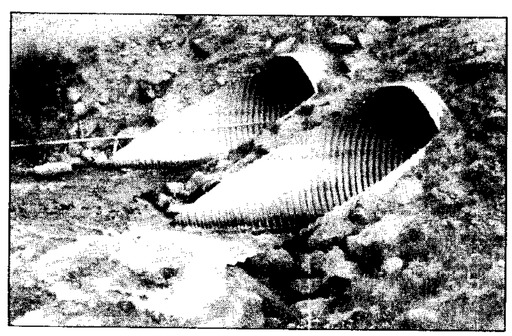
Municipality of Anchorage

FURROW CREEK

RABBIT CREEK



DRAINAGE STUDY

URS ENGINEERS
Anchorage, Alaska
February 1983

Alaska. Coastal Zone Management Program

FURROW CREEK-RABBIT CREEK DRAINAGE STUDY

for

Municipality of Anchorage

Department of Public Works

URS ENGINEERS

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FURROW CREEK-RABBIT CREEK DRAINAGE STUDY

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SUMMARY, CONCLUSIONS, & RECOMMENDATIONS

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The Municipality of Anchorage Department of Public Works, recognizing the need to improve and expand the existing stormwater drainage-related facilities in the south Anchorage areas of Furrow Creek and Rabbit Creek, authorized URS Company to prepare a comprehensive drainage study to meet the demands for the projected ultimate development of this area. The following paragraphs are a condensed review summarizing the major features of this drainage study, as well as stating specific conclusions and recommendations based upon the findings reached during the course of this study.

SUMMARY

The study area boundaries were established by the Municipality of Anchorage Department of Public Works. Topography was evaluated from existing Municipal contour maps. Existing land use patterns were determined from aerial photography. Existing drainage patterns within the study area resulted from a review of record drawings for subdivision plats, road improvement projects, storm drainage improvement projects and from site inspection. The existing storm drainage network was evaluated from a quality and quantity point of view through the review of various resource documents, computer simulation, public input, and field investigations.

An essential factor in the evaluation of existing and projected future systems in this storm drainage study is the prediction of land use patterns. Land use patterns used for the present were obtained from aerial photographs. The future land use was as published by the Municipality of Anchorage Planning Department. The existing and future land use patterns were used in the computation of runoff from the study area. Any deviation from this land use plan will decrease the value of this study in direct proportion to the magnitude of those changes.

Six alternative drainage/water quality control measures were formulated to meet the objectives of the defined goals. These alternatives were evaluated for each of the individual subcatchments within the thirteen subbasins. The criteria used to evaluate the alternatives and to size the proposed storm drainage networks were based upon future land use patterns, and storm drainage and water quality related goals of the Municipality of Anchorage. Population densities were based upon present and future population forecasts as published by the Municipal Planning Department. Planning criteria with respect to water quality objectives were as identified in the Anchorage 208 Water Quality Management Plan. Design rainfall storms were as identified from existing rainfall data information. A recommended alternative (or alternatives) is presented for each of the subcatchments which best fits the goals of the Municipality of Anchorage.

Compiling the recommended alternatives, a comprehensive storm drainage plan resulted. Cost estimates were developed for the major revisions or expansions to the existing and for proposed major storm drainage facilities.

CONCLUSIONS

1. The study area has a number of minor stormwater drainage collection facilities and in some cases, major stormwater drainage trunk systems which have been installed individually without considerations for an overall stormwater drainage network.

2. Presently, no water quality problems have been identified within the study area. Future stormwater runoff quality will not present any serious danger to the beneficial uses within the study area with minor exceptions to that of recreation and aesthetics in the Lower and Middle Furrow Creek segments.

3. The high percentage of localized flooding, inconvenience, safety hazards and maintenance problems associated with the storm drainage systems within the study area are the result of inadequate attention paid to the formation of ice and damage caused by ice within drainage structures.

4. Design, construction, and maintenance of storm drainage facilities within the study area have historically been directly associated with the design and construction of roadways and associated roadway improvements.

5. The areas which drain directly to Rabbit Creek and the areas flowing north out of the study area have minor and isolated stormwater quantity related problems but present no existing major stormwater/water quality related problems nor are any major quality problems projected for the future.

6. Areas within the Sunset Manor/Turnagain Park subdivisions (Subbasin G) which drain directly to Turnagain Arm have isolated cases of localized ponding of stormwater runoff. These problems are related to the undersizing of outfall structures and associated embankment erosion.

7. The Upper Furrow Creek drainage area, areas east of the New Seward Highway, is a developing area which needs immediate attention to an overall storm drainage network in order for development to proceed at a reasonable rate.

8. The Upper Furrow Creek segment does not have an adequate trunk system nor associated collection systems to convey existing and future stormwater runoff to the Middle Furrow Creek segment. The major cause of the overloading of the existing structures in the Middle and Lower Furrow Creek drainage networks is the result of the increased urbanization pressures in the Upper Furrow Creek segement.

9. The Middle Furrow Creek segment, the area between the New Seward Highway and the Alaska Railroad, has inadequate capacity for present or future flows for its entire length.

10. The Lower Furrow Creek segment, the area west of the Alaska Railroad to Turnagain Arm, exhibits isolated cases where inadequate capacity exists for both present and future runoff volumes. These areas are generally associated with railroad and street crossings along the creek corridor.

11. The maintenance of future wetland areas in their natural state is important to meet the goals and objectives of this study.

RECOMMENDATIONS

It is recommended that the following steps be taken to improve the existing storm drainage network within the study area:

1. Development of a stormwater drainage ordinance within the Municipality of Anchorage. This ordinance should identify the drainage criteria, construction, and inspection and operation of drainage structures. Implementation of this ordinance should be the responsibility of the Department of Public Works.

2. Areas presently not covered by road service districts should be either formed into local road improvement service districts or annexed into the Anchorage Road Service Area. These service areas will allow the legal mechanism to improve roadway conditions and stormwater networks for the area.

3. The construction of future storm drainage systems should be within public streets or right-of-ways or easements in conjunction with the expansion of the roadway network in the area.

4. A comprehensive stormwater drainage network should be designed and implemented per the recommendations for the individual subbasins and subcatchments covered by this report which would intertie all the isolated small trunk and collection systems and expand these systems into a comprehensive storm drainage network for the study area.

5. For areas of land identification for use as regional detention basins, the Municipality of Anchorage should purchase the land from the present land owner, thus ensuring its use as a detention basin.

6. The use of Level II control strategies as identified in the <u>208 Water Quality Management Plan</u> should be implemented for the management storm drainage related facilities with respect to water quality.

7. The installation of two precipitation gauges within the study area. One gauge should be located west of the Old Seward Highway and one gauge should be located east of the New Seward Highway.

8. In Subbasins L, M and portions of G which drain directly to Turnagain Arm, the existing outfall pipes should be removed

and installed with new outfall pipes with non-separating, non-leaking joints and designed to carry the identified capacity to avoid erosion of the bluff area.

9. In the Upper Furrow Creek segment, local depression and regional detention ponds should be incorporated into the drainage system throught the use of existing small depression areas and wetlands. As development pressures increase in Subbasins D, E, and F, a major stormwater trunk system network should be constructed to convey the collective stormwater runoff to the Middle Furrow Creek segment. These trunk systems should follow the existing natural corridors and street patterns to the maximum extent possible.

10. A study should be initiated to evaluate the various methods of increasing capacity of existing storm drainage network for the entire Middle Furrow Creek corridor. The study should be initiated immediately as severe constrictions exist for both present and future projected flows. Also, because of the potential for development at the west side of the Old Seward Highway and the intersection of Huffman, a method to transfer the collected upstream portions of water through the Alaska Railroad track foundations should be actively investigated and implemented.

11. In the Lower Furrow Creek segment, the existing road and stream crossings should be increased to carry the flows identified. Also, portions of the Lower Furrow Creek segment immediately west of the Alaska Railroad should be increased to carry projected

flows and alleviate a potential for severe embankment erosion for the Oceanview subdivision area.

12. All future designs and construction of stormwater drainage facilities within the study area should have methods for the control of ice formation and practical methods for maintenance crews to remove icing conditions at major street crossings.

13. In the design of future facilities, a preliminary feasibiity analysis should be performed by the developer and should be reviewed and approved by the Municipality.

CHAPTER ONE
INTRODUCTION

CHAPTER 1

INTRODUCTION

Anchorage is presently experiencing a high rate of growth. Areas which were previously undeveloped or contained limited development are now being impacted by the high growth rate resulting in rain and snowmelt flooding, glaciation and erosion. These problems will continue to increase in severity as growth continues, unless a comprehensive storm drainage plan is implemented.

This study analyzes the existing drainage system for current problem areas and predicts future system requirements within the study area of Furrow Creek and Lower Rabbit Creek. Information that was gathered includes existing and future land use, local hydrologic conditions, existing drainage facilities, soils and topography. This information was coded into the System Analysis Model (SAM) computer program to evaluate the hydrological and hydraulic response. The SAM output was then used in conjunction with information on future growth to establish drainage alternatives. Cost estimates were prepared for each chosen alternative. Pollutant load data was coded into the Storage Treatment Overflow Runoff Model (STORM) to compute the pollutant washoff loads and water quality effectiveness of each alternative.

This report sets forth the results, conclusions and recommendations from the storm drainage analysis for the study area.

Technical appendices have been included discussing the computer analysis and the problems and possible solutions of icing in culverts.

AUTHORIZATION

Recognizing the need to have a comprehensive storm drainage plan for the Furrow Creek-Rabbit Creek area, the Municipality of Anchorage authorized URS to conduct a storm drainage study for the Furrow Creek-Rabbit Creek area using the SAM and STORM computer models. This analysis has been completed in accordance with the terms of the Contract for Engineering Services by and between the Municipality of Anchorage, Alaska and URS Company, dated November 6, 1981.

SCOPE OF SERVICES

The scope of this study is as follows:

- Collect, with the assistance of the Municipality of Anchorage, the existing data necessary to complete the study.
- Review existing storm drainage and water quality planning and design requirements and modify to the study conditions.
- Establish the hydrologic boundaries within the approximate boundaries of the study area and delineate and identify major subbasins and their receiving waters.

- Establish subcatchments within each major subbasin.
- Select two single event design storms and one continuous period of average precipitation from spring break-up to winter freeze-up, using published available precipitation data.
- Develop, whenever possible, two to three conceptual drainage plans for each of the major subbasins.
- Simulate and evaluate the hydrologic and hydraulic response of the alternative plans to the selected design storms for each major subbasin using the Systems Analysis Model (SAM).
- Compute the pollutant wash-off loads and water quality effectiveness of the alternate plans over the selected continuous precipitation period. For each major subbasin use the Storage Treatment Overflow Runoff Model (STORM).
- Develop cost estimates for recommended plan.
- Provide the Municipality of Anchorage with input data for STORM and SAM computer models with an abstract outlining the procedure for the updating data file.
- Provide the Municipality of Anchorage with final computer output for each subbasin.
- Provide the Municipality of Anchorage with 1" = 1000' scale maps, to be incorporated into the final report.

° Arrange and conduct two community meetings with area residents to review alternatives and the final plan.

OBJECTIVE

It is the objective of this study to develop a comprehensive stormwater drainage and water quality plan for the Furrow Creek-Rabbit Creek area.

This study will provide a basis by which the Municipality of Anchorage Department of Public Works can make management decisions with respect to stormwater and water quality control measures for the area. As the study area grows in development, this plan can be used by both the public and private sector to implement an orderly development plan for the area.

PREVIOUS STUDIES

During the course of this study many resource documents were referred to for information. The following is a description of the information gathered from these documents. A summary list of references is located at the end of this report.

° Comprehensive Land Use Plan - published by the Municipal Planning Department in September 1981. In March 1982 additional land use and residential intensity map revisions became effective. The March 1982 land use was used as the basis for future land use within the study area.

- Anchorage Wetlands Management Plan - published by the Municipal Planning Department in October 1981. Supplemental revisions were added in February 1982. In April 1982 the plan was passed by the Assembly. The plan identifies the permitted use of wetlands within the study area. Land use within this report conforms to the Wetlands Management Plan.

- Hillside Wastewater Management Plan - published by Arctic Environmental Engineers and the Municipal Physical Planning Division in January 1982. The plan was passed by the Assembly in May 1982. The information contained in the plan was incorporated into the land use data of this study.

- Interim Snow Disposal Study - published by the Municipal Planning Department in January 1981. This study was used to identify the location of the existing snow disposal site within the study area as well as to gain information on the potential future snow disposal practices.

- Title 21 of the Anchorage Municipal Code - Land Use Regulation - became effective January 1, 1982. This document was used in identifying the local regulations for development.

- _208 Areawide Water Quality Management Plan, Anchorage, Alaska_ - published in August 1979. The 208 Plan provided information on water quality within the Anchorage area.

- Metropolitan Anchorage Urban Study, Volume 7, _Anchorage Area Soil Survey_, prepared by the Army Corps of Engineers in 1979. Soils information for this study area was obtained from this document.

- _Hydrology for Land Use Planning: the Hillside Area, Anchorage, Alaska_, Open File Report 75-105 - prepared by the Department of the Interior, 1975. Provided information on the Hillside area hydrology.

- _1995 Employment Population and Land Use Forecasts_ - prepared by the Municipal Planning Department in March 1977. Forecasts aided in establishing estimates of future conditions.

The following documents were referred to in using the SAM computer model:

- Campbell Creek Drainage Basin, Task Memorandum Number Seven, _Methodology Manual_, CH2M-Hill, January 1979.

- Wastewater Collection System Analysis Model (SAM), _User's Manual_, CH2M-Hill, June 1978.

- Storage Treatment Overflow Runoff Model (STORM), User's Manual, U.S. Army Corps of Engineers, 1976.

- Weather tape for 1963-1979, National Oceanic and Atmospheric Administration.

Reports which were referenced as general information on stormwater studies included:

- Drainage Management Plan for Homer, Alaska, CH2M-Hill, August 1979.

- Juanita Creek Drainage Plan, URS Engineers, 1977.

- Stormwater Drainage Study for the City of Soldotna, Ted Forsi & Associates, December 1979.

- Sand Lake Drainage and Water Quality Management Study, Quadra Engineering, August 1981.

- Urban Stormwater Management Special Report No. 49 - published by the American Public Works Association in 1981.

A number of researched reports were used in preparing the appendix on icing in storm drainage facilities. These reports included:

- "Solving Problems of Ice-Blocked Drainage Facilities", in Special Report 77-25, August 1977, K. L. Carey, Cold Regions Research and Engineering Laboratory.

- Soil Erosion and Sediment Control for Anchorage, Municipal Department of Public Works, December 1978.

- "Insulated Roadway Subdrains in the Subarctic for the Prevention of Spring Icings", H. Livingston and Eric Johnson.

- "Storm Drainage Design Considerations in Cold Regions", ASCE Conference Proceedings on Applied Techniques for Cold Environments, May 1978.

- "Icings Developed from Surface Water and Ground Water", CRREL Monograph 111-D3, Kevin Carey, May 1973.

- "Hydrologic Effects on Frozen Ground", CRREL Special Report 218, S. L. Dingman, March 1975.

Background information for this study was also obtained from:

- aerial photography
- record drawings
- flood plain insurance documents
- field investigation
- topographic maps
- public input

- Alaska Department of Environmental Conservation - water quality regulations
- Alaska Department of Fish and Game - fish and wildlife information

CHAPTER TWO
DESCRIPTION OF STUDY AREA

CHAPTER 2

DESCRIPTION OF STUDY AREA

LOCATION AND BOUNDARIES

The study area is located in the southwesterly portion of Anchorage, Alaska and is comprised of approximately 3825 acres. The area is bounded as follows:

- to the north by Klatt and O'Malley Roads
- to the east more or less by Cange and Elmore Roads
- to the south by Rabbit Creek Road
- to the west by unnamed wetlands and Turnagain Arm

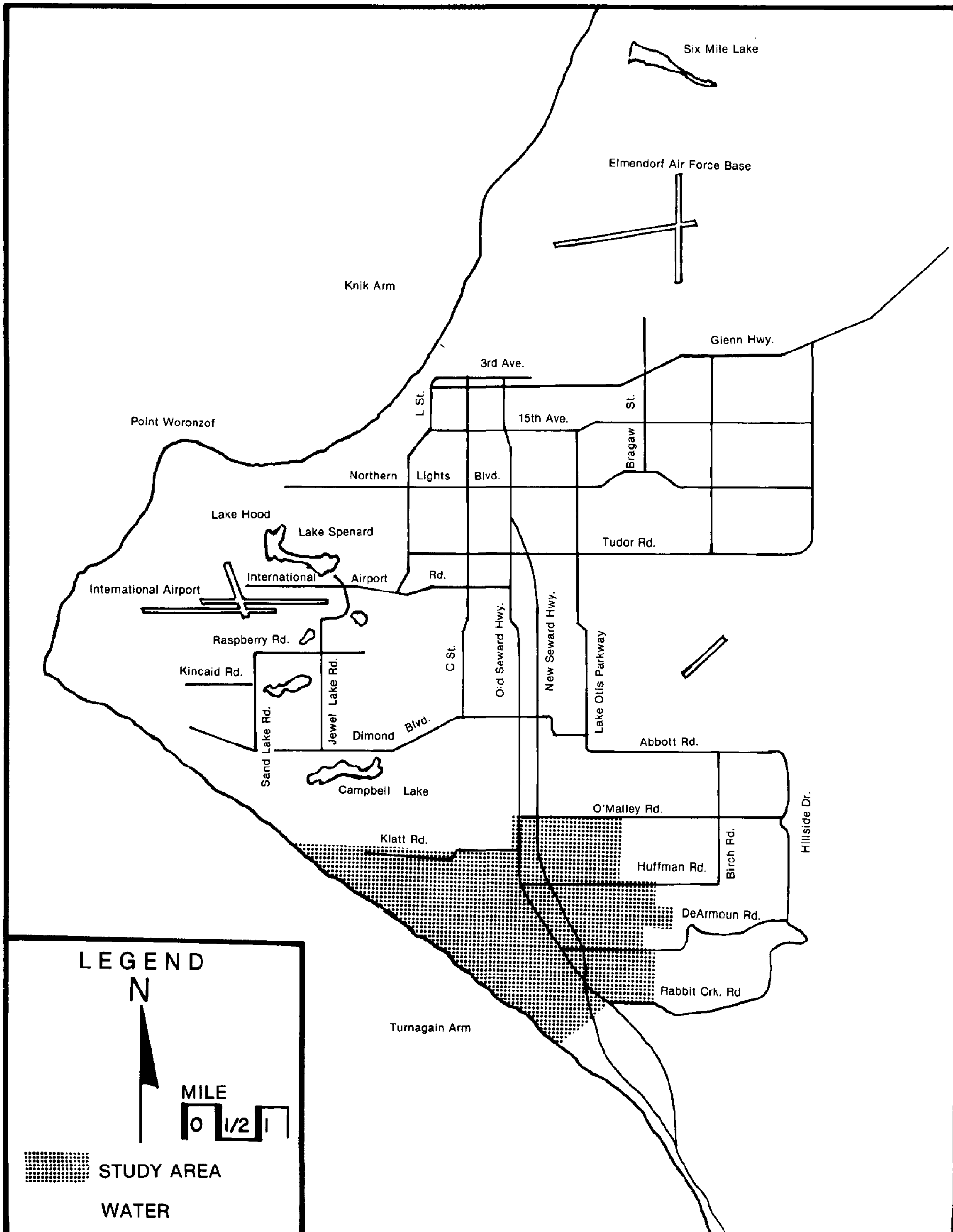
The study area location and boundaries are identified in Figure II-1.

It should be noted that while Rabbit Creek is a major stream which carries water from outside of the study area, only the subbasins which drain to Rabbit Creek within the study area boundaries are included in this study as per the contractual agreement.

CLIMATE

Anchorage Bowl

Within the Municipality of Anchorage, the "Anchorage Bowl", is that area bounded by the Glenn Highway to the north, Potter Marsh to the south, the Chugach Mountains to the east and Turnagain Arm to the west.



URS Engineers
Anchorage, Alaska

VICINITY MAP

February 1983 Figure II-1

The relatively moderate climate in the Anchorage bowl is due to the surrounding mountains. These mountain barriers shield the region from the temperature extremes of the Alaskan interior and the heavy precipitation of regions along the Gulf of Alaska.

Winters are not extremely cold with an average temperature from December through February of 14°F. The summer growing season averages 124 days with an average temperature from June through August of 56°F. The mean monthly temperature is about 35°F.

The mean annual precipitation in the region is 14.9 inches and 46 percent of the annual precipitation falls from July through September. The year's greatest monthly rainfall is 2.50 inches during September. The annual precipitation includes a mean snowfall of 70.2 inches and the greatest monthly snowfall is 15.1 inches during December.

Prevailing winds in the Anchorage area are from the northeast and normally light. This phenomenon results from the fact that air movement is normally from the cold ice-capped mountains to the warmer Cook Inlet waters and that strong outside winds are blocked by the mountains.

Furrow Creek-Rabbit Creek Area

Within the study area temperature extremes are probably a few degrees greater and precipitation volumes somewhat higher than the values presented for the Anchorage Bowl. The weather station for the Anchorage Bowl is located at the Anchorage International Airport which is about six miles from the study area. Weather at the airport station appears to be affected more by marine influences. Therefore, temperatures there are slightly warmer than those of the Furrow Creek-Rabbit Creek area. The proximity of the Chugach Mountains and their effects, such as strong winds, may have some impact upon the local weather in the study area. These conditions are the primary reason for the variance in climate between the local study area and the airport weather stations according to the National Weather Service.

Although it was acknowledged during this study that the climate in the Furrow Creek-Rabbit Creek area is not the same as that of the airport weather station, no adjustments were made to the airport station data. The reason for this decision was that there was no longterm, reliable data available upon which to base an adjustment factor. Therefore, the climatological data used for this study is the data available from the Anchorage International Airport station.

GEOLOGY

Anchorage Bowl

Within the Municipality of Anchorage, the "Bowl" is defined as the area bounded generally by the Glenn Highway to the north, Potter Marsh to the south, the Chugach Mountains to the east, and Turnagain Arm to the west.

Although glacial activity formed the physical features of the Anchorage Bowl area, the Cook Inlet Basin has been an area of low relief subjected to marine and continental deposition since Tertiary times. According to physical evidence, the basin was subjected to five distinct glacial movements during the Pleistocence times. Physical features formed during these times include the outwash plain, upon which most of Anchorage is located, the morainal hills, melt-water channels, and lakes and swamps.

Soils in the Anchorage Bowl area vary from free-draining sands and gravels to highly impervious silts and clays. Vegetation varies accordingly. Well-drained tracts are forested with evergreen and deciduous trees. Poorly-drained marsh lands are covered by mosses, sedges, grasses, and other marsh plants. Swamps deposited with water-saturated peat are located extensively throughout the lowlands.

The water-table depth in the planning area is relatively shallow and generally lies within 30 feet of the land surface.

Furrow Creek-Rabbit Creek Area

The study area is generally comprised of three types of geological surfaces.

1. Poorly sorted material deposited by glaciers

Almost half of the study area includes this kind of deposit which is marked by long ridges once signifying the margins of former glaciers.

2. Sand and gravel

Sand and gravel deposited mainly by streams (particularly along Rabbit Creek) and along the stream channels cover at least one third of the study area. The sand and gravel are generally well-stratified and well-sorted, referring to the grain size similarity. The portion of the study area west of the railroad is also mainly sand and gravel.

3. Silt and clay

Silt and clay deposited in former lakes and ponds in the lowlands (between lower Huffman Road and lower O'Malley Road) comprise a small portion of the study area.

PHYSICAL FEATURES/TOPOGRAPHY

The eastern limits of the study area lie in the foothills of the Chugach Mountains. Along this eastern boundary the highest elevation is approximately 400 feet. From the eastern limits, the land slopes westward to Turnagain Arm. Two major collectors of the foothills runoff water are Furrow Creek and Rabbit Creek. Rabbit Creek, lying in the southern portion of the study area, commences east of the study area boundary and flows into Turnagain Arm. Furrow Creek also has its origin in the Chugach foothills. However, in the vicinity of the New Seward

Highway, Old Seward Highway and the Alaskan Railroad, the flow in Furrow Creek is interrupted by manmade obstructions. Downstream of the Alaskan Railroad the land again begins to drain to a central location and Furrow Creek is again recognizable. As with Rabbit Creek, Furrow Creek empties into Turnagain Arm.

Mud flats extend along the entire length of the shoreline of the study area. West of Timberlane Road and south of Klatt Road the land becomes very flat until it attains a boggy quality. This bog, called the Klatt Bog, is identified in the map for Subbasin L, Figure V-12.

GROUNDWATER

Groundwater is primarily rain water and snow melt which has seeped into pores in the soil, rock and sediment, and includes all the water below the water table. Two principal groundwater systems exist for the Anchorage Bowl. An upper unconfined system (water table) is separated from a lower confined system by a continuous layer of clay. This separation is less distinct in South Anchorage (Furrow Creek-Rabbit Creek area) because of the glacial deposits and various levels of clay, sand and gravel.

Information on groundwater and aquifers is readily available for Anchorage and its more densely populated areas but for the outlying parts of the Municipality, the data is more scattered and less complete.

The following are some general observations about Anchorage groundwater characteristics which were abstracted from several water study reports:

- The chemical quality of groundwater in Anchorage is good to excellent.
- The temperature of confined and unconfined groundwater averages 37°F between the surface and to a depth of 300 feet.
- The estimated annual yield of groundwater in the Bowl is approximately 17 - 28 mgd.
- The hydraulic gradient of the groundwater closely conforms with the regional topographic gradient.
- The summer base flow of Furrow Creek is dependent for a large part on groundwater ex-filtration.

SUBBASIN DELINEATION

Using topographic maps as a basis the study area was divided into areas in which each area had a central point, or node, to which it drained. The boundaries of these basins were then modified to comply with existing drainage facilities such as ditches and culverts. Boundaries were also adjusted where physical features such as roads provided a barrier to the flow path; the barrier thus becoming a boundary.

By this method the area was divided into thirteen subbasins, labeled A through M. Figure II-5 identifies subbasin boundaries. Subbasins A, B, and C drain to Rabbit Creek. The tributary area to Furrow Creek within the study area is defined by Subbasins D, E, F, G, H and K. Subbasins I and J drain stormwater to the north, out of the study area. Subbasins L and M drain directly to Turnagain Arm. The acreage associated with each subbasin is listed in Table II-1.

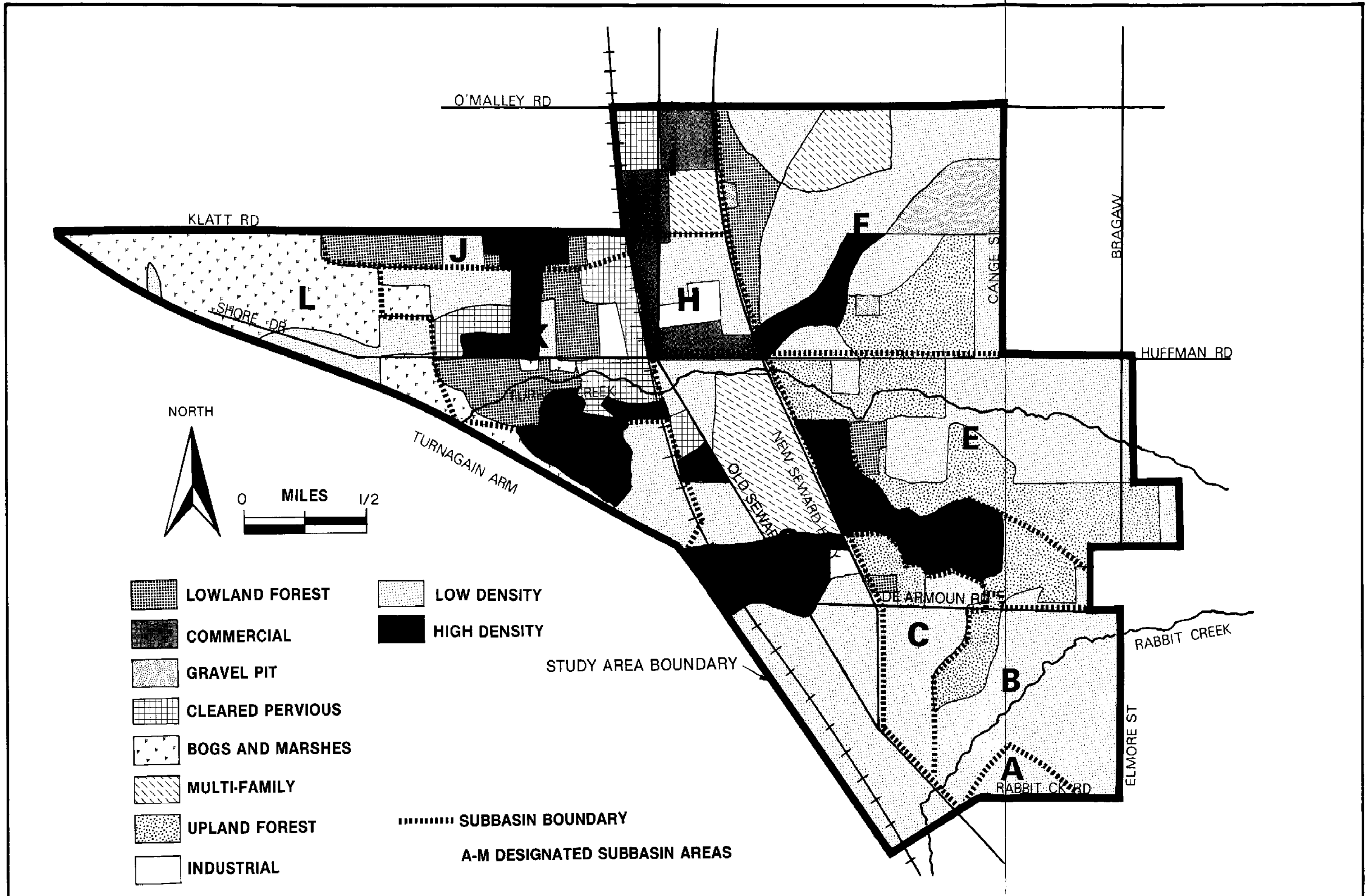
TABLE II-1

SUBBASIN ACREAGE

Subbasin		Area (acres)
A		27
B		351
C		168
D		198
E		592
F		767
G		526
H		138
I		131
J		81
K		385
L		314
M		148
	Total	3826

LAND USE

Land use was identified for the present and future cases. The existing land use was determined from aerial photography and field observations. The projected ultimate land use is as published in the revised land use and residential intensity maps produced by the Municipality of Anchorage Planning Department, dated March 1982. The existing and future land use is graphically depicted in Figures II-2, II-3, and II-4.

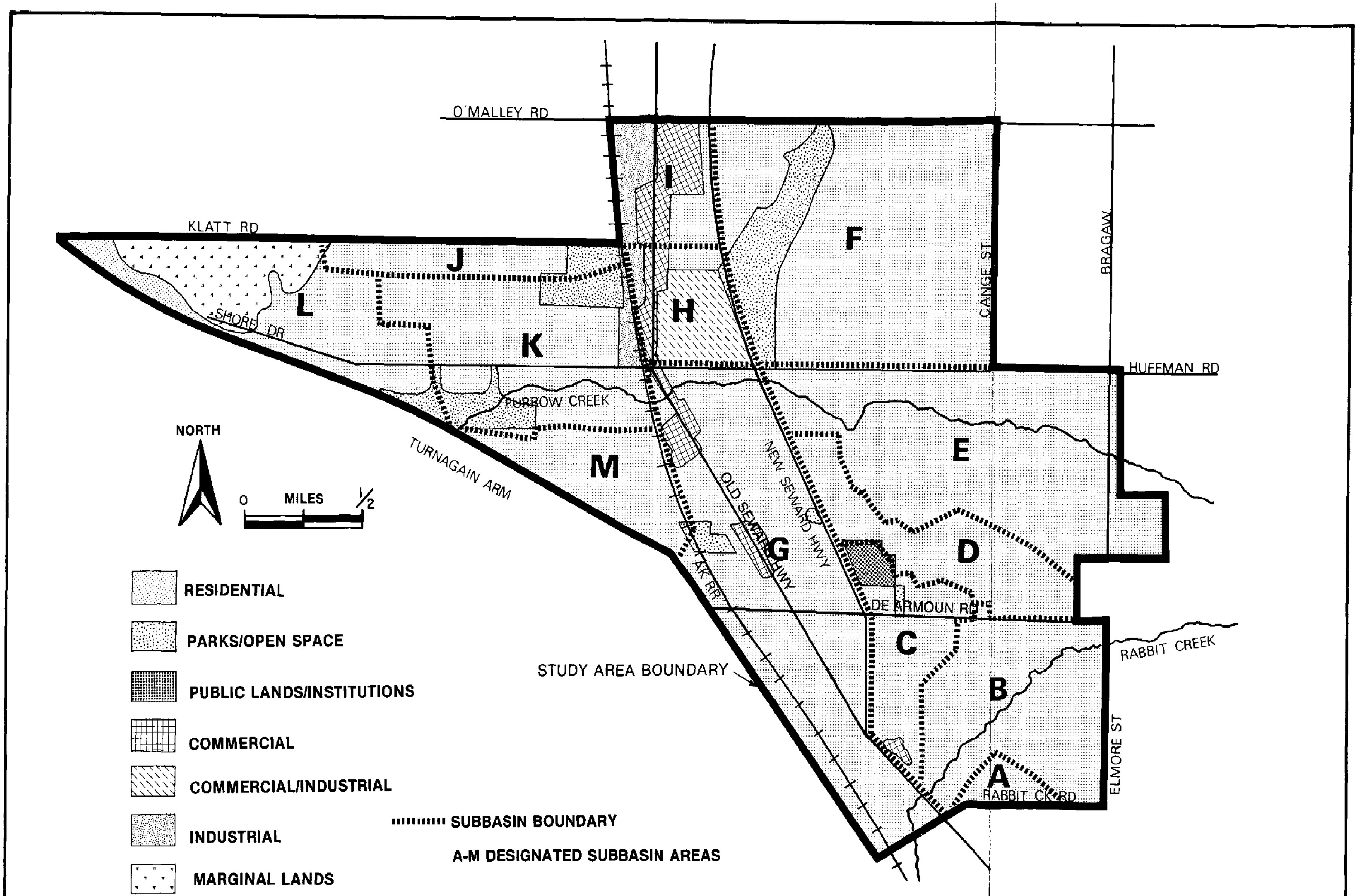


URS Engineers
Anchorage, Alaska

Per aerial photographs
September 10, 1980

EXISTING LAND USE MAP

February 1983 Figure II-2



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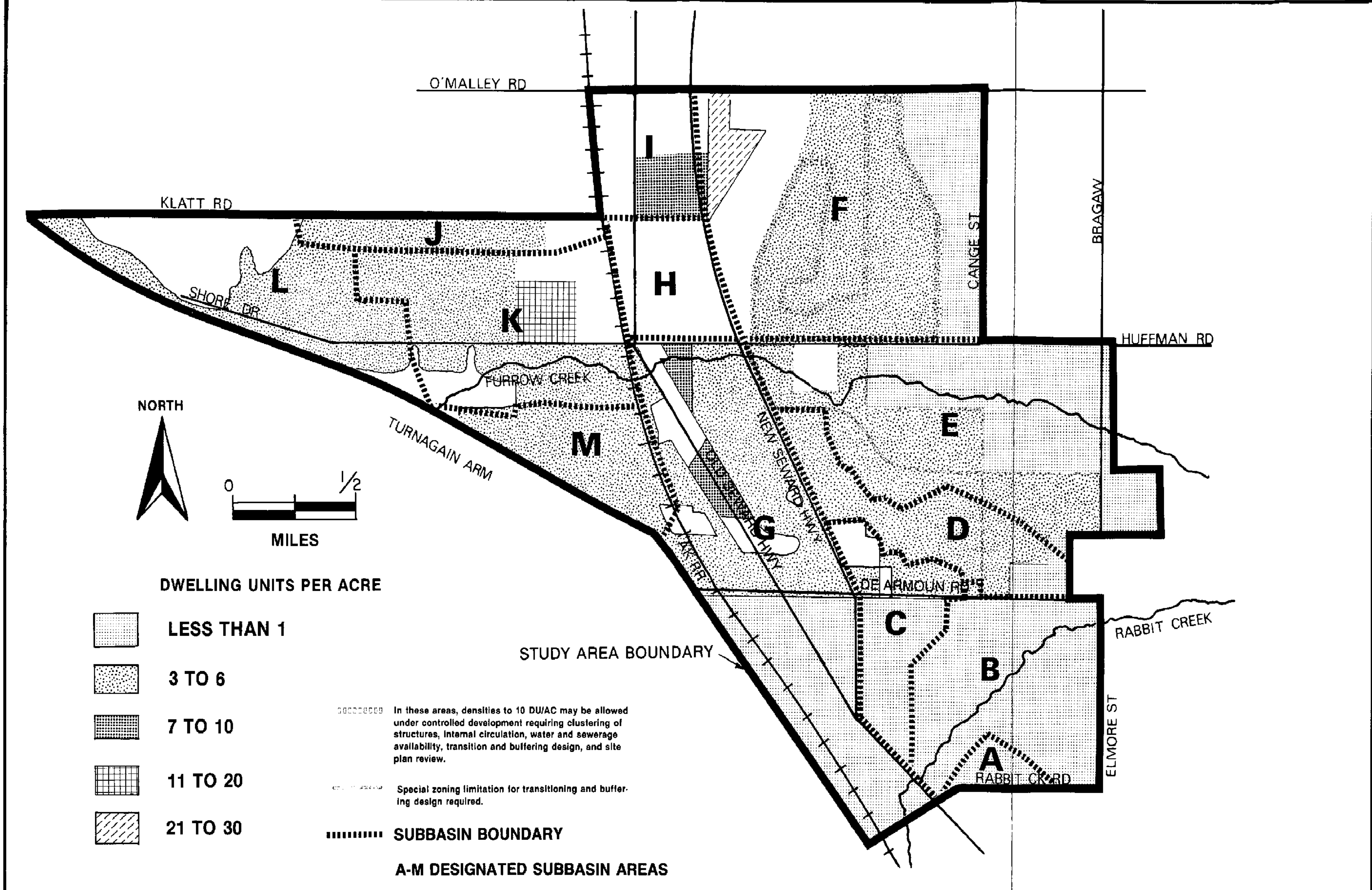
FUTURE LAND USE MAP

February 1983

Figure II-3



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FUTURE RESIDENTIAL INTENSITY MAP
February 1983 Figure II-4

Land use classifications for the computer analysis required modification from those identified in Figure II-3. The correlation between the land use classification for the computer and those of the Planning Department is presented in Table II-2.

TABLE II-2

LAND USE CLASSIFICATION CORRELATION

Computer Classification	Computer Code	Planning Department Land Use (Future)	Comments
Low density single family residential	LD	Less than 1 dwelling unit/acre	
High density single family residential	HD	3-6 dwelling units/ acre	1-2 dwelling units/ac are also included (Planning Dept. does not address this density.
Multi-family residential	MF	Greater than 6 dwelling units/ acre	It was assumed that a single family lot would not contain less than 7000 sq. ft. per lot.
Industrial	IN	Industrial plus 50% of Industrial/ Commercial	
Commercial	CO	Commercial plus 50% of Industrial/ Commercial	
Lowland Forest	LF	Public Lands/ Institutions and Parks	For existing land use, the New Seward Hwy was generally used as the division between upland and lowland forest with LF west of the New Seward Hwy.

TABLE II-2
(continued)

Computer Classification	Computer Code	Planning Department Land Use (Future)	Comments
Bogs and Marshes	BM	Marginal Lands	Tide flats were not included in the computer analysis.
Cleared Pervious	UP		Used solely for existing land use, indicating ground which has been cleared of vegetative cover and does not have man-made structure on it.
Upland Forest	UF		Used only for existing land use, generally identified as east of New Seward Hwy.
Gravel Pit	GP		Used only for existing land use to identify existing gravel pits.

Using the classifications given in the above table, the subbasins were assigned values for the area in computer classification. Summaries of the land use in each subbasin for both the existing and future cases are presented in Tables II-3 and II-4.

TABLE II-3

EXISTING LAND USE

Land Use Classification (acres)

Sub-basin	LD	HD	MF	IN	CO	LF	BM	UF	UP	GP	Total acres
A	27										27
B	306							45			351
C	139					8		21			168
D	27	131						40			198
E	280							287	25		592
F	350	21	50			71		191		84	767
G	279	101	130	4					12		526
H	62			20	56						138
I			37		49				45		131
J	7	25					39		10		81
K	71	65		12		111	23		103		385
L	46						265			3	144
M		134					14				148
	1594	477	217	36	105	190	341	584	195	87	3826

TABLE II-4

FUTURE LAND USE

Sub-basin	Land Use Classification (acres)										Total acres
	LD	HD	MF	IN	CO	LF	BM	UF	UP	GP	
A	27										27
B	346	5									351
C	97	34				37					168
D	34	151				13					198
E	291	301									592
F	127	387	45			208					767
G	103	265	75		58	25					526
H		2		50	86						138
I			37	45	49						131
J		66		3		12					81
K		256	41	30		58					385
L		240				10	64				314
M			141			7					148
	1025	1707	339	128	193	370	64				3826

EXISTING DRAINAGE PATTERNS

General

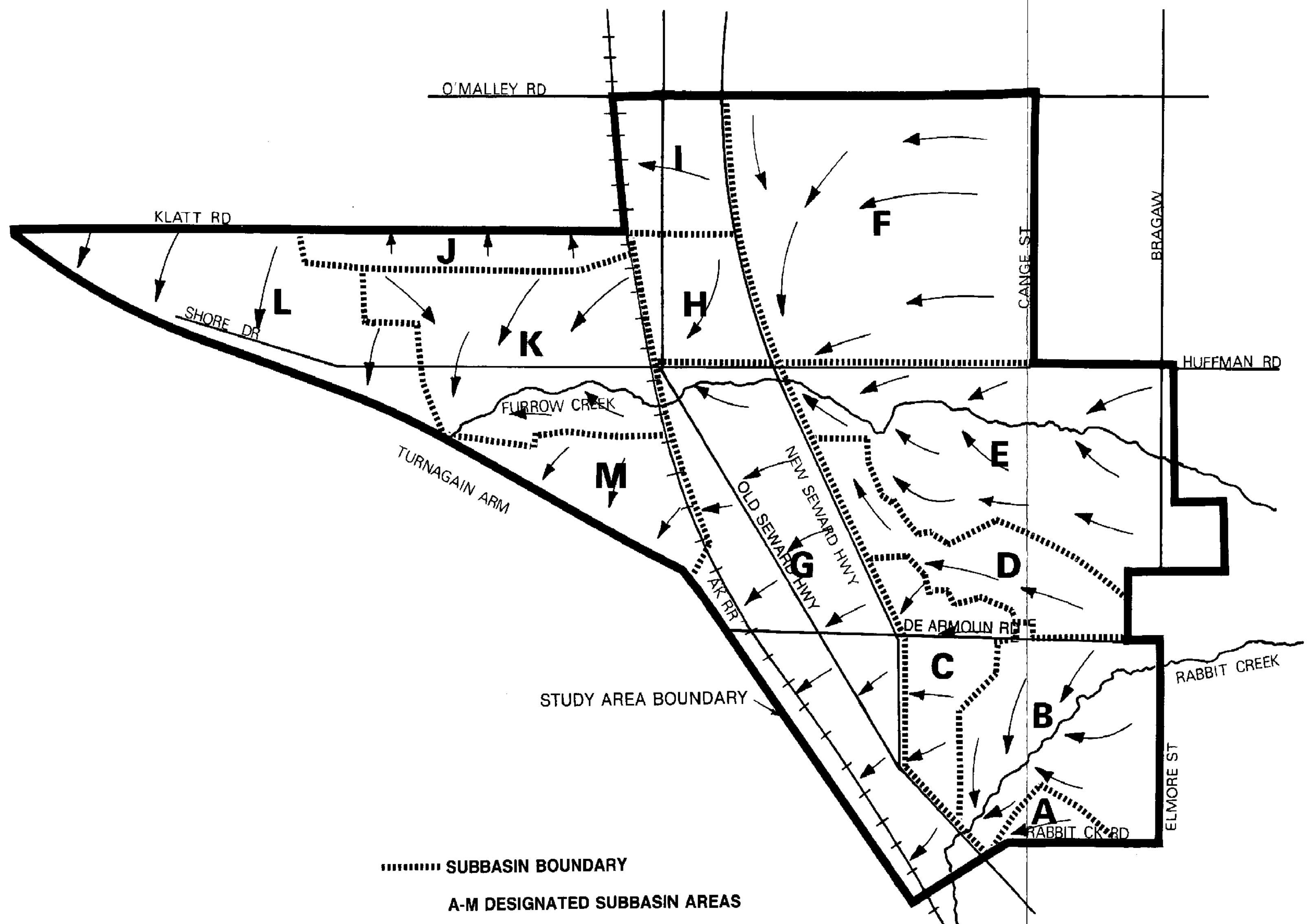
For simplicity in the following discussion of drainage patterns, the study area has been divided into three areas: east of the New Seward Highway, between the New Seward Highway and the Alaska Railroad, and west of the Alaska Railroad. These boundaries are the result of manmade structures (the highway and the railroad) which impede the flow of stormwater.

Figure II-5 depicts graphically the present general flow path of stormwater.

East of New Seward Highway

The portion of the study area lying east of the New Seward Highway can be divided into three parts based on drainage patterns. The approximate respective north/south limits of these three drainage areas are O'Malley and Huffman, Huffman and DeArmoun, and DeArmoun and Rabbit Creek Road.

Between Huffman Road and O'Malley Road stormwater drains via overland flow and roads west and south to the northeast corner of the intersection of Huffman and the New Seward Highway. At present, water is detained at this location. However, the intersection of Huffman Road and the New Seward Highway is being upgraded. A culvert is being extended to the northeast



URS Engineers
Anchorage, Alaska

FLOW PATTERNS/
SUBBASIN
DELINEATION
February 1983

Figure II-5

corner of the intersection. It is anticipated that during the present construction the currently detained water at this location will be allowed to cross the New Seward Highway and continue its flow downstream.

Stormwater flow between the approximate limits of Huffman and DeArmoun travels via road ditches and culverts and overland flow. In Turnagain View subdivision, a storm drainage system has been included in the construction of the subdivision. The travel path of Furrow Creek is not well defined east of the New Seward Highway. Flow from this area arrives at the southeast corner of the intersection of Huffman and New Seward Highway, and presently, does not traverse the New Seward Highway; rather, it is detained between the Frontage Road and the highway. With the completion of the new Huffman/New Seward Highway interchange flow will cross beneath the New Seward Highway and proceed west.

South of DeArmoun stormwater flow is west and south to Rabbit Creek via overland flow in most cases. Rabbit Creek crosses the Old Seward in a set of twin 72-inch culverts.

New Seward Highway to Alaska Railroad

In the area lying between the New Seward Highway and the Alaska Railroad, the natural flow path is interrupted by manmade structures (roads, culverts, and ditches). This area is approximately 30% developed north of Huffman and about 90% south of Huffman.

Between Klatt Road and O'Malley Road the flow is to the west to the railroad via roads, and ditch/culvert systems. At the railroad flow diverts to the north crossing the railroad tracks outside of the study area north of O'Malley Road.

For the area between Klatt Road and Huffman Road flow is west and south, traversing the Old Seward Highway and the Alaska Railroad approximately at Huffman. Flow is overland as well as via roads. Approximately 50% of the land area has been developed.

Downstream of Huffman the flow path varies. Between Kruge and Huffman, the flow is generally west and north, crossing the Old Seward Highway and the Alaska Railroad near Huffman. The flow from this area drains to Furrow Creek. From Karen to Kruge and east of the Old Seward Highway flow is west. Only in the vicinities of Kruge and Huffman do culverts exist for allowing an east to west flow path traversing the Old Seward Highway.

Along the strip of area between the Old Seward Highway and the Alaska Railroad, flow is to the west, ponding at the foundations of the railroad tracks. Between George Bell Circle and the intersection of the Old and New Seward Highways, a seepage drainage system has been constructed along the railroad tracks, allowing drainage of the ponded areas on the upstream side of the tracks. Drainage from this seepage system is via outfalls into Turnagain Arm.

West of the Alaska Railroad

North of Langnes Court and north of the portion of Klatt Road west of Toy Street stormwater drainage is to the north, exiting from the study area at Klatt Road. This land area is presently about 20% developed, with the undeveloped land being boggy.

Bounded by Timberlane Road and Alaska Railroad to the west and east, respectively, the Langnes Court and Galleon/Lighthouse Streets to the north and south, respectively, this area constitutes the downstream tributary area of Furrow Creek. Development of the land area is approximately 50%, with Furrow Creek being routed via culverts and greenbelts through the Oceanview subdivision. Furrow Creek exists at the southwest corner of this area to Turnagain Arm at Johns Park.

South of Galleon/Lighthouse Streets and bounded by Timberlane Road and the Alaska Railroad to the west and east, respectively, the land area is 100% developed. This area drains via roads and underground drainage system to Turnagain Arm via outfalls.

West of Timberlane Road the area is very boggy. Approximately one-third of this area has been designated as part of the Klatt Bog, and has the wetlands classification of "conservation-development with special considerations" which will limit its development. Flow is basically overland with two large drainage cuts existing to provide channels for discharge to Turnagain Arm.

CHAPTER THREE
PLANNING CRITERIA

CHAPTER 3

PLANNING CRITERIA

INTRODUCTION

This chapter contains topics which can be classified into two categories of criteria: conceptual and quantitative. The conceptual topics, planning goals and criteria for proper street drainage are presented first, followed by the quantitative topics of design storms and water quality criteria.

The section on planning goals presents the fundamentals which formed the framework for decision making in the course of this study. Street drainage criteria presented herein was used in the critique of the existing system. Both sections, goals and street drainage provide criteria which will be useful in the design and construction phases of future drainage facilities within the study area.

In order to proceed with the quantitative analyses in this study, it was necessary to establish the design storms used for the hydrologic investigation and the water quality parameters used as a basis for the evaluation of water quality. This information was used as input into the operation of the SAM and STORM computer models.

GOALS

Overview

The following paragraphs summarize the goals to be achieved through this planning process. This summary is a brief overview of the specific goals as outlined in the following paragraphs. Some of the more technical terms used in this section are discussed and defined in the appendicies of the report. The study area as outlined in the report has been divided into thirteen subbasins labeled A through M. These subbasins are grouped in four general categories: 1) Subbasins A, B & C which drain to Rabbit Creek. 2) Subbasins I & J which drain out of the study area into the south Anchroage storm drainage study area. 3) Subbasins D, E, F, G (northern portion), H, and K which comprise the drainage network of Furrow Creek. 4) Subbasin G (southern portion), L, and M which drain directly to Turnagain Arm. Depending on the natural topography, existing and future land use patterns, present manmade structures of each of the various subbasins and their drainage patterns, the goals for each of the subbasins will vary as to individual requirements and needs of each subbasin. It is the overall objective of this study to identify various alternatives for storm drainage control, both quantity and quality, and to recommend the best suited alternative for each particular subbasin/subcatchment to the goals and requirements of the Municipality of Anchorage Department of Public Works.

Of main concern is the identification of goals as they relate to the subbasins which comprise Furrow Creek. Furrow Creek has three different and distinct drainage patterns and land use patterns. These three distinct areas are: The Upper Furrow Creek segment, comprised of Subbasins D, E, and F, which is primarily the Furrow Creek subbasin east of the New Seward Highway; The Middle Furrow Creek segment, comprised of subbasins G and H, which is the area along Huffman Road between portions of the New Seward Highway and the Alaska Railroad tracks; and The Lower Furrow Creek segment, comprised of subbasin K, which is the area west of the Alaska Railroad through Johns Park and the outlet into Turnagain Arm.

A similarity exists between the various segments of Furrow Creek and the other subbasins within the study area which are not a part of the Furrow Creek drainage. Subbasins A and B, which drain to Rabbit Creek, are very similar to the Upper Furrow Creek segment; Subbasins I and J, which drain north, exiting from the study area and entering the South Anchorage Drainage Study area, are very similar to the middle segment of Furrow Creek; Subbasins L and M, which drain into Turnagain Arm, and the southerly portions of subbasin G, which also drains to Turnagain Arm, are very similar in nature to that of Lower Furrow Creek. In the identification of goals for this study area, it was concluded that goals set for Furrow Creek could be common to the overall study area because of the various distinctions within Furrow Creek itself. Therefore, goals defined below for Furrow Creek are also applicable to other portions of the study area similar to the respective segments of Furrow Creek.

Area-Wide Goals

Furrow Creek is to be preserved and enhanced as a valuable natural resource serving as arunoff corridor for stormwater. This creek serves as a natural drainage channel and is valuable open/greenbelt in an increasingly urbanized community. If maintained properly the creek increases the recreational and aesthetic values of the surrounding community. It also is a habitat providing food and shelter for numerous birds and small animals, and a few occasional moose.

Specific Goals

1. THE FURROW CREEK CORRIDOR SHOULD BE PROTECTED TO THE GREATEST EXTENT POSSIBLE.

The creek corridor is a strip of land of variable width on either side of Furrow Creek including the channel itself. It contains those sensitive areas which if altered could seriously degrade the stream and/or cause nuisance or monetary damage to surrounding businesses and homes.

Furrow Creek is recognized as a valuable natural resource that performs drainage and aesthetic functions, and provides a habitat for a variety of wildlife. New developments adjacent to the creek should leave a buffer strip on both sides of the channel adequate to preserve the drainage, habitat and aesthetic function of the creek. This buffer strip should be wide enough to include the following elements when they are directly associated with the creek channel and if their disruption would degrade the creek: vegetation along the channel, wetlands, slopes over 15%, or highly erosive soils. If the creek corridor must be disturbed, special precautions must be taken to preserve or replant vegetation adjacent to the stream, prevent erosion and the transport of sediment into the creek channel, maintain water quality, maintain bank stability and free-flowing open channel.

When new development or re-development occurs adjacent to the segments of the creek that have been previously degraded by land clearing and these modifications have created water quality or quantity problems at the impacted area or elsewhere in the system, creek channel and/or bank rehabilitation should be a condition of the development permit where feasible.

2. MAINTAIN THE NATURAL FUNCTIONS OF ALL ELEMENTS OF FURROW CREEK AND OTHER NATURAL DRAINAGE SYSTEMS WHERE POSSIBLE.

Natural stream channels convey and store water as well as permit infiltration of surface water to groundwater reservoirs. These channels slow the rate of flow of stormwater and delay flood peaks because of the high resistance of flow to rocky, grassy channels. All open channels should remain in use except for where road crossings are required to develop property. Such road crossings should be accomplished with a bridge or culvert of adequate width and depth to permit free-flowing conditions during the spring runoff when portions of the culvert may be plugged due to icing.

Wetlands, ponds, and lakes store water and purify runoff water through the settling and biological action as well as provide groundwater recharge and wildlife habitat. Various wetlands in the basin as identified in the Wetlands Management Plan provide significant storage capacity for stormwater. The storage and recharge capacity of these wetlands should not be reduced through filling as the result of development. This policy is in accordance with the classification of "conservation" assigned in the Wetlands Management Plan.

Presently, large portions of the study area are undeveloped. Existing topography is such that many small depression areas exist. These depressions become collection points for storm-water runoff and, as a result, decrease peak runoff quantities. The Municipality should encourage local developers to detain peak volumes of runoff throught the use of local detention methods to achieve runoff rates similar to natural conditions.

Aquifer recharge areas are areas with porous soils where the underlying geology absorb, store and purify vast amounts of precipitation as ground water. The stored water is later released and fed into the creek from springs during dry weather when runoff no longer contributes to stream flow. Development in the portions of the basin which act as recharge areas, should minimize impervious surfaces. Recognition of recharge basins as required for on-site detention facilities through the use of classified wetlands should be encouraged.

3. CONTROL MAXIMUM STREAM FLOWS WHILE MAINTAINING MINIMUM FLOWS DURING DRY WEATHER.

Stream flow quantites and velocities in response to storms are to be moderated through the use of on-site controls and coordinated with in-stream measures.

Stream controls to be provided will minimize flooding and private and public property damage, and will limit stream bank and bed erosion to non-destructive levels.

Groundwater recharge is to be used where feasible to limit runoff and assure a source of flow during dry weather.

4. MAXIMIZE THE USE OF EXISTING RUNOFF CAPACITIES OF THE EXISTING STORM DRAINAGE NETWORK WHILE USING A COMPREHENSIVE TRUNK SYSTEM TO CARRY EXCESSIVE STORMWATER FLOWS.

Capacities of existing channels and/or pipes should be identified. If necessary, these systems should be augmented with a comprehensive trunk system to carry peak flood flows.

A comprehensive planned development of existing subbasins in conjunction with the proposed ultimate land use of the area should be required. Development should minimize the effect on surrounding land owners both from a land use constraint and associated flooding problems.

The layout of the trunk stormwater drainage paths should be along naturally low spots in topography or in conjunction with road improvements. The existing culverts, roadside ditches and pipes should be utilized as the local collection system to convey drainage from all the land within the subcatchment to the trunk system. If these alignments prove to be inconsistent with future development plans for the areas indicated, the alignment of the trunk system may be adjusted in the future to reflect a properly designed and constructed alternative.

5. MAINTENANCE OF WATER QUALITY OBJECTIVES AS DEFINED IN THE MUNICIPALITY OF ANCHORAGE 208 AREA-WIDE WATER QUALITY MANAGEMENT PLAN.

The Anchorage 208 Water Quality Management Plan identifies the creeks and lakes within the Anchorage area as valuable recreational resources. These recreational resources can be impaired or even eliminated by poor water quality. The major causes of water quality degradation in the Anchorage Bowl are from: urban runoff, erosion which is primarily resulting from construction activities, runoff and percolation from snow disposal sites.

In the evaluation of alternative solutions to drainage problems, the ability of the various alternatives to meet the 208 Water Quality Management Plan goals shall be included.

Wherever possible, non-structural methods shall be used to control water quality problems that have been identified, these include: land use controls, urban cleanliness, soil erosion and sediment control plans, and comprehensive snow disposal programs.

STREET DRAINAGE

Both State and Municipal roads exist within the study area. Therefore, design storm frequency criteria used by the State of Alaska Department of Transportation and Public Facilities (DOT/PF) and the Municipality of Anchorage Department of Public Works were researched. The Alaska DOT/PF uses the following design storm frequencies:

Type of structures	Design storm frequency (years)
Culverts or primary highways	50
Culverts on secondary highways	10
Storm sewers	10
Roadway ditches, stormwater inlets, gutter flow	10

The Municipal Department of Public Works uses a twelve-year design storm frequency (in conjunction with the ILLUDAS computer model) for subdivision work. For the large area involved in this study, the Municipality has chosen to use 10- and 100-year storm frequencies. The flows calculated using the 10-year storm frequency are to be identified throughout the study area.
The flows generated using the 100-year storm frequency are to be identified at major road crossings and other important locations.

These storm frequencies coordinate well with the State DOT/PF storm frequencies detailed above.

Once the design storm has been selected for particular type of street, it becomes necessary to identify the amount of ponding which is allowable without causing danger to public safety or damage to surrounding property owners due to flooding conditions. Presently, neither the Municipality of Anchorage Public Works Department nor the Alaska Department of Transportation and Public Facilities have definitive criteria with respect to the amount of allowable stormwater accumulation during times of initial stormwater runoff. The location and size of inlets is based upon the allowable stormwater spread or depth of flow in the streets.

To properly identify the location of inlets for stormwater drainage in streets, the following is a synopsis of the criteria presently used by the Department of Transportation and Public Facilities:

- at low spots and/or changes of grade
- at all other points where necessary
- ahead (upstream) of street intersections
- at both sides of a street where water would flow towards the intersection
- ponding of stormwater on curbed streets shall be limited to 1/2 of the outer lane

As a guide, it is suggested that the design engineer selecting the location, size and capacity of gutter inlets, use an allowable depth of ponding between 6 to 8 inches. This has been identified as the most frequently used criteria by the American Public Works

Association (APWA) via a survey conducted in 1980 with respect to the stormwater management practice in the U.S. and Canada (Urban Stormwater Management Special Report No. 49, 1981).

DESIGN STORM

The design storms used in this study are not identical for the quantity and quality analyses. Therefore, in the following paragraphs the design storms are discussed for the separate cases of quantity and quality.

Quantity

Urban drainage facilities are designed to be capable of handling a storm runoff event of a certain recurrence frequency. Normally, for lack of long-term flow measurements, urban runoff events cannot be statistically defined. Therefore, in engineering calculations of runoff, it is assumed that the frequency of occurrence of a rainfall event is identical to the frequency of occurrence of the resulting runoff. It is important to know that this assumption is not entirely correct, since a given storm may produce runoffs of various magnitudes and frequencies depending on the antecedent characteristics of the catchment.

Ideally, the selection of the proper design frequency for drainage facilities is a compromise between periodic inconveniences, and damages due to flooding and the cost of preventing this flooding. However, as the drainage system in this project area is not very sophisticated, it does not warrant a detailed analysis of the relationship between the cost of flood protection and flood damage. Consequently, design periods as specified by the Alaska Department of Transportation and Public Facilities as well as by the trend set by other drainage basin planning studies are used.

The design event chosen for this project is a 10-year storm (i.e., a rainfall event of 10 years recurrence frequency) in conjunction with a snowmelt event. The subsurface condition during the design event is therefore considered to be frozen.

For information purposes, the 100-year storm event was also simulated. This data is presented in this report as information; it was not used herein for estimating capacities of facilities.

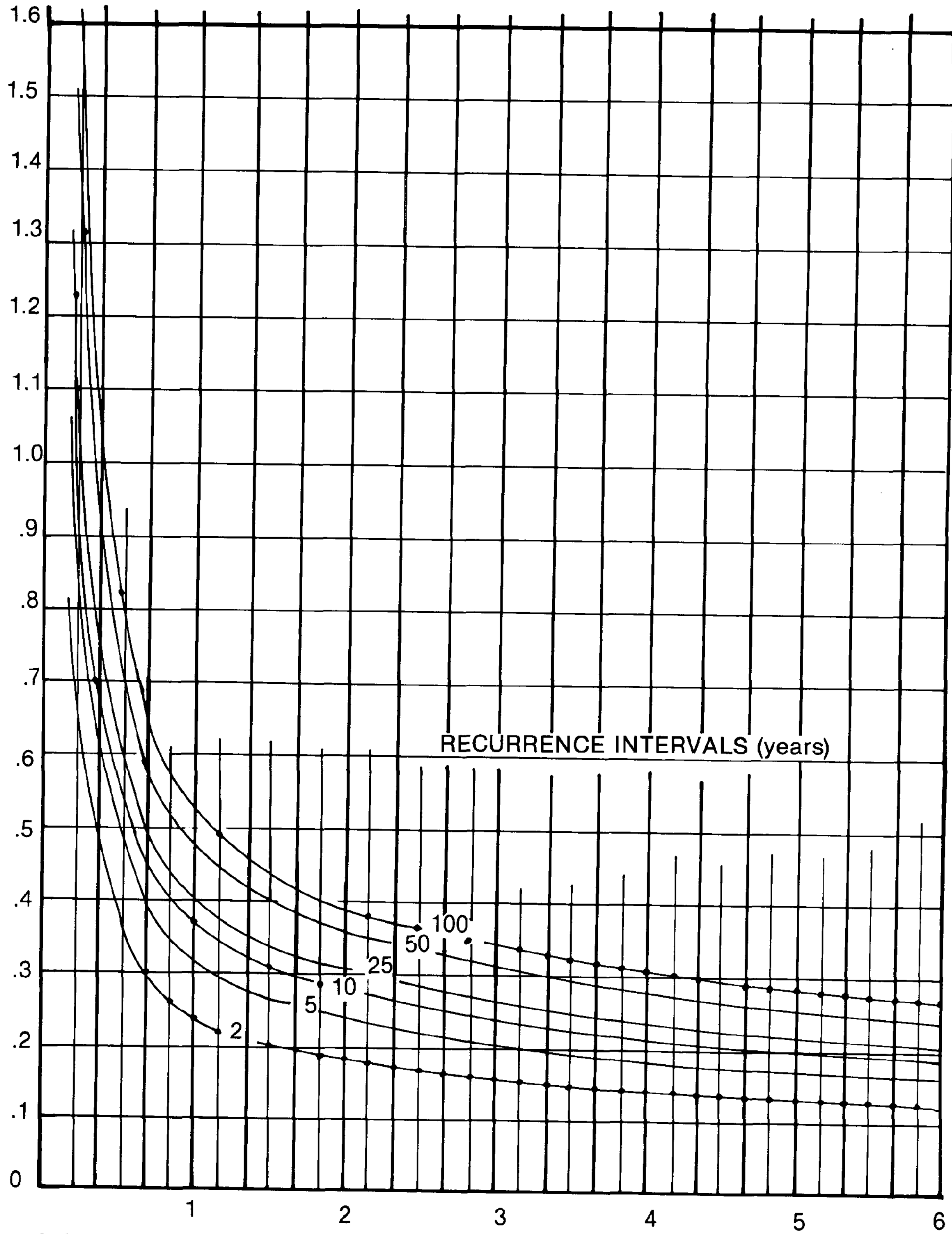
It is desirable that the design storm event is to be derived from historical precipitation records. The nearest weather station which has long-term precipitation records is located at the Anchorage International Airport, about six miles northwest of the project basin. Precipitation date from this station were used to derive the design storm event.

The International Airport station precipitation data were used as it is long-term. However, records from the Little Rabbit Creek weather station, located about 10 miles southeast of the study area at an elevation of 380 feet, show higher precipitation throughout the entire year than at the International Airport station. The cause of this variance in precipitation could be due to orographic effects of the Chugach Mountains. As a result of the difference in precipitation data between the Little Rabbit Creek and International Airport stations, the precipitation data to be used for the study area possibly could be higher than the International Airport data. However, resulting from the quantity of data available from the Little Rabbit Creek station, it was not possible to accurately interpolate between the data for the two stations. It is recommended that precipitation gauges be installed in the study area to obtain site-specific data.

In performing the runoff calculation, the SAM model requires the design storm be represented for input into the model by a storm hyetograph in which rainfall intensity varies with time as observed in nature. This design storm hyetograph was synthesized by distributing with respect to time the total volume of the design storm event to the entire storm duration. Profiles of the observed precipitation events as well as the intensity-duration relationships given by the intensity-duration-frequency (IDF) curves were used as references for establishing the design storm hyetograph.

The IDF curves for the International Airport have been developed as part of the South Anchorage Drainage Study (Ref: CH_2 M memo to Lee Browning, February 19, 1982). The curves, Figure III-1 were derived from 1953-1980 precipitation records and also included contributions from snowmelt during precipitation events in early spring.

Six-hour duration of storm was chosen which is considered



DATA BASE 1953—1980

REFERENCE: SOUTH ANCHORAGE DRAINAGE STUDY

URS Engineers
Anchorage, Alaska

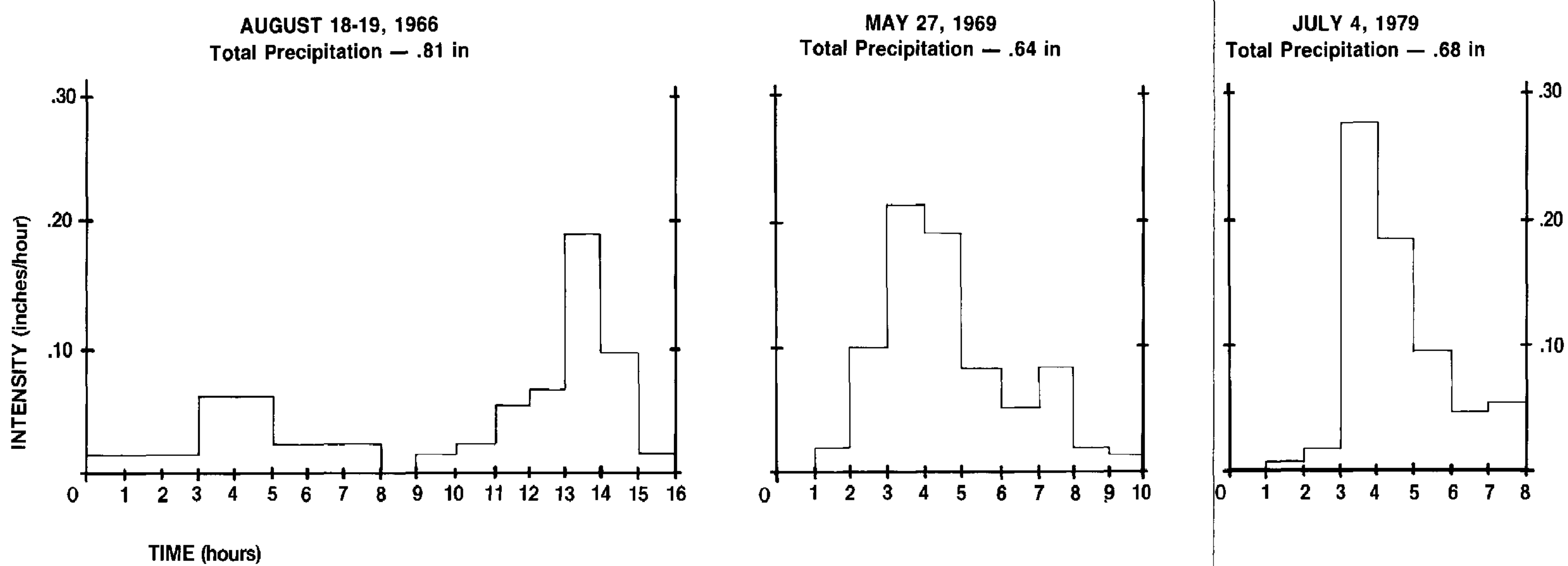
RAINFALL INTENSITY/ DURATION /FREQUENCY CURVES
for the Anchorage International Airport

Figure III-1

to be sufficiently long to ensure that the entire drainage basin contributes to the flow in the drainage system. Using a six-hour duration, 10-year storm, the IDF curves give a constant intensity of 0.18 inches per hour. The total volume of the design store is, therefore, 0.18 x 6 = 1.08 inches. Figure II-2 shows the magnitude and the time distribution of several historical large storm events. Figure III-3 shows the hourly distribution of the 10-year design storm and Figure III-4 shows the 10-year and 100-year design storm hyetograph as they are distributed in a 10-minute interval for a total of six hour duration.

QUALITY

The section on design storms for the quantity analysis dealt exclusively with runoff quantities. When water quality aspects are to be considered, a special analysis of the precipitation data may be required. The frequency of occurrence of pollutant loads of a given magnitude differs significantly from the frequency of occurrence of the corresponding storm. The reason for this is that pollutant load produced by an event depends not only on the event itself, but also on the length of the antecedent dry weather period. Consequently, the design storm approach is rarely used in quality-oriented drainage design. Instead, a continuous simulation of runoff quality and the associated costs of quality control are more frequently used and these provide a good basis for selecting a cost-effective means of runoff quality control.



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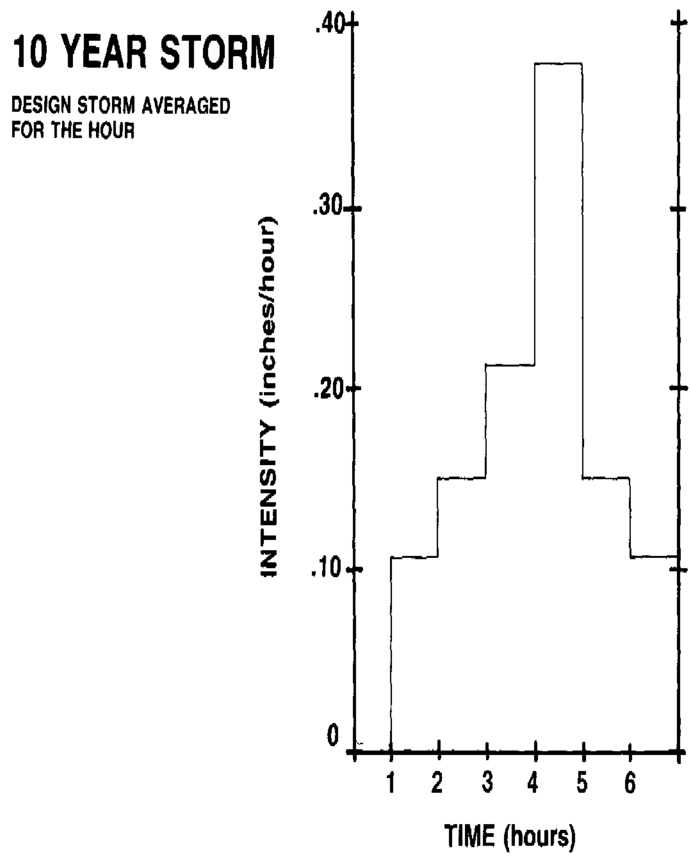
Anchorage, Alaska

HISTORICAL LARGE EVENT STORM

February 1983 Figure **III-2**



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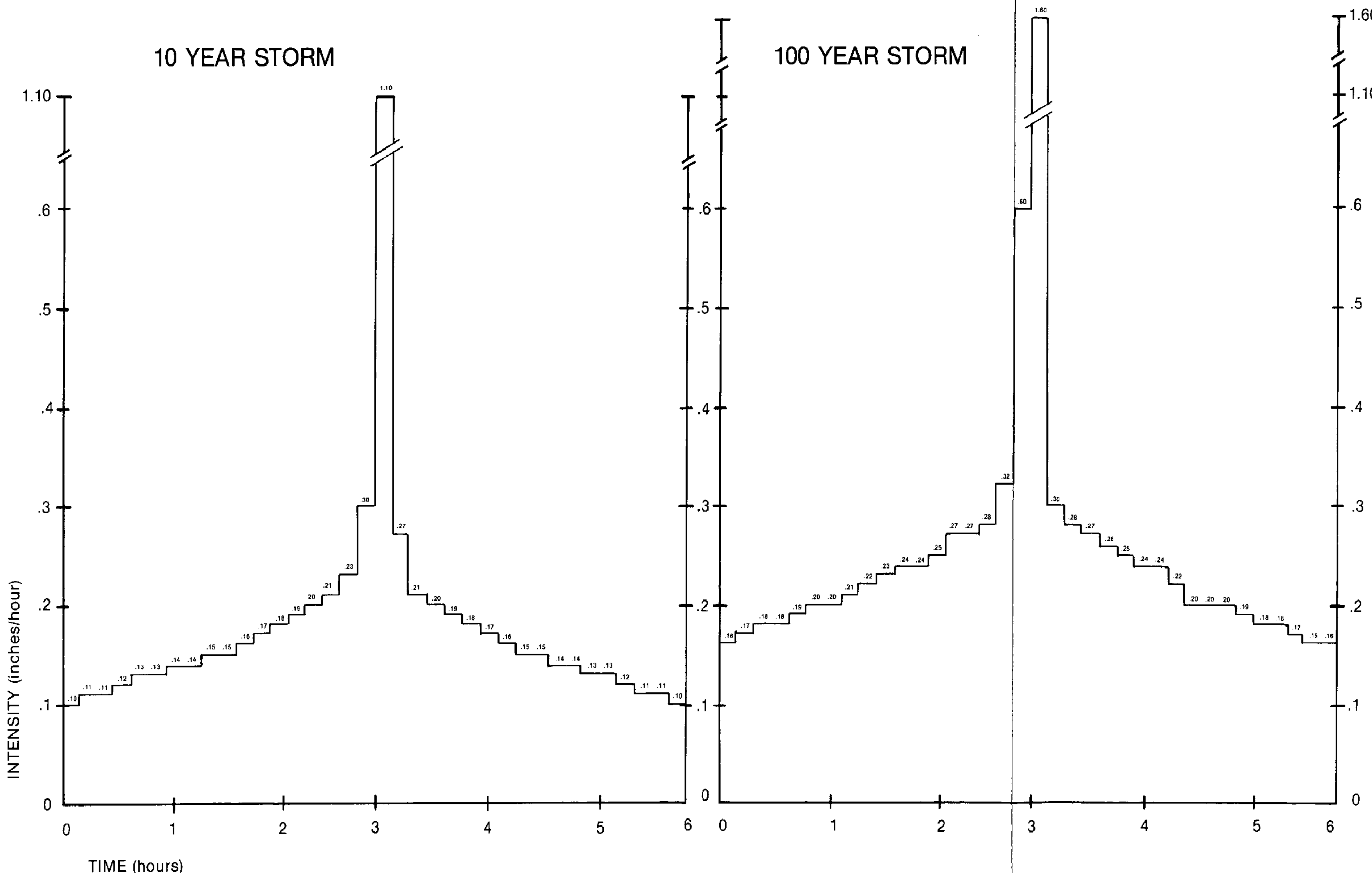
DESIGN STORM

February 1983 Figure III-3



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DESIGN STORMS

February 1983 Figure **III-4**

The same snowmelt event selected for the quantity analysis was used to represent the spring break-up event for the quality analysis. This event assumes a roughly five months (150 days) of pollutant loading before wash-off.

For the project area, two periods of time are critical to water quality. One is at the time of the spring break-up and the other is the period from spring break-up to winter freeze-up. Analysis of water quality was performed for each of the two critical periods.

Three five-month historical events (May to Sept.), representing three hydrological conditions - wet, average, and dry, were selected from the weather records to represent a variety of storm and pollutant wash-off events. These historical events are graphically presented in Figure IV-1. Hourly precipitation data from these events were inputed to STORM Model for continuous simulation of runoffs and resulting wash-off loads. The resulting statistics of pollutant loads and concentrations of the water quality constituents were used to determine the return periods and corresponding control measures.

WATER QUALITY

208 Water Quality Management Plan

The Anchorage 208 Water Quality Management Plan, mandated by Public Law 92-500 (the Clean Water Act) in conjunction with the State of Alaska's Water Quality Standards, sets forth the standards for water quality in the State of Alaska. It further identified the management strategies needed to achieve or protect the water quality uses in the study area. The current Alaska Water Quality Standards considers seven beneficial uses of surface waters. Of these seven, only Class C (water contact recreation) and Class D (growth and propagation of fish and other aquatic life) are applicable to the Furrow Creek-Rabbit Creek study area. It should be noted however, that the current water quality standards of the State of Alaska are undergoing review and revisions by the State. As a result of these revisions, a new list of freshwater uses will be developed for the Anchorage area, although the Municipality of Anchorage has not requested revisions to either Furrow Creek or Rabbit Creek water quality standards at this time.

The Anchorage 208 Management Plan identifies three levels of control strategies which can be used to meet water quality standards. As taken from the 208 Management Plan the three levels of control strategies are:

- Level 1 control strategy is the use of the existing programs and control measures. Through the use of existing comprehensive plans, zoning ordinances, subdivision regulations, and the review of various permit applications, the detrimental effect to water quality can be minimized. Level 1 controls will not meet the legal requirements established by the State of Alaska for water quality standards and will not protect beneficial uses.

- Level 2 control strategy is a set of control measures based upon existing municipal practices designed to reduce the four problem areas as identified in the 208 Plan: non-point pollution from urban runoff; erosion from construction sites; runoff from snow disposal site percolation, and failure of on-site wastewater disposal systems. Level 2 control measures will result in water quality levels sufficient for all existing uses but will not satisfy all requirements of the State water quality standards.

- Level 3 control strategy is a program which implements a system of interceptor storm sewers parallel to creeks and drainage swales and provides for the diversion of all stormwater runoff into Cook Inlet thereby decreasing the pollutant load in local receiving waters and meeting the State water quality standards.

The 208 Management Plan recommends that, whenever possible, the Level 2 control strategy be used to meet water quality standards for the beneficial use of the surface water in the area. In reviewing the options available and the planning criteria designated to meet water quality standards, the following items outlined in the 208 Management Plan and updated to meet present needs have been identified as being most applicable for the Furrow Creek-Rabbit Creek study area.

- Existing design criteria for stormwater controls should be amended to include more emphasis on stormwater detention and water quality protection.

- Stormwater controls may include sedimentation-type detention ponds, infiltration ponds, drywells, and multi-use areas among others.

- The present stream corridor protection program should be continued with the following additions:

  - All inwater construction work should be discouraged. That which is unavoidable should be conducted between June 1 and July 15 to avoid conflict with spawning salmon (Rabbit Creek Only).

  - Any planned road crossing in the vicinity of salmon spawning areas should be accomplished by bridge wherever possible (Rabbit Creek Only).

  - Disturbed stream banks should be returned to a slope no greater than two horizontal to one vertical with replacement of natural vegetation.

  - Flood plain regulations should be amended so that new developments should utilize swales and ditches to the extent possible through provisions provided by subdivision regulations and design criteria and improvement standards.

- Control of erosion and sediments at construction sites is recommended.

- Control of runoff from snow disposal sites should continue as outlined in the January 1981 disposal site study report.

Alaska State Standards

The two major uses of the water within the study area are fish and wildlife habitat and for recreational purposes. For these two uses of freshwater in the State of Alaska, the State standards for water quality criteria are as shown in Table III-1. Relevent water quality constituents include: dissolved oxygen (DO), pH, turbidity, dissolved inorganic substance, and toxic substances. Although there is little water quality data available in the study area, the existing and future land use with the majority of which is low density residential, would suggest little probability for the presence of toxic substances in the receiving waters. The only water quality recording station in the study area is at the mouth of Rabbit Creek. The water quality data collected from this station is presented in Table III-2. Comparing the information in Table III-2 to the standards in Table III-1 shows that the water at the recording station is meeting the water quality criteria.

TABLE III-1

STATE OF ALASKA WATER QUALITY STANDARDS

Parameter	Fresh Water Contact Recreation	Fresh Water Secondary Recreation	Fresh Water Growth and Propagation of Fish, Shellfish, Other Aquatic Life and Wildlife including Waterfowl and Furbearers
	1(B) (i)	1(B) (ii)	1(C)
Fecal Coliform Bacteria			
Mean Value (1)	20 FC/100 ml	200 FC/100 ml	200 FC/100 ml
90 Percentile Value (1)	40 FC/100 ml	400 FC/100 ml	400 FC/100 ml
Dissolved Oxygen (DO)			
Average Minimum (1)	4 mg/1	4 mg/1	4 mg/1
10 Percentile Minimum (1)	-	-	3 mg/1
Minimum	-	-	-
			For resident fish spawning waters, minimum DO 7 mg/1. Interstitial waters of the gravel bed minimum DO 5 mg/1.
pH	6.5 to 8.5	5.0 to 9.0	6.5 to 9.0
			Shall not vary more than 0.1 pH units from natural conditions.
Turbidity			
Maximum Increase When natural turbidity is less than 50 NTU	5 NTU	10 NTU	Maximum Secchi disk Depth Reduction 10%.
When natural turbidity is more than 50 NTU	10% or max. 25 NTU increase	20% or max. 50 NTU increase	Maximum turbidity increase. 25 BTU above natural conditions.

(1) Based on a minimum of five samples taken over a period of 30 days.

TABLE III-1
(Continued)

Parameter	Fresh Water Contact Recreation	Fresh Water Secondary Recreation	Fresh Water Growth and Propagation of Fish, Shellfish, Other Aquatic Life and Wildlife including Waterfowl and Furbearers.
	1(B) (i)	1(B) (ii)	1(C)
Temperature Maximum	30°C (86°F)	Not applicable	Natural +2°C (3.6°F) Max Rate 0.5°C (0.9°F)/hr
Dissolved Inorganic Substances	Not applicable	Not applicable	TDS Max = Natural conditions +33% or 1500 mg/1 whichever is lower
Sediment	No measurable increase	No measurable increase	Sediment 4.0 mm max. increase 5% Max = 30%
Toxic, deleterious Substances, including pesticides, related organic or inorganic material	Alaska Drinking Water Standards or EPA's <u>Criteria For Water</u>	Concentrations shall not pose hazards to immediate contact	0.1 96 hr LC50 value lowest measured value for most sensitive biologically important species. Not to exceed concentrations which impart undesirable taste and order. Dissolved gas limit 110% saturation.
Color	15 units - true color	Free of substance producing objectionable color	In combination with durbidity limit reduction of compensation point for photosynthetic activity depth by 10% and Secchi disk depth by 10% 0.01 96 hr LC50 value
Petroleum Hydrocarbons animal fats and vegetable oils	No visible sheen	No visible sheen	Prefer continuous flow test - static test acceptable. No deleterious chronic effects. Virtually free from floating oils.

TABLE III-1
(Continued)

Parameter	Fresh Water Contact Recreation	Fresh Water Secondary Recreation	Fresh Water Growth and Propagation of Fish, Shellfish, Other Aquatic Life and Wildlife including Waterfowl and Furbearers.
	1(B) (i)	1(B) (ii)	1(C)
Radioactivity	Shall not exceed limits in: • Alaska Drinking Water Standards • 10 CFR 20 Federal Regulations • National Bureau of Standards Handbook 69	Shall not exceed limits in: • Alaska Drinking Water Standards • 10 CFR 20 Federal Regulations • National Bureau of Standards Handbook 69	Shall not exceed limits in: • Alaska Drinking Water Standards • 10 CFR 20 Federal Regulations • National Bureau of Standards Handbook 69
Total Residual Chlorine	Not applicable	Not applicable	Salmonoid Fish 2.0 ug/1 10 ug/1.

TABLE III-2

SURFACE WATER ANALYSES
FOR
RABBIT CREEK AT OLD SEWARD HIGHWAY

Date Sampled	Stream Discharge (Ft3/s)	Temperature (°C)	Color Platinum-Cobalt Units	pH	Specific Conductance (micromhos at 25°)	Dissolved Solids	Total Hardness (as $CaCO_3$)
07-28-61			10	7.6	72	47	32
10-31-66		2.0		7.4	83	45	33
05-26-67							
09-10-71	24.8	6.0	0	7.6	83	53	38
04-06-72	4.8	.0	0	7.0	100	60	44
04-10-73	e 5.0	.5	7	7.4	103	68	48

	Calcium Ca	Magnesium Mg	Sodium Na	Potassium K	Bicarbonate HCO_3	Chloride Cl	Sulfate SO_4	Nitrate NO_3 as N	Fluoride F	Silica SiO_2	Cadmium Cd	Copper Cu	Iron Fe	Lead Pb	Manganese Mn
07-28-61	11	1.2	1.1	.2	31	1.0	8.0	.7	.0	8.4			50u		0u
10-31-66	10	1.9	.8	.0	30	.7	7.2	.9	.1	8.7			180u		0u
05-26-67	15	3.0	2.5	.3							16	56	40	0	15
09-10-71	12	2.0	1.6	.3	37	.5	9.2	1.3	.1	7.7			10d		10d
04-06-72	13	2.5	2.3	.1	41	.7	12	1.3	.1	8.5			50t		0t
04-10-73	15	2.5	2.1	.4	47	3.0	11	1.9	.1	8.3	0	4	30d	3	10d

SOURCE: USGS Open-file Report 75-105, Hydrology for Land-Use Planning: The Hillside Area, Anchorage, Alaska, Appendix A-5.
e: estimated

CHAPTER FOUR
EVALUATION OF EXISTING SYSTEM

CHAPTER 4

EVALUATION OF EXISTING SYSTEM

GENERAL

Information on the quantity and quality control of flow within the drainage system was gathered from resource documents, computer simulation, public input and field investigation. This information was compiled to obtain a clear understanding of the existing system and to make sound recommendations for the future system. This chapter presents the existing drainage system in terms of water quality and quantity.

QUANTITY

General

In the first half of this chapter the quantity of flow will be addressed by Subbasin. Land use as discussed in this chapter is as presented in Chapter 2. The capacities identified for the existing system are based on Manning's formula using "n" values of 0.04 for greenbelts, 0.03 for ditches, and 0.024 for CMP culverts. Flow values for both the existing and future system are based on the 10 year storm described in Appendix A, Computer Analysis.

It is possible to subdivide the study area into large areas which presently contain the same general type of drainage facility. The land lying north of Huffman Road consists generally of a ditch system. Between Huffman and DeArmoun Roads, the area is mainly curb and gutter west of Pintail Street, and a ditch system east of Pintail Street. South of DeArmoun Road ditches serve as the drainage system. It should be noted that one of the basic problems of the existing system is the lack of planned regional drainage facilities. The majority of the present systems are isolated and sized for local use and do not interlink to form a trunk system.

In the analysis of the existing system, a general trend was found regarding the culvert versus ditch size. Culverts were found to be of a significantly lower capacity than the adjoining ditch. At high flows, it is feasible that the ditch flow would surcharge the culvert and cause a culvert washout as well as local flooding. Recommendations for correction of this situation at key locations are presented in Chapter 5.

Another observation made in the field while gathering existing system information was the present state of culvert maintenance. Culverts were found to be up to one-third full of debris and rocks, especially on the upstream end of the culvert. If such lack of maintenance is to continue, the resulting reduction in pipe capacity must be taken into consideration during the final design of future facilities.

Throughout the quantity evaluation section of this chapter, reference is made to Table IV-1, "Capacity of Existing Systems", which is on the following pages.

TABLE IV-1

CAPACITY OF EXISTING SYSTEMS

Sub-BASIN	STREET	FROM	TO	SIZE (inches)	SLOPE (ft/ft)	APPROX. LENGTH. (ft)	ESTIMATED CAPACITY (cfs)	SYSTEM TYPE
A	Rabbit Ck Rd	Elmore	Old Sew. Hwy		0.065 *	340	70	Ditch
B	Old Sew. Hwy @ Rabbit Ck Rd	North	South	Twin 72	0.05 *	500	500 each 1100 Combined	CMP Culvert
C	Frontage Rd	E. 144th	Porcupine		0.033 *	1350	50	Ditch
	New Sew. Hwy	E. 144th	Porcupine		0.033 *	1350	450	Ditch
	Frontage Rd	Porcupine	Rabbit Ck		0.037 *	1700	30	Ditch
	E. New Swd. Hwy	Porcupine	Study Bdry.		0.058 *	1950	600	Ditch
	De Armoun	East of Staysail	New Sew. Hwy		0.032 *	2200	50	Ditch
	Westwind	Capstan	De Armoun	15	0.0052 and 0.012	550	3	CMP
	Frontage Rd	144th	De Armoun		0.002 *	1350	15	Ditch
	New Sew. Hwy	De Armoun	144th		0.004 *	1350	150	Ditch
	Frontage Rd	Tradewind	De Armoun		0.006 *	1900	20	Ditch
	New Sew. Hwy	Tradewind	De Armoun		0.032	1900	450	Ditch

* = (APPROX)

TABLE IV-1

CAPACITY OF EXISTING SYSTEMS (CONTINUED)

Sub-BASIN	STREET	FROM	TO	SIZE (inches)	SLOPE (ft/ft)	APPROX. LENGTH. (ft)	ESTIMATED CAPACITY (cfs)	SYSTEM TYPE
D	South of Seawind	Pintail	Spinnaker		0.013 *	800	50	Greenbelt
	Spinnaker	South	North	18	0.004	60	4	CMP
	South of Seawind	Spinnaker	Westwind		0.017 *	1550	60	Greenbelt
	Westwind	East	West	18	0.006	60	4	CMP
	Westwind	Spinnaker	Greenbelt	18 *	0.004	500	4	Subdrain CMP
	West of Westwind	Westwind	Tradewind		0.026	700	950	Greenbelt
	Tradewind	Greenbelt	Frontage Rd	24	0.006 and 0.012	1100	10	CMP
	Frontage Rd	Tradewind	North	24	0.006 *	220	10	CMP
	Frontage Rd	CMP	Steeple		0.008	1800	30	Ditch
	New Sew. Hwy	Tradewind	Steeple		0.014	1800	300	Ditch
	Legacy	Vern	Frontage Rd	10,15	0.05	160	4	CMP
	Hamilton Park Sub.	Sue's Way	Frontage Rd	15 *	0.02 *	850	4	CMP
E	Legacy	Biscayne	Lake Otis	18	0.013 *	750	10	CMP
	Lake Otis	Westwind	Legacy	18	0.023	150	10	CMP
	Lake Otis	Legacy	Nancy	18,21	0.003 and 0.007	550	5	CMP
	North of Sue's Way	Nancy	Steeple	21	0.015	550	10	CMP
	Huffman Cir.	Steeple	Back of Cir.	21,24	0.007 and 0.002	450	20	CMP
	Frontage Rd	Steeple	North of Steeple		0.017	1000	40	Ditch
	Frontage Rd	North of Steeple	South of Huffman		0.012	600	30	Ditch
	Frontage Rd	South of Huffman	Huffman		0.011	600	30	Ditch
	New Sew. Hwy	South of Huffman	Steeple		0.013	1700	300	
	New Sew. Hwy	South of Huffman	Huffman		0.019	700	350	

* = (APPROX)

TABLE IV-1

CAPACITY OF EXISTING SYSTEMS (CONTINUED)

Sub-BASIN	STREET	FROM	TO	SIZE (inches)	SLOPE (ft/ft)	APPROX. LENGTH. (ft)	ESTIMATED CAPACITY (cfs)	SYSTEM TYPE
F	Huffman	Bragaw	Gander		0.037 *	4200	60	Ditch
	Huffman	Gander	Lake Otis Pkwy		0.023 *	1300	40	Ditch
	Huffman	Lake Otis Pkwy	Gregory		0.032 *	2000	50	Ditch
	Huffman	Gregory	E. New Sew. Hwy	30	0.003	500	10	CMP
	Huffman	Northeast	E. New Sew. Hwy	8	0.02	150	2	Perf. Pipe
	Huffman	E. New Sew. Hwy	W. New Sew. Hwy	30	0.013	240	25	CMP
	Huffman	Northeast	W. New Sew. Hwy	30	0.012	350	25	CMP
	Frontage Rd	Chelea	O'Malley		0.032 *	1800	50	Ditch
	Frontage Rd	South of Chelea	Chelea		0.038	200	60	Ditch
	Frontage Rd	South of Chelea	North of Klatt		0.016	400	40	Ditch
	Frontage Rd	South of Klatt	North of Klatt		0.010 and 0.06	1200	50	Ditch
	Frontage Rd	Midpoint between Klatt & Huffman	South of Klatt		0.06	900	70	Ditch
	Frontage Rd	North of Huffman	Huffman		0.002	400	15	Ditch
	New Sew. Hwy	Chelea	O'Malley		0.026 *	1950	400	Ditch
	New Sew. Hwy	Chelea	South of Klatt		0.025 *	1000	400	Ditch
	New Sew. Hwy	South of Klatt	North of Huffman		0.039	500	500	Ditch
	New Sew. Hwy	North of Huffman	Huffman		0.008	800	200	Ditch
	O'Malley	Cange	Reader		0.045 *	4000	60	Ditch
	O'Malley	Tract A	Reader		0.068 *	1300	90	Ditch
	O'Malley	Tract A	New Sew. Hwy		0.095 *	1300	90	Ditch

* = (APPROX)

TABLE IV-1

CAPACITY OF EXISTING SYSTEMS (CONTINUED)

Sub-BASIN	STREET	FROM	TO	SIZE (inches)	SLOPE (ft/ft)	APPROX. LENGTH. (ft)	ESTIMATED CAPACITY (cfs)	SYSTEM TYPE
G	Jarvus	Karen	Old Sew. Hwy	15	0.004 *	1300	2	CMP
	Hace	Kruge	Harding	12,15	0.003 and 0.004	800	2	CMP
	Harding	Hace	Old Sew. Hwy	15	0.015	500	4	CMP
	South of Kruge	Troy	Hace	15	0.004	350	2	CMP
	Old Sew. Hwy	Karen	New Sew. Hwy		0.019 *	3150	90	Ditch
	Old Sew. Hwy	Karen	North of Dare		0.020 *	1900	90	Ditch
	Old Sew. Hwy @ Dare	West	East	24	0.017	150	20	CMP
	Old Sew. Hwy	Dare	Harding		0.009	1800	60	Ditch
	Old Sew. Hwy @ Harding	East	West	24	0.007	150	10	CMP
	Old Sew. Hwy @ Harding	East	West	24	0.007 *	200	10	CMP
	Old Sew. Hwy	North of Harding	Harding		0.004 *	1300	40	Ditch
	Old Sew. Hwy	North of Harding	Huffman		0.009 *	1100	60	Ditch
	Old Sew. Hwy Below Huffman			Two 24	0.007 *	150	10	CMP
	Alaska R. R.	Hancock	Rabbit Ck Rd	12	0.01 *	5100	2	Subdrain
	Alaska R. R.	Hancock	Rabbit Ck Rd	12,18,36	0.01 *		15	Crossdrain
	Jarvi @ Gwenn			12	0.003	200	2	Outfall DIP
	Alaska R. R. South of Huffman			36	0.01 *	200	40	CMP
	Huffman	New Sew. Hwy	Hace	36	0.006	500	30	CMP
	Huffman	Hace	East of Old Sew. Hwy	42,48	0.005 and 0.008	1500	60	CMP
	Huffman	East of Old Sew. Hwy	West of Old Sew. Hwy	48	0.002	250	30	CMP

* = (APPROX)

TABLE IV-1

CAPACITY OF EXISTING SYSTEMS (CONTINUED)

Sub-BASIN	STREET	FROM	TO	SIZE (inches)	SLOPE (ft/ft)	APPROX. LENGTH. (ft)	ESTIMATED CAPACITY (cfs)	SYSTEM TYPE
H	North of Huffman	East of New Sew. Hwy	West of Old Sew. Hwy	24	0.010 *	300	20	CMP
	Old Sew. Hwy	Klatt	Huffman		0.016 *	2650	80	Ditch
I	Old Sew. Hwy	O'Malley	112th		0.003	1000	30	Ditch
	Old Sew. Hwy	Klatt	112th		0.008	1600	60	Ditch
	O'Malley	Old Sew. Hwy	New Sew. Hwy		0.012	1100	30	Ditch
	O'Malley	Old Sew. Hwy	East of Old Sew. Hwy		0.040	600	60	Ditch
	O'Malley	Alaska R. R.	East of Old Sew. Hwy		0.029	350	50	Ditch
J	Johns Rd @ Deerfield			Two 12	0.01 *	60	2	Crossdrain CMP
	Klatt	Alaska R. R.	East of Ellen		0.014 *	3100	30	Ditch
	Klatt	Mary	East of Ellen		0.032 *	500	50	Ditch
	Klatt	Mary	Timberlane		0.032 *	500	50	Ditch

* = (APPROX)

TABLE IV-1

CAPACITY OF EXISTING SYSTEMS (CONTINUED)

Sub-BASIN	STREET	FROM	TO	SIZE (inches)	SLOPE (ft/ft)	APPROX. LENGTH. (ft)	ESTIMATED CAPACITY (cfs)	SYSTEM TYPE
K	Johns Rd	Klatt	Huffman		0.019	2600	200	Ditch
	Johns Rd	Langnes	Huffman	three 12	0.01 *	60	2 each	Crossdrain CMP
	Johns Rd	Huffman	Galleon		0.024	1050	200	Ditch
	Johns Rd	Huffman	Galleon	12	0.01 *	60	2 each	Crossdrain CMP
	West of Alaska R. R. and South of Huffman				0.005 *	300	150	Flow from pond
	East of Beach-comber @ Huffman				0.01 *	300	250	Flow to Huffman CMP
	Huffman	East of Beachcomber	West of Beachcomber	36	0.003	120	20	CMP
	Beachcomber	Huffman	Breakwater	12	0.001	500	1	CMP
	Clipper Ship Ct.	North	South	36	0.005	85	30	CMP
	Mariner	Beachcomber	Clipper Ship	18	0.006	280	4	CMP
	Johns Rd North of Galleon	East	West	24	0.026	40	20	Crossdrain CMP
	East of Clipper Ship Ct.	Clipper Ship Ct.	Mariner		0.013	530	550	Greenbelt
	South of Mariner	Mariner	Johns Rd		0.006	950	400	Greenbelt

* = (APPROX)

TABLE IV-1

CAPACITY OF EXISTING SYSTEMS (CONTINUED)

Sub-BASIN	STREET	FROM	TO	SIZE (inches)	SLOPE (ft/ft)	APPROX. LENGTH. (ft)	ESTIMATED CAPACITY (cfs)	SYSTEM TYPE
L	Klatt	Timberlane	Toy		0.018 *	2600	40	Ditch
	Klatt	Toy	Victor		0.006 *	3300	20	Ditch
	Victor	Klatt	Turnagain Arm		0.011 *	1300	100	Drainage Cut
	West of Victor	North of Klatt	Turnagain Arm		0.010	1250	100	Drainage Cut
M	Johns Rd @ High View			18	0.01 *	300	5	Outfall CMP
	Oceanview @ Admiralty	North	South	18	0.01 *	300	5	Outfall CMP
	Oceanview @ Gulf	North	South	18	0.01 *	300	5	Outfall CMP

* = (APPROX)

SUBBASIN A

Subbasin A (Figure V-1) has a southwesterly slope of 5-7% to its outlet at Rabbit Creek. Drainage within the subbasin is overland with ditch flow along roads. Along the north side of Rabbit Creek Road, the ditch capacity is approximately 70 cfs, as shown in Table IV-1, while the estimated design flow from Table V-3 is much lower, indicating that the ditch is of adequate capacity.

The existing and future land use classifications are both low density single family residential (LD). As a result of the land use classification and topography of the land it is anticipated that property loss due to flooding would be minimal in this subbasin.

SUBBASIN B

Subbasin B (Figure V-2) slopes southward at a rate of 4-7% to Rabbit Creek, indicating good overland drainage. Through a number of minor roads with ditches exist within the subbasin, the roadway drainage ditch system is for local collection and is not intertied to form a trunk system.

Land use for the present and future as identified in Tables II-2 and II-3 is almost 100% low density residential (LD), indicating that property loss in the case of flooding during a large storm would be minimal. Care must be exercised during construction not to encroach upon the flood plain of Rabbit Creek.

SUBBASIN C

Subbasin C (Figure V-3) flows westward to the New Seward Highway. The collected flow at the highway then travels south along the Frontage Road and the New Seward Highway to Potter Marsh. Capacity of these ditches along New Seward Highway and the Frontage Road are given in Table IV-1. Present drainage facilities in Subbasin C are ditches, pipes and overland flow.

Because of the existing low levels of development in the subbasin, the drainage systems in use are local, and not interconnected to form a trunk system at present. As a result of the localized drainage systems, ponding is prevalent within the subbasin.

In the Turnagain View Subdivision a drainage problem exists in the vicinity of the intersection of Westwind Drive and Capstan Drive. During periods of high runoff, water flowing overland to the north, reaching the houses on the south side of Capstan Drive, traverses the land and enters the drainage system on Westwind. During this study the manhole on Westwind immediately south of Capstan has been observed surcharging. Capacity of the Westwind storm drain was estimated at 3 cfs (Table IV-1), much less than the design flow presented in Table V-3.

Land use within the subbasin is presently low density single family residential or undeveloped. Future land use indicates high density single family residences will comprise approximately 20% of the land use within the subbasin with the remainder continuing in the present land use.

SUBBASIN D

Subbasin D (Figure V-4) flow is westward to the New Seward Highway and Frontage Road, at which point the flow changes its direction and travels north. Within the subbasin the storm runoff paths are: overland, via curb and gutter, pipes, and greenbelt.

In subcatchment 45, the most easterly subcatchment, the land is presently undeveloped or contains low density residential. However, future land use indicates that the entire area will be developed with half of the area in low density single family residential classification and the other half of the area in high density single family residential classification. Because of this impact the present overland flow cannot continue. Additionally, downstream facilities must be sized to handle the routed flow.

Subcatchments 46 and 47 are high density single family residential, and have a storm drainage network consisting of 18" cross drains under Spinnaker and Westwind and a greenbelt traversing the subcatchments. At Tradewind the greenbelt ends and a 24" underground storm drain system extending from east to west on Tradewind allows storm flow to travel to the Frontage Road. Flow collected on the streets travels via curb and gutter to the cross drains at Westwind and Spinnaker.

The 18" cross drains at Spinnaker and Westwind have a estimated capacity of 4 cfs each. However, to handle the design flows identified in Table V-3, a much larger capacity is needed.

The 24" underground storm drain on Tradewind is also of insufficient volume for projected future flows. Table IV-1 indicates a capacity of 10 cfs, much less than the design flow from Table V-3.

In subcatchments 48, 49, and 50, flow travels via curb and gutter ditch and pipes, arriving at the Frontage Road. At the Frontage Road, ponding occurs both on the east and west side of the road. Ponding also occurs on the east and west sides of the Frontage Road in subcatchment 47. The estimated Frontage Road ditch capacity ranges between 10 and 40 cfs and portions of this ditch are unable to handle the design flows of Table V-3.

Within Subbasin D, the overall need is to provide an integrated drainage system with the capability to handle regional needs and to avoid local ponding problems. The area will continue to develop, ultimately planned for approximately 75% high density single family residential and 25% low density single family residential. Much property damage could result from flooding during major storms, if no action is taken for future requirements.

<u>SUBBASIN E</u>

Stormwater runoff through Subbasin E (Figure V-5) flows from east to west in two drainageways which converge just north of Huffman Circle. The most northerly of the two branches is considered to be Furrow Creek. From this location the flow is northwest to the intersection of the New Seward Highway and Huffman. Drainage through the subbasin is accomplished via overland flow, curb and gutter, roadway ditch, pipes, channelized flow and greenbelt.

In the southeast portion of the subbasin are subcatchments 60 and 66. At present this area is undeveloped and flow is overland. However, future land use projects that the area will become developed, approximately 60% as high density single family residential and the remainder as low density single family residential (LD). It is therefore necessary in the design of such future developments to include an adequate storm drainage network.

Subcatchments 61 and 62 are in the northeast portion of the subbasin. In this area the present land use is low density single family residential and the future planned land use is the same. Flow through this area is overland and ditch.

In the southcentral portion of Subbasin E are subcatchments 67 and 68 which form the northernly portion of Turnagain View Subdivision. This area is developed and flow travels primarily via curb and gutter and overland.

Flow is presently primarily overland in subcatchments 64 and 65. However, this area will be developed in the future for low and high density single family residential. Therefore, a drainage network for this area must be addressed during the design phase of development for the area. This network must also handle the upstream contribution from subcatchments 60 and 66.

Flow from subcatchments 61 and 62 is to subcatchment 63. Downstream of subcatchment 63 is subcatchment 70 and 71. All three subcatchments (63, 70, and 71) are presently classified as low density single family residential. In the future it is planned that development will increase the density to high density single family residential. In this area flow travels by ditch, channelized flow and overland flow. Furrow Creek traverses these three subcatchments.

The curb and gutter system of subcatchment 69 routes the flow to the underground pipe system which ranges in size from 18" to 24". This pipe system has a present estimated capacity of 5-10 cfs (Table IV-1) which is much less than projected future flows as identified in Table V-3, indicating that the present pipe is inadequate in size.

In the northwest portion of the subbasin is located subcatchment 72. In this area the flow is overland and channelized as well as routed via ditches. This subcatchment is presently undeveloped. However, future land use planning is for high density single family residential. During development of the area, drainage facilities able to handle the flows of Table V-3 must be constructed.

In summary, for Subbasin E, attention must be given to providing a regional drainage system of adequate capacity to handle the flows of Table V-3. Within Turnagain View Subdivision, inadequately sized facilities must be augmented to provide sufficient capacity.

SUBBASIN F

Within Subbasin F (Figure V-6) flow is generally west and south. The area is presently used for single family residential housing with approximately 10% of the land area in use as a gravel pit, and approximately 25% of the land area in use as wetlands. Storm water flow at present through Subbasin F is overland and channelized flow in ditches along roads.

Subcatchments 82 and 83 drain west to the wetland. At present the area is low density single family residential. About one-half of the area is projected to become high density single family residential. The wetland in the western portion of subcatchment 83 is to be conserved (Wetlands Management Plan, 1982). Flow through this area is presently via ditches and overland flow. No existing problems are apparent in this area.

In the northcentral portion of the subbasin lie subcatchments 76 and 85. At present this area is a gravel pit in the eastern portion, and low density single family residential in the western portion. In the future it is planned that the area will become approximately 90% developed as low and high density residential. During this development a drainage system should be installed, draining to the wetland.

Subcatchments 75, 77, 78, and 79, are presently drained by overland flow, roadway ditches and channelized flow. Existing land use is undeveloped or low density single family residential. As the projected higher residential density development occurs, adequate drainage should be provided.

Two known problems exist within the area of subcatchments 75, 77, 78, and 79. On Cange Street south of the airstrip, the natural drainage has eroded the road. At the low spot in the road, approximately at Cleo Avenue, the flow traverses the width of Cange Road, forming a deep wash in the road.

Another problem is local flooding in the vicinity of Northern Raven Drive and Wilma Avenue. During snowmelt this spring cars were found stranded in this flooded area. Steps should be taken to alleviate this problem.

The wetland in the western portion of Subbasin F contains all or part of the following subcatchments: 80, 81. 83, 84, 85, and 86. According to the future land use plan of the Municipality, this wetland is to be preserved. The volume of this wetland is computed in Table V-2.

In summary, most of Subbasin F is projected to be generally developed as high density single family housing. As development occurs, a storm drainage network must be incorporated, possibly using the wetland as a detention area.

SUBBASIN G

Stormwater within Subbasin G (Figure V-7) flows in three directions. Subcatchments 105, 107, 109, and 110 drain north. Draining west to the Alaska Railroad are subcatchments 101, 102, 103, 106, 108, and 111. Subcatchments 100 and 104 drain south along the western edge of the New Seward Highway.

In the southern portion of the subbasin, subcatchment 104 drains directly to the western edge of the New Seward Highway. Subcatchment 100 drains to the Old Seward Highway and transfers to the new Seward Highway where the two highways intersect. In these subcatchments, the present method of drainage is via roadway ditches and overland flow. No known problems exist or are anticipated. The present land use classification is low density single family residential. The Municipality land use plan indicates that the area is projected to be classified ultimately as high density single family residential.

The subcatchments which drain to the Alaska Railroad (101, 102, 103, 106, 108, and 111) exhibit no drainage problems at this time, nor are any problems anticipated in the future. The present drainage systems are overland flow, curb and gutter, roadway ditches, and pipe systems. At present, the land use is low density single family residential, except in subcatchment 111, and no future increases are anticipated. For subcatchment 111, the present land use is 60% high density single family residential, and the remaining 40% is land which has been cleared

In summary, subbasin G has generally adequate drainage facilities for both present and future needs with the exception of the storm drain along Huffman Road and 36" cross culvert beneath the Alaska Railroad.

SUBBASIN H

Presently drainage in Subbasin H (Figure V-8) is handled with pipes, ditches, overland flow and channelized flow. At this time no problems are known to exist, and the capacities of the existing facilities are anticipated to handle the present and future estimated flows of Table V-3. However, an integrated trunk system should be incorporated in the design of future development in the subbasin.

SUBBASIN I

The present method of draining Subbasin I (Figure V-9) to the South Anchorage drainage study area is via ditches and overland flow. These systems have adequate estimated capacity for both present and future flows. The present system in Subbasin I is not well intertied within the subbasin for transfer of water ultimately to the subbasin outlet. It would be beneficial as the area continues to develop into an area projected to have industrial/commercial/ multi-family residential usage, that an intertied system within the subbasin be incorporated.

SUBBASIN J

Subbasin J (Figure V-10) drains north, out of this study area into the South Anchorage drainage study. The area presently is drained via overland flow and roadway ditches. No problems are known to exist of this subbasin. However, as development continues to its planned ultimate level (high density single family residential), attention should be given to creating an integrated drainage system.

SUBBASIN K

The most downstream portion of Furrow Creek is located in Subbasin K (Figure V-11). The area is approximately one-half developed at this time, with the majority of land being used as high density single family residential.

Ultimately the subbasin is planned to have approximately 90% developed with high density single family residential and multi-family housing.

Beginning at the Alaska Railroad 36" cross culvert, upstream flow from Subbasin G travels to the Huffman Road right-of-way. At the intersection of Beachcomber and Huffman the flow is routed through a 36" cross culvert beneath Beachcomber. Flow continues along the south edge of the Huffman right-of-way to the west of Division Street where it is diverted south across Clipper Ship Court and enters the greenbelt system of Oceanview Subdivision.

Prior to reaching Beachcomber, the flow of Furrow Creek is impeded by a recent fill on the south side of the stream. This fill encroaches upon Furrow Creek and has inadequate side slope for soil stabilization. This fill presents an immediate problem to the flow of Furrow Creek.

The greenbelt through Oceanview Subdivision extends from Clipper Ship Court to Johns Road. The capacity of the greenbelt itself is estimated to be adequate for the flows in Table V-3. However, the 18" to 36" culvert crossings at Clipper Ship, Mariner and Johns Road have estimated capacities (Table IV-1) less than the estimated design flow as shown in Table V-3.

As Furrow Creek continues downstream of Johns Road, the creek enters Johns Park. The portion of Johns Park lying within subcatchments 165 contains a channel of inadequate capacity (Table IV-1) for handling the estimated flow of Table V-3. However, in subcatchment 166, the capacity of the drainage channel improves and becomes capable of handling the estimated flow of Table V-3.

In the northeast portion of the subbasin is subcatchment 167. This area is approximately one-third developed at this time. Ultimately 80% of the land area is projected for development as residential and industrial usage. Present methods of handling drainage is overland flow and roadway ditches. No problems are known to exist presently in this area. As development progresses, an integrated drainage system should be incorporated to drain approximately at Division Street to Furrow Creek.

Subcatchments 168 and 169 in the northcentral portion of Subbasin K drain to Johns Road. These subcatchments are presently being developed, using overland flow and road ditches as drainage methods. Attention should be given to an area-wide drainage network during development. The capacity of the Johns Road system (Table IV-1), which includes both cross culverts under Johns Road and a ditch system along Johns Road are adequate for the estimated flows as shown in Table V-3.

In the northwest portion of the subbasin are subcatchments 170 and 185. These subcatchments drain to Timberlane Road and flow travels overland to the outlet of Furrow Creek into Turnagain Arm. The land area presently is approximately 25% developed. It is anticipated that the area will become almost entirely developed with high density single family residential housing. Presently the area is drained by overland flow and roadway ditches. As development progresses, an integrated drainage system should be implemented.

In summary, Subbasin K has present problems west of the Alaska Railroad, at the landfill east of Beachcomber, and at road crossings along the greenbelt. As development occurs north of Huffman, an area-wide drainage network should be implemented.

SUBBASIN L

Subbasin L (Figure V-12) drains to Turnagain Arm. The topography of the land is quite flat, with wetlands in the western portion. The present level of development is approximately 20%. It is anticipated that development will ultimately be approximately 80% of the land being used as high density single family residential. In the Wetlands Management Plan the Klatt Bog area of Subbasin L has been assigned "conservation status; i.e., development can occur on the fringes of the wetland but the natural character of the wetland must be returned to the greatest extent possible.

The present method of draining stormwater is via overland flow, roadway ditches and channelized flow. No known problems exist in the subbasin at this time. However, during future development proper consideration must be made to management of the Klatt Bog per the Wetlands Management Plan. Also, in areas designated in the land use plan for development, an integrated storm drainage system should be incorporated into the design.

SUBBASIN M

Subbasin M (Figure V-13) comprises a portion of the Oceanview Subdivision and is presently about 90% developed as high density single family residential. Storm runoff is routed via curb and gutter, overland flow and pipe systems to three 18" outfalls along the bluff north of the mudflats.

At present the three outfall pipes are exhibiting structural problems and the bluff is eroding away as a result. Estimated design flows of Table V-3 indicate that the outfall pipes have insufficient capacity (Table IV-1) to handle the projected future flows.

No other problems are known to exist.

QUALITY

General

At the present, there are no water quality problems identified in the project basin. Storm runoffs do not present any serious danger to the beneficial uses in the basin with the exception of recreation and aesthetics. The Furrow Creek-Rabbit Creek basin consists largely of low residential lands and a small percentage of commercial and industrial lands, most of which are located in subbasins I and J. Runoff from subbasins I and J drain to the South Anchorage drainage basin study area.

Table IV-2 shows the receiving water of subbasin runoffs in the Furrow Creek-Rabbit Creek basins. Channelized stream flow in the Upper and Middle Furrow Creek segments is difficult, if not impossible, to locate. Flow in this area is generally overland. The large wetland in subbasin F, Klatt Bog, and Turnagain Arm are receiving water bodies have no water quality concerns at present nor are problems anticipated in the future. The Alaska Department of Fish and Game maintains Dolly Varden and a small run of pink salmon in the Lower Rabbit Creek. Potter Marsh, the final outlet of the Rabbit Creek, provides important habitat for fish and wildlife. Lower Furrow Creek, however, has no recorded fish

TABLE IV-2

RECEIVING WATERS OF FURROW CREEK-RABBIT CREEK

STORM MODEL BASIN NAME	RECEIVING WATER
AB	Lower Rabbit Creek
C	Potter Marsh
D	Upper Furrow Creek
E1	Upper Furrow Creek
E2	Upper Furrow Creek
F1	Wetland
F2	Upper Furrow Creek
G1	Potter Marsh
G2	Turnagain Arm
G3	Middle Furrow Creek
H	Middle Furrow Creek
I	South Anchorage Study Area
J	South Anchorage Study Area
K1	Lower Furrow Creek
K2	Lower Furrow Creek
K3	Lower Furrow Creek
L1	Turnagain Arm
L2	Klatt Bog, Turnagain Arm
M	Turnagain Arm

population but its surrounding park setting makes Furrow Creek important in the aspect of recreation and aesthetics.

The portion of the study area which is tributary to Lower Rabbit Creek and Potter Marsh constitute a small percentage (7%) of the total contributing area. The area is mostly of low density residential land use. The water quality effects of storm runoff from the area tributary to Lower Rabbit Creek should be insignificant on the receiving water, as indicated by the low estimated pollutant loads in Table III-2.

The Lower Furrow Creek, due to presently observed land erosion and potential large quantity of flow from upstream basins, would be the only critical water quality area in the study area.

Estimated Pollutant Loads

For water quality evaluation purposes, pollutant loads from storm washoff were estimated using SAM and STORM models. (For details of the computer analysis - see Appendix A). The SAM model provides estimates of the pollutant loads at spring break-up. The STORM model computes the pollutant loadings for the period between spring break-up and winter freeze-up. Six water quality parameters, total dissolved solids, suspended solids, 5 day biological oxygen demand (BOD_5), grease and oil, fecal coliform and ammonia, which are critical to the beneficial uses were modeled. Both the SAM and the STORM used the pollution

TABLE IV-3

POLLUTANT BUILDUP MATRIX

Land Use	ID	TDS lbs/Ac/ Day	SUS-SOL lbs/Ac/ Day	BOD5 lbs/Ac/ Day	GRS-OIL lbs/Ac/ Day	FE-COL Billion MPN/Ac/ Day	AMMONIA lbs/Ac/ Day
				SUMMER/WINTER			
Commercial	CO	1.3/0.8	5.0/4.9	.43/.33	.23/.20	.15/.009	.010/.005
Industrial	IN	2.0/1.0	7.5/7.5	.60/.35	.34/.30	.01/.002	.010/.005
Multiple-Family Residential	MF	0.40/1.3	.50/1.6	.12/.34	.06/.10	.007/.0008	.005/.005
High Density Residential	HD	0.3/0.8	.20/1.0	.10/.49	.04/.03	.002/.002	.002/.016
Low Density Residential	LD	0.25/0.3	.15/0.7	.05/.10	.02/.01	.0015/.001	.002/.005
Cleared Pervious	UP	0.2/0.3	.10/0.5	.02/.05	.002/.001	.001/.001	.002/.0005
Bogs and Marshes	BM	0.1/0.2	0.0/0.0	.01/.02	.0001/0.0	.001/.0001	.0002/.0001
Lowland Forest	LF	0.1/0.1	0.0/0.0	.01/.01	.0001/0.0	.001/.0001	.0002/.0001
Upland Forest	UF	0.1/0.1	0.0/0.0	.01/.01	.0001/0.0	.001/.0001	.0002/.0001
Natural Pervious	GP	0.1/0.1	0.0/0.0	.01/.01	.0001/0.0	.001/.0001	.0002/.0001

Reference: Campbell Creek Drainage Study - Task memo No. 7, 1979) buildup factors in Table IV-3 which were derived from previous water quality studies for the greater Anchorage area (208 Water Quality Management Plan, 1979).

Using the SAM model it is possible to generate the total pollutant washoff loads at the spring break-up and the associated peak concentrations at the mouth of the Lower Furrow Creek under future drainage conditions. The pollutant washoff loads for Lower Furrow Creek are presented in Table IV-4. Under the existing drainage condition, numerous upstream pondings would result in decreasing pollutant loads and concentration at the mouth of the Lower Furrow Creek. The results of the SAM model in Table IV-4 are for the recommended system.

The STORM model computer runs provided long-term pollutant washoff loads which constituted numerous runoff events. Because the model has no routing feature, the pollutant washoff loads and corresponding average pollutant concentrations were generated on a subbasin basis. The first two runs performed used an average hydrologic year (1963) and compared long-term pollutant loads associated with existing and future land use. The two following runs made using the STORM model were to simulate the long-term pollutant loads and average pollutant concentrations under future land use were generated for two extreme hydrologic years (1967 and 1969), wet and dry. The results of the four computer runs are presented in Table IV-5. Figure IV-1 graphically depicts the monthly precipitation for the three summers (1963, 1967, 1969) chosen for use in this analysis.

TABLE IV-4

POLLUTANT WASHOFF LOADS

Water Quality Parameter	Total Pounds* of Washoff Load	Peak Concentration (mg/L)
Total Solids	5560	1580
Suspended Solids	4798	1355
BOD_{50}	325	92
Ammonia	5	1.5
Oil/Grease	196	55
Fecal Coliform	7.86**	0.52***

* Sum of the pollutants from time of 0 to 8 hr. 20 min.

** MPN in Billion

*** 10^3 MPN/L

Note: The values shown are for total pollutant washoff loads and corresponding peak concentration at the spring break-up at the mouth of the Lower Furrow Creek which have been generated for the design storm event - based on the recommended drainage system.

TABLE IV-5

ESTIMATED POLLUTANT LOADING
(1963 Summer (ave. summer), existing land use)

BASIN ID	TOTAL POUNDS OF WASHOFF FROM WATERSHED: Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal * Coliform	CONCENTRATION OF POLLUTANTS TO RECEIVING WATERS (Mg/1): Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal ** Coliform
AB	4941	2979	2256	236	789	38	75	47	25	3	8	1.24
C	1968	1156	1068	88	366	15	65	40	25	3	8	1.06
D	3113	1931	1897	145	660	22	72	47	31	3	10	1.19
E1	1671	955	804	66	260	11	64	38	25	3	6	1.19
E2	1442	609	735	52	148	27	38	17	12	1	2	0.72
F1												
F2	38420	34193	10121	1885	3713	124	908	818	182	44	57	4.16
G1	1125	692	579	40	198	2	71	45	27	3	9	0.96
G2	11006	9017	3945	543	1408	442	162	135	47	8	16	10.71
G3	8702	6720	3476	449	1371	115	115	91	37	6	14	2.69
H	29164	26139	7109	1433	2507	969	452	408	96	22	31	26.00
I	39635	35684	9578	1945	3429	920	531	481	114	26	38	22.00
J	2720	2115	1173	122	425	5	134	107	46	6	16	1.56
K1	4820	4276	1243	239	441	9	593	533	126	29	39	2.69
K2	1752	1035	1057	78	340	12	66	41	29	3	9	1.45
K3	950	485	452	37	104	11	53	28	18	2	4	1.27
L1	3957	2479	2663	179	965	32	65	42	33	3	11	1.25
L2	470	234	363	11	108	4	43	23	24	2	7	1.43
M	2645	1645	1768	110	636	19	65	42	34	3	12	1.31

* MPN in Billion

** 10^3 MPN/1

TABLE IV-5

ESTIMATED POLLUTANT LOADING
(1963 Summer (ave. summer), future land use)

BASIN ID	TOTAL POUNDS OF WASHOFF FROM WATERSHED: Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal * Coliform	CONCENTRATION OF POLLUTANTS TO RECEIVING WATERS (Mg/l): Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal ** Coliform
AB	4941	2979	2256	236	789	38	75	47	25	3	8	1.24
C	1968	1156	1068	88	366	15	65	40	25	3	8	1.06
D	3291	2043	2111	147	757	25	67	44	31	3	10	1.11
E1	2650	1610	1334	119	475	16	72	46	26	3	9	1.19
E2	6033	3754	3723	286	1345	55	67	44	31	3	11	1.19
F1												
F2	4924	3012	3248	226	1154	54	64	41	31	3	11	1.25
G1	1125	692	579	40	198	2	71	45	27	3	9	0.96
G2	11006	9017	3945	543	1408	442	162	135	47	8	16	11.00
G3	16296	13788	5336	817	1952	673	186	159	51	9	17	13.16
H	52494	47530	12403	2581	4383	1507	575	523	123	28	41	30.63
I	39635	35684	9578	1945	3429	920	531	481	114	26	38	22.37
J	2720	2115	1173	122	425	5	134	107	46	6	16	1.56
K1	14367	12906	3545	706	1296	33	716	648	152	35	51	4.00
K2	3163	2085	1931	148	720	33	70	48	33	3	12	1.63
K3	2377	1453	1645	103	588	18	62	40	32	3	11	1.16
L1	3957	2479	2663	179	965	32	65	42	33	3	11	1.25
L2	470	234	363	11	108	4	43	23	24	2	7	1.43
M	2645	1645	1768	110	636	19	65	42	34	3	12	1.31

* MPN in Billion

** 10^3 MPN/1

TABLE IV-5

ESTIMATED POLLUTANT LOADING
(1967 Summer (wet summer), future land use)

BASIN	TOTAL POUNDS OF WASHOFF FROM WATERSHED						CONCENTRATION OF POLLUTANTS TO RECEIVING WATERS (Mg/l)					
ID	Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal * Coliform	Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal ** Coliform
AB	5733	3397	2435	261	838	39	70	44	26	3	9	1.36
C	2235	1323	1176	92	393	8	63	39	30	3	10	1.39
D	3762	2308	2287	158	817	20	63	41	33	3	11	1.28
E1	3049	1834	1449	118	506	8	67	43	28	3	10	1.13
E2	6966	4276	4016	311	1434	52	63	41	31	3	11	1.24
F1												
F2	5595	3406	3545	255	1252	54	60	39	34	3	12	1.44
G1	1325	798	624	40	203	1	67	43	28	3	10	1.14
G2	12473	10168	4312	628	1517	470	156	130	47	8	16	10.60
G3	18671	15684	5818	920	2103	724	180	154	49	9	17	12.48
H	59954	54084	13609	2917	4714	1608	564	513	117	27	38	28.52
I	45273	40595	10503	2206	3691	981	521	417	109	25	36	20.93
J	3076	2385	1287	127	455	5	131	105	48	6	17	1.50
K1	16290	14617	3910	795	1416	32	709	643	153	34	52	4.03
K2	3595	2352	2092	161	767	29	68	46	34	3	12	1.70
K3	2700	1653	1789	106	639	14	60	39	35	3	12	1.35
L1	4549	2808	2886	203	1041	31	62	40	34	3	12	1.30
L2	472	250	410	10	115	4	40	22	32	1	10	1.51
M	3023	1867	1916	114	689	10	62	40	34	3	12	1.18

* MPN in Billion

** 10^3 MPN/l

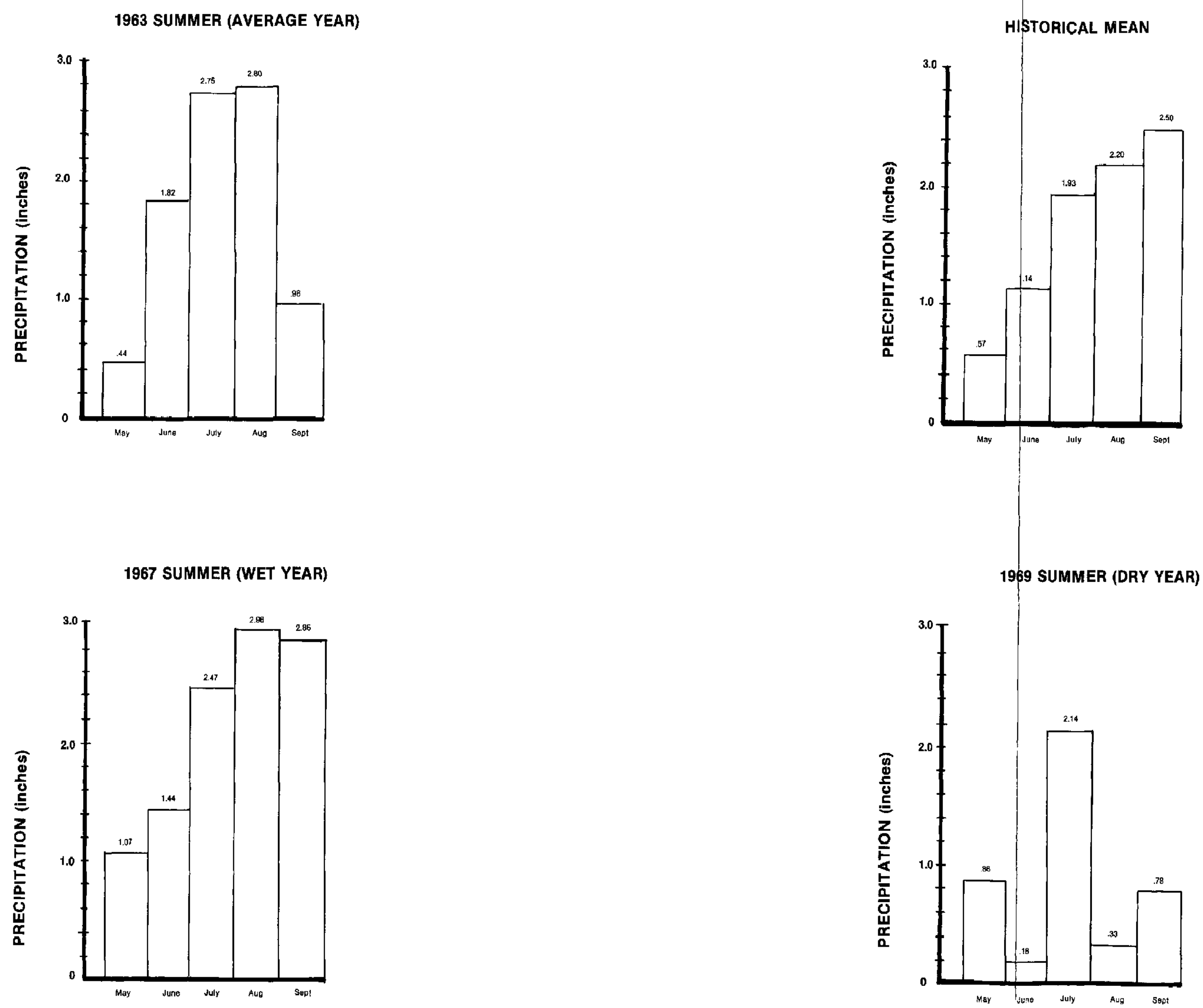
TABLE IV-5

ESTIMATED POLLUTANT LOADING
(1969 Summer (dry summer), future land use)

BASIN ID	TOTAL POUNDS OF WASHOFF FROM WATERSHED						CONCENTRATION OF POLLUTANTS TO RECEIVING WATERS (Mg/1)					
	Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal * Coliform	Total Solids	Suspended Solids	BOD	Ammonia	Oil/ Grease	Fecal ** Coliform
AB	2978	1934	1967	176	714	45	84	55	44	4	15	2.40
C	1150	737	906	65	320	22	72	47	48	4	17	2.51
D	1940	1295	1818	103	675	32	75	51	58	4	21	2.37
E1	1593	1044	1174	87	430	21	81	54	48	4	17	2.18
E2	3650	2433	3287	205	1223	61	78	53	58	4	21	2.41
F1												
F2	2869	1892	2748	159	1006	59	71	48	59	4	21	2.62
G1	677	445	505	35	183	9	81	54	49	4	17	2.42
G2	6713	5689	3252	360	1235	391	197	168	84	10	31	21.40
G3	10273	8934	4498	555	1764	621	237	207	91	12	34	26.40
H	33459	30813	10038	1708	3947	1376	772	714	214	39	81	62.04
I	25234	23130	7777	1293	3090	844	705	649	198	36	76	45.33
J	1629	1324	973	82	368	14	160	131	85	8	31	3.03
K1	8899	8154	2778	453	1123	39	896	823	262	45	103	8.19
K2	1869	1311	1647	102	631	38	83	59	64	4	24	3.35
K3	1383	916	1389	74	511	25	71	48	64	4	23	2.79
L1	2352	1575	2315	131	860	41	76	52	63	4	23	2.54
L2	269	148	296	13	95	9	47	26	39	2	12	2.72
M	1564	1048	1534	84	572	27	77	52	63	4	23	2.53

* MPN in Billion

** 10^3 MPN/1



URS Engineers
Anchorage, Alaska

SOURCE:

National Oceanic and Atmospheric Administration Asheville, N.C.

COMPARISON OF HISTORICAL SUMMER PRECIPITATION EVENTS

February 1983 Figure IV-1

Water Quality Effects and Control Measures

Water quality effects are evaluated in this study with respect to the beneficial uses. The beneficial use of Lower Furrow Creek is basically recreation. Key water quality parameters for Lower Furrow Creek are solids, oil/grease, and fecal coliform. At the spring break-up, the concentrations of these parameters are quite high in the creek. However, it is unlikely that much water recreation would occur on the creek during that period. During the summer, the height of the recreation season, the concentration of these parameters from each subbasin runoff are potentially high on the account of four basins, designated in the STORM model as F2, H, I, and K1. These basins all contain commercial and/or industrial land uses. It is important to note that the entirety of basins I and J drain to the South Anchorage storm drainage study area. Best management practices, such as improving road pavement, frequent catchbasin cleaning, regrading of disturbed areas and control of erosion would be sufficient to reduce, and potentially control, the pollutant loads.

The beneficial use of the Lower Rabbit Creek and Potter Marsh is for fisheries. Key water quality paramaters are: DO, fecal coliform, oil/grease, and solids. The total pollutant loads and average concentrations from the tributary basin designated in the STORM model as AB, C, and G1, are not high. The majority (at least 83%) of Rabbit Creek flow is from outside of this project area and should have better water quality than the tributary study area as the result of very little development. The low pollutant level in the tributary outside the project area should dilute the pollutants from the project area substantially. Specific water quality control measures are not deemed necessary in the basins AB, C, and G1.

CHAPTER FIVE
PRESENTATION OF ALTERNATIVES

CHAPTER 5

PRESENTATION OF ALTERNATIVES

INTRODUCTION

Six general alternatives were identified for stormwater planning within the study area. Each of the six alternatives are described in detail in this chapter. These general alternatives were applied to the study area in conjunction with the goals and planning criteria for the area, in order to identify the best alternative for each subcatchment.

An overview of the recommended alternatives for the study area is presented in this chapter. Following the overview, is a discussion of the alternatives for each subcatchment and their evaluation. The alternatives evaluated, as well as the recommended one(s) are summarized by subcatchment in Table V-1.

Also included in this chapter is a section which outlines the use of the plan for developers and design engineers.

DESCRIPTION OF GENERAL ALTERNATIVES

General

Historically, Furrow Creek-Rabbit Creek drainage area has experienced minimal urbanization pressures. The development pressures that have existed have been in small, concentrated, isolated location. These include: strip development along Old Seward Highway; Huffman Road; O'Malley Road; Oceanview Subdivision; and the Turnagain View Subdivision area. In the past, each developing area provided for their own stormwater drainage needs and associated control measures. But as urbanization pressures increased and the concentration of developments increased, the problems associated with the lack of a comprehensive stormwater management plan became apparent. In the Furrow Creek-Rabbit Creek urbanized setting, there are several stormwater runoff problems that require attention. These include: flooding, soil erosion, sedimentation, maintenance of drainage ways, water quality considerations, visual impact, and impedence to development.

In developing alternatives for this study, it was concluded that the major problem associated with stormwater runoff was the lack of a comprehensive storm drainage network. As a result, alternatives for controlling stormwater runoff within the study area for present and future conditions were placed into six general categories. These categories are consistent with the goals and criteria set forth in this chapter. The six alternatives are: no modifying action; local detention ponds and trunk system; trunk system; regional detention ponds; corridor/greenbelt systems; and diversion to other subcatchments. As the thirteen subbasins include varying storm drainage conditions which require different methods, no one alternative could be used throughout the entire study area. From the six alternatives listed above, it was possible to tailor modifications to existing facilities and to provide a comprehensive plan for future drainage networks.

The alternatives presented are for comprehensive storm drainage/water quality management plan for each subbasin in the study area. There are no recommendations for minor collection facilities which should be designed to fit specific needs.

ALTERNATIVE #1 - No Modifying Action

Under this alternative, no modifications will be made to the existing stormwater drainage system. This alternative is recommended in subcatchments where the quantity of runoff associated with the design storm can be handled adequately by the existing stormwater drainage network, and no water quality problems are apparent.

ALTERNATIVE #2 - Local Detention Ponds and Trunk System

Stormwater flows would be routed in this alternative through a trunk system which would be either an open system of roadside ditches and culverts, or a closed system of underground culverts, or an open channel or a combination of all of these systems, all of which would maximize the use of existing drainage networks or planned roadways and easements. Peak flow rates would be dampened by the use of local detention ponds. Under this alternative, the volume of stormwater runoff to be detained would equal the volume of stormwater necessary to keep runoff rates equal to pre-development rates. In this alternative development would not increase stormwater runoff rates, but might increase overall volume of stormwater runoff.

This alternative is best suited to areas which are not completely developed. Such areas still have the ability to incorporate into their development pattern the necessary space for detailing stormwater runoff. This alternative has the potential of avoiding enlargment of existing facilities in downstream developed areas, as a result of upstream urbanization.

ALTERNATIVE #3 - Trunk System

Stormwater flows would be routed in this alternative through a trunk system which would be either an open system of roadside ditches and culverts, a closed system of underground culverts, or an open channel system, or a combination of all of these, all of which would maximize the use of existing drainage networks, or planned roadways and easements. This trunk system would require the sizing of all ditches and pipes to capacities which would be large enough to handle the anticipated peak flow rates. This alternative is best suited in fully developed areas where land is at a premium and easements are mandatory.

ALTERNATIVE #4 - Regional Detention Ponds

By detaining stormwater runoff generated from large areas in man-made ponds, natural depressions, or existing identified wetlands, and controlling the release of the detained water, drainage facilities downstream will require smaller carrying capacities. These detention ponds also aid in the recharging of ground water levels and offer possible recreational value. However, the ability to incorporate a regional detention pond is limited to areas with low density levels, and areas containing existing wetlands or natural depressions. Additionally, it is necessary to incorporate adequate land in these areas for the detention pond system.

ALTERNATIVE #5 - Corridor/Greenbelt System

Under this alternative the natural drainageway would be preserved through the use of corridors and greenbelts. These facilities would be designed to carry the average runoff flows and also would have the capacity to handle peak flow rates in an open channel system.

The corridor/greenbelt system is aesthetically pleasing and provides a parkway appearance along the drainageway path. Since land must be reserved for this alternative, its use is limited to areas in which a corridor/greenbelt system can be incorporated into the development pattern for the area. It is mandatory to protect the corridor/greenbelt after its establishment from develpment encroachment through the use of zoning regulations.

ALTERNATIVE #6 - Diversion

Where inadequate downstream capacity exists to handle the peak runoff rates, it may be possible to divert all of or a portion of the upstream flow to another subbasin or subcatchment where the facilities are capable of handling the additional flow. This alternative of diversion is not an independent alternative, but rather a partial alternative to be used in conjunction with another alternative as presented previously.

OVERVIEW OF RECOMMENDED ALTERNATIVES

In recent years, this study area has experienced a rapid increase in urbanization. As a result of this urbanization, a number of minor stormwater drainage collection systems, and, in some cases major stormwater drainage trunk systems, have been installed to alleviate individual, isolated stormwater runoff problems. This study is the first composite analysis of the study area which analyzes these individual minor collection and/or major trunk systems on a comprehensive level.

There are two major problem areas which should be addressed in the Furrow Creek-Rabbit Creek basins: 1) the lack of a comprehensive storm drainage trunk network which would tie together all of the small collection or isolated trunk systems for the areas; and, 2) the Middle and Lower Furrow Creek segments in critical areas do not have adequate capacity for present and future stormwater flows.

The areas which drain directly to Rabbit Creek (subbasins A, B, C and portions of G) and the areas flowing north out of the study area to the South Anchorage Drainage Study area (subbasins I and J,) have minor and isolated stormwater related problems at present but no major stormwater related problems presently exist or are foreseen in the future. For these subbasins it was generally recommended that the alternative of a trunk system be implemented for situations where future flows would exceed capacities of existing systems or for areas where no stormwater systems presently exist. Because the drainage area for subcatchments within these subbasins is small, this trunk system will be a minor drainage system, such as a collection system, in most cases.

In subbasins L, M, and portions of G, which drain directly to the Turnagain Arm, only isolated cases of existing or projected future stormwater runoff related problems exist. These problem areas are located in subbasin M where the existing outfall pipes from the collection systems are undersized for present and future flows, and it is recommended that the capacity of these outfalls be increased. In the remaining portions of these subbasins, it is recommended that either no modifying action be implemented, because of the lack of existing or future stormwater runoff problems, or that a trunk system be implemented where no stormwater system presently exists and future land use indicates increased development. The drainage area of these subbasins are such that this trunk system will be a minor drainage facility, such as a local collection system discharging to an outfall pipe to Turnagain Arm.

The remaining subbasins are the three segments of Furrow Creek: 1) the Upper Furrow Creek segment east of the New Seward Highway; 2) the Middle Furrow Creek segment between the New Seward Highway and the Alaska railroad; 3) The Lower Furrow Creek segment from the Alaska Railroad to Turnagain Arm. In the evaluation of these three segments of Furrow Creek it was concluded that the main requirement for the Upper Furrow Creek segment for the future will be the installation of an adequate trunk and associated collection system to convey stormwater runoff to the Middle Furrow Creek segment. The main deficiency of the Middle Furrow Creek segment is inadequate capacity for present or future flows for its complete length. In the Lower Furrow Creek segment the major deficiencies are the isolated cases where inadequate capacity exists for both present and future runoff volumes, generally associated with streets and railroad crossings.

One of the major problem areas identified in this study, is the overloading of the Middle and Lower Furrow Creek segments as a result of increased urbanization pressures in the Upper Furrow Creek segments. In the evaluation of alternatives, it was determined that stormwater management techniques which would reduce the peak design stormwater flows from Upper Furrow to Middle Lower Furrow Creek segments should be implemented. To minimize this overloading condition alternatives which maximize the use of local depressions and regional detention ponds by using existing small depression areas or wetlands are recommended. These alternatives are recommended only in areas of single-family and multi-family dwellings or in identified wetland/open space areas, so as not to affect future development patterns. However, the projected peak flow rates are such that the Middle and Lower Furrow Creek segments will be overloaded. It is therefore recommended that the Municipality of Anchorage give consideration to additional regional detention ponds in areas where future land use patterns would be altered, particularly in subcatchments 72 and 71 (located in subbasin E) in order to further minimize peak runoff in those areas.

In addition to these detention facilities, a major trunk system layout has been recommended for subbasins D, E, and F. These trunk systems will convey collected stormwater runoff to the middle segment of Furrow Creek located approximately at the intersection of Huffman Road and the New Seward Highway. These trunk systems follow the existing natural drainage corridors or existing streets to the maximum extent possible. For the upper reaches of these subbasins, alternatives recommended are for a localized trunk/collection facilities which will control stormwater runoff.

The areas east of subbasins D, E, and F where overland flows cross the study area boundary, it is recommended that the overland flows be diverted away from the Furrow Creek drainage either to the Rabbit Creek drainage to the south or the Campbell Creek drainage to the north in order not to overload the recommended trunk systems as presented in this study.

Irrespective of which detention methodology is used in the Upper Furrow Creek segment, there exists a severe problem in the Middle Furrow Creek segment for both present and projected flows.

The Middle Furrow Creek segment, which is the northern portion of subbasin G and subbasin H between the New Seward

Highway and the Alaska Railroad, contains an existing stormwater drainage system. This system is inadequate for existing and projected stormwater runoff flows. Recommendations are presented in this chapter to alleviate this situation. Because of the close proximity of major highway corridors to this drainage system and existing business development in this segment, it is recommended that action be taken as soon as possible to alleviate this problem area.

The Lower Furrow Creek segment, or Subbasin K, presently is a corridor/greenbelt system in which flow constrictions exist which will cause localized ponding and flooding during times of peak runoff. Recommendations are presented for this subbasin which will alleviate these local constrictions by allowing an increase in the carrying capacity of the creek, while still maintaining its corridor/greenbelt nature for future projected flows.

In summary, all of the areas which drain directly to Turnagain Arm, out of the study area, or to Rabbit Creek, have minimal existing stormwater related problems, and by implementing the recommendations of this study, the potential future stormwater problems will be alleviated at minimal expense. The Furrow Creek drainage has no trunk system in the upper segment and has existing problems in the middle and lower segments. These situations can also be alleviated with the use of the alternatives as presented in this chapter. Refer to Table V-1 for an overview of the alternative evaluation for this study area.

TABLE V-1

SUBBASIN ALTERNATIVE EVALUATION

SUBBASIN	SUBCATCHMENT	EXISTING COLLECTION SYSTEM: Overland flow	Curb & Gutter	Roadway ditch	Pipe system	Channelized flow	Greenbelt	FUTURE % DEPRESSION AND/OR WETLAND	PROBLEMS	TRUNK SYSTEM I.D. *	ALTERNATIVES (RECOMMENDED ALTERNATIVE(S) CIRCLED): 1. No modifying action	2. Local detention & trunk system	3. Trunk system	4. Regional Detention	5. Corridor/green belt system	6. Diversion
A	1	*		*				30		RC Rd	*	*	*			
B	11	*		*				20		RC	*	*	*			
	12	*		*				30		RC	*	*	*			
	13	*		*				20		RC	*	*	*			
	14	*		*				20		RC	*	*	*			
	15	*		*				27		RC	*	*	*			
	16	*		*				30		OSH-2	*	*	*			
	17	*		*				20		RC	*	*	*			
C	25	*			*			0	ponding	NSH-E		*	*			*
	26	*		*				20	ponding	NSH-E		*	*			
	27	*		*				30	ponding	NSH-E		*	*			
	28	*		*				20	ponding	NSH-E		*	*			
	29	*		*				30	ponding	NSH-E		*	*			*
D	45	*						15		UFC-S1	*	*	*		*	
	46	*	*				*	0		UFC-S1	*		*			
	47	*	*		*		*	0	ponding	UFC-S1		*	*	*		*
	48	*	*		*			0	ponding	UFC-S1		*	*	*		*
	49	*	*		*			0	ponding	UFC-S1		*	*	*		
	50	*	*					0	ponding	UFC-S1		*	*	*		

* Trunk system I.D. corresponds to the trunk systems as shown on Figures V-1 through V-13.

TABLE V-1

SUBBASIN ALTERNATIVE EVALUATION

SUBBASIN	SUBCATCHMENT	EXISTING COLLECTION SYSTEM: Overland flow	Curb & Gutter	Roadway ditch	Pipe system	Channelized flow	Greenbelt	FUTURE % DEPRESSION AND/OR WETLAND	PROBLEMS	TRUNK SYSTEM I.D.	ALTERNATIVES (RECOMMENDED ALTERNATIVE(S) CIRCLED): 1. No modifying action	2. Local detention & trunk system	3. Trunk system	4. Regional Detention	5. Corridor/green belt system	6. Diversion
E	60	*						10		UFC-S3		*	*			
	61	*		*				20		UFC-S4	*	*	*			
	62	*		*				15		UFC-S4	*	*	*			
	63	*		*		*		16		UFC-S4	*	*	*		*	
	64	*		*				12		UFC-S3	*	*	*			
	65	*						0		UFC-S3	*	*	*	*	*	
	66	*						0		UFC-S3	*	*	*			
	67	*	*					0		UFC-S3	*	*	*			
	68	*	*		*			0		UFC-S3	*	*	*			
	69	*	*		*			0		UFC-S3	*	*	*			
	70	*		*		*		40		UFC-S4	*	*	*	*	*	
	71	*		*		*		0	ponding	UFC-S4		*	*	*	*	
	72	*		*		*		0		UFC-S2	*	*	*	*	*	
F	75	*		*		*		12	Cange St. ponding	UFC-N3		*	*			
	76	*						5		UFC-N2	*	*	*	*		
	77	*		*		*		0	flooding	UFC-N3		*	*			*
	78	*		*				0		UFC-N3	*	*	*			
	79	*		*				4	ponding	UFC-N3		*	*		*	
	80	*		*		*		25		UFC-N3	*	*	*	*	*	
	81	*		*		*		85	regional ponding	Wetlands		*	*	*	*	
	82	*		*				9		UFC-N1	*	*	*			
	83	*		*				30		UFC-N1	*	*	*	*		
	84	*						30		Wetlands	*	*	*	*		
	85	*		*		*		10	regional ponding	Wetlands		*	*	*		
	86	*		*		*		15	regional ponding	Wetlands		*	*	*	*	

TABLE V-1

SUBBASIN ALTERNATIVE EVALUATION

SUBBASIN	SUBCATCHMENT	EXISTING COLLECTION SYSTEM: Overland flow	Curb & Gutter	Roadway ditch	Pipe system	Channelized flow	Greenbelt	FUTURE % DEPRESSION AND/OR WETLAND	PROBLEMS	TRUNK SYSTEM I.D.	ALTERNATIVES (RECOMMENDED ALTERNATIVE(S) CIRCLED): 1. No modifying action	2. Local detention & trunk system	3. Trunk system	4. Regional Detention	5. Corridor/green belt system	6. Diversion
G	100	*		*				15		OSH-2	*	*	*			
	101	*	*	*	*			4		AK RR	*	*	*			
	102	*		*	*			10		AK RR	*	*	*			
	103	*		*	*			0		AK RR	*	*	*			
	104	*		*				0		NSH-W	*	*	*			
	105	*	*		*			0		OSH-1	*		*			
	106	*	*	*	*			0		AK RR	*	*	*			
	107	*	*					0		OSH-1	*		*			
	108	*	*	*	*			0		AK RR	*	*	*			
	109	*	*	*	*			0		OSH-1	*		*			
	110	*	*	*	*			0		MFC	*		*			
	111	*		*				0		AK RR	*	*	*			
H	125	*	*	*	*			0		MFC	*	*	*			*
	126	*	*	*	*			0		MFC	*	*	*			*
I	135	*		*				20		N. of study area	*	*	*	*		
	136	*		*				20		N. of study area	*	*	*			
J	145	*						5		N. of study area	*	*	*	*		
	146	*						0		N. of study area	*	*	*	*		
	147	*						0		N. of study area	*	*	*	*		

TABLE V-1

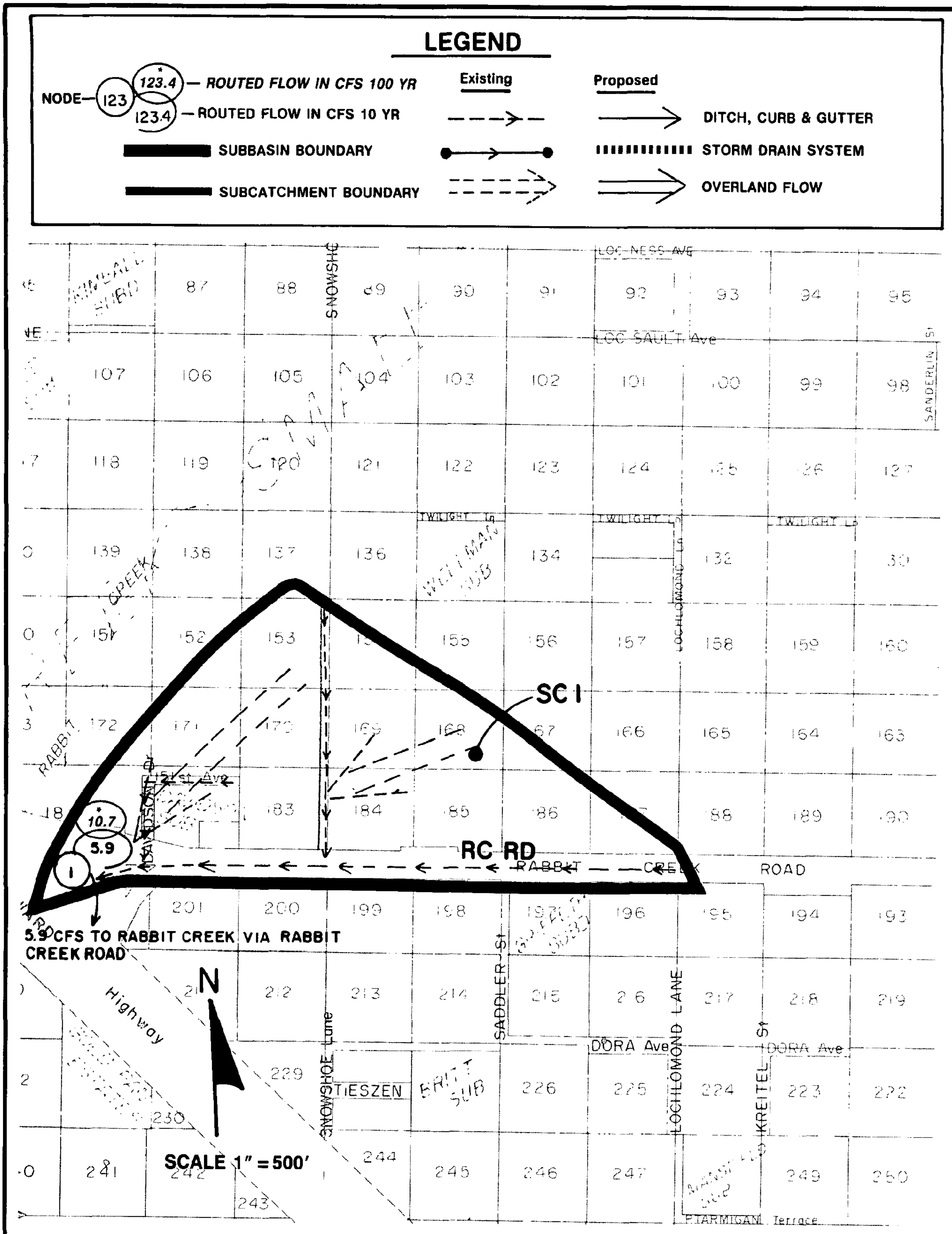
SUBBASIN ALTERNATIVE EVALUATION

SUBBASIN	SUBCATCHMENT	EXISTING COLLECTION SYSTEM: Overland flow	Curb & Gutter	Roadway ditch	Pipe system	Channelized flow	Greenbelt	FUTURE % DEPRESSION AND/OR WETLAND	PROBLEMS	TRUNK SYSTEM I.D.	ALTERNATIVES (RECOMMENDED ALTERNATIVE(S) CIRCLED): 1. No modifying action	2. Local detention & trunk system	3. Trunk system	4. Regional Detention	5. Corridor/green belt system	6. Diversion
K	160	*	*		*	*	*	0		LFC	*		*		*	
	161	*	*		*		*	0		LFC	*		*		*	
	162	*	*		*		*	0		LFC	*		*		*	
	163	*	*	*				0		LFC	*		*			
	164	*				*		5		LFC	*		*	*	*	
	165	*		*				12		LFC	*		*	*	*	
	166	*				*		14		LFC	*		*	*	*	
	167	*		*				5		LFC	*	*	*			
	168	*		*				8		Johns Rd.	*	*	*			
	169	*		*				0		Johns Rd.	*	*	*			
	170	*		*				0		LFC	*	*	*			
	185	*		*				0		LFC	*	*	*			
L	184	*		*				0		Turnagain Arm	*	*	*			*
	186	*		*				0		Turnagain Arm	*	*	*			
	187	*		*		*		80		Turnagain Arm	*	*	*	*		
	188	*		*		*		40		Turnagain Arm	*	*	*	*		
	189	*				*		16		Turnagain Arm	*	*	*	*		
M	200	*	*		*			0		Turnagain Arm	*		*			
	201	*	*		*			0		Turnagain Arm	*		*			
	202	*	*		*			2		Turnagain Arm	*		*			

SUBBASIN A

Subbasin A drains to Rabbit Creek. The various applicable alternatives evaluated are: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; and Alternative #3, trunk system. As shown on Table IV-1, the existing capacity of the ditch on the north side of Rabbit Creek is approximately 70 cfs. Both the existing and future flows projected for subbasin A are 5.9 cfs, which is much less than the ditch capacity.

It is recommended that Alternative #3, a trunk system be implemented throughout this subbasin. This system will be a minor drainage system and will resemble a local collection system. This system should be a series of roadside ditches and culverts designed to carry flows proportional to the area collected as identified on Table V-3 and ultimately discharging to Rabbit Creek via the ditch along the north side of Rabbit Creek Road. These collection systems should be constructed in conjunction with the expansion of the existing roadway network in the future to serve the ultimate growth of the area. There are no new major drainage structures recommended for this subbasin, as such the following map indicates only existing drainage systems and project flows from the area.



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SUBBASIN A

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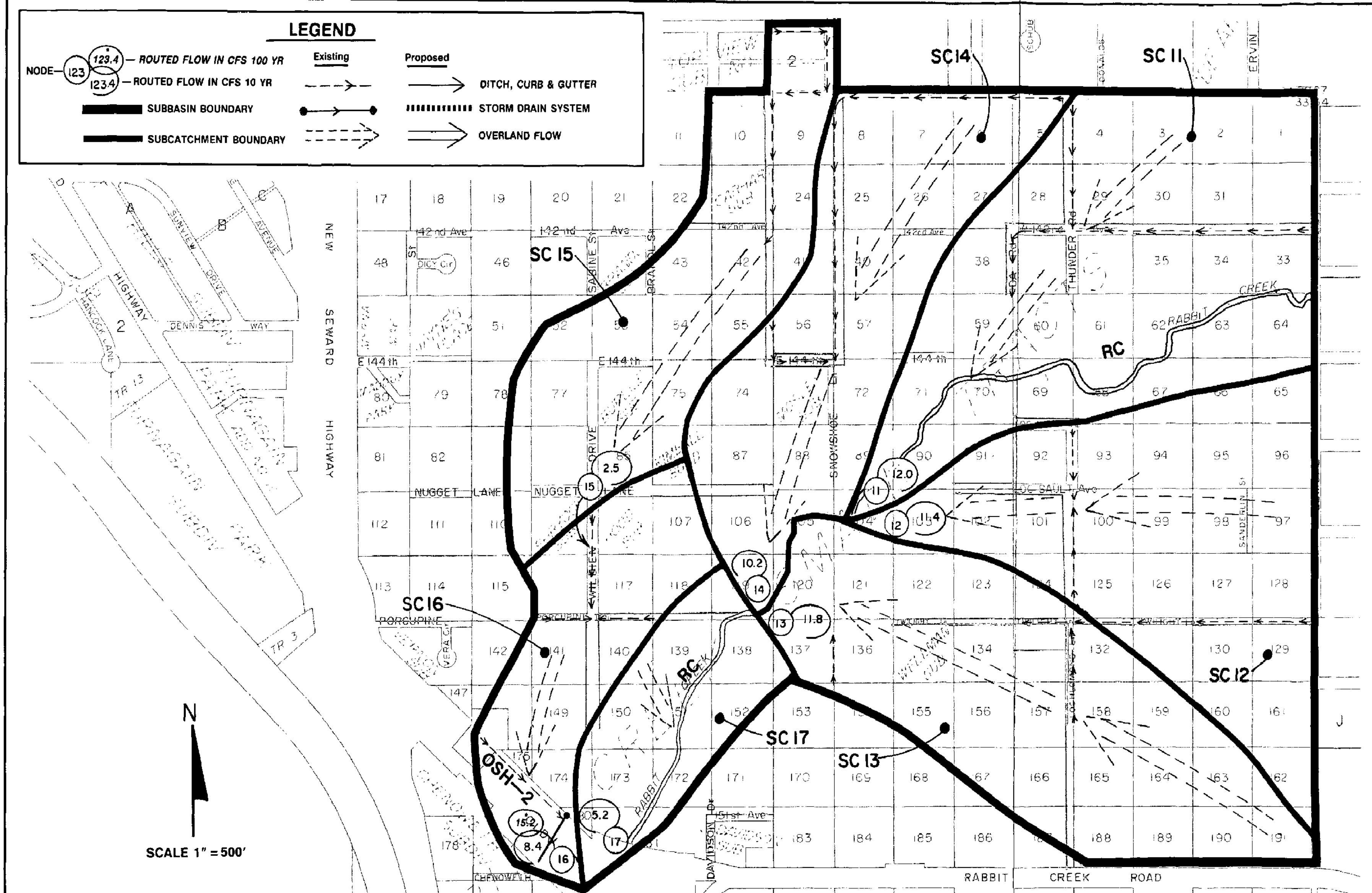
Figure **V-1**

SUBBASIN B

Subbasin B drains to Rabbit Creek. As shown on Table V-1, the various applicable alternatives evaluated in this subbasin are: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; and Alternative #3, a trunk system.

It is recommended that Alternative #3, a trunk system, be implemented throughout the subbasin. This system will be a minor drainage system and will resemble a series of local collection systems which individually, except for subcatchment #15, drain to Rabbit Creek and should be a series of roadside ditches and culverts. These collection systems should be constructed in conjunction with the expansion of the roadway network in the future and be designed to carry the flow associated with the area from which it is collected as identified in Table V-3. There are no new major drainage structures recommended for this subbasin, as such the following map indicates only existing drainage systems and project flows from the area.

The area to the east of the study area boundary, which presently drains into subbasin B, should be diverted away from subbasin B and into Rabbit Creek as a storm drainage network is developed in that area so as to not overload the proposed trunk/collection system.



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SUBBASIN B

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Figure V-2

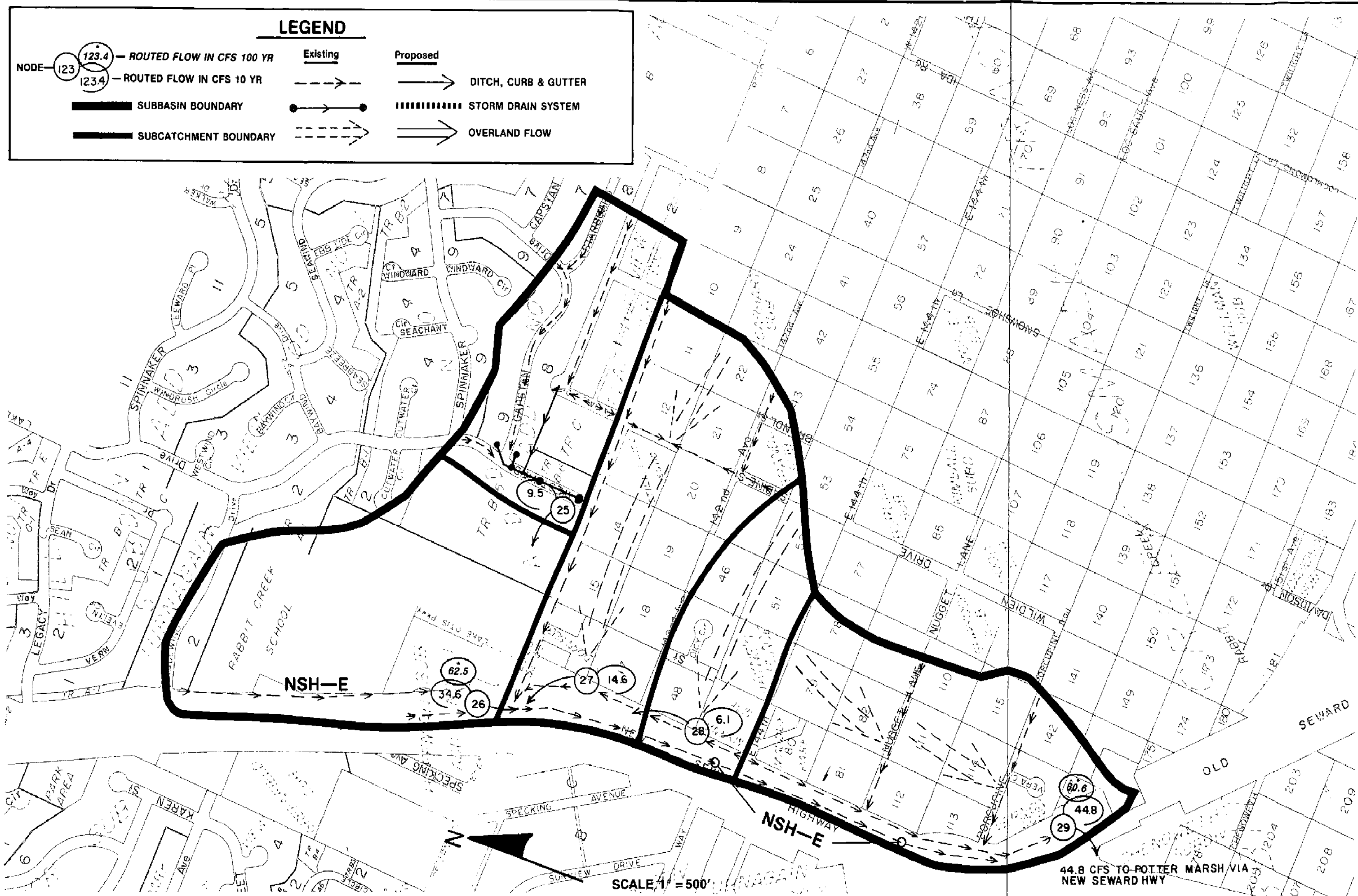
SUBBASIN C

Subbasin C drains to Potter Marsh. As shown on Table V-1, the various applicable alternatives evaluated in this subbasin are: Alternative #2, local detention and trunk system; Alternative #3, trunk system; and Alternative #6, diversion. This subbasin is presently experiencing problems with localized ponding, and as a result, the no modifying action alternative was not evaluated.

In the northeast and southwest areas of the subbasin, diversion of stormwater out of subbasin C into subbasin G was evaluated in order to decrease downstream stormwater flows. However, it was determined that storm drainage networks in this subbasin would not be overly impacted by existing or future flows. As such, no diversions were deemed necessary.

It is recommended that Alternative #3, a trunk system, be implemented throughout this subbasin. This trunk system should be either a series of roadside ditches and culverts or closed underground storm drainage networks, depending on the development pattern of the five subcatchments and should be designed to carry the flows as identified in Table V-3. The major trunk system for this subbasin is the existing stormwater drainage network along the Frontage Road and the drainage system of the New Seward Highway. Both of these trunk systems have existing capacity (Table IV-1) capable of handling the estimated future flows as shown on Table V-3. In conjunction with this trunk system, it is recommended that the upper portions of the subbasin implement a collection system designed to carry the proportionate amount of flows for the area to be served, as identified on Table V-3. This collection system should be constructed in conjunction with the expansion of the roadway network in the future and should consist of either a series of open ditches and culverts or closed underground storm drainage systems, depending on the ultimate development pattern of the area.

The State Department of Transportation and Public Facilities (DOT/PF) is presently designing improvements to the New Seward Highway for construction in the immediate future. It is recommended that DOT/PF be contacted to insure that adequate capacity in the inlet points are planned in the improvements to the New Seward Highway. The flows identified are for a 10-year design storm, and as such, DOT/PF should be contacted to identify what design storm is to be used for these systems, the 10-year or the 50-year design event.



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SUBBASIN C

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SUBBASIN D

Subbasin D forms the most southerly portions of the Upper Furrow Creek segment. As shown on Table V-1, the applicable alternatives evaluated in this subbasin were: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk system; Alternative #4, regional detention; Alternative #5, a corridor/greenbelt system; and Alternative #6, diversion.

It is recommended that a combination of Alternative #2, local detention and trunk system, and Alternative #3, trunk system, be implemented in subcatchments 45, 46, 47, and 48. The combination of these alternatives will form the upper portion of the trunk system UFC-S1. In subcatchments 49 and 50, Alternative #3, trunk system, and Alternative #4, regional detention, should be implemented. This trunk system is the middle portion of trunk system UFC-S1. The following paragraphs discuss each subcatchment in more detail.

In the easterly portion of the subbasin in subcatchment 45, the area east of the end of the Starboard Lane and Capstan Court, it is recommended that Alternative #2, local detention and trunk system, be implemented. This system will resemble a local collection system because of the drainage area involved and should consist of a series of roadside ditches and culverts or closed underground storm drainage network, depending on the ultimate development pattern of this subcatchment design to carry flows identified on Table V-3. This system will discharge to the UFC-S1 trunk and should be constructed in conjunction with the future expansion of the roadway network.

For the areas south of Legacy and west of DeArmoun Subdivision Addition No. 2, subcatchments 46, 47, and 48, it is recommended that Alternative #3, a trunk system, be implemented. This trunk system is identified as UFC-S1. The majority of this trunk system is already in place via the existing street and gutter system, pipe system, and open channel/greenbelt network. As identified on the subbasin map, there are portions of the existing system which are to become a part of the UFC-S1 trunk system which have inadequate capacity for the estimated future flows. These areas should be upgraded in the immediate future. In particular, along Spinnaker Drive, and Westwind Drive, where the greenbelt flows crosses the roadway through existing 18" culverts, it is recommended that these culverts be upsized in accordance with the flows as identified on Table V-3. Also, where the existing greenbelt discharges into a 24" closed culvert system along Tradewind Drive, there is inadequate capacity for future flows and this system should be increased either through the removal of the existing culvert and the installation of a culvert system designed for flows identified on Table V-3, or by paralleling the existing system with a second system designed to carry the additional capacity.

In the lower reaches of the subbasin, in subcatchments 49 and 50 between the Frontage Road and the New Seward Highway, it is recommended that Alternative #3, a trunk system, and Alternative #4, a regional detention pond, be implemented. This regional detention pond will dampen out the peak flows from Subbasin D prior to their confluence with the middle segment of Furrow Creek. This regional detention pond should be located between the Frontage Road and the New Seward Highway and be approximately sized in accordance with Table V-2. The Frontage Road along the New Seward Highway will be used as the lower UFC-S1 trunk system. The existing open ditch system does not have adequate capacity for estimated future flows as identified on Table V-3, and it is recommended the Frontage Road ditches cross-sectional area be increased to the carrying capacities as identified on Table V-3.

TABLE V-2

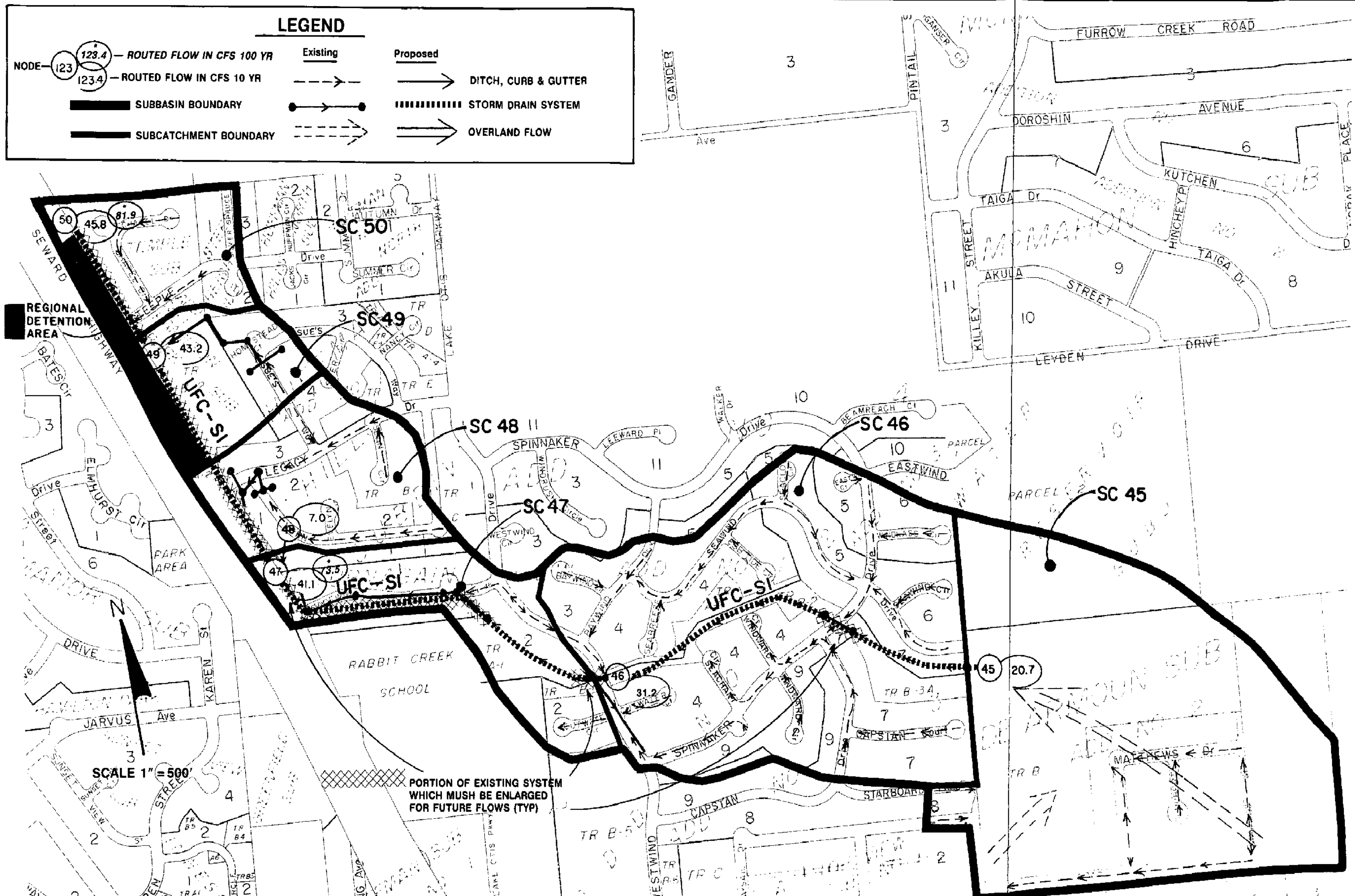
REGIONAL DETENTION VOLUME

1. Estimates for Subbasin D.

Subcatchment No.	Width (ft)	Length (ft)	Depth (ft)	Volume (ac-ft)
49	10	750	2.5	0.43
50	10	750	2.5	0.43
			Subbasin D Total Volume	0.86

2. Estimates for Subbasin F.

Subcatchment No.	Width (ft)	Length (ft)	Depth (ft)	Volume (ac-ft)
80	400	2600	2.5	59.75
81	450	2600	2.5	67.25
83	500	1850	2.5	53.00
84	450	1600	2.5	41.25
85	300	1220	2.5	21.00
86	500	1400	2.5	40.25
			Subbasin F Total Volume	282.50



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SUBBASIN D

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Figure V-4

SUBBASIN E

Subbasin E forms the middle portion of the Upper Furrow Creek segment. As shown on Table V-1, the various applicable alternatives evaluated were: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk system; Alternative #4, regional detention; and Alternative #5, a corridor/greenbelt system.

This subbasin is the largest contributor of flow to the Middle Furrow Creek segment. In this subbasin, Furrow Creek begins to fan out and as a result, it is recommended that four new main trunk/corridor systems should be implemented. These systems are:

1. The UFC-S4 trunk system in the northerly part of the subbasin. This trunk system drains the north portion of the subbasin, more or less paralleling Merganser Avenue. Because of the existing development pattern of this area, it is recommended that this trunk system be kept natural as an open greenbelt corridor as much as possible, following the natural path of this portion of the Upper Furrow Creek drainage.

2. The UFC-S3 trunk system which serves the existing subdivision in the south of the drainage basin and provides for a trunk system for the southeast corner of the developing basin.

This trunk system follows the existing storm drainage network to the greatest extent possible, but as a result of upstream development, the existing drainage network will require upgrading in the near future to handle the projected future flows.

3. The two above mentioned trunk systems, (UFC-S3 and UFC-S4), join in the northwest corner of the subbasin to form the UFC-S2 trunk system. This trunk system should be left in a open corridor/greenbelt state wherever possible and follow the natural drainage path of Furrow Creek.

4. The last trunk system in this subbasin is UFC-S1, which is a trunk system paralleling the Frontage Road draining from south to north, and originates in subbasin D.

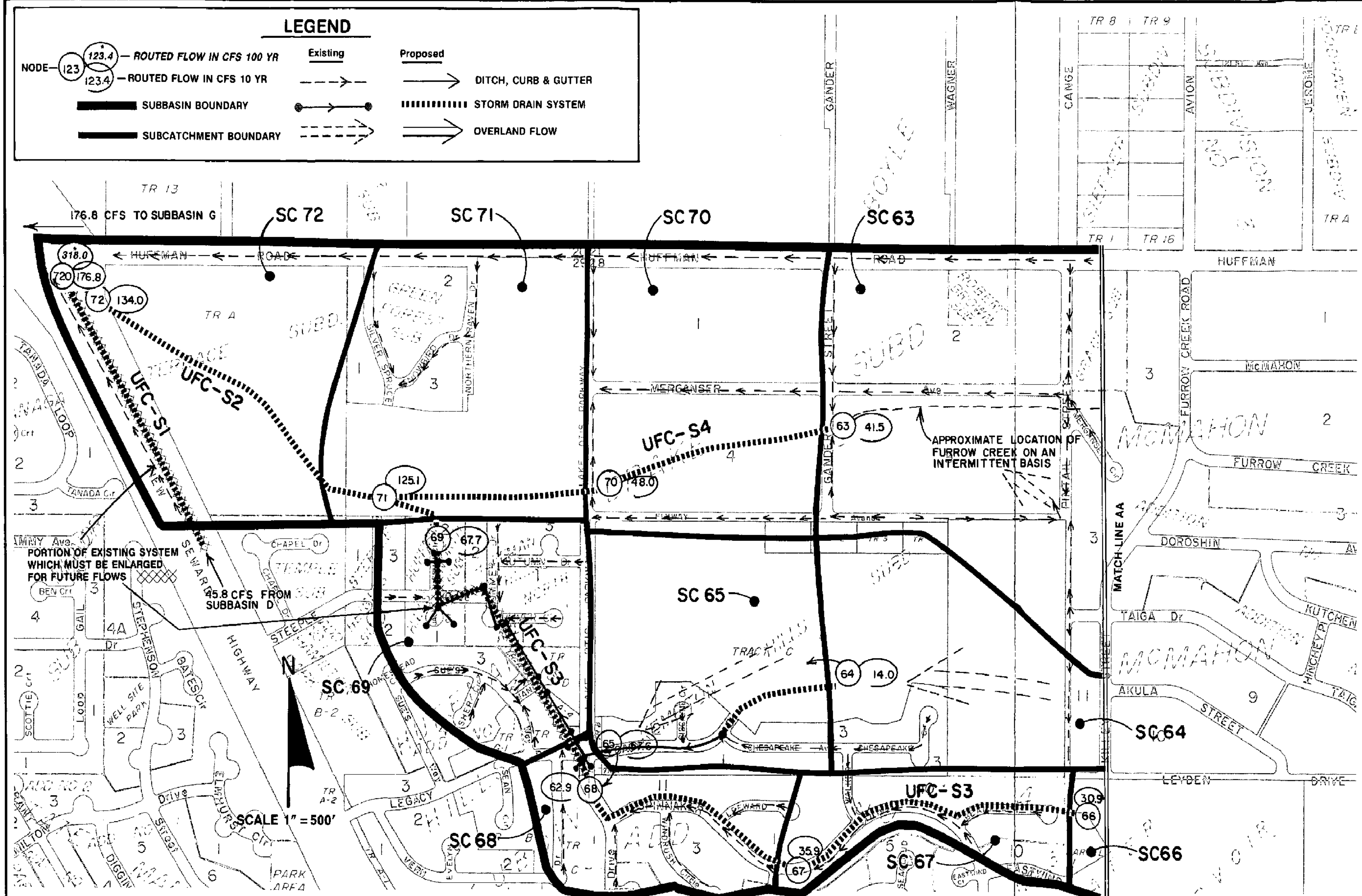
The following paragraphs discuss each of the subbasins and their particular requirements.

It is recommended in the southeasterly portions of the subbasin in subcatchments 60 and 66, the area bounded on the north by Leyden Drive, on the west by the extention of Pintail Street, on the east by the study area boundary, and on the south by Subbasin D, that Alternative #2, local detention and trunk system, be implemented. This system should be a trunk/collection system feeding trunk system UFC-S3, and designed to carry the flows as identified in Table V-3. This system should be constructed in conjunction with the expansion of the roadway network in the area and should consist of roadside ditches and culverts or closed underground storm drainage network, depending on the ultimate development pattern of this area.

It is recommended that in the middle portions of the subbasin, in subcatchments 64 and 65, the area bounded on the north by Flyway Avenue, on the south by Leyden Drive, on the east by Pintail Drive, and on the west by Lake Otis Parkway, that Alternative #2, a local detention and trunk/collection system feeding trunk system UFC-S3, should be implemented. This system should be constructed in conjunction with the expansion of the roadway network in the future and should be sized to accommodate the flows as identified in Table V-3.

For the middle segment of the northern portions of the subbasin in subcatchments 70 and 71, the areas east of Silver Spruce Drive, north of Flyway Avenue, west of the Gander Street, and south of Huffman Road, it is recommended that Alternatives #3, trunk system, and Alternative #5, corridor/greenbelt system, be implemented. This trunk system is identified as trunk UFC-S4, and should be designed to carry the flows identified on Table V-3. The routing of this system should follow the natural drainage path of Furrow creek to the maximum extent possible in an open channel/greenbelt type system which maximizes the recreational/aesthetic potential of Furrow Creek. In this portion of the subbasin is the lower part of trunk system, UFC-S3, which originates in subcatchment 69, and it is recommended that this trunk be a open-channel/greenbelt system also and join UFC-S4 in subcatchment 71. The combination of these two trunk systems becomes the trunk system UFC-S2.

In the northwesterly portion of this subbasin in subcatchment 72, the area bounded by Huffman Road on the north, Flyway Avenue on the south, Spruce Drive on the east and the New Seward Highway on the west, it is recommended that Alternative #3, a trunk system and Alternative #5, a corridor/greenbelt system, be implemented. There are two trunk systems in this subcatchment, UFC-S1 and UFC-S2. The trunk system identified as UFC-S2, should be designed to carry the flows as identified in Table V-3, and should follow the natural drainage path of Furrow Creek to the maximum extent possible in an open channel/greenbelt system which maximizes the recreational/aesthetic potential of the creek. Also in this subcatchment is the lower portion of trunk system UFC-S1, which originates in Subbasin D. This system (UFC-S1) should parallel the Frontage Road as an open ditch and join the UFC-S2 system at the northwest corner of subcatchment 72. Presently, the carrying capacity of UFC-S1 in subcatchment 72 is inadequate for the projected flows orginating in Subbasin D, and it is recommended at the time of improvements to the Frontage Road, that modifications to this open ditch be made in order to increase the capacities to those identified on Table V-3.



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SUBBASIN E
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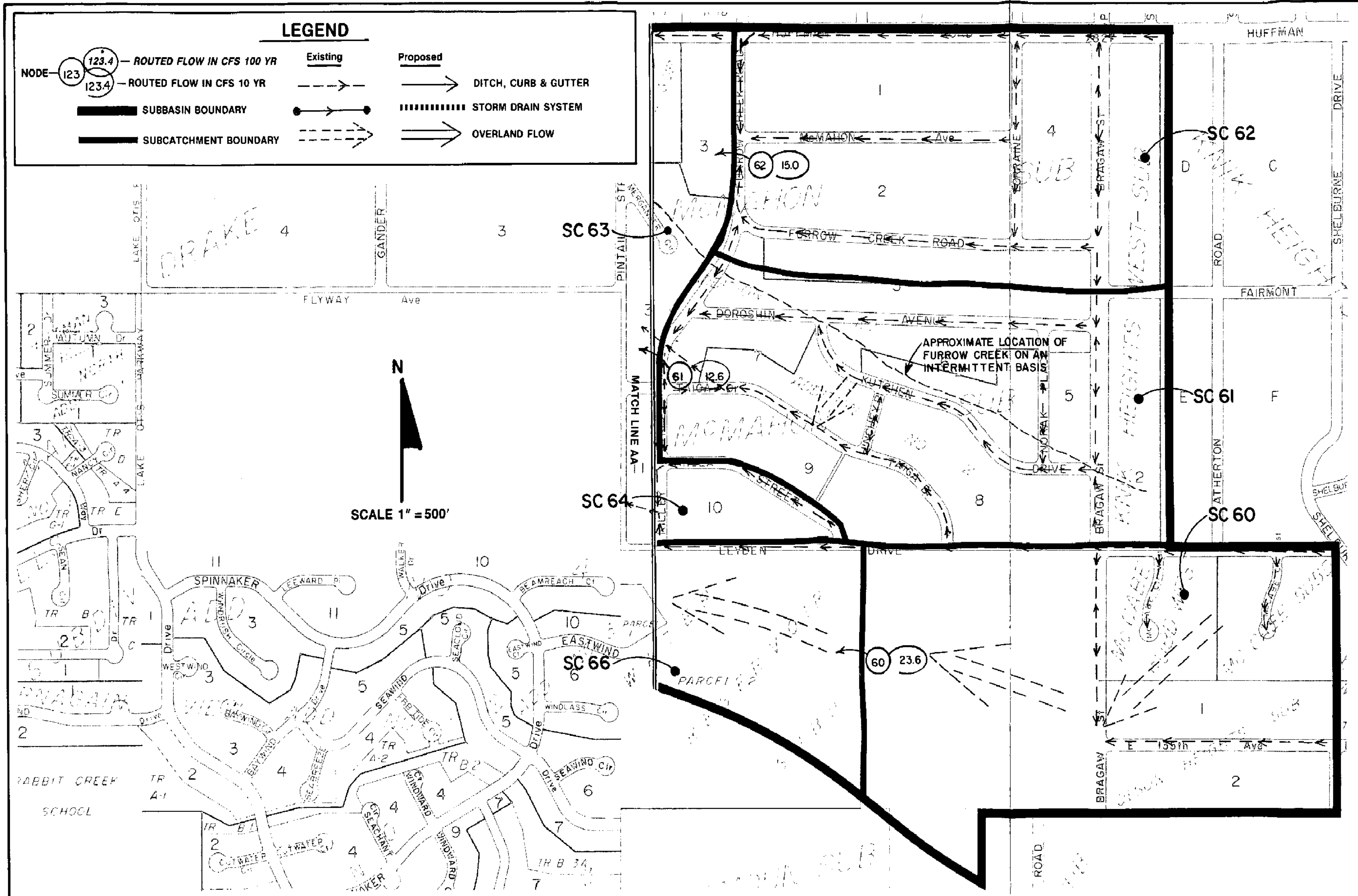
Figure **V-5**



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SUBBASIN E
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Figure V-5

SUBBASIN F

Subbasin F drains the northerly portion of the Upper Furrow Creek segment. As shown on Table V-1, the applicable alternatives evaluated were: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk system; Alternative #4, regional detention; alternative #5, corridor/greenbelt and Alternative #6, diversion.

In Subbasin F, there exists a large wetland area along the westerly side of the subbasin, more or less paralleling the New Seward Highway. All of the subcatchments in Subbasin F drain to this wetland <u>except</u> subcatchments 75, 77, 78, and 79 and portions of 80. The <u>Wetlands Management Plan</u> classifies this wetlands area as being under "conservation" status. Under this classification the area is to be managed in such a way as "to conserve the natural function and values to the maximum extent practicable while permitting uses to occur on wetland fringes and less critical wetlands areas". In the development of the fringes and less critical areas, it is paramount that enough of the wetland be preserved to handle the storm runoff. Table V-2 lists the runoff volume required.

There are three major trunk systems identified which are the future storm drainage network in Subbasin F. These trunk systems are: 1) the UFC-N1 trunk system which drains subcatchments 82 and 83 to the wetlands area; 2) the UFC-N2 trunk system which drains subcatchments 76 and 85 to the wetland area; and 3) the UFC-N3 trunk system which drains subcatchment 75, 77, 78, 79 and portions of 80 to Huffman Road. This trunk system ultimately discharges to the Middle Furrow Creek segment at the southwest corner of the subbasin.

In the northeast portions of the subbasin are subcatchments 82 and 83 which comprise the area bounded on the north by O'Malley, on the east by the study area boundary, on the south by approximately 112th Avenue, and on the west by wetlands. It is recommended that Alternative #3, a trunk system, be implemented throughout both subcatchments with the exception of the westerly portion of subcatchment 83 where it is recommended that Alternative #4, regional detention, be implemented. This trunk system is identified on the map as UFC-N1 and should be designed for the flows as shown on Table V-3. This system should be constructed in conjunction with the expansion of roadway network in the future and should consist of roadside ditches and culverts or closed underground storm drainage networks, depending upon the ultimate development pattern of this area. The ultimate discharge of UFC-N1 will be to the wetland. It has been estimated that the volume of flows associated with UFC-N1 will be absorbed into the wetland area (See Table V-2) and there will be no ultimate discharge from this trunk system into the Middle Furrow Creek segment.

In the north central portions of the subbasin are located subcatchments 76 and 85 which comprise the area bounded on the south by Klatt Road, on the north by an extension of 112th Avenue, on the east by the study area boundaries and on the west by a wetland area. It is recommended that Alternative #3, a trunk system, be implemented. This trunk system is identified as UFC-N2 and should be designed for the flows as identified on Table V-3, ultimately discharging into the wetland area. It is estimated that the volume of flows associated with UFC-N2 will be absorbed into the wetland area (See Table V-2) and there will be no ultimate discharge from this trunk system into the Middle Furrow Creek segment. This trunk system should be either a series of roadside ditches and culverts or a closed underground storm drainage system paralleling Klatt Road, depending upon the development pattern in the future. It is also recommended that in the westerly portions of subcatchment 85, that Alternative #4, regional detention pond, be implemented in the wetland area.

In the southerly portions of the subbasin are subcatchments 75, 77, 78, and 79, which are bounded on the east by the subbasin boundaries, on the north by Klatt Road, on the south by Huffman Road, and on the west by a wetland area. It is recommended that in the easterly sections of this area (subcatchment 75) that Alternative #2, local detention and trunk system, be implemented which consists of a collection system designed for the flows as identified in Table V-3, discharging to the upper portions of the UFC-N3 trunk system. For the remaining portions of the subbasin, (subcatchments 77, 78, and 79), it is recommended that Alternative #3, a trunk system be implemented. This trunk system should be designed to carry the flows as identified on Table V-3.

Though there presently exists a local collection system along the UFC-N3 proposed trunk routing, this system does not have the carrying capacity for the estimated future flows of the area. However, it is recommended that no action be taken at this time with respect to the replacement of the identified undersized pipes until the projected flow resulting from increased trunk/ collection systems in the contributing area exceeds the existing capacity, or at the time of street improvements to Gregory and Rainbow Avenues, which ever occurs first. In the routing of this trunk system which is identified as UFC-N3 on the subbasin map, for the areas east of Rainbow Avenue and west of Gertrude Street, more or less following Cleo Avenue to the east, there is a potential conflict. On the subbasin map, a suggested routing for this trunk system (UFC-N3) is shown but the selected route must be verified during a final design procedure. This potential conflict results from existing development in the area which is located on the proposed Cleo Avenue. It is suggested that this trunk system be a series of roadside ditches and culverts or underground storm drainage network, depending on the ultimate development pattern of the land.

In the westerly section of this subbasin, there exists a large wetland area, approximately paralleling the New Seward Highway between Huffman Road and O'Malley Road. Within this area exist portions of subcatchments 80, 81, 83, 84, 85, and 86. It is recommended for these areas that Alternative #4, regional detention, be implemented as shown on the subbasin map. For areas outside of the identified wetland area but within these subcatchment boundaries, a localized collection system consisting of roadside ditches should be implemented for the flows identified on Table V-3 draining to this wetland area. The volume of runoff discharging into this wetland area from these subcatchments (80, 81, 84, 85, and 86) and from trunk systems UFC-N1 and UFC-N2 is 24.2 Ac-ft and is shown on Table V-2. As such, the outlet point (node 81) as identified on the subbasin map has no contribution to the middle segment of Furrow Creek. All storm water flow will be assimulated and will either be lost through evapotranspiration or local ground water recharge.

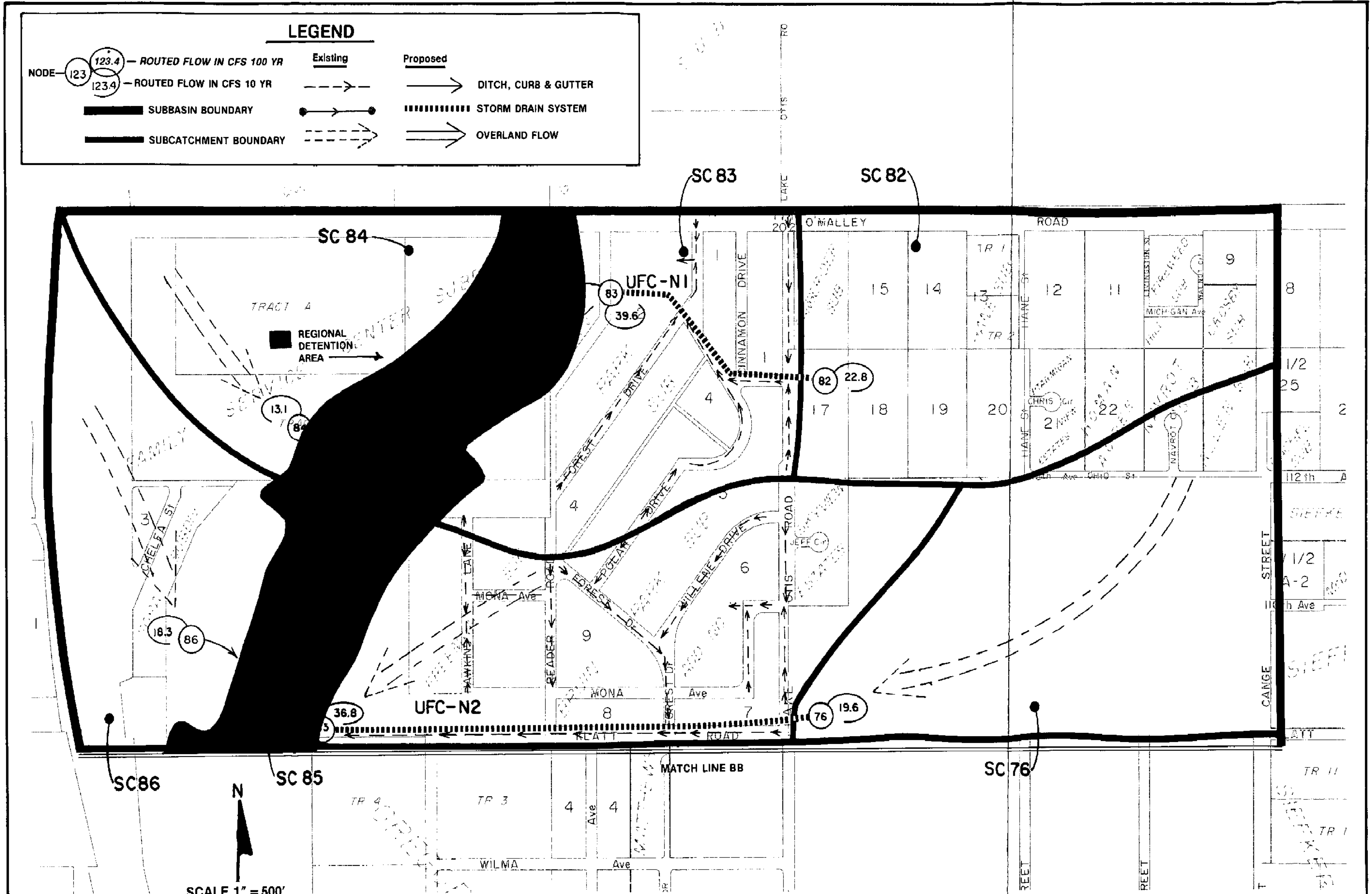

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Figure **V-6**

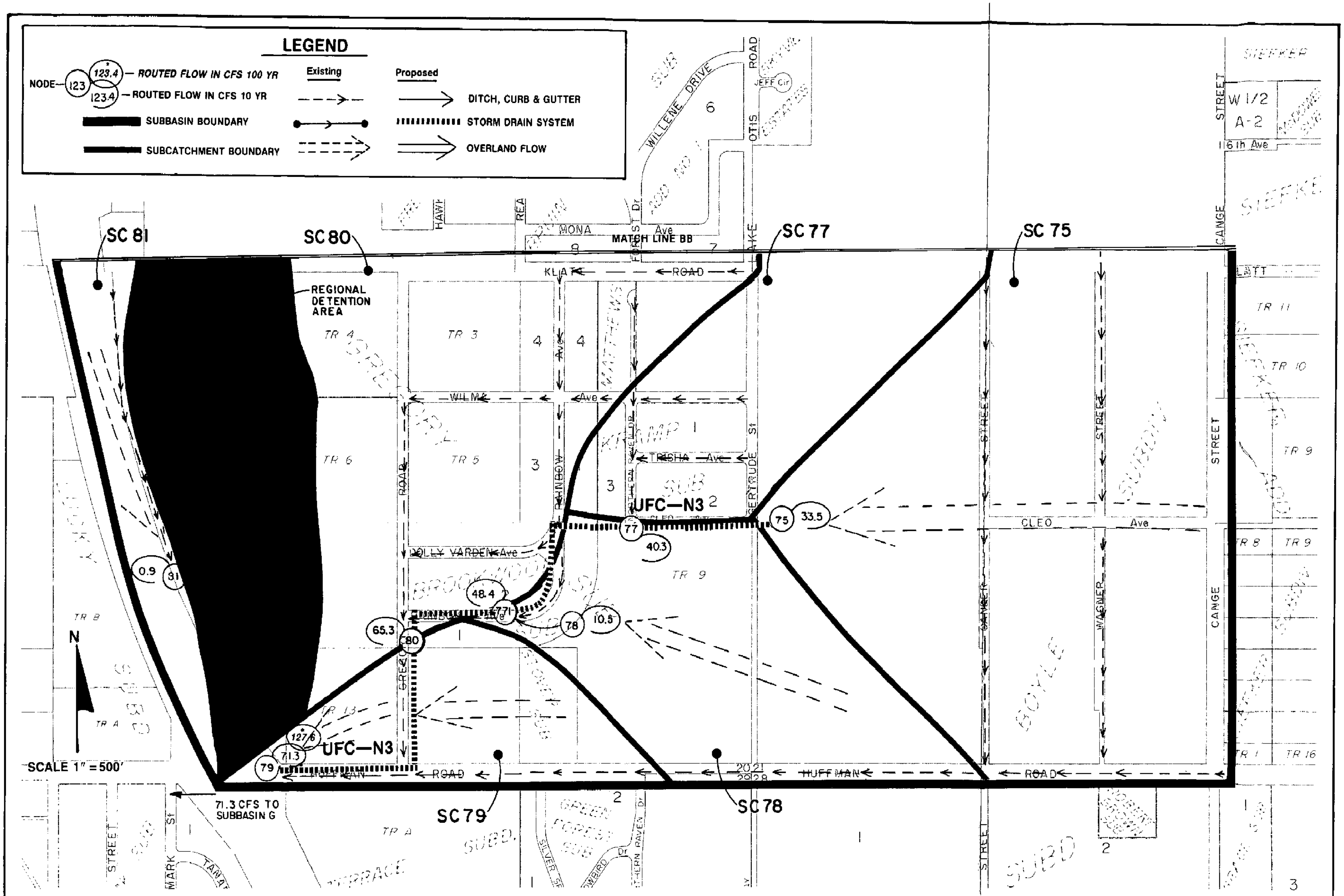

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Figure **V-6**

SUBBASIN G

Subbasin G drains to the Middle Furrow Creek segment in the north, to Turnagain Arm in the westerly portions and to the south along the Old Seward Highway (OSH). As shown on Table V-1, the various alternative evaluated in this subbasin are: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; and Alternative #3, trunk system.

It is recommended that for all but the northerly portions of Subbasin G (subcatchments 100-109 and 111), that Alternative #1, no modifying action be implemented. The present system, which consists of a street and gutter drainage network, roadway ditches, and a pipe system along the major highway corridors, collector streets and the Alaska Railroad tracks are adequate for the projected storm water flows.

It is recommended that in the area along Huffman Road between the New Seward and Old Seward Highways in subcatchment 110, that Alternative #3, a trunk system, be implemented. The projected flows from the Upper Furrow Creek drainage system indicate that the present 36" to 48" closed underground storm drainage system along Huffman Road between the Old and New Seward Highways is inadequate for both existing and future flows. It is recommended that a preliminary engineering report be initiated to identify methods to carry the flows as identified in Table V-3. Three routing options which should be looked at in this analysis are: 1) using open channels along Huffman Road; 2) using the now non-existant old drainage channel of Furrow Creek located approximately 300 to 500 ft. south of Huffman Road; and 3) an underground storm drainage system similar to the present system. In the analysis of capacity for these alternatives it is suggested that both a complete new system and a system which will carry the excess flows during times of high stormwater runoff be analyzed.

The flows from the Middle Furrow Creek segment are discharged into the Lower Furrow Creek segment through an existing 36" culvert located under the Alaska Railroad (ARR) tracks approximately 150 ft. south-southwest of the intersection of Huffman Road and the Old Seward Highway. Presently this land is undeveloped and in times of high runoff a large pond exists between the intersection and this culvert. In the future, however, development pressures will increase and this parcel will be filled and developed. The present carrying capacity of the 36" ARR cross culvert is estimated to be 40 cfs, as shown in Table IV-1, versus the estimated required carrying capacity of 317 cfs as shown in Table V-3. It is recommended that at the time this parcel of property is developed that a storm drainage network capable of handling the projected flows be constructed from the storm drainage system located immediately south of the intersection of Huffman and the Old Seward Highway to a series of culverts located beneath the ARR tracks. Because of the critical nature of this drainage system, it is recommended that special precautions for the prevention of icings (as discussed in Appendix B) be implemented during the design process to insure that the culverts under the ARR are capable of allowing runoff to be discharged to the Lower Furrow Creek segment at all times.

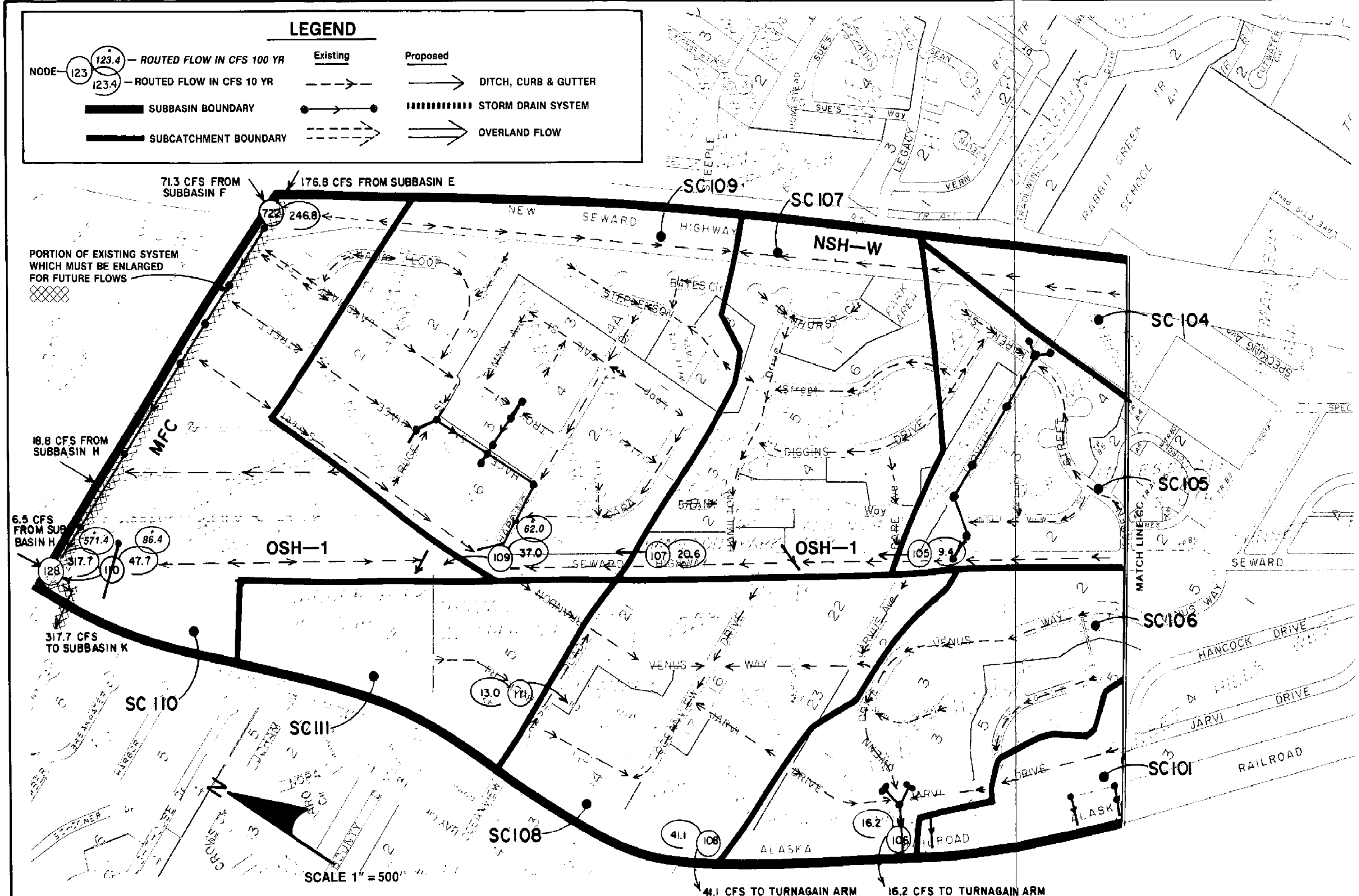

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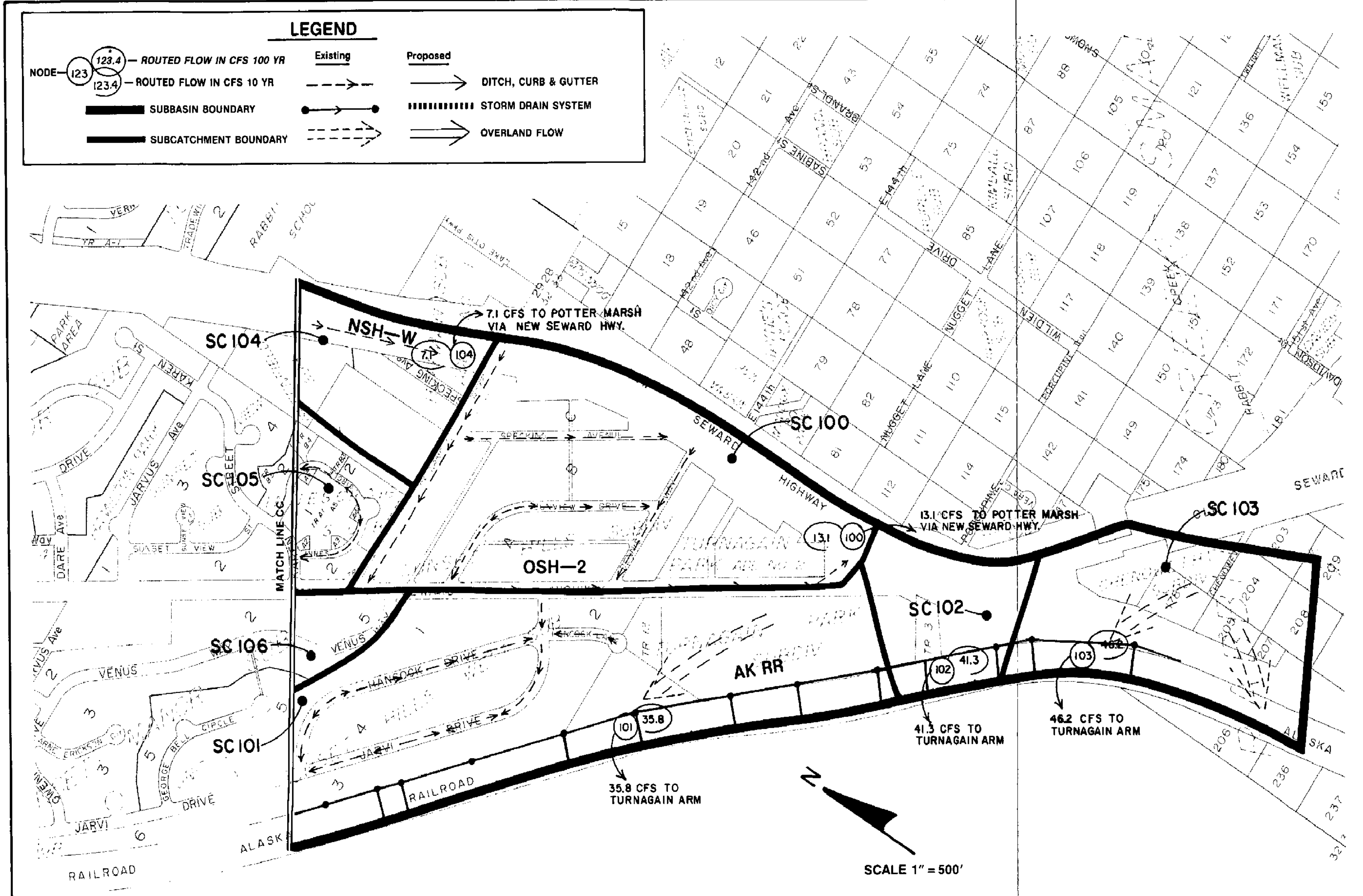
Figure V-7



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SUBBASIN G
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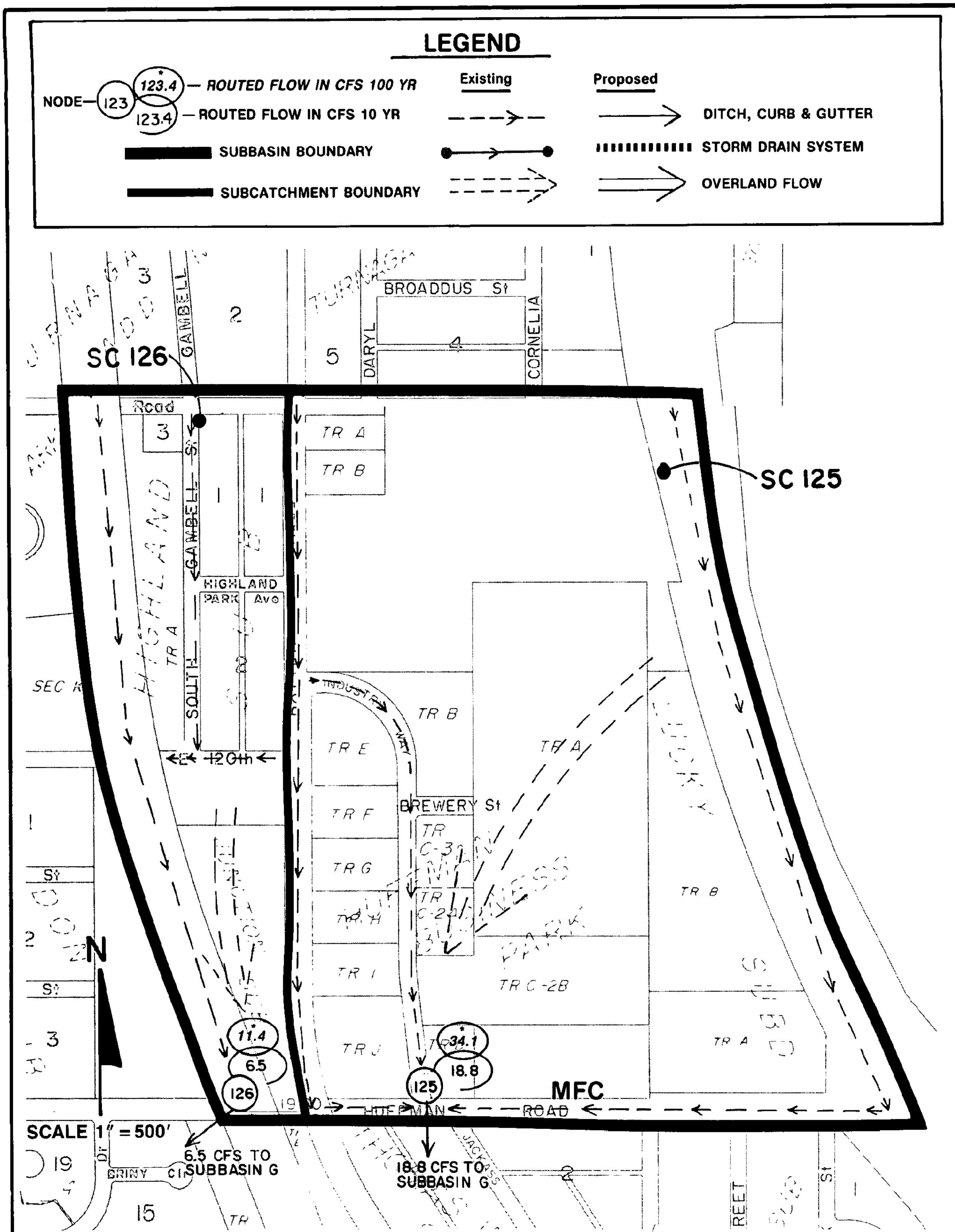
Figure V-7

SUBBASIN H

Subbasin H drains the northerly part of the Middle Furrow Creek segment to Furrow Creek along Huffman Road. The alternatives evaluated for this subbasin as shown on Table V-1 are: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk sytem; and Alternative #6, diversion.

It is recommended for this subbasin that Alternative #3, a trunk system, be implemented. This system, because of the drainage area involved and the existing roadway ditch system, will be a minor drainage system and will resemble a local collection system and should be a series of roadside ditches and culverts designed to carry the flows identified in Table V-3. These collection systems should be constructed in conjunction with the expansion of the existing roadway network in the future to serve the ultimate growth of the area. There are no new major drainage structures recommended for this subbasin; as such the following map indicates only existing drainage systems and projected flows from the area.



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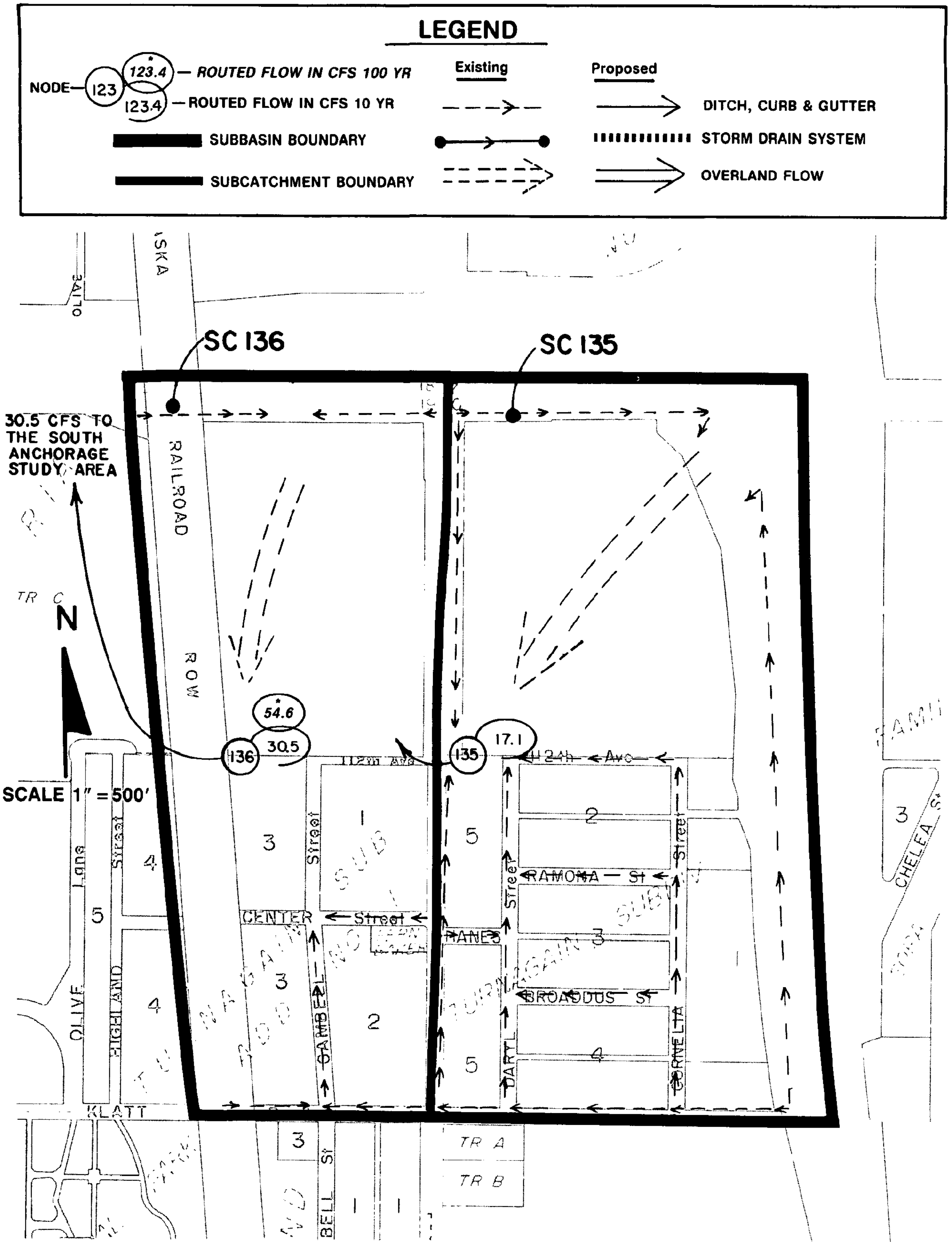
SUBBASIN H

February 1983 Figure **V-8**

SUBBASIN I

Subbasin I drains north, out of the study area. As shown in Table V-1 the various alternatives evaluated in this subbasin were: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk system; and Alternative #4, regional detention.

It is recommended that Alternative #3, a trunk system, be implemented throughout the subbasin. This system, because of the drainage area involved, will be a minor drainage system and will resemble a local collection system and should be a series of roadside ditches and culverts designed to carry the flows identified in Table V-3. These collection systems should be constructed in conjunction with the expansion of the existing roadway network in the future to serve the ultimate growth of the area. The South Anchorage Drainage Study should be consulted prior to design on any storm drainage facilities in this area to determine the proper routing of flows from this subbasin into the South Anchorage study area.



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SUBBASIN I

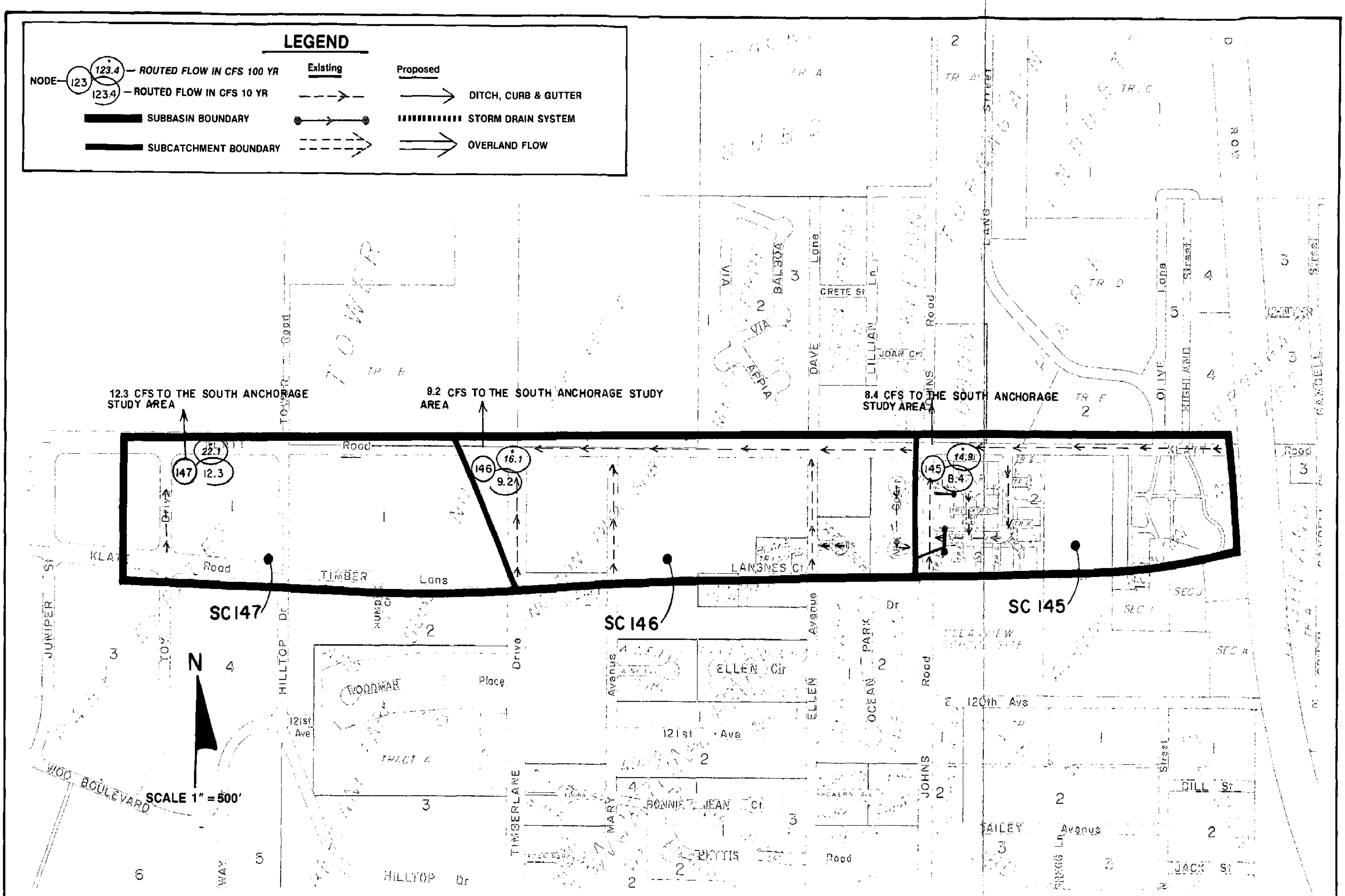
February 1983 Figure **V-9**

SUBBASIN J

Subbasin J drains north, out of the study area. As shown in Table V-1 the various alternatives evaluated in this subbasin were: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk system; and Alternative #4, regional detention.

It is recommended that Alternative #3, a trunk system, be implemented throughout the subbasin. This system, because of the drainage area involved, will be a minor drainage system and will resemble a local collection system and should be a series of roadside ditches and culverts designed to carry the flows identified on Table V-3. These collection systems should be constructed in conjunction with the expansion of the roadway network and urbanization in the area to serve the ultimate growth of the area. There are no new major drainage structures recommended for this subbasin. As such the following map indicates only existing drainage systems and projected flows from the area.

The South Anchorage Drainage Study should be consulted prior to design on any storm drainage facilities in this area to determine the proper routing of flows from this subbasin into the South Anchorage study area.



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SUBBASIN J

February 1983 Figure **V-10**

SUBBASIN K

Subbasin K forms the Lower Furrow Creek drainage segment. As shown in Table V-1, the various alternatives evaluated in this subbasin are: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk system; Alternative #4, regional detention; and Alternative #5, a corridor/greenbelt system.

This subbasin is the most complex basin from a stormwater management/plan point of view in this study. There are basically three types of systems within this subbasin: 1) the existing system of overland flow in undeveloped areas, 2) the existing storm drainage trunk and collection system which discharges to the Lower Furrow Creek segment, and, 3) the Lower Furrow Creek segment. In general, the recommendations for this subbasin are:

1. In areas of existing overland flow, a trunk or collection system should be implemented to collect and discharge the flows to the Lower Furrow Creek segment.

2. In areas where there is an existing trunk/collection systems discharging to the Lower Furrow Creek segment, no modifying action shall be taken, as these systems are adequate to carry the projected flows.

3. An enlargement of the corridor/greenbelt system to increase the channel carrying capacity of Lower Furrow Creek for the projected flows and the installation of suitable structures at major intersection crossings along Lower Furrow Creek capable of handling the projected flows. The following paragraphs discuss each of the subbasins and their particular requirements.

The main corridor of the Lower Furrow Creek segment is contained in subcatchments 160, 162, 164, and 166. It is recommended that in these subcatchments that Alternative #3, a trunk system, and Alternative #5, a corridor/greenbelt system, be implemented. The majority of this system is already in place. Presently, a corridor/greenbelt for Lower Furrow Creek exists from the outlet of Furrow Creek to the Turnagain Arm located in Johns Park to approximately the crossing of the creek at Clipper Ship Court located in subcatchment 160. It is recommended for this portion of the Lower Furrow Creek segment, that the existing channel capacity be expanded to carry the flows as identified in Table V-3, for areas identified on the subbasin map with inadequate capacity for future flows. Particular critical areas requiring upgrading are the Furrow Creek crossings of Johns Road, Mariner Drive and Clipper Ship Court. Modifications to these creek crossings should be implemented to increase their carrying capacity to the flows identified in Table V-3. Presently these crossings consist of 18" - 36" diameter corrugated metal pipe. Because of the nature of this corridor/greenbelt it is suggested that either a bridge or a plate arch culvert be constructed to increase the carrying capacity while simultaneously maximizing the aesthetic potential of this creek corridor. In addition with the use of these types of stream crossings, there will be minimal requirements for maintenance as related to icings, and there will be less potential for property damage resulting from flooding at these intersections during times of high volume runoff in the spring.

In the upper reaches of Lower Furrow Creek from the Alaska Railroad to approximately Clipper Ship Court, located in subcatchment 160, the existing channel of Furrow Creek does not have adequate capacity for existing or future flows, and the carrying capacity of the corridor/greenbelt for this area needs to be increased. Additionally, there is a recent fill on the south side of this portion of the stream segment which is encroaching upon Furrow Creek without adequate side slope soil stabillization. It is recommended that the side slopes of this fill area be stabilized immediately and that considerations of how this fill will affect the corridor/greenbelt nature of this creek segment be analyzed when improvements to the creek corridor are made.

In conjunction with the channel modifications to Furrow Creek, it is recommended that design procedures be used which will allow for the base flow of Furrow Creek to be channelized. Also within these subcatchments, in particular subcatchments 164 and 166, there exists a large future open space/park area, and for these areas it is recommended that Alternative #1, no modifying action, be implemented in order to maintain the natural characteristics of these areas to the maximum extent possible.

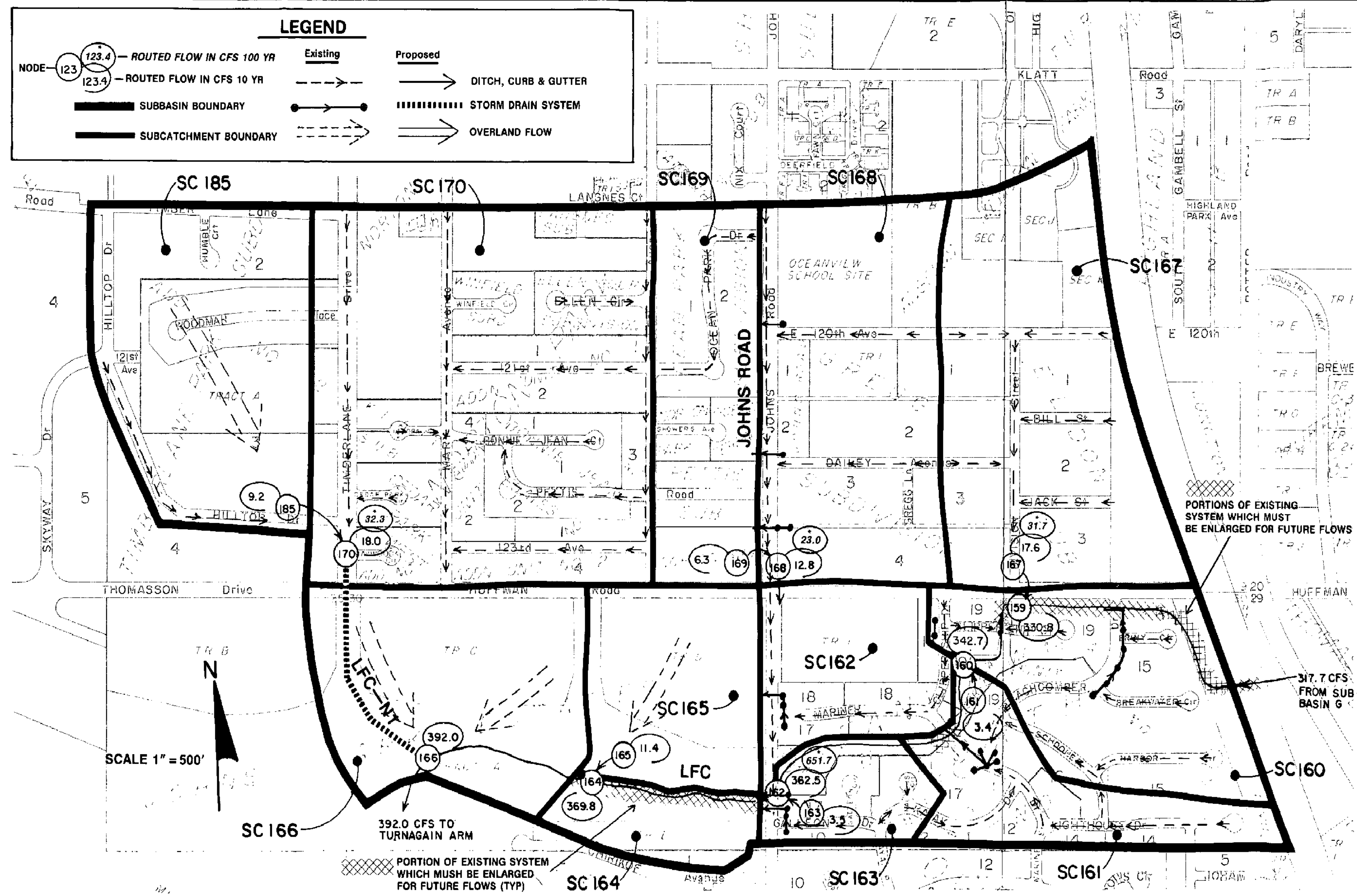
In the southeast section of this subbasin is subcatchment 167 which is bounded on the west by approximately Gregg Lane, on the north by the study area boundary and on the west by the Alaska Railroad, it is recommended that Alternative #3, a trunk system, be implemented for subcatchment 167. This system, because of the drainage area involved, will be sized as a local collection system and should be a series of roadside ditches and culverts designed to carry the flows identified in Table V-3. This system should be constructed in conjunction with the expansion of the existing roadway network in the future to serve the ultimate growth of the area, the system should discharge to Furrow Creek in the vicinity of Division Street.

In the northcentral portions of this subbasin, are subcatchments 168 and 169 which are bounded approximately on the east by Gregg Lane, on the south by Huffman Road, on the north by the study area boundary and on the west by Ellen Avenue. Alternative #1, no modifying action, is recommended for this area. Presently, these two subcatchments are in a developing state, and the existing trunk system for these subcatchments is an open ditch/ culvert system along Johns Road. As shown in Table V-3 and Table IV-1, the capacity of this system is adequate to handle projected flows.

In the northwest portions of the subbasin, the boundaries of Ellen Street on the east, Hilltop Drive on the west and south, and the study area boundary on the north identify subcatchments 170 and 185. It is recommended that Alternative #3, a trunk system, be implemented for this area. This system, because of the drainage area involved, will resemble a local collection system and should be a series of roadside ditches and culverts designed to carry the flows as identified in Table V-3. This system should be constructed in conjunction with the expansion of the existing roadway network in the future to serve the ultimate growth of the area and discharge to trunk system LFC-N1 at approximately the intersection of Timberlane Drive and Huffman Road as shown on Figure V-II. The LFC-N1 trunk system should be a closed, underground pipe corridor to its confluence with Furrow Creek, located approximately where Furrow Creek discharges to Turnagain Arm in subcatchment 166. To preserve the park/open space area located in 166 which the LFC trunk system traverses, it is recommended that no inlet points to this trunk system be planned.

Approximately in the center of the subbasin is subcatchment 165. For this subcatchment it is recommended that Alternative #1, no modifying action, be implemented. The discharge from subcatchments 168 and 169 flow through subcatchment 165 along Johns Road and the existing storm drainage network along Johns Road has adequate carrying capacity for the future volume of flows as indicated in Table V-3.

In the southeast portions of this subbasin are subcatchments 161 and 163. It is recommended that Alternative #1, no modifying action, be implemented in this area. Presently, the existing storm drainage network in these two subcatchments can handle the estimated future flows for the area involved.



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SUBBASIN K

February 1983 Figure **V-11**

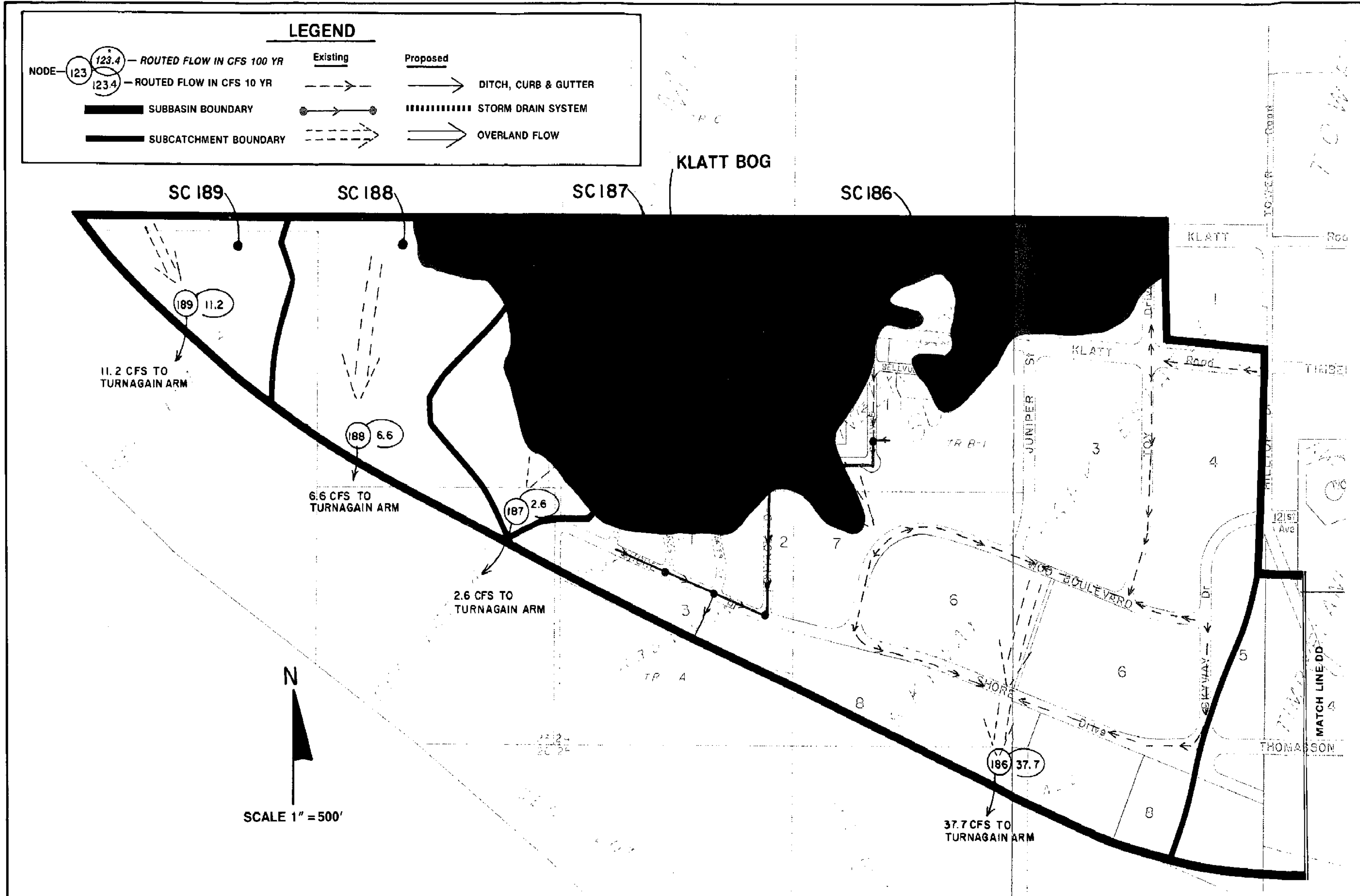
SUBBASIN L

Subbasin L drains to Turnagain Arm. As shown in Table V-1, the various alternatives evaluated are: Alternative #1, no modifying action; Alternative #2, local detention and trunk system; Alternative #3, trunk system; and Alternative #4, regional detention.

It is recommended that in the westerly portions of the subbasin (subcatchments 187, 188 and 189), that Alternative #1, no modifying action, and Alternative #3, a trunk system, be implemented. In portions of these subcatchments there are major wetlands designated to be conserved. Development should be such that it does not alter the drainage of the area. Therefore, in these areas no modifying action should be implemented. For the remaining area located in these three subcatchments, Alternative #3, a trunk system, should be implemented. This trunk system will be a minor drainage system and will resemble a collection system because of the drainage area involved. These collection systems should be sized for the flows as identified in Table V-3. Also in this area, there are two major open channel drainage corridors, which presently drain portions of Klatt Bog located north of Klatt Road. An analysis of these contributing flows should be conducted and these flows should become additive to the flows identified in Table V-3, in the design of future systems for these subcatchments. The flows in Table V-3 for those subcatchments which contain the designated wetland area, do not contain any flow routed from the wetland. Also, the design criteria of future drainage systems for the areas located in the near proximity of the wetland should contain considerations which insure these wetlands are not drained, thus insuring the future of this ecosystem.

It is recommended that Alternative #3, a trunk system, be implemented for the remaining subcatchments, subcatchments 184 and 186. This system will resemble a series of local collection facilities which drain to Turnagain Arm and should be a series of roadside ditches and culverts or a closed stormwater drainage network, depending upon the development patterns of the sub-catchments. This system should be constructed in conjunction with the expansion of the roadway network in the future and should be designed to carry the flows as identified in Table V-3.

The Wetlands Management Plan assigns the classification of "conservation" to the wetlands in Subbasin L. Under this designation the wetlands would be managed in such a way as to conserve the natural function and values to the maximum extent practicable while permitting uses to occur on wetland fringes and less critical wetland areas. It is imperative that in the potential development of part of this wetland, that the drainage characteristics of the wetland be retained, and not compromised or eliminated.



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SUBBASIN L
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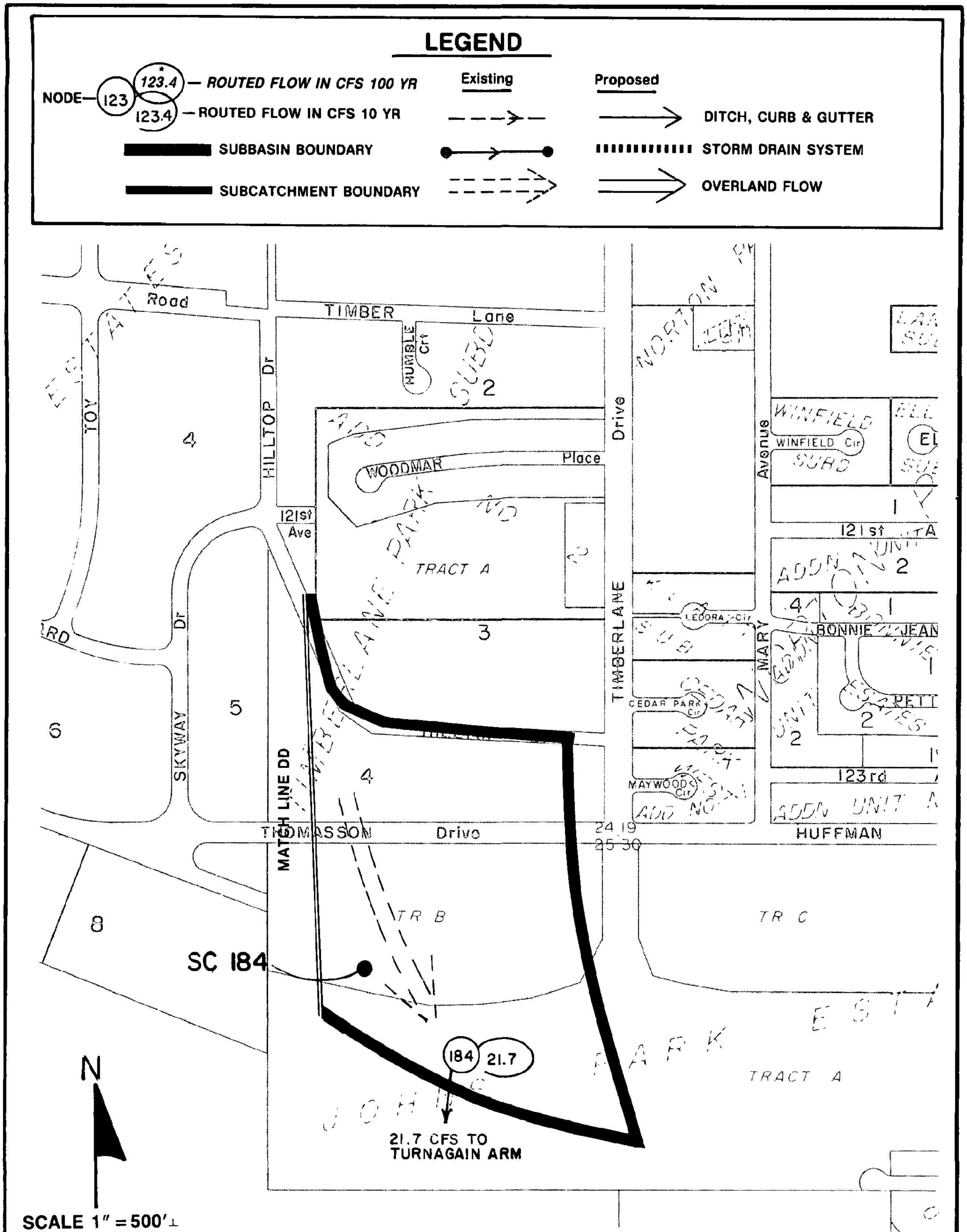

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SUBBASIN L
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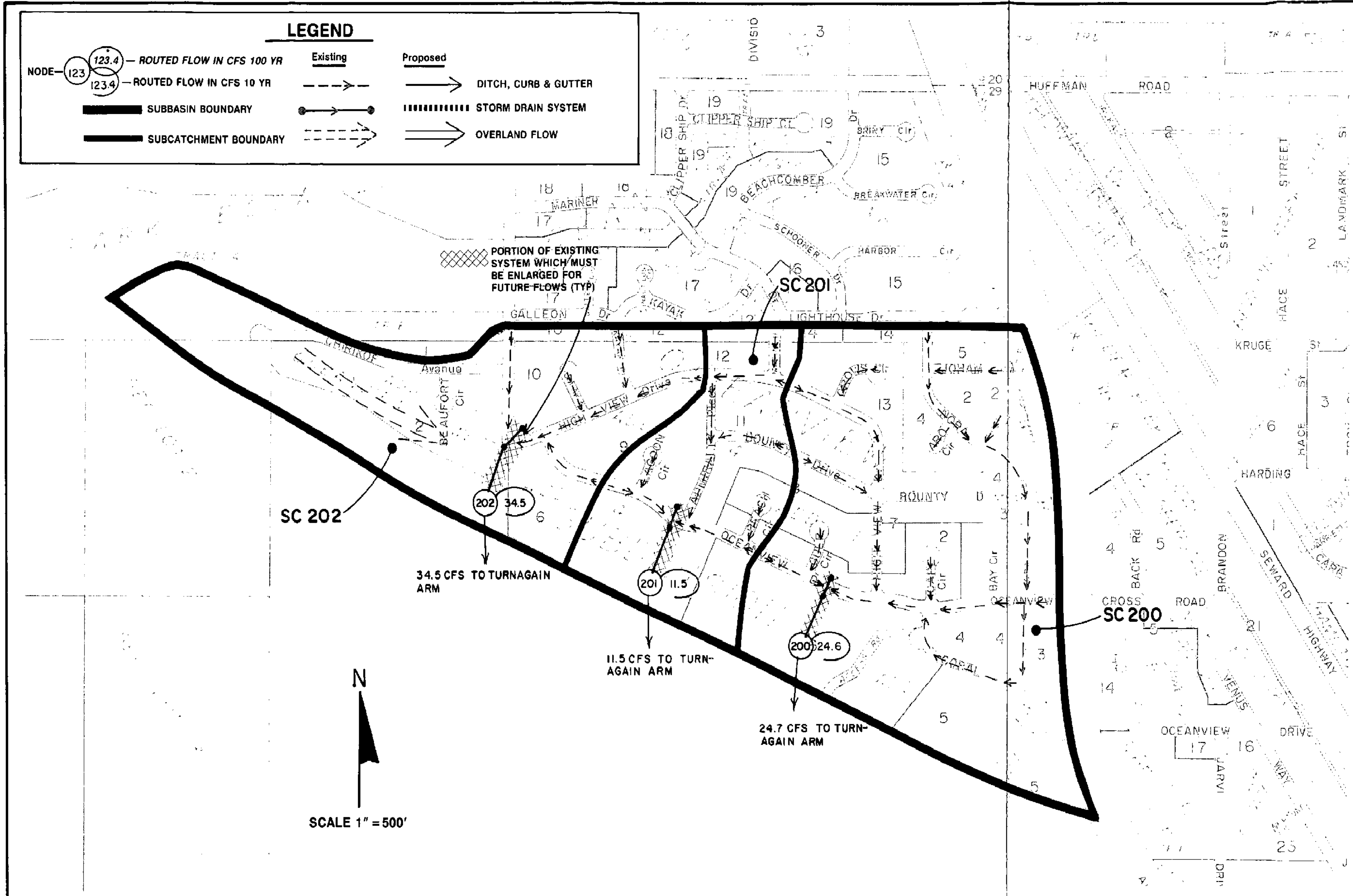
SUBBASIN M

Subbasin M drains to Turnagain Arm. As shown in Table V-1, the various alternatives evaluated in this subbasin were: Alternative #1, no modifying action; and Alternative #3, a trunk system.

There are two different types of drainage systems within this subbasin: 1) a collection system which consists primarily of street and gutter flow; and, 2) corrugated metal pipe outfalls from the collected storm drainage network which discharge to Turnagain Arm. The present collection system is adequate to handle the projected stormwater flows. But each of the three outfalls, one located in each of the subcatchments, 200, 201, 202, are inadequate to carry the combined discharge from the respective collected area.

As shown in Table IV-1, each of the outfall pipes have an estimated capacity of approximately 6 cfs, whereas the range of required capacity for these outfall pipes for projected flows, is between 11.5 and 34.5 cfs. It is recommended that each of the individual outfall pipes be removed and replaced with a storm drainage pipe of sufficient size to handle the capacity shown in Table V-3. It is also recommended, that because of the steep slope of the outfall pipe to Turnagain Arm over the bluff in this area, that design considerations be implemented to minimize the velocity of the runoff water and associated erosion of the bluff.



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SUBBASIN M

February 1983 Figure **V-13**

TABLE V-3

DESIGN PARAMETERS

SUBBASIN	SUBCATCHMENT /NODE	SUBCATCHMENT DESIGN FLOW (cfs) 10-Yr	SUBCATCHMENT DESIGN FLOW (cfs) 100-Yr	TRUNK SYSTEM ROUTING SUBCATCHMENT (NODE) From	TRUNK SYSTEM ROUTING SUBCATCHMENT (NODE) To	TRUNK SYSTEM DESIGN FLOW (cfs) 10-Yr	TRUNK SYSTEM DESIGN FLOW (cfs) 100-Yr	REGIONAL DETENTION POND VOLUME (Ac-ft)
A	1	5.9	10.7	1	Rabbit Creek	5.9	10.7	
B	11	12.0	21.6	11	Rabbit Creek	12.0	21.6	
	12	11.4	20.6	12	Rabbit Creek	11.4	20.6	
	13	11.8	21.3	13	Rabbit Creek	11.8	21.3	
	14	10.2	18.4	14	Rabbit Creek	10.2	18.4	
	15	2.5	4.5			2.5	4.5	
	16	5.9	10.7	15/16	16/Rabbit Creek	8.4	15.2	
	17	5.2	9.4	17	Rabbit Creek	5.2	9.4	
C	25	9.5	17.4			9.5	17.4	
	26	11.8	21.2	25	26	34.6	62.5	
	27	9.1	16.5	27	26	14.6	26.2	
	28	6.1	10.8	28	27	6.1	10.8	
	29	15.0	26.7	26/29	29/NSH(1)	44.8	80.6	
D	45	20.7	36.9			20.7	36.9	
	46	12.9	23.1	45	46	31.2	55.7	
	47	4.8	8.8	46	47	41.1	73.5	
	48	7.0	12.4			7.0	12.4	
	49	4.2	7.4	47/48	49	43.2	77.1	0.43
	50	3.3	6.0	49/50	50/(722)	45.8(2)	81.9(2)	0.43

(1) New Seward Highway

(2) Not Detained Flow
Flow/with Detention = 35.5 cfs, 10 Yr.

TABLE V-3

DESIGN PARAMETERS (CONTINUED)

SUBBASIN	SUBCATCHMENT /NODE	SUBCATCHMENT DESIGN FLOW (cfs)		TRUNK SYSTEM ROUTING SUBCATCHMENT (NODE)		TRUNK SYSTEM DESIGN FLOW (cfs)		REGIONAL DETENTION POND VOLUME (Ac-ft)
		10-Yr	100-Yr	From	To	10-Yr	100-Yr	
E	60	23.6	42.5			23.6	42.5	
	61	12.6	22.9			12.6	22.9	
	62	15.0	27.5			15.0	27.5	
	63	17.3	31.4	61/62	63	41.5	75.6	
	64	14.0	24.8			14.0	24.8	
	65	13.0	23.2	64	65	57.6	99.2	
	66	9.8	17.5	60	66	30.9	55.5	
	67	5.2	9.5	66	67	35.9	64.6	
	68	5.5	10.0	67/65	68	62.9	108.9	
	69	7.2	13.1	68	69	67.7	117.8	
	70	8.0	14.3	63	70	48.0	87.2	
	71	14.3	25.5	70/69	71	125.1	225.4	
	72 (720)	8.8	16.1	71/72/50	72/(720)/(720)	134.0 (176.8)	241.5 (318.0)	
F	75	33.5	60.4			33.5	60.4	
	76	19.6	35.4			19.6	35.4	
	77	8.5	15.4	75	77	40.3	72.7	
	78 (771)	10.5	19.0	77/78	(771)	10.5 (48.4)	 (87.4)	
	79	9.5	16.0	80/79	79(722)	71.3 (246.8)	127.6 (943.3)	
	80	26.1	47.1	(771)	80	65.3	117.9	59.75
	81	0.9	1.6	81	Wetlands	0.9	1.6	67.25
	82	22.8	40.7			22.8	40.7	
	83	20.0	35.5	82	83	39.6	70.5	53.00
	84	13.1	23.4	83	Wetlands	13.1	23.4	41.25
	85	24.1	36.0	76	85	36.8	60.3	21.00
	86	18.3	32.2	86	Wetlands	18.3	32.2	40.25

(1) New Seward Highway

TABLE V-3

DESIGN PARAMETERS (CONTINUED)

SUBBASIN	SUBCATCHMENT /NODE	SUBCATCHMENT DESIGN FLOW (cfs) 10-Yr	100-Yr	TRUNK SYSTEM ROUTING SUBCATCHMENT (NODE) From	To	TRUNK SYSTEM DESIGN FLOW (cfs) 10-Yr	100-Yr	REGIONAL DETENTION POND VOLUME (Ac-ft)
G	100	13.1	23.6	100	NSH[1]	13.1	23.6	
	101	35.8	64.6	101	Inlet	35.8	64.6	
	102	41.3	74.5	102	Inlet	41.3	74.5	
	103	46.2	83.3	103	Inlet	46.2	83.3	
	104	7.1	12.8	104	NSH[1]	7.1	12.8	
	105	9.4	17.0			9.4	17.0	
	106	16.2	29.2	106	Inlet	16.2	29.2	
	107 (1071)	8.5 (7.7)	15.4 (13.9)	105	107	20.6	37.3	
	108	28.1	50.7	111/108	108/Inlet	41.1	84.2	
	109	17.2	31.1	107	109	37.0	67.0	
	110 (128)	14.4	26.1	(722)/ 109/110	(128)/ 110/(128)	47.7 (317.7)	86.4 (571.4)	
	111	13.0	23.5	11	108	13.0	23.5	
H	125	18.8	34.1	125	(128)	18.8	34.1	
	126	6.5	11.4	126	(128)	6.5	11.4	
I	135	17.1	30.9	136	SA[3]	17.1	30.9	
	136	16.0	28.4	135	136	30.5	54.6	
J	145	8.4	14.9	145	SA[3]	8.4	14.9	
	146	16.7	16.1	146	SA[3]	9.2	16.1	
	147	27.8	22.1	147	SA[3]	12.3	22.1	

(1) New Seward Highway
(3) South Anchorage Study Area

TABLE V-3

DESIGN PARAMETERS (CONTINUED)

SUBBASIN	SUBCATCHMENT /NODE	SUBCATCHMENT DESIGN FLOW (cfs)		TRUNK SYSTEM ROUTING SUBCATCHMENT (NODE)		TRUNK SYSTEM DESIGN FLOW (cfs)		REGIONAL DETENTION POND VOLUME (Ac-ft)
		10-Yr	100-Yr	From	To	10-Yr	100-Yr	
K	160 (159)	11.7	21.6	(128)/ 167/(159)	(159)/ (159)/160	342.7 (300.8)	616.9 (595.0)	
	161	3.4	6.1			3.4	6.1	
	162	6.1	11.0	169/168/163	162	362.5	651.7	
	163	3.5	6.1			3.5	6.1	
	164	3.6	6.5	162	164	369.8	664.9	
	165	11.4	20.1			11.4	20.1	
	166	11.1	19.4	170/164	166	392.0	704.2	
	167	17.6	31.7	166	Inlet	17.6	31.7	
	168	12.8	23.0			12.8	23.0	
	169	6.3	10.7			6.3	10.7	
	170	9.9	16.7			18.0	32.3	
	185	9.2	16.6	185	170	9.2	16.6	
L	184	21.7	38.3	184	Inlet	21.7	38.3	
	186	37.7	69.2	186	Inlet	37.7	69.2	
	187	2.6	3.5	187	Inlet	2.6	3.5	
	188	6.6	8.6	188	Inlet	6.6	8.6	
	189	11.2	19.4	189	Inlet	11.2	19.4	
M	200	24.6	34.4	200	Inlet	24.6	34.4	
	201	11.5	20.8	201	Inlet	11.5	20.8	
	202	34.5	62.4	202	Inlet	34.5	62.4	

IMPLEMENTATION OF ALTERNATIVES

Existing and future drainage standards will be met by using the identified alternatives per each subbasin/subcatchment as presented in this chapter (summarized in Table V-1). The Municipality of Anchorage, in the implementation of their capital improvement program and in conjunction with private developers, should provide the major trunk systems necessary to convey stormwater runoff. For each of the recommendations contained in this study the developer should conduct a preliminary engineering feasibility analysis to: 1) identify the most acceptable type of drainage structure and system in accordance with the alternatives recommended herein, and, 2) identify the most feasible route alignment as per the land use pattern at the time of implementation. The results of this analysis will be subject to Municipal review and approval.

The design engineer prior to designing any of these systems for the study area, should use the following procedure as a guideline, but still rely on professional judgement where applicable.

- Use the composite map bound in the back of the report to identify the subbasins, subcatchments, flow paths, and major existing and proposed drainage ways for the areas involved.

- Review the alternatives and associated recommendations of each subbasin and/or subcatchment to be served, and update accordingly for the present land use patterns.

- From the appropriate subbasin map identify flow rates for subcatchments involved (particularly when identifying future collection systems), identify that portion of the subcatchment to be served, and compute the proportionate amount of contributing flow for each segment of the system to be designed.

- Identify the type of storm drainage system to be used. This system should meet the criteria as presented in Chapter 3 for this study, as well as the zoning regulations in effect at the time the design is performed.

- Use accepted Municipality of Anchorage Department of Public Works design criteria.

- Review Appendix B of this report (Discussion of Icings in Drainage Systems), use of this section will minimize the amount of icing associated with drainage structures. A deficiency of many of the existing drainage systems within this study area are the result of inadequate storm drainage design, as related to icings. With proper design considerations for icing conditions, operational and maintenance costs associated with icing conditions for these drainage structures can be minimized.

APPENDIX A
COMPUTER ANALYSIS

APPENDIX A

COMPUTER ANALYSIS

Two computer programs (mathematical models) were used in this study to perform the hydrologic/hydraulic and pollutant loading analysis. System Analysis Model (SAM), was used to compute 1) the runoff hydrographs and pollutographs from each subcatchment, 2) to route the hydrographs and pollutographs through the drainage system, and, 3) to size the required capacities for the drainage facilities, from a given design storm event. Storage Treatment Overflow Runoff Model (STORM) was used to compute the pollutant washoff loads in each subcatchment and the resulting pollutant concentrations in the runoff, from several long-term historical events (May to Sept.). The usages of both models are well documented in the respective user manuals (SAM: O'Neel et al, 1979; STORM: U.S. Army Corps of Engineers, 1976).

The following sections describe the compilation of the input data files for each model, the assumptions made, and how these files can be used in further analysis for the study area.

Analysis using SAM Model

Table A-1 lists the SAM input data requirements, and the data sources and remarks for compiling the input data files for the study area.

TABLE A-1

INFORMATION REQUIRED FOR SAM MODEL

Input Data Requirements	Data Sources	Remarks
1. A hyetograph for the event being simulated	Precipitation records from Anchorage Airport (1953 to 1980)	A design event of 10-year return frequency was used (see Design Storm, Chapter III)
2. Subsurface flow accounting parameters for previous areas; e.g. soil moisture zone storages, evapotranspiration indices, and ground water characteristics	Table 4 subsurface values, SAM methodology manual (Task memorandom No. 7), Campbell Creek Drainage Study	The surface runoff is not sensitive to the variability of these parameters for the design condition of the soil is assumed to be freezing.
3. Computation time increment	———	10 minutes
4. Pan evaporation rate	Weather data, Anchorage Airport, U.S. Weather Service	An evaporation rate typically for the time of the design event (April).
5. Interception and depression storage for impervious area	Field survey for the project area	Used 0.10 in. which is an average value for the impervious area.
6. Fraction of land use area which is impervious	Existing land use - Municipal Planning Dept. 1980 aerial photograph and field investigation; Future land use - Municipal Comp. Land Use Plan	For the design event when the freezing ground condition was assumed that 85 to 90% of the impervious area was treated to be impervious in the model.
7. Hydraulic characteristics of pipe, gutter, and channel cross section	Table B-1 "Land Use Classifications", Campbell Ck.Drain. Study, Task Memo. 7, and sections data D1 and D2	The hydraulic characteristics of closed conduits and natural channels are represented by dimensionless curves so that one set of coefficients can be applied to all cross sections of similar shape.

TABLE A-1
(Continued)

Input Data Requirements	Data Sources	Remarks
8. Daily rates of pollutant buildup in pounds per acre	Table 5, "Land Use Build-up Matrix", _Campbell Ck. Drain. Study, Task Memo. 7_	Data for winter months were used.
9. Pipe and gutter inverts, Lengths and other hydraulic characteristics	Record drawings for sub-divisions, and highway, and field investigations	Data for winter months were used.
10. Drainage basin area, slopes and overland flow length	Municipal Planning Dept. topographic maps and field investigation	Scale used was 1" = 500'.
11. Local retention (depression) storage area	Municipal Planning Dept. topographic maps and field investigation	Modeled as the fraction of the subcatchment area that contributes no runoff.

Much of the input data for the SAM model, such as the subsurface flow parameters, percent pervious in each land use type, and pollutant buildup rates, were calibrated during the investigation of Campbell Creek drainage basin. For lack of streamflow data in the Furrow Creek-Rabbit Creek study area, these constants were not able to be calibrated specifically for the study area. However, because of the similar hydrologic and land use settings between the Campbell Creek drainage basin and the study area, the model constants should represent the study area reasonably well. As local stream data for Furrow Creek and Rabbit Creek become available, the validity of these constants as applied to the study area should be evaluated.

The local depression storage areas were modeled as the portion of the subcatchment area that contributes no runoff. The extent of the existing depression storage area is measured from the basin aerial maps and later substantiated by field investigations. The possible future depression area in the basin is extrapolated from the future land use information. For example, the existing depression area will assume to be lost if the land use for a parcel of land is changed from a low density residential or undeveloped to high density residential, commercial or industrial land use classifications.

To execute the program, the input data files were organized in three groups:

1. Data base file - consists of all the data listed in Table A-1.

2. Analysis command file - instructs the model to compute overland flow, then the flow routing (using Kinematic wave method) and pollutant washoff.

3. Output command file - selects the portion of the output results to be printed; e.g., inlet and routed hydrographs and pollutographs.

For the convenience of basin analysis, the input data files for the following subbasins were grouped together:

Subbasin	Subbasin is tributary to:
A, B	Rabbit Creek
C, G (southern portion)	Potter Marsh & Turnagain Arm
D, E, F, G (remaining), H, K	Furrow Creek
I, J	South Anchorage Study Area
L, M	Turnagain Arm

The SAM model was run on the Boeing Computer Service (BCS) CTS system. Table A-2 lists the SAM input data files for this project. These files are stored on magnetic tape which is located in the office of the Municipality of Anchorage Department of Public Works. SAM input files can be easily retrieved and modified for future applications. The subcatchment (NODE) and drainage system (LINE) data files along with the design storm (HYETOGRAPH) data file will probably require revisions with time to simulate changes occuring in the future. These files, NODE, LINE, and HYETOGRAPH, are contained in the subbasin data files.

TABLE A-2

SAM INPUT DATA FILES

Subbasin	Data Base File*	Analysis Command File	Output Command File
1. A, B (existing land use)	ABEX FT12F001	Analyze FT12F001 **	OUTPT CNTL ***
(future land use)	ABFU FT12F001		AB NODEOUT
2. C, G (existing land use)	CGEX FT12F001		CG NODEOUT
(future land use)	CGFU FT12F001		
3. D, E, F, G, H, K	DEFGHK CNTL		DEFGHK NODEOUT
(existing land use)	DKEX SUBCAT		
(future land use)	DKFU SUBCAT		
4. I, J (existing land use)	IJEX FT12F001		IJ NODEOUT
(future land use)	IJFU FT12F001		
5. L, M (existing land use)	LMEX FT12F001		LM NODEOUT
	LMFU FT12F001		

* These data files are set up for running the quantity portion of the analysis; to conduct the quality runs, merge all files with POLLUT DATA.

** ANALYZE FT12F001 is a common file for all subbasins with the exception of the first card which specifies the unique basin name, e.g. AB, CG, DEFGHK, IJ and LM.

*** OUTPUT CNTL is a common file for all subbasins, it needs to merge with the accompanying file, e.g. AB NODEOUT to form an output command file.

The output of run using the SAM model typically contains inlet and routed hydrographs at selected locations and a table summarizing the drainage system, e.g. the peak flow, surcharge conditions, the existing and required capacity.

100-year Storm: Runoff and Routing

The computation of the runoff and routed flow quantities was performed in a somewhat different manner than that used for the 10-year storm. The following methods presents the method of calculation used for the 100-year storm event.

The computer run performed using the SAM model to generate the 100-year storm runoff was based on the assumption that all existing depression area will be destroyed in the future when the land is developed. However, later this assumption was modified such that a portion of the depression area will be retained in the future. Instead of performing another computer run based on this new assumption, a correction factor was applied to the design flow obtained in the computer run in order to account for the loss of runoff due to the depression area.

To produce the adjusted 100-year storm runoff values the flow ratio of the total non-depression area to total area was calculated for those subcatchments that have a significant amount of depression area. The corresponding subcatchment design flow was then adjusted by multiplying by the appropriate flow ratio.

The trunk system design flow was calculated using the peaking factor for the 10-year storm event and applying it to the 100-year storm event. For example:

Flow from node 1 is routed to node 2. The peak flows for node 1 and node 2 inlet hydrographs are 10 and 20 cfs, respectively.

The resultant routed hydrograph of node 2 from the model is 27 cfs. Then the peaking factor is $\frac{27}{10 + 20} = 0.9$.

Analysis using STORM Model

The STORM Model was used to compute the pollutant washoff loads from each subbasin. The Boeing Computer Service (BCS) EKS System was used to run the STORM model. To better characterize the pollutant loads, several of the subbasins defined for the use of the SAM model were further divided into several subbasins. The land use data used with the SAM model were used with the STORM model. Table A-3a and A-3b give the existing and future land use data for STORM model input.

Table IV-7, the pollutant buildup matrix shown in the main text of this report, was used to represent the summer pollutant loading rates for the model. The STORM model does not have the capability to route either the flow or the pollutant through the drainage system; so that the drainage data developed for the SAM model were not used.

TABLE A-3a

LAND USE DATA FOR STORM MODEL INPUT

(EXISTING SYSTEM)

STORM RUNOFF BASIN	SUB-CATCHMENTS	TOTAL AREA (AC)	LAND USE (% OF TOTAL BASIN AREA) LD	HD	MF	IN	CO	LF	BM	UP	UF	GP	Tc * (Hr)	DEPRESSION DEPTH (in)
AB	1,11,12,13,14 15,16,17	333	98	2									0.49	.100
C	25,26,27,28,29	168	58	20				22					0.64	.155
D	45,46,47,48, 49,50	198	14	66						20			1.28	.140
E1	61,62,63,70	252	70								30		1.39	.160
E2	60,64,65,66,67, 68,69,71,72	340	20	72						8	72		4.72	.260
F1	76,81,82,83,84, 85,86	423	54		11			17				18	1.27	.188
F2	75,77,78,79,80	344	35	7							32	26	2.08	.229
G1	100,104	71	75	25									0.59	.100
G2	101,102,103, 106,108,1081, 111	234	21	55	5		10	9					0.13	.123
G3	105,107,109, 110,1071	221	22	17	59	2							0.33	.100
H	125,126	138	45			15	40						0.45	.100
I	136,137	131			28	34	38						0.28	.100
J	145,146,147	81		81		4		15					0.12	.138
K1	167	60				20		30		50			1.02	.275
K2	160,161,162, 163,168,169	163	14	40				20	1	25			0.76	.203
K3	164,165,166, 170,185	162	30					37	14	19			1.67	.270
L1	184,186	223		95				5					0.56	.113
L2	187,188,189	91		30					70				0.61	.310
M	200,201,202	148		95				5					0.14	.113

*Tc, time concentration

TABLE A-3b

LAND USE DATA FOR STORM MODEL INPUT

(FUTURE SYSTEM)

STORM RUNOFF BASIN	SUB-CATCHMENTS	TOTAL AREA (AC)	LAND USE (% OF TOTAL BASIN AREA) LD	HD	MF	IN	CO	LF	BM	UP	UF	GP	Tc* (Hr)	DEPRESSION DEPTH (in)
AB	1,11,12,13,14 15,16,17	333	98	2									0.49	.100
C	25,26,27,28,29	168	58	20				22					0.64	.155
D	45,46,47,48, 49,50	198	17	76				7					1.11	.118
E1	61,62,63,70	252	78	22									1.39	.100
E2	60,64,65,66,67, 68,69,71,72	340	28	72									1.48	.100
F1	76,81,82,83,84, 85,86	423	18	37	11			34					.90	.185
F2	75,77,78,79,80	344	14	68				18					1.39	.145
G1	100,104	71	75	25									0.59	.100
G2	101,102,103, 106,108,1081, 111	234	21	56	5		9	9					0.17	.123
G3	105,107,109, 110,1071	221		53	29		16	2					0.28	.105
H	125,126	138		2		36	62						0.31	.100
I	135,136	131			28	34	38						0.28	.100
J	145,146,147	81		81		4		15					0.12	.138
K1	167	60			30	50		20					0.32	.150
K2	160,161,162, 163,168,169	163		76	14			10					0.52	.125
K3	164,165,166, 170,185	162		81				19					0.56	.148
L1	184,186	223		96				4					0.49	.110
L2	187,188,189	91		30					70				0.54	.310
M	200,201,202	148		96				4					0.14	.110

*Tc, time concentration

The major difference between SAM and STORM model input is the storm data. SAM uses a storm event which may cover several hours with a 10-minutes or less time steps. One can repeat the analysis using the SAM model for many storm events, but this effort is constrained by the cost of the computer runs. The STORM model uses a coarser time step (an hour) than the SAM model, therefore it is more suited for analyzing the runoff for continuous period such as the entire summer season. The hourly precipitation data for the STORM model were read directly from a tape (Format TD-9654) which was purchased from the National Climactic Center located in North Carolina. The data in the tape contains the most recent precipitation records (from 1963-79), measured at the Anchorage International Airport.

Table A-4 summarizes the input data files used for the simulation.

The model output gives results of both quantity and quality analysis. It also gives the number of storm events occured during the modeling period. The storm event for this project as defined to be, 1) it consists no more than 6 hours of dry period, and, 2) it produces a runoff of more than 0.01 in/hr.

TABLE A-4

STORM INPUT DATA FILES

- The input file for each run consists of a combination of three separate files:

1.	CNTL	Control data file
2.	WDATA67	Precipitation data, May-Sept., 1967
3.	FUTULU	Future land use and pollutant loading data

- Other relevant data files:

WDATA63	Precipication data, May-Sept., 1963
WDATA69	Precipitation data, May-Sept., 1969
ExistLU	Existing land use and pollutant loading data

- Following is a typical job file, called JSTORM, for the STORM model run:

```
TEST, CM205000,P02,T10
USER*
ATTACH, STORM
GET, CNTL, WDATA67, FUTULU.
COPYEI, CNTL, TANG.
COPYEI, WDATA67, TANG.**
COPYEI, FUTULU, TANG.
REWIND, TANG
STORM, PL=20000, TANG, TOUT1.***
EXIT, U.
COST, LO=F
DAYFILE(TOUT1)
REPLACE(TOUT1)
```

* User no. and password
** Input file
*** Output file

APPENDIX B
DISCUSSION OF ICINGS IN DRAINAGE SYSTEMS

APPENDIX B

DISCUSSION OF ICINGS IN DRAINAGE SYSTEMS

The purpose of this chapter is to supply information regarding the icing phenomenon and its related problems and to present methods of control and prevention. These methods were taken into consideration in the formulation of alternatives presented in this report.

The basic sources of icings are springs, streams and general seepage. Within the study area springs do not constitute a significant portion of the water supply sources (Hydrology for Land Use Planning: The Hillside Area, 1975, pg.15). Thus, streams and general seepage are the sources of icings within the study area.

The results of icing can be hazardous. In the case of complete blockage of the drainage facility, water may become ponded in areas or diverted to areas such as roadways, that were meant to be kept drained. Other potential hazards are soil erosion, increases in water levels in streams and channels, raised water tables, saturated fills and embankments and washouts.

EXAMPLES OF BLOCKAGE PROBLEMS

Two major factors involved in the formation of icings are heat loss and depth of flow. Heat loss is a function of the air temperature and exposure. As the air temperature decreases, the rate of ice formation increases. And as the area surrounding the water becomes less of a barrier to heat loss, the rate of heat loss increases.

Depth of flow is important in relation to the thickness of ice that can be formed for any particular heat loss condition. The problems of ice blockage occur when the depth of flow is the same or smaller than the thickness of ice that can form. The flow freezes solid, reducing the drainage facility cross section and forcing the flow to spread out on top of the already formed ice and become frozen itself.

Ice formation inside a culvert reduces its cross section and its cpacity to carry flow. Depending on hydraulic characteristics, such as slope, flow rate and inlet and outlet conditions, ice may build up uniformly throughout the length of a culvert, or form primarily at the entrance or exit of the culvert. In some cases, ice may begin forming in the upstream or downstream channel and progress to the culvert. A very common location of ice build-up is at the end of a culvert designed for free fall. A second common location is within a culvert having low flow where the water freezes in sheets gradually reducing the culvert cross section.

Icing in ditches can lead to ponding and overflow. Ice can form in ditch bottoms and progress upward, or may enter from the side of the ditch as a result of freezing backslope seepage. Debris in the ditch can become a contributor to icing problems. Snow, an insulator, can be a contributor if water seeps below the snow and freezes at the base of the snow, causing blockage in the ditch.

Subsurface drains which collect subsurface seepage and ground water can become blocked in two ways. First, if the frost level falls to the depth of the drain, freezing will occur. Secondly, and more commonly, is the occurance of ice blockage at the outlet of the subsurface drain where the drainage water first encounters low air temperatures. The second case essentially forms a plug, backing up the entire drain with water, which in turn may become frozen.

PROBLEM SOLUTIONS

The solutions to problems of ice-blocked drainage facilities fall into two categories: ice control and ice prevention. Each will be discussed in detail in the folowing paragraphs.

Methods of Icing Avoidance and Control

For drainage facilities where elimination of icings is not possible, a number of methods are available as control measures.

1. Transfer of location

   One of the most elementary yet most costly methods of solving icing problems is to relocate the drainage facility. This a "last resort" measure.

2. Raising grade

   By raising the grade of the structure, the seasonal encroachment of icings can be postponed. This measure must be used with caution as it may lead to washouts from ice blocked facilities as well as undesirable seepage effects.

3. Numerous and large drainage structures

   Providing more drainage facilities than might otherwise be required is based on the likelihood that icings will divert meltwatrer runoff to points normally dry. This method is used to protect against washouts.

4. Storage space

   By excavating an area for icings to form and grow, the icings will present no hazard to the area of concern.

5. Dams, dikes or barriers

   Facilities, such as ice fences, are used to limit the horizontal dimensions of icings. They may be temporary or permanent.

6. Culvert closures

In situations where the storage space for ice upstream from the culvert is large, and where the flow of water throughout the winter is very small and intermittent, it is possible that closures placed at the ends of a culvert in the autumn may facilitate opening the culvert to accept runoff in the spring. In this way, the culvert is prevented from becoming filled with snow and ice, and the maintenance effort to remove the closures may be less than the effort that would otherwise be required to remove ice from within the culvert.

7. Staggered culverts

Two or more culverts are used for one stream, one at the base of the roadway fill, and the other(s) at a higher elevation in the fill. The higher culvert is normally dry except during the spring when, because the lower culvert is blocked by the accumulated icing, the higher one carries the initial spring runoff over the icing. The lower culvert becomes cleared of ice as the spring thaw progresses, and eventually accommodates the entire flow, leaving the higher culvert dry again. The higher culvert may be placed to one side of the lower culvert. Thus, less vertical distance would be required for the installation, so that the initial amount of water blocked up during the spring is at a minimum. This method is most applicable where the topography permits or requires deep fills. The icing accumulation area must be large enough to store an entire winter's ice without having the icing reach the upper culvert or the elevation of the area being protected.

8. Filtration dikes

The filtration dike is an embankment composed of very coarse granular material. These dikes have been used in Russia with success. However, in the United States they have not achieved widespread use.

9. Heat

Icings are commonly controlled by the application of heat, the objective being not to prevent icings but to establish and maintain thawed channels through them to minimize their growth and to pass spring runoff.

10. Steam

This method is used to thaw culvert openings and to thaw channel into icings for collecting icing feed water or early spring runoff.

11. Fuel oil heaters

Known more commonly as "firepots", fuel oil heaters are widely used. Use of firepots is declining because of high maintenance requirements, energy inefficiency and the difficulty in preventing theft of fuel.

12. Electrical heating

Where electrical power is available, the use of insulated heating cables has proved successful. It requires less maintenance than steam thawing but may not be cost-effective if electricity costs are high.

13. Breaking and removing accumulated ice

This measure should be limited to use in emergencies.

14. Blasting

The process of blasting has two beneficial effects. First, blasting aids in the physical removal of ice. Secondly, fractures created by blasting provide paths for water flow deep within the icing where, protected from the atmosphere, it may not refreeze.

15. Chemicals

Chemicals such as sodium or calcium chloride are some times used to prevent refreezing of the drainage facility once it has been freed of ice by other means. Negative aspects of using these chemicals are the corrosive effects on metal piping and the detrimental effects of fish and wildlife, vegetation and other downstream water uses.

16. Solar heating

In recent years, tests have been undertaken to determine the practicality of using solar energy to thaw culverts (The Northern Engineer, Volume 13 No. 3, Pg 39). Results prove this method to be viable, especially as the technique becomes more refined.

Methods of Icing Prevention

A number of measures are available to avoid icings from forming including those described below.

1. Channel modifications

   Straightening and deepening a channel can prevent icings, although frequent maintenance is usually required to counteract the stream's tendency to resume natural configuration by erosion and.deposition. Rock-fill gabions have been used to create a deep, narrow channel for low winter discharges. Such deepened channels permit information of ice cover to normal thickness while providing adequate space beneath for flow. Deepening at riffles, rapids or drop structures is especially important as icings are most apt to form in these shallow areas.

2. Insulation of critical sections

   River icings may be prevented by insulating critical sections of the river where high heat losses cause an excessive thickening of the normal ice cover, leading to complete blockage of the flow and subsequent icing formation. These sections may be located under a bridge or at riffles and rapids. Insulating covers are generally expensive and time consuming.

3. Frost belts

   Also known as a permafrost belt, the frost belt is basically a ditch or cleared strip of land located upstream or upslope from the icing problem area. The area is cleared of vegetation and snow is removed during the first half of winter. This enables deep seasonal frost to act as a dam to the water seeping through the ground forcing it to surface where it will form an icing upstream or upslope from the belt. When used in a drainage channel situation, a belt is formed by periodically cutting transversely into the ice to cause the bottom of the ice cover to lower and merge with the bed. The icing is therefore induced to form away from the bridge or culvert entrance being protected.

4. Surface drainage

   In the region of icing development of the soil mantle can be drained by a network of drainage ditches. The ditches are made deep and narrow to expose only a small surface area to the atmosphere. Some ditches have insulated covers.

5. Subsurface Drainage

Though better defense against icings than surface drains, subsurface drains cannot be used in permafrost areas. The water collected is transferred to a point away from the area being protected. However, the drain outlet will still experience icing problems.

6. Insulation of the ground

In areas where deep seasonal frost penetration forms a dam to groundwater flow, icings have been avoided in some cases by insulating the ground. Caution must be exercised with this method to avoid the icing to simply be shifted to another problem area.

7. Earth embankments and impervious barriers

The earth embankment used in conjunction with the barrier impervious to groundwater flow is another technique for preventing the formation of ground icings. The embankment and barrier are placed well away from the area being protected. This technique functions in a manner similar to frost belts in that they dam seepage flow through the soil and induce an icing to form where it is harmless.

APPENDIX C
CAPITAL IMPROVEMENT COST ANALYSIS

APPENDIX C

CAPITAL IMPROVEMENT COST ANALYSIS

INTRODUCTION

In the main text of this report a comprehensive trunk system is described. The complete trunk system is shown on the map bound in the back of this report, with a color code designating whether the segment is existing and of adequate capacity, existing but requires upgrading, or a proposed new trunk line.

Cost estimates are presented by trunk system. The cost figures reflect recommended improvements to existing trunk systems or construction of recommended new trunk systems. All dollar amounts are for the year 1982. The costs shown are total project costs. Projected operation and maintenance costs are also presented.

The type of structure used in the cost estimates for each trunk improvement is as recommended in Chapter 5 of this report. It must be stressed that the choice of structure in each case is an engineering judgement based on experience and present conditions. The final decision as to the type of structure will be made by the Department of Public Works who will reevaluate the situation as time progresses and as community needs change.

PROJECT COSTS

The cost estimates presented in this appendix and summarized in Table C-13 located at the end of the appendix, are based on 1982 prices and represent total project costs. Project costs include construction cost plus a contingency of 10 percent as well as allied costs. Allied costs include services and costs such as project administration, engineering, Municipal administrative costs. These allied costs have been computed as 30 percent of the construction cost. Other costs such as easement acquisition, assessment roll, and bond and legal counsel may also be an allied cost, depending upon the project. Should these items arise, they must be added to the allied costs.

TABLE C-1

ALLIED PROJECT COSTS
AS A PERCENT OF CONSTRUCTION COST

Item	
Administration	1.0
Engineering Design	8.5
Construction Engineering	10.0
Bond Discount, Interest during Construction, Financial fees, etc.	10.5
TOTAL	30%

OPERATION AND MAINTENANCE COSTS/FACILITY SERVICE LIFE

Proper maintenance of storm drainage facilities is imperative for such facilities to function correctly as well as to extend the service life of the facilities for as long as possible. Maintenance measures which have been included in the estimate of maintenance costs are:

Closed pipe system:	Clean out entrances to pipe at inlets and manholes. Keep clear of debris and ice buildup. This is to be performed at regular intervals (estimated at two times per year).
Culvert/ditch system:	Seed every summer to keep channels lined with grass. Clean out entrances to culverts; keep entrances clear of debris and ice buildup. The clean-out process is estimated at three times per year: twice in spring/summer and once during a winter freeze/thaw cycle. Every other year one of the clean-up processes is to be replaced with retrenching of the ditches.
Greenbelt:	Operation and maintenance procedures comparable to the culvert/ditch system. However, clean-out process is estimated at two times per year, and retrenching every three years.
Outfall:	Repair gaskets and brackets on a yearly basis. Annually repair the area downstream of pipe which functions as an energy dissipator.

Service life for the closed pipe and outfall systems are estimated at twenty years. Ditch and culvert systems and greenbelts are also estimated to have a twenty year service life, with some retrenching required every two or three years, respectively, to maintain flow capacity.

Annual operation and maintenance costs for each system, based on a twenty-year life cycle are as follows:

System	Annual O & M Cost in 1982 dollars (per LF) (20 year life cycle)
Closed pipe	\$2.00
Culvert/ditch	\$1.50
Greenbelt	\$1.50
Outfall	\$1.00

CONSTRUCTION MATERIALS

All construction shall be performed per the Municipality of Anchorage Standard Specifications (MASS). For cost estimating purposes, all storm drain pipe was assumed to be corrugated metal pipe (CMP) and Class C bedding material. Ditches were assumed to have side slopes in the ratio of 2 (horizontal) to 1 (vertical). Ditch material was assumed to be the insitu material with seeding being performed on surface of ditch to prevent erosion.

Costs include appurtenances along trunk route such as catch basins and inlets. Road repair is included where applicable.

Where facilities are to be included in the development of an area, no dollar allotment has been made for road construction.

TRUNK SYSTEM IMPROVEMENTS

Cost estimates, as stated in the "Introduction" section of this appendix, are for the type of structures recommended in Chapter 5. Final determination of the facility type to be used for a given case will be made by the Department of Public Works.

Cost estimates are presented in the following paragraphs by trunk system.

A summary of the total cost for each trunk is presented at the end of this appendix in Table C-13.

Trunks Requiring No Improvements

The following trunk systems have no recommended improvements associated with them: Rabbit Creek Road (RC Rd), Rabbit Creek (RC), New Seward Highway - East and West (NSH-E, NSH,W), Old Seward Highway - 1 and 2 (OSH-1, OSH-2), Johns Road (Johns Rd), and the Alaska Railroad (Ak RR).

Upper Furrow Creek - South 1 (UFC-S1)

The upstream section of this trunk is presently a pipe/greenbelt system of adequate capacity except at two street crossings (Spinnaker and Westwind). For cost estimating it is assumed that the existing 18" culverts will be removed and 42" culverts inserted.

The downstream segment of UFC-S1 consists presently of pipe, culvert, and ditch systems. Along Tradewind the existing 24" pipe is slightly under the design capacity. It is recommended that the 24" pipe continue to be used unless severe drainage problems result. If such problems result, it is recommended that a 30" pipe be used to replace the 24" line.

Along the Frontage Road the culvert and ditch system presently in use is of inadequate size. It is assumed for cost estimating purposes that the present system will be enlarged in cross sectional area to be of the equivalent capacity of a 42" CMP pipe.

TABLE C-2
UFC-S1 COST ESTIMATE

Upgrade two (2) 18" cross culverts to 42" (50 LF each)		$ 12,000
Expand Frontage Road ditch (5000 LF)		66,900
Upgrade Tradewind pipe from 24" to 30" (if required) (1100 LF)		117,700
	CONSTRUCTION COST	$196,600
	10% Contingency	19,700
	30% Allied costs	59,000
	TOTAL PROJECT COST	$275,300
	O & M per year	$ 9,800

Upper Furrow Creek - South 2 (UFC-2)

This trunk system is presently the Upper Furrow Creek channel. A well defined greenbelt is recommended and is used herein for cost estimating purposes. The capacity of the greenbelt is assumed to be equivalent to a 54" pipe.

TABLE C-3

UFC-S2 COST ESTIMATE

Constuct Greenbelt (2300 LF)		$60,900
	CONSTRUCTION COST	$60,900
	10% Contingency	6,100
	30% Allied costs	18,300
	TOTAL PROJECT COST	$85,300
	O & M per year	$ 3,500

Upper Furrow Creek - South 3 (UFC-S3)

The upstream segment of this trunk is located in an area which will soon be developed. The trunk segment is proposed and costs are based upon the installation of a 30" pipe.

The downstream segment of UFC-S3 is presently 21" and 24" pipe. This approximately one quarter of the required design capacity. For cost estimating purposes, it is assumed that the present pipe will be replaced with 48" pipe.

TABLE C-4

UFC-S3 COST ESTIMATE

Install 30" pipe (3000 LF)		$248,300
Upgrading existing 21" - 24" pipe to 48" pipe (1900 LF)		308,600
	CONSTRUCTION COST	$556,900
	10% Contingency	55,700
	30% Allied costs	167,100
	TOTAL PROJECT COST	$779,700
	O & M per year	$ 9,800

Upper Furrow Creek - South 4 (UFC-S4)

The UFC-S4 trunk is a proposed system. The cost estimate is based upon an open greenbelt corridor following the channel of Upper Furrow Creek. The capacity of the greenbelt is assumed to be the equivalent of a 36" pipe.

TABLE C-5

UFC-S4 COST ESTIMATE

Construct greenbelt (2500 LF)		$38,400
	CONSTRUCTION COST	$38,400
	10% Contingency	3,800
	30% Allied costs	11,500
	TOTAL PROJECT COST	$53,700
	O & M per year	$ 3,800

Upper Furrow Creek - North 1 (UFC-N1)

The UFC-N1 trunk is a proposed sytem. It is recommended to be either a system of roadside ditches and culverts or a pipe system, depending upon the ultimate development pattern.

TABLE C-6

UFC-N1 COST ESTIMATE

Option 1:

Construct 21" pipe (1200 LF)		$105,800
	CONSTRUCTION COST	$105,800
	10% Contingency	10,600
	30% Allied costs	31,700
	TOTAL PROJECT COST	$148,100
	O & M per year	$ 2,400

Option 2:

Construct ditch/culvert system (1200 LF)		$ 25,000
	CONSTRUCTION COST	$ 25,000
	10% Contingency	2,500
	30% Allied costs	7,500
	TOTAL PROJECT COST	$ 35,000
	O & M per year	$ 1,800

Upper Furrow Creek - North 2 (UFC-N2)

The UFC-N2 trunk is a proposed system. In Chapter 5, the UFC-N2 trunk was recommended to be an open ditch/culvert system or a closed pipe system, depending upon the ultimate development pattern.

TABLE C-7

UFC-N2 COST ESTIMATE

Option 1:		
Construct 36" pipe (2700 LF)		$349,600
	CONSTRUCTION COST	$349,600
	10% Contingency	35,000
	30% Allied costs	104,900
	TOTAL PROJECT COST	$489,500
	O & M per year	$ 5,400
Option 2:		
Construct ditch/culvert system (2700 LF)		$ 56,200
	CONSTRUCTION COST	$ 56,200
	10% Contingency	5,600
	30% Allied costs	16,900
	TOTAL PROJECT COST	$ 78,700
	O & M per year	$ 4,100

Upper Furrow Creek - North 3 (UFC-N3)

The UFC-N3 trunk is a proposed system which is recommended to be either a series of roadside ditches and culverts or a pipe system. The cost estimates are based on 48" pipe system.

There is the potential of a routing problem during layout of the trunk. For this preliminary cost estimate the trunk location was assumed to follow the route delineated on the comprehensive map bound at the back of this report.

TABLE C-8

UFC-N3 COST ESTIMATE

Option 1:

Construct 48" pipe (4200 LF)		$672,900
	CONSTRUCTION COST	$672,900
	10% Contingency	67,300
	30% Allied costs	201,900
	TOTAL PROJECT COST	$942,100
	O & M per year	$ 8,400

Option 2:

Construct ditch/culvert system (4200 LF)		$ 96,900
	CONSTRUCTION COST	$ 96,900
	10% Contingency	9,700
	30% Allied costs	29,100
	TOTAL PROJECT COST	$135,700
	O & M per year	$ 6,300

Middle Furrow Creek (MFC)

The MFC trunk was recently upgraded to pipe ranging in size from 36" to 48". However, the design capacity necessary to handle the flows calculated in this study is 78". As described in Chapter 5, it is recommended that a preliminary engineering report be initiated to analyze in depth methods of handling the additional flow. However, for purposes of this estimate the costs are based on a 30" parallel pipe system.

Also included in the cost estimate for this trunk is dollar amount associated with jack and bore crossings at the New and Old Seward Highways and the Alaska Railroad using ductile iron pipe.

TABLE C-9

MFC COST ESTIMATE

Construct 30" Parallel Pipe System (2500 LF)		$417,100
Crossings at New Seward Highway, Old Seward Highway, and the Alaska Railroad (DIP pipe) (540 LF Total)		288,400
	CONSTRUCTION COST	$705,500
	10% Contingency	70,600
	30% Allied costs	211,700
	TOTAL PROJECT COST	$987,800
	O & M per year	$ 5,300

Lower Furrow Creek (LFC)

The LFC trunk follows the Furrow Creek channel. A number of segments of this trunk are of insufficient capacity and require upgrading.

The most upstream segment of the LFC trunk is presently a channel system which is of insufficient capacity. It is recommended that the corridor be enlarged and better defined. The enlarged greenbelt/corridor is recommended to have the equivalent capacity of an 84" pipe.

When Furrow Creek crosses Clipper Ship Court, Johns Road and Mariner Drive, the existing cross culverts are grossly undersized. A bridge or plate arch pipe is recommended. This cost estimate is based upon the use of a plate pipe arch for each location with a span of 8'7" and a rise of 5'11".

The segment of the LFC trunk immediately downstream of Johns Road requires upgrading as the present stream corridor is of inadequate size. It is recommended that the stream corridor be enlarged so that its capacity is equivalent to a 78" pipe.

TABLE C-10

LFC COST ESTIMATE

Upgrade Greenbelt Corridor (2100 LF)		$ 88,000
Upgrade Cross Culverts at Clipper Ship, Mariner, and Johns Road (60 LF each)		52,000
Upgrade Creek Corridor (1000 LF)		42,000
	CONSTRUCTION COST	$182,000
	10% Contingency	18,200
	30% Allied costs	54,600
	TOTAL PROJECT COST	$254,800
	O & M per year	$ 4,900

Lower Furrow Creek - North 1 (LFC-N1)

The proposed LFC-N1 trunk is recommended to be a pipe system. Costs are based upon a 21" pipe for the entire length of the trunk route.

TABLE C-11

LFC-N1 COST ESTIMATE

Construct 21" pipe system (1400 LF)		$152,700
	CONSTRUCTION COST	$152,700
	10% Contingency	15,300
	30% Allied costs	45,800
	TOTAL PROJECT COST	$213,800
	O & M per year	$ 2,800

Subbasin M Outfalls

The three 18" outfall pipes in Subbasin M are recommended to be removed and replaced. Two of the outfalls are recommended to be 36", while the third outfall is recommended to be 48". Design features to minimize flow velocity and bluff erosion were incorporated in the cost estimate.

TABLE C-12

SUBBASIN M OUTFALLS COST ESTIMATE

Replace existing outfalls:		
2 - 36" outfalls (500 LF each)		$ 61,000
1 - 48" outfalls (500 LF each)		41,600
	CONSTRUCTION COST	$102,600
	10% Contingency	10,300
	30% Allied costs	30,800
	TOTAL PROJECT COST	$143,700
	O & M per year	$ 1,500

COST ESTIMATE SUMMARY

In Table C-13 the total project cost is tabulated by trunk system. It should again be emphasized that the type of system (e.g. pipe, ditch) used in the cost estimate was as recommended in this report. Final determination of system type will be made by the Department of Public Works.

TABLE C-13

COST ESTIMATE SUMMARY

TRUNK		TOTAL PROJECT COST (1982 DOLLARS)	O & M PER YEAR (1982 DOLLARS)
UFC-S1		$275,300	$ 9,800
UFC-S2		85,300	3,500
UFC-S3		779,700	9,800
UFC-S4		53,700	3,800
UFC-N1	Pipe	148,100	2,400
	Ditch/Culvert	35,700	1,800
UFC-N2	Pipe	489,500	5,400
	Ditch/Culvert	78,700	4,100
UFC-N1	Pipe	942,100	8,400
	Ditch/Culvert	135,700	6,300
MFC		987,800	5,300
LFC		254,800	4,900
LFC-N1		213,800	2,800
OUTFALLS (M)		143,700	1,500
Total w/pipe option		4,373,800	57,600
Total w/ditch/culvert option		3,043,500	53,600

APPENDIX D

GLOSSARY OF TERMS

APPENDIX D

GLOSSARY OF TERMS

Depression storage - The fraction of precipitation that is trapped in depressions on the surface of the ground.

Design Criteria - Guidelines upon which planning and engineering decisions and judgments are based.

Design Storm - A precipitation event that, statistically, has a specified probability of occurring in any given year (expressed either in years or as a percentage).

Detention - Temporary storage of surface runoff-either on, below or above the ground surface-accompanied by controlled release of the stored water.

Detention Basin - A stormwater detention facility, natural or artificial, which normally drains completely between space runoff events; e.g., parking lot, rooftop, athletic field, dry well, oversized storm drain pipe.

Detention Pond - A stormwater detention facility natural or artifical, which maintains a fixed minimum water elevation between runoff events except for the lowering resulting from losses of water due to infiltration or evaporation.

Drainage - Interception, collection and removal of excess stormwater from an area into another area or into a receiving water body.

Drainage Area - The area from which flow of stormwater at a given point is derived. Since water flows downhill, water from a given drainage area will collect and flow through an outlet point. Drainage basins are subdivided into subbasins and further divided into subcatchments.

Easement - A right to control or use the property of another for designated purposes.

Event - An individual occurrence of precipitation or snowmelt.

Excess Runoff - Direct surface runoff that cannot be accommodated satisfactorily by the existing or planned drainage system.

Flow Routing - Path of travel of runoff through the drainage area.

Flood Control - Preventing the entry of stormwater into an area from another area, or from a stream or other water body.

Grade - The slope of a road, channel, or natural ground.

Hydrograph - A graph of runoff rate, inflow rate or discharge rate, versus time.

Hyetograph - A graph of intensity of a precipitation event versus time.

Infiltration - The process whereby water enters the surface of the soil and moves downward toward the water table.

Institutional Problems - Social, economic and political problems existing within or between agencies, organizations or groups-either public or private.

Intensity - The rate at which a precipitation event occurs, expressed as a depth of water per unit of time, such as inches of rain per hour.

Interception - Rainfall that is caught by foliage, branches, leaves, and other above-ground objects.

Invert - The lowest part of the internal cross section of a channel or conduit.

Lag - The time interval from the center of mass of excess rainfall to the peak rate of runoff.

Local Detention - Temporary storage of runoff on the same land development site where the runoff is generated-frequently required as a condition for subdivision plat approval.

Master Planning - A "systems" approach to the planning of facilities, programs and management organizations for comprehensive control and use of stormwater within a defined geographical area.

Minor Drainage System - The conveyance drainage system consisting of street gutters, storm sewers, small open channels, and swales, etc.

Off-stream Detention - Temprary storage accomplished off-line; i.e., not within a principal drainage system.

Outfall - The conduit through which water is discharged to a watercourse.

Percolation - The downward movement of water through soil.

Pollutant Loading - The arithmetic product of the pollutant concentration and the runoff rate.

Pollutograph - A graph of pollutant loading versus time (commonly referred to as a "mass emission pollutograph"). Units are kg/day or lbs/day, etc.

Ponding - The occurrence of excessive depths of stormwater after a rainfall or snowmelt event.

Precipitation - A basic part of the hydrologic cycle, precipitation is the falling of water in the form of rain, snow, sleet, or hail.

Receiving Waters - Streams, lakes, bays, etc., into which stormwaters are discharged.

Recurrence Interval - The average interval of time within which the magnitude of a given event is likely to be equalled or exceeded.

Regional Detention - Temporary storage of runoff for a large drainage area.

Retention Facility - Any type of detention facility not provided with a positive outlet.

Runoff - Water flowing overland or in a stream channel past any given section.

Sedimentation - The process of depositing particles of waterborne soil, rock, or other materials.

Storm Sewers - Usually, enclosed conduits that transport excess stormwater runoff toward points of dishcarge (sometimes called "storm drains").

Stormwater Management - Encompasses both "control" and "developmental" activities in which there is physical interaction with stormwater (a broader interpretation includes activities of an institutional nature - financing, staffing, etc.).

Stormwater Storage - Temporary storage of excess runoff on, below, or above the surface of the earth for the purpose of attenuating excess runoff.

Subbasin - A portion of a complete drainage area delineated by concentration of flow at a certain point, which contributes in turn to flows in the overall drainage area.

Subcatchment - A portion of a subbasin delineated by a concen-. tration of flow at a certain point.

Time of Concentration - The time period necessary for surface runoff to reach the outlet of a subbasin from the hydraulically most remote point in the tributary drainage area.

Transpiration - The process whereby moisture circulates through the structure of plants and is returned to the atmosphere.

Travel Time - The sum of the time intervals for overland flow, sewer or gutter flow, and pipe and channel flow from the hydraulically most remote point in the tributary to the discharge point of interest.

Trunk System - Major conveyance network which has the capacity to handle large areas.

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REFERENCES

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