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TOWN OF WINDSOR
MASTER DRAINAGE PLAN
WELD COUNTY, COLORADO

PREPARED FOR:

Town of Windsor
Engineering Department
301 Walnut Street
Windsor, CO 80550

PREPARED BY:

Anderson Consulting Engineers, Inc.
772 Whalers Way, Suite 200
Fort Collins, CO 80525
(ACE Project No. COTOW08)

October 3, 2003

Anderson Consulting Engineers, Inc
Civil • Water Resources • Environmental

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I. INTRODUCTION

The Town of Windsor, Colorado is presently experiencing rapid growth and development within the boundaries of several of its major drainage basins. As growth continues within these basins, the threat associated with flooding also grows. Flooding problems that presently exist will likely be exacerbated as future development occurs within these basins. Consequently, the construction of adequate storm drainage facilities that mitigate both existing and future flooding problems becomes increasingly important.

In support of the need to provide these facilities, the Town of Windsor contracted with Anderson Consulting Engineers, Inc. to develop a master drainage plan for those major drainage basins located within its urban growth management boundary. This report summarizes the results of the master planning efforts for these basins.

1.1 Project Goals and Objectives

The goal of this project was to develop a Master Drainage Plan (MDP) that can be utilized as a tool for making decisions related to stormwater management within the boundary of the Town of Windsor and the associated drainage basins. Completion of the master drainage plan involved the development of a planning document that would:

(a) identify long-term capital improvements and rehabilitation measures for the existing drainage system;

(b) be a tool for implementation of future improvements associated with new developments within the basin boundaries;

(c) provide a basis for prioritizing and scheduling required improvements (implementation plan);

(d) provide the flexibility to implement improvements that afford flood protection while being cost effective;

(e) address water quality issues;

(f) identify uniform criteria for the planning and design of major drainageway facilities and on-site detention requirements; and

(g) provide the basis for development of basin impact fees to fund construction of the stormwater capital improvement projects.

Sensitivity to these objectives was an important consideration during the preparation of the master drainage plan; however, the primary focus of the planning efforts was the reduction of both existing flooding and the potential for future flooding problems within the Town of Windsor.

1.2 Scope of Work

The scope of work followed during the development of the master drainage plan included the tasks described below.

1. Scoping and Coordination Meetings. This task included a scoping meeting to discuss project goals and objectives, schedule, deliverables, modeling criteria and level of detail, and planning concepts and evaluation procedures. Coordination meetings were also conducted to discuss project status, overall direction of the project and promote the exchange of information. Several workshops were held with members of the Town Board of Trustees and the Town Water and Sewer Board. During the coordination meetings, the scope of the project was often slightly modified to reflect information and results generated during the completion of the planning effort.

2. Data Collection and Field Investigation. A detailed review of all available reports, mapping and data pertinent to the planning efforts was conducted. This information included but was not limited to: (a) existing drainage criteria and policies; (b) existing and ongoing drainage studies; (c) zoning data; (d) pertinent soils, rainfall and runoff data; (e) inventories of existing facilities; (f) available mapping and utility databases; and (g) agreements with various entities including ditch companies and Weld County. In conjunction with the collection and review of available data, a field reconnaissance was conducted to further define and verify locations of existing drainage or flooding problems and formulate plans for conceptual improvements at these locations. Survey requirements were identified along with all existing facilities earmarked for evaluation during the hydraulic evaluation.

3. Mapping and Surveying. All available mapping sources were investigated. These sources included USGS quadrangle maps and mapping generated from existing and ongoing drainage studies. To facilitate the master planning efforts, aerial photography was utilized from a 1999 aerial flight. Survey requirements included the collection of data necessary to define existing structures (bridges, channels, culverts, outlet structures, etc.) with respect to geometric configuration, invert and overtopping elevations, etc.

4. Inventory of Existing Facilities. All information related to the major storm drainage facilities within the major drainageway of each basin was compiled. Existing detention facilities, culverts, bridges, road crossings, drainage channels, irrigation ditches, and major storm sewers were inventoried and evaluated. The capacity of

each structure was determined and the feasibility of integrating existing facilities into the master drainage plan investigated.

5. Hydrologic Analysis and Modeling. A detailed hydrologic model was developed and utilized to simulate the stormwater runoff for four scenarios: (a) existing or "as is" condition; (b) future development conditions with existing facilities and on-site detention; (c) future development conditions with existing facilities and on-site over-detention; and (d) future development conditions with on-site over-detention and proposed improvements.

6. Hydraulic Analysis and Modeling. A detailed hydraulic analysis was completed to evaluate the structures identified during the inventory of existing facilities. All proposed improvements were also evaluated during the hydraulic analysis and modeling efforts.

7. Drainageway Planning and Alternative Development. Storm drainage improvements were identified to mitigate the existing and potential future flood hazard problems. Alternative stormwater management plans were conceptually developed and internally reviewed and refined with input from the staff of the Town of Windsor and the Town Boards.

8. Basin Drainage Criteria. Criteria for the planning and design of drainage facilities were developed in conjunction with the staff of the Town of Windsor Engineering Department. The criteria specifically focused on the planning and design of major drainageway facilities, road crossings and on-site detention requirements.

9. Funding/Fee Assessment. An investigation of various funding mechanisms that could provide monies for the construction of the capital improvements was completed. Proposed modifications to the existing stormwater fee structure were developed. The results of the investigation along with a recommendation were presented to the Town Board of Trustees and Town Water and Sewer Board.

10. Reporting and Technical Documentation. The results of the master planning effort are summarized in this report. All recommended improvements are identified along with conceptual design drawings and detailed cost estimates. An implementation plan for the construction of the proposed improvements is presented. Documentation in the form of technical appendices and a project notebook is provided to allow replication of the steps taken to: (a) generate the design flows, (b) estimate the capacity of the existing and future facilities, (c) develop and evaluate the alternative plans, and (d) generate the cost estimates.

1.3 Acknowledgments

The preparation of this report along with the supporting documentation involved the dedicated efforts of several representatives of the Town staff, members of the Town Board of Trustees, Town Water and Sewer Board, Town Planning Commission as well as members of the community potentially impacted by the implementation of a master drainage plan. The assistance and cooperation of those entities and individuals identified below are acknowledged and greatly appreciated.

Town Board of Trustees (Mr. Wayne Miller, Mayor)
Town Water and Sewer Board Members (Mr. Kenton Brunner, Chairman)
Town Planning Commission (Mr. Gale Schick, Chairman)
Town of Windsor Staff:
- Mr. Rod Wensing, Administrator
- Mr. Dennis Wagner, Director of Engineering
- Mr. Terry Walker, Director of Public Works
- Mr. Joe Plummer, Director of Planning
- Mr. Dean Moyer, Director of Finance
- Mr. Rick Anderson, Civil Engineer

Mr. John Frey; Frey, Korb, Haggerty & Michaels, P.C. (Town Attorney)

In addition to those individuals and entities listed above, members and representatives of the Eastman Kodak Company, New Cache la Poudre Irrigating Company, Cache la Poudre Reservoir Company; Trollco, Inc. dba Water Valley, TST, Inc., Hall-Irwin, Inc., RMC, Inc., Office of the State Engineer, Weld County Engineering and Planning Departments, and the Colorado Department of Transportation provided invaluable information and details related to the existing facilities, historic flooding, and existing and proposed urban developments within the basins. The information provided by these individuals and entities is also gratefully acknowledged.

1.4 Mapping and Surveying

Digital USGS quadrangle mapping was utilized during this master planning effort. These maps were combined to form a single, contiguous topographic base map that encompassed the drainage boundaries of the major drainage basins. The following USGS quadrangle maps were obtained:

(a) Bracewell, Colorado, 1950, Photo revised 1980;
(b) Cobb Lake, Colorado, 1960, Photo revised 1978;
(c) Eaton, Colorado, 1960, Photo inspected 1975;
(d) Greeley, Colorado, 1950, Photo revised 1980;
(e) Nunn, Colorado, 1960, Photo inspected 1975;
(f) Severance, Colorado, 1960, Photo revised 1971;
(g) Timnath, Colorado, 1960, Photo revised 1971; and
(h) Windsor, Colorado, 1950 , Photo revised 1969.

All the quadrangles utilized the NAD29 Vertical Datum and incorporated a contour interval of 10 feet.

In addition to the USGS mapping, digital aerial photographic images were obtained from the Town of Windsor and utilized to illustrate the location of all proposed improvements. Ken Rushing Photographic Services of Drake, Colorado, provided the September 22, 1999 aerial photography to the Town of Windsor.

King Surveying, Inc. of Windsor, Colorado, provided additional surveying and topographic data necessary to complete the project.

1.5 Previous Studies

Previous studies related to stormwater management within the Town of Windsor were collected and reviewed during the completion of this project. Reports documenting stormwater drainage conditions and improvements in the Town of Windsor may be found in the reference section of the project notebook. The information gathered from these reports, including the available design drawings and specifications, were evaluated and utilized during the completion of this master planning effort.

With respect to flood studies within the basins, the Federal Emergency Management Agency (FEMA) has produced flood information and mapping along the Cache la Poudre River and the Law Ditch. Furthermore, FEMA has documented flooding in the Town of Windsor in the following reports:

(a) Flood Insurance Study, Town of Windsor, Colorado, Weld County, September 27, 1991; and

(b) Flood Insurance Study, Weld County, Colorado, Unincorporated Areas and Town of Eaton, Colorado; Revised September 27, 1991.

II. BASIN CHARACTERISTICS

2.1 General Information

The Town of Windsor is located in northwest Weld County approximately 60 miles north of the Denver metropolitan area. The Town is contained within a watershed that can generally be defined by Weld County Road 100 (WCR 100) on the north, Weld County Road 25 (WCR 25) on the east, U.S. Highway 34 on the south and Interstate 25 on the west. The watershed encompasses approximately 120 square miles. The primary land use is agricultural, ranching, and open space that comprises less than 67,000 acres or approximately 87% of the watershed. Less than 10% of the watershed is currently developed and of the area that is developed, the land use ranges from very low density to mixed-use residential housing and light to medium commercial/industrial development.

Several jurisdictional entities are responsible for regulating growth and development within the watershed. These entities include:

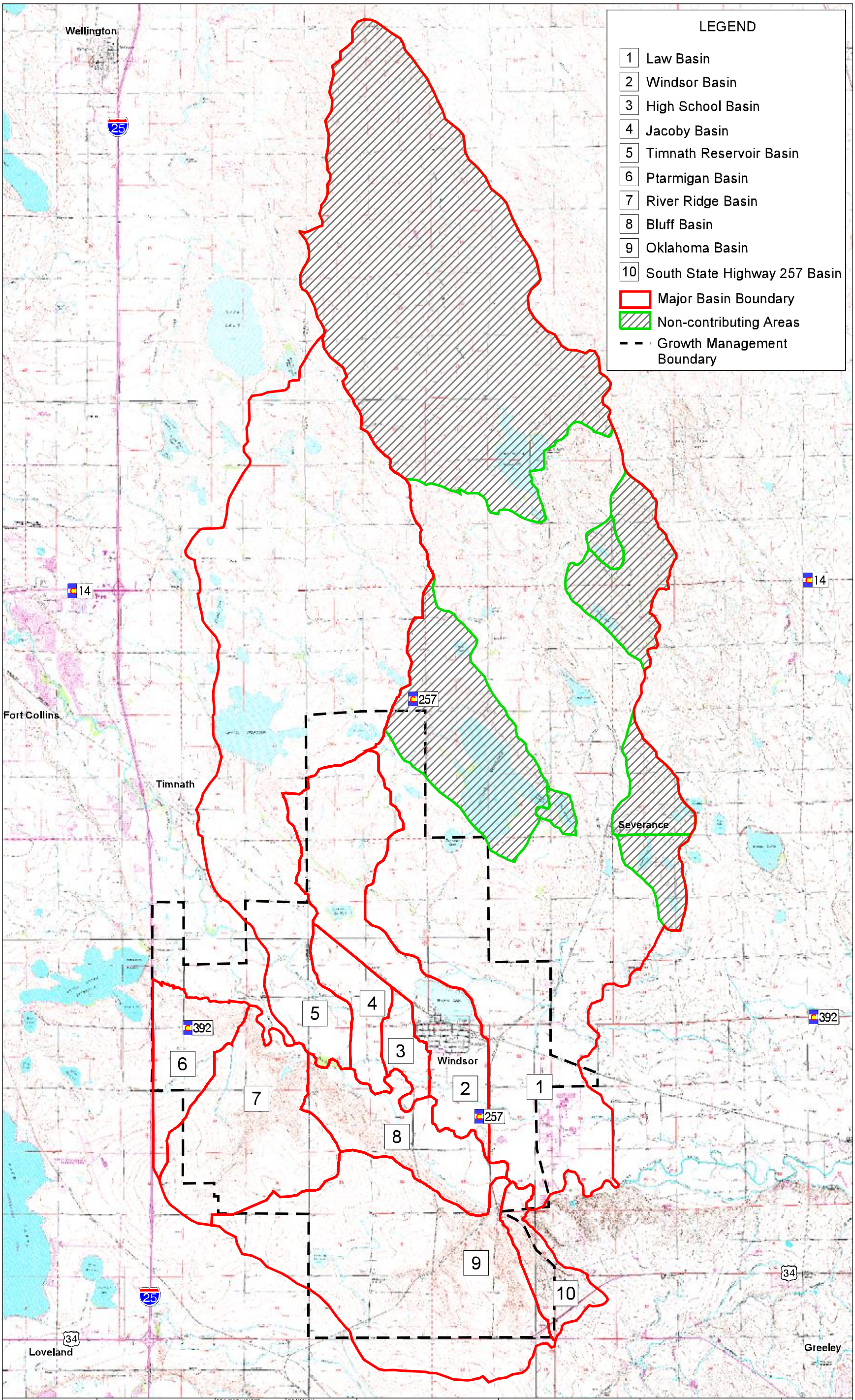
- Town of Windsor (~9,500 acres or 12.5%),
- Weld County (~53,200 acres or 70.0%),
- Larimer County (~11,400 acres or 15.0%),
- Town of Severance (~1,200 acres or 1.5%),
- City of Loveland (~700 acres or 0.9%), and
- City of Greeley (~90 acres or 0.1%).

Figure 2.1 provides a vicinity map illustrating the watershed boundary along with the Growth Management Boundary for the Town of Windsor.

2.2 Summary of Major Drainage Features

For the purposes of this master planning effort, the watershed was divided into ten major drainage basins. The major drainage basin boundaries are delineated on the vicinity map on Figure 2.1 and identified below:

> Law Basin	> Ptarmigan Basin
> Windsor Basin	> River Ridge Basin
> High School Basin	> Bluff Basin
> Jacoby Basin	> Oklahoma Basin
> Timnath Reservoir Basin	> South State Highway 257 Basin



REVISIONS:	N Contour Interval 10 feet 2000 0 2000 4000 Feet	PROJECT NUMBER: COTOW08	DRAWN BY: RJN	Anderson Consulting Engineers, Inc.	TOWN OF WINDSOR Master Drainage Plan	VICINITY MAP	FIGURE 2.1
		FILENAME: Windsor-masterplan.apr Vicinity Map 11x17	DESIGNED BY: CJP				
		DATE: 10/02/2003	CHECKED BY: BAA				

The Cache la Poudre River, which defines the southern boundary of the Law, Windsor, High School, Jacoby, and Timnath Basins and the northern boundary of the River Ridge, Bluff, Oklahoma, and South State Highway 257 Basins, represents the most notable drainage feature in the watershed. Fossil Creek represents the northern boundary and receiving watercourse for the Ptarmigan Basin. All stormwater runoff generated within the watershed is ultimately conveyed to the Cache la Poudre River. The 100-year floodplain associated with the Cache la Poudre River encompasses less than 5 percent (950 acres) of the total drainage area within the watershed.

Several irrigation facilities also traverse the ten major basins including:

- Greeley No. 2 Canal
- Lake Canal
- Law Ditch
- Whitney Ditch
- Timnath Reservoir Outlet
- Windsor Reservoir Outlet
- Louden Ditch
- Eaton Ditch

These ditches convey irrigation flows within the basin but generally offer limited value as drainage features that will convey stormwater runoff out of the basin. Due to the limited capacity of the ditch systems, coupled with the magnitude of the stormwater flows, these irrigation ditches do not appear to represent major drainage boundaries within the watershed.

The remainder of this chapter provides information related to the major drainage features within each of the ten basins. This information is generally limited to a description of the main flow paths including major crossings.

2.2.1 Law Basin

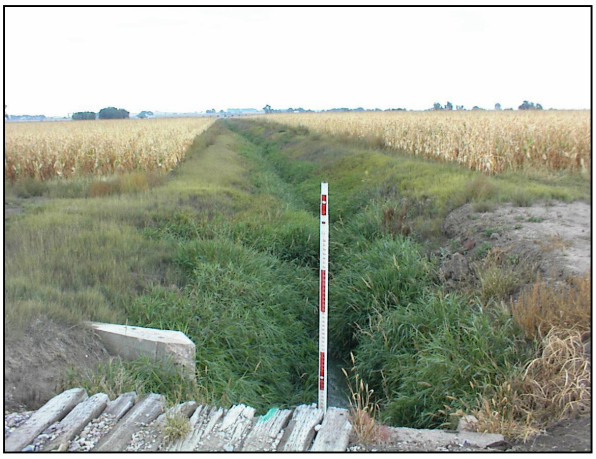
The Law Basin consists of a predominantly rural watershed with a well-defined major drainage channel. The major drainageway, referred to as the Law Ditch, (also known as The Slough above Law Reservoir, and The Consolidated Law Ditch below the Greeley No. 2 Canal) is the most predominant drainage feature and traverses the central portion of the basin in a southerly direction. A secondary drainageway associated with the Law Ditch, hereafter referred to as the West Tributary, collects stormwater runoff from the western portion of the basin and confluences with the main flow path immediately north of State Highway 392. Below the confluence with the West Tributary, the Law Ditch flows due south to its confluence with the Cache la Poudre River.

As indicated on Figure 2.1, the Law Ditch enters the Growth Management Boundary near the intersection of WCR 21 and the Greeley No. 2 Canal. As this location, the Law Ditch has limited capacity to convey the peak discharge associated with large flood events; consequently,

much of the flood flows will be conveyed in overbank areas. The stormwater conveyed along the Law Ditch will enter and overtop the Greeley Number 2 Canal and WCR 21 from the north and east. These flood flows will continue south and west utilizing the culvert crossing at State Highway 392 (4'H x 6'W arched CMP) and overtopping State Highway 392 at the Great Western Railroad crossing.

Flood flows associated with the West Tributary are conveyed along the alignment of the Windsor Reservoir Outlet (a.k.a. the Spring Ditch). Initially, the stormwater runoff captured by the Springer Ditch is conveyed through the crossings of both WCR 74 (8'H x 10'W bridge) and WCR 72 (7'H x 11.5'W arch CMP). South of the WCR 72 crossing, the West Tributary follows the alignment of the Springer Ditch adjacent to WCR 17 and ultimately to its confluence with the Greeley No. 2 Canal directly north of Windsor Lake (a.k.a. Kern Reservoir). At this location, the flows conveyed along the Springer Ditch overtop the Greeley No. 2 Canal and ultimately flow across WCR 10 and the Great Western Railroad embankment to State Highway 392. At State Highway 392, the Western Tributary confluences with the main flow path of the Law Ditch.

South of State Highway 392, flood flows are generally conveyed along the alignment of the Law Ditch to the crossing of the Colorado and Southern Railroad (4'H x 8'W RCBC). Stormwater flows conveyed through this crossing continue south and overtop the crossing of Eastman Park Drive (3'H x 5'W arch CMP). South of Eastman Park Drive, flood flows are conveyed through the Kodak property, ultimately overtopping the Whitney Ditch and terminating at the Cache la Poudre River.



Consolidated Law Ditch
Upstream of C&S Railroad

Within the Law Basin, several irrigation reservoirs and inadvertent storage areas exist which provide for a reduction in stormwater runoff. To significantly reduce or attenuate the peak discharge for major storm events, these reservoirs or storage areas must have substantial storage volumes available for detention of the stormwater runoff. Several storage facilities within the Law Basin were determined to have sufficient available capacity to fully retain the runoff from the major storm events. In general, these facilities were associated with irrigation reservoirs located outside the Growth Management Boundary and are more fully discussed with respect to their impact on the reduction on flood flows in Chapter 4. Two additional reservoirs (Loop Lake and Law Reservoir) also provide for significant reduction of the 100-year peak discharge within the Law Basin. These reservoirs are also discussed in more detail in Chapter 4.

Little erosion of the Law Basin channel is evident. In general, active erosion of the channel bed and banks is not prevalent throughout the basin. The Law Basin channel and floodplain are not heavily encroached with vegetation. The vegetation species vary from rangeland grasses and shrubs to farm crops in the overbank areas. The low flow channel is typically lined with dense stands of wetland grasses and shrubs.

2.2.2 Windsor Basin

The majority of the historic urbanization has occurred within the Windsor Basin and continues to occur through the construction of new residential developments. The Windsor Basin consists of an upper and lower basin that are separated by the Kern Reservoir (also known as Windsor Lake). The upper portion of the basin exhibits a more rural land use. The lower basin reflects a high level of development that consists of limited commercial and industrial facilities along with residential housing.

With the exception of a small residential development, stormwater in the upper portion of the basin is generated from agricultural lands, collected in a series of swales and conveyed in a southerly direction toward the Lake Canal in a well-defined drainage channel. Given the limited capacity of the Lake Canal in comparison to the peak discharge associated with the major storm events, stormwater runoff will likely be captured, spilled and conveyed to an existing irrigation reservoir, Lake Canal Reservoir No. 1, in a less defined swale. Based on a review of the operating criteria and available storage in Lake Canal Reservoir No. 1, minimal attenuation of the peak discharge associated with the stormwater runoff will occur. Stormwater spilled from the irrigation reservoir or conveyed in swales around this facility, will continue in a southeasterly direction toward WCR 15. It should be noted that the Colorado and Southern Railroad embankment represents a drainage boundary to direct the stormwater to the southeast within this basin.

Stormwater runoff conveyed to WCR 15 overtops the roadway since no crossing presently exists. Downstream of WCR 15, stormwater continues to flow to the southeast toward the Greeley No. 2 Canal and Osterhaut Lake. At this location, stormwater captured by the Greeley No. 2 Canal will either be spilled into Osterhaut Lake or conveyed by the canal. Stormwater conveyed to Osterhaut Lake will ultimately overtop the State Highway 257 crossing (24-inch circular pipe) and flow into Kern Reservoir. Stormwater captured by the Greeley No. 2 Canal will pass through the bridge crossing (bridge width of 35 feet) of State Highway 257 and is diverted into Kern Reservoir immediately east of State Highway 257. Assuming delivery of the maximum irrigation diversions, the Greeley No. 2 Canal provides limited capacity to convey stormwater.

In general, Kern Reservoir provides detention benefits for the stormwater conveyed to the reservoir. However, stormwater in excess of the principal outlet pipe will be conveyed over the existing emergency spillway (approximately 100 feet in length) located at the southeast corner of the reservoir and into the lower basin. Downstream of the Kern Reservoir, stormwater will be collected, combined with local runoff and conveyed within the existing network of streets and storm sewers through portions of the heavily urbanized areas of the Town. Specifically, stormwater runoff released from Kern Reservoir will overtop State Highway 392, pass under the Colorado and South Railroad, (bridge width of 26 feet) and be conveyed to a crossing structure (36-inch diameter pipe) at Walnut Street. Stormwater collected in the urbanized area will ultimately overtop the four existing culvert crossings of Eastman Park Drive (42"x 66" arch RCP, 54-inch RCP storm drain, 36-inch RCP, and twin 48"H x 72"W arch CMPs) and flow through the Water Valley residential development via the drainage channel constructed as part of the development. This drainage channel conveys stormwater over the Whitney Ditch to Rock Bridge Lake and subsequently, the Cache la Poudre River. Given the magnitude of the flows and the limited capacity of the existing facilities (storm sewers, streets, and culvert crossings), localized flooding is likely to occur throughout the lower portion of the basin.

Kern Reservoir in the Windsor Basin

Minimal to no erosion of the major drainageway is evident in this basin. Furthermore, the major drainageway and floodplain are not generally encroached with heavy, dense vegetation. The vegetation species vary from rangeland grasses and shrubs to farm crops in the upper basin. The major drainageway in the lower basin typically consists of grass-lined swales and channels as well as the local street and storm sewer system.

2.2.3 High School Basin

The High School Basin is largely encroached by urbanization; consequently, the majority of the stormwater generated in the basin is conveyed through the existing network of streets and storm sewer systems. Near the upper portion of the basin, the Greeley No. 2 Canal intercepts stormwater from a small residential development (approximately 35 acres) and conveys the stormwater, within the capacity of the ditch, into the Windsor Basin through a bridge crossing

(width of 54 feet) of the Colorado and Southern Railroad. Stormwater that exceeds the capacity of the ditch continues to the south, combines with the runoff generated high school facilities, and is conveyed to State Highway 392. At State Highway 392, the stormwater overflows the street and is conveyed in the existing streets, primarily 10th, 11th, and 12th Streets. Near Palisade Mountain Drive, a stormwater channel (bottom width of 2 feet and depth of 5 feet) conveys water collected and conveyed in 10th Street to a crossing at Stone Mountain Drive (three 30-inch CMPs) and into the Poudre Park Detention Pond. A minor storm sewer system located beneath 10th Street also conveys stormwater to the channel near Palisade Mountain Drive. Releases from the Poudre Park Detention Pond are siphoned (twin 30-inch diameter pipes) under the Whitney Ditch and ultimately reach the Cache la Poudre River.

10th Street Channel Upstream of Stone Mountain Drive

Stormwater conveyed within 11th Street continues to the south and is intercepted by the Whitney Ditch. That portion of the stormwater not captured by the ditch continues to the river.

Along the western portion of the basin, stormwater is conveyed primarily within 12th Street to an existing detention pond south of Walnut Street. Releases from the detention pond are directed toward the Whitney Ditch and ultimately to the river.

A small portion of the basin in the vicinity of Elm Street and Eighth Street is serviced by a minor storm sewer system. The minor storm sewer system conveys stormwater to the storm sewer system beneath 10th Street and ultimately to the stormwater channel near Palisade Mountain Drive.

Minimal to no erosion is evident in the stormwater facilities located within this basin. The majority of the main flow paths are paved streets or minor storm sewer systems. The main conveyance channel located south of 10th Street and Palisade Mountain Drive, is presently lined with concrete (lower 2 feet) and vegetation (upper 3 feet) and no erosion of the channel is apparent.

2.2.4 Jacoby Basin

The Jacoby Basin continues to experience changes in land use as predominantly agricultural land transitions to residential and commercial development. In general, the upper

portion of the Jacoby Basin north of State Highway 392 consists of agricultural land with residential development representing approximately 20% of the land use. South of State Highway 392, commercial and residential development represents the majority of the land use along with the regulatory floodway associated with the Cache la Poudre River.

Similar to the High School Basin, a single well-defined drainageway does not exist in the Jacoby Basin. Stormwater generally moves from the north to the south within the basin. Where commercial and residential development is encountered, stormwater is conveyed through the development in the network of streets and minor storm sewers.

The Greeley No. 2 Canal bisects the Jacoby Basin approximately one-quarter mile north of State Highway 392. While the canal does not represent a drainage boundary, a portion of the stormwater is captured and conveyed by the canal to the extent that capacity is available. That portion of the stormwater not captured by the canal, overtops the canal banks and is conveyed to the south toward State Highway 392. Near the intersection of State Highway 392 and WCR 15, the Town of Windsor recently installed the West Side Storm Sewer (72-inch RCP) to convey stormwater within the basin. The West Side Storm Sewer collects stormwater at this intersection, conveys the runoff approximately one-quarter mile to the west along the south side of State Highway 392, and continues south for almost three-quarters of a mile before discharging into the Cache la Poudre River. Near the outfall from the storm sewer system, an inverted siphon conveys the stormwater runoff under the Whitney Ditch. Less frequent releases associated with minor storm events are routed into a constructed wetland/water quality pond prior to release into the river.

West Side Storm Sewer Outlet Channel

Given the lack of a well-defined drainageway coupled with the structural nature of the street and storm sewer systems, minimal to no erosion of the existing stormwater conveyance systems is evident in this basin.

2.2.5 Timnath Reservoir Basin

The Timnath Reservoir Basin is characterized by its lack of development and largely agricultural land use. Timnath Reservoir represents the basin's dominant drainage feature and is

centrally located within the basin. The majority of the watershed tributary to Timnath Reservoir lies outside the Growth Management Boundary; less than 15% of the total watershed is located within the Growth Management Boundary.

The stormwater generated in the upper portion of the watershed is captured, attenuated and released by Timnath Reservoir. Irrigation releases from Timnath Reservoir are conveyed to the Greeley No. 2 Canal in a well-defined drainageway called the Timnath Reservoir Outlet (bottom width of 15 feet, depth of 12 feet). Given its location within the watershed, the Timnath Reservoir Outlet also collects and conveys all stormwater generated below Timnath Reservoir as well as stormwater captured and released from Timnath Reservoir.

The Timnath Reservoir Outlet approaches the Growth Management Boundary near the intersection of LCR 36 and the Colorado and Southern Railroad. At this location, the Timnath Reservoir Outlet conveys stormwater and irrigation releases under a bridge crossing (width of 15 feet) of LCR 36 as well as the bridge crossing of the Colorado and Southern Railroad (width of 13.5 feet). Unlike the Windsor Basin, the embankment associated with the Colorado and Southern Railroad does not represent a drainage boundary in the Timnath Reservoir Basin.

Timnath Reservoir Outlet
Downstream of C&S Railroad

The outlet ditch continues south until it joins the Greeley No. 2 Canal. Stormwater, captured during a major storm event, will overtop a low area along the southern bank of the Greeley Number 2 Canal and continue south into the floodplain of the Cache la Poudre River. Within the floodplain, the runoff will be conveyed through a series of abandoned gravel pits and agricultural fields prior to joining the river.

Given the duration and magnitude of the irrigation releases from Timnath Reservoir, some erosion of the Timnath Reservoir Outlet channel is evident. During a major storm event, erosion of the channel will likely be exacerbated. The Timnath Reservoir Outlet channel and floodplain are not heavily encroached with vegetation. The vegetation species vary from rangeland grasses and shrubs to farm crops in the overbank areas. The low flow channel is typically lined with dense stands of wetland grasses and shrubs. Where historic erosion is evident, the channel has been lined with concrete debris.

2.2.6 Ptarmigan Basin

In contrast to the major basins north of the Cache la Poudre River, the basins located south of the river have a more defined drainage system. The only exception is the Ptarmigan Basin, which has four minor drainage paths, all of which terminate in Fossil Creek.

Much of the lower and middle portions of the Ptarmigan Basin are developed. The recent development south of State Highway 392 incorporated stormwater runoff facilities consisting of a series of channels, detention ponds, and storm sewers. In general, these facilities convey runoff from the development to existing culvert crossings of State Highway 392. In this area, the stormwater facilities were designed to limit the release associated with development to the capacity of the existing culvert crossings. North of State Highway 392, the stormwater runoff is conveyed via a series of stormwater channels ultimately reaching Fossil Creek.

Culvert Crossing of State Highway 392 and Downstream Channel in Ptarmigan Basin

The upper portion of the Ptarmigan Basin south of LCR 30 is located outside the Growth Management Boundary and is presently undeveloped. The stormwater from this area is conveyed across LCR 30 and into the development located along the I-25 frontage road. The Louden Ditch located north of LCR 30 captures and conveys stormwater runoff to a limited extent based on its existing capacity. Runoff not captured by the Louden Ditch spills to the north and into the existing residential housing developments. The area between the Louden Ditch and LCR 30 continues to experience development pressure and several residential housing developments are presently planned for this area.

As stated previously, the existing drainage network in the area south of State Highway 392 consists primarily of channels, detention ponds and storm sewers. Minimal erosion of the conveyance facilities is evident. In those areas where development does not exist, the runoff is conveyed in existing swales with no noticeable erosion. With respect to the stormwater runoff north of State Highway 392, the existing drainage channels in the developed portion of this area are stabilized with man-made structures and minimal erosion is noted. Where development is not present, the drainage channels are experiencing minor to moderate erosion.

2.2.7 River Ridge Basin

The River Ridge Basin consists of primarily agricultural land. In recent years, however, the basin has experienced a transition from agricultural land to residential development in approximately 20% of the watershed.

Unlike the basins north of the Cache la Poudre River, the River Ridge Basin is characterized by a well-defined drainage system. A major drainageway traverses the central portion of the basin in a northeasterly direction. Several minor tributaries exist within the watershed to collect and convey stormwater runoff into the major drainageway and ultimately to the Cache la Poudre River. Near the lower portion of the watershed, runoff conveyed in the major drainageway flows into a small irrigation reservoir, also known as Joe Dee Reservoir. Runoff captured by the irrigation reservoir is released through an outlet pipe (36-inch RCP) or an emergency spillway. The stormwater passing through the reservoir is conveyed by the major drainageway through the River West residential development. Within the development, the major drainageway crosses both River West Drive and River Edge Road (crossing structures consisting of five 7-foot RCPs) prior to its confluence with the Cache la Poudre River.

River Edge Road Culverts

The existing network of natural channels has historically experienced moderate to severe channel erosion as indicated by the magnitude of the incised channel sections throughout the watershed. Minor to moderate erosion of the existing channels continues to occur.

2.2.8 Bluff Basin

The Bluff Basin is located immediately south of the Cache la Poudre River between WCR 13 and State Highway 257. Limited development has occurred within or adjacent to the basin. Development planning has been initiated in the portion of the basin located immediately west of WCR 17. The Bluff Basin also incorporates a large portion of the Cache la Poudre River floodplain between WCR 13 and State Highway 257.

Given the topographic relief within the basin, no major drainageway exists. Runoff generated within the upper portion of the watershed concentrates along the bluff and results in

the formation of gullies that convey the stormwater runoff to the lower portion of the watershed. At the base of the bluff, the drainage system transitions from the gullies to a less concentrated system of drainage swales. At this location, the Eaton Ditch, generally located along the base of the bluffs, intercepts stormwater generated within the watershed. Given the limited capacity of the Eaton Ditch, minimal stormwater will be conveyed out of the basin. That portion of the stormwater not captured and conveyed within the Eaton Ditch will overtop the ditch banks and spill onto the floodplain of the Cache la Poudre River.

Channel erosion associated with the drainage system is evident given the topographic relief associated with the basin and the magnitude of the existing gullies. The gullies continue to actively erode and produce sediment that is accumulated along the base of the bluff.

2.2.9 Oklahoma Basin

The Oklahoma Basin represents the largest of the drainage basins located south of the Cache la Poudre River. This basin is also characterized by its lack of development and largely agricultural land use. Limited development has occurred along WCR 15 adjacent to the Bluff Basin and along U.S. Highway 34, which traverses the upper portion of the basin.

Similar to the River Ridge Basin, a well-defined drainage network including the existence of the major drainageway conveys runoff generated within the basin. The channel associated with the major drainageway becomes evident west of the intersection of WCR 17 and WCR 60. From this point, stormwater captured by the main channel and its tributary channels flows in a northeasterly direction to the Cache la Poudre River. The main channel varies in bottom width from 5 feet to 10 feet and is moderately to deeply incised within the contributing watershed.

As stormwater runoff is collected by the main channel in the upper watershed, it is conveyed under WCR 17 in a reinforced concrete box culvert (8.5'H x 5.5'W). At this location, the road embankment associated with WCR 17 creates an inadvertent detention area within the deeply incised channel. Runoff released through the box culvert is conveyed in the main drainage channel located adjacent to the alignment of the Great Western Railroad. Approximately one mile downstream of the WCR 17 crossing, the main channel crosses the

Weld County Road 17 Culvert

Great Western Railroad tracks in an arched corrugated metal pipe (9'H x 12'W). East of the railroad crossing, the stormwater conveyed in the main channel is combined with runoff collected by a major tributary located along the east portion of the basin. Directly northeast of this location, storm flows are conveyed under the bridge crossing (width of 60 feet) of State Highway 257 before reaching the confluence of the Cache la Poudre River.

The main channel and tributaries are deeply incised within the watershed. Wetland grasses and shrubs dominate the vegetative cover along the bottom of the channel. Similar to the River Ridge Basin, the existing network of natural channels has historically experienced moderate to severe channel erosion as indicated by the magnitude of the incised channel sections throughout the watershed. Minor to moderate erosion of the existing channels continues to occur.

2.2.10 South State Highway 257 Basin

The watershed associated with the South State Highway 257 Basin is noted by its wide range of topographic relief and relatively small drainage area. In the upper portion of the basin, a relatively shallow slope is reflected by the largely agricultural land use. The shallow slope of the agricultural land quickly transitions to the middle portion of the basin that is characterized by moderate to steep terrain with numerous incised tributary channels and gulleys. The lower portion of the watershed, north of State Highway 257 reflects a landform typical of the floodplain area associated with the Cache la Poudre River. In general, land use within the South State Highway 257 Basin is largely agricultural with development encroaching along that portion of the upper basin located adjacent to the U.S. Highway 34 corridor.

Given the basin characteristics, stormwater runoff efficiently travels through the watershed from the southeast to the northwest. The main drainage channel is well defined, either as a channel or swale, which generally parallels State Highway 257 until it crosses under the highway and confluences with the Cache la Poudre River. At the crossing of State Highway 257, two reinforced concrete box culverts (two 6'H x 10'W) have been installed to convey storm flows within the channel.

State Highway 257 Culverts

Similar to the Oklahoma Basin, vegetation along the bottom of the channel

consists of wetland grass and shrubs. The overbank area is vegetated with dry rangeland grasses and shrubs. Consistent with the majority of the basins located south of the Cache la Poudre River, the existing channels have historically experienced moderate to severe channel erosion as indicated by the magnitude of the incised channel sections throughout the watershed. Minor to moderate erosion of the existing channels continues to occur.

The drainage features along with the limits of the 100-year floodplain associated with the Cache la Poudre River and the Law Ditch Drainageway are presented on the existing drainage facilities map provided on Sheets 1A and 1B.

Sheet 1A and 1B presents a map illustrating the major drainageways and major crossing structures within the basins along with the location of the major tributaries to the drainageways. A detailed description of the existing detention facilities/reservoirs, conveyance facilities and major road crossings, is provided in Chapter 3.

III. INVENTORY OF EXISTING FACILITIES

Within the Growth Management Boundary, the existing storm drainage facilities are largely associated with development that has historically occurred in the ten drainage basins. In general, small local detention ponds, storm sewers, and conveyance channels comprise the network of drainage facilities that provide flood control during the minor storm events within these developments. Drainage structures also occur at crossings of both natural and storm drainage outfall channels. These crossings are typically associated with the local, county and state roadways as well as railroads and the existing canals within the watershed.

During this master planning effort, an inventory of the existing storm drainage facilities within the Growth Management Boundary was conducted. The inventory and assessment of each structure involved: (a) field reconnaissance to document location, condition and additional data requirements; (b) review of available design and construction drawings; (c) collection of site-specific survey data; and (d) evaluation of the hydraulic capacity. Table 3.1 summarizes the results of the inventory and evaluation of the existing storm drainage facilities. Sheets 1A and 1B present the location and description of those storm drainage structures that were identified and inventoried. Specific information related to these facilities is provided in the following paragraphs. Detailed data and photographic documentation pertinent to each structure are provided in the project notebook submitted in conjunction with this report.

3.1 Detention Facilities

Within each subbasin, few detention facilities exist that substantially reduce the stormwater runoff during major storm events. Detention areas at two locations provide the storage necessary to significantly attenuate the peak runoff. In one location, storage is available at an existing irrigation reservoir and at the second location, inadvertent storage exists immediately upstream of a roadway crossing. Both detention facilities are described in the following sections.

Several detention facilities that presently exist within the Growth Management Boundary effectively reduce the peak runoff during minor storm events but offer limited benefits during the major storms. These detention facilities were identified but were not specifically inventoried or evaluated during the master planning effort.

Several major detention and retention facilities exist outside the Growth Management Boundary and therefore were not included in the detailed inventory of existing structures. These

Table 3.1 Inventory of Existing Drainage Facilities.

Name	Type	Location	Condition	EPA-SWMM ID	Maximum Storage Volume (acre-ft)	Maximum Discharge Capacity[1] (cfs)
LAW BASIN (MAIN CHANNEL)						
Eastman Park Drive	Arch CMP	0.75 mile East of State Highway 257	fair	L206	0	59
Colorado & Southern Railroad	RCB	C&S R.R. ¾ mile Southeast of State Highway 257	fair	L208	0	370
State Highway 392	Arch CMP	0.75 mile East of State Highway 257	good	L710	0	45[2]
Greeley No. 2 Confluence	CMP	WCR 21 Greeley No. 2 Crossing	fair	L226	0	56
LAW BASIN - WEST TRIBUTARY						
G.W. Railroad	CMP	300′ South of State Highway 392	good	L210	0	123
State Highway 392	CMP	1,500′ East of State Highway 257	good	L210	0	20
WCR 19	None	1,000′ North of State Highway 392	N/A	L211	N/A	0
Greeley No. 2 Canal	None	1,400′ West of WCR 19	N/A	L711	N/A	0
WCR 17 / State Highway 257	None	WCR 70	N/A	L812	N/A	0
WCR 72	Arch CMP	400′ East of State Highway 257	good	L219	0	606
WCR 74	Bridge	2,000′ East of State Highway 257	good	L220	0	580
WINDSOR BASIN						
Water Valley Parkway	RCB	WCR 64 ½	good	W204	0	1,814
Eastman Park Drive - A Crossing	Ellip.RCP	3rd Street to 500' East of 3rd Street	good	W210	0	230
Eastman Park Drive - B Crossing	RCP	Storm Sewer Outlet 500’ West of 1st Street	good	W113	0	N/A
Eastman Park Drive - C Crossing	RCP	230′ West of 1st Street	good	W215	0	87
Eastman Park Drive - D Crossing	2-Arch CMP	300′ East of 1st Street	good	W220	0	173
Folkstone Park Pond	Pipe Spillway	Garden Drive	fair	W323	13.5[3] 19.0[4]	6[3] 80[4]
Walnut Street	RCP	Kern Ditch Crossing of Walnut Street	good	W229	0	95
C&S Railroad	Bridge	Kern Ditch just upstream of Walnut Street	fair	W240	0	~700
State Highway 392	Pipe	Kern Reservoir Outlet Pipe to Kern Ditch	good	W240	0	60

Table 3.1 Inventory of Existing Drainage Facilities (Continued).

Name	Type	Location	Condition	EPA-SWMM ID	Maximum Storage Volume (acre-ft)	Maximum Discharge Capacity[1] (cfs)
WINDSOR BASIN (continued)						
Kern Reservoir	Pipe Spillway	Principal Spillway Kern Reservoir Emergency Spillway	good	W341	440[3] 1,030[4]	60[3] 834[4]
WCR 17 or State Highway 257	CMP	Outlet Osterhaut Lake	good	W142	66	31
WCR 15	None	WCR 70 ½	n/a	W243	0	0
HIGH SCHOOL BASIN						
Whitney Ditch	2-CMPs	1,100′ South/Downstream of Stone Mountain Dr.	fair	H205	0	211
Stone Mountain Drive	3-CMPs	1,200′ West of Seventh Street	good	H206	0	94
10th Street	Channel	Palisade Mountain Dr. to Whitney Ditch	good	H806	0	350
State Highway 392	None		n/a	H209	0	0
JACOBY BASIN						
State Highway 392	RCP	West Side Storm Sewer, From State Highway 392 to Cache la Poudre River	good	J207	0	200
WCR 68 ½	None	East of WCR15	n/a	J12	0	0
TIMNATH BASIN						
State Highway 392	RCP	1,100′ West of WCR 13	poor	T204	0	0
LCR 32 ½ (WCR 68 ½)	CMP	2,600′ West of WCR 13	fair	T205	0	10
LCR 36 (WCR 72)	Bridge	1,500′ West of WCR 13	good	T207	0	2,700/1,050[2]
C & S Railroad	Bridge	1,500′ West of WCR 13	good	T207	0	2,700/1,050[2]
PTARMIGAN BASIN						
Basin P03 Outlet	CMP/RCP	Under State Highway 392	good	P303	unknown	422
Basin P05 Outlet	RCP	Under State Highway 392	good	P205	0	47
Basin P09 Outlet	RCP	Under State Highway 392	poor	P209	0	0
Basin P11 Outlet	HDPE	Under State Highway 392	good	P711	0	30
Basin P14 East, at Royal Vista Dr.	CMP	Under State Highway 392	good	P214	0	25
Basin P14 West, North of Arbys	Arch CMP	Under State Highway 392	good	P214	0	24

Table 3.1 Inventory of Existing Drainage Facilities (Continued).

Name	Type	Location	Condition	EPA-SWMM ID	Maximum Storage Volume (acre-ft)	Maximum Discharge Capacity[1] (cfs)
RIVER RIDGE BASIN						
River Edge Rd.	5-RCPs	0.5 mile South of State Highway 392, 0.5 mile West of LCR1/WCR13	good	R704	0	2,535
River West Dr.	5-RCPs	0.5 mile South of State Highway 392, 0.5 mile West of LCR1/WCR13	good	R704	0	2,340
OKLAHOMA BASIN						
State Highway 257	Bridge	0.75 mile South Cache la Poudre River	good	O201	0	5,000/2,900[2]
G.W. Railroad	Arch CMP	1,300′ South of WCR 62	good	O103	0	770
WCR 17	RCB	1 mile North of US Hwy 34	poor	O308	235	1,240
SOUTH STATE HIGHWAY 257 BASIN						
State Highway 257	2-RCBs	1 mile South of Cache la Poudre River	good	S802B	0	1,075
BLUFF BASIN						
No structures inventoried in this basin.						
GREELEY NO. 2 CANAL						
State Highway 392	Bridge	900′ East of WCR 21	good	n/a	0	>1,500
WCR 21	Bridge	1,500′ North of State Highway 392	fair	n/a	0	>1,500
G.W. Railroad	Bridge	1,200′ Upstream of WCR 21	fair	n/a	0	>1,500
WCR 19	Bridge	3,800′ North of State Highway 392	fair	n/a	0	>1,500
Greeley No. 2 Canal	Bridge	WCR 17/State Highway 257	good	W142	0	3,500/330[2]
Greeley No. 2 Canal	Bridge	At C & S Railroad	good	n/a	0	2,630/2,000[2]
Greeley No.2 Canal	Bridge	At WCR 15	fair	J211	0	2600/630[2]
Greeley No. 2 Canal	None	1,500′ West of WCR 13	n/a	T206	0	0

[1] Maximum discharge capacity prior to overtopping, assuming no obstructions.
[2] Prior to overtopping the stream bank upstream of the structure.
[3] At the elevation of the emergency spillway crest.
[4] At the elevation of the dam crest

reservoirs or storage areas were determined to have sufficient capacity to fully retain the runoff from the major storm events. In general, these facilities were associated with irrigation reservoirs and were included in the hydrologic analysis to define the stormwater runoff during existing conditions as discussed in Chapter 4.

As stated previously, detention facilities located inside the Growth Management Boundary that are effective in reducing the peak discharge from the major storm events do not exist within the majority of the basins. These basins include the Law Basin, Jacoby Basin, Timnath Reservoir Basin, Ptarmigan Basin, River Ridge Basin, Bluff Basin and South State Highway 257 Basin. Kern Reservoir in the Windsor Basin and inadvertent storage in the Oklahoma Basin were determined to significantly reduce the stormwater runoff and are described in the paragraphs below.

3.1.1 Windsor Basin

Windsor Lake (Kern Reservoir) is an irrigation reservoir located in the middle of the Windsor Basin immediately north of State Highway 392. Windsor Lake represents the divide for the upper and lower portions of the Windsor Basin. The reservoir was originally created to provide irrigation water to the farmland immediately surrounding the Town of Windsor. The dam embankment starts in the southeast corner of the reservoir and extends approximately 2,500 feet to the west. The spillway facilities are located in the southeast corner of the reservoir and include: (a) a principle spillway consisting of a 42-inch standpipe and outlet; and (b) an emergency spillway. The principle spillway controls the operating level of the reservoir and also provides a mechanism for the diversion of water in to the Kern Ditch. The emergency spillway consists of a concrete spill structure (100 feet in width and 2 feet in depth) and riprap stabilization measures. Storage available in the reservoir is also utilized as equalization storage for the New Cache la Poudre Irrigating Company and the irrigation diversions conveyed in the Greeley No. 2 Canal. Irrigation water conveyed in the Greeley No. 2 Canal is diverted into the reservoir in the northwest corner near State Highway 257 through a controlled inlet structure. The irrigation flows are released from the reservoir and into the Greeley No. 2 Canal through two 10'H x 12'W radial gates located near the northeast corner of the reservoir.

Kern Reservoir (in background) and Outlet to Greeley Canal No. 2

Based on information in "Project Manual for Kern Reservoir Windsor Lake Dike Upgrade" [TST, Inc., 1984], the reservoir was reported to have 1,030 acre-feet of storage available for irrigation diversions between the Kern Ditch Outlet (4785.3 ft, 1929 NGVD) and the principle spillway (4792.0 ft, 1929 Datum). Prior to overtopping the existing dam embankment, the principle outlet and emergency spillway have a maximum discharge capacity of 834 cfs. During the Master Drainage Plan's 100-year storm event (existing condition land use), approximately 1980 cfs is conveyed into the reservoir. During the 100-year storm event, the principle spillway releases 58 cfs, while the reservoir stores approximately 290 acre-feet of floodwater. The reservoir's emergency spillway does not operate during the Master Plan's existing condition 100-year storm event.

Although it is not effective in attenuating the peak flow during the 100-year storm event, Osterhaut Lake is hydraulically connected to Windsor Lake and captures much of the stormwater runoff generated within the basin. Osterhaut Lake is located immediately west of Kern Reservoir with the inadvertent detention for small events created by the road embankment associated with State Highway 257. The lake is surrounded by the Greeley Number 2 Canal on the north and west sides and by a small subdivision on the south side. A 24-inch CMP under State Highway 257 provides the hydraulic connection to the Windsor Lake and is considered the principle outlet for Osterhaut Lake. The roadway embankment associated with State Highway 257 is elevated approximately 5 feet above the invert of the existing 24-inch CMP. The flood storage volume provided in Osterhaut Lake before overtopping is approximately 66 acre-feet with a maximum release rate of 31 cfs from the existing outlet pipe. The 100-year existing condition discharge that is captured and conveyed through Osterhaut Lake is estimated to be 1,860 cfs and corresponds to an overtopping depth of approximately 0.7 feet on State Highway 257. Although a hydraulic connection exists between Osterhaut Lake and Windsor Lake, the 24-inch CMP was assumed to be blocked.

3.1.2 Oklahoma Basin

In the Oklahoma Basin, an inadvertent detention pond is located approximately one mile north of U.S. Highway 34 on the west side of WCR 17. At this location, WCR 17 crosses the major drainageway of the basin. The roadway embankment associated with WCR 17 was constructed across a deeply incised portion of the major drainageway and is elevated approximately 32 feet above the channel invert. A crossing structure, consisting of an 8.5'H x 5.5'W RCB, releases runoff from the detention area. The volume provided behind the roadway embankment prior to overtopping is approximately 235 acre-feet with a corresponding release through the existing box culvert of approximately 1,240 cfs. The 100-year peak inflow (existing condition land use) to this detention

area is estimated to be 1,100 cfs, which corresponds to a storage volume of approximately 156 acre-feet with no overtopping of WCR 17.

3.2 Major Crossing Structures

As mentioned previously, the majority of the drainage structures that were inventoried and evaluated during this planning effort involved the crossings of both natural and storm drainage outfall channels. Crossing structures were identified, inventoried and evaluated in every basin with the exception of the Bluff Basin where no crossings presently exist. The existing crossings are generally associated with the local, county and state roadways as well as railroads and the existing canals within the watershed. The capacity of each crossing was calculated using the U.S. Army Corps of Engineer's HEC-2/HEC-RAS Water Surface Profile program and/or the Federal Highway Administration's HY-8 Culvert Analysis program. Table 3.1 summarizes the location, condition, and hydraulic capacity of each crossing structure. A brief description of the crossing structures that were inventoried within each basin is provided in the following paragraphs.

Within the Growth Management Boundary, several additional road crossings of minor tributaries to the major drainageways within each basin also exist. Depending on the location and the contributing drainage area, these tributaries convey significant storm flows that may require crossing structures or improvements to existing structures to prevent overtopping during relatively minor storm events. This master planning effort did not specifically identify or inventory these crossings. In general, these crossing structures lack the capacity to convey runoff generated during relatively minor storm events and several structures were presently experiencing sedimentation problems that further reduce their capacity. As development within each basin occurs, these structures should be identified, inventoried and evaluated.

3.2.1 Law Basin

Seven major road crossings, two railroad crossings, and two canal crossings were identified and evaluated on the major drainageways within the Law Basin. Those crossings where structures presently exist consisted of one bridge, one reinforced concrete box culverts (RCB), three arched corrugated metal pipes (CMP), and three circular CMPs. At three locations, no structures exist and storm water simply overtops the existing road or canal.

<u>Eastman Park Drive Culvert.</u> The crossing at Eastman Park Drive consists of a 3'H x 5'W arch corrugated metal pipe culvert. At this location, roadway overtopping occurs at a depth of

approximately 4 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 59 cfs. The 100-year existing condition discharge in the Law Basin at Eastman Park Drive is approximately 4,840 cfs.

Colorado & Southern Railroad Culvert. The crossing at Colorado & Southern Railroad consists of a 4'H x 8'W reinforced concrete box culvert. At this location, the railroad overtops at a depth of approximately 7.5 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 370 cfs. The 100-year existing condition discharge in the Law Basin at Colorado & Southern Railroad is approximately 5,080 cfs.

State Highway 392 Culvert. The crossing of the major drainageway and State Highway 392 consists of a 4'H x 6'W arch corrugated metal pipe culvert. At this location, culvert capacity is limited by the approximately 4-foot height of the right bank upstream of the culvert. The capacity of the culvert prior to overtopping the right bank is 45 cfs. Overtopping of State Highway 392 will occur on the right bank approximately 1,750 feet west of the culvert. The 100-year existing condition discharge in the main channel of the Law Basin upstream of State Highway 392 is 1,890 cfs.

Upstream Side of State Highway 392 Culvert on the Law Basin Main Channel

Greeley No. 2 Canal Culvert. Stormwater runoff conveyed within the major drainageway (also known as the Law Ditch at this location) is intercepted by the Greeley No. 2 Canal on the east side of WCR 21. Normal Law Ditch irrigation flows and minor storm water flows enter the Greeley No. 2 Canal through a 3-foot diameter corrugated metal pipe culvert. High flows enter the canal over the left bank immediately east of WCR 21. At this location, the culvert will overtop at a depth of flow approximately 4 feet above the invert of the culvert. The capacity of the culvert prior to overtopping is 56 cfs. The 100-year existing condition discharge in the Main Channel of the Law Basin at the Greeley No. 2 Canal is approximately 1,880 cfs.

Great Western Railroad Culvert. At this location, a western tributary to the major drainageway is conveyed under the Great Western Railroad in a 4-foot diameter corrugated metal pipe culvert. The culvert will overtop at a depth of flow approximately 7.6 feet from the invert of the culvert.

Greeley No. 2 Canal at Confluence with the Law Ditch

The capacity of the culvert before overtopping is 123 cfs. The 100-year existing condition discharge conveyed in the western tributary channel of the Law Basin at the Great Western Railroad is 3,180 cfs.

State Highway 392 Culvert-West Tributary. The crossing at State Highway 392 and the western tributary channel is a 2-foot diameter corrugated metal pipe culvert. At this location, roadway overtopping occurs at a depth of approximately 4.5 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 20 cfs. The 100-year existing condition discharge in the western tributary channel of the Law Basin at State Highway 392 is 3,180 cfs.

WCR 19-West Tributary. A crossing structure does not exist where the West Tributary of the Law Basin crosses WCR 19 approximately 1,000 feet north of State Highway 392. Overtopping of WCR 19 will occur for any significant event, especially events which may overtop the Greeley No. 2 Canal upstream of WCR 19. The 100-year existing condition discharge in the western tributary channel of the Law Basin at Weld County Road 19 is 2,970 cfs.

Greeley No. 2-West Tributary. The western tributary channel confluences with the Greeley No. 2 Canal approximately 1,400 feet west of WCR 19. Stormwater captured by the Springer Ditch is conveyed to the canal at this location. Large flow events will exceed the capacity of the canal thereby overtopping the right bank and spilling the stormwater to the southeast toward WCR 19. The 100-year existing condition discharge in the Western Tributary channel of the Law Basin at the Greeley No. 2 Canal is 2,850 cfs.

State Highway 257 (WCR 17). A crossing does not exist for State Highway 257 (WCR 17) near its intersection with WCR 70. At this location, stormwater generated in the western portion of the basin will spill over the road. The road embankment will overtop during any significant flood event. The 100-year existing condition discharge in Law Basin at State Highway 257 is 670 cfs.

WCR 72 Culvert-West Tributary. At this location, stormwater conveyed by the western tributary channel is essentially stormwater captured by the Springer Ditch (also known as the Windsor Reservoir Outlet). The crossing at WCR 72 consists of a 7'H x 11.5'W arched corrugated metal pipe culvert. The road embankment overtops at a depth of approximately 9.0 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 600 cfs. The 100-year existing condition discharge in the western tributary channel (Springer Ditch) at WCR 72 is 1,280 cfs.

WCR 74 Bridge-West Tributary. Stormwater at this location is captured and conveyed by the Springer Ditch through a steel bridge that incorporates an opening approximately 10 feet in width and 8 feet in height. The capacity of the bridge prior to overtopping the roadway embankment is 580 cfs. The 100-year existing condition discharge in the western tributary channel (Springer Ditch) at WCR 74 is 760 cfs.

3.2.2 Windsor Basin

There are ten locations where existing structures convey stormwater across the major drainageway in the Windsor Basin. These crossings consist of two steel/timber bridges, two reinforced concrete box culverts (RCB), four reinforced concrete pipes (RCPs), and two corrugated metal pipes (CMPs). At one location, no structure exists and storm water simply overtops the existing county road.

Water Valley Parkway Culvert. The crossing at Water Valley Parkway is a 12'H x 12'W reinforced concrete box culvert. At this location, roadway overtopping occurs at a depth of approximately 10.3 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 1,810 cfs. The 100-year existing condition discharge in the Windsor Basin's major drainageway at Water Valley Parkway is 620 cfs.

Eastman Park Drive-Crossing A. This structure is the western-most crossing of Eastman Park Drive and consists of a 5.5'H x 3.5'W elliptical reinforced concrete pipe culvert. At this location, roadway overtopping occurs at a depth of approximately 5 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 230 cfs. The 100-year existing condition discharge in the major drainageway at this location is estimated to be 295 cfs.

Eastman Park Drive-Crossing B. The structure represents a crossing directly east of Crossing A on Eastman Park Drive. Crossing B consists of a 54-inch reinforced concrete pipe storm sewer outlet. The storm sewer drains an area north of Eastman Park Drive in the older residential portion of the Town of Windsor. The storm sewer receives flow from area inlets on Oak Street at the intersections of Second Street, Third Street, and Fourth Street. Additional flow enters the storm sewer from two detention ponds north of Garden Drive; the Mountain View School Pond and the Condos Pond. Limited data exists for the storm sewer; however, it appears to have capacity in excess of 130 cfs. The 100-year existing condition discharge in Windsor Basin at Eastman Park Drive–Crossing B was estimated to be 50 cfs.

Eastman Park Drive-Crossing C. This structure is located east of Crossing B on Eastman Park Drive and consists of a 36-inch diameter reinforced concrete pipe culvert. At this location, roadway overtopping occurs at a depth of approximately 7.5 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 87 cfs. The 100-year existing condition discharge in the major drainage path at this location on Eastman Park Drive is 85 cfs.

Eastman Park Drive-Crossing D. This crossing represents the eastern-most crossing of Eastman Park Drive and consists of two 4'H x 6'W arch corrugated metal pipe culverts. At this location, roadway overtopping occurs at a depth of approximately 3.8 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 173 cfs. The 100-year existing condition discharge in the major drainage path at this location on Eastman Park Drive is 350 cfs.

Walnut Street Culvert. The crossing at Walnut Street is a 3-foot diameter corrugated metal pipe culvert. At this location, roadway overtopping occurs at a depth of approximately 6.7 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 95 cfs. The 100-year existing condition discharge at this location on Walnut Street is estimated to be 150 cfs.

Colorado & Southern Railroad Bridge. The crossing at the Colorado and Southern Railroad consists of a timber bridge with an opening that is approximately 26 feet wide. The depth of flow at which the railroad overtops is approximately 9 feet. The capacity of the bridge before overtopping the railroad was estimated to be approximately 700 cfs. The 100-year existing condition discharge in the major drainage path at the Colorado & Southern Railroad is estimated to be 76 cfs.

State Highway 392 Culvert. The reinforced concrete box culvert at this location has been abandoned and is not capable of passing any storm water. The 3.5-foot diameter outlet pipe from Kern Reservoir uses the culvert to cross under State Highway 392 prior to discharging into Kern Ditch. Stormwater generated from the small watershed tributary to this location will pass over State Highway 392 prior to combining with the releases from the Kern Reservoir south of the highway. The 100-year existing condition discharge in the Windsor Basin at this location along the major drainage path is 76 cfs, including the uncontrolled releases from Kern Reservoir.

State Highway 257 Culvert. This crossing of State Highway 257 (WCR 17) is located in the upper portion of Windsor Basin and consists of a 2-foot diameter corrugated metal pipe, which is the outlet for Osterhaut Lake. The depth of flow at which roadway overtopping occurs is approximately 5.0 feet. The capacity of the culvert prior to overtopping the roadway embankment was calculated to be 31 cfs. The 100-year existing condition discharge in the Windsor Basin at State Highway 257 is 1,860 cfs.

State Highway 257 Culvert in the Windsor Basin

Greeley No. 2 Canal. Stormwater runoff generated in the upper portion of the Windsor Basin will be captured by the Greeley No. 2 Canal immediately west of Osterhaut Lake. Stormwater will be conveyed in the canal, under State Highway 257 in a bridge, and finally to Kern Reservoir. Due to capacity limitations of the canal west of the State Highway 257 bridge, large runoff events will overtop the right bank of the canal and flow into Osterhaut Lake. The crossing at State Highway 257 is a 38-foot wide steel bridge with a capacity of approximately 3,500 cfs prior to overtopping the highway. However, the capacity of the canal west of State Highway 257 is limited to approximately 330 cfs. The 100-year existing condition discharge in Windsor Basin at this location is 1,860 cfs.

WCR 15. A crossing structure does not exist where the Windsor Basin Drainageway crosses WCR 15 approximately 1,000 feet north of the Colorado & Southern Railroad. Overtopping of WCR 15 will occur for any storm event that generates significant runoff. The 100-year existing condition discharge in the Windsor Basin at Weld County Road 15 is estimated to be 1,980 cfs.

3.2.3 High School Basin

There are two major crossings of the major drainageway in the High School Basin. Both crossings consist of installations of multiple corrugated metal pipes (CMPs). A single major crossing at State Highway 392 does not exist. The drainage basin upstream of State Highway 392 is relatively small and bisected by the Greeley No. 2 Canal. At this location, it should be noted that the Greeley No. 2 Canal has the capacity to capture and convey a portion of the basin runoff through a timber bridge under the C&S Railroad and into the Windsor Basin.

Whitney Ditch. The crossing at the Whitney Ditch consists of two, 3-foot diameter, corrugated metal pipes. At this location, ditch bank overtopping occurs at a depth of approximately 9 feet from the invert of the culvert. The capacity of the culverts prior to overtopping is 210 cfs. The 100-year existing condition discharge at this location along the major drainage path is estimated to be 530 cfs.

Stone Mountain Drive. The crossing at the Stone Mountain Drive consists of three, 2.5-foot diameter, corrugated metal pipes. At this location, roadway overtopping occurs at a depth of approximately 4.5 feet from the invert of the culvert. The capacity of the culvert prior to overtopping the roadway embankment is 95 cfs. The 100-year existing condition discharge at Stone Mountain Drive is 495 cfs.

Stone Mountain Drive Crossing in the High School Basin

3.2.4 Jacoby Basin

Only one major crossing presently exists within the Jacoby Basin. The crossing at State Highway 392 (immediately west of the 15th Street intersection) consists of a 5-foot diameter reinforced concrete storm sewer pipe. The 5-foot diameter storm sewer connects to a 6-foot diameter storm sewer prior to discharging into the Cache La Poudre River. The capacity of the storm sewer

prior to surcharging onto State Highway 392 is approximately 170 cfs. The 100-year existing condition discharge in the major drainage path at State Highway 392 is 450 cfs.

3.2.5 Timnath Reservoir Basin

There are four locations where existing structures convey stormwater across the major drainageway in the Timnath Reservoir Basin. These crossings consist of one steel bridge, one concrete bridge, one reinforced concrete pipe (RCP) culvert, and one corrugated metal pipe (CMP) culvert. It should be noted that no crossing structure of the Greeley No. 2 Canal exists along the major drainageway in the Timnath Reservoir Basin. Stormwater conveyed to the Greeley No. 2 Canal will either be captured or spilled at this location. Given the limited capacity of the canal compared to the magnitude of the storm runoff, it is assumed that the majority of the stormwater runoff will spill over the banks of the canal.

State Highway 392 Culvert. The crossing at State Highway 392 is a 3-foot diameter reinforced concrete pipe. The capacity of the culvert is very limited due to sediment deposition and debris blockage. Overtopping of State Highway 392 is likely during storm events that generate significant runoff. The 100-year existing condition discharge in the major drainage path at State Highway 392 is 2,210 cfs.

LCR 32½ Culvert. The crossing at LCR 32½ consists of a 2.5-foot corrugated metal pipe. At this location, roadway overtopping occurs at a depth of approximately 4 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 10 cfs. The 100-year existing condition discharge in the major drainageway at LCR 32 ½ is 2,199 cfs.

Looking Upstream at LCR 36 Bridge in Timnath Basin

LCR 36 Bridge. This concrete bridge incorporates an opening that is approximately 15 feet wide. The depth of flow at which the left bank of the drainageway overtops is approximately 8 feet. The capacity of the bridge before overtopping the left bank was determined to be approximately 1,050 cfs. Overtopping of the road would occur at approximately 2,700 cfs. The 100-year existing condition discharge along the major drainage path at LCR 36 is 1,570 cfs.

Colorado & Southern Railroad Bridge. This crossing consists of a timber bridge that incorporates an opening of approximately 13.5 feet in width. This structure is closely located to the LCR 36 bridge and reflects similar hydraulic characteristics. At this location, the depth of flow at

which the left bank of the drainageway overtops is approximately 8 feet. The capacity of the bridge before overtopping the left bank was calculated to be approximately 1,050 cfs. Overtopping of the railroad would occur at approximately 2,700 cfs. The 100-year existing condition discharge at this location along the major drainage path is 1,570 cfs.

3.2.6 Ptarmigan Basin

The crossings inventoried in the Ptarmigan Basin were limited to those crossings of State Highway 392. Presently, six crossing structures of State Highway 392 exist in the Ptarmigan Basin. The crossings are generally in good condition with the exception of the crossing immediately east of LCR 5, which appears to be plugged with sediment. Development upstream of the crossings has been required to over-detain storm water releases to meet the capacity constraints of the existing crossings of State Highway 392. Table 3.2 displays the crossing structures, capacity information and the 100-year existing condition discharge at each crossing.

Table 3.2 Ptarmigan Basin Existing Crossing Structures Summary.

Name	Type	Location	Maximum Discharge Capacity (cfs)	Existing Peak 100-Year Discharge (cfs)
PTARMIGAN BASIN				
Basin P03 Outlet	CMP/RCP	Under State Highway 392	422	68
Basin P05 Outlet	RCP	Under State Highway 392	47	92
Basin P09 Outlet	RCP	Under State Highway 392	0	160
Basin P11 Outlet	HDPE	Under State Highway 392	30	266
Basin P14 East, at Royal Vista Dr.	CMP	Under State Highway 392	25	195
Basin P14 West, North of Arbys	Arch CMP	Under State Highway 392	24	195

3.2.7 River Ridge Basin

There are two major crossings of the major drainageway in the River Ridge Basin. The crossings at River Edge Road and River West Drive are very similar. Both crossings consist of five, 7-foot diameter reinforced concrete pipes. The depth of flow upstream of the culverts is estimated to be approximately 11 feet prior to overtopping the road. The maximum capacity at River Edge Road and River West Drive prior to overtopping the road is 2,535 cfs and 2,340 cfs respectively. The 100-year existing condition discharge in the major drainageway at the two road crossings is 3,440 cfs.

River West Drive Crossing in the River Ridge Basin

3.2.8 Oklahoma Basin

There are three locations where existing structures convey stormwater across the major drainageway in the Oklahoma Basin. These crossing structures consist of one concrete bridge, one arch corrugated metal pipe (CMP) and one reinforced concrete box culvert (RCB).

State Highway 257 Bridge. The crossing at State Highway 257 consists of a 60-foot span concrete bridge. At this location, roadway overtopping occurs at a depth of approximately 19 feet from the invert of the bridge. However, left bank overtopping will occur at a depth of approximately 14 feet from the invert of the channel. The capacity of the bridge prior to overtopping the road and left bank is 5,000 cfs and 2,900 cfs respectively. The 100-year existing condition discharge at this location along the major drainageway is 5,010 cfs.

State Highway 257 Bridge in the Oklahoma Basin

Great Western Railroad Culvert. The Great Western Railroad crossing is single 12'H x 9'W arched CMP. Railroad overtopping occurs at a depth of approximately 12 feet above the invert of the culvert. The maximum capacity of the culvert before overtopping the railroad is 770 cfs. The

100-year existing condition discharge in the major drainage path at the Great Western Railroad is 3,700 cfs.

WCR 17 Culvert. This crossing of WCR 17 is a 8.5'H x 5.5'W reinforced concrete box culvert. The depth of flow at which roadway overtopping occurs is approximately 33 feet. The roadway embankment impounds stormwater associated with the existing WCR 17 detention area. The capacity of the culvert prior to overtopping the roadway was calculated to be 1,240 cfs. The 100-year existing condition discharge at this location is 1,100 cfs.

3.2.9 South State Highway 257 Basin

There is one crossing of the major drainageway in the South State Highway 257 Basin. A crossing structure presently exists at State Highway 257. This structure consists of two 6'H x 10'W reinforced concrete box culverts. At this location, roadway overtopping occurs at a depth of approximately 7.3 feet from the invert of the culvert. The capacity of the culvert prior to overtopping is 1,075 cfs. The 100-year existing condition discharge in the major drainageway at State Highway 257 is estimated to be 1,560 cfs.

3.3 Open Channels

Open channels that were identified and inventoried during this planning effort were limited to existing stormwater channels and irrigation ditches/canals that serve as a major drainageway within a basin. In general, channels with a primary purpose of conveying stormwater through the basin are noticeably absent and runoff is typically conveyed in existing swales. Few of the basins have natural channels that collect and convey the stormwater runoff. Where the natural channels exist, they are located in basins south of the Cache la Poudre River (River Ridge Basin, Oklahoma Basin, South State Highway 257 Basin) that area characterized by an increase in topographic relief with channels naturally incised into the bluffs in the upper portions of the watershed.

The majority of the open channels discussed in the following paragraphs are related to the conveyance channels associated with the releases from irrigation reservoirs. The remaining conveyance channels were constructed as stormwater facilities during development of the property within each basin.

3.3.1 Law Basin

The major drainage channel in the Law Basin is the Law Ditch upstream of the Greeley No. 2 Canal and the Consolidated Law Ditch downstream of the Greeley No. 2 Canal. Both ditches convey irrigation releases from irrigation reservoirs located in the upper portion of the basin and offer approximately 50-100 cfs of conveyance capacity. The Law Ditch conveys both irrigation releases and stormwater runoff into the Greeley No. 2 Canal immediately east of WCR 21. At this location, the magnitude of the stormwater runoff (1,890 cfs) from the major storm events (100-year) greatly exceeds the capacity of both the Law Ditch and the Greeley No. 2 Canal. Large stormwater flows will overtop the canal banks and continue south to State Highway 392. The Consolidated Law Ditch begins at the Greeley No. 2 Canal, immediately west of WCR 21, and continues in a southerly direction to property owned by Kodak Incorporated south of Eastman Park Drive. South of Eastman Park Drive, both irrigation diversions and stormwater captured by the Consolidated Law Ditch flow through a small irrigation pond, along the west side of the Kodak property, through a water quality pond and ultimately into Cache la Poudre River. Existing condition 100-year stormwater discharges along the Consolidated Law Ditch are approximately 5,000 cfs.

Great Western Railroad Culvert on the West Tributary of the Law Basin

The major flow path draining the western portion of the Law Basin, northwest of the intersection of State Highway 392 and the Great Western Railroad, will be referred to in this master drainage plan as the West Tributary of the Law Basin. A defined channel does not exist for the West Tributary between State Highway 392 and the Greeley No. 2 Canal. Upstream of the Greeley No. 2 Canal, the West Tributary is known as the Springer Ditch (also known as the Windsor Reservoir Outlet) and serves as the only major defined drainage channel for the area. In addition to irrigation releases from Windsor Reservoir, the Springer Ditch captures stormwater in the ditch facilities and ultimately conveys all flows into the Greeley No. 2 Canal.

Between the Windsor Reservoir and the Greeley No. 2 Canal the West Tributary/Springer Ditch has a capacity of 500 to 1,000 cfs and estimated 100-year stormwater flows range from 800 to 2,500 cfs. According to representatives of the New Cache la Poudre Irrigation Company, during the irrigation season the Springer Ditch could be conveying a maximum of 150 cfs of irrigation flows. For the purpose of this master planning effort, the maximum irrigation flows of 150 cfs are assumed to be conveyed by the Springer Ditch between WCR 74 and the Greeley No. 2 Canal.

A hydraulic capacity evaluation of the Greeley No. 2 Canal was performed between the Kern Reservoir and the State Highway 392 crossing to determine the canal's ability to capture and direct stormwater flows. The capacity of the Greeley No. 2 Canal between the Kern Reservoir radial gates and the confluence with the Springer Ditch was determined to be approximately 500 cfs. However, between the Springer Ditch confluence and State Highway 392, the Greeley No. 2 Canal minimum capacity was approximately 700 cfs. According to representatives of the New Cache la Poudre Irrigation Company, during the irrigation season the Greeley No. 2 Canal could be conveying a maximum of 600 cfs of irrigation flows. Therefore, the Greeley No. 2 has the capability to capture and divert approximately 100 cfs (700 cfs capacity – 600 cfs of irrigation flows) of stormwater flows downstream/east of the Springer Ditch confluence in the existing condition. When the Greeley No. 2 Canal's minimum capacity of 500 cfs is reached between the Kern Reservoir and the Springer Ditch confluence, flows will overtop the right bank of the canal. A defined West Tributary channel does not existing downstream of the Greeley No. 2 Canal, therefore, a wide shallow flooding zone will convey stormwater across WCR 19, the Great Western Railroad, and State Highway 392. The existing condition 100-year discharges along the West Tributary between the Greeley No. 2 Canal and State Highway 392 are approximately 3,000 cfs. Downstream of State Highway 392 the West Tributary flows will confluence with the Law Ditch Main Channel flows.

3.3.2 High School Basin

The primary drainage channel in the High School Basin is the 10th Street Channel. The channel is located between Palisade Mountain Drive and the Whitney Ditch. The channel has a 2-foot bottom width and is approximately 5 feet deep. Between Palisade Mountain Drive and Stone Mountain Drive, the channel bottom is lined partially with concrete to a height of 1.5 feet. The channel has a maximum conveyance capacity, prior to overtopping into adjacent properties, of 350 cfs. The 100-year existing condition discharge along the 10th Street Channel is approximately 500 cfs.

10th Street Channel between Palisade Mountain Drive and Stone Mountain Drive

3.3.3 Timnath Reservoir Basin

The Timnath Reservoir Outlet Ditch serves as the major drainageway for the Timnath Reservoir Basin. Both irrigation releases from Timnath Reservoir as well as stormwater runoff intercepted by the ditch facilities are conveyed to the Greeley No. 2 Canal. At this location, runoff in excess of the capacity of the Greeley No. 2 Canal is spilled to the south into the floodplain of the Cache la Poudre River. The Timnath Reservoir Outlet Ditch ranges from 10 to 15 feet in width and is characterized by relatively steep side slopes (1H:1V to 2H:1V). The ditch is deeply incised (typically 8 to 10 feet) and is lined with vegetation as well as concrete rubble/riprap to stabilize locations of historic erosion. The maximum discharge capacity of the ditch is estimated to range from 800 and 1,200 cfs. The 100-year existing condition discharge along the Timnath Reservoir Outlet Ditch varies from approximately 1,000 to 2,200 cfs.

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).

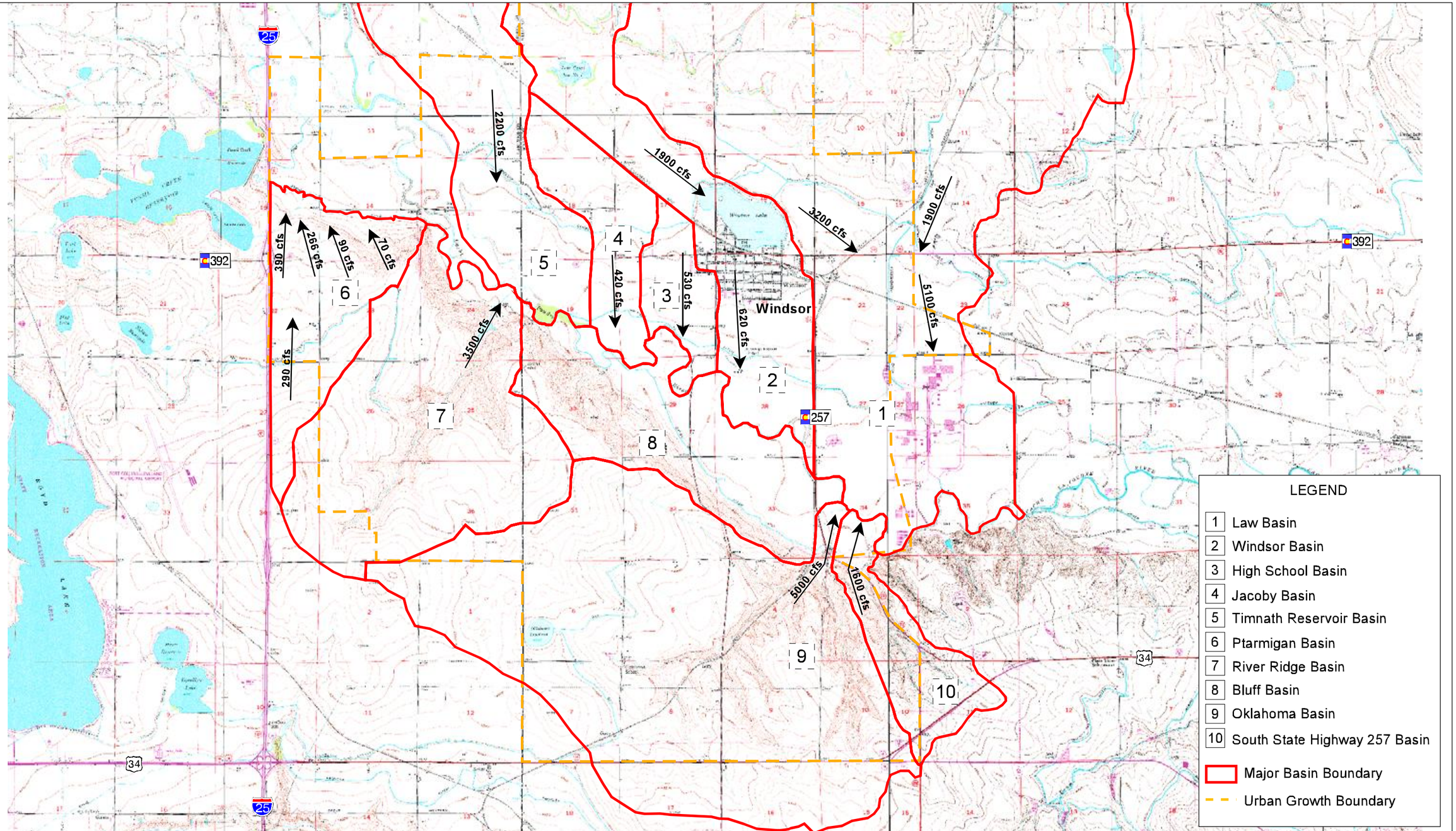
□ocation	EPA SWMM Elemen t	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr□	1□ yr□
WINDSOR BASIN – LOWER MISCELLANEOUS (continued)								
Oak St. Flows	W130	197	62	91	116	178	248	328
Elm St. at 7th St.	W131	77	33	51	62	85	114	123
Eastman Park Dr. at 54-inch Storm Sewer	W710	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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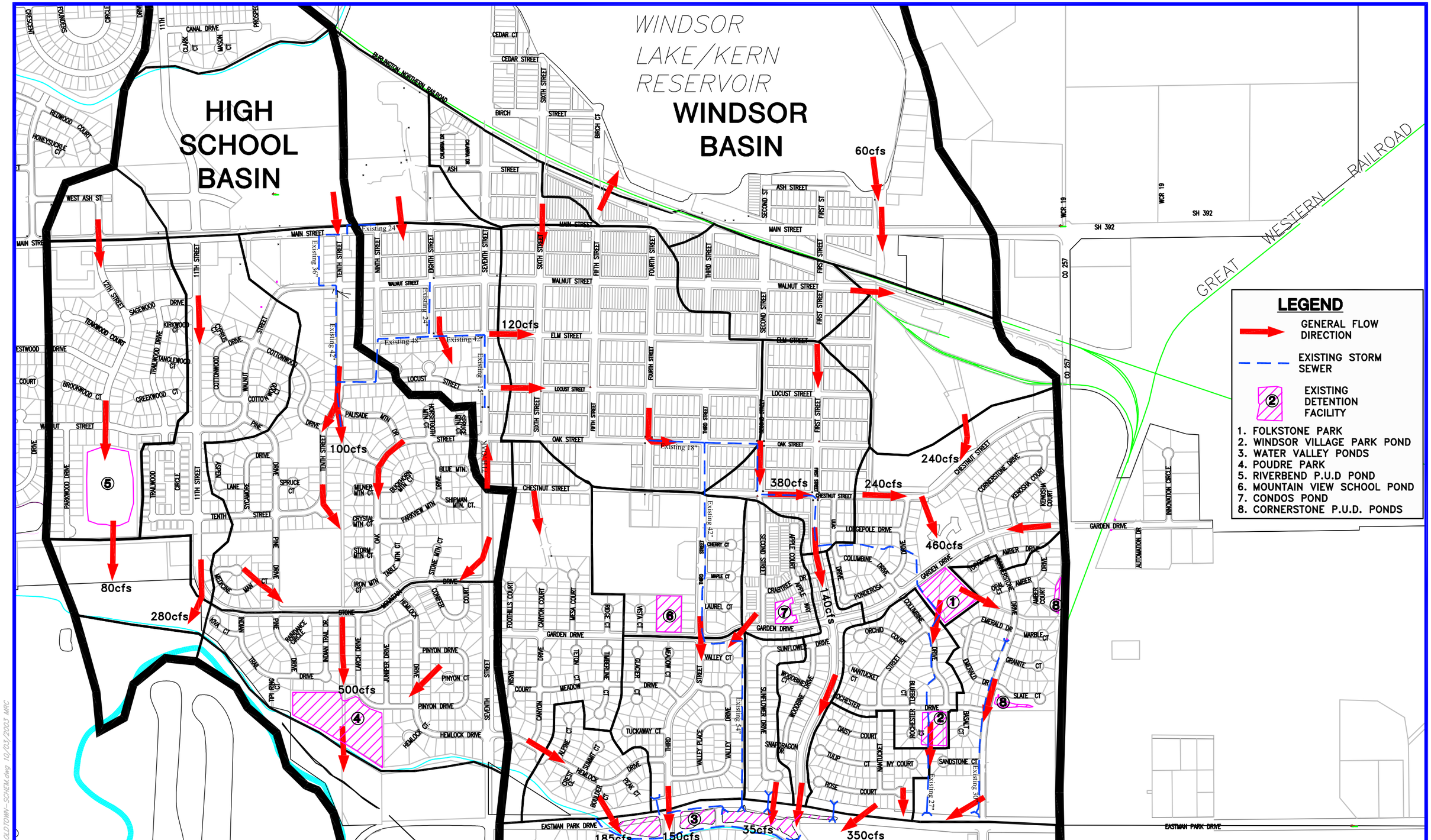
DATE: 10/02/2003	CHECKED BY: BAA

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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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).

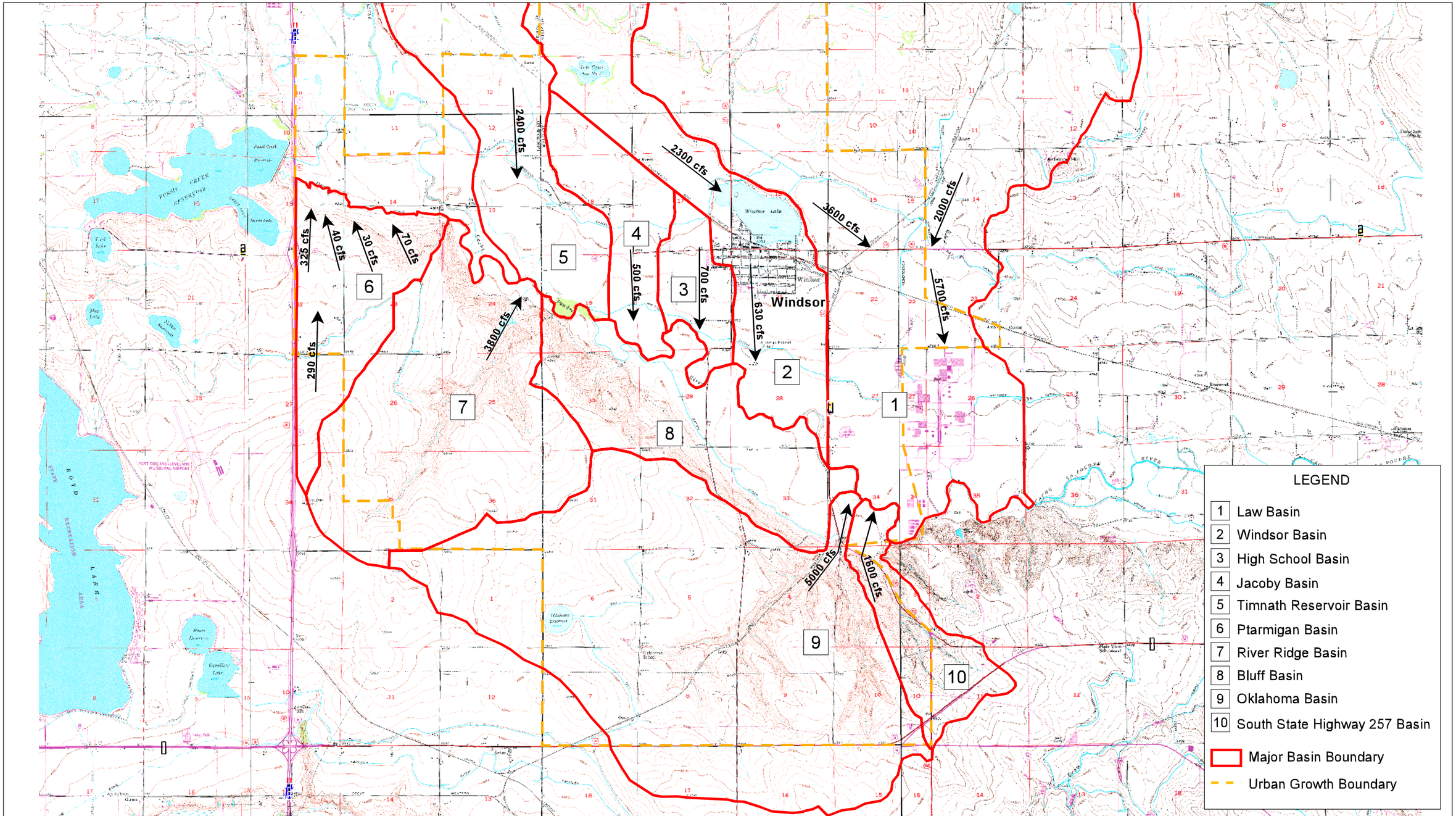
□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr [	□ yr	1□ yr [
WINDSOR BASIN – LOWER MISCELLANEOUS (continued)								
Oak St. Flows	W130	197	62	91	116	178	248	328
Elm St. at 7th St.	W131	77	33	51	62	85	114	123
Eastman Park Dr. at 54-inch Storm Sewer	W710	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).

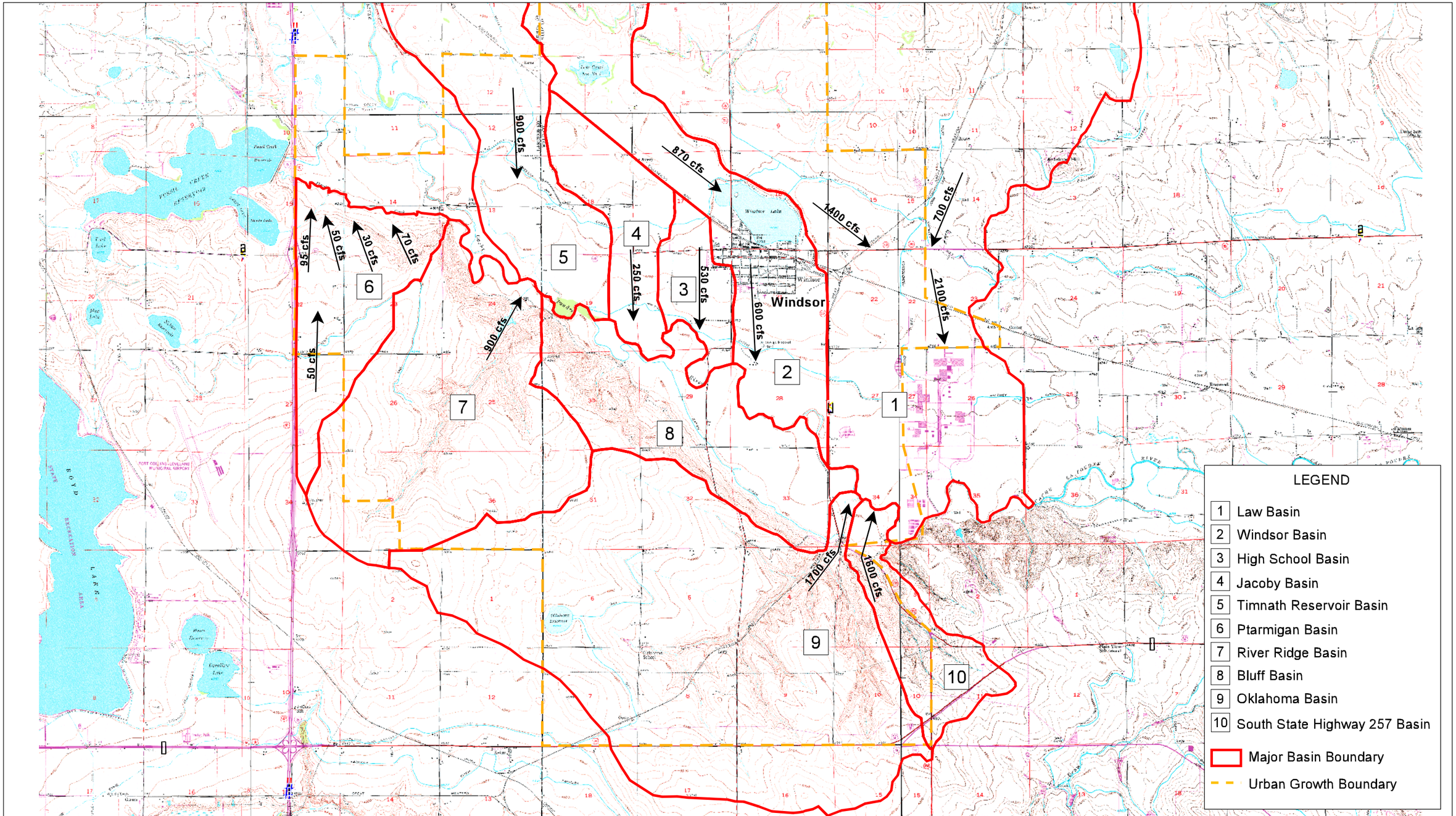
□ocation	EPA SWMM Element	Drainage Area (acres)	Pea□Discharge (cfs)					
			□yr□	□yr□	1□yr□	□ yr□	□ yr□	1□ yr□
WINDSOR BASIN – LOWER MISCELLANEOUS (continued)								
Oak St. Flows	W130	197	62	91	116	178	248	328
Elm St. at 7th St.	W131	77	33	51	62	85	114	123
Eastman Park Dr. at 54-inch Storm Sewer	W710	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.



REVISIONS:

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PROJECT NUMBER: COTOW08	DRAWN BY: RJN
FILENAME: Windsor-masterplan.apr Future Condition - Over 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
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.

V. DRAINAGE IMPROVEMENTS

5.1 Formulation of Drainage Improvements

As indicated in Chapter 4, 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. The results of this initial planning effort generated improvements that exceed a total construction cost of $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, it was determined that the formulation of drainage improvements would assume 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). The exceptions to this detention criteria are: (a) the Ptarmigan Basin where release rates have been previously determined based on the capacity of the existing structures; and (b) the Jacoby Basin where the area upstream of the Greeley No. 2 Canal and west of WCR 15 is limited to 0.2 cfs/acre during the 100-year storm event.

With the direction provided in meetings with the Town staff, drainage improvements were generated to mitigate the potential flooding assuming future land use conditions and over-detention in each drainage basin. The potential flooding problems were previously described in Chapter 4 for this hydrologic condition. This chapter presents the drainage improvements developed to mitigate these flooding problems.

In general, several improvements were initially identified as potential solutions for the drainage problems discussed in Chapter 4. These improvements were also intended to meet master planning objectives associated with the undeveloped portions within each drainage basin. The following items were considered during the formulation of a comprehensive drainage plan.

- Construction of detention ponds.
- Installation, removal or replacement of structures at specific problem areas.
- Improvement or enlargement of major drainage channels.
- Acquisition of property/structures within specific flooding areas and relocation of homeowners and/or businesses.
- Floodproof structures in specific flooding areas, where appropriate.

- Construction of berms and spill structures to contain additional stormwater captured by the irrigation canals/ditches and control the release of excess stormwater.

As previously indicated, over 90 percent of the watershed that contributes stormwater to the Town of Windsor is presently undeveloped and largely consists of land associated with agricultural, ranching, open space and floodplains. Within the Growth Management Boundary, flooding problems are predominantly associated with road crossings and limited channel capacity along the major drainageways, and limited on-site detention in developed areas. Due to the nature and location of the flooding problems, this master planning effort focused on providing alternatives that would significantly reduce the peak discharge in the major drainageways within each drainage basin.

Flooding problems are created at several of the road crossings within the Growth Management Boundary that do not adequately convey the 100-year flood event without overtopping the roadway. Currently, the responsibility for improving some of the structures may rest with either the county or state highway officials. Given that these structures are located within the Growth Management Boundary, however, this master planning effort assumed replacement of the structures as development occurs within the basin. Furthermore, replacement of these structures assumes the stormwater runoff is conveyed to these locations as indicated by the existing topographic mapping. Where drainage channels do not presently exist, channel improvements may ultimately be necessary to ensure that these stormwater flows are conveyed to the location of the crossing structures.

Several detention areas that presently exist behind roadway and railroad embankments do not meet existing drainage criteria. Nevertheless, these ponds may play an important role in reducing the 100-year peak discharge in the subbasins in which they are located. In addition, several irrigation reservoirs provide detention and/or retention benefits that reduce the stormwater runoff during major storm events. This master planning effort assumed that those ponds and reservoir, that provide effective detention during major storm events, will remain intact or will be replaced/incorporated into detention requirements associated with future development within each basin.

The general approach taken to provide solutions to the existing drainage problems consisted of the major improvements indicated below.

- Where appropriate, crossing structures with limited capacity should be improved and enlarged to meet existing drainage criteria or criteria identified in the master plan. These improvements will reduce backwater elevations and roadway overtopping.

- Open channels should be improved and enlarged to reduce channel overbank flooding and mitigate potential channel erosion.

- Regional detention ponds should be constructed where appropriate to reduce the peak discharges within a drainageway.

- On-site detention ponds should be constructed within the basins to limit the peak discharges in the drainageways, as well as the runoff from each subbasin, during the 100-year storm to the magnitude of the existing condition, 10-year storm event.

- Maintain or replace the existing roadway and railroad detention ponds where these facilities provide significant benefit during the major storm events. The releases from these structures should be limited to maximize the existing storage present at these locations.

- Construct spill structures and enlarge the canal section, where appropriate, along the irrigation canals/ditches to prevent flooding of adjacent properties. Divert the stormwater captured by the irrigation facilities to major outfall conveyance facilities or major drainageways.

5.2 Description of Drainage Improvements

The improvements discussed in the previous section were initially evaluated and screened for applicability. A preferred list of improvements was identified and a comprehensive plan was developed that would address flooding problems along the major drainageway in each drainage basin within the Growth Management Boundary. In the paragraphs that follow, the preferred improvements are discussed. Unless otherwise stated, these improvements assume that on-site detention is provided to limit the releases for future developments to the 10-year existing condition levels.

To facilitate the screening and evaluation of proposed improvements as well as the formulation of an implementation plan, each basin was evaluated on an individual basis, and if necessary, the major drainageway was divided into reaches as described below.

Law Basin-Main Channel

Reach L1: Cache la Poudre River to downstream of Eastman Park Drive
Reach L2: Downstream of Eastman Park Drive to State Highway 392
Reach L3: State Highway 392 to WCR 70

Law Basin-West Tributary

Reach L4: Colorado and Southern Railroad to State Highway 392
Reach L5: Kern Reservoir, WCR 19 to Greeley No. 2 Canal
Reach L6: WCR 19 to State Highway 257

Windsor Basin

Reach W1: Lower Basin-State Highway 392 to Eastman Park Drive
Reach W2: Lower Basin-State Highway 392 to Eastman Park Drive
Reach L5: Upper Basin, Kern Reservoir to Greeley No. 2 Canal

High School Basin

Reach H: State Highway 392 to Cache la Poudre River

Jacoby Basin

Reach J: Greeley No. 2 Canal to Cache la Poudre River

Timnath Reservoir Basin

Reach T: Cache la Poudre River to 1,400 feet Upstream of LCR 32 ½

Ptarmigan Basin

Reach P: 500 feet south of LCR 30 to State Highway 392

Oklahoma Basin

Reach O1: Cache la Poudre River to Great Western Railroad
Reach O2: WCR 17 and WCR 60

South State Highway 257 Basin

Reach S: Cache la Poudre River to State Highway 34

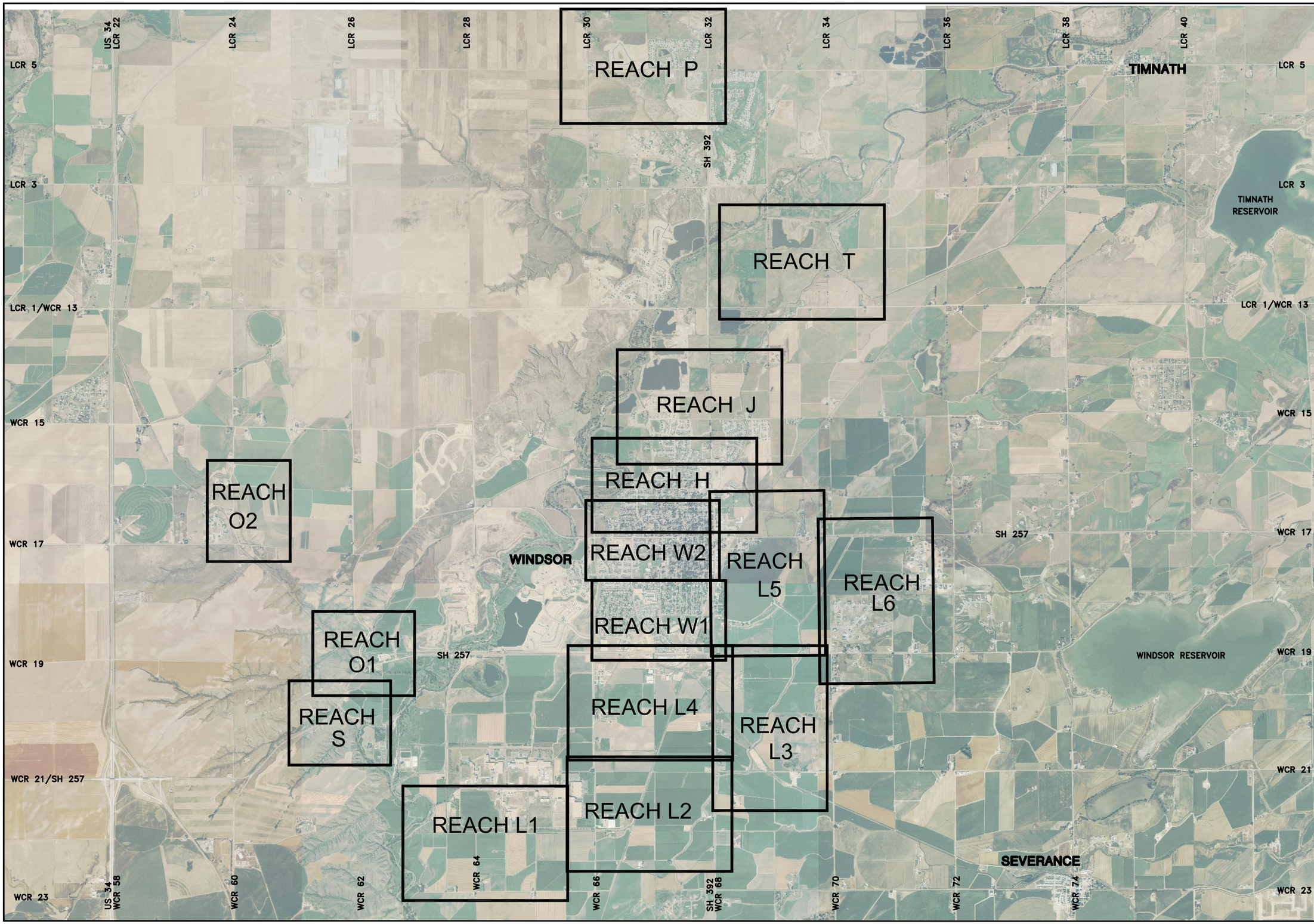
No drainage improvements have been identified within the River Ridge and Bluff Basins; consequently no reach delineations were necessary for these basins.

The drainage improvement plans for each reach are referenced in Figure 5.1. Figures 5.2 to 5.16 present the conceptual plans associated with the improvements within each drainage basin.

5.2.1 Law Basin-Reach L1

The proposed improvements to Reach L1 are presented below and in Figure 5.2.

<u>Stormwater Channel (Sta. 0+00 to 72+00).</u> Construct approximately 7,200 feet of channel improvements from the confluence of the Cache la Poudre River to approximately 1,200 south of Eastman Park Drive. The channel configuration includes a low flow channel along with an overbank area to convey flood flows. In addition, the



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SCALE
1" = 4000'

TOWN OF WINDSOR
MASTER DRAINAGE PLAN
REACH INDEX

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Project No.	COTOW08
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FIGURE 5.1

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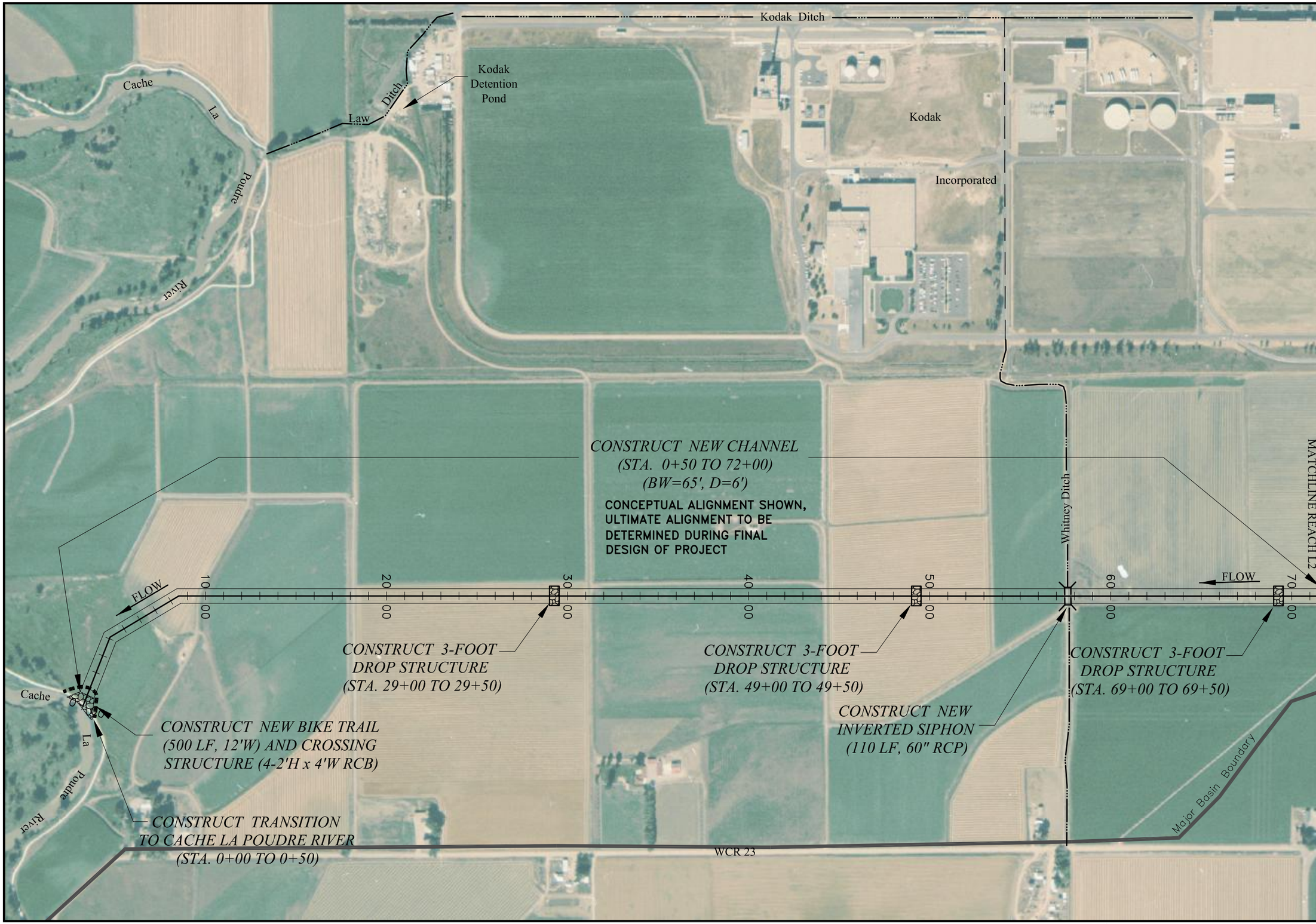

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SCALE
1" = 500'

TOWN OF WINDSOR
MASTER DRAINAGE PLAN
LAW BASIN
REACH L1

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FIGURE 5.2

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channel incorporates a grass lining, total bottom width of 65 feet including a bottom width of 20 feet for the low flow channel, maximum top width varying from 110 to 125 feet, sideslopes of 4H:1V, channel slope of 0.003 ft/ft, and design depth of approximately 5 feet. In conjunction with construction of the channel, a rock-lined transition structure into the Cache la Poudre River is required. To maintain a non-erosive channel slope, construction of three 3-foot grouted rock drop structures will be required.

Whitney Ditch Crossing. Construction of a 60-inch inverted siphon is required to convey irrigation diversions in the Whitney Ditch under the new stormwater channel. The siphon is approximately 110 feet in length and includes concrete headwalls at both the inlet and outlet to the siphon.

Bike Trail Crossing. Construction of the stormwater channel will required installation of a crossing for the existing bike trail along the Cache la Poudre River. The structure will convey the peak discharge associated with the future conditions 2-year storm event under the crossing. Events greater than the 2-year storm will be conveyed over the bike trail crossing. The structure consists of four 2'H x 4'W reinforced box culverts. Erosion protection will be provided along the embankments of the crossing to protect the trail during overtopping events between the 2-year and 100-year events.

5.2.2 Law Basin-Reach L2

The proposed improvements to Reach L2 focus on the new stormwater channel and crossing structures at Eastman Park Drive and the Colorado & Southern Railroad. Figure 5.3 presents the improvements in Reach L2.

Stormwater Channel (Sta. 72+00 to 136+00). Construct approximately 6,400 feet of channel improvements from approximately 1,200 south of Eastman Park Drive to a point along the north side of the Colorado & Southern Railroad. The channel improvements include a low flow channel along with an overbank area to convey flood flows. In addition, the channel incorporates a grass lining, bottom width ranging from 20 to 65 feet (high flow channel), maximum top width ranging from 90 to 120 feet, channel slope varying from 0.0013 ft/ft to 0.003 ft/ft, sideslopes of 4H:1V, and a design depth of varying from 6 to 8 feet.

Eastman Park Drive Crossing. Construct a new crossing consisting of a 6'H x 16'W reinforced concrete box culvert. Transitions from the existing channel to the culvert crossing are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing.

CONSTRUCT NEW CROSSING STRUCTURE (2-4'H x 8'W RCBs)
CONSTRUCT 2-FOOT DROP STRUCTURE (STA. 136+50 TO 137+00)
CONSTRUCT 3-FOOT DROP STRUCTURE (STA.149+00 TO 149+50)
FLOW
MATCHLINE REACH L4
CONSTRUCT NEW CHANNEL (STA. 136+00 TO 163+00) (BW=17', D=9')
CONSTRUCT 3-FOOT DROP STRUCTURE (STA. 8+00 TO 8+50)
CONSTRUCT NEW CHANNEL (STA. 0+00 TO 8+50) (BW=8', D=6')
CONSTRUCT 4-FOOT DROP STRUCTURE (STA. 0+00 TO 0+60)
MODIFY EXISTING CHECK STRUCTURE AND CONSTRUCT SIDE CHANNEL WEIR (BW=30', D=2')
Existing Check Structure
MATCHLINE REACH L3
SH 392
Kodak Incorporated
Kodak Incorporated
CONSTRUCT NEW CHANNEL (STA. 72+00 TO 83+00; BW=65', D=6') (STA. 83+00 TO 136+00; BW=20', D=8')
CONCEPTUAL ALIGNMENT SHOWN, ULTIMATE ALIGNMENT TO BE DETERMINED DURING FINAL DESIGN OF PROJECT
Eastman Park Drive
Metal Container Corporation
Kodak Railroad Spur
Colorado & Southern Railroad
Canal
No. 2
Greeley
Major Basin Boundary
MATCHLINE REACH L1
CONSTRUCT NEW CROSSING STRUCTURE (6'H x 16'W RCB)
FLOW
WCR 21 3/4
CONSTRUCT NEW CROSSING STRUCTURE (3-72" RCPs)
SCALE 1" = 500'
TOWN OF WINDSOR
MASTER DRAINAGE PLAN
LAW BASIN
REACH L2
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FIGURE 5.3
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Colorado & Southern Railroad Crossing. Construct a new crossing consisting of three 72-inch diameter RCPs. Transitions from the existing channel to the culvert crossing are required along with placement of stabilization measures to control the potential erosion near the crossing. Construction costs associated with the crossing assume the RCPs will be bored under the tracks to facilitate railroad operations during construction.

Stormwater Channel (Sta. 136+00 to 163+00). Construct approximately 2,700 feet of channel improvements from the Colorado & Southern Railroad to State Highway 392. The channel should incorporate a grass lining, a bottom width of 17 feet, maximum top width of 90 feet, channel slope ranging from 0.0024 ft/ft to 0.003 ft/ft, sideslopes of 4H:1V, and design depth of approximately 9 feet. A 2-foot drop structure and a 3-foot drop structure will be required along the new channel to maintain the non-erosive slope.

State Highway 392 Crossing. Construct a new crossing consisting of two 4'H x 8'W reinforced concrete box culverts. The crossing will be approximately 250 feet in length and will convey stormwater flows from the Main Channel of the Law Basin under the Greeley No. 2 Canal and State Highway 392. Transitions from the new channels to the culvert crossing are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing.

Stormwater Channel (Sta. 0+00 to 8+50). Construct approximately 850 feet of stormwater channel along the south side of State Highway 392 from the outlet of the proposed State Highway 392 crossing structure to the Greeley No. 2 Canal. The channel incorporates a grass lining, a bottom width of 8 feet, maximum top width of 60 feet, channel slope of 0.004 ft/ft, sideslopes of 4H:1V, and design depth of approximately 6 feet. A 3-foot drop structure and a 4-foot drop structure, respectively, will be required at the inlet and outlet to the channel.

Improvements to Greeley No. 2 Canal. Modifications to the existing check structure on the Greeley No. 2 Canal and construction of a new side-channel weir downstream of State Highway 392 will be required to divert as much as 210 cfs of stormwater flow conveyed by the canal due to upstream improvements. The side-channel weir incorporates a bottom width of 30 feet and a depth of 2 feet.

5.2.3 Law Basin-Reach L3

Improvements in Reach L3 focused on the alleviation of flooding problems at the intersection of WCR 21 and State Highway 392. The specific improvements to Reach L3 are presented below and in Figure 5.4.

Channel Improvements Sta. 166+00 to 186+00. Approximately 2,000 feet of channel improvements upstream of S.H. 392 will be required to capture and convey stormwater runoff captured by the main drainage channel in the Law Basin. The channel improvements should incorporate a grass lining, bottom width of 20 feet, maximum top

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LAW BASIN
REACH L3

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FIGURE 5.4

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width of 50 feet, channel slope of 0.003 ft/ft, sideslopes of 4H:1V, and design depth of 4 feet. At the upstream end of the channel improvements, approximately 600 feet of berming is required to capture the stormwater runoff and direct the flows into the proposed improvements.

Greeley No. 2 Canal Improvements. Approximately 2,600 lineal feet of the south and west bank of the Greeley No. 2 Canal between WCR 19 and State Highway 392 will be elevated. A maximum of 1.5 feet of fill will be required to provide approximately 1 foot of freeboard to convey the design discharge of 800 cfs (600 cfs of irrigation flows plus 200 cfs of stormwater during the 100-year event).

5.2.4 Law Basin-Reach L4

The proposed improvements to Reach L4 focus on the construction of new stormwater channels and the improvement of two existing crossing structures. Figure 5.5 presents the improvements for Reach L4.

Channel Improvements (Sta. 0+00 to 50+00). Construct approximately 5,000 feet of the West Tributary Channel from the Colorado & Southern Railroad confluence with the Main Channel to State Highway 392. The channel should incorporate a grass lining, a bottom width of 10 feet, maximum top width of 55 feet, channel slope of 0.0016 ft/ft, sideslopes of 4H:1V, and design depth of approximately 5 feet.

Consolidated Law Ditch Crossing. Construct a 36-inch diameter RCP, inverted siphon to convey Consolidated Law Ditch flows under the West Tributary stormwater channel improvements. The siphon will be approximately 100 feet in length.

Great Western Railroad Crossing. The existing 48-inch diameter CMP will be replaced with a crossing structure consisting of five 36-inch diameter RCPs. Transitions from the proposed channel to the culvert crossing are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing. The construction cost estimate assumes that the pipes will be bored under the tracks to facilitate railroad operations during construction. If construction will permit an open cut of the railroad tracks, a reinforced box culvert with an open area of approximately 40 square feet will be required.

State Highway 392 Crossing. The existing 24-inch diameter CMP will be replaced with a crossing structure consisting of two 3'H x 6'W reinforced concrete box culverts. Transitions from the existing channel upstream and the proposed channel downstream to the culvert crossing are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing.

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REACH L4

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FIGURE 5.5

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5.2.5 Law Basin-Reach L5

The proposed improvements to Reach L5 focus on the diversion of stormwater runoff into Kern Reservoir and the utilization of Kern Reservoir as a regional stormwater detention facility. The stormwater generated in both the Upper Windsor Basin and the West Tributary of the Law Basin will be captured and conveyed into Kern Reservoir. Modifications to the Greeley No. 2 Canal facilities are required to divert the stormwater into Kern Reservoir and convey the releases from the reservoir to the outfall channel improvements near State Highway 392. The proposed improvements encompass the West Tributary of the Law Basin as well as upper portion of the Windsor Basin. Figure 5.6 presents the improvements associated with Reach L5.

Kern Reservoir Enlargement. The sediment accumulated in Kern Reservoir will be excavated to create additional storage for irrigation water. This will allow the reservoir to be operated at a lower elevation (operating levels: current = 4,795 msl, NAVD 1988; proposed = 4,793.0 ft, msl, NAVD 1988) thereby creating additional storage for stormwater runoff. The required flood storage is 718 acre-feet.

Plug Existing Principal Spillway. The existing principal spillway presently controls the water surface in the reservoir at elevation 4,795.0 ft, msl, NAVD 1988 and releases as much as 60 cfs into the Lower Windsor Basin during the 100-year storm event. To prevent the stormwater releases into the Lower Windsor Basin during the 100-year event, it is recommended that the existing principal spillway be plugged. A new principal spillway can be incorporated into the new radial gate structure on the Greeley No. 2 Canal upstream of WCR 19.

Emergency Spillway Improvements. As a result of the proposed improvements to the Kern Reservoir, requirements for dam safety as promulgated by the Office of the State Engineer will need to be reviewed. It is anticipated that the results of this investigation will require modifications to the existing emergency spillway.

Greeley No. 2 Canal Improvements at Osterhaut Lake. The Greeley No. 2 Canal between the Colorado & Southern Railroad bridge and State Highway 257 (approximately 3,500 feet in length) will be enlarged to capture and convey stormwater emanating from the Upper Windsor Basin. The captured flows will be conveyed under State Highway 257 using the existing bridge and into Kern Reservoir using the existing inlet structure approximately 500 feet east of State Highway 257. The east bank of the canal will be elevated as much 5 feet and the canal cross section will need to enlarged. Channels necessary to convey stormwater runoff from the Upper Windsor Basin and transition into the Greeley No. 2 Canal will be required and are assumed to be provided during development of the property in the upper basin.

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LAW BASIN
REACH L5

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FIGURE 5.6

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Water Line Relocation. Currently, two raw water lines for the City of Greeley cross over the Greeley No. 2 Canal immediately downstream of the existing radial gates on the Kern Reservoir. As canal improvements are made to accommodate the capture and conveyance of stormwater in the basin, relocation of the raw water lines is anticipated.

Extend Pedestrian Bridge. Currently, a pedestrian bridge spans the Kern Reservoir outlet immediately upstream of the existing radial gates. When canal improvements are made to accommodate stormwater releases from the reservoir; it is anticipated that the pedestrian bridge will be extended.

Greeley No. 2 Canal/Kern Reservoir Outlet Improvements. To promote the capture and diversion of stormwater runoff from the West Tributary of the Law Basin into the Kern Reservoir, several improvements are necessary to the Greeley No. 2 Canal and the Kern Reservoir outlet canal. The banks of both canals will be raised a maximum of 5 feet. The banks should be elevated to contain and control the runoff from the future conditions 100-year storm event with out exceeding the elevation of the emergency spillway (4,797.0 ft, msl, NAVD 1988). The existing radial gates at the outlet will be relocated to Greeley No. 2 Canal in the vicinity of WCR 19. The existing measurement structure will also be relocated near WCR 19 Diversion of additional stormwater runoff from the Law Basin into Kern Reservoir will require coordination with the Office of the State Engineer to ensure all dam safety requirements are satisfied.

5.2.6 Law Basin-Reach L6

The proposed improvements to Reach L6 involve the diversion of stormwater runoff from the West Tributary of the Law Basin into Kern Reservoir, channel improvements to the Springer Ditch and installation of a crossing structure in State Highway 257. Figure 5.7 presents the improvements associated with Reach L6.

Channel Improvements. Construct approximately 2,400 feet of stormwater channel along the south side of WCR 70 west of WCR 19 to the confluence with the Springer Ditch. The channel will capture stormwater runoff from the Law Basin, specifically the Windsor Ranch Subdivision and the watershed tributary to the subdivision, and convey the runoff to the Springer Ditch. The channel incorporates a grass lining, a bottom width of 14 feet, maximum top width of 45 feet, channel slope ranging from 0.002 to 0.005 ft/ft, sideslopes of 4H:1V, and design depth of approximately 4 feet.

Springer Ditch Improvements (Sta. 15+00 to 46+00). Construct approximately 3,100 feet of improvements to the Springer Ditch from the confluence with the Greeley No. 2 Canal to State Highway 257. The improved channel should incorporate a grass lining, a bottom width of 15 feet, maximum top width of 65 feet, channel slope of 0.002 ft/ft, sideslopes of 4H:1V, and design depth of approximately 10 feet. Consideration should be given to a low-flow channel capable of conveying the normal irrigation flows in the Springer Ditch.

TOWN OF WINDSOR
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LAW BASIN
REACH L6

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FIGURE 5.7

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State Highway 257 Crossing. Construct a crossing structure consisting of a 3'H x 10'W reinforced concrete box culvert. The culvert is anticipated to be 40 feet in length and will convey stormwater flows generated west of State Highway 257 under the highway and into the Springer Ditch. Transitions from the new channels to the culvert crossing are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing. Minor berming may be required west of State Highway 257 to direct shallow sheet flooding to the crossing.

5.2.7 Windsor Basin-Reach W1

The improvements recommended in eastern portion of the Lower Windsor Basin are intended to minimize localized street flooding through the optimization of existing and proposed detention facilities and installation of storm sewer and channel improvements. Where appropriate, floodproofing measures have been recommended to minimize the damages from local street flooding. These improvements are presented in Figure 5.8.

Chimney Park Regional Detention Pond. To reduce peak flows in the lower Windsor Basin, a regional detention pond is proposed in the area east of Chimney Park. The proposed pond will incorporate a storage volume of 13 acre-feet and will require acquisition of 3 acres of land.

Folkstone Park Regional Detention Pond. To further reduce peak flows in the lower Windsor Basin, improvements are required to the existing detention facility at Folkstone Park. The existing pond will be enlarged to provide an additional 17.5 acre-feet of capacity. The park is currently owned by the Town of Windsor and therefore no property acquisition is required.

Chestnut Street Floodproofing Measures. Floodproofing measures are recommended for structures located along Chestnut Street. The exact number, location, and type of improvements should be determined following a detailed hydraulic study in this area.

Channel Improvements (Sta. 0+00 to 6+50). Construct approximately 650 feet of stormwater channel between Chestnut Street and Garden Drive. The channel will capture stormwater runoff on Chestnut Street and the drainage basin to the north and convey the runoff into the Folkstone Park Detention Pond. Due to space limitations, the proposed channel should incorporate vertical sideslopes, a channel slope of 0.0028 ft/ft, bottom width of 17 feet, and a design depth of approximately 4.5 feet.

Garden Drive Crossing. Construct a crossing structure consisting of a 4'H x 18'W reinforced concrete box culvert. The culvert is estimated to be 60 feet in length and will convey stormwater flows from north of Garden Drive under the road and into the Folkstone Park Detention Facility. Transitions from the new channel to the culvert crossing and from the culvert to the detention pond are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing.

SCALE
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TOWN OF WINDSOR
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WINDSOR BASIN
REACH W1

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FIGURE 5.8

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Folkstone Park Detention Pond Outfall. Install 2,900 feet of 36-inch diameter RCP storm sewer. The storm sewer will serve as an outfall facility for the Folkstone Park Detention Pond. The outfall pipe will convey stormwater to the roadside ditch located north of Eastman Park Drive.

5.2.8 Windsor Basin-Reach W2

The improvements recommended in the western portion of the Lower Windsor Basin are intended to minimize the damages associated with localized street flooding through the installation of floodproofing measures. These improvements are presented in Figure 5.9.

Oak Street Floodproofing Measures. Floodprofing measures are recommended for structures adjacent Oak Street between Third Street and Fourth Street. The exact number, location, and type of floodproofing measure should be determined following completion of a detailed hydraulic analysis.

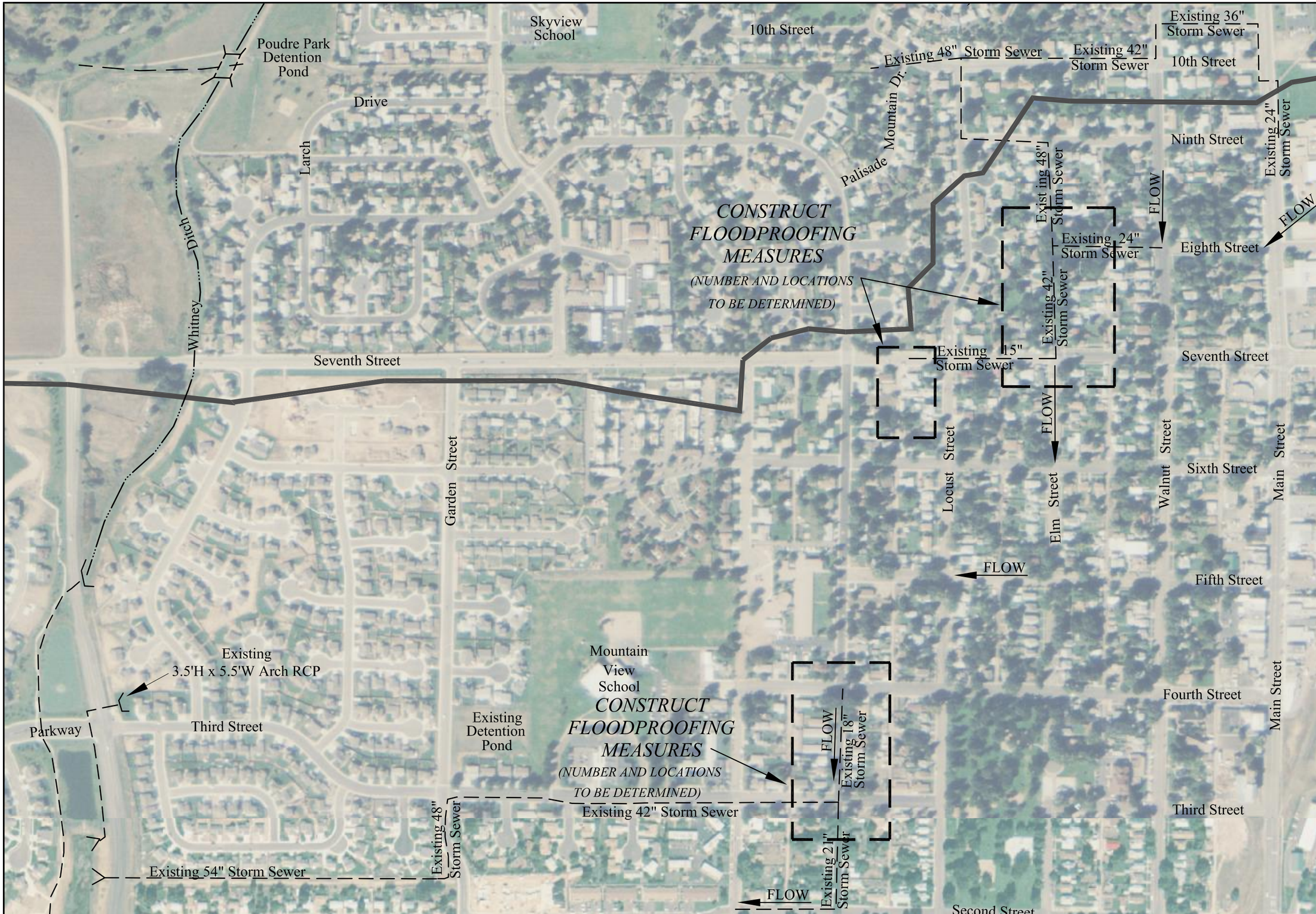
Locust Street Floodproofing Measures. Floodprofing measures are recommended for structures located between Locust Street and Oak Street directly east of Seventh Street. The exact number, location, and type of improvements should be determined following completion of a detailed hydraulic analysis.

Elm Street Floodproofing Measures. Floodprofing measures are recommended for structures along Elm Street between Seventh Street and Eighth Street. The exact number, location, and type of improvements should be determined following completion of a detailed hydraulic analysis.

5.2.9 High School Basin

Improvements in the High School Basin are intended to alleviate the potential flooding problems in the vicinity of Stone Mountain Drive. The specific improvements to the High School Basin are presented below and also on Figure 5.10.

10th Street Channel Outfall. Construct an outfall channel from the Poudre Park Detention Pond to the Cache la Poudre River. The outfall channel is approximately 600 feet in length and incorporates a bottom width of 40 feet, depth of 5 feet, channel slope of 0.001 ft/ft, grass-lined sideslopes of 3H:1V, and a stabilized transition into the Cache la Poudre River.



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TOWN OF WINDSOR
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REACH W2

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FIGURE 5.9

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TOWN OF WINDSOR
MASTER DRAINAGE PLAN
HIGH SCHOOL BASIN
REACH H

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FIGURE 5.10

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Whitney Ditch Siphon. A siphon will be installed under the outfall channel to convey irrigation diversions associated with the Whitney Ditch. The siphon will be approximately 60 feet in length and will consist of a 60-inch RCP with inlet and outlet transition structures.

10th Street Channel - Downstream of Stone Mountain Drive. Improve approximately 700 lineal feet of channel between the Poudre Park Detention Pond and Stone Mountain Drive. The channel improvements include a grass lining, bottom width of 10 feet, maximum top width of 35 feet, channel slope of 0.005 ft/ft, sideslopes of 3H:1V, and a design depth of approximately 4.0 feet.

Stone Mountain Drive Crossing Improvements. Replace the existing triple-30" CMP crossing of Stone Mountain Drive with two 4'H x 9'W reinforced box culverts. Transitions from the existing channel to the culvert crossing are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing.

10th Street Conveyance Improvements. During large storm events, runoff is collected and conveyed in 10th Street and ultimately the 10th Street Stormwater Channel. To promote the safe conveyance of stormwater into the 10th Street Channel, improvements are required at two locations. Improvements at the intersection of Palisade Mountain Drive and 10th Street are recommended to divert stormwater flows on the east side of the street into the 10th Street Channel. The improvement may include installation of a cross pan, localized lowering of the curbs/gutters and sidewalks, and construction of a swale from Palisade Mountain Drive to the outlet of the existing 48-inch diameter storm sewer. In addition, improvements are recommended at the end of the cul-de-sac where 10th Street turns to the west (see Figure 5.10). A drainage easement should be preserved at this location to allow stormwater runoff which collects in the cul-de-sac to be conveyed to the 10th Street Channel. The drainage improvements from the cul-de-sac may include construction of drainage swales, and lowering of street curbs/sidewalks.

5.2.10 Jacoby Basin

Improvements in Jacoby Basin focused on the alleviation of flooding problems at WCR 68 ½, the Greeley No. 2 Canal, and at State Highway 392. The specific improvements to Jacoby Basin are presented below and also on Figure 5.11.

WCR 15 Storm Sewer. Construct approximately 1,300 feet of 60-inch diameter, RCP storm sewer along the west side of WCR 15. The storm sewer will transition into the existing 60-inch RCP storm sewer at State Highway 392 and terminate on the north side of the Greeley No. 2 Canal. A 60-inch RCP siphon will be required near the Greeley No. 2 Canal to convey stormwater runoff beneath the canal facilities.

TOWN OF WINDSOR
MASTER DRAINAGE PLAN
JACOBY BASIN
REACH J

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FIGURE 5.11

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Limiting Stormwater Releases from Subbasins J11 and J12. It is recommended that future development (those parcels platted and approved for construction after June 1, 2000) north of the Greeley No. 2 Canal and west of WCR 15 limit the release of stormwater to approximately 0.2 cfs/acre. This recommended improvement will ensure that the capacity of the existing 72-inch diameter storm sewer is not exceeded.

5.2.11 Timnath Reservoir Basin

Improvements in the Timnath Reservoir Basin are intended to mitigate the potential flooding problems at LCR 32½ and the Greeley No. 2 Canal through the construction of a regional drainage channel, improved crossing structure and siphon/wasteway structure on the Greeley No. 2 Canal. The specific improvements in this basin are presented below and on Figure 5.12.

Channel Improvements. Construct approximately 6,000 feet of channel improvements from the Cache la Poudre River to the Timnath Reservoir Outlet Ditch located north of the Greeley No. 2 Canal. The channel configuration includes a compound channel, i.e., a low flow channel along with an overbank area to convey flood flows. In addition, the channel incorporates a grass lining, bottom width ranging from 35 to 40 feet, channel slope of approximately 0.003 ft/ft, sideslopes of 4H:1V, and a design depth ranging from 5 to 6.5 feet. At the downstream end of the channel improvements, a transition into the Cache la Poudre River is required to convey flows safely and in a non-erosive manner to the river. In a similar manner, a transition structure will be required near the Timnath Reservoir Outlet Ditch to convey stormwater into the channel improvements.

LCR 32½ Crossing Structure. Replace the existing 30-inch diameter CMP crossing of LCR 32½ with two 6'H x 10'W reinforced concrete box culverts. Transitions from the new channel to the culvert crossing are required along with placement of stabilization measures to control the potential erosion in the vicinity of the crossing.

Greeley Number 2 Improvements. In order to safely convey stormwater from the Timnath Reservoir Basin through the facilities associated with the Greeley No. 2 Canal, the following improvements are required: (a) an inverted siphon to convey irrigation flows underneath the new stormwater channel, (b) a wasteway structure upstream of the inverted siphon, and (c) a headgate/diversion structure at the upstream end of the channel improvements. The inverted siphon will consist of two 6'H x 10'W reinforced concrete box culverts. The wasteway structure will spill stormwater flows captured by the canal, which cannot pass through the new siphon. The headgate/diversion structure is required to allow conveyance of irrigation flows in the Timnath Reservoir Outlet Ditch into the Greeley No. 2 Canal while diverting stormwater flows into the channel improvements.

N

SCALE
1" = 500'

TOWN OF WINDSOR
MASTER DRAINAGE PLAN
TIMMATH RESERVOIR BASIN
REACH T

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Drawn:	MRC
Revisions:	05/05/2003
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FIGURE 5.12

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5.2.12 Ptarmigan Basin

The improvements in the Ptarmigan Basin are intended to alleviate the potential flooding identified at two crossings (Subbasin P14 and Subbasin P09) of State Highway 392. The specific improvements to Ptarmigan Basin are presented below and in Figure 5.13.

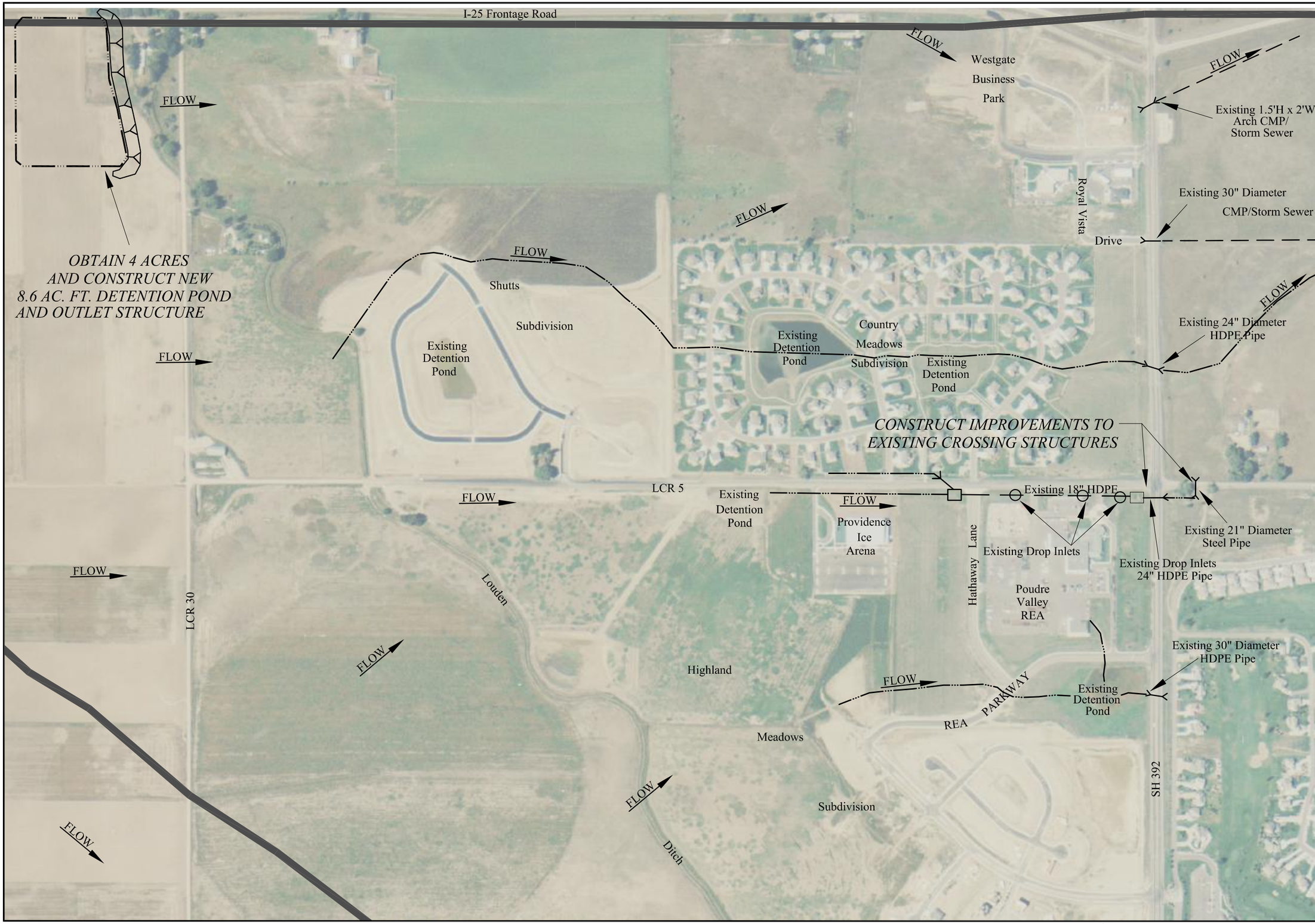
LCR 5 Crossing Structure. Improve the existing 21-inch diameter steel pipe crossing LCR 5 north of State Highway 392. The improvements consist of: (a) enlarging the existing drainage channel located north of State Highway 392 to the inlet of the 21-inch steel pipe, (b) installation of improved inlet and outlet structures (headwalls/flared end sections/trash rack), and (c) removal of all sediment within, and adjacent to the existing crossing structure.

State Highway 392 Crossing Structure. Improve the existing 24-inch diameter HDPE pipe crossing immediately east of LCR 5. The improvements consist of: (a) enlarging the existing drainage channel immediately south of the crossing, (b) constructing improvements to the existing drop inlet structures into the existing 18-inch diameter HDPE outfall pipe; (c) installing improved inlet and outlet structures (headwalls, flared end sections, trash rack, etc), and (d) removal of all sediment within, and adjacent to, the existing crossing structure.

LCR 30 Detention Pond. To reduce the peak flows at the State Highway 392 crossing in Subbasin P14, a regional detention pond is proposed south of LCR 30 and east of I-25. The pond is located upstream of the existing Louden Ditch. The pond requires 8.6 acre-feet of storage, which will necessitate the acquisition of approximately 4 acres of land along with the construction of an outlet structure under the Louden Ditch. The outfall channel from the detention pond to State Highway 392 is assumed to be included in the construction associated with development of the property between LCR 30 and State Highway 392.

5.2.13 River Ridge Basin

No flooding improvements in the River Ridge Basin are recommended assuming the enforcement of on-site over-detention within the basin. Special attention should be focused on potential channel stability problems within the main drainage channel as development occurs in the basin.



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TOWN OF WINDSOR
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PTARMIGAN BASIN
REACH P

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FIGURE
5.13

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5.2.14 Bluff Basin

No flooding improvements in the Bluff Basin are recommended assuming the enforcement of on-site over-detention within the basin. Similar to the River Ridge Basin, special attention should be focused on erosional issues as develop occurs in the basin.

5.2.15 Oklahoma Basin-Reach O1

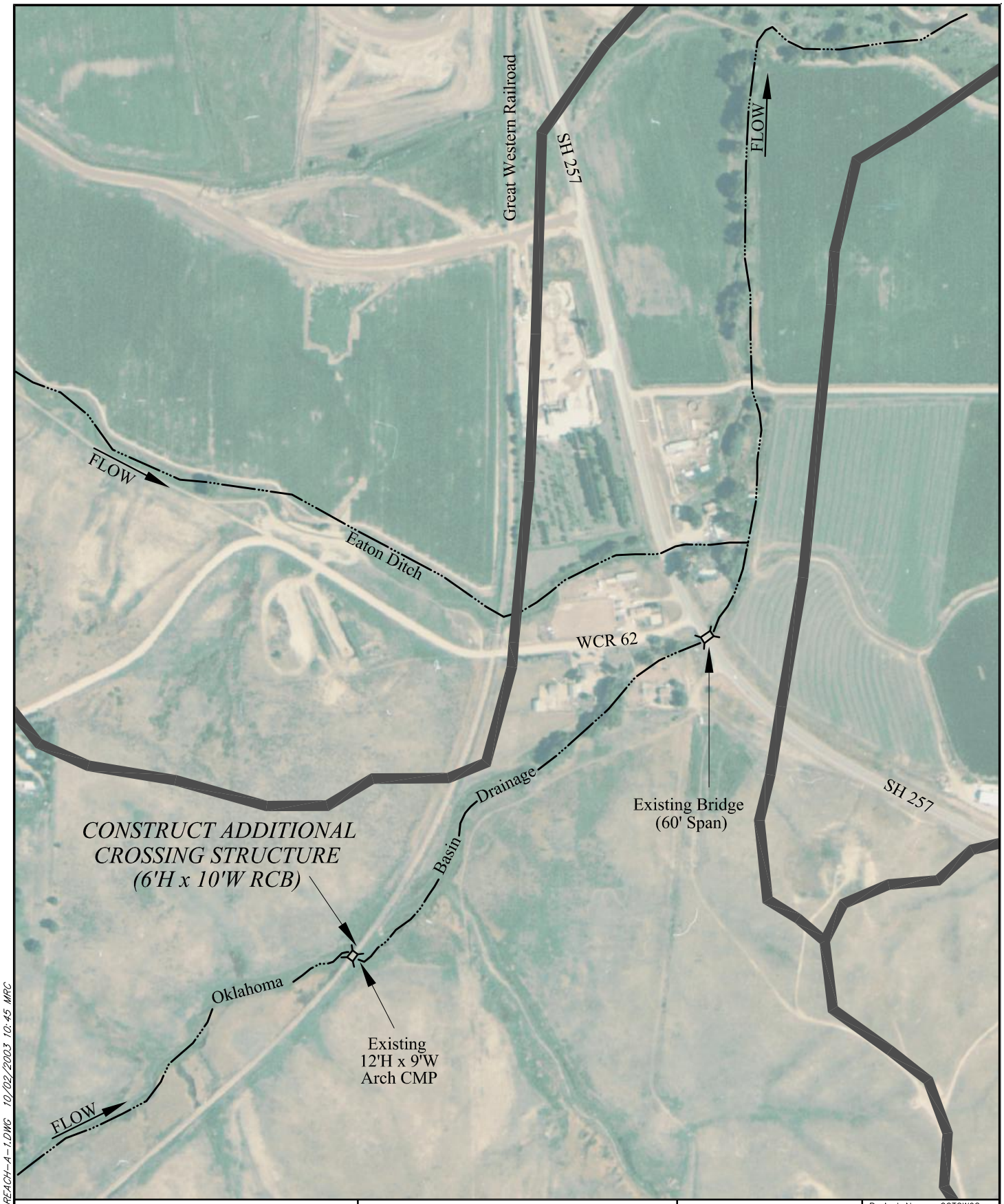
The improvements in Reach O1 involve the construction of an additional crossing structure at the Great Western Railroad. In addition to the existing 12'H x 9'W arch CMP, a 6'H x 10'W reinforced concrete box culvert is recommended. Erosion protection is also recommended for both structures. Localized channel work will be required to provide transitions from the existing channel to the improved culverts. These improvements are presented on Figure 5.14.

5.2.16 Oklahoma Basin-Reach O2

During the master planning effort, the longevity of the existing crossing structure at WCR was estimated to be less than 10 years due to the deterioration of the concrete box culvert. Furthermore, the inadvertent detention provided behind the road embankment provides significant reduction in the peak discharges during the major storm events. Consequently, the improvements in Reach O2 focused on: (a) replacement of the existing culvert at WCR 17; and (b) preservation of the existing detention area upstream of WCR 17. The specific improvements to Reach O2 are presented below and in Figure 5.15.

<u>WCR 17 Crossing.</u> This improvement involves the replacement of existing 8.5'H x 5.5'W reinforced box culvert which is in poor condition. The replacement structure recommended for this crossing should provide similar hydraulic characteristics as the existing culvert. Similar operational characteristics are required because this structure acts as the outlet control to the existing inadvertent/regional detention pond located on the upstream side of WCR 17. For the purposes of this master planning effort, an 8.5'H x 5.5'W RCB is recommended for this crossing.

<u>Regional Detention Pond.</u> To insure continued reduction of peak flows within the major drainageway of the Oklahoma Basin, the inadvertent storage/regional detention pond upstream of WCR 17 should be purchased and preserved. At this location, approximately 235 acre-feet of storage are available which will necessitate the acquisition of approximately 18 acres of land.



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TOWN OF WINDSOR
MASTER DRAINAGE PLAN
OKLAHOMA BASIN
REACH O1

SCALE
1" = 500'

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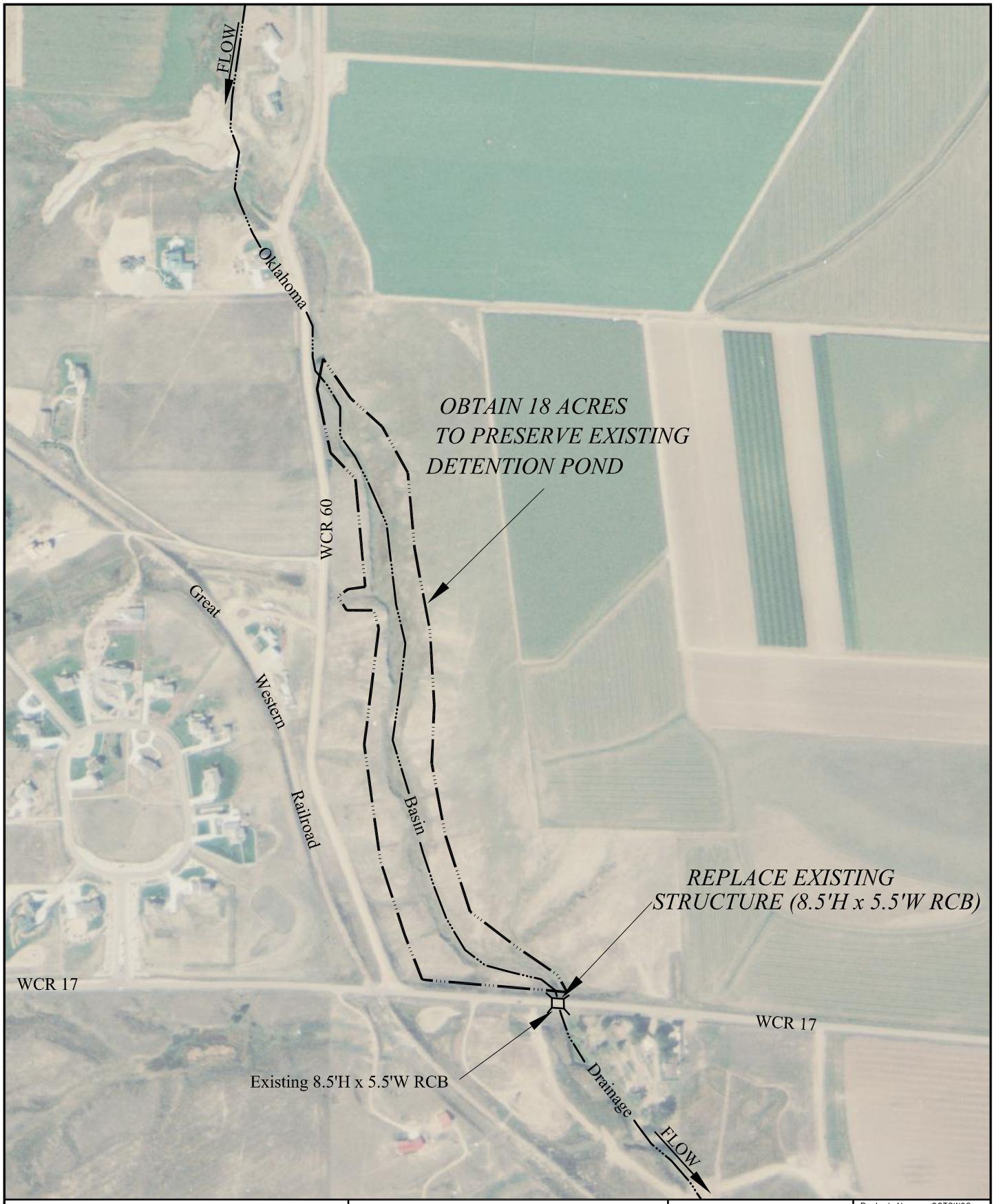
FIGURE 5.14



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TOWN OF WINDSOR
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OKLAHOMA BASIN
REACH O2

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FIGURE 5.15

5.2.17 South State Highway 257 Basin

The improvements identified for this basin are intended to reduce the potential overtopping of State Highway 257 during the 100-year flow event. The existing crossing structure consists of two 6'H x 10'W reinforced concrete box culverts with a capacity of 1,075 cfs prior to overtopping the roadway. Channel improvements along the west bank upstream of the crossing structure will increase the capacity of the existing structure to approximately 1,500 cfs prior to overtopping the roadway. The 100-year peak flow at this location is estimated to be 1,565 cfs. The recommended improvements consist of placing compacted fill along the west bank in the vicinity of the existing driveway access. The elevation of the compacted fill should not be less than 4,791 ft, msl. This improvement is presented on Figure 5.16.

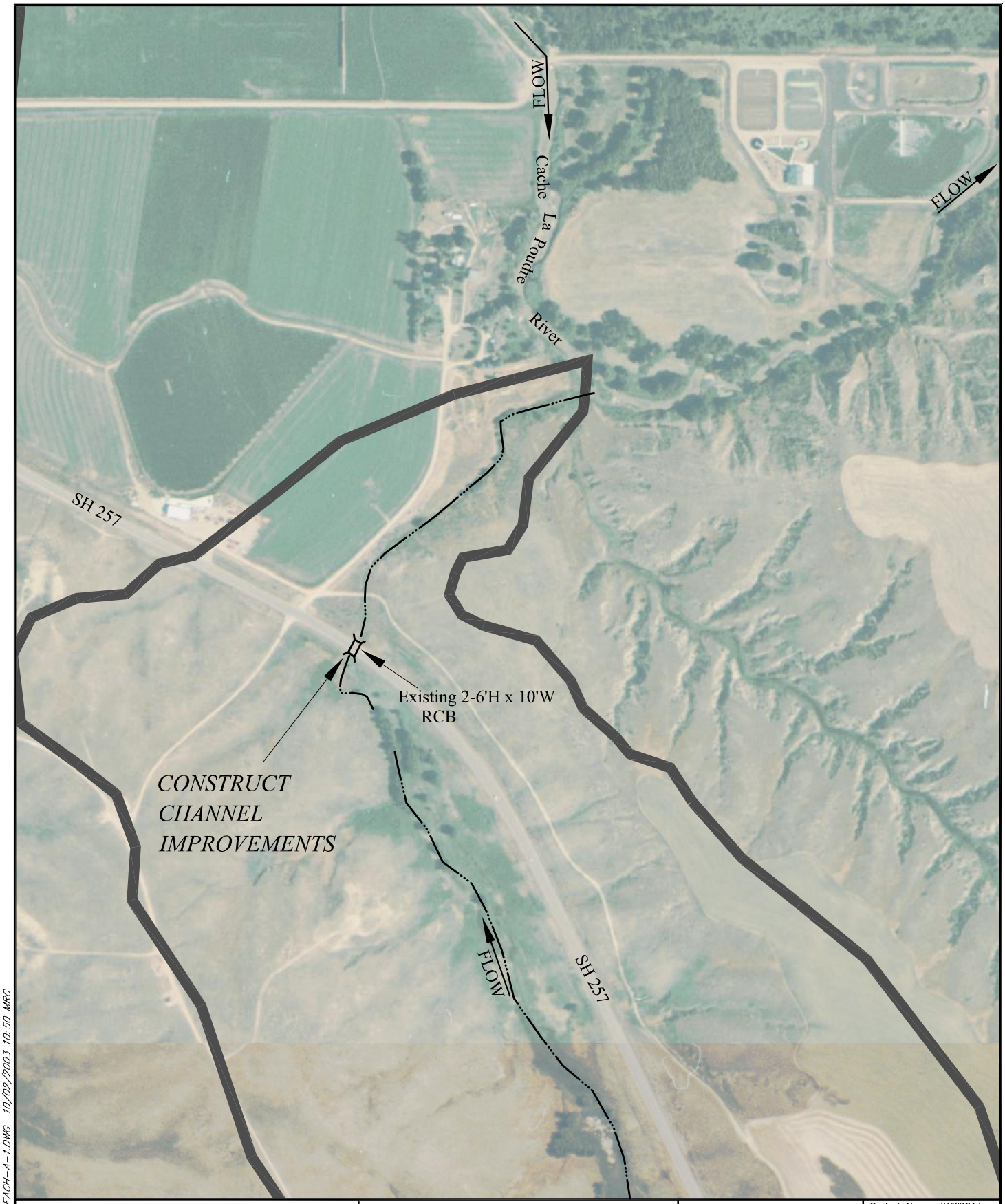
5.2.18 Additional Considerations

As indicated previously, road crossings of several tributaries to the major drainageways also exist. Several of these crossings lack the capacity to convey the peak discharge from the major storm events for existing conditions. In addition, several of the crossings were noted to be experiencing potential sediment and debris problems that would tend to reduce the conveyance capacity. As land development occurs within the basin, these crossing structures will require improvements to meet the existing drainage criteria. Improvements to these structures should consider the peak discharges conveyed through or over these crossings during the major storm event. In addition, the capacity and stability of downstream conveyance facilities should be considered.

As stated previously, the drainage improvements described in this chapter pertain to the major drainageways within each drainage basin. As future development occurs in each basin, it is assumed that on-site drainage facilities will be provided to convey stormwater runoff to the improvements identified in the master plan.

5.3 Drainage Criteria

It is assumed that land development within the growth management boundary will be governed by the guidelines and recommendations provided in this master planning document. The following information is presented to guide development within this area as well as areas contributing stormwater to the Town of Windsor.



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TOWN OF WINDSOR
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SOUTH SH 257 BASIN
REACH S

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Drawn:	MRC
Revisions:	11/12/2002
ACADFILE:	REACH-A-1.DWG

FIGURE 5.16

- Land development in the drainage basins will be required to limit the stormwater runoff from the 100-year storm event to the magnitude of the existing condition 10-year storm event.

- Land development in the Ptarmigan Basin will be required to limit the stormwater runoff from the 100-year storm event for the area located south of State Highway 392 to the available capacity of existing facilities under State Highway 392.

- Land development in the Jacoby Basin, upstream of the Greeley No. 2 Canal and west of WCR 15, will be required to limit stormwater runoff from the 100-year storm event to 0.2 cfs/acre.

- Stormwater generated from the development of land in each subbasin will be conveyed in a safe and stable manner to the major drainageway. This statement implies that stormwater runoff will be conveyed in an appropriate outfall facility such as a channel, storm sewer, etc.

- All existing detention and retention areas that are effective in reducing the runoff generated during the major storm events, must remain intact or be replaced.

5.4 Cost Estimates

Estimates of the costs were developed for the proposed improvements and provided the basis for the opinion of costs generated for the comprehensive plans for each basin. Data used to develop unit costs were obtained from bid tabulations, quotations from various suppliers and manufacturers, and information supplied by local contractors and the Town of Windsor Engineering Department. The unit pricing data compiled and used to generate the cost estimates is provided in the project notebook. The cost estimates for the comprehensive plans included the following categories: (a) general drainage infrastructure; and (b) engineering, permitting, legal, fiscal and administrative costs.

General drainage infrastructure costs are defined as the costs associated with the labor and materials for the construction of drainage improvements. This item also includes acquisition of the drainage ROW/easements or land necessary for the construction and maintenance of the drainage improvements. A contingency of 35% was included in the estimate of construction costs to include those real and intangible items not directly accounted for in the unit pricing data and the costs associated with mobilization, utility relocation, etc.

The final cost category was estimated to be approximately 20 percent of total construction costs. This category is intended to include professional engineering and construction services, labor required to obtain the necessary permits, and legal requirements.

Administrative costs relate to the implementation of the construction plan by the various governmental agencies.

The cost estimates for the comprehensive plans are presented in Tables 5.1 and 5.2. Table 5.1 presents the cost estimates generated for each improvement in each drainage basin. Table 5.2 provides a summary, by basin, of the total costs to construct the improvements.

5.5 Hydrologic Analysis

Following the formulation and evaluation of the drainage improvements, the future condition hydrologic model (with over-detention) was modified to reflect the improvements identified for the comprehensive plans. Hydrologically, the plans reflect the reduction in peak discharges provided by the improved facilities. Discharge data assuming implementation of the comprehensive plans are presented in tabular form in Table 5.3. For illustrative purposes, Figure 5.17 presents the 100-year peak discharge data at several locations within the watershed based on the hydrologic model of future conditions with over-detention along with construction of the proposed improvements..

Table 5.1 Cost Estimates for Drainage Improvements with Over-Detention.

Reach-Item	Description	Proposed Improvements/ General Drainage Infrastructure	Property Acquisition	Engineering, Permitting, Legal & Admin.	Total Cost
		LAW BASIN			
L1-1	*New Channel (Sta.0+00 to 72+00)*	$1,253,000	$209,000	$250,000	$1,712,000
L1-2	Bike Trail Crossing Structure	$67,000	$0	$13,000	$80,000
L1-3	Whitney Ditch Siphon	$38,000	$0	$7,000	$45,000
				Reach L1 Subtotal	**$1,837,000**
L2-1	New Channel (72+00 to 136+00)	$810,000	$152,000	$162,000	$1,124,000
L2-2	Eastman Park Crossing Structure	$125,000	$0	$25,000	$150,000
L2-3	C&S Railroad Crossing Structure	$130,000	$5,000(1)	$26,000	$161,000
L2-4	New Channel (Sta. 136+00 to 163+00)	$342,000	$59,000	$68,000	$469,000
L2-5	State Highway 392/Greeley No. 2 Crossing	$477,000	$2,000	$95,000	$574,000
L2-6	New Channel (Sta. 0+00 to 8+50)	$90,000	$14,000	$18,000	$122,000
L2-7	Check Structure and Wasteway	$28,000	$2,000	$5,000	$35,000
				Reach L2 Subtotal	**$2,635,000**
L3-1	New Channel (Sta. 166+00 to 186+00)	$89,000	$28,000	$18,000	$135,000
L3-2	Greeley No. 2 Bank Improvements WCR19/State Highway 392	$8,300	$0	$1,700	$10,000
				Reach L3 Subtotal	**$145,000**
L4-1	New Channel (Sta. 0+00 to 50+00)	$237,000	$74,000	$47,000	$358,000
L4-2	Consolidated Law Ditch Siphon	$22,000	$0	$4,000	$26,000
L4-3	G.W. Railroad Crossing Structure	$177,000	$5,000(1)	$35,000	$217,000
L4-4	State Highway 392 Crossing Structure	$126,000	$0	$25,000	$151,000
				Reach L4 Subtotal	**$752,000**
L5-1	Kern Reservoir Excavation	$3,027,000	$0	$606,000	$3,633,000
L5-2	Plug Existing Principal Spillway	$7,000	$0	$2,000	$9,000
L5-3	Emergency Spillway Improvements	$135,000	$0	$27,000	$162,000
L5-4	Greeley No. 2 Bank Improvements at Osterhaut Lake	$148,000	$40,000	$29,000	$217,000
L5-5	Lower Water Lines	$61,000	$0	$12,000	$73,000
L5-6	Extend Pedestrian Bridge	$41,000	$5,000	$8,000	$54,000
L5-7	Greeley No. 2 Channel/Kern Reservoir Outlet Improvements	$332,000	$22,000	$66,000	$420,000
				Reach L5 Subtotal	**$4,568,000**
L6-1	Windsor Ranch Channel	$68,000	$30,000	$13,000	$111,000
L6-2	Springer Ditch Improvements	$89,000	$46,000	$18,000	$153,000
L6-3	State Highway 257 Crossing Structure (Law Basin)	$65,000	$10,000	$13,000	$88,000
				Reach L6 Subtotal	**$352,000**
				Law Basin Total Cost	**$10,289,000**

Table 5.1 Cost Estimates for Drainage Improvements with Over-Detention (continued).

Reach-Item	Description	Proposed Improvements/ General Drainage Infrastructure	Property Acquisition	Engineering, Permitting, Legal & Admin.	Total Cost
WINDSOR BASIN					
W1-1	36" Storm Sewer	$367,000	$0	$73,000	$440,000
W1-2	Folkstone Detention Pond Improvements	$124,000	$0	$25,000	$149,000
W1-3	Garden Street Crossing Structure	$138,000	$0	$28,000	$166,000
W1-4	New Channel (Sta. 0+00 to 6+50)	$430,000	$13,000	$86,000	$529,000
W1-5	Chimney Park Detention Pond	$175,000	$150,000	$35,000	$360,000
W1-6	1st St. & Chestnut St. Floodproofing	$81,000	$0	$16,000	$97,000
				Reach W1 Subtotal	**$1,741,000**
W2-1	7th & Elm St. Floodproofing	$202,000	$0	$41,000	$243,000
W2-2	3rd St. & Oak St. Floodproofing	$203,000	$0	$40,000	$243,000
				Reach W2 Subtotal	**$486,000**
				Windsor Basin Total Cost	**$2,227,000**
HIGH SCHOOL BASIN					
H-1	10th Street Outfall Channel	$61,000	$11,000	$12,000	$84,000
H-2	Whitney Ditch Siphon	$31,000	$0	$6,000	$37,000
H-3	10th Street Channel Improvements	$30,000	$0	$6,000	$36,000
H-4	Stone Mountain Crossing Structure	$148,000	$0	$30,000	$178,000
H-5	10th Street Improvements	$31,000	$0	$6,000	$37,000
				High School Basin Total Cost	**$372,000**
JACOBY BASIN					
J-1	WCR 15 Storm Sewer	$385,000	$0	$77,000	$462,000
J-2	Over-Detention in Subbasins J11&J12	$0	$0	$0	$0
				Jacoby Basin Total Cost	**$462,000**
TIMNATH RESERVOIR BASIN					
T-1	Cache la Poudre Transition	$108,000	$3,000	$22,000	$133,000
T-2	New Channel (Sta. 1+00 to 15+70)	$228,000	$43,000	$46,000	$317,000
T-3	LCR 32 ½ Crossing Structure	$102,000	$0	$20,000	$122,000
T-4	New Channel (Sta. 16+00 to 59+50)	$322,000	$85,000	$64,000	$471,000
T-5	Greeley No. 2 Siphon, Wasteway, and Turnout	$319,000	$0	$64,000	$383,000
				Timnath Reservoir Basin Total Cost	**$1,426,000**

Table 5.1 Cost Estimates for Drainage Improvements with Over-Detention (continued).

Reach-Item	Description	Proposed Improvements/ General Drainage Infrastructure	Property Acquisition	Engineering, Permitting, Legal & Admin.	Total Cost
PTARMIGAN BASIN					
P-1	LCR 5 Culvert and Swale	$11,000	$0	$2,000	$13,000
P-2	State Highway 392 Culvert and Swale	$13,000	$0	$3,000	$16,000
P-3	LCR 30 Detention Pond	$88,000	$40,000	$17,000	$145,000
				Ptarmigan Basin Total Cost	**$174,000**
RIVER RIDGE BASIN					
No Master Plan Improvements recommended for this basin.					
BLUFF BASIN					
No Master Plan Improvements recommended for this basin.					
OKLAHOMA BASIN					
O1-1	Enlargement of G.W. Railroad Crossing	$136,000	$0	$27,000	$163,000
				Reach O1 Subtotal	**$163,000**
O2-1	Replacement of WCR 17 Crossing[(2)]	$244,000	$0	$49,000	$293,000
O2-2	WCR 17 Detention Pond	$0	$54,000	$5,000	$59,000
				Reach O2 Subtotal	**$352,000**
				Oklahoma Basin Total Cost	**$515,000**
SOUTH STATE HIGHWAY 257 BASIN					
S-1	Channel Improvements at State Highway 257 Crossing	$10,000	$0	$2,000	$12,000
				South State Highway 257 Basin Total Cost	**$12,000**

[(1)]In railroad right-of-way, cost to obtain permits from railroad.
[(2)]Possibly funded by Weld County during WCR17 improvements.

Table 5.2 Summary of Basin Costs.

Basin	Total Cost of Improvements With Over-Detention
Law Basin	$10,289,000
Windsor Basin	$2,227,000
High School Basin	$372,000
Jacoby Basin	$462,000
Timnath Reservoir Basin	$1,426,000
Ptarmigan Basin	$174,000
River Ridge Basin	$0
Bluff Basin	$0
Oklahoma Basin	$515,000
South State Highway 257 Basin	$12,000
Total Cost Drainage Improvements	**$15,477,000**

Table 5.3 Major Drainageway Peak Discharges for Future Conditions with Improvements.

Location	EPA SWMM Element	Drainage Area (acres)	Peak Discharge (cfs)					
			2-yr	5-yr	10-yr	25-yr	50-yr	100-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	94	140	178	343	592	1,012
C&S Railroad.	L208	16,135	82	122	154	309	550	972
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	L910	4,676	51	72	88	133	181	238
Great Western Railroad**	L210	4,676	651	672	688	733	781	838
Greeley No. 2 Canal at WCR 19**	W141	3,929	641	659	671	709	748	793
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	59	89	108	220	329	470
Eastman Park Drive East of 1st St.	W220	3,553	26	36	42	61	84	118
Folkstone Park Pond	W323	3,454	9	14	16	24	30	37
Garden Drive	W723	3,454	76	113	138	202	260	335
Chestnut Street	W223	3,392	59	86	104	148	197	247
Chimney Park	W128	3,158	6	9	11	16	21	26
Walnut Street	W229	3,116	30	46	55	77	93	113
C&S Railroad	W240	3,089	9	13	15	23	30	39
WINDSOR BASIN – UPPER								
Kern Reservoir Outflow**	W341	3,043	642	659	672	710	749	795
Kern Reservoir Inflow**	W241	3,043	1,170	1,455	1,647	2,039	2,289	2,748
State Highway 257**	W742	2,883	753	829	878	1,079	1,269	1,505
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

* Discharge values include assumed irrigation base flow of 150 cfs.
** Discharge values include assumed irrigation base flow of 600 cfs.
*** Includes Kern Reservoir releases carried in Greeley No. 2 Canal to be spilled into Law Ditch Main Channel.

Table 5.3 Major Drainageway Peak Discharges for Future Conditions with Improvements (continued).

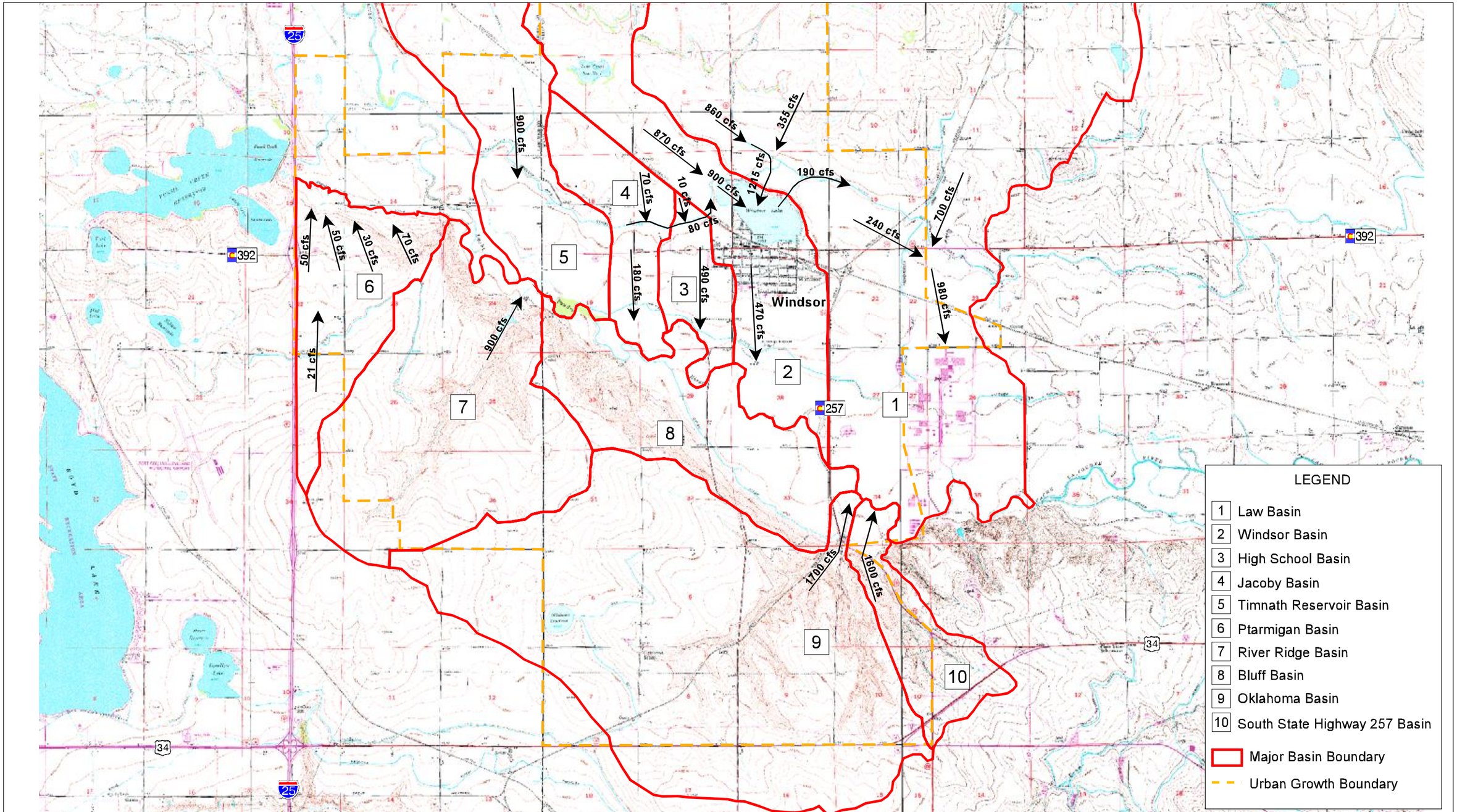
Location	EPA SWMM Element	Drainage Area (acres)	Peak Discharge (cfs)					
			2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
WINDSOR BASIN – UPPER (continued)								
Greeley No. 2 Canal at C&S Railroad**	GRNO2B	173	613	619	623	641	659	681
High School Basin at Greeley No. 2 Canal	H410	35	2	3	3	5	6	8
Jacoby Basin at Greeley No. 2 Canal	J208	138	11	16	20	37	53	73
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
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	4	6	7	10	13	16
Greeley No. 2 Canal	H410	n/a	Flows captured by canal and conveyed to Kern Reservoir.					
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	83	112
Walnut Street	J206	777	6	8	10	14	17	20
State Highway 392	J707	692	32	47	57	97	131	176
State Highway 392 East Path	J207	195	8	12	14	30	44	60
Greeley No. 2 Canal East Path	J208	n/a	Flows captured by canal and conveyed to Kern Reservoir.					
State Highway 392 West Path	J210	496	24	36	43	69	92	121
Greeley No. 2 Canal West Path	J211	457	19	28	34	53	69	89
72-inch Storm Sewer	J500	692	32	47	57	97	131	176
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

* Discharge values include assumed irrigation base flow of 150 cfs.
** Discharge values include assumed irrigation base flow of 600 cfs.
*** Includes Kern Reservoir releases carried in Greeley No. 2 Canal to be spilled into Law Ditch Main Channel.

Table 5.3 Major Drainageway Peak Discharges for Future Conditions with Improvements (continued).

Location	EPA SWMM Element	Drainage Area (acres)	Peak Discharge (cfs)					
			2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
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	41	60	73	114	145	184
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
State Highway 392 at I-25	P214	339	14	20	24	34	41	50
Inflow to Westgate	P215	298	6	9	11	20	27	35
LCR 30 at I-25 Frontage Road	P516	212	1	1	3	9	14	21
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 major drainageway for this subbasin. See subbasin peak runoff table (Appendix B.3) 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								
No improvements that will alter peak discharges in this basin; major drainageway peak discharges same as in future conditions.								

* Discharge values include assumed irrigation base flow of 150 cfs.
** Discharge values include assumed irrigation base flow of 600 cfs.
*** Includes Kern Reservoir releases carried in Greeley No. 2 Canal to be spilled into Law Ditch Main Channel.



REVISIONS:

N

Contour Interval
10 feet

2500 0 2500 5000 7500 10000
Feet

PROJECT NUMBER: COTOW08	DRAWN BY: RJN
FILENAME: Windsor-masterplan.apr Future Condition - Over Detention with Improvements 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
OVER-DETENTION (100-10)
WITH IMPROVEMENTS
100-YEAR PEAK DISCHARGES

FIGURE 5.17

VI. IMPLEMENTATION PLAN

6.1 Plan Evaluation

To promote the construction of the drainage improvements as funding becomes available, implementation priorities were established and an implementation plan developed. The implementation and phasing of the drainage improvements are dependent on several factors. The factors described below were utilized to establish the priority of implementation for the improvements.

- Improvements that reduced the health and safety hazards to the public and vehicular traffic were considered the highest priority. For example, crossings of State Highway 392 and State Highway 257 received a high priority as well as those areas where potential flooding occurs in highly urbanized areas.

- Improvements to areas likely to incur the most flood damages were considered to be the next highest priority.

- Construction phasing of adjacent improvements was considered. For example, improving a culvert crossing may significantly reduce flood damage upstream of the crossing; however, the downstream channel must be improved in conjunction with the roadway crossing to prevent an increase in flood damages on the downstream property.

6.2 Recommended Plan

Results of the implementation planning efforts are presented in Table 6.1. It should be noted that obstacles that hinder the implementation of the comprehensive plans are frequently encountered. In many instances, these obstacles could have been addressed or considered much earlier in the planning process. Consequently, administration of the plan should provide for immediate consideration of: (a) acquisition of property, easements and rights-of-way necessary to construct proposed improvements; (b) identification of potential utility conflicts that will require resolution prior to construction of the improvements; and (c) acquisition of permits required prior to construction of the improvements.

Table 6.1 Implementation Plan for Town of Windsor.

Reach-Item	Description	Cost	Subtotal	Priority
L2-5	State Highway 392/Greeley No. 2 Crossing	$574,000	$574,000	1
L3-1	New Channel (Sta. 166+00 to 186+00)	$135,000	$709,000	1
L4-4	State Highway 392 Crossing Structure	$151,000	$860,000	1
L5-1	Kern Reservoir Excavation	$3,633,000	$4,493,000	1
L5-3	Kern Reservoir Spillway Improvements	$162,000	$4,655,000	1
L5-5	Lower Water Lines	$73,000	$4,728,000	1
L5-6	Extend Pedestrian Bridge	$54,000	$4,782,000	1
L5-7	Channel/Reservoir Outlet Improvements	$420,000	$5,202,000	1
L6-3	State Highway 257 Crossing Structure (Law Basin)	$88,000	$5,290,000	1
W1-1	Storm Sewer (36-inch)	$440,000	$5,730,000	1
W1-2	Folkstone Detention Pond Improvements	$149,000	$5,879,000	1
W1-3	Garden Street Crossing Improvements	$166,000	$6,045,000	1
W1-4	Channel Improvements	$529,000	$6,574,000	1
W1-5	Chimney Park Detention Improvements	$360,000	$6,934,000	1
W1-6	1st St/Chestnut St Floodproofing	$97,000	$7,031,000	1
W2-1	7th St/Elm St Floodproofing	$243,000	$7,274,000	1
W2-2	3rd St/Oak St Floodproofing	$243,000	$7,517,000	1
H-1	10th Street Outfall Channel	$84,000	$7,601,000	1
H-3	10th Street Channel Improvements	$36,000	$7,637,000	1
H-4	Stone Mountain Crossing Structure	$178,000	$7,815,000	1
J-1	WCR 15 Storm Sewer	$462,000	$8,277,000	1
O2-1	Replacement of WCR 17 Crossing	$293,000	$8,570,000	1
S-1	Channel Improvements at State Highway 257 Crossing	$12,000	$8,582,000	1
L1-1	New Channel (Sta. 0+00 to 72+00)	$1,712,000	$10,294,000	2
L1-2	Bike Trail Crossing Structure	$80,000	$10,374,000	2
L1-3	Whitney Ditch Siphon	$45,000	$10,419,000	2
L2-1	New Channel (72+00 to 136+00)	$1,124,000	$11,543,000	2
L2-2	Eastman Park Crossing Structure	$150,000	$11,693,000	2
L2-3	C&S Railroad Crossing Structure	$161,000	$11,854,000	2
L2-4	New Channel (Sta. 136+00 to 163+00)	$469,000	$12,323,000	2
L2-6	New Channel (Sta. 0+00 to 8+50)	$122,000	$12,445,000	2
L2-7	Check Structure and Wasteway	$35,000	$12,480,000	2
L3-2	Greeley No. 2 Improvements WCR 19/State Highway 392	$10,000	$12,490,000	2
L4-1	New Channel (Sta. 0+00 to 50+00)	$358,000	$12,848,000	2
L4-2	Consolidated Law Ditch Siphon	$26,000	$12,874,000	2
L4-3	G.W. Railroad Crossing Structure	$217,000	$13,091,000	2
L5-2	Plug Existing Principal Spillway	$9,000	$13,100,000	2
L5-4	Greeley No. 2 Improvements at Osterhaut	$217,000	$13,317,000	2
L6-2	Springer Ditch Improvements	$153,000	$13,470,000	2
L6-1	Windsor Ranch Estates Channel	$111,000	$13,581,000	2
H-2	Whitney Ditch Siphon	$37,000	$13,618,000	2
H-5	10th Street Improvements	$37,000	$13,655,000	2
O1-1	Enlargement of G.W. Railroad Crossing	$163,000	$13,818,000	2
O2-2	WCR 17 Detention Pond	$59,000	$13,877,000	2
P-3	LCR 30 Detention Pond	$145,000	$14,022,000	3
P-2	State Highway 392 Culvert and Swale	$16,000	$14,038,000	3
P-1	LCR 5 Culvert and Swale	$13,000	$14,051,000	3
T-1	Cache la Poudre Transition	$133,000	$14,184,000	3
T-2	New Channel (Sta. 1+00 to 15+70)	$317,000	$14,501,000	3
T-3	LCR 32 ½ Crossing Structure	$122,000	$14,623,000	3
T-4	New Channel (Sta. 16+00 to 59+50)	$471,000	$15,094,000	3
T-5	Greeley No. 2 Siphon, Wasteway, Turnout	$383,000	$15,477,000	3

VII. STORMWATER IMPACT FEES

7.1 General

This chapter presents a summary of the funding and financing mechanism necessary to implement the drainage improvements identified within this master drainage plan. The underlying purpose of the funding and financing mechanism is to provide monies for the construction, operation and maintenance of stormwater drainage facilities necessary for the public health, safety, and welfare of the citizens of the Town of Windsor.

Typically, stormwater fees consist of: (a) annual operation and maintenance fees and, (b) basin impact fees. The annual operation and maintenance fee provides for the maintenance of all improvements and existing facilities as well as the administration of the stormwater program within the Town. Basin impact fees provide for the construction of the major and minor capital improvement projects. The basin impact fees provide a funding mechanism through which owners of properties that develop in the future share in the cost of projects built in anticipation of their needs. In addition, the basin impact fees may be allocated to individual users as a service charge for the construction of capital improvements associated with stormwater management.

The stormwater drainage improvements for each basin are identified in this master drainage plan. The revenue requirement, assuming implementation of these capital improvements, are estimated to be approximately $15 million. This revenue requirement provides the basis for establishing the rate structure for both the basin impact fees for new development as well as the monthly user fees.

It is acknowledged that the Town of Windsor presently assesses new development a storm water drainage fee (referred to as a plant investment fee) and residents a monthly storm water drainage fee. During the completion of this master drainage plan, revisions to the methods for calculating the fees became necessary given the revenue requirements associated with implementation of future drainage improvement projects. The paragraphs below summarize the methodologies and results related to the fees established for implementation of the storm drainage improvements, as well as the operation and maintenance associated existing and future storm water facilities.

7.2 Development of Basin Impact Fee

Development of the Basin Impact Fee is typically based on the potential for stormwater runoff from each parcel along with the cost of the drainage improvements. Specifically, the

calculation of the Basin Impact Fee for the Town of Windsor was based on the following formula:

Basin Impact Fee = (Impervious Rate Factor)*(Basin Impact Fee Factor)*(Area)

where,

- the Impervious Rate Factor is the average percent of impervious area associated with each property;
- the Basin Impact Fee Factor is based on the cost of the drainage improvements divided by the impervious area associated with future development ($/ft^2); and
- the Area is the size of the parcel (ft^2).

7.2.1 Impervious Rate Factor

The Impervious Rate Factor is commonly used to account for differences in runoff due to different types of land use. Typical impervious values were assigned to development categories as indicated below.

Development Category	% Impervious
Very Low Density Single Family Residential (VLDSFR)	10%
Single Family Residential (SFR)	40%
Light Industrial (LI)	80%
Heavy Industrial (HI)	90%
Commercial (C)	95%
Multi-Family Residential (MFR)	70%
Residential Mixed Use (RMU)	50%

7.2.2 Basin Impact Fee Factor

Development of the Basin Impact Fee Factor was based on the following assumptions:

- a planning window for implementation of the drainage improvements of 30 years;

- average growth of development in the last five years (approximately 400 building permits/yr);
- potential growth within the Growth Management Boundary during the 30-year planning window (50% of the undeveloped land within the Growth Management Boundary was assumed to be developed);
- future land use plans within the Growth Management Boundary that identified the categories of development; and
- a revenue requirement of $15 million associated with the drainage improvements.

With these assumptions, the basin impact fee factor was determined to be $0.1838/ft^2.

7.2.3 Basin Impact Fee Structure

Given this information, the formula for Basin Impact Fees was determined to be:

Basin Impact Fee = (Impervious Rate Factor)*($0.1838)*(Area)

Utilizing this formula, the fee structure for new development within the Growth Management Boundary was generated and is presented in Table 7.1.

Table 7.1 Basin Impact Fee Structure for New Development.

Development Category	Basin Impact Fee Formula	Basin Impact Fee
VLDSFR (1.5 AC)	1.5 AC x 43,560 ft^2/AC x $0.1838 x 0.1	$1,200
(2.5 AC)	2.5 AC x 43,560 ft^2/AC x $0.1838 x 0.1	$2,000
SFR	7,000 ft^2 x $0.1838 x 0.4	$515
	10,000 ft^2 x $0.1838 x 0.4	$735
Light Industrial	43,560 ft^2 x $0.1838 x 0.8	$6,400
Heavy Industrial	43,560 ft^2 x $0.1838 x 0.9	$7,200
Commercial	43,560 ft^2 x $0.1838 x 0.95	$7,600
MFR	43,560 ft^2 x $0.1838 x 0.7	$5,600
RMU	43,560 ft^2 x $0.1838 x 0.5	$4,000

7.3 Conceptual Financing Plan

The fee structure in Table 7.1 was utilized to prepare conceptual financing plans to demonstrate the economic feasibility associated with implementation of the drainage improvements. Several iterations were involved to determine the extent that monthly user fees are necessary to create income to cover the remainder of the debt service. The approach and assumptions that were used to create the conceptual financing plans are presented below.

- Funding for the proposed improvements was assumed to be through revenue bonds. The principal and interest on the bonds are assumed to be paid through Basin Impact Fees and, if necessary, monthly user fees. The monthly user fees, if required, are intended to be for all existing and future residents.

- Each revenue bond is assumed to be 20 years in duration, 6% interest rate, and fully amortized with constant principal and interest payments throughout the 20-year period of the bonds.

- Basin impact fees were assigned to all new development projected within the Growth Management Boundary. The projected growth assumed development of 50% of the undeveloped land within the Growth Management Boundary (400 new building units/yr).

- Projected revenue requirements associated with future development were based on the concept of an Equivalent Development Unit (EDU). A typical EDU within the Growth Management Boundary was determined to be:

Typical EDU	10,000 ft^2
Average % Impervious	26%
Impervious Area	2,600 ft^2
Cost/EDU (2,600 x $0.1838)	~$480

The results of the initial conceptual financing plan indicated that monthly user fees will be necessary to cover the debt service associated with the revenue bonds. The monthly fee associated with all existing and future residents was determined to be $0.000458/$ft^2$/month. The monthly fees relate to the impervious area associated with existing and future residents. Based on these results, the formula for calculating the monthly user fees is:

Monthly User Fee = (Impervious Rate Factor)*($0.000458)*(Area)

Utilizing this formula, an estimate of the monthly user fees associated with new development within the Growth Management Boundary was generated and is presented in Table 7.2.

Table 7.2 Monthly Basin User Fee Structure.

Development Category	Basin Impact Fee Formula	Basin Impact Fee
VLDSFR (1.5 AC)	1.5 AC x 43,560 ft^2/AC x \$0.000458 x 0.1	\$2.99
(2.5 AC)	2.5 AC x 43,560 ft^2/AC x \$0.000458 x 0.1	\$4.99
SFR	7,000 ft^2 x \$0.000458 x 0.4	\$1.28
	10,000 ft^2 x \$0.000458 x 0.4	\$1.83
Light Industrial	43,560 ft^2 x \$0.000458 x 0.8	\$15.96
Heavy Industrial	43,560 ft^2 x \$0.000458 x 0.9	\$17.96
Commercial	43,560 ft^2 x \$0.000458 x 0.95	\$18.95
MFR	43,560 ft^2 x \$0.000458 x 0.7	\$13.96
RMU	43,560 ft^2 x \$0.000458 x 0.5	\$9.98

The final conceptual financing plan utilized to demonstrate the economic feasibility associated with implementation of the drainage improvements is provided in Appendix C.

7.4 Operation and Maintenance Fees

Operation and maintenance fees are presently determined on a monthly basis in accordance with the following formula:

O&M Fee = \$1.70 + (Impervious Area x 0.00011)

The base fee of \$1.70 is necessary to pay for the routine operation and maintenance expenses. The remaining items account for the impervious area of a parcel and an operation and maintenance rate factor of 0.00011. Presently, a credit of 75% is also given to those parcels providing detention. This credit is applied to the product relating impervious area to the operation and maintenance rate factor.

Given the detention requirements associated with the master drainage plan, the formula for calculating the operation and maintenance fees was revised. This work effort focused on the determination of a revised operation and maintenance rate factor. Following an evaluation of existing fees presently collected by the Town, the revised rate factor was determined to be 0.00009 and resulted in the revised formula:

O&M Fee = $1.70 + (Impervious Area x 0.00009)

Based on the revised formula, an indication of the operation and maintenance fees associated with the property development within the Town is presented in Table 7.3.

Table 7.3 Monthly Operation and Maintenance Fees.

Development Category	O&M Fee Formula	O&M Fee
VLDSFR (1.5 AC)	1.5 AC x 43,560 ft^2/AC x $0.00009 x 0.1 + $1.70	$2.28
(2.5 AC)	2.5 AC x 43,560 ft^2/AC x $0.00009 x 0.1 + $1.70	$2.68
SFR	7,000 ft^2 x $0.00009 x 0.4 + $1.70	$1.95
	10,000 ft^2 x $0.00009 x 0.4 + $1.70	$2.06
Light Industrial	43,560 ft^2 x $0.00009 x 0.8 + $1.70	$4.84
Heavy Industrial	43,560 ft^2 x $0.00009 x 0.9 + $1.70	$5.23
Commercial	43,560 ft^2 x $0.00009 x 0.95 + $1.70	$5.42
MFR	43,560 ft^2 x $0.00009 x 0.7 + $1.70	$4.44
RMU	43,560 ft^2 x $0.00009 x 0.5 + $1.70	$3.66

Detailed documentation related to the development of the basin impact fees as well as the monthly operation and maintenance fees is provided in the project notebook.

VIII. SUMMARY AND RECOMMENDATIONS

This document summarizes the results of a master planning effort related to stormwater management within the Town of Windsor. The focus of the planning effort was the identification of improvements that would reduce both existing flooding and the potential for future flooding problems. Drainage improvements within each basin are described in detail in Chapter 5 along with the cost estimate associated with each improvement. A plan for implementing the drainage improvements was developed and is presented in Chapter 6. Finally, pertinent information related to funding and financing the drainage improvements was compiled and is presented in Chapter 7.

Following the approval and adoption of the master drainage plan, it is recommended that the Town of Windsor continue to diligently pursue implementation of the drainage improvements. Based on information obtained during the planning document, the following recommendations are also provided.

(1) Coordination with entities and/or property owners that may be impacted by the implementation of the drainage improvements should be conducted as early as possible. Potential conflicts associated with property acquisition, easements, and rights-of-way, may require resolution prior to construction of the proposed improvements. Identification and resolution of potential permitting conflicts should also be accomplished early in the planning process.

(2) Given the extent of the watershed that generates stormwater runoff, several jurisdictional boundaries are encountered. Close coordination with these jurisdictions should be conducted to ensure that detention requirements remain consistent with the assumptions presented in this master drainage plan.

(3) While funding and financing the drainage improvements is assumed to be the responsibility of the Town of Windsor, alternative sources of funding should be investigated. These funding sources may include Larimer and Weld Counties (for those crossing structures located on County roads), Colorado Department of Transportation (for those crossing structures located on State Highway 392), and the Colorado Water Conservation Board (projects related to reduction of potential flooding).

It should also be understood that the drainage improvements identified in this master drainage plan assume detention (construction of detention ponds that release a peak discharge associated with the 10-year event during the 100-year storm event) within the watershed. Consequently, this assumption means that 100-year protection will not be provided by the drainage improvements until

the upstream watershed is fully developed. As a minimum, 10-year protection will be provided and will incrementally increase as development occurs in the watershed.

Finally, it is important to remember that this planning effort culminated in the preparation of a comprehensive, flexible document that is intended to guide the decision-making process with respect to stormwater management within the basins. The document should be considered as a dynamic tool that can change as stormwater management issues within the basin arise. As the implementation of the drainage improvements progresses from this stage into preliminary and final design, revisions and modifications to the improvements may be necessary. These revisions or modifications should not change the underlying goal of this document, that is the reduction in the damages associated with existing flooding as well as potential flooding associated with future development in the basins.

APPENDIX A

SWMM MODELING NETWORK SCHEMATICS

APPENDIX A.1

EXISTING CONDITION

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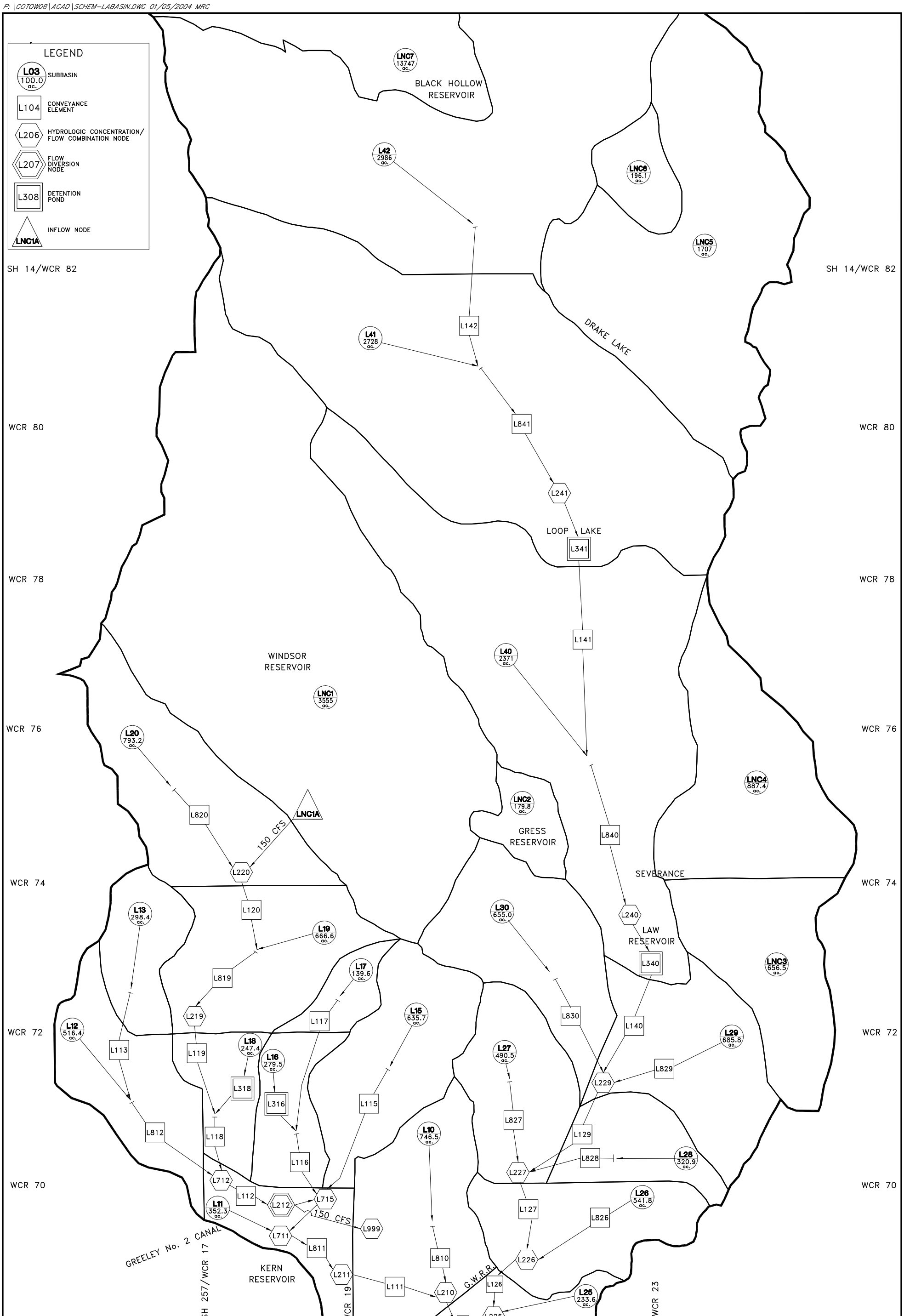

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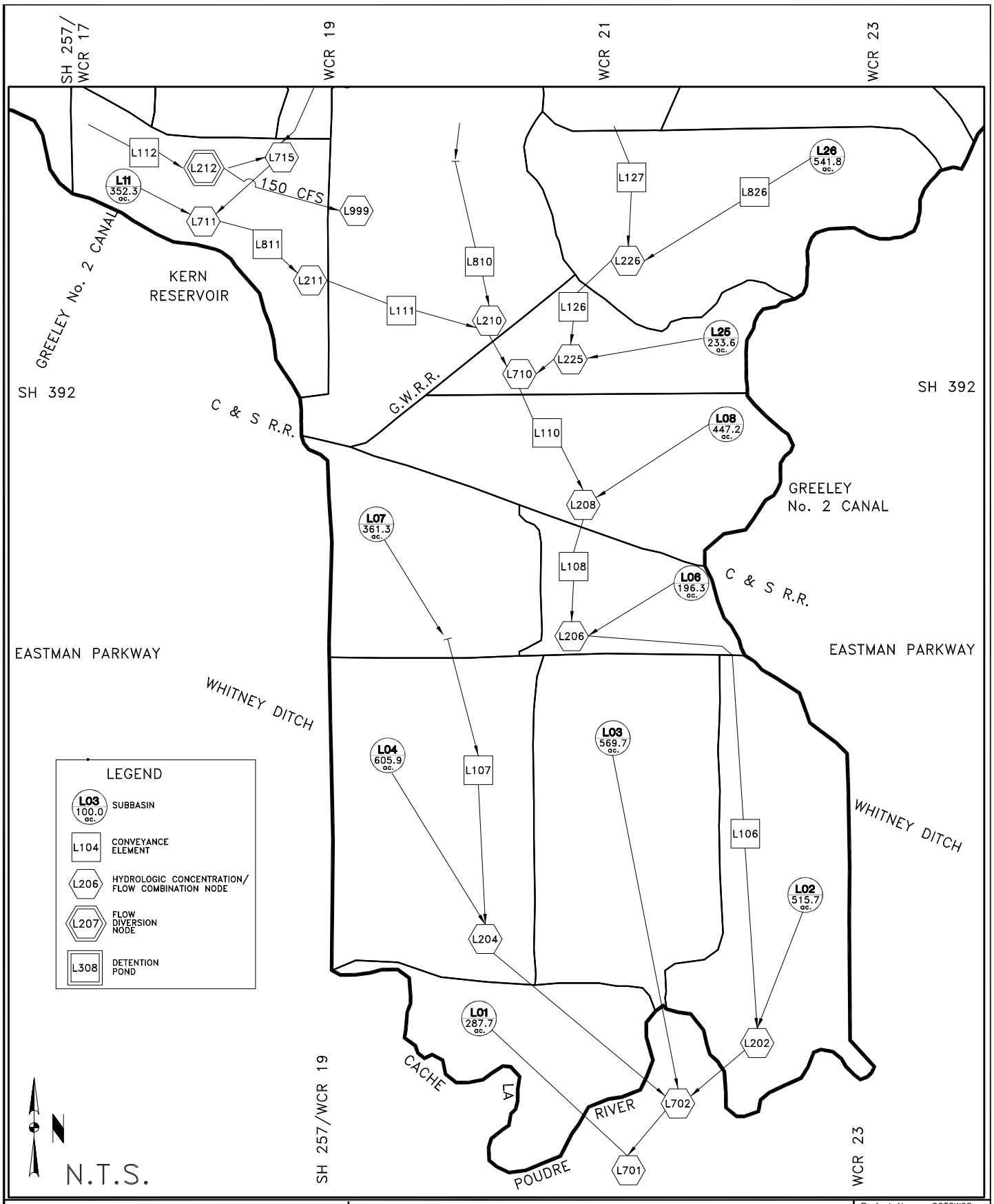
N.T.S.

TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
LAW BASIN-UPPER
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	JAN 5, 2004
ACADFILE:	SCHEM-LABASIN

A1-LA



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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
LAW BASIN-LOWER
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-LABASIN

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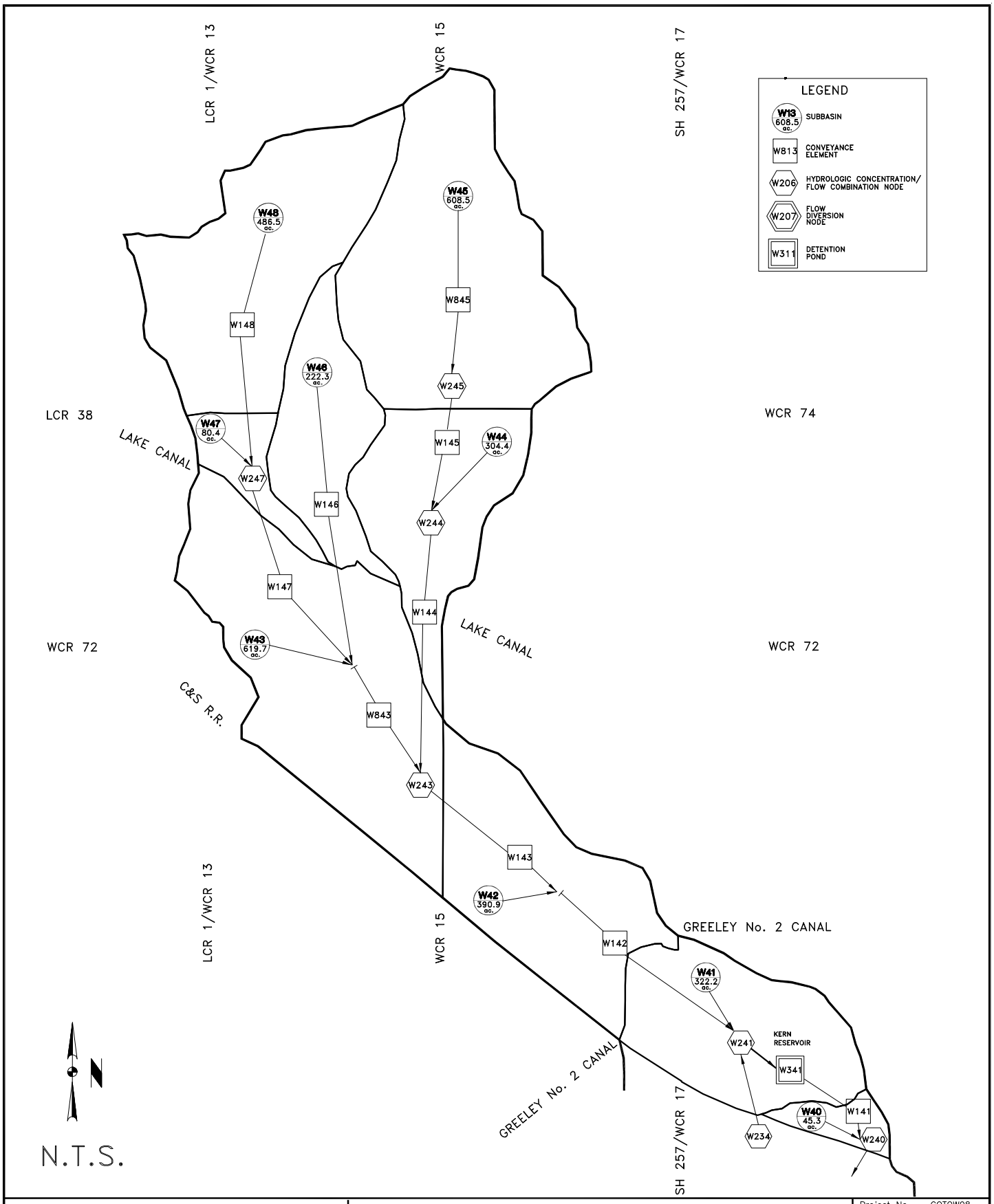

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
WINDSOR BASIN-UPPER
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE: SCHEM-WIBASIN	

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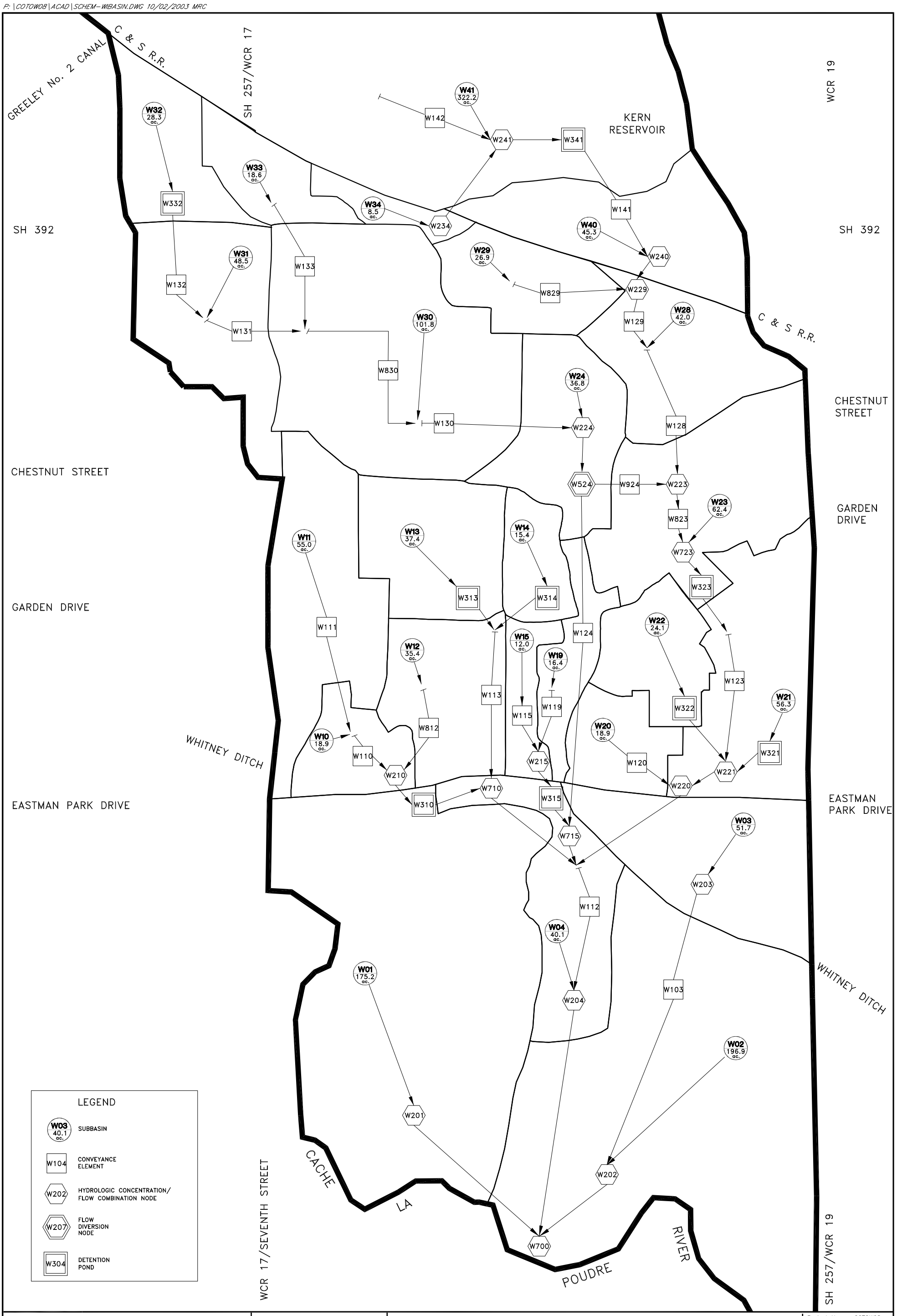

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MASTER DRAINAGE PLAN
WINDSOR BASIN-LOWER
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE: SCHEM-WIBASIN	

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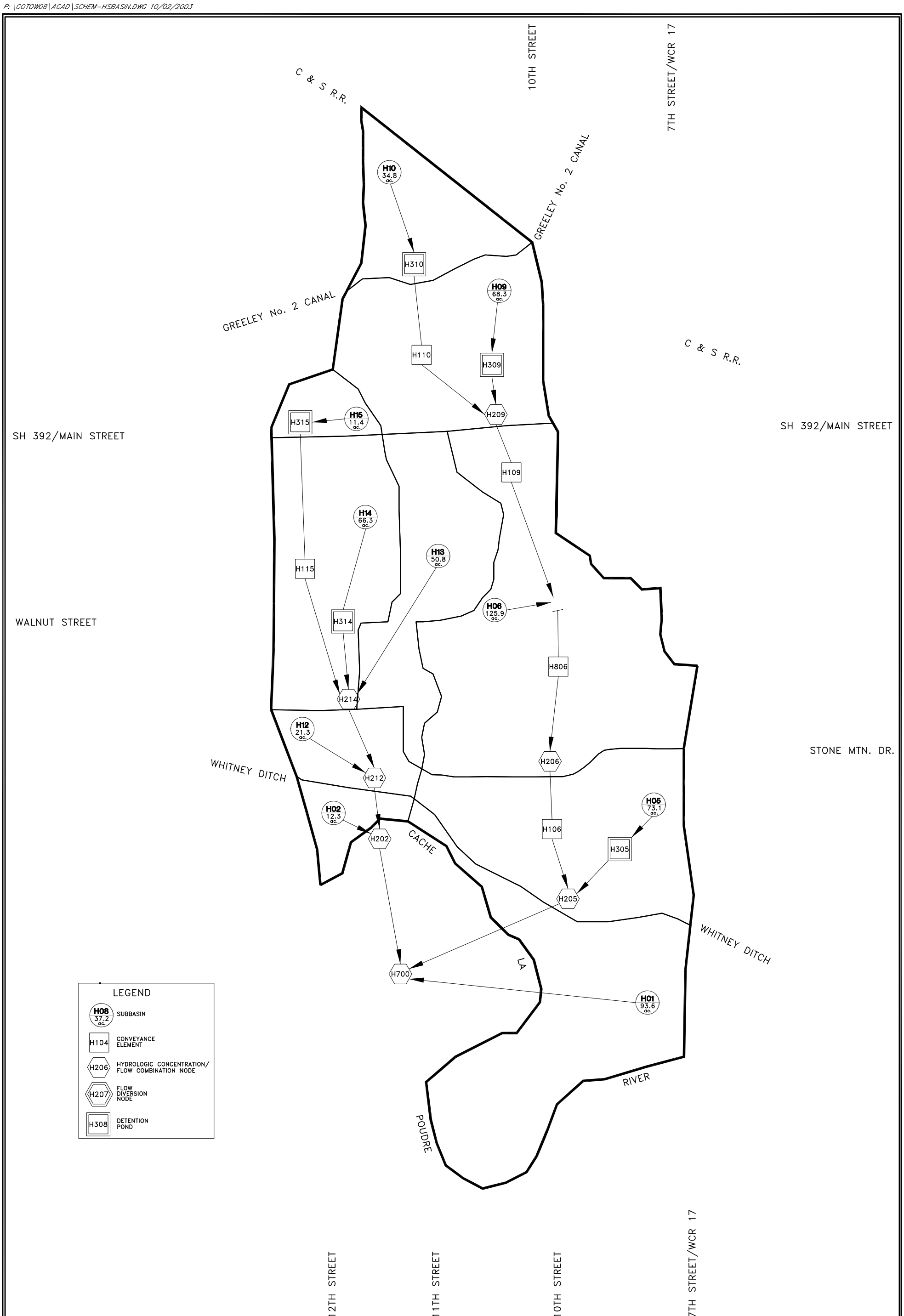

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
HIGH SCHOOL BASIN
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2001
ACADFILE: SCHEM-HSBASIN	

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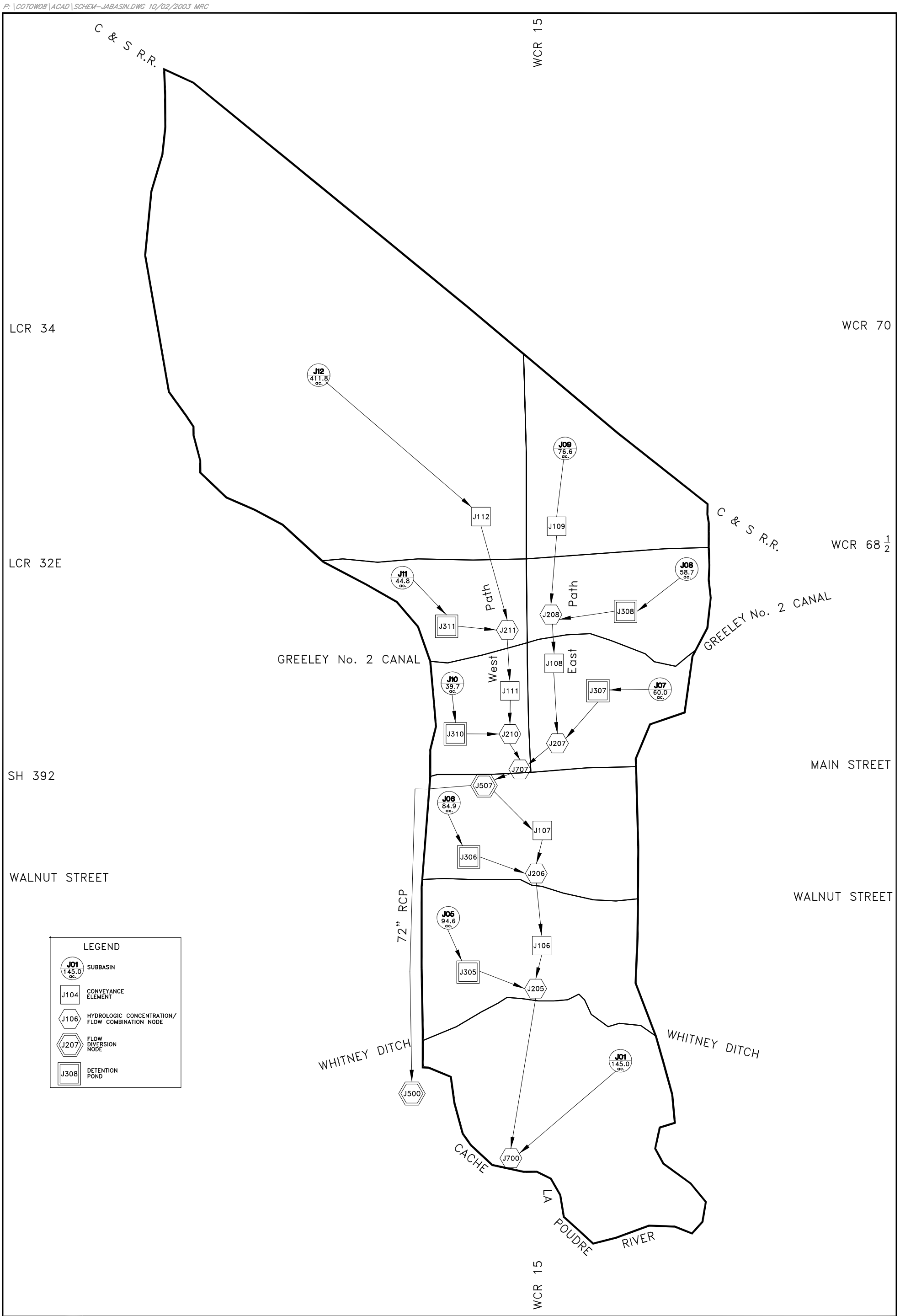

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
JACOBY BASIN
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	OCT, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002
ACADFILE:	SCHEM-JABASIN

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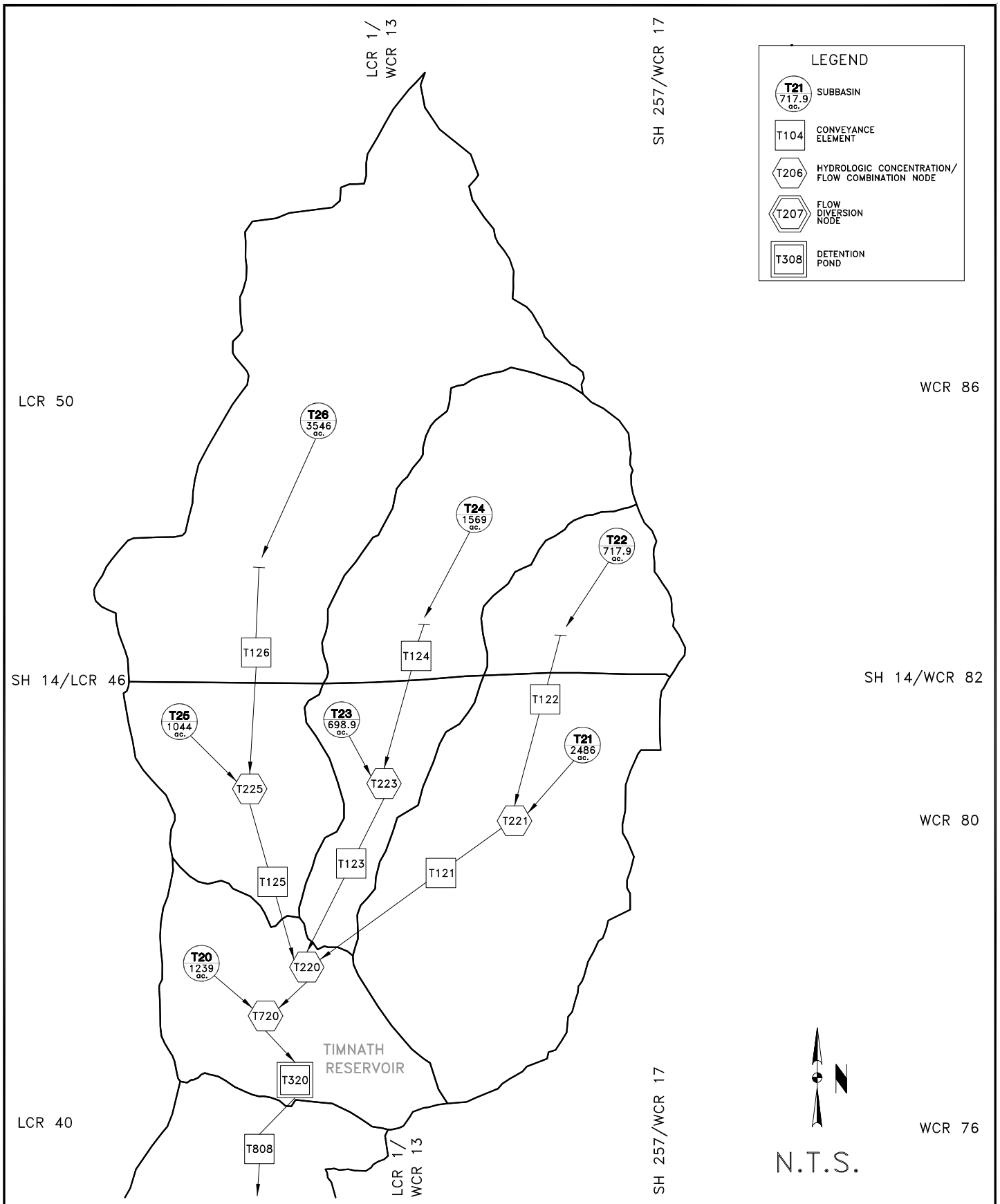

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
TIMNATH RESERVOIR BASIN-UPPER SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	
ACADFILE:	SCHEM-TRBASIN

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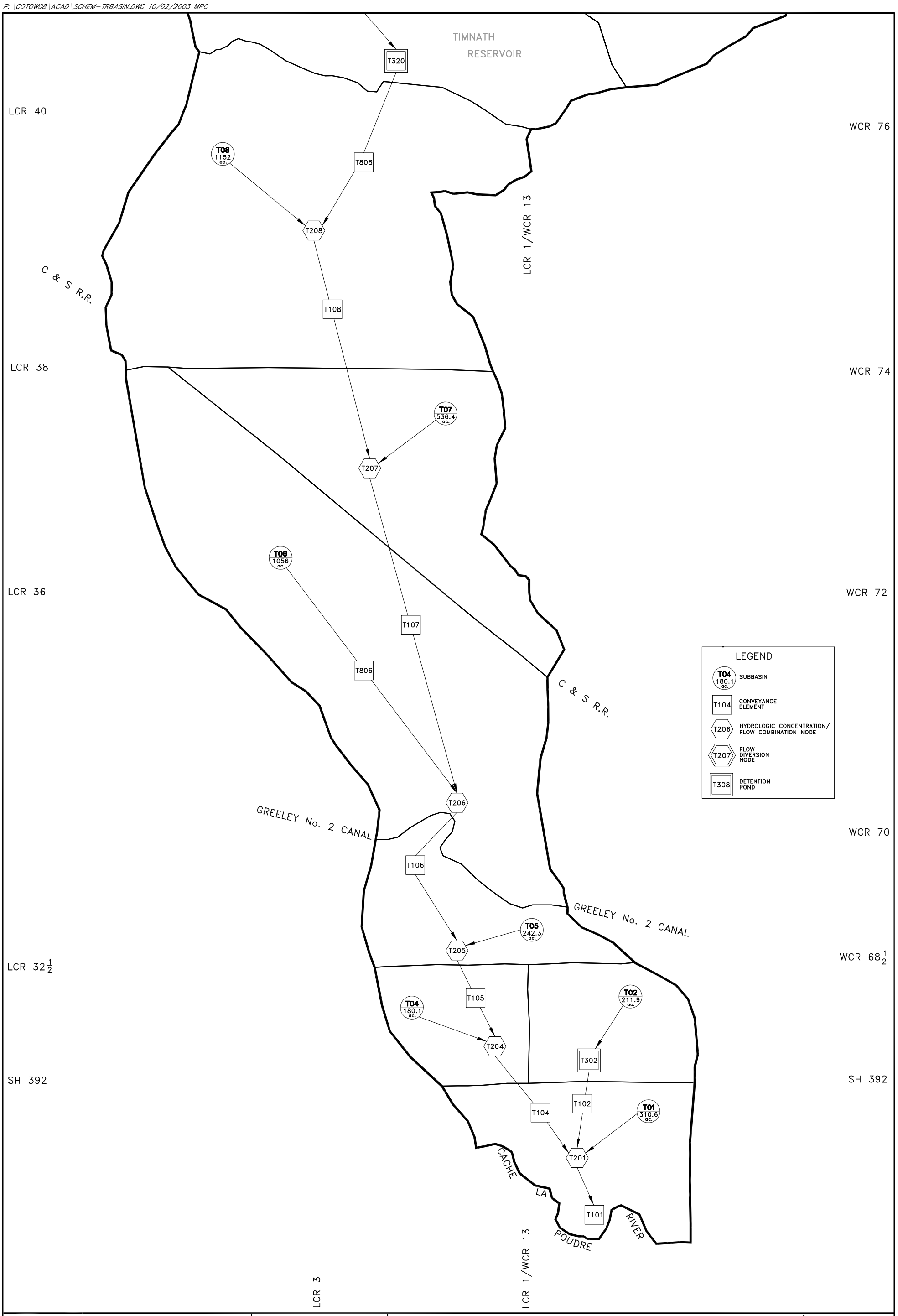

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
TIMNATH RESERVOIR BASIN-LOWER
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE: SCHEM-TRBASIN	

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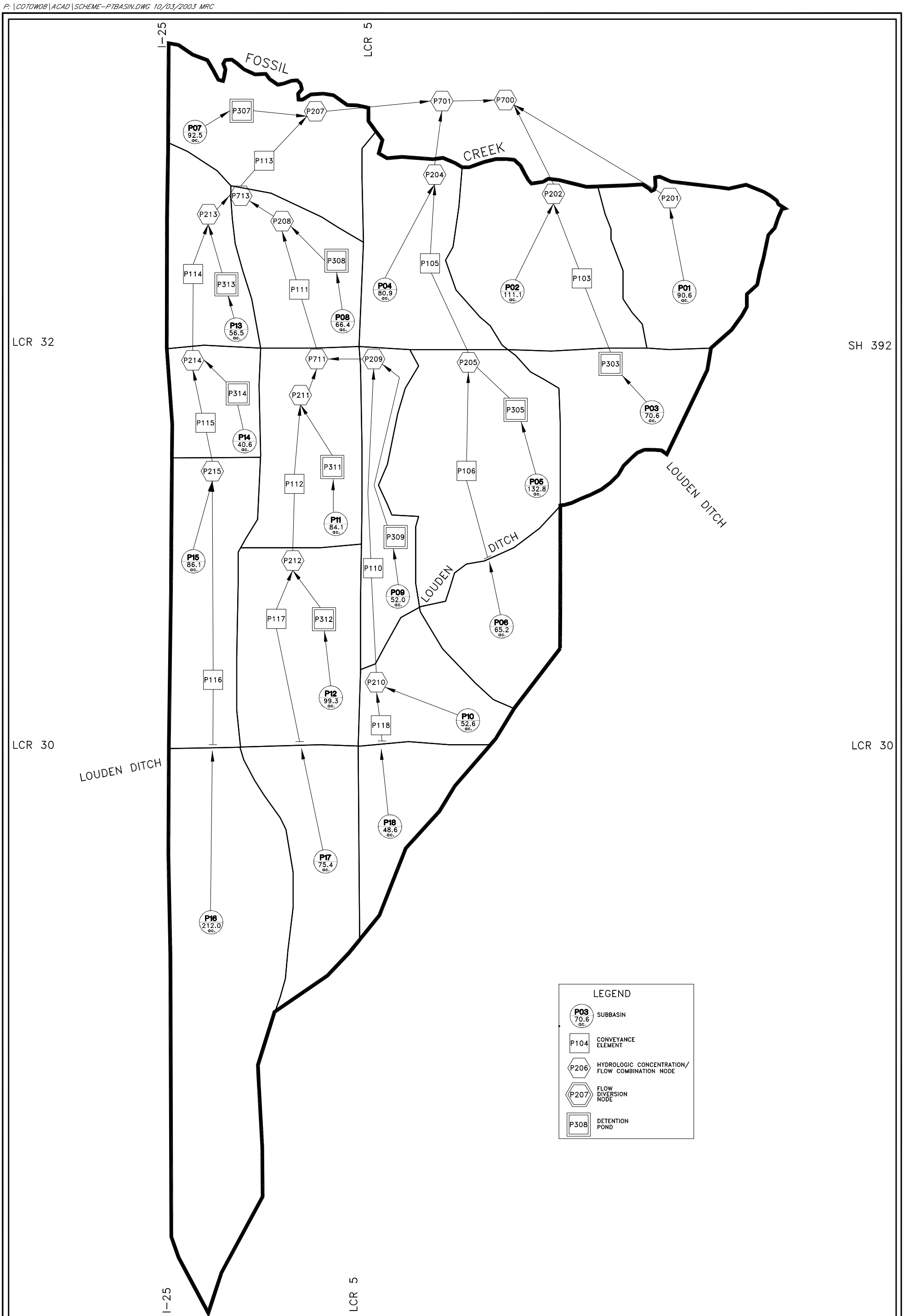

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
PTARMIGAN BASIN
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-PTBASIN

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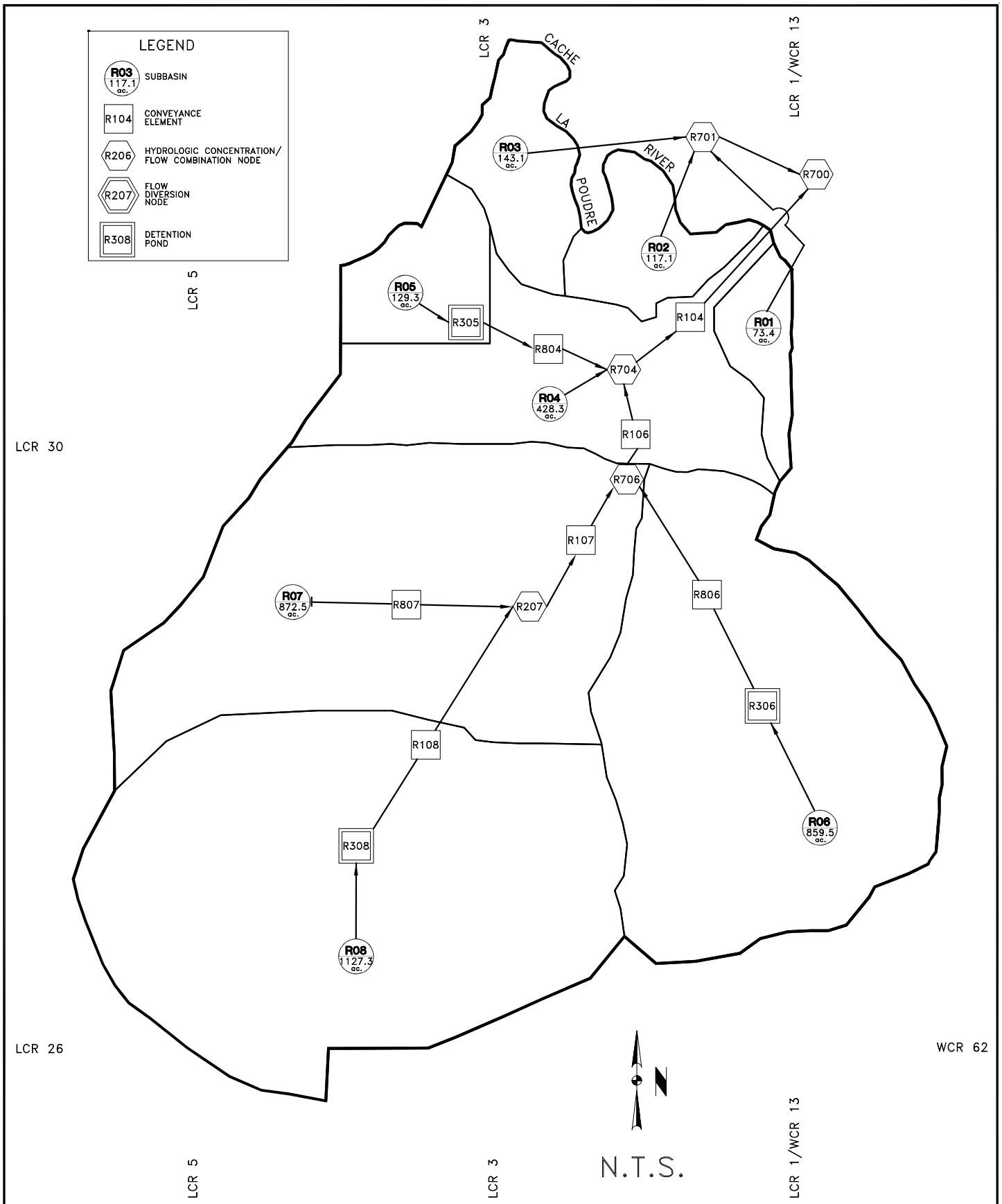

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
RIVER RIDGE BASIN
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-RRBASIN

A1-R

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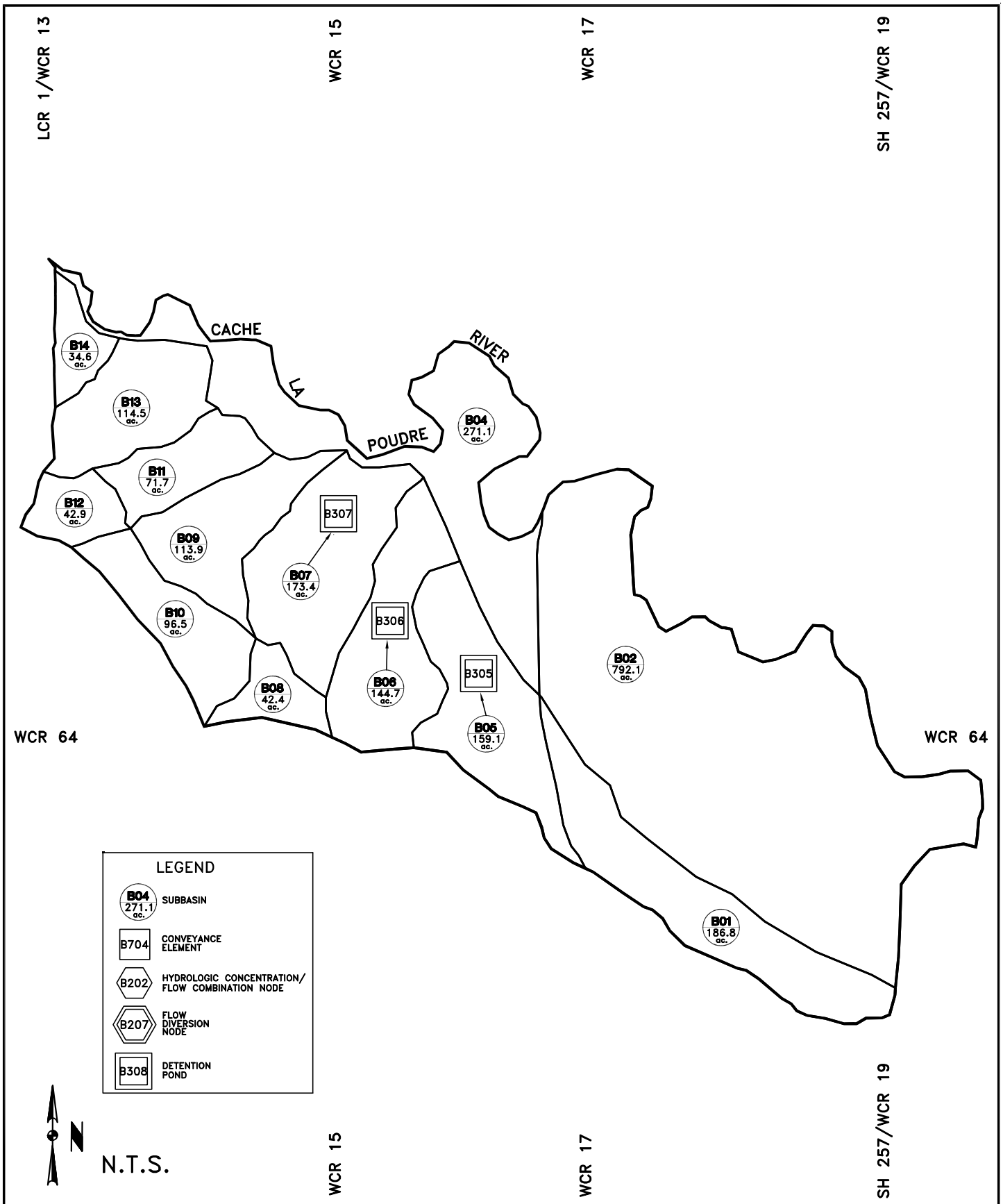

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
BLUFF BASIN
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002
ACADFILE:	SCHEM-BLBASIN

A.1-B

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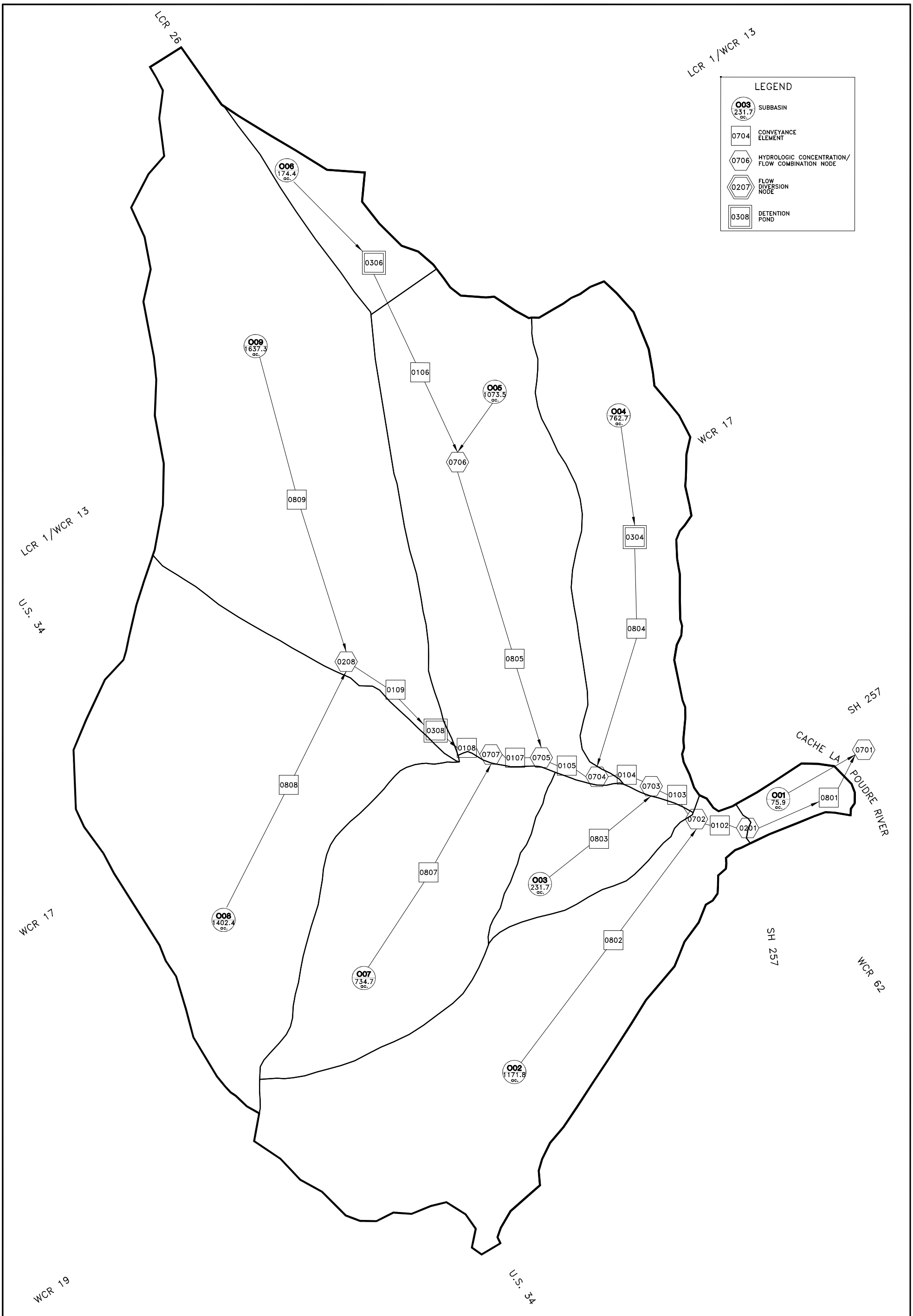

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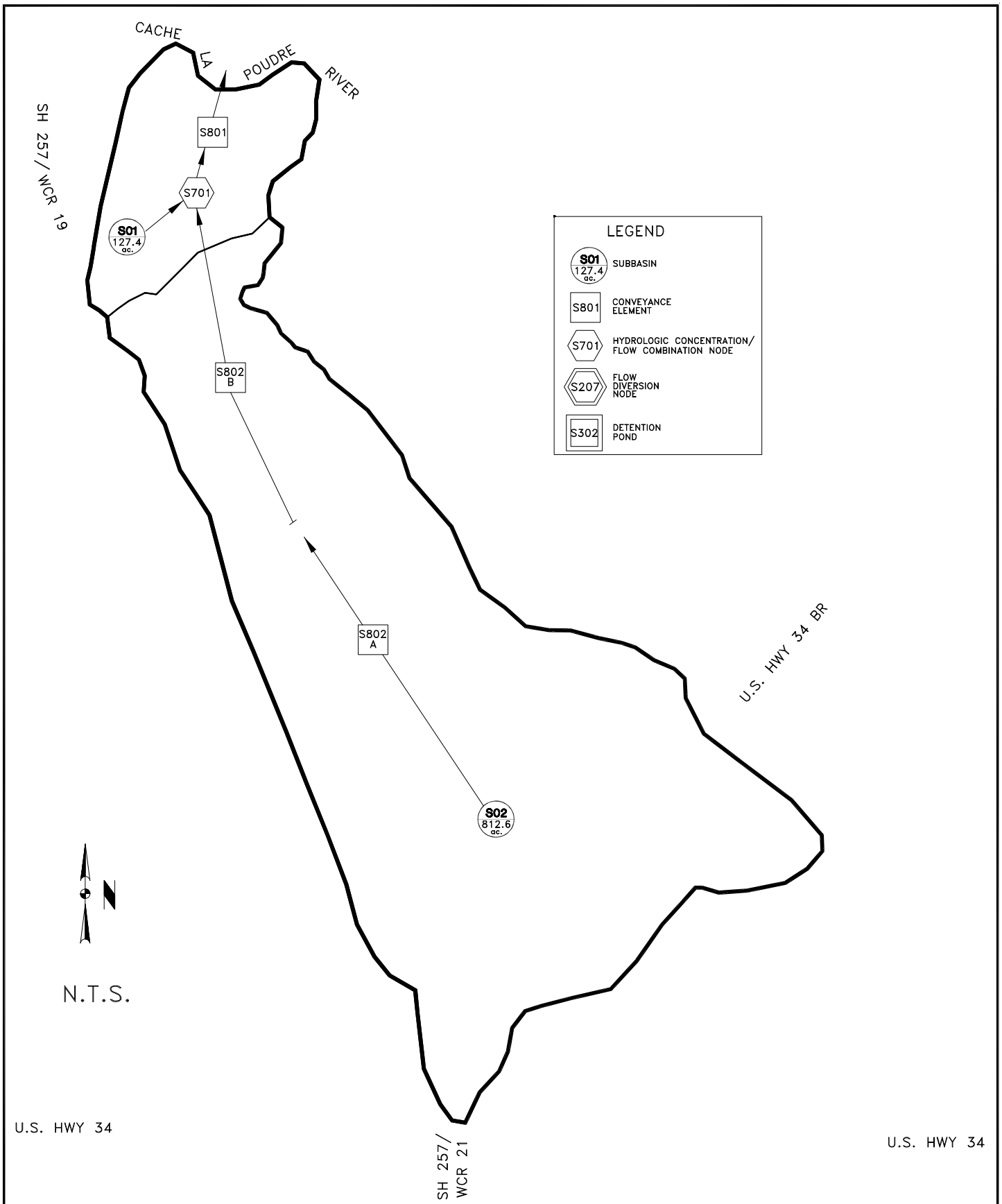
TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
OKLAHOMA BASIN
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	
ACADFILE: SCHEM-OKBASIN	

A1-O



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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
SOUTH HIGHWAY 257 BASIN
SWMM SCHEMATIC-EXISTING CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-SHBASIN

A.1-S

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APPENDIX A.2

FUTURE CONDITION

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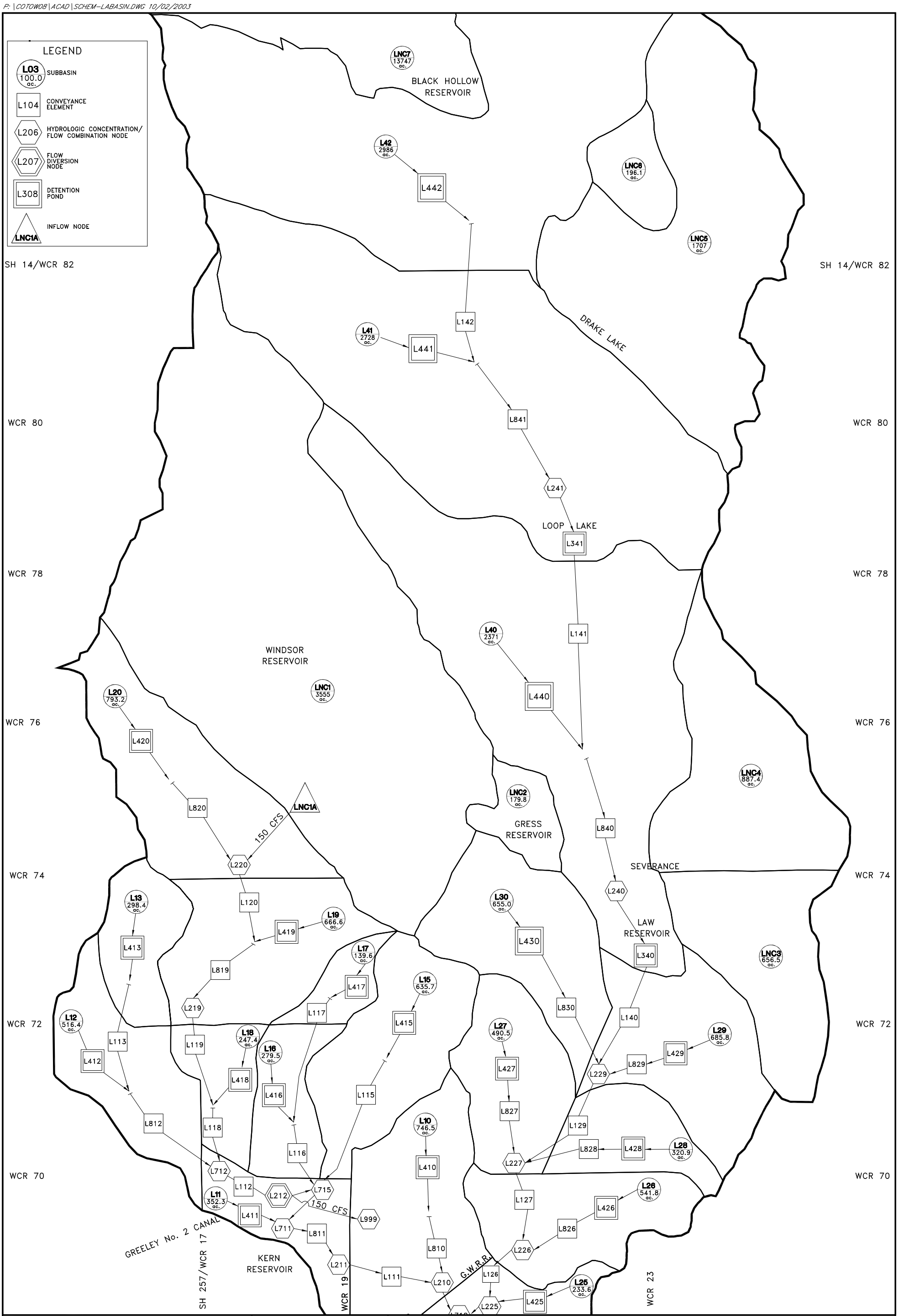

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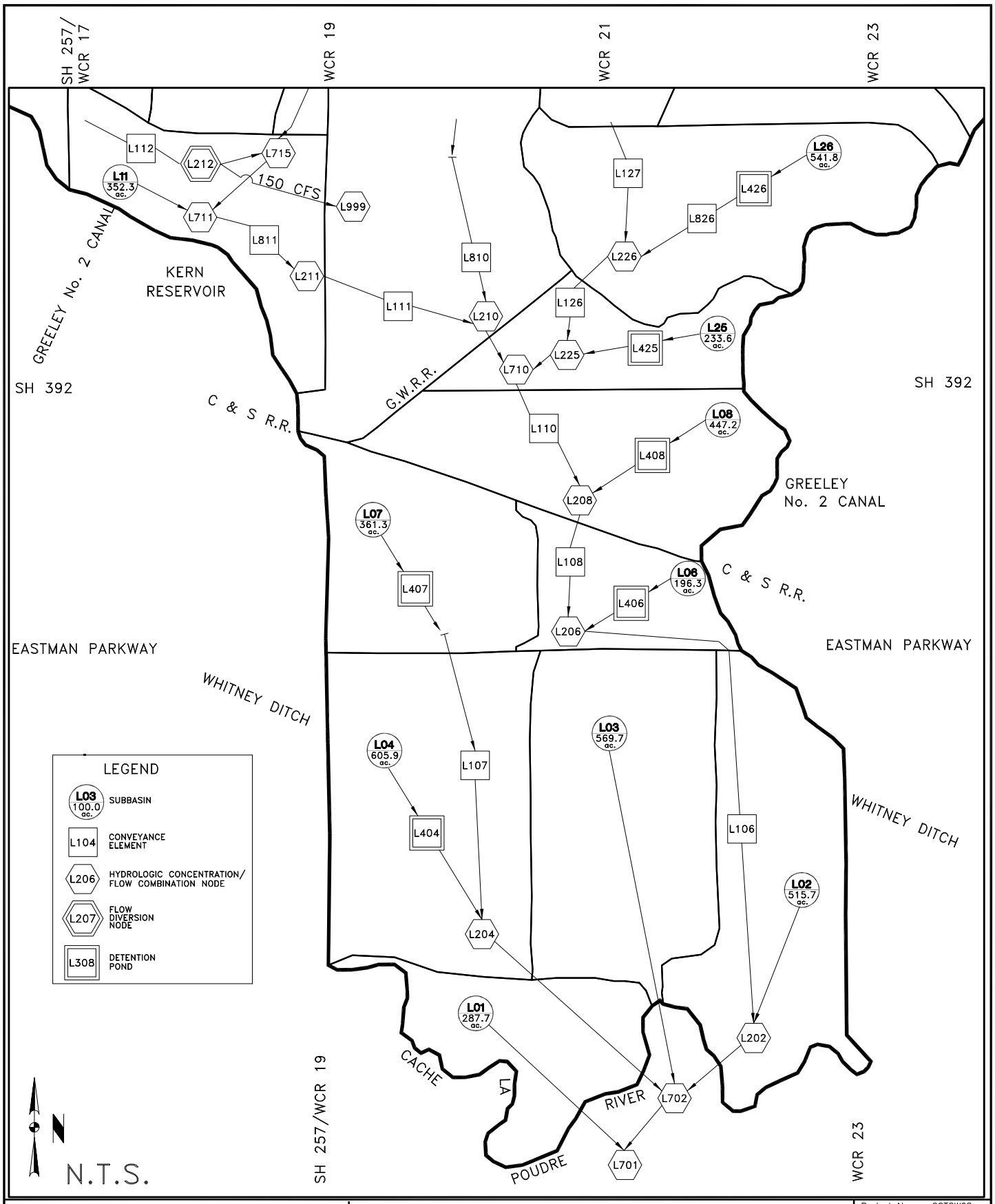
TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
LAW BASIN-UPPER
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-LABASIN

A2-LA



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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
LAW BASIN-LOWER
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-LABASIN

A2-LB

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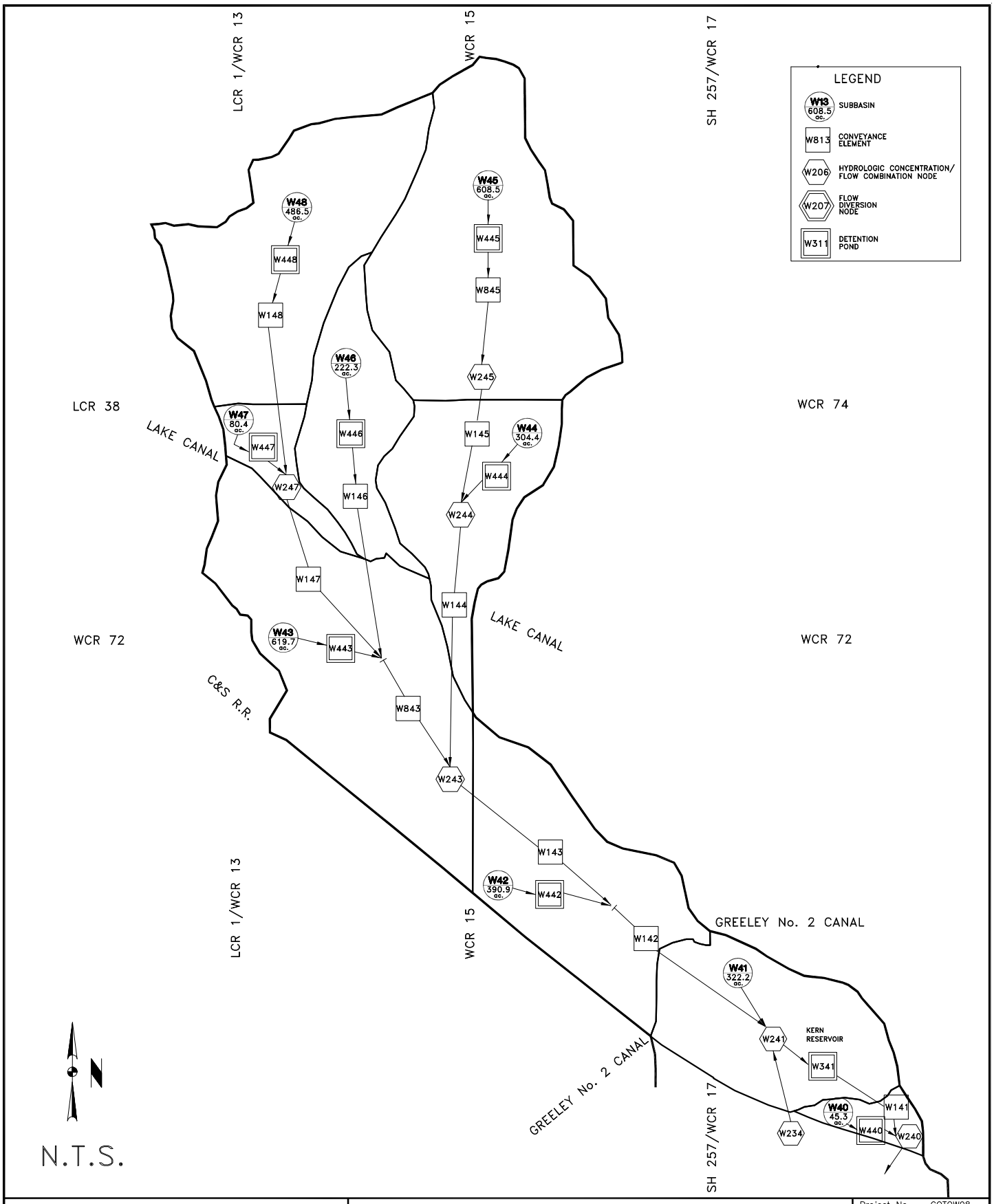

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
WINDSOR BASIN-UPPER
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-WIBASIN

A.2-WA

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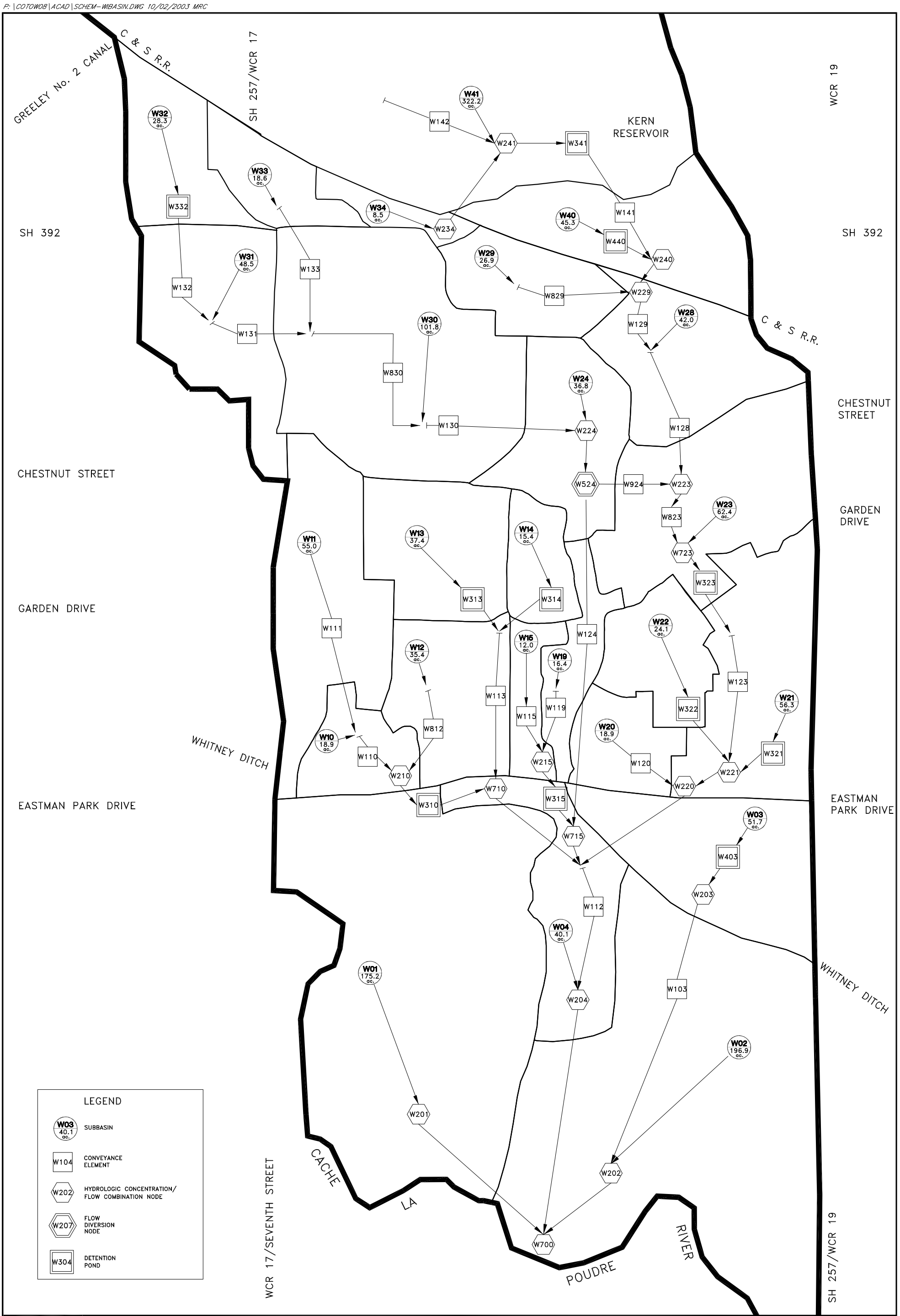

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
WINDSOR BASIN-LOWER
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-WIBASIN

A2-WB

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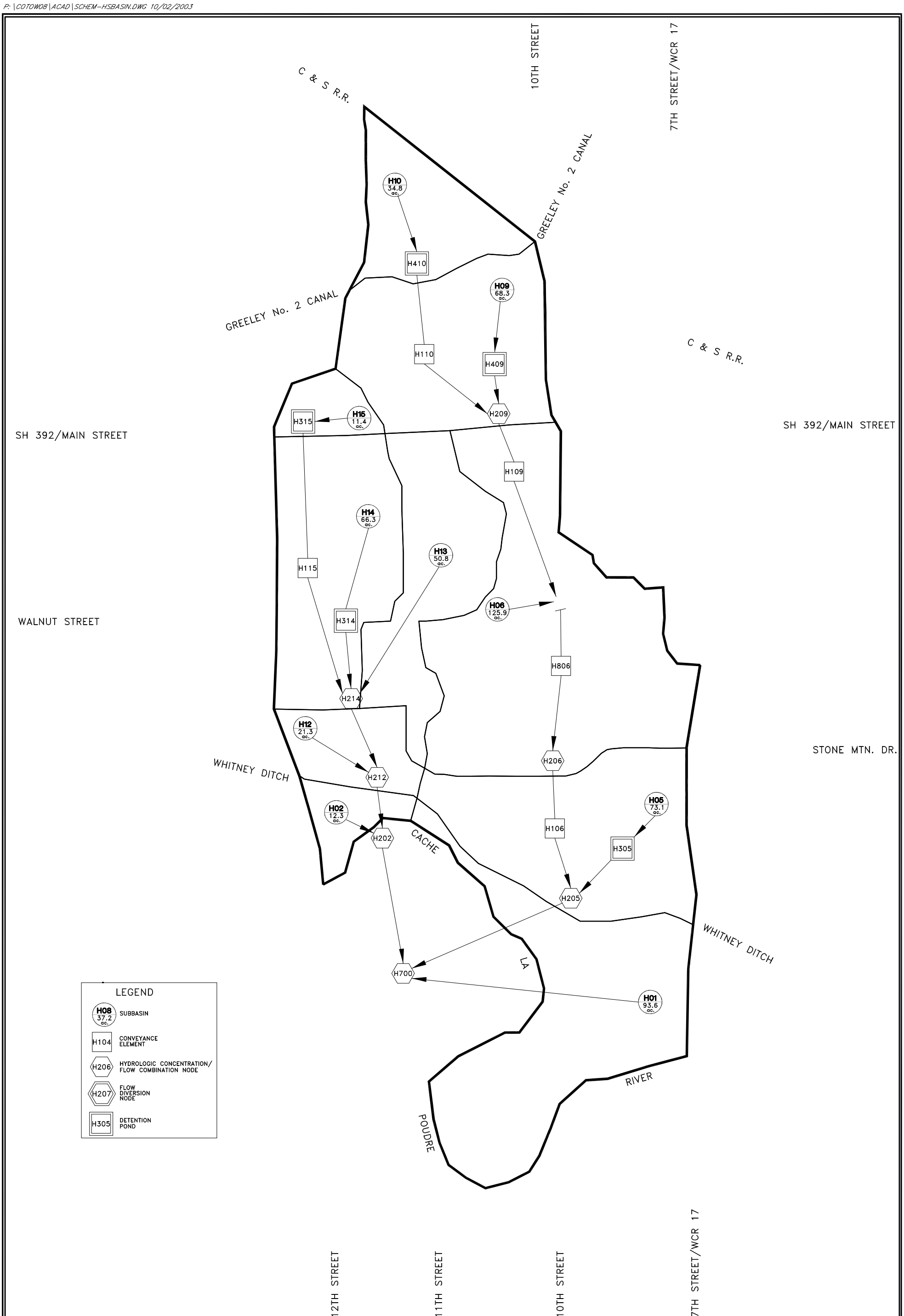

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
HIGH SCHOOL BASIN
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2001
ACADFILE: SCHEM-HSBASIN	

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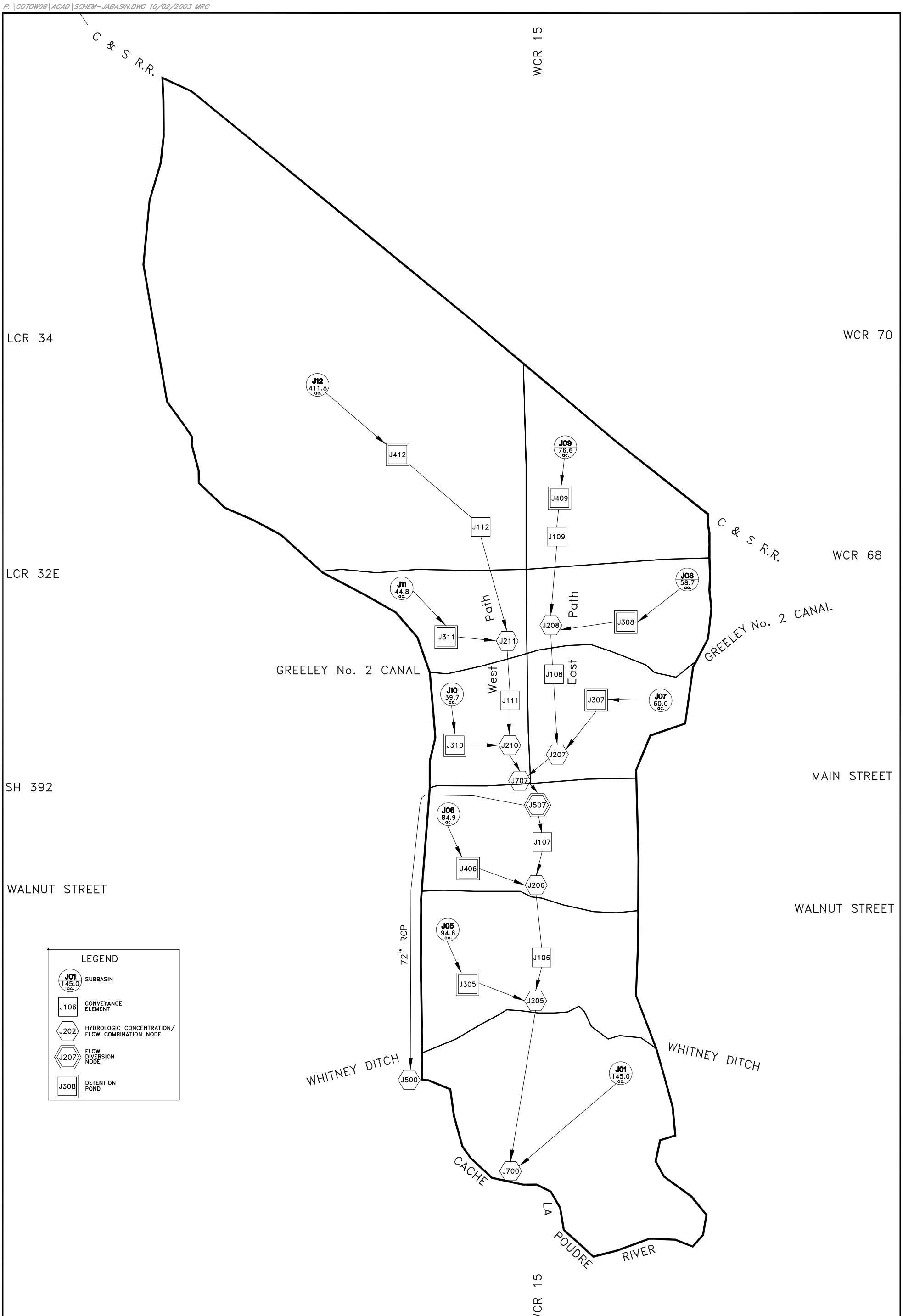

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
JACOBY BASIN
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	OCT, 2001
Design:	CJP
Drawn:	MRC
Revisions:	
ACADFILE:	SCHEM-JABASIN

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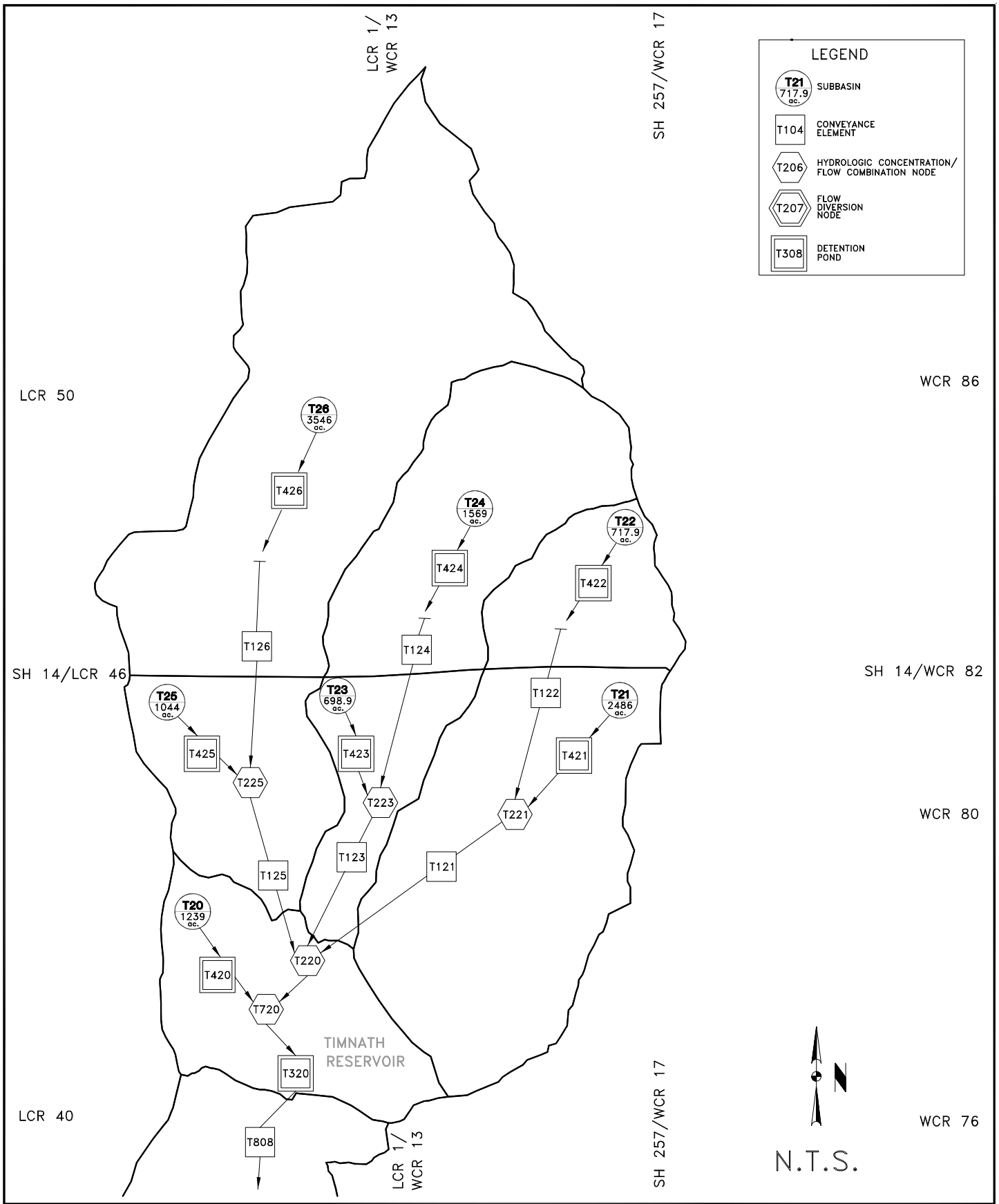

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
TIMNATH RESERVOIR BASIN-UPPER SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	
ACADFILE:	SCHEM-TRBASIN

A2-TA

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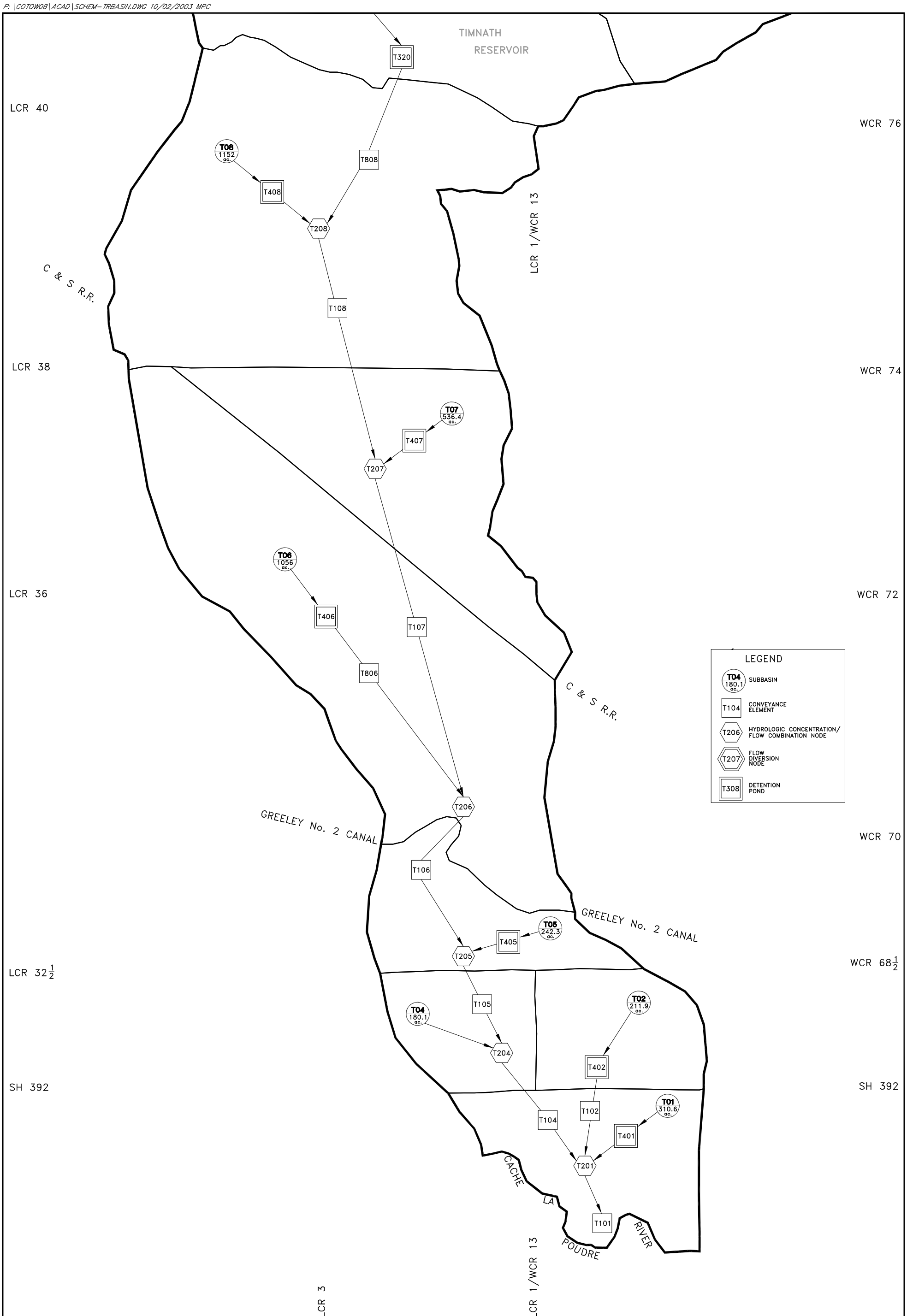

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
TIMNATH RESERVOIR BASIN-LOWER
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-TRBASIN

A2-TB

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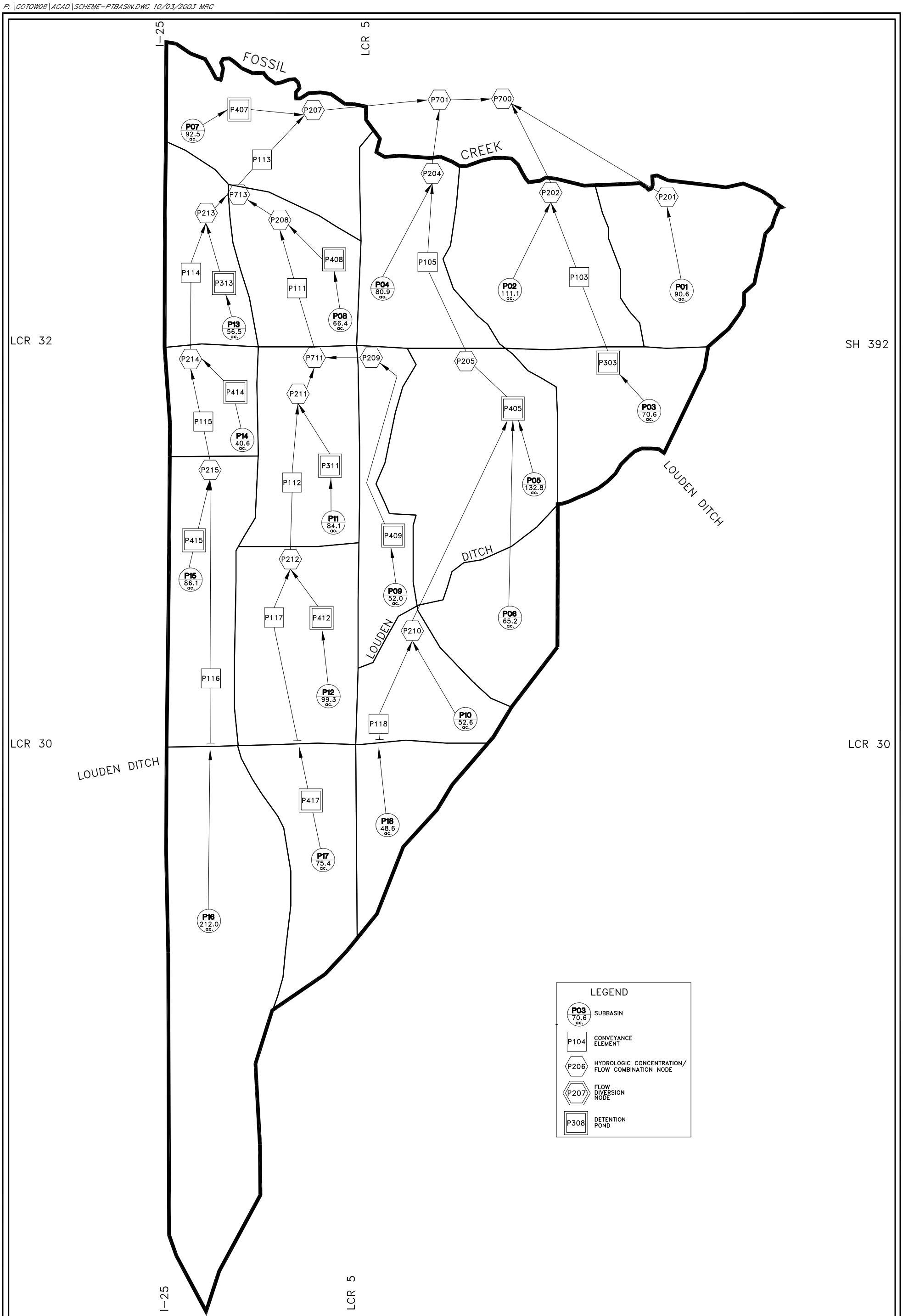

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
PTARMIGAN BASIN
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-PTBASIN

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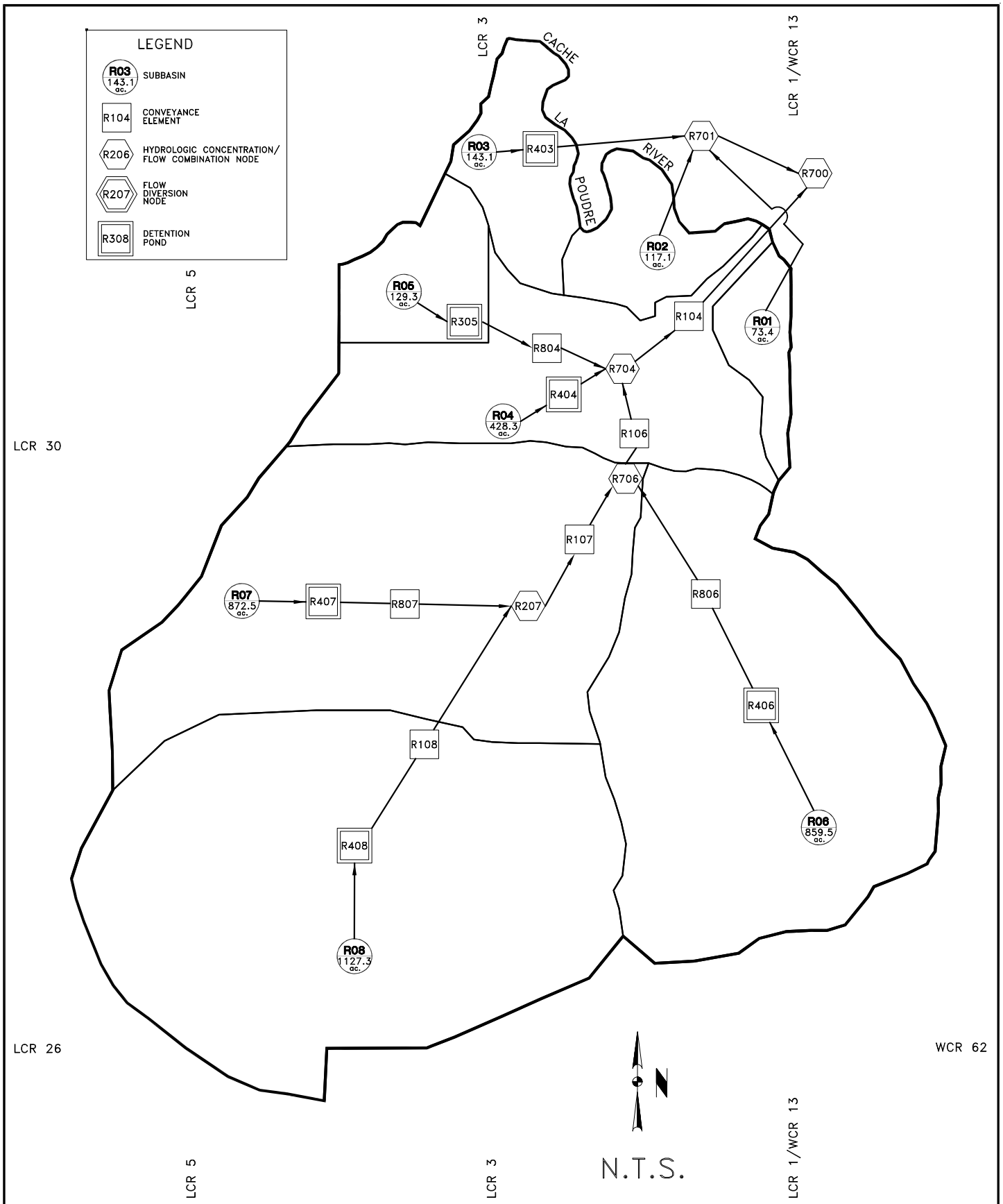

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
RIVER RIDGE BASIN
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-RRBASIN

A2-R

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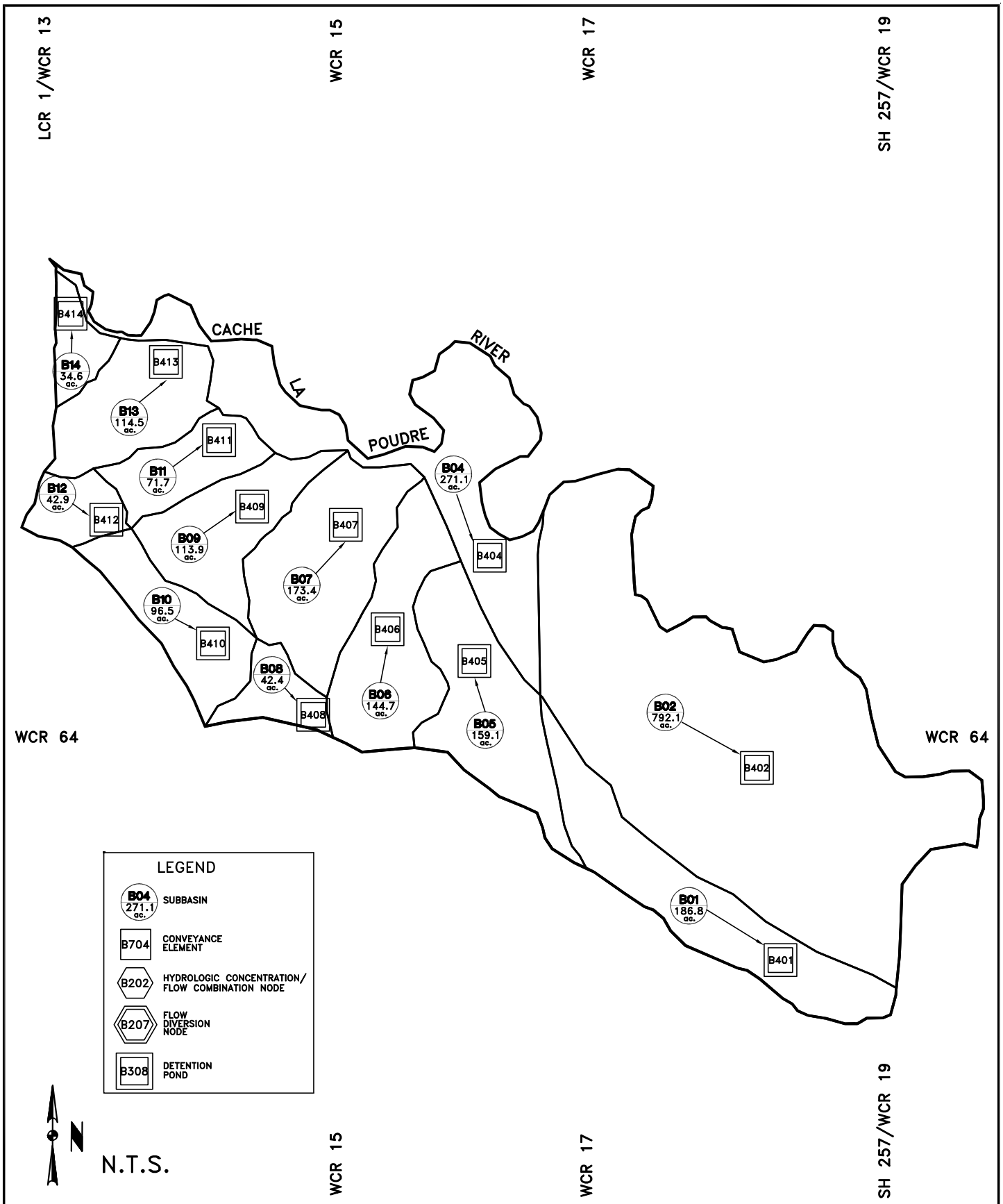

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
BLUFF BASIN
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	
ACADFILE:	SCHEM-BLBASIN

A2-B

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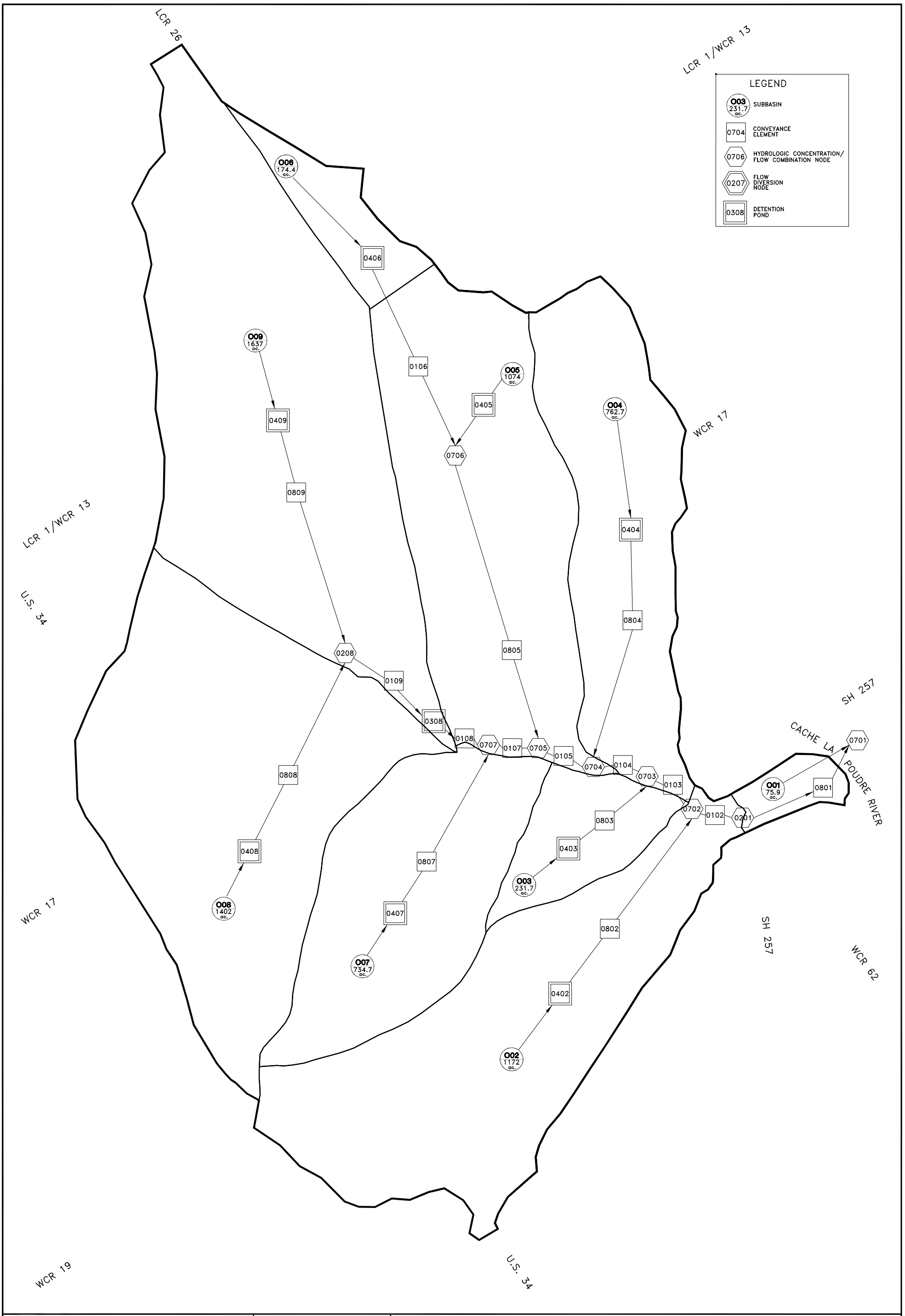

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
OKLAHOMA BASIN
SWMM SCHEMATIC-FUTURE CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	
ACADFILE: SCHEM-OKBASIN	

A2-O

APPENDIX A.3

FUTURE CONDITION WITH OVER-DETENTION

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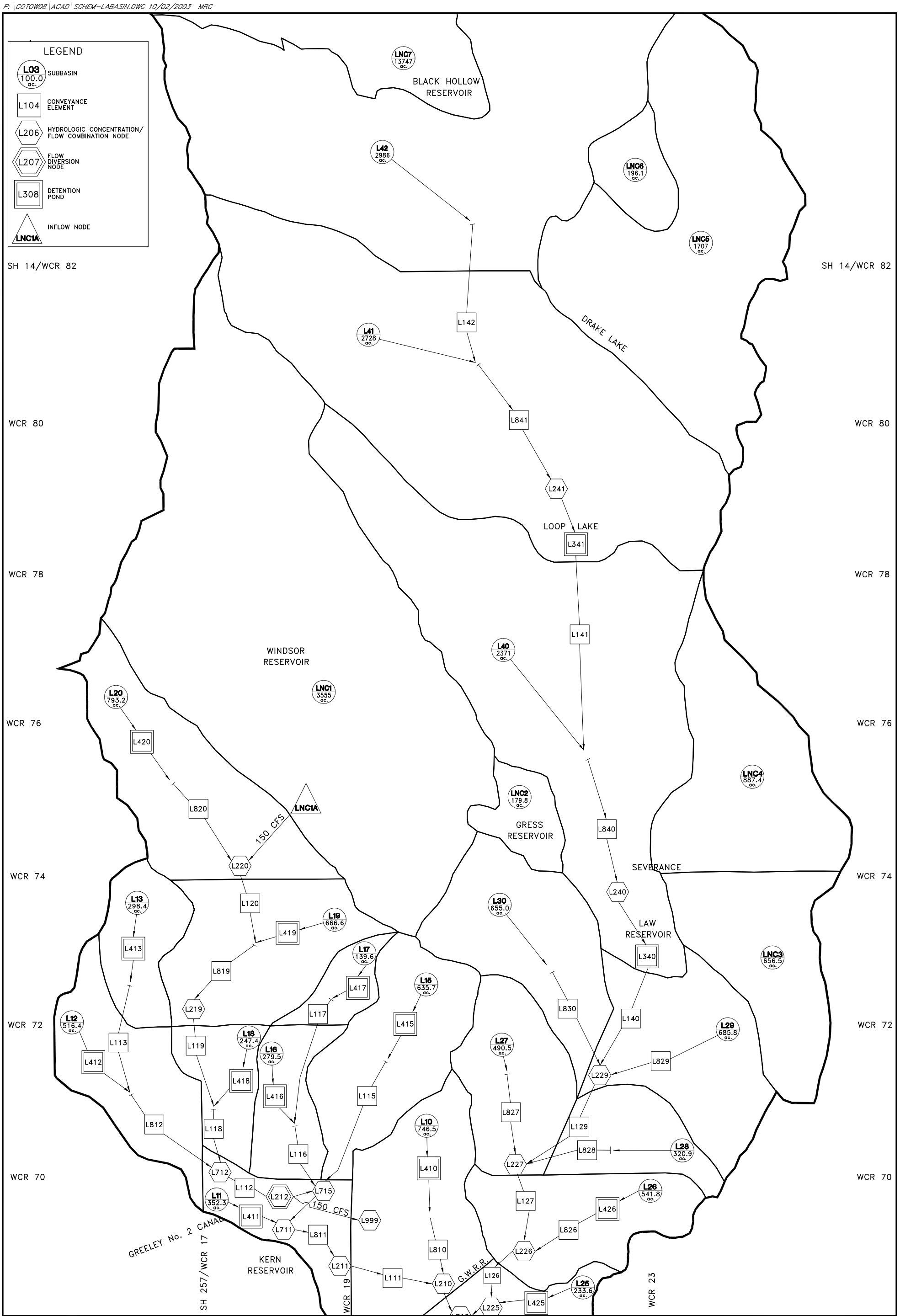

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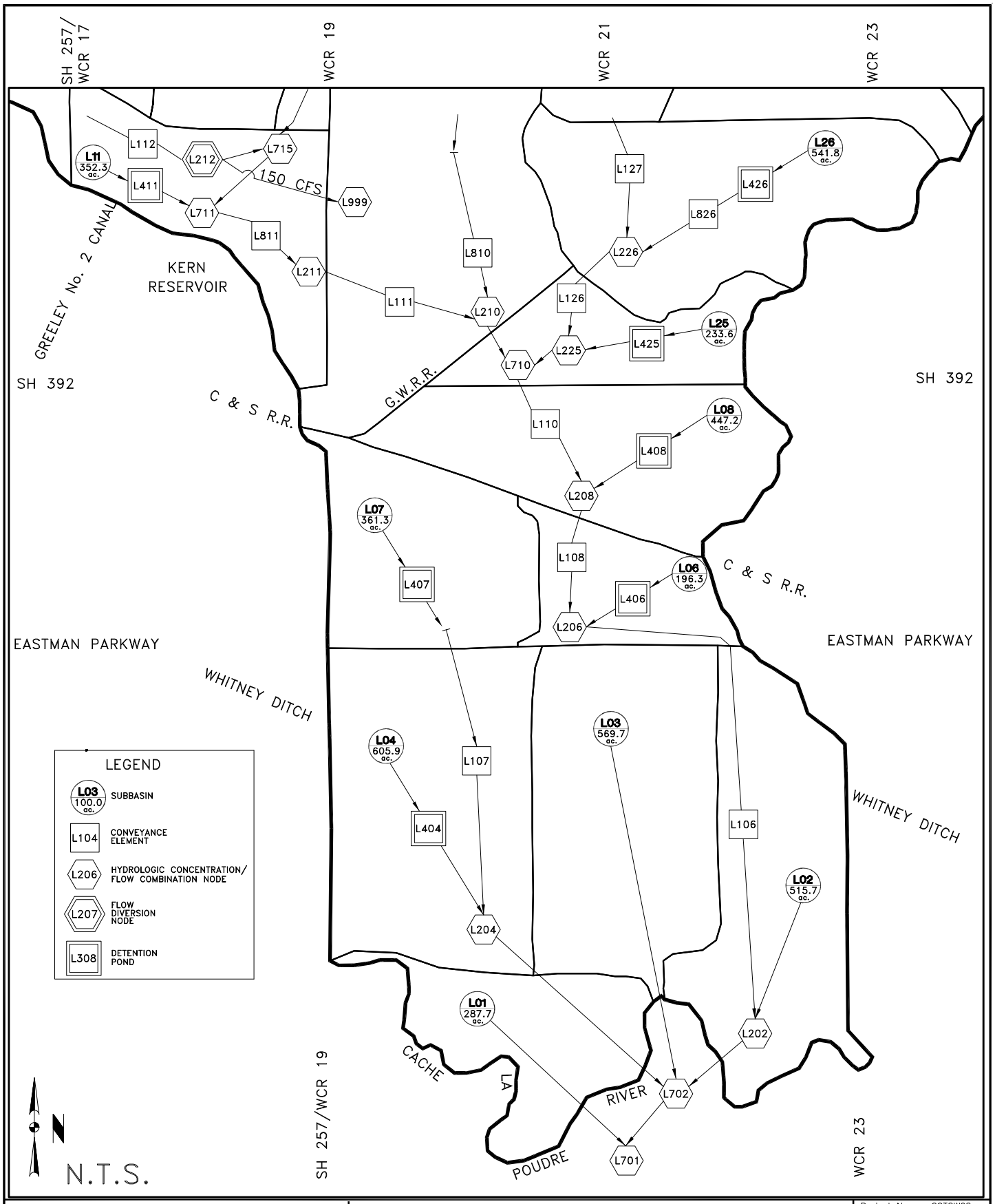
TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN

LAW BASIN-UPPER
SWMM SCHEMATIC-FUTURE CONDITION
WITH OVER-DETENTION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-LABASIN

A3-LA



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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN

LAW BASIN-LOWER
SWMM SCHEMATIC-FUTURE CONDITION WITH OVER-DETENTION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE:	SCHEM-LABASIN

A.3-LB

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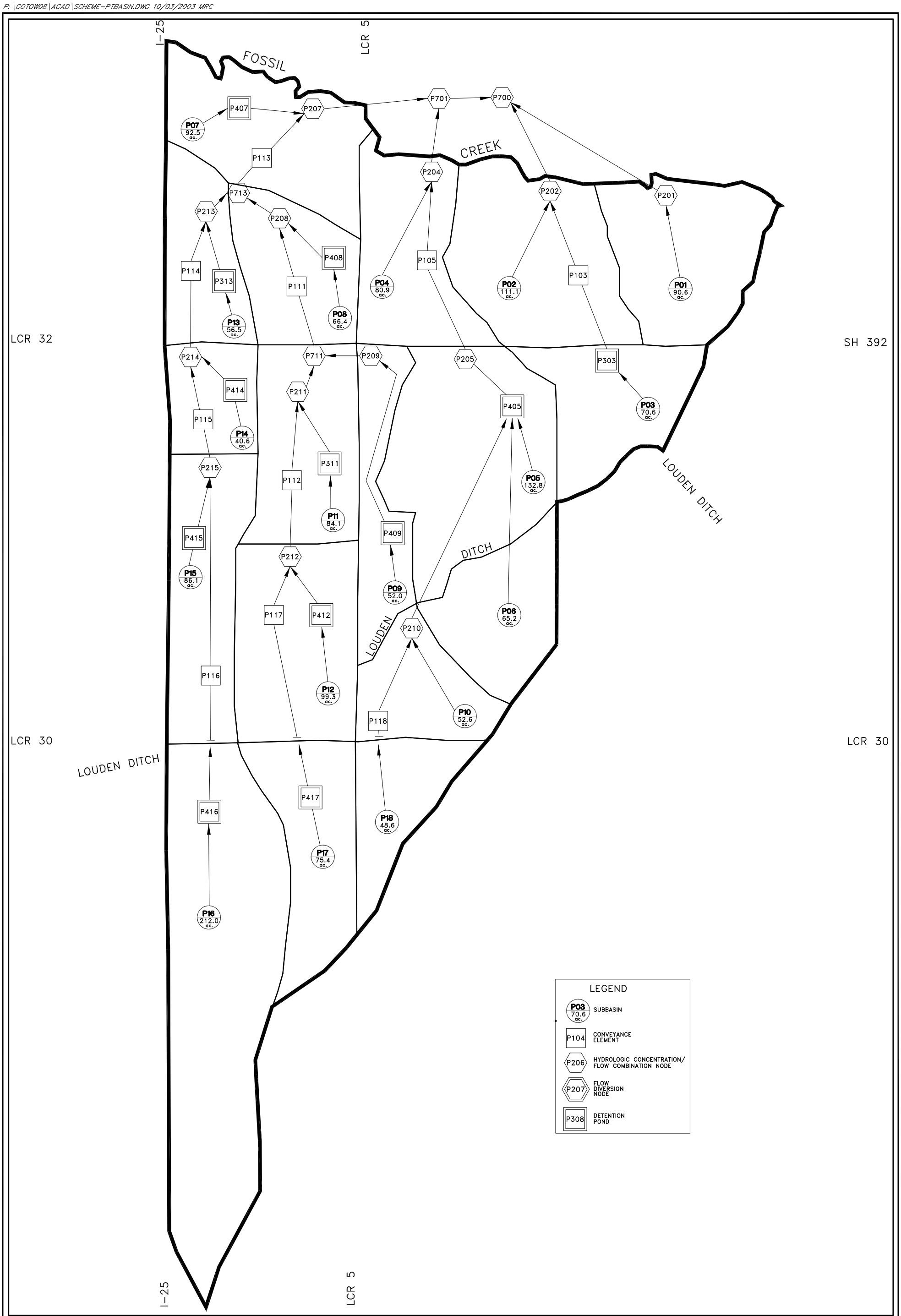

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN

PTARMIGAN BASIN
SWMM SCHEMATIC-FUTURE CONDITION
WITH OVER-DETENTION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT, 2002 TW
ACADFILE: SCHEM-PTBASIN	

A3-P

APPENDIX A.4

FUTURE CONDITION WITH IMPROVEMENTS

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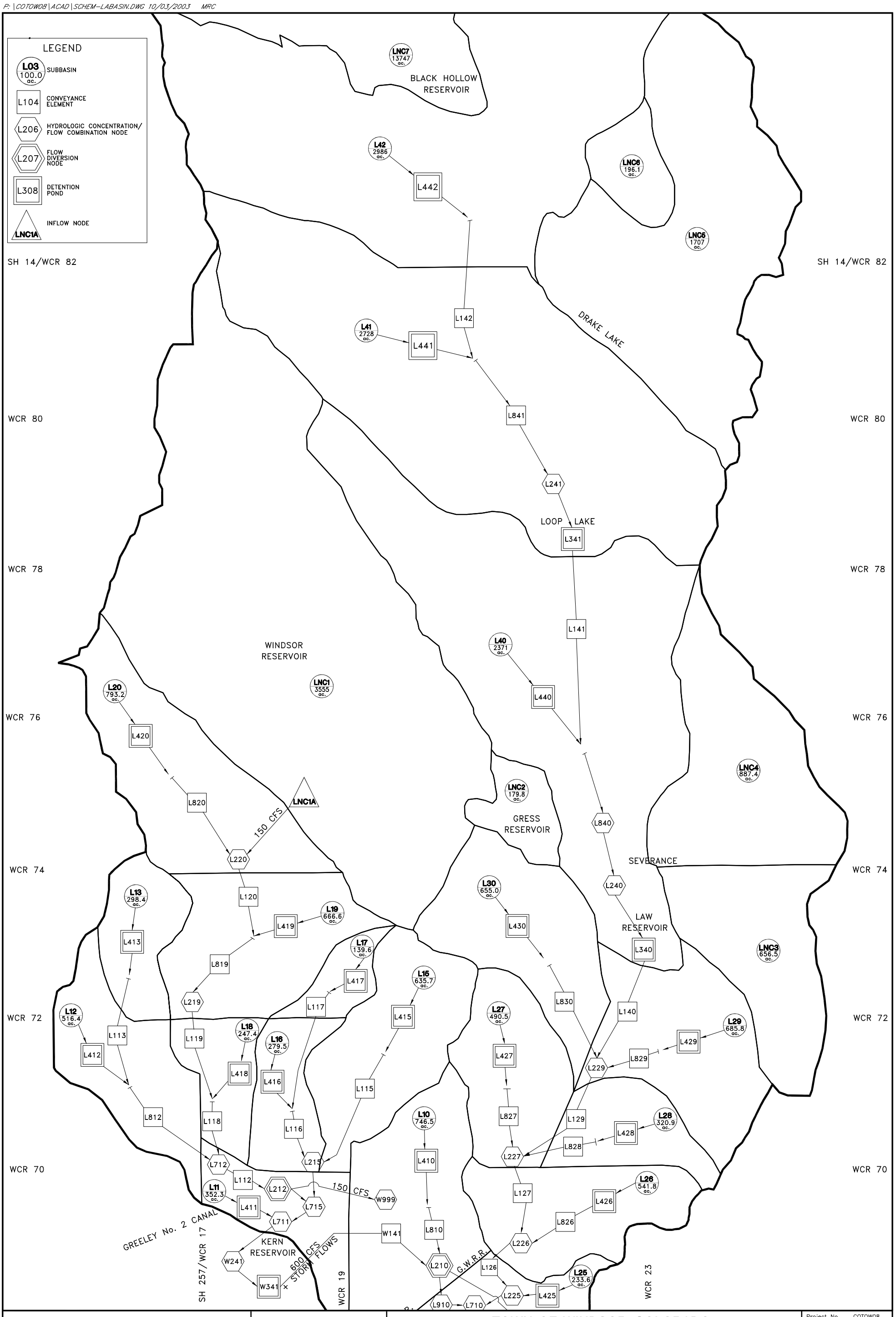

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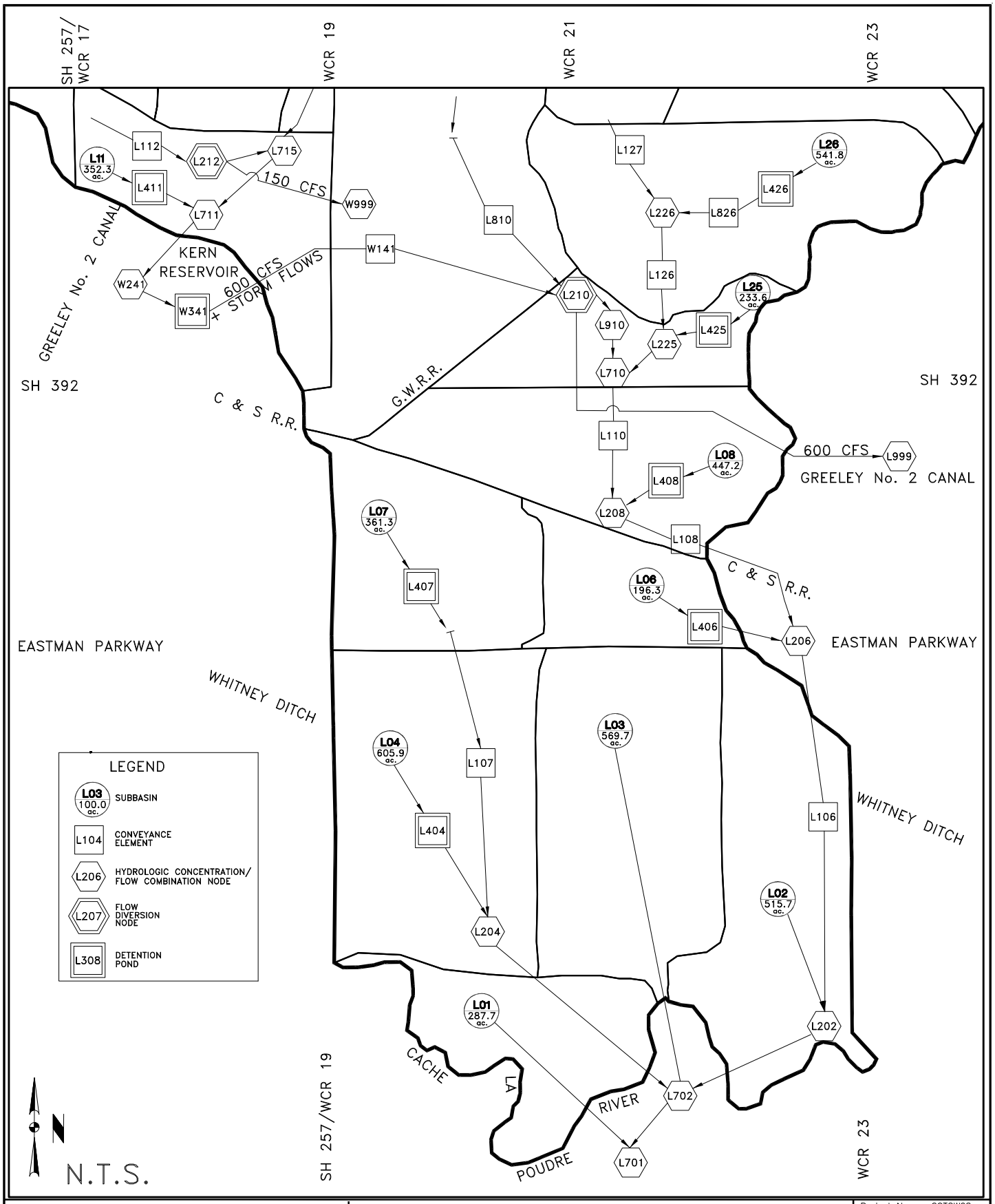
TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
LAW BASIN-UPPER
SWMM SCHEMATIC-IMPROVED CONDITION

Project No.	COTOW08
Date:	OCT, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT 31, 2002
ACADFILE: SCHEM-LABASIN	

A4-LA



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MASTER DRAINAGE PLAN
LAW BASIN-LOWER
SWMM SCHEMATIC-IMPROVED CONDITION

Project No.	COTOW08
Date:	OCT, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT 31, 2002
ACADFILE:	SCHEM-LABASIN

A4-LB

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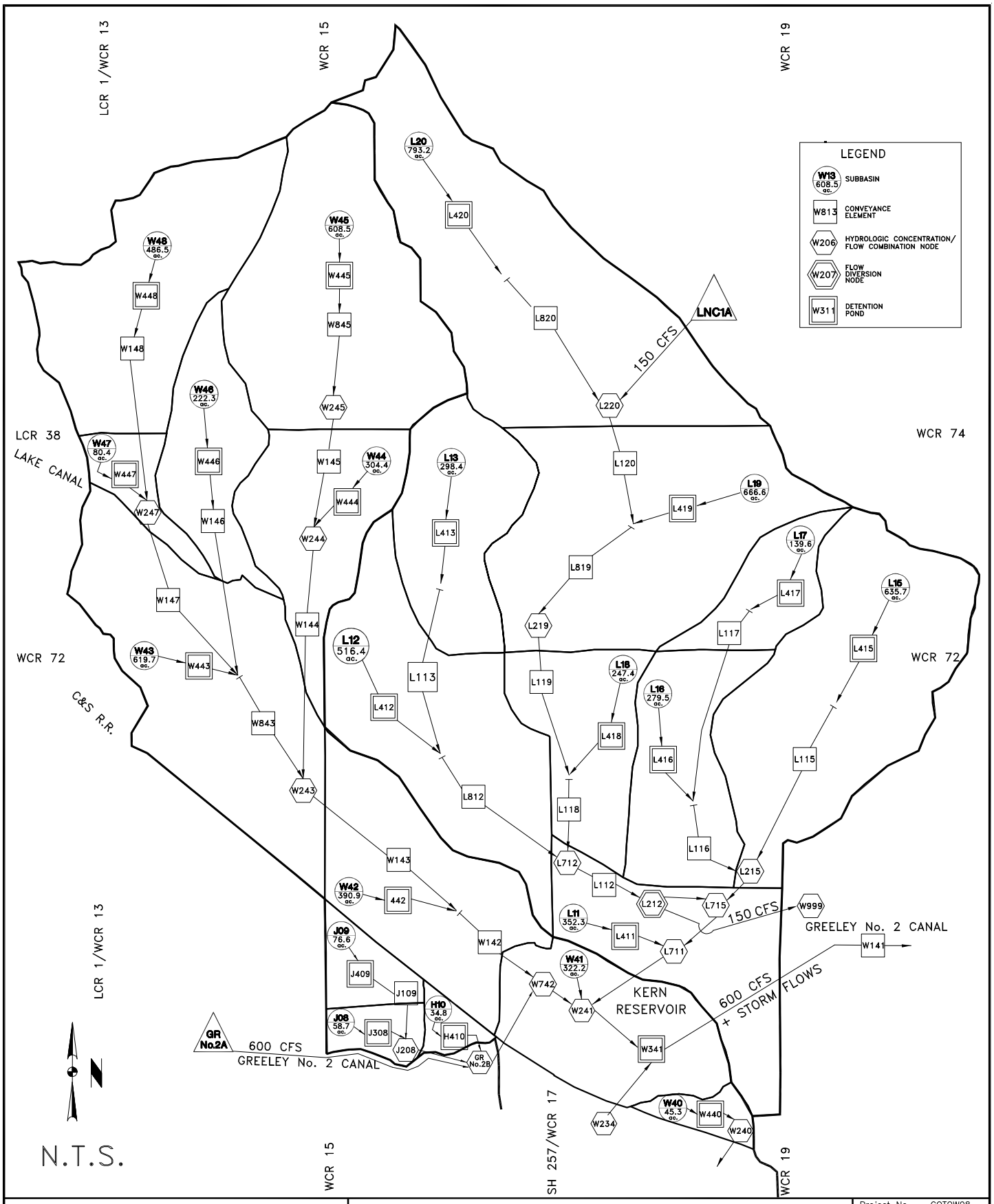

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MASTER DRAINAGE PLAN
WINDSOR BASIN-UPPER
SWMM SCHEMATIC-IMPROVED CONDITION

Project No.	COTOW08
Date:	OCT, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT 31, 2002
ACADFILE:	SCHEM-WIBASIN

A4-WA

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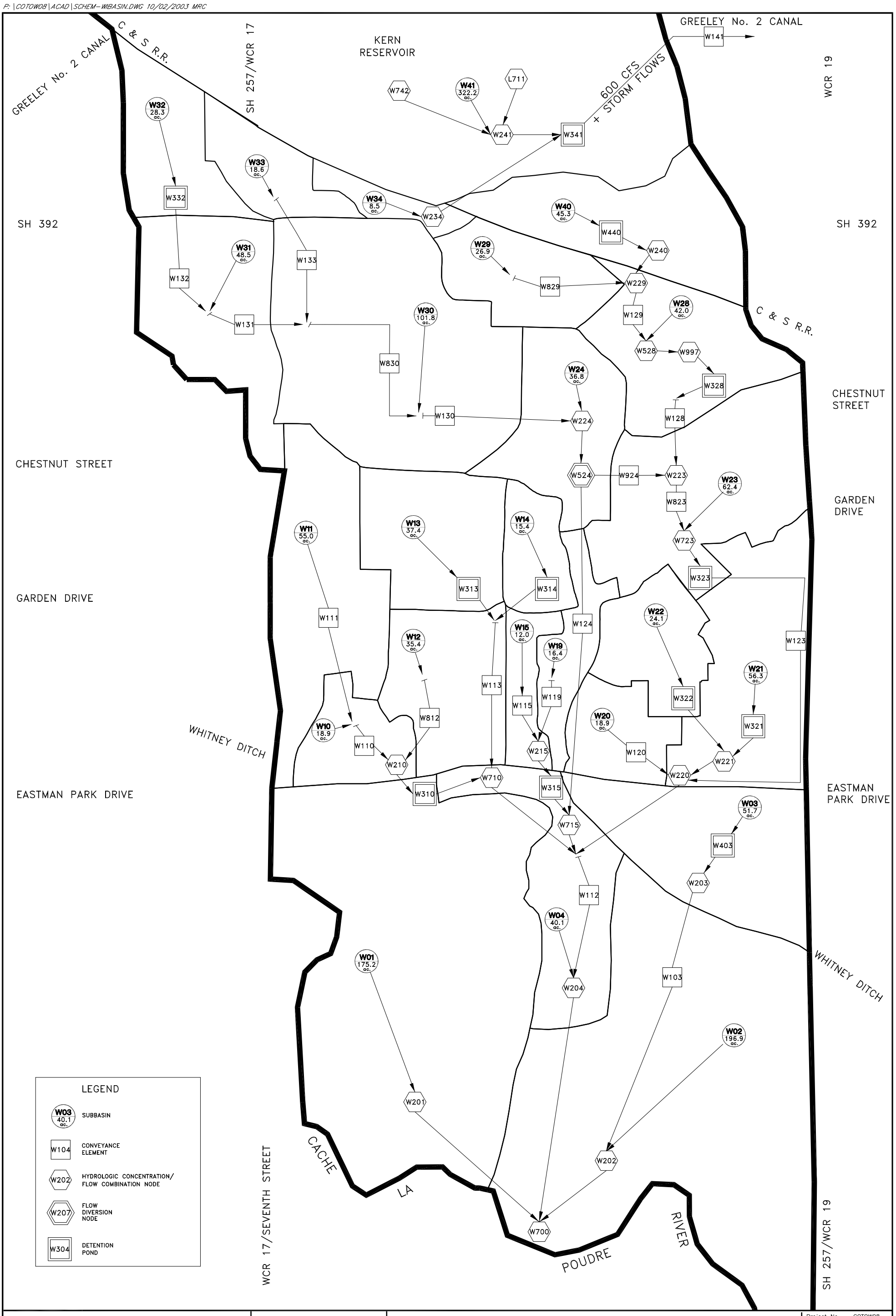

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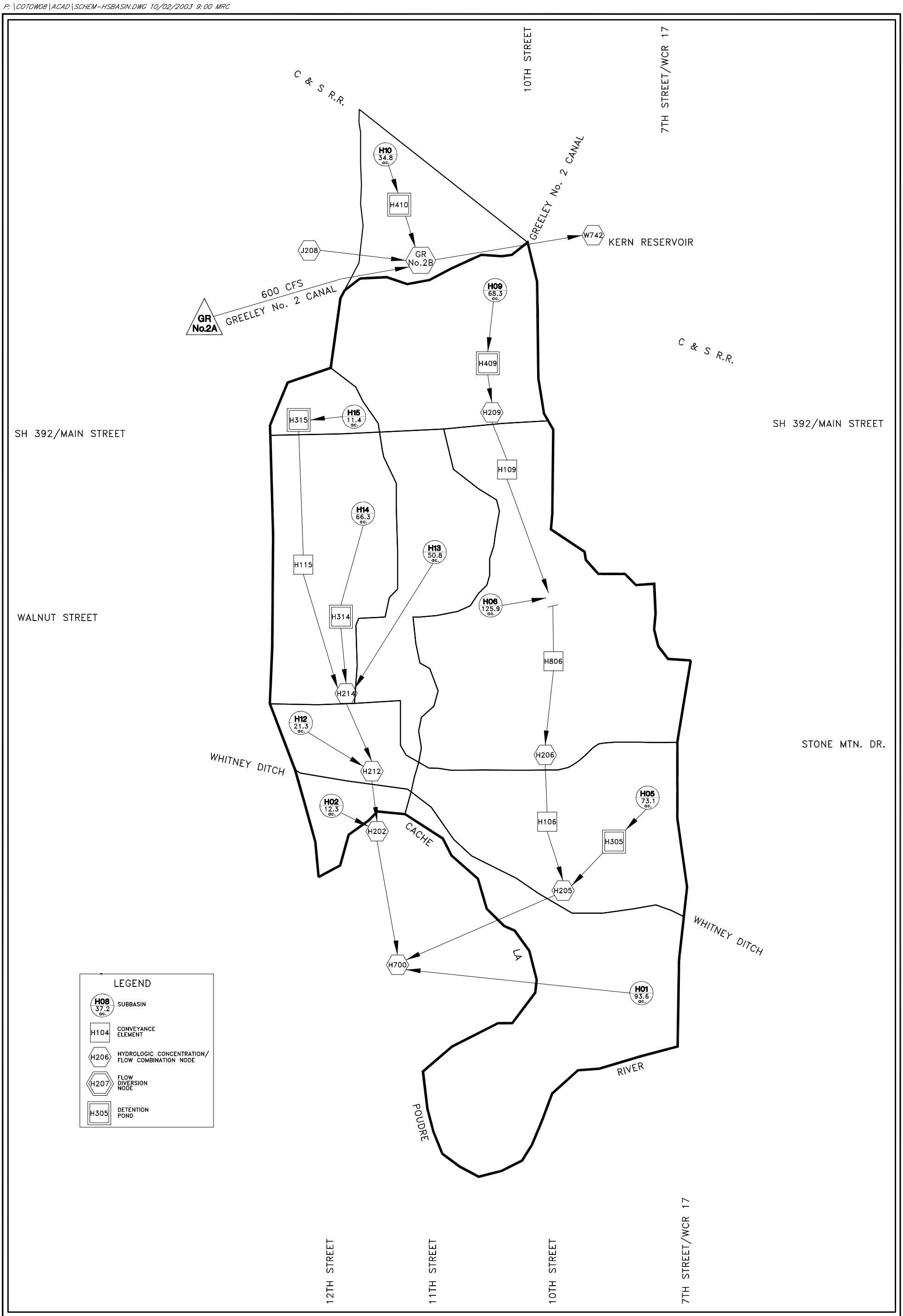
N.T.S.

TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
WINDSOR BASIN-LOWER
SWMM SCHEMATIC-IMPROVED CONDITION

Project No.	COTOW08
Date:	OCT, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT 31, 2002
ACADFILE: SCHEM-WIBASIN	

A4-WB



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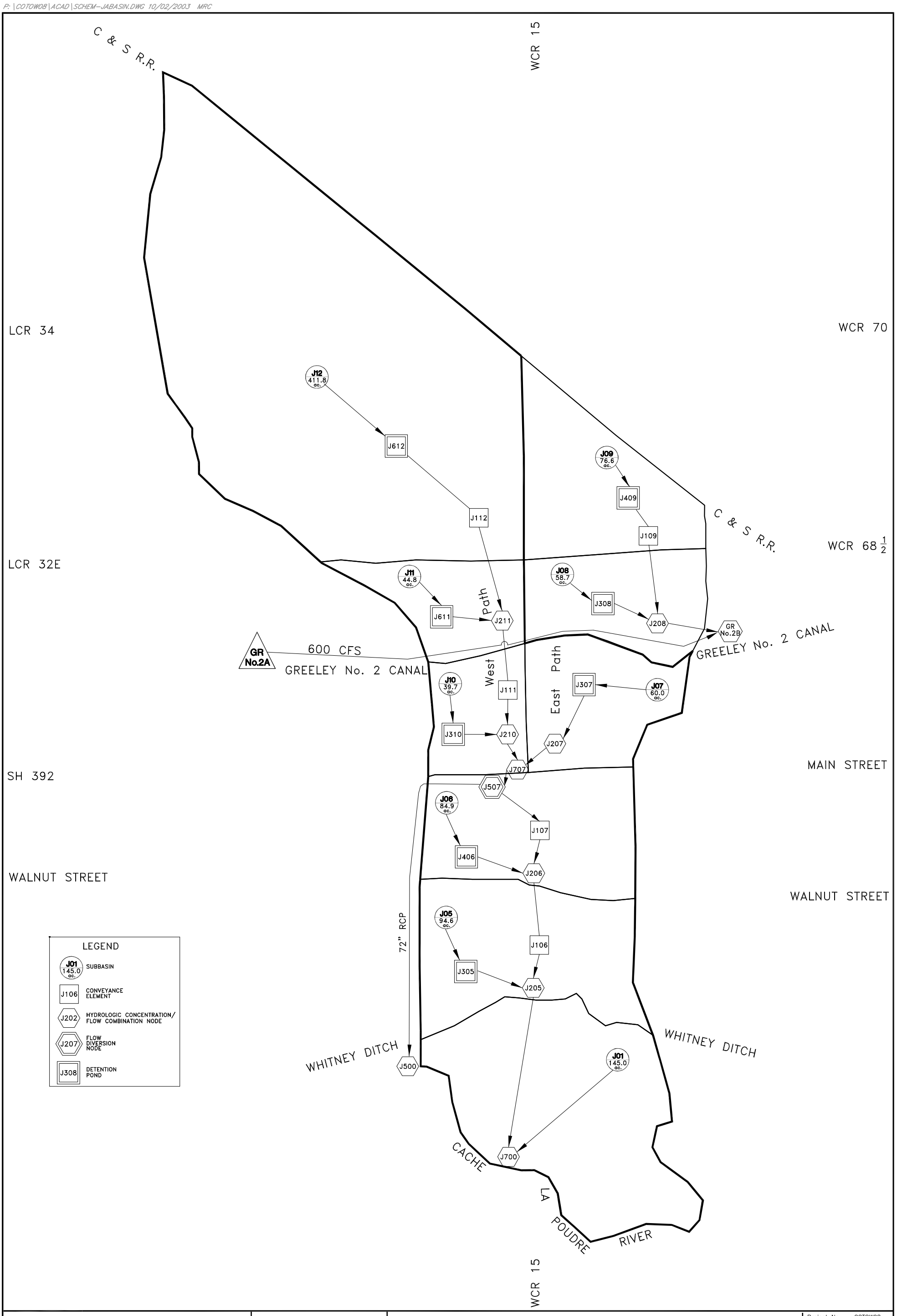
N.T.S.

TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
HIGH SCHOOL BASIN
SWMM SCHEMATIC-IMPROVED CONDITION

Project No.	COTOW08
Date:	OCT, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT 31, 2002
ACADFILE: SCHEM-HSBASIN	

A4-H

P:\COTOW08\ACAD\SCHEM-JABASIN.DWG 10/02/2003 MRC



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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
JACOBY BASIN
SWMM SCHEMATIC-IMPROVED CONDITION

Project No.	COTOW08
Date:	OCT. 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT 31. 2002
ACADFILE: SCHEM-JABASIN	

A4-J

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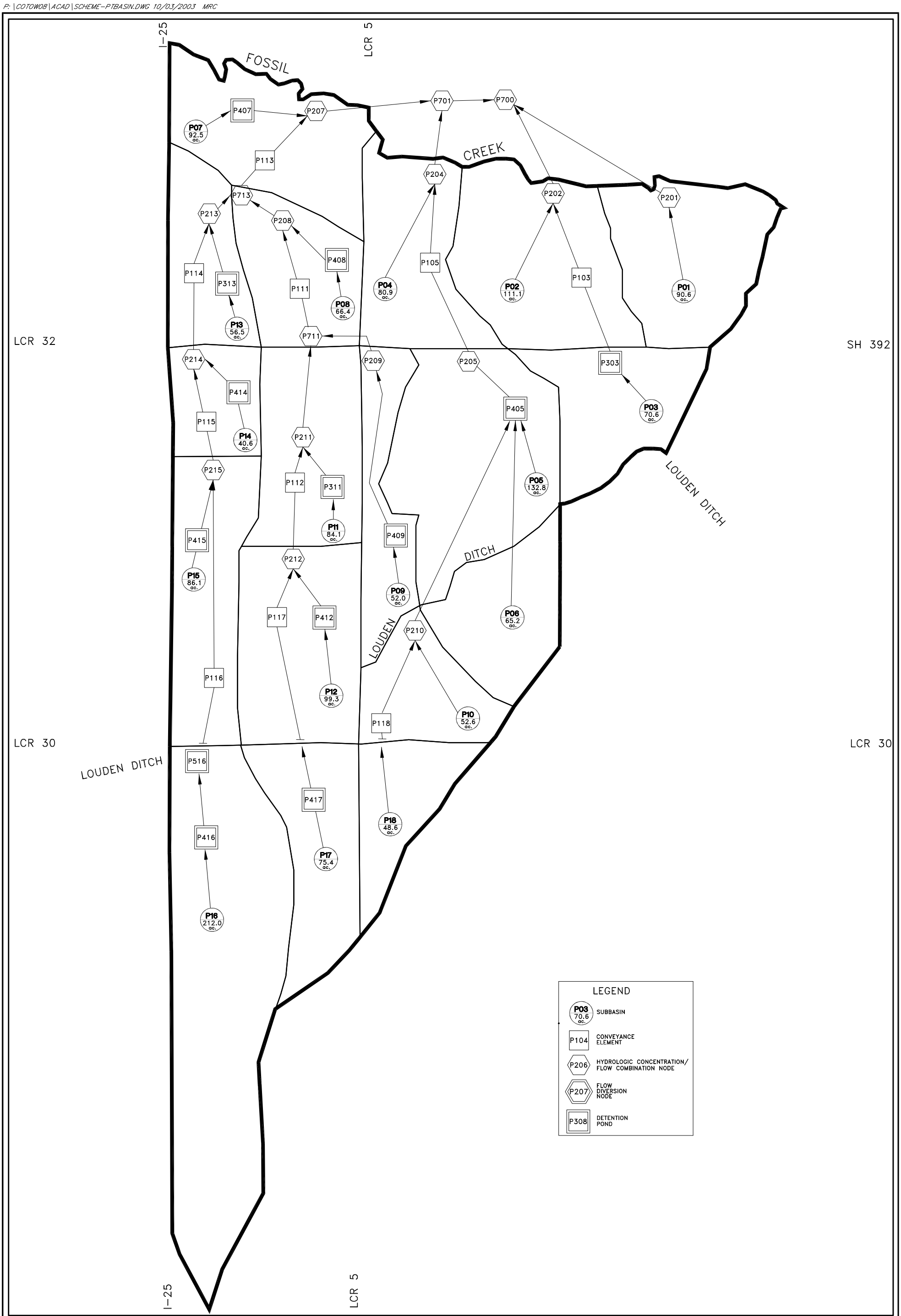

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TOWN OF WINDSOR, COLORADO
MASTER DRAINAGE PLAN
PTARMIGAN BASIN SWMM SCHEMATIC IMPROVED CONDITION

Project No.	COTOW08
Date:	JAN, 2001
Design:	CJP
Drawn:	MRC
Revisions:	OCT 31, 2002
ACADFILE:	SCHEM-PTBASIN

A4-P

APPENDIX B

SUBBASIN PEAK DISCHARGE TABLES

APPENDIX B.1

EXISTING CONDITION

Table B.1 Subbasin Peak Discharges for Existing Conditions.

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
LAW BASIN							
L01	287.7	150	217	261	354	424	508
L02	515.7	97	138	165	227	287	374
L03	569.7	617	900	1,083	1,431	1,690	2,002
L04	605.9	107	152	181	234	287	360
L06	196.3	51	73	87	116	142	174
L07	361.3	187	267	319	411	492	597
L08	447.2	56	79	94	134	201	327
L10	746.5	101	142	170	222	339	596
L11	352.3	45	64	76	99	149	264
L12	516.4	90	128	152	198	300	526
L13	298.4	63	89	106	136	173	252
L15	635.7	103	146	174	230	333	577
L16*	279.5	104	146	173	220	278	373
L17	139.6	33	47	56	73	93	137
L18*	247.4	111	155	181	233	298	382
L19	666.6	123	174	208	279	364	593
L20	793.2	166	236	281	370	466	630
L25	233.6	29	41	49	69	112	191
L26	541.8	91	128	153	201	267	461
L27	490.5	61	87	103	144	273	470
L28	320.9	50	70	84	109	154	271
L29	685.8	86	121	144	188	295	526
L30	655.0	83	118	141	183	289	514
L40	2,371.1	480	679	809	1,075	1,425	2,393
L41	2,727.5	552	781	930	1,226	1,797	3,009
L42	2,985.6	395	559	665	941	2,049	3,466
LNC1	3,554.6	2,433	3,462	4,132	5,612	6,951	8,629
LNC2	179.8	159	226	270	360	447	558
LNC3	656.5	335	477	568	754	929	1,165
LNC4	887.4	232	329	392	507	629	798
LNC5	1,707.3	388	549	654	852	1,095	1,636
LNC6	196.1	24	35	41	55	105	184
LNC7	13,747.4	2,781	3,936	4,687	6,104	8,165	13,990
WINDSOR BASIN							
W01	175.2	241	344	413	573	688	824
W02	196.9	269	382	457	641	776	936
W03	51.7	9	13	15	22	30	49
W04	40.1	44	62	74	101	122	149
W10	18.9	20	28	34	45	54	65
W11	55.0	57	82	98	128	153	183
W12	35.4	37	53	63	84	101	123
W13*	37.4	5	8	9	18	27	37

*Discharge represents a detained release off the subbasin.

Table B.1 Subbasin Peak Discharges for Existing Conditions (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
WINDSOR BASIN (continued)							
W14*	15.4	2	3	4	8	11	15
W15	12.0	13	18	21	28	34	42
W19	16.4	17	25	29	39	47	57
W20	18.9	20	28	34	45	54	65
W21*	56.3	8	11	13	27	40	56
W22*	24.1	3	5	6	12	17	24
W23	62.4	74	105	125	165	198	238
W24	36.8	44	62	74	98	119	144
W28	42.0	65	93	112	147	174	207
W29	26.9	32	45	54	72	87	105
W30	101.8	121	172	205	272	329	398
W31	48.5	51	72	86	114	136	163
W32*	28.3	4	6	7	14	20	28
W33	18.6	24	35	42	55	65	78
W34	8.5	20	29	34	45	53	61
W40	45.3	23	33	39	52	62	75
W41	322.2	550	823	1,008	1,381	1,622	1,899
W42	390.9	54	77	91	123	160	259
W43	619.7	165	233	278	378	487	656
W44	304.4	50	70	83	115	168	288
W45	608.5	164	231	276	380	506	755
W46	222.3	32	45	54	74	113	195
W47	80.4	11	15	18	23	36	72
W48	486.5	100	141	168	224	300	491
HIGH SCHOOL BASIN							
H01	93.6	65	92	111	163	202	248
H02	12.3	6	9	11	15	19	24
H05*	73.1	11	15	17	36	53	73
H06	125.9	160	227	271	381	481	601
H09*	68.3	10	14	16	30	43	59
H10*	34.8	5	7	8	16	23	33
H12	21.3	20	28	34	49	62	79
H13	50.8	82	116	138	190	234	286
H14*	66.3	9	13	16	34	48	66
H15*	11.4	2	2	3	6	8	11
JACOBY BASIN							
J01	145.0	95	137	165	231	278	334
J05*	94.6	11	16	23	49	69	95
J06*	84.9	12	17	20	39	56	76
J07*	60.0	8	12	14	30	44	60
J08*	58.7	8	12	14	29	42	59

*Discharge represents a detained release off the subbasin.

Table B.1 Subbasin Peak Discharges for Existing Conditions (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
JACOBY BASIN (continued)							
J09	76.6	11	15	18	24	31	47
J10*	39.7	5	8	9	20	28	40
J11*	44.8	6	9	11	22	32	45
J12	411.8	57	81	96	129	161	209
TIMNATH RESERVOIR BASIN							
T01	310.6	255	376	455	621	739	880
T02*	211.9	29	41	49	89	126	177
T04	180.1	24	34	40	57	73	110
T05	242.3	90	128	153	205	250	306
T06	1,056.0	144	203	242	332	454	775
T07	536.4	97	137	164	219	279	385
T08	1,151.6	153	216	258	380	668	1,142
T20	1,238.8	1,553	2,228	2,666	3,607	4,306	5,141
T21	2,486.3	608	862	1,026	1,383	1,765	2,451
T22	717.9	90	127	151	205	391	679
T23	698.9	91	128	153	204	317	550
T24	1,568.6	196	277	330	434	742	1,304
T25	1,043.6	381	541	645	855	1,053	1,318
T26	3,545.7	611	865	1,030	1,362	1,775	2,958
PTARMIGAN BASIN							
P01	90.6	96	135	162	231	297	376
P02	111.1	123	174	208	291	365	453
P03*	70.6	23	27	30	41	53	68
P04	80.9	89	126	150	215	270	337
P05*	132.8	6	9	11	17	22	28
P06	65.2	9	13	15	24	43	71
P07*	92.5	13	19	22	52	83	120
P08*	66.4	9	13	16	28	43	63
P09*	52.0	9	12	16	26	33	43
P10	52.6	7	10	12	28	48	73
P11*	84.1	1	1	1	2	2	3
P12*	99.3	4	6	10	18	25	33
P13*	56.5	8	11	13	29	41	57
P14*	40.6	7	10	11	18	24	31
P15	86.1	12	17	20	32	51	85
P16	212.0	29	42	50	110	193	292
P17	75.4	10	15	18	32	58	91
P18	48.6	7	10	11	20	35	54

*Discharge represents a detained release off the subbasin.

Table B.1 Subbasin Peak Discharges for Existing Conditions (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
RIVER RIDGE BASIN							
R01	73.4	71	100	120	166	211	267
R02	117.1	138	195	232	352	454	580
R03	143.1	38	53	64	110	195	328
R04	428.3	176	248	296	423	584	891
R05*	129.3	8	11	14	21	27	36
R06*	859.5	117	166	197	409	682	1,016
R07	872.5	116	164	196	367	696	1,101
R08*	1,127.3	155	220	258	531	848	1,267
BLUFF BASIN							
B01	186.8	26	37	44	155	286	474
B02	792.1	754	1,094	1,320	1,848	2,221	2,663
B04	271.1	246	352	422	593	720	872
B05*	159.1	22	32	37	100	167	246
B06*	144.7	20	28	34	90	139	195
B07*	173.4	24	34	41	101	176	267
B08	42.4	6	8	10	20	36	54
B09	113.9	16	23	27	101	197	322
B10	96.5	13	19	23	54	98	151
B11	71.7	10	14	17	67	127	207
B12	42.9	6	8	10	17	28	43
B13	114.5	20	28	33	80	154	262
B14	34.6	5	7	8	30	58	96
OKLAHOMA BASIN							
O01	75.9	37	52	62	83	104	131
O02	1,171.8	151	214	255	554	1,142	1,833
O03	231.7	29	41	49	67	163	281
O04*	762.7	99	140	165	273	539	888
O05	1,073.5	134	189	226	317	673	1,167
O06*	174.4	25	35	41	83	121	170
O07	734.7	108	153	183	274	599	992
O08	1,402.4	277	391	466	677	1,069	1,695
O09	1,637.3	515	729	869	1,184	1,512	1,959
SOUTH STATE HIGHWAY 257 BASIN							
S01	127.4	42	59	71	107	145	237
S02	812.6	111	157	187	564	1,062	1,967

*Discharge represents a detained release off the subbasin.

APPENDIX B.2

FUTURE CONDITION

Table B.2 Subbasin Peak Discharges for uture Conditions.

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
LAW BASIN							
L01	287.7	150	217	261	354	424	508
L02	515.7	97	138	165	227	287	374
L03	569.7	617	900	1,083	1,431	1,690	2,002
L04*	605.9	103	150	181	243	294	360
L06*	196.3	51	73	87	118	143	174
L07*	361.3	190	272	319	414	499	597
L08*	447.2	57	80	94	153	226	327
L10*	746.5	102	144	170	292	423	596
L11*	352.3	44	63	76	136	192	264
L12*	516.4	92	130	152	262	385	526
L13*	298.4	63	90	106	139	186	252
L15*	635.7	105	148	174	222	353	577
L16*	279.5	102	146	172	220	281	372
L17*	139.6	33	47	56	71	97	137
L18*	247.4	107	153	180	231	290	381
L19*	666.6	125	177	208	310	431	593
L20*	793.2	169	240	281	356	462	630
L25*	233.6	30	42	49	85	129	191
L26*	541.8	94	131	153	186	279	460
L27	490.5	61	87	103	144	273	470
L28	320.9	50	70	84	109	154	271
L29	685.8	86	121	144	188	295	526
L30	655.0	83	118	141	183	289	514
L40	2,371.1	480	679	809	1,075	1,425	2,393
L41	2,727.5	552	781	930	1,226	1,797	3,009
L42	2,985.6	395	559	665	941	2,049	3,466
LNC1	3,554.6	2,433	3,462	4,132	5,612	6,951	8,629
LNC2	179.8	159	226	270	360	447	558
LNC3	656.5	335	477	568	754	929	1,165
LNC4	887.4	232	329	392	507	629	798
LNC5	1,707.3	388	549	654	852	1,095	1,636
LNC6	196.1	24	35	41	55	105	184
LNC7	13,747.4	2,781	3,936	4,687	6,104	8,165	13,990
WINDSOR BASIN							
W01	175.2	241	344	413	573	688	824
W02	196.9	269	382	457	641	776	936
W03*	51.7	9	13	15	27	36	49
W04	40.1	44	62	74	101	122	149
W10	18.9	20	28	34	45	54	65
W11	55.0	57	82	98	128	153	183
W12	35.4	37	53	63	84	101	123
W13*	37.4	5	8	9	18	27	37

*Discharge represents a detained release off the subbasin.

Table B.2 Subbasin Peak Discharges for uture Conditions (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
WINDSOR BASIN (continued)							
W14*	15.4	2	3	4	8	11	15
W15	12.0	13	18	21	28	34	42
W19	16.4	17	25	29	39	47	57
W20	18.9	20	28	34	45	54	65
W21*	56.3	8	11	13	27	40	56
W22*	24.1	3	5	6	12	17	24
W23	62.4	74	105	125	165	198	238
W24	36.8	44	62	74	98	119	144
W28	42.0	65	93	112	147	174	207
W29	26.9	32	45	54	72	87	105
W30	101.8	121	172	205	272	329	398
W31	48.5	51	72	86	114	136	163
W32*	28.3	4	6	7	14	20	28
W33	18.6	24	35	42	55	65	78
W34	8.5	20	29	34	45	53	61
W40*	45.3	23	33	39	51	61	75
W41	322.2	559	838	1,027	1,407	1,652	1,932
W42*	390.9	54	77	91	147	196	259
W43*	619.7	168	237	278	382	504	656
W44*	304.4	51	71	83	143	207	288
W45*	608.5	168	236	276	409	564	755
W46*	222.3	33	46	54	95	139	195
W47*	80.4	11	15	18	28	44	72
W48*	486.5	100	142	168	239	339	491
HIGH SCHOOL BASIN							
H01	93.6	65	92	111	163	202	248
H02	12.3	6	9	11	15	19	24
H05*	73.1	11	15	17	36	53	73
H06	125.9	160	227	271	381	481	601
H09*	68.3	9	14	16	31	43	59
H10*	34.8	5	7	8	16	23	33
H12	21.3	20	28	34	49	62	79
H13	50.8	82	116	138	190	234	286
H14*	66.3	9	13	16	34	48	66
H15*	11.4	2	2	3	6	8	11
JACOBY BASIN							
J01	145.0	95	137	165	231	278	334
J05*	94.6	11	16	23	49	69	95
J06*	84.9	12	17	20	40	56	76
J07*	60.0	8	12	14	30	44	60
J08*	58.7	8	12	14	29	42	59

*Discharge represents a detained release off the subbasin.

Table B.2 Subbasin Peak Discharges for uture Conditions (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
JACOBY BASIN (continued)							
J09*	76.6	11	15	18	27	36	47
J10*	39.7	5	8	9	20	28	40
J11*	44.8	6	9	11	22	32	45
J12*	411.8	57	81	96	133	166	209
TIMNATH RESERVOIR BASIN							
T01*	310.6	261	381	455	602	727	879
T02*	211.9	28	40	49	92	129	177
T04	180.1	24	34	40	57	73	110
T05*	242.3	90	129	152	198	239	305
T06*	1,056.0	145	205	241	406	567	775
T07*	536.4	99	139	164	232	299	384
T08*	1,151.6	155	218	256	530	803	1,140
T20*	1,238.8	1,586	2,242	2,665	3,709	4,373	5,136
T21*	2,486.3	608	868	1,023	1,248	1,709	2,449
T22*	717.9	92	129	150	205	395	677
T23*	698.9	91	129	152	197	330	549
T24*	1,568.6	200	279	330	410	735	1,301
T25*	1,043.6	391	543	644	872	1,063	1,315
T26	3,545.7	611	865	1,030	1,362	1,775	2,958
PTARMIGAN BASIN							
P01	90.6	96	135	162	231	297	376
P02	111.1	123	174	208	291	365	453
P03*	70.6	23	27	30	41	53	68
P04	80.9	89	126	150	215	270	337
P05*	132.8	6	8	10	17	22	28
P06	65.2	72	102	122	176	222	278
P07*	92.5	12	18	22	57	86	119
P08*	66.4	9	13	16	32	45	63
P09*	52.0	9	13	16	26	34	43
P10	52.6	58	82	99	160	203	255
P11*	84.1	1	1	1	2	2	3
P12*	99.3	1	1	2	3	3	4
P13*	56.5	8	11	13	29	41	57
P14*	40.6	8	12	15	21	25	31
P15*	86.1	6	9	11	14	16	19
P16	212.0	29	42	50	110	193	292
P17*	75.4	1	1	2	6	10	15
P18	48.6	26	37	44	63	80	101

*Discharge represents a detained release off the subbasin.

Table B.2 Subbasin Peak Discharges for Future Conditions (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
RIVER RIDGE BASIN							
R01	73.4	71	100	120	166	211	267
R02	117.1	138	195	232	352	454	580
R03*	143.1	38	54	63	136	224	327
R04*	428.3	176	251	296	471	666	891
R05*	129.3	8	11	14	21	27	36
R06*	859.5	116	163	197	494	739	1,016
R07*	872.5	118	166	195	515	788	1,100
R08*	1,127.3	157	220	258	604	913	1,267
BLUFF BASIN							
B01*	186.8	27	38	44	200	326	474
B02*	792.1	776	1,103	1,320	1,780	2,180	2,663
B04*	271.1	246	352	421	550	694	872
B05*	159.1	23	32	37	112	174	246
B06*	144.7	19	26	34	94	142	195
B07*	173.4	24	35	40	115	185	267
B08*	42.4	5	8	10	26	39	54
B09*	113.9	16	23	27	137	223	322
B10*	96.5	13	18	23	70	108	151
B11*	71.7	10	15	17	85	140	206
B12*	42.9	6	8	10	22	31	43
B13*	114.5	20	28	33	118	186	262
B14*	34.6	5	7	8	43	69	96
OKLAHOMA BASIN							
O01	75.9	37	52	62	83	104	131
O02*	1,171.8	157	216	255	531	1,091	1,826
O03*	231.7	30	42	49	76	166	281
O04*	762.7	100	140	165	385	621	888
O05*	1,073.5	136	192	225	495	805	1,166
O06*	174.4	25	35	41	83	121	170
O07*	734.7	111	156	183	295	599	990
O08*	1,402.4	278	395	466	756	1,175	1,692
O09*	1,637.3	519	738	869	1,059	1,422	1,955
SOUTH STATE HIGHWAY 257 BASIN							
Future condition discharges are the same as existing conditions. Future development not anticipated in this basin.							

*Discharge represents a detained release off the sub-basin.

APPENDIX B.3

FUTURE CONDITIONS WITH OVER-DETENTION

Table B.□ Subbasin Peak Discharges for □uture Conditions □ith □ er-Detention.

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
LAW BASIN							
L01	287.7	150	217	261	354	424	508
L02	515.7	97	138	165	227	287	374
L03	569.7	617	900	1,083	1,431	1,690	2,002
L04*	605.9	53	77	93	126	151	181
L06*	196.3	23	33	40	58	71	87
L07*	361.3	82	117	140	202	254	319
L08*	447.2	14	20	25	48	69	94
L10*	746.5	34	50	60	94	127	170
L11*	352.3	16	24	29	44	58	76
L12*	516.4	28	41	49	81	113	152
L13*	298.4	19	27	33	54	77	106
L15*	635.7	16	23	28	62	110	174
L16*	279.5	30	43	51	86	123	173
L17*	139.6	8	11	14	25	38	56
L18*	247.4	33	47	55	90	129	181
L19*	666.6	35	52	62	109	153	208
L20*	793.2	46	67	81	140	201	281
L25*	233.6	8	12	14	24	35	49
L26*	541.8	13	19	23	52	95	153
L27*	490.5	5	7	9	32	62	103
L28*	320.9	5	8	9	24	49	84
L29*	685.8	8	12	14	39	83	144
L30*	655.0	8	11	14	38	82	141
L40*	2,371.1	59	85	102	266	498	809
L41*	2,727.5	66	95	114	295	568	930
L42*	2,985.6	32	46	55	207	407	665
LNC1	3,554.6	2,433	3,462	4,132	5,612	6,951	8,629
LNC2	179.8	159	226	270	360	447	558
LNC3	656.5	335	477	568	754	929	1,165
LNC4	887.4	232	329	392	507	629	798
LNC5	1,707.3	388	549	654	852	1,095	1,636
LNC6	196.1	24	35	41	55	105	184
LNC7	13,747.4	2,781	3,936	4,687	6,104	8,165	13,990
WINDSOR BASIN							
W01	175.2	241	344	413	573	688	824
W02	196.9	269	382	457	641	776	936
W03*	51.7	9	13	15	27	36	49
W04	40.1	44	62	74	101	122	149
W10	18.9	20	28	34	45	54	65
W11	55.0	57	82	98	128	153	183
W12	35.4	37	53	63	84	101	123
W13*	37.4	5	8	9	18	27	37

*Discharge represents a detained release off the subbasin.

Table B.□ Subbasin Peak Discharges for □uture Conditions □ith □ e□r-Detention (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
WINDSOR BASIN (continued)							
W14*	15.4	2	3	4	8	11	15
W15	12.0	13	18	21	28	34	42
W19	16.4	17	25	29	39	47	57
W20	18.9	20	28	34	45	54	65
W21*	56.3	8	11	13	27	40	56
W22*	24.1	3	5	6	12	17	24
W23	62.4	74	105	125	165	198	238
W24	36.8	44	62	74	98	119	144
W28	42.0	65	93	112	147	174	207
W29	26.9	32	45	54	72	87	105
W30	101.8	121	172	205	272	329	398
W31	48.5	51	72	86	114	136	163
W32*	28.3	4	6	7	14	20	28
W33	18.6	24	35	42	55	65	78
W34	8.5	20	29	34	45	53	61
W40*	45.3	9	13	15	23	30	39
W41	322.2	559	838	1,027	1,407	1,652	1,932
W42*	390.9	19	28	34	54	70	91
W43*	619.7	51	74	89	152	207	278
W44*	304.4	15	22	26	44	62	83
W45*	608.5	45	65	78	144	202	276
W46*	222.3	10	14	17	29	40	54
W47*	80.4	2	3	4	6	11	18
W48*	486.5	24	34	41	76	116	168
HIGH SCHOOL BASIN							
H01	93.6	65	92	111	163	202	248
H02	12.3	6	9	11	15	19	24
H05*	73.1	11	15	17	36	53	73
H06	125.9	160	227	271	381	481	601
H09*	68.3	4	6	7	10	13	16
H10*	34.8	2	3	3	5	6	8
H12	21.3	20	28	34	49	62	79
H13	50.8	82	116	138	190	234	286
H14*	66.3	9	13	16	34	48	66
H15*	11.4	2	2	3	6	8	11
JACOBY BASIN							
J01	145.0	95	137	165	231	278	334
J05*	94.6	11	16	23	49	69	95
J06*	84.9	6	8	10	14	17	20
J07*	60.0	8	12	14	30	44	60
J08*	58.7	8	12	14	29	42	59

*Discharge represents a detained release off the subbasin.

Table B.□ Subbasin Peak Discharges for □uture Conditions □ith □ e□r-Detention (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
JACOBY BASIN (continued)							
J09*	76.6	4	6	7	10	14	18
J10*	39.7	5	8	9	20	28	40
J11*	44.8	6	9	11	22	32	45
J12*	411.8	21	31	37	56	74	96
TIMNATH RESERVOIR BASIN							
T01*	310.6	261	381	455	602	727	879
T02*	211.9	11	17	20	31	39	49
T04	180.1	24	34	40	57	73	110
T05*	242.3	26	36	43	75	108	153
T06*	1,056.0	49	72	86	136	183	242
T07*	536.4	37	54	65	97	127	164
T08*	1,151.6	49	72	86	141	193	258
T20*	1,238.8	720	1,019	1,205	1,733	2,165	2,666
T21*	2,486.3	129	186	224	470	711	1,026
T22*	717.9	10	15	18	49	93	151
T23*	698.9	14	20	24	54	97	153
T24*	1,568.6	21	32	38	99	198	330
T25*	1,043.6	85	120	141	265	424	645
T26*	3,545.7	73	104	125	324	623	1,030
PTARMIGAN BASIN							
P01	90.6	96	135	162	231	297	376
P02	111.1	123	174	208	291	365	453
P03*	70.6	23	27	30	41	53	68
P04	80.9	89	126	150	215	270	337
P05*	132.8	6	8	10	17	22	28
P06	65.2	72	102	122	176	222	278
P07*	92.5	6	8	10	15	18	22
P08*	66.4	5	7	8	11	13	16
P09*	52.0	8	12	15	24	31	40
P10	52.6	58	82	99	160	203	255
P11*	84.1	1	1	1	2	2	3
P12*	99.3	1	1	2	3	3	4
P13*	56.5	8	11	13	29	41	57
P14*	40.6	8	12	15	21	25	31
P15*	86.1	6	9	11	14	16	19
P16*	212.0	2	3	6	21	34	50
P17*	75.4	1	1	2	6	10	15
P18	48.6	26	37	44	63	80	101

*Discharge represents a detained release off the subbasin.

Table B.□ Subbasin Peak Discharges for □uture Conditions □ith □ er□-Detention (continued).

Subbasin No.	Drainage Area (acres)	Peak Discharge (cfs)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
RIVER RIDGE BASIN							
R01	73.4	71	100	120	166	211	267
R02	117.1	138	195	232	352	454	580
R03*	143.1	6	8	10	28	44	63
R04*	428.3	51	72	86	156	219	296
R05*	129.3	8	11	14	21	27	36
R06*	859.5	32	47	60	111	149	197
R07*	872.5	29	43	54	107	146	196
R08*	1,127.3	42	61	74	137	191	258
BLUFF BASIN							
□ er□-detention □as not □odeled in this basin.							
OKLAHOMA BASIN							
O01	75.9	37	52	62	83	104	131
O02*	1,171.8	11	17	20	88	161	255
O03*	231.7	3	5	6	16	31	49
O04*	762.7	29	42	51	85	121	165
O05*	1,073.5	37	54	65	113	163	226
O06*	174.4	25	35	41	83	121	170
O07*	734.7	13	19	23	66	117	182
O08*	1,402.4	55	81	97	226	332	466
O09*	1,637.3	117	167	200	419	614	869
SOUTH STATE HIGHWAY 257 BASIN							
□uture condition discharges are the sa□e as existing conditions. □uture de□elo□ □ent not antici□ated in this basin.							

*Discharge represents a detained release off the subbasin.

APPENDIX C

FINANCING PLAN