City of Dubuque, Iowa

Drainage Basin Master Plan

THE CITY OF
DUBUQUE
Showing the Spirit

Fall 2001

HDR

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TECHNICAL APPENDICES

The primary objectives of this Drainage Basin Master Plan are to address the issue of storm water conveyance in major streams, identify existing and future problem areas, and develop recommendations for solutions to specific problem areas. The North Fork Catfish Creek and Bee Branch Drainage Basins were the only two drainage basins analyzed with the described goals.

Specifically, the objectives of this Master Plan include:

- Determining capacity of existing drainage system under ultimate development conditions for the 10-, 50-, 100- and 500-year return period storm events;
- Developing hydraulic models using aerial topographic mapping and GIS information for major drainage segments on North Fork Catfish Creek and the Bee Branch main trunk line storm sewer;
- Identifying problem areas in the stream segments studies and developing improvement plans for specific problem areas;
- Addressing water quality in a qualitative nature by developing a list of possible Best Management Practices (BMPs); and
- Identifying potential funding sources for improvement plans.

A total of nine (9) problem areas located within the North Fork Catfish Creek Drainage Basin were identified as out of compliance with the City's drainage standards/criteria. The majority of these problem areas are associated with limited hydraulic capacity of existing detention cells, natural channels, and culverts. The total cost for implementation of recommended improvements in the North Fork Catfish Creek Drainage Basin is estimated to be $1,673,000.

The Bee Branch Drainage Basin is composed of five (5) major subareas: West 32nd Street, Kaufmann Avenue, Locust Street, Central Business District – North, and Central Business District. Most of the specific problem areas identified in the Bee Branch Drainage Basin were located within the West 32nd Street Subarea. Seven (7) problem areas in the West 32nd Street Subarea, including one special problem area, exceeded the established design criteria. Most of the flooding problem areas are the result of limited hydraulic capacity of drainage structures. The West 32nd Street Subarea also was identified as a primary factor in the flooding hazards encountered in the low-lying, heavily developed area located in the lower portion of the Bee Branch Drainage Basin, also known as the Couler Valley area. The West 32nd Street Subarea was recognized as offering the best opportunity for storm water storage within the Bee Branch Drainage Basin; therefore, the recommended improvements focused on providing additional storage for storm water runoff. The total estimated capital cost for execution of the recommended improvements in the West 32nd Street Subarea is approximately $4,700,000. An itemized list of improvements can be found in Table 4.12 on page 4-19.

The primary drainage problems within the Kaufmann Avenue, Locust Street, Central Business District – North, and Central Business District Subareas have occurred where development has exceeded the capacity of the storm water conveyance system. The only viable detention storage option for these subareas was a small detention cell in the Kaufmann Avenue Subarea. The estimated capital cost for this detention cell is approximately $530,000.

Flooding problems in the upper portion of the main Bee Branch storm sewer trunk line, north of 24th Street, are greatly improved with the West 32nd Street Subarea improvements; however, the convergence of flood flows from Kaufmann, Locust, and Central Business District – North Subareas still result in significant flooding depths in the lower part of the drainage area south of 24th Street. The only alternative that significantly reduced 100-year flooding depths in the lower reaches of the Bee Branch was a flood control channel. This alternative consists of constructing a 150-foot wide, flood control channel to carry the flow of a 100-year flood event. Construction of this channel from the 16th Street Detention Cell to 24th Street was shown to remove approximately 99% of the homes and businesses from the 100-year floodplain along the main Bee Branch storm sewer trunk line, while requiring the purchasing or relocation of approximately 70 homes and/or businesses. Estimated cost for this alternative is $17.1 million.

1.0 INTRODUCTION

1.1 PROJECT BACKGROUND

The City of Dubuque (City) is located in the eastern portion of Dubuque County in eastern Iowa. The corporate limits of the City cover approximately 25 square miles and include a population of approximately 57,000 people. The City is located on the west or right bank of the Mississippi River and is characterized by numerous outcrops of limestone and steep slopes in the upland areas and generally flat low-lying floodplains in the lowlands. Earthen levees and floodwalls offer protection to the city against a Mississippi River flood. Along the river, numerous temporary storage sites are filled with storm water during a storm event and discharge into the Mississippi River through gravity outlets or pump stations when gravity drainage is not possible.

The streams and channels existing in the City of Dubuque predominately originate within the corporate limits and flow easterly to the Mississippi River. The City is principally drained by the Bee Branch Drainage Basin (Bee Branch), North Fork Catfish Creek Drainage Basin (North Fork), and their tributaries. The Bee Branch flows through the north end of the city and consists of several large tributary drainage areas including West 32nd Street, Kaufmann Avenue, Locust Street, Central Business District – North, and Central Business District. The low-lying, heavily developed areas located in the Central Business District – North and Central Business District are hereafter referred to as the Couler Valley area, while the North Fork Catfish Creek Drainage Basin consists primarily of one main channel with several small tributary drainage areas. The Bee Branch Drainage Basin flows into the 16th Street Detention Cell adjacent to the Mississippi River, and the North Fork Catfish Creek Drainage Basin empties into Middle Fork Catfish Creek. The Bee Branch and North Fork Catfish Creek Drainage Basins drain a total of 11.0 square miles and were identified by the City of Dubuque as the focus of the study. The contributing drainage areas of the Bee Branch and North Fork are shown in Figure 1-1.

Flooding periodically occurs along the streams and streets in Dubuque, with flood damage to streets, homes and businesses. As the city enjoys sustained growth through the years, runoff rates and flooding problems are likely to increase in many areas due to continued conversion of rural lands to urban uses.

A review of the rainfall records for the City of Dubuque shows that storms exceeding the magnitude of a 50-year and 100-year return period have occurred in the past and will likely continue to occur in the future. Daily rainfall has been recorded at the Dubuque Airport since 1896 and at Check Dam 11 located on the Mississippi River since 1937. Table 1.1 presents a summary of the ten greatest 24-hour rainfall measurements at the two (2) stations. It is noted that the maximum 24-hour rainfall events may not be taken within the same 24-hour period for each of the rainfall stations, such as the September 14, 1967 storm event.

Table 1.1
Summary of Maximum 24-Hour Rainfall Events for Dubuque, Iowa

	Dubuque Airport Station Nos. 132369 and 132367 (1896-2000)		Check Dam 11 Station No. 132364 (1937-2000)	
Rank	**Date**	**24-Hour Rainfall (inches)**	**Date**	**24-Hour Rainfall (inches)**
1	September 14, 1967	8.85	August 2, 1972	5.27
2	July 1, 1961	6.28	May 13, 1978	4.50
3	November 2, 1961	4.79	September 13, 1972	4.48
4	May 6, 1960	4.37	September 14, 1967	4.04
5	September 12, 1961	4.37	June 13, 1947	3.88
6	July 8, 1951	4.36	July 30, 1987	3.86
7	August 16, 1918	4.26	May 29, 1962	3.64
8	July 17, 1977	3.91	August 7, 1970	3.40
9	July 5, 1993	3.91	August 27, 1965	3.35
10	June 13, 2000	3.84	June 26, 1969	3.33

Urban development within a drainage area generally results in an increase in the percent impervious, i.e., more hard surfaces, with a concurrent increase in runoff associated with any given storm event. Therefore, stream channels and culverts that were adequate prior to urbanization may become inadequate as the drainage area develops. This results in more frequent stream channel flooding and backwater flooding from culverts unable to convey the higher discharges. The City of Dubuque addresses these problems, as funds allow, through street and drainage improvement projects.

1.2 PROJECT OBJECTIVES

This Drainage Basin Master Plan addresses the issue of storm water conveyance in major streams, the identification of existing and future problem areas that do not meet drainage criteria and the development of recommendations for solutions to specific problem areas. The primary objectives of this Drainage Basin Master Plan are the following:

1. Determine capacity of existing drainage system under ultimate development conditions for the 10-, 50-, 100- and 500-year return period storm events;

2. Develop hydraulic models using aerial topographic mapping and GIS information for major drainage segments on North Fork Catfish Creek and the Bee Branch main trunk line storm sewer;
3. Identify problem areas in the stream segments studies and develop improvement plans for specific problem areas;
4. Address water quality in a qualitative nature by developing a list of possible Best Management Practices (BMPs); and
5. Identify potential funding sources for improvement plans.

This Drainage Basin Master Plan addresses existing and projected flooding within the drainage areas. Portions of the drainage areas have been included in previous Federal Emergency Management Agency (FEMA) studies. While FEMA flood insurance studies are the official regulatory document for floodplain identification within Dubuque, they are lacking in three (3) areas: 1) they are based on very coarse hydrologic information, 2) they do not include drainage areas smaller than 1 square mile, and 3) they do not consider the impacts of ultimate development patterns. The Master Plan addresses these deficiencies by using more detailed hydrologic techniques. While not regulatory, the floodplains delineated in this Drainage Basin Master Plan are a more accurate representation of expected floodplains for planning purposes. Lastly, the floodplain delineation noted above includes considerations of ultimate development patterns.

Specifically, this Master Plan identifies the anticipated future hydrology (rainfall and runoff) for the drainage area considering reasonable land use changes based on ultimate development. Problem areas were identified for the existing system without any improvements at the future flows. Alternatives were evaluated and solutions recommended based on ultimate flows.

This Master Plan addresses water quality in a qualitative nature within the drainage areas. The levels of pollutants typically associated with urban runoff were not calculated. Although it was not directly addressed in this plan, implementation of best management practices (BMPs) and impacts on water quality were considered in the analysis of alternatives.

1.3 ORGANIZATION OF THE DRAINAGE BASIN MASTER PLAN

The Drainage Basin Master Plan is divided into five (5) main sections. Section 1 is the introduction. Section 2 provides a description of the methodologies used in the performance of this study including a description of the flood hydrology and stream hydraulic models, a discussion of the drainage criteria applied, and a description of the methods for the development of drainage cost improvement estimates. Sections 3 and 4 include the individual sub-section of each of the two (2) major drainage basins in Dubuque including North Fork Catfish Creek and the Bee Branch Drainage Areas. The Bee Branch Drainage Area includes: West 32nd Street, Kaufmann Avenue, Locust Street, Windsor Avenue, 8th Street, 11th Street, 15th Street, Lower and Upper Kerper and Dock Street and Hamilton Street Subareas. Each drainage basin sub-section include a description of the general characteristics of the drainage area, flood hydrology results, hydraulic capacity of roadway crossings, identification of problem areas, conceptual improvement plans to mitigate flooding in the problem areas and capital cost estimates for each improvement project. A ranking of the problem areas for each of the individual drainage basins was prepared to establish priorities for implementation of proposed projects. Although numerous criteria could be used to establish priorities for implementation of the proposed projects, the following criteria (arranged in order of decreasing importance) were considered:

- Severity of existing problem;
- Public safety;
- Capital cost;
- Preserving/enhancing existing property values;
- Development potential;
- Social/economic impacts; and
- Maintenance/operating costs.

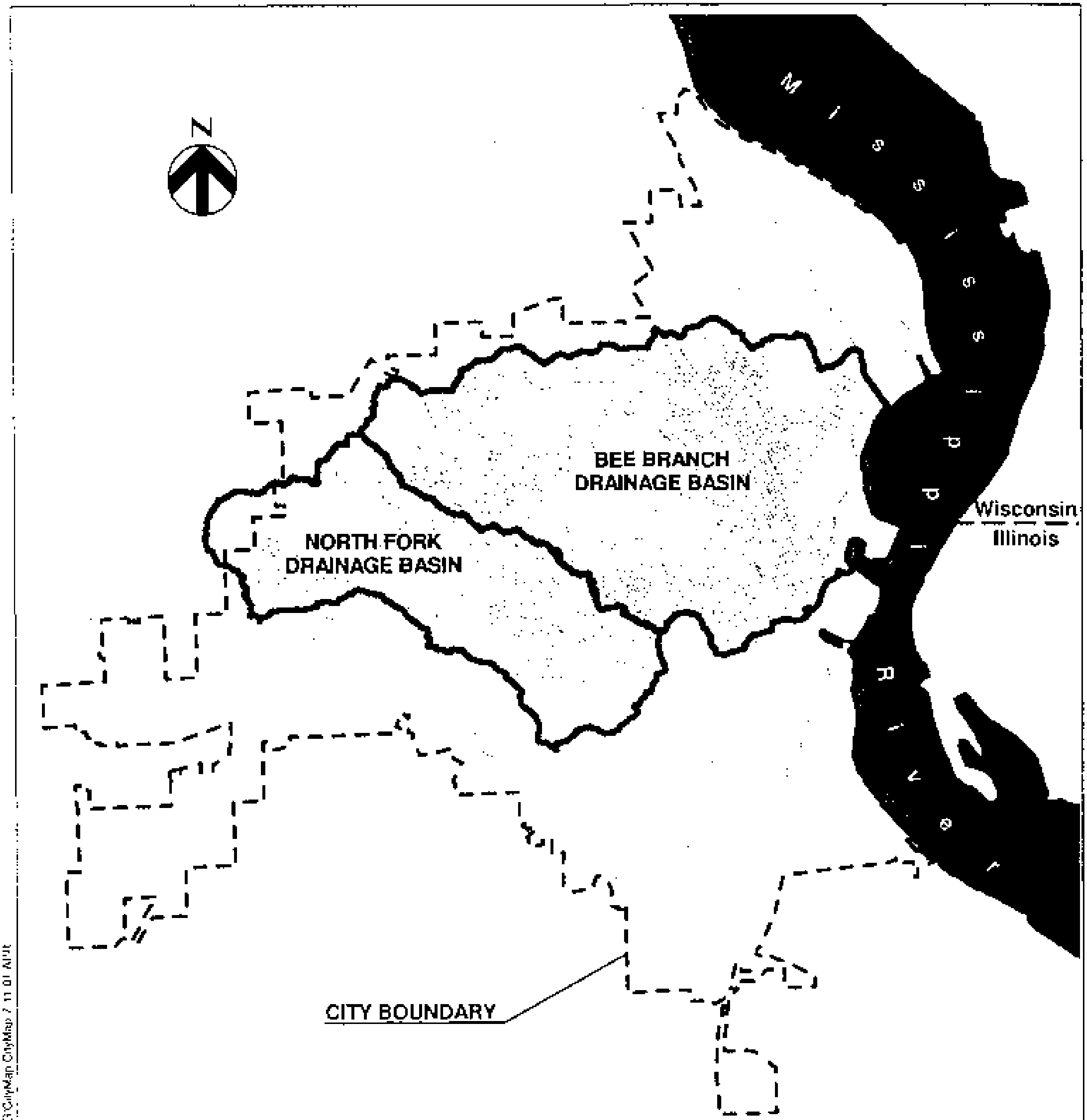
Flooding of residential, commercial and industrial buildings was given the highest priority for implementation of improvement projects. Roadway crossings failing to meet the drainage criteria were prioritized for improvement based on apparent traffic volumes, availability of alternate routes that are passable during flood events, frequency and degree of overtopping, and cost efficiency for mitigating the flooding problem. Section 5 addresses the financing of drainage improvements and operations.

1.4 NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM

The 1987 Amendments to the Clean Water Act recognized urban runoff as a major contributor to the Nation's water quality problem. Thereafter, storm water issues became as closely allied with water quality issues as they had been previously associated with flood control. In other words, *quality* became as important as *quantity*. In 1990, the U.S. Environmental Protection Agency (EPA) promulgated Phase I of the National Pollutant Discharge Elimination System (NPDES) permit coverage to address storm water runoff from "medium and large" municipal separate storm sewer systems (MS4s). Storm Water Phase II program is the next phase of EPA's effort to preserve, protect and improve the Nation's water resources from polluted storm water runoff.

Dubuque meets the definition of a small (MS4) sized municipality (population more than 50,000 but less than 100,000). The deadline for submittal of permit applications for Phase II designated small MS4s is March 10, 2003.

Six measures are to be included in a storm water management program to meet the conditions of its NPDES permit and include: 1) public education and outreach; 2) public participation/involvement; 3) illicit discharge detection and elimination; 4) construction site runoff control; 5) post-construction storm water management in new development and redevelopment and 6) municipal pollution prevention/good housekeeping. These measures comprise the range of Best Management Practices (BMPs) available to a municipality for the reduction of negative impacts resulting from storm water runoff. BMPs are defined as schedules of activities, prohibitions of practices, maintenance procedures, and other physical, structural, and/or managerial practices to prevent or reduce the pollution of waters of the United States.



Source: Dubuque Area Geographic Information System (DAGIS), dated May 2000

Drainage Basins Studied

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Drainage Basin Master Plan
City of Dubuque, Iowa

Figure
1-1

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2.0 METHODOLOGIES

Flood hydrology models were developed for each individual drainage basin, incorporating the unique characteristics of each basin to simulate runoff for specific storm events. Stream hydraulic models were developed for the segments included in this study incorporating the channel and floodplain geometry derived from aerial topographic maps, roughness characteristics of channel banks and floodplains and the numerous bridges and culverts that cross the streams and affect flood levels. The following sections describe the methodologies used in this study.

2.1 PHYSICAL CHARACTERISTICS

2.1.1 Topography

Topography of a drainage area refers to the characteristics and features of the land surface, such as slope and channel width. The slope of a drainage area influences the rate at which precipitation falling on the land surface will be conveyed to the outlet point of the drainage area. All other parameters considered equal, as the slope of a drainage area increases, the faster the water travels to the outlet point. Although there can be a great deal of variation in slope magnitude and direction within a drainage area, there are two main slope values of particular interest: 1) average overland slope and 2) average channel slope. Overland slope gives an indication of how fast runoff will travel on the land surface to a drainage channel, and channel slope relates how quickly the runoff will be routed to the outlet point of the drainage area. Drainage areas within the City typically have a much steeper overland slope than channel slope.

Elevation measurements and slope calculations were performed using the Dubuque Area Geographic Information System (DAGIS). The DAGIS included a digital terrain model (DTM) consisting of spot elevations and breaklines generated from aerial survey and ground control data. Two-foot elevation contour lines created from the DTM were also included in the DAGIS database. The DTM was used to produce two additional terrain models for use in the analysis. A triangular irregular network (TIN) terrain model, a continuous surface comprised of triangular faces, was created for use in calculating detention volumes, cutting stream cross-sections, and creating open channel hydraulic models. A digital elevation model (DEM), a grid comprised of 10-meter cells, was created from the TIN for use in delineating drainage areas, estimating hydrologic parameters, and creating hydrologic models.

2.1.2 Soil Types

The types of soils present in a drainage area have a significant impact on the amount of runoff a given storm will produce. This impact is influenced primarily by the infiltration characteristics of the soil.

Information on the soil types and characteristics for each drainage area was compiled by developing a digital soils database in GIS. Soil survey SSURGO and SATSGO databases developed by the Natural Resources Conservation Service (NRCS) were used. The SSURGO data set was used to provide specific information about each soil series within the drainage areas. Because the majority of the soils in the Dubuque area are classified as hydrologic soil group 'B,' the less detailed STATSGO database was used to develop hydrologic models. This information was then combined with land use data to obtain hydrologic characteristics for each polygon.

2.2 URBAN DEVELOPMENT CHARACTERISTICS

2.2.1 Land Use

Land use is a critical element for storm water planning. It impacts both the quantity and quality of water being routed through storm sewer systems and natural channels. The effect land use has on water quantity is generally linked to the amount of impervious area for a particular land use category. The more impervious area a tract of land has, the faster the water will be routed to the storm sewer system or channel due to lower infiltration losses into the ground and lower surface roughness of the land. In general, an area with a high percentage of impervious area will have a quicker time to peak and a higher peak, than a similar area with a lower percentage impervious.

The scope of this project was to model storm water quantity for ultimate development, so a land use database containing information for ultimate development was created. Ultimate land use was based on the City's comprehensive land use plan and supplemented with land use projections made by City personnel. The landuse categories within the drainage basins are shown in Table 2.1.

Table 2.1
Drainage Basin Land Use Groups

Land Use Group	Description
ST	Streets
CO	Commercial
IND	Industrial
INS	Institutional
HD	High Density Residential
MD	Medium Density Residential
LD	Low Density Residential
AG	Agricultural
OP	Open Space and Grass

2.3 HYDROLOGIC MODEL

The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) Hydrologic Modeling System (HEC-HMS) was selected to model the drainage areas in the city of Dubuque. HEC-HMS simulates precipitation-runoff processes of dendritic drainage systems. HEC-HMS computes runoff volume by computing the volume of water intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation.

HEC-HMS is designed to simulate the surface runoff response of a drainage basin to precipitation input. The model represents the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin commonly referred to as a subbasin. A component may be a surface runoff entity, a stream channel, or a reservoir. The result of the modeling is the computation of stream flow hydrographs at desired locations in the drainage area.

NRCS methodology was used to determine runoff volumes, direct runoff and channel routing. The advantage of the NRCS methodology is it converges quickly, resulting in a very stable model. Additionally, the input parameters are more commonly known and understood, resulting in easier applications. The disadvantage is the results are not as accurate as for non-linear routing, and differing land uses can only be accounted for via the runoff curve number. In the Drainage Basin Master Plan analysis, the NRCS methodology was used.

Key data required by the HEC-HMS model include:

- Drainage basin area;
- Precipitation depths;
- Runoff curve number;
- Unit hydrograph and basin lag time;
- Design storm characteristics; and
- Channel and reservoir routing parameters.

2.3.1 Model Schematic

HEC-HMS dynamically routes storm water through open channels. Hydraulic routing through drainage systems requires a mathematical framework from which numerical calculations can take place. HEC-HMS uses a link-node concept to idealize real-world systems. This concept requires a network of nodes or junctions and links or reaches represent the drainage system. A node is a discrete location in the drainage system where conservation of mass or continuity is maintained. Links are the connections between nodes and are used to transfer or convey water through the drainage system. The following general guidelines were used to locate nodes in the drainage area schematic:

1. Upstream and downstream of any structure (e.g., culverts, weirs, etc.);
2. Ponds and lakes (specifically storage nodes);
3. Channel junctions;
4. Downstream boundary;
5. Where channel geometry changes abruptly;
6. Where the channel bed slope changes abruptly; and
7. Where major surface inflows to the conveyance system.

By following the general guidelines, a schematic diagram of the drainage area conveyance system was developed. The drainage area drainage areas were delineated and subdivided using the DAGIS mapping. The two-foot contour interval on the GIS mapping provided useful information in determining the major drainage area divides and subbasin delineation. The drainage area was segmented into subbasins based on selected design points.

2.3.2 NRCS Runoff Curve Number

The Natural Resources Conservation Service (NRCS) runoff curve number procedure was used to compute abstractions for storm rainfall. Abstractions are defined as the physical process (such as soil infiltration and detention or retention by vegetation), which effectively reduces the volume of precipitation, which becomes runoff. The rainfall in excess of the abstractions becomes runoff and is referred to as excess rainfall. Excess rainfall is always less than or equal to the depth of precipitation. The curve number is a function of land use, soil type, condition of cover, and antecedent moisture condition. This information was used in conjunction with information from the Dubuque County Soil Survey, GIS mapping and city's drainage standards/criteria to develop a runoff curve number for each subbasin. The soils are generally characterized as hydrologic soil group 'B', which have moderate infiltration rates if thoroughly wetted, and consisting of deep or well drained soils with moderately fine to coarse textures. The average antecedent moisture condition (AMC-II) was assumed. The curve numbers are based on the tables published by the NRCS in Technical Report 55 (TR-55). Table 2.2 summarizes the land use classification and its respective curve number.

In subbasins where development is partially or fully developed, the hydrologic analysis was performed for ultimate land use development. In subbasins where agricultural development was present, the hydrologic analysis was performed as agricultural land use, because developers are required to provide on-site detention to maintain existing runoff releases.

Table 2.2
Drainage Area Land Use Groups and Curve Number

Land Use Group	Description	NRCS Curve Number
ST	Streets	99
IND	Industrial	88
CO	Commercial	92
INS	Institutional	88
HD	High Density Residential	85
MD	Medium Density Residential	75
LD	Low Density Residential	72
AG	Agricultural	73
OP	Open Space and Grass	69

2.3.3 NRCS Unit Hydrograph

The unit hydrograph method is the component in the rainfall-runoff model that transforms the rainfall excess into a surface runoff hydrograph. The unit hydrograph represents a typical hydrograph shape for a drainage area. The unit hydrograph for a drainage area is defined as a direct runoff hydrograph resulting from one inch of excess rainfall generated uniformly over the drainage area at a constant rate for a storm of a specified duration.

The NRCS unit hydrograph method relates hydrograph characteristics to a physical characteristic of the drainage area, the basin time to peak, t_p. The basin time to peak is defined as the time from the beginning of the rainfall event to the time at which the peak runoff rate is observed at the drainage area outlet. The time to peak can be estimated using the following empirical equation:

$$t_p = \frac{\Delta t}{2} + t_{lag}$$

where: t_p = time to peak, in hours
Δt = computational interval, in hours
t_{lag} = lag time, in hours

The lag time is defined as the time difference between the center of mass of the rainfall excess and the peak of the unit hydrograph. Lag times for each subbasin within the drainage area were computed by applying the curve number method in the GIS analysis. The lag time is give by the following equations:

$$t_{lag} = \frac{L^{0.8}(S+1)^{0.7}}{1900\ Y^{0.5}}$$

$$S = \frac{1000}{CN} - 10$$

where: t_{lag} = lag time, in hours
L = greatest flow length, in feet
Y = average drainage area slope, in percent
CN = runoff curve number, based on land use, land treatment and soil type

The NRCS unit hydrograph method was utilized in the HEC-HMS model for the drainage basins in the study.

2.3.4 Rainfall

The 24-hour rainfall depths for the 10-, 50-, 100- year frequency shown in Table 2.3 were based on the point (station) data and developed as isohyetal maps presented in the Midwestern Climate Center and Illinois State Water Survey publication, Bulletin 71, "Rainfall Frequency Atlas of the Midwest". The point data values are higher than the areal mean relations determined for each climatic section in the state of Iowa. The hydrologic analyses were conducted using the higher, more conservative point data values. The 500-year rainfall depth was extrapolated from the 10-, 50- and 100-year values. Area rainfall reduction factors were not used to reduce the point rainfall depth because the drainage areas were less than 10 square miles.

Table 2.3
City of Dubuque 24-Hour Total Rainfall Depths

Return Period	Rainfall Depth (inches)
10-Year	4.5
50-Year	6.0
100-Year	7.0
500-Year	11.0

In order to calibrate the hydrologic model, a comparison of the basin runoff to other hydrologic methods was made. An observed hydrograph, depicting flow rates over time, was not available for any storm events to calibrate; therefore, another method was sought. Hydrologic analysis has been conducted for Catfish Creek and its tributaries in the 1989 Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS). The North Fork of Catfish Creek was the first drainage basin to be studied; therefore, a comparison of the 100-year FIS results and 100-year HEC-HMS results were evaluated. North Fork of Catfish Creek is an ungaged stream, so synthetic methods were used to obtain the discharge-frequency relationships in the FIS. In addition, the Iowa Department of Transportation's (IaDOT) regression equations were compared. A summary of 100-year peak discharges is shown in Table 2.4.

Table 2.4 North Fork Catfish Creek Peak 100-Year Discharge Comparison at Confluence with Middle Fork Catfish Creek		
Peak Discharge Source	**Runoff Peak (cfs)**	**Comments**
IaDOT	2,500-3,140	Developed for rural Iowa drainage basins.
FEMA-FIS	3,600	Flood Insurance Study using regression equations. Based on existing land use conditions.
HEC-HMS	2,950	Existing land use conditions with no effective storage. Type-II distribution.
HEC-HMS	3,200	Existing land use conditions with no effective storage. Modified Type-II rainfall distribution.

From Table 2.4, the IaDOT results are lower then FIS or HEC-HMS results. It is because the IaDOT equations were derived for rural drainage basins and urban effects are not recognized. In order to simulate the FIS discharges, modifications to the NRCS Type-II rainfall distribution were made. The modification was performed to account for the quick runoff response of Dubuque soils. The hyetograph for each basin was developed using a 15-minute time increment and a modification of the NRCS Type-II rainfall distribution by including the 6-hour rainfall hyetograph within the 24-hour hyetograph. This technique maintained the depth and timing of the 24-hour storm while incorporating the intensity of the 6-hour storm. Table 2.5 tabulates the modified distribution. This modification produced favorable discharges to the FIS discharges.

Table 2.5 City of Dubuque 15-Minute Time Distribution for 24-Hour Storm Event									
	Return Period					Return Period			
Time Interval (hours)	10-Yr	50-Yr	100-Yr	500-Yr	Time Interval (hours)	10-Yr	50-Yr	100-Yr	500-Yr
0	0.000	0.000	0.000	0.000	8	0.017	0.023	0.030	0.053
0.25	0.007	0.009	0.012	0.021	8.25	0.021	0.027	0.036	0.064
0.5	0.010	0.014	0.018	0.032	8.5	0.024	0.032	0.042	0.074
0.75	0.010	0.014	0.018	0.032	8.75	0.024	0.032	0.042	0.074
1	0.010	0.014	0.018	0.032	9	0.024	0.032	0.042	0.074
1.25	0.010	0.014	0.018	0.032	9.25	0.028	0.036	0.058	0.085
1.5	0.010	0.014	0.018	0.032	9.5	0.042	0.056	0.063	0.085
1.75	0.010	0.014	0.018	0.032	9.75	0.042	0.056	0.063	0.096
2	0.010	0.014	0.018	0.032	10	0.046	0.061	0.068	0.096
2.25	0.010	0.014	0.018	0.032	10.25	0.056	0.075	0.084	0.112
2.5	0.010	0.014	0.018	0.032	10.5	0.056	0.075	0.084	0.127
2.75	0.010	0.014	0.018	0.032	10.75	0.070	0.094	0.105	0.159
3	0.010	0.014	0.018	0.032	11	0.070	0.094	0.105	0.191
3.25	0.010	0.014	0.018	0.032	11.25	0.095	0.127	0.142	0.223
3.5	0.010	0.014	0.018	0.032	11.5	0.119	0.160	0.179	0.276
3.75	0.010	0.014	0.018	0.032	11.75	0.193	0.259	0.289	1.104
4	0.014	0.018	0.024	0.042	12	1.495	2.007	2.242	2.930
4.25	0.014	0.018	0.024	0.042	12.25	0.396	0.531	0.593	0.791
4.5	0.014	0.018	0.024	0.042	12.5	0.172	0.230	0.257	0.343
4.75	0.014	0.018	0.024	0.042	12.75	0.108	0.146	0.163	0.244
5	0.014	0.018	0.024	0.042	13	0.087	0.118	0.131	0.191
5.25	0.014	0.018	0.024	0.042	13.25	0.077	0.103	0.116	0.159
5.5	0.014	0.018	0.024	0.042	13.5	0.066	0.089	0.100	0.138
5.75	0.014	0.018	0.024	0.042	13.75	0.056	0.075	0.084	0.117
6	0.014	0.018	0.024	0.042	14	0.053	0.071	0.079	0.106
6.25	0.017	0.023	0.030	0.053	14.25	0.042	0.056	0.063	0.096
6.5	0.017	0.023	0.030	0.053	14.5	0.042	0.056	0.063	0.085
6.75	0.017	0.023	0.030	0.053	14.75	0.042	0.056	0.063	0.084
7	0.017	0.023	0.030	0.053	15	0.039	0.052	0.058	0.077
7.25	0.017	0.023	0.030	0.053	15.25	0.024	0.032	0.042	0.074
7.5	0.017	0.023	0.030	0.053	15.5	0.021	0.027	0.036	0.064
7.75	0.017	0.023	0.030	0.053	15.75	0.021	0.027	0.036	0.064

Table 2.5
City of Dubuque 15-Minute Time Distribution for 24-Hour Storm Event

	Return Period					Return Period			
Time Interval (hours)	10-Yr	50-Yr	100-Yr	500-Yr	Time Interval (hours)	10-Yr	50-Yr	100-Yr	500-Yr
16	0.021	0.027	0.036	0.064	20.25	0.010	0.014	0.018	0.032
16.25	0.021	0.027	0.036	0.064	20.5	0.010	0.014	0.018	0.032
16.5	0.021	0.027	0.036	0.064	20.75	0.010	0.014	0.018	0.032
16.75	0.017	0.023	0.030	0.053	21	0.010	0.014	0.018	0.032
17	0.017	0.023	0.030	0.053	21.25	0.010	0.014	0.018	0.032
17.25	0.017	0.023	0.030	0.053	21.5	0.010	0.014	0.018	0.032
17.5	0.017	0.023	0.030	0.053	21.75	0.010	0.014	0.018	0.032
17.75	0.017	0.023	0.030	0.053	22	0.010	0.014	0.018	0.032
18	0.014	0.018	0.024	0.042	22.25	0.010	0.014	0.018	0.032
18.25	0.014	0.018	0.024	0.042	22.5	0.010	0.014	0.018	0.032
18.5	0.014	0.018	0.024	0.042	22.75	0.010	0.014	0.018	0.032
18.75	0.014	0.018	0.024	0.042	23	0.010	0.014	0.018	0.032
19	0.014	0.018	0.024	0.042	23.25	0.010	0.014	0.018	0.032
19.25	0.014	0.018	0.024	0.042	23.5	0.010	0.014	0.018	0.032
19.5	0.014	0.018	0.024	0.042	23.75	0.010	0.014	0.018	0.032
19.75	0.014	0.018	0.024	0.042	24	0.007	0.009	0.012	0.021
20	0.010	0.014	0.018	0.032	**Totals**	4.5	6.0	7.0	11.0

2.3.5 Channel Routing

Routing of flood flows from the outlet of an upstream subbasin to the next subbasin outlet was accomplished using the Muskingum routing method in HEC-HMS. Data input for the Muskingum consists of a storage correlation coefficient and a travel time for a reach. The storage correlation coefficient is a measure of how closely storage in the reach is related to outflow. Based on sensitivity analyses performed during the project it was shown to be a relatively insensitive variable. A value of 0.2 was used throughout the study area. The travel time through a given reach was calculated using GIS and based on an assumed velocity of 3.3 feet per second (1 meter per second).

2.3.6 Reservoir Routing

Reservoir routing was included in the model to account for the flood attenuation effects associated with roadway storage and existing and potential detention basins. The HEC-HMS Modified Puls routing routines were used to simulate flow through the reservoirs using the level pool routing procedure. This procedure assumes the reservoir water surface remains effectively level during the routing. Stage-storage-discharge relationships were developed where storage was effective by computing a stage-outflow relationship and combining it with the stage-storage relationship for the upstream reservoir pool. The stage-storage relationship was derived from GIS mapping. Stage-discharge rating tables were developed using information on the outlet works facilities obtained in the field. Assuming inlet control, a stage-discharge relationship was generated using nomographs contained in the Federal Highway Administration's (FHWA) Hydraulic Design Series No. 4 (HDS-4).

2.3.7 CRWR-PrePro

A preprocessor was developed by the Center for Research and Water Resources (CRWR) at the University of Texas, Austin, under the supervision of Dr. David Maidment. CRWR-PrePro was used to develop the input data for the hydrologic model.

CRWR-PrePro is a GIS preprocessor for the Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HEC-HMS). HEC-HMS is currently being developed by HEC as part of the NexGen program of research. The purpose of CRWR-PrePro is to summarize data from a GIS system for input to HEC-HMS. CRWR-PrePro uses stream and subbasin GIS layers as input data. Stream and subbasin data layers are required as input, and the software requires the use of metric units. The CRWR-PrePro analysis was executed using metric units and then the output data, consisting of a HEC-HMS basin file, was converted to English units. The system is written in ArcView Avenue programming language (Version 4.0.av).

The data sets must be in the same geographic coordinate system, and the input data must accurately describe the hydrologic properties of the area. Errors occur due to discrepancies among the stream and subbasin data layers.

The program code is oriented around identifying hydrologic elements and the relationship between these elements. Seven (7) hydrologic elements are identified: subbasins; sources; reaches; junctions; reservoirs; diversions and sinks.

The step-by-step methodology for developing a HEC-HMS basin file using CRWR-PrePro is presented below. These steps produce a HEC-HMS basin file, which is then imported into HEC-HMS.

Table 2.6 Spatial Data for CRWR-PrePro	
Data Set	**Description**
DEM	Digital elevation model (DEM), 10 meter grid of elevations describing topography, developed from the Digital Terrain Model (DTM) within the DAGIS dataset.
Rf	Shape file of streams or reaches developed by the EPA, augmented by DAGIS data.
LU	Land use shape file developed from DAGIS data.
STATSGO	State Soil Geographic database, soil classifications, developed by U.S. Geologic Survey
Aerial Photos	Aerial photography used for identifying structures and other features.

1. Develop a GIS Database- Spatial data representing the basin and streams is compiled in an ArcView project file. The required spatial data sets are shown in Table 2.6.
2. Intersect the stream shape file with the DEM to assure the streams delineated from the DEM match those from the EPA reach file (Rf).
3. Fill the DEM sinks so sumps do not cause incorrect flow directions.
4. Compute the flow direction for each grid point within the DEM.
5. Compute a flow accumulation grid based on the number of cells draining to each point.
6. Construct a stream network based on a user defined accumulation threshold.
7. Streams may be added to the stream network if they were not included in step 6.
8. Segment streams into reaches.
9. Place outlets at the junctions of each stream reach.
10. Add additional outlets where necessary (i.e. at structures).
11. Delineated drainage areas from each of the outlets using the DEM.
12. Streams and drainage area grids are converted to vector shapefiles.
13. Subbasins may be merged.
14. Calculate runoff curve numbers based on land use and soil classification.
15. Determine lag time based on basin topography.
16. Determine Muskingum coefficients based on channel characteristics.

17. Export the data set to a HEC-HMS basin file.
18. Import the HEC-HMS basin file into a HEC-HMS project file.

2.4 HYDRAULIC MODELS

Hydraulic models were developed for some of the drainage basins in the city of Dubuque for the purpose of assessing flood conditions including water surface elevations and hydraulic capacities of existing drainage structures. Peak runoff rates computed as part of the hydrologic modeling were used in conjunction with the GIS and limited field data to develop open channel and closed conduit hydraulic models. For the open channel model, water surface profiles were computed for the 10-, 50-, 100- and 500-year return period flood events. The resulting 100-year floodplain for ultimate development with and without project conditions was delineated using GIS. A portion of the North Fork Catfish Creek main channel was modeled with a hydraulic model. The closed conduit model was used to analyze the hydraulics of the Bee Branch main storm sewer trunk line. The 10-, 50-, and 100-year return period flood events were investigated. The following sections describe the key elements involved in the hydraulic modeling of the stream segments in the City of Dubuque.

The Hydrologic Engineering Center – River Analysis System (HEC-RAS) was used to analyze open channel hydraulics. HEC-RAS is a hydraulic model developed by the U.S. Army Corps of Engineers. The model is designed to perform one-dimensional hydraulic calculations for a network of natural and constructed open channels. The following assumptions are used by HEC-RAS in computing water surface profiles:

- Steady flow;
- Gradually varied flow;
- One-dimensional flow;
- Channel slopes are small, less than 1:10

Although some of the steeper channels may exhibit supercritical flow characteristics, it is conservative to base the hydraulic analyses on subcritical flow, since the depth of flow for subcritical flow conditions is greater than supercritical flow conditions.

XP-SWMM was used to analyze closed conduit hydraulics. XP-SWMM is proprietary storm water modeling software based on the U.S. Environmental Protection Agency model SWMM (Storm Water Management Model). XP-SWMM is capable of modeling unsteady flow allowing for analysis of changes in flow variables with time and attenuation of peak discharges as a result of storage. The following assumptions are used in hydraulic computations performed by XP-SWMM:

- Gradually varied flow;
- One-dimensional flow; and
- Subcritical flow

2.4.1 Model Schematic

For the open channel model, channel cross-section geometry and flow lengths were obtained from a triangular irregular model (TIN) developed from the digital terrain model (DTM). Cross-section geometry was generated from the TIN and the U.S. Army Corps of Engineer's HEC-GeoRAS software in conjunction with ArcView's 3D-Analyst which electronically generates the HEC-RAS input files within ArcView. Bridge and culvert geometry were obtained from field measurements. Manning's roughness coefficients were selected based on field observations and interpretations from aerial mapping. Guidelines contained in "Open Channel Hydraulics," by Chow, were used when estimating roughness coefficients.

The closed conduit XP-SWMM model was generated based primarily on information supplied by the City. The DAGIS storm sewer coverage provided the storm sewer alignment in the area of interest, and model geometry was based on storm sewer profile sheets with additional information obtained from the City's archive. Manning's roughness coefficients were selected based on conduit material information taken from storm sewer profile sheets and recommendations made by City engineering staff. Guidelines contained in "Open Channel Hydraulics," by Chow, were used when estimating roughness coefficients.

2.4.2 Model Calibration

Several high-water marks were evaluated for the May 16, 1999 storm event. This 1999 storm was estimated to be a 75-year return period. High-water marks were used for an order of magnitude assessment of the model results. No additional calibration of the hydraulic model was performed.

2.4.3 Channel and Structure Improvements

Channel improvements were evaluated for a number of problem areas identified in the study. HEC-RAS offers a convenient method for analyzing a range of channel improvement options and includes computational procedures for estimating excavation volumes and computing revised flood levels with the channel improvement in place. Channel shaping and clearing improvements were considered in several reaches of the study area.

Storm sewer improvements involved expansion of storage and conveyance through installation of additional conduits or construction of flood control channels. Improvements were iteratively incorporated into the XP-SWMM model and analyzed to assess their impact on flooding.

2.4.4 Drainage Criteria

The city plans to adopt drainage standards/criteria to be used as a guidance document for designing and evaluating drainage facilities within the city's jurisdiction. Storm drainage systems shall be designed to convey runoff from a return period storm, dependent on the type of drainage system facility.

In addition to providing storm drainage facilities for the design runoff, drainage policies dictate that provision shall be made to prevent significant property damage and loss of life from the 100-year return period storm.

2.4.5 Cost Estimates

Cost estimates were developed for recommended improvements at each of the problem areas identified on the major storm drainage system considered in the study. Component costs were estimated based on typical unit costs for construction. Contingencies (25%) were added to account for estimated quantities, unit price adjustments and miscellaneous work related items. An additional 25% was included for administrative, legal and engineering costs. Right-of-way, operation and maintenance and mitigation costs were not included.

Unit costs for specific components of improvement projects were obtained from the Iowa Department of Transportation 1999 bid tabulations. Unit price adjustments were made for large projects to account for economy of scale.

2.5 WATER QUALITY

Erosion and sedimentation processes are natural processes accelerated by human activities, especially during construction. Reducing erosion and preventing sediment from leaving construction sites offers the best opportunity to improve water quality of the environment. Rainfall on unprotected soil causes serious erosion and results in sediment being deposited in drainageways and a general degradation of the environment.

One major component of managing storm water runoff is the implementation of Best Management Practices (BMPs). Iowa State University has published an erosion control manual for construction site measures entitled "Iowa Construction Site Erosion Control Manual". The manual is to serve as a guide in selecting erosion control practices and preparing plans to reduce erosion on construction sites.

BMPs are operational techniques and/or structural facilities that can dramatically improve the quality of storm water runoff. Operational BMPs reduce the opportunity for pollution to come into contact with storm water runoff, whereas structural BMPs collect, concentrate, and/or treat runoff. The costs to implement operational and/or structural BMPs are usually significantly less than the costs associated with remediation damage resulting from inadequate storm water management. Operational BMPs are much more economical and simplistic, so they should generally be considered before structural BMPs.

When selecting any type of BMP, non-technical issues, as well as technical issues, should be considered. Technical issues vary with individual BMPs, but broadly deal with site feasibility, design considerations, and/or pollutant removal efficiencies. Technical issues are generally more involved for structural BMPs than operational BMPs. Non-technical issues deal with the economic, regulatory, and public aspects of selecting a BMP. These issues, among others, include: federal, state, and local regulations; real and perceived receiving water problems; economic feasibility of BMP being considered and public acceptance of BMP being considered.

2.5.1 Operational BMPs

The goals of operational BMPs are to prevent pollutants from coming in contact with storm water by controlling the pollutants at their source. For this reason, operational BMPs are often referred to as source control BMPs. Operational BMPs are non-structural controls generally associated with management practices that reduce contact between storm water and pollutants. The effectiveness of operational BMPs is often highly dependent on site-specific conditions, due to the high variability in pollutant source conditions; thus it is difficult to generate general removal efficiencies. Source controls for urban areas can be grouped into the following general categories:

- Public education
- Street/storm drain system maintenance
- On-site materials management
- Planning and regional management
- Illicit/accidental controls

Public education can be one of the most economical and effective pollution control strategies. The goal of public education is to change the way the public manages many of the constituents that end up in storm water runoff, through awareness. The methods in which many household products such as automotive fluids, cleaners, and fertilizers are used and disposed of can have a profound effect on the quantities of these substances that come into contact with storm water, and thus on the water quality of receiving waters. Many methods available for increasing public education include radio/television advertisements, mailings, public meetings, and others. Although public education is one of the simplest means of affecting storm water quality, its effectiveness is highly variable, and may be hard to directly measure.

Street and storm drain maintenance refers to the removal of pollutants from street surfaces and the periodic cleaning of storm drainage structures. This control may reduce the quantity of pollutants, most notably sediment, entering the storm sewer system. Examples of this type of pollution control include street sweeping, catch basin cleaning, curb and gutter cleaning, and road and bridge maintenance.

On-site materials management deals with the practice of use, storage, and disposal of substances that could pollute storm water runoff. There are many specific pollution controls for materials management; however, they can be generalized into three groups:

- Altering the activity to minimize generation of potential pollutants
- Covering pollutant sources, thus reducing their contact with precipitation and runoff
- Containing/segregating the activity containing source of pollutants from other activities, so pollutants may be handled and disposed of separately

Examples of on-site materials management include: storing materials inside or under cover on paved surfaces, minimizing storage and handling of hazardous materials, secondary containment to reduce leakage, and choosing safer alternative products.

Planning and regional management refers to practices by local governments aimed at reducing pollutants in storm water on a regional basis, especially those loadings from new development areas. Land use controls and floodplain management practices are the typical mechanisms for this type of pollution control. Examples of planning and regional management include: buffers and setbacks from all water bodies, zoning ordinances for open areas, regulations for sediment control measures in new developments, and use of vegetated natural channels.

Illicit and accidental control BMPs can be used to reduce introduction of pollutants to storm sewer systems through illegal or accidental activities. These activities are often related, because a responsible party may not even be aware of the detrimental impacts of an illegal or accidental discharge to the storm sewer system. Examples of illicit and accidental controls include:

detection, removal, and enforcement system for illegal connections/dumping through inspections or source testing; public notices; and accidental spill information boards/hotlines.

2.5.2 Structural BMPs

The goal of structural BMPs is to reduce non-point source pollution by collecting, concentrating, and/or treating storm water runoff. Unlike operational BMPs, which are often simply techniques for source control, structural BMPs are physical entities that are strategically located within a drainage area. The benefit of having purposefully located and designed entities is that it facilitates tabulation of general pollutant removal efficiencies for different structural BMPs. However, the disadvantages are higher initial cost, more complexity, and required maintenance. Overall, structural BMPs are most applicable to developing and redeveloping areas, since construction/implementation costs are less and site location is easier.

Structural BMPs are strategically located and designed to maximize their beneficial impact on storm water quality for an area, and to minimize implementation and operational costs. This benefit/cost feasibility analysis for selecting a structural BMP can be grouped into five general categories:

- Physical suitability
- Hydrologic conditions
- Pollutant characteristics and removal capabilities
- Environmental and aesthetic factors
- Operational factors

Physical suitability of a site refers to the technical feasibility criteria related to physical conditions, such as topography, required land area, contributing drainage area, soil types, and water availability. Physical suitability is often one of the first considerations when selecting a structural BMP since it is not feasible or possible to change many of the factors, and it can dramatically affect the usefulness of a given BMP.

Hydrologic criteria focus on the hydrologic characteristics for a given design storm event, such as storm water runoff volume, distribution, and peak discharge. It should be noted that the concepts in designing water quality controls are different than those for water quantity controls. The highest concentrations of pollutants are often found in the beginning of storms, often referred to as the "first flush" stage. In this stage, built-up pollutants are being washed off the land surface and potential dilution effects are negligible. Thus, water *quality* controls are designed for smaller, more frequent storms, whereas water *quantity* controls focus on larger, less frequent storms, which cause flooding and other damage.

Environmental and aesthetic factors refer to the impacts a structural BMP would have on the environment– how it would affect the aesthetics of the area. Examples of environmental and aesthetic factors include maintenance of low flows for aquatic life, streambank erosion, recreational benefits, and community acceptance.

Operational factors are mainly concerned with the amount and type of maintenance a given structural BMP requires. Generally, structural BMPs have a passive design, meaning that there is no active operation of mechanical or chemical equipment. However, almost all structural BMPs require periodic cleaning and maintenance to keep them working efficiently.

The following list includes a number of different structural BMPs that are commonly used to improve water quality of storm water runoff:

- Swales
- Filter strips and vegetative buffer zones
- Infiltration basins and percolation trenches
- Detention controls and constructed wetland basins
- Oil and water separators

Swales are shallow, vegetated, mildly- sloped channels that convey storm water runoff. They are designed for low velocity flows during small storms to allow infiltration of storm water into the swale bottom, and filtration and biological uptake of pollutants into the vegetative cover– collectively referred to as biofiltration. Swales are applicable in most mildly sloping areas, due to their relatively low space, cost, and maintenance requirements.

Filter strips are similar to swales, except they do not have side slopes, thus runoff is spread evenly through the filter strip area as sheet flow, rather than through small channels. Treatment, cost, maintenance, and applicability are similar to those of swales. Vegetative buffer zones are a specific type of filter strip surrounding or "buffering" a water body, so as to remove pollutants before reaching the receiving body.

Infiltration basins and percolation trenches are systems that enhance the potential for storm water runoff to percolate into the soil. These systems consist of a structure or trench filled with a filter media such as sand or gravel, which allows percolation into the soil. Infiltration basins and percolation trenches only work with porous soils, favorable site geology, and proper groundwater conditions. Infiltration devices are generally effective in the Dubuque area unless the silt loam layer is shallow and under lain by bedrock.

Detention controls consist of both dry detention basins, which completely drain out between storm events, and wet retention ponds, which maintain a designed level of water between storm events. Constructed wetland basins are complex wet retention facilities that have additional construction and biological requirements, but often provide increased pollutant removal. The primary mechanism for pollutant removal is sedimentation. Wet retention ponds provide additional removal through physical and biochemical processes, such as reduction in bottom scour and increased vegetative growth in the permanent pool. In addition to good pollutant removal, detention facilities also can provide reduction in peak runoff flows. Detention controls are most applicable where relatively large tracts of land are available, such as parks and industrial facilities.

2.6 DEVELOPMENT OF RECOMMENDED PLAN

2.6.1 General

Investigations of structural flooding and roadway overtopping were conducted for the future development conditions. Runoff from the future conditions was routed through the existing channels, culverts, and storm drains.

2.6.2 Structural Flooding

A 100-year floodplain delineation was created in ArcView using the existing conveyance elements and the ultimate land use runoff estimates for the segment of channels analyzed. Using the GIS topographic coverage and the 100-year floodplain, flooded structures were identified. Finished floor elevations were not surveyed.

2.6.3 Roadway Overtopping

Roadway overtopping is defined as transverse flow over a roadway resulting from flooding of an adjacent channel. Roadway overtopping was estimated for the 10-, 50- and 100-year storms. Roadway crest elevations were determined from the 1999 City of Dubuque 2-foot contour maps and the TIN generated from the City DTM. A majority of the roadway crest elevations at creek and tributary crossings were identified by survey using spot elevations. These spot elevations were verified by interpolation between roadway GIS contours at crossing locations.

Roadways adjacent to or crossing the storm drainage system were classified as residential, collector, minor arterial, and principal arterial. Roadway classifications were established using the following general definitions:

- Residential – interior streets in subdivisions and residential areas
- Collector – streets that direct subdivision and residential traffic to arterial roadways
- Minor Arterial – major streets directing collector traffic to other collector streets and freeways
- Principal Arterial – any U.S. or state designated roadway

The roadway overtopping criteria are summarized below in Table 2.7. In this table, the design storm is the flood event the culvert or storm sewer must pass to meet the criteria. For example, a residential roadway cannot flood in the 10-year storm to meet the criteria but may flood in the 50-year storm.

Table 2.7
Roadway Overtopping Design Storms

Roadway Classification	Design Storm (year)	100-Year Maximum Allowable Depth of Flow (feet)
Residential	10	No maximum
Collector	50	1.5
Minor Arterial	50	1.0
Principal Arterial	100	0.0

2.6.4 Flood Minimization Alternative Improvements

A list of flood minimization alternatives was compiled based on experience from past master planning and flood control activities with consideration for the unique topography in and around the City of Dubuque. Alternatives considered to have potential benefit in the City of Dubuque are shown in Table 2.8. A brief discussion of each alternative is given below.

Table 2.8
Flood Minimization Alternative Improvements

Nonstructural Alternatives	Structural Alternatives
Public Education/Outreach	Rehabilitate/Expand Capacity of Existing Facilities
Floodplain Buyout	Create Upstream Detention
Flood Proofing	Flood Control Channel
Flood Warning System	Relief Storm Sewer
Do Nothing	Transbasin Diversion
	Deep Storage/Pumping Tunnel
	Pressure Sewer System

2.6.4.1 Nonstructural Alternatives

Nonstructural alternatives focus on minimization of property damage or loss of life through means other than construction of detention and conveyance facilities. They involve public awareness of flood dangers, protection of property from flood damages, and removal of individuals from flood prone areas.

Public Education/Outreach

Public education programs can be instrumental in reducing flood losses and future flood causalities. Public outreach can include a development of public programs to provide emergency shelters and first aid during a flood event, emergency services to assist in evacuation of residences, and educational programs intended to inform citizens of required safety practices before, during, and after a flood event.

Floodplain Buyout

After the delineation of the 100-year limits of flooding, a program to acquire and remove flood-prone structures may be feasible in reducing or eliminating flooding problems. This approach may be considered for clearing the entire floodplain or as a partial solution in isolated areas where coverage by a structural solution exceeds the value of isolated structures. Floodplain acquisition programs have been used successfully in many communities and may be useful in reducing flood hazards in Dubuque.

Flood Proofing

When structural flood control alternatives are found to be cost prohibitive, flood proofing is an alternative that should be considered to reduce flood impacts. Installation of a variety of flood proofing systems would be required in order to meet the varied needs of the structures located within the flood-prone areas. Flood proofing facilities may range from structural modifications to reduce or eliminate damages from flooding to educational programs informing citizens how to protect their property or remain safe during a flood event.

Structural measures are usually implemented in commercial or industrial settings where personnel are available to operate and maintain flood proofing devices. In residential applications, flood proofing is typically limited to the relocation of vital residential systems such as heating, cooling, water heaters and laundry areas to safe flooding areas. The relocation of electrical services to areas above the anticipated water surface elevation also is required. Frequently casualties during flooding relate to structural failures of basement and foundation walls. Public education is an effective means to inform people of these dangers.

Flood Warning System

A flood warning system would be a critical element to the development of a flood proofing strategy. Flood warning systems can be designed to provide advance notice of a potential flood event by installing flood monitoring, rainfall indicators and storm sewer flow monitors in upstream areas. In the City of Dubuque, a flood warning system could be utilized to monitor flood conditions in various detention facilities including Ice Harbor, Maus Lake, the 16th Street Detention Cell and the West 32nd Street Detention Cell. This information would be valuable for managing operations such as pumping and gate closure operations as well as provide information as to when flood warning alarms should be sounded.

Because of the steep basins and high rate of storm water runoff in Dubuque, a flood warning system would not provide a meaningful amount of time. The City also should consider the possibility that a flood warning system may encourage people to leave their homes and try to evacuate when such evacuation is not possible due to the short notice time and high congestion of the area. If the flood warning system were used, it would be important to provide a public education/outreach program that would inform the people of the correct responses to the flood warning system.

Do Nothing

If the public is not concerned about the frequency and magnitude of flooding problems in the community, no action is an alternative. Flooding problems should be monitored and appropriate steps taken to eliminate the loss of life.

2.6.4.2 Structural Improvement Alternatives

Structural alternatives involve the construction or improvement of municipal facilities with the specific purpose of limiting the extent and depth of flooding and thereby reducing the potential for property damage or loss of life. Structural alternatives include detention areas to capture runoff, expansion of conveyance through flood prone areas, or redirection of flood flows away from populated areas.

Rehabilitate/Expand Capacity of Existing Facilities

Increasing the volume of existing detention cells or modifying detention cell outlet works to discharge flood flows could have a significant effect on flooding. Detention of larger volumes of water in the upland areas of the City would reduce flooding in low areas near the Mississippi River and provide relief for storm sewer systems unable to convey runoff from extreme storm events. Increase in volume or the size of gravity outlets and pumps in detention cells adjacent to the Mississippi River would draw water out of the Couler Valley area and discharge it to the Mississippi River more quickly, thereby further reducing flooding depths.

Repair or replacement of storm water conveyance systems where development has exceeded the system's capacity could decrease or eliminate flooding problems due to ponding in both the upland and lowland areas.

Create Upstream Detention

Creation of detention in undeveloped, upland areas would decrease peak discharges and delay the large volume of storm water runoff draining toward the Couler Valley area. Detention could be created by constructing embankments to contain the water in the steep valleys and ravines characteristic of the terrain in the Dubuque area. When an upstream open space is not available for the development of detention, it becomes necessary to identify occupied sites that can be converted to potential storage locations.

Flood Control Channel

Conveyance of runoff through flat, heavily developed areas may require capacity in excess of what can be provided by a closed-conduit storm sewer system. Construction of a large flood control channel through the developed areas would provide a significant increase in conveyance and storage and could have a large impact of the flooding problem. This requires the purchase of private and commercial property and the relocation of individuals, businesses, roads, and utilities. Investigations should also be undertaken to ensure the pathways runoff takes during the design flood event to reach the flood control channel are non-damaging pathways.

Relief Storm Sewer

Construction of a relief storm sewer to expand the capacity of major trunk lines in the system would have a similar, although less dramatic, effect than a flood control channel. The increase in conveyance would deliver water to the Mississippi River more quickly and decrease flooding in the low-lying areas. The benefit/cost ratio would be substantially lower than that of the flood control channel; however, its construction would have a less dramatic effect on the community.

Transbasin Diversion

Rather than convey runoff from the upland areas through the heavily developed areas, water could be diverted away and allowed to take another path to the Mississippi River. One such opportunity would be the diversion of flows entering the Couler Valley area from the north to the Little Maquoketa River.

Deep Storage/Pumping Tunnel

Considering current tunneling technologies, the construction of a deep tunnel far beneath the surface is an alternative that may be considered. This alternative would construct a facility consisting of a large diameter tunnel shaft in the lowland area of the city to be used as an underground storage reservoir. Existing sewers could be connected to this facility by service shafts at appropriate high flow connection points. A pumping station would be required to evacuate the system after storm events. This type of facility would be technically very challenging; however, it would provide minimal impact to existing development and utilities.

Pressure Sewer System

An effective alternative to reduce downstream flooding would be to provide piping from upland areas that transport large volumes of floodwater directly to the Mississippi River and bypassing the 16th Street Detention Cell. These new sewers would likely be located in the existing street rights-of-way and would require extensive reconstruction of existing utilities as well as street surfacing reconstruction. To minimize the impact of construction on existing facilities, the pressure sewer system could also be constructed using rock/earth tunneling technology.

3.0 NORTH FORK CATFISH CREEK DRAINAGE BASIN

3.1 GENERAL DRAINAGE BASIN DESCRIPTION

The North Fork Catfish Creek (North Fork) Drainage Basin is located in the southern vicinity of the Dubuque municipal limits and is shown on Figure 3-1. The drainage basin measures approximately 3.9 square miles, with a majority of the drainage area being contained within the Dubuque boundary limits. A small portion in the west part of the drainage basin extends into the City of Asbury's jurisdiction. The drainage area is roughly bounded by Asbury Road to the north, Pennsylvania Avenue, University Avenue and Brunskill Road to the south, Radford Road to the west and Grandview Avenue to the east.

The North Fork Drainage Basin is relatively steep, with an average terrain slope of around 6 percent. The overall slope of the main channel is approximately 1 percent. Elevations in the drainage basin range from 680 ft NGVD at the confluence of North Fork with Middle Fork Catfish Creek to 950 ft NGVD in the upper reaches of the drainage basin. Figure 3-2 shows the range of slopes for the North Fork Drainage Basin. The steepest slopes of 15% or greater are located along the main channel and near the confluence with Middle Fork Catfish Creek.

Information on the soil types and characteristics in the North Fork Drainage Basin was compiled by developing a digital soils database in GIS. Table 3.1 shows the relative representation and general hydrologic characteristics for the different soil series found in the North Fork Drainage Basin. The North Fork Drainage Basin consists of over 25 different soil types, of which the Fayette-Urban land complex and the Rozetta-Eleroy silt loam series account for close to 50 percent of the total drainage basin area. The majority of the Fayette-Urban series are located in the lower two-thirds of the drainage area while the Rozetta-Eleroy series are primarily located along the channel west of Northwest Arterial and north of Hillcrest Road. For modeling purposes, the different soil types were grouped by the NRCS hydrologic soil type as Type A, B, C, or D. Nearly the entire drainage basin consists of Type B soils, as depicted in Figure 3-3.

Table 3.1
North Fork Catfish Creek Drainage Basin
Soil Type Summary

Soil Series	General Hydrologic Characteristics	Texture	Number of Polygons	% Area
Fayette-Urban (4163C, 4163D)	(5 to 14% slopes) Moderately sloping well-drained soil on short, convex side slopes in the uplands. Moderate permeability with rapid runoff.	Silt Loam	17	37.6
Rozetta-Eleroy (563E2, 563D2)	(9 to 18% slopes) Moderately eroded, strongly sloping, moderately well drained soils on convex side slopes of the uplands. Rozetta soil on the upland areas and Eleroy adjacent to drainageways. Moderate permeability with rapid runoff.	Silt Loam	18	9
Orthents (5040B)	Gently sloping soils in cut and fill areas, highly variable drainage, moderate to slow permeability, runoff is slow to medium	Loam	11	7.7
Fayette Silt Loam (163C2, 163D2)	(5 to 14 % slopes) Moderately eroded, moderately sloping, well drained, moderate permeability, medium runoff	Silt Loam	46	7.3
	Various soils, 18 soil types ranging from 0.01% to 4.5% area.	Silt Loam	84	38.4
	Total Percent Area			**100.0%**

Source: Soil Survey of Dubuque County, Iowa: SCS, December 1985.

The drainage system in the North Fork Drainage Basin consists of both natural channel and closed conduit sections. The main channel is a natural earthen channel and numerous storm sewers convey runoff to the natural channel. The majority of the storm water conveyance system consists of open channels, and the 18 miles of the drainage system modeled consist entirely of open channel sections. Of the total 18 miles of conveyance length modeled, 11 miles are major creeks and tributaries. The remaining 7 miles are smaller tributaries and drainageways. Although there are a number of smaller creeks in the North Fork Drainage Basin, North Fork is the only major creek. The flooding problems in the North Fork Drainage Basin are confined to the open channels system; therefore, the storm sewer system was not modeled.

A land use database containing information for ultimate development was created based on the DAGIS 1999 Comprehensive Land Use Plan and supplemented with land use projections made by City staff. Land use classifications in North Fork range from open spaces to industrial, with the majority of the drainage basin being classified as low density and medium density residential and commercial land uses. The breakdown of land use within the North Fork Drainage Basin for ultimate development is shown in Table 3.2 and Figure 3.4.

Table 3.2
North Fork Catfish Creek Drainage Basin
Land Use Summary

Land Use Classification	Area (acres)	% of Area
Streets	235	9.2
Industrial	6	0.2
Commercial	345	13.5
Institutional	139	5.5
High Density Residential	119	4.7
Medium Density Residential	396	15.6
Low Density Residential	808	31.7
Agricultural	190	7.5
Open Space and Grass	307	12.1
Total	**2,545**	**100.0%**

Source: City of Dubuque, Iowa Comprehensive Land Use Plan, 1999.
Note: Water bodies are incorporated into adjacent parcel land use categories.

Few flood control measures have been implemented in the North Fork Drainage Basin, other than minor channel modifications on the main channel and some of the tributaries. The North Fork Drainage Basin is one of the few drainage basins in which regional detention of storm water runoff may be a viable alternative for flood control. Regional detention is most effective when applied in the upper portions of the drainage basin. Natural detention upstream of several drainage structures offers an opportunity to reduce the discharge and water surface elevations downstream. As the drainage basin becomes more developed, the number of available detention sites is reduced and detention options are thereby eliminated. Regional detention sites were analyzed along with channel improvements that can be implemented as a potential means of flood control in the North Fork Drainage Basin.

3.2 FLOOD HYDROLOGY

The HEC-HMS model was utilized to compute the peak runoff rates for the 10-year, 50-year, 100-year and 500-year return period storm events. Runoff hydrographs were developed for each storm event for ultimate development condition, as defined by the City's comprehensive land use plan. Figure 3-5 depicts the subbasin delineation, while Figure 3-6 is a schematic of the HEC-HMS model for the North Fork Drainage Basin.

Table 3.3 provides a summary of ultimate peak runoff rates for selected storm events at key locations in the North Fork Drainage Basin. A summary of the peak runoff rates for all sub-basin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 3.3
North Fork Catfish Creek Drainage Basin
Peak Runoff Summary for Existing Drainage System Conditions

Structure Id. No.[2]	HEC-HMS Node No.[3]	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs)[1,4]			
				10-Year	50-Year	100-Year	500-Year
Main Channel							
NF-ST-1	26	Brunskill Road	3.8	1,930	2,720	3,130	4,220
NF-ST-2	22	US 20 (Dodge St.)	3.2	1,680	2,210	2,490	3,000
NF-ST-3	49	University Ave.	3.0	1,630	2,150	2,420	2,930
NF-ST-4[5]	18	J.F. Kennedy Rd. & Pennsylvania Ave.	2.1	1,400	1,880	2,080	2,580
NF-ST-5	9	Keyway	1.7	1,230	1,900	2,280	3,250
NF-ST-6	35	Rosemont St.	0.94	660	930	1,070	1,930
NF-ST-7	10	Northwest Arterial	0.52	240	460	650	1,160
NF-ST-8	36	Sunnyslope	0.26	460	670	770	1,080
NF-ST-9	3	Radford Road	0.16	270	400	470	660
NF-ST-10	5	Saratoga Road	0.06	90	130	160	230
Tributary No. 1							
NF-T1-ST-1	57	Brunskill Road	0.41	460	710	850	1,240
NF-T1-ST-2	54	US 20 (Dodge St.)	0.14	170	270	320	470
Tributary No. 2							
NF-T2-ST-1	33	Hillcrest Road	0.41	370	650	800	1,230
NF-T2-ST-2	30	Asbury Road	0.15	180	290	360	550
NF-T2-ST-3	29	Asbury Road	0.03	30	50	60	100
Tributary No. 3							
NF-T3-ST-1	12	NW Arterial	0.21	270	390	460	550
NF-T3-ST-2	13	Embassy West Dr.	0.10	110	170	210	300

Notes:
1. Peak runoff rates based on ultimate land use condition and simulation of a 24-hour storm event.
2. See Figure 3-1 for location of structure identification number.
3. See Figure 3-6 for location of HEC-HMS node and identification number.
4. Peak discharges reported are outflows from the specified nodes.
5. Peak discharges are attenuated by storage upstream of the Pennsylvania Avenue roadway embankment and flooding of the J. F. Kennedy Road/Pennsylvania Avenue intersection.

3.3 STREAM HYDRAULICS

HEC-HMS and HEC-RAS were used to determine the stream hydraulics of the channel and the bridges and culverts on the main channel and tributaries studied. A total of 16 road crossings were analyzed in the North Fork Drainage Basin. A summary of the hydraulic capacity for each of the crossings studied is presented in Table 3.4 for the 10-, 50- and 100-year storm events.

Table 3.4
North Fork Catfish Creek Drainage Basin
Existing Hydraulic Capacity of Stream Crossings Summary

Structure Identification No.	Location	Minimum Overtopping Elevation[1]	Depth of Overtopping (ft)[2]		
			10-Year	50-Year	100-Year
Main Channel					
NF-ST-1	Brunskill Road	704.4	No hydraulic analysis required.		
NF-ST-2	US 20 (Dodge Street)	758.2	0.0	0.0	0.0
NF-ST-3	University Avenue	743.9	0.0	0.0	0.0
NF-ST-4	J.F. Kennedy Road & Pennsylvania Ave.[3]	780	0.0	0.9	3.4
NF-ST-5	Keyway[3]	789	1.3	2.1	2.4
NF-ST-6	Rosemont Street[3]	812.7[4]	1.5	1.9	2.4
NF-ST-7	Northwest Arterial	835.6	0.0	1.2	1.7
NF-ST-8	Sunnyslope	858.4[4]	0.0	0.2	0.4
NF-ST-9	Radford Road	864.2[4]	0.0	0.2	0.3
NF-ST-10	Saratoga Road	869.9[4]	1.1	1.2	1.2
Tributary No. 1					
NF-T1-ST-1	Brunskill Road	703.8[4]	0.0	0.0	0.0
NF-T1-ST-2	US 20 (Dodge St.)	814.9	0.0	0.0	0.0
Tributary No. 2					
NF-T2-ST-1	Hillcrest Road	810.8	0.0	0.1	0.5
NF-T2-ST-2	Asbury Road	855.8	0.0	0.2	0.3
NF-T2-ST-3	Asbury Road	900.1[4]	0.0	0.0	0.0
Tributary No. 3					
NF-T3-ST-1	NW Arterial	835.6	0.0	0.0	0.0
NF-T3-ST-2	Embassy West Drive	843.1[4]	0.0	0.0	0.0

Notes:

1. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
2. Depth of overtopping obtained from HEC-HMS analysis, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.
4. Minimum overtopping elevation based on minimum roadway elevation obtained by interpolating DAGIS mapping.

The standards/criteria for passing the design flood event without roadway overtopping were used to evaluate each crossing. A summary of the hydraulic capacity and return period for each of the crossings studied is presented in Table 3.5.

Table 3.5
North Fork Catfish Creek Drainage Basin
Existing Hydraulic Capacity and Return Period of Stream Crossings Summary

Structure Identification No.	Location	Existing Structure Type	Roadway Classification	Hydraulic Capacity Return Period	
				Required	Actual
Main Channel					
NF-ST-1	Brunskill Road	Bridge	Collector	No hydraulic analysis required.	
NF-ST-2	US 20 (Dodge St.)	26.8' x 18.5' CAP	Principal Arterial	100-yr with 0' overtop	GT 100-yr
NF-ST-3	University Avenue	12' x 12.2' RCB	Minor Arterial	50-yr with 1' overtop for 100-yr	GT 100-yr
NF-ST-4	J.F. Kennedy Road & Pennsylvania Ave.	14.5 CMP	Principal Arterial	100-yr with 0' overtop	0.9' overtop for 50-yr & 3.4' for 100-yr
NF-ST-5	Keyway	2- 6.5' RCP	Collector	50-yr with 1.5' overtop for 100-yr	2.1' overtop for 50-yr & 2.4' for 100-yr
NF-ST-6	Rosemont Street	6' RCP	Collector	50-yr with 1.5' overtop for 100-yr	1.9' overtop for 50-yr & 2.4' for 100-yr
NF-ST-7	Northwest Arterial	6' RCP	Principal Arterial	100-yr with 0' overtop	1.2' overtop for 50-yr & 1.7' for 100-yr
NF-ST-8	Sunnyslope	3- 4' RCP	Residential	10-yr with no 100-yr max. overtop	10-yr with 0.4' overtop for 100-yr
NF-ST-9	Radford Road	2- 3.5' RCP	Collector	50-yr with 1.5' overtop for 100-yr	0.2' overtop for 50-yr & 0.3' for 100-yr
NF-ST-10	Saratoga Road	30" RCP/ 3' RCP	Residential	10-yr with no 100-yr max. overtop	1.1' overtop for 10-yr & 1.2' for 50-yr
Tributary No. 1					
NF-T1-ST-1	Brunskill Road	9.75' x 5.6' RCB	Collector	50-yr with 1' overtop for 100-yr	GT 100-yr
NF-T1-ST-2	US 20 (Dodge St.)	4'-9" x 6'-1" RCAP/ 4.9' x 5.8' RCB	Principal Arterial	100-yr with 0' overtop	GT 100-yr

Table 3.5
North Fork Catfish Creek Drainage Basin
Existing Hydraulic Capacity and Return Period of Stream Crossings Summary

Structure Identification No.	Location	Existing Structure Type	Roadway Classification	Hydraulic Capacity Return Period	
				Required	Actual
Tributary No. 2					
NF-T2-ST-1	Hillcrest Road	7' RCP	Collector	50-yr with 1.5' overtop for 100-yr	0.1' overtop for 50-yr & 0.5' for 100-yr
NF-T2-ST-2	Asbury Road	5' RCP	Minor Arterial	50-yr with 1' overtop for 100-yr	0.2' overtop for 50-yr & 0.3' for 100-yr
NF-T2-ST-3	Asbury Road	3.5' RCP	Minor Arterial	50-yr with 1' overtop for 100-yr	GT 100-yr
Tributary No. 3					
NF-T3-ST-1	NW Arterial	6' RCP	Principal Arterial	100-yr with 0' overtop	GT 100-yr
NF-T3-ST-2	Embassy West Drive	3- 3.5' RCP	Residential	10-yr with no 100-yr max. overtop	GT 100-yr

Notes:
1. Hydraulic capacity at minimum roadway elevation.
2. CAP – concrete arch pipe, RCB-reinforced concrete box culvert, RCP – reinforced concrete pipe, CMP – corrugated metal pipe.
3. GT – Greater Than; LT – Less Than
4. Roadway classification based on City of Dubuque's street classification index.

3.4 PROBLEM AREAS

The flood hydrology and stream hydraulics models provide the results needed for identification of areas that are not in compliance with the City's drainage standards/criteria. Problem areas in the North Fork Drainage Basin range from flooding of residential structures to inadequate conveyance systems. Figure 3-7 illustrates the locations of the identified problem areas. A description of each of the identified problem areas is also presented in Table 3.6 and the following section.

Table 3.6
North Fork Catfish Creek Drainage Basin
Identified Problem Area Summary

Structure Identification No.	Location	Criteria Violation
Main Channel		
NF-ST-4	J.F. Kennedy Road & Pennsylvania Ave.	50-year flood event overtops for ultimate land use conditions, but is contained within the street.
NF-ST-5	Keyway	10-year flood event overtops for ultimate land use conditions. Four (4) structures are within the 100-year flood level for ultimate runoff conditions.
NF-ST-6	Rosemont Street	10-year flood event overtops for ultimate land use conditions. Eight (8) structures are within the 100-year flood level for ultimate runoff conditions.
NF-ST-7	Northwest Arterial	50-year flood event overtops for ultimate land use conditions.
NF-ST-9	Radford Road	50-year flood event overtops for ultimate land use conditions.
NF-ST-10	Saratoga Road	10-year flood event overtops for ultimate land use conditions.
Tributary No. 2		
NF-T2-ST-1	Hillcrest Road	50-year flood event overtops for ultimate land use conditions.
NF-T2-ST-2	Asbury Rd	50-year flood event overtops for ultimate land use conditions.
Special Problem Area		
	Hillcrest Road & Rosemont Street	100-year flood event overtops for existing land use conditions.

The J.F. Kennedy Road & Pennsylvania Ave. (NF-ST-4) culvert is overtopped during flood events exceeding the 50-year event. The flooding is restricted to the street; therefore, no action is required.

The main channel and the drainage structures from the Northwest Arterial to Keyway cause structural damages and are overtopped during flood events with less than a 10-year return period. The drainage improvements for this reach are discussed in a subsequent section.

Saratoga Road (NF-ST-10) crosses the North Fork with a 30-inch RCP /36-inch RCP. Ultimate development conditions are projected to increase the peak flow at this site such that the 10-year flood event will overtop the roadway. Improvements recommended to upgrade Saratoga Road to pass a 10-year flood event include enlarging the existing culvert by adding a 36-inch RCP.

However, these improvements are within the City of Asbury's jurisdiction, so the City of Dubuque would not be directly responsible for the estimated $21,000 required for the proposed improvement.

The Hillcrest Road structure (NF-T2-ST-1) crosses Tributary No. 2 between St. Celia Street and Winnie Court. Two (2) options are recommended which includes either detention storage or drainage structure improvements. The 100-year peak discharge is nearly 800 cfs and the total runoff volume is 96 acre-feet for the existing land use condition (agricultural). To contain the peak storm volume, the detention volume behind Hillcrest Road must be increased from an existing storage volume of 7 acre-feet to 16.3 acre-feet to eliminate roadway overtopping at Hillcrest Road. The estimated cost for a detention proposed improvement is $76,000. If detention is not viable, the culvert must be increased from an 84-inch RCP to a 108-inch RCP in order to pass the existing peak flow without roadway overtopping. The estimated cost for the proposed drainage structure improvement is $100,000.

The Asbury Road structure (NF-T2-ST-2) crosses Tributary No. 2 between Northwest Arterial and St. Celia Street. Ultimate development conditions are projected to cause minor overtopping of the roadway for the 50-year flood event. Since the flooding is less than 0.3-foot, no proposed improvements are required.

Another location where local flooding is a problem is at the intersection of Rosemont Street and Hillcrest Road. Two (2) options are recommended which either includes detention storage or drainage structure improvements. The peak 100-year runoff near Rosemont Street is approximately 110 cfs and the total runoff volume is 12 acre-feet for the existing land use condition (agricultural). In order to contain the entire volume of the storm and eliminate roadway overtopping at Hillcrest, a 12 acre-feet detention pond is required. The estimated cost for a detention proposed improvement is $23,000. If detention is not viable, it is recommended to construct a 42-inch storm sewer to convey the peak discharge downstream and to eliminate roadway overtopping at Hillcrest. The estimated cost for the proposed storm sewer improvement is $90,000.

3.5 DEVELOPMENT OF ALTERNATIVE SOLUTIONS

3.5.1 Detention

Detention offers a means of controlling major flood events to prevent damage to downstream properties and infrastructure. Detention basins function by impounding runoff from an upstream basin and releasing it at a controlled rate to minimize downstream flooding. Table 3.7 presents a summary of the detention basins considered in this study. Figures 3-8 and 3-9 show a potential layout configuration for the Northwest Arterial and Pennsylvania Avenue detention storage sites.

Located along the northern branch of North Fork upstream of the Northwest Arterial is an existing storage area. A 6-foot RCP conveys the water downstream. The inlet structure has been modified to encourage upstream storage and regulate the downstream flow. The existing storage volume is approximately 52 acre-feet at the top of the road elevation or elevation 838. The maximum depth of water was computed at 19 feet. Prior to roadway overtopping, the residential lawns located along the right bank or southern bank flood.

Additional storage upstream of the Northwest Arterial can be achieved by excavating approximately 12-feet from the maintained lawns or from elevation 836 to elevation 821 at a 3H:1V slope. This would increase the storage to 110 acre-feet and require the excavation of approximately 95,500 cubic yards of material. A two-staged inlet would be constructed to create a detention cell.

Upstream of Pennsylvania, along the North Fork, is another opportunity to increase storage. Between Keyway and Pennsylvania, along the main channel, the area is heavily wooded and storage is available. Additional storage can be obtained through excavation. Excavation was limited to 4-feet because soil boring logs conducted for a private developer showed bedrock within 6-feet of the surface. Excavation of 35,000 cubic yards of material would increase storage from 40 acre-feet to 62 acre-feet or excavation of 70,000 cubic yards of material would increase storage from 40 acre-feet to 87 acre-feet.

Table 3.7
North Fork Catfish Creek Drainage Basin
Detention Storage Summary

Location	Drainage Area Controlled (square miles)	Existing			Alt. No. 1[1]			Alt. No. 2[1]			Alt. No. 3[1]		
		100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)
		Inflow	Outflow		Inflow	Outflow		Inflow	Outflow		Inflow	Outflow	
Northwest Arterial (NF-ST-7)	0.52	1,280	650	47	1,280	590	33	1,280	180	81[2]	1,280	180	81[2]
Pennsylvania (NF-ST-4)	2.05	2,650	2,080	42	2,850	2,080	63[3]	2,540	2,040	37	2,540	1,830	82[4]

Notes:

1. Alternative 1 – excavate storage upstream of Pennsylvania and modify outlet structure at Northwest Arterial; improve channel between Northwest Arterial and Keyway.
   Alternative 2 – excavate storage upstream of Northwest Arterial and improve channel between Northwest Arterial and Keyway.
   Alternative 3 – excavate storage upstream of Northwest Arterial and upstream of Pennsylvania and improve channel between Northwest Arterial and Keyway.
2. Excavate approximately 95,500 cubic yards of material for additional storage and construct 2-stage outlet structure at Northwest Arterial.
3. Excavate approximately 35,000 cubic yards of material for additional storage and build structural wall at Pennsylvania Avenue.
4. Excavate approximately 70,000 cubic yards of material for additional storage and build structural wall at Pennsylvania Avenue.

3.5.2 Channel and Drainage Structure Improvements

3.5.2.1 Upstream of University Avenue

The segment of North Fork between Northwest Arterial and University Avenue was identified as a problem area. Numerous structures are located within the 100-year floodplain and are relatively low in relation to the creek and likely incur frequent flooding. Major channel improvements would be required to lower the 100-year level below the floor elevation of the structures. It is proposed to improve the creek channel by clearing, shaping and/or increasing the capacity of several drainage structures.

A HEC-RAS hydraulic model was developed to describe the hydraulic conditions of the channel and drainage structures between Northwest Arterial and University Avenue. The peak discharges generated from the HEC-HMS model were used. Three (3) alternatives were evaluated. Alternative No. 1 proposes to increase the storage upstream of Pennsylvania Avenue, while Alternative No. 2 proposes to increase the storage upstream of the Northwest Arterial and Alternative No. 3 proposes to increase the storage upstream of both Pennsylvania Avenue and Northwest Arterial.

A baseline HEC-RAS model with current channel geometry was used to compare with proposed improvements. Cross-sections were based on data obtained from the digital terrain model (DTM) provided by the City. The channel geometry was assumed to be adequate for this study; however, surveyed channel inverts may be lower than those shown in the model.

Rosemont Street, Keyway, and Pennsylvania Avenue cross the main channel. The existing culverts and roadway geometry were modeled based on both DTM and survey information. A single 72-inch reinforced concrete pipe (RCP) culvert, twin 78-inch RCP culverts, and a single 14.5-foot corrugated metal pipe are located at the Rosemont Street, Keyway, and Pennsylvania Avenue crossings, respectively.

Manning's coefficients and entrance loss coefficients for the culverts were estimated based on site visits and guidelines presented in the HEC-RAS User's Manual. Manning's coefficients of 0.011 and 0.028 were used for concrete and corrugated metal, respectively. An entrance loss coefficient of 0.5 was used at Rosemont Street and Keyway while 0.7 was used at Pennsylvania Avenue because of the inlet configuration.

Manning's coefficients for the channel and overbanks were based on guidelines in "Open Channel Hydraulics," by Chow. A value of 0.035 was used for the channel upstream of Keyway, while a higher value of 0.040 was used downstream where the channel is more congested with trees. Overbank Manning's values ranged from 0.050 to 0.10 representing conditions ranging from grassy areas with few trees to heavy stands of timber.

The baseline hydraulic model showed all three roadways to be overtopped during 100-year flood discharges. In addition, several yards and homes were affected by flooding. Proposed condition models were created to determine measures to significantly reduce or prevent damage to private property and infrastructure caused by 100-year flood flows. Improvement options studied included channel clearing, channel shaping, and/or drainage structure improvements. Manning's coefficients were decreased in some locations to represent channel clearing; a minimum value of 0.035 was assumed. Channel shaping was accomplished with the channel modification option in HEC-RAS. Channel geometry was modified to create a trapezoidal shape with 3H:1V side slopes. The bottom width of the channel was changed to accommodate larger culverts or to increase channel conveyance; thereby, reducing water surface elevations. The channel downstream of Pennsylvania Avenue was not reshaped for the proposed conditions; however, the roughness was decreased to reflect channel clearing.

An improved entrance loss coefficient of 0.2 was used for all culverts. Culverts were replaced or supplemented to decrease or prevent a depth of flow over the roadways. A maximum depth over the roadways of 0.5 feet was used as the criteria for this analysis.

Table 3.8 presents a summary of the channel and drainage structure improvements and an estimated capital cost within the North Fork Drainage Basin. Figures 3-10, 3-11, 3-12 and 3-13 note the proposed channel and structural improvements between Northwest Arterial and University Avenue.

It is proposed to improve the existing entrance of the 14.5-foot corrugated metal pipe at the intersection of Pennsylvania Avenue and J.F. Kennedy Road by reestablishing the embankment around the pipe and installing rock riprap and a concrete collar around the pipe. In addition, a portion of the upstream channel would be realigned and a 310-foot long structural concrete wall built at the top of the slope above the inlet. The concrete wall would be 3-feet high, allowing an additional 2 feet of ponding with 1 foot of freeboard (top of wall elevation approximately 783 feet). The ponding elevation is limited because storm sewer inlets along Pennsylvania Avenue and J.F. Kennedy Road would begin to allow water to flow into the road from the culvert should the water surface upstream become too high. Alternative No. 3 requires the 14.5-foot opening be restricted to a 10-foot opening by constructing a concrete inlet that blocks the opening. The 10-foot opening does not cause any backwater effects on Keyway and provides 1.5-foot of freeboard on the structural wall.

The proposed improvements described above drastically reduce flooding limits on the study reach. For the Alternative No. 2 flow condition, roadway overtopping has been eliminated at Rosemont Street and Keyway and backyard flooding is reduced to the channel upstream of

Table 3.8
North Fork Catfish Creek Drainage Basin
Main Channel and Structural Improvement Summary

Location	Alternative 1		Alternative 2		Alternative 3	
	Proposed Improvements	**Capital Costs**	**Proposed Improvements**	**Capital Costs**	**Proposed Improvements**	**Capital Costs**
Northwest Arterial (NF-ST-7)	Modify outlet – increase opening area	$3,800	Excavate upstream detention and build two-stage outlet structure	$587,300	Excavate upstream detention and build two-stage outlet structure	$587,300
Northwest Arterial to Rosemont Reach	Trap. channel with b_w of 10' and side slopes of 3H:1V	$20,200	Trap. channel with b_w of 10' and side slopes of 3H:1V	$20,200	Trap. channel with b_w of 10' and side slopes of 3H:1V	$20,200
Rosemont (NF-ST-6)	Build 2 additional 72" RCP or 85 SF of total opening required	$88,400	Build 1 additional 72" RCP or 57 SF of total opening required	$61,800	Build 1 additional 72" RCP or 57 SF of total opening required	$61,800
Rosemont to Keyway Reach	Rosemont to Ellen: trap. channel with b_w of 10', side slopes of 3H:1V; Ellen to Keyway: trap. channel with b_w of 25' and side slopes of 3H:1V	$293,000	Rosemont to Ellen: trap. channel with b_w of 10', side slopes of 3H:1V; Ellen to Keyway: trap. channel with b_w of 25' and side slopes of 3H:1V	$293,000	Rosemont to Ellen: trap. channel with b_w of 10', side slopes of 3H:1V; Ellen to Keyway: trap. channel with b_w of 25' and side slopes of 3H:1V	$293,000
Keyway (NF-ST-5)	Remove exist. structure and build 3 – 10'x 8' RCBs or 240 SF of total opening required	$331,800	Remove exist. structure and build 3 – 10'x 8' RCBs or 240 SF of total opening required	$331,800	Remove exist. structure and build 3 – 10'x 8' RCBs or 240 SF of total opening required	$331,800
Keyway to Pennsylvania Reach (Approx. 530')	Trap. channel with b_w of 25' and side slopes of 3H:1V	$122,500	Trap. channel with b_w of 25'and side slopes of 3H:1V	$122,500	Trap. channel with b_w of 25'and side slopes of 3H:1V	$122,500
Pennsylvania (NF-ST-4)	Excavate upstream detention and build concrete structural wall. Improve inlet entrance	$539,900	Build concrete structural wall. Improve inlet entrance	$157,400	Excavate upstream detention and build concrete structural wall. Improve inlet entrance and restrict opening to 10'	$838,400
Total Costs		**$1,399,600**		**$1,574,000**		**$2,255,000**
Optional Costs	Channel clearing J.F. Kennedy to University	$99,000	Channel clearing J.F. Kennedy to University	$99,000	Channel clearing J.F. Kennedy to University	$99,000

Notes:

1. Alternative 1 – excavate storage upstream of Pennsylvania and modify outlet structure at Northwest Arterial; improve channel between Northwest Arterial and Keyway.
   Alternative 2 – excavate storage upstream of Northwest Arterial and improve channel between Northwest Arterial and Keyway.
   Alternative 3 – excavate storage upstream of Northwest Arterial and upstream of Pennsylvania Avenue and improve channel between Northwest Arterial and Keyway.
2. RCB- reinforced concrete box culvert; RCP – reinforced concrete pipe; b_w – bottom width; SF – square feet; trap. – trapezoidal; 3H:1V – ratio of horizontal to vertical
3. Contingencies (25%) were added to account for estimated quantities, unit price adjustments and miscellaneous work related items. An additional 25% was included for administrative, legal and engineering costs. Right-of-way, operation and maintenance and mitigation costs were not included. Costs based on Iowa Department of Transportation 1999 unit prices.

Keyway. For the Alternative No. 1 flow condition, roadway overtopping has also been eliminated and flooding is generally kept away from structures upstream of Keyway, although several entire back yards remain inundated.

3.5.2.2 Downstream of University Avenue

An apartment complex and one (1) home immediately downstream of the Pennsylvania Ave. culvert would still experience flooding with the improvements in place. A field investigation was performed and it was found that the first floor elevation of the apartments located closest to the channel was 767.5 and that the apartment building is constructed on piles without a basement. The existing 100-year water surface elevation was determined to be at elevation 765.5; therefore, the storm water affects the substructure of the apartments, but the first habitable floor is dry. The 100-year water surface elevation of Alternative No. 3 was 764.6, thereby reducing the 100-year water surface elevation less than 1-foot. The reduction in flooding of Alternative No. 3 quickly dissipates downstream of Pennsylvania Avenue/J.F. Kennedy Road and the associated costs quickly exceed the resulting benefit of the proposed alternatives.

There have been complaints of frequent flooding of agricultural ground in the reach immediately upstream of University Avenue. To investigate flooding problems in this area, the existing conditions hydraulic model described above was extended downstream to include University Avenue, U.S. Highway 20, and the reach immediately downstream of U.S. Highway 20. Under existing conditions, a 12-foot by 12-foot reinforced concrete box approximately 280 feet in length passes under University Avenue and a 26.5-foot by 18.5-foot arched concrete culvert passes under U.S. Highway 20. A normal depth boundary condition was used for this model.

Using this extended model, the installation of an additional structure at University was analyzed. With the existing model as a baseline, the effect of constructing an additional12-foot by 12-foot RCB culvert was investigated. Based on the model results, the additional structure would produce a maximum decrease in water surface elevation of 5.5 feet and reduce the inundated area by approximately 3.5 acres. An estimate of the cost required to construct an additional 12-foot by 12-foot RCB beneath University Ave. is included in the Opinion of Probable Construction Costs Appendix. Given the high cost of construction and the relatively small impact to the inundated agricultural ground adjacent to the stream, it is not recommended to add an additional structure.

3.5.3 Flood Inundation

Figures 3-14, 3-15, 3-16, and 3-17 depict the 100-year existing, Alternative No. 1, and Alternative No. 2 flood inundation. These figures show the approximate limits of flooding. The flood inundation for Alternative No. 3 is similar to Alternatives Nos. 1 and 2 and was not shown on the flood inundation figures.

3.6 RECOMMENDATIONS FOR IMPROVEMENT ALTERNATIVES

The program developed for the City consists of the recommended solutions for the North Fork Drainage Area. These recommended solutions are located within the city limits and could be implemented by the City. The peak discharges associated with all three North Fork Drainage Basin alternatives are summarized in Table 3.9.

Table 3.9
North Fork Catfish Creek Drainage Basin
Peak Runoff Summary for Existing and Proposed Hydraulic Conditions

Structure Id. No. [2]	HEC-HMS Node No. [3]	Location	Drainage Area (sq. mi)	100-Year Peak Runoff Rate (cfs) [1,4]			
				Existing	Alt. No. 1[5]	Alt. No. 2[5]	Alt. No. 3[5]
Main Channel							
NF-ST-1	26	Brunskill Rd.	3.8	3,130	3,120	3,120	3,100
NF-ST-2	22	US 20 (Dodge St.) Highway	3.2	2,490	2,430	2,460	2,270
NF-ST-3	49	University Avenue	3.0	2,420	2,370	2,390	2,070
NF-ST-4	18	J.F. Kennedy Rd. & Pennsylvania Ave.	2.1	2,080	2,080	2,040	1,830
NF-ST-5	9	Keyway	1.7	2,280	2,500	2,160	2,160
NF-ST-6	35	Rosemont St.	0.94	1,070	1,320	950	950
NF-ST-7	10	Northwest Arterial	0.52	650	590	180	180
NF-ST-8	36	Sunnyslope	0.26	770	770	770	770
NF-ST-9	3	Radford Rd.	0.16	470	470	470	470
NF-ST-10	5	Saratoga Rd.	0.06	160	160	160	160

Table 3.9
North Fork Catfish Creek Drainage Basin
Peak Runoff Summary for Existing and Proposed Hydraulic Conditions

Structure Id. No.[2]	HEC-HMS Node No.[3]	Location	Drainage Area (sq. mi)	100-Year Peak Runoff Rate (cfs)[1,4]			
				Existing	Alt. No. 1[5]	Alt. No. 2[5]	Alt. No. 3[5]
Tributary No. 1							
NF-T1-ST-1	57	Brunskill Rd	0.41	850	850	850	850
NF-T1-ST-2	54	US 20 (Dodge St.)	0.14	320	320	320	320
Tributary No. 2							
NF-T2-ST-1	33	Hillcrest Rd.	0.41	800	800	800	800
NF-T2-ST-2	30	Asbury Road	0.15	360	360	360	360
NF-T2-ST-3	29	Asbury Rd.	0.03	60	60	60	60
Tributary No. 3							
NF-T3-ST-1	12	NW Arterial	0.21	460	460	460	460
NF-T3-ST-2	13	Embassy West Dr.	0.10	210	210	210	210

Notes:
1. Peak runoff rates based on ultimate land use conditions and simulation of a 24-hour storm event
2. See Figure 3-1 for location of structure identification number.
3. See Figure 3-6 for location of HEC-HMS node and identification number.
4. Peak discharges reported are outflows from the specified nodes.
5. Alternative No. 1 – Build additional detention storage at Pennsylvania. Build channel improvements from Northwest Arterial to approximately 530 feet downstream of Keyway. Improve outlet at Northwest Arterial.
   Alternative No. 2 – Build additional detention storage at Northwest Arterial. Build channel improvements from Northwest Arterial to approximately 530 feet downstream of Keyway.
   Alternative No. 3 – Build additional detention storage at Pennsylvania Avenue and Northwest Arterial. Build channel improvements from Northwest Arterial to approximately 530 feet downstream of Keyway.

It is recommended to construct Alternative No. 2 and expand the existing storage upstream of the Northwest Arterial. This area aids in reducing the peak discharges downstream and provide a water quality benefit as the sediment-laden water is provided an opportunity to settle-out. While this alternative is not the least cost alternative, the additional incremental impact on flooding is substantial relative to the increased cost. Additionally, it is more aesthetically desirable, as it does not require destruction of the heavily wooded area upstream of Pennsylvania Avenue. It is also recommended to obtain 100-year flowage easements on private property and purchase flood prone properties as they become available. Commercial development opportunities exist along the left overbank, parallel to J.F. Kennedy Road. It is recommended that any development require a 2:1 excavation to fill ratio.

3.7 PROJECT PHASING

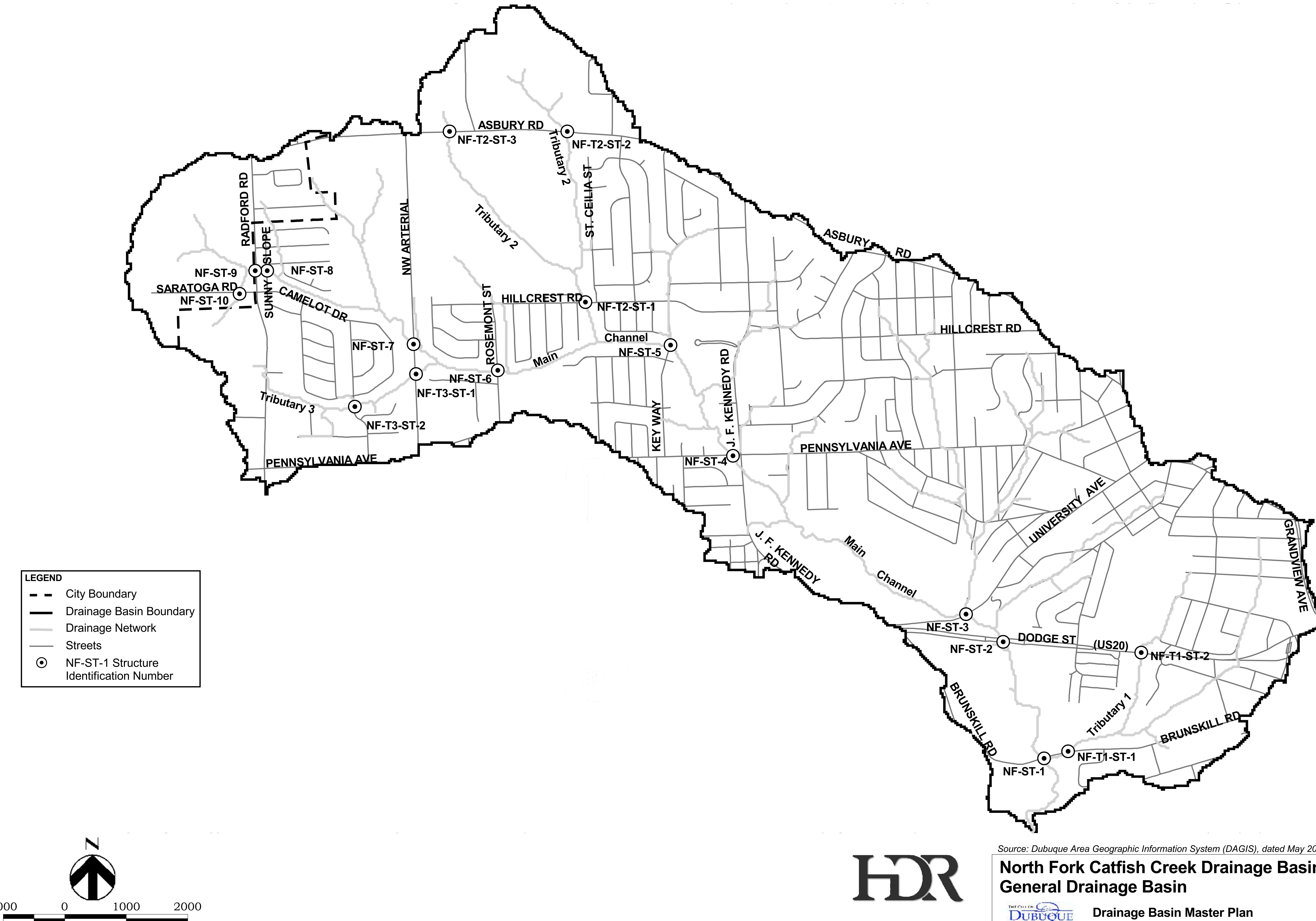
The problem areas and recommended improvements were ranked based on the resulting benefits in comparison to the costs of improvements in order to prioritize proposed improvements in the North Fork Drainage Basin. Table 3.10 presents a drainage basin priority based on other proposed improvements within the drainage basin. It is recommended to conduct detention improvements from the most upstream first and then proceed downstream. Channel improvements are to be constructed from downstream to upstream.

Table 3.10
North Fork Catfish Creek Drainage Basin
Recommended Improvements Summary

Drainage Basin Priority	Location	Recommended Improvements	Estimated Capital Cost[1]
1	Northwest Arterial (NF-ST-7)	Excavate upstream detention and build two-stage outlet structure.	$587,300
2	Pennsylvania (NF-ST-4)	Build concrete structural wall. Improve inlet.	$157,400
3	Keyway to Pennsylvania Reach (approx. 530')	Trap. channel with b_w of 25' and side slopes of 3H:1V.	$122,500
4	Keyway (NF-ST-5)	Remove existing structure and build 3 – 10' x 8' RCBs or 240 SF of total opening required	$331,800
5	Rosemont to Keyway Reach	Rosemont to Ellen: trap. channel with b_w of 10', side slopes of 3H:1V; Ellen to Keyway: trap. channel with b_w of 25' and side slopes of 3H:1V.	$293,000
6	Rosemont Street (NF-ST-6)	Build 1 additional 72" RCP or 57 SF of total opening required.	$61,800
7	Rosemont Street & Hillcrest Road (Special Problem Area)	Provide 12 AF of storage ($23,000) or Build 42" storm sewer ($90,000)	$23,000[3]
8	Northwest Arterial to Rosemont Street Reach	Trap. channel with b_w of 10' and side slopes of 3H:1V.	$20,200
9	Hillcrest Road (NF-T2-ST-1)	Provide 16.3 AF of storage ($76,000) or Remove existing structure and build 108" RCP ($110,000)	$76,000[3]
Total Recommended Improvement Cost			**$1,673,000**
	Saratoga Road (NF-ST-10) (Asbury Jurisdiction)[2]	Build 1 additional 3' RCP.	$21,000

Notes:

1. Estimated capital costs include contingencies (25%) to account for estimated quantities, unit price adjustments and miscellaneous work related items. An additional 25% was included for administrative, legal and engineering costs. Right-of-way, operation and maintenance and mitigation costs were not included. Costs based on Iowa Department of Transportation 1999 unit prices.
2. The Saratoga Road (NF-ST-10) improvements are within the jurisdiction of the City of Asbury; therefore, the improvement cost was not included with the other drainage basin improvements.
3. Total recommended improvement cost includes minimum cost for locations with multiple options.



Layout: General Drainage Basin
ENVIRO/CITY_OF_DUBUQUE/007-134_stormwater/GIS_MODELING/NCATFISH/REPORT_10_22_01.APR

Source: Dubuque Area Geographic Information System (DAGIS), dated May 2000

HDR

HDR Engineering, Inc.

North Fork Catfish Creek Drainage Basin
General Drainage Basin

The City of Dubuque

Drainage Basin Master Plan
City of Dubuque, Iowa

Date: FALL 2001

Figure: 3-1

4.0 BEE BRANCH DRAINAGE BASIN

4.1 GENERAL DRAINAGE BASIN DESCRIPTION

The Bee Branch Drainage Basin (Bee Branch) is located in the central vicinity of the Dubuque municipal limits and is shown on Figure 4-1. The drainage basin measures approximately 7.1 square miles, and flows in a southeasterly direction to the 16th Street Detention Cell and discharges into the Mississippi River through a 12-foot by 12-foot RCB equipped with dual sluice gates. The main Bee Branch channel is primarily located along West 32nd Street and Washington Street. The drainage area is roughly bounded by West 32nd Street to the north, Asbury Road and University Avenue to the south, Northwest Arterial to the west and the Mississippi River to the east.

The basin consists of several large subareas draining from large bluffs into a flat, densely populated lowland area within the old Mississippi River floodplain, hereafter referred to as the Couler Valley area. The subareas include West 32nd Street, Kaufmann Avenue, Locust Street, Washington Street (main Bee Branch trunk line storm sewer), Windsor, 11th Street, 14th Street, Upper Kerper and Lower Kerper. During flood events on the Mississippi River, runoff is diverted from Dock Street (at elevation 600.5 or stage 15) and Hamilton Street (at elevation 603.5 or stage 18) subareas through a 60-inch RCP located between Hamilton Street and Dock Street and a 78-inch RCP between Dock Street and a ditch south to Fengler Street. At elevation 598.5 or stage 13, the 8th Street Subarea is diverted into the 16th Street Detention Cell. Table 4.1 displays the drainage area for each subarea of the Bee Branch Drainage Basin.

Table 4.1 Bee Branch Drainage Basin Drainage Areas for Subareas	
Subarea	**Drainage Area (sq. mi.)**
West 32nd Street	1.90
Kaufmann Avenue	1.31
Locust Street	0.90
Windsor Avenue	0.39
Washington Street	1.15
Hamilton Street	0.16
Dock Street	0.16
Upper Kerper	0.28
Lower Kerper	0.09
14th Street	0.16
11th Street	0.21
8th Street	0.41
Total Bee Branch Drainage Basin Area:	**7.12**

The Bee Branch Drainage Basin is relatively steep, with an average terrain slope of approximately 37 percent. The overall slope of the main channel in the upland areas is approximately 2 percent, while the slope of the main channel in the flat Couler Valley area to the outlet is approximately 0.5 percent. Elevations in the drainage basin range from 594 feet NGVD at the 16th Street Detention Cell at the Mississippi River to 962 feet NGVD in the upper reaches of the drainage basin. Figure 4-2 shows the range of slopes for the Bee Branch Drainage Basin.

Information on the soil types and characteristics in the Bee Branch Drainage Basin was compiled by developing a digital soils database in GIS. Table 4.2 shows the relative representation and general hydrologic characteristics for the different soil series found in the Bee Branch Drainage Basin. The Bee Branch Drainage Basin consists of over 19 different soil types, of which the Fayette-Urban Land Complex and the Fayette Silt Loam series account for over 40 percent of the total drainage basin area. For modeling purposes, the different soil types were grouped by the NRCS hydrologic soil type as Type A, B, C, or D. Nearly the entire drainage basin consists of Type B soils, as depicted in Figure 4-3.

Table 4.2
Bee Branch Drainage Basin
Soil Type Summary

Soil Series	General Hydrologic Characteristics	Texture	Number of Polygons	% Area
Fayette-Urban Land Complex (4163C1, 4163D1, 4163E1)	(5 to 20% slopes) Moderately to strongly sloping, well-drained soil and urban land on side slopes in uplands within the City of Dubuque. Moderate permeability with medium to rapid runoff.	Silt Loam	39	27.3
Fayette Silt Loam (163C1, 163C2, 163D1, 163D2, 163E1, 163E2, 163F1, 163F2)	(5 to 25% slopes) Moderately to strongly sloping, well drained soil on side slopes in uplands. Moderate permeability with medium to rapid runoff.	Silt Loam	65	14.4
Nordness Rock Outcrop Complex (478G)	(18 to 60% slopes) Steep and very steep, well drained soils and rock outcrop on convex side slopes and escarpments. Moderate permeability with rapid runoff.	Silt Loam	8	14.1
Psamments-Urban Land (5070)	(0 to 2% slopes) Areas where material dredged from the Mississippi River has been deposited. Rapid to very rapid permeability with slow runoff.	Variable – Typically Coarse Sand	1	7.9
Urban Land-Dorchester Complex (4158B)	(2 to 5% slopes) Gently sloping areas of urban land with well drained Dorchester soil on wide bottomlands and along narrow drainage ways within the City of Dubuque. Moderate permeability with slow runoff.	Silt Loam	1	7.7
Urban Land-Lamont Complex (4110B)	(2 to 7% slopes) Gently sloping to moderately sloping areas of urban land with well-drained Lamont soil located on ridges and side slopes on high stream terraces within the City of Dubuque. Moderately rapid permeability with medium runoff.	Fine Sandy Loam	2	6.4
Dorchester-Volney Complex (496B)	(2 to 5 % slopes) Gently sloping, moderately well-drained to well-drained soils on alluvial fans and in the lower part of narrow drainageways.		4	4.1
	Various soils. 12 soil types ranging from 0.08% to 3.3% area.		131	18.1
	Total Percent Area			**100.0%**

Source: Soil Survey of Dubuque County, Iowa; SCS, December 1985.

The drainage system in the Bee Branch Drainage Basin consists of both natural channel and closed conduit sections. The majority of the drainage basin is highly developed and therefore much of the runoff is conveyed through storm sewer systems. Generally, natural channels are only present in less densely populated upland areas, specifically the West 32nd Street Subarea.

A land use database containing information for ultimate development was created based on the City's 1999 GIS Comprehensive Land Use Plan and supplemented with land use projections made by City staff. Land use classifications in Bee Branch range from open spaces to industrial, with the majority of the drainage basin being classified as low density and medium density residential and commercial land uses. The breakdown of land use within the Bee Branch Drainage Basin for ultimate development is shown in Table 4.3 and Figure 4-4.

Table 4.3
Bee Branch Drainage Basin
Land Use Summary

Land Use Classification	Area (acres)	% of Area
Streets	437	9.6
Industrial	195	4.3
Commercial	289	6.3
Institutional	624	13.7
High Density Residential	239	5.3
Medium Density Residential	1,377	30.2
Low Density Residential	205	4.5
Agricultural	146	3.2
Open Space and Grass	1,045	22.9
Total	**4,557**	**100.0%**

Source: City of Dubuque, Iowa Comprehensive Land Use Plan, 1999.
Note: Water bodies are incorporated into adjacent parcel land use categories.

While local flooding problems exist in the upland areas of the basin, the primary flooding problem in the Bee Branch occurs in the heavily developed Couler Valley area located in the former Mississippi River floodplain. While this area is protected from high Mississippi River stages by levees, flooding problems persist due to interior drainage. During large storm events, runoff from the steep upland areas rapidly drains toward the Couler Valley area. The flat topography of the Couler Valley area and the system of levees then slow the progression of the floodwaters to the Mississippi River. Additionally, existing storm sewer systems intended to collect and convey flood flows do not have the capacity to provide significant relief during extreme events. These problems combine to make the Couler Valley area of Dubuque prone to serious flooding during large storm events. Figure 4-5 depicts flooding from the main storm sewer trunk line of the Bee Branch for a 100-year 24-hour rainfall event and is an indication of the severity of the problem.

Few flood control measures have been implemented in the Bee Branch Drainage Basin, other than several regional detention cells on the main channel. The Bee Branch Drainage Basin is one of the few drainage basins in which regional detention of storm runoff is used and expanding existing detention cells may be a viable alternative for flood control.

Regional detention is most effective when applied in the upper portions of the drainage basin. Natural detention upstream of several drainage structures offers an opportunity to reduce the peak discharges and water surface elevations downstream. As the drainage basin becomes more developed, the number of available detention sites is reduced and detention options are eliminated or limited to expansion of existing detention cells. Regional detention sites were analyzed along with channel improvements that can be implemented as a potential means of flood control in the Bee Branch Drainage Basin.

Conveyance alternatives may also be a viable alternative for flood control in the lower reaches of Bee Branch. The topography of the lower reaches of Bee Branch does not provide any viable alternatives for detention sites, so increasing conveyance becomes the primary mechanism for minimizing flood impacts. Increasing the hydraulic capacity of the storm sewer system through resizing or adding relief sewers may reduce flooding impacts for smaller flood events.

The following sections describe each of the five (5) drainage subareas of the Bee Branch. The improvement alternative discussed is limited to the specific subarea. For this study, the main Bee Branch channel improvements along West 32nd Street are described within the West 32nd Street Subarea and the Bee Branch storm sewer trunk line sections.

4.2 WEST 32ND STREET DRAINAGE SUBAREA

4.2.1 General Subarea Description

The West 32nd Street Drainage Subarea is located in the upper reaches of the Bee Branch Drainage Basin. The drainage subarea measures approximately 1.9 square miles and drains into the West 32nd Street Detention Cell and then discharges into the Bee Branch storm sewer trunk line through a 10-foot x 9-foot concrete arch pipe. The drainage area is roughly bounded by West 32nd Street to the north, Asbury Road, Carter Road and Kane Street to the south, Northwest Arterial to the west, and Wildwood Drive to the east.

4.2.2 Flood Hydrology

The HEC-HMS model was utilized to compute the peak runoff rates for the 10-year, 50-year, 100-year and 500-year return period storm events. Runoff hydrographs were developed for each storm event for ultimate development conditions, as defined by the City's comprehensive land use plan. Figure 4-6 depicts the subbasin delineation, while Figure 4-7 is a schematic of the HEC-HMS model.

Table 4.4 provides a summary of peak runoff rates for selected storm events at key locations in the West 32nd Street Drainage Subarea. A summary of the peak runoff rates for all subbasin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 4.4
West 32nd Street Drainage Subarea Peak Runoff Summary
Existing Drainage System Conditions

Structure Id. No.[1]	HEC-HMS Node No.[2]	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs)[3,4]			
				10-Year	50-Year	100-Year	500-Year
West 32nd – Main Channel							
W32-ST-1	4	West 32nd St. Detention Cell[4]	1.9	860	1,750	2,140	3,300
W32-ST-2	3	Fink St.	1.8	1,120	1,770	2,140	3,290
W32-ST-5	10	West 32nd St.	1.8	1,120	1,770	2,140	3,280
W32-ST-6	2	Wildwood Dr.	1.7	1,100	1,730	2,100	3,220
W32-ST-9	1	Grandview Ave.	1.6	1,080	1,700	2,050	3,140
W32-ST-12	14	Carter Road	1.0	850	1,290	1,540	2,430
W32-ST-13	7	Pedestrian Bridge	0.38	360	460	560	1,160
W32-ST-14	21	J. F. Kennedy Road	0.30	310	400	630	1,070
W32-ST-15	18	Northwest Arterial	0.05	30	60	70	110
West 32nd Street - Tributary No. 1							
W32-T1-ST-1	90	Carter Road	0.19	160	260	320	500
W32-T1-ST-3	12	Kerry Ct.	0.15	130	210	270	410
W32-T1-ST-4	12	Killarney Ct.	0.15	130	210	270	410

Notes:
1. See Figure 4-6 for location of structure identification number.
2. See Figure 4-7 for location of HEC-HMS node and identification number.
3. Peak runoff rates based on ultimate land use conditions and simulation of a 24-hour storm event.
4. Peak discharges reported are outflows from the specified node.
5. Peak runoff rates for West 32nd Detention Cell represent peak outflows from the structure, not inflows into the detention cell.

4.2.3 Stream Hydraulics

HEC-HMS was used to determine the depth of overtopping for the drainage structures analyzed in the main channel and tributaries. At design points where a stage-storage-discharge relationship was analyzed by HEC-HMS, the peak stage was compared to the minimum overtopping elevation. At design points where the storage was negligible, an independent stage-discharge relationship was established using the inlet control nomograph from HDS-5. Weir flow and pipe flow were combined to find a peak stage, and this peak stage was then compared to the minimum overtopping elevation. A total of twelve road crossings were analyzed in the West 32nd Street Drainage Subarea. A summary of the hydraulic capacity for each of the crossings studied is presented in Table 4.5 for the 10-, 50-, and 100-year storm events.

Table 4.5
West 32nd Street Drainage Subarea
Existing Hydraulic Capacity of Stream Crossings Summary

Structure Identification No.	Location	Minimum Overtopping Elevation [2]	Depth of Overtopping (ft) [1]		
			10-Year	50-Year	100-Year
Main Channel					
W32-ST-1	West 32nd Street Detention Cell[4]	644.0	0.0	1.4	1.7
W32-ST-2	Fink Street	648.2	0.0[4]	0.0[4]	0.0[4]
W32-ST-5	West 32nd Street	660.4[3]	0.6	1.1	1.4
W32-ST-6	Wildwood Drive	662.9	0.3	1.1	1.4
W32-ST-9	Grandview Avenue	672.7	0.0	0.8	1.1
W32-ST-12	Carter Road	719.0	0.8	1.5	1.8
W32-ST-13	Pedestrian Bridge	796.3	0.0	0.0	0.0
W32-ST-14	J.F. Kennedy Road	820	0.0	0.0	0.7
W32-ST-15	Northwest Arterial	925.2	0.0	0.0	0.0
Tributary No. 1					
W32-T1-ST-1	Carter Road	710.3[3]	0.0	0.0	0.0
W32-T1-ST-3	Kerry Court	787.3[3]	0.0	0.4	0.6
W32-T1-ST-4	Killarney Court	810.5[3]	0.6	0.9	1.2

Notes:
1. Depth of overtopping obtained from HEC-HMS analysis, unless otherwise noted.
2. Minimum overtopping elevation based on topographic survey, unless otherwise noted.
3. Minimum overtopping elevation based on minimum roadway elevation obtained by interpolating City's DAGIS mapping.
4. Assumes reconstructed outlet structure. See stage-storage-discharge relationship in Hydrologic and Hydraulic Appendices.

The drainage standards/criteria of passing the design flood event without roadway overtopping was evaluated for each crossing. A summary of the return period for each of the crossings studied is presented in Table 4.6.

Table 4.6
West 32nd Street Drainage Subarea
Existing Hydraulic Capacity and Return Period of Stream Crossings Summary

Structure Identification No.	Location	Existing Structure Type[2]	Roadway Classification[3]	Hydraulic Capacity Return Period[1]	
				Required	Actual
West 32nd Street -Main Channel					
W32-ST-1	West 32nd St. @ Detention Cell[4]	Earthen Berm w/ Concrete Riser	Minor Arterial	50-yr with 1' overtop for 100-yr	1.4' overtop for 50-yr & 1.7' for 100-yr
W32-ST-2	Fink St.	22' x 11' RCB	Residential	10-yr with no 100-yr max. overtop	GT 100-yr[4]
W32-ST-5	West 32nd St.	9.8' x 7.7' RCB	Minor Arterial	50-yr with 1' overtop for 100-yr	0.6' overtop for 10-yr & 1.1' for 50-yr
W32-ST-6	Wildwood Dr.	2 – 10' x 6.3' RCB	Residential	10-yr with no 100-yr max. overtop	0.3' overtop for 10-yr & 1.1' for 50-yr
W32-ST-9	Grandview Ave.	2 – 10.5' x 7' RCAP	Minor Arterial	50-yr with 1' overtop for 100-yr	0.8' overtop for 50-yr & 1.1' for 100-yr
W32-ST-12	Carter Road	2 – 7' RCP	Collector	50-yr with 1.5' overtop for 100-yr	0.8' overtop for 10-yr & 1.5' for 50-yr
W32-ST-13	Pedestrian Bridge	7' RCP	N/A		GT 100-yr
W32-ST-14	J. F. Kennedy Rd.	5' CMP (in)/ 5'x 5.7' RCB (out)	Minor Arterial	50-yr with 1' overtop for 100-yr	50-yr with 0.7' overtop for 100-yr
W32-ST-15	Northwest Arterial	4' CMP (out)/ 4' RCP (in)	Principal Arterial	100-yr with 0' overtop	GT 100-yr
West 32nd Street - Tributary No. 1					
W32-T1-ST-1	Carter Road	2- 5' RCP	Collector	50-yr with 1.5' overtop for 100-yr	GT 100-yr
W32-T1-ST-3	Kerry Court	4' CMP	Residential	10-yr with no 100-yr max. overtop	0.0' overtop for 10-yr & 0.4' for 50-yr
W32-T1-ST-4	Killarney Court	3' RCP	Residential	10-yr with no 100-yr max. overtop	0.6' overtop for 10-yr & 0.9' for 50-yr

Notes:
1. Hydraulic capacity at minimum roadway elevation.
2. RCB – reinforced concrete box culvert, RCAP – reinforced concrete arch pipe, RCP – reinforced concrete pipe, CMP – corrugated metal pipe.
3. Roadway classification based on City of Dubuque's street classification index.
4. Assumes reconstructed outlet structure at West 32nd Street Detention Cell. Backwater impacts from West 32nd Street Detention Cell are reflected at Fink Street.

4.2.4 Problem Areas

The flood hydrology model provides the results needed for identification of areas that are not in compliance with the City's drainage standards/criteria. Problem areas in the West 32nd Street Drainage Subarea range from flooding of residential structures to inadequate drainage structures. A description of each of the identified problem areas is presented in Table 4.7 and Figure 4-21 shows the location of the identified problem areas.

Table 4.7
West 32nd Street Drainage Subarea
Identified Problem Area Summary

Structure Identification No.	Location	Criteria Violation[2]
Main Channel		
W32-ST-1	West 32nd Street Detention Cell	50-year flood event overtops for ultimate land use conditions
W32-ST-5	West 32nd Street	50-year flood event overtops for ultimate land use conditions
W32-ST-6	Wildwood Drive	10-year flood event overtops for ultimate land use conditions
W32-ST-9	Grandview Ave.	50-year flood event overtops for ultimate land use conditions
W32-ST-12	Carter Road	50-year flood event overtops for ultimate land use conditions
Tributary No. 1		
W32-T1-ST-4	Killarney Court	10-year flood event overtops for ultimate land use conditions
Special Problem Area		
	Kaufmann Avenue and Martin Drive	City staff identified problem area

Notes:
1. Roadway classification based on City of Dubuque street classification index.
2. Criteria violations based on roadway overtopping design storms presented in Table 2.7.

Although few problems exist within the West 32nd Street Subarea, the subarea is a major contributor to the flooding problems in the Couler Valley area. It is the largest of the upper drainage subareas flowing toward the Couler Valley area, and the West 32nd Street Detention Cell controls storm water runoff, thereby reducing the flooding downstream.

The six structures listed in Table 4.6 exceed the criteria presented in Table 2.7. The structures are overtopped at their required design storm and several structures have overtopping depths for the 100-year flood event in excess of the maximum allowable depth.

Special Study Area

A depressed area in the upper portion of the West 32nd Street Subarea also was investigated as a problem area. The area is located along Kaufmann Avenue east of the Kaufmann/J.F. Kennedy Road intersection. An existing storm sewer system adjacent to Kaufmann Avenue drains storm water to a tributary north of Kaufmann Avenue near the Kaufmann and Martin Drive intersection. The storm sewer trunk line was analyzed based on the assumption of full pipe flow using peak discharges from the HEC-HMS hydrologic model of the subarea. The existing pipe sizes were compared to the pipe sizes calculated to handle the 100-year peak discharge. It was determined that the storm sewer system is undersized to meet a 100-year design standard. Based on this analysis, the storm sewer along Kaufmann Avenue west of the Kaufmann/J.F. Kennedy Road intersection should be replaced with a 60-inch RCP to handle the 100-year peak discharge. The trunk line east of the Kaufmann/J.F. Kennedy intersection was shown to have capacity in excess of the 100-year peak discharge. It was noted there were few storm sewer inlets located in the depressed area. This should be investigated further to insure no flooding would occur due to lack of inlet capacity in this location.

4.2.5 Development of Alternative Solutions

The West 32nd Street Subarea predominantly consists of well-drained uplands and therefore contains few areas with flooding risks. Properties situated in the narrow valley of the Bee Branch running adjacent to West 32nd Street are at risk due to their proximity to the stream and their location in the lower portion of the subarea.

As mentioned previously, the West 32nd Street Subarea is a primary factor in the flooding hazards encountered in the Couler Valley area; however, expansion of existing detention cells and/or construction of additional storage areas in the West 32nd Street Subarea would reduce peak discharges and retain large volumes of storm water runoff, potentially further reducing the flooding problems downstream. Construction of storage areas in the upland portions of the subarea would have an impact on the flood damages experienced, not only in the Couler Valley but also, in the low lying areas of the West 32nd Street Subarea.

Channel improvements would have limited benefit because they would impact only a small number of properties located adjacent to the well-defined stream in the lower portion of the subarea. Flood proofing or property buyout would be a more effective alternative to address chronic flooding problems. The purchase of property located adjacent to the West 32nd Street Detention Cell also may be necessary to allow for storage capacity expansion.

4.2.5.1 Detention

Detention offers a means of controlling major flood events to prevent damage to downstream properties and infrastructure. Detention basins function by impounding runoff from an upstream basin and releasing it at a controlled rate to minimize downstream flooding. Within the Bee Branch Drainage Area, the West 32nd Street Subarea offers the best opportunity for storage. Table 4.8 summaries the detention sites investigated, while Figure 4-8 depicts the location of the sites.

Table 4.8
West 32nd Street Drainage Subarea
Detention Sites Investigated

Structure Identification No.[1]	Location	Description
W32-ST-1	West 32nd Street Detention Cell	The West 32nd Street Detention Cell is an existing detention cell with a gated outlet control. This site was investigated further.
W32-ST-9	Grandview Avenue	A storage area excavated upstream of the Grandview Avenue crossing south of West 32nd Street. The site was shown to produce a small decrease in peak discharge in the reach immediately downstream of Grandview Avenue. Due to the limited benefit to a relatively small number of properties located between Grandview Avenue and the West 32nd Street Detention Cell, this site was not investigated further. Greater impacts over a broader area would be realized if these efforts were focused on expanding storage at the West 32nd Street Detention Cell.
W32-DET-1	Former Ski Area	An earthen embankment constructed within the former ski area subbasin was investigated. Due to location of rock outcrops, the dam embankment would be situated in the upper reach of the subbasin. The contributing drainage area and the storage volume would be reduced making this site unfavorable. This site was not investigated further.
W32-DET-2	West 32nd Street	An earthen embankment constructed across West 32nd Street located east of the Carter Blvd and West 32nd Street intersection. This detention site would control the runoff from the upper 2/3 of the drainage subarea. This site would require relocating or eliminating a portion of West 32nd Street and relocating the Carter Blvd intersection. Several homes located within the flood pool would be relocated. Due to the extensive relocations, this site was not investigated further.

Table 4.8
West 32nd Street Drainage Subarea
Detention Sites Investigated

Structure Identification No.[1]	Location	Description
W32-DET-5	West 32nd Street/Carter Boulevard	The West 32nd Street drainage channel along Carter Blvd. is a natural depressional area and offers an opportunity to construct an embankment, on the east side of Carter Blvd. to create detention storage. This would require modifications to the Carter Blvd. intersection and the construction of a berm parallel to Carter Blvd. Due to the limited amount of storage, this site was not investigated further.
W32-ST-12	Carter Boulevard	The West 32nd Street drainage channel along Carter Blvd. is a natural depressional area and offers an opportunity to construct an embankment, on the west side of Carter Blvd. to create detention storage. Due to the limited amount of storage, this site was not investigated further.
W32-DET-3	Arabian Trail	Located on the west side of the West 32nd Street drainage channel along Carter Blvd and near Arabian Trail is a possible detention storage site. Due to the limited amount of storage, this site was not investigated further.
W32-DET-4	Upper Carter Boulevard	A possible detention site exists where the West 32nd Street drainage channel turns northeasterly along Carter Blvd. Constructing an earthen embankment and outlet system that would block the natural ravine in this area would restrict outflow. Due to the large storage potential and limited interference with utilities, this site was investigated further.
W32-ST-13	Pedestrian Crossing	By increasing the height of the pedestrian crossing berm, storage can be increased substantially with minor construction. This site was investigated further.
W32-ST-14	J.F. Kennedy	Excavating upstream of J.F. Kennedy would create additional storage. Due to the potential storage, this site was investigated further.
W32-ST-15	Northwest Arterial	Modifying the drainage structure by constricting the existing 4-foot opening will increase the peak storage behind the roadway embankment. The increased flood pool would be restricted to the park area. This site was investigated further.

Note:
1. See Figure 4-8 for structure identification number.

Table 4.9 presents a summary of the detention improvement alternatives considered for further investigation. Figure 4-9 shows a layout configuration for the potential detention sites identified. Each alternative considered either multiple detention cells or one large regional detention facility. All alternatives assumed the existing outlet works at the West 32nd Street Detention Cell would be removed and replaced to improve the hydraulics at the outlet. Expanding the West 32nd Street Detention included excavating, increasing the existing berm elevation, or a combination of the two. Table 4.10 summarizes the peak 100-year inflow and outflow discharges resulting from the five (5) improvement alternatives.

Table 4.9
West 32nd Street Subarea
Detention Improvement Alternative Summary

Alternative No.	Proposed Improvement Alternative
W32-1	Construct multiple upstream detention at J.F. Kennedy, pedestrian bridge, and upper Carter. Excavate additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
W32-2	Construct multiple upstream detention at J.F. Kennedy, pedestrian bridge, and upper Carter. Increase existing berm elevation to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
W32-3	Construct one large upstream detention at upper Carter. Excavate additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
W32-4	Construct one large upstream detention at upper Carter. Increase existing berm elevation to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
W32-5	Construct one large upstream detention at upper Carter. Increase existing berm elevation and excavate existing area to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.

The Upper Carter Boulevard (W32-DET-4) location and expansion of the West 32nd Street Detention Cell (W32-ST-1) are the two primary sources of potential storage capacity within the West 32nd Street Subarea. A natural ravine area is located along Carter Blvd in the West 32nd Street Subarea and this natural depressional area offers the opportunity to store the entire upstream 100-year runoff volume if a controlled gate is installed. Construction of either of these structures would need to be in accordance with the Iowa Department of Natural Resources, Class 3 dam classification.

Table 4.10
West 32nd Street Subarea
Detention Storage and Discharge Summary

Location	Drainage Area Controlled (square miles)	Existing			Alt. No. W32-1[1]			Alt. No. W32-2[2]			Alt. No. W32-3[3]			Alt. No. W32-4[4]			Alt. No. W32-5[5]		
		100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)	100-Year Discharge (cfs)		100-Year Peak Storage (Acre-Feet)
		Inflow	Outflow		Inflow	Outflow		Inflow	Outflow		Inflow	Outflow		Inflow	Outflow		Inflow	Outflow	
West 32nd St. Detention Cell (W32-ST-1)	1.90	2,150	2,140[6]	46	1,500	850	58	1,500	950	51	1,500	850	58	1,500	950	51	1,500	470	94
Upper Carter Blvd. (W32-DET-4)	0.80	1,250	1,250	0	830	0	176	830	0	176	1,240	0	182	1,240	0	182	1,240	0	182
Pedestrian Crossing (W32-ST-13)	0.38	630	560	3	230	180	17	230	180	17	610	540	3	610	540	3	610	540	3
John F. Kennedy (W32-ST-14)	0.30	650	630	7	640	200	33	640	200	33	640	590	7	640	590	7	640	590	7
Northwest Arterial (W32-ST-15)	0.05	70	70	0	70	50	1	70	50	1	70	50	1	70	50	1	70	50	1

Notes:

1. Alternative W32-1 - Construct multiple upstream detentions at J.F. Kennedy, pedestrian crossing, and upper Carter. Excavate additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
2. Alternative W32-2 - Construct multiple upstream detentions at J.F. Kennedy, pedestrian crossing, and upper Carter. Increase existing berm elevation to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located withan the 100-year flood pool.
3. Alternative W32-3 -Construct one large upstream detention at upper Carter. Excavate additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
4. Alternative W32-4 -Construct one large upstream detention at upper Carter. Increase existing berm elevation to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
5. Alternative W32-5 -Construct one large upstream detention at upper Carter. Increase existing berm elevation and excavate existing area to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.
6. Assumes reconstructed outlet structure. See stage-storage-discharge relationship included in Hydrologic and Hydraulic Appendices.

Construction of the three (3) detention cells located upstream of the Northwest Arterial (W32-ST-15), J. F. Kennedy Road (W32-ST-14) and the pedestrian crossing (W32-ST-13) would provide similar storage to the construction of the Upper Carter Boulevard detention cell: but at a greater capital cost. The West 32nd Street Detention Cell has an existing storage capacity of 46.0 acre-feet. Excavation, increasing the earthen berm elevation, or a combination of the two can obtain additional storage up to 94.0 acre-feet as shown in Table 4.10. Proposed improvements to the West 32nd Street Detention Cell are shown in Figure 4-10.

Table 4.11 summarizes an opinion of probable construction costs for each of the detention improvement alternatives within the West 32nd Street Subarea. Considering cost and impact on downstream flows, Alternative W32-5 was determined to be the most efficient and effective alternative. This alternative was then used as the basis to develop the downstream alternatives so a realistic comparison could be made without evaluating a complex matrix of interrelated options. Impacts from this alternative affecting the Couler Valley area are discussed in subsequent sections.

Table 4.11
West 32nd Street Subarea
Detention Improvement Alternative and Estimated Construction Cost Summary

Alternative No.	Proposed Improvement Alternative	Estimated Opinion of Probable Construction Costs[1]
W32-1	Construct multiple upstream detention at J.F. Kennedy, pedestrian bridge, and upper Carter. Excavate additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.	$5,250,000
W32-2	Construct multiple upstream detention at J.F. Kennedy, pedestrian bridge, and upper Carter. Increase existing berm elevation to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.	$4,000,000
W32-3	Construct one large upstream detention at upper Carter. Excavate additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.	$4,700,000
W32-4	Construct one large upstream detention at upper Carter. Increase existing berm elevation to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.	$3,500,000
W32-5	Construct one large upstream detention at upper Carter. Increase existing berm elevation and excavate existing area to provide additional storage at West 32nd Detention Cell and remove and replace outlet structure. Purchase properties located within the 100-year flood pool.	$4,700,000

Note:

1. Contingencies (25%) were added to account for estimated quantities, unit price adjustments and miscellaneous work related items. An additional 25% was included for administrative, legal and engineering costs. Rights-of-way, operation and maintenance and mitigation costs were not included. Costs based on Iowa Department of Transportation 1999 unit prices.

4.2.5.2 Channel and Drainage Structure Improvements

No opportunity was found to make significant impacts on flood damages through channel or storm sewer improvements in the West 32nd Street Subarea.

4.2.6 Recommendations for Improvement Alternatives

The program developed for the City of Dubuque consists of the recommended solutions for the West 32nd Street Drainage Subarea and could be implemented by the City.

It is recommended to implement the items contained in detention improvement alternative W32-5. This alternative includes construction of one large upstream detention cell at upper Carter, increasing the existing berm elevation and excavating existing area to provide additional storage at the West 32nd detention cell, removing and replacing the outlet structure at the West 32nd Street Detention Cell, and purchasing properties located within the 100-year flood pool of the West 32nd Street Detention Cell. While this alternative was not the least cost alternative, the additional incremental impact on flooding is substantial relative to the increased cost.

4.2.7 Project Phasing

The recommended improvements were ranked based on the resulting benefits in comparison to the costs of improvements. In this manner, the proposed West 32nd Street Subarea improvements were prioritized, as shown in Table 4.12. It is recommended that detention improvements at the most upstream areas are built first and then proceed downstream.

Table 4.12
West 32nd Street Drainage Subarea
Recommended Improvements Summary

Drainage Subarea Priority	Location	Recommended Improvements	Estimated Capital Cost[1]
1	Northwest Arterial (W32-ST-15)	Modify the drainage structure by constricting the existing 4-foot opening to increase the peak storage behind the roadway embankment.	$3,000
2	Upper Carter Blvd. (W32-DET-4)	Construct an earthen embankment and outlet system where the West 32nd Street drainage channel turns northeasterly along Carter Blvd. to block the natural ravine area and restrict outflow.	$874,000
3	West 32nd Street Detention Cell (W32-ST-1)	Increase existing berm elevation and excavate existing area to provide additional storage. Remove and replace outlet structure, and purchase properties located within the 100-year flood pool.	$3,831,000
		Total Estimated Capital Cost:	**$4,700,000**

Note:
1. Estimated capital costs include contingencies (25%) to account for estimated quantities, unit price adjustments, and miscellaneous work related items. An additional 25% was included for administrative, legal, and engineering costs. Right-of-way, operation and maintenance, and mitigation costs were not included. Costs based on Iowa Department of Transportation 1999 unit prices.

4.3 KAUFMANN AVENUE DRAINAGE SUBAREA

4.3.1 General Subarea Description

The Kaufmann Avenue Drainage Subarea (Kaufmann Subarea) is located in the west central portion of the Bee Branch Drainage Area. The drainage subarea measures approximately 1.3 square miles and drains in an easterly direction into the Bee Branch storm sewer trunk line through a 6-foot x 3-foot oval pipe. The drainage area is roughly bounded by Kane Street to the north, Clarke Drive to the south, Carter Road to the west, and North Main Street to the east.

Elevations in the subarea range from 914 feet in the upper portion to 618 feet at the outlet. The main drainage path through the subarea follows Kaufmann Avenue, where water is conveyed in the storm sewer and the street. The overall slope along this path is 2 percent.

4.3.2 Flood Hydrology

The HEC-HMS model was utilized to compute the peak runoff rates for the 10-year, 50-year, 100-year and 500-year return period storm events. Runoff hydrographs were developed for each storm event for ultimate development condition, as defined by the City's comprehensive land use plan.

Figure 4-11 depicts the subbasin delineation, while Figure 4-12 is a schematic of the HEC-HMS model. Table 4.13 provides a summary of peak runoff rates for selected storm events at key locations in the Kaufmann Avenue Drainage Subarea. A summary of the peak runoff rates for subbasin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 4.13
Kaufmann Avenue Drainage Subarea Peak Runoff Summary
Existing Drainage System Conditions

HEC-HMS Node No.[1]	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs)[2,3]			
			10-Year	50-Year	100-Year	500-Year
Kaufmann – Main Channel						
2	Kaufmann & Heeb	1.30	1,630	2,400	2,800	3,960
20	Kaufmann & Hempstead	1.22	1,620	2,370	2,760	3,920
10	Kaufmann & Valeria	1.15	1,600	2,350	2,740	3,880
6	Kaufmann & Kane	1.04	1,530	2,240	2,600	3,680
7	Kaufmann & Tributary	0.93	1,520	2,210	2,570	3,620
9	Kaufmann & Grandview	0.83	1,480	2,150	2,490	3,500
3	Kaufmann & Grandview	0.44	820	1,190	1,370	1,930
4	Kaufmann & Tributary (N)	0.34	770	1,120	1,290	1,810
16	Kaufmann & Tributary (S)	0.29	690	990	1,140	1,590
18	Kaufmann & Maryville Drive	0.22	550	790	910	1,270
12	Kaufmann & Tributary (S)	0.19	500	720	820	1,140
Kaufmann - Tributary No. 1						
11	Bunker Hill Golf Course	0.33	650	940	1,090	1,540
14	Bunker Hill Golf Course	0.18	500	740	860	1,210
21	Bunker Hill Road	0.05	180	260	290	390
22	St. Ambroise	0.06	160	250	300	430

Notes:
1. See Figure 4-12 for location of HEC-HMS node and identification number.
2. Peak runoff rates based on ultimate land use conditions and simulation of a 24-hour storm event.
3. Peak discharges reported are outflows from the specified node.

4.3.3 Stream Hydraulics

The main channel is along Kaufmann Avenue, where the storm sewer and the street convey flow. Flow in tributaries also is conveyed through streets and storm sewer systems. A simplified street cross-section and Manning's equation were used to determine the hydraulics in the streets on the main channel and tributaries studied. A rectangular cross-section using the average longitudinal street slope, curb height, and street width along with a Manning's roughness coefficient of 0.013 was assumed to determine the street or curb full capacity. An average longitudinal street slope and Manning's equation for full pipe flow determined the capacity of the existing storm sewer system. The existing hydraulic capacity of the system was equal to the summation of the pipe and street flow. The total conveyance was then compared to the 2-year and 10-year peak discharges. The flow in excess of the storm sewer capacity (not including curb full capacity) was used to size a proposed relief storm sewer system. The additional capacity required for the proposed relief sewer system was determined by subtracting the existing storm sewer pipe capacity from the peak discharges for the 2- and 10-year flood events. Manning's equation for full pipe flow and the existing average longitudinal street slope were used to calculate the pipe size required for the additional capacity.

A summary of the hydraulic capacity at several locations along the Kaufmann Avenue Drainage Subarea is presented in Table 4.14. The additional capacity required for a proposed relief sewer to convey the 2- and 10-year flood events is presented in the two right-hand columns of Table 4.14. Twelve of the storm sewer segments evaluated do not provide the hydraulic capacity necessary for a 2-year flood event, and all fifteen storm sewer segments analyzed fail to provide the hydraulic capacity needed for a 10-year flood.

4.3.4 Problem Areas

The hydrologic and hydraulic analyses provided the information needed for identification of areas not in compliance with the City's drainage standards/criteria. The frequency and hazards associated with particular flood events must be taken into account; therefore, the flood protection required may vary from street to street. Consequently, the sizing of storm sewers must be performed on a case-by-case basis, while considering the impact of each portion on the entire system.

Table 4.14
Kaufmann Avenue Drainage Subarea
Existing Hydraulic Capacity Summary

HEC-HMS Node No.[1]	Location	Roadway Classification	Existing Storm Sewer Size (in)	Roadway Slope (%)	Storm Sewer Capacity[2] (cfs)	Typical Curb Height (in)	Typical Street Width (ft)	Street Capacity - Curb Full[3] (cfs)	Total Existing Hydraulic Capacity - Street & Sewer (cfs)	Discharge (cfs)		Additional Hydraulic Capacity Required (Circular Pipe Size, inches)[4]	
										2-Year	10-Year	2-Year	10-Year
Kaufmann – Main Channel													
2	Kaufmann & Heeb	Minor Arterial	54 and 60	1.1	480	7	36	170	650	750	1,630	60	108
20	Kaufmann & Hempstead	Minor Arterial	54 and 60	1.3	520	7	36	190	710	750	1,620	60	102
10	Kaufmann & Valeria	Minor Arterial	84	1	640	7	36	170	810	740	1,600	42	102
6	Kaufmann & Kane	Minor Arterial	84	0.8	570	11	36	310	880	710	1,530	54	102
7	Kaufmann & Tributary	Minor Arterial	78 - 84	1.1	550	1	36	10	560	710	1,520	54	102
9	Kaufmann & Grandview	Minor Arterial	72	1.5	520	8	36	250	770	690	1,480	48	96
3	Kaufmann & Grandview	Minor Arterial	72	1.7	550	12	36	520	1070	390	820	None	60
4	Kaufmann & Tributary (N)	Minor Arterial	54 - 60	2.4	300	9	36	390	690	380	770	36	66
16	Kaufmann & Tributary (S)	Minor Arterial	54	2.3	300	8	36	310	610	340	690	30	60
18	Kaufmann & Maryville Drive	Minor Arterial	48 - 54	2.6	230	6	36	210	440	280	550	30	54
12	Kaufmann & Tributary (S)	Minor Arterial	48	3.1	250	5.5	36	200	450	250	500	None	48
Kaufmann - Tributary No. 1													
11	Bunker Hill Golf Course	N/A	48 - 54	2.2	210	-	40	-	210	340	650	42	66
14	Grandview	Minor Arterial	36 - 54	2.2	100	6	40	210	310	240	500	42	66
21	Bunker Hill Road	Residential	32	4.8	110	5	20	110	220	90	180	None	30
22	St. Ambroise	Collector	24 - 48	3.9	40	6	30	210	250	70	160	24	36

Notes:
1. See Figure 4-12 for location of HEC-HMS node and identification number.
2. Assumed Manning Roughness Coefficient of n=0.013 and full pipe flow conditions.
3. Assumed a rectangular cross-section for curb full flow conditions.
4. Additional capacity required for pipe flow only – no street flow.

4.3.5 Development of Alternative Solutions

Because the Kaufmann Avenue Drainage Subarea is located in the upland areas of the Bee Branch basin, construction of detention cells could potentially have an effect on flooding in the Couler Valley area. Detention storage in the upper portion of the subarea would provide relief where development has exceeded the capacity of the storm water conveyance system located downstream. Expansion of the capacity of storm sewer inlets and pipes may also significantly reduce flooding streets and adjacent properties within the Kaufmann Avenue Drainage Subarea.

4.3.5.1 Detention

While detention storage in the Kaufmann Avenue Drainage Subarea may have a significant impact on flooding problems in the Couler Valley area, few potential sites for construction of a detention cell exist. Only one site in the Kaufmann Avenue Drainage Subarea was identified as a possible location for a detention cell. The site is located on the roadway connecting Grandview Avenue and Kaufmann Avenue (Grandview/Kaufmann connector), as shown in Figure 4-13. To take advantage of the storage volume available at this location, an earthen embankment would be constructed across the roadway, thereby eliminating the Grandview/Kaufmann connector. During extreme storm events, the Grandview Avenue intersection would be closed to traffic and a detour would be posted on the approaching segment of each roadway.

The proposed embankment would pond water on Grandview to the northwest and south up to an elevation of approximately 720 feet. This is approximately the elevation at which water would begin to spill over the crest of the hill on Grandview Avenue. A maximum volume of 43.5 acre-ft would be stored at this elevation.

The impact of the construction of this detention cell was evaluated by modifying the HEC-HMS hydrologic model. The stage-storage relationship for the detention cell was estimated using topographic information from the DAGIS. Stage-discharge data was created assuming a 48-inch reinforced concrete pipe outlet with inlet control using FHWA nomographs and orifice discharge equations.

The 100-year peak inflow to the proposed Grandview/Kaufmann Detention Cell was estimated at 1,120 cfs. The storage provided by the proposed detention cell attenuated the peak discharge by 855 cfs (76%), to 265 cfs, immediately downstream of the detention cell and reduced the peak discharge by 30% (from 2,800 cfs to 1,950 cfs) at the outlet of the Kaufmann Avenue Subarea. However, flooding may still occur downstream because of the coincidence in peak discharges from adjacent subareas.

Construction of this detention cell would result in maximum ponding depths on Grandview Avenue up to 20 feet, so safety issues must be addressed. During storm events, the impacted section of Grandview Avenue would be closed to traffic. A system of advanced warning signs and barricades alerting people to the danger as well as prohibiting access to the area would be installed. The estimated cost for the Grandview/Kaufmann Detention Cell is approximately $530,000. A detailed breakdown of this cost estimate is provided in the Opinion of Probable Construction Costs Appendix.

4.3.5.2 Channel and Drainage Structure Improvements

The hydraulic capacities (pipe sizes) required for conveyance of the 2-year and 10-year flood events are reported in Table 4.14.

4.3.6 Recommendations for Improvement Alternatives

The program developed for the City of Dubuque consists of recommended solutions for the Kaufmann Avenue Drainage Subarea and could be implemented by the City. It is recommended to implement the proposed Grandview/Kaufmann Detention Cell. This proposal includes construction of a 20- to 25-foot earthen berm, installation of a 48-inch RCP outlet structure, and providing adequate advanced warning signs and proper street lighting.

4.3.7 Project Phasing

The only recommendation for the Kaufmann Avenue Drainage Subarea is the proposed Grandview/Kaufmann Detention Cell; therefore, no project phasing is required for the Kaufmann Avenue Drainage Subarea at this time. The recommended improvement is summarized in Table 4.15.

Table 4.15
Kaufmann Avenue Drainage Subarea
Recommended Improvements Summary

Drainage Basin Priority	Location	Recommended Improvements	Estimated Capital Cost[1]
1	Grandview/Kaufmann Detention Cell	Construct 20- to 25-foot earthen berm, install 48-inch RCP outlet structure, and provide adequate warning signs and lighting.	$530,000
		Total Estimated Capital Cost:	**$530,000**

Note:
1. Estimated capital costs include contingencies (25%) to account for estimated quantities, unit price adjustments, and miscellaneous work related items. An additional 25% was included for administrative, legal, and engineering costs. Right-of-way, operation and maintenance, and mitigation costs were not included. Costs based on Iowa Department of Transportation 1999 unit prices.

4.4 LOCUST STREET DRAINAGE SUBAREA

4.4.1 General Subarea Description

The Locust Street Drainage Subarea (Locust Subarea) is located in the upper reaches of the Bee Branch Drainage Subarea. The drainage subarea measures approximately 0.9 square miles and drains into the Bee Branch storm sewer trunk line through a 10.5-foot x 15-foot RCB. The drainage area is roughly bounded by Clarke Drive to the north, University Avenue to the south, Avoca Street to the west, and Central Street to the east.

Elevations in the subarea range from 900 feet in the upper portion to 620 feet at the outlet. The main drainage path through the subarea follows Locust Street, where water is conveyed in the storm sewer and the street. The overall slope along this path is 2 percent.

4.4.2 Flood Hydrology

The HEC-HMS model was utilized to compute the peak runoff rates for the 10-year, 50-year, 100-year and 500-year return period storm events. Runoff hydrographs were developed for each storm event for ultimate development conditions, as defined by the City's comprehensive land use plan.

Figure 4-14 depicts the subbasin delineation, while Figure 4-15 is a schematic of the HEC-HMS model. Table 4.16 provides a summary of peak runoff rates for selected storm events at key locations in the Locust Subarea. A summary of the peak runoff rates for all sub-basin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 4.16
Locust Street Drainage Subarea Peak Runoff Summary
Existing Drainage System Conditions

HEC-HMS Node No.[1]	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs)[2,3]			
			10-Year	50-Year	100-Year	500-Year
Locust– Main Channel						
2	16th and Cedar	0.90	870	1,330	1,580	2,310
1	17th and Central	0.88	860	1,320	1,570	2,290
3	17th & Dorgan Place	0.87	850	1,300	1,560	2,280
4	Locust and Clark	0.82	820	1,250	1,490	2,170
8	Locust and Pierce	0.70	740	1,130	1,340	1,960

Table 4.16 Locust Street Drainage Subarea Peak Runoff Summary Existing Drainage System Conditions						
HEC-HMS Node No. [1]	**Location**	**Drainage Area (sq. mi)**	**Peak Runoff Rate (cfs)** [2,3]			
			10-Year	**50-Year**	**100-Year**	**500-Year**
6	Locust and Kirkwood	0.64	690	1,060	1,260	1,840
5	Locust and Rosedale	0.54	650	990	1,180	1,720
7	Rosedale and Glen Oak	0.44	520	800	960	1,400
10	Rosedale and Adair	0.29	360	560	660	970
12	Alta Place	0.05	70	100	120	180
Locust- Tributary No. 1						
9	Loras and Cox	0.04	60	100	120	170
Locust- Tributary No. 2						
11	Vernon and Glen Oak	0.06	90	140	160	240
Locust- Tributary No. 3						
13	Custer and Grandview	0.05	70	100	120	180

Notes:
1. See Figure 4-15 for location of HEC-HMS node and identification number.
2. Peak runoff rates based on ultimate land use conditions and simulation of a 24-hour storm event.
3. Peak discharges reported are outflows from the specified node.

4.4.3 Stream Hydraulics

The main channel is along Locust Street, where the storm sewer and the street convey flow. Flow in tributaries also is conveyed through streets and storm sewer systems. A simplified street cross-section and Manning's equation were used to determine the hydraulics in the streets on the main channel and tributaries studied. A rectangular cross-section using the average longitudinal street slope, curb height, and street width along with a Manning's roughness coefficient of 0.013 was assumed to determine the street or curb full capacity. An average longitudinal street slope and Manning's equation for full pipe flow determined the capacity of the existing storm sewer system. The existing hydraulic capacity of the system was equal to the summation of the pipe and street flow. The total conveyance was then compared to the 2-year and 10-year peak discharges. The flow in excess of the storm sewer capacity (not including curb full capacity) was used to size a proposed relief storm sewer system. The additional capacity required for the proposed relief sewer system was determined by subtracting the existing storm sewer pipe capacity from the peak discharges for the 2- and 10-year flood events. Manning's equation for full pipe flow and the existing average longitudinal street slope were used to calculate the pipe size required for the additional capacity.

A summary of the hydraulic capacity at several locations along the Locust Subarea is presented in Table 4.17. The additional capacity required for a proposed relief sewer to convey the 2- and 10-year flood events is presented in the two right-hand columns of Table 4.17. Seven of the storm sewer segments evaluated do not provide the hydraulic capacity necessary for a 2-year flood event, and all thirteen storm sewer segments analyzed, except for the Vernon Street segment, fail to provide the hydraulic capacity needed for a 10-year flood.

4.4.4 Problem Areas

The hydrologic and hydraulic analyses provided the information needed for the identification of areas not in compliance with the City's drainage standards/criteria. The frequency and hazards associated with particular flood events must be taken into account; therefore, the flood protection required may vary from street to street. Consequently, the sizing of storm sewers must be performed on a case-by-case basis, while considering the impact of each portion on the entire system.

An evaluation of the existing storm sewer system along Rosedale Avenue from Grandview Avenue to Locust Street was performed for the 2-year and 10-year flood events. This segment of storm sewer corresponds to HEC-HMS nodes 6, 5, 7, 10, and 12 in Table 4.16 and 4.17. The hydraulic capacities presented in Table 4.17 are for a relief sewer to supplement the existing system. The pipe capacity required for a complete replacement of the existing sewer system also was performed. The results of this analysis are presented in Table 4.18 for the 2-year and 10-year flood events.

Table 4.17
Locust Street Drainage Subarea
Existing Hydraulic Capacity Summary

HEC-HMS Node No.[1]	Location	Roadway Classification	Existing Storm Sewer Size (in)	Roadway Slope (%)	Storm Sewer Capacity[2] (cfs)	Typical Curb Height (in)	Typical Street Width (ft)	Street Capacity - Curb Full[3] (cfs)	Total Existing Hydraulic Capacity - Street & Sewer (cfs)	Discharge (cfs)		Additional Hydraulic Capacity Required (Circular Pipe Size, inches)[4]	
										2-Year	10-Year	2-Year	10-Year
Locust Street – Main Channel													
2	16th and Cedar	Collector	54	2.0	280	5.5	36	160	440	400	870	42	72
1	17th and Central	Minor Arterial	54	6.3	490	11	36	870	1360	390	860	None	54
3	17th and Dorgan Place	Minor Arterial	54 – 60	2.9	340	20	36	1550	1890	390	850	30	66
4	Locust and Clark	Minor Arterial	72	0.8	380	18	36	690	1070	370	820	None	78
8	Locust and Pierce	Minor Arterial	72	0.9	400	6	36	120	520	340	740	None	72
6	Locust and Kirkwood	Minor Arterial	60	1.4	310	4.5	40	100	410	310	690	12	66
5	Locust and Rosedale	Minor Arterial	36 – 72	1.7	90	7	36	210	300	300	650	54	78
7	Rosedale and Glen Oak	Collector	36 – 42	2.4	100	7	36	260	360	240	520	42	66
10	Rosedale and Adair	Collector	15	3.0	10	8	36	360	370	160	360	42	60
12	Alta Place	Residential	24 – 36	3.6	40	5.5	30	180	220	30	70	None	24
Locust Street - Tributary No. 1													
9	Loras and Cox	Minor Arterial	12 – 36	5.5	10	5.5	24	170	180	30	60	18	24
Locust Street - Tributary No. 2													
11	Vernon and Glen Oak	Residential	36 – 48	6.4	170	7	24	270	440	40	90	None	None
Locust Street - Tributary No. 3													
13	Custer and Grandview	Minor Arterial	24 – 30	3.6	40	5	24	120	160	30	70	None	24

Notes:
1. See Figure 4-15 for location of HEC-HMS node and identification number.
2. Assumed Manning Roughness Coefficient of n=0.013 and full pipe flow conditions.
3. Assumed rectangular cross-section for curb full flow conditions.
4. Additional capacity required for pipe flow only – no street flow.

Table 4.18 Locust Street Drainage Subarea Hydraulic Capacity for Storm Sewer Replacement						
HEC-HMS Node No. [1]	Location	Existing Storm Sewer Size (in)	Discharge (cfs)		Total Hydraulic Capacity Required (Circular Pipe Size, inches) [2]	
			2-Year	10-Year	2-Year	10-Year
Locust Street – Main Channel						
6	Locust & Kirkwood	60	310	690	66	84
5	Locust & Rosedale	36 – 72	300	650	60	78
7	Rosedale & Glen Oak	36 – 42	240	520	54	66
10	Rosedale & Adair	15	160	360	42	60
12	Alta Place	24 – 36	30	70	24	30

Notes
1. See Figure 4-15 for location of HEC-HMS node and identification number
2. Total capacity required for replacement of existing storm sewer.

4.4.5 Development of Alternative Solutions

Although alternatives were developed to address the special problem area along Rosedale Avenue, general alternatives focused on the entire subarea were not established. No available sites for regional detention exist because of the topography and land use of the Locust Subarea. It should also be noted that the expansion of the capacity of storm sewer inlets and pipes might significantly reduce street and property flooding within the Locust Subarea.

4.4.5.1 Detention

Regional detention is not viable within the Locust Subarea, as the subarea is fully developed.

4.4.5.2 Channel and Drainage Structure Improvements

The hydraulic capacity (pipe sizes) required for conveyance of the 2- and 10-year flood events are reported in Tables 4.17 and 4.18.

4.4.6 Recommendations for Improvement Alternatives

The sizing of individual storm sewers should be performed on a case-by-case basis. The potential for flood damage posed by the various storm events should be weighed against the cost of improvement.

4.4.7 Project Phasing

No project phasing is required for the Locust Subarea at this time.

4.5 CENTRAL BUSINESS DISTRICT – NORTH SUBAREAS

4.5.1 General Subarea Description

The Central Business District – North Drainage Subareas (Central Business District – North) are located in the center of the Bee Branch Drainage Subarea and includes Washington Street, Windsor, Hamilton, Dock and Upper Kerper Drainage Subareas. The drainage subarea measures approximately 1.9 square miles and is roughly bounded by West 32nd Street to the north, 17th Street to the south, Central Street to the west, and Peosta Channel to the east.

Elevations in the subarea range from 644 feet in the upper portion to 594 feet at the 16th Street Detention Cell. The main drainage path through the subarea follows Washington Street, where water is conveyed in the storm sewer and the street. The overall slope along this path is 0.5 percent.

The main Bee Branch storm sewer trunk line is the main channel in the Bee Branch Drainage Area. The storm sewer begins approximately 625-feet west of the intersection of West 32nd Street and Saunders Street then travels in a southeasterly direction to Washington Street and 28th Streets, as shown in Figures 4-18A and 4-18B. The trunk line then follows Washington Street to 24th Street where the alignment changes to Elm Street. Near 21st Street the alignment leaves the street and transverses under commercial and industrial properties to its outflow into the 16th Street Detention Cell. The storm sewer begins as a 10-foot by 9-foot concrete arch and terminates as a 20-foot x 12-foot stone box. The City has inspected and cleaned the storm sewer within the last couple of years and has rated the condition of the storm sewer as "good". The total length of the truck line is approximately 10,400-feet and the pipe falls approximately 40 feet over its length for an average slope of 0.4%.

Numerous collector storm sewers enter into the Bee Branch trunk line system. The West 32nd, Kaufmann, and Locust Subareas all intersect the Bee Branch trunk line storm sewer system with a variety of collector pipe sizes. The largest collector pipe is a 10.5-foot x 15-foot reinforced concrete box draining the Locust Subarea entering the Bee Branch trunk line at 15th and Sycamore Streets. A 10-foot x 9-foot concrete arch pipe discharges into the Bee Branch trunk line from the West 32nd Subarea, and a 6-foot x 3-foot oval pipe drains the Kaufmann Subarea into the Bee Branch trunk line. Several 12-inch pipes also join the trunk line in the upper portion of the storm sewer trunk line.

The 16th Street Detention Cell is located at the outlet of the Bee Branch trunk line storm sewer. It is an interior drainage ponding area adjacent to and protecting the Couler Valley area from Mississippi River floodwaters. The 16th Street Detention Cell pump station is an outdoor installation consisting of two pumps rated at 90,000-gpm at an 18.7-foot total dynamic head (TDH) and one pump rated at 20,000-gpm at a 25.4-foot TDH. Twin 12-foot by 12-foot box culverts serve as a gravity outlet into the Peosta Channel. During periods of high river stages, the culverts are closed on the riverside with sluice gates mounted on the discharge headwall of the outlets. When the gates are closed, the culverts serve as a sump and intake bay for the pumps.

4.5.2 Flood Hydrology

The HEC-HMS model was utilized to compute the peak runoff rates for the 10-year, 50-year, 100-year and 500-year return period storm events. Runoff hydrographs were developed for each storm event for ultimate development conditions, as defined by the City's comprehensive land use plan. Washington Street is the primary street where the Bee Branch storm sewer trunk line is located. The HEC-HMS model was used to route hydrographs to the Bee Branch storm sewer trunk line. Where subareas feed into the Bee Branch storm sewer trunk line, the hydrographs were exported from HEC-HMS to XP-SWMM. The XP-SWMM model was then used to route and combine hydrographs along the Bee Branch storm sewer trunk line itself.

Figure 4-16 depicts the subbasin delineation, while Figure 4-17 is a schematic of the HEC-HMS model. Tables 4.19 and 4.20 provide a summary of peak runoff rates for selected storm events at key locations in the Central Business District – North. A summary of the peak runoff rates for the subbasin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 4.19
Central Business District - North Drainage Subareas
Peak Runoff Summary for Existing Drainage System Conditions

HEC-HMS Node No.[1]	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs)[2,3]			
			10-Year	50-Year	100-Year	500-Year
Windsor Avenue– Main Channel						
BB_17	24th Street and Washington	0.39	490	720	840	1,200
53	Windsor and Burden	0.27	430	620	730	1,020
Hamilton Street– Main Channel						
Hamilton	Peosta Channel	0.16	150	230	280	420
Dock Street – Main Channel						
Dock	Peosta Channel	0.16	180	280	320	470
Upper Kerper						
Upper Kerper	16th Street Detention Cell	0.24	150	240	290	440
BB_27B	15th Street and Sycamore	0.05	50	70	80	120

Notes

1. Peak runoff rates based on ultimate land use conditions and simulation of a 24-hour storm event
2. See Figure 4-17 for location of HEC-HMS node and identification number
3. Peak discharges reported are outflows from the specified node
4. See Table 4.20 for peak runoff rates along Bee Branch trunk line.
5. Peak discharges from subareas and subbasins calculated in HEC-HMS, peak discharges along Bee Branch storm sewer trunk line calculated in XP-SWMM

4.5.3 Stream Hydraulics

The complexity and importance of the Bee Branch storm sewer trunk line suggested a separate, detailed analysis should be performed. The Bee Branch storm sewer trunk line was analyzed using an XP-SWMM model, and a simplified street cross-section and Manning's equation were used to analyze the hydraulics of other storm sewers and streets not directly impacted by the main Bee Branch trunk line.

Table 4.20
Washington Street Drainage Subarea Peak Runoff Summary
Existing Drainage System Conditions Along Bee Branch Trunk line

Location	XP-SWMM Node	Comment	Total Flow (cfs)[1]			Street Flow (cfs)			Pipe Flow (cfs)		
			10-Yr	50-Yr	100-Yr	10-Yr	50-Yr	100-Yr	10-Yr	50-Yr	100-Yr
16th Street Detention Cell	28	See Note 2 below.	3,100	4,640	7,050	-	-	-	-	-	-
15th Street & Sycamore	27	Locust Street Subarea Inflow. See Note 4 below.	3,100	4,640	7,050	0	0	1,330	3,100	4,640	5,720
18th/19th Streets & Railroad	24	See Note 4 below.	2,210	3,730	4,790	150	1,610	2,730	2,060	2,120	2,060
22nd Street & Elm	19	Kaufmann Avenue Subarea Inflow	2,300	3,560	4,580	960	2,230	3,260	1,340	1,330	1,320
24th Street & Elm	17	Windsor Avenue Subarea Inflow	1,380	2,210	2,850	390	1,180	1,820	990	1,030	1,030
26th Street & Washington	14		940	1,750	2,230	80	770	1,270	860	980	960
27th Street & Washington	12		850	1,720	2,180	10	720	1,200	840	1,000	980
30th Street & Washington	8		800	1,660	2,090	0	630	1,060	800	1,030	1,030
West 32nd Street & Saunders Street	2		780	1,640	2,040	0	680	1,080	780	960	960
West 32nd Street Detention Cell	1	See Note 3 below.	770	1,590	1,980	-	-	-	-	-	-

Notes
1. Peak runoff rates based on ultimate land use conditions and simulation of a 24-hour storm event
2. Peak inflow into 16th Street Detention Cell from Bee Branch storm sewer trunk line.
3. Peak existing conditions outflow from West 32nd Street Detention Cell
4. Street and pipe flow locations differ due to overland flow path. Reported street flow for 15th and Sycamore (XP-SWMM Node 27) is located at the railroad crossing and 15th Street. Reported street flow for railroad between 18th and 19th Streets (XP-SWMM Node 24) is located at 19th and Elm Street
5. See Figure 4-18A and 4-18B for location of XP-SWMM node and identification number.

4.5.3.1 XP-SWMM Analysis

XP-SWMM is a proprietary program developed by XP Software and is an enhanced version of the Environmental Protection Agency (EPA) Storm Water Management Model (SWMM). XP-SWMM models unsteady closed conduit and open channel flow using a series of links and nodes. Links represent pipe segments and channel reaches, while nodes represent manholes, junctions, and storage cells. Nodes connect links to create linear, branched, or looped systems making it possible to model complex networks of pipes and channels.

An XP-SWMM model was developed to assess flooding problems along the Bee Branch trunk line located in the Couler Valley area of the City of Dubuque. The Bee Branch trunk line model contains several broad assumptions and is intended to be a useful tool in evaluating alternative flooding impacts.

4.5.3.1.1 Model Assumptions

<u>Main Storm Sewer Trunk line</u>

The model geometry of the Bee Branch trunk line was based on profile drawings provided by the City. Information available on this profile included slope, shape, size and material of the segments making up the trunk line. Twenty-eight (28) nodes were created in the XP-SWMM model where a change in slope, shape, size or material occurred. Several additional nodes were inserted where minor tributary sewer lines connected to the main line and it was not reasonable to shift the junction to the next upstream node. The twenty-eight (28) nodes along the main trunk line of the storm sewer are shown in Figures 4-18A and 4-18B.

Friction losses in XP-SWMM are calculated based on Manning's roughness coefficients. In general, minor losses were not considered on the main truck line storm sewer. Losses at manholes were neglected because manholes do not involve expansion and contraction of flow. Manholes are not a barrel section, but are a tap in the top of the conduit and are small relative to the conduit cross-section. Junction losses were also neglected due to the small amount of flow coming from the tributaries relative to the flow in the main trunk line. In cases where major bend losses were apparent, an equivalent Manning's roughness coefficient in the link where the bend occurs was calculated to include these effects. Where pipe size changed by more than 20%, entrance or exit losses were included to account for contraction or expansion of flow. Expansion and contraction coefficients of 0.7 and 0.3 were used respectively. Entrance and exit loss coefficients of 0.5 and 1, respectively, were used for flows entering and exiting detention cells.

Inlets along the main trunk line were assumed to have negligible capacity based on field observations and were, therefore, not modeled. Each node along the main line was "sealed" to prevent overflow along the main trunk line and force any excess flow into the streets via the tributary pipes. Generally, very few inlets exist along the main trunk line and the majority of the inlets are located along the tributaries. See Connection Between Storm Sewer and Street for further discussion.

Tributary Storm Sewer Lines

Tributaries to the main trunk line were identified using the DAGIS. Tributary pipes were assumed to have a slope of 0.5% based on the slope from two representative storm sewers. A 50-foot section of each pipe was included in the model to partition inflow hydrographs between the main trunk line and the street and to allow flow to reenter the trunk line when capacity is available. See Connection Between Storm Sewer and Street for further discussion.

Streets

The street geometry was approximated with a simplified rectangular cross-section consisting of a 33-foot wide street (based on aerial photos) with a 1-foot high curb (based on field observation) and a 16.5-feet overbank on either side of the street (based on aerial photos), or a total flow width of 66-feet. When effective flow was anticipated across more than one street, differences in elevation between streets inverts were taken from the DAGIS and used to create a representative cross-section. The effective flow area was limited to a single street cross-section in the upper portion of the trunk line, then changed to a double street cross-section near Jackson Street and finally to a triple street cross-section near 15th Street to the 16th Street Detention Cell.

In general, invert elevations for the street sections were based on the ground profile elevations described on the storm sewer profile sheets provided by the City. Reach lengths were based on the steepest path through the Couler Valley area determined from the DAGIS. Where the steepest path differed significantly from the path of the trunk line, elevations were established by the DAGIS. Typically, the steepest path follows the main trunk line of the storm sewer, but near 20th and Elm, where the storm sewer turns to the southeast, the steepest path continues to follow Elm and then 15th Street to the 16th Street Detention Cell. Links representing the streets were connected to links representing the tributary sewer pipes allowing for exchange of flow between the street and the sewer.

Connection Between Storm Sewer and Street

The connections between the storm sewer and the street are modeled in the nodes common to the tributary storm sewer pipe and street. Inlet capacity was neglected and the connection merely consists of two street links perched above the sewer link at the same node. Inflow hydrographs were input at these nodes. When the capacity of either the main or tributary storm sewer limits inflow, water is forced into the street and when capacity is not limited in the pipe, water from the street can enter or reenter the storm sewer. Inlets directly connected to the main storm sewer line were assumed to have a negligible effect based on field observations and were not modeled; therefore, connections only exist at the upstream end of each tributary pipe.

16th Street Detention Cell

The 16th Street Detention Cell's stage-storage relationship is based on information obtained from the U.S. Army Corps of Engineers (USACE), Rock Island District Design Memorandum, dated 1966. Water surface elevations in the cell are dependent on Mississippi River stage and operation of the outlet works.

The 16th Street Detention Cell outlet works were designed to discharge interior drainage by gravity at low Mississippi River stages and pump at high Mississippi River stages. The modeled outlet works geometry, twin 12-foot x 12-foot RCBs, is based on information contained in the USACE Design Memorandum. During high river stages, the outlet is sealed with sluice gates and flows are diverted over the levee by three (3) pumps: two 90,000-gpm pumps, rated at 18.7-feet total dynamic head (TDH), and one 20,000-gpm pump, rated at 25.4 TDH. The geometry and pump curves for the 90,000-gpm pumps are based on information in the USACE Design Memorandum. Less information was available on the 20,000-gpm pump, and its pump curve was assumed to have the same characteristics as the other pumps. All three pumps operate simultaneously with a minimum water surface elevation of 591.5 feet (below 591.5 feet, cavitation occurs).

Diversions

Three (3) subareas are diverted directly into the 16th Street Detention Cell during high Mississippi River stages. Depending on the tailwater condition created by the Mississippi River, different hydrographs were applied to XP-SWMM Node 28. The following subareas and diversion elevations are as follows:

- 8th Street Subarea – 598.5 feet
- Dock Street Subarea – 600.5 feet
- Hamilton Street Subarea – 603.5 feet

4.5.3.1.2 16th Street Operating Scenarios

Three (3) operating scenarios: normal, current and minimum, were modeled based on the following constraints:

1. Normal Operating Conditions. Mississippi River water surface elevation is 594.3 feet or the elevation in which 50% of the time the Mississippi River is equal to or exceeded. Hamilton Street, Dock Street and 8th Street Subareas are not diverted to

the 16th Street Detention Cell at elevation 594.3; therefore, they are not included in this operating scenario.

2. Current Gate Closure Operating Conditions. The City's current operating procedure is to close the sluice gates when the Mississippi River's water surface elevation is at 598.5 feet. Only 8th Street Subarea is diverted at elevation 598.5; therefore, it was included in this operating scenario.

3. Minimum Water Surface Elevation Operating Conditions. The minimum allowable water surface elevation in the 16th Street Detention Cell is at elevation 591.5 feet. This scenario assumes the Mississippi River water surface elevation is at or above the gate closure elevation and the 16th Street Detention Cell was pumped down to elevation 591.5 feet in anticipation of large storm water discharges. Also, 8th Street Subarea flows are diverted to the 16th Street Detention Cell under this scenario because the Mississippi River water surface elevation is assumed to be at or above the gate closure elevation.

4.5.3.1.3 Street Flooding Depths

The XP-SWMM model, assuming normal operating conditions, provides a depth of flooding or ponding at various nodes along the Bee Branch storm sewer trunk line. Figure 4-5 depicts the limits and depth of ponding along the Bee Branch main trunk line. The greatest depth of ponding is between 24th Street and the 16th Street Detention Cell. Table 4.21 summaries the depth of flooding for the 10-, 50- and 100- year events. In addition, from Table 4.21 it is noted that the Bee Branch storm sewer trunk line system has a capacity of less than the 10-year storm event.

Table 4.21
Central Business District – North Drainage Subareas
Existing Hydraulic Capacity Summary

XP-SWMM Node	Location	Depth of Ponding (ft)[1]		
		10-Year	50-Year	100-Year
Washington Street Drainage Subarea				
28	16th Street Detention Cell[2]	8.02	10.6	12.9
27	15th Street and Railroad[3,4]	0	0	0.7
24	19th and Elm[1]	1.5	4.0	4.7
19	22nd Street and Elm	3.4	4.8	5.5
17	24th Street and Elm	2.7	4.2	5.0
14	26th Street and Jackson[3]	0.5	1.8	2.2
12	27th Street and Jackson[3]	0.2	1.8	2.2
8	30th Street and Jackson[3]	0	1.7	2.2
2	West 32nd Street and Saunders Street	0	1.7	2.1
1	West 32nd Street Detention Cell[2]	11.0	13.5	13.8

Notes:
1. Depth of ponding based on rectangular street section, average longitudinal street slope and peak discharge.
2. Depth of ponding represents peak stage in detention cell.
3. Street node location differs from storm sewer node location.
4. Depth of ponding represents depth of overtopping at railroad. Railroad profile is elevated relative to surrounding topography.

4.5.3.2 Manning's Analysis

The main channel is along Washington Street, where the storm sewer and the street convey flow. Flow in tributaries also is conveyed through streets and storm sewer systems. A simplified street cross-section and Manning's equation were used to determine the hydraulics in the streets on the main channel and tributaries studied. A rectangular cross-section using the average longitudinal street slope, curb height, and street width along with a Manning's roughness coefficient of 0.013 was assumed to determine the street or curb full capacity. An average longitudinal street slope and Manning's equation for full pipe flow determined the capacity of the existing storm sewer system. The existing hydraulic capacity of the system was equal to the summation of the pipe and street flow. The total conveyance was then compared to the 2-year and 10-year peak discharges. The flow in excess of the storm sewer capacity (not including curb full capacity) was used to size a proposed relief storm sewer system. The additional capacity required for the proposed relief sewer system was determined by subtracting the existing storm sewer pipe capacity from the peak discharges for the 2- and 10-year flood events. Manning's equation for full pipe flow and the existing average longitudinal street slope were used to calculate the pipe size required for the additional capacity.

A summary of the hydraulic capacity at several locations along Windsor, Hamilton, and Dock Subareas is presented in Table 4.22. The additional capacity required for a proposed relief sewer to convey the 2- and 10-year flood events is presented in the two right-hand columns of Table 4.22. Some of the storm sewer segments evaluated do not provide the hydraulic capacity necessary for a 2- or 10-year flood event.

4.5.4 Problem Areas

The flood hydrologic and hydraulic analyses provide the information needed for identification of areas not in compliance with the City's drainage standards/criteria. The frequency and hazards associated with particular flood events must be taken into account; therefore, the flood protection required may vary from street to street. Consequently, the sizing of storm sewers must be performed on a case-by-case basis, while considering the impact of each portion on the entire system.

The City requested analysis of a particular problem area in the Windsor Subarea. An evaluation of the existing storm sewer system along Windsor Avenue from Burden Street to Sutter Street was performed for the 2-year and 10-year flood events. The hydraulic capacities presented in Table 4.22 are for a relief sewer to supplement the existing system. The pipe capacity required for a complete replacement of the existing sewer system also was performed. The results of this analysis are presented in Table 4.23 for the 2-year and 10-year flood events. The existing sewer system consists of a 36-, 24-, 48-, and 54-inch system starting at the intersection of Burden and Windsor. According to the analysis summarized in Table 4.23, a 48-inch pipe would be required to convey the 2-year event or a 66-inch pipe to convey the 10-year event.

Table 4.22
Central Business District – North Drainage Subareas
Existing Hydraulic Capacity Summary

HEC-HMS Node No.[1]	Location	Roadway Classification	Existing Storm Sewer Size (in)	Roadway Slope (%)	Storm Sewer Capacity[2] (cfs)	Typical Curb Height (in)	Typical Street Width (ft)	Street Capacity - Curb Full[3] (cfs)	Total Existing Hydraulic Capacity - Street & Sewer (cfs)	Discharge (cfs)		Additional Hydraulic Capacity Required (Circular Pipe Size, inches)[4]	
										2-Year	10-Year	2-Year	10-Year
Windsor Avenue – Main Channel													
BB_17	24th & Elm	Collector	54	2.5	310	8	36	330	640	240	490	None	48
53	Windsor & Burden	Minor Arterial	12 - 24	2.5	10	6	36	200	210	210	430	48	66

Notes:
1 See Figure 4-17 for location of HEC-HMS node and identification number.
2 Assumed Manning Roughness Coefficient of n=0.013 and full pipe flow conditions.
3 Assumed rectangular cross-section for curb full flow conditions.
4. Additional capacity required for pipe flow only – no street flow.
5 Hydraulic capacities for Hamilton, Dock and Upper Kerper subareas were not included because no applicable roadway slopes were available.

Table 4.23 Central Business District – North Drainage Subareas Hydraulic Capacity for Storm Sewer Replacement						
HEC-HMS Node No.[1]	Location	Existing Storm Sewer Size (in)	Discharge (cfs)		Total Hydraulic Capacity Required (Circular Pipe Size, inches)[2]	
			2-Year	10-Year	2-Year	10-Year
Windsor Avenue – Main Channel						
BB_17	24th and Elm	54	240	490	54	66
53	Windsor and Burden	12 - 24	210	430	48	66
Notes: 1. See Figure 4-17 for location of HEC-HMS node and identification number. 2. Total capacity required for replacement of existing storm sewer.						

The combination of an inadequate conveyance system and poor drainage from the flat topography produces the undesirable flooding conditions shown in Figure 4-5 for the 100-year flood event. The alternative improvements to the Bee Branch storm sewer trunk line will be discussed in Section 4.7 in more detail.

4.5.5 Development of Alternative Solutions

Although alternatives were developed to address the special problem area along Windsor Avenue, general alternatives focused on the entire subarea were not established. No available sites for regional detention exist because of the topography and land use of the Central Business District – North. Increasing the capacity of storm sewer inlets and pipes will significantly reduce street and property flooding within the Central Business District – North. Further development of alternatives in the Washington Street Subarea is discussed in Section 4.7.

4.5.5.1 Detention

While the 16th Street Detention Cell has a significant impact on the Bee Branch Drainage Basin, few detention sites exist within the Central Business District – North. Table 4.24 summarizes the existing 16th Street Detention Cell storage capacity. Topographic constraints prohibit enlarging the capacity of the 16th Street Detention Cell.

Table 4.24
Central Business District – North Drainage Subareas
Detention Storage Summary

Location	Drainage Area Controlled (sq. miles)	Existing		Comments
		Flood Pool Area (Acres)	Flood Storage (Acre-Feet)	
16th Street Detention Cell	6.4	63	595	Additional storage not viable due to topographic constraints.

4.5.5.2 Channel and Drainage Structure Improvements

The hydraulic capacities (pipe sizes) required for conveyance of the 2- and 10-year flood events are reported in Tables 4.22 and 4.23. Other conveyance improvements for the Bee Branch Drainage Basin are discussed in Section 4.7.

4.5.6 Recommendations for Improvement Alternatives

No recommendations are presented for the Central Business District – North; however, specific recommendations are discussed in Section 4.7 for the Bee Branch Drainage Basin.

4.5.7 Project Phasing

No project phasing is required for the Central Business District -- North.

4.6 CENTRAL BUSINESS DISTRICT SUBAREAS

4.6.1 General Subarea Description

The Central Business District Drainage Subareas (Central Business District) are located in the lower reaches of the Bee Branch Drainage Subarea and includes 8th Street, 11th Street, 14th Street and Lower Kerper Drainage Subareas. The drainage area measures approximately 0.9 square miles and is roughly bounded by 15th Street to the north, 5th Street to the south, Alpine Street to the west, and the Peosta Channel to the east.

Elevations in the subarea range from 902 feet in the upper portion to 594 feet at the 16th Street Detention Cell. The main channel through the subareas follows 8th, 11th, and 14th Streets, where water is conveyed in the storm sewer and street. The overall slope along the main channel is 4 percent.

4.6.2 Flood Hydrology

The HEC-HMS model was utilized to compute the peak runoff rates for the 10-year, 50-year, 100-year and 500-year return period storm events. Runoff hydrographs were developed for each storm event for ultimate development conditions, as defined by the City's comprehensive land use plan.

Figure 4-19 depicts the subbasin delineation, while Figure 4-20 is a schematic of the HEC-HMS model. Table 4.25 provides a summary of peak runoff rates for selected storm events at key locations in the Central Business District. A summary of the peak runoff rates for all subbasin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 4.25
Central Business District Drainage Subareas Peak Runoff Summary
Existing Drainage System Conditions

HEC-HMS Node No.[1]	Location	Drainage Area (sq. mi)	Peak Runoff Rate[2,3] (cfs)			
			10-Year	50-Year	100-Year	500-Year
Lower Kerper						
96	16th St. Detention Cell and Kerper Blvd.	0.42	450	650	760	1,060
8th Street – Main Channel						
103	8th and Washington	0.41	320	480	570	830
116	8th and White	0.34	310	480	570	830
11th Street – Main Channel						
99	11th and U.S. Hwy 61	0.21	240	350	400	560
14th Street – Main Channel						
98	14th and U.S. Hwy 61	0.12	130	200	240	340
15th Street – Main Channel						
BB_27A	15th and Sycamore	0.04	60	80	100	130

Notes:
1 See Figure 4-20 for location of HEC-HMS node and identification number
2 Peak runoff rates based on ultimate land use conditions and simulation of a 24-hour storm event.
3 Peak discharges reported are outflows from the specified node

4.6.3 Stream Hydraulics

The main channel is along 8th, 11th, and 14th Streets, where the storm sewer and the street convey flow. Flow in tributaries is also conveyed through streets and storm sewer systems. A simplified street cross-section and Manning's equation were used to determine the hydraulics in the streets on the main channel and tributaries studied. A rectangular cross-section using the average longitudinal street slope, curb height, and street width along with a Manning's roughness coefficient of 0.013 was assumed to determine the street or curb full capacity. An average longitudinal street slope and Manning's equation for full pipe flow determined the capacity of the existing storm sewer system. The existing hydraulic capacity of the system was equal to the summation of the pipe and street flow. The total conveyance was then compared to the 2-year and 10-year peak discharges. The flow in excess of the storm sewer capacity (not including curb full capacity) was used to size a proposed relief storm sewer system. The additional capacity required for the proposed relief sewer system was determined by subtracting the existing storm sewer pipe capacity from the peak discharges for the 2- and 10-year flood events. Manning's equation for full pipe flow and the existing average longitudinal street slope were used to calculate the pipe size required for the additional capacity. A summary of the hydraulic capacity at several locations along 8th Street, 11th Street, 14th Street, and Lower Kerper Subareas is presented in Table 4.26. The additional capacity required for a proposed relief sewer to convey the 2- and 10-year flood events is presented in the two right-hand columns of Table 4.26. Some of the storm sewer segments evaluated do not provide the hydraulic capacity necessary for a 2- or 10-year flood event.

4.6.4 Problem Areas

The flood hydrologic and hydraulic analyses provide the information needed for identification of areas not in compliance with the City's drainage standards/criteria. The frequency and hazards associated with particular flood events must be taken into account and the flood protection required may vary from street to street. Consequently, the sizing of storm sewers must be performed on a case-by-case basis, while considering the impact of each portion on the entire system.

4.6.5 Development of Alternative Solutions

General alternatives focused on the entire Central Business District were not established. No available sites for regional detention exist because of the topography and land use within the Central Business District. Increasing the capacity of storm sewer inlets and pipes has the potential for reducing street and property flooding within the Central Business District.

4.6.5.1 Detention

Regional detention is not viable within the Central Business District.

4.6.5.2 Channel and Drainage Structure Improvements

The hydraulic capacities (pipe sizes) required for conveyance of the 2-year and 10-year flood events within the Central Business District Drainage Subarea are reported in Table 4.26.

4.6.6 Recommendations for Improvement Alternatives

The sizing of individual storm sewers should be performed on a case-by-case basis. The potential for flood damage posed by the various storm events should be weighed against the cost of improvement.

4.6.7 Project Phasing

No project phasing is required for the Central Business District.

4.7 BEE BRANCH STORM SEWER TRUNK LINE

Figure 4.21 summarizes the problem areas in the Bee Branch Drainage Basin. The majority of the problems are along the Bee Branch storm sewer trunk line and in the West 32nd Street Subarea. Due to the large magnitude of construction, cost, and impact on the community, improvements to the Bee Branch storm sewer trunk line are addressed separately. Because expansion of detention storage in the upper subareas of the Bee Branch Drainage Basin is not sufficient to eliminate flooding problems in the low-lying, heavily developed north end of the city, improvements must be made to the conveyance system in the lower subareas to reduce flood damages.

Table 4.26
Central Business District Drainage Subareas
Existing Hydraulic Capacity Summary

HEC-HMS Node No.[1]	Location	Roadway Classification	Existing Storm Sewer Size (in)	Roadway Slope (%)	Storm Sewer Capacity[2] (cfs)	Typical Curb Height (in)	Typical Street Width (ft)	Street Capacity - Curb Full[3] (cfs)	Total Existing Hydraulic Capacity - Street & Sewer (cfs)	Discharge (cfs)		Additional Hydraulic Capacity Required (Circular Pipe Size, inches)[4]	
										2-Year	10-Year	2-Year	10-Year
14th Street – Main Channel													
98	14th & Hwy 61	Collector		1.0		5	40			60	130		
15th Street – Main Channel													
BB_27A	15th & Sycamore	Collector		0.7		5	37			30	60		

Notes
1. See Figure 4-20 for location of HEC-HMS node and identification number.
2. Assumed Manning Roughness Coefficient of n=0.013 and full pipe flow conditions.
3. Assumed rectangular cross-section for curb full flow conditions.
4. Additional capacity required for pipe flow only – no street flow.
5. Hydraulic capacities for Lower Kerper, 11th Street and part of 8th Street subareas were not included because no applicable roadway slopes were available.

4.7.1 Development of Alternative Solutions

The available improvement alternatives applicable to the Bee Branch Drainage Basin are summarized in Table 4.27. A discussion of each alternative is given below.

Table 4.27
Bee Branch Drainage Basin
Flood Minimization Alternative Improvements

Nonstructural Alternatives	Structural Alternatives
• Public Education/Outreach Prepare an educational program alerting residents of the risk of flooding and methods to minimize flood damage. Provide subsidized flood insurance.	• Expand Existing Detention Cell Capacity Expand existing detention cell storage volume, gate outlet capacity and/or pump capacity.
• Floodplain Buyout Purchase buildings located within the floodplain.	• Create Upstream Detention Purchase un-occupied property and construct detention cells.
• Flood Proofing Remove or minimize flood damage by elevating homes and businesses, moving electrical/mechanical devices to non-flooding elevation, install flood panels at flooding points (e.g. doors and windows).	• Rehabilitate/Expand Capacity of Existing Facilities Repair damaged or increase conveyance system where development has exceeded the system's capacity
• Do Nothing Accept continued occurrence of chronic flooding and storm water damages.	• Open Channel Floodway Create an open channel conveyance system
	• Relief Storm Sewer Construct a parallel trunk line storm sewer system where development has exceeded the capacity of the storm water conveyance system

4.7.1.1 Nonstructural Alternatives

Education/Outreach

Public education programs can be instrumental in reducing flood losses and future flood casualities. Public outreach can include development of public programs to provide emergency shelters and first aid during a flood event, emergency service to assist in evacuation of residences, and educational programs intended to inform citizens of required safety practices before, during and after a flood event.

Floodplain Buyout

A program to acquire and remove flood prone structures within the 100-year floodplain may be feasible in reducing or eliminating flooding problems. This approach may be considered as a major approach for clearing the entire area subject to flooding.

Flood Proofing

Flood proofing of structures subject to flooding may be a cost effective alternative to reduce flood damages. Installation of a variety of flood proofing systems would be required in order to meet the varied needs of the structures located within the flood-prone areas. Flood proofing facilities may range from structural modifications to reduce or eliminate damages from flooding to educational programs that inform people how to protect their property or remain safe during a flood event. Structural measures are usually implemented in commercial or industrial settings where personnel are available to operate and maintain flood proofing devices. In residential applications, flood proofing is usually limited to the relocation of vital residential systems such as heating, cooling, water heaters and laundry areas to safe flooding areas. The relocation of electrical services to areas above the anticipated water surface elevation is also required. Frequently, casualties during flooding relate to structural failures of basement and foundation walls. Public education is an effective means to inform people of these dangers.

Do Nothing Alternative

If the public is not concerned about the current frequency and magnitude of flooding problems in the community, it may be a viable alternative to take no action.

4.7.1.2 Structural Alternatives

Expand Existing Detention Cell Capacity

Increasing the capacity of the 16th Street Detention Cell volume or the ability of the detention cell outlet works to discharge flood flows could have a significant effect on flooding in the Couler Valley area.

Create Upstream Detention

No opportunities exist for upstream detention along the alignment of the Bee Branch trunk line. However, potential upstream detention sites in subareas located in the upland portion of the Bee Branch have a substantial impact on peak discharges in the trunk line. Maximizing the capacity of the West 32nd Street Detention Cell provides the greatest potential for reducing peak discharges in the Bee Branch storm sewer trunk line. Limited opportunities for upstream detention exist in the other subareas.

Rehabilitate/Expand Capacity of Existing Facilities

Repair or replacement of storm water conveyance systems where development has exceeded the system's capacity could decrease or eliminate flooding problems due to ponding in both the upland and lowland areas. Smaller sewer systems feeding into the Bee Branch trunk line could be improved to more effectively convey runoff to the trunk line and reduce localized flood damages.

Open Channel Floodway

Conveyance of runoff through the flat, heavily developed Couler Valley area of Dubuque may require capacity in excess of the Bee Branch trunk line. Construction of a large flood control channel through the Couler Valley area would provide a significant increase in conveyance and storage and could have a large impact on the flooding problem. This would require the purchase of private and commercial property in the Couler Valley area and the relocation of individuals, businesses, roads, and utilities.

Relief Storm Sewer

Construction of a relief storm sewer to expand the capacity of the Bee Branch storm sewer trunk line would have a similar, although less dramatic, effect to that of a flood control channel. The increase in conveyance would deliver water to the Mississippi River more quickly and decrease flooding in the low-lying areas of the City. The benefit/cost ratio would be substantially lower than that of the flood control channel; however, its construction would require purchase of fewer properties and relocation of fewer households and businesses.

4.7.2 Recommendations for Improvement Alternatives

Analysis of the existing condition for the Bee Branch storm sewer trunk line indicates the major flooding problems occur throughout the Bee Branch Drainage Basin for the 100-year storm (See Figure 4-5). The selected alternative for the West 32nd Street Subarea, Alternative W32-5, has the potential to reduce flooding in the Bee Branch Drainage Basin from West 32nd Street to approximately 24th Street to approximately ½ to 1-½ feet of flow in the street. Figure 4-22 illustrates the reduction in flooding depths along the Bee Branch storm sewer trunk line with the recommended Alternative W32-5 improvement. Below Windsor Avenue the impact of increasing detention in the West 32nd Street Subarea is negligible for the 100-year flood. The magnitude of the flooding indicates a significant increase in conveyance would be required to affect a change below 24th Street. An alternative involving a dramatic increase in conveyance would be required to reduce flooding downstream of 24th Street; therefore, available alternatives addressing conveyance along the Bee Branch trunk line were investigated further. Specifically, expansion of the 16th Street Detention Cell outlet works, a relief storm sewer and a flood control channel were analyzed as potential solutions to flooding problems.

4.7.2.1 Existing Conditions

The XP-SWMM model described in Section 4.5 was used to evaluate the Bee Branch storm sewer trunk line. Existing conditions analyses involved modeling the existing storm water system in the lower portion of the Bee Branch Drainage Basin to assess current flooding problems and to provide a baseline condition for comparison with improvement alternatives. All existing condition analyses assumed no improvements in the Bee Branch Drainage Basin. The normal Mississippi River stage (elevation 594.3 feet) was used as the downstream boundary condition for existing conditions analyses and represents the elevation which the Mississippi River water surface equals or exceeds 50% of the time. At this stage, under current operating procedures, the 16th Street Detention Cell gravity outlet gates are open and the three (3) pumps are not activated.

The analyses show the existing facilities have capacity for flows associated with an event less than the 10-year flood (estimated at approximately the 3-year flood). Model results indicate 10-year flooding depths ranging from 0.5 feet near 26th Street and Jackson Street to 3.4 feet near 19th and Elm Streets. Peak flooding depths for the 50-year range from 2 feet near 30th and Jackson Streets to 4.8 feet near 22nd and Elm Streets. Peak flooding depths for the 100-year range from 2.3 feet near 30th and Jackson Streets to 5.8 feet near 22nd and Elm Streets. A 100-year interior rainfall event with a Mississippi River stage of 594.3 feet inundates approximately 1,170 homes and businesses.

An additional model analysis was performed to evaluate the effect of activating the pumps while the gravity outlet gates are open. This analysis used the existing 100-year flows and the normal Mississippi River stage as its boundary conditions. Activation of the pumps did not reduce peak flooding depths or flow rates upstream of the 16th Street Detention Cell for these conditions but did result in a 5-percent increase in peak outflow from the drainage basin. Based on these results, operation of the pumps has little impact on flows and flooding depths while the gates are open.

4.7.2.2 West 32nd Street Improvements

A second set of analyses was performed to evaluate the impact of the West 32nd Subarea improvements on the Couler Valley area. The hydrographs associated with the most effective West 32nd Street improvement, Alternative W32-5, were used as the boundary condition at the

West 32nd Street Detention Cell outlet. The remaining subbasin and subarea inflow hydrographs were identical to those in the existing conditions analyses. The model was executed with the normal Mississippi River stage and the 16th Street Detention Cell gravity outlet in operation, and the pumps turned off. The 10-, 50-, and 100-year conditions were investigated.

Figure 4-22 shows the 100-year flooding depths in the Couler Valley area for Mississippi River stage 594.3 feet with the West 32nd Street Subarea improvements implemented. When compared with the existing flooding depths shown in Figure 4-5, the benefits of the improvements are apparent. The improvements result in approximately 200 fewer properties or 970 homes and businesses inundated for the 100-year flood with Mississippi River stage 594.3 feet. In general, the West 32nd improvements substantially reduced peak flows and flooding depths in the upper portion of the Washington Street Subarea but had little to no effect below the Windsor Subarea outlet at 24th Street, the first major inflow downstream of West 32nd Street.

For 10-year conditions, flow in the storm sewer is significantly reduced in the upper portion of the subarea, but there is little effect on street flooding. This suggests inlet improvements are needed to alleviate flooding in the upper portion for a 10-year design. For 50-year conditions, flooding depths are reduced by as much as 1.7 feet at 32nd and Saunders Streets in the upper portion with less significant effects in the lower portions (for example, 0.3 feet at 22nd and Elm Streets). Flooding depths are reduced by as much as 1.9 feet (at 32nd and Central Streets) in the upper portion also with less significant effects in the lower portions (for example, 0.5 feet at 24th and Washington Streets) for 100-year conditions. The results of the 50-year and 100-year analyses show West 32nd Alternative W32-5 in combination with improvements to the Bee Branch storm sewer system may significantly reduce or eliminate flooding in the downstream reaches of the Bee Branch; therefore, subsequent investigations included Alternative W32-5 as an upstream boundary condition.

4.7.2.3 Relief Sewer

Construction of a relief sewer in the lower reaches of the Bee Branch was then investigated to supplement the capacity of the existing trunk line sewer. Because West 32nd Alternative W32-5 is shown to significantly impact flooding upstream of the Windsor Subarea outlet, trunk line improvements were modeled beginning at 24th Street and extending downstream to the 16th Street Detention Cell. A second conduit identical to the existing Bee Branch trunk line was input into the model effectively doubling the capacity. This geometry was evaluated with 10-, 50- and 100-year flows.

The Alternative W32-5 hydrograph was used as the upstream boundary condition since it significantly reduces downstream flooding. Three (3) operating conditions at the 16th Street Detention Cell were analyzed as downstream boundary conditions, as shown in Table 4.28.

Table 4.28 Bee Branch Drainage Basin Downstream Boundary Conditions for Relief Sewer Alternative						
Scenario	**Operating Condition**	**Mississippi River WSEL[1] (ft)**	**16th St. Detention Cell WSEL[1] (ft)**	**Pump Operation**	**Sluice Gate Position**	**Comments**
1	Normal	594.3	594.3	Off	Open	N/A
2	Current Gate Closure	598.5	598.5	Off	Open	N/A
3	Minimum Water Surface Elevation	598.5	591.5	On	Closed	Pump cell down in anticipation of large storm water discharges

Note

1 WSEL – water surface elevation

Scenarios 2 and 3 in Table 4-28 include diversion of the 8th Street Subarea flows into the 16th Street Detention Cell. Modeling of the three downstream boundary conditions show how the proposed facilities will operate over a wider range of scenarios.

For a 10-year flood, the maximum system flooding depths occur at 24th and Elm Streets, with depths of 1.6 feet, 1.9 feet, and 1.6 feet for the three downstream boundary condition scenarios, respectively. Flooding depths at 24th and Elm Streets were reduced by a maximum of 1.5 feet with the addition of a relief sewer. Flooding depths throughout the areas adjacent to the trunk line were reduced to less than 1 foot. This result indicates a relief sewer may be a viable option to achieve a 10-year level of protection. A potential relief sewer alignment is shown in Figure 4-23. Estimated construction costs for a relief sewer with a 10-year flood capacity is approximately $18.7 million.

For a 50-year flood, the maximum flooding depths also occur at 24th and Elm Streets with depths of 2.5 feet, 2.7 feet, and 2.5 feet for the three downstream boundary condition scenarios, respectively. While the relief sewer was shown to reduce flooding depths by a maximum of 1.4 feet, significant flow still exists in the street, including a depth of 3.2 feet at 22nd and Elm Streets. Because of the street flooding, a relief sewer is not considered an effective option for a 50-year level of protection.

For a 100-year flood, only Scenario 1 was analyzed. It also showed a reduction of flooding depth up to 1.4 feet, but significant flow was left in the street with depths remaining as high as 3.9 feet at 22nd and Elm Streets. Because the street flooding is excessive, a relief sewer is not considered an effective alternative for a 100-year level of protection. In addition, Scenario 1 showed the relief sewer to be ineffective for the 100-year return period; therefore, no other scenarios were investigated.

Further benefit may be gained through improvements to tributary pipes feeding the Bee Branch trunk line, assuming capacity of the storm sewer is expanded to handle the additional flows. Increase in the capacity of inlets and pipes in theses systems could reduce or eliminate flood damages due to localized ponding. Limited information was available on the tributary systems; therefore, they were not included in the analysis of the trunk line.

4.7.2.4 Flood Control Channel

A relief sewer is not an effective option for the 100-year return period; therefore, construction of an open channel capable of conveying 100-year flood flows was investigated. Improvements were modeled from 24th Street to the 16th Street Detention Cell, because the West 32nd Street Alternative W32-5 effectively reduces flooding above 24th Street. The channel replaced the Bee Branch trunk line in this reach, maintained the same invert as the trunk line, and was modeled as an equivalent rectangular channel with a 100-foot bottom width. For the purposes of this study, a preliminary alignment was chosen to assess the magnitude of homes and businesses impacted by the channel. The exact alignment of the proposed channel requires further study. Figure 4-24 illustrates the preliminary alignment of Phase I (Point 1 to 2) and Phases I and II (Point 1 to 2 to 3). The West 32nd Street Alternative W32-5 hydrograph was used as the upstream boundary condition in all of the analyses. Because there would be marginal difference in the cost of constructing a channel for 10-, 50-, or 100-year protection, only the 100-year flows were analyzed.

The first flood control channel analyses were performed to determine the size of channel required to convey the flow assuming an unlimited outlet capacity at the 16th Street Detention Cell. These analyses assumed the capacity of the gravity outlet would be increased to convey flood flows without a rise in stage above that of the Mississippi River, and therefore resulted in the minimum possible channel cross-section. This was accomplished by assuming a constant water surface elevation in the 16th Street Detention Cell. By using a constant water surface elevation, backwater effects from rising stages in the detention cell were eliminated and flow was not limited by the capacity of the gravity outlet. These analyses were performed with two different downstream boundary conditions at the 16th Street Detention Cell: constant water surface elevations of 594.3 feet and 598.5 feet, the starting water surface elevations for Scenarios 1 and 2, respectively. Operating at a constant water surface elevation of 591.5 feet was not considered, as this would be a gate closure condition and require a very large pump.

A sensitivity analysis examining the effect of improvements to storm sewer inlets and tributary pipes was evaluated. The sizes of the tributary pipes in the model were increased so as to not limit passage of flow from the street to the storm sewer. In this manner, the effect of inlets and tributary pipes on the trunk line could be evaluated and a channel sized to carry the total street and storm sewer flows.

The results of the first series of analyses indicates that for a constant water surface elevation of 594.3 feet in the 16th Street Detention Cell, a 10-foot-deep grass lined trapezoidal channel with a 60-foot bottom width and 3H:1V side slopes would be required to convey flows associated with the 100-year flood. For a constant water surface elevation of 598.5 feet in the 16th Street Detention Cell, a 10-foot-deep grass-lined trapezoidal channel with a 66-foot bottom width and 3H:1V side slopes would be required.

While the improvement of the storm sewer inlets and tributary pipes decreased flooding depths, the improvement had no impact on the flood control channel size. The analyses also showed the 16th Street Detention Cell was not the factor limiting the conveyance of the Bee Branch storm sewer trunk line. Reduction in flooding of the Couler Valley area therefore requires modification to the Bee Branch storm sewer trunk line itself.

A second series of runs was performed to determine the effect of backwater from the 16th Street Detention Cell. Improvements to tributary pipes were assumed to size the channel for the maximum predicted peak discharges. These runs included the three (3) downstream boundary conditions listed in Table 4.29.

Table 4.29
Bee Branch Drainage Basin
Downstream Boundary Conditions for Flood Control Channel Alternative

Scenario	Operating Condition	Mississippi River WSEL[1] (ft)	16th St. Detention Cell WSEL[1] (ft)	Pump Operation	Sluice Gate Position	Comments
1	Normal	594.3	594.3	On	Open	N/A
2	Current Gate Closure	598.5	598.5	On	Open	N/A
3	Minimum Water Surface Elevation	598.5	591.5	On	Closed	N/A

Note
1 WSEL – water surface elevation

The results of the second series of analyses show for Scenario 1, a slightly larger trapezoidal channel with a 10-foot depth, 76-foot bottom width, and 3H:1V side slopes is required to convey the 100-year flood flows. Scenarios 2 and 3 require a slightly deeper channel to contain the flow: a 13.5-foot-deep trapezoidal channel with a 55-foot bottom width and 3H:1V side slopes. In all three scenarios, flooding is significantly reduced with maximum flooding depths of 0.2 feet, 0.6 feet, and 0.1 feet for each of the three downstream boundary conditions, respectively.

A channel beginning downstream at the 16th Street Detention Cell and terminating at Garfield Street was assumed as an initial phase, Phase I. Consequently, the existing storm sewer trunk line was modeled from the West 32nd Street Detention Cell to Garfield Street with the flood control channel constructed downstream. The results of this analysis for the Phase I flood control channel are shown in Figure 4-25. The model results indicated the construction of the initial phase of the flood control channel would only have significant impact on flooding for the 100-year storms downstream of Garfield Street. Water surface elevations were decreased by one foot or more as far upstream as 25th Street; however, flooding depths remain two feet and higher in these locations. This analysis demonstrates that partial construction of the project will not provide adequate flood protection for the upper portion of the Bee Branch trunk line. Estimated construction costs for the Phase I Flood Control Channel from the 16th Street Detention Cell to Garfield Avenue are approximately $6.9 million.

Further analysis was conducted to determine the effect of extending the flood control channel to provide flood protection for the upper portion of the Bee Branch storm sewer trunk line. The original flood control channel was extended up to 24th Street for this analysis, Phase II. The combined effect of the West 32nd Subarea improvements and construction of Phase I and II of the Flood Control Channel for a Mississippi River stage of 594.3 feet is illustrated in Figure 4-26. For the 100-year flood with Mississippi River stage 594.3 feet and tributary improvements, the flood control channel in conjunction with the West 32nd Street improvements resulted in fewer than 10 properties inundated. Construction costs for Phase I and II of the Flood Control Channel from the 16th Street Detention Cell to 24th Street are estimated at $17.1 million. The demolition of the estimated 71 homes/businesses is included in the cost estimate.

Further benefit may be gained through tributary pipe improvements feeding the Bee Branch trunk line. Increase in the capacity of inlets and pipes in theses systems could reduce or eliminate flood damages due to localized ponding. Limited information is available on these systems; therefore, they were not analyzed in detail.

4.7.2.5 Additional Comments

An important consideration in the design of the flood control system is the effect of the downstream boundary condition. It became apparent, through the course of these analyses, that the worst-case downstream boundary condition is not the same for every return period. Comparing the results of the three downstream boundary conditions modeled, it was found that the critical condition for the 100-year flood is when the gates on the gravity outlet are closed. Less volume of runoff is produced by the 10- and 50-year flood events, so the storage volume in the 16th Street Detention Cell is not consumed as quickly when the gates are closed. Therefore, the critical condition for the 10- and 50-year floods becomes the Mississippi River water surface elevation of 598.5 feet with the gates open.

4.7.2.6 Summary

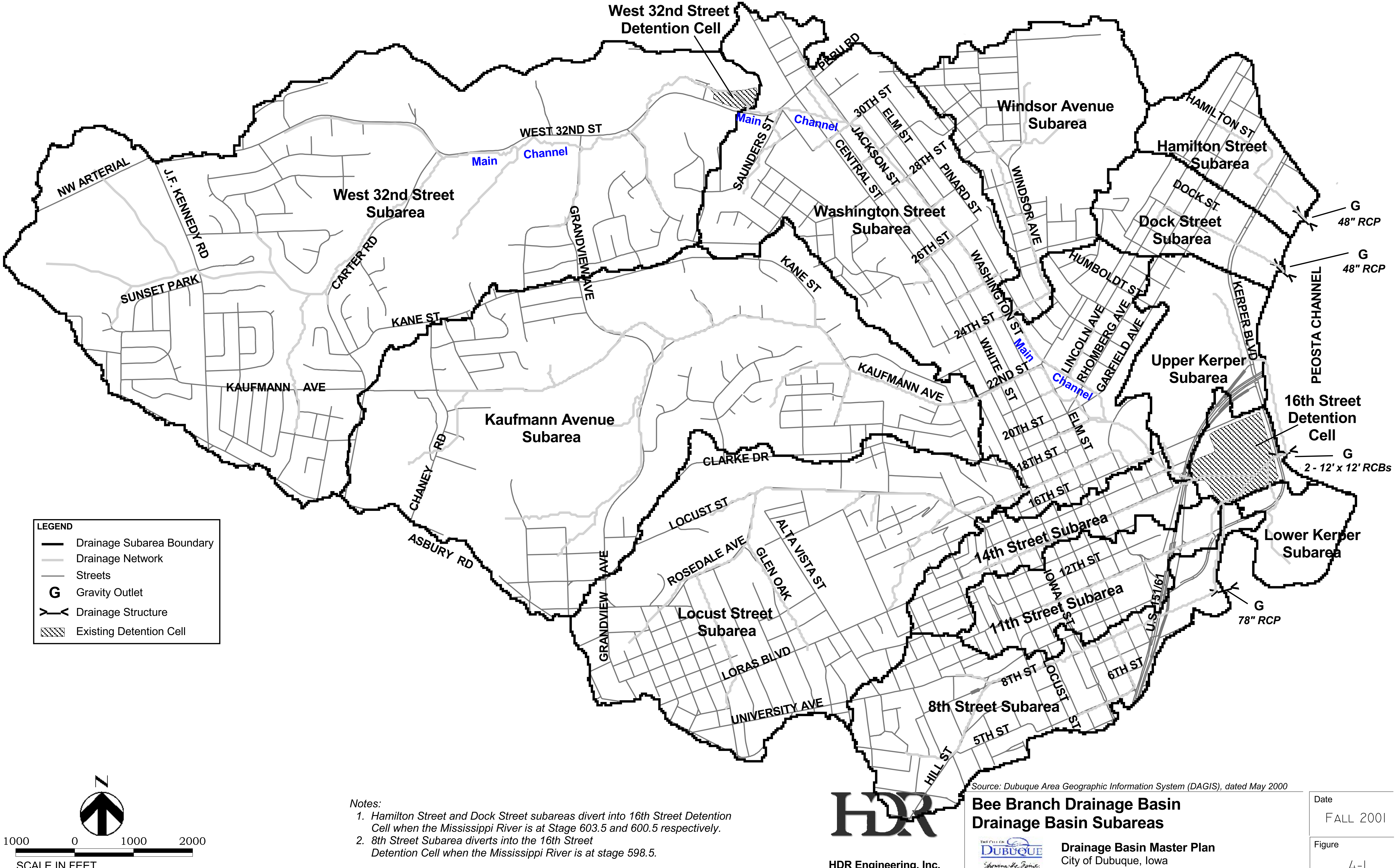
Analysis of the Bee Branch storm sewer trunk line indicates that implementation of West 32nd Subarea Alternative W32-5 would have a significant impact on 100-year flood depths along the Bee Branch from 32nd to 24th Streets, with a lesser impact further downstream. West 32nd Subarea improvements result in approximately 200 properties removed from the floodplain at a Mississippi River stage of 594.3 feet.

To further reduce flooding along the Bee Branch storm sewer trunk line, construction of a relief storm sewer from 24th Street to the 16th Street Detention Cell was analyzed. It was determined the relief sewer option was not viable for flood discharges in excess of the 10-year storm. It would take an additional four (4) relief sewers equivalent in size to the existing Bee Branch trunk line to eliminate the flooding depths produced by the 100-year event.

Construction of a flood control channel from the 16th Street Detention Cell to 24th Street was then investigated. Improvements to tributary pipes were assumed to maximize the anticipated 100-year peak discharges used for sizing the channel. A grass-lined trapezoidal channel with approximately a 10-foot depth, 76-foot bottom width, and 3H:1V side slopes was analyzed. The flood control channel in conjunction with the West 32nd Street improvements was shown to remove all but 4 of the 1,155 properties in the Washington Subarea from the 100-year floodplain at a Mississippi River stage of 594.3 feet. Construction costs for Phases I and II of the flood control channel from the 16th Street Detention Cell to 24th Street are estimated at $17.1 million.

4.7.3 Project Phasing

Improvements made to the Bee Branch storm sewer trunk line should progress from downstream to upstream. If initial improvements were to be made upstream, resulting increases in peak discharges would be realized in the unimproved downstream reaches of the trunk line, increasing flood damages.



Notes:
1. *Hamilton Street and Dock Street subareas divert into 16th Street Detention Cell when the Mississippi River is at Stage 603.5 and 600.5 respectively.*
2. *8th Street Subarea diverts into the 16th Street Detention Cell when the Mississippi River is at stage 598.5.*

Source: Dubuque Area Geographic Information System (DAGIS), dated May 2000

HDR

HDR Engineering, Inc.

Bee Branch Drainage Basin
Drainage Basin Subareas

The City of Dubuque — Masterpiece on the Mississippi

Drainage Basin Master Plan
City of Dubuque, Iowa

Date: FALL 2001

Figure: 4-1

Layout: Drainage Basin Subareas
ENVIRO/CITY_OF_DUBUQUE/007-134_stormwater/GIS_MODELING/DOWNTOWN/REPORT_fall-2001.APR

5.0 FINANCING DRAINAGE IMPROVEMENTS AND OPERATIONS

Historically, many cities and towns in Iowa have considered municipal drainage a function of public works and have funded drainage improvements similar to the methods used to finance street and road improvements. Traditional tax revenues accruing to the General Fund have historically been relied upon to fund the annual operation and maintenance expense of urban drainage. General Obligation bonds have been the debt tools for funding major public projects of which drainage is a component.

5.1 GENERAL FUND FINANCING

Drainage activities and improvements are supported by the municipality's General Fund or from wastewater or sewage utility fees. Drainage projects are one of many "line items" in the General Fund that are supported with the combined pool of general revenues from ad valorem taxes, sales taxes and other revenue. Capital financing is typically accomplished through cash transfers for small projects and general obligation bonds for major improvements. Operational activities are usually funded with general revenues.

Advantages of the simple general revenue funding approach include:

- A broad base of financial support (all taxpayers pay), and
- Customers can deduct local taxes from Federal income taxes.

Disadvantages include:

- Competition for funding with other general services,
- A perceived lack of identity as a significant municipal utility function that must be addressed with on-going efforts, and
- Inequities arising from tax liabilities not equated with contribution to drainage problems.

With increasing attention given to the water quality aspects of urban drainage, especially with respect to the EPA's Total Maximum Daily Loads (TMDLs) program, more municipalities are moving this function into the water and/or wastewater enterprise fund, with some communities establishing specific storm water enterprise funds. An enterprise fund is a self-supporting component of municipal government that depends upon rates and fees, and frequently development impact fees, to fund its activities. Water and wastewater utilities are examples of municipal enterprises that are intended to be self-supporting. Enterprise funds typically finance major capital improvements with revenue bonds that only require the approval of the local governing body, such as the City Council, rather than a public vote, to approve the issuing of bonds. Being within an enterprise fund facilitates a sustainable storm water program because it allows the utility to set rates and charges on the basis of its actual direct and indirect costs of providing this service.

As mentioned above, increased attention to urban drainage has created the demand for more contemporary methods of funding drainage improvements. These newer methods of drainage financing seek to: 1) acknowledge the drainage problem as a formal utility function, and 2) seek to place a greater financial burden for remediation or prevention of drainage and flooding problems upon those activities contributing to the problem.

Numerous methods are available to finance drainage improvements and operations. As monies for drainage projects become competitive with other city projects, the need to evaluate financing alternatives is necessary. The remainder of this section reviews the methods that enterprise fund-based storm water utilities can use to: (1) finance ongoing operation and maintenance activities; (2) provide up-front financing for current and future capital projects; and (3) repay any indebtedness that results from financing the capital projects.

5.2 FUNDING OPERATION AND MAINTENANCE ACTIVITIES

For storm water utilities that are either a stand-alone enterprise or are a component of the water/wastewater enterprises, user charges are counted upon to fund ongoing activities. These user charges are billed in a manner and frequency similar to that of water/wastewater charges, such as monthly, bi-monthly, quarterly, or annual billing. Typically, the estimated annual operation and maintenance (O&M) expenditures include labor costs, materials, machinery, and some portion of General and Administrative (G&A) expenditures.

The basis for the drainage charge is frequently the volume of impervious area, such as rooftops, sidewalks, driveways, streets, and other structures, in relation to total area. Impervious area is generally indexed on a single-family residential equivalent basis (SFR). Impervious areas for non-residential customers are often measured as a multiple of SFRs.

5.3 CAPITAL FUNDING

There is a wide range of sources of funds, including funds from public and private sources.

5.3.1 Pay-As-You-Go

Pay-As-You-Go financing is what its name implies. Improvements are made as sufficient reserves are collected. This method is low risk, but considering that projects need to be constructed and on-line in order to generate revenue, the funds are often not available when needed. However, for long-term capital improvement programs, it is often possible to phase the improvements in a manner in which pay-as-you-go financing can comprise the majority of the project's financing.

5.3.2 General Obligation Bonds

General obligation bonds are long-term municipal bonds that are backed by the full faith and credit of the City. This means that the local government pledges to use all of its taxing and other revenue-raising powers to repay bondholders. General obligation bonds require a two-thirds approval by voters. Since general obligation bonds have low risk due to excellent collateral, interest rates are usually one half to one percent lower than other municipal bonds.

5.3.3 Revenue Bonds

Revenue bonds are backed by the revenue from the enterprise backing the project, including user charges and, potentially, development impact fees. They also do not require a public referendum, but only the approval of the city council. If defaulted, bondholders have rights to the project revenues but not the project property. Revenue bonds are most typically used by water supply and wastewater utilities.

Advantages of revenue bonds include:

- Credit analysis is relatively straight-forward compared to other types of bonds
- The primary beneficiaries pay for the facility
- Default on the issue does not burden local taxpayers
- Debt is not normally subject to a debt ceiling
- Improved financial management is promulgated and
- A voter referendum may not be required.

Disadvantages of revenue bonds include:

- Interest rate charges to the issuer are generally higher than rates charged for general obligation bonds
- Revenue bond ordinance usually contain restrictive covenants which may constrain operations
- The market for revenue bond debt is not as broad as for general obligation bonds.

5.3.4 Grants

Several grant programs are available for funding storm water-related activities. One of the most common is the EPA's Non-point Source Implementation Grants, also known as Section 319 Grants[1]. These grants are intended to promote the use of Best Management Practices in minimizing and/or mitigating nonpoint source water pollution, from a watershed perspective. Section 319 Grants require a 40 percent cost share for studies and projects. The EPA has a formula-based system for allocating their $200+ million dollar annual contribution to the States' lead agencies (in Iowa, the Department of Natural Resources).

Other EPA grant programs include Water Quality Cooperative Agreements and Watershed Assistance Grants. These programs can also be used for storm water-related facilities, but their funding levels are minimal compared to the Section 319 program.

Other programs include:

- Department of Interior: Land and Water Conservation Fund Grants to States. As indicated this program awards moneys to states for disbursement to individual communities and projects. Though all states are eligible, the funds originate from offshore oil leasing revenues and projects tend to focus on coastal areas.
- Department of Agriculture, Natural Resource Conservation Service: Watershed Protection and Flood Prevention Program. Technical assistance and cost sharing opportunities are available through this program, also known as the PL 565 Program and the "Small Watershed Program". The level of cost sharing varies by project. This program provides assistance for Best Management Practices in relatively small watersheds (less than 250,000 acres).
- Department of Housing and Urban Development (HUD): Community Development Block Grants. Though these grants are typically targeted for urban re-development, they can also be used for infrastructure improvement, to the extent that these improvements benefit the existing urban area. Most urban areas of 50,000 or more typically receive some CDBG assistance. Annual grants range from $500,000 to $750,000, with some cost-sharing involved. In some cases, municipal policies dictate how these funds can be used.

[1] Clean Water Act, Section 319(h).

5.3.5 Developer Contributions

The need for a storm water facility addition in a community is often linked to new development. Developers can also be obligated when existing downstream facilities will not handle flow increases from upstream construction. Cities and counties must also frequently call on their residents and current revenue sources to install oversized system that are not needed now but will be if anticipated growth occurs. Existing property owners do not always feel they can or should bear the cost of improvements, which are needed primarily to facilitate growth; therefore, developer contributions enable communities to meet these kinds of demands on the system.

Charges are levied on new developments after the improvement is constructed, as a means of balancing financial participation. The intent is to enable a community to achieve excess capacity improvements in advance of growth, yet place an equitable portion of the cost on those properties, which later develop and make use of the extra capacity built into the systems.

5.4 CAPITAL RECOVERY

5.4.1 Monthly User Charges

For purposes of obtaining debt financing and meeting debt service coverage requirements, monthly user charges must be set at a level that will generate sufficient annual revenue to cover all O&M and debt service costs. More typically, total enterprise revenues must be anywhere from 1.10 to 1.30 times higher than the sum of O&M and debt service costs.

5.4.2 Impact Fees

Development impact fees are a method of recovering capital costs that have been used to construct new facilities for new customers. That is, drainage facilities constructed to accommodate new growth should be paid for exclusively by the new residents benefiting from these new facilities. Impact fees are used in most states for this purpose and have been upheld by courts.

When rapid growth in the late 1970's and early 1980's hit many Iowa municipalities, a noticeable number of municipalities implemented capital recovery (impact) fee programs for new water and wastewater connections. Some implemented such fees for drainage as well. These fees were targeted at making new growth "pay for itself." The intent of these up-front fees were to gather cash for the purpose of partial or full financing of public capital improvements attributable to new growth.

Advantages of the capital recovery fee approach include:

- Partial or full funding of growth-induced drainage problems is borne by new development,
- Specific funding becomes available for the sole use of drainage capital projects, and
- Incorporation into the mortgage financing, the interest is Federally tax deductible.

Disadvantages include:

- Raises the cost of new homes and lessens financing eligibility for home buyers,
- May re-locate some new development to nearby communities with lower or no fees,
- Takes time to accumulate enough fee revenue to make substantial contribution to new project financing when needs may be immediate,
- Can create double-charge inequities arising from "growth" having paid once up-front for drainage improvements and again over longer-term through taxes,
- Still leaves "existing" drainage and flooding problems subject to the difficulties of General Fund financing mentioned above.

5.5 MUNICIPAL DRAINAGE UTILITIES

Iowa Legislature enacted a law (Iowa Code Sections 384.80-384.94) specifically authorizing the creation of municipal drainage utilities. This Act allowed drainage utilities to be formed as an enterprise fund function of municipal government on a par with the financial and operational capabilities of municipal water/wastewater and electric utility funds. Typically, separate revenue and (capital and operating) expense accounting is maintained with fund income arising from drainage fee (rate) revenue and collection or transfers from other funds. Most common is a periodic drainage fee (i.e. rate charge) that is usually made monthly and included on the water/wastewater billing. This monthly drainage fee usually reflects a flat charge for single family residential or a unit charge per amount of impervious cover for multi-family, commercial, industrial, municipal, religious, and institutional land uses. The drainage fee levies should be equitable, related to the extent of problem drainage caused by the land use, and produce a targeted level of overall revenue recovery for the drainage utility. Equity includes reduction or elimination of fees for low income and elderly customers. The income of a drainage utility can also include the drainage capital recovery levy previously described.

Advantages of the municipal drainage utility approach include:

- Provides continuing stream of income for on-going drainage improvements and operational activities,
- Allows for the issuance of utility revenue bonds to fund capital improvements,
- With proper fee design, a reasonable charge can be levied that is equitable between new development and longer-term residents and also equitable among differing land uses, and
- Raises the chronic drainage issue to a higher profile level and better targets needed actions.

Disadvantages include:

- In gathering revenues as a monthly rate charge, this source of financing is not deductible by rate-payers on Federal tax returns, and
- The City may incur slightly more administrative overhead due to the separate enterprise fund accounting and potentially expanded drainage programs.

Municipal drainage utilities have been implemented by a number of cities in Iowa to fund projects to mitigate existing drainage problems. A list of cities in Iowa is provided in Table 5.1.

Table 5.1
Municipal Drainage Utilities in Iowa

Municipality	
• Des Moines	• Ames
• Sioux City	• Burlington
• Cedar Rapids	• Boone
• Garner	

Base residential fees charged by municipal drainage utilities in Iowa were found to range from as low as $1.50 per month for the City of Ames to as high as $4.60 per month for Des Moines. An estimate of revenues that could potentially be generated by the City of Dubuque with a comparable fee structure as some of the cities surveyed is presented in Table 5.2. As shown in Table 5.2, annual revenues for the City of Dubuque with a comparable fee structure as the six cities shown would range from about $415,600 to $2,124,300 per year. This type of revenue would provide a means for the City to implement a number of projects for identified problem areas over a period of five to ten years without the use of the general revenue fund or issuance of capital improvement bonds.

Table 5.2
Estimated Annual Revenue for the City of Dubuque With Implementation of Municipal Drainage Utility with Comparable Fee Structures

City	Population	Base Residential Fee	Municipal Drainage Utility Annual Revenue	City of Dubuque Estimated Annual Revenue with Comparable Fee Structure[2]
Des Moines	193,190	$4.60	$7,200,000.00	$2,124,300
Sioux City	82,970	$1.84	$1,100,000.00	$755,700
Cedar Rapids	114,560	$2.25	$970,000.00	$482,600
Ames	48,415	$ 1.50	$353,000.00	$415,600
Burlington	26,855	$3.00	$234,000.00	$496,700
Boone	12,755	$1.95	$183,880.00	$821,700
Garner	2,915	$2.63	$55,000.00	$1,075,500

Notes:

1. Estimated number of customers for City of Dubuque is 57,000.
2. Estimated annual revenue determined by using the ratio of each city's population with Dubuque's population, then multiplying each city's municipal drainage utility annual revenue by the respective population ratio.

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