July 24, 2003

City of Ontario, Oregon

Storm Water Master Plan

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1.0 - EXECUTIVE SUMMARY

The City of Ontario has undertaken this Storm Water System Master Plan in order to evaluate the condition and capacity of their storm water facilities and to evaluate those facilities in light of future conditions. This master plan evaluates the existing system in regards to capacity, City standards and future conditions. Recommendations for improvements and upgrades necessary to solve existing problems, accommodate future growth, and meet City, State and Federal objectives in regards to storm water runoff and storm water quality are presented.

Key to the development of the recommended alternatives was the participation of the technical review committee. This committee consisted of the Public Works Director, Steve Gaschler; Engineering staff, Tom Davis and Dan Shepard; Wastewater Treatment Supervisor, Glen Schoeneman; Storm/Wastewater Collection Supervisor, Don Brown; and staff from Keller Associates.

1.1 STUDY AREA

Ontario is located along the western border of Idaho along the Snake River in an area dominated by agricultural activities. The area studied includes the entire area within the Urban Growth Boundary in addition to other areas that drain into the City's storm water system and discharges into the Snake River. The study area is shown in Figure 1 in Appendix A.

The City's economy is primarily based on agriculture, livestock and food processing. The top employers in Ontario include Heinz Frozen Foods, the Snake River Correctional Institution and the Holy Rosary Medical Center. The City of Ontario is the largest city in Malheur County, Oregon having a population of 11,140 people. By 2022, the population is expected to grow to 17,000 people.

Parts of the City of Ontario are located in the flood plains of both the Malheur River and the Snake River, as shown in Figure 3 in Appendix A. The City has experienced major floods, with the latest occurring in 1982 due to excessive rain and snowmelt. During periods of extreme high river levels, the City's storm system outfalls may be submerged preventing discharge into the river.

1.2 DESIGN CRITERIA

The storm water collection system was evaluated based upon design criteria determined by the Technical Review Committee (TRC). Initially the Heinz Frozen Foods (HFF) drainage basin was evaluated to determine the method to model the sub-basins of each major drainage basin. The results of this evaluation are presented in Technical Memo #3 included in Appendix B. Based upon the HFF analysis and the preliminary findings from the other basin models, the TRC also decided that the recommended improvements should eliminate all flooding, and minimize surcharging caused by the design storm event.

1.2.1 Assumptions

In the analysis of the storm water system, several assumptions, agreed upon by the TRC, were made to develop the model and evaluate Ontario's storm water system. These assumptions include the following:

- For this analysis "free" outfalls to the Snake River or Dork Canal were assumed. Occasional high river levels that submerge the storm water outfalls to the Snake River were assumed to not coincide with the design storm event. A detailed evaluation of river induced flooding was beyond the scope of this study.

- The pipes and catch basins in the network are clean of excessive sediment build-up, collapse or other problems that could impede flow.

- Only the major storm water pipelines (primarily 10-inches and larger) were evaluated.

- A 25-year, 24-hour, 1-inch cumulative rainfall design storm was used in the analysis of the pipeline network.

One additional assumption used in the evaluation of future needs was that the City would adopt a policy that would limit post development storm water runoff to the pre-development conditions. This would prevent storm water runoff conditions in the future from becoming more severe because of new development.

1.3 HYDROLOGIC MODEL

The computer model XP-SWMM was used to calculate the surface water runoff using the SCS curve number method, and dynamically route the runoff through the storm water collection system.

The City of Ontario was delineated into 12 major existing drainage basins, which were then delineated into smaller sub-basins dependent upon elevation, slopes and runoff direction. Each of the sub-basins was evaluated in regards to certain model parameters, as discussed in Section 4.

1.4 EXISTING STORM COLLECTION SYSTEM EVALUATION AND RECOMMENDATIONS

1.4.1 Existing Pipeline Structural Condition

The City has video recorded approximately 15% of their storm water lines, with an emphasis on problematic areas. Figure 8 in Appendix A illustrates the condition of the pipes. The condition of the video recorded lines is rated in terms of their priority need of replacement or repair.

Keller Associates recommends that the City of Ontario establish an annual pipeline replacement program. As part of this replacement program, it is recommended that pipelines less than 12-inches in diameter be replaced with at least a 12-inch pipeline. Assuming an average remaining pipeline life of 40 years, approximately 5,900 feet of pipe should be replaced annually costing approximately $273,000 a year.

1.4.2 Irrigation and Drainage Districts

The majority of the storm conveyance system within the City of Ontario is under the jurisdiction of the City with the exceptions of the Stewart Carter Canal and the 48-inch pipeline that conveys the Stewart Carter flows through Ontario, the Dork Canal, and a 24-inch line that runs from south of SW 4th Avenue north through the Waterford Subdivision and discharges in to the Dork Canal. These lines are under the jurisdiction of the Owyhee Ditch Company and Irrigation District and the Malheur Drainage District. A jurisdictional map is included as Figure 9 in Appendix A. These districts provide irrigation water to the agricultural land surrounding and throughout the City of Ontario and they transport the runoff from these lands to the river. There is an overlap between these lines and the storm water system requiring that cooperation be maintained between the City and these entities so that the joint use of the facilities can be continued. Interagency agreements should be reviewed and updated between the City and the Districts to outline each entity's responsibilities in regards to water quality, storm water runoff and the maintenance and operation of the jointly used facilities. Elements that should be included in this interagency agreement were discussed with the TRC and are included in Appendix E.

1.4.3 Collection System Evaluation and Recommendations

Refer to Figures 6, 7, and 11 through 16 in Appendix A for illustrations of the basin delineations, and Figures 18 and 21 for an illustration of the recommended improvements. Evaluations and recommendations are presented for each of the major basin areas below.

Wal-Mart Drainage Basin (see Figure 11)
This drainage basin is approximately 53 acres located just north of East Idaho Avenue and east of I-84. The storm water drains to the north to a trunk line that drains east and discharges through a 36-inch tidal flex valve into the Snake River. Model results indicate that the design storm results in no flooding and only minimal surcharging through the pipes in this basin. The TV logs showed that the pipelines in this basin were in fair to good condition with no areas needing repair at this time. No improvements are recommended at this time.

Kmart Drainage Basin (see Figure 11)
This 28 acre drainage basin is located just south of East Idaho Avenue and between I-84 and the Snake River. The area is primarily paved parking lots and large commercial buildings. The storm water flows to the north and then to the east to ultimately discharge into the Snake River. The pipelines in this basin are surcharged during a design storm event resulting in some flooding, as shown in Figure 10. As shown in Figure 8, the lines that were video recorded in this basin were determined to be in fair condition. Refer to Figures 8, 11 and 21 in Appendix A.

The following are recommended improvements for this basin:

- Upsize the 15-inch line along the north side of the Ernst parking lot from the outfall west to the manhole just west of East lane with a 24-inch pipeline.

- Upsize the north half of the 15-inch line along East Lane with a 24-inch pipeline.

- Add a cleanout at the south end of the eastern most line in the drainage basin.

Heinz Frozen Foods (HFF) Drainage Basin (see Figure 12)
The Heinz Frozen Foods (HFF) drainage basin was initially evaluated to determine the method to model the sub-basins of each drainage basin, with the discussion and results reported in Technical Memo #3 included in Appendix B. This basin covered an area of approximately 330 acres, with 32 acres of on-site retention. The pipelines in this basin were found to be surcharged with flooding in some areas during a design storm. The pipes in this drainage basin were also found to be in relatively good condition where video recorded with only one area needing repair in the next 20 years. See Figures 8, 12, and 21 in Appendix A.

The following are the recommended improvements for this basin:

- Correct the adverse pipeline grade or reverse flow condition between SE 5th Avenue and SE 4th Avenue along SE 4th Street

Double Trunk Line Drainage Basin-Downtown Ontario (see Figure 13)
This drainage basin covers the area immediately west of the railroad and has two major trunk lines running north through downtown Ontario. One of the trunk lines carries the storm water runoff from the majority of the downtown area, with the other trunk line carrying primarily agricultural runoff from south of SW 18th Avenue and the TVCC ball parks, and storm water runoff from a relatively small residential area between SW 18th Avenue and the Alameda soccer fields. The 24-inch trunk line that conveys the agricultural runoff is one of the older lines in the City of Ontario and have areas needing replacement or repair. Both lines are surcharged during the design storm event with significant flooding near the Treasure Valley Community College (TVCC) campus (see Figure 13).

The recommended improvements are outlined under the Park Boulevard Basin discussion.

Park Boulevard Drainage Basin (see Figure 14)
The Park Boulevard drainage basin encompasses a 550 acre area east of SW Alameda Drive and Verde Street and west of the Double Trunk line drainage basin. The pipes in this basin are in fair to good condition with only a few inventoried areas needing replacement or repair in the next 15-20 years. This line does surcharge during a design storm event with areas of flooding throughout the basin. The reason for the flooding can mostly be attributed to inadequate capacity in the downstream storm lines.

The recommended improvements in this basin include combining this basin with the Double Trunk Line Basin. Improvement alternatives are discussed in Section 7.1.4. The following outlines the recommended improvement plan for these two drainage basins (refer to Figure 18 in Appendix A):

- Upsize and re-route the trunk line from SW 11th Avenue north to the outfall from a 24-inch line to a 42-inch line.

- Connect the Park Boulevard trunk line to the above mentioned 42-inch trunk line along SW 4th Avenue with a 42-inch line and also near the TVCC campus with a 24-inch line.

- Upsize the 12-inch line along SW Alameda Drive from just south of SW 6th Avenue where the irrigation water enters the system, north to SW 4th Avenue with a 15-inch line and limit the inflow at this sight to 0.33 cfs.

- Replace the 8-inch line along SW 10th Avenue between SW 2nd Street and SW 3rd Street with a 12-inch line.

- Redirect the irrigation water south of SW 18th Avenue and to SW 11th Street to flow south.

- Limit the irrigation inflow upstream of the TVCC ballparks to 3.5 cfs.

- Limit the irrigation inflow at the intersection of Alameda Drive and SW 14th Avenue to 1.0 cfs.

- Construct desiltation/sedimentation basins before the irrigation water enters the storm water pipe network at the TVCC ball parks and at SW 8th Street.

- Repair the sewer crossing on NW 2nd Avenue between NW 3rd Street and NW 4th Street.

Stewart Carter Canal and Verde Road Basins (see Figure 15)
The Stewart Carter Canal and Verde Road Drainage Basins overlap areas with the Stewart Carter Canal conveying mainly irrigation water with minimal inflow from storm water, and the Verde Road storm water system conveying the City's storm water runoff. The Stewart Carter Canal is piped through the City along Verde Road parallel to the Verde Road trunk line and both ultimately discharge into the Dork Canal. There is one area in need of repair along the Verde trunk line, and both lines are slightly surcharged during the design storm event. There is no flooding predicted by the model. See Figures 8 and 15 in Appendix A.

The only recommended improvement in this basin is to fix the collapsing 30-inch pipe at Verde and NW 4th Avenue.

NW 7th Avenue Drainage Basin (see Figure 16)
The NW 7th Avenue basin covers a small area of 36 acres near the western end of Hunter Lane and along NW 7th Avenue. There are no flooding or pipe condition problems known in this basin. There are no recommended improvements in this basin.

Waterford Subdivision Drainage Basin (see Figure 16)
This basin encompasses the new development in the Waterford Subdivision between SW 4th Avenue and NW 4th Avenue. The line through this basin is under the jurisdiction of the Malheur Drainage District and a new pipe was constructed with a new alignment through this subdivision. This basin also carries subsurface agricultural runoff from

the fields southwest of Ontario to discharge into the Dork Canal. According to the model, the pipes in this basin have adequate capacity to carry the storm water runoff and agricultural runoff without major surcharging and no flooding. There are no recommended improvements in this basin.

Dorian Street Drainage Basin (see Figure 16)
This basin consists of a combination of open ditches and storm water. For this analysis only the storm water pipelines were evaluated. This basin does convey some irrigation water in addition to the storm water runoff through the basin to discharge into an open channel that flows to the Dork Canal. There is no flooding predicted by the model and the pipelines seem to have adequate capacity for a 24-hour 1-inch storm in addition to the irrigation runoff. There are no recommended improvements in this basin.

SW 4th Avenue Drainage Basin (see Figure 16)
The SW 4th Avenue drainage basin facilities were constructed by the Oregon Department of Transportation as part of a SW 4th Avenue improvement project. This system conveys the runoff from SW 4th Avenue in addition to other small areas of runoff south of the road. The runoff from this basin discharges into the Dork Canal just north of the airport. The storm water lines are surcharged during the design storm event, but no flooding is predicted.

The only improvement recommended for this basin is to construct a desiltation basin for the agricultural runoff before entering the storm water pipeline (see Figure 21).

1.5 WATER QUALITY ISSUES

Historically storm water management has emphasized flood control. With recent federal regulations addressing the potential impacts of storm water on groundwater and surface water quality, water quality issues have been gaining importance. Federally regulated programs include the following: the Underground Injection Control (UIC) program, the National Pollutant Discharge Elimination System (NPDES) program, Total Maximum Daily Loads (TMDLs) and Water Quality Management Plans prepared for 303(d)-listed water quality limited waters.

1.5.1 UIC Program

The Department of Environmental Quality (DEQ) administers the Underground Injection Control (UIC) Program in Oregon. This program was initially enacted in 1974 to manage fluid injection underground, in order to protect groundwater aquifers from contamination. Injection activities must all be authorized through DEQ, including storm water systems.

1.5.2 NPDES Program

DEQ has selected Ontario to be evaluated in regards to storm water controls. At this time, Ontario has not been included in the NPDES program, but may be in the future as a result of the completion and approval of the Total Maximum Daily Loads (TMDLs).

1.5.3 Total Maximum Daily Loads (TMDLs) and Water Quality Management Plan (WXMP)

As part of a watershed approach to water quality problems, the State of Oregon is developing a Total Maximum Daily Load (TMDL) and Water Quality Management Plan (WQMP) for each water body that does not meet water quality standards. Ontario is located on the upstream Snake River segment of the Snake River-Hells Canyon TMDL reach. This segment is listed as water quality limited due to concerns over bacteria, dissolved oxygen, mercury, nutrients, pH, sediment and temperature.

1.5.4 Ontario Storm Water Pollutant Loadings

A water quality monitoring project, funded through an Oregon Watershed Enhancement Board (OWEB) grant, was initiated to assess pollutant loadings of urban storm water and agricultural runoff from the City of Ontario system. The OWEB project was initiated to obtain baseline water quality and flow information necessary for an assessment of existing conditions, and to identify potential storm water impacts in the Snake River watershed. The OWEB project was also intended to help clarify the water quality and quantity being generated from the City and the Drainage Districts, since future water quality improvements will involve shared costs and responsibilities with irrigation and drainage districts.

1.5.5 Storm Water Management Practices

Should a Phase II MS4 permit be required in the future, the City would be obligated to implement storm water management practices that achieve the six minimum measures for reducing storm water pollution impacts. Ordinances, policies and programs need to specifically address these six measures:

- Public Education
- Public Involvement
- Illicit Discharge Detection and Elimination
- Construction Site Runoff Control

- Post-Construction Runoff Control
- Pollution Prevention in Municipal Operations

The permit requires that the City establish measurable goals for evaluating the success of each of these measures. Target dates for implementing each measure would then be identified.

1.5.6 Maintenance Impacts

Since sediments can carry attached pollutant loads, such as nutrients and mercury, reduction of sediment loading can provide multiple water quality benefits. Reduction of solids concentrations may allow Ontario's storm water discharges to meet the TMDL for both TSS and phosphorus.

1.6 MASTER PLAN

As the City of Ontario grows and more land becomes developed, greater storm water runoff volumes will result due to the increased impervious land area. The City of Ontario is encouraged to limit post-development flows to pre-development conditions, by encouraging onsite detention and retention facilities. Retention facilities are designed to hold or retain all storm water on-site. Detention facilities also provide on-site storage, but allow for storm water to leave the site through a controlled lower flow rate.

In addition to the priority improvements discussed previously and shown in Figures 11, 12, 15, 16, and 18, Figure 21 illustrates further improvements to be completed as part of the Master Plan. These include the following:

- Construct an onsite retention basin for the drainage area immediately west of the railroad and south of I-84.

- Encourage onsite retention facilities for the industrial area south of the HFF drainage basin (SE 9th Street) and east of the railroad as additional development occurs.

- As development occurs near Kendall Pit, construct a new 18-inch storm water pipeline to carry the runoff from SE 10th Street and SE 9th Avenue south to Kendall Pit.

- Preserve lands identified in Figure 21 for future water quality treatment needs.

1.7 SUMMARY OF COSTS AND RATE IMPACTS

1.7.1 Estimate of Most Probable Cost

Table 1.1 summarizes the improvements and their associated costs. A more detailed estimate of most probable cost is presented in Appendix F.

TABLE 1.1
ESTIMATE OF MOST PROBABLE COST

Priority I Improvements (2003)		**Project Cost***
Double Trunk Line and Park Blvd Basin:	Downtown Improvement-Alternative A	$1,456,200
Double Trunk Line Basin:	TVCC Sedimentation Basin/Structure	$25,000
	Repair Sanitary Sewer Pipe Crossing on NW 2nd Ave	$4,000
	Repair area around Pennington Drive	$15,000
HFF Basin:	Correct reverse grade on SE 4th St. between SE 4th Ave and SE 5th Avenue	$14,000
Verde Street Basin:	Replace 30" Collapsing Pipe Section at Verde and NW 4th Avenue	$3,040
Kmart Basin:	Add Cleanout south of ODOT office on SE Kendall Road	$2,000
	Upsize 15-inch along East Ln and north of Kmart/Ernst Parking	$67,350
Miscellaneous Basins:	Replace 30-ft section of concrete pipe on SE 11th Avenue	$4,375
	Miscellaneous Smaller Sedimentation Basins/Structures	$20,000
	Kendall Pit 18-inch Pipeline from SE 9th Ave to SE 5th Ave	$53,475
	TOTAL PRIORITY I	**$1,664,440**

Priority II Improvements (2010)		
Double Trunk Line and Park Blvd Basin:	Downtown Improvement-Alternative A	$101,770
Double Trunk Line Basin:	Replace 15-inch on NW 4th Ave. between NW 2nd St. and NW 3rd St.	$13,180
	TOTAL PRIORITY II	**$114,950**

Priority III Improvements (2015)		
Double Trunk Line Basin:	Replace 21-inch along SW 11th Ave north of TVCC ballparks	$22,050
Park Blvd Basin:	Replace 24-inch along SW 4th Ave west of Park Blvd.	$37,580
	Replace 8-inch with 12-inch south of Sears Dr.	$6,900
	Replace 12-inch along SW 5th Ave west of SW 12th St.	$8,770
	TOTAL PRIORITY III	**$75,300**
	TOTAL PROJECT COST	**$1,854,690**

*Project Cost includes engineering and contingencies

1.7.2 Total Estimated Annual Costs

Table 1.2 below summarizes the projected annual costs for operation, maintenance, repair, replacement and Priority I bonding. Details of these annual costs are provided in Appendix F. Future storm water fees are projected on the basis of generating annual revenue of $652,262.00 to equal the projected annual costs.

TABLE 1.2
ANNUAL COST SUMMARY

Item	Annual Cost	Running Totals
Sewer line/catch basin cleaning	$76,117	$76,117
Pipeline TV Inspection	21,204	$97,321
Labor Costs	37,000	$134,321
Equipment and Testing	50,000	$184,321
Bond Repayment (4.5% for 20 yrs)	$127,955	$312,276
Misc. Capital Improvements	40,000	352,276
Projected Replacement Costs	$273,286	$652,562
TOTALS	**$652,562**	**$652,562**

1.7.3 Existing Storm Water User Fees

In 1980 the City of Ontario passed a resolution instituting storm drainage user fees, which have been only raised once in 1984 by approximately 5 percent. The initial residential and industrial user rates were developed based on the cost per square foot amount of impervious area. It was assumed that a residential user would have 2,500 square feet of impervious area per lot. This 2,500 square feet of impervious area was assumed to equal one equivalent storm water user. The actual impervious area on industrial properties was divided by this 2,500 square feet area to determine the equivalent storm water user rate for each industrial user. A flat rate of $6.41 per commercial business was established, which was not based on impervious area. There is a significant disparity between the dollar amounts paid by commercial users and the amount of impervious area on their individual lots. Thus there is a significant disparity between the rates paid and the volume of storm water generated on commercial lots. Due to these disparities, a new rate structure is proposed in the next section for the City's consideration.

1.7.4 Potential User Rate Structure and Impact

A new user rate is hereby proposed that will be based completely on impervious area of each user. A summary of the various user categories is shown below in Table 1.3. This table indicates that there are 4,393 residential storm water users and that each user is assumed to have 2,500 square feet of impervious area per lot. For commercial and industrial users, it has been assumed that 85 percent of the commercial area will be impervious and 90 percent of the industrial area will be impervious.

The new impervious area assessment structure could be applied under two separate conditions:

> Condition 1 - All Existing Zoned Area: This condition assumes that storm water assessments will be made to all zoned land within the City whether it is developed or not.
>
> Condition 2 - Existing Developed Area: This condition assumes that storm water assessment will be made only to land within the City that is presently developed.

Table 1.3 indicates the total number of residential lots and commercial and industrial acreage that would be assessed under both conditions.

As can be seen, the major difference would be in the acreage of industrial land being assessed. An industrial acreage reduction of 90 percent from 541 acres for existing zoned area to 54 acres for existing developed area would be seen.

TABLE 1.3
SUMMARY OF STORM WATER USERS AND AREAS

Category	Number of Users	Impervious Area Assumption	Existing Zoned Area	Assumed Percent Developed	Existing Developed Area
Industrial	4393	2,500 sq ft/lot	4,393 lots	100%	4393 lots
Commercial	536	85% of Area	569.9 Acre	70%	399 Acres
Residential	18±	90% of Area	541.4 Acre	10%	54 Acres

General Assumptions:
1) All storm water users will be charged on the basis of impervious area on their property
2) Actual monthly user charges were calculated and are presented in this report on the basis of:
   - Residential – Per Equivalent Dwelling Unit (2,500 sq ft)
   - Commercial – Per 10,00 square feet of commercial property

− Industrial – Per 10,000 square feet of industrial property

Implementing this new user rate philosophy, and assuming that the new user rates would need to generate the projected annual cost of $625,562, the projected storm water user fees are shown below in Table 1.4. Table 1.4 makes a comparison between the 1984 existing storm water rates and the rates that would be charged under both Conditions 1 and 2 as described above. As can be seen from Table 1.4, the residential rates would double if all zoned land were assessed and would almost quadruple if only developed land were assessed. Table 1.4 also illustrates that commercial user charges would increase astronomically, while industrial user charges would basically double for zoned land and quadruple for only developed land. If the total zoned land (condition 1) user rates were assessed to only the developed property, (condition 2) the annual storm water revenues that would be generated would be $326,490 as shown by Note 5 in Table 1.4.

This annual revenue of $326,490 would cover the cost for annual operation, maintenance, and bond costs for Priority I improvements (a cost of $312,276 – see Table 1.2).

TABLE 1.4
PROPOSED MONTHLY USER RATES VERSUS EXISTING RATES

Category	1984	2003 Zoned Land[1]	2003 Developed Land[2]
Residential	$1.16 Per ERU	$2.44 Per ERU	$4.68 Per ERU
Commercial	$6.41 Per Business	$8.31 Per 10,000 sq ft	$15.90 Per 10,000 sq ft
½ acre lot	$6.41	$18.10	$34.63
1 acre lot	$6.41	$36.20	$69.26
5 acre lot	$6.41	$180.99	$346.30
Industrial	$4.18[3] Per 10,000 sq ft	$8.80 Per 10,000 sq ft	$16.83 Per 10,000 sq ft
½ acre lot	$9.10	$19.17	$36.66
1 acre lot	$18.21	$38.33	$73.31
5 acre lot	$91.04	$191.66	$366.56
Annual Revenue	**$113,610**	**$625,562[4]** **$326,490[5]**	**$625,562**

Notes:

1) Assumes that 100% of the land zoned as commercial and industrial within the City of Ontario will be assessed a storm water fee
2) Assumes that only the developed lands within the City of Ontario will be assessed a storm water fee. Assumes that only 10% of the industrial zoned land and 70% of the commercial zoned land is currently developed and, therefore, will be assessed
3) Assumes for the existing rates that existing industrial lots are 90% impervious

4) Assumes that 100% of the zoned land is assessed a storm water fee
5) Assumes that only 70% of the commercial zoned land and 10% of the industrial zoned land is developed and assessed a storm water fee at the same rate as if all, developed and undeveloped, zoned land was assessed

In light of the fact that the storm water user rates could significantly increase, it may be appropriate to complete a separate, more detailed storm water fee user rate analysis. A new rate structure could be developed that would provide a gradual increase in storm water fees to accomplish only the most urgent and needed storm water improvements with perhaps other City general fund revenues.

2.0 – STUDY AREA

2.1 SERVICE AREA

The City of Ontario, Oregon is the largest city in Malheur County with a population of 11,140 people. It is located just west of the border of Idaho along the Snake River in an area dominated by agricultural activity. The study area for this storm water study and master plan includes the area within the Urban Growth Boundary and additional areas that drain into the City storm water system. Approximately 4,010 acres that drain into the storm water conveyance system and an additional 3,520 acres on the perimeter of Ontario were considered in the master planning portion of this study. The area studied does not include the municipal airport, which recently constructed an independent on-site storm water detention facility, and the adjacent municipal golf course. Figure 1 illustrates the study area boundaries for both the existing and future systems.

2.1.1 Geography and Soils

The City of Ontario is located at the confluence of the Malheur and Snake River, with the Snake River running along the eastern and northeastern edge of the City and the Malheur River running north of the City parallel to the Dork Canal, a major drainage canal. The City is not bound by any natural geographical features to the west and south, which is primarily agricultural land. The City is located on an old lake terrace and alluvial valley fill.

The geologic history of the Ontario region shows the effects of both volcanic and glacial forces in forming the present valley. The soils that have resulted from volcanic sediments, lava flows, and continuous deposits influence the runoff rate for precipitation; therefore, significantly impacting the planning, design and construction of a storm water system for the area.

The United States Department of Agriculture Natural Resource Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), maintains soil maps that were used to identify soil types for this study. Table 3-1 summarizes the characteristics of each soil type and the approximate extent of each soil is shown in Figure 4. SCS classifications indicate the effect soils will have on the infiltration and drainage of storm water.

TABLE 2.1
ONTARIO SOIL TYPES

Soil #	Soil Type	Runoff	Erosion Potential	Transmission Rate (in/hr)	Hydrologic Soil Group	Hydrologic Soil Group Description
20	Notus-Falk Variant Complex	Slow	Severe	0.05-0.15	C	Sandy Clay Loam
31	Stanfield Silt Loam	Slow	Slight	0.05-0.15	C	Silt Loam
34	Umapine Silt Loam	Slow	Slight	0.0-0.15	C/D	Silt Loam
12A	Garbutt, 0-2% slopes	Slow	Slight	0.15-0.30	B	Silt Loam
13A	Greenleaf, 0-2% slopes	Slow	Slight	0.15-0.30	B	Silt Loam
25A	Owyhee, 0-2% slopes	Slow	Slight	0.15-0.30	B	Silt Loam
25B	Owyhee, 2-5% slopes	Slow	Slight	0.15-0.30	B	Silt Loam
25C	Owyhee, 5-8% slopes	Medium	Moderate	0.15-0.30	B	Silt Loam
25D	Owyhee, 8-12% slopes	Medium	Moderate	0.15-0.30	B	Silt Loam
25E	Owyhee, 12-20% slopes	Rapid	Severe	0.15-0.30	B	Silt Loam
30A	Sagehill, 0-2% slopes	Slow	Slight	0.15-0.30	B	Fine Sandy Loam
33A	Turbyfill, 0-2% slopes	Slow	Moderate	0.15-0.30	B	Fine Sandy Loam
33B	Turbyfill, 2-5% slopes	Slow	Moderate	0.15-0.30	B	Fine Sandy Loam

As Figure 4 and Table 3-1 show, the majority of the soil in the study area consists of silt loam or sandy clay loam. The hydrologic soil group designation is dependent upon the infiltration rate of the soil. The soil designations that are prevalent in Ontario are B, C and D, and are defined in the Urban Hydrology for Small Watersheds Technical Release 55 (TR-55) revised in June 1986, included in Appendix C and summarized as follows:

Group B: These soils have moderate infiltration rates when thoroughly wetted and are primarily moderately deep to deep. These soils are moderately well to well drained soils, have moderately fine to moderately coarse textures, and a moderate rate of water transmission

Group C: These soils have a low infiltration rate when thoroughly wetted, and are primarily soils with a layer that impedes downward movement of water. These soils have moderately fine and fine texture, and a low rate of water transmission

Group D: These soils have a high runoff potential, due to a very low infiltration rate when thoroughly wetted. These soils are primarily clay soils with high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material, and have a very low rate of water transmission.

The Hydrologic Soil Group influences the selection of the SCS "Curve Number (CN)" used in hydrologic models for predicting storm water runoff. The selection of the SCS Curve Number is addressed later.

2.1.2 Topography

The elevation in the area ranges from about 2,130 feet along the Snake River to about 2,200 near the Ontario Airport to the west of the City. The City is relatively flat with a gentle slope towards the north and northeast. There is a relatively significant slope, dropping north, just south of the Dork Canal forming a distinct ridge along the north edge of Ontario. This feature and other elevation changes are illustrated in Figure 2.

2.1.3 Climate

Oregon State University's College of Oceanic and Atmospheric Sciences regularly monitors and records climatic conditions around Oregon from stations located in nine different climatic zones. Ontario is located in Climatic Zone 9 with stations at Ontario KSRV and the Malheur Experimental Station.

The City of Ontario experiences weather similar to southwest Idaho, but significantly different than the western half of Oregon. The average temperature in the winter is about 25°F with the summer average being 75°F. The extremes range from -25°F to 110°F, with annual average temperature of 50.5°F.

Snowfall generally occurs between November and March with an average yearly snowfall of 18.7 inches. The total precipitation average is 9.45 inches per year. The driest months of the year are July and August with an average total monthly precipitation of about 0.30 inches.

2.1.4 Historical Flooding

The City of Ontario is situated such that the Snake River forms a large bend along the north and east side of Ontario. The Malheur River runs along the north side of Ontario before discharging into the Snake River. According to the 1983 Storm Water Management Plan developed by Century West Engineering Corporation, the Malheur and Snake Rivers have been responsible for numerous flooding problems in Ontario. Two large flooding events occurred in 1952 and 1957 as a result of rain and snowmelt. A similar event occurred on February 15, 1982 requiring homes to be evacuated due to the Malheur River flooding. This same event caused the level of the Snake River to rise 18 inches in two days.

The Boley Creek dam (1963), Warm Springs dam and the Agency dam alleviate the flooding along the Snake and Malheur Rivers. Flood hazard maps showing the 100-year and the 500-year flood were prepared by the Federal Emergency Management Agency (FEMA). The FEMA 100-year and 500-year flood boundaries were superimposed on the City's base mapping to create Figure 3. In July 2001, Holladay Engineering Co. compared the FEMA maps to previous work completed by the Army Corp. of Engineers (1974) for a development near the Dork Canal. They suggested that the flood plain boundaries near the Dork Canal may extend further to the south than the FEMA maps suggest for both the 100-year and the 500-year events.

According to interviews with City staff, historical flooding has also occurred as a result of ice jams that cause water to backup. The ice jams occur as large chunks of ice become logged at locations such as the bridge crossing between Ontario and Fruitland.

The majority of the storm water runoff from the City of Ontario ultimately discharges to the Snake River. High water levels in the Snake River have resulted in flooding as river water has backed up into storm water lines. More recently, the City of Ontario installed 24" and 42" Tidal-flex valves that allow flow to leave the storm drain system while prohibiting flow from entering the storm system. Figure 5 shows the location of the valves. A Tidal-flex valve is designed to allow flow to leave the storm drain system with as little as one inch of head differential. The valves are also designed to prevent flow from entering the storm system from the river for back flow pressures up to 5 psi (11.5 feet).

During periods of high river levels, it may be difficult or impossible for the City's storm system to provide enough head or pressure to allow the storm water to leave the City, thus causing flooding within the City of Ontario. This study does not address the frequency or probability of a storm event in Ontario occurring simultaneously with high river levels. For the purposes of this study, the storm water outfalls were assumed to be free, or unrestricted, outfalls.

2.2 ECONOMY

Originally the City of Ontario's economy was primarily gold mining, but is currently driven mainly through agriculture, livestock and food processing. The primary crops are onions, russet potatoes, sugar beets and peppermint. Within the City of Ontario itself there are a total of 17 manufacturing companies with the top five employers being Heinz Frozen Foods (9.9% of Ontario's population), Snake River Correctional Institution (9.0%), Holy Rosary Medical Center (4.3%), School District 8-C (3.4%) and Treasure Valley Community College (2.3%).

2.3 POPULATION

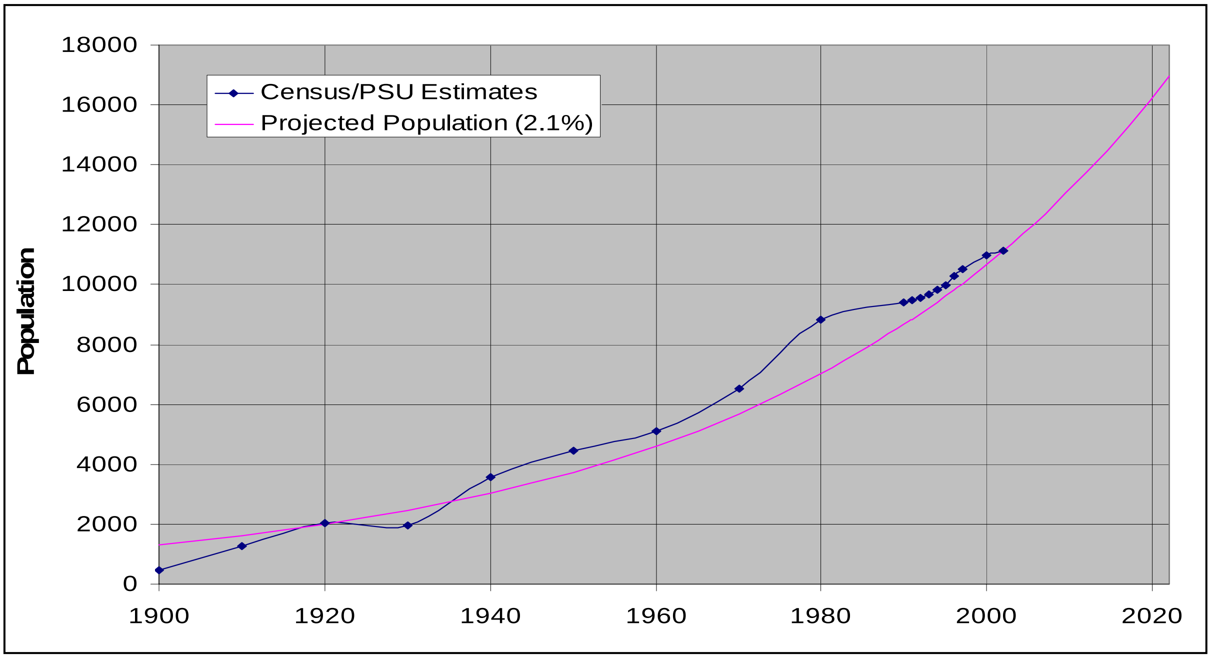
The City of Ontario currently has a population of 11,140 according to the Portland State University (PSU) Center for Population Research July 2002 estimates based upon past Census data and housing and group quarters information. Census data from 1900 to 2000, and annual population estimates from PSU from 1990 to 2002 form the basis for the twenty year growth projection used by this study.

Table 2-2 shows the population since 1900. This data indicates that Ontario has a historical 2.1% growth rate. Chart 2-1 compares the historical statistics and estimates to a 2.1% exponential growth curve.

TABLE 2.2
POPULATION PROJECTIONS

Year	Census	PSU
1900	445	
1910	1,248	
1920	2,039	
1930	1,941	
1940	3,551	
1950	4,465	
1960	5,101	
1970	6,523	
1980	8,814	
1990	9,392	
2000	10,985	
2001		11,140
2002		11,140
2007		12,373
2012		13,743
2017		15,265
2022		17,000

CHART 2.1
ONTARIO, OREGON PROJECTED POPULATION



This report uses a design population of 17,000 for the year 2022, which is consistent with previously adopted water and sewer master plans.

3.0 - DESIGN CRITERIA

3.1 TECHNICAL REVIEW COMMITTEE AND DESIGN CRITERIA

The storm water collection system was evaluated based upon design criteria determined by the Technical Review Committee (TRC). This committee consisted of the Public Works Director, Steve Gaschler; Engineering staff, Tom Davis and Dan Shepard; Wastewater Treatment Supervisor, Glen Schoeneman; Storm/Wastewater Collection Supervisor, Don Brown; and staff from Keller Associates. This committee met monthly throughout the development of the Master Plan to discuss and evaluate system analysis results and make decisions regarding the recommended improvements.

Initially the Heinz Frozen Foods (HFF) drainage basin was evaluated to determine the method to model the sub-basins of each major drainage basin. The results of this evaluation are presented in Technical Memo #3 included in Appendix B. As part of this evaluation the HFF basin was delineated into 55 sub-basins with every storm water pipe inputted in the model. In comparison with this detailed model, a simplified version with 15 basins and only the storm water lines with a diameter greater than 12-inches were inputted in the model. When compared, the results indicated that the peak flows from the detailed model and the skeletonized model did not differ significantly. It was concluded that the major basins did not need to be delineated into many smaller basins in order to produce reliable results.

As part of the analysis of the HFF Drainage Basin improvements were developed based on two design criteria: improvements to reduce all surcharging and improvements to only reduce flooding. These two alternative improvements were presented to the TRC, along with preliminary results from the other drainage basins throughout Ontario. Based upon the amount of surcharging throughout the existing storm water system during the 25-year design storm event (refer to Figure 10), the TRC decided that the recommended improvements should eliminate all flooding and minimize surcharging.

3.2 ASSUMPTIONS

In the analysis of the storm water system, several assumptions were made to develop the model. These assumptions include the following:

- During extreme events, the river level may be high enough that the storm water would not leave the City of Ontario. For this analysis “free” outfalls to the Snake River or Dork Canal were assumed.

- The pipes and catch basins in the network are clean of sediment build-up, collapse or other problems that could impede flow. The catch basins were also assumed to have adequate capacity to accept all the storm water runoff and discharge directly into the storm water collection system.

- Only the major storm water pipelines (primarily 10-inches and larger) were evaluated. No in-depth analysis was completed for the collector lines.

- Currently, the western half of the storm water system discharges into the Dork Canal, which then carries the runoff to the Snake River. It was assumed that the City will continue to use the Dork Canal as part of the storm water system and as such, consideration was taken when determining future water quality improvements to meet state and federal requirements.

- The study area defined in Figure 1 was used as the outer limit of the future storm water system expansion. The areas not currently developed should be required to limit the post development runoff volumes to the predevelopment or current runoff levels. This requirement will allow the expansion and development of the City without increasing the volume or adversely affecting the water quality of the storm water discharge into the Snake River.

- Several sections of natural channels were modeled as part of the ditch and culvert analysis presented in Section 5. The cross sections of these open channels were based upon City provided survey information and assumed to be constant for the lengths between the cross sections.

- Where irrigation runoff entered the City's storm water system, the flow was approximated. Average peak month flows at these sites were assumed to coincide with the design storm event. Where actual flow data was not available, base irrigation flows were estimated based on the testimony of Ontario staff.

4.0 - HYDROLOGIC MODEL DEVELOPMENT

4.1 INTRODUCTION

The modeling program XP-SWMM2000 v8.5 was used to estimate storm water runoff from the study area. The XP-SWMM2000 program simulates the surface water runoff for storm water basins and dynamically routes the runoff through a storm water collection system to the outfall. The model requires several input parameters which determine the runoff: basin characteristics, design storm size and time, and pipeline and open channel characteristics.

4.2 MODEL PARAMETERS

The City of Ontario was delineated into 12 major Drainage Basins, which were further divided into sub-basins. The basin delineation is shown in Figures 6 and 7 and the basin parameters are summarized in Table 4-1. Appendix C contains a summary of the parameters and location description of each of the sub-basins.

TABLE 4.1
RUNOFF BASIN PARAMETERS

Drainage Basin	Sub-basin	Area (acre)	% Imperv.	Length (ft)	Width (ft)	Slope	CN	Tc (min)
Double Trunk Line Storm water Line	A1	7.0	24.1%	490	624	0.004	76	22.0
	A2	11.7	24.1%	600	849	0.004	76	24.8
	A3	12.5	24.1%	700	779	0.004	76	20.7
	A4	14.9	27.6%	550	1183	0.002	77	20.8
	A5	15.9	27.6%	600	1152	0.003	77	17.6
	A6	23.0	27.6%	700	1429	0.004	77	19.5
	A7	27.9	27.6%	1100	1104	0.002	77	34.8
	A8	15.6	27.6%	525	1291	0.001	77	25.0
	A9	46.6	27.6%	1375	1477	0.004	77	33.2
	A10	23.0	27.6%	550	1825	0.003	77	24.5
	A11	8.7	24.1%	400	943	0.005	76	7.0
	A12	57.0	24.1%	1400	1773	0.002	76	38.6
	A13	67.6	24.1%	1200	2455	0.003	76	29.3
	A14	25.3	20.7%	600	1837	0.004	75	22.1
	A15	68.9	20.7%	800	3752	0.003	75	32.8

	Sub-basin	Area (acre)	% Imperv.	Length (ft)	Width (ft)	Slope	CN	Tc (min)
Double Trunk line Irrigation Line	B0	60.1	Minimal	4500	582	0.001	65	137.6
	B1	92.1	9.4%	1800	2228	0.015	72	45.0
	B2	36.5	13.1%	1075	1480	0.003	73	31.4
	B3	80.0	5.5%	2800	1245	0.016	71	43.0
	B4	68.3	0.9%	2000	1488	0.012	69	46.0
	B5	3.3	20.5%	250	577	0.003	75	13.8
	B6	15.5	39.0%	480	1407	0.005	80	18.1
	B7	3.2	10.5%	645	216	0.001	75	14
Park Boulevard	C0	74.4	3.3%	3915	828	0.004	70	71.9
	C1	21.8	16.9%	800	1187	0.004	74	21.7
	C2	29.7	13.1%	900	1439	0.012	73	21.8
	C3	34.1	20.0%	1150	1292	0.007	75	29.9
	C4	17.3	21.6%	750	1006	0.016	75	20.9
	C5	33.0	28.8%	1400	1027	0.006	77	27.1
	C6	24.7	37.2%	800	1187	0.004	74	21.7
	C7	22.8	65.9%	1300	763	0.014	88	26.0
	C8	70.4	32.7%	1400	2190	0.012	78	30.0
	C9	80.2	21.0%	1200	2076	0.006	75	30.5
	C10	57.0	21.9%	1000	2481	0.005	75	36.4
	C11	85.5	4.9%	1400	2660	0.024	70	26.7
	C12	34.6	6.0%	1100	1368	0.004	71	67.1
Stewart Carter Canal	D0	102.7	0.0%	4000	1118	0.012	69	34.4
	D1	34.7	17.7%	700	2162	0.002	74	47.0
	D2	14.0	76.5%	550	1112	0.007	91	17.9
	D3	26.0	19.7%	800	1416	0.011	75	27.1
	D7	19.2	23.1%	550	1,524	0.010	76	18.2
Verde Street	D4	43.1	18.6%	650	2886	0.006	74	22.9
	D5	11.2	38.4%	600	814	0.006	80	19.9
	D6	36.9	21.2%	1100	1463	0.001	75	36.0
	D8	30.7	14.5%	800	1672	0.014	73	30.9
	D9	51.9	13.6%	1200	1885	0.005	73	26.1
	D10	157.0	2.9%	3000	2279	0.012	70	177.0
Water-ford	E0	1252.2	Minimal	8740	6241	0.001	65	276.1
	E1	140.2	5.4%	3500	1745	0.001	71	33.6
Dorian	E2	71.1	12.3%	2000	1549	0.009	73	19.0
NW 7th Ave	E3	35.7	9.2%	1400	1110	0.002	72	9.0
SW 4th Ave	SW1	22.3	90.0%	5910	190	0.004	95	41.8
	SW2	29.4	13.6%	1010	1269	0.003	73	37.7
	SW3	43.8	16.4%	1120	1704	0.010	74	46.0

	Sub-basin	Area (acre)	% Imperv.	Length (ft)	Width (ft)	Slope	CN	Tc (min)
Wal-Mart	W1	18.5	27.5%	1660	486	0.014	85	22.6
	W2	18.0	73.8%	1340	586	0.002	93	27.5
	W3	12.1	95.2%	1550	341	0.010	97	21.3
	WD	0.9	100.0%	350	114	0.010	98	10.4
	WO	3.1	100.0%	150	891	0.012	98	7.9
Kmart	K1	5.2	60.7%	600	379	0.016	91	6.9
	K2	14.6	90.9%	1000	635	0.005	96	19.3
	K3	7.8	59.0%	550	617	0.009	91	13.8
Heinz Frozen Foods	A	13.1	26.8%	800	715	0	74	23
	B	19.3	23.7%	750	1123	0	73	22
	C	24.6	24.0%	1000	1073	0	73	27
	D	12.1	19.3%	840	627	0	71	31
	E	33.0	29.6%	800	1797	0	72	26
	F	27.1	55.3%	1100	1075	0	83	46
	G	33.8	29.3%	1020	1445	0	75	38
	H	32.6	53.7%	900	1576	0	83	33
	I	20.1	65.0%	450	1950	0	86	15
	J	46.5	0.0%	2600	779	0	86	240
	K	11.5	70.0%	500	1002	0	91	17
	L	4.3	65.0%	100	1893	0	88	5
	M	1.0	70.0%	500	86	0	94	22
	N	1.4	64.5%	100	614	0	86	8
	O	13.8	90.0%	500	1198	0	94	22
	P	22.7	80.0%	950	1042	0	92	22
	Q	12.9	80.0%	600	936	0	92	13
Sanitary Sewer Connection/Onsite Retention	SS1	1.9	19.4%				75	
	SS2	2.3	29.4%				78	
	SS3	0.8	29.1%				77	
	SS4	13.6	20.5%	300	1976	0.006	75	23.0
	SS5	0.8	20.1%				75	
	SS6	0.4	100.0%				98	
	SS7	0.6	32.5%				78	
	SS8	0.5	25.1%				76	
	O1	35.2	17.8%	1300	1178	0.001	74	48.4
	O2	1.4	26.5%				77	
	O3	15.1	0.0%				69	
	O4	35.8	4.9%				70	

	Sub-basin	Area (acre)	% Imperv.	Length (ft)	Width (ft)	Slope	CN	Tc (min)
Future Runoff Basins	F0	77.5	Minimal	1000	3376	0.001	65	23.2
	F1	562.4	Minimal	2800	8750	0.002	65	57.9
	F2	105.4	Minimal	800	5736	0.018	65	3.9
	F3	380.6	Minimal	1800	9212	0.001	65	49.3
	F4	77.8	Minimal	800	4236	0.016	65	4.3
	F5	46.0	Minimal	800	2507	0.041	65	2.7
	F6	195.4	Minimal	3000	2837	0.001	65	88.5
	F7	231.0	Minimal	3000	3354	0.001	65	88.5
	F8	242.6	Minimal	2130	4962	0.015	65	15.4
	F9	327.6	Minimal	4100	3481	0.002	65	88.0
	F10	112.4	Minimal	2600	1883	0.001	65	75.5
	F11	681.4	Minimal	5500	5396	0.001	65	170.2
	Airport	479.0	Minimal				65	

4.2.1 Basin Parameters: Area

In developing the hydrologic model, the study area boundary was defined and then subdivided into runoff basins for each major trunk line and outfall. Each basin drains into an independent trunk line with one outfall, except for the major storm water pipeline network immediately west of the railroad tracks and the irrigation trunk line that parallels the storm water trunk line, which are interconnected in two places.

Each major basin was further divided into sub-basins using contour maps and by conducting field observations of roadway slopes and the flow of storm water and irrigation runoff to drainage ditches. City personnel and previous storm water studies were used to confirm these sub-basin boundaries and direction of runoff. The resultant basins and sub-basins are shown in Figure 7, with the areas of each shown in tables in Appendix C.

The area of each sub-basin directly correlates with the volume of runoff that enters the storm water system. The sub-basin boundaries indicate the area that produces runoff to a point along the trunk line either overland or through the collector pipe network.

4.2.2 Basin Parameters: Base Irrigation Flows

Where irrigation runoff entered the City's storm water system the flow was approximated. The flow data from the previously completed *Oregon Watershed Enhancement Board Water Quality Monitoring Project Report for the City of Ontario* was used to estimate the inflow for many irrigation sources into the City's system. Average peak month flows were assumed to coincide with the design storm event. Where actual flow data was not available, base irrigation flows were estimated based on the testimony of Ontario staff.

4.2.3 Basin Parameters: Length and Width

In order to accurately model the runoff from each of the sub-basins, the geometric properties of the sub-basins were calculated. If the sub-basin were a perfect rectangle, the width of the flow would equal the width of the sub-basin. However, because most sub-basins are not rectangular or symmetrical, the width is assumed to be equal to the area divided by the average length of overland flow.

In modeling the City's system, the overland flow length was assumed as the average length the runoff has to travel in order to reach the point where the sub-basin drains into the trunk line. The average length was used to calculate the width and the maximum length was used to calculate the time of concentration. The width is one of the key parameters, affecting the runoff pattern and runoff volume.

4.2.4 Basin Parameters: Slope

The slope is the average slope along the flow path. The slope is computed by dividing the difference of the beginning elevation and the inlet elevation, by the length of the overland flow. This parameter is dimensionless.

4.2.5 Basin Parameters: Pervious and Impervious Area

It is necessary to determine the percentage of the sub-basin area that is impervious or pervious. Pervious surfaces are those which are covered primarily with vegetation and permit the infiltration of water. Impervious areas are those which inhibit infiltration of water, such as pavement, roadways, sidewalks, and roofs.

In calculating the percent impervious of a sub-basin, only the impervious areas connected directly to the storm water system, (i.e. curb and gutters, streets, driveways, etc) were calculated. The impervious areas that drain to adjacent pervious areas (i.e. roofs), were not included in the percent impervious. In general, the impervious areas in Ontario consist mainly of roadways, sidewalks, parking lots and the roofs of larger buildings that drain on to paved parking lots.

The percent impervious is a key parameter used to determine a composite Soil Conservation Service Curve Number (SCS CN) and the time of concentration (Tc). Generally, as the percent impervious increases the infiltration decreases, resulting in more rapid runoff, shorter Tc and greater CN, resulting in higher peak runoff rates.

The City of Ontario has a variety of residential, commercial and industrial areas. The permeability of each of these areas varies according to their use. The residential areas typically have greater areas of pervious land; whereas, commercial and industrial areas usually have larger areas of pavement. Field observations, City provided information and maps, and aerials were used to determine the percent impervious for each sub-basin. These values are listed for each sub-basin in Table 4-1.

4.2.6 Basin Parameters: Soil Conservation Service Curve Numbers

The Soil Conservation Service's "The Urban Hydrology for Small Watersheds Technical Release 55" (TR-55) outlines the process for computing the SCS Curve Number (CN) for sub-basins. The SCS CN is used as an index of the potential runoff from a storm event from each basin. As stated previously, the higher the CN, the higher the runoff.

The curve number is based on the hydrologic soil group, ground cover, percent impervious and land use. The following table from TR-55 shows average CN for a variety of land uses, hydrologic soil groups and ground cover.

TABLE 4.2
RUNOFF CURVE NUMBERS FOR URBAN AREAS

Cover Description			CN for Hydrologic Soil Group-			
Cover type and hydrologic condition		Average % Impervious[2]	A	B	C	D
Fully developed urban areas (vegetation established)						
Open space (lawns, parks, golf courses, cemetaries, etc.)[3]:						
	Poor Condition (grass cover <50%)		68	79	86	89
	Fair Condition (grass cover 50% to 75%)		49	69	79	84
	Good Condition (grass cover >75%)		39	61	74	80
Impervious Areas:						
	Paved Parking Lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
	Streets and Roads:					
	Paved; curbs and storm sewers (including right-of-way)		98	98	98	98
	Paved; open ditches (including right-of-way)		83	89	92	93
	Gravel (including right-of-way)		76	85	89	91
	Dirt (including right-of-way)		72	82	87	89
Western Desert Urban Areas:						
	Natural desert landscaping (pervious areas only)[4]		63	77	85	88
	Artificial desert landscaping (impervious weed barrier, desert shrub with 1-to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban Districts:						
	Commercial and Business	85	89	92	94	95
	Industrial	72	81	88	91	93
Residential Districts by Average Lot Size:						
	1/8 acre or less (town houses)	65	77	85	90	92
	1/4 acre	38	61	75	83	87
	1/3 acre	30	57	72	81	86
	1/2 acre	25	54	70	80	85
	1 acre	20	51	68	79	84
	2 acres	12	46	65	77	82
Developing Urban Areas						
Newly graded areas (pervious area only, no vegetation)[5]			77	86	91	94

Notes:

(1) Average runoff condition, and Initial Abstraction (Ia) = 0.2S

(2) The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4 (in TR-55)

(3) CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type

(4) Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN=98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition

(5) Composite CN's to use for the design of temporary measures during grading and construction should be computed using figures 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

In determining the CN values for the sub-basins in Ontario, a weighted average, or composite curve number, was computed using the percent impervious and cover type of the pervious area. For hydrologic Soil Group E, the CN for the impervious and pervious areas were 98 and 69, respectively. These weighted averages were compared to the values presented in Table 4-2 and were found to be comparable.

4.2.7 Basin Parameters: Time of Concentration

According to SCS TR-55 the time of concentration (Tc) is "the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed." The Tc is the sum of the travel times of each of the flow components of the sub-basin: overland flow, sheet flow, shallow concentrated flow, open channel flow and flow in pipes. The time of concentration influences the shape of the storm hydrograph and the peak discharge. Typically, as an area urbanizes, the Tc decreases, increasing the peak runoff.

Several parameters affect the time of concentration: surface roughness, channel shape and flow patterns and slope. The surface roughness is a factor of the vegetative cover and/or surface covering. A roughness coefficient, Manning's n, is used in the equations for sheet flow for determining Tc. The following Table 4-3, from Table 3.1 on page 3-3 of TR-55 summarizes the different roughness coefficients for sheet flow.

TABLE 4.3
ROUGHNESS COEFFICIENTS (MANNING'S N) FOR SHEET FLOW

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

Notes: **(1)** The n values area a composite of information compiled by Engman (1986)

(2) Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures
(3) When selecting n, consider cover to a height of about 0.1 feet. This is the only part of the plant cover that will obstruct sheet flow

The channel shape influences the Tc due to more efficient hydraulic characteristics that typically reduce the Tc. The more urbanized an area becomes, the shorter the overland flow lengths become due to the presence of curb and gutter, allowing flow to enter a channel sooner, where the flow velocity is faster. The slope of the flow path varies greatly, with no real correlation with urbanization. A larger slope shortens the Tc, resulting in a larger peak discharge.

In determining the Tc for the sub-basins in Ontario, the equations presented below were used. A minimum Tc of 10 minutes was also assumed.

- Sheet flow (flow path < 300 feet): $\mathbf{Ts=0.007*(nL)^{0.8}/(P_2)^{0.5}s^{0.4}}$
  Where: Ts=travel time for sheet flow (hr)
  n=Manning's roughness coefficient (Table 4-3)
  L=flow length (ft)
  P_2=2-year, 24-hour rainfall (in)
  s=slope of a hydraulic grade line (ft/ft)
  Assumed: n=0.011
  P_2=0.85 inches

- Shallow Concentrated Flow (flow path >300 feet):
  - Slopes > 0.005: **Tsc=L/V**
    Where: Tsc=travel time for shallow concentrated flow with slopes > 0.005 (sec)
    L=flow length (ft)
    V=flow velocity (ft/sec) determined from Figure 3-1 in TR-55, included in Appendix C
  - Slopes < 0.005: $\mathbf{Tss=L/20.3282s^{0.5}}$
    Where: Tss=travel time for shallow concentrated flow with slopes < 0.005 (seconds)
    L=flow length (ft)
    s=slope of a hydraulic grade line (ft/ft)
- Pipe Flow: **Tp=L/2.0**
  Where: Tp=travel time for pipe flow (seconds)
  L=flow length (ft)
  Assumed: Pipe flow velocity = 2.0 ft/sec

- Total Time of Concentration**: Tc=Ts+Tsc+Tss+Tp**

The Tc values for each of the sub-basins in Ontario are reported in Table 4-1.

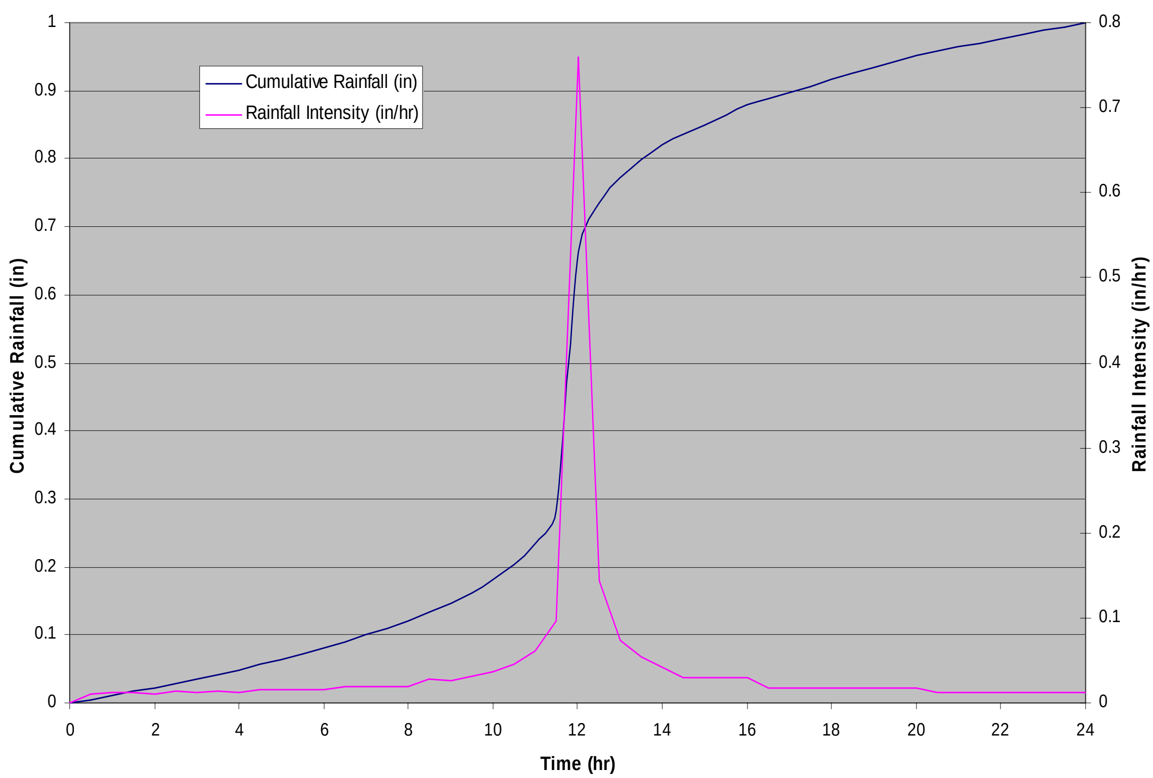
4.2.8 Basin Parameters: Initial Abstraction Fraction

The initial abstraction represents the amount of precipitation that is lost or detained prior to the runoff. It is reported as either a total depth of precipitation in inches or as a fraction of the amount of total precipitation (between 0 and 1). In calibrating the Ontario model the initial abstraction fraction was found to be between 0.045 and 0.2, with the smaller number representing the more urbanized areas of Ontario and the larger numbers being used for the basins that include agricultural and undeveloped lands.

4.2.9 Design Storm

In accordance to the recommendations outlined in Technical Memo #2 in Appendix B, the design storm used was a 25-year 24-hour storm with a cumulative rainfall of 1-inch. The following hydrograph is an example of a SCS Type II storm hydrograph similar to the one used for this analysis. A SCS Type II storm is recommended for this region of the United States.

CHART 4.1
SCS TYPE II STORM EVENT



It is important to select a storm duration that will accurately estimate the runoff. A duration that is too short will underestimate the runoff, whereas a storm event that is too long will over-exaggerate the runoff volume. For purposes of the Ontario storm water model the general process was to allow between 6 and 12 hours for the constant irrigation/drainage inflow to fill the pipes to steady state conditions. Once the steady state conditions were achieved, a 24-hour storm was superimposed, with an additional 24-hour period for the basins to drain. This allowed for the pipes to accurately model a storm occurring during peak conditions when the pipes are used for both irrigation and storm water discharge.

4.2.10 Pipeline Characteristics: Roughness Coefficient

The XP-SWMM2000 model accounts for pipe friction while dynamically routing the flow through the pipeline network. This equation requires the input of the roughness of the pipe, which when not flowing full is assumed to be equivalent to the roughness of an open channel of the same material. For this model it was assumed that Manning's n for all channels was constant and equal to 0.014.

5.0 – EXISTING STORM DRAINAGE SYSTEM CONDITION AND EVALUATION

5.1 INTRODUCTION

The City of Ontario storm drainage system consists of surface flow to catch basins, a subsurface network of pipes, and open drainage ditches. The system collects and transports the runoff from several drainage basins. This runoff ultimately drains by gravity to discharge into the Snake River northeast of the City. The evaluation of the storm water collection system was conducted based upon the criteria developed by the Technical Review Committee (TRC), as discussed previously in Section 3.

5.2 EXISTING PIPELINE CONDITION

The City of Ontario has just completed video recording approximately 15% of their storm water lines with an emphasis on problematic areas. Figure 8 illustrates the areas determined to have problems, i.e. broken, cracked, collapsed, heavy sedimentation or otherwise needing repair. This figure prioritizes the problem areas according to the urgency of repair needed.

Keller Associates recommends that the City begin replacing the pipelines highlighted in Figure 8 in the order of priority shown. In addition, an annual pipeline replacement program should be established and the priority improvement plan should be continually updated as additional video recording is performed.

Sedimentation build-up in the pipelines reduces the capacity of the pipelines. Approximately 50 percent of the pipelines originally video recorded had to be cleaned before the video equipment could pass through the pipe segment. Substantial sediment problems have been reported downstream of the irrigation discharge points. Sedimentation basins located upstream of inlets gathering irrigation return could substantially reduce sedimentation problems prevalent in many sections of City pipelines. In addition, the City should also begin a regular cleaning program to reduce the sedimentation in the storm lines.

5.3 IRRIGATION AND DRAINAGE DISTRICTS

Several irrigation and drainage districts exist within the City of Ontario storm water study area. The irrigation districts provide irrigation water to fields used for agricultural purposes and the drainage districts carry the excess water from the fields and transport it to the river. Figure 9 illustrates the jurisdiction of each entity.

Recently several issues regarding the irrigation/drainage district jurisdiction and impact on the City's storm water system have been highlighted As outlined in the City of Ontario and Oregon Watershed Enhancement Board Water Quality

Monitoring Report (OWEB Report) completed in the Summer of 2002, the irrigation/drainage ditches and channels carry most of the water which flows through the City. The districts also play a critical role in the quality of the water discharged into the Snake River. In addition, the drainage water discharged into the City's system reduces the capacity of these lines to handle a storm event. A short summary of each of the drainage and/or irrigation districts involved with these issues is presented below.

5.3.1 Malheur Drainage District

The Malheur Drainage District was formed to handle the agricultural runoff from cultivated lands and transport that runoff to the river. Most of the irrigation return that Malheur Drainage District picks up originates from the Owyhee Ditch Company. The District has maintained an agreement with the City for over 10 years. This agreement allows the City of Ontario to discharge storm water from a drainage basin defined in the 1983 Storm Water Management Plan. To compensate the District for this service, the City pays the District $300 annually. In exchange, the District maintains the drainage channels. The canals and lines that the Malheur Drainage District has jurisdiction over include the Dork Canal, a 24-inch line from south of SW 4^{th} Avenue north through the Waterford Subdivision to the Dork Canal, and a network of collection lines south of the City, as shown in Figure 5.

5.3.2 Owyhee Ditch Company and Owyhee Irrigation District

The Owyhee Ditch Company has jurisdiction over the Stewart Carter Canal. They receive water from the Owyhee Irrigation District, which is responsible for transporting the water from the Owyhee Dam to the Stewart Carter Canal. The Ditch Company maintains open channel facilities. At the time of a site visit in the summer of 2002, the ditch was found to be very clean and free of vegetation. The City has assisted the Ditch Company in the past by cleaning and video recording portions of the buried pipe located within the City limits. The Company's jurisdiction does not include delivery from the Canal, which is the responsibility of each user.

5.3.3 Warm Springs Irrigation District

The Warm Springs Irrigation District does not have jurisdiction over any water within the city limits. The water for this district is supplied by the Warm Springs Reservoir and it is owned by a group do farmers within the District. It controls the Blanton Ditch to the northwest of Ontario and flows west, just south of the wastewater treatment plant and then north to discharge in the Malheur River.

5.3.4 Interagency Agreement

An agreement between the City of Ontario and the different irrigation and drainage districts should be negotiated for future use and maintenance of canals and pipelines used for both storm water and irrigation water. A detailed discussion and summary of items to include in the interagency agreements was prepared and reviewed with the technical review committee and can be found in Appendix E. An overview of the items discussed is presented below.

1. Capacity Limitations of use
   a. City to irrigation facilities
   b. Tailwater to City facilities

2. Quality standards to discharge
   a. TSS – concentration or % removal
   b. City to irrigation vs. tailwater to City
   c. Monitoring

3. Maintenance
   a. Cleaning – pipes vs. ditches
   b. Frequency - for water quality goals, carrying capacity
   c. Responsibility for pipes, ditches, other (if have settling ponds, may want agreement to share material removed from settling ponds with property owners)
   d. Cost sharing

4. Repairs – responsibility
   a. Pipes: City
   b. Ditches/Canals: Respective drainage district/ ditch company

5. Insurance responsibilities

6. Emergency procedures – notifications

7. Capital improvement plans
   a. Potential locations for improvements (e.g. tailwater settling ponds) – *consider relative to TMDL, cost-effectiveness vs. increased cleaning frequency*
   b. Cost sharing

8. Public education (storm water & agricultural BMPs)

9. Future monitoring
   a. Locations, parameters, frequency
   b. Responsibility

5.4 EXISTING DRAINAGE BASINS

This section discusses the condition of the existing drainage basins (see Figure 7, Appendix A) within the City of Ontario. This evaluation includes the hydraulic results of the computer modeling include areas of surcharging and flooding. This discussion also includes an evaluation of the maintenance issues and pipe conditions determined from the video records.

5.4.1 **Wal-Mart Drainage Basin** (Figure 11)

This drainage basin encompasses the commercially zoned area east of Interstate 84 and north of Idaho Avenue. This basin is illustrated in Figure 11. The developed area contributing to the public and private storm water systems in this basin totals about 52.7 acres. The storm water flows north and east to discharge through a 36-inch diameter tidal flex valve into the Snake River. There are several areas within this basin that have on-site retention and do not contribute to the runoff.

The computer model for this basin indicated that for a 24-hour 1-inch storm that the maximum runoff would equal approximately 19.3 cfs, with an average outflow of 0.53 cfs. This model was run over the 24-hour period of the storm with another 24-hour period to allow for runoff. The total volume of runoff over these 48-hours equaled about 91,100 cubic feet. The pipes had minimal, if any, surcharging and no flooding. In summary, this basin has adequate storm water utilities for the developed area enclosed in this runoff basin. Several of the lines in this basin were video recorded and were found to be in fair to good condition. No capital improvements are recommended at this time.

5.4.2 **Kmart Drainage Basin** (Figure 11)

This storm water runoff basin encompasses the area immediately south of the Wal-Mart drainage basin on the south side of Idaho Avenue. This area is primarily large commercial buildings with paved parking lots. The area served by storm water lines is approximately 27.6 acres. The rest of the area south of Idaho Ave and East of I-84 either has on-site retention or is not served by a storm water collection system. The Kmart basin storm water flows north and then to the east to discharge into the Snake River.

The model results indicate that with a 24-hour storm and an additional 24-hour period allowed for runoff, the peak runoff would be approximately 8.0 cfs, with a total volume of about 60,600 cubic feet. However, there are areas within this basin that do flood during the storm event for 1-2 hours. The lines south and north of Kmart and the line that extends to the Ernst parking lot both are surcharged and reach flooding levels. The other lines are also surcharged, but do not flood. Figure 11 illustrates the storm water system and Figure 10 shows the areas of flooding in this basin.

Some of the lines in this basin were video recorded and were found to be in fair condition as shown in Figure 8. It was recommended, however, to add a cleanout at the south end of the north south line just east of Kendall Road.

5.4.3 **Heinz Frozen Foods Drainage Basin** (Figure 12)

The Heinz Frozen Foods (HFF) drainage basin was initially evaluated to determine the modeling of runoff basins and whether the number of basins affected the results. It was found that the skeletonized model, that which included larger and fewer basins, had results similar to a more detailed model with smaller basins. Following these results, the rest of the basins in Ontario were evaluated in a similar manner.

A detailed evaluation of this basin was completed early in the study process to assist the City in planning a joint storm water facility with HFF. The evaluation of this basin and its results are included in Tech Memo #3 included in Appendix B. In summary, the basin covers an area of approximately 330 acres, 32 of which have onsite retention or drain directly to the HFF wastewater treatment plant. The existing storm water system is not capable of handling the 25-year storm event without surcharging many of the interceptor lines. Much of the excess runoff is stored in the manholes and in a few locations, rises to the surface, resulting in brief flooding. The maximum outflow is approximately 14.8 cfs with the total outflow volume being about 202,000 cubic feet. The surcharging of the trunk line is shown in Figure 7 of Tech Memo #3, which reflects the profile of the trunk line during a storm event at maximum flow.

Undersized pipes are a major factor in causing this surface ponding and Figure 5 in Tech Memo #3 illustrates the pipelines with a diameter less than 12-inches. Another factor that causes the flooding is sections of adverse grade. Sections of adverse grade not only cause hydraulic problems (i.e. flooding), but also increase the sedimentation in the storm line and cause maintenance problems. The ponding in this area is shown in Figure 10 in Appendix A and the trunk line running north and south in this runoff basin is also shown as being surcharged.

Recommendations to remediate this flooding and surcharging are discussed in the following chapter.

5.4.4 **Double Trunk Line Drainage Basin** (Figure 13)

Two parallel drainage lines running north and south collect irrigation return and storm water for much of the Ontario downtown area. These two lines and the areas of runoff that they collect are shown in Figure 13 in Appendix A. The smaller and older 24-inch pipeline collects agricultural runoff from two locations, groundwater from the Treasure Valley Community College (TVCC), and some storm water from the residential area south of the Alameda soccer park. The area of runoff for this trunk line is approximately 360 acres. From north of the campus until the outfall at the Snake River, this line does not collect storm water, but is connected in two locations to the parallel storm water trunk line that collects the storm water for the majority of downtown Ontario, an area of about 425 acres.

Portions of these trunk lines were video recorded and their condition evaluated. Figure 8 shows the results of the pipeline conditions survey and evaluation. It was found that the line carrying the majority of the irrigation runoff water was in poor condition and portions in immediate need of replacement. This line has historically had an issue with sediment build-up due to the agricultural runoff. This line has required regular cleaning due to the irrigation water.

In addition to the condition of the pipes in this basin, the lines are surcharged during a design storm event, especially when the storm occurs when irrigation water is flowing. Figure 10 highlights the lines that are surcharged and the areas of flooding. The majority of the problem areas in this basin are along the 24-inch trunk line carrying the irrigation water and some storm water runoff. Much of the surcharging in the larger storm water trunk line occurs for a short period of time and the one area of flooding is due to an undersized (less than 12-inch) pipe. Another area of flooding is near Pennington Drive in an area of low elevation and irrigation inflow. This flooding is currently in an area of low impact in an open field. Improvements to eliminate this flooding should be coordinated with future development.

5.4.5 **Park Boulevard Drainage Basin** (Figure 14)

The Park Boulevard Drainage Basin encompasses the area east of SW Alameda Drive and Verde Street, an area of about 550 acres. The topography of this area slopes to the east with a relatively significant slope just west of Park Boulevard. This basin and the sub-basin delineation is shown in Figure 14.

The structural condition of the pipes in this basin is shown in the Figure 8. The pipes in general are in fair to good condition. Those that are in less than fair condition will need to be replaced in the next 10 to 20 years.

The trunk line in this drainage basin is surcharged during the 25-year design storm event with some areas flooding. The reason for this surcharging can be attributed to undersized downstream lines. When the downstream lines do not have sufficient capacity, the upstream lines also surcharge, and possibly floods. According to City personnel some of the areas where the model predicts flooding have not historically had problems. One reason that the model results may show flooding in those areas could be due to the assumption that more storm water is collected by the system than is seen in the field, causing the model to overpredict the volume of storm water in the system. The main areas of concern are in the southern half of the basin near SW 11th Avenue and the southern end of Park Boulevard.

The improvements to minimize the flooding and minimize the surcharging are discussed in the following chapter.

5.4.6 Stewart Carter Canal and Verde Road Basin (Figure 15)

The Stewart Carter Ditch Basin overlaps the Verde Road Drainage Basin, both of which encompass an area of approximately 370 acres. The Stewart Carter basin includes the area along the Stewart Carter Canal with minimal inflow from City storm water runoff. The storm water that flows into the ditch is primarily south of SW 4th Avenue, upstream of the City of Ontario, before the canal is piped north along Verde to just east of Hunter Lane where it discharges and flows north to the Dork Canal. Flow in the Stewart Carter Canal upstream of the study area is complex and partially regulated. Flow into the basin was assumed based on historical flow data.

The Verde Road storm water drainage basin encompasses an area of approximately 174 acres west of Verde with a small development east of Verde near the north of the trunk line. The main trunk line parallels the Stewart Carter Canal pipe along Verde Street, but collects the storm water along the street and from other collection lines.

The condition of these pipes is generally unknown, having only a small section video recorded and evaluated. There is one section at the intersection of NW 4th Avenue and Verde Road that needs repair. Both lines are, however, at capacity and slightly surcharged during a storm event. The Stewart Carter Canal also stresses the capacity of the culverts under SW 7th Avenue and SW 4th Avenue during the design storm event. There is no flooding and the surcharging is minimal when compared to other drainage basin trunk line surcharging throughout Ontario.

5.4.7 NW 7th Avenue Drainage Basin (Figure 16)

This basin only encompasses an area of about 36 acres along NW 7th Avenue and Hunter Lane. There is a single storm water line that collects the runoff from this area and drains into the Dork Canal. This is a relatively new line and according to the model results, there are no surcharging or flooding problems.

5.4.8 Waterford Subdivision Drainage Basin (Figure 16)

This drainage basin encompasses approximately 140 acres west of Dorian Street and east of the new highway, and approximately 1,200 acres south of the SW 4th Avenue drainage basin and west of the Stewart Carter Canal. The Malheur Drainage District has a pipe network throughout the agricultural land west of the Stewart Carter Canal that collects the irrigation runoff (primarily through subsurface infiltration into the pipeline) and is carried north, through the Waterford Subdivision and discharges into the Dork Canal.

In developing the model for this basin a constant irrigation inflow based upon the area and pipe sizes was inputted at one of the upstream pipe junctions, as shown in Figure 10. When the design storm is superimposed on this drainage basin, the pipes are only slightly surcharged and there is no flooding. These pipes appear to have adequate capacity to carry the runoff from the design storm and the agricultural runoff from the large area south of the City which drains into this system.

5.4.9 Dorian Street Drainage Basin (Figure 16)

The Dorian Street Drainage Basin has a combination of open ditches and storm pipes to collect and convey the storm water runoff from an area of approximately 70 acres mainly east of Dorian Street. In the model, just the storm pipelines were evaluated for capacity and were found to be surcharged during a storm event in addition to the irrigation water that flows constantly during the summer months. As seen in Figure 10, a constant irrigation inflow of 0.6 cfs was assumed. There is no flooding predicted by the model and the pipelines seem to have adequate capacity for a 24-hour 1-inch storm in addition to the irrigation runoff.

5.4.10 SW 4th Avenue Drainage Basin (Figure 16)

The SW 4th Avenue drainage facilities are mainly those constructed by the Oregon Department of Transportation during a road improvement project in the summer of 1983. There is an area of about 75 acres just to the south of SW 4th Avenue that drains into this system, in addition to some minor agricultural runoff. This area drains into the storm water system, which conveys the flow west along SW 4th Avenue and north of the airport to discharge into the Dork Canal. This system is entirely separate from the airport's storm water system, which is all contained onsite in retention basins. During a design storm event and with a small constant inflow the line is surcharged, but there is no flooding predicted by the model. These lines are relatively new and are assumed to be in good condition.

5.4.11 Summary of Existing Drainage Basins (Figures 7 and 10)

The following table summarizes the existing drainage basin model results. The amount of time allowed for the pipes to reach steady state condition, pipe fill duration, and the time allowed after the 24-hour storm, runoff duration, both affect the outflow volume. The maximum outflow, however, is more a function of the 24-hour design storm than the pipe fill and runoff times. The areas that are surcharged and/or flooded are illustrated in Figure 10 in Appendix A.

TABLE 5.1
EXISTING DRAINAGE BASIN SUMMARY

Basin	Area (acres)	Irrigation Inflow (cfs)	Max Outflow (cfs)	Outflow Volume (cu. Ft.)	Surcharged (Yes/No)	Flooding (Yes/No)
Wal-Mart	52.6	0	19.3	91,100	Yes	No
Kmart	27.6	0	8.0	60,600	Yes	Yes
Heinz Frozen Foods	329.8	0	14.8	202,000	Yes	Yes
Double Trunk Line	484.6	6.5	32.8/11.0	275,400/ 1,395,000	Yes	Yes
Park Blvd.	585.5	0.57	38.5	609,800	Yes	Yes
Stewart Carter Canal	196.6	50	52.1	9,477,800	Yes	No
Verde Street	330.8	0	13.1	100,600	Yes	No
NW 7th Avenue	35.7	0	4.8	18,500	Yes	No
Waterford Subdivision	1392.4	6.2	12.3	1,406,000	Yes	No
Dorian Street	71.1	0.6	8.6	153,200	Yes	No
SW 4th Avenue	95.5	5.3	10.7	1,093,000	Yes	No

5.5 DITCH ANALYSIS (Figure 17)

A capacity analysis was performed on four canals/ditches in the city of Ontario. These canals included the Stewart Carter, the ditch west of Skyline, the Dork

Canal, and an un-named ditch north of the fairgrounds, as shown in Figure 17 in Appendix A. The analysis involved collecting topographic and geographic data for each of the canals/ditches at selective locations along their reach. This data was used to calculate a flow based on surveyed channel cross-sections and slopes. The flow was computed at a minimum of two different depths: full depth (capacity) and one foot of freeboard. Full depth is defined as the water level at the top of the bank elevation of the canal/ditch. One foot of freeboard occurs when the stage of the canal/ditch is one foot below the top of the bank elevation.

5.5.1 Results

A complete summary of the input data and the resulting flows for each location analyzed on the canals is located in the Appendix. A synopsis of the flow results is provided below.

TABLE 5.2
CANAL CAPACITY SUMMARY

Canal/Ditch	**Stewart Canal**	**Ditch West of Skylane Drive**	**Dork Canal**		**Un-named Ditch**
Location	South of SW 4th Avenue	South of SW4th Avenue	Downstream of Malheur Drive	Upstream of Verde Drive	North of Fairgrounds
Full Depth Flow (cfs)	131	17	318	136	159
1' Freeboard Flow (cfs)	71	8	201	95	103
High Water Flow* (cfs)	47	-	-	-	-

*The high water mark was recorded for the Stewart Carter Canal and was used to confirm a close correlation between calculated flows/capacity with recorded flows reported in the OWEB study.

5.5.1.1 Stewart Carter Canal

The Stewart Carter Canal delivers water north from the Owyhee Canal to the agricultural land south of Ontario, and conveys runoff north to the Dork Canal. It is piped through the City of Ontario from SW 4th Avenue north along Verde Drive to Hunter Lane where it discharges into an open channel, which drains into the Dork Canal east of Verde Drive. The table above outlines the capacity at a location near the mouth of the canal.

In 2001 the City of Ontario, in cooperation with the Oregon Watershed Enhancement Board conducted a water quality monitoring project. As part of this project the flow in various open channels was measured. One of the measured sites was located along the Dork Canal just upstream of the discharge into the Snake River. Recognizing that the Stewart Carter Canal flows into the Dork Canal and making the conservative assumption that all the flow in the Dork Canal originated from the Stewart Canal, the maximum flow for 2001 was measured at approximately 75 cfs (see Water Quality Monitoring Project Report, p. 1-12). The capacity of Stewart Canal at full depth, as shown above, is 131 cfs, which provides sufficient capacity for this peak flow.

5.5.1.2 Ditch West of Skylane Drive

The ditch west of Skylane Drive conveys irrigation water and agricultural runoff from the agricultural lands southwest of Ontario. It drains into the Ontario storm water pipe network just south of SW 4th Avenue and eventually discharges into the Dork Canal. The table above outlines the capacity at a location just south of SW 4th Avenue.

As cited in the Water Quality Monitoring Project Report (p. 1-14), the maximum reported flow at this location for 2001 was approximately 25 cfs, which is slightly higher than the maximum capacity calculated by Keller Associates. Keller Associates believes that the stage-storage relationship developed previously may not accurately reflect conditions and that previously reported flows were higher than actual flows.

5.5.1.3 Dork Canal

The Dork Canal receives a large portion of the City's storm water runoff and flow from the Stewart Carter Canal. The table above outlines the capacities at two locations along the canal. As expected, the capacity of the canal increases towards the mouth of the canal which discharges into the Snake River. The maximum flow in 2001 at a site just downstream of the discharge into the Snake River was approximately 75 cfs. The capacity of Dork Canal ranges from 131 cfs just upstream of Verde Drive to 318 cfs downstream of Malheur Drive. The Dork Canal's capacity of 131 to 318 cfs is sufficient to carry the flow data measured.

5.5.1.4 Un-named Ditch

The un-named ditch north of the fairgrounds conveys runoff, groundwater and the discharge from the storm water trunk line that runs along Park Boulevard. It drains northeast and eventually discharges into the Snake River. The table above outlines the capacity at a location near the mouth of the canal, south of the freeway. As reported in the Water Quality Monitoring Project Report (p. 1-14, the maximum flow in 2001 at this location was approximately 5 cfs. Keller Associates calculated this ditch's capacity at 159 cfs, which is more than adequate for the measured flows.

5.6 CULVERT ANALYSIS (Figure 17)

A capacity analysis was performed on the culverts located along three canals/ditches in the city of Ontario: the Stewart Carter, the Dork Canal, and the un-named ditch. The locations of the analyzed culverts are shown in Figure 17 in Appendix A. The analysis involved collecting topographic and geographic data for each of the culverts along the canals of interest. This data was entered into a XP-SWMM2000, a computer software program that models culvert and open channel flow behavior. The computer model calculated culvert capacities, with a flow in the channels equal to the flow at full depth. Full depth is defined as the water level at the top of the bank elevation of the canal/ditch, at which point the channel is close to flooding.

5.6.1 Corrective Measures Based on Control Type

Culvert capacities are controlled by their inlet or outlet flow conditions. Generally, long, rough culverts with submerged exits and flat slopes have capacities that are controlled by outlet flow conditions. Steep, smooth, and short culverts with submerged entrances have capacities that are controlled by inlet flow conditions. Corrective measures for increasing culvert capacities are dependent on the culvert control type. Decreasing the headloss at the entrance can increase capacities for culverts with inlet control. For example, adding headwalls and wing walls to the pipe or changing the angle of the wing walls can increase the culvert capacity. Changing edges from square to bevel can also increase the capacity. Capacities for culverts with outlet control can be increased by reducing the stage of the canal downstream of the culvert, reducing the roughness of the culvert, or by decreasing the length of the culvert. For either control type, increasing the diameter of the culvert or adding parallel pipes will also increase the capacity of a culvert.

5.6.2 Results

A complete summary of the input data and the resulting flows for each culvert analyzed is located in Appendix D. A summary of the culvert analysis results is provided below.

TABLE 5.3
RUNOFF BASIN PARAMETERS

Canal	Location	Capacity (cfs)	Control Type
Dork Canal	Malheur Drive	125	Inlet
Dork Canal	West of Verde Drive	35	Outlet
Dork Canal	Private Driveway	47	Outlet
Dork Canal	South of WWTP	63	Outlet
Stewart Carter Canal	SW 4th Avenue	44+	Outlet
Stewart Carter Canal	SW 7th Place	63	Inlet
Un-named Ditch	NW 9th Street	67	Outlet

5.6.2.1 Stewart Carter Canal

The bottleneck in the Stewart Carter Canal is the SW 4th Avenue culvert. The canal capacity flowing full is approximately 130 cfs, while the capacity of the SW 4th Avenue culvert is only approximately 44 cfs. It should be emphasized that this capacity is at flooding, and the calculated capacity of 44 cfs is a conservative value. This culvert is a box culvert and has a larger inlet, (9' x 3') than outlet (4' x 2.5'). The 44 cfs reflects the capacity of a uniform box culvert with the smaller of the two dimensions, 4' x 2.5'. The high water mark observed in the Stewart Carter Canal South of SW 4th Avenue of 2' would equate to a flow of 47 cfs. Although flows possible at high water marks actually slightly exceed the culvert capacity, historically there has been no flooding at this culvert according to City personnel. Therefore, the culvert at SW 4th Ave. warrants future investigation and monitoring.

The other culvert along the Stewart Carter Canal at SW 7th Place has a capacity of 63 cfs. This capacity satisfies the high water mark flows of 47 cfs calculated along this canal.

According to City personnel, debris and rubbish blockage during peak load events such as storm and irrigation events have occurred at the entrance of the pipe north of SW 4th Avenue and west of Sunset Drive and at the outfall at Hunter Lane. The blockage at the outfall has caused the flow to back up and flood at two sites along the pipeline that runs along Verde Road. Debris and blockage at other culverts along Stewart Carter Canal is a potential problem and can be minimized with a regular maintenance program.

5.6.2.2 Dork Canal

The culvert with the least capacity along the Dork Canal is the culvert directly west of Verde Drive, which has a capacity of 35 cfs. This culvert is 705 feet long, which results in large head losses and creates an outlet control condition. Typical flows for this portion of the Dork Canal are unknown, but the Canal's capacity at full depth flow is approximately 136 cfs, significantly higher than the 35 cfs capacity of the culvert. However, the typical flows are assumed to be less than the 35 cfs capacity due to no historical flooding attributed to this culvert. In addition, the Stewart Carter Canal is the most significant inflow to the Dork Canal just downstream of the east end of this culvert, east of Verde Drive. This culvert may present a potential problem due to its limited capacity if the flows upstream increase dramatically due to either increased irrigation flows or a significant storm event.

The Stewart Carter Canal discharges directly downstream from the culvert at Verde Drive with recorded flows of up to 65 cfs. The only culverts downstream from this discharge point are the parallel culverts under Malheur Drive. The total capacity of these parallel culverts, approximately 125 cfs, is smaller than the capacity of the Dork Canal downstream of Malheur Drive, approximately 320 cfs. However, the 125 cfs capacity substantially exceeds the 2001 peak recorded flow of 75 cfs (see Water Quality Monitoring Project Report, p. 1-12). The capacity of the culverts under Malheur Drive is dependent upon inlet control. Therefore, by adding a headwall, changing the corners from square to bevel, or adding an additional parallel pipe could potentially expand the capacity. At this time, Keller Associates recommends to continue to monitor flows in the Dork Canal.

The culverts under the private driveway and South of the WWTP are upstream of both the discharge of Stewart Carter Canal and the culvert west of Verde Drive. These culverts have capacities of 47 cfs and 63 cfs, respectively. Due to these culverts being upstream of the culvert at Verde Drive and having larger capacities, the flooding potential is less critical than at Verde Drive. Furthermore, the capacities of these two culverts exceed observed flows.

Flooding during peak load events like storm and irrigation events have been observed by City personnel at culverts along the Dork Canal due to debris and rubbish blockage. This problem can be minimized with a regular maintenance program.

5.6.2.3 Un-named Ditch

The only culvert on the un-named ditch is the culvert at NW 9th Street. This culvert has a capacity of approximately 67 cfs and easily satisfies the observed maximum flows of 5 cfs recorded in 2001 downstream of the culvert (see Water Quality Monitoring Project Report, p. 1-12). This ditch can accommodate flows of up to 160 cfs. Both the culvert and ditch have sufficient capacity to satisfy growth and peak flows. According to City personnel, neither the ditch nor the culvert has a history of flooding.

6.0 - WATER QUALITY ISSUES

6.1 OVERVIEW

In the past, the primary objective of storm water management was flood control. Water quality issues have been gaining importance, with recent federal regulations addressing the potential impacts of storm water on groundwater and surface water quality. Both are potentially affected by Ontario's storm water system, which involves discharge to the Snake River for about 94% of the area served and onsite disposal for the remainder.

The Underground Injection Control (UIC) program covers storm water impacts on groundwater, while the municipal storm water National Pollutant Discharge Elimination System (NPDES) program addresses surface water quality issues. Total Maximum Daily Loads (TMDLs) and Water Quality Management Plans prepared for 303(d)-listed water quality limited waters also relate to the potential impact of storm water on surface water.

6.2 UNDERGROUND INJECTION CONTROL (UIC) PROGRAM

6.2.1 Background

The Underground Injection Control (UIC) Program was enacted in 1974 for management of fluid injection underground, in order to protect groundwater aquifers from contamination. The primary goal of the UIC Program is to preserve groundwater for beneficial use such as drinking water. All groundwater aquifers in Oregon are deemed suitable for drinking water.

The Department of Environmental Quality (DEQ) administers the UIC program in Oregon. Oregon Administrative Rules (OAR 340-044) govern underground injection activities of all types. Final UIC rules went into effect in September 2001. The OAR define types of injection systems and applicable underground injection control requirements. Injection activities must be authorized through DEQ, either by registering the injection system and meeting general regulatory requirements ("rule authorized") or by obtaining a permit.

The DEQ UIC Program covers storm water systems such as sumps, infiltration galleries, drywells and french drains, which are designated as Class V injection systems under the Federal UIC program. EPA Region X interprets Class V systems as "any system, structure, or activity that is created to emplace fluid directly into the subsurface." A dug hole or even a trench with piping qualifies as a Class V system if its purpose is subsurface discharge through infiltration or injection. The UIC Program excludes *single residential* roof or footing drains that receive only storm water.

DEQ has developed guidance documents and forms to facilitate compliance with the UIC Program. Several of these are included in Appendix G, along with a list of documents referenced in preparing this summary.

6.2.2 UIC Storm Water Management Requirements

Identification of all public water suppliers and potential sources of contamination, including injection systems, is required by the year 2003. An owner or operator of an existing or new injection system must register the system, and be authorized to use it by DEQ. Registration involves inventorying the system (Form UICJUR-1005) and submitting the inventory data to DEQ.

Registration information is used by DEQ to determine if the injection well qualifies as "rule authorized" (no separate permit required). To qualify, a system must meet a "non-endangerment" standard to ensure the well is located, constructed, operated and maintained such that it does not endanger underground sources of drinking water. Specific requirements are summarized in "Underground Injection Control Rule Authorized Storm Water Injection Requirements" (UIC Storm RA 1101).

In addition to the non-endangerment standard, storm water injection systems will qualify as "rule authorized" only if no other disposal option is appropriate. OAR 340-044-0030 specifically prohibits injection wells with depths greater than their largest surface dimension, if any other treatment or disposal method which "affords better protection of public health or water resources is reasonably available or possible."

All "rule authorized" systems must meet the *General Requirements* in RA 1101. Further provisions in *Basic Requirements* must be met by all injection systems except roof drains. Additional specific *Category Requirements* (per RA 1101) apply to the following categories:

- Municipal systems with 50 or more injection wells
- Municipal systems with less than 50 injection wells
- Industrial/commercial facilities with hazardous substances
- Industrial/commercial facilities without hazardous substances
- Large parking lots
- Small parking lots
- Residential systems included in the UIC Program (e.g. garage floor and driveway drains)

Owners of any category of "rule authorized" storm water injection systems (except residential) must prepare and implement a storm water management plan. The required elements of the plan vary depending on the size of the system. Certain elements – system assessment; Best Management Practices (BMPs) for source control and treatment; spill prevention and response; maintenance plan; employee and public education; and evaluation of plan effectiveness – are required for any size system. For municipal systems with 50 or more injection systems, storm water management plans must also have monitoring and record-keeping plans.

DEQ has developed recommendations for source control measures, spill response, storm water maintenance standards, education outreach, and monitoring. These are documented in "DEQ UIC Class V BMPs for Groundwater."

If an injection system does not qualify as "rule authorized", the Owner may be required to either: 1) modify the system so it meets the criteria for rule authorized; 2) close the injection system; 3) discharge to a municipal storm sewer, if available; or 4) apply for a WPCF Permit. DEQ will be developing a general WPCF storm water permit for Class V systems which fail to meet Rule Authorization requirements.

Municipalities with over 50 injection systems need to develop a Decommissioning Plan for injection systems that do not meet the *Basic Requirements* (OAR 340-044-0018). DEQ documents (Ref. 5 and 6) outline evaluation steps needed, and suggest closure standards for storm water injection systems. DEQ Storm water Management Guidelines outline different methods to remove pollutants from storm water prior to groundwater discharge, including alternatives to injection wells.

Municipalities also have the option to negotiate an area-wide permit or memorandum of agreement with DEQ for systems that fail to meet Rule Authorization requirements. (As of March 2002, no area-wide UIC Class V agreements had yet been negotiated.) An area-wide permit would need to include the following elements:

- Quarterly inventory reporting of new injection systems
- Use of DEQ database spread sheet
- GPS location data
- Monitoring and maintenance plans
- Maintenance schedule
- Storm water management
- Screening for hazardous areas
- Spill plans
- Closure and remediation requirements

- Inspection and enforcement options
- Information on existing land uses and any available data on unsuitable areas (soils)

In summary, any owner or operator of a Class V storm water system is required to:

1) Register system prior to use, and provide inventory data to DEQ.

2) Meet "non-endangerment" performance standard to prevent contamination of groundwater by storm water.

3) Submit a closure plan to DEQ, and then properly decommission a banned system or any system when it is no longer in use.

4) Comply with other local, state and federal regulations (including requirements of the State Groundwater Act and the Safe Drinking Water Act Standards).

6.3 NPDES PROGRAM

Phase I of the municipal storm water NPDES program addressed medium and large systems, in places with more than 100,000 people. Phase II, adopted in 1999, applies to regulated small municipal separate storm sewer systems within urbanized areas. "Urbanized areas" are defined as areas with a population of 50,000 or more, and a population density of at least 1000 people per square mile. Since neither the City of Ontario nor Malheur County have sufficient population to be classified as an urbanized area, current NPDES requirements do not apply.

However, certain municipalities outside urbanized areas will be evaluated by DEQ to determine the need for an NPDES permit. Ontario was one of the cities selected for evaluation. If DEQ determines that storm water controls are needed – on the basis of a TMDL allocation, contribution to a violation of water quality standards, or a significant contribution of pollutants – an NPDES permit may be required.

DEQ has drafted a general NPDES permit to address storm water discharge from small municipal systems (MS4). The MS4 permit requires a permittee to "develop, implement, enforce and measure the effectiveness of, a storm water management program designed to reduce the discharge of pollutants… to the maximum extent practicable…" The management program must include the following 6 minimum control measures:

- Public Education and Outreach on storm water impacts
- Public Involvement/Participation in program development
- Illicit Discharge Detection and Elimination

- Construction Site Runoff Control
- Post-Construction Runoff Control in new development and redevelopment
- Pollution Prevention/Good Housekeeping

At the time this report was prepared, DEQ was not requiring Ontario to participate in the permit program since there was insufficient data linking storm water impacts from the MS4 to receiving water quality. Inclusion in the NPDES program may occur in the future as a result of completion and approval of Total Maximum Daily Loads (see below).

6.4 TOTAL MAXIMUM DAILY LOADS (TMDL) AND WATER QUALITY MANAGEMENT PLAN (WQMP)

6.4.1 Background

As part of a watershed approach to water quality problems, the State of Oregon is developing a Total Maximum Daily Load (TMDL) and Water Quality Management Plan (WQMP) for each water body that doesn't meet water quality standards. The Malheur and Snake River are among these. The Snake River TMDL process is expected to be complete by the end of 2002, and the Malheur River TMDL is scheduled for completion in 2003.

Since Oregon and Idaho share the Snake River, development of the Snake River-Hells Canyon TMDL and WQMP was a bi-state effort. Recreational groups; city, county, state and federal governments; and industrial, agricultural and tribal interests were also involved. The Draft Snake River-Hells Canyon TMDL and WQMP was issued for public comment in December 2001. A public hearing was held April 11, 2002. As of January 2003, responses to review comments were being finalized.

6.4.2 Overview (Snake River)

Ontario is on the Upstream Snake River segment of the Snake River-Hells Canyon TMDL reach. This segment encompasses a 74-mile section of the Snake River from the Oregon/Idaho border downstream to Farewell Bend. Most of the agricultural and urban land use along the river occurs within this segment. All but two of the major tributary inflows to the entire 221-mile Snake River-Hells Canyon (SR-HC) TMDL reach enter the river within this segment. Flow within the Upstream Snake River segment is related to snowmelt and precipitation, upstream/tributary impoundments, and irrigation (diversion and returns).

Upstream Snake River segment waters are listed as water quality limited due to concerns over bacteria, dissolved oxygen, mercury, nutrients, pH, sediment and temperature. Since available data for the upper section of the river showed no exceedences of bacteria or pH limits, the SR-HC

TMDL recommends de-listing the upper 62 miles of the Upstream Snake River segment for these two pollutants. Ontario is included in that portion of the river.

6.4.3 SR-HC TMDL Summary

6.4.3.1 Dissolved Oxygen/Nutrients

Dissolved oxygen and nutrients, interconnected in relation to water quality, are considered together in the TMDL.

Dissolved oxygen is vital for fish and aquatic life. Dissolved oxygen (DO) concentrations below 5 mg/l cause stress to aquatic species. Low DO also affects water and sediment chemistry, which can increase the concentration and mobility of nutrients and toxins (e.g. phosphorus, ammonia and mercury).

Low DO may result from high nutrient loadings that cause nuisance aquatic growth (algae and rooted plants). In addition to harmful effects on aquatic life, algae blooms can adversely affect recreational uses through the production of toxins that cause skin irritation and animal illness/death if ingested. Nutrient enrichment of drinking water supplies has also been identified as a serious health problem.

Available data collected in the Upstream Snake River segment show seasonal dissolved oxygen levels averaging from 9 to 11 mg/l at the mouth of the Malheur River. Dissolved oxygen concentrations vary with temperature and with flow. Lower dissolved oxygen levels are most common in late summer, with high temperatures and low water levels.

Total phosphorus is the nutrient of concern in the SR-HC TMDL reach. Available data show excessive total phosphorus concentrations in the Upstream Snake River segment, and nuisance algae blooms have been observed to occur routinely in this segment. A reduction in total phosphorus correlates to a reduction in algal biomass, and a corresponding improvement in dissolved oxygen levels.

A target level of 6.5 mg/l dissolved oxygen has been established as an absolute minimum to support aquatic species. The SR-HC TMDL established a target level for in-stream total phosphorus of 0.07 mg/l, to achieve water quality standards and support designated beneficial uses. Dissolved oxygen targets will be applied year-round, while attainment of total phosphorus targets is critical for May through September.

6.4.3.2 Mercury

Mercury in the river translates to mercury in fish, which is of concern for human consumption. In Oregon, the level of concern for mercury in fish tissue (for issuing fish consumption advisories) is 0.35 mg/kg (0.5 mg/kg in Idaho). The US FDA has established an action level of 1.0 mg/kg mercury in fish tissue to protect against potential health risks from human consumption. Criteria promulgated by the US EPA identified a maximum of 0.050 μg/l for mercury in the water column.

All fish tissue data in the Upstream Snake River segment were positive for mercury, and Oregon and Idaho levels of concern were exceeded by 80% and 52%, respectively. Although the available data do not show direct impairment of aquatic life uses due to mercury, the designated beneficial use of fishing is considered impaired because of fish consumption advisories issued by Oregon and Idaho.

Target levels for mercury established by the SR-HC TMDL include 0.35 mg/kg fish tissue concentration and 0.012 μg/l water column concentration. Mercury targets will be applied year round. Lack of data currently prevents determination of the TMDL; data collection will be accomplished during the first phase of implementation following plan approval. After existing loading is determined, waste load (point source) and load (non-point source) allocations will be made.

6.4.3.3 Sediment

Sediment loading can directly impair aquatic life uses, and is also of concern due to the attached pollutant loads (mercury, pesticides and nutrients) that the sediment carries. The available data show that over 95% of the sediment loading into the SR-HC reach originates from the Upstream Snake River segment. No duration data for sediment is available to assess the extent of use impairment in the Upstream Snake River segment. Sources of unmeasured load may include human activities, runoff, unidentified small tributaries and drains.

Targets of 50 mg/l total suspended solids (TSS) as a monthly average, and 80 mg/l for no longer than 14 days have been established to protect aquatic life uses. While duration data is being collected, sources currently discharging less than 50 mg/l monthly average will be assigned no net increase load allocations. Sediment targets will be applied year-round.

6.4.3.4 Temperature

High temperatures can be harmful to fish, especially occurring with other habitat limitations such as low dissolved oxygen or poor food supply. Acceptable temperature ranges vary for different species of fish. Criteria have been established for maximum temperatures that support the aquatic life needs of various species, as follows:

- For cold water biota and salmonid rearing, 17.8 °C as a 7-day average of daily maximums

- For salmonid spawning (when & where it occurs), 13.0 °C instantaneous, or 9 °C average daily mean

Since salmonid spawning is not observed to occur in the Upper Snake River segment, the salmonid spawning beneficial use designation and accompanying water quality targets will apply only to tributaries with that designated use. State-specific targets will be applied to those areas of the tributaries designated for salmonid spawning.

Summertime water temperatures exceeding 24 °C have been routinely measured in the Upper Snake River segment. Natural atmospheric heat sources are the dominant influence on water temperature in the river. Because natural influences cause temperature exceedences, cumulative man-caused effects are allowed to have no measurable increase over natural background levels. Oregon State standards define “no measurable increase” as 0.14 °C.

The critical conditions for target temperatures are from June through September for cold-water biota and salmonid rearing.

6.4.4 Load Allocations

Load allocations for non-point sources have been established for phosphorus, sediment and temperature. A dissolved oxygen allocation has been established for Brownlee Reservoir for the period between the last of July to the first week of October; dissolved oxygen concentrations are expected to improve with reductions in total phosphorus. Storm water, as well as agriculture and forestry, is included in the load allocations for ungaged flow.

TABLE 6.1
LOAD ALLOCATIONS FOR UPSTREAM SNAKE RIVER

	Total	Ungaged Flow
Phosphorus (May through Sept), kg/day	2,495	134
Sediment, kg/day	1,684,019	281,656
Temperature (June through September)	Existing Load	Existing Load

Implementation of procedures for nonpoint sources to meet TMDL targets are expected to begin after completion of site-specific implementation plans, within 18 months after approval of the SR-HC TMDL. If monitored trends subsequently indicate that sediment concentrations are increasing despite implementation efforts, load allocations will be revised downward.

6.5 ONTARIO STORM WATER POLLUTANT LOADINGS

6.5.1 Background

The City of Ontario's storm drain system collects and transports runoff water from various drainage areas in the City to the Snake River. Four trunk lines, including those serving the business core, discharge directly to the river. Four trunk lines discharge to the river indirectly by way of canals, combining urban storm water with irrigation water conveyance and return flow facilities.

A water quality monitoring project, funded through an Oregon Watershed Enhancement Board (OWEB) grant, was initiated to assess pollutant loadings of urban storm water and agricultural runoff from the City of Ontario system. (Storm water, agricultural runoff and groundwater, which intermingle in the City of Ontario system, are also considered in combination for TMDL load allocations.)

The OWEB project was initiated to obtain baseline water quality and flow information necessary for an assessment of existing conditions, and to identify potential storm water impacts in the Snake River watershed. It was also intended to help clarify the correlation between the drainage and irrigation facilities entering and exiting the City, since future water quality improvements will involve coordination with irrigation and drainage districts.

The OWEB project included setting up eleven monitoring sites where samples were collected over a one-year period, and tested for 22 parameters (including dissolved oxygen, nutrients, mercury, sediment and temperature). Water quality tests were done on a monthly basis in addition to three storm events. Flow at eight of the sites (Sites 1-9) was monitored hourly, in addition to a staff gauge reading five times a week.

Three of the monitoring sites - Sites 3, 5 and 10 - are just upstream of outfalls to the Snake River. Site 3 is on the Dork Canal, which receives flow from two storm water trunk lines. Sites 5 and 10 represent direct storm water discharge points.

6.5.2 Summary of OWEB Data Relative to TMDL

Flow rates at the most downstream sites along the Stewart Carter Ditch and the Dork Canal were more sensitive to storm events than monitoring sites upstream of the City of Ontario. This indicates that the city's storm water runoff does contribute to the flow in these irrigation canals. However, it should be noted that storm water runoff has a minimal impact on overall flow. Total flow from storm events represents less than 1% of the total annual flow through the monitored sites. Even at sites not located along irrigation canals, the flows through all (except Site 5) were influenced by irrigation and return flows.

Water quality results relative to Snake River TMDLs are summarized in Table 6-1 for the primary storm water discharge points monitored. (Combined flow from Sites 1 and 2 was used for Site 10 flows, which were not measured.) Complete results of the OWEB water quality monitoring project are included in the City of Ontario/OWEB Water Quality Monitoring Project Report, July 15, 2002.

TABLE 6.2
OWEB MONITORING RESULTS

		Flow, cfs						TSS, mg/l			Dissolved Oxygen, mg/l			Temperature		
		Site 3	Site 5	Sites 1&2	Site 3	Site 5	Site 10	Site 3	Site 5	Site 10	Site 3	Site 5	Site 10	Site 3	Site 5	Site 10
4/17/01	Regular	-	-				NF	15	13	NF	22	20	NF	11.8	19.2	NF
5/14/01	Regular	-	0.2	1.3/-	0.25	0.20	0.20	267	40	46	9	8	8	13.8	15.3	14.6
6/4/01	Regular	69.8	0.4	5.5/0.1	0.31	0.21	0.68	142	15	834	10	11	9	12.0	13.8	13.4
6/27/01	Storm Event	41.8	0.7	0.3/0.5	0.66	0.75	0.67	582	230	1866	8	6	7	22.2	23.2	23.0
7/2/01	Regular	50.2	0.1	0.5/1.9	0.35	0.31	0.28	223	9	89	8	3	8	19.8	15.7	20.0
8/20/01	Regular	59.1	0.4	0/9.0	0.37	0.35	0.24	104	15	26	7	5	8	17.2	14.9	12.0
9/17/01	Regular	50.2	0.4	0/0	0.22	0.53	0.37	43	18	3	8	7	8	15.7	15.1	18.5
10/16/01	Regular	37.1	0.3	0/1.3	0.11	0.26	0.19	32	13	0	9	7	7	9.7	10.1	14.6
10/30/01	Storm Event	2.9	0.7	0/0	0.32	0.59	0.42	55	54	70	8	7	7	11.8	13.5	14.0
11/13/01	Regular	0.8	0.5	0/0	0.20	1.00	0.32	9	35	117	9	8	8	8.0	13.4	13.9
12/13/01	Regular	0.4	0.6	0/0	0.29	0.25	0.24	14	2	13	10	9	7	1.5	5.9	13.9
1/14/02	Regular	0.4	0.5	0/0	0.20	0.25	0.23	4	0	1	7	6	5	2.5	5.6	12.4
2/11/02	Regular	0.4	0.5	0/0	0.13	0.22	0.23	13	27	9	10	9	9	1.3	3.7	11.9
3/11/02	Storm Event	0.1	0.7	0/0	0.11	0.19	0.25	6	2	1	7	7	6	6.1	7.0	11.9
3/18/02	Regular	0	0.8	0/0	0.14	0.18	0.23	7	1	1	10	10	11	1.2	1.6	11.6
4/15/02	Regular	48.3	0.2	0/0	0.58	0.22	0.21	276	10	1	10	7	8	8.3	7.0	11.9
	Maximum	69.8	0.8		0.66	1.00	0.68	582	230	1866	22	20	11	22.2	23.2	23.0
	Minimum	0	0.1		0.11	0.18	0.19	4	0	0	**7**	**3**	**5**	1.2	1.6	11.6
	Average	25.8	0.5		0.28	0.37	0.32	112	30	206	9	8	8	10.2	11.6	14.5
	Avg regular	28.8	0.4		0.26	0.33	0.29	88	15	96	10	8	8	9.5	10.9	14.1
	Avg storm	14.9	0.7		0.36	0.51	0.45	214	95	646	8	6	7	13.4	14.6	16.3
	Annual CF	894,456,360	13,969,188	95,250,960												

6.5.2.1 Dissolved Oxygen/Nutrients

Monthly dissolved oxygen (DO) levels measured at the three discharge points ranged from 3 -22 mg/l, with an annual average of 8 -9 mg/l. The DO levels were generally lowest during the summer months. Dissolved oxygen levels also appeared to drop during storm events. These discharges apparently have minimal impact on the river, as data in the SR-HC TMDL for the Upstream Snake River showed no DO concentrations less than the 6.5 mg/l target value.

Total phosphorus concentrations in the flow from Sites 3 and 5 ranged from 0.11-1.0 mg/l, averaging 0.28-0.37 mg/l on an annual basis. Site 3, which accounts for about 90% of the measured flow from the 3 discharge points, contributes most of the phosphorus loading. Daily loads up to 69 kg/day were measured at Site 3 during the critical May-September period. This represents *over half* of the load allocation for ungaged flows in the Upstream Snake River Segment.

6.5.2.2 Mercury

Heavy metal concentrations were measured once during non-storm conditions. Mercury was too low (<0.2 μg/l) to detect any differences between sites.

6.5.2.3 Sediment

Total suspended solids measured at Sites 3 and 10 during non-storm periods exceeded the TMDL monthly and bi-weekly target levels of 50 mg/l and 80 mg/l during several months. Maximum monthly values of 276 mg/l and 834 mg/l were measured at Site 3 and 10, respectively. Average yearly values (non-storm) for both Sites 3 and 10 were 200% of the monthly average target level.

Even higher concentrations – up to 1866 mg/l at Site 10 – were experienced during storm events. Increased agricultural activity and initial flushing of the irrigation canals appeared to have the greatest impact on suspended solids concentrations in the storm runoff.

Because the OWEB data are single daily measurements (rather than monthly or bi-weekly averages), they cannot be used to determine if the SR-HC TMDL sediment targets are being exceeded. However, the fact that sediment values were substantially higher than the 80 mg/l target over several consecutive months makes it likely that there are TMDL exceedences.

Based on OWEB measurements of flow and total suspended solids concentrations, the average sediment load calculated for Sites 3 and 5 is 12,000 kg/day, or about 4% of the load allocation for ungaged flows in the Upstream Snake River Segment.

6.5.2.4 Temperature

Single daily temperature measurements in July were above the cold water biota/salmonid rearing target temperature of 17.8 °C at both Sites 3 and 10. Site 10 was also above the target temperature in September. Flows at all 3 discharge points were above 22 °C during a storm event in late June.

Ambient temperatures in the river, upstream of these discharge points, are similarly high. Continuous exceedences of the target value are observed in the Upstream Snake River segment in July and August, and 20% of the historical data in these months showed temperatures above 22 °C.

The temperature values measured in the OWEB project represent the "existing load" TMDL for these discharges. The TMDL allows no measurable increase over these background levels.

6.5.3 Unmeasured Loads

The OWEB project was limited to portions of the Ontario storm water system anticipated to be impacted by agricultural uses. The three discharge points monitored in the OWEB study drain an estimated 1638 acres, representing over 2/3 of the area in Ontario that drains to the Snake River. The remaining trunks that discharge to the river carry runoff from about 772 acres, including the downtown business core.

Pollutant loads for an area can be estimated from the type of land use and imperviousness of the area. For commercial/industrial areas in Boise, average concentrations of pollutants during storm events are estimated at 155 mg/l TSS and 0.88 mg/l total phosphorus ("Land Use and Pollutant Load Characterization Requirements", Ada County Highway District). Using these values for Ontario results in estimated loads of 500 and 3 kg/*year* for TSS and total phosphorus, respectively, in the unmeasured storm water runoff.

6.6 STORM WATER MANAGEMENT PRACTICES

6.6.1 General

Should a Phase II MS4 permit (Section 6.3) be required in the future, the City would be obligated to implement storm water management practices that achieve the six minimum measures for reducing storm water pollution impacts. Ordinances, policies and programs need to specifically address these six measures: public education, public involvement, illicit discharge detection and elimination, construction site runoff control, post-construction runoff control, and pollution prevention in municipal operations.

The permit also requires that the City establish measurable goals for evaluating the success of each of these measures. Target dates for implementing each measure should be identified.

6.6.2 Public Education

Education helps create a knowledgeable public that supports the storm water program and voluntarily complies with BMPs. The public needs to be informed on the impacts of storm water discharges, actions they can take to reduce storm water pollution, and how to become involved in the City's storm water program.

A public outreach program should include the following steps:

- Select target audiences for the public education program, including specific entities or groups likely to create significant storm water impacts.
- Target pollutant sources the education program is designed to address.
- Develop an outreach strategy, including mechanisms to reach the specific target audiences. Using a variety of strategies is recommended to provide maximum coverage.

The City may use educational materials (brochures or fact sheets), sponsor speaking engagements before community groups, provide public service announcements, implement educational programs targeted at school age children, and sponsor cleanups. Brochures and fact sheets can be adapted to fit local conditions by modifying materials developed by others. These can then be distributed at City hall, at community events, by direct mailing or as utility bill inserts.

Cleanups and storm drain stenciling are cost-effective activities that can be implemented through the use of volunteer organizations. Storm drain stenciling is an effective method of raising public awareness of the impacts of storm water runoff on water quality. Stenciling neighborhood storm drains reminds car owners not to dump their motor oil down the drain. Storm drain stenciling programs can be started by any local group, such as the Boy Scouts, a school class, or a neighborhood association.

Specific audiences can be targeted for education or training, if the City has identified specific activities or commercial activities as problems. Educational brochures and posters can be developed and distributed which outline specific BMPs that reduce the pollutants from these sources. Examples of pollutant "hotspots" are gas stations or vehicle repair shops.

Measurable goals for evaluating the success of the public education program may include development of various educational materials, completion of storm drain stenciling, and/or elimination of a number of storm water pollutant sources.

6.6.3 Public Involvement

For the public to play an active role in development and implementation of the storm water program, the City must implement public involvement measures. At a minimum, this includes compliance with applicable state and local public notice requirements, which includes public notice of new ordinances or ordinance revisions.

Ideally, the public should be involved in the development, implementation and ongoing review of the storm water program. This will provide for broader public support and a broader base of expertise. A public meeting for program review on an annual basis is recommended.

An annual cleanup week, including incentives such as free pickup and disposal of solid waste, is an effective public participation activity. Community cleanup activities can be enhanced and used as a method for public education and public involvement, through publicity, educational activities, volunteer involvement of schools and community organizations, and specific cleanup activities in public areas.

Other methods for complying with the public involvement requirement include public meetings, volunteer activities (e.g. storm drain stenciling, citizen watch groups), and the use of advisory groups for development of specific program elements.

Stakeholder involvement in advisory groups is important when these interests will be most affected by the program. The use of an advisory group is recommended for development of the construction site control program (Section 6.6.5).

Measurable goals may include public meeting notices, volunteer involvement, establishment of an advisory group, and participation in community clean-up activities.

6.6.4 Illicit Discharge Detection and Elimination

Illicit discharges (non-storm water sources) include both illicit connections and illegal dumping. Illicit connections can be from industrial or business connections that should be connected to sanitary sewer. Other illicit discharges include improper disposal of recreational sewage and failing septic systems.

Some non-storm water discharges may contribute high levels of pollutants to the storm water system. A program to detect and eliminate those illicit discharges identified as significant contributors of pollutants requires the following:

- A storm sewer system map, showing the location of all outfalls and the names and location of all waters of the United States that receive discharges from those outfalls.
- The prohibition through ordinance, or other regulatory mechanism, of non-storm water discharges into the storm sewer system, including appropriate enforcement procedures and actions (such as stop work orders, fines, bonding requirements, permit denials).
- A plan to detect and address illicit discharges, including illegal dumping, into the MS4.
- The education of public employees, businesses, and the general public about the hazards associated with illegal discharges and improper disposal of waste.

A storm sewer system map is being developed as part of this project. This map may be used to help identify priority areas with higher likelihood of illicit connections (e.g. areas with older sewer lines) for detailed screening.

Activities to detect illicit connections include building inspections for new development, routine street and drainage system inspections and maintenance, and ongoing code enforcement activities. Also recommended is periodic outfall screening during wet weather events and dry weather, and documentation of existing operational and discharge activities. Additional procedures may be needed to trace the source of an illicit discharge.

The requirement to control illegal dumping can be addressed through the public education component of the program and through ordinances. The public education program should encourage citizen reporting of illegal dumpers, who can then be fined, or be required to perform community service.

Proper enforcement must be implemented to discourage others from performing illegal dumping. Additional enforcement remedies are needed as well as clear enforcement guidelines. A storm water management ordinance would allow the City to access private property from which an illicit discharge originates, and to establish further prohibitions against unauthorized connections and illegal dumping.

Measurable goals could include implementation of a recycling program for household hazardous waste, completion of public employee training, participation in household hazardous waste collection days, and detection and elimination of a number of illicit discharges.

6.6.5 Construction Site Runoff Control

The Phase II rule requires the City to develop, implement, and enforce a program to reduce pollutants in storm water runoff from construction activities that result in a land disturbance of greater than or equal to one acre. The requirements include:

- An ordinance or other regulatory mechanism requiring the implementation of erosion and sediment controls, and controls for other construction site wastes
- Pre-construction review of construction site plans
- Regular inspections during construction
- Procedures for site inspection and penalties for non-compliance
- Procedures for the receipt and consideration of information submitted by the public

The City of Ontario currently does not have a program that addresses discharges from construction sites. However, the Public Works Department does have existing plan review and site inspection capabilities.

Existing reviewers and inspectors can be cross-trained in this program, or new staff may be hired. Plan review and inspections can require considerable resources, depending on the number of construction projects underway at one time. However, the City is only required to regulate construction sites larger than one acre.

Cost savings may be realized by sharing an inspector with another community if the workload does not warrant a full-time person. Another approach is to require the developer to hire a private inspector in certain circumstances, such as when the development is very large or when the sites under construction present significant management problems.

Establishment of a construction site program will entail development of an ordinance requiring permits, review and approval, plans, design requirements, inspections and enforcement. Compliance with this program can also be achieved through fines, non-monetary penalties (e.g. restoration work), stop work orders, or bonding requirements. Modifying an EPA model ordinance or an ordinance developed by others can expedite this requirement.

Measurable goals could include adoption of a regulatory mechanism, establishment of procedures for information submitted by the public, implementation of site inspection procedures, and reduction of sediment loads.

6.6.6 Post-construction Runoff Control

This minimum measure addresses pollutant reduction in runoff from new development and redevelopment projects that result in the land disturbance of greater than or equal to one acre. The MS4 owner is required to have a plan to implement structural and/or non-structural best management practices (BMPs) and ensure adequate long-term operation and maintenance of such BMPs. The City may implement their own BMPs or require that developers provide control for post-construction runoff.

As part of this project, a Storm Water Management Design Manual has been drafted to implement post-construction runoff control in new development and redevelopment in the City of Ontario. The manual addresses non-structural and structural BMPs, and operation/maintenance responsibilities. The Storm Water Management Design Manual should be implemented, with periodic reviews and updating as needed (see Public Involvement). A draft of this manual is included in Appendix G.

Non-structural controls may also reduce post-construction impacts to water quality. Non-structural methods include directing growth to identified areas; protecting sensitive areas; maintaining and/or increasing open space; providing buffers along sensitive water bodies; minimizing impervious surfaces; and policies or ordinances that encourage infill development in areas with existing infrastructure or higher density. These controls could be incorporated in a subdivision ordinance or comprehensive plan.

Measurable goals may include development of strategies (including structural and non-structural BMPs), codification of strategies through ordinance or other regulatory mechanism, reduction in percentage of new impervious surfaces, and reduction in sediment loads.

6.6.7 Pollution Prevention in Municipal Operations

The Phase II requirements include pollution prevention for municipal operations. Municipal operations include streets, parking lots, parks and open spaces, and storage and vehicle maintenance areas. This program element includes:

- Development of an operation and maintenance program
- Employee training on how to incorporate pollution prevention/good housekeeping techniques into municipal operations

Park operations that may impact the storm water system include application of herbicides and fertilizers to park grounds, maintenance of walkways adjacent to waterways, irrigation practices, and control of aquatic plants in park ponds. Municipal operations implemented by the Public Works Department include annual maintenance of sand and grease traps and catch basins, routing street sweeping, snow and ice control on roadways, and drainage system repair.

Recommended Phase II compliance activities include the development of an Operation and Maintenance (O&M) Plan that describes maintenance activities, maintenance schedules, inspection procedures, and waste disposal practices. The following represent some recommended maintenance frequencies:

1. Street sweeping: establish frequency to limit sediment/debris accumulations to 1 cu. ft. per 1000 sq. ft.

2. Inspection of storm water system

   - Frequency based on sediment accumulated, but at least every 5 years for conveyance system; semi-annually & after storm events for other facilities

   - Keep records of noted conditions & corrective actions

3. Cleaning frequency

   - Conveyance system: when accumulated sediment/trash at 20% of pipe diameter, or if inhibiting facility operation

   - Catch basins: when accumulated sediment/trash blocking 1/3 of pipe diameter

   - Vegetated systems (swales or filter strips): when accumulated sediment exceeds 2” in depth

   - Sand filter: when accumulated sediment exceeds ½” in depth

   - Oil/water separator: when accumulated sediment exceeds 1’ in depth; oil accumulation exceeds 1”

The O&M plan should also include a description of controls for reducing or eliminating the discharge of pollutants from areas such as roads and parking lots, maintenance and storage yards (including salt/sand storage and snow disposal areas), and waste transfer stations.

A record-keeping system to document operation and maintenance activities should also be created to document compliance with both the Phase II and TMDL requirements. Employee training should be provided and could consist of training for new employees on the elements of the Operation and Maintenance Plan, with an annual refresher for all employees. Training materials developed by other entities could be modified for use by the City.

6.7 MAINTENANCE IMPACTS

Since sediments can carry attached pollutant loads, such as nutrients and mercury, reduction of sediment loading can provide multiple water quality benefits. Reduction of solids concentrations may allow Ontario’s storm water discharges to meet the TMDL for both TSS and phosphorus.

Reduction of sediment loads in storm water management areas served by irrigation canals/ditches will depend largely on use of agricultural BMPs (operational or structural). Operational measures may include fertilizer management, erosion control (chemical or mulch), conservation tillage, and reuse of surface runoff. Structural measures such as conservation buffers or sedimentation ponds may be used. Implementation of agricultural BMPs may be encouraged through the public education program, in cooperation with the Malheur Drainage District and Owyhee Ditch Company.

Good housekeeping practices and maintenance activities, such as street sweeping and storm system cleaning, will have a greater effect on pollutant reduction in areas not impacted by agricultural uses. Conventional mechanical broom and vacuum-assisted wet sweepers can reduce annual sediment loading from a residential street by 5 to 30%, while vacuum-assisted dry sweepers may achieve a reduction of 35 to 80%. The percent reduction of nutrients is about half the sediment reduction.

Since irrigation and return flows are by far the largest component of flow in Ontario's storm water system, implementation of agricultural BMPs is expected to have the greatest impact on pollutant loadings to the Snake River.

7.0 – ALTERNATIVES AND SUMMARY IMPROVEMENTS

7.1 EXISTING SYSTEM

The recommendations for improvements to the Ontario storm water collection pipelines and facilities are illustrated in Figures 11, 12, 13, 14, 15, and 16, with summaries shown in Figures 18 and 21 of Appendix A. Several options were discussed with the Technical Review Committee on how to best handle future development. It was the general consensus that the City of Ontario adopt a policy limiting post-development runoff to pre-development conditions for all undeveloped areas. Adopting this policy will prevent conditions from becoming worse in the future. In addition, solutions necessary to correct existing flooding problems will not need to be upsized or substantially expanded as a result of future growth.

Alternative improvements for each basin are summarized below and illustrated in Figures 18, 19, 20 and 21 in Appendix A.

7.1.1 Wal-Mart Drainage Basin (Figure 11)

No recommended improvements at this time.

7.1.2 Kmart Drainage Basin (Figure 11)

The Kmart Drainage Basin does have several areas of surcharging and flooding as illustrated in Figure 10 in Appendix A. These flooding problems are primarily due to the downstream pipes not being large enough to handle the runoff from the basin. In order to correct these flooding problems and minimize the surcharging, Keller Associates recommends that the 15-inch line along the north side of the Ernst Parking lot from the outfall west to the manhole just west of East Lane be replaced with a 24-inch line. In addition the 15-inch line running north and south along East Lane should be upsized to a 24-inch line from the manhole where the 12-inch from the Ernst parking lot joins the north-south line along East lane, north to the east-west line, approximately 340 feet. These improvements are illustrated in Figure 21.

7.1.3 Heinz Frozen Food Drainage Basin (Figure 12)

This basin was initially evaluated and recommendations given based upon a more stringent design standard than was decided upon in later stages of the system evaluation. The design standard presented in Tech Memo #3 was to fix all flooding problems and eliminate surcharging in the lines. The City later agreed upon a design standard of just eliminating all flooding due to the large quantity of surcharged lines located in other parts of the City.

With this new standard the recommendations in Tech Memo #3 were reduced to the following: correct the reverse flow between SE 5th Avenue and SE 4th Avenue along SE 4th Street. However, as future development occurs, the other improvements outlined in Tech Memo #3 will become more imperative depending on future storm water detention/retention practices initiated in the undeveloped area immediately east of HFF.

7.1.4 Double Trunk Line and Park Boulevard Drainage Basin

(Figures 13 and 14)

The Double Trunk line Basin and the Park Boulevard Basin are close enough in proximity to develop a joint improvement plan that will benefit both basins while reducing the overall costs. Three alternatives were developed to help reduce the flooding in these basins and to address the pipe condition issues prevalent in the irrigation line in the Double Trunk line Basin. The improvements included a combination of limiting the inflow from irrigation canals, replacing pipelines and interconnecting the two basins. All three alternatives limit the irrigation inflow to the values shown in Figures 18, 19 and 20 in Appendix A. By limiting or redirecting the inflow, the amount of storm water runoff the pipelines are capable of carrying increases.

All three alternatives include the following improvements:

- Upsize the 12-inch line along SW Alameda Drive from just south of SW 6th Avenue where the irrigation water enters the system, north to SW 4th Avenue with a 15-inch line and limit the inflow at SW 6th Avenue and SW Alameda Drive to 0.33 cfs.

- Replace the 8-inch line along SW 10th Avenue between SW 2nd Street and SW 3rd Street with a 12-inch line

- Redirect the irrigation water south of SW 18th Avenue to flow south into the Malheur Drainage District system

- Limit the irrigation inflow just southeast of the TVCC ballparks to 3.5 cfs.

- Limit the irrigation inflow at the intersection of Alameda Drive and SW 14th Avenue to 1.0 cfs.

- Construct desiltation/sedimentation basins before the irrigation water enters the storm water pipe network at the TVCC ball parks and at SW 8th Street.

- Repair the sewer crossing on NW 2nd Avenue between NW 3rd Street and NW 4th Street.

- Repair the area near Pennington Drive in coordination with development and as the City feels necessary

<u>Alternative A – (Figure 18)</u>
The first alternative, Alternative A, is illustrated in Figure 18 in Appendix A. This alternative interconnects the Park Boulevard system with the irrigation system and replaces the irrigation line with a larger diameter line with greater capacity. The lines highlighted in red are Priority I Improvements to be completed in the immediate future. The main component of this improvement is upsizing the irrigation line from a 24-inch line to a 42-inch line from SW 11th Avenue north to the outfall on the Snake River. There are a few areas of realignment that allow for the storm pipeline to be constructed in the roads and not through private property. The priority II improvements for this alternative include upsizing and replacing the lines in the southern half of the Park Boulevard Basin.

<u>Alternative B – (Figure 19)</u>
Alternative B also combines the Park Boulevard and Double Trunk line Basins, however, rather than upsizing the entire irrigation line, it provides for a detention facility to handle the excess runoff from both basins. The detention facility is located at the City Park along SW 4th Avenue and just east of SW 9th Street. This detention facility would need to be excavated down three to five feet, over an area of 2.3 acres (about 310'x 325'). This alternative allows for both the irrigation line and the Park Boulevard trunk line to discharge into this detention facility. During the 25-year design storm event the water in this facility would reach a depth of less than a foot and a volume of between 84,000 and 87,000 cubic feet. This option also has an inter-tie from the storm pipeline along Park Boulevard to the irrigation line through the TVCC campus. There is also an inter-tie between the irrigation line and the storm water trunk line that runs through downtown Ontario. This option eliminates the need to upsize the entire irrigation line, however, the line downstream of the detention facility will need to be replaced at some time in the future due to the poor condition of the pipe. Figure 19 illustrates the proposed pipe sizes and alignments for this alternative.

<u>Alternative C – (Figure 20)</u>
Alternative C includes an improvement similar to Alternative B, but with no interconnection between the Park Boulevard and Double Trunk line

Basins except at the detention facility at the City park. Without that additional connection the entire line along Park Boulevard from SW 4th Avenue south to the intersection of SW 12th Street and SW 12th Avenue should be upsized to reduce the flooding predicted by the model and minimize the surcharging through the entire line. As with Alternative B, the irrigation line will need to be upsized from south of the TVCC campus to the detention facility and the line downstream of the detention facility should be replaced in the future with a 24-inch line. This Alternative is shown in Figure 20.

The City has not reached a final decision on the preferred alternative. Although Alternative B is the slightly cheaper option, there are other non-monetary issues which the Technical Review Committee feels may make the more expensive Alternative A the preferred alternative. Some of these issues include concerns over using the City Park as a detention facility in part due to the mature trees in the park that would need to be removed. Keller Associates recommends that the improvements outlined in Alternative A be prioritized as shown in Figure 18. In summary these improvements include:

Alternative A: Priority I Improvements:

- Upsize the existing 24-inch irrigation trunk line from SW11th Avenue north to the discharge to the Snake River with a 42-inch trunk line with the alignment re-aligned at the southern and northern ends, as shown in Figure 18

- Interconnect the Park Boulevard trunk line with the new 42-inch trunk line along SW 4th Avenue with a 42-inch and at the TVCC campus with a 24-inch and 42-inch as shown in the Figure 18

- Upsize the 12-inch line along SW Alameda Drive from just south of SW 6th Avenue where the irrigation water enters the system, north to SW 4th Avenue with a 15-inch line and limit the inflow at this sight to 0.33 cfs

- Replace the 8-inch line along SW 10th Avenue between SW 2nd Street and SW 3rd Street with a 12-inch line

- Redirect the irrigation water south of SW 18th Avenue to flow south

- Limit the irrigation inflow at the TVCC ballparks to 3.5 cfs

- Limit the irrigation inflow at the intersection of Alameda Drive and SW 14th Avenue to 1.0 cfs

Alternative A: Priority II Improvements:

- Upsize the 15-inch along SW 12th Avenue from SW 12th Street to SW 11th Street and north along SW 11th Street to just north of SW 11th Avenue

- Construct a 24-inch line north from between SW 11th Avenue and SW 10th Avenue to SW 9th Avenue and east along SW 9th Avenue to Park Boulevard

Alternative A: Future Improvements

- Abandon the existing 12-inch line that runs east from SW 11th Avenue through the cemetery to Park Boulevard and north along Park Boulevard to SW 9th Avenue
- As the pipe condition deteriorates, abandon the existing 24-inch storm line through the TVCC campus from approximately SW 9th Avenue north to SW 4th Avenue

7.1.5 **Stewart Carter Canal and Verde Road Drainage Basins** (Figure 15)

This basin does not appear to have any flooding problems. The only recommended improvement in these two drainage basins is to fix a section of collapsing 30-inch pipe at Verde and NW 4th Avenue.

In the future, additional water quality measures may be needed along the Stewart Carter Canal and the Dork Canal to help minimize the impact on the Snake River from urban storm water runoff. The corridor along the Dork Canal and neighboring ponds should be preserved as wetlands and potential water quality treatment methods.

7.1.6 **NW 7th Avenue, Waterford Subdivision, Dorian Street and SW 4th Avenue Drainage Basins** (Figure 16)

The storm water system in these basins is generally in good condition and there are no flooding problems or significant surcharging issues that need to be remediated. It is recommended that a desiltation basin be constructed to treat the irrigation runoff south of SW 4th Avenue near Dorian Street.

7.1.7 **Miscellaneous Basin Improvements** (Figures 16 and 21)

Improvements not outlined in the individual basin discussion include those for the basins that either have onsite retention or are areas of future growth and development. The River Street Basin, basin O1 as shown in Figure 6, immediately west of the railroad and south of I-84 initially drained through a pipe that went underneath the freeway to discharge into the Snake River. Over time this pipe has become plugged and the ends of the pipe cannot be located. The runoff is now retained onsite in a depression in the NE corner of this basin. It is recommended that on site retention of storm water be continued for this area.

Another basin of interest is the basin encompassing the industrial complexes south of SE 9th Avenue and immediately east of the railroad, designated as basin F10 in Figure 6. This area naturally drains westward towards the railroad. There are no existing storm water facilities in this area and the water is contained in natural depressions. It is recommended that an onsite retention facility be constructed east of SE 1st Street to contain the future storm water runoff from basin F10.

One additional improvement is for the drainage basin west of I-84 and south of Idaho Avenue, basin F9 in Figure 6, including an area that naturally drains to the Kendall Pit. There are existing pipelines that drain east along SE 9th Avenue to an open drainage canal, and another that drains south along SE 10th Street to SE 5th Avenue. Keller Associates recommends that a new 18-inch storm pipe be constructed between SE 5th Avenue and SE 9th Avenue to carry the flow to the Kendall Pit. The existing open channel that drains the storm water south of SE 9th Avenue to Kendall Pit should be preserved as a wetland corridor to provide additional water quality treatment. In addition, the Kendall Pit should be preserved as a wetland.

7.2 MASTER PLAN (Figure 21)

As the City of Ontario continues to grow, more land will become developed, thus creating greater potential for storm water runoff due to the increased impervious land area. The City of Ontario is encouraged to limit post-development runoff to pre-development conditions for future developments. Figure 21 in Appendix A illustrates the Ontario Master Plan with the priority improvements outlined above in red and future improvements in green.

As outlined in Figure 21, the future improvements are shown in green and include mostly future water quality improvements:

- Preserve an area along the Stewart Carter Canal before it enters the City limits for a future desiltation/sedimentation facility

- Work with the Malheur Drainage District to reroute the irrigation tailwater from the land south of SW 18th Avenue to flow south

- Preserve a corridor along the Dork Canal from Verde Street to the Snake River for wetlands and future water treatment

- Preserve ponds located adjacent to the Dork Canal for future water treatment

- Preserve the wetland corridor along the channel north of the fairgrounds to the Snake River

- Preserve the Beck Ponds for future treatment facilities

- Preserve the ponds north of I-84 and Wal-Mart for future treatment facilities

- Preserve the Kendall Pit for future water treatment

- Preserve the wetland corridor running north from Kendall Pit to SE 9th Avenue

- Preserve land east of the city limits for future pretreatment of the Dork Canal

- As pipelines are replaced, replace pipelines less than 12-inch in diameter with at least a 12-inch pipeline

8.0 – SUMMARY OF COSTS AND RATE IMPACTS

8.1 SUMMARY OF COSTS AND RATE IMPACTS

8.1.1 Estimate of Most Probable Cost

Table 1.1 summarizes the improvements and their associated costs. A more detailed estimate of most probable cost is presented in Appendix F.

TABLE 8.1
ESTIMATE OF MOST PROBABLE COST

Priority I Improvements (2003)		**Project Cost***
Double Trunk Line and Park Blvd Basin:	Downtown Improvement-Alternative A	$1,456,200
Double Trunk Line Basin:	TVCC Sedimentation Basin/Structure	$25,000
	Repair Sanitary Sewer Pipe Crossing on NW 2nd Ave	$4,000
	Repair area around Pennington Drive	$15,000
HFF Basin:	Correct reverse grade on SE 4th St. between SE 4th Ave and SE 5th Avenue	$14,000
Verde Street Basin:	Replace 30" Collapsing Pipe Section at Verde and NW 4th Avenue	$3,040
Kmart Basin:	Add Cleanout south of ODOT office on SE Kendall Road	$2,000
	Upsize 15-inch along East Ln and north of Kmart/Ernst Parking	$67,350
Miscellaneous Basins:	Replace 30-ft section of concrete pipe on SE 11th Avenue	$4,375
	Miscellaneous Smaller Sedimentation Basins/Structures	$20,000
	Kendall Pit 18-inch Pipeline from SE 9th Ave to SE 5th Ave	$53,475
	TOTAL PRIORITY I	**$1,664,440**

Priority II Improvements (2010)		
Double Trunk Line and Park Blvd Basin:	Downtown Improvement-Alternative A	$101,770
Double Trunk Line Basin:	Replace 15-inch on NW 4th Ave. between NW 2nd St. and NW 3rd St.	$13,180
	TOTAL PRIORITY II	**$114,950**

Priority III Improvements (2015)		
Double Trunk Line Basin:	Replace 21-inch along SW 11th Ave north of TVCC ballparks	$22,050
Park Blvd Basin:	Replace 24-inch along SW 4th Ave west of Park Blvd.	$37,580
	Replace 8-inch with 12-inch south of Sears Dr.	$6,900
	Replace 12-inch along SW 5th Ave west of SW 12th St.	$8,770
	TOTAL PRIORITY III	**$75,300**
	TOTAL PROJECT COST	**$1,854,690**

*Project Cost includes engineering and contingencies

8.1.2 Total Estimated Annual Costs

Table 1.2 below summarizes the projected annual costs for operation, maintenance, repair, replacement and Priority I bonding. Details of these annual costs are provided in Appendix F. Future storm water fees are projected on the basis of generating annual revenue of $652,262.00 to equal the projected annual costs.

TABLE 8.2
ANNUAL COST SUMMARY

Item	Annual Cost	Running Totals
Sewer line/catch basin cleaning	$76,117	$76,117
Pipeline TV Inspection	21,204	$97,321
Labor Costs	37,000	$134,321
Equipment and Testing	50,000	$184,321
Bond Repayment (4.5% for 20 yrs)	$127,955	$312,276
Misc. Capital Improvements	40,000	352,276
Projected Replacement Costs	$273,286	$652,562
TOTALS	**$652,562**	**$652,562**

8.1.3 Existing Storm Water User Fees

In 1980 the City of Ontario passed a resolution instituting storm drainage user fees, which have been only raised once in 1984 by approximately 5 percent. The initial residential and industrial user rates were developed based on the cost per square foot amount of impervious area. It was assumed that a residential user would have 2,500 square feet of impervious area per lot. This 2,500 square feet of impervious area was assumed to equal one equivalent storm water user. The actual impervious area on industrial properties was divided by this 2,500 square feet area to determine the equivalent storm water user rate for each industrial user. A flat rate of $6.41 per commercial business was established, which was not based on impervious area. There is a significant disparity between the dollar amounts paid by commercial users and the amount of impervious area on their individual lots. Thus there is a significant disparity between the rates paid and the volume of storm water generated on commercial lots. Due to these disparities, a new rate structure is proposed in the next section for the City's consideration.

8.1.4 Potential User Rate Structure and Impact

A new user rate is hereby proposed that will be based completely on impervious area of each user. A summary of the various user categories is shown below in Table 1.3. This table indicates that there are 4,393 residential storm water users and that each user is assumed to have 2,500 square feet of impervious area per lot. For commercial and industrial users, it has been assumed that 85 percent of the commercial area will be impervious and 90 percent of the industrial area will be impervious.

The new impervious area assessment structure could be applied under two separate conditions:

> Condition 1 - All Existing Zoned Area: This condition assumes that storm water assessments will be made to all zoned land within the City whether it is developed or not.
>
> Condition 2 - Existing Developed Area: This condition assumes that storm water assessment will be made only to land within the City that is presently developed.

Table 1.3 indicates the total number of residential lots and commercial and industrial acreage that would be assessed under both conditions.

As can be seen, the major difference would be in the acreage of industrial land being assessed. An industrial acreage reduction of 90 percent from 541 acres for existing zoned area to 54 acres for existing developed area would be seen.

TABLE 8.3
SUMMARY OF STORM WATER USERS AND AREAS

Category	Number of Users	Impervious Area Assumption	Existing Zoned Area	Assumed Percent Developed	Existing Developed Area
Industrial	4393	2,500 sq ft/lot	4,393 lots	100%	4393 lots
Commercial	536	85% of Area	569.9 Acre	70%	399 Acres
Residential	18±	90% of Area	541.4 Acre	10%	54 Acres

General Assumptions:

1) All storm water users will be charged on the basis of impervious area on their property
2) Actual monthly user charges were calculated and are presented in this report on the basis of:
   - Residential – Per Equivalent Dwelling Unit (2,500 sq ft)
   - Commercial – Per 10,00 square feet of commercial property

- Industrial – Per 10,000 square feet of industrial property

Implementing this new user rate philosophy, and assuming that the new user rates would need to generate the projected annual cost of $625,562, the projected storm water user fees are shown below in Table 1.4. Table 1.4 makes a comparison between the 1984 existing storm water rates and the rates that would be charged under both Conditions 1 and 2 as described above. As can be seen from Table 1.4, the residential rates would double if all zoned land were assessed and would almost quadruple if only developed land were assessed. Table 1.4 also illustrates that commercial user charges would increase astronomically, while industrial user charges would basically double for zoned land and quadruple for only developed land. If the total zoned land (condition 1) user rates were assessed to only the developed property, (condition 2) the annual storm water revenues that would be generated would be $326,490 as shown by Note 5 in Table 1.4.

This annual revenue of $326,490 would cover the cost for annual operation, maintenance, and bond costs for Priority I improvements (a cost of $312,276 – see Table 1.2).

TABLE 8.4
PROPOSED MONTHLY USER RATES VERSUS EXISTING RATES

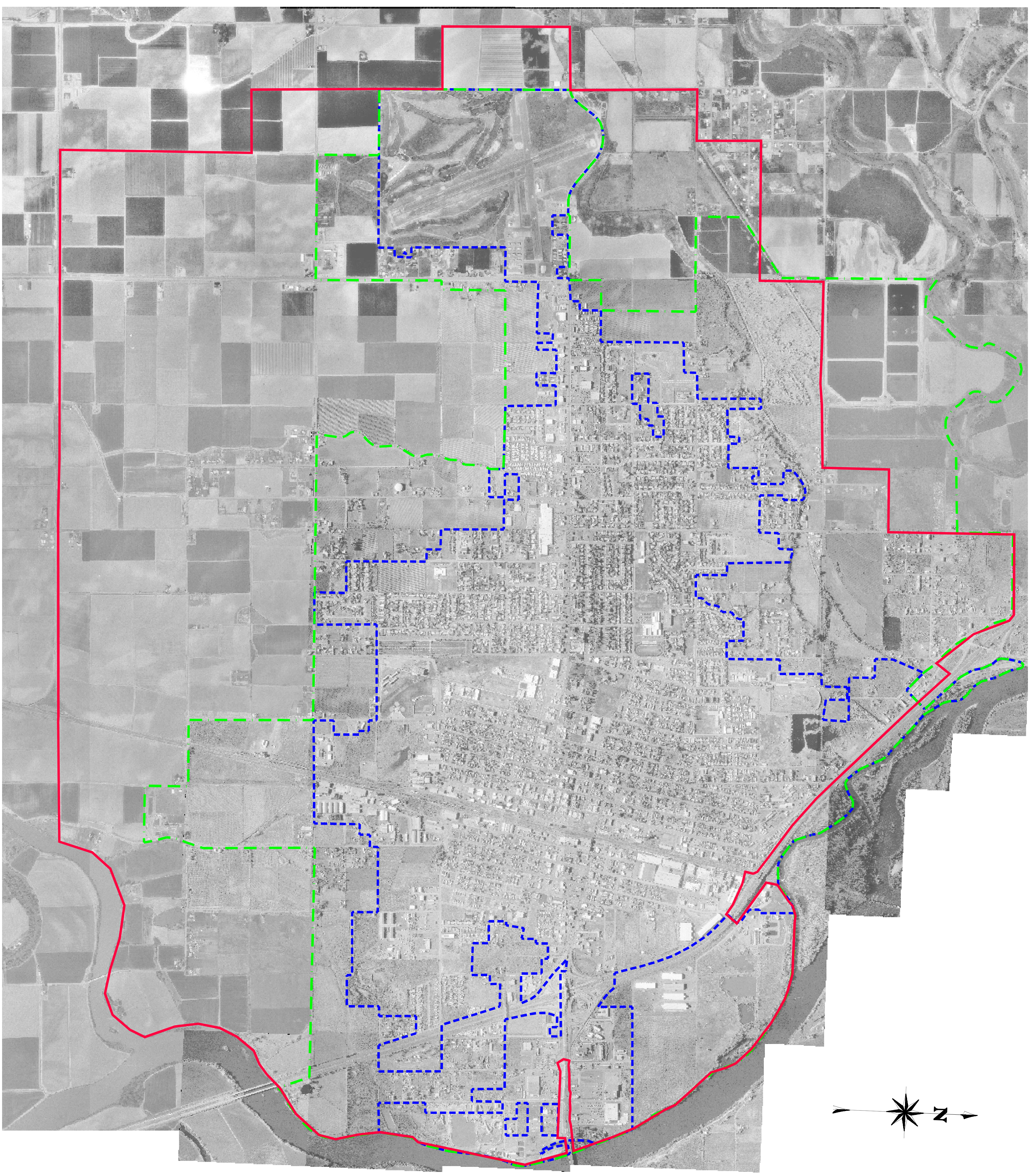
Category	1984	2003 Zoned Land[1]	2003 Developed Land[2]
Residential	$1.16 Per ERU	$2.44 Per ERU	$4.68 Per ERU
Commercial	$6.41 Per Business	$8.31 Per 10,000 sq ft	$15.90 Per 10,000 sq ft
½ acre lot	$6.41	$18.10	$34.63
1 acre lot	$6.41	$36.20	$69.26
5 acre lot	$6.41	$180.99	$346.30
Industrial	$4.18[3] Per 10,000 sq ft	$8.80 Per 10,000 sq ft	$16.83 Per 10,000 sq ft
½ acre lot	$9.10	$19.17	$36.66
1 acre lot	$18.21	$38.33	$73.31
5 acre lot	$91.04	$191.66	$366.56
Annual Revenue	**$113,610**	**$625,562[4]** **$326,490[5]**	**$625,562**

Notes:

1) Assumes that 100% of the land zoned as commercial and industrial within the City of Ontario will be assessed a storm water fee
2) Assumes that only the developed lands within the City of Ontario will be assessed a storm water fee. Assumes that only 10% of the industrial zoned land and 70% of the commercial zoned land is currently developed and, therefore, will be assessed
3) Assumes for the existing rates that existing industrial lots are 90% impervious

Appendix A

Figures

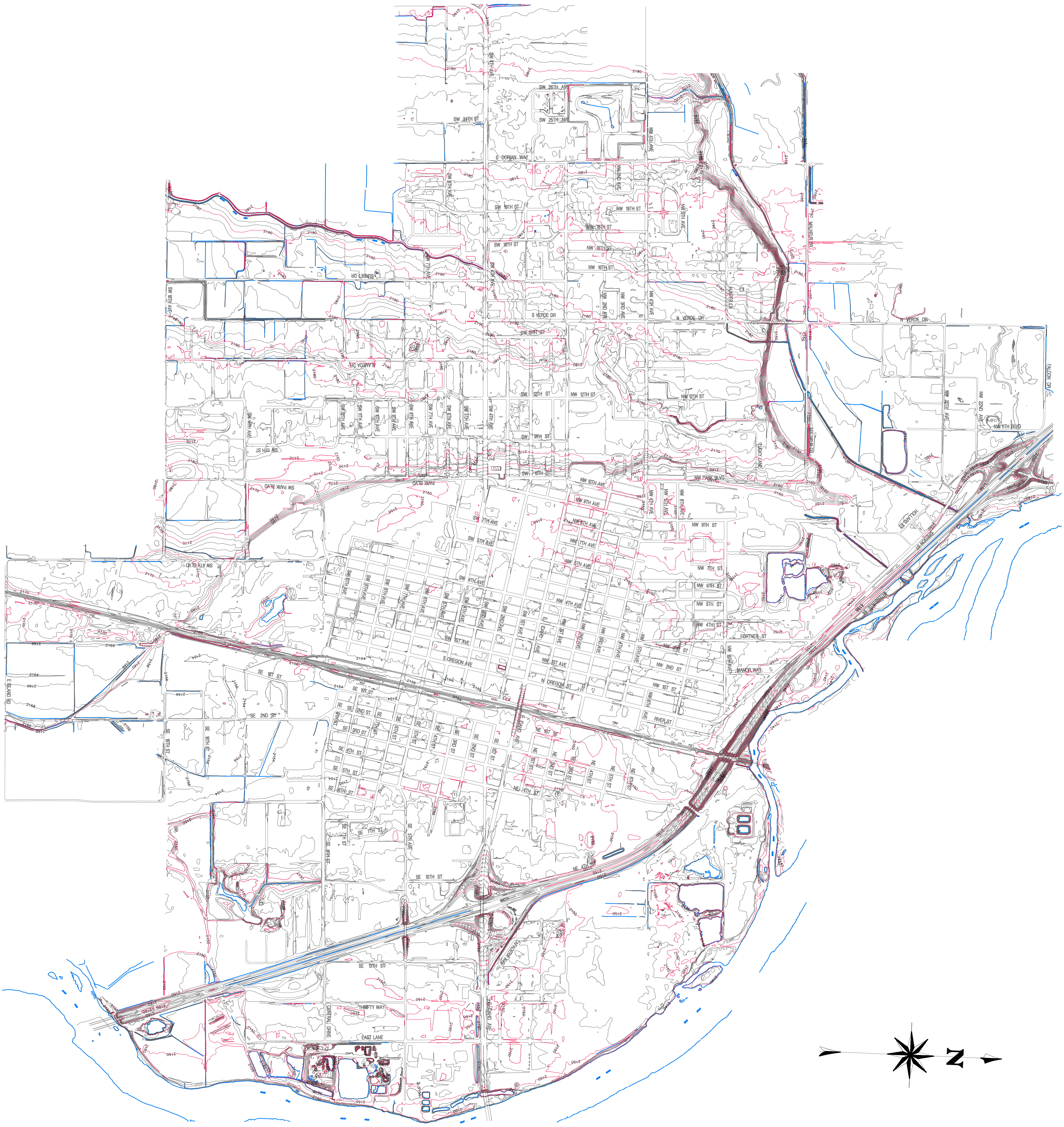


LEGEND

City Limits
Urban Growth Boundary
Study Area Boundary

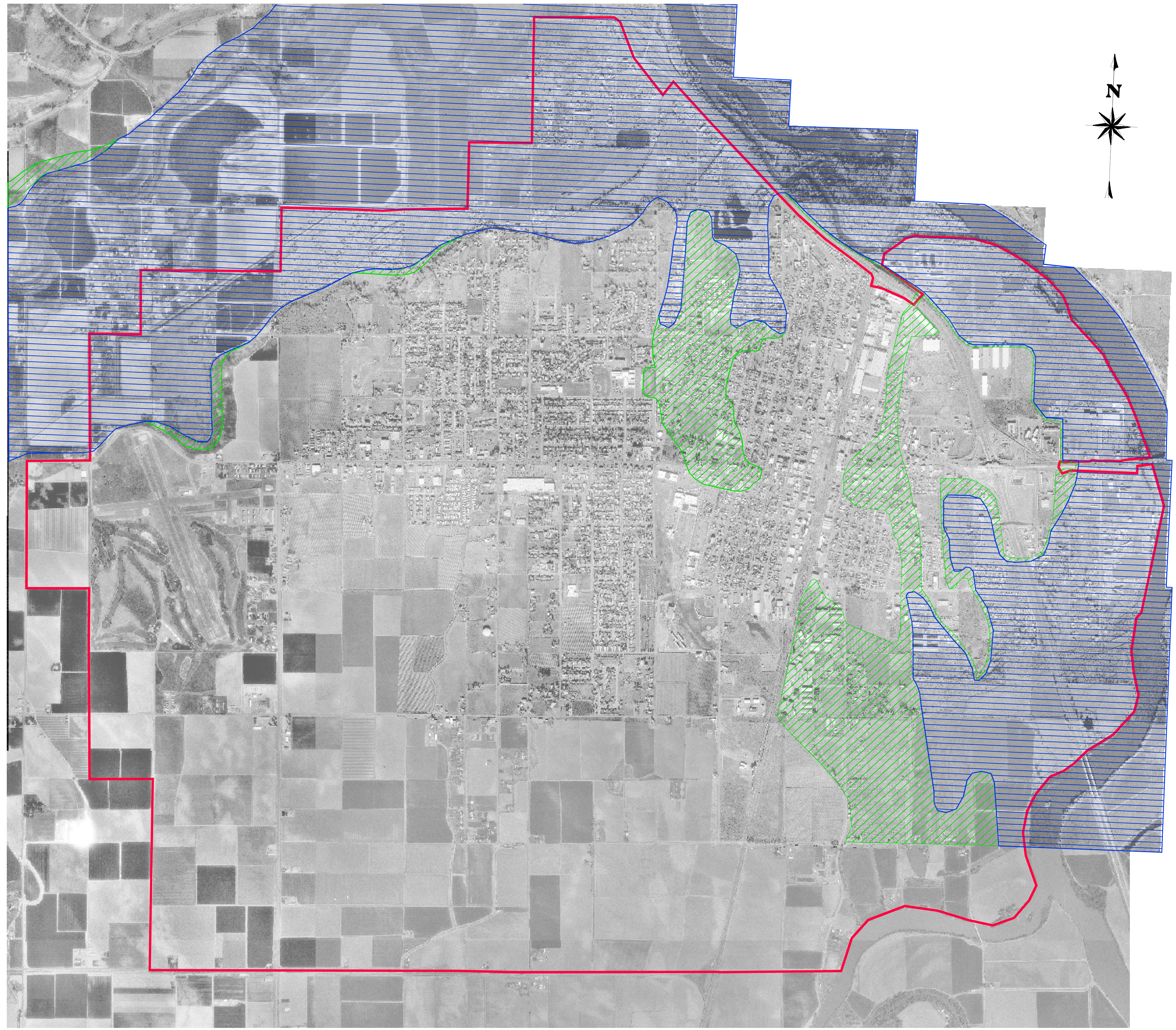
CITY OF
Ontario, Oregon
STUDY AREA

FIGURE NO. 1	ONTARIO STORM WATER MASTER PLAN STUDY AREA	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG01a.dwg



CITY OF
Ontario, Oregon
TOPOGRAPHIC MAP

FIGURE NO. 2	ONTARIO STORM WATER MASTER PLAN TOPOGRAPHIC MAP	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME



CITY OF

Ontario, Oregon

FLOOD PLAIN MAP

LEGEND

Standard Project Flood

100 Year Flood:
Intermediate Regional Flood

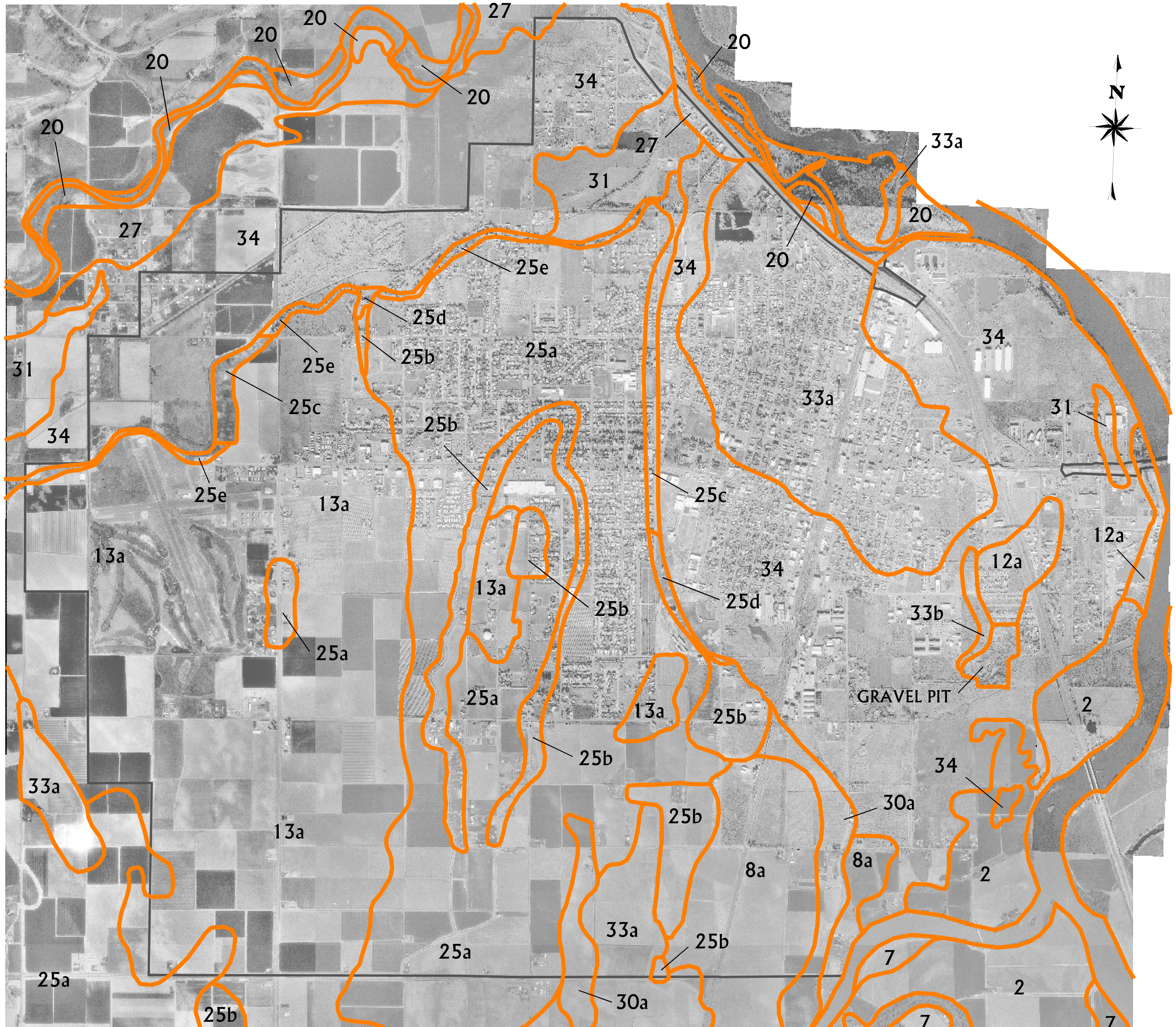
Study Area Boundary

(SOURCE: Flood Plain Information Snake, Malheur Rivers, City of Ontario prepared by the Department of the Army, May 1974)

FIGURE NO. 3	ONTARIO STORM WATER MASTER PLAN FLOOD PLAIN MAP	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG03a.dwg

CITY OF Ontario, Oregon

SOILS MAP



LEGEND

——	Study Area Boundary
2	Baldock Silt Loam
7	Falk Variant Fine Sandy Loam
8a	Feltam Loamy Fine Sand, 0 to 2% slopes
12a	Garbutt Silt Loam, 0 to 2% slopes
13a	Greenleaf Silt Loam, 0 to 2% slopes
20	Notus-Falk Variant Complex
25a	Owyhee Silt Loam, 0 to 2% slopes
25b	Owyhee Silt Loam, 2 to 5% slopes
25c	Owyhee Silt Loam, 5 to 8% slopes
25d	Owyhee Silt Loam, 8 to 12% slopes
25e	Owyhee Silt Loam, 12 to 20% slopes
27	Powder Silt Loam
30a	Sagehill Fine Sandy Loam, 0 to 2% slopes
31	Stanfield Silt Loam
33a	Turbyfill Fine Sandy Loam 0 to 2% slopes
33b	Turbyfill Fine Sandy Loam, 2 to 5% slopes
34	Umapine Silt Loam

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PROJECT NO. 197032
FILENAME 197032FIG04a.dwg

City of Ontario, Oregon

ONTARIO STORM WATER MASTER PLAN

SOILS MAP

FIGURE NO. 4

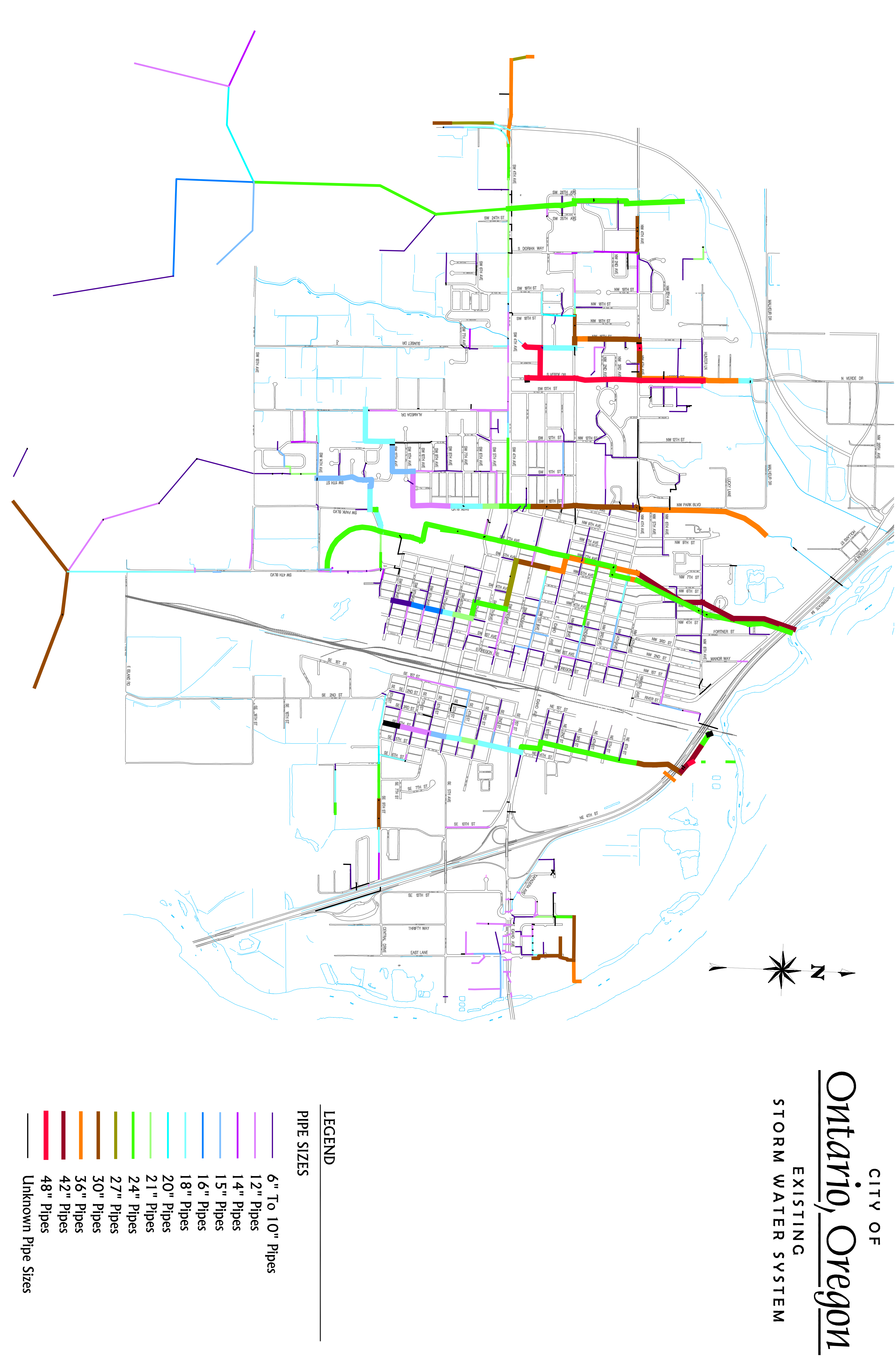
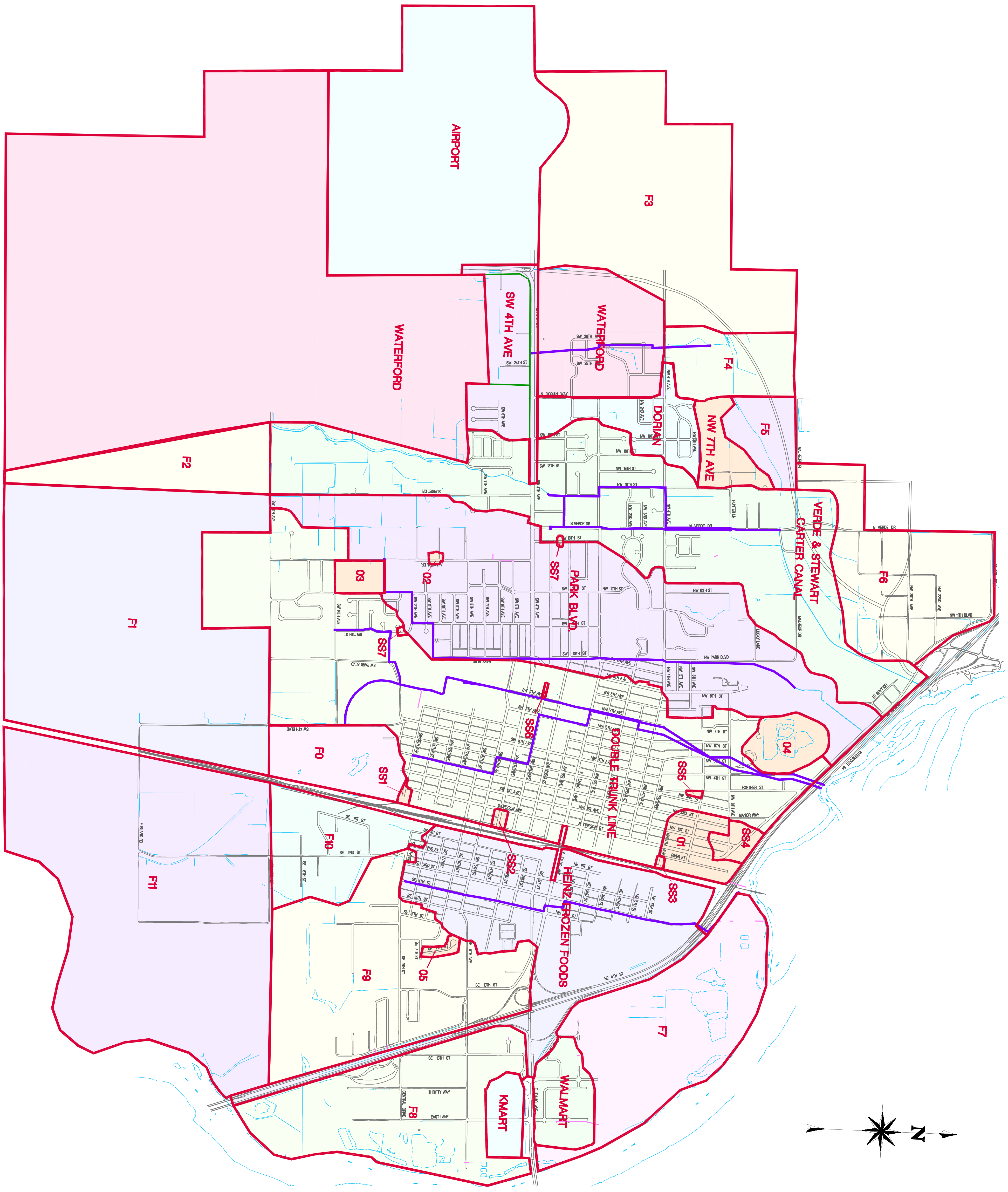


FIGURE NO. 5	ONTARIO STORM WATER MASTER PLAN EXISTING STORM WATER SYSTEM	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG05a.dwg

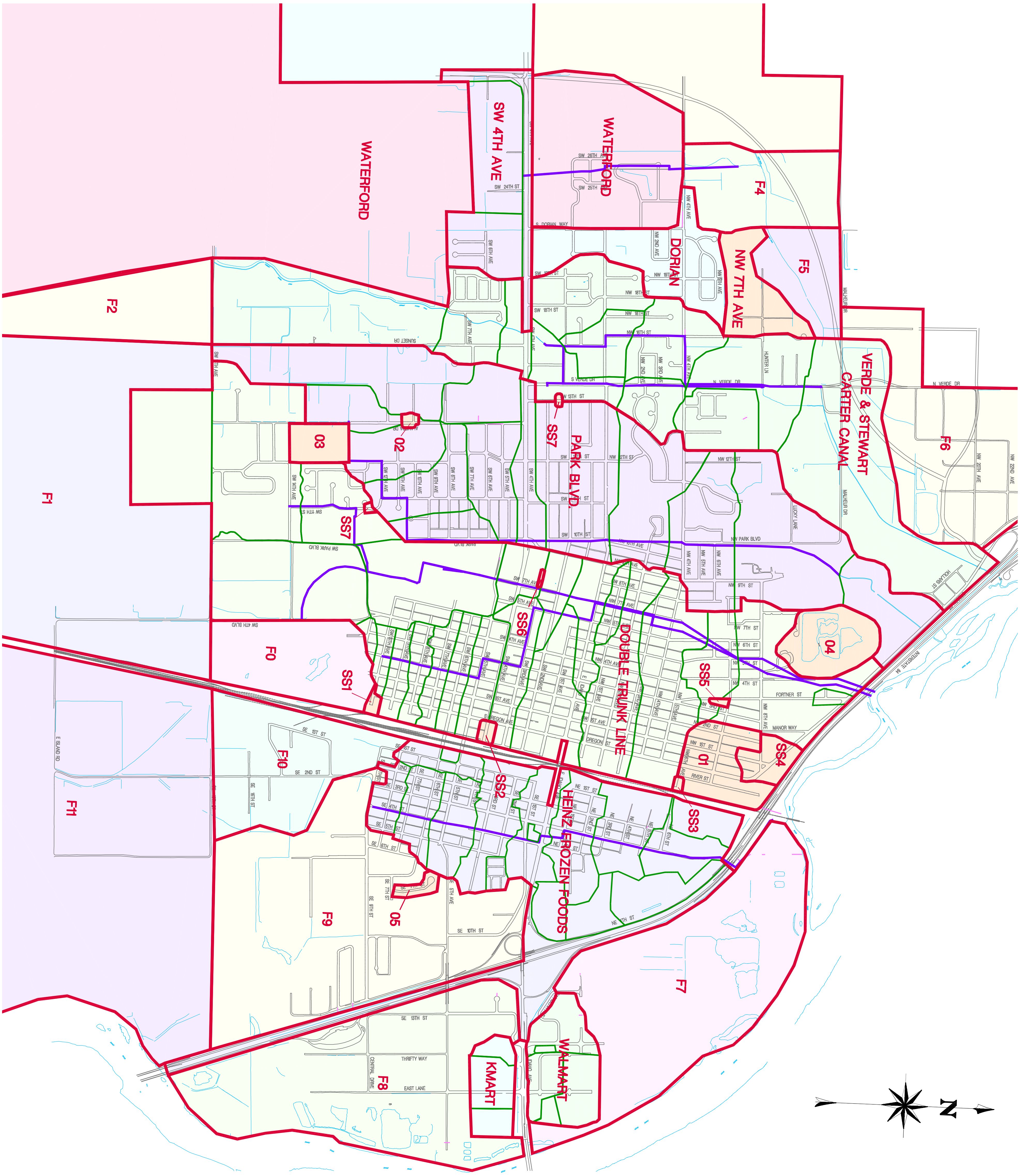


LEGEND

Trunklines

CITY OF
Ontario, Oregon
EXISTING
BASIN DELINEATION

FIGURE NO.	ONTARIO STORM WATER MASTER PLAN EXISTING BASIN DELINEATION	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG06a.dwg
6				



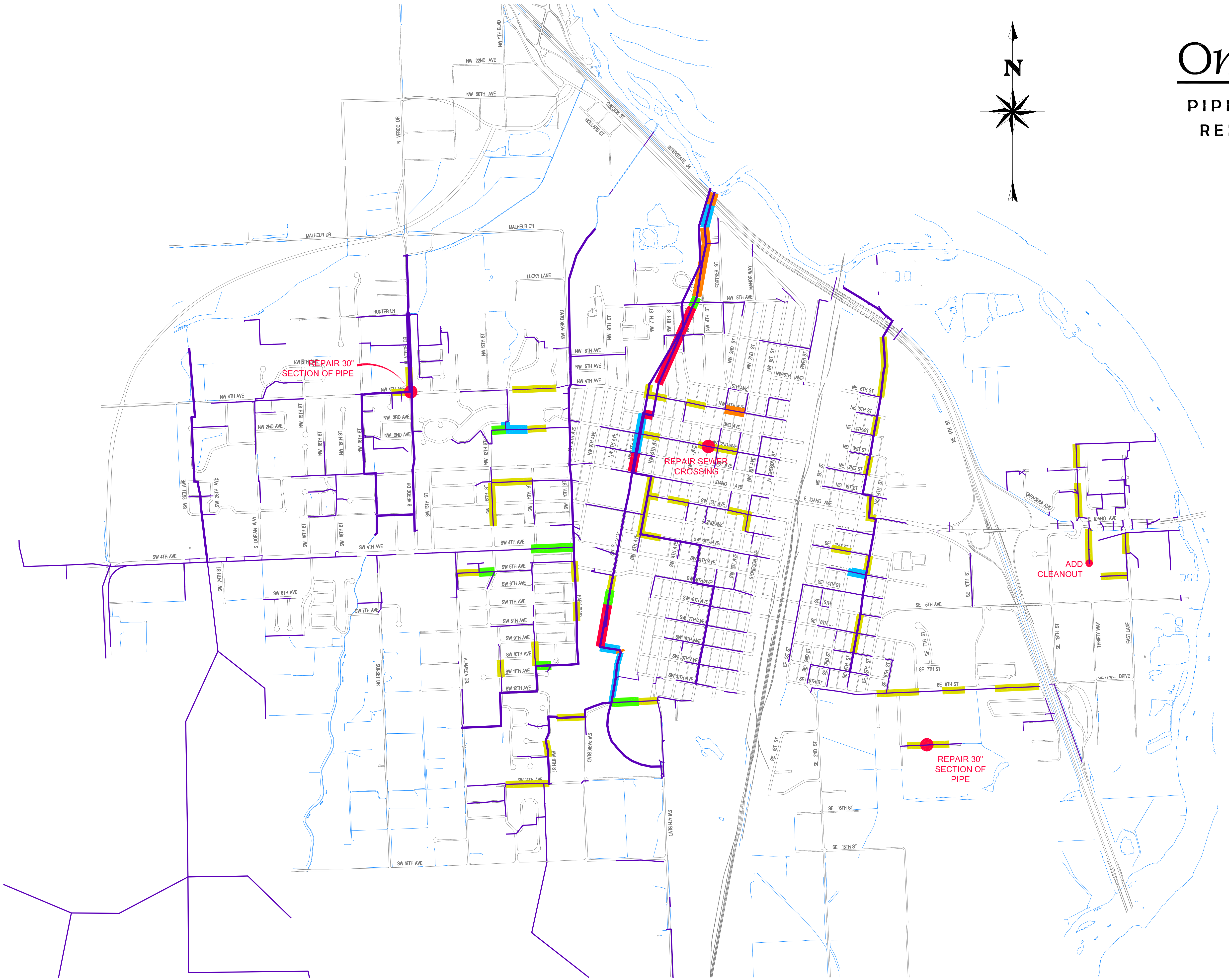
CITY OF
Ontario, Oregon
EXISTING
SUB-BASIN DELINEATION

LEGEND
Trunklines
Sub-basin Delineations

FIGURE NO. 7	ONTARIO STORM WATER MASTER PLAN EXISTING SUB- BASIN DELINEATION	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG07a.dwg

CITY OF Ontario, Oregon

PIPELINE REPLACEMENT & REHABILITATION NEEDS



LEGEND

improvements

- Priority 1 (2003)
- Priority 2 (2010)
- Priority 3 (2015)
- Priority 4 (2020±)
- Pipeline In Fair To Good Condition

NOTE:

Approximately 15% of City's Pipelines Were Video Recorded.

ONTARIO STORM WATER MASTER PLAN

REHABILITATION/REPLACEMENT NEEDS

FIGURE NO. 8

City of Ontario, Oregon

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PROJECT NO. 197032

FILENAME 197032FIG08A.DWG

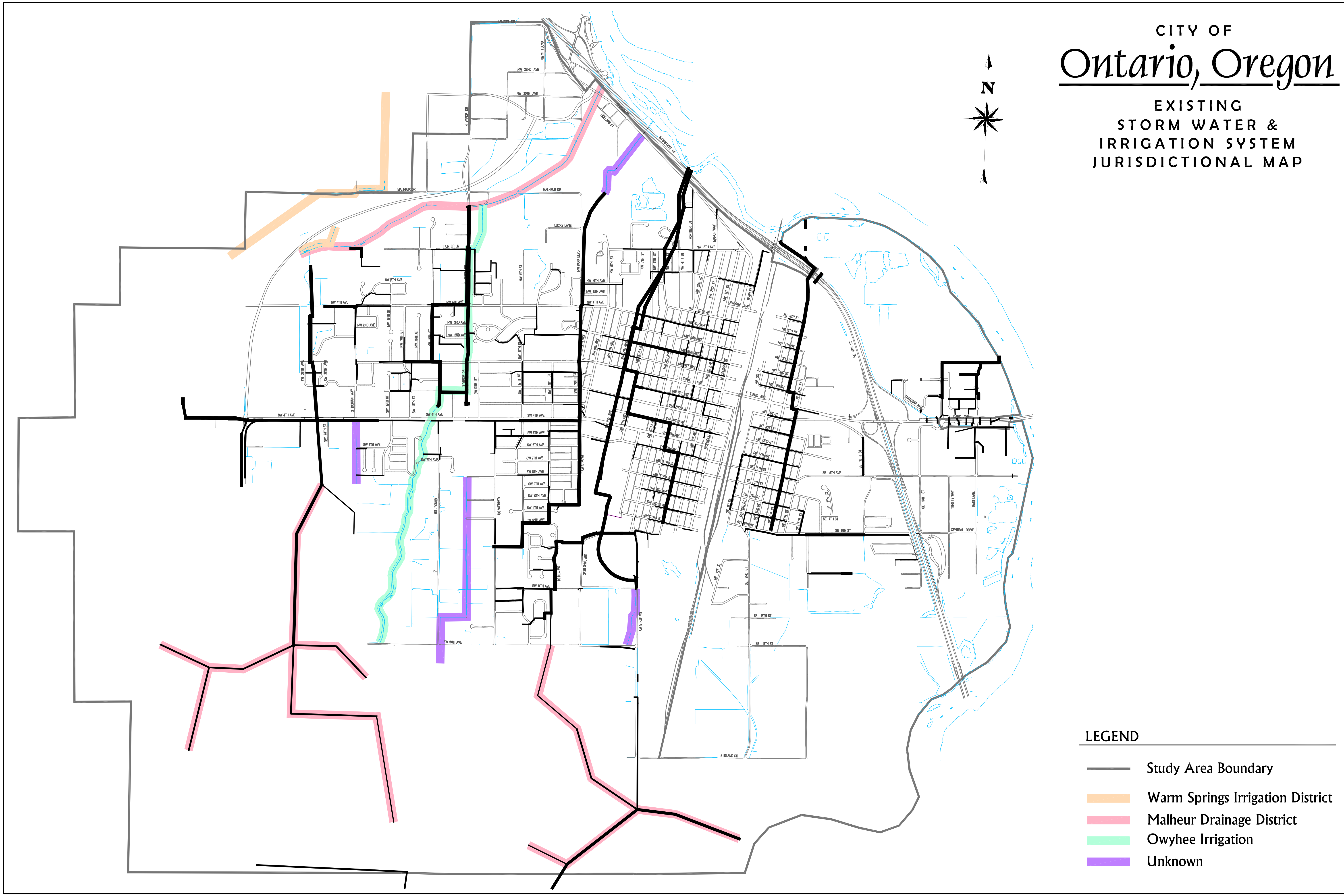


FIGURE NO. 9	ONTARIO STORM WATER MASTER PLAN JURISDICTIONAL MAP	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG09a.dwg

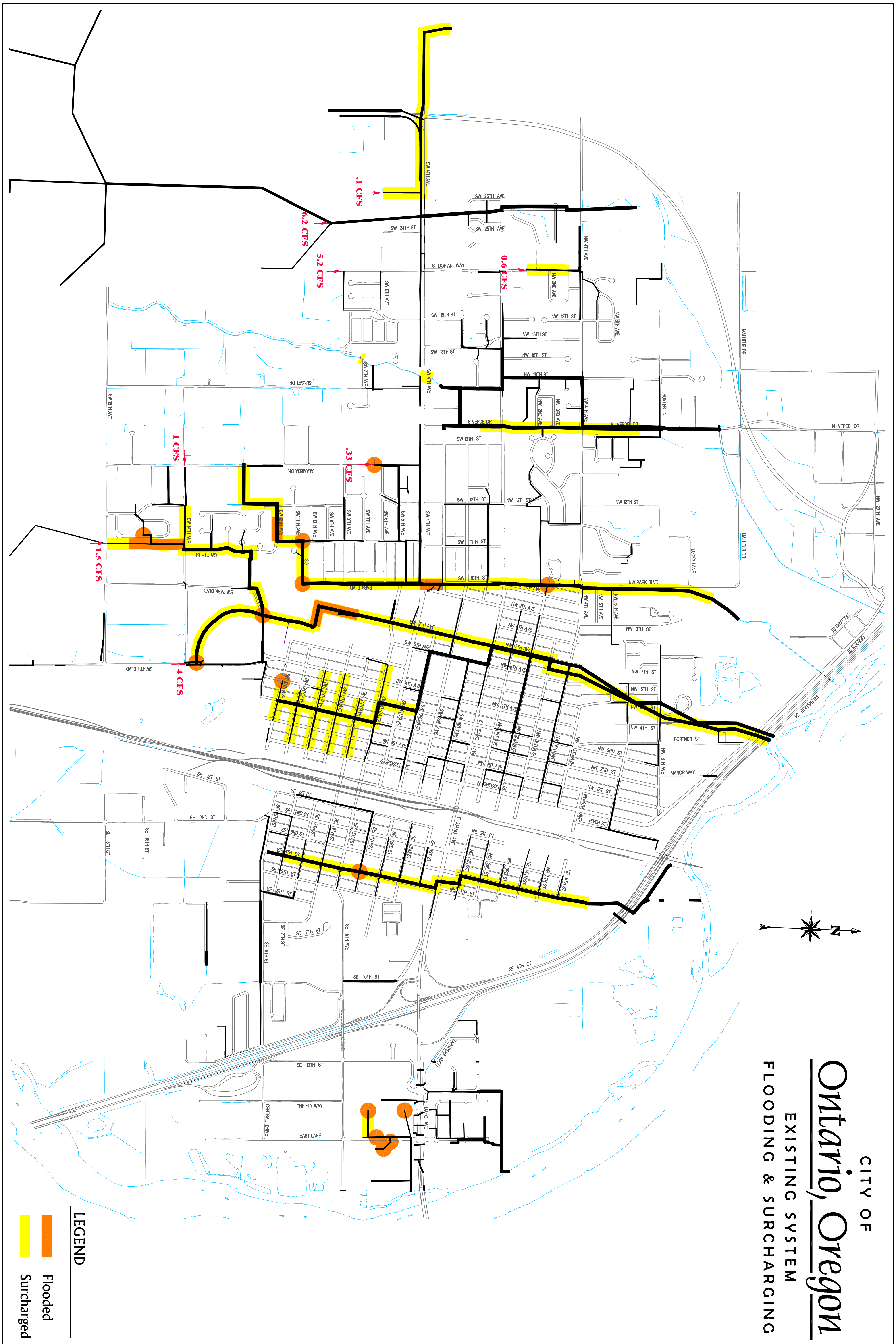
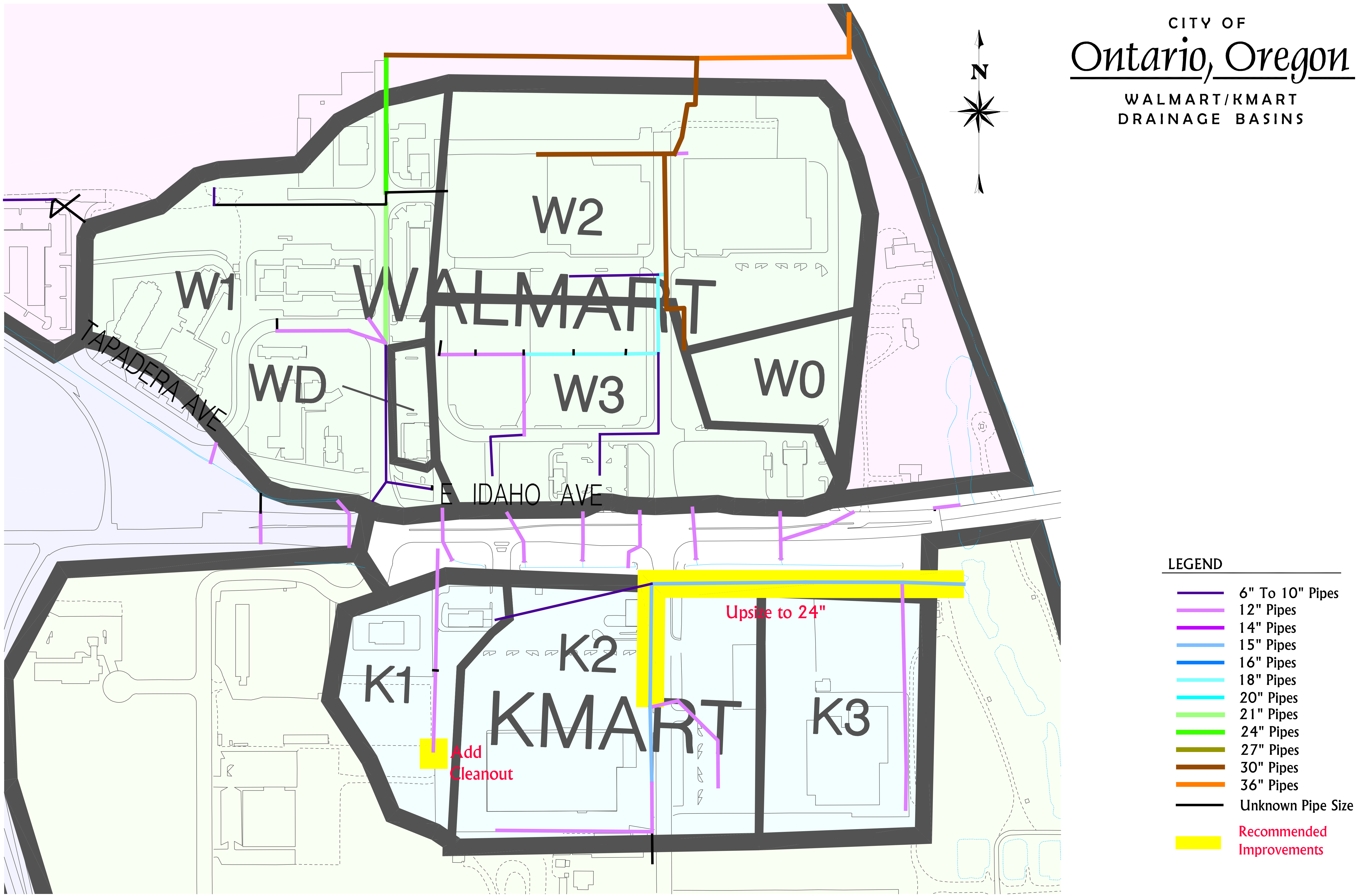


FIGURE NO. 10	ONTARIO STORM WATER MASTER PLAN EXISTING FLOODING & SURCHARGING	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992

PROJECT NO. 197032
FILENAME 197032FIG10a.dwg



CITY OF
Ontario, Oregon
WALMART/KMART DRAINAGE BASINS

PROJECT NO. 197032
FILENAME 197032FIG11a.dwg
KELLER ASSOCIATES
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Meridian, Idaho 83642
(208) 288-1992

City of Ontario, Oregon

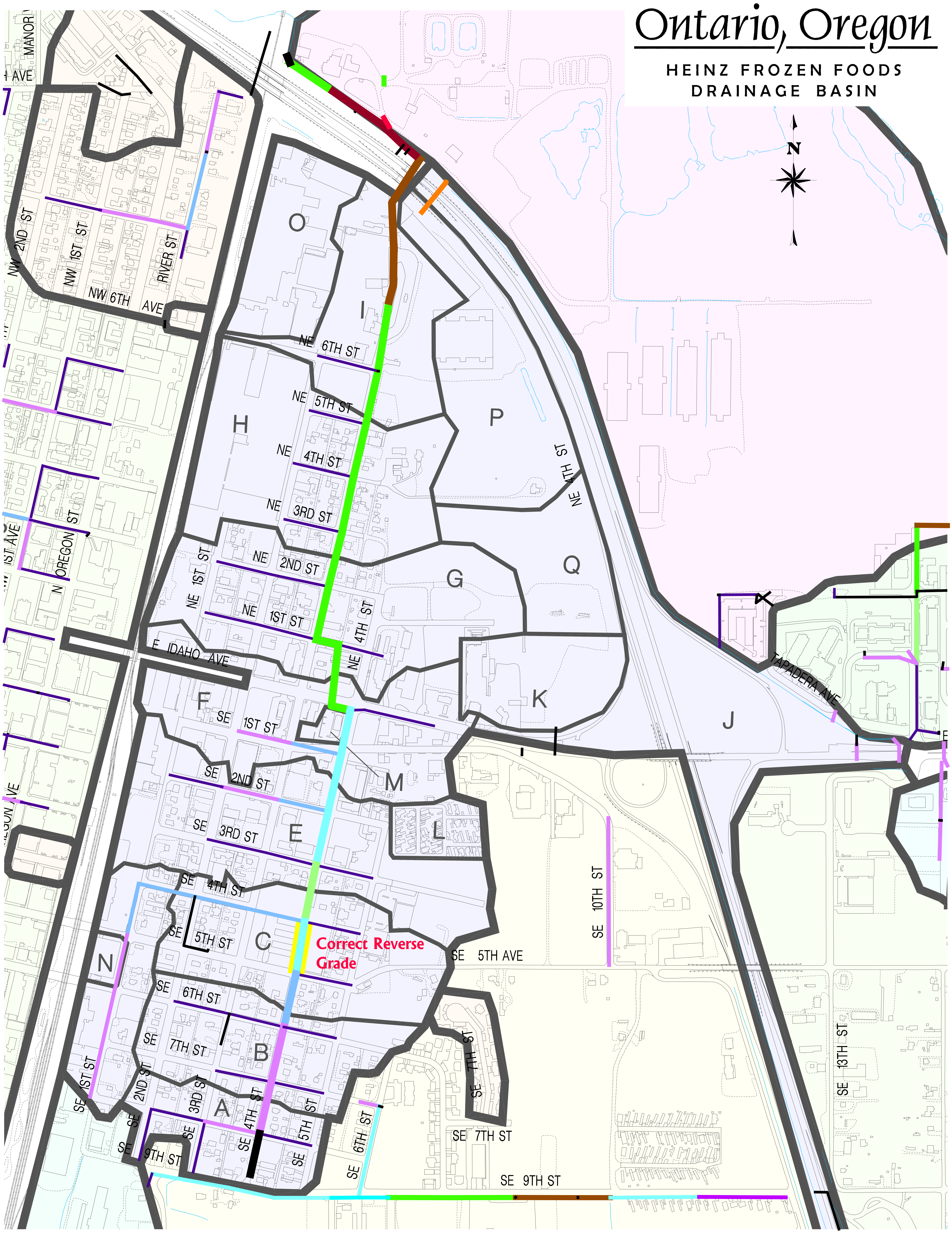
ONTARIO STORM WATER MASTER PLAN
WALMART/KMART DRAINAGE BASINS

FIGURE NO. 11

CITY OF

Ontario, Oregon

HEINZ FROZEN FOODS DRAINAGE BASIN



LEGEND

Unknown Pipe Size	20" Pipes
6" To 10" Pipes	21" Pipes
12" Pipes	24" Pipes
14" Pipes	27" Pipes
15" Pipes	30" Pipes
16" Pipes	36" Pipes
18" Pipes	Recommended Improvements

FIGURE NO. 12

ONTARIO STORM WATER MASTER PLAN
HEINZ FROZEN FOODS DRAINAGE BASIN

City of Ontario, Oregon

KELLER ASSOCIATES
131 SW 5th Avenue, Suite A
Meridian, Idaho 83642
(208) 288-1992

PROJECT NO. 197032
FILENAME 197032FIG12a.dwg

CITY OF

Ontario, Oregon

DOUBLE TRUNKLINE
DRAINAGE BASIN

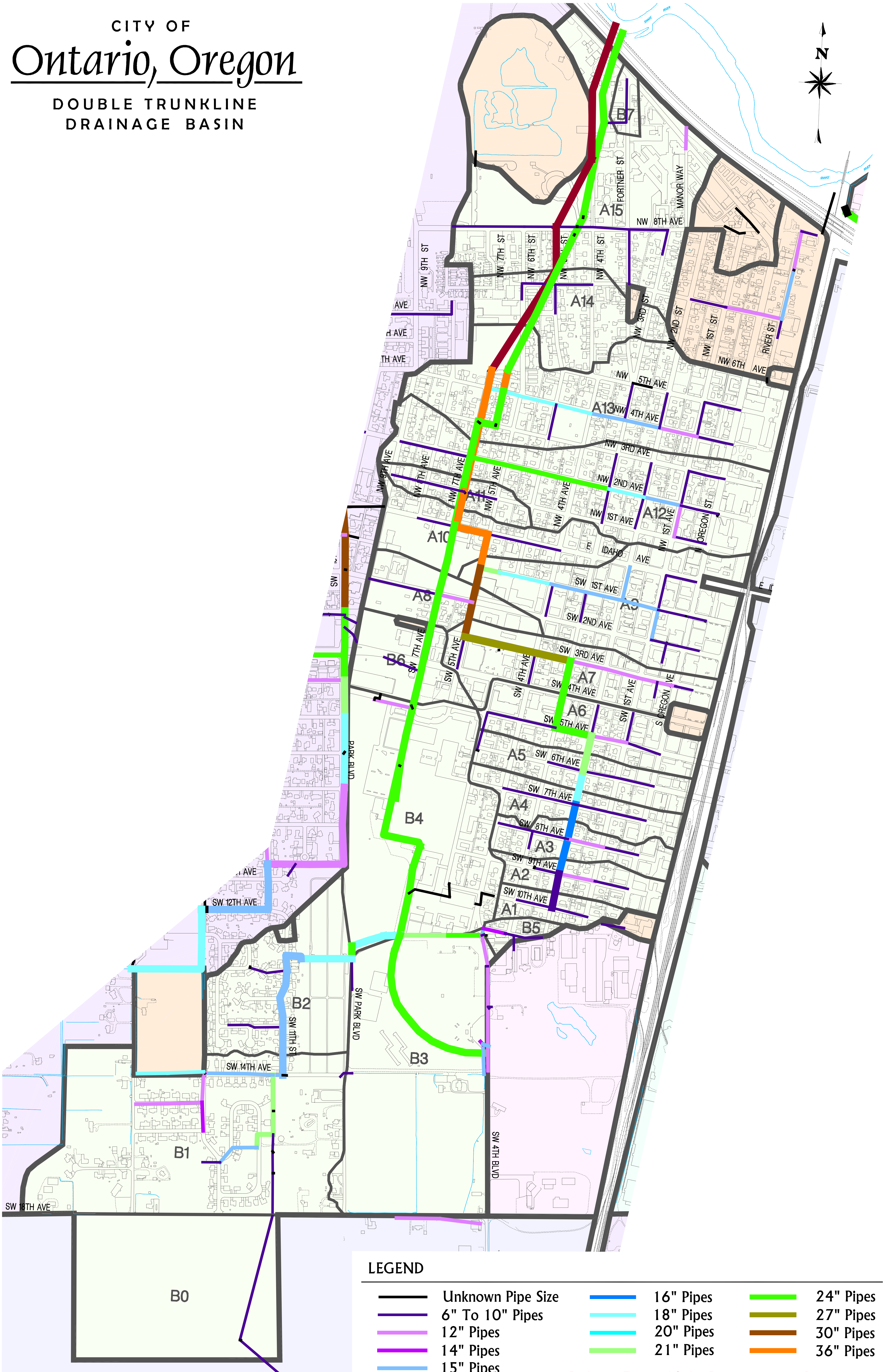
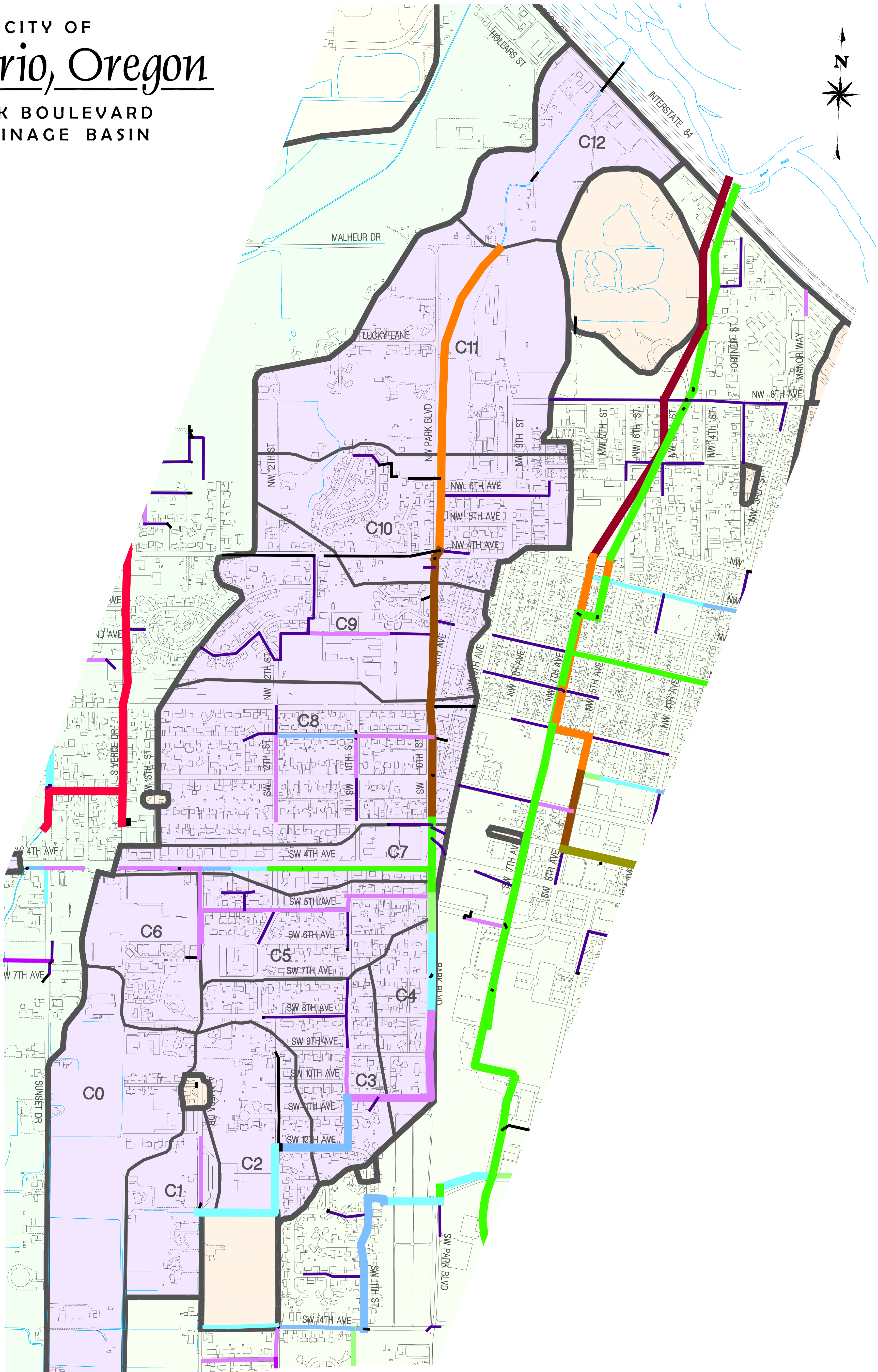


FIGURE NO. 13	ONTARIO STORM WATER MASTER PLAN DOUBLE TRUNK LINE DRAINAGE BASIN	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG13a.dwg

CITY OF

Ontario, Oregon

PARK BOULEVARD DRAINAGE BASIN



LEGEND

Unknown Pipe Size	16" Pipes	24" Pipes
6" To 10" Pipes	18" Pipes	27" Pipes
12" Pipes	20" Pipes	30" Pipes
14" Pipes	21" Pipes	36" Pipes
15" Pipes		

FIGURE NO. 14

ONTARIO STORM WATER MASTER PLAN

PARK BOULEVARD DRAINAGE BASIN

City of Ontario, Oregon

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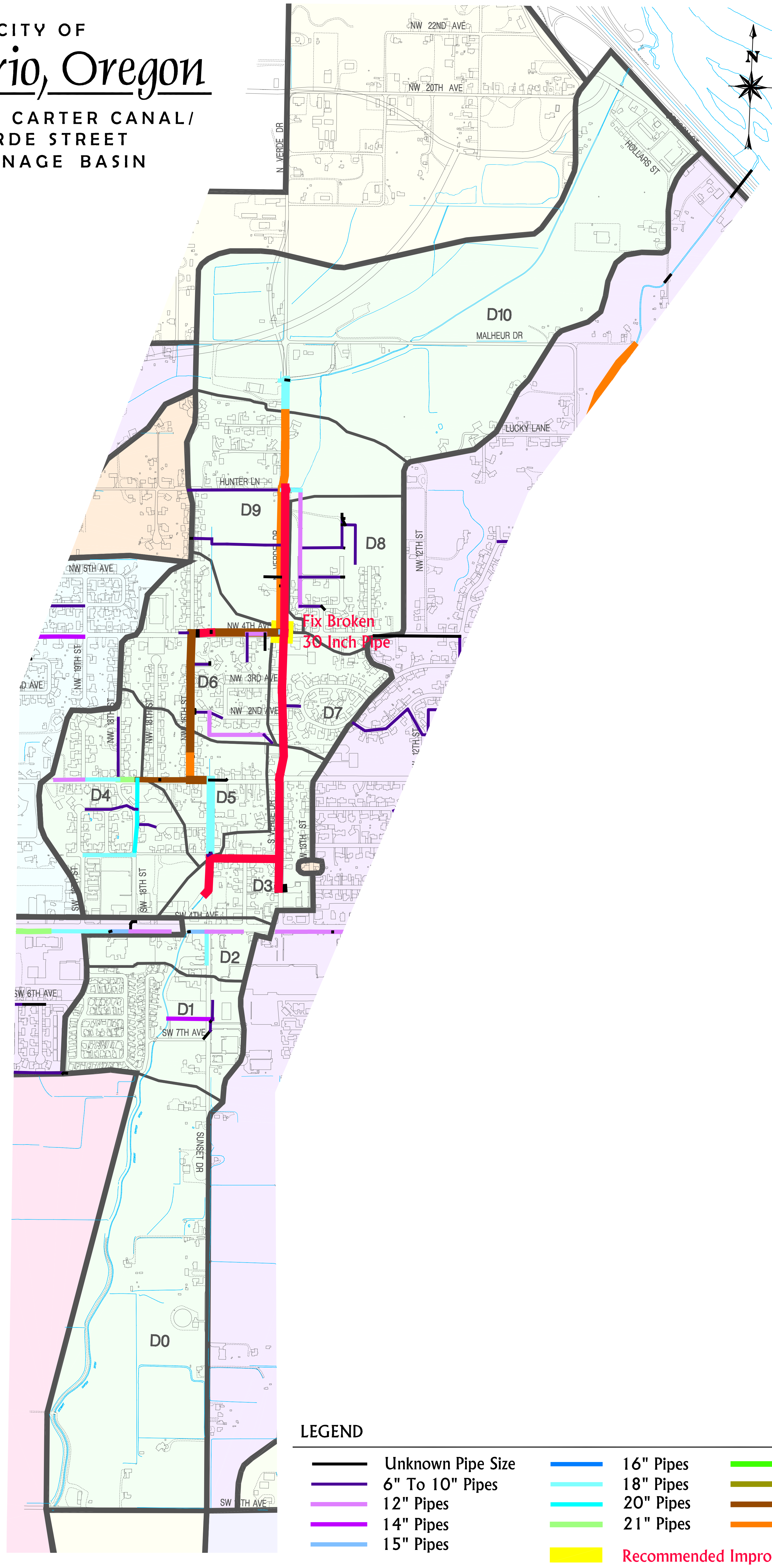
PROJECT NO. 197032

FILENAME 197032FIG14a.dwg

CITY OF

Ontario, Oregon

STEWART CARTER CANAL/
VERDE STREET
DRAINAGE BASIN



LEGEND

Unknown Pipe Size	16" Pipes	24" Pipes
6" To 10" Pipes	18" Pipes	27" Pipes
12" Pipes	20" Pipes	30" Pipes
14" Pipes	21" Pipes	36" Pipes
15" Pipes	Recommended Improvements	

FIGURE NO. 15

ONTARIO STORM WATER MASTER PLAN
STEWART CARTER CANAL/VERDE STREET

City of
Ontario, Oregon

KELLER ASSOCIATES
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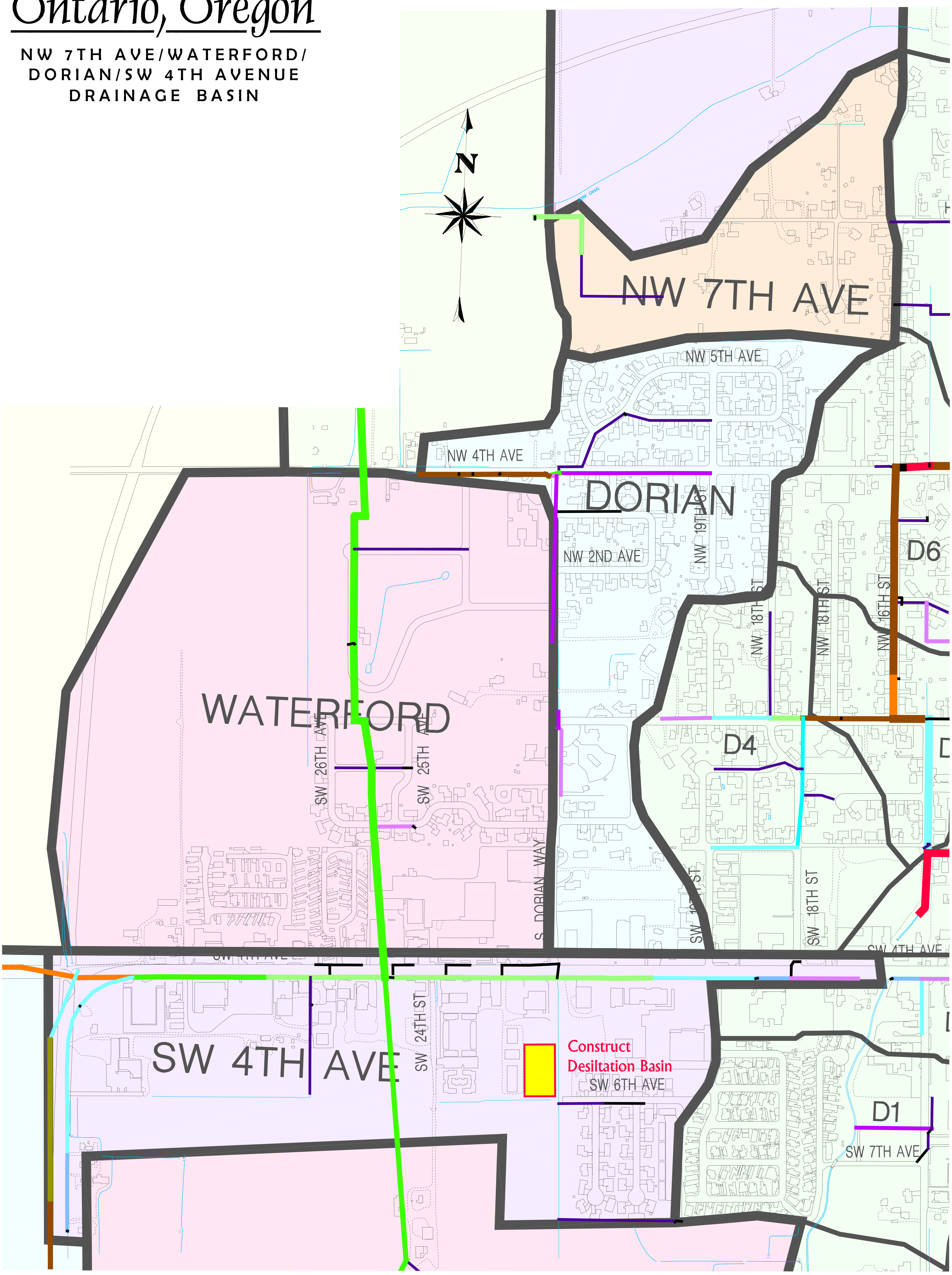
(208) 288-1992

PROJECT NO. 197032
FILENAME 197032FIG15a.dwg

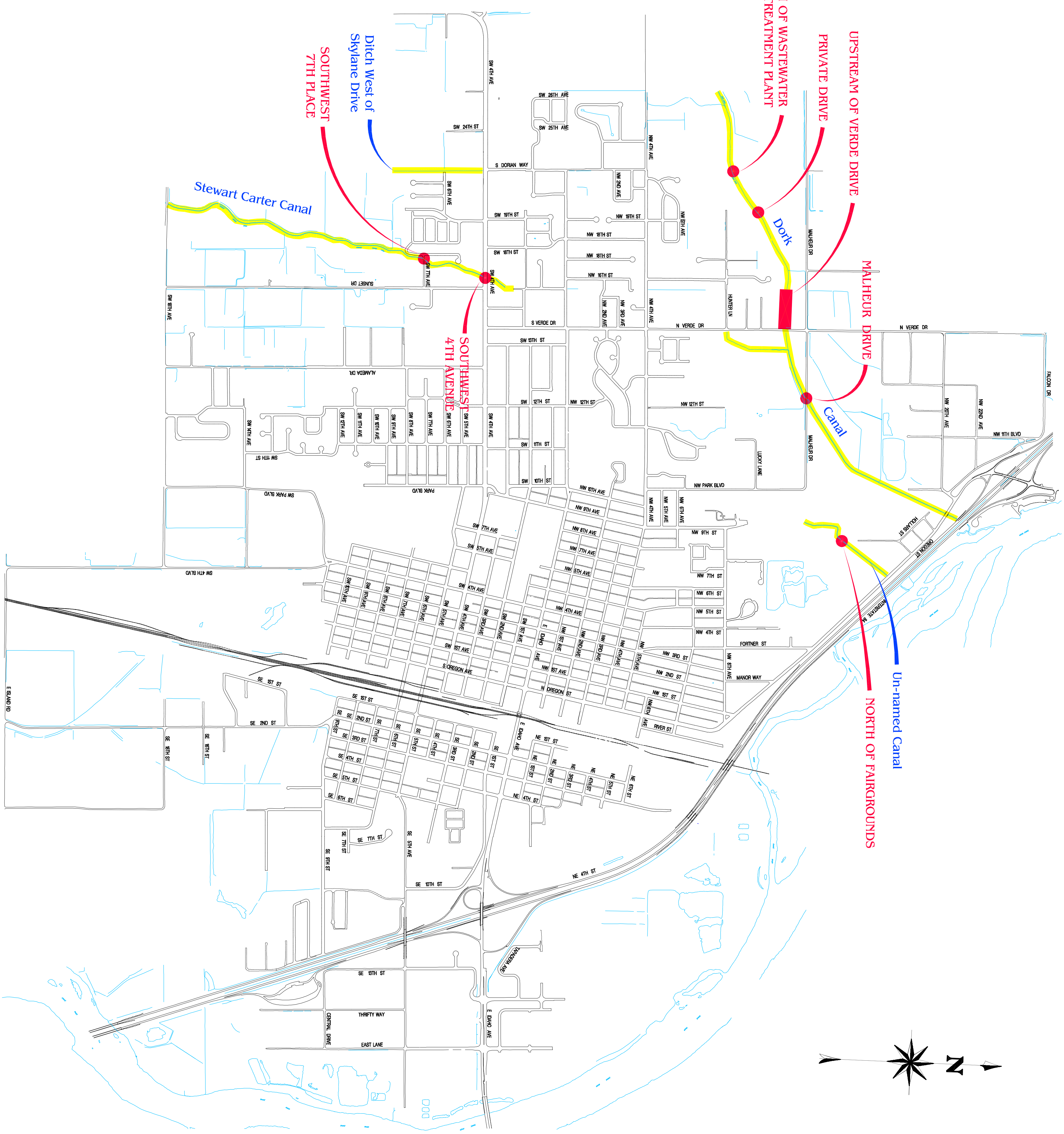
CITY OF
Ontario, Oregon
NW 7TH AVE/WATERFORD/ DORIAN/SW 4TH AVENUE DRAINAGE BASIN



LEGEND

Unknown Pipe Size	16" Pipes	24" Pipes
6" To 10" Pipes	18" Pipes	27" Pipes
12" Pipes	20" Pipes	30" Pipes
14" Pipes	21" Pipes	36" Pipes
15" Pipes	Recommended Improvements	

FIGURE NO. 16	ONTARIO STORM WATER MASTER PLAN MISCELLANEOUS DRAINAGE BASINS	City of Ontario, Oregon	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME 197032FIG16a.dwg



LEGEND

Culverts

Open Channel

CITY OF
Ontario, Oregon
DITCH & CULVERT ANALYSIS LOCATIONS

FIGURE NO. 17

ONTARIO STORM WATER MASTER PLAN

DITCH & CULVERT ANALYSIS LOCATIONS

City of Ontario, Oregon

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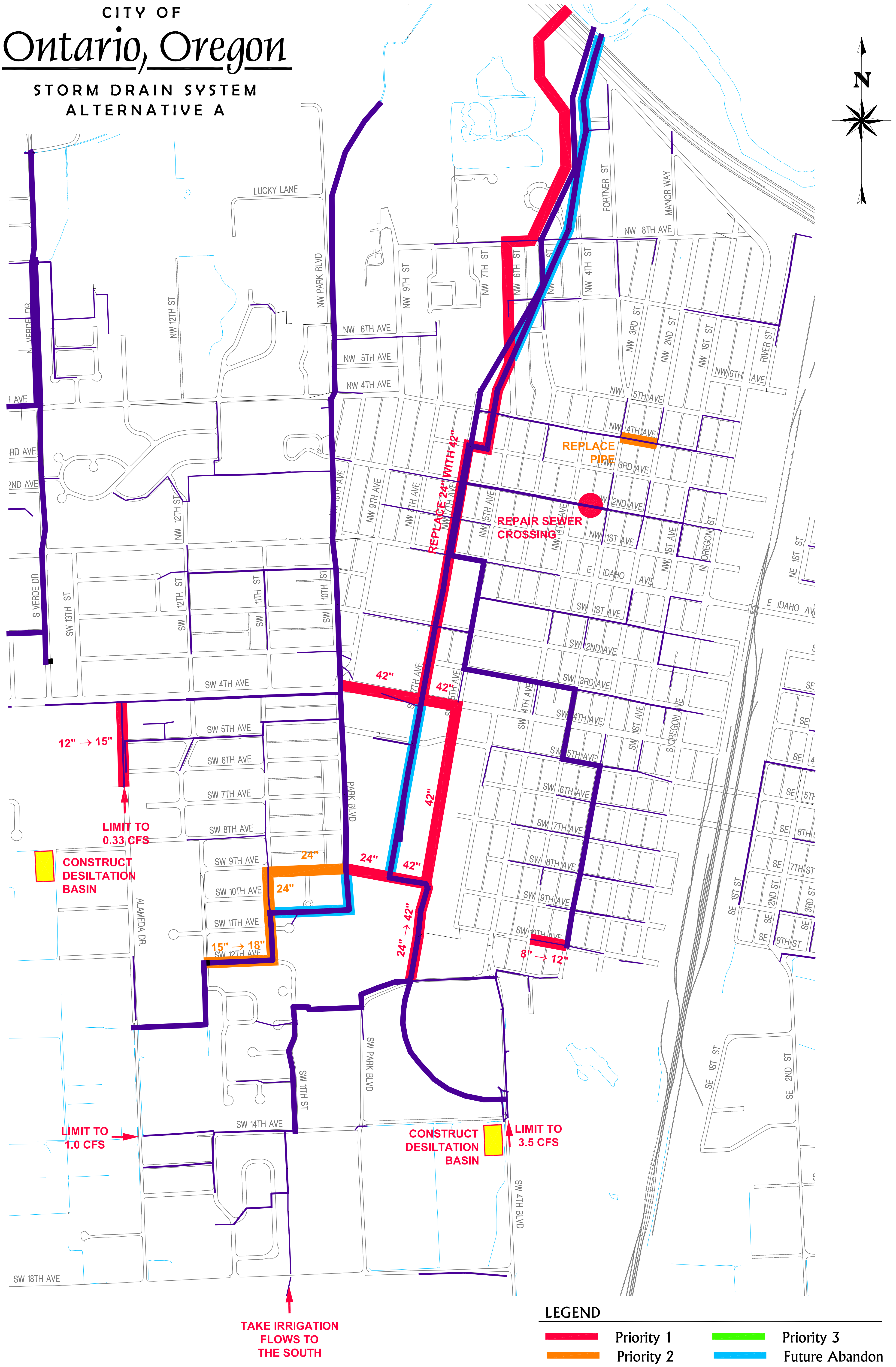
(208) 288-1992

PROJECT NO. 197032

FILENAME 197032FIG17a.dwg



ONTARIO STORM WATER MASTER PLAN

ALTERNATIVE A

FIGURE NO. 18

City of Ontario, Oregon

KELLER ASSOCIATES

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Meridian, Idaho 83642
(208) 288-1992

PROJECT NO. 197032

FILENAME 197032FIG1820.dwg

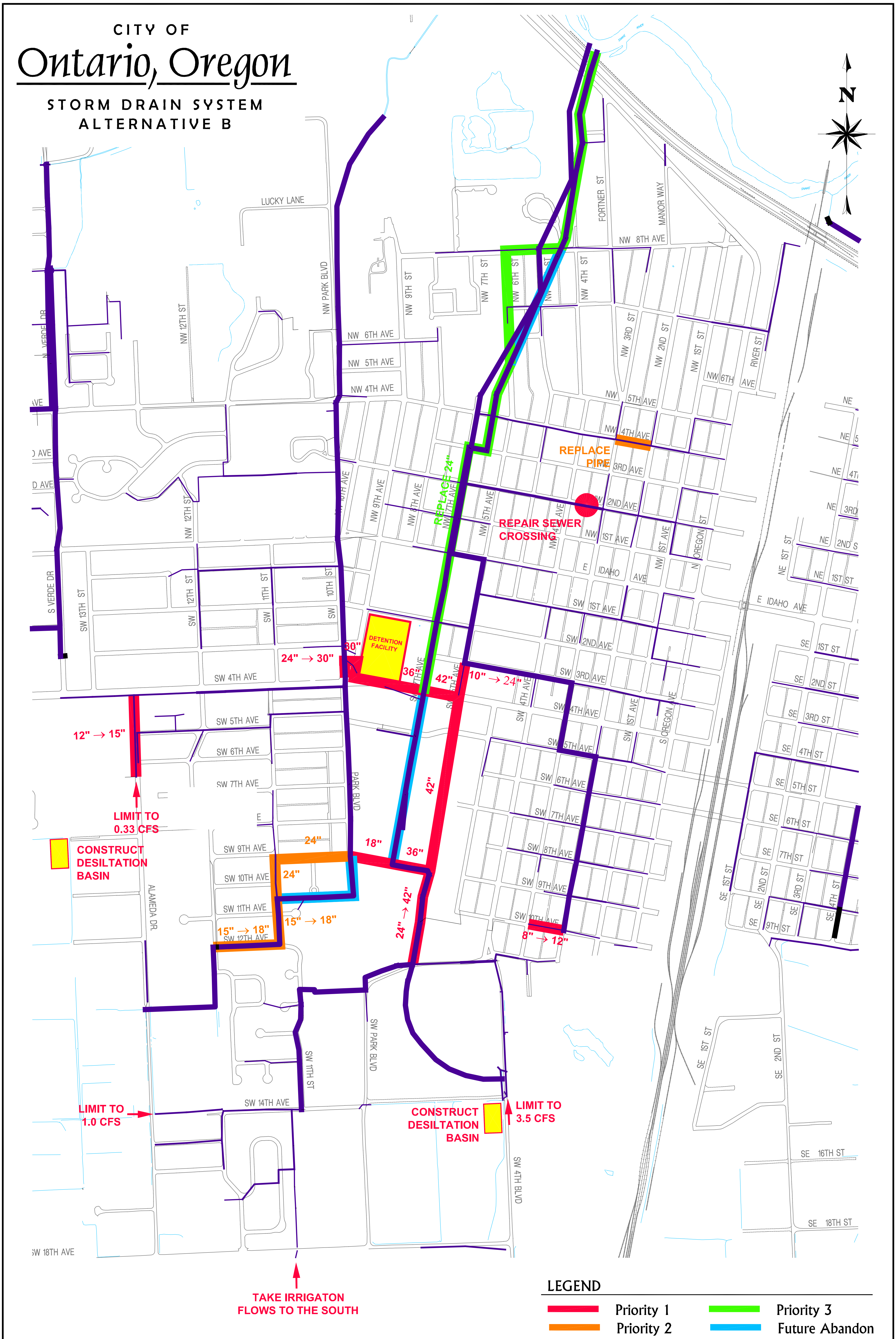

FIGURE NO. 19

ONTARIO STORM WATER MASTER PLAN

ALTERNATIVE B

City of Ontario, Oregon

KELLER ASSOCIATES

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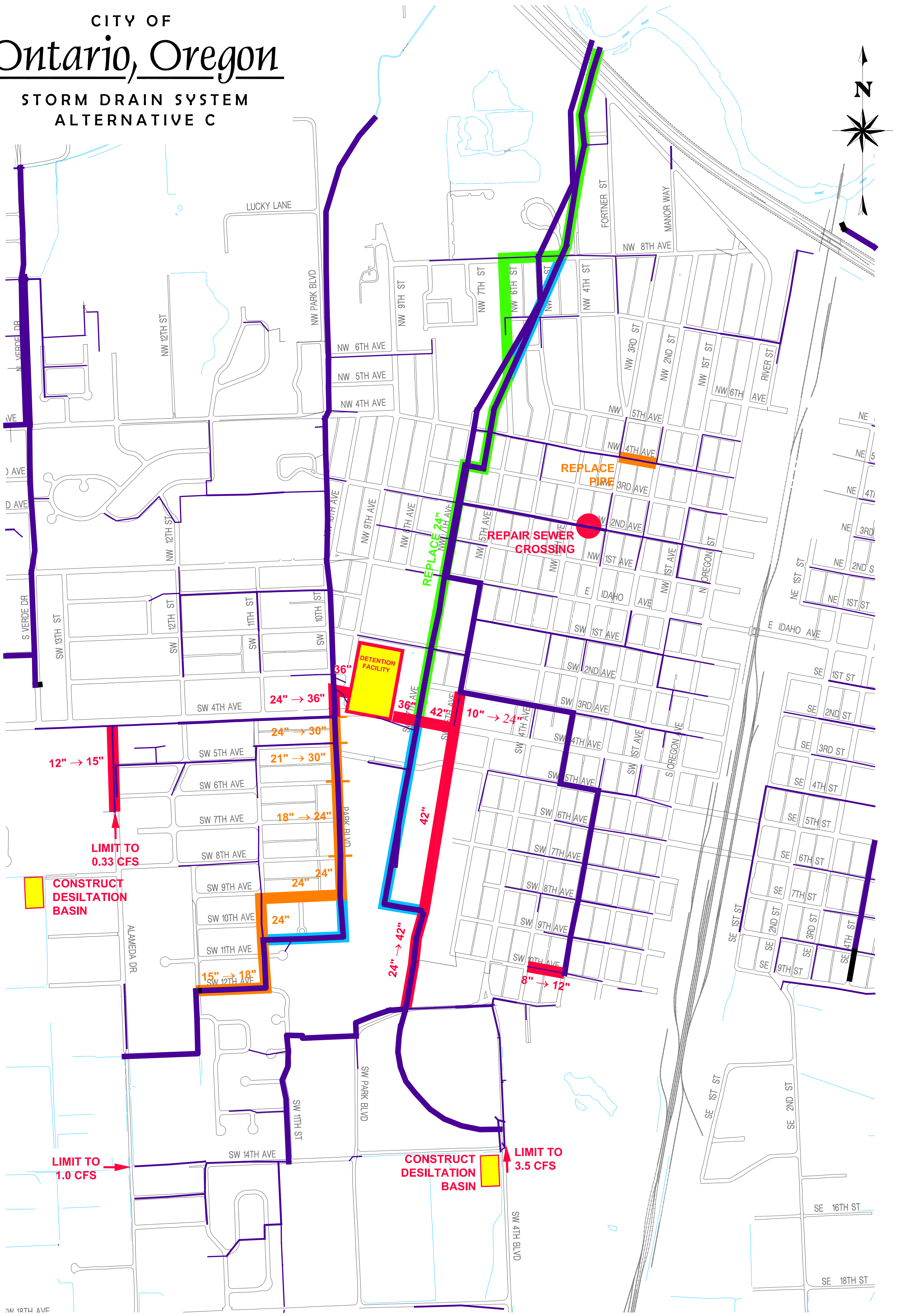
Meridian, Idaho 83642
(208) 288-1992

PROJECT NO. 197032

FILENAME 197032FIG1820.dwg

CITY OF Ontario, Oregon

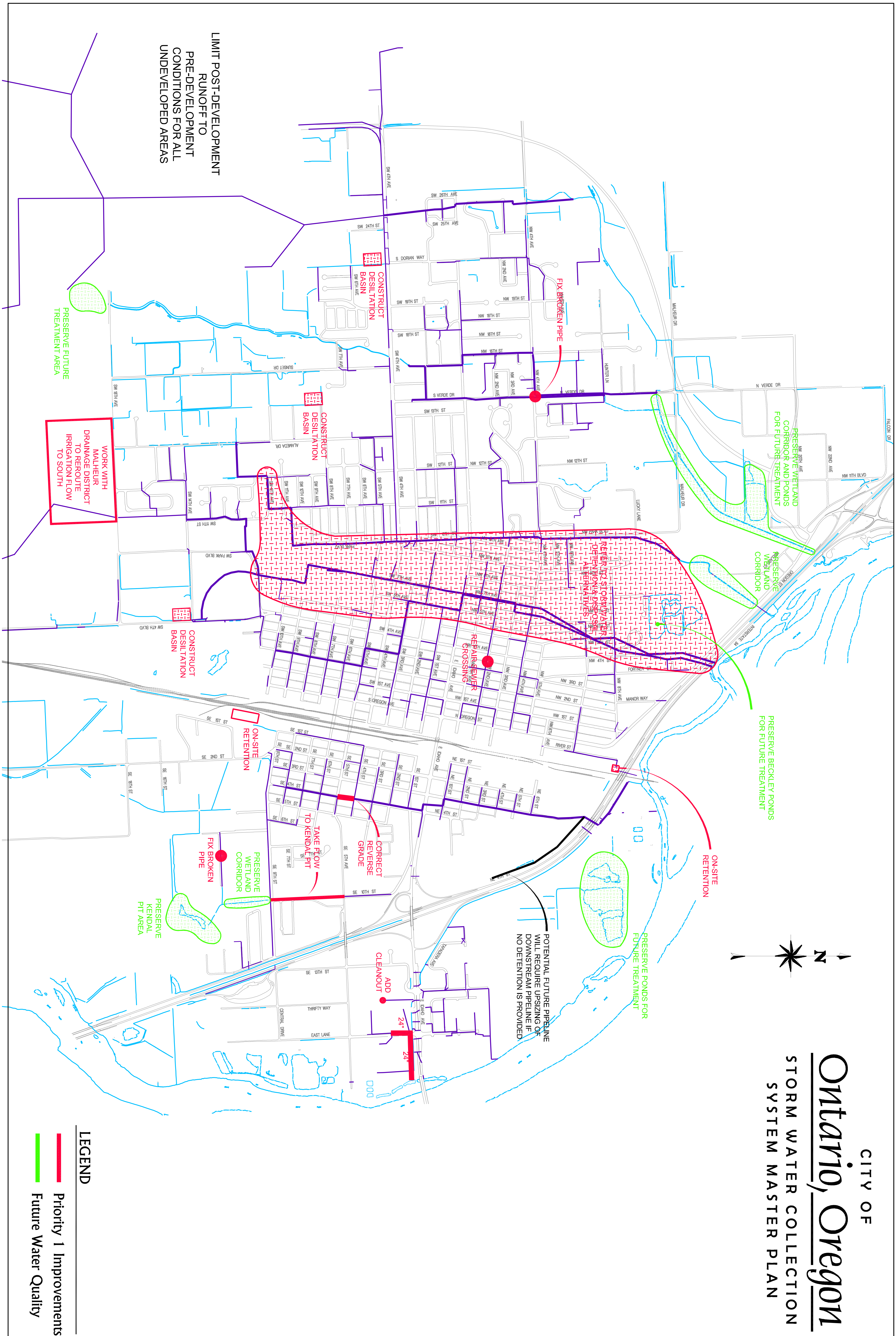
STORM DRAIN SYSTEM ALTERNATIVE C



LEGEND

- Priority 1
- Priority 2
- Priority 3
- Future Abandon

FIGURE NO.	ONTARIO STORM WATER MASTER PLAN / ALTERNATIVE C	City of Ontario, Oregon	KELLER ASSOCIATES, 131 SW 5th Avenue, Suite A, Meridian, Idaho 83642, (208) 288-1992	PROJECT NO. 197032 / FILENAME 197032FIG1820.dwg
20				



ONTARIO STORM WATER MASTER PLAN	City of Ontario, Oregon	KELLER ASSOCIATES — 131 SW 5th Avenue, Suite A, Meridian, Idaho 83642, (208) 288-1992	PROJECT NO. 197032
STORM WATER MASTER PLAN			FILENAME 197032FIG21a.dwg

FIGURE NO. 21

Appendix B

Technical Memorandums

Technical Memorandum #1

To: City of Ontario

From: Susan Burnham, P.E., Keller Associates, Inc.
James Bledsoe, E.I.T., Keller Associates, Inc.

Date: September 20, 2001

Re: Scope Item 4.3 - Special Design Storm Evaluation

Project #: 197032

INTRODUCTION

Part of the stormwater management planning effort included an evaluation of the design storms for the City of Ontario. The evaluation was conducted by applying the design storms to two different theoretical sites. The sites were a 10-acre development assumed to be commercial land, and a 40-acre development assumed to be residential.

Conveyance and storage systems were sized for each of the sites for each of the design storms. Two storage systems were sized for each site, one for detention and one a retention-type basin. Order of magnitude cost estimates for each of the stormwater facilities was prepared.

BACKGROUND

A set of design storms was selected by the consultant after a thorough review of available data. The design storms were constructed from 50 years of precipitation data and methods provided by the NOAA Atlas 2. Real data was compared to the results of the analysis to confirm that the computed storms were realistic for the Ontario area. The total rainfall amounts calculated were comparable to actual values.

The design storms are defined by Intensity-Duration-Frequency (IDF) Curves. Table A is a summary of the data defining the IDF Curves for the Ontario design storms.

Each of the storms was applied to the theoretical 10- and 40-acre sites described in the previous section. Conveyance and storage systems were laid out and preliminary cost estimates were prepared. A review of the sample facilities and their associated costs will provide a feel for the magnitude of each of the design storms, and their impacts that on development. This analysis should be used as a guide in setting future stormwater policy.

TABLE A

Rainfall Intensity (in/hr)				
	Return Period (years)			
Minutes	**5**	**10**	**25**	**50**
5	1.18	1.28	1.41	1.76
10	0.92	1.00	1.10	1.37
15	0.78	0.84	0.92	1.15
30	0.54	0.58	0.64	0.80
60	0.34	0.37	0.41	0.51
120	0.20	0.22	0.24	0.29
180	0.15	0.16	0.18	0.22
360	0.09	0.11	0.11	0.13
720	0.06	0.07	0.07	0.08
1440	0.04	0.04	0.04	0.05

NOAA IDF curves generated from 50 years precipitation data with methods in the Atlas 2 publication.

METHODS

The purpose of this section is to guide the reader through the analysis process used by the consultant and to document the assumptions made that are critical to the outcome of the analysis.

Typical layouts of the example sites were made (Figures 1 and 2). The 10-acre site shown in Figure 1 was assumed to be a commercial development with 30% of the area occupied by the building. The remaining land area is laid out in parking and access roads. The entire site is assumed to be impervious, including the roof of the building which drains into the stormwater system.

It was assumed that the 40-acre site was residential, with 25% of the area in streets and roofs, and the remaining 75% landscaped. The entire site (Figure 2) was assumed to drain into the stormwater system, with C values of 0.9 for streets and roof areas and 0.1 for landscaped areas.

StormCAD computer software (by Haestad Methods) was used to size the conveyance systems. Pipe IDs are shown on Figure 3 and Figure 4. Pipe sizes determined from the model, and assuming a minimum pipe diameter of 12 inches, are summarized in Table 1.

Storage was evaluated for both a detention and a retention-type design. A detention facility temporarily stores water, while retention suggests permanent storage. Infiltration systems and closed depressions function more as retention facilities, while open ponds, tanks, and vaults perform more like detention facilities. However, they both eventually release their stored water, either directly as outflow, or through infiltration and evapotranspiration.

Required detention volumes were calculated by using a simplified SCS triangle hydrograph. The peak of the hydrograph was the peak flow as calculated by StormCAD using the Rational Method. The time to peak, Tc, was also calculated by StormCAD, again using the Rational Method. The base time of the hydrograph was assumed to be 2.67*Tc.

Outflow from the detention pond was assumed to be constant at the pre-development flow rate. Using these assumptions, a detention volume was calculated for each of the design storms on both of the sites. The detention volume was the area under the SCS triangular hydrograph minus the area represented by the constant outflow from the pond. Table 2 shows a summary of the required detention volumes and the assumptions used to obtain them.

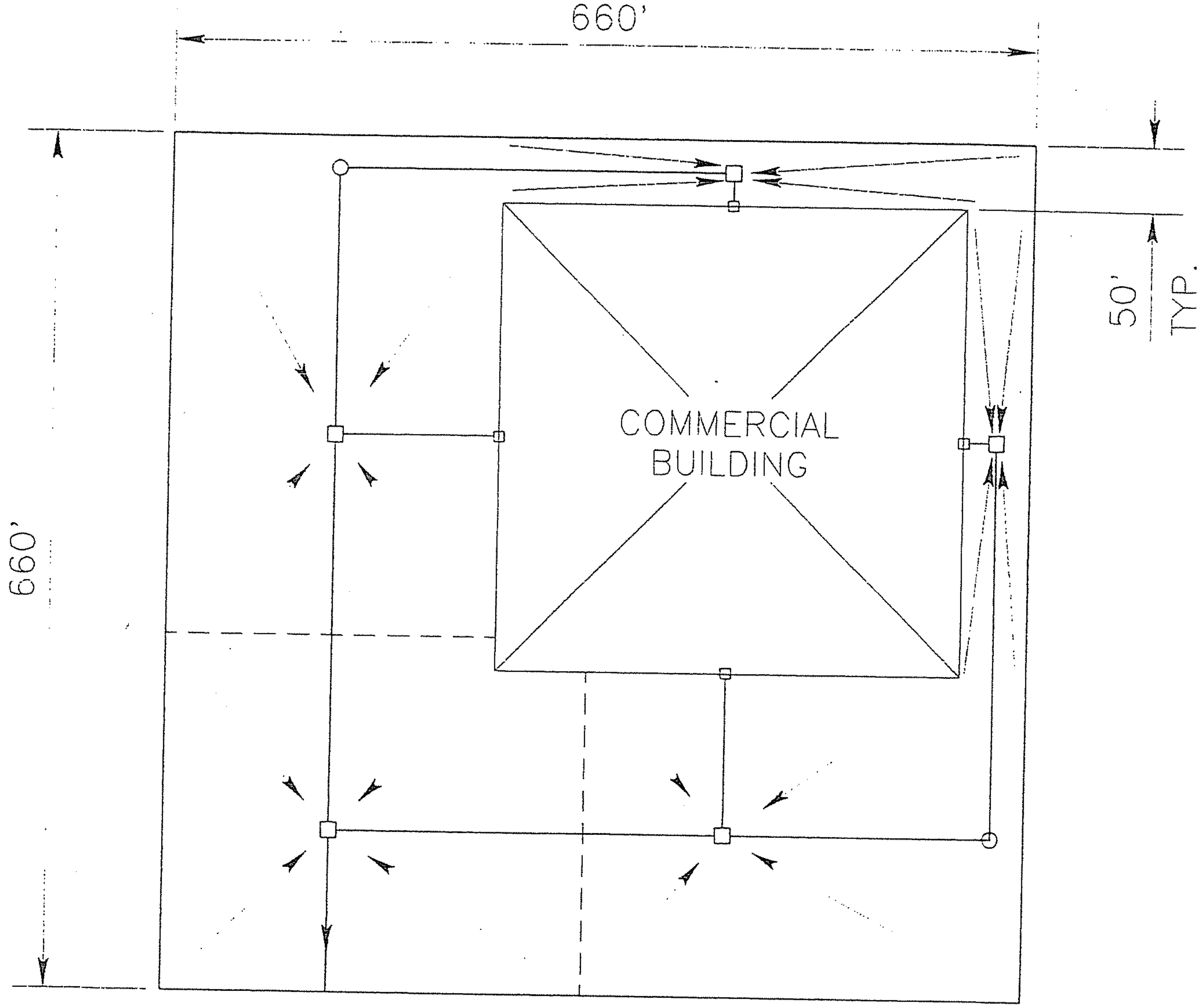
Required retention volumes were calculated by simply multiplying the area times the C factor times the depth of rainfall for each of the design storms. Because the total volume of the storm is expected to be stored in a retention pond, the calculation is free of any hydrographs or routing methods. Table 2 also shows a summary of the retention pond calculations.

Costs were estimated based on the quantities and sizes calculated. Since many assumptions were necessary regarding costs and quantities, it should be noted that the analysis provides "order of magnitude cost estimates". Because all assumptions were applied to each of the scenarios, the relative costs are valid for comparison to each other.

Table 3 shows the unit costs and the quantities used for the stormwater systems with detention ponds. Table 4 shows the costs for stormwater systems with retention ponds (unit costs and quantities are the same as for detention ponds). Table 5 is a summary of the costs for each design storm applied to both of the theoretical sites for both detention and retention ponds.

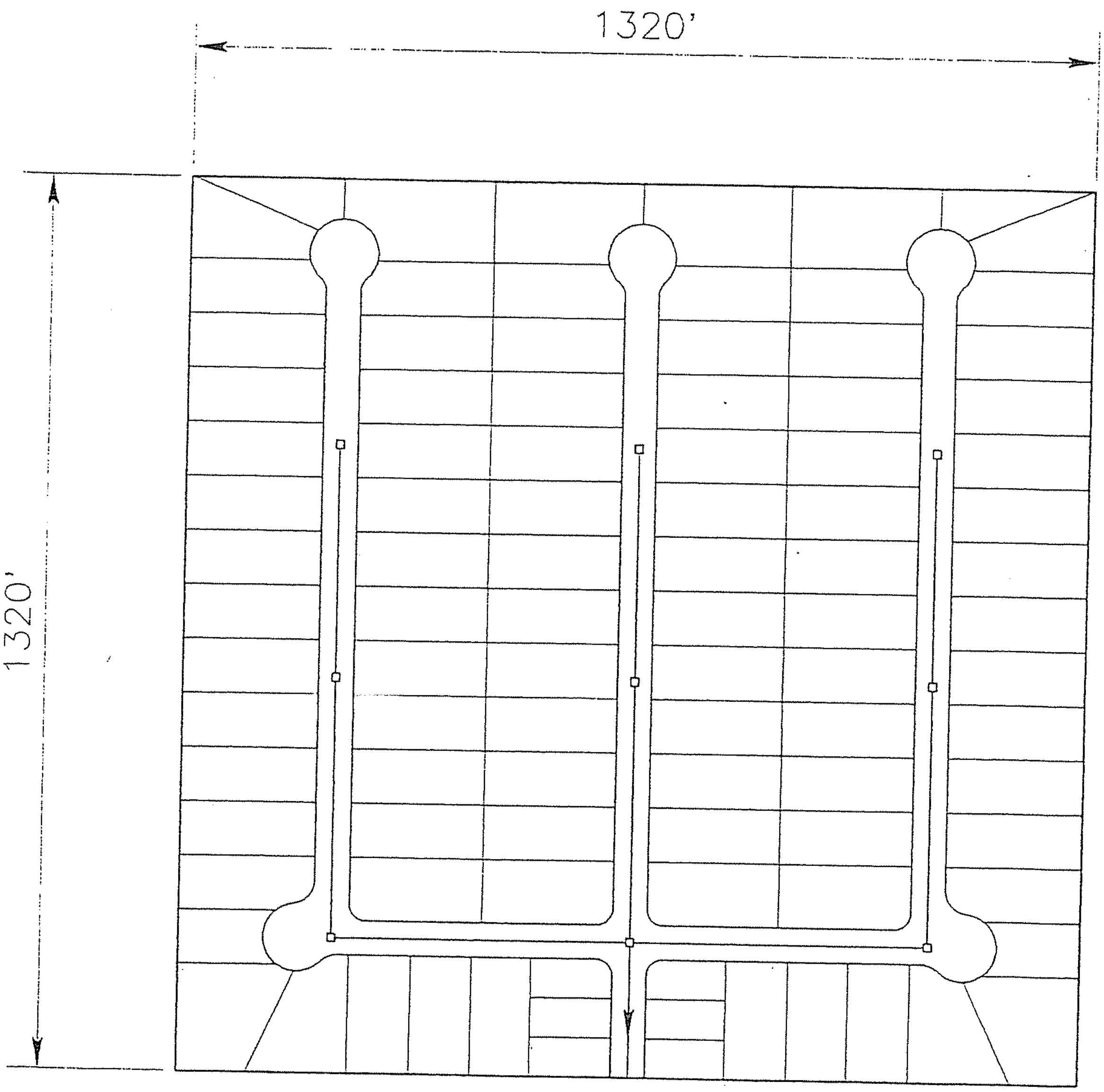
CONCLUSION

Design storms were chosen by the consultant to represent the best estimate of the expected precipitation in the Ontario area. As a check on the validity of the design storms, they were applied to two example sites. Conveyance and detention systems were sized and cost estimated in order to provide a comparison to those wanting to evaluate the design storms. This data should be used to select an appropriate design storm for the Stormwater Management Plan, and perhaps should be used as a guide in setting policy for new development within the city.



TYPICAL 10 ACRE SITE
(COMMERCIAL)

FIGURE 1



TYPICAL 40 ACRE SITE
(RESIDENTIAL)

FIGURE 2

KELLER ASSOCIATES, INC.

Special Design Storm Evaluation

Ontario Stormwater Management Plan
Pipe Quantities

10 Acre Commercial Site

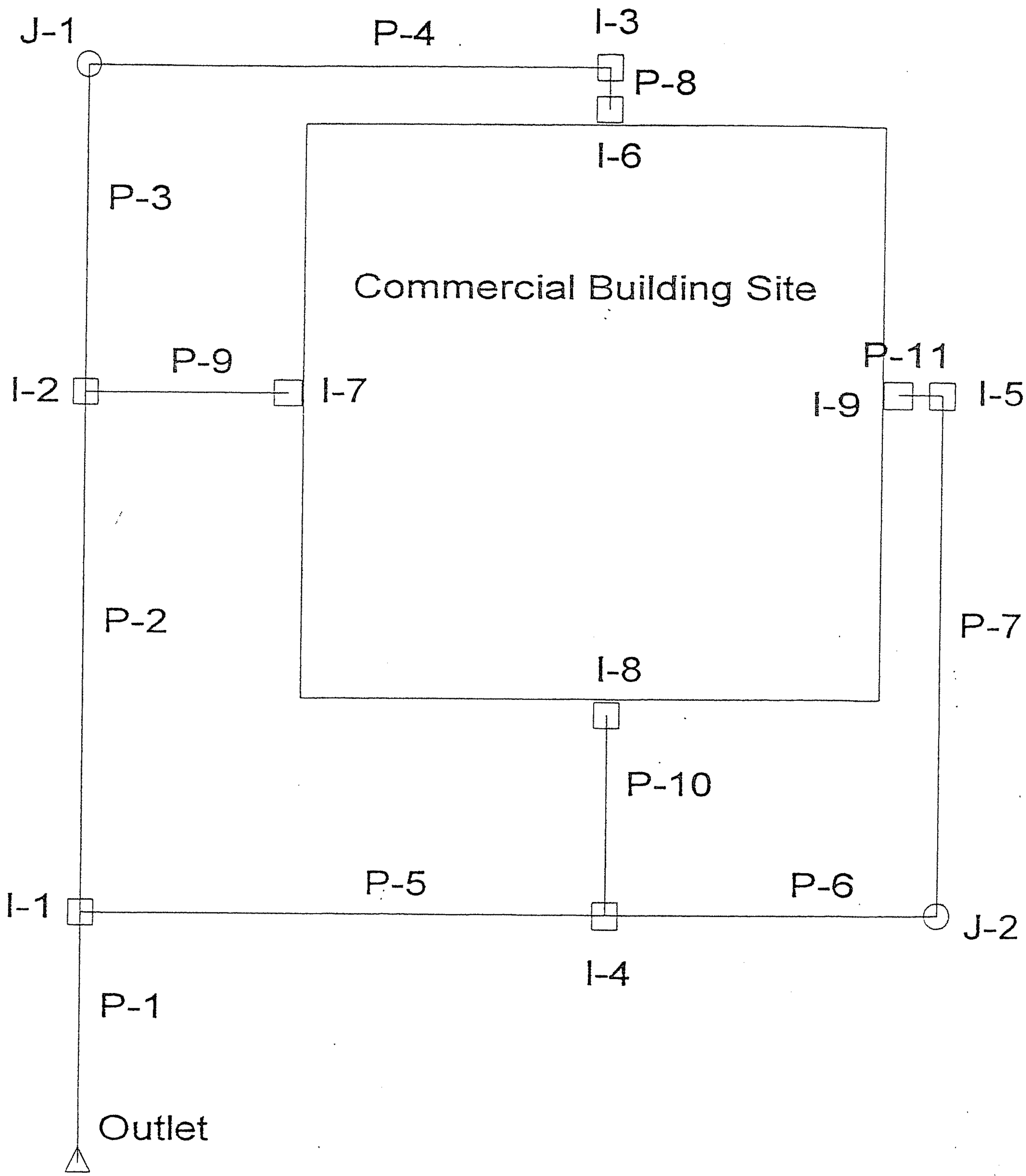
Storm Recurrence (Yr)		5 Year	10 Year	25 Year	50 Year
Pipe	Length (ft)	Size (in)	Size (in)	Size (in)	Size (in)
P-11	25	12	12	12	12
P-8	25	12	12	12	12
P-7	320	12	12	12	12
P-4	350	12	12	12	12
P-6	200	12	12	12	12
P-10	125	12	12	12	12
P-9	125	12	12	12	12
P-3	200	12	12	12	12
P-5	320	15	18	18	24
P-2	320	15	18	18	18
P-1	150	24	24	24	30

40 Acre Residential Site

Storm Recurrence (Yr)		5 Year	10 Year	25 Year	50 Year
Pipe	Length (ft)	Size (in)	Size (in)	Size (in)	Size (in)
P-10	400	12	12	12	12
P-7	400	12	12	12	12
P-9	400	12	12	12	12
P-6	400	12	12	12	12
P-3	400	12	12	12	12
P-8	440	12	12	12	15
P-5	440	12	12	12	15
P-2	400	12	12	12	12
P-1	100	18	18	18	18

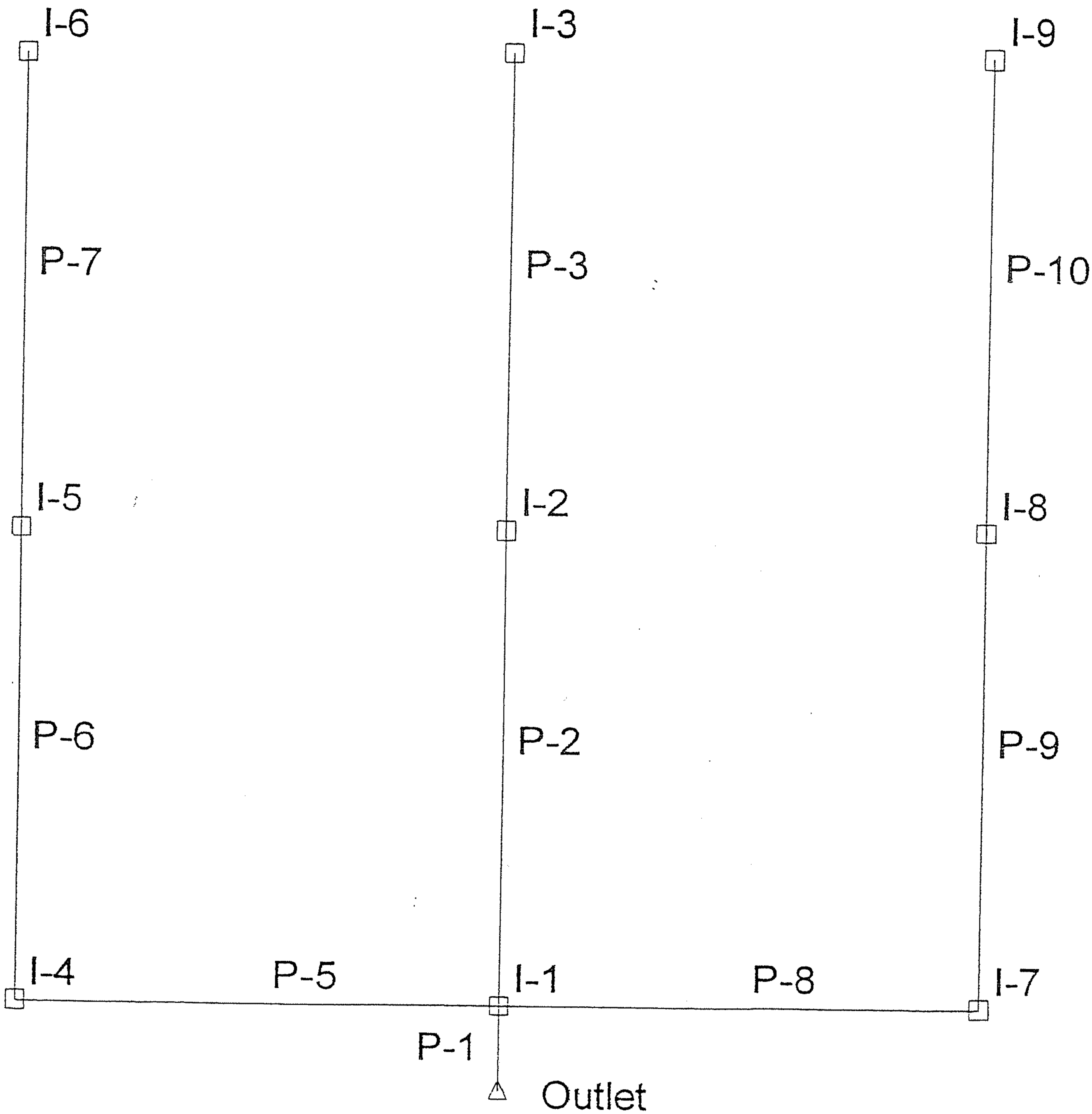
NOTE: A 12" mimimum pipe diameter was assumed

FIGURE 3



J-1
P-4
I-3
P-8
I-6
P-3
Commercial Building Site
P-9
I-2
I-7
P-11
I-9
I-5
P-2
P-7
I-8
P-10
P-5
P-6
I-1
J-2
I-4
P-1
Outlet

FIGURE 4



Special Design Storm Evaluation
Ontario Stormwater Management Plan

Detention Volume Computations

Site Size Acres	Recurrence Interval	Pre-Develop. Tc (min)	Pre-Develop. Qp (cfs)	Post Develop. Tc (min)	Post Develop. Qp (cfs)	Vol (cf)	Vol (cy)
Commercial							
10	5	45	0.44	8.19	10.02	5,998	222
	10	45	0.48	8.12	10.85	6,438	238
	25	45	0.52	7.99	12.02	7,023	260
	50	45	0.65	7.6	15.25	8,488	314
Residential							
40	5	50	1.62	52.84	4.73	6,279	233
	10	50	1.76	52.73	5.13	6,798	252
	25	50	1.94	52.59	5.69	7,658	284
	50	50	2.42	52.28	7.12	9,576	355

ASSUMPTIONS:

Post Developed Hydrograph Base Factor, Tb =	2.67
Extra area needed for finished detention pond =	30%
Pond Depth (ft) =	2
Predevelopment C =	0.1

Retention Volume Computations

Site Size	Recurrence Interval	24 Hour Rainfall Depths (in)	Total Retention Volume (cf)
Commercial			
10	5	0.93	33,759
	10	0.98	35,574
	25	1.01	36,663
	50	1.12	40,656
Residential			
40	5	0.93	40,511
	10	0.98	42,689
	25	1.01	43,996
	50	1.12	48,787

Special Design Storm Evaluation

Ontario Stormwater Management Plan

Concept Cost Estimates with Detention Basins

Recurrence Interval	No.	Item (10 Acre Sites)	Quantity		Unit Cos	Amount	No.	Item (40 Acre Sites)	Quantity		Unit Cost	Amount
10 Year	1	12" Pipe (installed)	1370	LF	$23	$31,510	1	12" Pipe (installed)	3280	LF	$23	$75,440
	2	15" Pipe (installed)	640	LF	$26	$16,640	2	15" Pipe (installed)	0	LF	$26	$0
	3	18" Pipe (installed)	0	LF	$31	$0	3	18" Pipe (installed)	100	LF	$31	$3,100
	4	24" Pipe (installed)	150	LF	$40	$6,000	4	24" Pipe (installed)	0	LF	$40	$0
	5	30" Pipe (installed)	0	LF	$48	$0	5	30" Pipe (installed)	0	LF	$48	$0
	6	Inlets	5	EA	$1,150	$5,750	6	Inlets	18	EA	$1,150	$20,700
	7	Manholes	2	EA	$1,750	$3,500	7	Manholes	6	EA	$1,750	$10,500
	8	Quality Box	1	EA	$3,500	$3,500	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	222	CY	$3.50	$777	9	Earthwork	233	CY	$3.50	$814
	10	Basin	166	SY	$40	$6,631	10	Basin	174	SY	$40.00	$6,942
	11	Land (purchase)	3,899	SF	$4.50	$17,543	11	Land (purchase)	4,081	SF	$2.10	$8,571
			Estimated Total =			$91,852			Estimated Total =			$129,567

Recurrence Interval	No.	Item (10 Acre Sites)	Quantity		Unit Cos	Amount	No.	Item (40 Acre Sites)	Quantity		Unit Cost	Amount
10 Year	1	12" Pipe (installed)	1370	LF	$23	$31,510	1	12" Pipe (installed)	3280	LF	$23	$75,440
	2	15" Pipe (installed)	0	LF	$26	$0	2	15" Pipe (installed)	0	LF	$26	$0
	3	18" Pipe (installed)	640	LF	$31	$19,840	3	18" Pipe (installed)	100	LF	$31	$3,100
	4	24" Pipe (installed)	150	LF	$40	$6,000	4	24" Pipe (installed)	0	LF	$40	$0
	5	30" Pipe (installed)	0	LF	$48	$0	5	30" Pipe (installed)	0	LF	$48	$0
	6	Inlets	5	EA	$1,150	$5,750	6	Inlets	18	EA	$1,150	$20,700
	7	Manholes	2	EA	$1,750	$3,500	7	Manholes	6	EA	$1,750	$10,500
	8	Quality Box	1	EA	$3,500	$3,500	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	238	CY	$3.50	$835	9	Earthwork	252	CY	$3.50	$881
	10	Basin	178	SY	$40.00	$7,118	10	Basin	188	SY	$40.00	$7,516
	11	Land (purchase)	4,185	SF	$4.50	$18,832	11	Land (purchase)	4,419	SF	$2.10	$9,279
			Estimated Total =			$96,884			Estimated Total =			$130,916

Recurrence Interval	No.	Item (10 Acre Sites)	Quantity		Unit Cos	Amount	No.	Item (40 Acre Sites)	Quantity		Unit Cost	Amount
25 Year	1	12" Pipe (installed)	1370	LF	$23	$31,510	1	12" Pipe (installed)	3280	LF	$23	$75,440
	2	15" Pipe (installed)	0	LF	$26	$0	2	15" Pipe (installed)	0	LF	$26	$0
	3	18" Pipe (installed)	640	LF	$31	$19,840	3	18" Pipe (installed)	100	LF	$31	$3,100
	4	24" Pipe (installed)	150	LF	$40	$6,000	4	24" Pipe (installed)	0	LF	$40	$0
	5	30" Pipe (installed)	0	LF	$48	$0	5	30" Pipe (installed)	0	LF	$48	$0
	6	Inlets	5	EA	$1,150	$5,750	6	Inlets	18	EA	$1,150	$20,700
	7	Manholes	2	EA	$1,750	$3,500	7	Manholes	6	EA	$1,750	$10,500
	8	Quality Box	1	EA	$3,500	$3,500	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	260	CY	$3.50	$910	9	Earthwork	284	CY	$3.50	$993
	10	Basin	194	SY	$40.00	$7,765	10	Basin	212	SY	$40.00	$8,466
	11	Land (purchase)	4,565	SF	$4.50	$20,542	11	Land (purchase)	4,978	SF	$2.10	$10,453
			Estimated Total =			$99,317			Estimated Total =			$133,152

Recurrence Interval	No.	Item (10 Acre Sites)	Quantity		Unit Cos	Amount	No.	Item (40 Acre Sites)	Quantity		Unit Cost	Amount
50 Year	1	12" Pipe (installed)	1370	LF	$23	$31,510	1	12" Pipe (installed)	2400	LF	$23	$55,200
	2	15" Pipe (installed)	0	LF	$26	$0	2	15" Pipe (installed)	880	LF	$26	$22,880
	3	18" Pipe (installed)	640	LF	$31	$19,840	3	18" Pipe (installed)	0	LF	$31	$0
	4	24" Pipe (installed)	0	LF	$40	$0	4	24" Pipe (installed)	100	LF	$40	$4,000
	5	30" Pipe (installed)	150	LF	$48	$7,200	5	30" Pipe (installed)	0	LF	$48	$0
	6	Inlets	5	EA	$1,150	$5,750	6	Inlets	18	EA	$1,150	$20,700
	7	Manholes	2	EA	$1,750	$3,500	7	Manholes	6	EA	$1,750	$10,500
	8	Quality Box	1	EA	$3,500	$3,500	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	314	CY	$3.50	$1,100	9	Earthwork	355	CY	$3.50	$1,241
	10	Basin	235	SY	$40.00	$9,385	10	Basin	265	SY	$40.00	$10,587
	11	Land (purchase)	5,517	SF	$4.50	$24,829	11	Land (purchase)	6,225	SF	$2.10	$13,072
			Estimated Total =			$106,614			Estimated Total =			$141,680

Special Design Storm Evaluation

Ontario Stormwater Management Plan

Concept Cost Estimates with Retention Basins

10 Acre Sites

Recurrence Interval	No.	Item	Quantity		Unit Cost	Amount
10 Year	1	12" Pipe (installed	1370	LF	$23	$31,510
	2	15" Pipe (installed	640	LF	$26	$16,640
	3	18" Pipe (installed	0	LF	$31	$0
	4	24" Pipe (installed	150	LF	$40	$6,000
	5	30" Pipe (installed	0	LF	$48	$0
	6	Inlets	5	EA	$1,150	$5,750
	7	Manholes	2	EA	$1,750	$3,500
	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	1,250	CY	$3.50	$4,376
	10	Basin	933	SY	$40	$37,323
	11	Land (purchase)	21,943	SF	$4.50	$98,745
			Estimated Total =			**$207,345**

40 Acre Sites

No.	Item	Quantity		Unit Cost	Amount
1	12" Pipe (installed	3280	LF	$23	$75,440
2	15" Pipe (installed	0	LF	$26	$0
3	18" Pipe (installed	100	LF	$31	$3,100
4	24" Pipe (installed	0	LF	$40	$0
5	30" Pipe (installed	0	LF	$48	$0
6	Inlets	18	EA	$1,150	$20,700
7	Manholes	6	EA	$1,750	$10,500
8	Quality Box	1	EA	$3,500	$3,500
9	Earthwork	1,500	CY	$3.50	$5,251
10	Basin	1,120	SY	$40	$44,788
11	Land (purchase)	26,332	SF	$2.10	$55,297
		Estimated Total =			**$218,577**

Recurrence Interval	No.	Item	Quantity		Unit Cost	Amount
10 Year	1	12" Pipe (installed	1370	LF	$23	$31,510
	2	15" Pipe (installed	0	LF	$26	$0
	3	18" Pipe (installed	640	LF	$31	$19,840
	4	24" Pipe (installed	150	LF	$40	$6,000
	5	30" Pipe (installed	0	LF	$48	$0
	6	Inlets	5	EA	$1,150	$5,750
	7	Manholes	2	EA	$1,750	$3,500
	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	1,318	CY	$3.50	$4,611
	10	Basin	983	SY	$40	$39,330
	11	Land (purchase)	23,123	SF	$4.50	$104,054
			Estimated Total =			**$218,095**

No.	Item	Quantity		Unit Cost	Amount
1	12" Pipe (installed	3280	LF	$23	$75,440
2	15" Pipe (installed	0	LF	$26	$0
3	18" Pipe (installed	100	LF	$31	$3,100
4	24" Pipe (installed	0	LF	$40	$0
5	30" Pipe (installed	0	LF	$48	$0
6	Inlets	18	EA	$1,150	$20,700
7	Manholes	6	EA	$1,750	$10,500
8	Quality Box	1	EA	$3,500	$3,500
9	Earthwork	1,581	CY	$3.50	$5,534
10	Basin	1,180	SY	$40	$47,196
11	Land (purchase)	27,748	SF	$2.10	$58,270
		Estimated Total =			**$224,240**

Recurrence Interval	No.	Item	Quantity		Unit Cost	Amount
25 Year	1	12" Pipe (installed	1370	LF	$23	$31,510
	2	15" Pipe (installed	0	LF	$26	$0
	3	18" Pipe (installed	640	LF	$31	$19,840
	4	24" Pipe (installed	150	LF	$40	$6,000
	5	30" Pipe (installed	0	LF	$48	$0
	6	Inlets	5	EA	$1,150	$5,750
	7	Manholes	2	EA	$1,750	$3,500
	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	1,358	CY	$3.50	$4,753
		Basin	1,013	SY	$40	$40,534
	11	Land (purchase)	23,831	SF	$4.50	$107,239
			Estimated Total =			**$222,626**

No.	Item	Quantity		Unit Cost	Amount
1	12" Pipe (installed	3280	LF	$23	$75,440
2	15" Pipe (installed	0	LF	$26	$0
3	18" Pipe (installed	100	LF	$31	$3,100
4	24" Pipe (installed	0	LF	$40	$0
5	30" Pipe (installed	0	LF	$48	$0
6	Inlets	18	EA	$1,150	$20,700
7	Manholes	6	EA	$1,750	$10,500
8	Quality Box	1	EA	$3,500	$3,500
9	Earthwork	1,629	CY	$3.50	$5,703
	Basin	1,216	SY	$40	$48,641
11	Land (purchase)	28,597	SF	$2.10	$60,054
		Estimated Total =			**$227,638**

Recurrence Interval	No.	Item	Quantity		Unit Cost	Amount
50 Year	1	12" Pipe (installed	1370	LF	$23	$31,510
	2	15" Pipe (installed	0	LF	$26	$0
	3	18" Pipe (installed	640	LF	$31	$19,840
	4	24" Pipe (installed	0	LF	$40	$0
	5	30" Pipe (installed	150	LF	$48	$7,200
	6	Inlets	5	EA	$1,150	$5,750
	7	Manholes	2	EA	$1,750	$3,500
	8	Quality Box	1	EA	$3,500	$3,500
	9	Earthwork	1,506	CY	$3.50	$5,270
	10	Basin	1,124	SY	$40	$44,949
	11	Land (purchase)	26,426	SF	$4.50	$118,919
			Estimated Total =			**$240,438**

No.	Item	Quantity		Unit Cost	Amount
1	12" Pipe (installed	2400	LF	$23	$55,200
2	15" Pipe (installed	880	LF	$26	$22,880
3	18" Pipe (installed	0	LF	$31	$0
4	24" Pipe (installed	100	LF	$40	$4,000
5	30" Pipe (installed	0	LF	$48	$0
6	Inlets	18	EA	$1,150	$20,700
7	Manholes	6	EA	$1,750	$10,500
8	Quality Box	1	EA	$3,500	$3,500
9	Earthwork	1,807	CY	$3.50	$6,324
10	Basin	1,348	SY	$40	$53,938
11	Land (purchase)	31,712	SF	$2.10	$66,595
		Estimated Total =			**$243,637**

Special Design Storm Evaluation

Ontario Stormwater Management Plan

Cost Summaries for Detention

10 Acre Sites		40 Acre Sites	
Recurrence Interval	Total Estimated Costs	Recurrence Interval	Total Estimated Costs
5	$91,852	5	$129,567
10	$96,884	10	$130,916
25	$99,317	25	$133,152
50	$106,614	50	$141,680

Cost Summaries for Retention

10 Acre Sites		40 Acre Sites	
Recurrence Interval	Total Estimated Costs	Recurrence Interval	Total Estimated Costs
5	$207,345	5	$218,577
10	$218,095	10	$224,240
25	$222,626	25	$227,638
50	$240,438	50	$243,637

Technical Memorandum #2

To: City of Ontario

From: Susan Burnham, P.E., Keller Associates, Inc.
James Bledsoe, E.I.T., Keller Associates, Inc.

Date: October 18, 2001

Re: Design Storm Analysis

Project #: 197032

METHODS

Potential design storms were analyzed based on available data. Sources used were the Western Regional Climate Center (WRCC), Oregon Department of Transportation (ODOT), the National Oceanic and Atmospheric Administration (NOAA), and the 1983 City of Ontario Storm Water Management Plan (SWMP). Data from the Malheur Agricultural Experiment Station was also reviewed.

From the available data, it appears that the ODOT information was derived directly from 1973 NOAA data. The most complete precipitation information came from WRCC, which provided daily records from July 1948 through September 2000. This data was used with methods from NOAA Atlas 2 to generate potential design storms for the City of Ontario.

The WRCC data was compiled as a partial series, with storms arranged in descending order of magnitude and assigned a ranking (m). The storm frequency (recurrence interval T_r) was calculated as $T_r = \frac{n+1}{m}$, where n is the number of years of record.

Since daily rainfall records represent observations for calendar days, a 24-hour storm (which can span more than one calendar day) is not necessarily the same as a daily observation. Per the statistical relation discussed in the NOAA atlas, 24-hour rainfall values were derived from observed daily values multiplied by a factor of 1.13.

Values for other storm durations were extrapolated from the WRCC data, based on relationships developed from NOAA data. Two durations were considered in the analysis: a 1-hour storm, and a 24-hour storm. A comparison of total rainfall (over the specified duration) from the various sources is included in Table 1. The lowest values are those from the 1983 SWMP, while the highest are from ODOT/NOAA.

TABLE 1. COMPARISON OF DESIGN STORMS

1-Hr Storm Event (Inches)

Source	**5-YR**	**10-YR**	**25-YR**	**50-YR**
ODOT / NOAA (1973)	0.47	0.56	0.69	0.81
Storm Water Master Plan (1983)	0.18	0.21	0.25	0.29
Based on WRCC data (7/48 – 9/00)	**0.34**	**0.37**	**0.41**	**0.51**

24-Hr Storm Event (Inches)

Source	**5-YR**	**10-YR**	**25-YR**	**50-YR**
NOAA (1973)	1.35	1.55	1.76	1.8
Based on WRCC data (7/48 – 9/00)	**0.93**	**0.98**	**1.01**	**1.12**

DISCUSSION

In the 1983 SWMP, 5- and 10-year storm frequencies were used to analyze storm sewer requirements for residential and commercial/industrial areas respectively. Intensity-duration-frequency (IDF) curves for 15-minute to 15-hour storms were developed. The storm intensity was apparently based on a duration equal to the time of concentration in the respective drainage basin.

It is recommended that the WRCC data be used for design storms for the City of Ontario. For the 5- and 10-year storms, the WRCC values average about 80% greater than 1983 SWMP design criteria (though less than ODOT values).

Conveyance system design should be based on a storm duration equal to the drainage area time of concentration. Because the times of concentration for most sites in the city will be less than 30 minutes, the conveyance systems will be sized for a short duration event by default. Short times of concentration lead to higher intensities, which will result in conservatively-sized conveyance systems.

The remaining decision to be made by the City is the frequency considered appropriate for the design storm. The selection of the appropriate design frequency should involve a consideration of the cost versus the flooding risk that citizens of the community are willing to accept. Use of a less frequent-more intense storm may be justified in high property value areas, where flooding will cause considerable property damage. Use of a more frequent-less intense storm may be practical when minor flooding can be tolerated, or if limited funds are available.
Several communities in the region use a 50-year storm for design, with an intensity of 1 inch/hour. Utilizing a 50-year storm for design, particularly with the higher IDF values

based on WRCC data, would result in design rainfall events nearly 3 times greater than the 1983 SWMP values for Ontario.

On the other hand, a 25-year storm would produce about twice the rainfall used in the 1983 SWMP. Based on WRCC data, a 25-year 24-hour storm would generate about one inch total rainfall. If applied to stormwater detention/retention facilities, this would provide storage equivalent to the 1 inch/hour storm used by many other communities.

Based on the assumptions and estimates in Tech Memo #1, a new residential stormwater system designed for a 25-year frequency would cost just 3-4% more than one designed for a 5-year storm. Similarly, a 25-year frequency stormwater system for a new commercial/industrial development would cost about the same as a 10-year system. Providing a 50-year system would cost 6-8% more than a 25-year system.

RECOMMENDATIONS

It is recommended that the Intensity-Duration-Frequency (IDF) curves developed from WRCC data be used for the City of Ontario (see Figure 1).

Conveyance system design should be based on a storm duration equal to the drainage area time of concentration, which in most cases will be less than 30 minutes. A minimum time of concentration of 10 minutes should be specified.

It is recommended that a 24-hour duration storm be used for design of detention and retention facilities.

Design based on a 25-year storm frequency would result in a lower risk of flooding than using a more frequent-less intense storm. For a 24-hour duration, the 25-year storm represents 1 inch total rainfall. A 25-year frequency appears to be economically feasible for new development, though it may be cost-prohibitive for the existing collection system. The choice of design storm frequency can be reevaluated after its impact on capacity of the existing storm sewer system has been analyzed.

Ontario IDF Curves

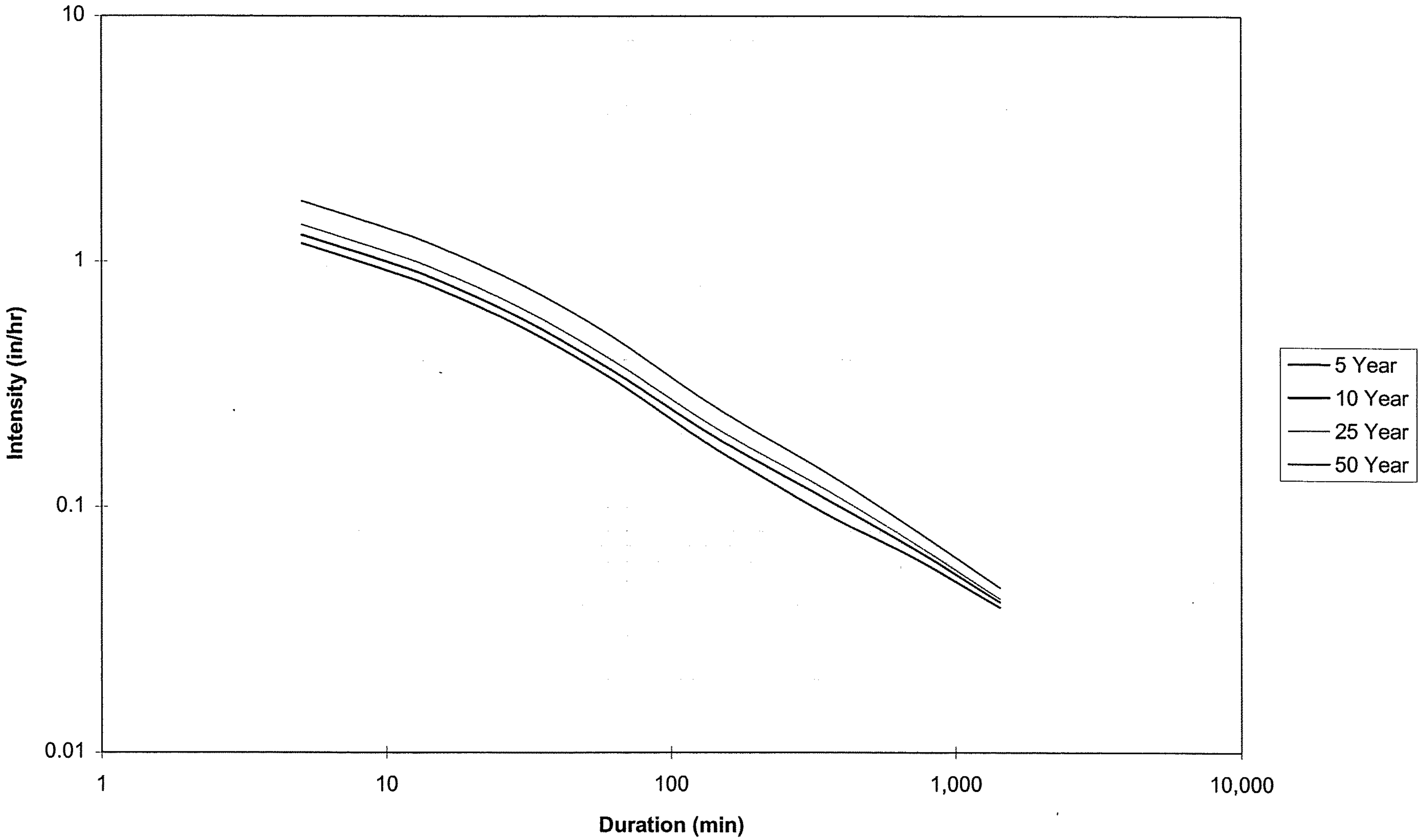


Figure 1

Keller Associates, Inc.

Technical Memorandum #3

Storm Water Analysis of the Heinz Frozen Foods Basin

To: City of Ontario

From: James Bledsoe, E.I.T., Keller Associates, Inc.
Hailey Wappett, E.I.T., Keller Associates, Inc.

Date: January 18, 2002

Project #: 197032

Introduction

Keller Associates has been contracted to do the Storm Water Master Plan for the City of Ontario. As part of the Storm Water Master Plan, the City has been divided into drainage basins. The first basin to be analyzed is the area east of the railroad tracks and west of Interstate 84, encompassing the industrial area of Heinz Frozen Foods (HFF) and Americold. This memorandum outlines the model analysis and results of this watershed, hereafter referred to as the Heinz Frozen Foods basin, or HFF basin. The information in this memorandum will be useful in the design of a joint detention/retention facility that will serve the City of Ontario and HFF.

Model Development

XP-SWMM2000 was used to create a hydraulic model of the HFF Sub-basin. The software integrates storm water runoff models (including those using the SCS runoff equations) with dynamic routing of pipe networks. XP-SWMM requires input data for each manhole, pipe, and storm water basin. The model has the ability to calculate the flow through each manhole, junction, pipe and ultimately the maximum flow and total volume at the outfall of the basin.

The HFF basin has an area of 330 acres, 32 acres of which do not contribute to the basin's storm sewer system. Of these 32 acres, 18.2 acres have onsite storm water retention and the remaining 13.8 acres include the HFF complex, and therefore, drains to the HFF wastewater treatment plant. The topology of the HFF basin area is relatively flat; however, runoff tends to drain in a northerly direction. The delineation of the rainfall basin is shown in Figure 1. The storm sewer interceptor lines convey rainfall runoff north along SE 4th Street and NE 3rd

Street, then through the HFF complex to an outfall near the Snake River. The storm sewer pipe network is shown in Figure 2.

Field data and City maps were used to develop the storm water model, which consists of two parts--a collection system model and runoff model. The collection system parameters include the following: pipe diameter, invert and ground elevations, slope, length, and pipe roughness. The runoff basin parameters include: area, percent of sub-basin that is impervious, SCS curve number, time of concentration, slope, width and initial abstraction. A description of each of these parameters follows:

- Pipe roughness/Mannings coefficient: a coefficient that is based upon the age and condition of the pipe. For this model, it was assumed that the pipes are clean and in good condition.
- Area: drainage area of a sub-basin.
- Percent Impervious: the percent of the sub-basin that is impervious and is directly connected to the drainage system. Rooftops that drain onto pervious areas are not included in the percent impervious.
- Curve Number: a coefficient based upon the hydrologic soil group, cover type, treatment, hydrological condition, and antecedent moisture conditions.
- Time of Concentration: the time it takes for rainfall from the furthest point in the sub-basin to reach the storm sewer system.
- Slope: the average slope of the sub-basin, reflecting the average slope along the pathway of overland flow to the catch basin location.
- Width: the width of the sub-basin area, as approximated by dividing the area of the sub-basin by the average path length of overland flow.
- Initial Abstraction: this coefficient accounts for the runoff that does not ultimately reach the storm sewer system as a result of the impervious area and pervious area combination, surface depression storage, evapotranspiration, interception, etc.

The following assumptions were made in developing the model:

- A 25 year event, 24 hour 1-inch rainfall
- SCS Rainfall, Type II Storm Distribution
- Pipes are relatively free of accumulated debris (n = 0.014)
- Outfall is free of any obstruction
- Hydrologic Soil Group B: Silt loam or loam, as determined from soil maps, was used to determine the SCS curve number.
- A weighted Curve Number (CN) was calculated for each sub-basin assuming a CN of 98 and for impervious areas, and 65 for pervious areas.
- An Initial Abstraction of 0.1S, where S=(1000/CN)-10

A further assumption made concerns the Americold food processing industry. As shown in Figure 2, the storm network captures not only surface runoff, but the defrost wastewater from Americold. The defrosting process is done periodically and the waste water is discharged directly into the stormwater system. It was assumed for this model that the Americold defrost discharge did not coincide with the storm event, and therefore, does not affect the maximum outflow. Defrosting will potentially impact the storage volume required for a detention/retention facility. It is further recommended that Americold be advised to not defrost during major storm events, in order to prevent surcharging the system.

Two storm water model runoff models were developed for the existing HFF basin. The first involved 15 sub-basins and represents a skeletonized model of the basin. The second model divided the HFF basin into 55 sub-basins. The calculated and assumed parameters for the 15 and 55 sub-basin delineations are included in Tables 1 and 2, respectively.

Future Development

In order to provide accurate design criteria for the detention/retention basin, the effect of future development on the HFF basin stormwater system was analyzed. This basin has approximately 36 acres of undeveloped land in the northeast portion of the basin, which currently contributes minimally to the stormwater runoff. In the future, this area has potential as a sight for industrial/commercial expansion. As it develops the runoff from this area will greatly increase. In order to accurately model this development, the undeveloped area was divided into two new sub-basins, basins P and Q, shown in Figure 4, and a new stormwater pipeline was placed to capture the runoff, as shown in Figure 5. Industrial/commercial expansion would result in large impervious areas, affecting the runoff characteristics of the basin. The SCS curve number was increased to 92, a typical value for industrial/commercial drainage basins. The remaining basin parameters for these two new basins are summarized in Table 3.

In addition to the anticipated additional development in the HFF basin, the drainage from the HFF complex, basin O as shown in Figure 1, which currently drains to the HFF wastewater treatment plant and not the stormwater system, may be connected to the storm water system in the future (Figure 5). Therefore, in order to accurately reflect future conditions, this basin, along with the two for future development was added to the skeletonized model of the HFF basin.

Model Results

Both the skeletonized model and the more detailed 55-basin model resulted in similar hydrographs including peak flows, runoff volumes, and time to peak (see Table 3 below). Both models showed some ponding. The skeletonized model resulted in slightly more conservative peak flows. This is attributed to the additional attenuation of sub-basin runoff in the more detailed model. The 55 basin model projected a 15 percent higher runoff volume. This is believed to be an anomaly of the SCS method which, depending on the size of the storm event, may allow for more runoff volume from multiple sub-basins with various CN's, than from a single basin having a composite CN. Figure 3 illustrates the runoff hydrograph for the 15 basin model.

The addition of the two new basins reflecting future development and the HFF complex basin, resulted in a 17 basin skeletonized model. This model resulted in nearly double the peak flow of the 15 basin model, and a 32% increase in the runoff volume. Figure 6 illustrates the runoff hydrograph for future conditions.

Table 4: HFF Basin Stormwater Model Results

25 Year, 1" Storm	**Peak Flow (cfs)**	**Runoff Volume (ft^3)**
55 Basin Model	12.2	231,000
15 Basin Model	14.8	202,000
Future Development	28.5	266,600
Pipeline Improvements (see below)	28.1	255,700

It should be noted that there is no calibration data, or observed flow measurements for the HFF basin. According to City personnel, there has not historically been ponding in the streets. This observation of City staff is supported by model results which show only 12 minutes of ponding occurring from a 25-year event imposed on the skeletonized model.

Future Pipeline Improvements

The existing storm water system is not capable of handling a 25 year storm event without surcharging all the interceptor lines. Much of the excess runoff is stored in the manholes and in a few locations, rises to the surface, resulting in ponding. This surcharging is shown in Figure 7, which reflects the profile of the trunk line during a storm event at maximum flow. Undersized pipes are a major factor causing the surface ponding. Idaho Standards for Public Works Construction (ISPWC) requires that a storm water pipeline be not less than 12-inches in diameter. The existing system has several sections that are less than this, resulting in flooding in these areas. Figure 5 illustrates the pipelines with a diameter less than the 12-inch minimum. When these waterlines are upsized to reflect the 12-inch minimum pipe diameter, the ponding problem is remediated. It is recommended that all future storm water waterlines meet the 12-inch minimum diameter requirement.

As the existing system ages and requires replacement, Keller Associates recommends that some of the pipelines be upsized to allow for the entire storm event to be conveyed through the pipes, thus preventing any surcharging in the manholes or ponding (Figure 8). The areas of recommended improvements are highlighted in Figure 5 and summarized below:

- Two sections of reverse flow should be corrected to allow for continuous northerly flow: between SE 5th Avenue and SE 4th Avenue, and between SE 1st Avenue and E Idaho Avenue.
- The section of pipeline between NE 2nd Avenue and the manhole one block north of NE 6th Avenue be upsized from 24-inch to 30-inch.
- The section of pipeline between SE 8th Avenue and SE 6th Avenue be upsized from 12-inch to 18-inch.
- The section of pipeline between SE 3rd Avenue and E Idaho Street be upsized from 18-inch to 24-inch.
- The section of pipeline under Interstate 84 and before the 90-degree turn be upsized from 30-inch to 36-inch.

With these improvements all flow will be contained within the pipelines, as shown in Figure 8. The effects of these improvements on the peak flow at the outfall and runoff volume is shown in Table 4. These improvements aid in reducing the peak flow and runoff volume resulting from the future development of the basin. As future development and expansion occur, these improvements will become more imperative.

HFF Retention/Detention Facility Recommendations

It is recommended that for design of the bypass piping and *retention* facility on the HFF property, that a **minimum peak flow of 30 cfs** and a **minimum storage volume of 300,000 cubic feet** be used. Additional storage for freeboard should also be provided. If a *detention* facility is designed with a continuous outflow, significantly less storage may be necessary depending on the allowed discharge rate. The volume of storage needed for a detention facility could be more accurately calculated using the hydrograph presented in Figure 3.

A potential diversion point to the new retention/detention facility is located on NE 3rd Street just north of I-84 where the pipeline takes a 90 degree turn to the west. This is shown in Figure 2.

In designing the new retention/detention facility the City should ensure that there is adequate fall between the existing collection system and the retention facilities, and that the retention facility meets state requirements specifically in regards to groundwater separation distances. According to the Oregon Department of Environmental Quality (DEQ) there are specific requirements in regards to stormwater management In regard to retention/detention facilities the applicable guidelines are summarized as follows:

- Retention/Detention facilities can be, dependent upon their design, effective for the removal of urban pollution present in stormwater runoff.
- In order to prevent contamination of the groundwater any facility located in soils with a minimum infiltration rate or permeability of more than 9 inches per hour must have a lining system designed to prohibit infiltration. This may be a 12-inch layer of Bentonite clay or a commercial heavy plastic pond liner with a minimum 18-inch thick compacted top soil over the liner.
- The permanent depth of a retention/detention facility is recommended to be between 3 and 6 feet, plus storage for sediment. If the facility is greater than 6 feet there is a high likelihood of groundwater contamination.
- In order to deal with nutrients a shallow marsh system is recommended.
- Currently stormwater runoff from an industrial area requires a formal Stormwater Pollution Prevention Plan (SWPPP) for a National Pollutant Discharge Elimination Permit System (NPDES) stormwater permit under 40 CFR 122.26 for the discharge surface waters, including wetlands.
- A detention facility's discharge will be impacted by Phase II NPDES regulations. It is recommended that the design of a detention facility meet these future regulations.

Keller Associates understands that the City is considering installing a pumping station to lift the runoff into the future retention facility. Installation of a pump station may be costly both from a construction cost as well as a continued operation and maintenance. This is particularly true in light of the periodic defrosting from Americold. In our brief review of groundwater and elevation data it appears that there would be adequate grade to transport the flow from the existing pipe network to the proposed retention site. The City should consider constructing a vegetated channel conveyance system. This channel could serve also as a treatment process similar to the channel receiving the storm water runoff from SE 9th Avenue and thereafter conveying the runoff to the Kendall Pit.

These retention/detention design recommendations are based upon a 25 year event: 1" rainfall in a 24 hour time period. According to Oregon DEQ, overflow/emergency provisions are required to ensure adequate storage in the likelihood of a 50 or 100 year event. These provisions may include additional storage at the proposed retention/detention facility and/or a separate storage facility.

FIGURE NO. 1	ONTARIO STORM WATER MASTER PLAN HEINZ FROZEN FOODS WATERSHED	City of **Ontario, Oregon**	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME SSW Sub-basin

City of Ontario
Stormwater Improvements

Alternative A: Upsize Irrigation Line and 2 Tie-ins from Park Blvd (42-inch)

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
12-inch						
Open Field 5'-10' deep	LF		$23.00	$0		
5'-10' deep w/ asphalt repair	LF	340	$27.00	$9,180		
Open Field 10'-15'	LF		$31.00	$0		
10'-15' deep w/ asphalt repair	LF		$35.00	$0		
Bore	LF		$300.00	$0	$9,180	$11,475
15-inch						
Open Field 5'-10' deep	LF		$27.00	$0		
5'-10' deep w/ asphalt repair	LF	587.91	$31.00	$18,225		
Open Field 10'-15'	LF		$35.90	$0		
10'-15' deep w/ asphalt repair	LF		$40.00	$0		
Bore	LF		$300.00	$0	$18,225	$22,782
24-inch						
Open Field 5'-10' deep	LF	400	$37.00	$14,800		
5'-10' deep w/ asphalt repair	LF		$41.75	$0		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0		
Bore	LF		$450.00	$0	$14,800	$18,500
42-inch						
Open Field 5'-10' deep	LF	2181.39	$69.00	$150,516		
5'-10' deep w/ asphalt repair	LF	6535.26	$79.00	$516,286		
Open Field 10'-15'	LF	2079.88	$87.00	$180,950		
10'-15' deep w/ asphalt repair	LF		$93.00	$0		
Bore	LF	275	$1,000.00	$275,000	$1,122,751	$1,403,439
Alternative A Total Construction Cost					**$1,164,956**	
25% Contingencies					**$291,239**	
TOTAL ESTIMATED COST					**$1,456,195**	

Priority II: South Park Boulevard (SW 9th Ave South to SW 12th Avenue) Improvements

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
18-inch						
Open Field 5'-10' deep	LF		$31.00	$0		
5'-10' deep w/ asphalt repair	LF	1019.77	$35.00	$35,692		
Open Field 10'-15'	LF		$40.75	$0		
10'-15' deep w/ asphalt repair	LF		$44.75	$0		
Bore	LF		$300.00	$0	$35,692	$44,615
24-inch						
Open Field 5'-10' deep	LF		$37.00	$0		
5'-10' deep w/ asphalt repair	LF	1095.23	$41.75	$45,726		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0		
Bore	LF		$450.00	$0	$45,726	$57,157
Alternative A Priority II Total Construction Cost					**$81,418**	
25% Contingencies					**$20,354**	
TOTAL ESTIMATED COST					**$101,772**	

ESTIMATE OF MOST PROBABLE COST **$1,557,968**

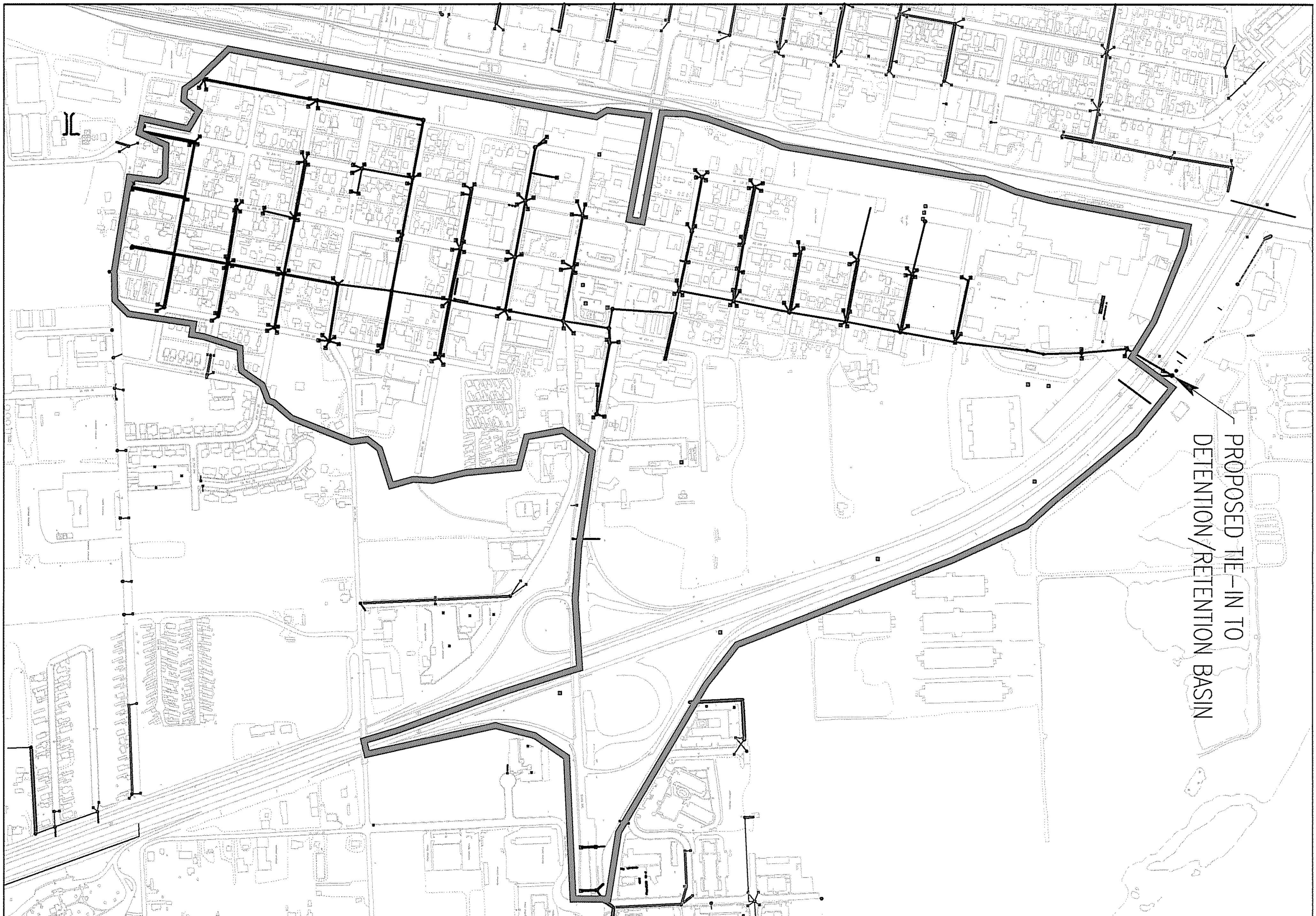
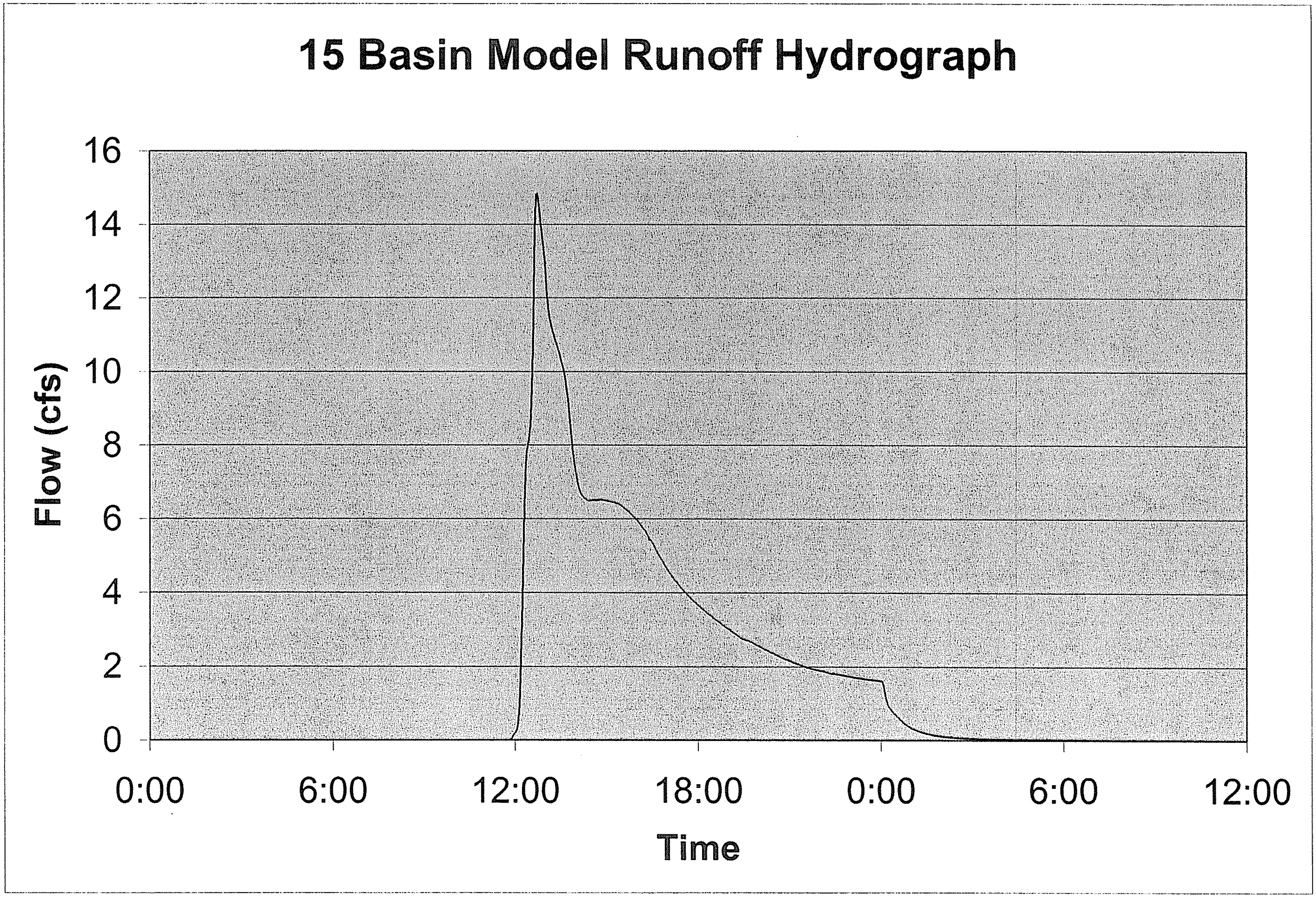
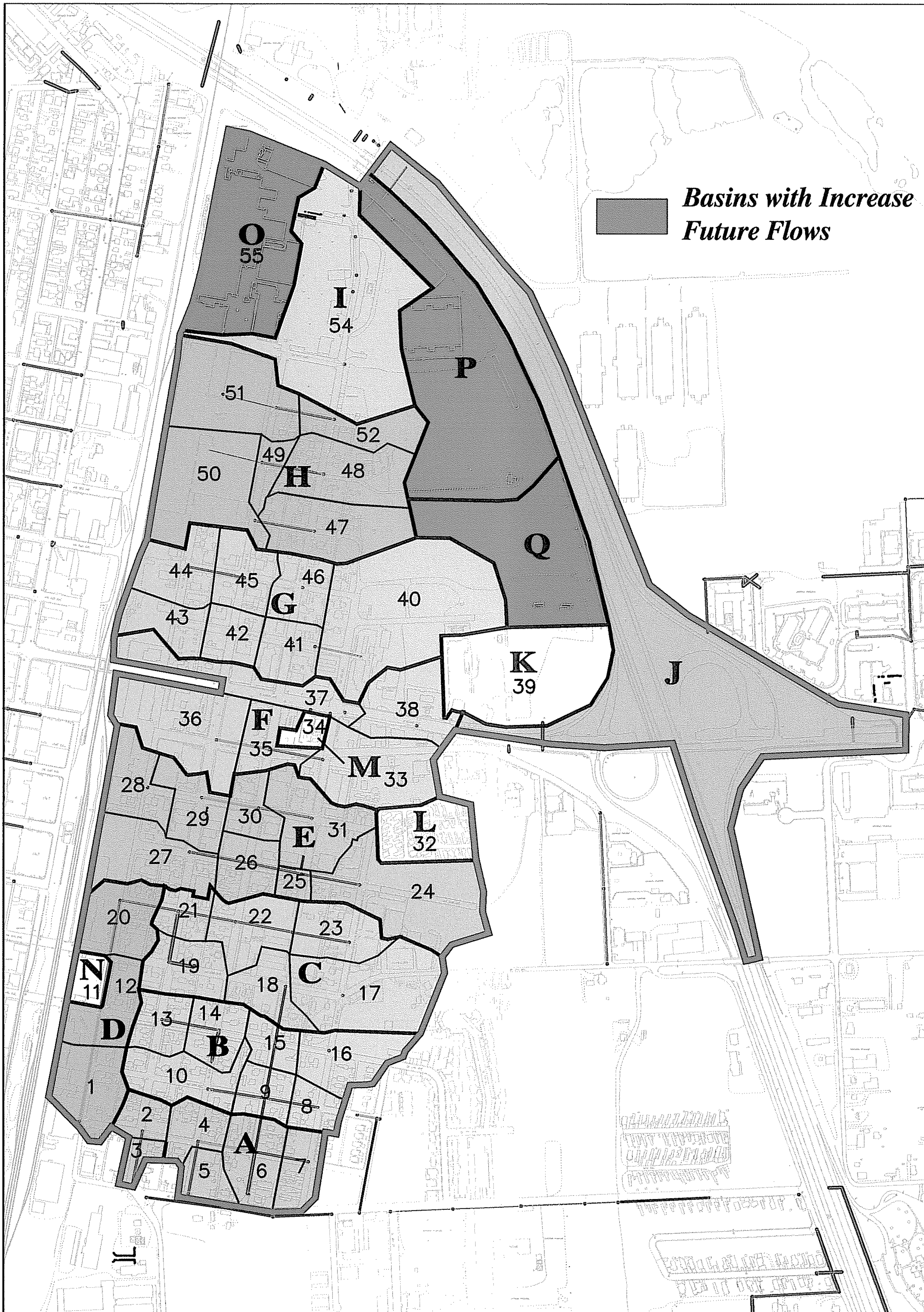


FIGURE NO. 2	ONTARIO STORM WATER MASTER PLAN HFF STORM WATER SYSTEM	City of **Ontario, Oregon**	KELLER ASSOCIATES 131 SW 5th Avenue, Suite A Meridian, Idaho 83642 (208) 288-1992	PROJECT NO. 197032 FILENAME Strmwtrsys.dwg

Figure 3


15 Basin Model Runoff Hydrograph
Flow (cfs)
16
14
12
10
8
6
4
2
0
0:00
6:00
12:00
18:00
0:00
6:00
12:00
Time



PROJECT NO. 197032

FILENAME ...Sub-basin future

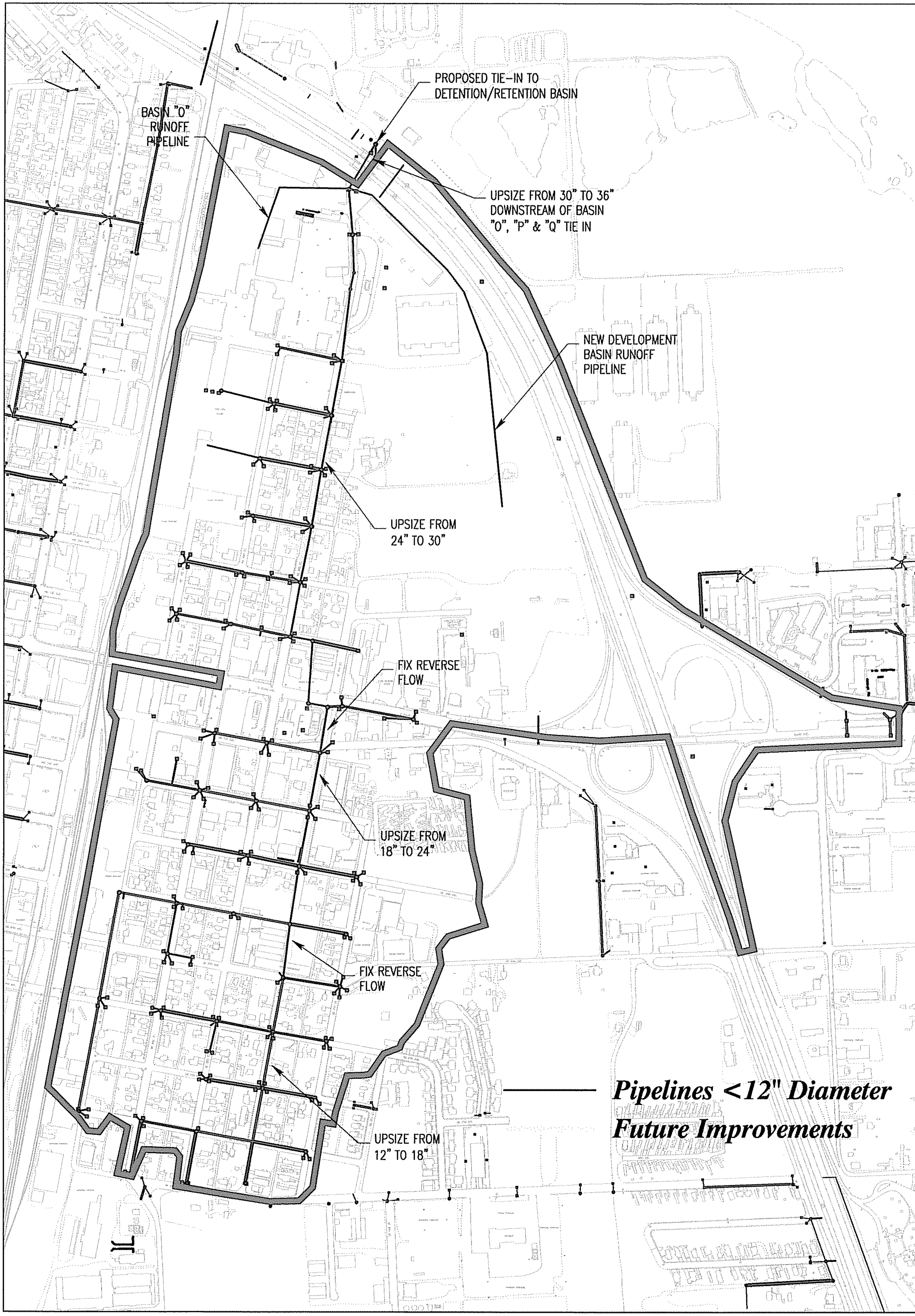
KELLER ASSOCIATES

131 SW 5th Avenue, Suite A
Meridian, Idaho 83642
(208) 288-1992

City of
Ontario, Oregon

ONTARIO STORM WATER MASTER PLAN
HFF WATERSHED FUTURE DEVELOPMENT

FIGURE NO.
4



PROJECT NO. 197032

FILENAME Strmwtrsys.dwg

KELLER ASSOCIATES

131 SW 5th Avenue, Suite A
Meridian, Idaho 83642
(208) 288-1992

City of
Ontario, Oregon

ONTARIO STORM WATER MASTER PLAN

HFF STORM SYS. FUTURE DEVELOPMENT

FIGURE NO.
5

Figure 6

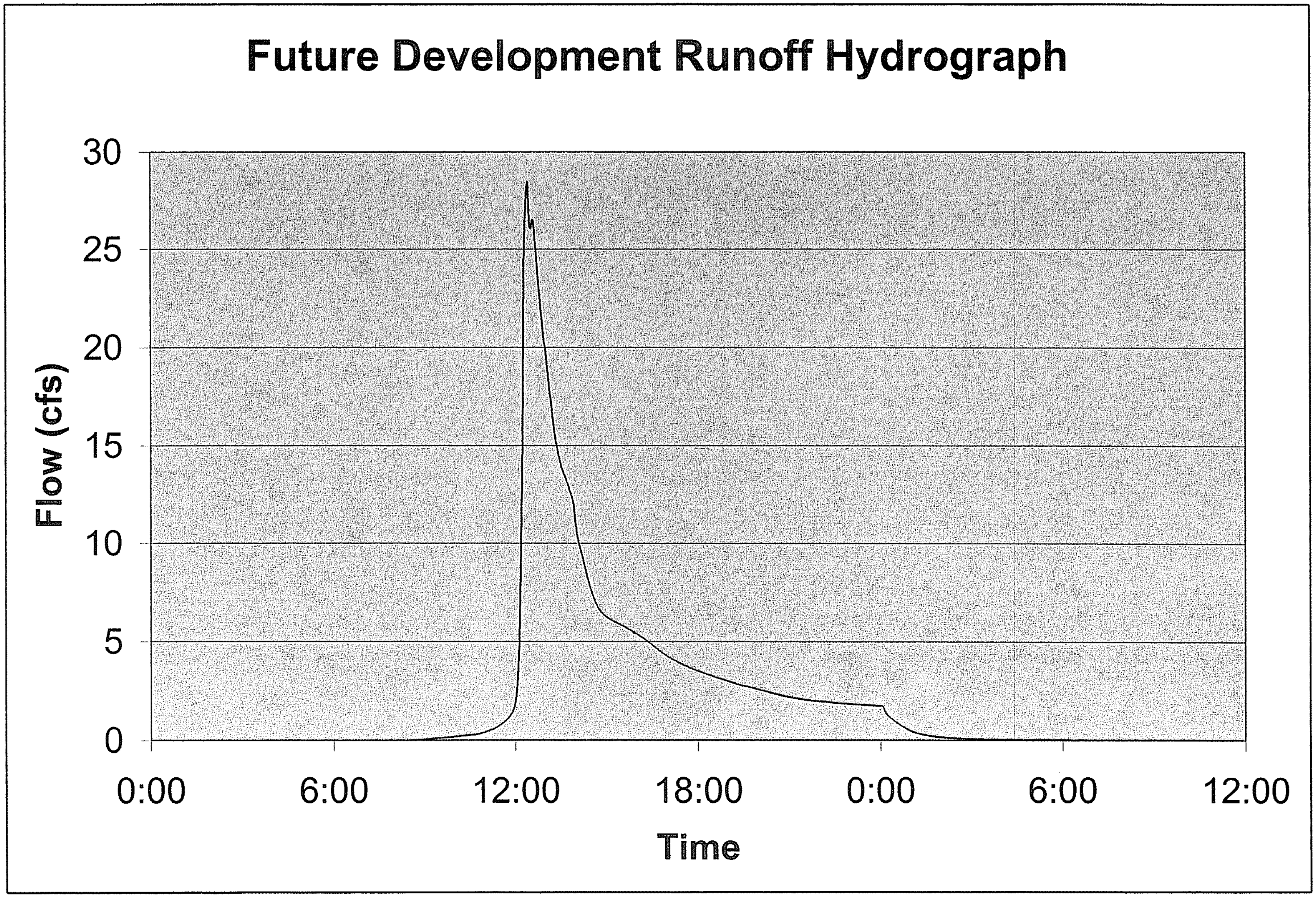

Future Development Runoff Hydrograph
Flow (cfs)
30
25
20
15
10
5
0
0:00
6:00
12:00
18:00
0:00
6:00
12:00
Time

Figure 7

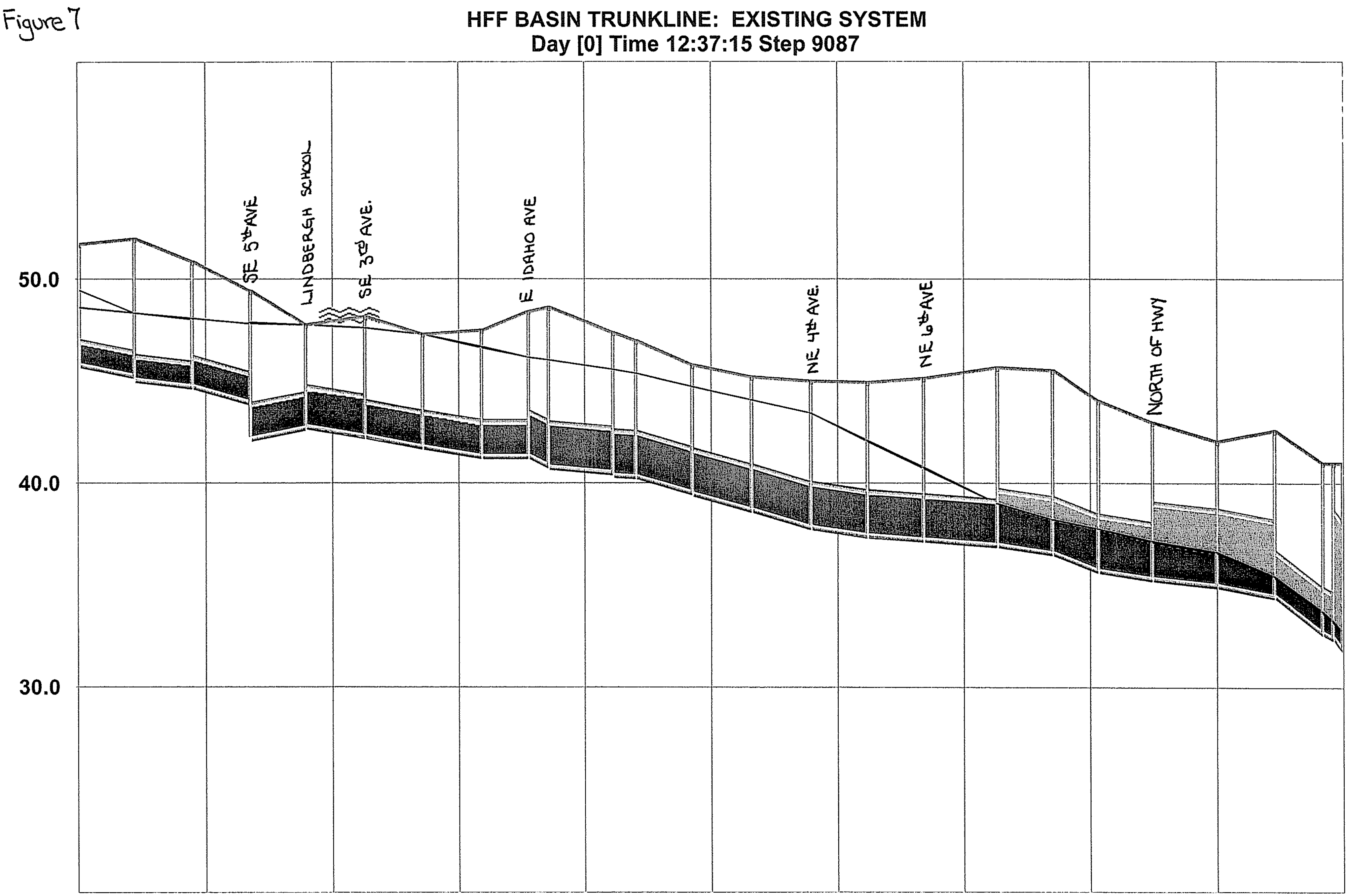
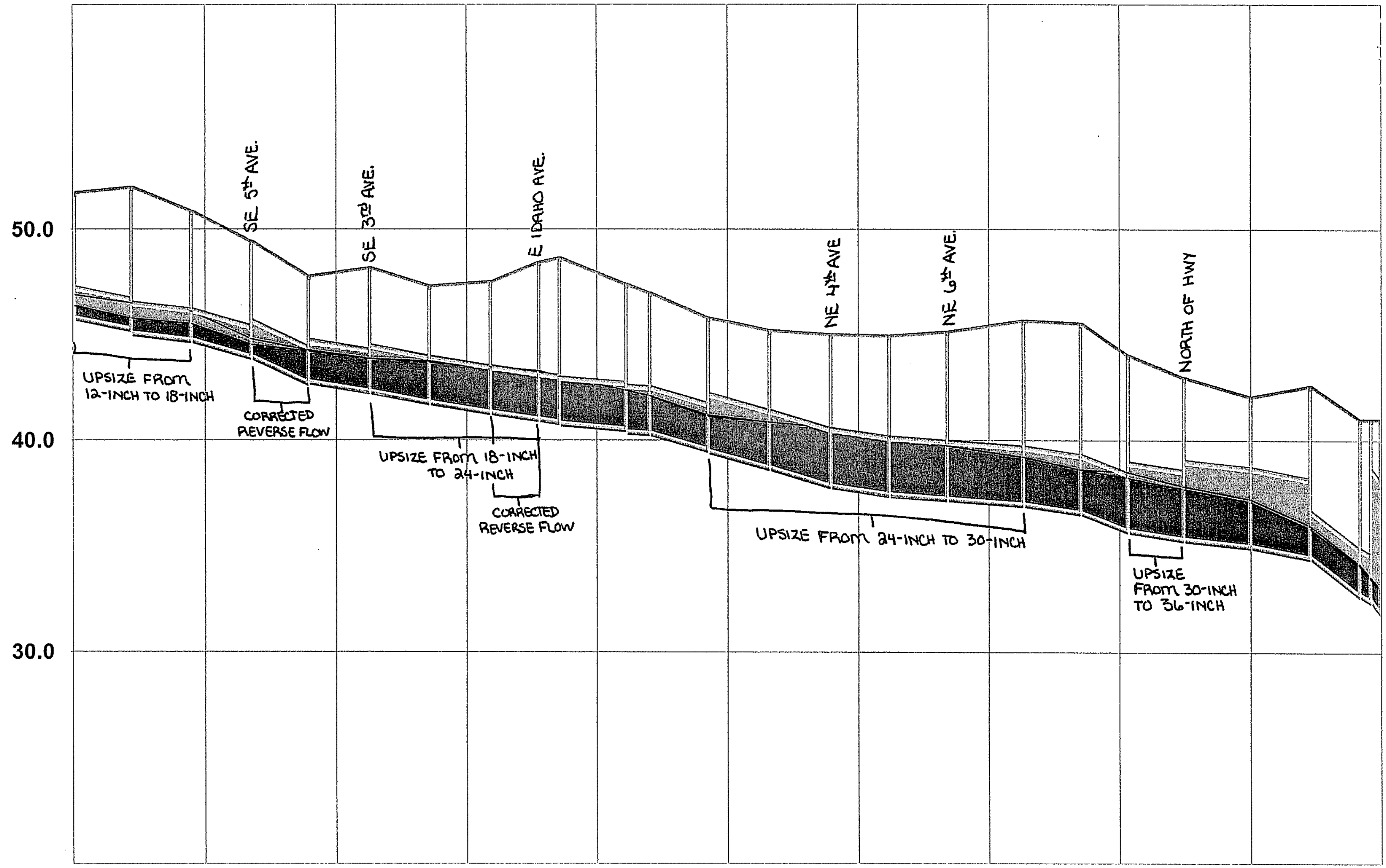

HFF BASIN TRUNKLINE: EXISTING SYSTEM
Day [0] Time 12:37:15 Step 9087
50.0
40.0
30.0
SE 5th AVE
LINDBERGH SCHOOL
SE 3rd AVE.
E IDAHO AVE
NE 4th AVE
NE 6th AVE
NORTH OF HWY

Figure 8

HFF BASIN TRUNKLINE WITH FUTURE DEVELOPMENT AND PIPELINE IMPROVEMENTS

Day [0] Time 12:40:15 Step 9123



Appendix C

Model Parameters

Table 1

15 Basin Parameters

Basin	Area (acre)	Impervious %	Length (ft)	Width (ft)	Slope	CN	Time of Conc (min)	Notes
A	13.14	26.82%	800	715	0.003	74	23	
B	19.34	23.67%	750	1123	0.003	73	22	
C	24.64	23.96%	1000	1073	0.003	73	27	
D	12.08	19.27%	840	627	0.002	71	31	
E	33.01	29.55%	800	1797	0.003	72	26	
F	27.14	55.33%	1100	1075	0.002	83	46	
G	33.83	29.30%	1020	1445	0.002	75	38	
H	32.57	53.70%	900	1576	0.003	83	33	
I	20.15	65.00%	450	1950	0.004	86	15	
J	82.14	0.00%	2600	1376	0.002	86	240	Highway
K	11.50	70.00%	500	1002	0.004	91	17	Onsite
L	4.35	65.00%	100	1893	0.008	88	5	Onsite
M	0.98	70.00%	500	86	0.002	94	22	Onsite
N	1.41	64.50%	100	614	0.001	86	8	Onsite
O	13.75	90.00%	500	1198	0.008	94	22	To WTP

Table 2

55 Basin Parameters

Basin	Area (acre)	% Impervious	Length (ft)	Width (ft)	Slope	Cn	Tc (min)	Notes
1	4.37	13.50%	350	543.93	0.002	69	20.49	
2	1.68	39.13%	100	729.70	0.0024	78	13.22	
3	0.98	32.61%	175	244.45	0.003	76	14.04	
4	2.33	35.66%	200	508.22	0.004	77	11.3	
5	1.89	23.29%	225	366.32	0.006	73	10.43	
6	3.78	14.49%	300	548.21	0.003	70	16.58	
7	2.48	29.34%	250	431.78	0.001	75	20.06	
8	1.74	27.17%	100	759.95	0.002	74	14.22	
9	2.18	24.88%	100	950.49	0.002	73	14.22	
10	4.05	26.83%	250	705.81	0.003	74	16.42	
11	1.41	65.00%	100	613.85	0.001	86	8	onsite
12	3.53	21.68%	200	767.88	0.002	72	18.85	
13	2.50	26.14%	100	1090.35	0.003	74	12.1	
14	2.77	24.23%	125	963.97	0.002	73	14.22	
15	1.87	26.95%	75	1083.45	0.002	74	15.3	
16	4.23	15.26%	250	736.39	0.003	70	16.98	
17	8.50	19.68%	300	1234.20	0.002	71	25.16	
18	3.06	27.13%	175	762.50	0.002	74	13.59	
19	3.55	26.15%	125	1237.52	0.002	74	17.62	
20	4.19	23.26%	250	729.86	0.003	73	15.8	
21	1.92	30.41%	100	836.96	0.002	75	14.22	
22	3.93	28.72%	250	685.28	0.003	74	13.59	
23	3.67	20.63%	150	1066.49	0.004	72	12.98	
24	9.36	23.44%	300	1359.39	0.005	65	15.08	
25	0.83	55.61%	150	240.66	0.001	65	18.77	
26	2.86	33.65%	200	622.55	0.002	65	16.09	
27	5.43	20.40%	350	675.71	0.003	72	17.33	
28	3.47	32.33%	250	605.04	0.002	76	16.05	
29	3.37	54.07%	250	586.79	0.004	83	16.05	
30	2.25	55.61%	100	981.83	0.001	83	5.78	
31	5.44	22.27%	250	947.32	0.001	76	22.66	
32	4.35	65.00%	100	1893.28	0.008	88	5	onsite
33	6.72	31.10%	350	835.96	0.003	75	20.32	
34	0.98	70.00%	500	85.77	0.002	94	22	onsite
35	2.80	66.86%	150	811.70	0.005	87	11.02	
36	7.68	72.15%	250	1337.51	0.002	89	32.65	
37	5.12	46.66%	650	342.87	0.001	80	45.99	
38	4.83	64.80%	250	841.96	0.001	86	23.96	
39	11.50	40.00%	500	1002.31	0.004	91	17	onsite
40	15.45	17.71%	500	1345.98	0.002	71	31.64	
41	2.82	83.46%	150	818.73	0.004	87	10.78	
42	2.33	65.58%	150	677.07	0.002	87	14.97	
43	2.95	13.24%	150	856.01	0.005	69	9.86	
44	4.00	17.23%	250	696.36	0.002	71	16.05	
45	3.37	23.84%	150	978.87	0.001	73	20.25	
46	2.91	48.41%	150	845.54	0.003	81	13.68	
47	6.07	10.97%	300	881.42	0.005	76	13.92	
48	5.79	12.19%	250	1009.22	0.005	69	12.18	
49	1.40	32.26%	100	610.59	0.002	76	12.29	
50	8.21	100.00%	500	715.04	0.002	95	27.05	
51	7.95	87.45%	300	1154.36	0.002	94	20.56	
52	3.15	16.05%	300	457.04	0.004	70	14.67	
53	82.14	0.00%	2600	1376.18	0.002	72	240	Hwy
54	20.15	65.00%	450	1950.19	0.004	86	15	
55	13.75	90.00%	500	1198.28	0.008	94	22	to WTP

Table 3

Future 17 Basin Parameters

Basin	Acre	Imp %	Length (ft)	Width (ft)	Slope	CN	Tc	Notes
A	13.14	26.82%	800	715	0.003	74	23	
B	19.34	23.67%	750	1123	0.003	73	22	
C	24.64	23.96%	1000	1073	0.003	73	27	
D	12.08	19.27%	840	627	0.002	71	31	
E	33.01	29.55%	800	1797	0.003	72	26	
F	27.14	55.33%	1100	1075	0.002	83	46	
G	33.83	29.30%	1020	1445	0.002	75	38	
H	32.57	53.70%	900	1576	0.003	83	33	
I	20.15	65.00%	450	1950	0.004	86	15	
J	46.49	0.00%	2600	779	0.002	86	240	Highway
K	11.50	70.00%	500	1002	0.004	91	17	Pilot Truck Stop, Onsite Retention
L	4.35	65.00%	100	1893	0.008	88	5	Trailer Park, Onsite Retention
M	0.98	70.00%	500	86	0.002	94	22	Taco Bell, Onsite Retention
N	1.41	64.50%	100	614	0.001	86	8	Industrial, Onsite Retention
O	**13.75**	**90.00%**	**500**	**1198**	**0.008**	**94**	**22**	**Connect to Stormwater System**
P	**22.72**	**80.00%**	**950**	**1042**	**0.002**	**92**	**22**	**Future Development**
Q	**12.90**	**80.00%**	**600**	**936**	**0.002**	**92**	**13**	**Future Development**

KELLER
ASSOCIATES
131 S.W. 5th Avenue, Suite A • Meridian, ID 83642

May 11, 1999

Mr. Carl P. Malone, P.E.
Public Works Director
City of Ontario
444 S.W. 4th Street
Ontario, OR 97914

RE: Regional Stormwater Facility Evaluation for the ODOT Gravel Pit Site

Dear Carl:

Please find enclosed the evaluation for creating a regional or citywide stormwater facility at the ODOT Gravel Pit Site in Ontario. We find that physical constraints limit the amount of stormwater that can be conveyed to the site via gravity. However, the site is currently being used for stormwater purposes and should continue to provide this function in the future, so the City should have a continued interest in site modifications. Additional stormwater could be discharged to the site, primarily from undeveloped areas to the south, but it does not provide the "huge" regional or citywide solution that we discussed.

(541) 963-1351
REGION 5, GEOLOGY (LA GRANDE)

We made contact with Richard Fredricksen, ODOT, regarding the status of the site reclamation plan. He noted that roadway project bidding is scheduled for next year and the construction contract will be set up as a "contractor furnished source"; meaning the contractor can use different approved source sites, and is not limited to this particular site. This site will be an available option to the contractor. Mr. Fredricksen did note that there are still some unresolved concerns regarding the adequacy of haul roads. ODOT plans to develop the site reclamation plan, but it won't be completed until about August 1999.

We hope this evaluation provides the information you need at this time. We can incorporate these findings into the City's Stormwater Management Plan at a later date. Please call me at (208) 288-1992 if you have any questions.

Sincerely,

KELLER ASSOCIATES, INC.

Rod J. Linja, P.E.
Project Manager

Enclosure

A Company of Professional Engineers and Scientists
Ph: (208) 288-1992 • Fax: (208) 288-1999

REGIONAL STORMWATER FACILITY EVALUATION FOR THE ODOT GRAVEL PIT SITE IN ONTARIO

PURPOSE

The Oregon Department of Transportation (ODOT) plans to use an existing site within the City of Ontario as a materials source. ODOT plans to excavate approximately 200,000 m^3 of material from the site for construction of a new roadway project. The site currently serves as a stormwater facility within the City. The purpose of this evaluation is to assess the potential of creating a major (regional) stormwater facility within the City. If possible, the City of Ontario could coordinate the overall plan with ODOT to create this regional stormwater disposal and BMP facility. Multi-purpose uses, primarily recreation and wildlife, could also be incorporated into the concept. Therefore, the primary purpose of this evaluation is to assess the feasibility of creating this regional or citywide facility given the existing natural and man-made constraints.

EXISTING CONDITIONS AND SITE DESCRIPTION

The existing site is approximately 37 acres and is located south of the intersection of 9th and Cedar. The site acts as a "retention" area and there are three stormwater discharges into the site. The existing sub-basin that drains to the site is approximately 185 acres. Interstate 84 bounds the sub-basin to the east. Relatively flat slopes and natural grades limit the sub-basin boundary in all other directions. The three discharges into the site are described as follows; (1) a 42-inch discharge along the north boundary, (2) a 24-inch discharge along the west boundary, and (3) a small diameter discharge from the trailer park in northeast corner. The discharges along the north and west boundary flow within a ten-foot storm easement through the ODOT site.

The north discharge collects stormwater mainly along SE 9th Street. Pipes vary in size between 12 and 30 inches with a 42-inch outlet pipe. Some stormwater is collected east of SE 6th Street and directed to this system. The discharge on the west boundary is composed mainly of surface water runoff from SE 6th Street (south of 9th) and also stormwater from the industrial area in the vicinity. Pipes vary in size from 18 to 24 inches for this system with a 24-inch outlet pipe to the site. The outlet near the northeast corner is a small diameter pipe that collects stormwater runoff from the trailer park site. Figure 2 shows the gravel pit site and the discharge pipe locations.

We understand that there are no direct outlets from the site and runoff that enters the site is retained and eventually infiltrates into the ground or is lost through evaporation. Aerial mapping does show a drainage way just to the south of the site. Our effort did not include field verification to confirm whether or not there is any "connectivity" between this drainage way and the site.

Regional Facility Evaluation

The existing site is large (37 acres) compared to the existing drainage sub-basin (185 acres). In order to create an effective regional solution for the City, it is desirable to increase the size of the drainage sub-basin; thereby directing considerably more stormwater to the site. There are several natural and man-made limitations that effect the feasibility of doing this such as groundwater elevations, sub-basin and site topography, existing storm drain pipelines, and a goal to avoid the construction of stormwater pump stations.

Further, the concept behind utilizing this site as a regional facility involves excavating below groundwater levels and creating some wetland ponds to retain stormwater in the area. These improvements could create a park atmosphere, probably focusing on a wildlife refuge with trails, benches, and, perhaps, interpretive signs and opportunities. This concept would require more "relief" and contouring compared to a flat or level park to be used for recreation, ball fields, and stormwater. Either option could attract people and wildlife to this location. Therefore, the goal is to create a multi-use facility for stormwater and other purposes.

Several parameters acted as constraints to size the proposed or enlarged sub-basin in utilizing the site as a regional stormwater facility and are summarized below.

- ***Groundwater***: The gravel pit site has low spots in the southern portion that hold water year round. These elevations were assumed to be the groundwater level. The water level probably fluctuates with the elevation of the Snake River throughout the year. The invert elevation of any new stormwater discharges should be above this groundwater elevation to avoid stormwater storage in the piped system and a surcharged condition.

- ***Gravity Flow System***: The sub-basin should allow water to reach the site via gravity. The land in the area is very flat with natural depressions scattered throughout. Minimum slopes must be used on storm drain systems to convey water from the outer sub-basin boundaries to the project site. Therefore, existing ground elevations near the outer sub-basin boundaries compared to the site groundwater elevations provide data on "available fall or head". This data is used to determine how the sub-basin can be enlarged. Pumping stations could be constructed to further expand sub-basin boundaries, but is not recommended due to the high capital, operation, and maintenance costs associated with stormwater pump stations. Large pumps are normally required to handle the design storm and reliability concerns during power outages typically result in the need for standby power systems.

  To the north, a natural crown of the land exists between SE 5th and SE 7th Streets. This makes it difficult to route water to the south because of trying to reverse the natural flow of the runoff. To the east, Interstate 84 acts as a sub-basin boundary. To the south, the Snake River is relatively close with depressed areas that maintain surface water throughout a majority of the year. Finally to the west, some depressed or low-lying areas have been developed for industrial uses. The Union Pacific Railroad parallels this land further to the west. Routing stormwater to the site via gravity is, perhaps, the most difficult aspect to deal with, due to reversing storm drain flows compared to natural land flow directions.

- ***Existing Infrastructure***: To minimize costs of developing this site, the existing stormwater infrastructure played a role in developing the expanded sub-basin boundary. Large diameter pipe (varying between 18" and 30") exists along SE 9th Street. A 42-inch diameter outlet to the gravel pit site exists along the north boundary. The invert elevation of this 42-inch pipe at the site is only a couple feet above the estimated groundwater elevation. Even if this storm drain system was replaced, the resultant increase in the sub-basin boundary is not significant. Therefore, this system should be retained avoiding the high replacement costs of constructing new and deeper storm drains. Sub-basin area expansion is based on connecting smaller storm drains to the existing system.

- ***Site Layout & Contours***: With the excavation of 200,000 m^3 of the material by ODOT, rough calculations show that significant cuts must be made and the resultant contours will be very low in elevation. Elevations above groundwater may range between 1 and 2 feet. The contours of the excavated land would be approximately 10 feet below the existing grades on 9th Street and at the trailer park in the northeast corner. The site may result in a large depression and present difficulties in developing a "natural looking" park and wildlife refuge.

ENLARGED SUB-BASIN FINDINGS

By using the described constraints, an enlarged sub-basin was delineated. The proposed sub-basin, via gravity flow only, is approximately 310 acres. The most significant addition to the sub-basin is undeveloped and rural land to the south totaling about 85 acres. As development occurs, storm drains could be constructed to direct stormwater runoff to this site. This would require a network of underground pipes, probably constructed as development occurs. To the north, only about 40 acres is added to the existing sub-basin. New storm drains would either connect to existing pipes or a new line may be required with another discharge to the site (depends on system capacity, density of development, etc.). Figure 2, the attached 11"x17" aerial photo shows the existing and enlarged sub-basin boundary.

SUMMARY AND RECOMMENDATIONS

The concept of constructing a regional stormwater facility to serve a major part of the City is limited, primarily due to the relatively flat topography within the City and high groundwater elevations influenced by the Snake River. (Note, even if groundwater levels were lower, it would be impractical to build extremely deep storm drains to the site). The site currently serves as a stormwater retention facility and should continue to serve this purpose in the future. Our analysis yielded that additional stormwater could be routed to this site, primarily from undeveloped land to the south.

Therefore, the City should have a continued interest in the site. The large site area could present a park opportunity to the City (more so than a huge stormwater facility). However, the excavation of 200,000 m^3 of material requires major cuts and will create a large depression that may limit some park uses. The City should evaluate park needs and goals in this area and determine what type of facility is desired. Further, the City should request a site reclamation plan from ODOT to review finished contours and impacts to the site that may impact future stormwater and park uses.


Scale: 1"=200'
ODOT Gravel Pit Site
TRAILER PARK
OUTLET
24" OUTLET
12" OUTLET
10' STORM EASMENT
SE 9th ST.

FIGURE 1

Existing Sub basin
PROPOSED SUB BASIN
ODOT Gravel Pit
Scale: 1"=500'
SE 5th ST.
SE 6th ST.
SE 7th ST.
RAILROAD
I-84
SE 9th ST.
SE 6th ST.

FIGURE 2

City of Ontario
Stormwater Study
Model Sub-basin Descriptions

Basin #	Drainage Basin	Location Description
A1	Double Trunkline Basin	SW 10th Ave
A2	Double Trunkline Basin	SW 9th Ave
A3	Double Trunkline Basin	SW 8th Ave
A4	Double Trunkline Basin	SW 7th Ave
A5	Double Trunkline Basin	SW 6th Ave
A6	Double Trunkline Basin	SW 6th Ave to SW 4th Ave
A7	Double Trunkline Basin	SW 4th Ave to SW SW 2nd Ave
A8	Double Trunkline Basin	Tiger Gym area
A9	Double Trunkline Basin	SW 3rd Ave north to NW 1st Ave and
A10	Double Trunkline Basin	Park by St. Peter's Catholic School
A11	Double Trunkline Basin	NW 8th St and NW NW 1st Ave
A12	Double Trunkline Basin	NW 1st Ave to NW 3rd Ave
A13	Double Trunkline Basin	NW 3rd Ave to NW 5th Ave
A14	Double Trunkline Basin	NW 5th Ave to NW 7th Ave
A15	Double Trunkline Basin	Outfall to Snake River
B1	Double Trunkline Basin	From Irrigation canal east to SW Park Blvd
B2	Double Trunkline Basin	SW 11th St south of SW 12th Ave
B3	Double Trunkline Basin	South College Basin/Ball parks
B4	Double Trunkline Basin	North half of College
B5	Double Trunkline Basin	SW 11th Ave east of SW 4th St.
B6	Double Trunkline Basin	SW 7th St south of tennis courts to SW 5th Ave
B7	Double Trunkline Basin	Oregon St and Fortner St.
C0	Park Boulevard Basin	Irrigation runoff from West of Alameda
C1	Park Boulevard Basin	Alameda and NW of soccer fields and elementary school
C2	Park Boulevard Basin	North of Alameda soccer fields
C3	Park Boulevard Basin	SW 11th St from SW 7th Ave south to SW 12th Ave
C4	Park Boulevard Basin	Southern end of Park Boulevard from SW 5th Ave to the south end
C5	Park Boulevard Basin	SW 5th Ave from just south of SW 4th Ave south to SW 7th Ave
C6	Park Boulevard Basin	Mall
C7	Park Boulevard Basin	SW 4th Ave
C8	Park Boulevard Basin	North of SW 4th Ave to south of High School/W Idaho Ave
C9	Park Boulevard Basin	High school (W Idaho Ave) north
C10	Park Boulevard Basin	Loveridge Area
C11	Park Boulevard Basin	Fairgrounds
C12	Park Boulevard Basin	Open Channel and outfall to Snake River
D0	Dork Canal/Verde St Basin	Stewart Carter Ditch
D1	Dork Canal/Verde St Basin	Winger Drive Trailer Court east to Sunset Drive
D2	Dork Canal/Verde St Basin	South of SW 4th Ave to north end of trailer court
D3	Dork Canal/Verde St Basin	Irrigation line along Verde St. to W Idaho Ave
D4	Dork Canal/Verde St Basin	North of SW 4th Ave north to north of W Idaho Ave
D5	Dork Canal/Verde St Basin	Between D2 and D3
D6	Dork Canal/Verde St Basin	West of Verde and with NW 4th Ave in middle and NW 16th St
D7	Dork Canal/Verde St Basin	East of Verde and Arata Way
D8	Dork Canal/Verde St Basin	Hayden Subdivision
D9	Dork Canal/Verde St Basin	Hunter Lane and West of Verde
D10	Dork Canal/Verde St Basin	Dork Canal and outfalls
E0	Waterford Basin	South of SW 4th Ave-Agricultural runoff
E1	Waterford Basin	Waterford Subdivision
E2	Dorian Street Basin	Ridge Way Subdivision/Dorian Basin
E3	NW 7th Ave Basin	Hunter Lane and top of the bluff above the Dork Canal
SW 1	SW 4th Avenue Basin	SW 4th Ave western half
SW 2	SW 4th Avenue Basin	Irrigation runoff from Skylane and SW 6th Ave, connected to SW 4th
SW3	SW 4th Avenue Basin	SW 24th St west to SW 30th St.
W1	Walmart Basin	East of Super 8 and Denny's
W2	Walmart Basin	Walmart and north half of parking lot

City of Ontario
Stormwater Study
Model Sub-basin Descriptions

Basin #	Drainage Basin	Location Description	
W3	Walmart Basin	North of Idaho Avenue and south half of parking lot	
Wdenny	Walmart Basin	Denny's and parking lot-onsite	
Wonsite	Walmart Basin	Parking in the SE corner of Walmart Basin	
K1	Kmart Basin	East of Kmart	
K2	Kmart Basin	Kmart, Kmart and Ernst Parking Lots	
K3	Kmart Basin	Ernst and Ernst Parking Lot	
A	Heinz Frozen Foods Basin	South of SE 7th Ave and north of SE 9th Ave	
B	Heinz Frozen Foods Basin	North of SE 7th Ave and south of SE 5th Ave	
C	Heinz Frozen Foods Basin	North of SE 5th Ave and south of SE 4th Ave	
D	Heinz Frozen Foods Basin	SE 1st Street south of SE 4th Ave	
E	Heinz Frozen Foods Basin	SE 3rd Ave to SE 2nd Ave	
F	Heinz Frozen Foods Basin	SE 1st Ave and Idaho Avenue	
G	Heinz Frozen Foods Basin	North of Idaho Avenue and south of NE 2nd Ave	
H	Heinz Frozen Foods Basin	North of NE 3rd Ave and south of NE 5th Ave-Americold	
I	Heinz Frozen Foods Basin	North of NE 5th Ave and South of I-84	
J	Heinz Frozen Foods Basin	Independent System-I-84	
K	Heinz Frozen Foods Basin	Onsite-Pilot Truck Stop north of Idaho Ave and west of I-84	
L	Heinz Frozen Foods Basin	Onsite-Trailer Court east of SE 2nd Ave	
M	Heinz Frozen Foods Basin	Onsite-Taco Bell	
N	Heinz Frozen Foods Basin	Onsite-east of RR and south of SE 5th Ave	
O	Heinz Frozen Foods Basin	Independent System-Heinz Frozen Foods	
P	Heinz Frozen Foods Basin	Future Development-Existing Onsite	
Q	Heinz Frozen Foods Basin	Future Development-Existing Onsite	
F0	Ontario Periphery	East of SW 4th Street, South of SW 10th Ave and North of SW 18th Ave	Not Modeled
F1	Ontario Periphery	West of the RR and South of town, drains ESE	
F2	Ontario Periphery	Stewart Carter Ditch	
F3	Ontario Periphery	WNW of the Waterford Subdivision	
F4	Ontario Periphery	North of Waterford Subdivision	
F5	Ontario Periphery	North of Hunter Lane extension and NW 7th Ave	
F6	Ontario Periphery	North of town and the Dork Canal and SW of the freeway	
F7	Ontario Periphery	North of Freeway and north of Walmart	
F8	Ontario Periphery	South of Kmart and East of freeway	
F9	Ontario Periphery	South of Idaho and West of freeway	
F10	Ontario Periphery	South of HFF basin and East of RR	
F11	Ontario Periphery	South of SW 19th Ave and East of RR	
Airport	Ontario Periphery	Airport and surrounding golf course	
SS1	Sanitary Sewer	S Oregon and SW 10th St	
SS2	Sanitary Sewer	SW 4th Ave just west of RR	
SS3	Sanitary Sewer	NW 6th Ave just west of RR	
SS4	Sanitary Sewer	Oregon St. and NW 1st St	
SS5	Sanitary Sewer	NW 7th Ave and Fortner St.	
SS6	Sanitary Sewer	Parking on SW 3rd Ave	
SS7	Sanitary Sewer	SW 11th St	
SS8	Sanitary Sewer	SW 15th St and SW 2nd Ave	
O1	Onsite	River Street Basin	
O2	Onsite	French Drain on Alameda	
O3	Onsite	Alameda Soccer Field	
O4	Onsite	Kiwanis Park and Pond	

City of Ontario
Storm Water Study
Runoff Basin Parameters

	Model	Area		Parameters					Time of Conc		
Basin #	Node	sq ft	acre	Impervious %	Length (ft)	Width (ft)	Slope	CN	min	Model	
A1	T2	305,640	7.02	24.14%	490	624	0.0040	76	22.0	Double RR-Pipe Fill.xp	Combined System.xp
A2	T5	509,586	11.70	24.14%	600	849	0.0040	76	24.8	Double RR-Pipe Fill.xp	
A3	T11	545,621	12.53	24.14%	700	779	0.0040	76	20.7	Double RR-Pipe Fill.xp	
A4	T16	650,596	14.94	27.59%	550	1183	0.0020	77	20.8	Double RR-Pipe Fill.xp	
A5	T19	691,479	15.87	27.59%	600	1152	0.0030	77	17.6	Double RR-Pipe Fill.xp	
A6	T26	1,000,258	22.96	27.59%	700	1429	0.0037	77	19.5	Double RR-Pipe Fill.xp	
A7	T35	1,214,564	27.88	27.59%	1100	1104	0.0020	77	34.8	Double RR-Pipe Fill.xp	
A8	T41	677,899	15.56	27.59%	525	1291	0.0010	77	25.0	Double RR-Pipe Fill.xp	
A9	T45	2,030,378	46.61	27.59%	1375	1477	0.0040	77	33.2	Double RR-Pipe Fill.xp	
A10	T63	1,003,568	23.04	27.59%	550	1825	0.0030	77	24.5	Double RR-Pipe Fill.xp	
A11	T67	377,395	8.66	24.14%	400	943	0.0050	76	7.0	Double RR-Pipe Fill.xp	
A12	T72	2,481,993	56.98	24.14%	1400	1773	0.0020	76	38.6	Double RR-Pipe Fill.xp	
A13	T89	2,945,504	67.62	24.14%	1200	2455	0.0030	76	29.3	Double RR-Pipe Fill.xp	
A14	T105	1,101,948	25.30	20.69%	600	1837	0.0040	75	22.1	Double RR-Pipe Fill.xp	
A15	T113	3,001,480	68.90	20.69%	800	3752	0.0030	75	32.8	Double RR-Pipe Fill.xp	
B0	DT1	2,617,957	60.10	Minimal	4500	582	0.0010	65	137.6	DT Irrigation-Pipe Fill.xp	
B1	DT4	4,010,371	92.07	9.36%	1800	2228	0.0150	71.7	45.0	DT Irrigation-Pipe Fill.xp	
B2	DT12	1,591,460	36.53	13.08%	1075	1480	0.0033	72.8	31.4	DT Irrigation-Pipe Fill.xp	
B3	DT20	3,486,845	80.05	5.54%	2800	1245	0.0160	70.6	43.0	DT Irrigation-Pipe Fill.xp	
B4	DT28	2,975,018	68.30	0.87%	2000	1488	0.0122	69.3	46.0	DT Irrigation-Pipe Fill.xp	
B5	DT19	144,295	3.31	20.54%	250	577	0.003	75.0	13.8	DT Irrigation-Pipe Fill.xp	
B6	DT30	675,414	15.51	39.01%	480	1407	0.0048	80.3	18.1	DT Irrigation-Pipe Fill.xp	
B7	DT45	139,297	3.20	10.50%	645	216	0.001	75	14	DT Irrigation-Pipe Fill.xp	
C0	N38	3,242,779	74.44	3.26%	3915	828	0.0037	69.9	71.9	Park Blvd BasinCalib.xp	
C1	N38	949,705	21.80	16.91%	800	1,187	0.0043	73.9	21.7	Park Blvd BasinCalib.xp	
C2	N4	1,294,954	29.73	13.11%	900	1,439	0.0117	72.8	21.8	Park Blvd BasinCalib.xp	
C3	N8	1,485,409	34.10	20.04%	1,150	1,292	0.0069	74.8	29.9	Park Blvd BasinCalib.xp	
C4	N10	754,467	17.32	21.59%	750	1,006	0.0156	75.3	20.9	Park Blvd BasinCalib.xp	
C5	N15	1,438,419	33.02	28.84%	1,400	1,027	0.0059	77.4	27.1	Park Blvd BasinCalib.xp	
C6	N35	1,077,715	25	37.18%	800.00	1187.13	0.0043	73.9	21.7	Park Blvd BasinCalib.xp	
C7	N27	991,519	22.76	65.92%	1,300	763	0.0143	88.1	26.0	Park Blvd BasinCalib.xp	
C8	N44	3,066,449	70.40	32.67%	1,400	2,190	0.0119	78.5	30.0	Park Blvd BasinCalib.xp	
C9	N57	3,491,343	80.15	21.01%	1,200	2,076	0.0057	75.1	30.5	Park Blvd BasinCalib.xp	
C10	N76	2,480,949	56.95	21.94%	1,000	2,481	0.0051	75.4	36.4	Park Blvd BasinCalib.xp	
C11	N78	3,723,900	85.49	4.95%	1,400	2,660	0.0239	70.4	26.7	Park Blvd BasinCalib.xp	
C12	Canal	1,505,037	34.55	5.97%	1,100	1,368	0.0037	70.7	67.1	Not Modeled	
D0	A	4,472,423	102.67	0%	4000	1118	0.012	69	34.4	Stewart Carter Oct 1.xp	
D1	B	1,513,339	34.74	17.69%	700	2,162	0.0024	74.1	47.0	Stewart Carter Oct 1.xp	
D2	D	611,822	14.05	76.48%	550	1112	0.0065	91.2	17.9	Stewart Carter Oct 1.xp	
D3	J	1,132,465	26.00	19.74%	800	1416	0.0105	74.7	27.1	Stewart Carter Oct 1.xp	
D4	N6	1,875,684	43.06	18.59%	650	2886	0.0063	74.4	22.9	Vstorm.xp	
D5	N9	488,656	11.22	38.35%	600	814	0.0055	80.1	19.9	Vstorm.xp	
D6	N24	1,608,939	36.94	21.20%	1100	1463	0.0010	75.1	36.0	Vstorm.xp	
D7	M	838,169	19.24	23.05%	550	1524	0.0100	75.7	18.2	Stewart Carter Oct 1.xp	
D8	N45	1,337,477	30.70	14.50%	800	1672	0.0137	73.2	30.9	Vstorm.xp	
D9	N40	2,262,408	51.94	13.59%	1200	1885	0.0054	72.9	26.1	Vstorm.xp	
D10	Dork	6,837,606	156.97	2.90%	3000	2279	0.0117	69.8	177.0	Not Modeled	
E0	B	54,547,693	1,252.24	Minimal	8740	6241	0.0010	65.0	276.1	Waterford.xp	
E1	D	6,106,408	140.18	5.41%	3500	1745	0.0010	70.6	33.6	Waterford.xp	
E2	D4	3,098,657	71.14	12.30%	2000	1549	0.0090	72.6	19.0	Dorian.xp	
E3	NW2	1,554,194	35.68	9.17%	1400	1110	0.0017	71.7	9.0	NW 7th Ave.xp	
SW1	Q	969,720	22.26	90.0%	5910	190	0.0035	95.1	41.8	SW 4th Ave.xp	
SW2	F3	1,281,487	29.42	13.65%	1010	1269	0.0030	73.0	37.7	SW 4th Ave.xp	
SW3	M	1,908,878	43.82	16.35%	1120	1704	0.0100	73.7	46.0	SW 4th Ave.xp	
F0		3,376,393	77.51	Minimal	1000	3376	0.001	65	23.2	Not Modeled	
F1		24,499,510	562.43	Minimal	2800	8750	0.0020	65.0	57.9	Not Modeled	
F2		4,589,060	105.35	Minimal	800	5736	0.0180	65.0	3.9	Not Modeled	
F3		16,580,823	380.64	Minimal	1800	9212	0.0010	65.0	49.3	Not Modeled	
F4		3,388,723	77.79	Minimal	800	4236	0.0160	65.0	4.3	Not Modeled	
F5		2,005,565	46.04	Minimal	800	2507	0.0410	65.0	2.7	Not Modeled	
F6		8,512,110	195.41	Minimal	3000	2837	0.0010	65.0	88.5	Not Modeled	
F7		10,061,447	230.98	Minimal	3000	3354	0.0010	65.0	88.5	Not Modeled	
F8		10,568,762	242.63	Minimal	2130	4962	0.0150	65.0	15.4	Not Modeled	
F9		14,271,205	327.62	Minimal	4100	3481	0.0020	65.0	88.0	Not Modeled	
F10		4,896,141	112.40	Minimal	2600	1883	0.0010	65.0	75.5	Not Modeled	
F11		29,680,560	681.37	Minimal	5500	5396	0.0010	65.0	170.2	Not Modeled	
Airport		20,863,410	478.96	Minimal				65.0		Not Modeled	
SS1		83,729	1.92	19.41%				74.6		Not Modeled	
SS2		100,567	2.31	29.41%				77.5		Not Modeled	
SS3		36,538	0.84	29.07%				77.4		Not Modeled	
SS4		592,718	13.61	20.47%	300	1,976	0.0060	74.9	23.0	Not Modeled	
SS5		34,870	0.80	20.07%				74.8		Not Modeled	
SS6		18,858	0.43	100.00%				98.0		Not Modeled	
SS7		24,304	0.56	32.48%				78.4		Not Modeled	
SS8		23,929	0.55	25.07%				76.3		Not Modeled	

City of Ontario
Storm Water Study
Runoff Basin Parameters

	Model	Area		Parameters					Time of Conc	
Basin #	Node	sq ft	acre	Impervious %	Length (ft)	Width (ft)	Slope	CN	min	Model
O1		1,531,778	35.16	17.84%	1300	1178	0.0010	74.2	48.4	Not Modeled
O2		61,922	1.42	26.53%				76.7		Not Modeled
O3		657,709	15.10	0.00%				69.0		Not Modeled
O4		1,557,543	35.76	4.87%				70.4		Not Modeled
W1		806,353	18.51	27.46%	1660	486	0.0140	84.9	22.6	WalmartAllNodes.xp
W2		785,744	18.04	73.78%	1340	586	0.0022	93.3	27.5	WalmartAllNodes.xp
W3		527,864	12.12	95.16%	1550	341	0.0098	97.1	21.3	WalmartAllNodes.xp
Wdenny		40,013	0.92	100%	350	114	0.0097	98.0	10.4	WalmartAllNodes.xp
Wonsite		133,616	3.07	100%	150	891	0.0120	98.0	7.9	WalmartAllNodes.xp
K1		227,374	5.22	60.71%	600	379	0.0157	90.9	6.9	KmartBasinAll.xp
K2		635,004	14.58	90.89%	1000	635	0.0051	96.4	19.3	KmartBasinAll.xp
K3		339,321	7.79	59.04%	550	617	0.0085	90.6	13.8	KmartBasinAll.xp
A	4	572,223	13.14	26.82%	800	715	0.003	74	23	ExistingHeinz.xp
B	6	842,383	19.34	23.67%	750	1123	0.003	73	22	ExistingHeinz.xp
C	8	1,073,378	24.64	23.96%	1000	1073	0.003	73	27	ExistingHeinz.xp
D	28	526,418	12.08	19.27%	840	627	0.002	71	31	ExistingHeinz.xp
E	10	1,437,896	33.01	29.55%	800	1797	0.003	72	26	ExistingHeinz.xp
F	12	1,182,075	27.14	55.33%	1100	1075	0.002	83	46	ExistingHeinz.xp
G	15	1,473,516	33.83	29.30%	1020	1445	0.002	75	38	ExistingHeinz.xp
H	18	1,418,730	32.57	53.70%	900	1576	0.003	83	33	ExistingHeinz.xp
I	21	877,585	20.15	65.00%	450	1950	0.004	86	15	ExistingHeinz.xp
J		2,025,177	46.49	0.00%	2600	779	0.002	86	240	ExistingHeinz.xp
K		501,153	11.50	70.00%	500	1002	0.004	91	17	ExistingHeinz.xp
L		189,328	4.35	65.00%	100	1893	0.008	88	5	ExistingHeinz.xp
M		42,884	0.98	70.00%	500	86	0.002	94	22	ExistingHeinz.xp
N		61,385	1.41	64.50%	100	614	0.001	86	8	ExistingHeinz.xp
O		599,141	13.75	90.00%	500	1198	0.008	94	22	ExistingHeinz.xp
P		989,833	22.72	80.00%	950	1042	0.002	92	22	ExistingHeinz.xp
Q		561,785	12.90	80.00%	600	936	0.002	92	13	ExistingHeinz.xp

City of Ontario Stormwater Study

Ontario Soil Types and Hydrologic Classification

			SCS CN			
Soil Type		Hydrologic Soil Group	Streets/Roads	Commercial	Residential	Open Space
20	Notus-Falk Variant Complex	C	98	94	83	79
31	Stanfield silt loam	C	98	94	83	79
34	Umapine Silt loam	C/D	98	94/95	83/87	79/84
12A	Garbutt silt loam, 0 to 2 percent slopes	B	98	92	75	69
13A	Greenleaf silt loam, 0 to 2 percent slopes	B	98	92	75	69
25A	Owyhee silt loam, 0 to 2 percent slopes	B	98	92	75	69
25B	Owyhee silt loam, 2 to 5 percent slopes	B	98	92	75	69
25C	Owyhee silt loam, 5 to 8 percent slopes	B	98	92	75	69
25D	Owyhee silt loam, 8 to 12 percent slopes	B	98	92	75	69
25E	Owyhee silt loam, 12 to 20 percent slopes	B	98	92	75	69
30A	Sagehill fine sand loam, 0 to 2 percent slopes	B	98	92	75	69
33A	Turbyfill fine sandy loam, 0 to 2 percent slopes	B	98	92	75	69
33B	Turbyfill fine sandy loam, 2 to 5 percent slopes	B	98	92	75	69

Assumptions:

Open space has fair condition (grass cover 50%-75%)
Residential Area averages 1/4 acre lots
Impervious area CN=98
Pervious Area CN=65 (61-B, 74-C, 80-D)

In a representative profile, the surface layer is grayish brown silt loam about 10 inches thick. The underlying material is grayish brown silt loam to a depth of 30 inches. Below this is a grayish brown, silica-cemented hardpan. The soil is moderately alkaline throughout.

Permeability is moderate above the hardpan and slow to very slow in the hardpan. Available water capacity is 4 to 6 inches. Effective rooting depth is 20 to 40 inches. A seasonal water table is at a depth of 1 to 2 feet during the irrigation period. Rare flooding occurs during spring runoff.

These soils are used for irrigated pasture, hay, occasional row crops, and wildlife habitat.

Representative profile of the Ahtanum silt loam about 100 feet north of a barn, 20 feet east of a north-south fence in SW1/4SW1/4SW1/4 section 30, T. 16 S., R. 44 E.:

Ap—0 to 6 inches; grayish brown (10YR 5/2) silt loam, very dark brown (10YR 2/2) moist; weak fine subangular blocky structure; slightly hard, friable, sticky and plastic; many fine roots; many fine tubular pores; moderately alkaline; abrupt smooth boundary.

A12—6 to 10 inches; grayish brown (10YR 5/2) silt loam, very dark grayish brown (10YR 3/2) moist; moderate fine subangular blocky structure; slightly hard, friable, sticky and plastic; many fine roots; many fine tubular pores; moderately alkaline; abrupt smooth boundary.

C1—10 to 23 inches; grayish brown (10YR 5/2) silt loam, very dark grayish brown (10YR 3/2) moist; weak fine subangular blocky structure; slightly hard, friable, slightly sticky and slightly plastic; many fine roots; many fine tubular pores; moderately calcareous; moderately alkaline; gradual wavy boundary.

C2—23 to 30 inches; grayish brown (10YR 5/2) silt loam, dark brown (10YR 3/3) moist; weak very fine subangular blocky structure; slightly hard, friable, slightly sticky and slightly plastic; many fine roots; many fine tubular pores; 20 percent durinodes 1/2 inch diameter; moderately alkaline; abrupt wavy boundary.

Ccasim—30 to 32 inches; grayish brown (10YR 5/2) silica-cemented duripan of silty material, very dark brown (10YR 2/2) moist; massive; strongly cemented; very hard, very firm, nonsticky and nonplastic; very few roots; very few tubular pores; strong effervescence; abrupt wavy boundary.

The A horizon is silt loam that is more than 18 percent clay. It is moderately to strongly alkaline and high in sodium. The lower part of the C horizon contains 5 to 30 percent weakly silica-cemented durinodes 1/4 to 1/2 inch in diameter. Depth to the silica-cemented duripan ranges from 20 to 40 inches but is generally less than 30 inches. The duripan may have a root mat on the upper surface.

1—Ahtanum silt loam. This soil is in irregularly shaped areas on low fans and flood plains. Slope are 0 to 2 percent.

Included with this soil in mapping were about 10 percent Harana soils. Also included were about 10 percent soils that are similar to Ahtanum soils but that are 10 to 20 inches deep to the hardpan.

Runoff is slow, and the hazard of erosion is slight. Capability unit IVw-1.

Baldock series

The Baldock series consists of somewhat poorly drained soils that formed on low terraces, fans, and bottom lands in medium textured alluvium. Slopes are 0 to 2 percent. Elevation is 2,100 to 2,600 feet. The native vegetation was saltgrass, giant wildrye, rabbitbrush, greasewood, and big sagebrush. Average annual precipitation is 9 to 11 inches, average annual air temperature is 48 degrees to 52 degrees F, and the frost-free period is 120 to 160 days.

In a representative profile, the surface layer is light brownish gray silt loam about 16 inches thick. The subsoil is grayish brown silt loam about 16 inches thick. The substratum is light gray silt loam to a depth of 60 inches or more. The soil is strongly alkaline above a depth of about 32 inches and very strongly alkaline below a depth of 32 inches.

Permeability is moderate to moderately slow. Available water capacity is 8 to 10 inches. Effective rooting depth is 60 inches or more. A seasonal water table is at a depth of 2 to 3 feet during the irrigation period. Rare flooding occurs during spring runoff.

These soils are used for irrigated pasture, hay, small grain, sugar beets, and wildlife habitat.

Representative profile of Baldock silt loam about 850 feet east of section corner in the SE1/4SW1/4SW1/4 section 21, T. 17 S., R. 47 E.:

Ap—0 to 9 inches; light brownish gray (10YR 6/2) silt loam, very dark grayish brown (10YR 3/2) moist; moderate medium to fine granular structure; slightly hard, friable, slightly sticky and slightly plastic; few fine roots; many fine irregular pores; moderately calcareous; strongly alkaline; abrupt smooth boundary.

A1—9 to 16 inches; light brownish gray (10YR 6/2) silt loam, very dark grayish brown (10YR 3/2) moist; weak coarse prismatic structure; slightly hard, friable, slightly sticky and slightly plastic; few fine roots; many fine irregular pores; moderately calcareous; strongly alkaline; gradual wavy boundary.

B2—16 to 32 inches; grayish brown (10YR 5/2) silt loam, very dark gray (10YR 3/1) moist; few faint yellowish brown (10YR 5/4) mottles, dark brown (10YR 3/3) moist; moderate fine angular blocky structure; hard, firm, sticky and plastic; few fine roots; many fine tubular pores; moderately calcareous; strongly alkaline; gradual wavy boundary.

Cca—32 to 60 inches; light gray (10YR 7/2) silt loam, brown to dark brown (10YR 4/3) moist; massive; slightly hard, friable, slightly sticky and slightly plastic; very few fine roots; few fine tubular pores; strongly calcareous; very strongly alkaline.

The A horizon is light brownish gray or light gray loam or silt loam. It is more than 18 percent clay. It is calcareous and moderately alkaline to strongly alkaline. The 10- to 40-inch control section is loam or silt loam.

2—Baldock silt loam. This soil is in irregularly shaped areas on stream bottoms, alluvial fans, and low terraces.

Included with this soil in mapping were about 10 percent Umapine soils and about 5 percent Stanfield soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIIw-1.

Bully series

The Bully series consists of well drained soils that formed on bottom lands and fans in mixed alluvium that is high in diatomaceous material. Slopes are 0 to 2 percent. Elevation is 2,500 to 2,700 feet. The native vegetation was giant wildrye, bluebunch wheatgrass, hopsage, and big sagebrush. Average annual precipitation is 7 to 9 inches, average annual air temperature is 48 degrees to 50 degrees F, and the frost-free period is 120 to 150 days.

In a representative profile, the surface layer is light gray silt loam about 9 inches thick. The underlying material is light gray silt loam to a depth of 60 inches or more. The soil is mildly alkaline above a depth of 9 inches and moderately alkaline below that depth.

Permeability is moderate. Available water capacity is 10 to 12.5 inches. Effective rooting depth is 60 inches or more.

These soils are used for irrigated corn, potatoes, onions, small grain, alfalfa hay, pasture, and wildlife habitat.

Representative profile of Bully silt loam about 1/2 mile northwest of Harper, 40 feet north and 40 feet east of the southwest corner of the SE1/4 section 31, T. 19 S., R. 42 E.:

Ap—0 to 9 inches; light gray (10YR 7/2) silt loam, brown (10YR 5/3) moist; weak very fine granular structure; soft, very friable, slightly sticky and slightly plastic; few roots; many fine round pores; mildly alkaline; abrupt smooth boundary.

C1—9 to 30 inches; light gray (10YR 7/2) silt loam, brown (10YR 5/3) moist; massive; soft, very friable, slightly sticky and slightly plastic; few roots; common fine tubular pores; few white soft diatomite fragments; noncalcareous; moderately alkaline; diffuse wavy boundary.

C2—30 to 60 inches; light gray (10YR 7/2) silt loam, brown (10YR 5/3) moist; massive; soft, very friable, slightly sticky and slightly plastic; few roots; few fine tubular pores; few soft diatomite fragments in the matrix, increasing with depth; few thin lenses of very fine sandy loam; noncalcareous; moderately alkaline.

The A and C horizons are silt loam, loam, or very fine sandy loam. Silica-cemented durinodes as large as 1/2 inch in diameter make up 2 to 10 percent of the C horizon.

3—Bully silt loam. This soil is in irregularly shaped areas on flood plains and fans.

Included with this soil in mapping were about 10 percent Powder soils, about 5 percent Umapine soils, and about 5 percent Stanfield soils.

Runoff is slow. The hazard of erosion is severe because of the light weight of the soil, its tendency to undercut in corrugations and furrows, and gullies. Capability unit IIe-1.

Cencove series

The Cencove series consists of well drained soils that formed on medium and low terraces in mixed alluvium. Slopes are 0 to 12 percent. Elevation is 2,200 to 2,500 feet. The native vegetation was Sandberg bluegrass, bluebunch wheatgrass, and big sagebrush. Average annual precipitation is 9 to 11 inches, average annual air temperature is 50 degrees to 52 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is pale brown fine sandy loam about 9 inches thick. The upper part of the underlying material, to a depth of 24 inches, is brown fine sandy loam, and the lower part is very gravelly sand to a depth of 60 inches or more. The soil is mildly alkaline above a depth of about 9 inches and moderately alkaline below that depth.

Permeability is moderately rapid in the upper part and very rapid in the very gravelly sand. Available water capacity is 4 to 6 inches. Effective rooting depth is 20 to 40 inches.

These soils are used for irrigated potatoes, onions, corn, sugar beets, small grain, alfalfa seed, alfalfa hay, pasture, and wildlife habitat.

Representative profile of Cencove fine sandy loam, 0 to 2 percent slopes, about 1,320 feet north and 750 feet east of the country road, SW1/4SE1/4 section 25, T. 16 S., R. 47 E.:

Ap—0 to 9 inches; pale brown (10YR 6/3) fine sandy loam, very dark grayish brown (10YR 3/2) moist; weak fine granular structure; slightly hard, very friable, nonsticky and nonplastic; many fine roots; many fine irregular pores; mildly alkaline; abrupt smooth boundary.

C1ca—9 to 16 inches; brown (10YR 5/3) fine sandy loam, very dark grayish brown (10YR 3/2) moist; massive; very friable, slightly hard, nonsticky and slightly plastic; many fine roots; few fine tubular pores; weakly calcareous; moderately alkaline; gradual wavy boundary.

C2ca—16 to 24 inches; brown (10YR 5/3) fine sandy loam, dark brown (10YR 3/3) moist; massive; very friable, slightly sticky and slightly plastic; many fine roots; many fine tubular pores; moderately calcareous; moderately alkaline; abrupt wavy boundary.

IIC3—24 to 60 inches; multicolored very gravelly sand; single grained; loose; weakly calcareous; gravel coated with lime on lower side; moderately alkaline.

The A horizon has a value of 5 or 6 when dry and a chroma of 2 or 3 when dry or moist. It is sandy loam or fine sandy loam. The C1 and C2 horizons have a value of 5 or 6 when dry and a chroma of 2 or 3 when dry or moist. They are fine sandy loam or sandy loam. Depth to the very gravelly sand IIC horizon ranges from 20 to 40 inches.

4A—Cencove fine sandy loam, 0 to 2 percent slopes. This soil is in irregularly shaped areas. It has the profile described as representative of the series.

Included with this soil in mapping were about 5 percent Nyssa soils, gravel substratum; about 5 percent Nyssa soils; about 5 percent Sagehill soils; and about 5 percent Truesdale soils.

Runoff is slow, and the hazard of water erosion is slight. The hazard of soil blowing is moderate. Capability unit IIIs-5.

4B—Cencove fine sandy loam, 2 to 5 percent slopes. This soil is in irregularly shaped areas.

Included with this soil in mapping were about 5 percent Nyssa soils, gravel substratum; about 5 percent Nyssa soils; about 5 percent Sagehill soils; and about 5 percent Truesdale soils.

Runoff is slow, and the hazard of water erosion is slight. The hazard of soil blowing is moderate. Capability unit IIIe-3.

4C—Cencove fine sandy loam, 5 to 8 percent slopes. This soil is in irregularly shaped areas.

Included with this soil in mapping were about 5 percent Nyssa soils, gravel substratum; about 5 percent Nyssa soils; and about 5 percent Truesdale soils.

Runoff is medium, and the hazard of water erosion is moderate. The hazard of soil blowing is moderate. Capability unit IIIe-3.

Runoff is slow, and the hazard of water erosion is slight. The hazard of soil blowing is moderate to severe. Capability unit IIIe-3.

9C—Feltham sandy loam, 5 to 8 percent slopes. This soil is in irregularly shaped areas. It has a profile similar to the one described as representative of the series, but the surface layer is sandy loam and the underlying material is loamy coarse sand and coarse sandy loam to a depth of 60 inches or more.

Included with this soil in mapping were about 10 percent Quincy soils, about 10 percent Cencove soils, about 2 percent Feltham loamy fine sand, and about 2 percent Feltham Variant loamy fine sand.

Runoff is slow, and the hazard of water erosion is slight. The hazard of soil blowing is moderate to severe. Capability unit IIIe-3.

9D—Feltham sandy loam, 8 to 12 percent slopes. This soil is in irregularly shaped areas. It has a profile similar to the one described as representative of the series, but the surface layer is sandy loam and the underlying material is loamy coarse sand and coarse sandy loam to a depth of 60 inches or more.

Included with this soil in mapping were about 15 percent Cencove soils and about 5 percent Feltham loamy fine sand.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IVe-2.

Feltham Variant

The Feltham Variant consists of excessively drained soils that formed on terraces and fans in sandy alluvium that has been modified by wind. Slopes are 0 to 2 percent. Elevation is 2,100 to 2,600 feet. The native vegetation was needlegrass, Indian ricegrass, rabbitbrush, and big sagebrush. Average annual precipitation is 8 to 10 inches, average annual air temperature is 50 degrees to 52 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is brown loamy fine sand about 5 inches thick. The upper part of the underlying material, to a depth of 26 inches, is brown and yellowish brown loamy sand; the middle part, to a depth of 31 inches, is yellowish brown fine sandy loam; and the lower part is multicolored very gravelly sand to a depth of 60 inches or more. The soil is mildly alkaline throughout.

Permeability is moderately rapid in the upper 31 inches and very rapid in the very gravelly sand. Available water capacity is 2 to 4 inches. Effective rooting depth is 30 to [illegible] inches.

These soils are used for irrigated corn, potatoes, onions, [illegible] grain, alfalfa hay, alfalfa seed, and wildlife habitat.

Representative profile of Feltham Variant loamy fine [illegible] about 20 feet west of a farm road, 200 feet south of [illegible] in the NW1/4SE1/4NW1/4 section 19, T. 16 S., R. [illegible]

[illegible] inches; brown (10YR 5/3) loamy fine sand, very dark grayish [illegible] (10YR 3/2) moist; weak fine granular structure; loose, nonsticky and nonplastic; many roots; many fine tubular pores; mildly alkaline; abrupt smooth boundary.

C1—5 to 14 inches; brown (10YR 5/3) loamy sand, very dark grayish brown (10YR 3/2) moist; massive; slightly hard, friable, nonsticky and nonplastic; many roots; many fine tubular pores; mildly alkaline; gradual wavy boundary.

C2—14 to 26 inches; yellowish brown (10YR 5/4) loamy sand, dark brown (10YR 3/3) moist; massive; loose; nonsticky and nonplastic; many roots; many fine tubular pores; mildly alkaline; gradual wavy boundary.

C3—26 to 31 inches; yellowish brown (10YR 5/4) fine sandy loam, dark brown (10YR 3/3) moist; massive; soft, very friable, nonsticky and nonplastic; many roots; irregular pores; slightly calcareous; mildly alkaline; abrupt wavy boundary.

IIC4—31 to 60 inches; multicolored very gravelly sand; single grained; loose; mildly alkaline.

The A horizon is grayish brown, light grayish brown or brown loamy fine sand. The soil is mildly or moderately alkaline throughout.

10—Feltham Variant loamy fine sand. This soil is in elongated areas. Slopes are 0 to 2 percent.

Included with this soil in mapping were about 5 percent Feltham loamy fine sand and about 5 percent Quincy soils. Also included were about 15 percent soils that are similar to Feltham Variant loamy fine sand but that have slopes of 2 to 8 percent.

Runoff is slow, and the hazard of water erosion is slight. The hazard of soil blowing is moderate to severe. Capability unit IVs-3.

Frohman series

The Frohman series consists of well drained soils that formed on high terraces in loess-capped, medium textured old lacustrine material and very gravelly alluvium. Slopes are 0 to 20 percent. Elevation is 2,100 feet to 2,600 feet. The native vegetation was bluebunch wheatgrass, Sandberg bluegrass, big sagebrush, and annual forbs. Average annual precipitation is 9 to 11 inches, average annual air temperature is 50 degrees to 54 degrees F, and the frost-free period is 130 to 170 days.

In a representative profile, the surface layer is light brownish gray silt loam about 8 inches thick. The subsoil is light brownish gray silt loam about 4 inches thick. The upper part of the substratum is a pale brown, silica-cemented hardpan 6 inches thick. Below this hardpan is light gray silt loam 18 inches thick. Below this is a hardpan of very gravelly silica-cemented material. The soil is mildly alkaline above a depth of 8 inches and moderately alkaline below a depth of 8 inches.

Permeability is moderate above the hardpan and very slow in the hardpan. Available water capacity is 2 to 4 inches. Effective rooting depth is 10 to 20 inches.

These soils are used for irrigated small grain, alfalfa hay, pasture, and wildlife habitat.

Representative profile of the Frohman silt loam, 0 to 2 percent slopes, 30 feet north of the country road and 60 feet west of the center line in the SE1/4SW1/4 section 27, T. 18 S., R. 44 E.:

Ap—0 to 8 inches; light brownish gray (10YR 6/2) silt loam, dark grayish brown (10YR 4/2) moist; weak medium platy structure parting to weak fine subangular blocky; slightly hard, friable, slightly sticky and slightly plastic; many very fine and fine roots; many very fine and fine tubular pores; mildly alkaline; abrupt smooth boundary.

B2—8 to 12 inches; light brownish gray (10YR 6/2) silt loam, dark grayish brown (10YR 4/2) moist; moderate medium prismatic structure; hard, friable, slightly sticky and slightly plastic; many very fine and fine roots; many fine and very fine tubular pores; moderately alkaline; abrupt smooth boundary.

C1sim—12 to 18 inches; pale brown (10YR 6/3) indurated duripan of silt loam, dark brown (10YR 3/3) moist; massive; extremely hard, extremely firm; thin light gray (10YR 6/1) laminar cappings on the surface of the duripan; root mat on surface; many very fine tubular pores; moderately alkaline; clear wavy boundary.

C2—18 to 28 inches; light gray (10YR 7/2) silt loam, brown (10YR 5/3) moist; massive; hard, friable, nonsticky and nonplastic; few very fine tubular pores; moderately alkaline; clear wavy boundary.

C3—28 to 36 inches; light gray (10YR 7/2) silt loam, pale brown (10YR 6/3) moist; massive; hard, friable, nonsticky and nonplastic; few very fine tubular pores; strongly calcareous; moderately alkaline; abrupt smooth boundary.

C4casim—36 to 42 inches; very gravelly indurated duripan; massive; extremely hard, extremely firm; pebbles coated with lime.

The A horizon is light brownish gray or pale brown when dry and very dark grayish brown, dark grayish brown, or dark brown when moist. The A and B horizons above the duripan are silt loam or very fine sandy loam. They are less than 18 percent clay. Depth to the duripan ranges from 10 to 20 inches. Depth to very gravelly material ranges from 20 to 40 inches. Depth to bedrock is more than 60 inches. The underlying very gravelly duripan is indurated or strongly cemented in the upper few inches and becomes less cemented with depth.

11A—Frohman silt loam, 0 to 2 percent slopes. This soil is in irregularly shaped areas on terraces. It has the profile described as representative of the series.

Included with this soil in mapping were about 2 percent Nyssa soils and about 5 percent Virtue soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IVs-1.

11B—Frohman silt loam, 2 to 5 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 2 percent Nyssa soils and about 5 percent Virtue soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IVe-1.

11C—Frohman silt loam, 5 to 8 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 2 percent Nyssa soils and about 5 percent Virtue soils.

Runoff is moderate, and the hazard of erosion is moderate. Capability unit IVe-1.

11D—Frohman silt loam, 8 to 12 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 5 percent Virtue soils and about 2 percent Nyssa soils.

Runoff is moderate, and the hazard of erosion is moderate. Capability unit IVe-3.

11E—Frohman silt loam, 12 to 20 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 2 percent Nyssa soils and about 2 percent Xeric Torriorthents, moderately steep.

Runoff is moderate, and the hazard of erosion is high. Capability unit VIe-1.

Garbutt series

The Garbutt series consists of well drained soils that formed on fans, bottom lands, and low terraces in mixed, medium textured alluvium. Slopes are 0 to 5 percent. Elevation is 2,100 to 2,400 feet. The native vegetation was needlegrass, Sandberg bluegrass, giant wildrye, big sagebrush, bursage, and fourwing saltbush. Average annual precipitation is 8 to 10 inches; average annual air temperature is 51 degrees to 53 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is very pale brown silt loam about 10 inches thick. The underlying material is pale brown and very pale brown silt loam to a depth of 62 inches. The soil is moderately alkaline throughout.

Permeability is moderate. Available water capacity is 10 to 12 inches. Effective rooting depth is 60 inches or more.

These soils are used for irrigated onions, potatoes, sugar beets, corn, small grain, alfalfa hay, alfalfa seed, and wildlife habitat.

Representative profile of Garbutt silt loam, 0 to 2 percent slopes, about 60 feet west of Alkali Creek, 50 feet south of the county road in the NW1/4SW1/4 of section 11, T. 22 S., R. 46 E.:

Ap—0 to 10 inches; very pale brown (10YR 7/3) silt loam, brown (10YI 4/3) moist; weak fine subangular blocky structure parting to weal fine granular; soft, very friable, nonsticky and nonplastic; many fin roots; many fine irregular and tubular pores; moderately alkaline abrupt smooth boundary.

C1—10 to 23 inches; pale brown (10YR 6/3) silt loam, brown (10YR 4/ moist; weak very coarse prismatic structure; slightly hard, very fr able, nonsticky and slightly plastic; many fine roots; many fine tu bular pores; moderately calcareous; moderately alkaline; gradu wavy boundary.

C2ca—23 to 38 inches; very pale brown (10YR 7/3) silt loam, brow (10YR 4/3) moist; massive; slightly hard, very friable, nonsticky an slightly plastic; few fine roots; few fine tubular pores; moderatel calcareous; moderately alkaline; gradual wavy boundary.

C3ca—38 to 62 inches; very pale brown (10YR 7/3) silt loam, brow (10YR 5/4) moist; massive; slightly hard, very friable, nonsticky a slightly plastic; few fine roots; few fine tubular pores; moderate calcareous; moderately alkaline.

The A horizon is dark grayish brown, very pale brown, or white wh dry and dark grayish brown, grayish brown, or brown when moist. T 10- to 40-inch control section ranges from silt loam to very fine san loam, averages 12 to 18 percent clay and less than 15 percent partic coarser than very fine sand, and commonly contains much coarse s The soil is generally calcareous throughout. Some pedons have no fr lime in the A horizon and very small amounts below, increasing w depth.

12A—Garbutt silt loam, 0 to 2 percent slopes. Th soil is in irregularly shaped areas. It has the profi described as representative of the series.

Included with this soil in mapping were about 10 pe cent Umapine soils, about 10 percent Turbyfill soils, a about 3 percent Stanfield soils.

Runoff is slow, and the hazard of erosion is slig Capability unit I-1.

12B—Garbutt silt loam, 2 to 5 percent slopes. T soil is in irregularly shaped areas.

Included with this soil in mapping were about 10 percent Owyhee soils and about 3 percent Turbyfill soils.

Runoff is slow, and the hazard of erosion is moderate. Capability unit IIe-2.

Greenleaf series

The Greenleaf series consists of well drained soils that formed on low and medium terraces in loess-capped, medium and fine textured old lacustrine material. Slopes are 0 to 5 percent. Elevation is 2,100 to 2,300 feet. The native vegetation was bluebunch wheatgrass, needlegrass, Sandberg bluegrass, big sagebrush, shadscale, and rabbitbrush. Average annual precipitation is 8 to 10 inches, average annual air temperature is 50 degrees to 52 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is light brownish gray silt loam about 8 inches thick. The subsoil is light brownish gray and brown silty clay loam about 23 inches thick. The substratum is very pale brown and light gray silt loam to a depth of 60 inches or more. The soil is mildly alkaline to a depth of 21 inches and moderately alkaline below a depth of 21 inches.

Permeability is moderately slow. Available water capacity is 10 to 13 inches. Effective rooting depth is 60 inches or more.

These soils are used for irrigated onions, potatoes, sugar beets, corn, small grain, alfalfa hay, alfalfa seed, and wildlife habitat.

Representative profile of the Greenleaf silt loam, 0 to 2 percent slopes, in the SW1/4SW1/4 section 9, T. 16 S., R. 47 E.:

Ap—0 to 8 inches; light brownish gray (10YR 6/2) silt loam, dark grayish brown (10YR 4/2) moist; weak fine granular structure; hard, friable, slightly sticky and plastic; few fine roots; few fine tubular pores; mildly alkaline; abrupt smooth boundary.

B1—8 to 14 inches; light brownish gray (10YR 6/2) silty clay loam, dark grayish brown (10YR 4/2) moist; weak medium subangular blocky structure; hard, friable, sticky and plastic; few fine roots; few fine tubular pores; mildly alkaline; abrupt wavy boundary.

B21t—14 to 21 inches; brown (10YR 5/3) light silty clay loam, dark brown (10YR 3/3) moist; weak coarse prismatic structure parting to moderate medium subangular blocky; hard, firm, sticky and very plastic; few fine roots; few fine tubular pores; continuous thin clay films on peds; mildly alkaline; gradual wavy boundary.

B22t—21 to 31 inches; brown (10YR 5/3) light silty clay loam, brown to dark brown (10YR 4/3) moist; moderate medium subangular blocky structure; hard, firm, sticky and very plastic; few fine roots; few fine tubular pores; continuous thin clay films on peds; moderately alkaline; abrupt wavy boundary.

C1ca—31 to 36 inches; very pale brown (10YR 7/3) silt loam, pale brown (10YR 6/3) moist; massive; hard, friable, slightly sticky and plastic; few fine roots; few fine tubular pores; strongly calcareous; moderately alkaline; abrupt smooth boundary.

C2ca—36 to 60 inches; light gray (10YR 7/2) silt loam, brown to dark brown (10YR 6/3) moist; fine and medium platy laminations of old alluvium; hard, friable, sticky and plastic; no roots; few fine irregular pores; moderately alkaline.

The A horizon has a value of 5 to 7 when dry and 3 or 4 when moist and a chroma of 2 or 3 when dry and moist. It is not darker in value than 5.5 when dry and 3.5 when moist, both when broken and when rubbed. The Bt horizon averages 22 to 28 percent clay and is less than 15 percent particles coarser than very fine sand. It has common thin to nearly continuous moderately thick clay films on the faces of peds and walls of pores. Depth to laminated material is 20 to 40 inches.

13A—Greenleaf silt loam, 0 to 2 percent slopes. This soil is in large, rounded areas on broad terraces. It has the profile described as representative of the series.

Included with this soil in mapping were about 5 percent Owyhee soils and about 5 percent Nyssa soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit I-1.

13B—Greenleaf silt loam, 2 to 5 percent slopes. This soil is in large, rounded areas on broad terraces.

Included with this soil in mapping were about 5 percent Owyhee soils, about 5 percent Nyssa soils, and about 5 percent Greenleaf soils that have slopes of 5 to 12 percent.

Runoff is slow, and the hazard of erosion is moderate. Capability unit IIe-2.

Harana series

The Harana series consists of moderately well drained soils that formed on bottom lands in moderately fine textured, mixed alluvium. Slopes are 0 to 2 percent. Elevation is 2,200 to 2,600 feet. The native vegetation was giant wildrye, sod-forming bluegrass, big sagebrush, and related forbs. Average annual precipitation is 9 to 11 inches, average annual air temperature is 48 degrees to 52 degrees F, and the frost-free period is 130 to 160 days.

In a representative profile, the surface layer is dark gray and gray silty clay loam about 24 inches thick. The upper part of the underlying material, to a depth of 43 inches, is dark gray silty clay loam, and the lower part is black clay to a depth of 60 inches or more. The soil is mildly alkaline to a depth of about 16 inches and moderately alkaline below that depth.

Permeability is moderately slow. Available water capacity is 10 to 12 inches. Effective rooting depth is 60 inches or more. Rare flooding occurs during spring runoff.

These soils are used for irrigated alfalfa hay, small grain, sugar beets, corn, and wildlife habitat.

Representative profile of Harana silty clay loam 60 feet west of Willow Creek, 600 feet north of the section line (fence line), SW1/4SE1/4SW1/4 section 32, T. 15 S., R. 43 E.:

Ap—0 to 8 inches; dark gray (10YR 4/1) silty clay loam, black (10YR 2/1) moist; strong fine granular structure; hard, firm, sticky and plastic; many very fine to medium roots; many fine irregular pores; mildly alkaline; abrupt smooth boundary.

A11—8 to 16 inches; dark gray (10YR 4/1) silty clay loam, black (10YR 2/1) moist; weak medium blocky structure parting to strong very fine subangular blocky; hard, firm, sticky and plastic; many very fine to medium roots; many very fine tubular pores; mildly alkaline; gradual wavy boundary.

A12—16 to 24 inches; gray (10YR 5/1) silty clay loam, black (10YR 2/1) moist; strong very fine subangular blocky structure; very hard, firm, sticky and plastic; many very fine to medium roots; many very fine tubular pores; moderately alkaline; gradual wavy boundary.

C1ca—24 to 34 inches; dark gray (10YR 4/1) silty clay loam, black (10YR 2/1) moist; weak very fine subangular blocky structure; very hard, firm, sticky and plastic; many very fine to medium roots; many very fine tubular pores; calcareous, with segregated lime; moderately alkaline; gradual smooth boundary.

C2—34 to 43 inches; dark gray (10YR 4/1) silty clay loam, black (10YR 2/1) moist; weak very fine subangular blocky structure; very hard, firm, sticky and plastic; many very fine to medium roots; many very fine tubular pores; moderately alkaline; gradual wavy boundary.

C3—43 to 60 inches; black (N 2/) clay, black (N 2/) moist; massive; very hard, very firm, very sticky and very plastic; few medium roots; few fine irregular pores; moderately alkaline.

The soil is silty clay loam throughout the control section and averages 27 to 35 percent clay and less than 15 percent material coarser than very fine sand. The C horizon below a depth of 40 inches has a value of 2 or 3 when moist and a chroma of 1 or less; high chroma mottles are in some pedons. The C horizon below a depth of 40 inches is silty clay loam, silty clay, or clay.

14—Harana silt loam. This soil is in regularly shaped areas on bottom lands along perennial streams. It has a profile similar to the one described as representative of the series, but the surface layer is silt loam 11 to 24 inches thick.

Included with this soil in mapping were about 10 percent Harana silty clay loam; about 5 percent Harana silty clay loam, alkali; about 5 percent Powder soils; and about 5 percent Umapine soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIc-1.

15—Harana silty clay loam. This soil is in regularly shaped areas on bottom lands along perennial streams. It has the profile described as representative of the series.

Included with this soil in mapping were about 10 percent Harana silt loam; about 5 percent Harana silty clay loam, alkali; about 5 percent Powder soils; and about 5 percent Umapine soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIc-1.

16—Harana silty clay loam, alkali. This soil is in regularly shaped areas on bottom lands along perennial streams. It has a profile similar to the one described as representative of the series, but the surface layer is gray, about 9 inches thick, strongly alkaline, and high in exchangeable sodium.

Included with this soil in mapping were about 10 percent Harana silty clay loam, about 10 percent Umapine soils, and about 5 percent Harana silt loam.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIIs-3.

Kiesel series

The Kiesel series consists of well drained alkali soils that formed on low terraces in mixed alluvium. Slopes are 0 to 2 percent. Elevation is 2,100 to 2,200 feet. The native vegetation was saltgrass, greasewood, and rabbitbrush. Average annual precipitation is 9 to 11 inches, average annual air temperature is 51 degrees to 53 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is light gray silt loam about 4 inches thick. The subsoil is light brownish gray and light gray silty clay about 18 inches thick. The upper part of the substratum is light gray silt loam to a depth of 40 inches, and the lower part is light gray loam and very fine sandy loam to a depth of 60 inches or more. The soil is strongly alkaline in the upper 4 inches and very strongly alkaline below a depth of 4 inches.

Permeability is slow. Available water capacity is 7 to 9 inches. Effective rooting depth is 60 inches or more. A seasonal water table is at a depth of 3 to 4 feet in spring.

These soils are used mostly for irrigated and subirrigated pasture and for some alfalfa hay.

Representative profile of Kiesel silt loam 400 feet south and 100 feet west of the section corner in the NE1/4NE1/4 of section 8, T. 19 S., R. 47 E.:

A1—0 to 4 inches; light gray (10YR 7/2) silt loam, brown (10YR 4/3) moist; massive; slightly hard, friable, slightly sticky and slightly plastic; few very fine and fine roots; many very fine round pores; weakly calcareous; strongly alkaline; abrupt smooth boundary.

B21t—4 to 7 inches; light brownish gray (10YR 6/2) silty clay, dark grayish brown (10YR 4/2) moist; thin light gray (10YR 7/2) and grayish brown (10YR 5/2) silt coatings; weak fine prismatic structure parting to moderate fine and medium subangular blocky; hard, firm, very sticky and very plastic; common very fine roots; many very fine tubular pores; common thin and few moderately thick clay films; strongly calcareous; very strongly alkaline; clear smooth boundary.

B22t—7 to 11 inches; light gray (10YR 7/2) silty clay, brown (10YR 4/3) moist; strong fine subangular blocky structure; hard, firm, sticky and very plastic; many very fine roots; common very fine tubular pores; clay films as above; strongly calcareous; very strongly alkaline; clear smooth boundary.

B23t—11 to 22 inches; light gray (10YR 7/2) silty clay, brown (10YR 4/3) moist; strong fine subangular blocky structure; hard, firm sticky and very plastic; common very fine roots; many very fine tubular pores; clay films as above; strongly calcareous; very strongly alkaline; clear smooth boundary.

C1ca—22 to 40 inches; light gray (10YR 7/1) heavy silt loam, brown (10YR 5/3) moist; massive; slightly hard, friable, slightly sticky and plastic; few fine roots; few very fine tubular pores; violently calcareous; very strongly alkaline; clear wavy boundary.

C2ca—40 to 60 inches; light gray (10YR 7/1) loam and very fine sandy loam, brown (10YR 5/3) moist; slightly hard, friable slightly sticky and slightly plastic; strongly calcareous; very strongly alkaline.

The A horizon has a value of 6 or 7 when dry and a chroma of 2 or when moist. The B2t horizon is silty clay or clay and contains a average of 40 to 50 percent clay. It contains more than 15 percen exchangeable sodium and has weak prismatic or columnar structur parting to moderate or strong subangular blocky structure. The C horizon has a value of 6 or 7 when dry, and a chroma of 1 or 2 when dr and 2 or 3 when moist. Is is silt loam or silty clay loam in the C1c horizon and silt loam, loam, or very fine sandy loam in the C2ca horizon.

17—Kiesel silt loam. This soil is in irregularly shape areas on low terraces adjacent to the Snake River.

Included with this soil in mapping were about 10 percent Baldock soils, about 5 percent Umapine soils, an about 5 percent Stanfield soils.

Runoff is slow, and the hazard of erosion is sligh Capability unit IVs-2.

Kimberly series

The Kimberly series consists of well drained soils th formed on bottom lands and fans in moderately coar

In a representative profile, the surface layer is very pale brown silt loam about 9 inches thick. The subsoil is very pale brown silt loam about 11 inches thick. The upper part of the substratum, to a depth of 50 inches, is very pale brown light silty clay loam, and the lower part is stratified silty clay loam and silt loam to a depth of 60 inches. The soil is moderately alkaline to a depth of 9 inches and strongly alkaline below a depth of 9 inches.

Permeability is moderately slow. Available water capacity is 8 to 12 inches. Effective rooting depth is 60 inches or more.

These soils are used for irrigated small grain, alfalfa hay, pasture, an occasional crop of sugar beets, and wildlife habitat.

Representative profile of McLoughlin silt loam, 0 to 2 percent slopes, about 700 feet north of Succor Creek channel, NE1/4SE1/4SE1/4 section 2, T. 23 S., R. 46 E.:

Ap1—0 to 3 inches; very pale brown (10YR 7/3) silt loam, dark brown (10YR 4/3) moist; strong thin platy structure; slightly hard, friable, slightly sticky and slightly plastic; many roots; many very fine irregular pores; very weakly calcareous in spots; moderately alkaline; abrupt smooth boundary.

Ap2—3 to 9 inches; very pale brown (10YR 7/3) silt loam, dark brown (10YR 4/3) moist; moderate fine and medium granular structure; soft, very friable, sticky and plastic; many roots; many very fine irregular pores; very weakly calcareous in spots; moderately alkaline; abrupt smooth boundary.

B2—9 to 20 inches; very pale brown (10YR 7/3) silt loam, dark yellowish brown (10YR 4/4) moist; weak coarse prismatic structure; hard, friable, sticky and plastic; common large roots; common very fine tubular pores; weakly calcareous; strongly alkaline; gradual wavy boundary.

C1ca—20 to 28 inches; very pale brown (10YR 7/3) light silty clay loam, yellowish brown (10YR 5/4) moist; massive; hard, friable, sticky and plastic; common large roots; common very fine tubular pores; moderately calcareous; strongly alkaline; gradual wavy boundary.

C2ca—28 to 50 inches; very pale brown (10YR 7/3) light silty clay loam, yellowish brown (10YR 4/4) moist; massive; hard, friable, sticky and plastic; strongly calcareous; strongly alkaline; gradual wavy boundary.

C3—50 to 60 inches; stratified silty clay loam and silt loam, similar to above in color and consistency; massive.

The profile to a depth of 40 inches or more has a value of 6 or 7 when dry, 4 or 5 when moist, and a chroma of 3 or 4 when dry and moist. The solum is silt loam, loam, or silty clay loam. It is 18 to 35 percent clay and less than 15 percent fine sand and coarser. Depth to bedrock is more than 60 inches. The solum is stratified with medium and moderately fine textured alluvium washed from adjacent uplands of silty lacustrine sediments.

19A—McLoughlin silt loam, 0 to 2 percent slopes. This soil is in regularly shaped areas on alluvial fans. It has the profile described as representative of the series.

Included with this soil in mapping were about 10 percent Garbutt soils, about 5 percent Stanfield soils, and about 5 percent Umapine soils.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IIIs-3.

19B—McLoughlin silt loam, 2 to 5 percent slopes. This soil is in regularly shaped areas on alluvial fans.

Included with this soil in mapping were about 10 percent Garbutt soils.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IIIs-4.

19C—McLoughlin silt loam, 5 to 8 percent slopes. This soil is in regularly shaped areas on alluvial fans.

Included with this soil in mapping were about 5 percent McLoughlin silt loam that has slopes of 8 to 12 percent.

Runoff is medium, and the hazard of erosion is moderate to high. Capability unit IIIs-4.

Notus series

The Notus series consists of moderately well drained or somewhat poorly drained soils that formed on bottom lands and very low terraces in gravelly sandy alluvium. Slopes are 0 to 2 percent. Elevation is 2,200 to 2,500 feet. The native vegetation was needlegrass, squirreltail, rabbitbrush, and big sagebrush. Average annual precipitation is 9 to 11 inches. Average annual air temperature is 50 degrees to 52 degrees F, and the frost-free period is 120 to 170 days.

In a representative profile, the surface layer is light brownish gray coarse sandy loam about 8 inches thick. The substratum is light brownish gray very gravelly loamy coarse sand to a depth of 60 inches. The soil is moderately alkaline throughout.

Permeability is moderately rapid above a depth of 11 inches and very rapid below a depth of 11 inches. Available water capacity is 2 to 4 inches. Effective rooting depth is 60 inches or more. A seasonal water table is at a depth of 2 to 4 feet in summer. Occasional flooding occurs during spring runoff.

These soils are used for irrigated pasture and wildlife habitat.

Representative profile of Notus coarse sandy loam in an area of Notus-Falk Variant complex, about 200 feet north of the Malheur River, about 200 feet west of the fence along the county road in the NW1/4NW1/4 section 16, T. 48 S., R. 46 E.:

Ap—0 to 8 inches; light brownish gray (10YR 6/2) coarse sandy loam, dark brown (10YR 4/3) moist; weak fine granular structure; loose, very friable, slightly sticky and nonplastic; many fine and coarse roots; many irregular pores; moderately alkaline; abrupt smooth boundary.

C1—8 to 11 inches; light brownish gray (10YR 6/2) coarse sandy loam, dark brown (10YR 4/3) moist; common fine faint dark yellowish brown (10YR 4/4) mantle; massive; soft, very friable, slightly sticky and nonplastic; many fine coarse roots; many irregular pores; moderately alkaline; abrupt wavy boundary.

IIC2—11 to 60 inches; light brownish gray (10YR 6/2) very gravelly loamy coarse sand, dark brown (10YR 4/3) moist; massive; soft, very friable, nonsticky and nonplastic; few coarse roots; many irregular pores; moderately alkaline.

The A horizon has a value of 5 or 6 when dry and a chroma of 2 or 3 when moist and dry. The A horizon is coarse sandy loam or sandy loam and contains 10 to 30 percent coarse fragments. The IIC horizon has a value of 5 or 6 when dry and a chroma of 2 or 3 when dry and moist. Texture is loamy coarse sand, loamy sand, or sand and contains 35 to 80 percent coarse fragments. Depth to the IIC horizon ranges from 10 to 20 inches.

20—Notus-Falk Variant complex. This complex consists of about 50 to 60 percent Notus coarse sandy loam, about 20 to 30 percent Falk Variant fine sandy loam, about 15 percent Kimberly fine sandy loam, and about 5 percent Poden silt loam. It is in irregularly shaped areas.

Runoff is slow, and the hazard of erosion by overflow from adjacent rivers is severe. Capability unit IVw-2.

Nyssa series

The Nyssa series consists of well drained soils that formed on terraces in medium textured material over old lake-laid sediments. Slopes range from 0 to 20 percent but generally are less than 8 percent. Elevation is 2,100 to 2,600 feet. The native vegetation was needlegrass, Sandberg bluegrass, annual fescue, big sagebrush, shadscale, bud sagebrush, and forbs. Average annual precipitation is 9 to 11 inches, the average annual air temperature is 50 degrees to 54 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is light brownish gray silt loam about 13 inches thick. The subsoil is pale brown silt loam about 7 inches thick. The substratum is light gray silt loam to a depth of 25 inches. Below this is a silica-cemented hardpan about 6 inches thick. Below the hardpan, the substratum is very pale brown laminated silt loam, fine sandy loam, and loam to a depth of 60 inches. The soil is mildly alkaline above a depth of 13 inches, neutral between depths of 13 and 20 inches, moderately alkaline between depths of 20 and 31 inches, and very strongly alkaline below a depth of 31 inches.

Permeability is moderate above the hardpan and slow to very slow in the hardpan and the underlying laminated sediments. Available water capacity is 4 to 8 inches. Effective rooting depth is 20 to 40 inches.

These soils are used for irrigated onions, potatoes, sugar beets, small grain, vegetable seeds, alfalfa seed, alfalfa hay, and wildlife habitat.

Representative profile of Nyssa silt loam, 0 to 2 percent slopes, about 80 feet southeast of vertical standpipe turnout, NW1/4SW1/4SW1/4 section 11, T. 20 S., R. 46 E.:

Ap—0 to 13 inches; light brownish gray (10YR 6/2) silt loam, dark grayish brown (10YR 4/2) moist; weak medium granular structure; slightly hard, friable, slightly sticky and slightly plastic; few fine roots; many very fine tubular pores; mildly alkaline; clear smooth boundary.

B2—13 to 20 inches; pale brown (10YR 6/3) silt loam, dark brown (10YR 3/3) moist; weak coarse subangular blocky structure; slightly hard, friable, slightly sticky and slightly plastic; few fine roots; many very fine tubular pores; neutral; clear wavy boundary.

C1casi—20 to 25 inches; light gray (10YR 7/2) silt loam, grayish brown and brown (10YR 5/2 and 5/3) moist; massive; hard, firm, slightly sticky and slightly plastic; few roots; common very fine tubular pores; 45 percent calcareous durinodes 1/4 to 1 inch by 1/4 to 1/2 inch; strongly calcareous; moderately alkaline; abrupt wavy boundary.

C2casim—25 to 31 inches; light gray (10YR 7/2) indurated duripan of silty material, brown (10YR 5/3) moist; weak very thick platy structure; thin silica-indurated lenses on top of plates with a matting of roots and reddish brown organic matter on top of silica lenses; extremely hard, extremely firm, nonsticky and nonplastic; no roots except for mats on lenses; many very fine and fine tubular pores; strongly calcareous; moderately alkaline; clear wavy boundary.

C3—31 to 60 inches; very pale brown (10YR 7/3) silt loam, fine sandy loam, and loam; laminated; hard; strongly calcareous; very strongly alkaline.

The A horizon has a value of 3 or 4 when moist and a chroma of 2 or 3 when moist or dry. The A and B horizons above the duripan are silt loam or very fine sandy loam. They are less than 18 percent clay and less than 15 percent fine and coarser sand. The A and B horizons above the duripan are neutral to moderately alkaline and commonly noncalcareous, but in some pedons the B horizon is weakly calcareous. Depth to the duripan ranges from 20 to 40 inches but is dominantly 20 to 30 inches. Some pedons have very gravelly coarse sand below the duripan. Depth to bedrock is more than 60 inches.

21A—Nyssa silt loam, 0 to 2 percent slopes. This soil is in irregularly shaped areas on terraces. It has the profile described as representative of the series.

Included with this soil in mapping were about 20 percent soils that are similar to Nyssa soils but that are 15 to 20 inches deep to the hardpan. Also included are about 10 percent Owyhee soils; about 5 percent Nyssa soils, gravel substratum; about 5 percent Truesdale soils; and about 1 percent Virtue soils.

Runoff is slow, and the erosion hazard is slight. Capability unit IIIs-1.

21B—Nyssa silt loam, 2 to 5 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent soils that are similar to Nyssa soils but that are 15 to 20 inches deep to the hardpan. Also included are about 10 percent Owyhee soils; about 5 percent Nyssa soils, gravel substratum; about 5 percent Truesdale soils; and about 1 percent Virtue soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIIe-1.

21C—Nyssa silt loam, 5 to 8 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 15 percent soils that are similar to Nyssa soils but that are 15 to 20 inches deep to the hardpan. Also included are about 10 percent Owyhee soils; about 5 percent Nyssa soils, gravel substratum; and about 5 percent Truesdale soils.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IIIe-1.

21D—Nyssa silt loam, 8 to 12 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 20 percent soils that are similar to Nyssa soils but that are 15 to 20 inches deep to the hardpan. Also included are about 10 percent Owyhee soils; about 5 percent Nyssa soils, gravel substratum; and about 5 percent Truesdale soils.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IVe-4.

21E—Nyssa silt loam, 12 to 20 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 25 percent soils that are similar to Nyssa soils but that are 15 to 20 inches deep to the hardpan. Also included are about 10 percent Owyhee soils; about 5 percent Nyssa soils, gravel substratum; and about 5 percent Truesdale soils.

Runoff is moderate, and the hazard of erosion is moderate to severe. Capability unit VIe-2.

The A and AC horizons have a value of 5.5 to 6 when dry and 3 to 4 when moist. They are silt loam or very fine sandy loam. The C horizon has a value of 6 or 7 when dry and 3 or 4 when moist and a chroma of 3 when dry or moist. It is silt loam. Content of durinodes ranges from 0 to 60 percent. Depth to the duripan is 10 to 20 inches. Depth to bedrock is more than 60 inches and typically is many feet.

24—Otoole silt loam. This soil is in rounded areas on low terraces.

Included with this soil in mapping were about 10 percent Stanfield soils, about 5 percent Umapine soils, and about 2 percent Powder soils.

Flooding occurs rarely on this soil. Runoff is slow, and the hazard of erosion is slight. Capability unit IVw-1.

Owyhee series

The Owyhee series consists of well drained soils that formed on terraces in loess-capped old lacustrine materials. Slopes range from 0 to 20 percent but are generally less than 8 percent. Elevation is 2,100 to 2,500 feet. The native vegetation was bluebunch wheatgrass, needlegrass, Sandberg bluegrass, big sagebrush, shadscale, and rabbitbrush. Average annual precipitation is 8 to 10 inches, average annual air temperature is 50 degrees to 52 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is pale brown silt loam about 10 inches thick. The subsoil is pale brown silt loam about 6 inches thick. The upper part of the substratum, to a depth of 28 inches, is white silt loam, and the lower part is light gray laminated silt loam and very fine sandy loam to a depth of 60 inches. The soil is moderately alkaline above a depth of 28 inches, strongly alkaline between depths of 28 and 38 inches, and moderately alkaline below a depth of 38 inches.

Permeability is moderate in the upper part and moderately slow in the laminated sediments. Available water capacity is 9 to 12 inches. Effective rooting depth is 60 inches or more.

These soils are used for irrigated onions, potatoes, sugar beets, small grain, vegetable seeds, alfalfa seed, alfalfa hay (fig. 2), and wildlife habitat.

Representative profile of Owyhee silt loam, 0 to 2 percent slopes, about 200 feet north and 30 feet east of the section corner, SW1/4SW1/4SW1/4 section 33, T. 20 S., R. 46 E.:

Ap—0 to 10 inches; pale brown (10YR 6/3) silt loam, dark brown to brown (10YR 4/3) moist; weak fine granular structure; soft, very friable, slightly sticky and slightly plastic; many fine roots; many fine tubular pores; moderately alkaline; abrupt smooth boundary.

B—10 to 16 inches; pale brown (10YR 6/3) silt loam, dark brown to brown (10YR 4/3) moist; moderate coarse prismatic structure; slightly hard, very friable, slightly sticky and slightly plastic; many fine roots; many fine tubular pores; moderately alkaline; abrupt wavy boundary.

C1ca—16 to 28 inches; white (10YR 8/2) silt loam, brown (10YR 5/3) moist; massive; hard, firm, nonsticky and nonplastic; few coarse roots; few fine tubular pores; strongly calcareous; moderately alkaline; abrupt wavy boundary.

C2—28 to 38 inches; light gray (10YR 7/2) silt loam, grayish brown (10YR 5/2) moist; laminated lacustrine sediments in moderate medium plates that part to weak fine and medium angular and subangular blocks; hard, firm, nonsticky and nonplastic; few coarse roots; few medium tubular pores; moderately calcareous; strongly alkaline; abrupt smooth boundary.

C3—38 to 50 inches; light gray (10YR 7/2) very fine sandy loam, grayish brown (10YR 5/2) moist; laminated lacustrine sediments in weak medium plates that part to weak fine and medium angular and subangular blocks; hard, friable, nonsticky and nonplastic; few coarse roots; few medium tubular pores and many fine irregular pores; moderately calcareous; moderately alkaline; abrupt smooth boundary.

C4—50 to 60 inches; light gray (10YR 7/2) silt loam, yellowish brown (10YR 5/4) moist; laminated lacustrine sediments in weak medium plates that part to weak fine and medium angular and subangular blocks; hard, friable, slightly sticky and slightly plastic; few fine irregular pores; weakly calcareous; moderately alkaline.

The A horizon has a value of 5.5 to 6.5 when dry and 3.5 to 4.5 when moist and a chroma of 2 to 3. It is noncalcareous and neutral to moderately alkaline. It is silt loam or loam. The B horizon is noncalcareous to a depth of 12 to 24 inches. The upper boundary of the Cca horizon is below a depth of 16 inches. In places the Cca horizon contains few or common, hard, firm, rounded nodules of soil material or cicada krotovinas 0.5 to 0.8 inch in diameter. Depth to the underlying laminated sediments ranges from about 20 to 35 inches.

25A—Owyhee silt loam, 0 to 2 percent slopes. This soil is in regularly shaped areas on terraces. It has the profile described as representative of the series.

Included with this soil in mapping were about 10 percent Nyssa soils, about 5 percent Greenleaf soils, and about 5 percent Nyssa silt loam, gravel substratum.

Runoff is slow, and the hazard of erosion is slight. Capability unit I-1.

25B—Owyhee silt loam, 2 to 5 percent slopes. This soil is in regularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent Nyssa soils and about 5 percent Nyssa silt loam, gravel substratum.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIe-2.

25C—Owyhee silt loam, 5 to 8 percent slopes. This soil is in regularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent Nyssa soils and about 5 percent Nyssa silt loam, gravel substratum.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IIIe-1.

25D—Owyhee silt loam, 8 to 12 percent slopes. This soil is in regularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent Nyssa soils and about 5 percent Nyssa silt loam, gravel substratum.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IVe-4.

25E—Owyhee silt loam, 12 to 20 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent Nyssa soils and about 5 percent Nyssa silt loam, gravel substratum.

Runoff is rapid, and the hazard of erosion is severe. Capability unit VIe-2.

30A—Sagehill fine sandy loam, 0 to 2 percent slopes. This soil is in irregularly shaped areas on terraces. It has the profile described as representative of the series.

Included with this soil in mapping were about 10 percent Truesdale soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIs-2.

30B—Sagehill fine sandy loam, 2 to 5 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent Truesdale soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is slow, and the erosion hazard is slight. Capability unit IIe-2.

30C—Sagehill fine sandy loam, 5 to 8 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent Truesdale soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is slow, and the hazard of erosion is moderate. Capability unit IIIe-3.

30E—Sagehill fine sandy loam, 12 to 20 percent slopes. This soil is in irregularly shaped areas on terraces.

Included with this soil in mapping were about 10 percent Truesdale soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is medium, and the hazard of erosion is high. Capability unit VIe-2.

Stanfield series

The Stanfield series consists of moderately well drained soils that formed in medium textured old alluvium on low terraces and bottom lands. Slopes are 0 to 2 percent. Elevation is 2,100 to 2,600 feet. The native vegetation was saltgrass, giant wildrye, and greasewood. Average annual precipitation is 9 to 11 inches, average annual air temperature is 48 degrees to 54 degrees F, and the frost-free period is 120 to 170 days.

In a representative profile, the surface layer is very pale brown silt loam about 12 inches thick. The upper part of the underlying material, to a depth of 22 inches, is very pale brown silt loam, and the lower part is a silica- and calcium-cemented hardpan. The soil is very strongly alkaline throughout.

Permeability is moderate above the hardpan and very slow in the hardpan. Available water capacity is 2.5 to 7 inches. Effective rooting depth is 20 to 40 inches.

These soils are used for irrigated and subirrigated pasture and wildlife habitat.

Representative profile of Stanfield silt loam about 3 miles southwest of Vale, SW1/4NE1/4SE1/4 section 2, T. 19 S., R. 44 E.:

Ap—0 to 8 inches; very pale brown (10YR 7/3) silt loam, dark brown (10YR 4/3) moist; weak fine subangular blocky structure; soft, very friable, nonsticky and slightly plastic; many fine roots; many fine tubular and irregular pores; strongly calcareous; very strongly alkaline; abrupt smooth boundary.

AC—8 to 12 inches; very pale brown (10YR 7/3) silt loam, dark brown to brown (10YR 4/3) moist; weak fine subangular blocky structure; soft, very friable, nonsticky and slightly plastic; many fine roots; many fine tubular pores; strongly calcareous; very strongly alkaline; abrupt smooth boundary.

C1—12 to 22 inches; very pale brown (10YR 7/3) silt loam, dark brown (10YR 4/3) moist; massive; soft, very friable, nonsticky and slightly plastic; common fine roots; few fine tubular pores; 10 to 20 percent calcium coated durinodes 1/4 to 3/4 inches in diameter; strongly calcareous; very strongly alkaline; abrupt smooth boundary.

C2casim—22 to 26 inches; very pale brown (10YR 7/3) strongly calcium- and silica-cemented duripan, dark brown (10YR 4/3) moist; strongly calcareous; very strongly alkaline.

The A, AC, and C horizons have a value of 5.5 to 7 when dry and 3.5 or 4 when moist and a chroma of 2 or 3 when dry and moist. They are silt loam or very fine sandy loam. The duripan ranges from strongly cemented to indurated but is indurated in some part in every pedon. Depth to the pan ranges from about 20 to 40 inches. The pan ranges from 4 to 30 inches in thickness, but it is generally 7 to 20 inches thick. In some pedons there is a series of pans with friable horizons between the pans.

31—Stanfield silt loam. This soil is in irregularly shaped areas on low terraces and bottom lands.

Included with this soil in mapping were about 10 percent Umapine soils and about 5 percent Powder soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IVw-1.

Truesdale series

The Truesdale series consists of well drained soils that formed on terraces in moderately coarse textured material over old lacustrine sediments. Slopes are 0 to 12 percent. Elevation is 2,200 to 2,600 feet. The native vegetation was needlegrass, Sandberg bluegrass, big sagebrush, hopsage, and shadscale. Average annual precipitation is 9 to 11 inches, average annual air temperature is 50 degrees to 54 degrees F, and the frost-free period is 150 to 170 days.

In a representative profile, the surface layer is pale brown and brown fine sandy loam about 13 inches thick. The upper part of the underlying material is pale brown fine sandy loam to a depth of 26 inches, the middle part is white very fine sandy loam to a depth of 36 inches, and the lower part is a silica- and calcium-cemented hardpan. This soil is moderately alkaline throughout.

Permeability is moderately rapid above the hardpan and very slow in the hardpan. Available water capacity is 3 to 6 inches. Effective rooting depth is 20 to 40 inches.

These soils are used for irrigated onions, potatoes, sugar beets, small grain, alfalfa seed, alfalfa hay, and wildlife habitat.

Representative profile of Truesdale fine sandy loam, 0 to 2 percent slopes, 2 1/2 miles south of Ontario, 50 feet west of water drop from the ditch going east, SE1/4SE1/4 section 29, T. 18 S., R. 47 E.:

Ap1—0 to 5 inches; pale brown (10YR 6/3) fine sandy loam, dark brown (10YR 3/3) moist; weak fine granular structure; soft, very friable, nonsticky and nonplastic; many fine roots; many fine tubular pores; moderately alkaline; abrupt smooth boundary.

Ap2—5 to 13 inches; brown (10YR 5/3) fine sandy loam, dark brown (10YR 3/3) moist; massive; soft, very friable, nonsticky and slightly plastic; many fine roots; many fine tubular pores; moderately alkaline; abrupt smooth boundary.

C1—13 to 26 inches; pale brown (10YR 6/3) fine sandy loam, dark brown (10YR 3/3) moist; massive; soft, very friable, nonsticky and slightly plastic; many fine roots; many fine tubular pores; moderately alkaline; abrupt wavy boundary.

C2ca—26 to 36 inches; white (10YR 8/2) very fine sandy loam, pale brown (10YR 6/3) moist; massive; very hard, very friable, nonsticky and slightly plastic; few fine roots; few fine tubular pores; 10 to 20 percent durinodes 1/4 to 3/4 inch in diameter; strongly calcareous; moderately alkaline; gradual wavy boundary.

C3sicam—36 to 42 inches; pale brown (10YR 6/3) very fine sandy loam, yellowish brown (10YR 5/4) moist; white (10YR 8/2) coatings; massive; very hard, very firm, nonsticky and nonplastic; thin horizontal strongly lime-silica-cemented lenses 1/2 to 3 inches apart, concentrated in upper part; matrix very weakly cemented; few fine tubular pores; strongly calcareous; moderately alkaline.

The A horizon has a value of 5 or 6 when dry and 3 or 4 when moist and a chroma of 2 or 3. The C horizon has a value of 6, 7, or 8 when dry and 3 or 4 when moist. Chroma is 2, 3, or 4. Texture is fine sandy loam or coarse sandy loam. Depth to the duripan ranges from 20 to 40 inches but is generally about 30 inches. The duripan ranges from a strongly silica-lime-cemented horizon to a weakly lime-silica-cemented horizon. The weaker duripans have few very thin strongly cemented lenses in a very weakly cemented matrix containing 15 to 50 percent very hard nodules, some of which are durinodes.

32A—Truesdale fine sandy loam, 0 to 2 percent slopes. This soil is in irregularly shaped areas on medium and high terraces. It has the profile described as representative of the series.

Included with this soil in mapping were about 10 percent Sagehill soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIIs-2.

32B—Truesdale fine sandy loam, 2 to 5 percent slopes. This soil is in irregularly shaped areas on medium and high terraces.

Included with this soil in mapping were about 10 percent Sagehill soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IIIe-3.

32C—Truesdale fine sandy loam, 5 to 8 percent slopes. This soil is in irregularly shaped areas on medium and high terraces.

Included with this soil in mapping were about 10 percent Sagehill soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IIIe-3.

32D—Truesdale fine sandy loam, 8 to 12 percent slopes. This soil is in irregularly shaped areas on medium and high terraces.

Included with this soil in mapping were about 10 percent Sagehill soils, about 5 percent Cencove soils, and about 5 percent Nyssa soils.

Runoff is medium, and the hazard of erosion is moderate. Capability unit IVe-2.

Turbyfill series

The Turbyfill series consists of well drained soils that formed on bottom lands and fans in mixed, moderately coarse textured alluvium. Slopes are 0 to 5 percent. Elevation is 2,100 to 2,600 feet. The native vegetation was Sandberg bluegrass, giant wildrye, and big sagebrush. Average annual precipitation is 8 to 11 inches, average annual air temperature is 47 degrees to 52 degrees F, and the frost-free period is 120 to 160 days.

In a representative profile, the surface layer is pale brown fine sandy loam about 6 inches thick. The underlying material is pale brown and very pale brown fine sandy loam to a depth of 60 inches. The soil is moderately alkaline throughout.

Permeability is moderately rapid. Available water capacity is 7 to 9 inches. Effective rooting depth is 60 inches or more.

These soils are used for irrigated onions, potatoes, sugar beets, small grain, alfalfa seed, alfalfa hay, and wildlife habitat.

Representative profile of Turbyfill fine sandy loam, 0 to 2 percent slopes, about 1,400 feet east and 60 feet north of section corner, SE1/4SW1/4 section 6, T. 21 S., R. 47 E.:

Ap—0 to 6 inches; pale brown (10YR 6/3) fine sandy loam, very dark grayish brown (10YR 3/2) moist; weak fine granular structure; soft, very friable, nonsticky and nonplastic; many fine and coarse roots; many fine irregular and tubular pores; moderately alkaline; abrupt smooth boundary.

C1—6 to 20 inches; pale brown (10YR 6/3) fine sandy loam, very dark grayish brown (10YR 3/2) moist; massive; soft, very friable, nonsticky and nonplastic; many fine and coarse roots; many fine irregular and tubular pores; slightly calcareous; moderately alkaline; gradual wavy boundary.

C2ca—20 to 37 inches; pale brown (10YR 6/3) fine sandy loam, dark brown to brown (10YR 4/3) moist; massive; soft, very friable, nonsticky and nonplastic; many fine and coarse roots; many fine irregular and tubular pores; strongly calcareous; moderately alkaline; gradual wavy boundary.

C3ca—37 to 60 inches; very pale brown (10YR 7/3) fine sandy loam, dark brown to brown (10YR 4/3) moist; single grained; soft, very friable, nonsticky and nonplastic; few fine roots; many fine irregular pores; strongly calcareous; moderately alkaline.

The A horizon has a value of 5 to 6 when dry and 3 to 5 when moist and a chroma of 2 to 3 when dry and moist. It is fine sandy loam or sandy loam that is less than 18 percent clay and less than 15 percent coarse fragments. It is noncalcareous to moderately calcareous and is neutral to moderately alkaline. The C1 horizon is moderately to strongly calcareous and moderately alkaline. The Cca horizon has accumulations of carbonates but contains less than 15 percent in any layer at least 6 inches thick, and it is not cemented.

33A—Turbyfill fine sandy loam, 0 to 2 percent slopes. This soil is in irregularly shaped areas on bottom lands. It has the profile described as representative of the series.

Included with this soil in mapping were about 10 percent Garbutt soils, about 5 percent Quincy soils, and about 5 percent Cencove soils.

Runoff is slow, and the hazard of erosion is moderate. Capability unit I-1.

33B—Turbyfill fine sandy loam, 2 to 5 percent slopes. This soil is in irregularly shaped areas on bottom lands.

Included with this soil in mapping were about 10 percent Garbutt soils, about 5 percent Quincy soils, and about 5 percent Cencove soils.

Runoff is slow, and the hazard of erosion is moderate. Capability unit IIe-2.

Umapine series

The Umapine series consists of somewhat poorly drained soils that formed on bottom lands and low terraces in medium textured old alluvium. Slopes are 0 to 2 percent. Elevation is 2,100 to 2,600 feet. The native vegetation was saltgrass, giant wildrye, and greasewood. Average annual precipitation is 9 to 11 inches, average annual air temperature is 48 degrees to 54 degrees F, and the frost-free period is 120 to 170 days.

In a representative profile, the surface layer is pale brown silt loam about 11 inches thick. The upper part of the underlying material, to a depth of 23 inches, is very pale brown silt loam, and the lower part is light gray silt loam to a depth of 60 inches. The soil is very strongly alkaline above a depth of 6 inches, strongly alkaline between depths of 6 and 23 inches, and moderately alkaline below a depth of 23 inches.

Permeability is moderately slow. Available water capacity is 7 to 12 inches. Effective rooting depth is 60 inches or more. A seasonal water table is at a depth of 2 to 5 feet in winter and spring. Rare flooding occurs during spring runoff.

These soils are used for irrigated small grain, alfalfa hay, pasture, and wildlife habitat.

Representative profile of Umapine silt loam about 20 feet northeast of 1/16 corner SE1/4NE1/4 section 1, T. 19 S., R. 44 E.:

A11—0 to 2 inches; pale brown (10YR 6/3) silt loam, dark brown (10YR 3/3) moist; weak very fine granular structure; soft, very friable, nonsticky and plastic; many fine and coarse roots; many fine round pores; strongly calcareous; very strongly alkaline; abrupt smooth boundary.

A12—2 to 6 inches; pale brown (10YR 6/3) silt loam, dark brown to brown (10YR 4/3) moist; strong thin platy structure; soft, very friable, slightly sticky and plastic; many fine and coarse roots; many fine tubular pores; strongly calcareous; very strongly alkaline; abrupt smooth boundary.

A13—6 to 11 inches; pale brown (10YR 6/3) silt loam, dark brown to brown (10YR 4/3) moist; moderate thick platy structure; soft, very friable, slightly sticky and plastic; many fine and coarse roots; many fine tubular pores; strongly calcareous; strongly alkaline; gradual wavy boundary.

C1—11 to 23 inches; very pale brown (10YR 7/3) silt loam, dark brown to brown (10YR 4/3) moist; weak coarse prismatic structure; soft, very friable, slightly sticky and plastic; many fine and coarse roots; many fine tubular pores; strongly calcareous; strongly alkaline; abrupt smooth boundary.

C2—23 to 30 inches; light gray (10YR 7/2) silt loam, dark brown (10YR 3/3) moist; massive; hard, friable, slightly sticky and plastic; few fine roots; few fine tubular pores; 50 percent rounded 1/4- to 1/2-inch silica- and calcium-cemented nodules; strongly calcareous; moderately alkaline; gradual wavy boundary.

C3—30 to 60 inches; light gray (10YR 7/2) silt loam, dark brown (10YR 3/3) moist; massive; hard, friable, slightly sticky and plastic; few fine roots; few fine tubular pores; strongly calcareous; moderately alkaline.

The A horizon has a value of 5 or 6 when dry and 4 or 5 when moist and a chroma of 2 to 3. It is silt loam, very fine sandy loam, or fine sandy loam. The upper 40 inches is moderately to very strongly alkaline. Content of exchangeable sodium exceeds 15 percent in the upper 20 inches. These soils are calcareous in all parts between depths of 10 and 20 inches.

34—Umapine silt loam. This soil is in irregularly shaped areas on bottom lands and low terraces.

Included with this soil in mapping were about 10 percent Stanfield soils and about 5 percent Powder soils.

Runoff is slow, and the hazard of erosion is slight. Capability unit IIIw-1.

Virtue series

The Virtue series consists of well drained soils that formed on terraces in medium textured old alluvial material over a cemented hardpan. Slopes range from 0 to 20 percent but are generally less than 8 percent. Elevation is 2,300 to 2,600 feet. The native vegetation was bluebunch wheatgrass, Sandberg bluegrass, and big sagebrush. Average annual precipitation is 9 to 11 inches, average annual air temperature is 48 degrees to 52 degrees F, frost-free period is 110 to 170 days.

In a representative profile, the surface layer is light brownish gray and pale brown silt loam about 14 inches thick. The subsoil is yellowish brown silty clay loam about 12 inches thick. An indurated, silica-lime hardpan is at a depth of about 26 inches. The soil is neutral in the upper 5 inches, mildly alkaline between depths of 5 and 24 inches, and moderately alkaline below a depth of 24 inches.

Permeability is moderately slow. Available water capacity is 5 to 8.5 inches. Effective rooting depth is 20 to 40 inches.

These soils are used for irrigated small grain, alfalfa seed, alfalfa hay, occasional row crops, and wildlife habitat.

Representative profile of Virtue silt loam, 0 to 2 percent slopes, about 400 feet west of gravel pit, 200 feet south of fence, NW1/4NW1/4 section 12, T. 17 S., R. 43 E.:

A1—0 to 5 inches; light brownish gray (10YR 6/2) silt loam, dark brown (10YR 3/3) moist; moderate fine to medium platy structure; soft, very friable, slightly sticky and slightly plastic; many fine roots; many fine tubular pores; neutral abrupt smooth boundary.

A3—5 to 14 inches; pale brown (10YR 6/3) silt loam, dark brown (10YR 4/3) moist; weak coarse prismatic structure parting to weak medium subangular blocky; soft, very friable, slightly sticky and slightly plastic; many fine roots; many fine tubular pores; mildly alkaline; clear smooth boundary.

B21t—14 to 19 inches; yellowish brown (10YR 5/4) silty clay loam, dark brown (10YR 4/3) moist; light gray (10YR 7/2) coatings; moderate medium prismatic structure parting to moderate fine subangular blocky; hard, firm, sticky and plastic; few coarse roots; few fine tubular pores; few thin clay films on surfaces of peds and in pores; mildly alkaline; clear wavy boundary.

B22t—19 to 24 inches; yellowish brown (10YR 5/4) silty clay loam, dark brown (10YR 3/3) moist; moderate medium subangular blocky structure; hard, firm, sticky and plastic; few coarse roots; few fine tubular pores; few thin clay films on surfaces of peds and in pores; mildly alkaline; clear wavy boundary.

Appendix F: Equations for figures and exhibits

This appendix presents the equations used in procedure applications to generate figures and exhibits in TR-55.

Figure 2-1 (runoff equation):

$$Q = \frac{\left[P - 0.2\left(\frac{1000}{CN} - 10\right)\right]^2}{P + 0.8\left(\frac{1000}{CN} - 10\right)}$$

where

Q = runoff (in),
P = rainfall (in), and
CN = runoff curve number.

Figure 2-3 (composite CN with connected impervious area):

$$CN_c = CN_p + (P_{imp}/100)(98 - CN_p)$$

where

CN_c = composite runoff curve number,
CN_p = pervious runoff curve number, and
P_{imp} = percent imperviousness.

Figure 2-4 (composite CN with unconnected impervious areas and total impervious area less than 30%):

$$CN_c = CN_p + (P_{imp}/100)(98 - CN_p)(1 - 0.5R)$$

where R = ratio of unconnected impervious area to total impervious area.

Figure 3-1 (average velocities for estimating travel time for shallow concentrated flow):

Unpaved $V = 16.1345\,(s)^{0.5}$
Paved $V = 20.3282\,(s)^{0.5}$

where

V = average velocity (ft/s), and
s = slope of hydraulic grade line (watercourse slope, ft/ft).

These two equations are based on the solution of Manning's equation (Eq. 3-4) with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, ft). For unpaved areas, n is 0.05 and r is 0.4; for paved areas, n is 0.025 and r is 0.2.

Exhibit 4 (unit peak discharges for SCS type I, IA, II, and III distributions):

$$\log(q_u) = C_0 + C_1 \log(T_c) + C_2 [\log(T_c)]^2$$

where

q_u = unit peak discharge (csm/in),
T_c = time of concentration (hr) (minimum, 0.1; maximum, 10.0), and
C_0, C_1, C_2 = coefficients from table F-1.

Figure 6-1 (approximate detention basin routing through single- and multiple-stage structures for 24-hour rainfalls of the indicated type):

$$V_s/V_r = C_0 + C_1 (q_o/q_i) + C_2 (q_o/q_i)^2 + C_3 (q_o/q_i)^3$$

where

V_s/V_r = ratio of storage volume (V_s) to runoff volume (V_r),
q_o/q_i = ratio of peak outflow discharge (q_o) to peak inflow discharge (q_i), and
C_0, C_1, C_2, C_3 = coefficients from table F-2.

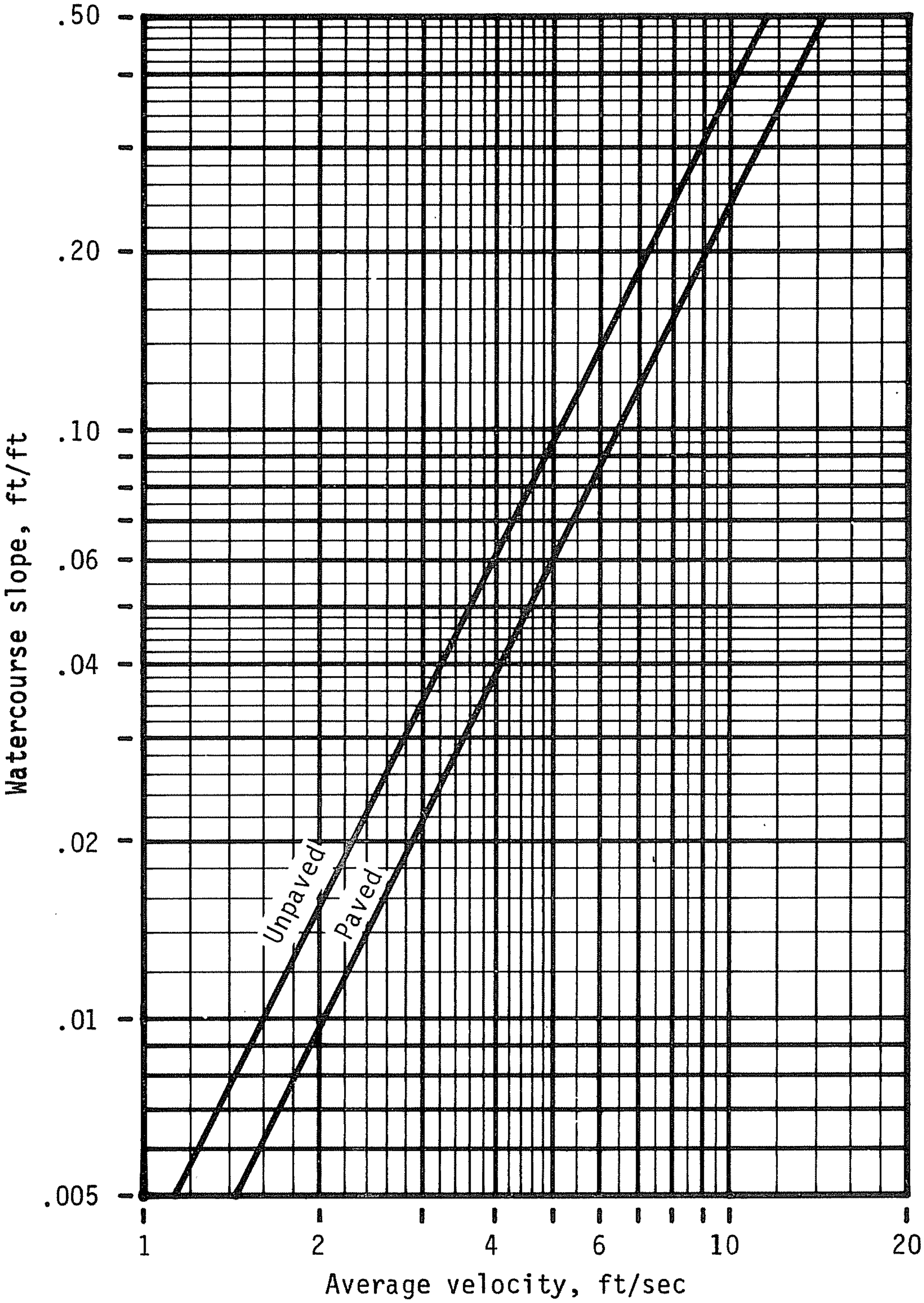


Figure 3-1.—Average velocities for estimating travel time for shallow concentrated flow.

Appendix D

Ditch and Culvert Analysis

DORK CANAL

Malheur Drive Culvert #1

0.8' freeboard

```
*================================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).            |
| Use this section for culvert comparisons                      |
*================================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	63.384	0.981	0.420	2.402	4.000	2147.400	2145.712	Max Flow

```
*===========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the         |
|         culvert was in a particular classification        |
|         during the simulation.  The time is in minutes.   |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification      |
|         condition is based on the HW and TW depths.       |
*===========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.250	0.000	0.000	0.000	0.000	0.000	29.750	Projecting

Full Depth

```
*==============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).            |
| Use this section for culvert comparisons                      |
*==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	79.769	1.312	0.563	2.705	4.000	2148.200	2146.015	Max Flow

```
*=========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|         culvert was in a particular classification       |
|         during the simulation.  The time is in minutes.  |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification     |
|         condition is based on the HW and TW depths.      |
*=========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	0.000	0.000	0.000	30.000	Projecting

Malheur Drive Culvert #2

1' freeboard

N/A

Full Depth

```
*==============================================================*
 | Table E13. Channel losses(H), headwater depth (HW), tailwater |
 | depth (TW), critical and normal depth (Yc and Yn).            |
 | Use this section for culvert comparisons                      |
 *==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	69.942	1.089	0.434	2.527	2.775	2148.200	2145.867	Max Flow

```
*========================================================*
 | CULVERT ANALYSIS CLASSIFICATION, and the time the      |
 |        culvert was in a particular classification      |
 |        during the simulation.  The time is in minutes.  |
 |        The Dynamic Wave Equation is used for all conduit|
 |        analysis but the culvert flow classification     |
 |        condition is based on the HW and TW depths.      |
 *========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	0.000	0.000	0.000	30.000	Projecting

Culvert East of North Verde Drive

1' freeboard

```
*==============================================================*
 | Table E13. Channel losses(H), headwater depth (HW), tailwater |
 | depth (TW), critical and normal depth (Yc and Yn).            |
 | Use this section for culvert comparisons                      |
 *==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	37.519	0.497	3.014	1.824	4.000	2149.860	2146.184	Max Flow

```
*==========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|         culvert was in a particular classification       |
|         during the simulation.  The time is in minutes.  |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification     |
|         condition is based on the HW and TW depths.      |
*==========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	28.250	1.750	0.000	0.000	0.000	0.000	0.000	0.000	Mitered to Slope

Full Depth

```
*=============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).           |
| Use this section for culvert comparisons                     |
*=============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	43.879	0.621	3.757	1.980	4.000	2150.860	2146.340	Max Flow

```
*==========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|         culvert was in a particular classification       |
|         during the simulation.  The time is in minutes.  |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification     |
|         condition is based on the HW and TW depths.      |
*==========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	1.000	0.000	0.000	0.000	0.000	0.000	29.000	0.000	Mitered to Slope

Culvert under private driveway

1' freeboard

```
*==============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).            |
| Use this section for culvert comparisons                      |
*==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	-77.101	1.655	-0.374	2.657	2.915	2148.497	2150.460	Max Flow

```
*=========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|         culvert was in a particular classification       |
|         during the simulation.  The time is in minutes.  |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification     |
|         condition is based on the HW and TW depths.      |
*=========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	30.000	0.000	0.000	0.000	Projecting

Full Depth

```
*==============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).            |
| Use this section for culvert comparisons                      |
*==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	-95.525	2.218	-0.499	2.961	4.000	2148.801	2151.460	Max Flow

```
*============================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the          |
|        culvert was in a particular classification          |
|        during the simulation.  The time is in minutes.     |
|        The Dynamic Wave Equation is used for all conduit|
|        analysis but the culvert flow classification        |
|        condition is based on the HW and TW depths.         |
*============================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	30.000	0.000	0.000	0.000	Projecting

Culvert South of the Wastewater Treatment Plant

1' freeboard

```
*=================================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).            |
| Use this section for culvert comparisons                      |
*=================================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	82.041	1.639	0.519	2.744	4.000	2151.570	2149.314	Max Flow

```
*==========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|         culvert was in a particular classification       |
|         during the simulation.  The time is in minutes.  |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification     |
|         condition is based on the HW and TW depths.      |
*==========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	29.500	0.500	0.000	0.000	0.000	0.000	0.000	0.000	Projecting

Full Depth

```
*==============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).           |
| Use this section for culvert comparisons                     |
*==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	101.095	2.261	0.708	3.046	4.000	2152.570	2149.616	Max Flow

```
*==========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|         culvert was in a particular classification       |
|         during the simulation.  The time is in minutes.  |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification     |
|         condition is based on the HW and TW depths.      |
*==========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	0.000	0.000	30.000	0.000	Projecting

STEWART CARTER CANAL

SW 4th Avenue Culvert

1' freeboard

N/A

Full Depth

```
*==============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).           |
| Use this section for culvert comparisons                     |
*==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	47.233	0.752	0.531	1.630	2.500	2177.370	2175.920	Max Flow

```
*==========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|         culvert was in a particular classification       |
|         during the simulation.  The time is in minutes.  |
|         The Dynamic Wave Equation is used for all conduit|
|         analysis but the culvert flow classification     |
|         condition is based on the HW and TW depths.      |
*==========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	3.000	27.000	0.000	0.000	0.000	0.000	0.000	0.000	45 deg Wingwall Flares

SW 7th Pl. Culvert

1' freeboard

N/A

Full Depth

```
*===============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).            |
| Use this section for culvert comparisons                      |
*===============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	64.246	1.825	0.364	2.512	1.933	2185.810	2182.993	Max Flow

```
*=========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the        |
|        culvert was in a particular classification        |
|        during the simulation.  The time is in minutes.   |
|        The Dynamic Wave Equation is used for all conduit|
|        analysis but the culvert flow classification      |
|        condition is based on the HW and TW depths.       |
*=========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	0.000	0.000	0.250	29.750	Groove End Projecting

UN-NAMED DITCH

Upstream from OWEB Site #5

1' freeboard

```
*==============================================================*
| Table E13. Channel losses(H), headwater depth (HW), tailwater |
| depth (TW), critical and normal depth (Yc and Yn).            |
| Use this section for culvert comparisons                      |
*==============================================================*
```

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	53.511	1.930	1.662	2.375	3.000	2146.000	2142.355	Max Flow

```
*==========================================================*
| CULVERT ANALYSIS CLASSIFICATION, and the time the          |
|        culvert was in a particular classification          |
|        during the simulation.  The time is in minutes.     |
|        The Dynamic Wave Equation is used for all conduit|
|        analysis but the culvert flow classification        |
|        condition is based on the HW and TW depths.         |
*==========================================================*
```

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	0.000	0.000	30.000	0.000	Projecting

Full Depth

Table E13. Channel losses(H), headwater depth (HW), tailwater depth (TW), critical and normal depth (Yc and Yn).
Use this section for culvert comparisons

Conduit Name	Maximum Flow	Head Loss	Friction Loss	Critical Depth	Normal Depth	HW Elevat	TW Elevat	
Culvert	61.125	2.397	2.067	2.521	3.000	2147.000	2142.501	Max Flow

CULVERT ANALYSIS CLASSIFICATION, and the time the culvert was in a particular classification during the simulation. The time is in minutes. The Dynamic Wave Equation is used for all conduit analysis but the culvert flow classification condition is based on the HW and TW depths.

Conduit Name	Mild Slope Critical D Outlet Control	Mild Slope TW Control Outlet Control	Steep Slope TW Insignf Entrance Control	Slug Flow Outlet/ Entrance Control	Mild Slope TW > D Outlet Control	Mild Slope TW <= D Outlet Control	Outlet Control	Inlet Control	Inlet Configuration
Culvert	0.000	0.000	0.000	0.000	0.000	0.000	30.000	0.000	Projecting

Appendix E

Interagency Agreement

Interagency cooperative use agreements with irrigation districts:
(Malheur Drainage District, Owyhee Ditch Company)

1. Limitations of use *(any capacity limitations on quantity?)*
   a. City to irrigation facilities
   b. Tailwater to City facilities
2. Quality standards to discharge
   a. TSS – concentration or % removal?
   b. City to irrigation vs. tailwater to City
   c. Monitoring
3. Maintenance
   a. Cleaning – pipes vs. ditches
   b. Frequency - for water quality goals, carrying capacity
   c. Responsibility for pipes, ditches, other (if have tailwater settling ponds, may want agreement to share material removed from settling ponds with property owners)
   d. Cost sharing
4. Repairs – responsibility
   a. Pipes: City
   b. Ditches/canals: respective drainage district/ ditch company
5. Insurance responsibilities
6. Emergency procedures - notifications
7. Capital improvement plans
   a. Potential locations for improvements (e.g. tailwater settling ponds) – *consider relative to TMDL, cost-effectiveness vs. increased cleaning frequency*
   b. Cost sharing
8. Public education (stormwater & agricultural BMPs – *see list below for examples*)
9. Future monitoring
   a. Locations, parameters, frequency
   b. Responsibility

Examples of agricultural BMPs:

A. Operational
   1. Use soil moisture measurements to help plan irrigation
   2. Fertilizer management
   3. Apply PAM (PAM-chemical combinations even better), or mulch furrows to prevent erosion
   4. Drip or sprinkler vs. furrow irrigation
   5. Contour strip farming
   6. Conservation tillage (≥30% soil covered by crop residues)
   7. Retirement of highly erodible land
   8. Gather & reuse surface runoff
B. Structural
   1. Conservation buffers (remove up to 80% sediment & 40% phosphorus)
      a. Filter strips near surface water
      b. Contour buffer strips
      c. Field borders
      d. Windbreaks
      e. Wetlands
   2. Sedimentation ponds (modified stock ponds)

Draft Interagency Agreement

Between City of Ontario and the Malheur Irrigation District

Inasmuch as the Malheur Drainage District, hereafter referred to as the District, and the City of Ontario, hereafter referred to as the City, currently have interconnecting drainage systems, this agreement is intended to clarify the responsibilities of each party.

Malheur Drainage District Agrees to

1) Accept storm water runoff from developments within the Ontario City limits provided that
   - There is adequate downstream capacity.
   - The connection to the District is according to accepted District standards and engineering practices.

2) Clean and maintain open channel sections currently under the Districts jurisdiction. Also clean and maintain pipelines outside of the City limits.

3) Pay all replacement and repair costs of District owned pipelines and open channels.

4) Encourage removal of Total Suspended Solids (TSS) from irrigation return, with emphasis on reducing the TSS entering City owned and maintained pipelines and ditches.

5) Be responsible for any damage resulting from failure (i.e. collapse of pipe) of District facilities and pipelines.

6) Maintain a minimum liability insurance of two million dollars ($2,000,000.00).

7) As recommended by regulatory agencies, or as negotiated with the City, the District will actively participate in the reduction of pollutants by
   - Participating in education and source control programs for agricultural users within the District's drainage boundaries.
   - Participating in future upstream and downstream monitoring of flow and potential pollutants.
   - Participating in potential future capital improvements intended to meet future discharge requirements. Where capital improvements, flow and water quality monitoring, and other programs are necessary to meet future discharge requirements at the Snake River, the District's share shall be proportioned based on the following table

Location of Monitoring Site or Improvement or Program	% District*	% City*	Comment
Upstream of City Limits and within District boundaries	100	0	
From District facilities entering City limits downstream to Stewart Carter Canal discharge @ the Dork Canal	50	50	
The Dork Canal downstream of Stewart Carter Canal	33 1/3	33 1/3	Remaining cost carried by others
City pipelines and channels downstream of District Facilities (i.e. Pump Station)	25	75	Currently runoff from 60 ± acres pumped into City system
District pipelines downstream of City limits (south of town)	TBD-% of upstream drainage area outside of City limits	TBD-% of upstream drainage area within City limits	

*Where a capital improvement, monitoring requirement, or program is believed to be required largely because of a particular type of discharger (i.e. agricultural versus urban), then the District and City may negotiate a more fair cost distribution.

8) Allow City to use irrigation return and groundwater infiltration collected in the District's lines and ditches for the purpose of irrigating properties within the City.

The City of Ontario Agrees to

1) Encourage post development runoff to be limited to predevelopment conditions.

2) Periodically assess the capacity of District facilities including ditches and pipelines so as to ensure that additional flows from the Ontario storm system do not overly tax the existing system. Where additional downstream facilities are required to meet increased runoff requirements, the City will be solely responsible for constructing and maintaining said facilities. In addition, where damage is determined to be caused by storm water runoff from the addition of new developments within the City, the City will be responsible for correcting said damages.

3) Clean annually and TV inspect every five (5) years the District pipelines located within the City limits.

4) Continue to clean and maintain City pipelines located downstream of existing District facilities. Also, maintain any irrigation facilities constructed for the purpose of withdrawing water for City irrigation.

5) As recommended by regulatory agencies, actively participate in the reduction of pollutants by
   - Participating in education and source control programs for urban users contributing flow to the District lines and ditches
   - Supervising and directing all monitoring efforts, providing personnel and lab facilities for water quality testing of many parameters for which the City is currently set up to test, including parameter such as TSS and BOD. City will maintain records of time and materials to be partially reimbursed based on the cost distribution described above).
   - Assisting in financing the City's share of capital improvements, monitoring, and other programs required by regulatory agencies, or as mutually agreed upon.

This agreement should be reviewed every five (5) years and updated to reflect changes in development, additional flow monitoring data, future engineering studies, and changes in regulatory requirements.

Draft Interagency Agreement

Between City of Ontario and the Owyhee Ditch Company

Inasmuch as the Owyhee Ditch Company, and the City of Ontario, hereafter referred to as the City, currently have interconnecting drainage systems, this agreement is intended to clarify the responsibilities of each party.

Owhyee Ditch Company Agrees to

1) Accept storm water runoff from developments within the Ontario City limits provided that
    - There is adequate downstream capacity.
    - The connection to the Stewart Carter Canal is according to accepted District standards and engineering practices.

2) Clean and maintain open channel sections currently under the Districts jurisdiction. Also clean and maintain pipelines outside of the City limits.

3) Pay all replacement and repair costs of the Owyhee Ditch Company facilities, pipelines, and canals.

4) Be responsible for any damage resulting from failure (i.e. collapse of pipe) of District facilities and pipelines.

5) Maintain a minimum liability insurance of two million dollars ($2,000,000.00).

6) As recommended by regulatory agencies, or as negotiated with the City, the District will actively participate in the reduction of pollutants by
    - Participating in education and source control programs for agricultural users within the District's drainage boundaries.
    - Participating in future upstream and downstream monitoring of flow and potential pollutants.
    - Participating in potential future capital improvements and programs intended to meet future discharge requirements. Where capital improvements, monitoring, and other programs are necessary to meet future discharge requirements at the Snake River, the Owyhee Ditch Company's share shall be proportioned based on the following table

Location of Monitoring Site or Improvement or Program	% Owhyee Ditch Company*	% City*	Comment
Upstream of City limits	100	0	
Downstream of City limits to the Dork Canal	50	50	
The Dork Canal downstream of Steward Carter Canal	33 1/3	33 1/3	Remaining cost to be carried by others

*Where a capital improvement, monitoring requirement, or program is believed to be required largely because of a particular type of discharger (i.e. agricultural versus urban), then the District and City may negotiate a more fair cost distribution.

<u>The City of Ontario Agrees to</u>

1) Encourage post development runoff to be limited to predevelopment conditions.

2) Periodically assess the capacity of District facilities including ditches and pipelines so as to ensure that additional flows from the Ontario storm system do not overly tax the existing system. Where additional downstream facilities are required to meet increased runoff requirements, the City will be solely responsible for constructing and maintaining said facilities. In addition, where damage is determined to be caused by storm water runoff from the addition of new developments within the City, the City will be responsible for correcting said damages.

3) Clean annually and TV inspect every five (5) years the Ditch Company pipelines located within the City limits.

4) As recommended by regulatory agencies, actively participate in the reduction of pollutants by
   - Participating in education and source control programs for urban users contributing flow to the District lines and ditches
   - Supervising and directing all monitoring efforts, providing personnel and lab facilities for water quality testing of many parameters for which the City is currently set up to test, including parameter such as TSS and BOD. City will maintain records of time and materials to be partially reimbursed based on the cost distribution described above).

- Assisting in financing the City's share of capital improvements, monitoring, and other programs required by regulatory agencies, or as mutually agreed upon.

This agreement should be reviewed every five (5) years and updated to reflect changes in development, additional flow monitoring data, future engineering studies, and changes in regulatory requirements.

Appendix F

Estimate of Cost

City of Ontario
Storm Water Master Plan
Estimate of Most Probable Cost

Priority I Improvements

Downtown Improvement A

Item	Unit	Amount	Unit Cost	Cost	Total	Total with 25% Contingencies
12-inch						
Open Field 5'-10' deep	LF		$23.00	$0		
5'-10' deep w/ asphalt repair	LF	340	$27.00	$9,180		
Open Field 10'-15'	LF		$31.00	$0		
10'-15' deep w/ asphalt repair	LF		$35.00	$0	$9,180	$11,475
15-inch						
Open Field 5'-10' deep	LF		$27.00	$0		
5'-10' deep w/ asphalt repair	LF	587.91	$31.00	$18,225		
Open Field 10'-15'	LF		$35.90	$0		
10'-15' deep w/ asphalt repair	LF		$40.00	$0	$18,225	$22,782
24-inch						
Open Field 5'-10' deep	LF	400	$37.00	$14,800		
5'-10' deep w/ asphalt repair	LF		$41.75	$0		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0	$14,800	$18,500
42-inch						
Open Field 5'-10' deep	LF	2181.39	$69.00	$150,516		
5'-10' deep w/ asphalt repair	LF	6535.26	$79.00	$516,286		
Open Field 10'-15'	LF	2079.88	$87.00	$180,950		
10'-15' deep w/ asphalt repair	LF		$93.00	$0		
Bore	LF	275	$1,000.00	$275,000	$1,122,751	$1,403,439
Priority I: Alternative A Cost					**$1,155,776**	**$1,456,195**

Miscellaneous Pipeline Improvements

Item	Unit	Amount	Unit Cost	Cost	Total	Total with 25% Contingencies
Replace 15-inch with 24-inch by Kmart						
Open Field 5'-10' deep	LF	1006	$37.00	$37,222		
5'-10' deep w/ asphalt repair	LF	399	$41.75	$16,658		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0	$53,880	$67,350
Kendall Pit 18-inch from SE 9th Avenue to SE 5th Avenue						
Open Field 5'-10' deep	LF	1380	$31.00	$42,780		
5'-10' deep w/ asphalt repair	LF		$35.00	$0		
Open Field 10'-15'	LF		$40.75	$0		
10'-15' deep w/ asphalt repair	LF		$44.75	$0	$42,780	$53,475
Correct Reverse Grade on SE 4th Street on 18-inch						
Open Field 5'-10' deep	LF		$31.00	$0		
5'-10' deep w/ asphalt repair	LF	320	$35.00	$11,200		
Open Field 10'-15'	LF		$40.75	$0		
10'-15' deep w/ asphalt repair	LF		$44.75	$0	$11,200	$14,000
Replace 30" CMP Collapsing Pipe Section at Verde Road and NW 4th Avenue						
Open Field 5'-10' deep	LF		$49.00	$0		
5'-10' deep w/ asphalt repair	LF	45	$54.00	$2,430		
Open Field 10'-15'	LF		$61.50	$0		
10'-15' deep w/ asphalt repair	LF		$66.50	$0	$2,430	$3,038
Repair Sanitary Sewer Pipe Crossing on 24-inch on NW 2nd Avenue between NW 3rd and 4th Streets						
Pipe, connections, asphalt repair, etc.	EA	1	$3,200.00	$3,200	$3,200	$4,000
Replace Section of 18-inch concrete pipe on SE 11th Ave.						
Open Field 5'-10' deep	LF		$31.00	$0		
5'-10' deep w/ asphalt repair	LF	100	$35.00	$3,500		
Open Field 10'-15'	LF		$40.75	$0		
10'-15' deep w/ asphalt repair	LF		$44.75	$0	$3,500	$4,375
TVCC Sedimentation Basin/Structure	EA	1	$20,000	$20,000		
Smaller Sedimentation Basins/Structures	EA	2	$8,000.00	$16,000	$36,000	$45,000
Add Cleanout (Near ODOT)	EA	1	$1,600.00	$1,600	$1,600	$2,000
Pennington Drive Improvements	EA	1	$12,000	$12,000	$12,000	$15,000
Priority I: Miscellaneous Pipeline Improvement Cost					**$166,590**	**$208,238**

PRIORITY I TOTAL PROJECT COST **$1,664,433**

City of Ontario
Storm Water Master Plan
Estimate of Most Probable Cost

Priority II Improvements (2010)

Downtown Improvement A

Item	Unit	Amount	Unit Cost	Cost	Total	Total with 25% Contingencies
18-inch						
Open Field 5'-10' deep	LF		$31.00	$0		
5'-10' deep w/ asphalt repair	LF	1019.77	$35.00	$35,692		
Open Field 10'-15'	LF		$40.75	$0		
10'-15' deep w/ asphalt repair	LF		$44.75	$0	$35,692	$44,615
24-inch						
Open Field 5'-10' deep	LF		$37.00	$0		
5'-10' deep w/ asphalt repair	LF	1095.23	$41.75	$45,726		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0	$45,726	$57,157
Priority II: Alternative A Cost					**$81,418**	**$101,772**

Miscellaneous Pipeline Improvements

Item	Unit	Amount	Unit Cost	Cost	Total	Total with 25% Contingencies
Replace 15-inch on NW 4th Avenue between NW 2nd and 3rd Street						
Open Field 5'-10' deep	LF		$27.00	$0		
5'-10' deep w/ asphalt repair	LF	340	$31.00	$10,540		
Open Field 10'-15'	LF		$35.90	$0		
10'-15' deep w/ asphalt repair	LF		$40.00	$0	$10,540	$13,175
Priority II: Miscellaneous Pipeline Improvement Cost					**$10,540**	**$13,175**

PRIORITY II TOTAL PROJECT COST **$114,947**

Priority III Improvements (2015)

Miscellaneous Pipeline Improvements

Item	Unit	Amount	Unit Cost	Cost	Total	Total with 25% Contingencies
Replace 21-inch along SW 11th Avenue north of TVCC ballpark						
Open Field 5'-10' deep	LF		$34.00	$0		
5'-10' deep w/ asphalt repair	LF	460	$38.35	$17,641		
Open Field 10'-15'	LF		$43.12	$0		
10'-15' deep w/ asphalt repair	LF		$47.47	$0	$17,641	$22,051
Replace 24-inch along SW 4th Avenue west of Park Boulevard						
Open Field 5'-10' deep	LF		$37.00	$0		
5'-10' deep w/ asphalt repair	LF	720	$41.75	$30,060		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0	$30,060	$37,575
Replace 8-inch with 12-inch MH F12-16 to F12-17 (south of Sears Dr.)						
Open Field 5'-10' deep	LF	240	$23.00	$5,520		
5'-10' deep w/ asphalt repair	LF		$27.00	$0		
Open Field 10'-15'	LF		$31.00	$0		
10'-15' deep w/ asphalt repair	LF		$35.00	$0	$5,520	$6,900
Replace 12-inch along SW 5th Avenue west of SW 12th Street						
Open Field 5'-10' deep	LF		$23.00	$0		
5'-10' deep w/ asphalt repair	LF	260	$27.00	$7,020		
Open Field 10'-15'	LF		$31.00	$0		
10'-15' deep w/ asphalt repair	LF		$35.00	$0	$7,020	$8,775
Miscellaneous Priority III Total Cost					**$60,241**	**$75,301**

PRIORITY III TOTAL PROJECT COST **$75,301**

TOTAL PROJECT COST **$1,854,682**

CITY OF ONTARIO

Priority Improvement Cost Summary

Downtown Improvement Alternatives		Project Cost
Alternative A: Upsize Irrigation Line and 2 tie-ins from Park	$1,456,195	
South Park Boulevard Pipe Upsizing	$ 101,772	
Total		$ 1,557,968
Alternative B: Detention Basin and 42" Pipeline near TVCC	$ 657,779	
South Park Boulevard Pipe Upsizing	$ 101,772	
24" Replacement of TVCC "Wet" Line	$ 518,158	
Total		$ 1,277,709
Alternative C: Detention Basin at City Park	$ 624,166	
South Park Boulevard Pipe Upsizing	$ 193,153	
24" Replacement of TVCC "Wet" Line	$ 518,158	
Total		$ 1,335,478

Summary of Priority Improvements

Priority 1 Improvements		
Preferred Downtown Improvement Alternative A	$ 1,456,195	
Kmart Improvements	$ 67,350	
Kendall Pit Pipeline from SE 9th Avenue to SE 5th Avenue	$ 53,475	
Correct Reverse Grade on SE 4th Street on 18-inch	$ 14,000	
Replace 30" CMP Collapsing Pipe Section at Verde Rd. and NW 4th Ave.	$ 3,038	
Repair Sanitary Sewer Pipe Crossing on NW 2nd Ave at NW 3rd and 4th St.	$ 4,000	
Replace Section of 18-inch concrete pipe on SE 11th Ave.	$ 4,375	
TVCC Sedimentation Basin/Structure	$ 25,000	
Smaller Sedimentation Basins/Structures	$ 20,000	
Add Cleanout (Near ODOT)	$ 2,000	
Pennington Drive Improvements	$ 15,000	
Total Priority 1		**$ 1,664,433**
Priority II Improvements		
Preferred Downtown Improvement Alternative A	$ 101,772	
Replace 15-inch on NW 4th Avenue between NW 2nd and 3rd Street	$ 13,175	
Total Priority 2		**$ 114,947**
Priority III Improvements		
Replace 21-inch along SW 11th Avenue north of TVCC ballpark	$ 22,051	
Replace 24-inch along SW 4th Avenue west of Park Boulevard	$ 37,575	
Replace 8-inch with 12-inch MH F12-16 to F12-17 (south of Sears Dr.)	$ 6,900	
Replace 12-inch along SW 5th Avenue west of SW 12th Street	$ 8,775	
Total Priority 3		**$ 75,301**

TOTAL PROJECT COST (Alternative A)	**$ 1,854,682**

CITY OF ONTARIO

Annual Operation and Maintenance Costs and Replacement Costs

Assumptions

Clean (6 to 30")	$	0.50	per Foot (every 3 years)
Clean (36"+)	$	0.75	per Foot (every 3 years)
TV	$	0.35	per Foot (every 5 years)
Clean CB	$	50	each CB (every 2 years)
Bill/Clerk 1/4 FTE	$	10,000	per Year
GIS/Maint 1/4 FTE	$	12,000	per Year
Supervisor 1/4 FTE	$	15,000	per Year
Ave. Pipe Life		40	years remaining
Loan Amount	$	1,664,433	Priority I Improvements
Loan Life		20	years (Priority Improvements)
Interest Rate		4.5%	

Replacement costs include engineering and contingency
Field labor cost are included in cleaning cost, TV cost, and replacement cost

Calculations

Pipe Diameter (inch)	Total Length (ft)	Malheur Drainage Pipe Length (ft)	Stewart Carter Pipe (ft)	City Maintained Pipelines		O&M Costs		Replacement Costs	
				Length (ft)	Length (miles)	Clean Cost	TV Cost	Cost/Foot	Annual Cost
6-inch	3,060			3,060	0.58	$ 510	$ 214	$ 35	$ 2,678
8-inch	38,011	4,100		33,911	6.42	$ 5,652	$ 2,661	$ 35	$ 29,672
10-inch	30,885	10,863		20,022	3.79	$ 3,337	$ 2,162	$ 35	$ 17,519
12-inch	38,168	10,272		27,896	5.28	$ 4,649	$ 2,672	$ 35	$ 24,409
14-inch	7,635	4,250		3,385	0.64	$ 564	$ 534	$ 40	$ 3,385
15-inch	15,897	2,144		13,753	2.60	$ 2,292	$ 1,113	$ 40	$ 13,753
16-inch	4,265	3,600		665	0.13	$ 111	$ 299	$ 40	$ 665
18-inch	20,497	5,071		15,426	2.92	$ 2,571	$ 1,435	$ 45	$ 17,354
20-inch	4,215	2,100		2,115	0.40	$ 353	$ 295	$ 45	$ 2,379
21-inch	5,814			5,814	1.10	$ 969	$ 407	$ 50	$ 7,268
24-inch	37,058	14,426		22,632	4.29	$ 3,772	$ 2,594	$ 55	$ 31,119
27-inch	2,229			2,229	0.42	$ 372	$ 156	$ 55	$ 3,065
30-inch	14,613	4,565		10,048	1.90	$ 1,675	$ 1,023	$ 70	$ 17,584
36-inch	10,039			10,039	1.90	$ 2,510	$ 703	$ 80	$ 20,078
42-inch	4,947			4,947	0.94	$ 1,237	$ 346	$ 90	$ 11,131
48-inch	5,460		4572	888	0.17	$ 1,365	$ 382	$ 100	$ 2,220
Unknown	20,240	320		19,920	3.77	$ 5,060	$ 1,417	$ 40	$ 19,920
Catch Basin Pipes	39,878	435		39,443	7.47	$ 9,970	$ 2,791	$ 35	$ 34,513
Catch Basins	1,166					$ 29,150		$ 500	$ 14,575
Manholes	580								
TOTAL	302,911	62,146		236,193	44.7	$ 76,117	$ 21,204		$ 273,286

Summary of Annual Costs

Annual Cleaning Cost	$ 76,117	
Annual TV Cost	$ 21,204	
Labor Costs (office only)*	$ 37,000	
Building, Truck, Fuel, Office Supplies, Utilities, Monitoring/Testing	$ 50,000	
Sub-Total		$ 184,321
Annual Replacement Costs*		$ 273,286
Annual Bond Payment for Priority Improvements		$ 127,955
Miscellaneous Capital Improvements (water quality, software, new pipelines, etc.)		$ 40,000
Estimated Total Annual Cost		**$ 625,562**

*Assumes replacement of all existing pipe less Malheur Drainage District's pipe

City of Ontario
Stormwater Improvements

Alternative B: 1 Tie-In, Parallel 42-inch at TVCC and Detention Basin @ City Park

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
12-inch						
Open Field 5'-10' deep	LF		$23.00	$0		
5'-10' deep w/ asphalt repair	LF	340	$27.00	$9,180		
Open Field 10'-15'	LF		$31.00	$0		
10'-15' deep w/ asphalt repair	LF		$35.00	$0		
Bore	LF		$300.00	$0	$9,180	$11,475
15-inch						
Open Field 5'-10' deep	LF		$27.00	$0		
5'-10' deep w/ asphalt repair	LF	587.91	$31.00	$18,225		
Open Field 10'-15'	LF		$35.90	$0		
10'-15' deep w/ asphalt repair	LF		$40.00	$0		
Bore	LF		$300.00	$0	$18,225	$22,782
18-inch						
Open Field 5'-10' deep	LF	400	$31.00	$12,400		
5'-10' deep w/ asphalt repair	LF		$35.00	$0		
Open Field 10'-15'	LF		$40.75	$0		
10'-15' deep w/ asphalt repair	LF		$44.75	$0		
Bore	LF		$300.00	$0	$12,400	$15,500
24-inch						
Open Field 5'-10' deep	LF		$37.00	$0		
5'-10' deep w/ asphalt repair	LF	305	$41.75	$12,734		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0		
Bore	LF		$450.00	$0	$12,734	$15,917
36-inch						
Open Field 5'-10' deep	LF	336.2	$59.00	$19,836		
5'-10' deep w/ asphalt repair	LF	1130.78	$66.00	$74,631		
Open Field 10'-15'	LF		$73.00	$0		
10'-15' deep w/ asphalt repair	LF		$78.50	$0		
Bore	LF		$600.00	$0	$94,467	$118,084
42-inch						
Open Field 5'-10' deep	LF	1720.19	$69.00	$118,693		
5'-10' deep w/ asphalt repair	LF	1175	$79.00	$92,825		
Open Field 10'-15'	LF		$87.00	$0		
10'-15' deep w/ asphalt repair	LF		$93.00	$0		
Bore	LF		$1,000.00	$0	$211,518	$264,398
Detention Facility						
Earthwork	CY	13060.19	$6.00	$78,361		
Sprinklers	SF	100750	$0.15	$15,113		
Re-seed	SF	100750	$0.30	$30,225		
Tree Removal	EA	25	$600.00	$15,000		
Water line Re-alignment	LF	600	$25.00	$15,000		
Check Valve Vault	EA	1	$8,000.00	$8,000		
Drop and Seal Sewer Manhole	EA	1	$1,000.00	$1,000		
Outfall Structure	EA	1	$5,000	$5,000	$167,699	$209,623

Alternative B Total Construction Cost **$526,223**
25% Contingencies **$131,556**
TOTAL ESTIMATED COST **$657,779**

Priority II: South Park Boulevard (SW 9th Ave South to SW 12th Avenue) Improvements

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
18-inch						
Open Field 5'-10' deep	LF		$31.00	$0		
5'-10' deep w/ asphalt repair	LF	1019.77	$35.00	$35,692		
Open Field 10'-15'	LF		$40.75	$0		
10'-15' deep w/ asphalt repair	LF		$44.75	$0		
Bore	LF		$300.00	$0	$35,692	$44,615
24-inch						
Open Field 5'-10' deep	LF		$37.00	$0		
5'-10' deep w/ asphalt repair	LF	1095.23	$41.75	$45,726		
Open Field 10'-15'	LF		$48.00	$0		
10'-15' deep w/ asphalt repair	LF		$52.75	$0		
Bore	LF		$450.00	$0	$45,726	$57,157

Alternative B Priority II Total Construction Cost **$81,418**
25% Contingencies **$20,354**
TOTAL ESTIMATED COST **$101,772**

Priority III: 24-inch Replacement (SW 4th Ave North to Outfall)

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
24-inch						
Open Field 5'-10' deep	LF	1443.9	$37.00	$53,424		
5'-10' deep w/ asphalt repair	LF	2478.96	$41.75	$103,497		
Open Field 10'-15'	LF	213.895	$48.00	$10,267		
10'-15' deep w/ asphalt repair	LF	2342.915	$52.75	$123,589		
Bore	LF	275	$450.00	$123,750	$414,527	$518,158

Alternative B Priority III Total Construction Cost **$414,527**
25% Contingencies **$103,632**
TOTAL ESTIMATED COST **$518,158**

ESTIMATE OF MOST PROBABLE COST **$1,277,709**

City of Ontario
Stormwater Improvements

Alternative C: Detention Basin @ City Park

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
12-inch						
Open Field 5'-10' deep	LF		$23.00	$0		
5'-10' deep w/ asphalt repai	LF	340	$27.00	$9,180		
Open Field 10'-15	LF		$31.00	$0		
10'-15' deep w/ asphalt repai	LF		$35.00	$0		
Bore	LF		$300.00	$0	$9,180	$11,475
15-inch						
Open Field 5'-10' deep	LF		$27.00	$0		
5'-10' deep w/ asphalt repai	LF	587.91	$31.00	$18,225		
Open Field 10'-15	LF		$35.90	$0		
10'-15' deep w/ asphalt repai	LF		$40.00	$0		
Bore	LF		$300.00	$0	$18,225	$22,782
24-inch						
Open Field 5'-10' deep	LF		$37.00	$0		
5'-10' deep w/ asphalt repai	LF	305	$41.75	$12,734		
Open Field 10'-15	LF		$48.00	$0		
10'-15' deep w/ asphalt repai	LF		$52.75	$0		
Bore	LF		$450.00	$0	$12,734	$15,917
36-inch						
Open Field 5'-10' deep	LF		$59.00	$0		
5'-10' deep w/ asphalt repai	LF	1211.78	$66.00	$79,977		
Open Field 10'-15	LF		$73.00	$0		
10'-15' deep w/ asphalt repai	LF		$78.50	$0		
Bore	LF		$600.00	$0	$79,977	$99,972
42-inch						
Open Field 5'-10' deep	LF	1720.19	$69.00	$118,693		
5'-10' deep w/ asphalt repai	LF	1175	$79.00	$92,825		
Open Field 10'-15	LF		$87.00	$0		
10'-15' deep w/ asphalt repai	LF		$93.00	$0		
Bore	LF		$1,000.00	$0	$211,518	$264,398
Detention Facility						
Earthwork	CY	13060.19	$6.00	$78,361		
Sprinklers	SF	100750	$0.15	$15,113		
Re-seed	SF	100750	$0.30	$30,225		
Tree Remova	EA	25	$600.00	$15,000		
Water line Re-alignmen	LF	600	$25.00	$15,000		
Check Valve Vault	EA	1	$8,000.00	$8,000		
Drop and Seal Sewer Manhole	EA	1	$1,000.00	$1,000		
Outfall Structure	EA	1	$5,000	$5,000	$167,699	$209,623

Alternative C Total Construction Cost **$499,333**
25% Contingencies **$124,833**
TOTAL ESTIMATED COST **$624,166**

Priority II: South Park Boulevard (SW 9th Ave South to SW 12th Avenue) Improvements

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
18-inch						
Open Field 5'-10' deep	LF		$31.00	$0		
5'-10' deep w/ asphalt repai	LF	1019.77	$35.00	$35,692		
Open Field 10'-15	LF		$40.75	$0		
10'-15' deep w/ asphalt repai	LF		$44.75	$0		
Bore	LF		$300.00	$0	$35,692	$44,615
24-inch						
Open Field 5'-10' deep	LF		$37.00	$0		
5'-10' deep w/ asphalt repai	LF	2101.23	$41.75	$87,726		
Open Field 10'-15	LF		$48.00	$0		
10'-15' deep w/ asphalt repai	LF		$52.75	$0		
Bore	LF		$450.00	$0	$87,726	$109,658
30-inch						
Open Field 5'-10' deep	LF		$49.00	$0		
5'-10' deep w/ asphalt repai	LF	576	$54.00	$31,104		
Open Field 10'-15	LF		$61.50	$0		
10'-15' deep w/ asphalt repai	LF		$66.50	$0		
Bore	LF		$450.00	$0	$31,104	$38,880

Alternative C Priority II Total Construction Cost **$154,522**
25% Contingencies **$38,631**
TOTAL ESTIMATED COST **$193,153**

Priority III: 24-inch Replacement (SW 4th Ave North to Outfall)

Item	Unit	Amount	Unit Cost	Cost	Total	Total with Contingencies
24-inch						
Open Field 5'-10' deep	LF	1443.9	$37.00	$53,424		
5'-10' deep w/ asphalt repai	LF	2478.96	$41.75	$103,497		
Open Field 10'-15	LF	213.895	$48.00	$10,267		
10'-15' deep w/ asphalt repai	LF	2342.915	$52.75	$123,589		
Bore	LF	275	$450.00	$123,750	$414,527	$518,158

Alternative C Priority III Total Construction Cost **$414,527**
25% Contingencies **$103,632**
TOTAL ESTIMATED COST **$518,158**

ESTIMATE OF MOST PROBABLE COST **$1,335,478**

City of Ontario
Stormwater Master Plan
Rate Analysis

In 1980 the City of Ontario passed a resolution instituting storm drainage user fees, which have been raised once in 1984 by 5 percent. For residential and industrial areas, the original fees were determined based upon the average amount of impervious area that properties in each of the following categories contained in relationship to the total impervious area within the City. It was assumed that each residential lot/user equaled approximately 2,500 sq feet of impervious area. In order to determine the rates for industrial areas, the actual measured impervious area of the industrial lot was multiplied by the residential rate and divided by 2,500. Unlike the residential and industrial rates, commercial rates were set using a rate per business rather than per area.

In evaluating potential rate impacts a new rate schedule for Ontario similar to the existing rate structure was assumed, basing the rates on area and percent impervious area. The following are the assumptions:

Residential Lot	2,500 square feet of impervious area
Commercial Lot	85% Impervious
Industrial Lot	90% Impervious

The rates for commercial and industrial lots were based on an equivalent residential unit, using the assumed percent impervious and the assumption of 2,500 square feet of impervious area being equivalent to one ERU. This method results in the following rate structure:

Category	Monthly Rate per Unit (Monthly Total/Amount)	Unit	Monthly Total (% Imp x Total Monthly Revenue)
Industrial	$8.80	10,000 sq ft	$20,756
Commercial	$8.31	10,000 sq ft	$20,634
Residential	$2.44	ERU	$10,739
Total			**$52,130**

The following table illustrates the difference in rates from 1984 and the proposed rate structure.

Category	**1984**		**2003**	
Residential	$1.16	Per ERU	$2.44	Per ERU
Commercial	$6.41	Per Business	$8.31	Per 10,000 sq ft
Monthly Bill				
½ acre lot	$6.41		$18.10	
1 acre lot	$6.41		$36.20	
5 acre lot	$6.41		$180.99	
Industrial	$4.18*	Per 10,000 sq ft	$8.80	Per 10,000 sq ft
Monthly Bill				
½ acre lot	$9.10		$19.17	
1 acre lot	$18.21		$38.33	
15 acre lot	$91.04		$191.66	

*Assumed that industrial lots were 90% impervious

The City of Ontario may also choose to adopt rates based upon cost per acre for commercial and industrial or calculate the ERU for each establishment

City of Ontario
Stormwater Master Plan
Rate Analysis

Assumptions

	1984	2003
Residential	Rate per 2500 sq ft impervious area	
Commercial	Rate per Business	85% Impervious
Industrial	Rate Calculated	90% Impervious
Annual Revenue	$112,000	$625,600

Monthly Rates

	1984	2003	
		Option 1 Developed Area	**Option 2** Total Area within City Limits
Residential	$1.16/ERU	$4.68	$2.44
Commercial	$6.41/Business	$15.90	$8.31
Industrial	$4.18/10,000 sq ft	$16.83	$8.80

Notes: 2003 Commercial and Industrial Rates are per 10,000 square feet total area and Residential Rates are per ERU

Option 1: Rates assumed $4.68/2500 sq ft impervious area. No revenue is generated from undeveloped or pervious areas. *Industrial zones and commercial zones were assumed to be 10% and 70% developed, respectively.*

Option 2: Rates determined similar to A, except that *all areas within the City Limits, developed and undeveloped, were assessed.*

Commercial Monthly Rate Comparison

	1984	2003	
		Option 1	**Option 2**
½ Acre Lot	$6.41	$34.63	$18.10
1 Acre Lot	$6.41	$69.26	$36.20
5 Acre Lot	$6.41	$346.30	$180.99

City of Ontario

Stormwater Master Plan

Rate Comparison

	Pop.	Area (sq mi)	Monthly User Fees
ACHD (ID)	312,337	1,055	No Stormwater Utility
Pocatello, ID	51,466	28	Forming a Stormwater Utility
Spokane, WA	423,261	1,764	**Residential:** $0.83 for 3160 sq ft impervious area
Kennewick, WA	54,693	20	No stormwater utility
Baker, OR	9,860	7	No Stormwater Utility
Bend, OR	52,029	14	No Stormwater Utility
Pendleton, OR	16,354	9	No Stormwater Utility
Portland, OR	513,325	130	**Residential:** $11.42 per 2400 sq ft impervious area **Commercial:** $51.70 per 10,000 sq feet
Fort Collins, CO	126,848	49.43	**Residential:** $9.42 per 2500 sq ft impervious area monthly **Commercial:** $30.15 per 10,000 sq ft monthly Monthly Fees = (Lot area+Open Space) x Base Rate x Rate Factor Base Rate=$0.0037685

Appendix G

Storm Water Management Design Manual

City of Ontario

Storm Water Management Design Manual

Draft

February 2003

Chapter 1.0 - Introduction

1.1 Purpose and Applicability

Stormwater management is important for both flood control and protection of groundwater and surface water quality. Development, with its rooftops and paved surfaces, increases the rate and amount of runoff, and also the level of pollutants that are discharged to groundwater or to the Malheur and Snake Rivers. Pollution reduction is vital to protecting the area's waters for beneficial uses, including fish and wildlife habitat, recreation, and drinking water.

The purpose of this manual is to define standards for the design, construction and maintenance of drainage facilities in Ontario that will prevent flooding from stormwater runoff, and protect groundwater and surface water quality. The manual provides requirements for reducing the impacts of stormwater runoff and pollution from new development and redevelopment.

The standards in this manual apply to new development and redevelopment, public or private. All projects that create over 500 square feet of impervious area are to comply with the requirements of this manual, as are projects of any size that are classified as high risk (as discussed in Section 1.3). Required submittals are outlined in Chapter 2.0, design criteria are defined in Chapter 3.0, and Chapter 4.0 presents information on practices and facilities for stormwater management.

1.2 Authority and Policies

The City of Ontario, as a municipal corporation, has constitutional authority to promulgate stormwater discharge regulations to minimize flooding, protect groundwater resources and prevent surface water pollution. *Refer to ordinance?*

Under regulations mandated by the Clean Water Act of 1987, Ontario may be required to obtain a National Pollutant Discharge Elimination System (NPDES) Storm Water Discharge Permit to control urban stormwater pollution. The permit obligates Ontario to develop, implement, enforce and measure the effectiveness of, a storm water management program designed to reduce the discharge of pollutants and protect water quality.

The waters in the Upstream Snake River are listed as water quality limited due to concerns over dissolved oxygen, mercury, nutrients, sediment and temperature. Stormwater from Ontario discharges to this segment of the Snake River. Reduction of sediment load has been identified as a first step in achieving water quality goals. Additional pollutant removal requirements are under consideration.

1.3 Stormwater Management Criteria

Pollution reduction is required in the City of Ontario for all new development creating over 500 square feet of impervious surface (or redevelopment footprint over 500 square feet). Flow control measures are also required to limit peak flows to the capacity of the conveyance system. Design storms for water quality (pollution reduction) and water quantity (flow control) are defined in Chapter 3.0.

A development may discharge to an existing publicly-owned stormwater system provided the following conditions are met:

- Private on-site flow control facilities are used to the maximum extent practicable
- On-site systems provide required pollution reduction
- The system to which the development is discharging has adequate carrying capacity

Long-term stormwater management requirements are based on the land use and the amount of impervious area resulting from the project. (Impervious area includes rooftops plus asphalt and concrete surfaces such as parking lots, driveways, roads and sidewalks.) Projects involving construction activities that disturb one acre or more have separate stormwater management requirements for the construction phase.

Mitigation measures are encouraged for all projects to reduce stormwater management needs. Mitigation may decrease the volume of stormwater for disposal, reduce peak flows, increase evapotranspiration, reduce pollutants, or facilitate infiltration. Mitigation measures may include limiting the extent of impervious and disturbed areas, stabilizing pervious areas to prevent erosion, and landscaping for flow and pollution control.

Stormwater management criteria, applicability and requirements are summarized in the table below.

Management Category and Criteria	**Stormwater Management Requirements**
New impervious area equal to or greater than 500 square feet, or Redevelopment footprint equal to or greater than 500 square feet	▪ Pollution reduction and flow control measures required ▪ Simplified Approach *(Section 3.3)* may be used (combined flow/pollution control)
Higher risk uses* (see below)	▪ Pollution reduction and flow control measures required ▪ Performance Approach *(Section 3.4)* - separate flow and pollution controls ▪ NPDES permit required for point source discharge from EPA-listed industries *(see Section 2.7)*
Stormwater disposed of by infiltration	▪ Satisfy UIC program requirements *(see Section 2.7)*
Construction activities disturb land area equal to or greater than 1 acre	▪ NPDES permit from Oregon DEQ *(see Section 2.7)* ▪ Erosion and sediment control BMPs

*Higher risk uses are those land use categories that may generate higher pollutant concentrations. These include industrial and commercial land uses such as fuel dispensing facilities; vehicle/equipment service and cleaning; vehicle parking and storage; aboveground storage of liquids (chemicals, oils, solvents); exterior storage of erodible bulk materials (e.g. landscape, sand); material transfer areas/loading docks; and solid waste storage areas.

1.4 Acknowledgements

Several concepts utilized in developing this plan were based on previous work by other cities, particularly Portland and Boise. Their contribution is gratefully acknowledged.
The "Catalog of Stormwater BMPs for Idaho Cities and Counties" has been adopted as a community standard for Ontario, and is incorporated in this manual by reference.

Chapter 2.0 - General Stormwater Requirements

2.1 Introduction

Stormwater management should be an integral part of the development process from the concept stage through design, construction, and occupancy. All developments must submit a storm water management plan that includes a site evaluation, proposed development information, a drainage analysis and a plan for facility operation and maintenance.

2.2 Site Evaluation

Site constraints may limit stormwater management options, which in turn can affect potential land uses and location of improvements. Therefore, evaluation of site suitability should be completed before preparing development concepts or plans. Site conditions that should be evaluated include:

- Size of drainage area
- Possible land uses and related contaminant types
- Site slope and geometry
- Proximity to drinking water supply or surface water
- Past uses as related to soil and groundwater contamination
- Soil types and permeability below drainage facilities
- Subsurface conditions: depth to bedrock, high groundwater

2.3 Proposed Development Information

Development information should include pre-development and post-development site conditions (including change in impervious area), and a determination of the applicable stormwater management criteria.

A scaled site map that provides the following general information is required.

- Vicinity map
- Proposed layout showing buildings, impervious areas, landscaping, utilities, etc.
- Topographic information: existing & proposed site contours, elevations of building(s), grades of impervious surfaces
- Easements (if applicable)
- Landscape plan (if applicable)
- Mitigation measures, as applicable
- Plan of stormwater facilities, with system dimensions

2.4 Drainage Analysis

Two options - the simplified approach and the performance approach - are available for designing stormwater management facilities. These approaches are discussed in Chapter 3.0. The simplified approach utilizes sizing factors for combined pollution reduction and flow control facilities. Facilities designed in accordance with the simplified approach are presumed to comply with the City's pollution reduction and flow control requirements. Detailed hydrologic calculations are not required for these facilities.

Assumptions used in determining the sizing factors for the simplified approach may result in conservative sizing for some developments. Manual users have the option of following the performance approach to submit engineering calculations for alternative facility sizing. The performance approach is also applicable to other types of facilities that are not included in the simplified approach.

If the performance approach is used, submittals should include hydrologic calculations for peak flow rate and runoff volume (pre- and post-development), and the basis for design (including method used, equations, references, graphs, etc.).

2.5 Operation & Maintenance Plan

An Operation and Maintenance (O&M) Plan shall include the following:

- Stormwater system owner(s)
- Party responsible for long-term operation and maintenance of system
- Emergency contacts and response procedures
- Source controls (see Chapter 4.0)
- Schedule for stormwater system inspection and maintenance activities
- Specific maintenance techniques for system components
- Documentation requirements for inspection and maintenance (records of noted conditions and corrective actions to be maintained for at least 5 years)

2.6 Review and Approval Process

A pre-application or predesign conference is recommended for large developments.

Submittals for all developments shall include a site evaluation, proposed development information, drainage calculations (simplified or performance approach) and an operation and maintenance plan, as discussed above.

The City of Ontario will review all submittals for compliance with the requirements of this manual. Approval does not relieve the applicant from responsibility for ensuring facility suitability, performance and safety, nor does it constitute a guarantee of system performance.

2.7 State and Federal Requirements

As noted in Chapter 1.0, certain stormwater discharges require permitting through DEQ. These include:

- Point source stormwater discharges from EPA-listed industries
- Construction activities that disturb one or more acres
- Stormwater disposal by infiltration

Industrial and construction activities are addressed by NPDES storm water regulations. Stormwater disposal by infiltration systems is covered under the Underground Injection Control (UIC) program. Information on these requirements can be accessed at the Oregon DEQ Water Quality Program website at www.deq.state.or.us/wq. (On the Water Quality Program page "Features" menu, select "Permits" for NPDES information, or "UIC" for infiltration disposal system requirements.) Information is also available from the Pendleton office of DEQ (800-452-4011) for NPDES, and from the Portland office (503-229-5495).

Chapter 3.0 - Design Criteria

3.1 Pollution Reduction

Stormwater BMPs shall be designed for removal of total suspended solids (TSS), under post-development conditions, at the rates shown below:

% Parcel Area that is Impervious	% TSS removal required
30 or less	40
35	47
40	53
45	59
50	62
55	66
60	68
65	70
70	72
75	74
80	75
85	77
90	78
95	79
100	80

Removals shall apply to runoff generated from a water quality design storm, defined as 0.26" per hour with a duration of 1 hour.

3.2 Flow Control

Flow controls are intended to maintain post-development peak flows at pre-development levels for most storm events, to keep flows within the capacity of the conveyance system. On-site flow control shall be designed such that the runoff flow rate from a proposed land development shall not exceed the rate of runoff prior to the development, for 5-, 10- and 25-year storms.

Conveyance systems shall be designed to carry the peak flow from the 25-year storm without flooding. Peak volumes for detention and retention facilities shall be based on a 24-hour, 25-year storm event generating 1" total rainfall.

3.3 Simplified Approach

The simplified approach, which utilizes sizing factors for some types of facilities, provides a rapid method to select and design combined pollution reduction and flow control facilities. Facilities designed in accordance with the simplified approach are presumed to comply with the City's pollution reduction and flow control requirements. Detailed hydrologic calculations are not required.

Since each simplified approach facility can handle flow from a limited amount of impervious surface, larger projects must divide impervious surfaces into smaller management areas.

Sizing factors for simplified approach facilities are presented in the table below. Descriptions and general specifications for these BMPs are included in Chapter 4.0. Note that these BMPs can also be used for larger areas than noted, if sized according to the performance approach.

BMP	Simplified Approach Sizing Factor for BMP Area	Max. Impervious Area for Simplified Approach
Bioretention Swale	0.06 x impervious area	15,000 sf
Vegetative Filter Strip	0.13 x impervious area	1,000 sf per strip
Bioretention Basin	0.06 x impervious area	15,000 sf

3.4 Performance Approach

Facilities designed using the performance approach shall provide pollution reduction for the water quality storm and flow control for the water quantity storms, as specified above.

Hydrologic calculations shall provide peak flow rate and runoff volume (for pre- and post-development conditions), and the basis for design (including method used, equations, references, graphs, etc.). The Rational Method may be used for the water quality storm for any size development, but is limited to areas less than 10 acres for water quantity storms. If the Rational Method is used, peak flow rates for water quantity storms shall be based on time of concentration (t_c) and associated intensity for the various storms (t_c no less than 10 minutes). Refer to Intensity-Duration-Frequency curves.

Facilities designed in accordance with the criteria in the "Catalog of Stormwater BMPs for Idaho Cities and Counties" are presumed to provide the pollutant removal efficiencies noted in the Catalog.

Chapter 4.0 - Best Management Practices (BMPs)

4.1 Introduction

Best Management Practices (BMPs) are physical, structural, and/or managerial practices that prevent or reduce water pollution. The two types of BMPs are source control (pollution prevention), and treatment. Most treatment BMPs involve structural measures (e.g. detention ponds and oil/water separators). Source control typically involves non-structural measures, and is generally more cost-effective than treatment for preventing stormwater impacts.

Stormwater management plans should incorporate all practical source control measures before considering treatment facilities.

Source control and treatment BMPs are addressed in detail in the "Catalog of Stormwater BMPs for Idaho Cities and Counties" (hereinafter referred to as the Catalog). The Catalog has been adopted as a community standard for Ontario, and is incorporated in this manual by reference. The Catalog is available on the Idaho DEQ website at www.deq.state.id.us (click on Water Quality, then Catalog of Stormwater BMPs under Stormwater heading).

4.2 Source Control BMPs for Pollution Reduction

Source control practices involve erosion control, spill prevention and other housekeeping practices intended to prevent pollutants from entering the drainage system. Public education, activity schedules, prohibition of practices, maintenance procedures, operating methods, and practices to control runoff, spillage or leaks are possible source controls. Source controls apply to construction, operation and maintenance practices.

Construction projects that disturb one acre or more must have an erosion and sediment control plan approved by DEQ prior to any on-site activities. Best Management Practices applicable to construction can be found in the Catalog. The construction/temporary BMP section of the Catalog covers general site guidelines, housekeeping, slope protection, storm drain and channel protection, sediment collection and runoff diversion. The housekeeping BMPs may also be used as long-term source control measures.

Other source control BMPs include street sweeping, and inspection/cleaning of the stormwater system. The following are recommended frequencies for various maintenance activities:

- Street sweeping: Establish frequency to limit sediment/debris accumulations to 1 cu. ft. per 1000 sq. ft.
- Inspection of stormwater system: Establish frequency based on accumulated sediment, but at least every 5 years for conveyance system; semi-annually and after storm events for other facilities
- Cleaning of stormwater system
  - Conveyance system: when accumulated sediment/trash at 20% of pipe diameter, or if inhibiting facility operation
  - Catch basins: when accumulated sediment/trash blocking 1/3 of pipe diameter
  - Vegetated systems (swales or filter strips): when accumulated sediment exceeds 2" in depth

- Sand filter: when accumulated sediment exceeds ½" in depth
- Oil/water separator: when accumulated sediment exceeds 1' in depth; oil accumulation exceeds 1"

4.3 Structural BMPs for Combined Pollution Reduction/Flow Control

Several BMPs provide both pollution reduction and flow control, and are readily integrated into the site landscaping. These BMPs are used in the simplified approach (sizing criteria are presented in Section 3.3), and can also be sized using the performance approach.

4.3.1 Bioretention Swale

Description: Bioretention swales are long narrow vegetated facilities that provide conveyance as well as treatment and infiltration. Flow through the grass slows the water and facilitates sedimentation and infiltration.

Design Criteria: (Acceptable for sandy loam or soils with loam texture, with infiltration rates 0.5-3.0 in/hr.) Maximum swale slope is 5%; minimum is 1%. All swales will require additional means for disposal. Required setback from property line 2 feet. See Figure 4.1.

4.3.2 Vegetated Filter Strip

Description: Vegetated filter strips are gently sloping areas used to filter, slow and infiltrate stormwater flows that enter the strip as sheet flow. The key to successful vegetated filter strip performance is avoiding concentrated flows. Utilization of site landscape areas as vegetated filters is encouraged.

Design Criteria: (Acceptable for all soil types.) Minimum slope is 1%; maximum is 10%. If slopes exceed 5%, provide check dams (e.g. BMP #22) at 5' intervals. All filter strips will require additional means for disposal. See Figure 4.2.

4.3.3 Bioretention Basin

Description: Bioretention basins are shallow vegetated depressions that provide a temporary ponding area for infiltration and evapotranspiration. Pollutants are removed by settling as the water infiltrates.

Design Criteria: (Acceptable for soils with loam texture, with infiltration rates 0.5-3.0 in/hr.) An overflow to an approved conveyance/disposal facility is required. Required setback from property line is 5 feet. See Figure 4.3.

A dense growth of vegetation is important for proper functioning of all these systems. Fine, close-growing, water-resistant grasses are recommended (suggested species are listed in the Catalog under BMP #38a). Maintenance guidelines are also found in the Catalog.

4.4 Other BMPs

Other BMPs approved by the City may be used separately or in combination for pollution reduction, flow control, and stormwater disposal.

The simplified approach stormwater management facilities address pollution reduction and flow control, but do not necessarily dispose of stormwater adequately in all cases. Disposal facilities may include on-site infiltration facilities, ditches, canals and off-site storm sewers. Infiltration facilities are classified as "injection systems" under the federal Underground Injection Control (UIC) Program. These systems must be either Rule Authorized or permitted by DEQ. Treatment of stormwater prior to disposal in infiltration facilities is required.

The following list summarizes the BMPs that are detailed in the Catalog of Stormwater BMPs for Idaho Cities and Counties. A selection matrix included as part of the Stormwater BMP Selection Suitability Decision Tree may be used to evaluate physical constraints, as well as suitability for flow control and removal of various pollutants.

A. Construction/Temporary

1. General Construction Site Guidelines
   - BMP #1 Timing of Construction
   - BMP #2 Staging Areas
   - BMP #3 Preservation of Existing Vegetation
   - BMP #4 Clearing Limits
   - BMP #5a Stabilization of Construction Entrance and Roads
   - BMP #5b Erosion Prevention on Temporary and Private Roads

2. Housekeeping
   - BMP #6 Dust Control
   - BMP #7 Cover for Materials & Equipment
   - BMP #8 Spill Prevention & Control
   - BMP #9 Vehicle/Equipment Washing & Maintenance
   - BMP #10 Waste Management

3. Slope Protection
   - BMP #11 Mulching
   - BMP #12 Hydromulching
   - BMP #13 Geotextile
   - BMP #14 Matting
   - BMP #15 Pipe Slope Drain
   - BMP #16 Slope Roughening
   - BMP #17 Gradient Terracing
   - BMP #18 Retaining Walls

4. Storm Drain and Channel Protection
   - BMP #19 Gabions
   - BMP #20 Riprap Slope and Outlet Protection
   - BMP #21 Inlet Protection
   - BMP #22 Check Dams
   - BMP #23 Temporary Stream Crossing

5. Sediment Collection and Runoff Diversion
   - BMP #24 Straw Bales/Biofilter Bags
   - BMP #25 Silt Fence
   - BMP #26 Vegetative Buffer Strip

BMP #27 Sedimentation Trap (Basin)
BMP #28 Portable Sediment Tank
BMP #29 Temporary Swale
BMP #30 Earth Dike
BMP #31 Perimeter Dike/Swale
BMP #32 Temporary Berms (Sandbags)
BMP #33 Temporary Storm Drain Diversion

B. Post-Construction/Permanent

1. Slope Protection & Stabilization
BMP #34 Topsoiling
BMP #35 Seeding
BMP #36 Sodding
BMP #37 Planting

2. Stormwater Filters
BMP #38a Biofiltration Swale (Vegetated Swale)
BMP #38b Bioinfiltration Swale (Bioretention Swale)
BMP #39 Vegetative Filter Strip
BMP #40 Sand Filter
BMP #41 Compost Stormwater Filter
BMP #42 Catchbasin Inserts

3. Infiltration Facilities
BMP #43 Infiltration Trench
BMP #44 Bioretention Basin

4. Detention Facilities
BMP #45 Wet Pond (Conventional Pollutants)
BMP #46 Wet Pond (Nutrients)
BMP #47 Wet Extended Detention Pond
BMP #48 Dry Extended Detention Pond
BMP #49 Biodetention Basin
BMP #50 Presettling/Sedimentation Basin
BMP #51 Wet Vault/Tank

5. Other Structural Controls
BMP #52 Oil/Water Separator
BMP #53 Level Spreader

APPENDIX

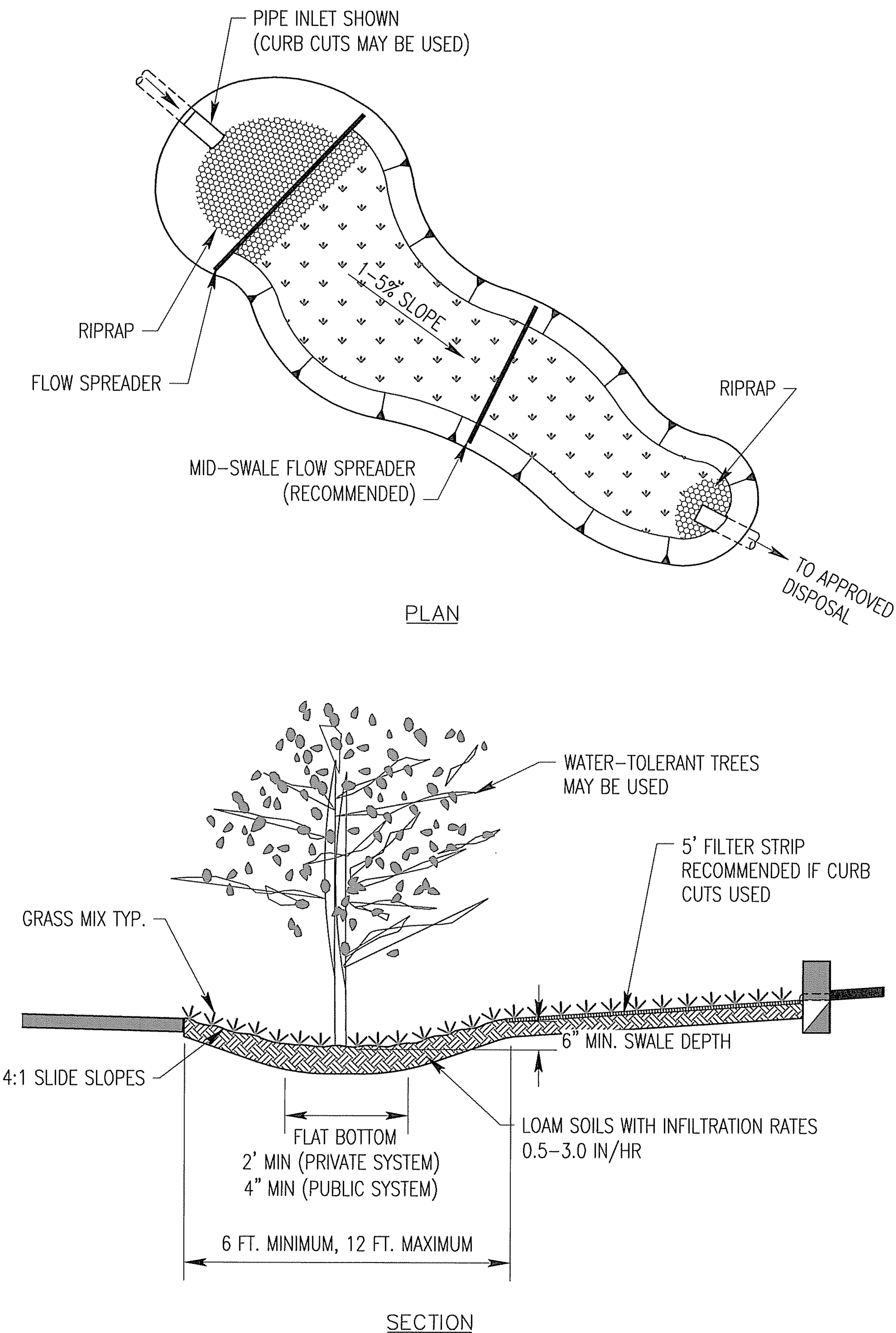


FIGURE 4.1
BIORETENTION SWALE

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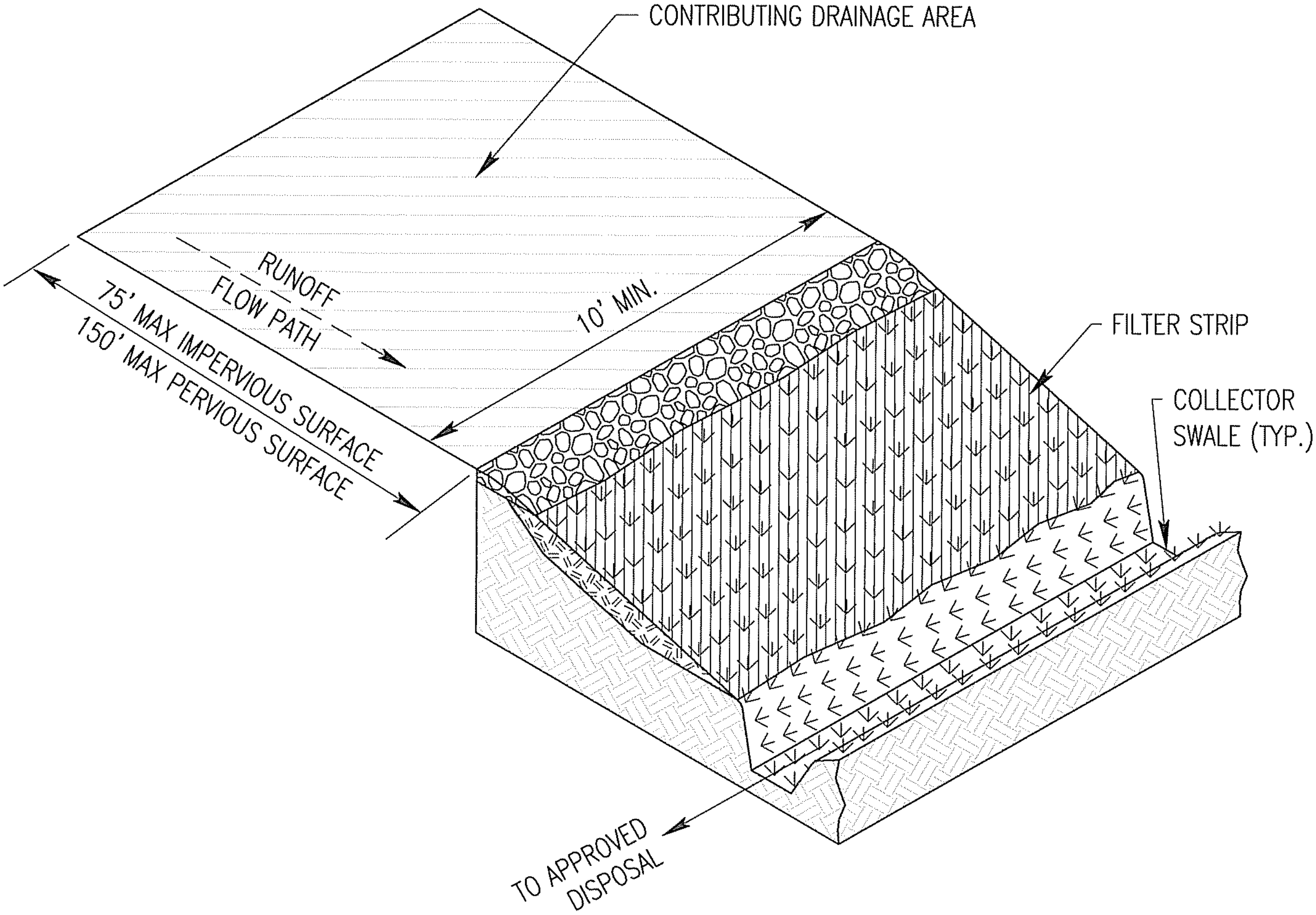

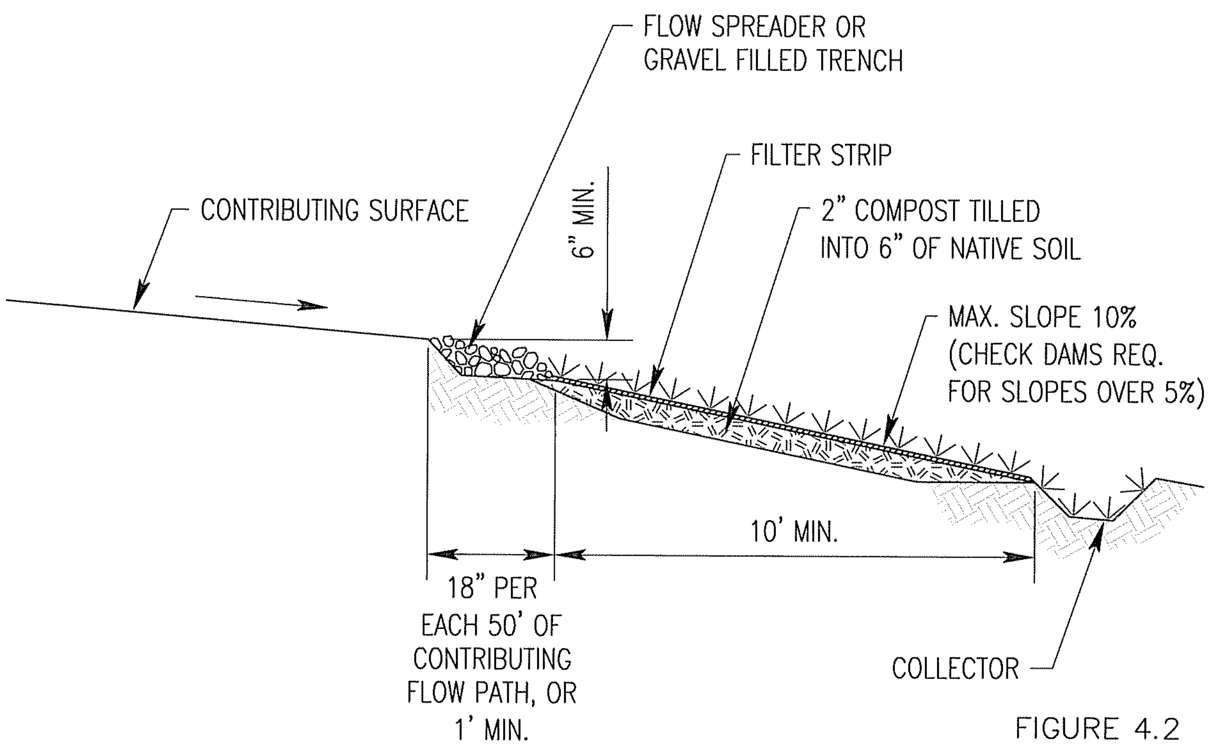

FIGURE 4.2
VEGETATED FILTER STRIP

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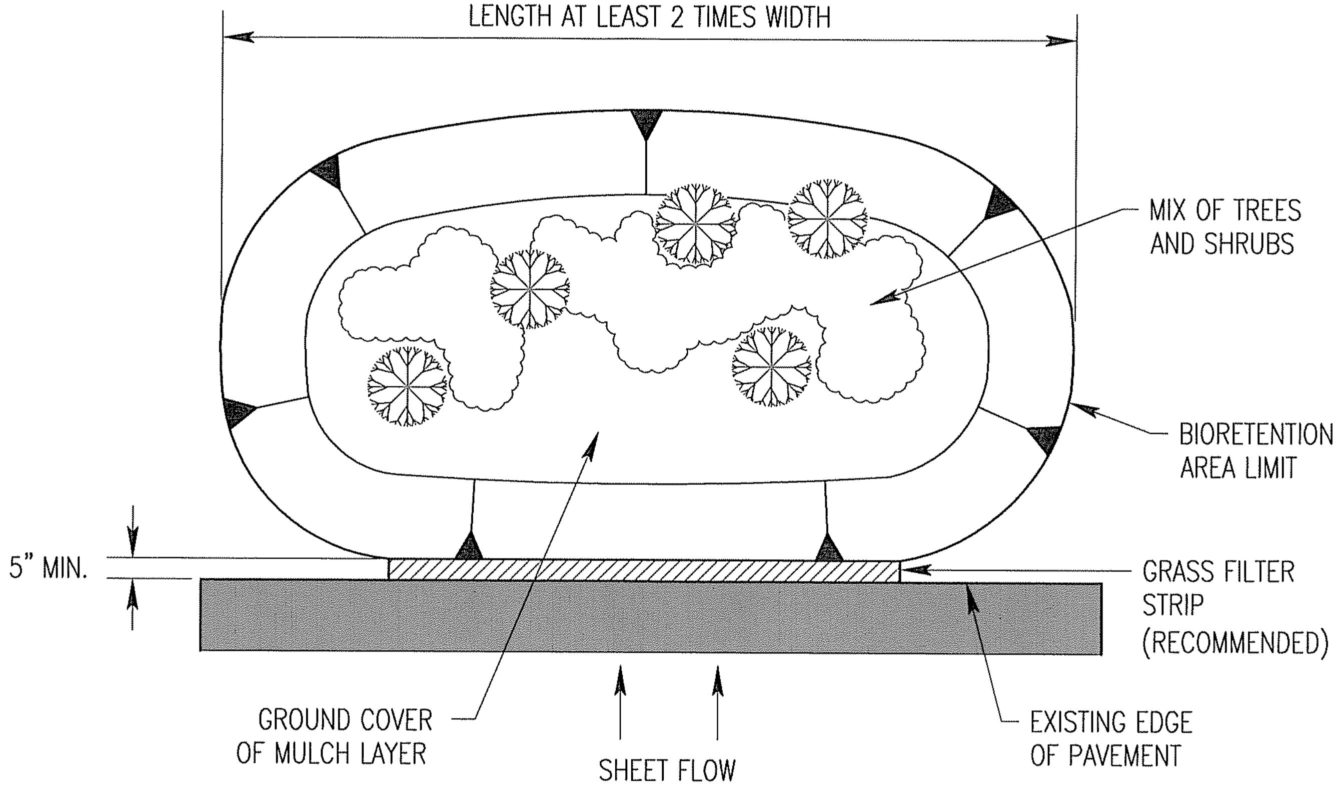

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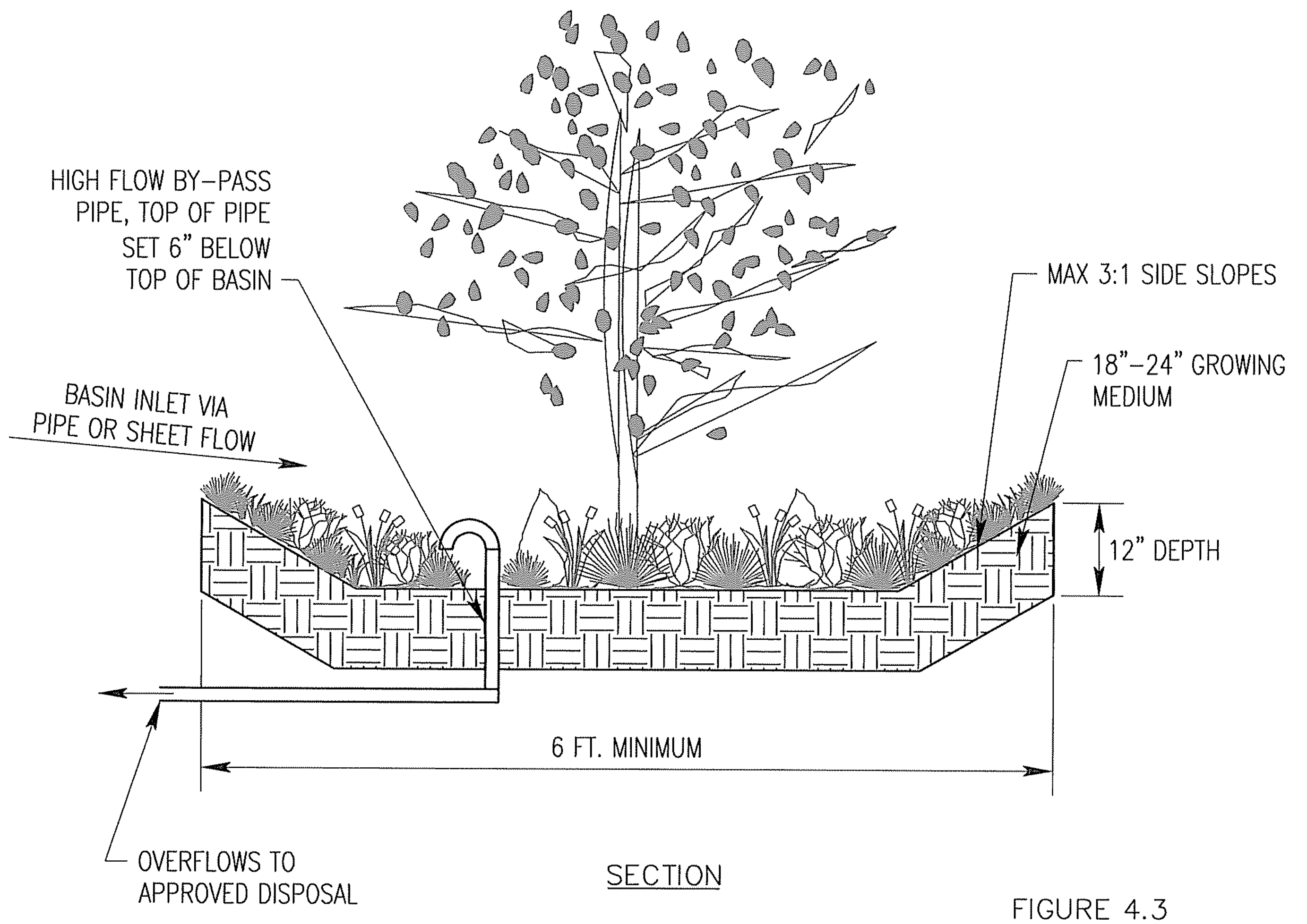

SECTION

FIGURE 4.3
BIORETENTION BASIN

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4) Assumes that 100% of the zoned land is assessed a storm water fee
5) Assumes that only 70% of the commercial zoned land and 10% of the industrial zoned land is developed and assessed a storm water fee at the same rate as if all, developed and undeveloped, zoned land was assessed

In light of the fact that the storm water user rates could significantly increase, it may be appropriate to complete a separate, more detailed storm water fee user rate analysis. A new rate structure could be developed that would provide a gradual increase in storm water fees to accomplish only the most urgent and needed storm water improvements with perhaps other City general fund revenues.