IV. HYDROLOGIC ANALYSES AND MODELING

4.1 Formulation of Hydrologic Model

The primary objective of the hydrologic analyses and modeling efforts was the development of peak discharge and hydrograph data at various locations throughout the contributing drainage basins within the watershed. This information, combined with the capacity of the existing drainage facilities, provided insight to existing and future flooding problems and promoted the development of storm water management plans for the basins.

The hydrologic analyses for all ten of the major drainage basins were conducted for the 2-, 5-, 10-, 25-, 50-, and 100-year return periods. The model simulated four scenarios: (a) existing conditions with existing facilities; (b) future development conditions, assuming on-site detention (using Town of Windsor drainage criteria) with existing facilities; (c) future development conditions, assuming on-site over-detention (detaining 100-year flows to a 10-year existing condition event) with existing facilities; and (d) future development conditions, assuming on-site over-detention and selected drainage improvements. The first three scenarios are discussed and documented in this chapter. The results of the hydrologic analysis associated with the last scenario (involving selected drainage improvements) are presented in Chapter 5.

4.1.1 Model Description

The modeling approach chosen to simulate the runoff generated within and routed through the ten drainage basins involved the application of a single computer model, the U.S. Environmental Protection Agency Storm Water Management Model (EPA SWMM). Specifically, PCSWMM 2000 was obtained from Computational Hydraulics Int. (CHI) of Ontario, Canada. The PCSWMM 2000 software is an interface for the EPA SWMM model, allowing the user to use up to four different versions of EPA SWMM (Version 4.31 was chosen as the basis for this study). The EPA SWMM model is a hydrologic simulation program developed in 1970 for the U.S. Environmental Protection Agency, and is used to generate and route storm hydrographs for a number of basin sub-catchments. The program has two components: the runoff block and the transport block. The runoff block requires input of a design storm as either a 2- or 3-hour hyetograph (5 minute increments, storm duration depending on the size of the drainage basin) distribution from which a total precipitation depth is computed. The methodology used in developing the design storm is outlined in the National Oceanic and Atmospheric Administration's (NOAA) "Precipitation-Frequency Atlas of the Western United States, Volume III – Colorado" (1973). Storm hydrographs for each subbasin were generated

by the runoff block for the 2-, 5-, 10-, 25-, 50-, and 100-year return periods; these hydrographs were written to a file to be used by the transport block.

The transport block simulates the quantity of runoff from a drainage basin and the routing of flow through the basin. The hydrologic analyses and modeling for the ten drainage basins utilized the transport block to develop flood hydrographs at various locations throughout each basin. In addition, storm flow hydrographs generated by the runoff block are routed through the transport block to depict the actual network of detention ponds and open channels that exist within each basin.

4.1.2 Network Development

The network incorporated into EPA SWMM's runoff and transport blocks is a model of the basin drainage network, representing each of the drainage subbasins and facilities along the major drainageway. The first step in forming the network is to conceptualize and develop a schematic linking the drainage subbasins to the drainage facilities within the major drainageway. Identification of each drainage facility is based on the information compiled from the field reconnaissance and surveying efforts. EPA SWMM incorporates hydrologic features into the modeling network in accordance with the following:

- sewer element data or conveyance elements (conduits, open channels, junctions, nodes, and manholes),
- sub-catchments (or subbasins),
- storage elements (detention ponds), and
- flow dividers (diversions).

The subbasin delineation was accomplished through the use of mapping and survey data generated during the project. Drainage network schematics were developed for the ten drainage basins for the four scenarios previously discussed. The network schematic diagrams are provided in the project technical notebook that accompanies this report.

An alphanumeric scheme was integrated into the modeling network to allow easy identification of each type of facility. The alphabetic and numeric convention is presented below.

B-W	Drainage basin name (eg. Bluff Basin, Windsor Basin, etc.)
1 - 99	Subbasins
100–199, 800-899	Conveyance elements (i.e., storm sewers and open channels)
200-299, 500-599	Nodes (local and regional design points; flow diversions)
300 - 399	Existing on-site and regional detention facilities
400 - 499	Future on-site detention facilities (approved and future development)
700 - 799	Nodes (regional or combination design points)

4.2 Rainfall Design Storms

The rainfall design storms used in the hydrologic analysis of the ten drainage basins were based on information presented in the "Precipitation-Frequency Atlas of the Western United States, Volume III – Colorado" (1973). One-hour rainfall values were obtained for the drainage area from the NOAA Atlas, and were used to develop two and three-hour unadjusted and adjusted design storms, depending on the size of the particular drainage basin. The two and three-hour storms developed for each return period are presented in the project technical notebook which accompanies this document. Further documentation and details regarding the development of the design storms can also be found in the Town of Windsor Storm Drainage Design Criteria Manual (Engineering Professionals, Inc., 1990).

4.3 Hydrologic Modeling Parameters

The hydrologic modeling of the ten drainage basins involved the determination of several hydrologic parameters associated with each subbasin. These parameters, required by the runoff block of EPA SWMM, are summarized in the following paragraphs. Table 4.1 presents hydrologic modeling parameters developed for the existing condition analyses.

4.3.1 Subbasin Delineation and Basin Characteristics

The ten drainage basins were subdivided into smaller subbasins ranging in size from approximately 9 acres to over 13,700 acres. The delineation of subbasins was based on several considerations including the location of drainage facilities, road crossings, and potential flooding problems. The location of major drainage facilities and the desire to obtain peak discharge data at several design points resulted in the wide range of drainage areas noted above.

The subbasin delineation for the ten drainage basins is presented on Sheets 2A and 2B. USGS topographic mapping (scale: 1" = 2000', 10-foot contour interval) was utilized to determine the subbasin characteristics and significant hydrologic parameters. These data included subbasin area, width (ratio of subbasin area to average length of overland flow), and basin slope.

Table 4.1 Hydrologic Parameters for Existing Conditions.

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
LAW BASIN											
L01	31,330	287.7	22.3	0.0029	0.011	0.17	0.1	0.4	2.6	0.4	0.0014
L02	44,930	515.7	7.6	0.0044	0.011	0.17	0.1	0.4	3.4	0.5	0.0018
L03	82,720	569.7	47.2	0.0056	0.011	0.21	0.1	0.4	3.5	0.5	0.0018
L04	52,790	605.9	7.1	0.0052	0.011	0.17	0.1	0.4	4.3	0.6	0.0017
L06	19,000	196.3	10.6	0.0024	0.011	0.17	0.1	0.4	3.4	0.5	0.0018
L07	39,350	361.3	21.1	0.0105	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L08	38,960	447.2	5.0	0.0056	0.011	0.17	0.1	0.4	3.5	0.5	0.0018
L10	65,040	746.5	5.4	0.0163	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
L11	30,690	352.3	5.1	0.0133	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
L12	44,990	516.4	7.0	0.0385	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L13	28,890	298.4	8.4	0.0140	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L15	55,380	635.7	6.5	0.0225	0.011	0.17	0.1	0.4	4.3	0.6	0.0018
L16	34,790	279.5	18.1	0.0143	0.011	0.21	0.1	0.35	4.5	0.6	0.0018
L17	13,510	139.6	9.5	0.0214	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
L18	35,920	247.4	24.3	0.0200	0.011	0.24	0.1	0.35	4.4	0.6	0.0017
L19	58,070	666.6	7.4	0.0160	0.011	0.17	0.1	0.4	4.0	0.6	0.0018
L20	69,100	793.2	8.4	0.0107	0.011	0.17	0.1	0.4	4.1	0.6	0.0018
L25	22,610	233.6	5.0	0.0100	0.011	0.17	0.1	0.4	3.8	0.6	0.0018
L26	47,200	541.8	6.7	0.0176	0.011	0.17	0.1	0.4	4.3	0.6	0.0018
L27	42,730	490.5	5.0	0.0242	0.011	0.17	0.1	0.4	4.0	0.6	0.0018
L28	31,060	320.9	6.2	0.0163	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L29	59,750	685.8	5.0	0.0154	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L30	57,060	655.0	5.1	0.0167	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L40	206,570	2,371.1	8.1	0.0281	0.011	0.17	0.1	0.4	4.2	0.6	0.0018
L41	237,620	2,727.5	8.1	0.0477	0.011	0.17	0.1	0.4	4.2	0.6	0.0017
L42	260,110	2,985.6	5.3	0.0544	0.011	0.17	0.1	0.4	4.1	0.6	0.0018
LNC1	387,100	3,554.6	27.7	0.0240	0.011	0.17	0.1	0.4	3.0	0.4	0.0014
LNC2	22,380	179.8	35.7	0.0429	0.011	0.17	0.1	0.4	3.0	0.4	0.0012
LNC3	71,490	656.5	20.6	0.0176	0.011	0.17	0.1	0.4	3.6	0.5	0.0016
LNC4	85,900	887.4	10.5	0.0127	0.011	0.17	0.1	0.4	4.3	0.6	0.0017
LNC5	165,270	1,707.3	9.1	0.0200	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
LNC6	17,080	196.1	5.0	0.0300	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
LNC7	1,197,670	13,747.4	8.1	0.0366	0.011	0.17	0.1	0.4	4.3	0.6	0.0017
WINDSOR BASIN											
W01	50,880	175.2	52.9	0.0060	0.011	0.24	0.1	0.35	2.3	0.4	0.0011
W02	85,770	196.9	51.9	0.0042	0.011	0.24	0.1	0.35	2.4	0.4	0.0011
W03	4,500	51.7	6.6	0.0118	0.011	0.15	0.1	0.35	4.5	0.6	0.0018
W04	11,650	40.1	41.2	0.0095	0.011	0.24	0.1	0.35	3.9	0.6	0.0018
W10	8,230	18.9	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W11	15,970	55.0	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018`
W12	15,420	35.4	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W13	10,680	37.4	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W14	6,710	15.4	70.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W15	5,230	12.0	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W19	7,140	16.4	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W20	8,230	18.9	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018

Table 4.1 Hydrologic Parameters for Existing Conditions (continued).

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
WINDSOR BASIN (continued)											
W21	17,480	56.3	45.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W22	10,500	24.1	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W23	25,530	62.4	45.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W24	16,030	36.8	45.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W28	9,150	42.0	60.0	0.0125	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W29	11,720	26.9	45.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W30	44,340	101.8	45.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W31	21,130	48.5	40.0	0.0020	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W32	12,330	28.3	70.0	0.0020	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W33	8,100	18.6	50.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W34	7,410	8.5	90.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W40	19,730	45.3	19.4	0.0006	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W41	140,350	322.2	76.5	0.0006	0.011	0.24	0.1	0.35	1.5	0.2	0.0006
W42	34,060	390.9	5.2	0.0060	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
W43	59,990	619.7	10.0	0.0156	0.011	0.17	0.1	0.4	4.1	0.6	0.0017
W44	26,520	304.4	6.1	0.0188	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
W45	58,900	608.5	10.1	0.0286	0.011	0.17	0.1	0.4	4.3	0.6	0.0018
W46	19,370	222.3	5.4	0.0162	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
W47	7,780	80.4	5.0	0.0429	0.011	0.17	0.1	0.4	4.6	0.7	0.0014
W48	47,090	486.5	7.7	0.0218	0.011	0.17	0.1	0.4	4.5	0.6	0.0017
HIGH SCHOOL BASIN											
H01	11,650	93.6	25.4	0.0100	0.011	0.16	0.1	0.4	2.6	0.4	0.0015
H02	1,530	12.3	19.0	0.0130	0.011	0.15	0.1	0.4	3.7	0.5	0.0015
H05	31,840	73.1	35.2	0.0080	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
H06	109,680	125.9	45.9	0.0070	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H09	19,830	68.3	42.9	0.0020	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H10	15,160	34.8	35.6	0.0030	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H12	9,280	21.3	33.7	0.0130	0.011	0.24	0.1	0.35	4.1	0.6	0.0018
H13	44,260	50.8	58.0	0.0070	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H14	28,880	66.3	51.9	0.0090	0.011	0.24	0.1	0.35	4.2	0.6	0.0018
H15	9,930	11.4	90.0	0.0090	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
JACOBY BASIN											
J01	18,050	145.0	24.9	0.0030	0.011	0.17	0.1	0.4	2.5	0.4	0.0015
J05	41,210	94.6	36.9	0.0100	0.011	0.24	0.1	0.35	3.2	0.5	0.0018
J06	24,660	84.9	45.4	0.0070	0.011	0.24	0.1	0.35	4.2	0.6	0.0018
J07	26,140	60.0	56.0	0.0070	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
J08	25,570	58.7	39.2	0.0040	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
J09	6,670	76.6	5.0	0.0040	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
J10	34,590	39.7	83.9	0.0080	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
J11	19,520	44.8	39.5	0.0040	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
J12	35,880	411.8	5.0	0.0020	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
TIMNATH RESERVOIR BASIN											
T01	27,100	310.6	34.9	0.0050	0.011	0.15	0.1	0.4	2.5	0.4	0.0012
T02	18,500	211.9	33.4	0.0080	0.011	0.20	0.1	0.4	3.5	0.5	0.0016
T04	15,700	180.1	5.0	0.0030	0.011	0.20	0.1	0.4	3.6	0.5	0.0018
T05	21,100	242.3	14.2	0.0050	0.011	0.17	0.1	0.4	3.5	0.5	0.0016

TIMNATH RESERVOIR BASIN (continued)

Table 4.1 Hydrologic Parameters for Existing Conditions (continued).

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
T06	92,000	1,056.0	5.1	0.0080	0.011	0.17	0.1	0.4	4.3	0.6	0.0018
T07	46,700	536.4	6.8	0.0070	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
T08	100,300	1,151.6	5.0	0.0420	0.011	0.24	0.1	0.4	4.0	0.6	0.0017
T20	212,900	1,238.8	51.9	0.0130	0.011	0.24	0.1	0.4	2.0	0.3	0.0009
T21	216,600	2,486.3	9.8	0.0220	0.011	0.20	0.1	0.4	3.8	0.5	0.0017
T22	62,500	717.9	5.0	0.0370	0.011	0.20	0.1	0.4	4.2	0.6	0.0018
T23	60,900	698.9	5.2	0.0190	0.011	0.20	0.1	0.4	4.2	0.6	0.0018
T24	136,600	1,568.6	5.0	0.0270	0.011	0.20	0.1	0.4	4.4	0.6	0.0018
T25	90,900	1,043.6	14.7	0.0180	0.011	0.20	0.1	0.4	3.6	0.5	0.0016
T26	308,900	3,545.7	6.9	0.0210	0.011	0.20	0.1	0.4	4.2	0.6	0.0018
PTARMIGAN BASIN											
P01	26,310	90.6	38.1	0.0560	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
P02	24,200	111.1	40.0	0.0530	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
P03	15,380	70.6	40.0	0.0480	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
P04	23,490	80.9	39.6	0.0310	0.011	0.24	0.1	0.35	4.1	0.6	0.0018
P05	38,570	132.8	43.6	0.0240	0.011	0.24	0.1	0.35	3.7	0.5	0.0017
P06	5,680	65.2	5.0	0.0180	0.011	0.15	0.1	0.4	4.2	0.6	0.0018
P07	11,510	92.5	21.4	0.0360	0.011	0.15	0.1	0.4	3.7	0.6	0.0017
P08	14,460	66.4	22.1	0.0240	0.011	0.21	0.1	0.4	4.5	0.6	0.0018
P09	15,100	52.0	49.4	0.0130	0.011	0.24	0.1	0.35	3.7	0.5	0.0018
P10	4,580	52.6	5.0	0.0180	0.011	0.15	0.1	0.4	3.1	0.5	0.0018
P11	36,630	84.1	42.6	0.0130	0.011	0.24	0.1	0.35	4.1	0.6	0.0017
P12	28,840	99.3	23.1	0.0170	0.011	0.24	0.1	0.35	3.1	0.5	0.0018
P13	24,610	56.5	82.1	0.0270	0.011	0.21	0.1	0.35	4.4	0.6	0.0018
P14	5,900	40.6	44.4	0.0110	0.011	0.21	0.1	0.35	4.5	0.6	0.0018
P15	8,330	86.1	5.0	0.0080	0.011	0.15	0.1	0.4	3.8	0.6	0.0018
P16	18,470	212.0	5.0	0.0240	0.011	0.17	0.1	0.4	3.2	0.5	0.0018
P17	6,570	75.4	5.0	0.0180	0.011	0.17	0.1	0.4	3.5	0.5	0.0018
P18	4,230	48.6	5.0	0.0130	0.011	0.17	0.1	0.4	3.4	0.5	0.0018
RIVER RIDGE BASIN											
R01	15,990	73.4	36.3	0.0700	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
R02	39,240	117.1	44.2	0.0824	0.011	0.24	0.1	0.35	3.3	0.5	0.0014
R03	17,810	143.1	9.9	0.1059	0.011	0.15	0.1	0.4	3.9	0.6	0.0018
R04	53,304	428.3	15.4	0.0760	0.011	0.17	0.1	0.4	4.2	0.6	0.0018
R05	28,161	129.3	40.0	0.0227	0.011	0.24	0.1	0.4	4.5	0.6	0.0018
R06	85,090	859.5	12.4	0.0369	0.011	0.20	0.1	0.4	3.5	0.5	0.0018
R07	76,012	872.5	5.0	0.0304	0.011	0.17	0.1	0.4	3.7	0.5	0.0018
R08	98,210	1,127.3	18.5	0.0302	0.011	0.17	0.1	0.4	3.6	0.6	0.0017
BLUFF BASIN											
B01	27,100	186.8	5.0	0.1300	0.011	0.17	0.1	0.4	3.8	0.6	0.0018
B02	69,000	792.1	36.9	0.0100	0.011	0.15	0.1	0.4	2.2	0.4	0.0012
B04	23,600	271.1	33.7	0.0200	0.011	0.17	0.1	0.4	2.4	0.4	0.0012
B05	23,100	159.1	13.1	0.0900	0.011	0.15	0.1	0.4	4.0	0.6	0.0018
B06	16,800	144.7	19.4	0.0700	0.011	0.17	0.1	0.4	3.4	0.5	0.0018
B07	25,200	173.4	8.7	0.0600	0.011	0.15	0.1	0.4	4.0	0.6	0.0018
B08	3,700	42.4	5.0	0.0200	0.011	0.17	0.1	0.4	3.3	0.5	0.0018
B09	19,800	113.9	5.0	0.1400	0.011	0.15	0.1	0.4	4.0	0.6	0.0018

Table 4.1 Hydrologic Parameters for Existing Conditions (continued).

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
BLUFF BASIN (continued)											
B10	10,500	96.5	5.0	0.0300	0.011	0.17	0.1	0.4	3.6	0.5	0.0018
B11	12,500	71.7	5.0	0.1300	0.011	0.15	0.1	0.4	3.8	0.6	0.0018
B12	3,700	42.9	5.0	0.0100	0.011	0.17	0.1	0.4	3.5	0.5	0.0018
B13	16,600	114.5	6.2	0.0900	0.011	0.15	0.1	0.4	4.3	0.6	0.0018
B14	7,500	34.6	5.1	0.0800	0.011	0.15	0.1	0.4	3.8	0.6	0.0017
OKLAHOMA BASIN											
O01	6,600	75.9	19.5	0.0230	0.011	0.15	0.1	0.4	3.8	0.5	0.0018
O02	127,600	1,171.8	5.16	0.1250	0.011	0.20	0.1	0.4	3.8	0.6	0.0018
O03	24,400	231.7	5.0	0.0780	0.011	0.20	0.1	0.4	4.4	0.6	0.0018
O04	66,400	762.7	10.0	0.0690	0.011	0.17	0.1	0.4	4.2	0.6	0.0018
O05	93,500	1,073.5	5.0	0.0420	0.011	0.17	0.1	0.4	4.1	0.6	0.0018
O06	19,000	174.4	27.7	0.0100	0.011	0.24	0.1	0.35	3.5	0.5	0.0018
O07	73,200	734.7	5.9	0.0980	0.011	0.20	0.1	0.4	4.1	0.6	0.0018
O08	122,200	1,402.4	7.9	0.0450	0.011	0.20	0.1	0.4	3.7	0.5	0.0018
O09	142,600	1,637.3	12.6	0.0300	0.011	0.20	0.1	0.4	3.6	0.5	0.0017
SOUTH STATE HIGHWAY 257 BASIN											
S01	11,100	127.4	12.4	0.05	0.011	0.17	0.1	0.4	3.5	0.5	0.0017
S02	101,100	812.6	5.1	0.12	0.011	0.15	0.1	0.4	3.9	0.6	0.0018

4.3.2 Land Use

The majority of the existing land use in the watershed is agricultural or open space. Nine basins (Bluff, Jacoby, Law, Oklahoma, Ptarmigan, River Ridge, SH 257, Timnath, and Windsor) have primarily undeveloped agricultural lands and open space, with limited low to medium density housing and commercial development. The remaining basin (High School Basin) consists predominantly of medium density residential housing with commercial development concentrated along State Highway 392.

For existing conditions, the percent imperviousness of each subbasin was estimated from aerial photography provided by Town of Windsor and verified during field investigations. This photography displays existing developments through the Fall of 1999. Existing conditions with respect to this master planning effort assumed the land use that existed, including those developments approved for construction, as of June 1, 2000. For future development conditions, the Town of Windsor and Weld County provided land use and zoning maps. The existing and future land use information is provided on Sheets 4 and 5, respectively, located in the project notebook.

Depending on the nature of the land use, the runoff block requires Manning's roughness values for overland flow for pervious and impervious areas of the subbasin. Roughness values were chosen based on the dominant land use in the subbasin. Subbasins containing more than one type of land use utilized an area-weighted average to determine roughness values.

4.3.3 Soils, Infiltration, and Depression Storage

Soils information was obtained from the Soil Survey of Weld County, Southern Part, Colorado (Soil Conservation Service, 1980) and the Soil Survey of Larimer County Area, Colorado (Soil Conservation Service, 1980). The soil types specified from the soil surveys were classified into the four hydrologic soil groups. These four groups classify the soils according to infiltration rates, from "A" representing well drained soils to "D" representing poorly drained soils. The soil types represented within the ten drainage basins can be generally classified in the B to C hydrologic soil groupings. Soils mapping is provided on Sheets 3A and 3B, located in the project notebook.

The Urban Drainage and Flood Control District (UDFCD) conducted a detailed analysis of rainfall/runoff data for each of the hydrologic soil groups and established recommended values for infiltration rates and decay coefficients for use with CUHP (USDCM, Vol. 1, 2001). These values are also utilized in the runoff block of EPA SWMM. The values recommended for each of the soil groups are reproduced in Table 4.2. For subbasins containing more than one soil group classification, the coverage of each soil group was measured and an area-weighted average determined.

Table 4.2. Hydrologic Soil Groups Recommended Values (USDCM, Vol. 1, 2001).

SCS Hydrologic Soil Group	Infiltration (in/hr)		Horton's Decay Coefficient
	Initial	Final	
A	5.0	1.0	0.0007
B	4.5	0.6	0.0018
C	3.0	0.5	0.0018
D	3.0	0.5	0.0018

Surface depression storage losses and water intercepted by trees, bushes, and other vegetation play an important role in the hydrologic cycle and the determination of rainfall available for runoff. The CUHP method requires estimation of these losses for both impervious and pervious areas to facilitate the calculation of the effective rainfall for each storm event. Values for surface depression storage and interception losses were selected in accordance with the values presented in Volume 1 of the USDCM (UDFCD, 2001). Infiltration, depression storage, and decay coefficient values incorporated into the hydrologic model are presented in Table 4.1.

4.4 Hydraulic Modeling Parameters

Several hydraulic modeling parameters are required by the transport block of the EPA SWMM model to simulate the routing of storm flows through the storm sewers and open channels. For the modeling of open channels, the hydraulic parameters required by the transport block of the EPA SWMM model are as follows:

1. Bottom width of channel or channel cross section
2. Length of channel
3. Invert slope
4. Channel sideslopes (1H: XV)
5. Manning's *n*
6. Maximum flow depth

Where appropriate, data for each of the parameters were recorded during the field reconnaissance and surveying efforts.

4.5 Special Modeling Functions Utilized

In addition to the basic channel routing functions incorporated into the EPA SWMM model, special modeling functions are often required in the hydrologic model to simulate the more intricate drainage systems. The EPA SWMM model incorporates the capability to simulate detention storage facilities and flow diversions. For the modeling efforts associated with the ten major drainage basins, the capability to simulate detention storage facilities and flow diversions was utilized extensively.

As previously discussed in Chapter 3, those detention facilities that provided a significant reduction in peak discharge during the major storm events (50-year and 100-year storm events) were identified and inventoried. Information related to the existing detention facilities or reservoirs was collected from previous drainage studies or reports provided by the Town of Windsor and supplemented by information collected from the State Engineer's Office. As mentioned previously, several other detention facilities also exist within the watershed; however, the effectiveness of these ponds in providing reduction of peak discharges is primarily limited to the more frequent storm events. During the master planning effort, forty-four existing detention ponds were identified and incorporated into the EPA SWMM for the ten drainage basins. At these locations, stage-storage-discharge relationships were developed for each detention pond or system of ponds and are summarized in the project technical notebook. The EPA SWMM model utilized these relationships to determine the volume of storm water detained in each pond and the corresponding discharge that was released from the pond.

In addition to the detention ponds, seven retention areas were identified in the watershed and specifically within the Law Basin. The retention areas are enclosed subbasins from which no discharge releases are anticipated for flood events up to the 100-year event. The retention or non-contributing areas include several irrigation reservoirs that have adequate capacity to store storm water runoff above the normal operating levels of the reservoir. Those retention ponds and areas identified as non-contributing areas are shown on Sheets 1 and 2.

Five flow diversion locations were incorporated into the modeling efforts completed during the hydrologic analyses. For the existing conditions, future conditions , and future conditions with over-detention scenarios, these diversions are itemized and described below.

<u>Law Basin:</u> A diversion was located at the end of the Springer Ditch to remove 150 cfs of irrigation flow assumed to be present in the ditch between Windsor Reservoir and the Greeley No. 2 Canal confluence. The irrigation flows were assumed to remain in the Greeley No. 2 Canal.

Windsor Basin: A flow diversion was located at the First Street and Chestnut Street intersection to model the splitting of storm water runoff at this location.

Jacoby Basin: A flow diversion was located adjacent to State Highway 392 to divert storm water runoff into the West Side Storm Sewer.

Similar flow diversions were utilized for the scenario assuming future conditions with over-detention and selected drainage improvements with one exception. The diversion of the 600 cfs irrigation flow into the Kern Reservoir (Windsor Basin) was relocated to the diversion of 600 cfs from the Greeley No. 2 Canal at State Highway 392 (Law Basin).

4.6 Summary of Existing Conditions

4.6.1 Definition of Existing Conditions

As mentioned previously, the definition of existing conditions includes all development that presently exists or was approved for construction prior to June 1, 2000. All basin development after this date is considered under the future conditions analyses. Results of the hydrologic analyses and modeling identified locations where major problems exist; these locations include areas of potential flooding as well as locations where the storm drainage criteria, as defined in the next section, are violated.

4.6.2 Storm Drainage Criteria

Storm drainage in the Town of Windsor is currently regulated by the Town of Windsor Storm Drainage Design Criteria Manual (Engineering Professionals, Inc., 1990). The purpose of this manual is to set forth the technical criteria to be used in the analysis and design of drainage systems within the Town of Windsor. The manual focuses on the criteria that pertain to: (a) runoff methodologies; (b) road crossings; (c) storm sewers; (d) open channels; (e) culverts; and (f) detention ponds. Discussions with Town Staff, as well as the information contained in the design criteria manual, resulted in the recommendations below with respect to supplemental storm drainage criteria that would guide the development and evaluation of the master drainage plan.

Major Road Crossings:	For major storm events, the criteria in the manual will govern except on State Highway 257 and State Highway 392. On State Highway 257 and State Highway 392, no overtopping of the roadway during major storm events will be allowed.
Detention Ponds:	The releases from on-site detention facilities (associated with future development) were limited to the 10-year existing condition peak discharge during the 100-year storm event. The over-detention scenario was applied to every subbasin with undeveloped area except in the Ptarmigan Basin, where proposed development south of State Highway 392 will limit releases based on the available capacity of existing structures under State Highway 392 as reported in previously approved drainage studies.

These drainage criteria were utilized to identify potential problems along the major drainageways. In general, violations related to the criteria were specifically noted where road crossings exceeded maximum allowable overtopping depths or detention facilities overtopped the pond embankments during the storm events.

4.6.3 Modeling Results

Based on the hydrologic analyses of existing conditions, several facilities or structures lack the capacity to safely convey the 100-year design storm thereby creating potential flooding problems within the basin. The results of the hydrologic modeling of existing conditions for each subbasin are presented in Appendix B.1. The hydrologic modeling results for locations along the major drainageways are presented in Table 4.3. For illustrative purposes, Figure 4.1 presents the 100-year peak discharge at several locations within the watersheds. At the request of the Town Staff, a more detailed analysis of the surface runoff in the urbanized portions of the Windsor Basin and High School Basin (those portions south of State Highway 392) was conducted. This work effort provided an indication of the flow distribution within the street networks in this portion of the community. The results of this work effort are also presented in Table 4.3 and depicted on Figure 4.2.

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Existing Conditions.

□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr□	1□ yr□
LAW BASIN (MAIN CHANNEL)								
Cache la Poudre River	L701	18,671	978	1,423	1,717	2,313	2,899	5,168
Eastman Park Dr.	L206	16,331	293	429	521	1,324	2,738	4,837
C&S Railroad	L208	16,135	323	468	563	1,387	2,871	5,082
State Highway 392	L225	11,012	96	139	166	444	1,015	1,890
Greeley No. 2 Canal/WCR 21	L226	10,778	103	145	173	438	1,009	1,879
WCR 70	L227	10,236	101	139	168	357	846	1,589
WCR 70 ½	L229	9,425	86	123	144	208	519	1,045
Law Reservoir Outflow	L340	8,084	0	0	0	79	310	1,073
Law Reservoir Inflow	L240	8,084	210	301	354	554	1,189	2,136
Loop Lake Outflow	L341	5,713	0	0	0	0	0	1,752
Loop Lake Inflow	L241	5,713	331	494	587	1,171	2,616	4,738
State Highway 14/WCR 82	L42	2,986	395	559	665	941	2,049	3,466
LAW BASIN - WEST TRIBUTARY								
State Highway 392 & Great Western Railroad	L210	4,676	237	344	418	923	1,832	3,182
WCR 19	L211	3,929	246	357	426	867	1,706	2,972
Basin L11 Concentration Point	L711	3,929	271	383	458	884	1,740	3,033
Greeley No. 2 Canal	L715	3,577	265	373	447	835	1,635	2,849
Basin L12 Inflow Downstream of State Highway 257*	L712	2,522	353	439	495	755	1,314	2,173
Ventana Way*	L118	1,707	307	372	412	588	946	1,500
WCR 72*	L219	1,460	255	304	334	504	802	1,281
WCR 74*	L220	793	247	290	315	380	515	759
Basin L20 Inflow	L820	793	97	140	165	230	365	609
State Highway 257 (WCR 17)	L812	815	51	71	85	167	372	673
WINDSOR BASIN – LOWER								
Cache la Poudre River	W700	4,208	555	790	947	1,326	1,600	1,925
Water Valley Parkway	W204	3,784	58	85	114	244	401	618
Eastman Park Drive East of 1st St.	W220	3,553	26	41	66	144	229	350
Folkstone Park Pond	W323	3,454	13	32	52	116	185	284
Garden Drive	W723	3,454	131	195	235	355	452	570
Chestnut Street	W223	3,392	118	173	207	284	353	457
Chimney Park	W128	3,158	60	92	113	164	208	244
Walnut Street	W229	3,116	36	52	62	90	111	146
C&S Railroad	W240	3,089	23	33	39	52	63	76
WINDSOR BASIN – UPPER								
Kern Reservoir Outflow	W341	3,043	10	14	17	30	47	58
Kern Reservoir Inflow	W241	3,043	570	852	1,042	1,426	1,675	1,978
Upstream of Greeley No.2 Canal/State Highway 257	W142	2,713	105	157	192	519	1,062	1,860
WCR 15	W243	2,322	129	195	232	553	1,127	1,978
Basin W44 Outlet Point	W144	913	69	102	122	272	543	910
Lake Canal	W244	913	93	140	168	313	596	975
State Highway 68/WCR 74	W245	609	111	170	207	298	455	728
WINDSOR BASIN – LOWER MISCELLANEOUS								
Eastman Park Dr. at 1st St.	W715	262	16	25	34	71	111	162
1st St. Flows	W124	234	12	19	27	57	92	136
Chestnut St. Flows	W924	234	58	85	103	142	190	238
1st St. at Chestnut St.	W224	234	82	124	153	222	290	381

*Discharge values include assumed irrigation base flow of 150 cfs.

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Existing Conditions (continued).

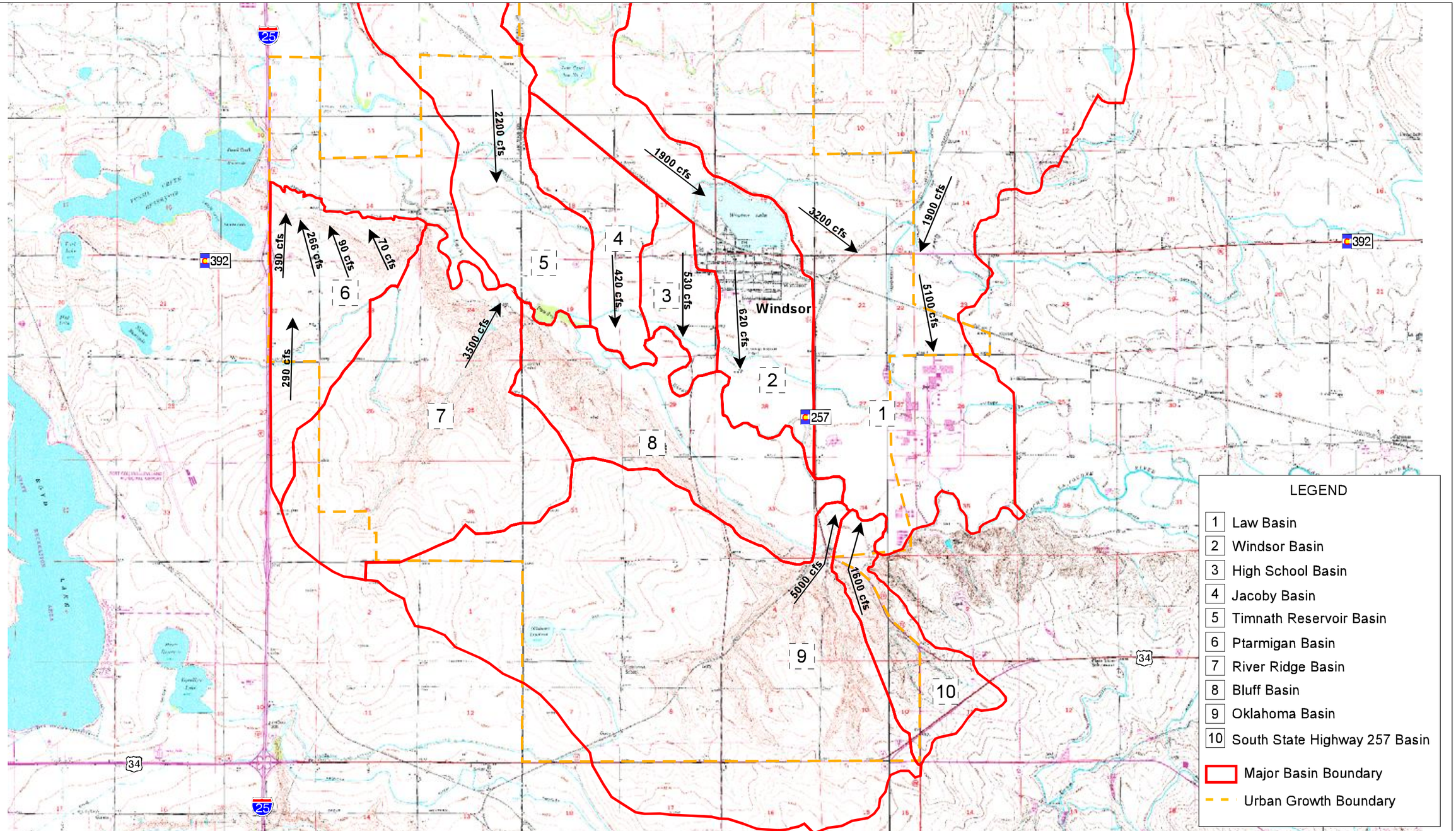
□ocation	EPA SWMM Elemen t	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr□	1□ yr□
WINDSOR BASIN – LOWER MISCELLANEOUS (continued)								
Oak St. Flows	W130	197	62	91	116	178	248	328
Elm St. at 7th St.	W131	77	33	51	62	85	114	123
Eastman Park Dr. at 54-inch Storm Sewer	W710	162	22	32	38	78	115	161
Eastman Park Dr. at 3rd St.	W210	109	72	112	137	194	238	295
HIGH SCHOOL BASIN								
Cache la Poudre River	H700	558	191	288	353	548	721	943
Whitney Ditch – East	H205	302	116	172	206	307	406	528
Stone Mountain Dr. – East	H206	229	121	178	214	309	393	495
State Highway 392 at High School	H209	103	14	20	24	45	65	90
Greeley No. 2 Canal	H310	35	5	7	8	16	23	33
Whitney Ditch – West	H212	150	106	150	180	251	310	381
Stone Mountain Dr. – West	H214	129	86	122	146	202	248	302
State Highway 392 at 12th Street	H315	11	2	2	3	6	8	11
JACOBY BASIN								
Cache la Poudre River	J700	1,016	101	146	175	249	304	564
Whitney Ditch	J205	871	23	32	43	87	206	416
Walnut Street	J206	777	12	17	20	39	139	326
State Highway 392	J707	692	59	86	105	168	285	452
State Highway 392 East Path	J207	195	20	29	34	71	111	164
Greeley No. 2 Canal East Path	J208	135	14	20	24	41	69	105
State Highway 392 West Path	J210	496	43	67	81	120	174	288
Greeley No. 2 Canal West Path	J211	457	53	76	90	124	166	251
72-inch Storm Sewer	J500	692	59	86	105	168	200	200
TIMNATH RESERVOIR BASIN								
Cache la Poudre River	T201	14,989	393	509	585	780	1,378	2,243
State Highway 392	T204	14,466	264	313	349	712	1,308	2,207
LCR 32E	T205	14,286	267	320	351	701	1,291	2,189
Greeley No. 2 Canal	T206	14,044	280	329	361	679	1,277	2,184
C&S Railroad & LCR 36	T207	12,988	269	321	351	520	957	1,567
LCR 38	T208	12,451	303	366	408	529	824	1,297
Timnath Reservoir Outlet	T320	11,300	163	170	177	204	230	258
Inflow to Timnath Reservoir	T720	11,300	1,673	2,493	3,023	4,276	5,045	7,945
PTARMIGAN BASIN								
Fossil Creek	P701	1,245	97	139	166	404	673	1,027
Basin P02 Outlet	P202	182	124	176	211	301	376	466
State Highway 392	P303	71	23	27	30	41	53	68
Basin P04 Outlet	P204	279	89	126	150	215	270	338
State Highway 392 at REA Parkway	P205	198	9	13	15	33	60	92
Basin P07 Outlet	P207	966	64	93	116	339	565	858
Downstream of State Highway 392 & LCR 5	P711	412	20	29	39	110	178	266
State Highway 392 at LCR 5	P209	153	16	22	27	67	109	160
State Highway 392 at Country Meadows	P211	259	7	10	16	46	75	114
Shutts	P212	175	9	13	15	45	75	114

*Discharge values include assumed irrigation base flow of 150 cfs.

Table 4. Major Drainageway Peak Discharges for Existing Conditions (continued).

Location	EPA SWMM Element	Drainage Area (acres)	Peak Discharge (cfs)					
			5 yr	10 yr	100 yr	500 yr	1000 yr	10000 yr
PTARMIGAN BASIN (continued)								
State Highway 392 at I-25	P214	339	24	35	42	141	252	391
Inflow to Westgate	P215	298	18	27	32	123	230	365
LCR 30 at I-25 Frontage Road	P16	212	29	42	50	110	193	292
RIVER RIDGE BASIN								
Cache la Poudre River	R700	3,751	290	429	517	1,409	2,350	3,664
River West Drive	R104	3,417	235	343	423	1,304	2,208	3,409
Basin R04 Concentration Point	R704	3,417	239	346	427	1,312	2,227	3,443
Basin R06 Inflow	R706	2,859	211	308	377	1,173	1,990	3,053
Basin R07 Concentration Point	R207	2,000	139	202	247	783	1,330	2,068
Basin R08 Inflow	R308	1,127	155	220	258	531	848	1,267
BLUFF BASIN								
No major drainageway for this subbasin. See subbasin peak runoff table (Appendix B.1) for results.								
OKLAHOMA BASIN								
Cache la Poudre River	O701	7,264	305	432	515	1,668	3,070	4,935
State Highway 257	O201	7,189	307	433	515	1,674	3,113	5,006
Great Western Railroad	O703	6,017	286	397	470	1,351	2,379	3,726
Basin O04 Concentration Point	O704	5,785	284	391	463	1,308	2,259	3,511
Basin O05 Concentration Point	O705	5,022	247	341	401	1,099	1,762	2,672
Basin O07 Concentration Point	O707	3,774	216	293	344	817	1,143	1,611
WCR 17 Pond Outflow	O308	3,040	197	269	316	687	926	1,099
WCR 17 Pond Inflow	O109	3,040	370	540	657	1,158	2,142	3,382
Basin O08 Concentration Point	O208	3,040	464	667	786	1,212	2,187	3,428
Basin O09 Inflow	O809	1,637	292	420	498	697	1,162	1,801
SOUTH STATE HIGHWAY 257 BASIN								
Cache la Poudre River	S701	940	53	80	98	508	993	1,744
State Highway 257	S802B	813	48	71	86	451	880	1,565

*Discharge values include assumed irrigation base flow of 150 cfs.



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PROJECT NUMBER: COTOW08	DRAWN BY: RJN
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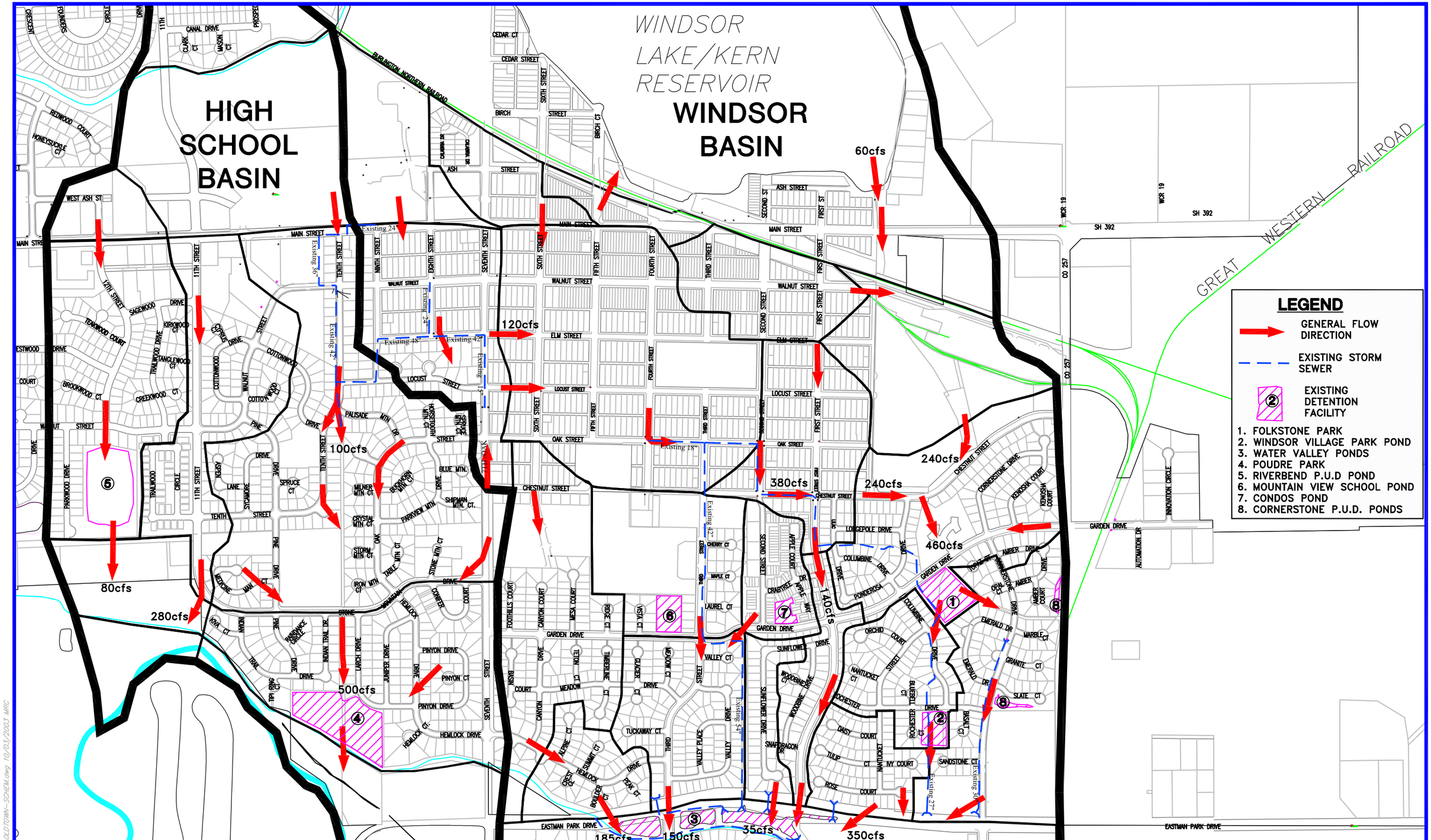
DATE: 10/02/2003	CHECKED BY: BAA

Anderson Consulting Engineers, Inc.
Civil • Water Resources • Environmental

TOWN OF WINDSOR
Master Drainage Plan

EXISTING CONDITION
100-YEAR PEAK DISCHARGES

FIGURE 4.1



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Phone (970) 226-0120 / Fax (970) 226-0121

TOWN OF WINDSOR - MASTER DRAINAGE PLAN
HIGH SCHOOL AND LOWER WINDSOR BASINS
EXISTING CONDITION
100-YEAR PEAK DISCHARGES AND FLOW PATHS

Project No.	COTOW08
Date:	09/23/2003
Design:	BLV/CJP
Drawn:	MRC
Revisions:	
ACADFILE: OLDTOWN-SCHEM	

FIGURE 4.2

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4.6.4 Law Basin: Summary of Existing Drainage Problems

During the 100-year flood event, storm water runoff typically exceeds the capacity of the natural channel (typically the Law Ditch). The limits of flooding were approximately defined during a floodplain analysis of the Law Ditch drainageway shown on the currently effective Federal Emergency Management Agency (FEMA) flood insurance rate maps. The regulatory floodplain information is also provided on Sheet 1A and 1B of this master plan study. Flooding along the Law Ditch drainageway is exacerbated where road crossings exist. It is also recognized that potential flooding problems will likely occur along several of the major tributaries to the major drainageway in the Law Basin, especially at the road crossings where undersized culverts and sediment accumulation limit the capacity of the crossing structures to convey stormwater runoff. A brief summary of the major problem areas noted during this master planning effort is presented in the following paragraphs. This summary is limited, however, to those locations along the Main Channel and West Tributary of the Law Basin that are within the Growth Management Boundary.

Main Channel

Eastman Park Drive Culvert. The 100-year peak discharge in the Main Channel of the Law Basin at Eastman Park Drive is estimated to be 4,840 cfs. This flow greatly exceeds the capacity of the existing crossing structure, which is estimated to be 60 cfs. In addition, the capacity of the Main Channel is greatly exceeded by the magnitude of the 100-year peak discharge. Stormwater runoff exceeding the channel capacity will flow primarily to the east along Eastman Park Drive before overtopping the road. These circumstances result in an extensive floodplain near the confluence of the Main Channel and Eastman Park Drive. An improved conveyance channel and crossing structure could significantly reduce the localized flooding problems at this location.

Colorado & Southern Railroad Culvert. Low flows conveyed within Main Channel cross under the Colorado & Southern Railroad in a reinforced box culvert. The capacity of the box culvert prior to overtopping the railroad is estimated to be 370 cfs. The 100-year existing condition discharge at this location is 5,080 cfs. Large storm event flows will spill to the east overtopping the railroad for approximately 4,000 feet. Installation of channel improvements and a larger crossing structure will reduce the flooding at this location.

State Highway 392 Culvert. The capacity of this arched-CMP culvert is 45 cfs prior to overtopping the channel banks and subsequently the highway. Large storm events will extend to the west overtopping the highway until ultimately coinciding with storm water runoff conveyed by the West Tributary at a low spot near the intersection of the Great Western Railroad and State Highway 392. The 100-year existing condition discharge in the Main Channel at this location is 1,890 cfs.

Channelization of storm water runoff and an improved culvert crossing could reduce flooding problems at this location.

Greeley No. 2 Canal Confluence. All storm water runoff in the Main Channel enters the Greeley No. 2 Canal east of WCR 21. Since a stormwater crossing facility does not exist at this location, there is minimal capacity to handle large storm events. This crossing lacks the capacity to convey the 100-year peak discharge of 1,880 cfs at this location. Channelization and a crossing structure could reduce the flooding problems at this location.

West Tributary

Great Western Railroad Culvert. The 100-year existing condition discharge in the West Tributary at the Great Western Railroad is 3,180 cfs. This compares to a culvert capacity, prior to overtopping the railroad, of 120 cfs. An improved crossing structure could significantly reduce the localized flooding problems at this location.

State Highway 392 Culvert. At this location, the West Tributary is conveyed under the highway in a CMP culvert. The capacity of the culvert prior to overtopping the road is estimated to be 20 cfs. The 100-year existing condition discharge in West Tributary at State Highway 392 is 3,180 cfs. An improved crossing structure could reduce the localized flooding problems at this location and prevent overtopping of the highway.

WCR 19. A stormwater crossing structure does not exist at this location. The 100-year existing condition discharge is 2,970 cfs. A crossing structure could reduce the localized flooding problems at this location.

Greeley No. 2 Canal. A stormwater crossing structure does not exist at this location. Stormwater captured and conveyed within and adjacent to the Windsor Reservoir Outlet (Springer Ditch) is intercepted by the Greeley No. 2 Canal. Limited capacity is available in the Greeley No. 2 Canal to convey stormwater; consequently, the majority of the stormwater will overtop the canal facilities. The 100-year existing condition discharge is 2,850 cfs. A crossing structure could reduce the localized flooding problems at this location.

State Highway 257/WCR 17. A stormwater crossing structure does not exist at this location. The 100-year existing condition discharge is 670 cfs. A crossing structure and minor channelization could reduce potential for flood events to overtop State Highway 257 at this location.

WCR 72 Culvert. The capacity of the culvert prior to overtopping the west bank upstream of the crossing of WCR 72 is 610 cfs. The 100-year existing condition discharge in West Tributary (Springer Ditch) at Weld County Road 72 is 1,280 cfs. An improved crossing structure and channel improvements could reduce the localized flooding problems at this location.

WCR 74 Bridge. The capacity of the bridge before overtopping the west bank upstream of WCR 74 is 580 cfs. The 100-year existing condition discharge in the West Tributary (Springer Ditch) at Weld County Road 74 is 760 cfs. An additional crossing structure at the overtopping location would reduce flooding problems at this location.

Within the Law Basin, road crossings of several additional tributaries to the Law Ditch major drainageway also exist. These tributaries convey significant storm flows that may require crossing structures capable of safely passing the 100-year existing condition discharge or peak discharges generated during minor storm events (i.e., 10-year or 50-year design storms). This master planning effort did not inventory or specifically identify these road crossings. It is noted, however, that several of these crossing structures provide inadequate capacity to convey runoff generated during relatively minor storm events and were experiencing sedimentation problems. As land development occurs within the basin, these structures should be more closely inventoried, evaluated and replaced where appropriate.

4.6.5 Windsor Basin: Summary of Existing Drainage Problems

As mentioned previously, the Windsor Basin does not contain a single, well-defined major drainageway. Stormwater runoff is conveyed in swales in the upper portion of the basin and ultimately is captured by Kern Reservoir. Runoff generated in the more urbanized lower basin is conveyed through the residential street network. Flooding within the Windsor Basin is characterized by shallow flooding behind road embankments in the upper basin and adjacent to the streets and intersections in the lower basin.

Lower Basin - Upstream of Eastman Park Drive. Flooding in the Lower Windsor Basin (downstream of the Kern Reservoir) is typically shallow street flooding. However, flooding depths greater than 1-foot have been reported at numerous locations in the basin. For example, an evaluation of flow distribution at several intersections indicates that approximately 570 cfs peak discharge will be conveyed to the Folkstone Park Detention facility. The existing condition analysis indicates that the Folkstone Park Detention facility will be surcharged. Flooding problems in the vicinity of the Folkstone Detention facility could be reduced by enlarging the pond and the outlet structure.

State Highway 257/WCR 17 Culvert. The capacity of the culvert prior to overtopping the roadway was calculated to be 30 cfs. The 100-year existing condition discharge in Windsor Basin at WCR 17 is 1,860 cfs. An improved crossing structure will allow flows to pass from Osterhaut Lake to the Kern Reservoir without overtopping State Highway 257/WCR 17.

Greeley No. 2 Canal. A stormwater crossing structure does not exist at this location. Flows up to 330 cfs can be captured by the canal and conveyed under the State Highway 257 Bridge and into the Kern Reservoir. Canal flows in excess of 330 cfs will overtop the east bank of the canal upstream of the State Highway 257 Bridge and flow into Osterhaut Lake. The 100-year existing condition discharge is 1,860 cfs. Improvements to the canal to increase carrying capacity and direct more water to the existing State Highway 257 Bridge over the Greeley No. 2 Canal may eliminate the need for culvert improvements between Osterhaut Lake and the Kern Reservoir.

WCR 15. A stormwater crossing structure does not exist at this location. The 100-year existing condition discharge is 1,980 cfs. A crossing structure could reduce the flooding problem at this location.

4.6.6 High School Basin: Summary of Existing Drainage Problems

Given the urbanized nature of the basin, the existing drainage problems are created by local street flooding and inadequate capacity associated with the existing network of streets. In general, stormwater is collected and conveyed from north to south in 10th, 11th and 12th Streets. Localized flooding is likely to occur at the majority of the major intersections during the major storm events.

Whitney Ditch Culvert. The capacity of the culvert prior to overtopping is 210 cfs. The 100-year, existing condition discharge in the High School Basin at the Whitney Ditch is 530 cfs. An improved crossing structure and downstream channelization could reduce the flooding problems at this location.

Stone Mountain Drive Culvert. This culvert, located near an elementary school, has a capacity prior to overtopping Stone Mountain Drive of 95 cfs. The 100-year, existing condition discharge in the 10th Street Channel at Stone Mountain Drive is 495 cfs. An improved crossing structure and upstream channelization could significantly reduce the localized flooding problems at this location.

10th Street Channel. The 100-year, existing condition discharge in the 10th Street Channel is 495 cfs. The existing capacity of the channel with no freeboard is 350 cfs. An improved channel cross section could reduce the flooding problems.

4.6.7 Jacoby Basin: Summary of Existing Drainage Problems

Very few structures presently exist within the Jacoby Basin for conveyance of storm water. The potential drainage problems are related to road crossings as well as the capture and spilling of

storm water by the Greeley No. 2 Canal. As development continues to occur South of State Highway 392, local street flooding may also exist at the intersection of 15th Street and Walnut Street during the major storm events.

State Highway 392. At the intersection of State Highway 392 and WCR 15, a storm sewer (60-inch to 72-inch) presently exists with a maximum capacity of approximately 200 cfs. The 100-year existing condition discharge in Jacoby Basin at State Highway 392 is 450 cfs. An improved crossing structure and additional channel or storm sewer could eliminate existing condition flooding problems at this location and those properties located downstream of State Highway 392. Alternately, limiting the storm water release from those undeveloped portions of the basin upstream of the existing storm sewer could reduce the flooding conditions at this location in the future.

Greeley No. 2 Canal. The Greeley No. 2 Canal captures and conveys storm water from the Jacoby Basin as well as basins located to the west (Timnath Reservoir Basin). Limited capacity exists to convey the additional storm water intercepted during major storm events. A high potential exists for breaching or spilling storm water from the delivery canal and into adjacent properties and neighborhoods. The 100-year existing condition discharge into the Greeley No. 2 Canal is estimated to be 250 cfs east of WCR 15 and an additional 105 cfs west of WCR 15. Facilities that separate the irrigation flows from the storm water should be considered (siphons, flumes, etc.) as well as side-channel weirs structures that spill storm water flows captured by the canal to drainage improvements capable of conveying these storm water flows.

Finally, no crossing structure presently exists to convey storm water across Weld County Road 68½ . During major storm events, water ponds and overtops this county road (by approximately 0.3 feet) west of the intersection with WCR 15. It should be noted that the overtopping depth at this location is within the overtopping criteria regulated by the Town of Windsor. As development of the upstream properties occur, a crossing structure along with upstream detention should reduce the overtopping of the road crossing and limit the potential ponding at this location.

4.6.8 Timnath Reservoir Basin: Summary of Existing Drainage Problems

Potential flooding in the Timnath Reservoir Basin is typically related to inadequate capacity of crossing structures during the major storm events. Since the major drainage channel in the upper basin is the Timnath Reservoir Outlet, the capacity of the irrigation facilities is greatly exceeded by the storm water runoff and localized flooding will occur along this channel. Where the outlet channel confluences with the Greeley No. 2 Canal, the capacity of the conveyance facilities associated with the Greeley No. 2 Canal will be exceed and overtopping will occur.

LCR 32 ½ Culvert. The capacity of the culvert prior to overtopping LCR 32 ½ is 10 cfs. The 100-year existing condition discharge in the Timnath Reservoir Basin at LCR 32 ½ is 2,200 cfs. An improved crossing structure and channel work could reduce the localized flooding problems at this location.

Greeley No. 2 Canal. A stormwater crossing structure does not exist at this location. Stormwater in the Timnath Reservoir Outlet Ditch enters the Greeley No. 2 Canal through an inlet channel constructed in the north bank and will exit the canal along the lower southern bank. The 100-year existing condition discharge is 2,190 cfs. Improvements at this location should focus on separating irrigation flows and stormwater flows. Minimizing the amount of stormwater captured by the canal at this location will prevent stormwater from being conveyed to the east and into developed portions of the Town of Windsor.

LCR 36 Bridge. The capacity of the bridge prior to overtopping the east bank and subsequently LCR 36 is 1,050 cfs. The 100-year existing condition discharge at this location is 1,570 cfs. An improved crossing structure and channelization work would reduce flooding problems at this location.

Colorado & Southern Railroad Bridge. The capacity of the bridge before overtopping the east bank is also 1,050 cfs. As overtopping occurs, the stormwater ultimately is conveyed to the east along the railroad embankment and subsequently overtops LCR 36. The 100-year existing condition discharge at this location is 1,570 cfs. An improved crossing structure and channelization work would reduce flooding problems at this location and at LCR 36 immediately downstream.

4.6.9 Ptarmigan Basin: Summary of Existing Drainage Problems

Within the Ptarmigan Basin, the majority of the flooding problems are related to existing crossings of State Highway 392. The existing crossing structures are limited in capacity and existing stormwater runoff will likely overtop the highway at these locations.

Subbasin P14 Culverts under State Highway 392. The combined capacity of the existing culverts at this location prior to overtopping State Highway 392 is approximately 50 cfs. The 100-year existing condition discharge at this location is estimated to be 390 cfs. While existing and planned development between State Highway 392 and LCR 30 (Subbasins P14 and P15) are over-detaining runoff to meet the limitations of the existing facilities, it appears that the existing drainage studies did not include an off-site area contributing runoff to this location. The off-site area is Subbasin P16 (see Sheet 2A) south of LCR 30. An improved crossing structure and/or storm sewer would reduce flooding problems at this location. Alternatively, as properties develop within the contributing drainage subbasin, additional detention could be provided to limit the releases to the

capacity of the existing crossing structures and/or a regional detention facility could be constructed to eliminate the potential for overtopping State Highway 392 at this location.

Subbasin P11 Culvert under State Highway 392. The capacity of the existing culvert at this location prior to overtopping State Highway 392 is approximately 30 cfs. The 100-year existing condition discharge at this location is approximately 270 cfs. While existing and planned development upstream of the Subbasin P11 culvert is planning on releasing flows to meet the capacity of the existing structure, a large portion of the basin is currently undeveloped. Until development occurs, potential flooding and overtopping of the highway at this location is likely during the major storm events.

Subbasin P5 Culvert under State Highway 392. The capacity of the existing culvert at this location prior to overtopping State Highway 392 is approximately 50 cfs. The 100-year existing condition discharge at this location is estimated to be 90 cfs. While existing and planned development upstream of the Subbasin P5 culvert is planning on releasing flows to meet the capacity of the existing structure, a large portion of the basin is currently undeveloped. Until development occurs, potential flooding and overtopping of the highway at this location is likely during the major storm events.

4.6.10 River Ridge Basin: Summary of Existing Drainage Problems

Potential flooding within the River Ridge Basin is largely related to crossings of the major drainage channel that were created during development of the lower portion of the basin.

River Edge Road Culvert. The capacity of the culvert prior to overtopping River Edge Road is 2,535 cfs. The 100-year existing condition discharge in the River Ridge Basin is 3,440 cfs. An improved crossing structure and channel work could reduce the localized flooding problems at this location. Alternately, limiting the storm water release (through detention) from those undeveloped portions of the basin upstream of the existing road crossing could reduce the flooding conditions at this location in the future.

River West Drive Culvert. The capacity of the culvert before overtopping River West Drive is 2,340 cfs. The 100-year existing condition discharge in the River Ridge Basin is 3,440 cfs. An improved crossing structure and channel work could reduce the localized flooding problems at this location. Alternately, limiting the storm water release (through detention) from those undeveloped portions of the basin upstream of the existing road crossing could reduce the flooding conditions at this location in the future.

4.6.11 Bluff Basin: Summary of Existing Drainage Problems

Given the topographic conditions (i.e., high bluffs transitioning into very flat floodplain areas) and the lack of major drainage paths within the basin, no potential flooding problems were identified. As development occurs within the basin, potential flooding will be minimized through the construction of drainage facilities to convey stormwater runoff that follow the criteria established in the Town of Windsor Storm Drainage Design Criteria Manual (Engineering Professionals, Inc., 1990). Control of erosion from stormwater generated along the bluffs is an issue that must be addressed during the development of the property within the basin.

4.6.12 Oklahoma Basin: Summary of Existing Drainage Problems

Similar to the majority of drainage basins located south of the Cache la Poudre River, potential flooding problems within the Oklahoma Basin are related to crossings of the major drainage channel.

State Highway 257 Bridge. The capacity of the bridge prior to overtopping the north bank of the channel was calculated to be 2,900 cfs. The capacity of the bridge before overtopping State Highway 257 at the bridge site was calculated to be 5,000 cfs. The 100-year existing condition discharge in the Oklahoma Basin at State Highway 257 is 5,010 cfs. Channel improvements upstream of the existing bridge could reduce the flooding potential upstream of this location.

Great Western Railroad Culvert. The 100-year existing condition discharge in the Oklahoma Basin at the Great Western Railroad is 3,700 cfs. This compares to a culvert capacity, prior to overtopping the railroad, of 770 cfs. An improved crossing structure could significantly reduce the localized flooding problems at this location.

4.6.13 South State Highway 257 Basin: Summary of Existing Drainage Problems

The flooding problems within this drainage basin are limited to the crossing of the major drainage channel and State Highway 257. The 100-year existing condition discharge in the South State Highway 257 Basin at this location is 1,565 cfs. This compares to a culvert capacity, prior to overtopping the highway, of 1,075 cfs. An improved crossing structure could reduce the localized flooding problems at this location.

4.7 Summary of Future Conditions

4.7.1 Definition of Future Conditions

In this master planning effort, the generation of future condition hydrology is based on the land use plans as defined by the various communities' zoning and land use maps (see Sheets 5A and 5B located in the project notebook). Furthermore, future developments, according to the existing storm drainage criteria described previously, are required to provide on-site detention limiting releases to the 10-year existing condition peak discharge during the 10-year storm event and to the 100-year existing condition peak discharge during the 100-year storm event. The results of the future conditions model were utilized to develop and evaluate alternative strategies for further reducing discharges within the basin.

Based on the projected land uses, the hydrologic parameters describing each basin were modified. The revised hydrologic parameters reflecting projected land use for the ten drainage basins are presented in Table 4.4.

4.7.2 Modeling Results

The results of the hydrologic modeling of future conditions for each subbasin are presented in Appendix B.2. The hydrologic modeling results for locations along the major drainageways are presented in Table 4.5. Given the assumptions related to detention within the drainage basins, similar results were anticipated for the future conditions hydrologic model, i.e., several facilities or structures lack the capacity to safely convey the 100-year design storm thereby creating potential flooding problems within the basin. It should be noted that the results of detention within the basin generally reflect an increase in the peak runoff due to the timing of the flood peaks. Figure 4.3 presents the 100-year peak discharge data at several locations within the watershed based on the hydrologic model of future conditions.

4.7.3 Law Basin: Summary of Future Condition Drainage Problems

Within the Law Basin, the impact of future development on the stormwater runoff generated during the major storm events is not significant. This is largely attributed to the placement of on-site detention ponds that limit the releases during the 100-year storm event to the 100-year existing condition runoff. It is likely, however, that an increase in the peak flows within the major drainageway will occur with the on-site detention facilities. Delayed peaks resulting from the

Table 4.4 Hydrologic Parameters for Future Conditions.

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
LAW BASIN											
L01	31,330	287.7	22.3	0.0029	0.011	0.17	0.1	0.4	2.6	0.4	0.0014
L02	44,930	515.7	7.6	0.0044	0.011	0.17	0.1	0.4	3.4	0.5	0.0018
L03	82,720	569.7	47.2	0.0056	0.011	0.21	0.1	0.4	3.5	0.5	0.0018
L04	175,950	605.9	72.0	0.0052	0.011	0.24	0.1	0.35	4.3	0.6	0.0017
L06	57,010	196.3	66.7	0.0024	0.011	0.24	0.1	0.35	3.4	0.5	0.0018
L07	104,920	361.3	69.8	0.0105	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
L08	48,700	447.2	20.9	0.0056	0.011	0.17	0.1	0.4	3.5	0.5	0.0018
L10	130,070	746.5	33.7	0.0163	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
L11	153,460	352.3	40.7	0.0133	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
L12	149,960	516.4	33.6	0.0385	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
L13	64,990	298.4	30.0	0.0140	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
L15	61,540	635.7	10.6	0.0225	0.011	0.17	0.1	0.4	4.3	0.6	0.0018
L16	60,880	279.5	30.6	0.0143	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
L17	20,270	139.6	20.0	0.0214	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
L18	53,880	247.4	32.7	0.0200	0.011	0.24	0.1	0.35	4.4	0.6	0.0017
L19	145,190	666.6	29.0	0.0160	0.011	0.24	0.1	0.35	4.0	0.6	0.0018
L20	115,170	793.2	24.5	0.0107	0.011	0.24	0.1	0.35	4.1	0.6	0.0018
L25	29,070	233.6	22.7	0.0100	0.011	0.17	0.1	0.4	3.8	0.6	0.0018
L26	52,450	541.8	9.4	0.0176	0.011	0.17	0.1	0.4	4.3	0.6	0.0018
L27	42,730	490.5	5.0	0.0242	0.011	0.17	0.1	0.4	4.0	0.6	0.0018
L28	31,060	320.9	6.2	0.0163	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L29	59,750	685.8	5.0	0.0154	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L30	57,060	655.0	5.1	0.0167	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
L40	206,570	2,371.1	8.1	0.0281	0.011	0.17	0.1	0.4	4.2	0.6	0.0018
L41	237,620	2,727.5	8.1	0.0477	0.011	0.17	0.1	0.4	4.2	0.6	0.0017
L42	260,110	2,985.6	5.3	0.0544	0.011	0.17	0.1	0.4	4.1	0.6	0.0018
LNC1	387,100	3,554.6	27.7	0.0240	0.011	0.17	0.1	0.4	3.0	0.4	0.0014
LNC2	22,380	179.8	35.7	0.0429	0.011	0.17	0.1	0.4	3.0	0.4	0.0012
LNC3	71,490	656.5	20.6	0.0176	0.011	0.17	0.1	0.4	3.6	0.5	0.0016
LNC4	85,900	887.4	10.5	0.0127	0.011	0.17	0.1	0.4	4.3	0.6	0.0017
LNC5	165,270	1,707.3	9.1	0.0200	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
LNC6	17,080	196.1	5.0	0.0300	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
LNC7	1,197,670	13,747.4	8.1	0.0366	0.011	0.17	0.1	0.4	4.3	0.6	0.0017
WINDSOR BASIN											
W01	50,880	175.2	52.9	0.0060	0.011	0.24	0.1	0.35	2.3	0.4	0.0011
W02	85,770	196.9	51.9	0.0042	0.011	0.24	0.1	0.35	2.4	0.4	0.0011
W03	45,040	51.7	88.4	0.0118	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W04	11,650	40.1	41.2	0.0095	0.011	0.24	0.1	0.35	3.9	0.6	0.0018
W10	8,230	18.9	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W11	15,970	55.0	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W12	15,420	35.4	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W13	10,860	37.4	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W14	6,710	15.4	70.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W15	5,230	12.0	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W19	7,140	16.4	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W20	8,230	18.9	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018

Table 4.4 Hydrologic Parameters for Future Conditions (continued).

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
WINDSOR BASIN (continued)											
W21	17,480	56.3	45.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W22	10,500	24.1	40.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W23	25,530	62.4	45.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W24	16,030	36.8	45.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W28	9,150	42.0	60.0	0.0125	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W29	11,720	26.9	45.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W30	44,340	101.8	45.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W31	21,130	48.5	40.0	0.0020	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W32	12,330	28.3	70.0	0.0020	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W33	8,100	18.6	50.0	0.0035	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W34	7,410	8.5	90.0	0.0050	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W40	19,730	45.3	39.8	0.0006	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W41	140,350	322.2	78.2	0.0006	0.011	0.24	0.1	0.35	1.5	0.2	0.0006
W42	170,280	390.9	40.0	0.0060	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
W43	134,970	619.7	34.6	0.0156	0.011	0.24	0.1	0.35	4.1	0.6	0.0017
W44	66,300	304.4	30.4	0.0188	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
W45	132,530	608.5	30.0	0.0286	0.011	0.24	0.1	0.35	4.3	0.6	0.0018
W46	48,420	222.3	30.0	0.0162	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
W47	8,760	80.4	13.0	0.0429	0.011	0.24	0.1	0.35	4.6	0.7	0.0014
W48	60,550	486.5	19.3	0.0218	0.011	0.24	0.1	0.35	4.5	0.6	0.0017
HIGH SCHOOL BASIN											
H01	11,650	93.6	25.4	0.0100	0.011	0.16	0.1	0.4	2.6	0.4	0.0015
H02	1,530	12.3	19.0	0.0130	0.011	0.15	0.1	0.4	3.7	0.5	0.0015
H05	31,840	73.1	35.2	0.0080	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
H06	109,680	125.9	45.9	0.0070	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H09	29,750	68.3	51.7	0.0020	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H10	15,160	34.8	40.0	0.0030	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H12	9,280	21.3	33.7	0.0130	0.011	0.24	0.1	0.35	4.1	0.6	0.0018
H13	44,260	50.8	58.0	0.0070	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
H14	28,880	66.3	51.9	0.0090	0.011	0.24	0.1	0.35	4.2	0.6	0.0018
H15	9,930	11.4	90.0	0.0090	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
JACOBY BASIN											
J01	18,050	145.0	24.9	0.0030	0.011	0.17	0.1	0.4	2.5	0.4	0.0015
J05	41,210	94.6	36.9	0.0100	0.011	0.24	0.1	0.35	3.2	0.5	0.0018
J06	30,600	84.9	70.1	0.0070	0.011	0.24	0.1	0.35	4.2	0.6	0.0018
J07	26,140	60.0	56.0	0.0070	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
J08	25,570	58.7	39.2	0.0040	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
J09	33,400	76.6	40.0	0.0040	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
J10	34,590	39.7	83.9	0.0080	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
J11	19,520	44.8	39.5	0.0040	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
J12	179,400	411.8	40.0	0.0020	0.011	0.17	0.1	0.4	4.5	0.6	0.0018
TIMNATH RESERVOIR BASIN											
T01	31,500	310.6	48.5	0.0050	0.011	0.15	0.1	0.4	2.5	0.4	0.0012
T02	27,100	211.9	47.3	0.0080	0.011	0.20	0.1	0.4	3.5	0.5	0.0016
T04	15,700	180.1	5.0	0.0030	0.011	0.20	0.1	0.4	3.6	0.5	0.0018
T05	26,200	242.3	22.7	0.0050	0.011	0.17	0.1	0.4	3.5	0.5	0.0016

TIMNATH RESERVOIR BASIN (continued)

Table 4.4 Hydrologic Parameters for Future Conditions (continued).

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
T06	287,500	1,056.0	36.17	0.0080	0.011	0.17	0.1	0.4	4.3	0.6	0.0018
T07	148,300	536.4	44.9	0.0070	0.011	0.17	0.1	0.4	4.4	0.6	0.0018
T08	202,900	1,151.6	32.9	0.0420	0.011	0.24	0.1	0.4	4.0	0.6	0.0017
T20	224,200	1,238.8	56.5	0.0130	0.011	0.24	0.1	0.4	2.0	0.3	0.0009
T21	271,800	2,486.3	18.3	0.0220	0.011	0.20	0.1	0.4	3.8	0.5	0.0017
T22	65,825	717.9	7.1	0.0370	0.011	0.20	0.1	0.4	4.2	0.6	0.0018
T23	67,900	698.9	9.5	0.0190	0.011	0.20	0.1	0.4	4.2	0.6	0.0018
T24	141,400	1,568.6	6.4	0.0270	0.011	0.20	0.1	0.4	4.4	0.6	0.0018
T25	92,800	1,043.6	15.2	0.0180	0.011	0.20	0.1	0.4	3.6	0.5	0.0016
T26	308,900	3,545.7	6.9	0.0210	0.011	0.20	0.1	0.4	4.2	0.6	0.0018
PTARMIGAN BASIN											
P01	26,310	90.6	38.1	0.0560	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
P02	24,200	111.1	40.0	0.0530	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
P03	15,380	70.6	40.0	0.0480	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
P04	23,490	80.9	39.6	0.0310	0.011	0.24	0.1	0.35	4.1	0.6	0.0018
P05	57,850	132.8	45.9	0.0240	0.011	0.24	0.1	0.35	3.7	0.5	0.0017
P06	28,400	65.2	40.0	0.0180	0.011	0.24	0.1	0.35	4.2	0.6	0.0018
P07	40,290	92.5	62.4	0.0360	0.011	0.24	0.1	0.35	3.7	0.6	0.0017
P08	57,850	66.4	73.1	0.0240	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
P09	22,650	52.0	52.1	0.0130	0.011	0.24	0.1	0.35	3.7	0.5	0.0018
P10	22,910	52.6	39.9	0.0180	0.011	0.24	0.1	0.35	3.1	0.5	0.0018
P11	36,630	84.1	42.6	0.0130	0.011	0.24	0.1	0.35	4.1	0.6	0.0017
P12	28,840	99.3	38.5	0.0170	0.011	0.24	0.1	0.35	3.1	0.5	0.0018
P13	24,610	56.5	82.1	0.0270	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
P14	35,370	40.6	77.4	0.0110	0.011	0.24	0.1	0.35	4.5	0.6	0.0018
P15	75,010	86.1	89.7	0.0080	0.011	0.24	0.1	0.35	3.8	0.6	0.0018
P16	18,470	212.0	5.0	0.0240	0.011	0.17	0.1	0.4	3.2	0.5	0.0018
P17	6,570	75.4	5.0	0.0180	0.011	0.17	0.1	0.4	3.5	0.5	0.0018
P18	8,470	48.6	19.2	0.0130	0.011	0.24	0.1	0.4	3.4	0.5	0.0018
RIVER RIDGE BASIN											
R01	15,990	73.4	36.3	0.0700	0.011	0.24	0.1	0.35	4.4	0.6	0.0018
R02	39,240	117.1	44.2	0.0824	0.011	0.24	0.1	0.35	3.3	0.5	0.0014
R03	41,556	143.1	15.0	0.1059	0.011	0.15	0.1	0.4	3.9	0.6	0.0018
R04	124,378	428.3	35.7	0.0760	0.011	0.17	0.1	0.4	4.2	0.6	0.0018
R05	28,161	129.3	40.0	0.0227	0.011	0.24	0.1	0.4	4.5	0.6	0.0018
R06	187,199	859.5	30.0	0.0369	0.011	0.20	0.1	0.4	3.5	0.5	0.0018
R07	190,030	872.5	26.5	0.0304	0.011	0.17	0.1	0.4	3.7	0.5	0.0018
R08	327,368	1,127.3	28.7	0.0302	0.011	0.17	0.1	0.4	3.6	0.6	0.0017
BLUFF BASIN											
B01	40,700	186.8	21.2	0.1300	0.011	0.17	0.1	0.4	3.8	0.6	0.0018
B02	96,700	792.1	49.8	0.0100	0.011	0.15	0.1	0.4	2.2	0.4	0.0012
B04	28,800	271.1	41.1	0.0200	0.011	0.17	0.1	0.4	2.4	0.4	0.0012
B05	28,800	159.1	27.7	0.0900	0.011	0.15	0.1	0.4	4.0	0.6	0.0018
B06	21,800	144.7	30.8	0.0700	0.011	0.17	0.1	0.4	3.4	0.5	0.0018
B07	26,000	173.4	21.7	0.0600	0.011	0.15	0.1	0.4	4.0	0.6	0.0018
B08	9,200	42.4	30.0	0.0200	0.011	0.17	0.1	0.4	3.3	0.5	0.0018
B09	23,500	113.9	23.5	0.1400	0.011	0.15	0.1	0.4	4.0	0.6	0.0018
BLUFF BASIN (continued)											
B10	21,000	96.5	30.0	0.0300	0.011	0.17	0.1	0.4	3.6	0.5	0.0018

Table 4.4 Hydrologic Parameters for Future Conditions (continued).

Subbasin No.	Basin Width (ft)	Basin Area (acres)	Percent Impervious	Average Basin Slope (ft/ft)	Manning's Roughness		Depression Storage (inches)		Infiltration Rates (in/hr)		Horton's Decay Rate
					Impervious	Pervious	Impervious	Pervious	Initial	Final	
B11	14,200	71.7	18.8	0.1300	0.011	0.15	0.1	0.4	3.8	0.6	0.0018
B12	9,340	42.9	30.0	0.0100	0.011	0.17	0.1	0.4	3.5	0.5	0.0018
B13	35,600	114.5	35.7	0.0900	0.011	0.15	0.1	0.4	4.3	0.6	0.0018
B14	15,100	34.6	40.1	0.0800	0.011	0.15	0.1	0.4	3.8	0.6	0.0017
OKLAHOMA BASIN											
O01	6,600	75.9	19.5	0.0230	0.011	0.15	0.1	0.4	3.8	0.5	0.0018
O02	129,100	1,171.8	5.6	0.1250	0.011	0.20	0.1	0.4	3.8	0.6	0.0018
O03	26,100	231.7	8.3	0.0780	0.011	0.20	0.1	0.4	4.4	0.6	0.0018
O04	164,300	762.7	29.8	0.0690	0.011	0.17	0.1	0.4	4.2	0.6	0.0018
O05	192,500	1,073.5	26.4	0.0420	0.011	0.17	0.1	0.4	4.1	0.6	0.0018
O06	19,000	174.4	30.1	0.0100	0.011	0.24	0.1	0.35	3.5	0.5	0.0018
O07	76,900	734.7	9.1	0.0980	0.011	0.20	0.1	0.4	4.1	0.6	0.0018
O08	185,100	1,402.4	19.4	0.0450	0.011	0.20	0.1	0.4	3.7	0.5	0.0018
O09	203,800	1,637.3	22.3	0.0300	0.011	0.20	0.1	0.4	3.6	0.5	0.0017
SOUTH STATE HIGHWAY 257 BASIN											
S01	11,100	127.4	12.4	0.05	0.011	0.17	0.1	0.4	3.5	0.5	0.0017
S02	101,100	812.6	5.5	0.12	0.011	0.15	0.1	0.4	3.9	0.6	0.0018

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Future Conditions.

□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr	1□ yr
LAW BASIN (MAIN CHANNEL)								
Cache la Poudre River	L701	18,671	904	1,320	1,587	2,671	4,160	6,182
Eastman Park Dr.	L206	16,331	687	1,019	1,237	2,330	3,684	5,537
C&S Railroad	L208	16,135	689	1,017	1,226	2,329	3,746	5,682
State Highway 392	L225	11,012	125	181	218	511	1,121	2,045
Greeley No. 2 Canal/WCR 21	L226	10,778	112	161	193	457	1,054	1,957
WCR 70	L227	10,236	101	139	168	357	846	1,589
WCR 70 ½	L229	9,425	86	123	144	208	519	1,045
Law Reservoir Outflow	L340	8,084	0	0	0	79	310	1,073
Law Reservoir Inflow	L240	8,084	210	301	354	554	1,189	2,136
Loop Lake Outflow	L341	5,713	0	0	0	0	0	1,752
Loop Lake Inflow	L241	5,713	331	494	587	1,171	2,616	4,738
State Highway 14/WCR 82	L42	2,986	395	559	666	942	2,049	3,466
LAW BASIN - WEST TRIBUTARY								
State Highway 392 & Great Western Railroad	L210	4,676	570	841	1,010	1,750	2,549	3,586
WCR 19	L211	3,929	506	741	887	1,546	2,257	3,191
Basin L11 Concentration Point	L711	3,929	516	755	900	1,565	2,291	3,239
Greeley No. 2 Canal	L715	3,577	473	692	824	1,434	2,110	2,997
Basin L12 Inflow Downstream State Highway 257*	L712	2,522	538	715	823	1,255	1,686	2,254
Ventana Way*	L118	1,707	414	535	610	886	1,164	1,532
WCR 72*	L219	1,460	388	493	558	790	1,017	1,326
WCR 74*	L220	793	291	352	390	493	606	772
Basin L20 Inflow	L820	793	141	202	240	343	456	622
State Highway 257 (WCR 17)	L812	815	126	183	217	369	523	721
WINDSOR BASIN – LOWER								
Cache la Poudre River	W700	4,208	554	789	945	1,323	1,596	1,921
Water Valley Parkway	W204	3,784	58	86	123	258	414	631
Eastman Park Drive East of 1st St.	W220	3,553	26	49	74	157	243	365
Folkstone Park Pond	W323	3,454	24	39	60	129	199	300
Garden Drive	W723	3,454	130	189	231	352	447	566
Chestnut Street	W223	3,392	113	170	206	287	368	465
Chimney Park	W128	3,158	56	85	105	157	190	239
Walnut Street	W229	3,116	45	67	81	111	133	160
C&S Railroad	W240	3,089	23	33	39	57	65	81
WINDSOR BASIN – UPPER								
Kern Reservoir Outflow	W341	3,043	21	30	37	57	58	59
Kern Reservoir Inflow	W241	3,043	579	867	1,061	1,452	1,747	2,375
Upstream of Greeley No. 2 Canal/State Highway 257	W142	2,713	348	523	633	1,147	1,632	2,263
WCR 15	W243	2,322	350	517	620	1,107	1,589	2,225
Basin W44 Outlet Point	W144	913	165	240	286	510	726	998
Lake Canal	W244	913	188	269	318	534	756	1,028
State Highway 68/WCR 74	W245	609	153	217	255	401	557	750
WINDSOR BASIN – LOWER MISCELLANEOUS								
Eastman Park Dr. at 1st St.	W715	262	16	25	34	71	111	162
1st St. Flows	W124	234	12	19	27	57	92	136
Chestnut St. Flows	W924	234	58	85	103	142	190	238
1st St. at Chestnut St.	W224	234	82	124	153	222	290	381

*Discharge values include assumed irrigation base flow of 150 cfs.

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Future Conditions (continued).

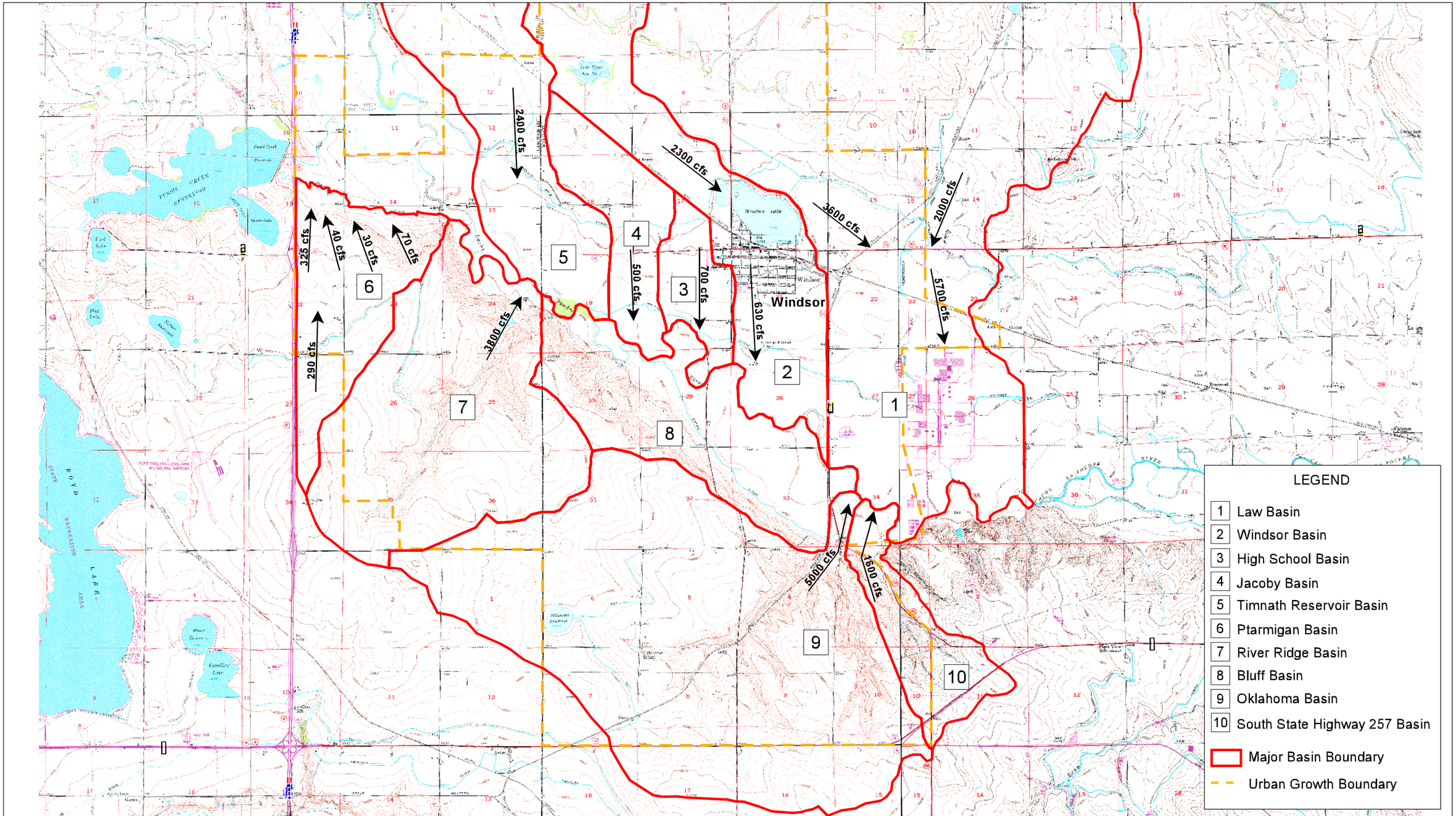
□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr [	□ yr	1□ yr [
WINDSOR BASIN – LOWER MISCELLANEOUS (continued)								
Oak St. Flows	W130	197	62	91	116	178	248	328
Elm St. at 7th St.	W131	77	33	51	62	85	114	123
Eastman Park Dr. at 54-inch Storm Sewer	W710	162	22	32	38	78	115	161
Eastman Park Dr. at 3rd St.	W210	109	72	112	137	194	238	295
HIGH SCHOOL BASIN								
Cache la Poudre River	H700	558	191	288	353	548	721	943
Whitney Ditch – East	H205	302	116	172	206	307	406	528
Stone Mountain Dr. – East	H206	229	121	178	214	309	393	495
State Highway 392 at High School	H209	103	14	20	24	46	65	90
Greeley No. 2 Canal	H410	35	5	7	8	16	23	33
Whitney Ditch – West	H212	150	106	150	180	251	310	381
Stone Mountain Dr. – West	H214	129	86	122	146	202	248	302
State Highway 392 at 12th Street	H315	11	2	2	3	6	8	11
JACOBY BASIN								
Cache la Poudre River	J700	1,016	101	145	175	248	343	531
Whitney Ditch	J205	871	22	32	42	141	259	407
Walnut Street	J206	777	12	17	20	96	195	320
State Highway 392	J707	692	94	136	161	257	341	447
State Highway 392 East Path	J207	195	26	38	46	85	118	161
Greeley No. 2 East Path	J208	135	18	26	32	55	76	103
State Highway 392 West Path	J210	496	68	98	115	172	223	287
Greeley No. 2 West Path	J211	457	63	90	106	153	196	251
72-inch Storm Sewer	J500	692	94	136	161	200	200	200
TIMNATH RESERVOIR BASIN								
Cache la Poudre River	T201	14,989	518	693	804	1,307	1,802	2,428
State Highway 392	T204	14,466	508	679	786	1,291	1,770	2,380
LCR 32E	T205	14,286	511	682	791	1,284	1,757	2,360
Greeley No. 2 Canal	T206	14,044	509	676	783	1,266	1,732	2,326
C&S Railroad & LCR 36	T207	12,988	391	497	565	905	1,239	1,664
LCR 38	T208	12,451	306	372	412	690	967	1,313
Timnath Reservoir Outlet	T320	11,300	167	176	184	212	236	264
Inflow to Timnath Reservoir	T720	11,300	1,626	2,355	2,841	4,178	5,115	8,098
PTARMIGAN BASIN								
Fossil Creek	P701	1,245	95	135	161	314	479	692
Basin P02 Outlet	P202	182	124	176	211	301	376	466
State Highway 392	P303	71	23	27	30	41	53	68
Basin P04 Outlet	P204	380	89	126	150	215	270	337
State Highway 392 at REA Parkway	P205	299	6	8	10	17	22	28
Basin P07 Outlet	P207	865	59	84	113	269	412	591
Downstream of State Highway 392 & LCR 5	P711	311	10	14	19	31	41	54
State Highway 392 at LCR 5	P209	52	9	13	16	26	34	43
State Highway 392 at Country Meadows	P211	259	2	3	4	10	16	22
Shutts	P212	175	1	2	3	9	13	19

*Drainage values include assumed irrigation base flow of 150 cfs.

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Future Conditions (continued).

□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr	1□ yr
PTARMIGAN BASIN (continued)								
State Highway 392 at I-25	P214	339	25	36	45	131	219	324
Inflow to Westgate	P215	298	19	28	34	112	195	298
LCR 30 at I-25 Frontage Road	P16	212	29	42	50	110	193	292
RIVER RIDGE BASIN								
Cache la Poudre River	R700	3,751	438	647	797	1,875	2,798	3,965
River West Drive	R104	3,417	415	606	736	1,763	2,650	3,741
Basin R04 Concentration Point	R704	3,417	419	609	738	1,774	2,664	3,764
Basin R06 Inflow	R706	2,859	352	506	615	1,528	2,306	3,249
Basin R07 Concentration Point	R207	2,000	244	351	422	1,053	1,598	2,257
Basin R08 Inflow	R408	1,127	157	220	258	604	913	1,267
BLUFF BASIN								
No ma□or drainage□ay for this subbasin. See subbasin pea□runoff table (Appendix B.□) for results.								
OKLAHOMA BASIN								
Cache la Poudre River	O701	7,264	594	861	1,029	2,134	3,365	5,004
State Highway 257	O201	7,189	597	865	1,034	2,144	3,405	5,047
Great Western Railroad	O703	6,017	580	838	999	1,815	2,705	3,862
Basin O04 Concentration Point	O704	5,785	575	829	988	1,762	2,584	3,646
Basin O05 Concentration Point	O705	5,022	489	704	836	1,433	2,031	2,836
Basin O07 Concentration Point	O707	3,774	358	510	604	940	1,213	1,608
WCR 17 Pond Outflow	O308	3,040	338	477	561	848	1,033	1,189
WCR 17 Pond Inflow	O109	3,040	566	818	967	1,693	2,494	3,529
Basin O08 Inflow Concentration Point	O208	3,040	597	868	1,027	1,710	2,509	3,546
Basin O09 Inflow	O809	1,637	365	531	629	973	1,364	1,899
SOUTH STATE HIGHWAY 257 BASIN								
Future condition discharges are the same as existing conditions. Future development not anticipated in this basin.								

*Drainage values include assumed irrigation base flow of 150 cfs.



REVISIONS:

Contour Interval 10 feet

2500 0 2500 5000 7500 10000

Feet

PROJECT NUMBER: COTOW08	DRAWN BY: RJN
FILENAME: Windsor-masterplan.apr Future Condition - Detention 11x17	DESIGNED BY: CJP
DATE: 10/02/2003	CHECKED BY: BAA

Anderson Consulting Engineers, Inc.
Civil • Water Resources • Environmental

TOWN OF WINDSOR
Master Drainage Plan

FUTURE CONDITION
DETENTION (100-100, 10-10)
100-YEAR PEAK DISCHARGES

FIGURE 4.3

attenuation of the stormwater runoff typically results in minor increases in the peak discharges along the Law Basin's major drainageways when compared to the existing conditions. For example, peak discharges downstream of State Highway 392 increased approximately 500 to 1,000 cfs (10 to 20 percent). Although, the peak discharge may not increase significantly, the volume of the stormwater runoff will increase, thereby, significantly increasing the duration of flooding.

Given the information discussed in the previous paragraph, the potential flooding problems identified in the existing condition hydrologic analysis will be slightly exacerbated. Consequently, the flooding problems will continue to persist and be slightly greater in magnitude at the following locations:

Main Channel

- Eastman Park Drive culvert crossing
- Colorado and Southern Railroad crossing
- State Highway 392 crossing
- Greeley No. 2 Canal crossing

West Tributary

- Great Western Railroad crossing
- State Highway 392 crossing
- WCR 19 crossing
- Greeley No. 2 crossing
- State Highway 257 crossing
- WCR 72 crossing
- WCR 74 crossing

4.7.4 Windsor Basin: Summary of Future Condition Drainage Problems

For the Windsor Basin, the impact of future development on the stormwater runoff generated during major storm events was minimal. The lower portion of the basin is almost fully developed by in the existing condition. The upper portion of the Windsor Basin, where potential land development is significant, noted a flow increase of approximately 400 cfs into Kern Reservoir. In general, the flood problems presently experienced within the upper portion of the Windsor Basin will be slightly exacerbated with future development. As stated previously, the increase in flood peaks is attributable to the placement of on-site detention ponds which may extend the duration of the peak discharge.

In summary, those structures within the Windsor Basin that experienced potential flooding problems during the existing condition analysis continue to experience flooding problems during the

future conditions analysis. In the upper basin, flooding problems will be slightly increased at the following locations:

- State Highway 257 crossing
- Greeley No. 2 Canal crossing
- WCR 15 crossing

4.7.5 High School Basin: Summary of Future Condition Drainage Problems

The High School Basin reflects a fully developed basin in the existing condition; therefore, no increase in peak discharges were observed during the future conditions analysis. The flooding problems identified previously will continue to persist assuming future land use conditions.

4.7.6 Jacoby Basin: Summary of Future Condition Drainage Problems

For the future conditions, peak discharges in the Jacoby Basin generally remained the same. A slight reduction in discharges is evident due to the delay attributable to on-site detention associated with future development in the upper basin. In general, flooding problems will continue to persist and be similar in magnitude at the following locations:

- State Highway 392 crossing structure
- Greeley No. 2 Canal crossing

4.7.7 Timnath Reservoir Basin: Summary of Future Condition Drainage Problems

The peak discharge data for the Timnath Reservoir Basin increased slightly (less than 200 cfs) during the hydrologic analysis of future land use conditions. Consequently, flooding problems will continue to persist, and be slightly increased, at the following locations:

- LCR 32½ crossing structure
- Greeley Canal No. 2 crossing
- LCR 36 bridge structure
- Colorado & Southern Railroad bridge structure

4.7.8 Ptarmigan Basin: Summary of Future Condition Drainage Problems

In the Ptarmigan Basin, future condition peak discharges were generally lower than in the existing condition. In the existing condition, the Ptarmigan Basin reflects partial development of the tributary watershed. As development occurs in this basin, over-detention was assumed, and is consistent with the detention requirements associated with the existing development. As indicated previously, the over-detention requirement was adopted to meet the available capacity of the existing drainage facilities at State Highway 392. With this assumption, the peak discharges associated with the future land use conditions are reduced compared to the peak discharges for the existing condition. Consequently, flooding problems are eliminated at the following locations:

- Subbasin P11 culvert under State Highway 392
- Subbasin P5 culvert under State Highway 392

Flooding problems continue to persist at the Subbasin P14 culverts under State Highway 392.

4.7.9 River Ridge Basin: Summary of Future Condition Drainage Problems

Discharge data in the major drainageway of the River Ridge Basin were less than 10% higher for the future land use conditions when compared to existing condition discharges. The minor increase in discharges is attributed to the on-site detention ponds, the longer duration of the peak flows out of the ponds, and coinciding of the peak discharges as stormwater is conveyed through the basin. The minor increases in the magnitude of the peak discharge data reflects a small increase in potential flooding at the following locations:

- River Edge Road culvert crossing
- River West Drive culvert crossing

4.7.10 Bluff Basin: Summary of Future Condition Drainage Problems

As stated previously, major drainageway channels do not exist in the Bluff Basin. Each subbasin directly contributes runoff into the Cache la Poudre River. Consequently, the peak discharge data generated from the hydrologic analysis of future land use conditions (with on-site detention) is identical to the peak discharges from the existing condition analysis. It is noted that no flooding problems were identified for the Bluff Basin during the hydrologic analysis of existing conditions.

4.7.11 Oklahoma Basin: Summary of Future Condition Drainage Problems

In the Oklahoma Basin, only a slight increase (less than 5%) in future conditions peak discharge data were observed along the major drainageway when compared with the existing condition results. The flooding problems identified previously will continue to persist, and be slightly increased in magnitude, at the following locations:

- State Highway 157 bridge structure
- Great Western Railroad culvert crossing

4.7.12 South State Highway 257 Basin: Summary of Future Condition Drainage Problems

Future land use plans in the South State Highway 257 Basin were not available during this master planning effort although development is presently encroaching into the upper portion of the basin adjacent to US Highway 34. Given the lack of available information, major drainageway peak discharges in the future are assumed to remain the same as the peak discharge data for the existing condition. The flooding problems at the State Highway 257 crossing structure will continue to persist.

4.8 Summary of Future Conditions with Over-Detention

4.8.1 Definition of Future Conditions with Over-Detention

Following the completion of the hydrologic analysis associated with future land use conditions, drainage improvements were identified and evaluated to mitigate the flooding problems identified within each drainage basin. Conceptual level cost estimates were prepared for the drainage improvements and presented to the Town staff, Water and Sewer Board and Board of Trustees. The preliminary cost estimate associated with the drainage improvements exceeded $22 million. Given the magnitude of the cost estimate along with the potential increase associated with the construction of the improvements on the existing stormwater fees, an additional scenario of hydrologic conditions was investigated. This scenario assumed the construction of on-site detention ponds associated with future development within each basin. The on-site detention ponds assumed 100-year future condition flows were detained and released at the 10-year existing condition discharge rate (i.e., over-detention requirements). As stated previously, the only exception to this detention criteria is in

the Ptarmigan Basin where release rates have been previously determined based on the capacity of the existing structures. All other modeling parameters remained identical to the hydrologic analysis of future land use conditions (see Table 4.4).

The future condition with over-detention scenario has a couple of inherent implementation issues that need to be addressed and recognized. Construction of improved major drainageway facilities, based on the over-detention criteria, will only provide 10-year storm protection until the basin realizes full development. At the point in time a basin is fully developed, the entire on-site detention pond system will be operational and 100-year protection will be afforded by the drainage improvements. As development occurs within the contributing watershed, the protection provided by the drainage improvements will increase from 10-year protection (that protection provided under the existing conditions within the watershed) to 100-year protection (provided following full development of the tributary watershed with over-detention). Additionally, since this master planning effort encompasses several jurisdictional boundaries, detention requirements associated with these jurisdictions should be discussed and informal agreements, as a minimum, entered into to ensure over-detention is provided for those areas outside the jurisdiction of the Town of Windsor. It should be noted that Weld County, Larimer County and the Town of Severance presently require future development to incorporate detention ponds assuming 100-year future condition flows are detained and released at the 10-year or 2-year existing condition discharge rate. The Town of Timnath is presently developing stormwater criteria that also includes a requirement to over-detain stormwater runoff associated with future development.

4.8.2 Modeling Results

The results of the hydrologic modeling of future conditions with over-detention for each subbasin are presented in Appendix B.3. Hydrologic modeling results for specific locations along the major drainageways are presented in Table 4.6. Given the assumptions related to over-detention within the drainage basins, the results generally reflect a decrease in the peak runoff throughout the basin. Figure 4.4 presents the 100-year peak discharge data at several locations within the watershed based on the hydrologic model of future conditions with over-detention.

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Future Conditions □ith □ver□Detention.

□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr□	1□ yr□
LAW BASIN (MAIN CHANNEL)								
Cache la Poudre River	L701	18,671	880	1,282	1,541	2,062	2,458	2,938
Eastman Park Dr.	L206	16,331	308	451	548	967	1,481	2,148
C&S Railroad	L208	16,135	294	432	523	940	1,454	2,123
State Highway 392	L225	11,012	42	61	75	185	371	697
Greeley No. 2 Canal/WCR 21	L226	10,778	35	52	63	166	344	667
WCR 70	L227	10,236	25	37	44	125	288	604
WCR 70 ½	L229	9,425	15	23	27	74	240	509
Law Reservoir Outflow	L340	8,084	0	0	0	36	169	362
Law Reservoir Inflow	L240	8,084	53	79	95	248	468	762
Loop Lake Outflow	L341	5,713	0	0	0	0	0	346
Loop Lake Inflow	L241	5,713	78	116	141	403	811	1,349
State Highway 14/WCR 82	L442	2,986	32	46	55	207	407	665
LAW BASIN - WEST TRIBUTARY								
State Highway 392 & Great Western Railroad	L210	4,676	242	355	429	724	1,033	1,427
WCR 19	L211	3,929	210	310	373	640	922	1,283
Basin L11 Concentration Point	L711	3,929	211	311	375	643	926	1,289
Greeley No. 2 Canal	L715	3,577	195	287	346	599	868	1,215
Basin L12 Inflow Downstream of State Highway 257*	L712	2,522	298	368	413	595	776	1,007
Ventana Way*	L118	1,707	254	303	334	465	595	762
WCR 72*	L219	1,460	230	268	291	396	498	629
WCR 74*	L220	793	195	217	230	289	349	428
Basin L20 Inflow	L820	793	45	67	80	139	199	278
State Highway 257 (WCR 17)	L812	815	46	67	81	133	183	247
WINDSOR BASIN – LOWER								
Cache la Poudre River	W700	4,208	554	789	945	1,323	1,596	1,921
Water Valley Parkway	W204	3,784	58	85	118	243	392	600
Eastman Park Drive East of 1st St.	W220	3,553	26	45	70	147	226	337
Folkstone Park Pond	W323	3,454	20	35	56	120	184	273
Garden Drive	W723	3,454	120	178	216	329	419	528
Chestnut Street	W223	3,392	105	156	186	260	335	423
Chimney Park	W128	3,158	51	78	96	139	169	205
Walnut Street	W229	3,116	30	46	56	79	95	115
C&S Railroad	W240	3,089	18	26	31	49	57	59
WINDSOR BASIN – UPPER								
Kern Reservoir Outflow	W341	3,043	18	26	31	49	57	59
Kern Reservoir Inflow	W241	3,043	579	867	1,061	1,452	1,704	1,994
Upstream of Greeley No.2 Canal/State Highway 257	W142	2,713	145	217	263	455	639	867
WCR 15	W243	2,322	134	198	240	420	592	809
Basin W44 Outlet Point	W144	913	58	86	103	184	258	350
Lake Canal	W244	913	59	87	104	187	262	356
State Highway 68/WCR 74	W245	609	45	65	78	143	201	275
WINDSOR BASIN – LOWER MISCELLANEOUS								
Eastman Park Dr. at 1st St.	W715	262	16	25	34	71	111	162
1st St. Flows	W124	234	12	19	27	57	92	136
Chestnut St. Flows	W924	234	58	85	103	142	190	238
1st St. at Chestnut St.	W224	234	82	124	153	222	290	381

*Discharge values include assumed irrigation base flow of 150 cfs.

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Future Conditions □ith □ver□Detention (continued).

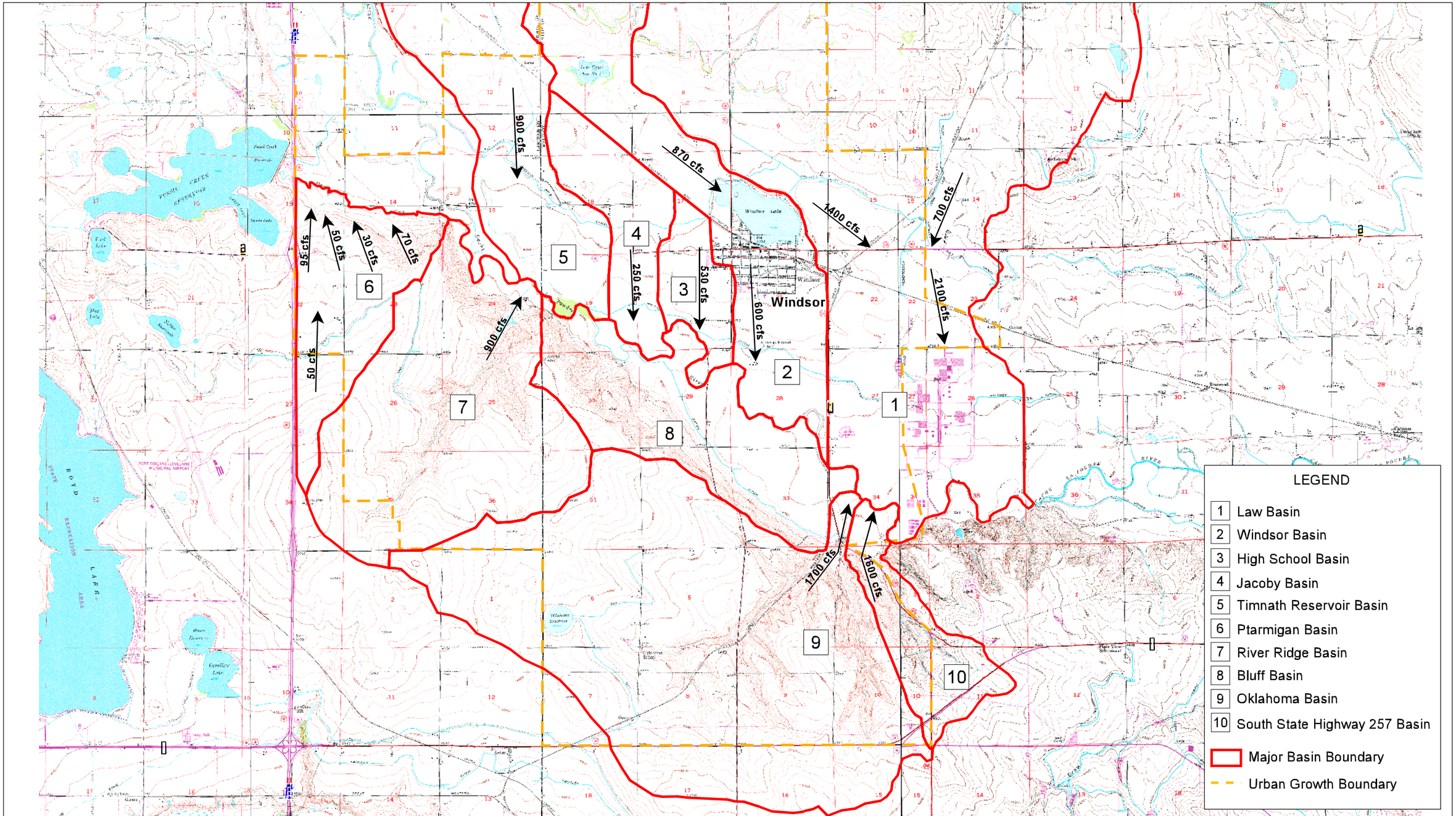
□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr□	1□ yr□
WINDSOR BASIN – LOWER MISCELLANEOUS (continued)								
Oak St. Flows	W130	197	62	91	116	178	248	328
Elm St. at 7th St.	W131	77	33	51	62	85	114	123
Eastman Park Dr. at 54-inch Storm Sewer	W710	162	22	32	38	78	115	161
Eastman Park Dr. at 3rd St.	W210	109	72	112	137	194	238	295
HIGH SCHOOL BASIN								
Cache la Poudre River	H700	558	191	288	353	548	720	942
Whitney Ditch – East	H205	302	116	172	206	307	405	527
Stone Mountain Dr. – East	H206	229	121	178	214	309	392	494
State Highway 392 at High School	H209	103	6	8	10	15	19	24
Greeley No. 2 Canal	H410	35	2	3	3	5	6	8
Whitney Ditch – West	H212	150	106	150	180	251	310	381
Stone Mountain Dr. – West	H214	129	86	122	146	202	248	302
State Highway 392 at 12th Street	H315	11	2	2	3	6	8	11
JACOBY BASIN								
Cache la Poudre River	J700	1,016	101	145	175	247	297	356
Whitney Ditch	J205	871	16	23	31	61	95	205
Walnut Street	J206	777	6	8	10	14	36	120
State Highway 392	J707	692	49	73	88	158	221	301
State Highway 392 East Path	J207	195	19	28	34	67	95	132
Greeley No. 2 Canal East Path	J208	135	11	16	20	37	53	73
State Highway 392 West Path	J210	496	31	45	55	93	127	171
Greeley No. 2 Canal West Path	J211	457	26	38	46	74	101	135
72-inch Storm Sewer	J500	692	49	73	88	158	200	200
TIMNATH RESERVOIR BASIN								
Cache la Poudre River	T201	14,989	415	536	611	760	880	1,024
State Highway 392	T204	14,466	292	359	403	563	713	904
LCR 32E	T205	14,286	292	360	404	564	713	897
Greeley No. 2 Canal	T206	14,044	286	350	392	534	665	823
C&S Railroad & LCR 36	T207	12,988	240	283	311	405	490	593
LCR 38	T208	12,451	205	232	250	313	372	443
Timnath Reservoir Outlet	T320	11,300	160	167	174	199	225	250
Inflow to Timnath Reservoir	T720	11,300	753	1,083	1,285	1,926	2,495	3,331
PTARMIGAN BASIN								
Fossil Creek	P701	1,245	91	129	154	224	280	348
Basin P02 Outlet	P202	182	124	176	211	301	376	466
State Highway 392	P303	71	23	27	30	41	53	68
Basin P04 Outlet	P204	380	89	126	150	215	270	337
State Highway 392 at REA Parkway	P205	299	6	8	10	17	22	28
Basin P07 Outlet	P207	865	42	62	78	131	171	222
Downstream of State Highway 392 & LCR 5	P711	311	9	14	18	30	40	52
State Highway 392 at LCR 5	P209	52	8	12	15	24	31	40
State Highway 392 at Country Meadows	P211	259	2	3	4	10	16	22
Shutts	P212	175	1	2	3	9	13	19

*Discharge values include assumed irrigation base flow of 150 cfs.

Table 4.□ Ma□or Drainage□ay Pea□Discharges for Future Conditions □ith □ver□Detention (continued).

□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr□	1□ yr□
PTARMIGAN BASIN (continued)								
State Highway 392 at I-25	P214	339	15	23	30	53	72	95
Inflow to Westgate	P215	298	8	11	16	35	50	69
LCR 30 at I-25 Frontage Road	P416	212	2	3	6	21	34	50
RIVER RIDGE BASIN								
Cache la Poudre River	R700	3,751	212	300	359	547	745	991
River West Drive	R104	3,417	145	214	265	494	678	906
Basin R04 Concentration Point	R704	3,417	147	216	266	495	680	910
Basin R06 Inflow	R706	2,859	102	149	185	352	482	642
Basin R07 Concentration Point	R207	2,000	70	103	126	242	333	447
Basin R08 Inflow	R408	1,127	42	61	74	137	191	258
BLUFF BASIN								
No ma□or drainage□ay for this subbasin. See subbasin pea□runoff table (Appendix B.□) for results.								
OKLAHOMA BASIN								
Cache la Poudre River	O701	7,264	260	377	456	940	1,298	1,699
State Highway 257	O201	7,189	259	377	455	942	1,302	1,699
Great Western Railroad	O703	6,017	249	362	436	865	1,153	1,469
Basin O04 Concentration Point	O704	5,785	246	357	430	851	1,125	1,427
Basin O05 Concentration Point	O705	5,022	218	316	380	770	1,010	1,273
Basin O07 Concentration Point	O707	3,774	163	237	284	607	779	981
WCR 17 Pond Outflow	O308	3,040	151	219	263	552	688	849
WCR 17 Pond Inflow	O109	3,040	165	243	291	633	929	1,308
Basin O08 Inflow Concentration Point	O208	3,040	166	243	292	635	933	1,313
Basin O09 Inflow	O809	1,637	111	163	197	411	603	851
SOUTH STATE HIGHWAY 257 BASIN								
□ver□detention not modeled in this basin□ ma□or drainage□ay pea□discharges same as in future conditions.								

*Discharge values include assumed irrigation base flow of 150 cfs.



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FILENAME: Windsor-masterplan.apr Future Condition - Over Detention 11x17	DESIGNED BY: CJP
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Anderson Consulting Engineers, Inc.
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TOWN OF WINDSOR
Master Drainage Plan

FUTURE CONDITION
OVER-DETENTION (100-10)
100-YEAR PEAK DISCHARGES

FIGURE 4.4

4.8.3 Law Basin: Summary of Future Condition with Over-Detention

In the Law Basin, a reduction of 50% or more was observed when over-detention was applied to the future condition peak discharges. The peak discharge to the Cache la Poudre River was reduced from approximately 6,200 cfs in the future condition to approximately 2,900 cfs in the future condition with over-detention. In the over-detention scenario, flooding problems still persist at the locations indicated below given the limited capacity of the existing facilities to convey stormwater runoff. The magnitude of the flooding problems is greatly reduced by the over-detention criteria associated with future land development.

Main Channel

- Eastman Park Drive culvert crossing
- Colorado and Southern Railroad crossing
- State Highway 392 crossing
- Greeley No. 2 Canal crossing

West Tributary

- Great Western Railroad crossing
- State Highway 392 crossing
- WCR 19 crossing
- Greeley No. 2 crossing
- State Highway 257 crossing
- WCR 72 crossing
- WCR 74 crossing

Flooding problems were minimized or eliminated at the crossings of WCR 72 and WCR 74.

4.8.4 Windsor Basin: Summary of Future Condition with Over-Detention

In the lower portion of the Windsor Basin (downstream of the Kern Reservoir), the peak discharge data is not significantly changed and reflects the extent of development south of Kern Reservoir. The upper portion of the Windsor Basin, however, realized a significant reduction (in excess of 50%) in peak discharges through the placement of detention ponds with the revised release criteria. As indicated previously, the upper portion of the Windsor Basin is largely undeveloped and the potential to reduce runoff through over-detention becomes increasingly important. While the peak discharge from major storm events is lowered in the upper Windsor Basin, flooding problems continue to persist at State Highway 257 crossing structure and the crossing of the Greeley No.2

Canal. The potential for flood damages and overtopping at the crossing of WCR 15 was reduced to a level that very nearly satisfies the criteria for overtopping roadways in the Town of Windsor Storm Drainage Design Criteria Manual (Engineering Professionals, Inc., 1990).

4.8.5 High School Basin: Summary of Future Condition with Over-Detention

Given the extent of development within the High School Basin, no change in peak discharges were observed during the hydrologic analysis of future land use conditions with over-detention.

4.8.6 Jacoby Basin: Summary of Future Condition with Over-Detention

In the Jacoby Basin, future development of the upper portion of the basin resulted in a reduction in the peak discharges from the major storm events. The existing 72-inch storm sewer outfall south of State Highway 392 has a capacity of approximately 200 cfs. Assuming development of the upstream portion of the basin with over-detention criteria, the existing storm sewer receives a peak discharge of approximately 170 cfs. Stormwater runoff conveyed to the Greeley No. 2 Canal is also reduced to potential inflows of approximately 70 cfs east of WCR 15 and 135 cfs west of WCR 15. It should be noted that the 135 cfs west of WCR 15 is assumed to spill over the Greeley No. 2 Canal and is conveyed to State Highway 392 and ultimately to the 72-inch storm sewer outfall. Similarly, the 70 cfs east of WCR 15 is assumed to spill over the Greeley No. 2 Canal and is conveyed to State Highway 392. Based on these results, potential flooding problems are reduced but continue to persist along the Greeley No. 2 Canal.

4.8.7 Timnath Reservoir Basin: Summary of Future Condition with Over-Detention

The peak discharges generated during the major storm events in the lower portion of the Timnath Reservoir Basin (below Timnath Reservoir) decreased approximately 60% for this detention scenario. Similarly, this detention scenario in the upper portion of the Timnath Reservoir Basin produced a substantial reduction in peak discharge. It is noteworthy, however, that the outflow from the Timnath Reservoir remained relatively constant (260 cfs to 250 cfs) for all hydrologic modeling scenarios. Therefore, it appears that future development with detention in the upper Timnath Reservoir Basin will have little impact on storm water planning efforts in the lower portion of the Timnath Reservoir Basin. Given the results with this detention scenario:

- Flooding problems continue to persist due to the capacity of the existing crossing structure at LCR 32½.
- The capacity of the Greeley No. 2 Canal is limited and spills continue to occur at this location.
- The capacity of the crossing structures located along LCR 36 and the Colorado & Southern Railroad eliminated the flooding problems at these locations.

4.8.8 Ptarmigan Basin: Summary of Future Condition with Over-Detention

In the Ptarmigan Basin, peak discharges are lower with this detention scenario. Over-detention applied to undeveloped lands in Subbasin P16 reduces but does not eliminate the potential flooding problem along the Subbasin P14 corridor. As stated previously, the combined capacity of the existing culverts at this location prior to overtopping State Highway 392 is approximately 50 cfs. The future condition with over-detention peak discharge for the 100-year storm event at this location is estimated to be 95 cfs.

4.8.9 River Ridge Basin: Summary of Future Condition with Over-Detention

Assuming the detention requirements with this hydrologic scenario, the peak discharges are reduced by approximately 75% when compared with the future condition discharges in the River Ridge Basin. Flooding problems at River Edge Road and River West Drive are eliminated.

4.8.10 Bluff Basin: Summary of Future Condition with Over-Detention

As expected, stormwater from each subbasin in the future condition with over-detention will be significantly reduced compared to the peak discharges from the future condition.

4.8.11 Oklahoma Basin: Summary of Future Condition with Over-Detention

In the Oklahoma Basin, reductions in major drainageway peak discharges of approximately 65% were observed when the results of the future condition with over-detention scenario were compared with the future condition results. Flooding problems at State Highway 257 are eliminated but flooding problems continue to persist at the crossing structure associated with the Great Western Railroad. As indicated previously, the capacity of the Great Western Railroad crossing structure is

770 cfs. The future condition with over-detention peak discharge for the 100-year storm event at this location is estimated to be 1,470 cfs.

4.8.12 South State Highway 257 Basin: Summary of Future Condition with Over-Detention

Similar to the future condition scenario, major drainageway peak discharges assuming future conditions with over-detention are assumed to remain the same as the peak discharge data for the existing conditions. Consequently, a flooding problem continues to persist at the crossing of State Highway 257. As indicated previously, the culvert capacity at this location is 1,075 cfs prior to overtopping the highway. The peak discharge for the 100-year storm event is estimated to be 1,570 cfs.