and change in storage across the entire grid system are assessed at the end of each time step to ensure that volume is conserved within tolerance levels. If the volume is not conserved sufficiently, the model will automatically reduce the time step and resolve for flow depth at each node.

The rheological behavior of hyperconcentrated sediment flows involves the interaction of several complex physical processes, including turbulence, fluid-particle interaction, particle-particle interaction, and fluid viscosity. The total shear stress in hyperconcentrated sediment flows, including mud floods and mudflows, can be calculated from the summation of five shear stress components:

$$\tau = \tau_c + \tau_{mc} + \tau_v + \tau_t + \tau_d \tag{6}$$

Where: τ = Total Shear Stress

τ_c = Cohesive yield stress

τ_{mc} = Mohr - Columb Shear Stress

τ_v = Viscous shear stress

τ_t = Turbulent shear stress

τ_d = Dispersive shear stress

The following quadratic equation was developed by the authors of FLO-2D and used in FLO-2D to account for the various shear stresses.

$$\tau = \tau_y + \eta \frac{dV}{dy} + C\left(\frac{dv}{dy}\right)^2 \tag{7}$$

Where:	τ_y	=	$\tau_c + \tau_{mc}$
	η	=	Dynamic Viscosity
	$\frac{dV}{dy}$	=	Rate of Shear
	C	=	Inertial Shear Stress Coefficient

The first two terms in Equation 7 are referred to as the Bingham shear stresses and represent the internal resistance stresses of a Bingham fluid. The sum of the cohesive yield stress and viscous stress defines the shear stress of a cohesive, hyperconcentrated sediment fluid in a viscous flow regime. The last term is the sum of the dispersive and turbulent shear stresses, which is a function of the square of the velocity gradient.

3. Methodology

The model was first run for clear water only. Rainfall (the 100-year and 10-year storm events) was applied over the entire study area. Nodes were selected up-gradient (south) of the City, and the flow hydrograph at each of these nodes was determined.

These outflow nodes and hydrographs were then converted to inflow nodes and hydrographs. After examination of the hydrographs, sediment concentrations were assigned for each time period and for each outflow node. The peak sediment concentration by volume, C_V, was assigned to the time that the peak flow occurred. Flows less than 10 cfs were assigned a sediment concentration of 20%. Sediment concentrations for flows between the peak flow and 10 cfs were varied to provide a smooth transition between the two values.

These hydrographs with their associated sediment concentrations are then input through the nodes up-gradient of the City, and the flow of water and sediment is analyzed as it flows through the City to the Roaring Fork River.

The peak sediment concentration was varied until the water and sediment flow just reached the river. This concentration was used to provide a conservative estimate of the effect of a mudflow or mud flood on the City or any future development. This value optimizes the area covered against the maximum depth of flow. Higher sediment concentrations would cause a greater depth of flow but would not cover as wide an area. Lower sediment concentrations would cause the flow to cover a greater area, but the depth of flow would be less.

4. Input Data

Most of the input data relates to how the model operates, the output it generates, or is non-controversial in nature (i.e. topographic data). This data is not presented in this report. The data which most effects the results of the modeling effort are presented herein.

For this study 6,508 nodes were established at a spacing of 100 feet. The amount of area over the entire basin that is available to store water and allow water infiltration is 90%. The study assumes that impervious objects (i.e. trees, rocks, buildings, etc.) occupy a significant portion of the basin (10%). The study also assumes that the maximum Froude Number for flow down steep slopes is 0.9.

NOAA precipitation maps provided estimates of the 100-year, 1-hour rainfall event and the 10-year, 1-hour rainfall event. The CUHP program then converted these precipitation amounts to 2-hour quantities and developed the rainfall distribution that was used in the FLO-2D model.

Surface detention and abstraction was assumed to be 0.025 inches. For use in FLO-2D, the study assumed that the soil type (loam in this case) was the same over the entire basin. The porosity was 0.4 feet per feet, and the initial and final saturation contents were 0.9 and 1.0, respectively. The hydraulic conductivity of the soil was assumed to be 0.3 inches per hour and the capillary suction was 4.8 inches.

The peak sediment concentration used in the inflow hydrographs was 0.45. The model was run for a range of values varying between 0.35 and 0.48. A concentration of 0.45 allowed the flow to just enter the Roaring Fork River during a 100-year event.

C. RESULTS

1. Initial Mudflow Analysis

The output from the model is the maximum depth of flow at each node. Drawing 1 shows a contour map of the maximum depth of flow through the City of Aspen assuming that the water contains no sediment (C_V=0.0). Drawing 2 provides the maximum depth of flow assuming that the peak sediment concentration is 0.45. By comparing the two drawings, it is apparent that sediment has a great effect on the depth of flow and ultimately the design of any mitigation alternatives.

It is important to note that Drawings 1 and 2 do not include the effect of buildings and streets. It was modeled as if there were no obstructions to the flow.

Drawings 3 and 4 provide a contour map of the water surface elevation generated by the runoff from the 10-year, 2-hour storm with a peak sediment concentration of 0.0 and 0.45, respectively. These drawings also do not model the effect of the city (buildings and streets).

2. Model Revisions

Once the initial model was completed and verified, two revisions were made to the model to account for changes in topography and for inclusion of City streets and buildings.

a. Revised Topography

The topography on Aspen Mountain directly south of the City between Mill Street and Monarch Street was modified according to the World Cup Grading Plan. This re-grading of the area cut 24,600 cubic yards of material from the west side of the project and used this material as fill on the east side of the project. Drawing 5 shows the mudflow depth with this change in topography for a 100-year mudflow event. This drawing does not account for the streets and buildings within the City.

b. In-City

Accounting for the effect that streets and buildings have on a mudflow can have a large effect on the depth of a mudflow. Streets are model as depressed (0.5 feet) channels with a relatively low roughness factor, so the mud flows "easier" down the streets than it would over unimproved areas. Drawing 6 provides the estimated 100-year mudflow depth for the FLO-2D model that accounts for the effect that streets will have on the mudflow.

Buildings reduce the area for the mud to flow through and also reduce the area that can store a volume of mud. Both are important factors when modeling the mudflow event. Drawing 7 is the maximum mudflow depth when both streets and buildings are included in the model.

3. New Development

One of the purposes of this FLO-2D model development is to determine what effect, if any, new development or redevelopment will have on a mudflow event. There are several ways that a development can be modeled, and there are different levels of detail to which a development may be modeled.

To estimate the effect that new developments may have on flow adjacent to and through the development, an example development was analyzed. A preliminary plan for the Top of the Mill development was received from the City. The proposed development, which is located at the south end of Mill Street, was then coded into the model.

When buildings are placed in the flow path of flowing water, the buildings reduce the width available to pass the flow and cause the water surface elevation to increase. This increase in water surface elevation may be localized and only effect the area where the buildings are located, but this increase in water surface elevation can also expand and effect areas upstream of the buildings, downstream of the buildings, and adjacent to the buildings.

FLO-2D allows the flow area through designated nodes to be reduced. The hypothetical development has a total width of approximately 400 feet and the width of the houses is about 320 feet in the direction of flow. Based on these values, the width available to pass flow through the nodes associated with this proposed development were reduced by 80%. The area available to store water and allow infiltration was also reduced.

A FLO-2D model was developed that used a "generic" width and area reduction for the hypothetical development. Generic, in this case, means that all nodes within the area of the hypothetical development are given the same flow reduction factor values. Every node within the hypothetical development was given a width reduction value of 0.8 (80% of the flow path was blocked and would not pass flow).

Drawing 9 shows the maximum mudflow depth in the area south of Mill Street without the hypothetical development. This drawing represents an estimate of the maximum depth of flow if a mudflow would occur under the existing conditions. The node spacing is 100 feet, and the city (streets and buildings) is modeled generically.

Drawing 10 shows the maximum mudflow depth in the area south of Mill Street with a hypothetical development constructed. The development is modeled generically, as described above. The city, down-gradient of the development, is also modeled generically and, the node spacing is 100 feet.

Mudflow through the hypothetical development was also modeled using "specific" area reduction factors. Specific means that each node is defined differently. Some nodes within the development will have a very low reduction factor (i.e. 20%), and other nodes will be completely blocked (i.e. those representing buildings). Increasing the level of detail increases the time and effort necessary to code the data for the model and increases the model run time slightly. Drawing 11 represents the maximum mudflow depth modeling these conditions.

A FLO-2D model with a 25-foot node spacing was developed for hypothetical development and the area around the hypothetical development. A specific area reduction factor was given to each node in the hypothetical new development and in the existing developed area. The effort and time needed to create the 25-foot node spacing is much greater than with the 100-foot model, and the run-time is also much greater, even though the number of nodes is less. The results of this modeling effort are shown on Drawing 12.

The hypothetical development was then removed from the 25-foot spacing model, and the model was re-run. The results of this model are shown in Drawing 13.

The effect of the new development as modeled with a 25-foot spacing was then estimated by subtracting the mudflow depths resulting from the existing development model (Drawing 13) from the mudflow depths from the hypothetical development model (Drawing 12). Drawing 14 is the difference in the maximum mudflow depth caused by the hypothetical development.

D. DISCUSSION AND RECOMMENDATIONS

The purpose of the various FLO-2D model runs was to determine the best way to model the effect that a new development will have on a mudflow event. Table 11 provides a summary of the modeling features that were used in FLO-2D to generate each of the drawings (Drawings 1 through 14). Due to the uncertainty of the data and the modeling assumptions, the depths of flow provided by the FLO-2D model should be considered as qualitative values rather than absolute values.

In comparing Drawings 1, 2, 3, and 4, it is apparent that increasing the storm return frequency from 10-years to 100-years increases the depth of flow through the City. Also, by modeling a mudflow event (adding sediment to the water flow), the depth of flow increases.

By comparing Drawing 5 to Drawing 2 and Drawing 7 to Drawing 8, very little difference in the flow depth is noticeable at the south end of Mill Street. Revising the topography has very little effect on the depth and direction of a possible mudflow event down Pioneer Gulch.

Including the "Streets" option in the FLO-2D model has very little effect on the flow depth (compare Drawing 5 to Drawing 6). Including the "Buildings" feature of FLO-2D has a significant effect on the flow depth (compare Drawing 6 to Drawing 7). It increases the flow depth by 5 or 6 feet in some locations.

In a comparison of Drawings 9 and 10, it is shown that a new development can increase the depth of flow at the new development and also adjacent to the development on other's property if proper development layout and mudflow facility designs are not used..

FLO-2D allows the modeler to either model buildings generically or specifically. Modeling the buildings specifically (Drawing 11) rather than generically (Drawing 10) decreases the depth of flow slightly (1 or 2 feet) t also changes the path of the mudflow and the location of the deepest flow.

Decreasing the node spacing from 100 feet (Drawing 11) to 25 feet (Drawing 12) further revises the depth of flow and the flow path. Since it is more detailed, it could be assumed that the model with the 25-foot spacing would be more accurate.

Drawing 14 shows the effect that a new development may have using the 25-foot spacing model. From this comparison it appears that the new development would not negatively impact the adjacent property because it would intercept the mud before it reached the downstream development. The depth of flow upstream of the proposed new development would increase by 6 feet though.

Based on these analyses, it is recommended that a maximum node spacing of 50 feet be used to model the effect of future proposed developments. A 50-foot spacing should provide a sufficient resolution to model buildings accurately (most buildings are wider than 50 feet), and the effort needed to develop the model should be reasonable. A smaller spacing provides good results, but it takes a significantly longer time to develop and run the model.

It appears that FLO-2D could be an important tool for the City of Aspen to identify "high risk" areas that may be subject to flooding and/or mudflows and mud floods. The analysis shows the effect that buildings have on the depth of flow of the runoff on themselves and their neighbors. With further refinement, the FLO-2D model may also be useful in designing mitigation measures.

E. FLO-2D RUNOFF VALIDATION

To ensure the validity of the results of the FLO-2D model, the study calculated and compared the flow from Vallejo Gulch and Spar Gulch using different models and different methodologies. Table 12 presents the results of this analysis. Table 13 provides many of the input data values that were used in the analysis. The data between models was kept as consistent as possible, although it did vary slightly in some cases due to model limitations.

In Vallejo Gulch, the CUHP/SWMM model produced a peak flow for the 100-year, 2-hour storm that was slightly less than the peak flow produced by FLO-2D and by HEC-1 using the Green-Ampt and Holtan loss equations. It appears that the uniform loss method produces a much higher flow rate than the other methods. This is probably due to the other methods having a high initial infiltration rate that decreases over time.

In Spar Gulch, the CUHP/SWMM 100-year, 2-hour peak flow is very similar to that generated by HEC-1, but the flow generated by FLO-2D using the same infiltration methodology is much less (532 cfs versus 340 cfs). This large difference in flow may be due to FLO-2D's interpretation of the direction of flow off of the alluvial fan. FLO-2D may route some of the flow into a different watershed.

IV. SNOWMELT ANALYSIS

A. NATURAL SNOWMELT

1. Introduction

Snowmelt from Aspen Mountain in the Spring and early Summer causes flow and sediment problems in the City of Aspen. Usually the magnitude of the flow associated with snowmelt is relatively small but of a long duration. As the sediment-laden runoff encounters the relatively flat slopes in the City, the sediment settles out of the flow and is deposited in the stream, channels, streets, and storm sewers. The quantity of sediment is relatively small in comparison to the mud floods and mudflows previously discussed. These flows and deposits constitute more of a maintenance problem as opposed to the risks of property losses and loss of life that the mudflows and mud floods produce.

2. HEC-1

a. Introduction

HEC-1, written by the Corps of Engineers, was used to calculate the runoff associated with snowmelt. HEC-1 can produce estimates of snowmelt by two different methods: the degree-day method and the energy-budget method. The degree-day method was chosen mainly due to the availability of the data for use in the model. The input data needed for the degree-day method is the temperature as it varies over time. In this case, it is the hourly temperature over the course of a day. The energy-budget method requires values of shortwave radiation, the dew point temperature, and wind speed over time.

b. Theory

The degree-day method of estimating snowmelt used by HEC-1 incorporates the following equation:

$$\text{Snowmelt} = C * (\text{Temp} - 32^{\circ}\text{F}) \tag{8}$$

Where: $C = 0.07$

Temp = Air Temperature

HEC-1 is capable of using several different types of synthetic unit hydrographs. For this study the Soil Conservation Service (SCS) dimensionless unit hydrograph method was chosen. The time-to-peak and the peak flow are calculated as follows:

$$T_p = 0.5 * \Delta t + T_{LAG} \tag{9}$$

Where: T_p = Time to peak of unit hydrograph

Δt = Duration of excess rainfall

$T_{LAG} = 0.6 * T_C$ = Lag between the center of mass of the excess rainfall and the peak of the unit hydrograph.

T_c = Time of concentration

$$Q_p = 484 * \frac{A}{T_p} \tag{10}$$

Where: Q_p = Peak flow of the unit hydrograph

A = Sub-watershed Area

c. Input Data

The temperature data used by the model was based on actual temperature data recorded on Aspen Mountain and recovered from the Colorado Climate Center via the Internet. Two recording stations have been located on Aspen Mountain. Station 50370 (Aspen) recorded from 1900 to 1979, and climatic data collected from 1980 to 1997 from the second station, 50372 (Aspen 1 SW), was also used. This data was combined for this study. The mean monthly maximum temperatures and the mean monthly minimum temperatures are provided in Tables 14 and 15, respectively.

These tables also arrange the temperature data in the month of May from highest to lowest. Based on the inspection of daily temperature readings for the month of May in 1989, peak daily minimum and maximum temperature values are approximately 15 °F greater than their respective average monthly values (NOAA, 1990).

Using this data, the maximum high temperature and the maximum low temperature that would be associated with a 2-year, 5-year, 10-year, 50-year, and 100-year snowmelt event was estimated. For instance, it was assumed that the 10-year average maximum monthly temperature would occur 10 percent of the time. According to the collected data, this would correspond to a temperature of 67° F. The 10-year maximum daily temperature would then be 15 °F higher than that or 83 °F.

The maximum minimum daily temperature was estimated in the same manner. The hourly temperature fluctuation between the maximum and minimum daily temperature was then estimated. The estimates of the hourly fluctuation of the daily temperature used in the model for the 2-year, 5-year, 10-year, 50-year, and 100-year snowmelt events are shown in Table 16.

Table 16 also supplies the temperature fluctuation of the average monthly temperature. Since this is used to estimate the average monthly snowmelt in the month of May, 15 ^{o} F is not added to the average monthly values.

The adiabatic rate of cooling or lapse rate for the cooling of temperature with a given rise in elevation must also be estimated and input into the model. Assuming saturated air, an elevation range of 8,000 feet to 11,000 feet, and an expected temperature range from 36^{o} F to 68^{0} F, Figure 3.2 in V.T. Chow's Handbook of Applied Hydrology (1964) shows a lapse rate of about 2.5^{o} F for every 1000-foot rise in elevation.

It was also assumed that there would be approximately 56 inches of snow on the ground in May and that the water content of the snow would be 1 inch of water for every 11 inches of snow. For every 1000-foot increase in elevation, the amount of remaining snow in May was assumed to increase by about 18 inches.

d. Methodology

The runoff from snowmelt in two watersheds were estimated – Spar Gulch and Vallejo Gulch. Spar Gulch consists of Sub-watersheds 14, 22, 23, and 24 as defined in the CUHP/SWMM analysis, and Vallejo Gulch is Sub-watershed 16. In the CUHP/SWMM analysis the runoff from the Sub-watersheds was routed down to Original Street. To simplify the analysis, the HEC-1 model combines the four basins comprising Spar Gulch into one basin and there is no routing analysis. The outflow from Vallejo Gulch is calculated at Durant Street.

HEC-1 also can model the infiltration of water into the ground by several different methods. This study used a uniform infiltration rate to estimate the amount of infiltration from snowmelt. It was assumed that snowmelt would have been occurring previously and the ground would already be saturated. Infiltration would then occur at a constant rate, and there would also be no initial loss due to depression storage, etc. The model also ignored the possible effect of base flow due to previous ground water infiltration reentering the runoff further down-gradient.

e. Results

Tables 9 and 10 provide the results of snowmelt analysis for Spar Gulch and Vallejo Gulch, respectively. The peak flow expected from snowmelt for these size watersheds should vary little from year to year and from day to day. The magnitude of the peak snowmelt runoff is also very small in comparison to rainfall runoff, although it will occur over a much longer period of time

An analysis was also performed to determine the effect that the uniform loss rate has on the peak flow generated by a rain on snow event. If the ground is assumed to be frozen, the infiltration rate would be low (i.e. 0.05 inches per hour). Typically though the ground beneath a snow pack is unfrozen. The layer of snow provides an insulation layer from the cold air, and the hear from the earth rises to warm the ground near the surface. In this situation the infiltration would be higher (i.e. 0.5 to 0.85 inches per hour). As would be expected, the results show that the lower the infiltration rate, the greater the rate of runoff, although the increase is not large.

f. Discussion

Snowmelt does not appear to pose a risk to property or life in Aspen, although it probably causes a maintenance problem in the streams and streets in the City. Infiltration from snowmelt may also saturate the ground, which could cause landslides. Landslides are different than the land flows discussed in this report. Landslides are governed by different processes and analysis methods than land flows, and an analysis of the landslide potential of Aspen Mountain is beyond the scope of this project.

B. RAIN ON SNOW ANALYSIS

1. Introduction

During the spring and early summer, there is a possibility that a major rainfall event could occur when there is still snow on the ground. If the snow is ripe (partial melting has already taken place and the snow has little capacity to "absorb" water), rainfall can produce runoff that is much greater than would be expected by rain alone or snowmelt alone.

2. Methodology and Input Data

For the purpose of this analysis, the rainfall event was assumed to occur in the afternoon (2:00 PM to 4:00 PM) when the temperature was the warmest. Rainfall amount and distribution was the same as was developed for CUHP/SWMM. The average monthly temperature was used for this analysis (see Table 16). Since snowmelt was currently occurring, the uniform loss rate was used with a hydraulic conductivity of 0.5 inches per hour and no initial losses.

Since the Degree-Day method of calculating snowmelt does not account for the melt created by the heat transfer from the rain to the snow, a value of snowmelt created by the rain was estimated and added to the hyetegraph. This snowmelt is estimated by the following equation (Chow, 1964):

$$M_P = 0.007 * P * \left(T_a - 32\right) \qquad (11)$$

Where:

M_p = Daily snowmelt from rain

P = Daily rainfall

T_a = Saturated air temperature

This assumes that the temperature of the rain is the same as the surrounding air. While this equation is for daily values, it can be subdivided into smaller time intervals (i.e. 5 minutes).

Table 17 supplies a table of the quantity of snowmelt for a given rainfall depth at a given temperature. This table was used in conjunction with the hourly temperature values to provide an estimate of the snowmelt caused by the rain. Table 18 contains the results of the snowmelt caused by rain during the duration of the storm and calculates a new rainfall hectograph, which is then used in the HEC-1 model.

3. Results

The results of this analysis are provided in Tables 9 and 10 for Spar Gulch and Vallejo Gulch, respectively. The return period associated with these values is in respect to the rainfall amount and distribution only. In reality, the occurrence of a major rain event while snow was still on the ground would involve a multiple probability analysis, which is beyond the scope of this study. The actual return frequency of these flows occurring would be much less than those shown.

Some of the reasons that the Rain on Snow event is so much greater than the Rain Only flow may be:

a. The Rain Only event includes an abstraction amount of 0.2 inches while the Rain on Snow event has no abstraction.

b. The Rain Only event has a greater final infiltration rate.

c. The Rain on Snow event would carry more runoff due to the melted snow caused by the air temperature and the rain.

d. The Rain Only event used the Green-Ampt infiltration method and the Rain on Snow analysis used the uniform loss rate.

4. Discussion

A rain on snow event could cause a very serious problem for the City of Aspen. In reviewing Tables 9 and 10, it is apparent that a 2-year storm occurring on snow would be equivalent to about a 25-year storm occurring in the summer (without snow), and a 10-year storm occurring on snow would produce almost a 100-0year event without snow. It is strongly recommended that a joint probability analysis be performed to determine the actual likelihood of a given frequency rainstorm falling on snow. It would allow the City to balance the level of risk (frequency of occurrence) of a very large flow event caused by rain on snow against the cost of building remediation projects to prevent damage to the City and its people.

C. NATURAL AND MANMADE SNOWMELT

1. Introduction

The effect that manmade snow would have on the peak runoff from the watershed was also estimated. It was assume that an additional 18 inches of manmade snow would be applied on the Spar Gulch basin and the Vallejo Gulch basin between an elevation of 8,000 feet and 9,000 feet. All other factors and were identical to those previously discussed for natural snow runoff.

2. Results

As shown in Tables 9 and 10, the artificial manufacturing of snow will not effect the peak runoff expected due to snowmelt for most instances. The main effect of manmade snow is to extend the runoff season by the period of time necessary to melt the additional snow, and it has a slightly different water content. An instance when the manmade snow could effect runoff is if a major storm took place on Aspen Mountain during the time that this additional manmade snow was still melting, but the natural ow would have all disappeared. The likelihood of this event occurring involves multiple probabilities and is beyond the scope of this study.

3. Discussion

Man-made snow should have little effect on the potential for mudflows or mud floods. The exception would be if a large rain event occurred on snow and the snow would not normally still be there except for the addition of manmade snow

V. ALTERNATIVE DEVELOPMENT

A. INTRODUCTION

Three alternatives were also investigated that would convey runoff (water only) through the City of Aspen, from the base of Aspen Mountain to the Roaring Fork River. Two design alternatives were generated that would prevent or reduce the magnitude of a mudflow event on Aspen Mountain, and a third alternative was devised to regulate development in consideration of a mudflow event but not prevent a mudflow event from occurring. The alternatives developed for Aspen Mountain (on-mountain) are independent of the alternatives developed for the in-City (i.e. Alternative 1 for Aspen Mountain can be used with Alternative 2 for the City).

For this analysis, the City was divided into 3 different storm drainage systems. System 1 drains eastern portion of town and the runoff from Spar Gulch. System 2 lies in the center of Aspen and conveys runoff from Vallejo Gulch (Sub-watershed 16) to the Roaring Fork River. System 3 covers the western portion of Aspen and conveys runoff from Pioneer Gulch to the Roaring Fork River.

B. STORMWATER ALTERNATIVES

1. Alternative 1

Alternative 1 collects and conveys runoff from Aspen Mountain and the City for the 100-year rainfall event. This alternative is shown in Drawing 15.

During the 100-year flood the runoff from Spar Gulch (System 1) will flow as sheet flow down Aspen Alps South Road to Ute Avenue. This flow will be collected by a concrete channel that is 300 feet long, four feet wide, and varies from 3 feet to 11 feet deep. Runoff collected by this concrete channel will then be conveyed through an 84-inch RCP from Ute Avenue north down West End Street and then west and south to the intersection of Spring and Main. The proposed storm sewer also collects flow from the existing Original Street and Spring Street storm sewer systems. From the intersection of Spring and Main, a 96-inch RCP will convey the flow north to the proposed water quality extended detention basin, Basin A12B1. The basin is for water quality purposes only and will not reduce the peak inflow to the Roaring Fork River. This basin will require a storage volume of about 7.0 acre-feet.

The water quality capture volume (WQCV) that guides the design of the water quality detention facility is determined by the estimated runoff from the watershed during the water quality design storm. The water quality design storm is the rainfall depth of the 80th percentile storm taken from rainfall events along the front range.

Essentially the existing storm sewer in System 2 has sufficient capacity to convey the 100-year runoff down Mill Street from Aspen Mountain to the water quality extended detention basin, Basin A12B2. The two exceptions are an extension of the existing storm sewer from 200 feet south of Durant Avenue to the end of Mill Street. The other location is at the north end of the storm sewer system, where the flow is diverted through a 48-inch RCP into Basin A12B2 instead of directly into the Roaring Fork River. This water quality basin will require a storage volume of 8.8 acre-feet.

There are two storm sewer systems to collect and convey runoff in System 3. The first storm sewer system collects runoff from Pioneer Gulch at Gilbert Street and conveys the flow north down Monarch Street to the water quality extended detention basin, Basin A12B2. For most of its length down Monarch Street the storm sewer is a 42-inch RCP. From about Puppy Smith Road to Basin A12B2, the storm sewer is a 54-inch RCP.

The second storm sewer system collects runoff along Garmisch Street and Francis Street and conveys it to Basin A12B2. This storm sewer will replace the existing system with a larger storm sewer. The storm sewer along Garmisch Street varies from a 48-inch RCP to a 72-inch RCP. A 48-inch RCP collects runoff along Main Street and conveys it to the Garmsich Street storm sewer. A new storm sewer, a 24-inch RCP, along Francis Street would also be required. In order to reduce the required capacity and size of the storm sewer, curb and gutter would be constructed along the street so some of the flow could be conveyed by the street. From the corner of Garmisch and Francis Streets an 84-inch RCP will be used to convey the flow to Basin A12B2.

2. Alternative 2

Alternative 2 also collects and conveys runoff from Aspen Mountain and the City for the 100-year rainfall event. This alternative is shown in Drawing 17.

Runoff from Spar Gulch is collected at the south end of Aspen Alps South Road by an inlet structure and an 84-inch RCP. Figure 8 is a sketch of the profile of this inlet structure. The purpose of the inlet is to collect the runoff before it has a chance to spread and become sheet flow. The 84-inch RCP will be constructed down Aspen Alps South Road to Ute Avenue where it combines with a proposed 18-inch RCP along Ute Avenue. The 84-inch RCP will continue down Original Street to the intersection of Main and Spring. It will collect flow from existing storm sewers on Cooper Avenue, Spring Street, and Main Street. A 96-inch storm sewer will conduct the flow from this intersection to the water quality extended detention basin, Basin A12B1, discussed in Alternative 1.

The proposed storm sewer system for System 2 is the same as in Alternative 1.

There is only one storm sewer system in System 3 in this alternative. The upgradient end of Alternative 2 is the same as in Alternative 1, with a storm sewer along Gilbert Street and down Monarch Street. In Alternative 2 the proposed storm sewer system turns into a 60-inch RCP at the intersection of Durant Avenue and Monarch Street and proceeds down Durant Avenue to Garmisch Street. The main trunk of the storm sewer system proceeds north on Garmisch Street increasing in size to an 84-inch RCP. A lateral storm sewer collects runoff from Sub-watersheds 6 and 7 and directs it east on Hopkins Avenue through an 18-inch RCP and a 27-inch RCP to Garmisch Street. A new 33-inch RCP will be laid beneath Main Street between Second Street and Garmisch Street. The 42-inch RCP proposed along Francis Street will pass the entire 100-year runoff, and no curb and gutter will be constructed along the road. A 96-inch RCP will deliver the combined Francis Street flow and Garmisch Street flow to the water quality extended detention basin, Basin A12B2. This basin is the same as described in Alternative 1.

3. Alternative 3

Alternative 3 collects and transports runoff from Aspen Mountain and the City for the initial rainfall event, as defined in the City's Storm Drainage Criteria Manual. This alternative assumes that the entire runoff from the initial storm must be carried in the storm sewer and not by the street. The return period of the storm event for the initial storm depends on the land use type. Higher density housing and construction has a larger return period. The storm sewer systems in System 1 and System 2 run mainly through commercial buildings, so the proposed storm sewer is designed for the 10-year runoff event. In System 3, the proposed storm sewer along Main Street collects runoff from medium density housing so it is designed for the 5-year return period storm. The housing along Francis Street is low-density residential so the storm sewer is designed for the 2-year runoff event. The south end of Garmisch Street is commercial so it is designed to convey the 10-year flow. The remainder of the Garmisch Street storm sewer system is also designed for the 10-year return period so the return period will not decrease as it proceeds downstream. This alternative is shown in Drawing 19.

A proposed 30-inch storm sewer adjacent to Ute Avenue will collect runoff from Spar Gulch and conduct it to the existing storm sewer system beneath Original Street. A 24-inch and 27-inch RCP will also be placed beneath Spring Street from the chair lift to Main Street. At Main Street the existing 36-inch RCP will guide the flow to the proposed water quality extended detention basin, Basin A3B1. This storage volume of this basin is approximately 9.6 acre-feet. It will not reduce the peak flow from the initial storm runoff.

In Drainage System 2, a new 21-inch RCP from the south end of Mill Street to the end of the existing storm sewer, which is about 400 feet south of Durant Avenue. A 48-inch RCP will also be placed just south of Rio Grande to transfer the runoff from the existing 48-inch storm sewer beneath Mill Street to Basin A3B1.

In System 3, a new storm sewer system will collect runoff from Pioneer Gulch along Gilbert Street and Monarch Street and convey the flow through a 24-inch RCP beneath Aspen Street to the existing 30-inch corrugated metal pipe (CMP) at Aspen Street ad Durant Avenue. A new storm sewer system is also needed along Garmisch Street from Hopkins Avenue to Francis Street. This storm sewer will increase in size from 36 inches to 54 inches. A new 27-inch RCP is needed beneath Francis Street from Seventh Street to Francis Street. No curb and gutter will be constructed. A 54-inch RCP will be laid from the intersection of Garmisch Street and Francis Street to the proposed water quality extended detention basin, Basin A3B2. The design storage volume of Basin A3B2 is 6.6 acre-feet.

C. MUDFLOW ALTERNATIVES

There are several different ways to approach alternatives managing potential mudflow events. Alternative 1 is proposed to attempt to prevent a mudflow from occurring. Alternative 2 is proposed to limit the magnitude of the mudflow event. Alternative 3 does nothing to prevent or reduce the magnitude of a mudflow event, but it does regulate new construction to minimize the effect that future mudflows may have on the City. Several other conceptual alternatives were examined but will not be discussed in detail.

1. Alternative 1 - Drain and Channel

This alternative constructs a boulder-lined channel and a drain system beneath the main channels on Aspen Mountain. A plan view of the location of the channel and drain is provided by Drawings 15 and 16. A cross-sectional view of this alternative is shown in Figure 9 and a typical profile is shown in Figure 10. The boulder-lined channel prevents concentrated flow from eroding the bottom of the channels. The coarse material beneath the boulder allows ground water to flow easily to the perforated PVC pipe. The PVC pipe then directs the ground water flow down gradient to the City's storm sewer system. Mudflows are usually caused by the failure of saturated soil on steep slopes. The underdrain system should keep the ground at the bottom of the gulches unsaturated and thus more stable.

The coarse material and large perforations in the PVC pipe will allow sediment carried in runoff to enter the pipe without plugging the coarse material above it. This will allow the system to be relatively maintenance free. The steepness of the drain will ensure that all sediment will be carried to the City storm sewer system, where a junction box will be constructed to collect the sediment and allow it to be removed at the City's convenience. The relatively fine granular filter material that separates the existing ground from the coarse granular material allows water to pass through but prevents the existing soil to flow into the underdrain system with the ground water.

A cutoff wall will be constructed at the very upstream end of the channel. The purpose of the cutoff wall is to stop a mudflow that has already started flowing. The proposed cutoff walls would be 50 feet long and 10 feet deep.

2. Alternative 2 - Cutoff Walls

Alternative 2 consists of a series of cutoff walls along the bottom of the gulches. As previously mentioned, these walls would not prevent a mudflow from occurring, but they would limit the magnitude of the event to very small slides. The cutoff walls would be 50 feet long and 10 feet high and spaced every 100 feet. The cutoff walls would be buried so that they would not be visible. As erosion continues to occur on the mountain, the walls may require some maintenance to keep them buried and hidden. If the depth to bedrock is less than 10 feet, the walls will be keyed into the bedrock. Drawings 17 and 18 supply a plan view of the cutoff wall system. Drain systems behind the walls are needed to prevent the walls from blocking subsurface flows.

3 Alternative 3 - Mudflow Hazard Regulations

The third alternative regarding mudflows off of Aspen Mountain is establishing regulations to account for a mudflow event in new development and construction. In general, new construction should not increase the depth of the 100-year mudflow depth and new construction must be designed and built to withstand the static and dynamic forces of the mudflow event.

4. Other Alternatives

Several other alternative were also examined. A collection system and a storm sewer system was designed to collect a mudflow event after it had occurred, but before it enters the City. The size of these facilities (i.e. 12-foot by 12-foot reinforced concrete box culverts) made the cost of this alternative very high and its construction infeasible.

The use of abandoned mines to collect runoff and mudflows was also analyzed. After gathering maps and reports regarding mining on Aspen Mountain and personnel correspondence with several individuals who are currently mining the mountain, it was concluded that the use of the mines would be too costly and too dangerous. The shafts and galleries would have to be treated to ensure that the introduced water would not flow through the mountain to undesirable locations. Mudflows could not be allowed to freefall down the shafts, and the cost of constructing a pipe that would contain the debris carried by a mudflow would be enormous.

D. DISCUSSION AND RECOMMENDATIONS

1. Stormwater Solutions

The main factors in comparing the alternatives for conveying storm runoff in the City are cost and the level of protection the alternatives provide. All of the alternatives consist of storm sewers and water quality detention basins. Other types of drainage facilities (i.e detention basins, open channels, etc.) are not practical because the City is almost fully developed and there is not enough available land to contain these types of facilities. A comparison of the various alternatives is provided in Table 19. The total cost of Alternative 1 is $13,297,000. The cost of the drainage facilities for Drainage Systems 1, 2 and 3 are $5,870,000, $443,000, and $6,984,000, respectively. This alternative is designed for the 100-year event. Flow was allowed to be conveyed in the street at a maximum depth of 0.5 feet due to the lack of slope away from the top curb at some locations. The remaining flow was conveyed in the storm sewer.

The total cost of Alternative 2 is $17,501,000. The cost of the drainage facilities for Drainage Systems 1, 2 and 3 are $7,170,000, $443,000, and $9,888,000, respectively. This alternative is also designed for the 100-year event, and flow was allowed to be conveyed in the street at a maximum depth of 0.5 feet. The remaining flow was conveyed in the storm sewer.

The total cost of Alternative 3 is $6,204,000. The cost of the drainage facilities for Drainage Systems 1, 2 and 3 are $2,280,000, $455,000, and $3,469,000, respectively. This alternative is designed for the initial storm, which varies from a 2-year to a 10-year event. The storm sewers were designed to convey the entire runoff of the initial storms as is required by the proposed Storm Drainage Criteria Manual.

Typically, retrofit projects are designed for the 2-year to 10-year storm event due to the high cost of providing 100-year protection in areas that are already fully developed. Alternative 3 is half the cost of Alternative 1 and about a third of the cost of Alternative 2. In addition to its lower cost, Alternative 3 will provide protection against the more frequent storms that will occur in Aspen. Based on this analysis, Alternative 3 is the recommended alternative.

2. Mudflow Solutions

There are several factors that can be compared when analyzing the Aspen Mountain mudflow alternatives. Construction cost, operation and maintenance cost, the aesthetic appeal, and the risk of a mudflow occurring even with the alternative in place are the factors that were used to compare alternatives. This comparison is provided in Table 20.

Alternative 1 will cost almost $11,000,000, which is more costly than the other alternatives, but it should have lower operation and maintenance costs. The use of boulders should blend in with the

mountain
environment, and the risk of a mudflow occurring with this alternative is less than with the other alternatives.

Alternative 2 will cost approximately $7,800,000. Since this alternative does not prevent the minor mudflows that may occur or the annual erosion that occurs, the operation and maintenance costs will be relatively high. Since the cutoff walls will be buried, they should have no effect on the aesthetic appeal of the mountain. Mudflows on alluvial fans frequently change course during the event (typically this is on shallower fans), and thus they may flow around the edge of the proposed cutoff walls. The risk of failure is probably higher than with Alternative 1.

Alternative 3 does not have a capital construction cost, but new developments or re-developments that are located in the designated mudplain would have increased costs. These structures would have to be designed to have no effect on the depth of the mudflow and designed to withstand the force of the mudflow. The operation and maintenance costs could be extremely high with this alternative. If a major mudflow event were to occur, the damages could easily be in the 10's of millions of dollars and cleanup costs for a mudflow event are typically more than the damage that they cause. The probability that a mudflow event will occur on Aspen Mountain is relatively high. Mudflows have historically occurred on the Mountain. Geologic maps published by the U.S. Geological Survey show large areas on Aspen Mountain directly above the City that are defined as potentially unstable.

The actual risk of a mudflow event occurring on Aspen mountain is not precisely known. Further study and analysis may yield better data and tools that can define the location and hazard of mudflow events on Aspen Mountain. Alternatives 1 and 2 are conceptual solutions which need additional investigation and analysis prior to selection of a recommended alternative. The construction of these alternatives on the extremely steep slopes of Aspen Mountain would be very difficult and may be infeasible. Before Alternatives 1 or 2 are implemented, it is strongly recommended that a test site be constructed to test the effectiveness and feasibility of these alternatives. Alternatives 1 and 2 are also very costly, and the City does not currently have the financing available to construct either of these alternatives. Based on this analysis, it is recommended that regulatory controls be initiated until financing becomes available to construct alternatives that can control or prevent mudflow events and until the designs used in Alternatives 1 and 2 can be refined and tested. The proposed mudflow hazard area is delineated on Figure 11.

VI. CONCEPTUAL DESIGN

The stormwater design alternative selected by the City for a more detailed analysis of the needed drainage facilities for the City is essentially Alternative 3 with some minor modifications. Plan and profiles of this conceptual design are provided in Drawings 20 through 24.

The modifications to Alternative 3 include a grass-lined open channel along Ute Avenue to collect the runoff from Spar Gulch. Also, the design and shape of the detention basins was changed slightly. Since the design was described in the previous section, it will not be described again.

A more detailed cost estimate was produced based on this conceptual design (see Table 21). The total cost of the project would be about $6,204,000. Costs associated with Drainage Systems 1, 2, and 3 would be $2,280,000, $455,000, and $3,469,000, respectively.

This design will provide the City of Aspen with a financially feasible solution to convey relatively frequent runoff events to the Roaring Fork River. The water quality basins incorporated into the design also treat the runoff before it enters the Roaring Fork River to assist in protecting this valuable resource for the City and the region.

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APPENDIX A

EXISTING STORM SEWER INVENTORY

TABLES

TABLE 1: CUHP SUB-WATERSHED PARAMETERS

WATERSHED DESIGNATION	BASIN AREA (acres)	BASIN AREA (mi2)	BASIN LENGTH (feet)	BASIN LENGTH (miles)	CENTROID LENGTH (feet)	CENTROID LENGTH (miles)	IMPERVIOUS AREA (%)	SLOPE (%)	Tc (minutes)	PERVIOUS DETENTION (inches)	IMPERVIOUS DETENTION (inches)	INFILTRATION INITIAL RATE (in/hr)	INFILTRATION DECAY COEFF. (sec-1)	INFILTRATION FINAL RATE (in/hr)	HYDROLOGIC SOIL GROUP
1	63.85	0.100	2300	.44	800	.15	6.1	4.47	32.2	.35	0.1	4.50	0.0018	0.60	B
2	37.33	0.058	2200	.42	600	.11	25.5	1.57	22.2	.35	0.1	4.50	0.0018	0.60	B
3	55.83	0.087	3350	.63	1200	.23	70.0	1.19	28.6	.35	0.1	4.50	0.0018	0.60	B
4	52.84	0.083	1900	.36	1000	.19	30.0	4.49	21.3	.35	0.1	4.50	0.0018	0.60	B
5	31.47	0.049	3200	.61	1500	.28	19.8	4.02	22.6	.35	0.1	3.00	0.0018	0.50	C
6	44.19	0.069	2000	.38	850	.16	10.0	9.31	12.4	.35	0.1	3.00	0.0018	0.50	D
7	25.78	0.040	1800	.34	600	.11	40.0	1.31	20.0	.35	0.1	4.50	0.0018	0.60	B
8	53.77	0.084	2050	.39	600	.11	20.0	1.44	21.4	.35	0.1	4.50	0.0018	0.60	B
9	42.08	0.066	2850	.54	1460	.28	15.0	8.32	15.2	.35	0.1	3.00	0.0018	0.50	C
10	9.52	0.015	650	.12	350	.07	10.0	4.03	13.6	.35	0.1	4.50	0.0018	0.60	B
11	49.04	0.077	4010	.76	2070	.39	65.0	3.26	32.3	.35	0.1	4.50	0.0018	0.60	B
12	47.2	0.074	3600	.68	1200	.23	40.0	2.44	30.0	.35	0.1	4.50	0.0018	0.60	B
13	40.88	0.064	3190	.60	1900	.36	10.0	6.78	23.0	.35	0.1	4.50	0.0018	0.60	B
14	118.9	0.186	5300	1.00	3000	.57	10.0	7.84	N/A	.35	0.1	4.50	0.0018	0.60	B
15	7.61	0.012	1500	.28	380	.07	10.0	7.17	13.1	.35	0.1	4.50	0.0018	0.60	B
16	107.05	0.167	5250	.99	2550	.48	25.0	7.73	N/A	.35	0.1	4.50	0.0018	0.60	B
17	28.97	0.045	2400	.45	840	.16	70.0	2.66	19.9	.35	0.1	4.50	0.0018	0.60	B
18	31.57	0.049	2100	.40	800	.15	60.0	2.99	20.2	.35	0.1	4.50	0.0018	0.60	B
19	50.9	0.080	3600	.68	1330	.25	70.0	2.82	22.8	.35	0.1	4.50	0.0018	0.60	B
20	51.44	0.080	3150	.60	1500	.28	10.0	8.68	15.4	.35	0.1	3.00	0.0018	0.50	C
21	79.88	0.125	3800	.72	2000	.38	10.0	8.62	18.0	.35	0.1	4.50	0.0018	0.60	B
22	117.5	0.184	4950	.94	2400	.45	10.0	7.53	N/A	.35	0.1	3.00	0.0018	0.50	D
23	247.96	0.387	5250	.99	1650	.31	10.0	7.03	N/A	.35	0.1	3.00	0.0018	0.50	D
24	95.23	0.149	3950	.75	1550	.29	10.0	8.04	N/A	.35	0.1	3.00	0.0018	0.50	D
25	12.66	0.020	1100	.21	500	.09	40.0	3.35	16.7	.35	0.1	4.50	0.0018	0.60	B

TABLE 2: RAINFALL DISTRIBUTION

2-YEAR, 1-HOUR PRECIPITATION (inches) = 0.64
5-YEAR, 1-HOUR PRECIPITATION (inches) = 0.80
10-YEAR, 1-HOUR PRECIPITATION (inches) = 1.00
50-YEAR, 1-HOUR PRECIPITATION (inches) = 1.40
100-YEAR, 1-HOUR PRECIPITATION (inches) = 1.63

		2-YEAR, 2-HOUR		5-YEAR, 2-HOUR		10-YEAR, 2-HOUR		50-YEAR, 2-HOUR		100-YEAR, 2-HOUR	
TIME (minutes)	TIME (hours)	RAINFALL DISTRIBUTION % Distribution	INCREMENTAL RAINFALL (inches)	RAINFALL DISTRIBUTION % Distribution	INCREMENTAL RAINFALL (inches)	RAINFALL DISTRIBUTION % Distribution	INCREMENTAL RAINFALL (inches)	RAINFALL DISTRIBUTION % Distribution	INCREMENTAL RAINFALL (inches)	RAINFALL DISTRIBUTION % Distribution	INCREMENTAL RAINFALL (inches)
5	0.0833	2.0%	0.01	2.0%	0.02	2.0%	0.02	1.3%	0.02	1.0%	0.02
10	0.1667	4.0%	0.03	3.7%	0.03	3.7%	0.04	3.5%	0.05	3.0%	0.05
15	0.2500	8.4%	0.05	8.7%	0.07	8.2%	0.08	5.0%	0.07	4.6%	0.07
20	0.3333	16.0%	0.10	15.3%	0.12	15.0%	0.15	8.0%	0.11	8.0%	0.13
25	0.4167	25.0%	0.16	25.0%	0.20	25.0%	0.25	15.0%	0.21	14.0%	0.23
30	0.5000	14.0%	0.09	13.0%	0.10	12.0%	0.12	25.0%	0.35	25.0%	0.41
35	0.5833	6.3%	0.04	5.8%	0.05	5.6%	0.06	12.0%	0.17	14.0%	0.23
40	0.6667	5.0%	0.03	4.4%	0.04	4.3%	0.04	8.0%	0.11	8.0%	0.13
45	0.7500	3.0%	0.02	3.6%	0.03	3.8%	0.04	5.0%	0.07	6.2%	0.10
50	0.8333	3.0%	0.02	3.6%	0.03	3.2%	0.03	5.0%	0.07	5.0%	0.08
55	0.9167	3.0%	0.02	3.0%	0.02	3.2%	0.03	3.2%	0.04	4.0%	0.07
60	1.0000	3.0%	0.02	3.0%	0.02	3.2%	0.03	3.2%	0.04	4.0%	0.07
65	1.0833	3.0%	0.02	3.0%	0.02	3.2%	0.03	3.2%	0.04	4.0%	0.07
70	1.1667	2.0%	0.01	3.0%	0.02	3.2%	0.03	2.4%	0.03	2.0%	0.03
75	1.2500	2.0%	0.01	2.5%	0.02	3.2%	0.03	2.4%	0.03	2.0%	0.03
80	1.3333	2.0%	0.01	2.2%	0.02	2.5%	0.03	1.8%	0.03	1.2%	0.02
85	1.4167	2.0%	0.01	2.2%	0.02	1.9%	0.02	1.8%	0.03	1.2%	0.02
90	1.5000	2.0%	0.01	2.2%	0.02	1.9%	0.02	1.4%	0.02	1.2%	0.02
95	1.5833	2.0%	0.01	2.2%	0.02	1.9%	0.02	1.4%	0.02	1.2%	0.02
100	1.6667	2.0%	0.01	1.5%	0.01	1.9%	0.02	1.4%	0.02	1.2%	0.02
105	1.7500	2.0%	0.01	1.5%	0.01	1.9%	0.02	1.4%	0.02	1.2%	0.02
110	1.8333	2.0%	0.01	1.5%	0.01	1.9%	0.02	1.4%	0.02	1.2%	0.02
115	1.9167	1.0%	0.01	1.5%	0.01	1.7%	0.02	1.4%	0.02	1.2%	0.02
120	2.0000	1.0%	0.01	1.3%	0.01	1.3%	0.01	1.4%	0.02	1.2%	0.02
	TOTAL	115.7%	0.74	115.7%	0.93	115.7%	1.16	115.6%	1.62	115.6%	1.88

NOTE: 2-hour precipitation quantities are 1.16 times the 1-hour amount for a given frequency.

TABLE 3: CUHP FLOW SUMMARY

BASIN	2-YEAR	5-YEAR	10-YEAR	50-YEAR	100-YEAR
1	1	2	3	35	56
2	6	9	12	34	49
3	32	40	51	88	110
4	11	16	21	55	78
5	4	6	8	31	43
6	4	6	8	54	80
7	7	10	14	30	41
8	7	10	14	48	73
9	5	8	11	51	75
10	0	1	1	8	13
11	22	28	36	65	81
12	11	15	20	46	63
13	2	3	5	31	47
14	3	5	7	48	73
15	0	1	1	7	11
16	12	18	25	79	111
17	20	25	32	54	67
18	17	21	28	50	64
19	33	42	53	90	112
20	4	6	9	57	85
21	4	8	12	68	110
22	3	6	9	58	86
23	9	15	23	152	224
24	4	6	9	59	87
25	4	5	7	16	22

TABLE 4: SWMM ROUTING PARAMETERS

CONVEYANC ELEMENT	DOWNSTREA ONVEYAN ELEMENT	TYPE OF ONVEYANC ELEMENT	PIPE DIAMETER R CHANNEL WIDTH (feet)	LENGTH (feet)	LONGITUDIN SLOPE (feet/feet)	LEFT SIDE SLOPE (feet/feet)	RIGHT SIDE SLOPE (feet/feet)	IANNING'S n	OVERBANK/ SURCHARGE DEPTH (feet)
101			0	1	0.001	0	0	0.001	10
104			0	1	0.001	0	0	0.001	10
102	202		0	1	0.001	0	0	0.001	10
202	108	pe w/ Overfl	1.3	1000	0.065	0	0	0.024	1.25
			1	1000	0.065	50	50	0.035	10
108			0	1	0.001	0	0	0.001	10
121	221		0	1	0.001	0	0	0.001	10
221	120	Channel	10	1280	0.281	12	12	0.13	20
120	220		0	1	0.001	0	0	0.001	10
220	109	nnel w/ Ove	0.5	1140	0.07	12	12	0.045	0.5
			10	1140	0.07	50	50	0.14	20
109	209		0	1	0.001	0	0	0.001	10
209	107	pe w/ Overfl	2	1500	0.013	0	0	0.024	2
			1	1500	0.013	50	50	0.02	10
105	205		0	1	0.001	0	0	0.001	10
205	107	pe w/ Overfl	1.8	1050	0.01	0	0	0.015	1.75
			1	1050	0.01	50	50	0.02	10
106	206		0	1	0.001	0	0	0.001	10
206	107	pe w/ Overfl	2	1460	0.007	0	0	0.024	2
			1	1460	0.007	50	50	0.02	10
107	207		0	1	0.001	0	0	0.001	10
207	119	pe w/ Overfl	3	620	0.008	0	0	0.024	3
			1	620	0.008	50	50	0.02	10
119	219		0	1	0.001	0	0	0.001	10
219	103	pe w/ Overfl	4	300	0.01	0	0	0.024	4
			1	300	0.01	50	50	0.02	10
103	203		0	1	0.001	0	0	0.001	10
203	118	pe w/ Overfl	3	920	0.065	0	0	0.024	3
			1	920	0.065	50	50	0.035	10
118			0	1	0.001	0	0	0.001	10
116	216		0	1	0.001	0	0	0.001	10
216	117	pe w/ Overfl	3	2120	0.024	0	0	0.015	3
			1	2120	0.024	50	50	0.02	10
117	217		0	1	0.001	0	0	0.001	10
217	110	pe w/ Overfl	2	820	0.024	0	0	0.024	2
			1	820	0.024	50	50	0.02	10
123	223		0	1	0.001	0	0	0.001	10
223	122	Channel	10	1600	0.469	12	12	0.14	20
124	122		0	1	0.001	0	0	0.001	10
122	222		0	1	0.001	0	0	0.001	10
222	114	Channel	10	5220	0.343	12	12	0.053	20
113	213		0	1	0.001	0	0	0.001	10
213	114	nnel w/ Ove	0.5	110	0.133	12	12	0.045	0.5
			10	110	0.133	50	50	0.02	20
114	214		0	1	0.001	0	0	0.001	10
214	115	nnel w/ Ove	0.5	300	0.064	12	12	0.045	0.5
			10	300	0.064	50	50	0.02	20
115	215		0	1	0.001	0	0	0.001	10
215	112	pe w/ Overfl	3	1680	0.023	0	0	0.024	3
			1	1680	0.023	50	50	0.02	10
112	212		0	1	0.001	0	0	0.001	10
212	111	pe w/ Overfl	3	750	0.026	0	0	0.24	3
			1	750	0.026	50	50	0.035	10
111	211		0	1	0.001	0	0	0.001	10
211	110	pe w/ Overfl	3	1400	0.029	0	0	0.015	3
			1	1400	0.029	50	50	0.035	10
110			0	1	0.001	0	0	0.001	10

TABLE 5: ROUTING ELEMENT FLOW SUMMARY

ROUTING ELEMENT DESIGNATION	2-YEAR (cfs)	5-YEAR (cfs)	10-YEAR (cfs)	50-YEAR (cfs)	100-YEAR (cfs)
101	0	2	3	35	56
102	6	9	12	34	49
103	82	110	139	355	509
104	11	16	21	55	78
105	4	6	8	31	43
106	4	6	8	54	80
107	19	31	43	211	323
108	12	18	24	80	116
109	8	14	21	132	204
110	58	82	117	466	713
111	32	50	74	386	587
112	18	33	51	345	528
113	2	3	5	31	47
114	12	24	40	313	477
115	12	24	40	318	485
116	12	18	25	79	111
117	29	39	52	123	160
118	97	122	154	391	556
119	50	69	89	278	410
120	5	9	15	113	173
121	4	8	12	68	110
122	14	23	36	262	389
123	9	15	23	152	224
124	4	6	9	59	87
125	4	5	7	16	22
202	6	8	11	32	47
203	82	103	134	350	503
205	4	6	8	27	39
206	2	4	7	38	58
207	19	30	40	208	316
209	7	13	18	124	193
211	32	50	74	378	578
212	18	32	51	342	522
213	2	3	5	31	46
214	12	24	40	314	478
215	12	24	39	311	481
216	12	17	24	79	104
217	27	37	49	122	157
219	51	70	88	274	407
220	4	9	14	98	154
221	2	5	8	61	99
222	10	19	31	246	372
223	7	13	20	147	219

TABLE 6: COLLECTOR STORM SEWER FLOW CAPACITY

LOCATION	FROM	TO	REACH DESIGNATION	PIPE TYPE	STORM SEWER SIZE (inches)	WETTED PERIMETER (ft)	FLOW AREA (ft^2)	MANNING'S n	STORM SEWER SLOPE (ft/ft)	STORM SEWER CAPACITY (cfs)	STORM SEWER VELOCITY (fps)	LIMITED STORM SEWER VELOCITY (fps)	LIMITED STORM SEWER CAPACITY (cfs)
SYSTEM 1													
Cooper	Cleveland	Original	A	HDPE	18	4.71	1.77	0.012	0.012	13	7.1	7.1	13
Ute	Original	Alps	B	CMP	18	4.71	1.77	0.024	0.017	8	4.2	4.2	7
Original	Ute	Cooper	C	CMP	36	9.42	7.07	0.024	0.019	50	7.1	7.1	50
Original	Cooper	Spring & Main	D	CMP	36	9.42	7.07	0.024	0.019	50	7.1	7.1	50
Spring	Chair Lift	Main	E	CMP	24	6.28	3.14	0.024	0.025	19	6.2	6.2	19
Spring	Main	Outlet @ Ponds	F	RCP	36	9.42	7.07	0.015	0.025	92	13	12	85
SYSTEM 2													
Gelena	Snark	Durant	G	RCP	18	4.71	1.77	0.015	0.06	22	12.7	12	21
Durant	Gelena	Mill	H	RCP	24	6.28	3.14	0.015	0.0077	17	5.5	5.5	17
Mill		Durant	I	RCP	24	6.28	3.14	0.015	0.05	44	14	12	38
Mill	Durant	Cooper	J	RCP	36	9.42	7.07	0.015	0.042	119	16.8	12	85
Mill	Cooper	Hyman	K	RCP	36	9.42	7.07	0.015	0.018	78	11	11	78
Mill	Hyman	Main	L	CMP	48	12.57	12.57	0.024	0.017	102	8.1	8.1	102
Mill	Main	Rio Grande	M	CMP	48	12.57	12.57	0.024	0.046	167	13.3	12	151
Mill	Rio Grande	Roaring Fork River	N	CMP	48	12.57	12.57	0.024	0.029	133	10.6	10.6	133
SYSTEM 3													
Aspen	Durant	Hopkins	O	CMP	30	7.85	4.91	0.024	0.025	35	7.2	7.2	35
Hopkins	Aspen	Garmisch	P	CMP	36	9.42	7.07	0.024	0.027	60	8.4	8.4	59
Hopkins	First	Garmisch	Q	CMP	24	6.28	3.14	0.024	0.02	17	5.5	5.5	17
Garmisch	Durant	Hyman	R	CMP	24	6.28	3.14	0.024	0.022	18	5.8	5.8	18
Garmisch	Hyman	Hopkins	S	CMP	36	9.42	7.07	0.024	0.04	73	10.3	10.3	73
Garmisch	Hopkins	Main	T	CMP	36	9.42	7.07	0.024	0.0073	31	4.4	4.4	31
Main	Fifth	Second	U	RCP	18	4.71	1.77	0.015	0.03	16	9	9	16
Main	Second	Garmisch	V	RCP	24	6.28	3.14	0.015	0.0035	12	3.7	3.7	12
Garmisch	Main	Hallam	W	RCP	36	9.42	7.07	0.015	0.007	49	6.9	6.9	49
Garmisch	Hallam	Francis	X	CMP	48	12.57	12.57	0.024	0.007	65	5.2	5.2	65
Francis	Seventh	Garmish	Y	HDPE	15	3.93	1.23	0.012	0.01	7	5.7	5.7	7
	Francis & Garmisch	Roaring Fork River	Z	HDPE	36	9.42	7.07	0.012	0.038	141	20	12	85

Storm sewer slope is assumed to be equal to the street slope.
Storm sewer is assumed to be flowing full with no surcharge
HDPE storm sewer is assumed to have smooth walls.

Manning's n	
RCP =	0.015
HDPE =	0.012
CMP =	0.024

TABLE 7: STREET FLOW CAPACITY

LOCATION	FROM	TO	REACH DESIGNATION	WATER/ GUTTER DEPTH (feet)	STREET WIDTH (feet)	LONGITUDINAL SLOPE (ft/ft)	CROSS SLOPE (ft/ft)	FLOW AREA (ft^2)	SLOPE LENGTH (feet)	WETTED PERIMETER (feet)	MANNING'S n	STREET CAPACITY (cfs)	STREET VELOCITY (fps)
SYSTEM 1													
Cooper	Cleveland	Original	A	0.5	30	0.012	0.02	10.5	15	31	0.016	52	5
Ute	Original	Alps	B	0.5	30	0.017	0.02	10.5	15	31	0.016	61.9	5.9
Original	Ute	Cooper	C	0.5	45	0.019	0.02	12.375	22.5	46	0.016	66.2	5.3
Original	Cooper	Spring & Main	D	0.5	45	0.019	0.02	12.375	22.5	46	0.016	66.2	5.3
Spring	Chair Lift	Main	E	0.5	45	0.025	0.02	12.375	22.5	46	0.016	75.9	6.1
Spring	Main	Outlet @ W.Q. Ponds	F	0.5	45	0.025	0.02	12.375	22.5	46	0.016	75.9	6.1
SYSTEM 2													
Gelena	Snark	Durant	G	0.5	45	0.06	0.02	12.375	22.5	46	0.016	117.6	9.5
Durant	Gelena	Mill	H	0.5	45	0.0077	0.02	12.375	22.5	46	0.016	42.1	3.4
Mill		Durant	I	0.5	45	0.05	0.02	12.375	22.5	46	0.016	107.4	8.7
Mill	Durant	Cooper	J	0.5	45	0.042	0.02	12.375	22.5	46	0.016	98.4	8
Mill	Cooper	Hyman	K	0.5	45	0.018	0.02	12.375	22.5	46	0.016	64.4	5.2
Mill	Hyman	Main	L	0.5	45	0.017	0.02	12.375	22.5	46	0.016	62.6	5.1
Mill	Main	Rio Grande	M	0.5	45	0.046	0.02	12.375	22.5	46	0.016	103	8.3
Mill	Rio Grande	Roaring Fork River	N	0.5	45	0.029	0.02	12.375	22.5	46	0.016	81.8	6.6
SYSTEM 3													
Aspen	Durant	Hopkins	O	0.5	45	0.025	0.02	12.375	22.5	46	0.016	75.9	6.1
Hopkins	Aspen	Garmisch	P	0.5	45	0.027	0.02	12.375	22.5	46	0.016	78.9	6.4
Hopkins	First	Garmisch	Q	0.5	45	0.02	0.02	12.375	22.5	46	0.016	67.9	5.5
Garmisch	Durant	Hyman	R	0.5	45	0.022	0.02	12.375	22.5	46	0.016	71.2	5.8
Garmisch	Hyman	Hopkins	S	0.5	45	0.04	0.02	12.375	22.5	46	0.016	96.1	7.8
Garmisch	Hopkins	Main	T	0.5	45	0.0073	0.02	12.375	22.5	46	0.016	41	3.3
Main	Fifth	Second	U	0.5	65	0.03	0.02	12.5	25	51	0.016	79	6.3
Main	Second	Garmisch	V	0.5	65	0.0035	0.02	12.5	25	51	0.016	27	2.2
Garmisch	Main	Hallam	W	0.5	45	0.007	0.02	12.375	22.5	46	0.016	40.2	3.2
Garmisch	Hallam	Francis	X	0.5	45	0.007	0.02	12.375	22.5	46	0.016	40.2	3.2
Francis	Seventh	Garmish	Y	0.5	0	0.01	0.02	0	0	1	0.016	0	0
	Francis & Garmisch	Roaring Fork River	Z	0	NA	0.038	NA	NA	NA	NA	NA	N/A	N/A

TABLE 8: COMPARISON OF FLOW TO DRAINAGE FACILITIES CAPACITY

LOCATION	FROM	TO	REACH DESIGNATION	PEAK FLOW 2-YEAR (cfs)	PEAK FLOW 5-YEAR (cfs)	PEAK FLOW 10-YEAR (cfs)	PEAK FLOW 50-YEAR (cfs)	PEAK FLOW 100-YEAR (cfs)	STORM SEWER CAPACITY (cfs)	STREET CAPACITY (cfs)	Channel Capacity (cfs)	TOTAL COMBINED CAPACITY (cfs)
SYSTEM 1												
Cooper	Cleveland	Original	A	9	12	15	36	49	13	52	0	65
Ute	Original	Alps	B	12	24	40	318	485	7	62	2	72
Original	Ute	Cooper	C	18	33	50	340	520	50	66	0	116
Original	Cooper	Spring & Main	D	18	33	51	345	528	50	66	0	116
Spring	Chair Lift	Main	E	19	24	31	56	70	19	76	0	95
Spring	Main	Outlet @ Ponds	F	32	50	74	386	587	85	76	0	161
SYSTEM 2												
Gelena	Snark	Durant	G	12	18	25	79	111	21	118	0	139
Durant	Gelena	Mill	H	12	18	25	79	111	17	42	0	59
Mill	0	Durant	I	12	18	25	79	111	38	107	0	145
Mill	Durant	Cooper	J	25	33	45	105	137	85	98	0	183
Mill	Cooper	Hyman	K	26	34	46	109	141	78	64	0	142
Mill	Hyman	Main	L	27	37	49	116	151	102	63	0	164
Mill	Main	Rio Grande	M	29	39	52	123	160	151	103	0	254
Mill	Rio Grande	Roaring Fork River	N	29	39	52	123	160	133	82	0	215
SYSTEM 3												
Aspen	Durant	Hopkins	O	4	9	14	98	154	35	76	0	111
Hopkins	Aspen	Garmisch	P	4	9	14	98	154	59	79	0	138
Hopkins	First	Garmisch	Q	2	4	7	38	58	17	68	0	85
Garmisch	Durant	Hyman	R	42	57	74	231	340	18	71	0	89
Garmisch	Hyman	Hopkins	S	44	60	77	242	357	73	96	0	169
Garmisch	Hopkins	Main	T	45	62	80	250	368	31	41	0	72
Main	Fifth	Second	U	4	6	8	31	43	16	79	0	95
Main	Second	Garmisch	V	19	26	33	104	153	12	27	0	39
Garmisch	Main	Hallam	W	50	69	89	278	410	49	40	0	89
Garmisch	Hallam	Francis	X	76	102	128	328	470	65	40	0	106
Francis	Seventh	Garmish	Y	19	24	31	54	67	7	0	0	7
	Francis & Garmisch	Roaring Fork River	Z	82	103	134	350	503	85	0	0	85

TABLE 9 : FLOW COMPARISON OF SPAR GULCH AT ORIGINAL STREET

(DRAINAGE BASINS 14, 22, 23, AND 24)

RETURN FREQUENCY	PEAK FLOW			
		SNOWMELT ONLY		
	RAIN ONLY (cfs)	NATURAL SNOW ONLY (cfs)	NATURAL AND MANMADE SNOW (cfs)	RAIN ON SNOW (cfs)
AVERAGE ANNUAL	-	10	10	-
2-YEAR	70	15	15	226
5-YEAR	100	16	16	299
10-YEAR	155	17	17	387
50-YEAR	345	17	17	637
100-YEAR	474	18	18	794

NOTES:

Rain on snow event assumes 2-hour rain event on the average annual snowmelt event.

The flows shown in this table are estimated using the HEC-1 Computer Program. These flows may vary from the flows at the same location generated using CUHP/SWMM.

TABLE 10 : FLOW COMPARISON OF VALLEJO GULCH AT DURANT STREET

(DRAINAGE BASIN 16)

RETURN FREQUENCY	PEAK FLOW			
		SNOWMELT ONLY		
	RAIN ONLY (cfs)	NATURAL SNOW ONLY (cfs)	NATURAL AND MANMADE SNOW (cfs)	RAIN ON SNOW (cfs)
AVERAGE ANNUAL	-	2	2	-
2-YEAR	22	3	3	78
5-YEAR	32	4	4	102
10-YEAR	53	4	4	132
50-YEAR	117	4	4	200
100-YEAR	160	4	4	243

NOTES:

Rain on snow event assumes 2-hour rain event on the average annual snowmelt event.

The flows shown in this table are estimated using the HEC-1 Computer Program. These flows may vary from the flows at the same location generated using CUHP/SWMM.

TABLE 11 : DRAWING IDENTIFICATION

DRAWING NUMBER	(1) CITY STREETS	(1) CITY BUILDINGS	(2) REVISED TOPOGRAPHY	MODEL NODE SPACING (feet)	(3) NEW DEVELOPMENT	(4) METHOD USED TO MODEL DEVELOPMENT	STORM RETURN PERIOD (years)	FLOOD TYPE
1	NO	NO	NO	100	NO	--	100	WATER
2	NO	NO	NO	100	NO	--	100	MUD
3	NO	NO	NO	100	NO	--	10	WATER
4	NO	NO	NO	100	NO	--	10	MUD
5	NO	NO	YES	100	NO	--	100	MUD
6	YES	NO	YES	100	NO	--	100	MUD
7	YES	YES	YES	100	NO	GENERIC	100	MUD
8	YES	YES	NO	100	NO	GENERIC	100	MUD
9	YES	YES	YES	100	NO	SPECIFIC	100	MUD
10	YES	YES	YES	100	YES	GENERIC	100	MUD
11	YES	YES	YES	100	YES	SPECIFIC	100	MUD
12	YES	YES	YES	25	YES	SPECIFIC	100	MUD
13	YES	YES	YES	25	YES	SPECIFIC	100	MUD
14	YES	YES	YES	25	-	-	100	MUD

(1) Yes means that the FLO-2D model accounts for the buildings and streets in the City of Aspen.

(2) Yes means that the FLO-2D model accounts for the grading that was done south of Monarch Street.

(3) Yes means that the FLO-2D model includes a hypothetical development constructed south of Mill Street.

(4) Generic means that all nodes within the area of the hypothectical development are given the same flow reduction factors. Specific means that the nodes are defined differently. Nodes that fall on buildings will not allow flow through them and the other nodes will allow flow to pass through them with little reduction in flow width.

TABLE 12 : DIFFERENCE IN RUNOFF DUE TO INFILTRATION RATES AND METHODOLOGY

COMPUTER PROGRAM	LOSS METHODOLOGY	RUNOFF TYPE	LOCATION	TOTAL RAINFALL (inches)	TOTAL LOSS (inches)	TOTAL EXCESS (inches)	100-YR; 2-HR PEAK FLOW (cfs)
CUHP/SWMM	Horton's	RAIN	Vallejo Gulch	1.88	1.02	0.86	111
HEC-1	SCS	RAIN	Vallejo Gulch	1.9	1.29	0.61	81
HEC-1	Uniform	RAIN	Vallejo Gulch	1.9	0.67	1.23	229
HEC-1	Holtan's	RAIN	Vallejo Gulch	1.9	1.18	0.72	145
HEC-1	Green-Ampt	RAIN	Vallejo Gulch	1.9	1.14	0.76	154
FLO-2D	Green-Ampt	RAIN	Vallejo Gulch	1.88	-	-	158
CUHP/SWMM	Horton's	RAIN	Spar Gulch	1.88	-	-	477
HEC-1	Green-Ampt	RAIN	Spar Gulch	1.9	1	0.9	532
FLO-2D	Green-Ampt	RAIN	Spar Gulch	1.88	-	-	340
HEC-1	Uniform (K=0.05 in/hr)	Rain and Snowmelt	Vallejo Gulch	1.9			280
HEC-1	Uniform (K=0.5 in/hr)	Rain and Snowmelt	Vallejo Gulch	1.9			243
HEC-1	Uniform (K=0.85 in/hr)	Rain and Snowmelt	Vallejo Gulch	1.9			215

NOTE: Infiltration parameters were estimated based on the soil types and conditions expected to be found within the two sub-watersheds. When applicable, the values of the infiltration parameters were kept constant for the different methodologies.

TABLE 13 : INPUT DATA USED TO GENERATE FLOWS SHOWN IN TABLE 16

COMPUTER PROGRAM	LOSS METHODOLOGY	RUNOFF TYPE	LOCATION	ABSTRACTION (inches)	IMPERVIOUSNESS (%)	SATURATED HYDRAULIC CONDUCTIVITY (in/hr)	WETTING FRONT SUCTION (inches)	GROWTH INDEX (GIA)	AVAILABLE SOIL MOISTURE CAPACITY (in)	CURVE NUMBER	INITIAL HYDRAULIC CONDUCTIVITY (in/hr)	DECAY COEFFICIENT
CUHP/SWMM	Horton's	RAIN	Vallejo Gulch	0.1	25	0.6					4.5	0.0018
HEC-1	SCS	RAIN	Vallejo Gulch	0.2	20					55		
HEC-1	Uniform	RAIN	Vallejo Gulch	0.2	20	0.5						
HEC-1	Holtan's	RAIN	Vallejo Gulch		20	0.5		1	1.5			
HEC-1	Green-Ampt	RAIN	Vallejo Gulch	0.2	20	0.85	4.3					
FLO-2D	Green-Ampt	RAIN	Vallejo Gulch	0.2	20	0.5	4.8					
CUHP/SWMM	Horton's	RAIN	Spar Gulch	0.1	10	0.5					3	0.0018
HEC-1	Green-Ampt	RAIN	Spar Gulch	0.2	20	0.3	6.2					
FLO-2D	Green-Ampt	RAIN	Spar Gulch	0.2	20	0.3	6.2					
HEC-1	Uniform (K=0.05 in/hr)	Rain and Snowmelt	Vallejo Gulch	0	20	0.05						
HEC-1	Uniform (K=0.5 in/hr)	Rain and Snowmelt	Vallejo Gulch	0	20	0.5						
HEC-1	Uniform (K=0.85 in/hr)	Rain and Snowmelt	Vallejo Gulch	0	20	0.85						

NOTE: Infiltration parameters were estimated based on the soil types and conditions expected to be found within the two sub-watersheds. When applicable, the values of the infiltration parameters were kept constant for the different methodologies.

TABLE 14 : MONTHLY MEAN MAXIMUM TEMPERATURE

YEAR	JAN (ºF*10)	FEB (ºF*10)	MAR (ºF*10)	APR (ºF*10)	MAY (ºF*10)	JUN (ºF*10)	JUL (ºF*10)	AUG (ºF*10)	SEP (ºF*10)	OCT (ºF*10)	NOV (ºF*10)	DEC (ºF*10)	ANNUAL (ºF)	MAY (ºF*10)	YEAR	ORDER	PERCENTILE
				.													
1914	354	356	451	536	649	731	753	770	726	593	546	322	56.6	M	1934		
1915	328	401	413	583	603	705	793	773	681	614	447	365	55.9	M	1948		
1916	340	408	475	542I	619I	750I	789	740I	697I	572	441	297	55.6	M	1978		
1917	298	354	370	482	553	725	816	M	M	601I	518	406		M	1980		
1918	282	375	478	493	641	788	776	776	680	587	396	332	55	M	1991		
1919	341	334	431	561	668	742I	M	M	M	M	M	M		681I	1969		
1934	M	M	M	M	M	770I	835	804	684	643	449	368		666I	1942		
1935	395	403	432	504	575	746	809	782	717	587	441	374	56.4	665I	1958		
1936	317	342	447	557	692	774	820	772	702	595I	439	375	56.9	620I	1979		
1937	269	352	416	520	673	725	768	795	727	594	449	378	55.5	619I	1916		
1938	360	378	419	535	592	730	778	785	707	606	410	365	55.5	604I	1990		
1939	332	269	445	577	682	739	842	801	722	592	514	440	58	692	1936	1	1.7%
1940	321	368	451	540	668	772	829	788	692	542	350	383	55.9	689	1974	2	3.4%
1941	347	407	419	475	632	686	777	769	683	558	443	381	54.8	682	1939	3	5.1%
1942	328	305	386	544I	666I	744	804	807	728I	M	M	407I		673	1937	4	6.8%
1943	327	422	459	654	659	751	811	771	730	595	470	371	58.5	669	1966	5	8.5%
1944	335	356	391	482	619	731	788	801	754	619	451	375	55.9	668	1919	6	10.2%
1945	375	391	432	474	645	691	803	760	703	615	480	336	55.9	668	1940	7	11.9%
1946	350	415	481	606	611	771	809	790	749	562	467	435	58.7	664	1956	8	13.6%
1947	333	406	432	518	658	686	800	777	736	645	391	362	56.2	664	1996	9	15.3%
1948	M	M	M	M	M	M	M	777	766	615	376	359		660	1954	10	16.9%
1949	295I	355	444	560	632	690	771	758	699	577I	533	349	55.5	659	1943	11	18.6%
1950	350I	384	397	528	603	753	763I	746I	670I	661I	494	407	56.3	658	1947	12	20.3%
1951	327	382	409	504	633	683	802	749I	713	558I	421	312	54.1	657	1963	13	22.0%
1952	338I	335	370I	540I	619	756I	775	747I	718I	646	405	338I	54.9	657	1977	14	23.7%
1953	390	360	432	487I	583	762	794	767	743	610	476	350	56.3	653	1970	15	25.4%
1954	424	447	405	590	660	760	813	782	692	604	496	355	58.6	652	1961	16	27.1%
1955	320	319	410	516	620	700	804	783	737	619I	405I	379	55.1	652	1994	17	28.8%
1956	386	329	441	523	664	768	782I	752	750	623	379	339	56.1	649	1914	18	30.5%
1957	307	421	418	472	568	692	760	760	690	571	376	372	53.4	647	1960	19	32.2%
1958	335	401	390	475	665I	751	780	812	715	613	466	435	57	647	1972	20	33.9%
1959	401	M	488	557	642	769	794	767	677	565	479	423		645	1945	21	35.6%
1960	350I	323	458	583	647	765	816I	805	731	591	474	380	57.7	642	1959	22	37.3%
1961	381	402	451	508	652	764	797	785	621	590	426	320	55.8	642	1989	23	39.0%
1962	326	421	421	585	633	731	777	782	706	619	473	376	57.1	641	1918	24	40.7%
1963	292	403	415	536	657	727	829	750	741	660	473	344	56.9	640	1964	25	42.4%
1964	288	300	360	496	640	719	825	747	692	632	419	323	53.7	638	1992	26	44.1%
1965	337	361	353	526	619	703	773	740	621	642	462	368	54.2	636	1976	27	45.8%
1966	312	M	474	556	669	738	815	787	726	584	484	312		633	1951	28	47.5%
1967	330	375	487	548	599	691	796	780	712	618	470	304	55.9	633	1962	29	49.2%
1968	340	374	440	471	612	755	787	721	677	612	388	317	54.1	632	1941	30	50.8%
1969	377	363	389	576	681I	682	803	817	698	489I	421	355	55.4	632	1949	31	52.5%
1970	319	415	397	463	653	728I	799	801	666	519	424	344	54.4	632	1984	32	54.2%
1971	330	347	421	541	607	756	806	804I	680	581	422	285	54.8	628	1973	33	55.9%
1972	324	392	490	M	647	750	808	778I	679	557I	364	289		625	1988	34	57.6%
1973	301	376	422I	462	628	715	764	774	682	622	469	322	54.5	620	1955	35	59.3%
1974	280	357	475	513	689	762	789I	766	699	602	428	293	55.4	620	1985	36	61.0%
1975	320	342	407	482	597	704	791I	785	710	632	432	364I	54.7	619	1944	37	62.7%
1976	326	419I	418	554	636	753	821	765	698I	585I	476	386	57	619	1952	38	64.4%
1977	324I	407	420I	586	657	799	804	785	731I	607I	451	364I	57.8	619	1965	39	66.1%
1978	M	M	M	M	M	795I	832	799	721	619	465I	261		618	1986	40	67.8%
1979	249	329	431I	528I	620I	740I	824	M	752	619	362I	M		614	1987	41	69.5%
1980	M	M	M	M	M	M	782	743	693	561	453	452		612	1968	42	71.2%
1981	426	395	425	567	586	738	758	743	694	528	467	349	55.6	611	1946	43	72.9%
1982	307	357	417	480	582	671	753	750	656	519	407	343I	52	610	1993	44	74.6%
1983	374	392I	413	445	549	672	780	766	709	582	412	287	53.2	609	1997	45	76.3%
1984	306	392	414	460	632	676	777	743	677	469	430	352	52.7	607	1971	46	78.0%
1985	322	328	433	525	620	731	761	770	638	571	388	360	53.7	603	1915	47	79.7%
1986	M	393	508	519	618	732	739	759	626	531	435	359		603	1950	48	81.4%
1987	332	418	450	559	614	747	779	746	694	608	433	312	55.8	599	1967	49	83.1%
1988	289	399	421	545	625	766	811	780	672	620	390	330	55.4	597	1975	50	84.7%
1989	338	373	500	551	642	721	802	740	705	605I	M	332		592	1938	51	86.4%
1990	347	403	M	541	604I	M	M	752	M	580I	482I	M		586	1981	52	88.1%
1991	M	428I	423I	533I	M	724	M	M	684	614I	398I	351		583	1953	53	89.8%
1992	358	404I	462	569	638	692	722	734	684	615	339I	311	54.4	582	1982	54	91.5%
1993	356	351	454	477	610	692	747	725	667	550	M	360		575	1935	55	93.2%
1994	366	368	472	521	652	767	778	776	694	553	407	M		568	1957	56	94.9%
1995	342	448	452	485	542	691	733	757	698	559	464	M		553	1917	57	96.6%
1996	329	416	M	504	664	731	785	741	638	564	444	326		549	1983	58	98.3%
1997	342	378	483	471	609	729	773	735	695	567	419	348	54.6	542	1995	59	100.0%
AVERAGE	335.4	374.7	434.3	525.9	628.3	730.3	790.7	770.5	698.6	591.1	441.8	352.6	55.6				
MINIMUM	249.0	269.0	353.0	445.0	542.0	671.0	722.0	721.0	621.0	469.0	350.0	261.0	52.0				
MAXIMUM	426.0	448.0	508.0	654.0	692.0	799.0	842.0	817.0	766.0	660.0	546.0	452.0	58.7				

TABLE 15 : MONTHLY MEAN MINIMUM TEMPERATURE

YEAR	JAN (°F*10)	FEB (°F*10)	MAR (°F*10)	APR (°F*10)	MAY (°F*10)	JUN (°F*10)	JUL (°F*10)	AUG (°F*10)	SEP (°F*10)	OCT (°F*10)	NOV (°F*10)	DEC (°F*10)	ANNUAL (°F)
1914	129	91	192	293	379	356I	458	425	375	285	172	30	26.5
1915	37	123	181	299	313	368	426	406	370	269	156	93	25.3
1916	93	131	201	255I	289I	365I	448	418I	353I	290	121	57	25.2
1917	20	100	93	207	291	371	443	410	363	251	220	166	24.5
1918	70	97	194	225	310	420	437	421	357	301	125	56	25.1
1919	4	40	131	264	327	319I	M	M	M	M	M	M	
1934	M	M	M	M	M	404I	478	477	364	305	211	105	
1935	110	116	172	269	325	390	447	464	370	288	205	100	27.1
1936	94	135	179	273	357	426	472	461	373	283I	162	105	27.7
1937	-6	62	169	247	347	397	467	450	387	312	217	136	26.5
1938	95	138	194	265	320	397	418	447	384	309	103	57	26.1
1939	52	-36	129	240	308	324	420	400	353	256	179	117	22.8
1940	61	118	195	280	344	395	458	432	403	275	119	106	26.6
1941	30	102	148	246	355	370	416	420	357	303	170	69	24.9
1942	1	-4	72	277I	333I	393	442	433	349I	181	131	92I	22.5
1943	36	-5	16	196	277	375	440	422	350	270	158	71	21.7
1944	16	84	124	208	305	364	441	418	371	285	181	39	23.6
1945	63	115	120	166	313	344	410	435	343	274	140	65	23.2
1946	9	50	174	278	286	361	435	415	319	251	115	105	23.3
1947	-8	87	136	239	305	335	409	436	369	291	103	60	23
1948	M	M	M	M	M	M	M	402	376	280	127	95	
1949	37I	55	200	277	334	383	439	406	361	245I	211	82	25.3
1950	80I	98	139	238	279	349	410I	381I	375I	320I	223	130	25.2
1951	60	94	167	248	330	347	435	422I	335	270I	126	75	24.2
1952	64I	36I	87I	259I	340	396I	437	433I	364I	264	119	44I	23.7
1953	145	58	171	225I	296	406	454	435	356	279	195	69	25.7
1954	116	167	170	286	338	374	469	428	381I	292	192	34	27.1
1955	26	-8	109	196	302	346	425	440	349	246I	148I	134	22.6
1956	147	28	120	252	347	406	418I	386	370	290	100	48	24.3
1957	64	159	180	246	315	374	443	438	341	309	148	84	25.8
1958	30	157	144	231	326I	403	427	445	376	275	160	110	25.7
1959	60	118	150	232	315	420	426	436	345	246	148	95	24.9
1960	56I	36	172	248	313	381	444I	439	390	285	172	85	25.2
1961	49	121	191	244	332	405	441	460	332	262	141	52	25.2
1962	14	148	105	268	304	377	423	404	352	296	227	135	25.4
1963	35	158	170	270	356	372	446	454	404	328	182	76	27.1
1964	22	12	79	228	326	385	475	425	369	300	188	104	24.3
1965	140	100	131	266	333	399	470	443	367	297	222	119	27.4
1966	33	M	161	242	338	389	464	413	358	270	217	73	
1967	102	101	230	259	318	384	461	427	367	282	195	80	26.7
1968	74	143	152	211	317	387	455	425	343	293	158	85	25.4
1969	132	97	118	274	363I	375	471	470	387	249I	139	120	26.6
1970	95	143	151	195	321	397I	453	454	353	239	212	113	26
1971	111	102	146	255	318	401	460	449I	332	274	169	81	25.8
1972	77	135	229	M	322	416	453	451I	389	323I	141	59	
1973	58	68	169I	206	332	388	448	449	370	290	211	99	25.7
1974	59	72	221	234	339	411	452I	420	359	323	170	60	26
1975	79	82	165	217	318	372	450I	437	358	260	153	116I	25.1

MAY (°F*10)	YEAR	ORDER	PERCENTILE
M	1934		
M	1948		
M	1978		
M	1980		
363I	1969		
342I	1991		
335I	1979		
333I	1942		
326I	1958		
289I	1916		
379	1914	1	1.7%
368	1996	2	3.3%
358	1985	3	5.0%
357	1936	4	6.7%
357	1984	5	8.3%
356	1963	6	10.0%
355	1941	7	11.7%
355	1987	8	13.3%
355	1994	9	15.0%
351	1997	10	16.7%
347	1937	11	18.3%
347	1956	12	20.0%
347	1989	13	21.7%
345	1976	14	23.3%
344	1940	15	25.0%
344	1977	16	26.7%
341	1992	17	28.3%
340	1952	18	30.0%
339	1974	19	31.7%
339	1988	20	33.3%
338	1954	21	35.0%
338	1966	22	36.7%
338	1981	23	38.3%
338	1990	24	40.0%
336	1986	25	41.7%
334	1949	26	43.3%
333	1965	27	45.0%
332	1961	28	46.7%
332	1973	29	48.3%
330	1951	30	50.0%
327	1919	31	51.7%
326	1964	32	53.3%
325	1935	33	55.0%
322	1972	34	56.7%
321	1970	35	58.3%
321	1982	36	60.0%
320	1938	37	61.7%
319	1995	38	63.3%

TABLE 15 : MONTHLY MEAN MINIMUM TEMPERATURE

YEAR	JAN (°F*10)	FEB (°F*10)	MAR (°F*10)	APR (°F*10)	MAY (°F*10)	JUN (°F*10)	JUL (°F*10)	AUG (°F*10)	SEP (°F*10)	OCT (°F*10)	NOV (°F*10)	DEC (°F*10)	ANNUAL (°F)
1976	75	152I	136	264	345	379	462	423	383I	252I	170	74	26
1977	77I	125	127I	272	344	426	478	465	406I	298I	259	M	
1978	M	M	M	M	M	439I	468	420	380	298	194I	69	
1979	54	131	173I	248I	335I	388I	457	M	382	308	127	M	
1980	M	M	M	M	M	M	483	457	398	279	173	193	
1981	117	120	183	290	338	418	482	461	402	301	218	119	28.7
1982	86	89	184	217	321	391	450	478	397	258	157	95	26
1983	81	128I	195	215	301	391	467	483	408	313	203	111	27.5
1984	38	82	158	216	357	393	468	468	392	260	185	115	26.1
1985	65	44	192	286	358	421	468	461	359	303	193	114	27.2
1986	M	168	240	280	336	443	463	459	383	286	193	98	
1987	77	133	158	270	355	421	456	453	371I	326	180	94	27.5
1988	61	111	147	261	339	458	478	471	362	320	184	81	27.3
1989	35	88	224	284	347	401	478	441	391I	280	199	90I	27.2
1990	71	124	238	296	338	438	466	455	421	288	185	43	28
1991	46	145	184	236	342I	421	452	456	387I	294I	157I	60	26.5
1992	25	111I	200	285	341	368	413	411I	372	303	112	37	24.8
1993	86	89	170	217	317	360	396	429	347	282	M	53	
1994	96	90	191	259	355	430	453	468	379	289	157	M	
1995	102	194	206	254	319	391	447	483	395	260	218	M	
1996	80	156	M	255	368	422	479	459	372	274	201	83	
1997	102	81	184	230	351	430	460	457	411	278	189	80	27.1
AVERAGE	65.5	97.7	163.6	248.4	327.9	391.4	450.2	440.2	369.6	283.7	171.3	87.8	25.6
MINIMUM	-8.0	-36.0	16.0	166.0	277.0	324.0	396.0	386.0	319.0	181.0	100.0	30.0	21.7
MAXIMUM	147.0	194.0	240.0	299.0	379.0	458.0	483.0	483.0	421.0	328.0	259.0	193.0	28.7

MAY (°F*10)	YEAR	ORDER	PERCENTILE
318	1967	39	65.0%
318	1971	40	66.7%
318	1975	41	68.3%
317	1968	42	70.0%
317	1993	43	71.7%
315	1957	44	73.3%
315	1959	45	75.0%
313	1915	46	76.7%
313	1945	47	78.3%
313	1960	48	80.0%
310	1918	49	81.7%
308	1939	50	83.3%
305	1944	51	85.0%
305	1947	52	86.7%
304	1962	53	88.3%
302	1955	54	90.0%
301	1983	55	91.7%
296	1953	56	93.3%
291	1917	57	95.0%
286	1946	58	96.7%
279	1950	59	98.3%
277	1943	60	100.0%

TABLE 16 : APPROXIMATE MAXIMUM DAILY TEMPERATURE FLUCTUATION IN MAY

(HYPOTHETICAL)

TIME	AVERAGE MONTHLY TEMPERATURE (°F)	2-YEAR MAXIMUM DAILY TEMPERATURE (°F)	5-YEAR MAXIMUM DAILY TEMPERATURE (°F)	10-YEAR MAXIMUM DAILY TEMPERATURE (°F)	50-YEAR MAXIMUM DAILY TEMPERATURE (°F)	100-YEAR MAXIMUM DAILY TEMPERATURE (°F)
12 PM	33	48	50	51	53	55
1 AM	33	48	50	51	53	55
2 AM	33	48	50	51	53	55
3 AM	33	48	50	51	53	55
4 AM	33	48	50	51	53	55
5 AM	33	48	50	51	53	55
6 AM	35	50	52	53	55	57
7 AM	38	53	55	56	58	60
8 AM	43	58	60	61	63	65
9 AM	48	63	65	66	68	70
10 AM	53	68	70	71	73	75
11 AM	58	73	75	76	78	80
12 AM	60	75	77	78	80	83
1 PM	62	77	79	80	82	85
2 PM	63	78	81	82	84	87
3 PM	63	78	81	82	84	87
4 PM	63	78	81	82	84	87
5 PM	61	76	79	80	82	85
6 PM	57	72	75	76	78	80
7 PM	52	67	70	71	73	75
8 PM	47	62	65	66	68	70
9 PM	43	58	60	61	63	65
10 PM	39	54	55	57	59	61
11 PM	36	51	52	53	55	58
12 PM	33	48	50	51	53	55

These values are based on the monthly mean minimum temperatures and the monthly mean maximum temperature as described in the report.

TABLE 17 : SNOWMELT ONLY DUE TO HEAT TRANSFER FROM RAIN TO SNOW

TEMPERATURE (°F)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)	5 MINUTE RAINFALL DEPTH (inches)	5 MINUTE SNOWMELT DEPTH (inches)
32	0.02	0.0000	0.05	0.0000	0.1	0.0000	0.15	0.0000	0.2	0.0000	0.25	0.0000	0.3	0.0000	0.35	0.0000	0.4	0.0000
34	0.02	0.0003	0.05	0.0007	0.1	0.0014	0.15	0.0021	0.2	0.0028	0.25	0.0035	0.3	0.0042	0.35	0.0049	0.4	0.0056
36	0.02	0.0006	0.05	0.0014	0.1	0.0028	0.15	0.0042	0.2	0.0056	0.25	0.0070	0.3	0.0084	0.35	0.0098	0.4	0.0112
38	0.02	0.0008	0.05	0.0021	0.1	0.0042	0.15	0.0063	0.2	0.0084	0.25	0.0105	0.3	0.0126	0.35	0.0147	0.4	0.0168
40	0.02	0.0011	0.05	0.0028	0.1	0.0056	0.15	0.0084	0.2	0.0112	0.25	0.0140	0.3	0.0168	0.35	0.0196	0.4	0.0224
42	0.02	0.0014	0.05	0.0035	0.1	0.0070	0.15	0.0105	0.2	0.0140	0.25	0.0175	0.3	0.0210	0.35	0.0245	0.4	0.0280
44	0.02	0.0017	0.05	0.0042	0.1	0.0084	0.15	0.0126	0.2	0.0168	0.25	0.0210	0.3	0.0252	0.35	0.0294	0.4	0.0336
46	0.02	0.0020	0.05	0.0049	0.1	0.0098	0.15	0.0147	0.2	0.0196	0.25	0.0245	0.3	0.0294	0.35	0.0343	0.4	0.0392
48	0.02	0.0022	0.05	0.0056	0.1	0.0112	0.15	0.0168	0.2	0.0224	0.25	0.0280	0.3	0.0336	0.35	0.0392	0.4	0.0448
50	0.02	0.0025	0.05	0.0063	0.1	0.0126	0.15	0.0189	0.2	0.0252	0.25	0.0315	0.3	0.0378	0.35	0.0441	0.4	0.0504
52	0.02	0.0028	0.05	0.0070	0.1	0.0140	0.15	0.0210	0.2	0.0280	0.25	0.0350	0.3	0.0420	0.35	0.0490	0.4	0.0560
54	0.02	0.0031	0.05	0.0077	0.1	0.0154	0.15	0.0231	0.2	0.0308	0.25	0.0385	0.3	0.0462	0.35	0.0539	0.4	0.0616
56	0.02	0.0034	0.05	0.0084	0.1	0.0168	0.15	0.0252	0.2	0.0336	0.25	0.0420	0.3	0.0504	0.35	0.0588	0.4	0.0672
58	0.02	0.0036	0.05	0.0091	0.1	0.0182	0.15	0.0273	0.2	0.0364	0.25	0.0455	0.3	0.0546	0.35	0.0637	0.4	0.0728
60	0.02	0.0039	0.05	0.0098	0.1	0.0196	0.15	0.0294	0.2	0.0392	0.25	0.0490	0.3	0.0588	0.35	0.0686	0.4	0.0784
62	0.02	0.0042	0.05	0.0105	0.1	0.0210	0.15	0.0315	0.2	0.0420	0.25	0.0525	0.3	0.0630	0.35	0.0735	0.4	0.0840
64	0.02	0.0045	0.05	0.0112	0.1	0.0224	0.15	0.0336	0.2	0.0448	0.25	0.0560	0.3	0.0672	0.35	0.0784	0.4	0.0896
66	0.02	0.0048	0.05	0.0119	0.1	0.0238	0.15	0.0357	0.2	0.0476	0.25	0.0595	0.3	0.0714	0.35	0.0833	0.4	0.0952
68	0.02	0.0050	0.05	0.0126	0.1	0.0252	0.15	0.0378	0.2	0.0504	0.25	0.0630	0.3	0.0756	0.35	0.0882	0.4	0.1008
70	0.02	0.0053	0.05	0.0133	0.1	0.0266	0.15	0.0399	0.2	0.0532	0.25	0.0665	0.3	0.0798	0.35	0.0931	0.4	0.1064
72	0.02	0.0056	0.05	0.0140	0.1	0.0280	0.15	0.0420	0.2	0.0560	0.25	0.0700	0.3	0.0840	0.35	0.0980	0.4	0.1120
74	0.02	0.0059	0.05	0.0147	0.1	0.0294	0.15	0.0441	0.2	0.0588	0.25	0.0735	0.3	0.0882	0.35	0.1029	0.4	0.1176
76	0.02	0.0062	0.05	0.0154	0.1	0.0308	0.15	0.0462	0.2	0.0616	0.25	0.0770	0.3	0.0924	0.35	0.1078	0.4	0.1232
78	0.02	0.0064	0.05	0.0161	0.1	0.0322	0.15	0.0483	0.2	0.0644	0.25	0.0805	0.3	0.0966	0.35	0.1127	0.4	0.1288
80	0.02	0.0067	0.05	0.0168	0.1	0.0336	0.15	0.0504	0.2	0.0672	0.25	0.0840	0.3	0.1008	0.35	0.1176	0.4	0.1344
82	0.02	0.0070	0.05	0.0175	0.1	0.0350	0.15	0.0525	0.2	0.0700	0.25	0.0875	0.3	0.1050	0.35	0.1225	0.4	0.1400
84	0.02	0.0073	0.05	0.0182	0.1	0.0364	0.15	0.0546	0.2	0.0728	0.25	0.0910	0.3	0.1092	0.35	0.1274	0.4	0.1456
86	0.02	0.0076	0.05	0.0189	0.1	0.0378	0.15	0.0567	0.2	0.0756	0.25	0.0945	0.3	0.1134	0.35	0.1323	0.4	0.1512
88	0.02	0.0078	0.05	0.0196	0.1	0.0392	0.15	0.0588	0.2	0.0784	0.25	0.0980	0.3	0.1176	0.35	0.1372	0.4	0.1568

From V.T. Chow, Handbook of Applied Hydrology, 1964

$M = 0.007*P*(T-32)$

Where M = Daily Snowmelt (in.)

P = Daily Rainfall (in.)

T = Temperature 10' above ground with saturated air (°F)

This relationship should hold for any duration of rain.

TABLE 18: RAINFALL AND SNOWMELT DISTRIBUTION

		2-YEAR, 2-HOUR			5-YEAR, 2-HOUR			10-YEAR, 2-HOUR			50-YEAR, 2-HOUR			100-YEAR, 2-HOUR		
TIME (minutes)	TIME (hours)	INCREMENTAL RAINFALL (inches)	SNOWMELT CAUSED BY RAIN (inches)	INCREMENTAL SNOWMELT AND RAINFALL (inches)	INCREMENTAL RAINFALL (inches)	SNOWMELT CAUSED BY RAIN (inches)	INCREMENTAL SNOWMELT AND RAINFALL (inches)	INCREMENTAL RAINFALL (inches)	SNOWMELT CAUSED BY RAIN (inches)	INCREMENTAL SNOWMELT AND RAINFALL (inches)	INCREMENTAL RAINFALL (inches)	SNOWMELT CAUSED BY RAIN (inches)	INCREMENTAL SNOWMELT AND RAINFALL (inches)	INCREMENTAL RAINFALL (inches)	SNOWMELT CAUSED BY RAIN (inches)	INCREMENTAL SNOWMELT AND RAINFALL (inches)
5	0.0833	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02
10	0.1667	0.03	0.006	0.03	0.03	0.006	0.04	0.04	0.009	0.05	0.05	0.011	0.06	0.05	0.011	0.06
15	0.2500	0.05	0.011	0.06	0.07	0.015	0.08	0.08	0.020	0.10	0.07	0.015	0.09	0.07	0.015	0.09
20	0.3333	0.10	0.022	0.12	0.12	0.025	0.15	0.15	0.033	0.18	0.11	0.024	0.14	0.13	0.027	0.16
25	0.4167	0.16	0.034	0.19	0.20	0.043	0.24	0.25	0.054	0.30	0.21	0.044	0.25	0.23	0.049	0.28
30	0.5000	0.09	0.019	0.11	0.10	0.022	0.13	0.12	0.025	0.15	0.35	0.077	0.43	0.41	0.090	0.50
35	0.5833	0.04	0.009	0.05	0.05	0.011	0.06	0.06	0.013	0.07	0.17	0.037	0.21	0.23	0.049	0.28
40	0.6667	0.03	0.006	0.04	0.04	0.009	0.04	0.04	0.009	0.05	0.11	0.024	0.14	0.13	0.027	0.16
45	0.7500	0.02	0.004	0.02	0.03	0.006	0.03	0.04	0.009	0.05	0.07	0.015	0.09	0.10	0.022	0.12
50	0.8333	0.02	0.004	0.02	0.03	0.006	0.03	0.03	0.006	0.04	0.07	0.015	0.09	0.08	0.020	0.10
55	0.9167	0.02	0.004	0.02	0.02	0.004	0.03	0.03	0.006	0.04	0.04	0.009	0.05	0.07	0.015	0.08
60	1.0000	0.02	0.004	0.02	0.02	0.004	0.03	0.03	0.006	0.04	0.04	0.009	0.05	0.07	0.015	0.08
65	1.0833	0.02	0.004	0.02	0.02	0.004	0.03	0.03	0.006	0.04	0.04	0.009	0.05	0.07	0.015	0.08
70	1.1667	0.01	0.002	0.01	0.02	0.004	0.03	0.03	0.006	0.04	0.03	0.006	0.04	0.03	0.006	0.04
75	1.2500	0.01	0.002	0.01	0.02	0.004	0.02	0.03	0.006	0.04	0.03	0.006	0.04	0.03	0.006	0.04
80	1.3333	0.01	0.002	0.01	0.02	0.004	0.02	0.03	0.006	0.03	0.03	0.006	0.03	0.02	0.004	0.02
85	1.4167	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.03	0.006	0.03	0.02	0.004	0.02
90	1.5000	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02
95	1.5833	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02
100	1.6667	0.01	0.002	0.01	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02
105	1.7500	0.01	0.002	0.01	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02
110	1.8333	0.01	0.002	0.01	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02
115	1.9167	0.01	0.002	0.01	0.01	0.002	0.01	0.02	0.004	0.02	0.02	0.004	0.02	0.02	0.004	0.02
120	2.0000	0.01	0.002	0.01	0.01	0.002	0.01	0.01	0.002	0.02	0.02	0.004	0.02	0.02	0.004	0.02
	TOTAL	0.74	0.15	0.89	0.93	0.19	1.12	1.16	0.25	1.41	1.62	0.35	1.96	1.88	0.41	2.29

NOTE: Snowmelt only caused by heat transfer from rain. Air temperature, short-wave radiation etc. is not considered for snowmelt here.
HEC-1 considers air temperature, short-wave radiation, etc. in it calculations of snow melt.
Temperature of rain based on average monthly temperature in May.

TABLE 19: COMPARISON OF CITY ALTERNATIVES

	CONCEPTUAL COST ESTIMATE	LEVEL OF PROTECTION	COMMENTS
ALTERNATIVE 1	$6,100,000		Flow conveyed by street and storm sewer
System 1	$2,693,000	100-Year	
System 2	$203,000	100-Year	
System 3	$3,204,000	100-Year	
ALTERNATIVE 2	$8,028,000		Flow conveyed by street and storm sewer
System 1	$3,289,000	100-Year	
System 2	$203,000	100-Year	
System 3	$4,536,000	100-Year	
ALTERNATIVE 3	$2,846,000		Flow conveyed by storm sewer only
System 1	$817,000	10-Year	
System 2	$179,000	10-Year	
System 3	$1,850,000	2- to 10-Year	

NOTES: In general, alternatives for each system are independent from other systems

TABLE 20: COMPARISON OF MOUNTAIN ALTERNATIVES

	CONCEPTUAL COST ESTIMATE	AVERAGE ANNUAL OPERATION AND MAINTENANCE EXPENSE	IMPACT ON MOUNTAIN AESTHETICS	RELATIVE RISK OF FAILURE	COMMENTS
ALTERNATIVE 1- CHANNEL/DRAIN	$10,969,000	Low	Low	Low	Stability analysis will need to be performed
ALTERNATIVE 2- CUTOFF WALL	$7,758,000	Medium	Low	Medium	Potential for erosion to expose walls
ALTERNATIVE 3- REGULATORY CONTROL	$0	High	None	High	Potential for 10's of millions of dollars of damage and loss of life. It will cost new development to implement regulations.

NOTES: In general, alternatives for each system are independent from other systems

TABLE 21: CONCEPTUAL DESIGN COST ESTIMATE

IN-CITY - SYSTEM 1

ITEM DESCRIPTION	QUANTITY	UNITS	UNIT COST ($/unit)	COST ($)
Storm Sewer - 18-Inch RCP	62	l.f.	150	$9,300
Storm Sewer - 24-Inch RCP	630	l.f.	100	$63,000
Storm Sewer - 27-Inch RCP	1,143	l.f.	110	$125,730
Storm Sewer - 30-Inch RCP	190	l.f.	210	$39,900
Culvert Inlet (Headwall/Wingwall/Riprap) (0 to 200 cfs)	4	each	20,000	$80,000
Culvert Outlet (Headwall/Wingwall/Riprap) (0 to 200 cfs)	4	each	20,000	$80,000
Manhole	9	each	7,000	$63,000
Channel Excavation w/Haul	540	c.y	16	$8,640
Basin Excavation (0 to 100,000 C.Y.)	88,800	c.y	10	$888,000
Seeding, Mulching, and Fertilizing	6.00	acre	8,700	$52,200
Clearing and Grubbing	6.00	acre	8,500	$51,000
Topsoil (Remove, Stockpile, and Replace)	8,400	c.y	9	$75,600
Curb and Gutter Removal and Replacement	1,500	l.f.	45	$67,500
Asphalt Pavement/Base Coarse Removal and Replacement	1,770	s.y.	22	$38,940
			TOTAL CONSTRUCTION COST	**$1,642,800**

	% of Subtotal	
Engineering	15.0%	$246,420
Contractor Mob/Demob	5.0%	$82,140
Construction Contingency	10.0%	$164,280
Traffic Control	0.8%	$13,142
Utility Relocation	8.0%	$131,424
	TOTAL CONTINGENCY COST	**$637,400**
	TOTAL COST	***$2,280,000***

TABLE 21: CONCEPTUAL DESIGN COST ESTIMATE (CONT.)

IN-CITY - SYSTEM 2

ITEM DESCRIPTION	QUANTITY	UNITS	UNIT COST ($/unit)	COST ($)
Storm Sewer - 21-Inch RCP	1,002	l.f.	90	$90,180
Storm Sewer - 48-Inch RCP	201	l.f.	220	$44,220
Inlets - Double Grated	3	each	5,000	$15,000
Manholes	6	each	8,000	$48,000
Curb and Gutter Removal and Replacement	1,022	l.f.	45	$45,990
Asphalt Pavement/Base Coarse Removal and Replacement	1,100	s.y.	22	$24,200
			TOTAL CONSTRUCTION COST	**$267,600**

	% of Subtotal	
Engineering	15.0%	$40,140
Contractor Mob/Demob	5.0%	$13,380
Construction Contingency	10.0%	$26,760
Traffic Control	10.0%	$26,760
Utility Relocation	30.0%	$80,280
	TOTAL CONTINGENCY COST	**$187,300**
	TOTAL COST	***$455,000***

Note: The cost of the water quality extended detention basin is included in the System 1 cost estimate.

TABLE 21: CONCEPTUAL DESIGN COST ESTIMATE (CONT.)

IN-CITY - SYSTEM 3

ITEM DESCRIPTION	QUANTITY	UNITS	UNIT COST ($/unit)	COST ($)
Storm Sewer - 18-Inch RCP	419	l.f.	100	$41,900
Storm Sewer - 24-Inch RCP	882	l.f.	110	$97,020
Storm Sewer - 27-Inch RCP	2,462	l.f.	95	$233,890
Storm Sewer - 36-Inch RCP	280	l.f.	150	$42,000
Storm Sewer - 48-Inch RCP	630	l.f.	160	$100,800
Storm Sewer - 54-Inch RCP	1,022	l.f.	270	$275,940
Inlets - Single Combination	50	each	4,000	$200,000
Manholes	23	each	5,000	$115,000
Culvert Inlet (Headwall/Wingwall/Riprap) (0 to 200 cfs)	1	each	20,000	$20,000
Culvert Outlet (Headwall/Wingwall/Riprap) (0 to 200 cfs)	2	each	30,000	$60,000
Seeding, Mulching, and Fertilizing	3.50	acre	8,700	$30,450
Clearing and Grubbing	3.50	acre	12,000	$42,000
Basin Excavation (0 to 100,000 C.Y.)	32,200	c.y	20	$644,000
Topsoil (Remove, Stockpile, and Replace)	4,360	c.y	15	$65,400
Curb and Gutter Removal and Replacement	2,450	l.f.	45	$110,250
Asphalt Pavement/Base Coarse Removal and Replacement	5,750	s.y.	20	$115,000
Building Removal or Destruction	3	each	30,000	$90,000
			TOTAL CONSTRUCTION COST	**$2,283,700**

	% of Subtotal	
Engineering	15.0%	$342,555
Contractor Mob/Demob	5.0%	$114,185
Construction Contingency	10.0%	$228,370
Traffic Control	4.1%	$93,632
Utility Relocation	17.8%	$406,499
	TOTAL CONTINGENCY COST	**$1,185,200**

TOTAL COST	***$3,469,000***
GRAND TOTAL COST	***$6,204,000***

FIGURES

APPENDIX B

MEETING MINUTES

DRAWINGS