Storm Drainage Master Plan

Volume II

City of Elk Grove

June 2011

Eco-Friendly

Storm Drainage Master Plan
Volume II

Prepared for

City of Elk Grove

June 2011

WEST YOST
ASSOCIATES
Consulting Engineers

In Association with

DAVID FORD
CONSULTING
ENGINEERS

REGISTERED PROFESSIONAL ENGINEER
MARK O.
KUBIK
No. 50963
Exp. 9-30-13
CIVIL
STATE OF CALIFORNIA

296-00-05-01

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CHAPTER 1. INTRODUCTION

INTRODUCTION

Contained within this document are the technical components that support the City of Elk Grove Storm Drainage Master Plan (SDMP). Included in this Volume II of the SDMP is a description of the planning criteria used to evaluate the drainage systems; evaluation of the performance level of the existing drainage conveyance and flood control facilities; identification of performance deficiencies; identification of potential impacts of future development on existing major facilities; and identification of existing and new facilities upgrades to serve buildout conditions of the City's General Plan.

BACKGROUND

The City is located in Sacramento County, just south of the City of Sacramento (see Figure 1-1). The City encompasses an area of over 26,000 acres. Storm drainage within the City is conveyed through a storm drainage system consisting of about 400 miles of underground pipes and 60 miles of natural and constructed channels. The City incorporated on July 1, 2000, and took over ownership and the responsibility for the operation and maintenance of the drainage system from Sacramento County on July 1, 2003. Sacramento County continues to collect development impact fees through their Zone 11A fee program to provide funding for major drainage facilities.

The City has experienced rapid urban growth since incorporating and additional growth is planned in the future. The major future development areas in the City are shown on Figure 1-2. The SDMP addresses the drainage need to accommodate the future development in these areas as well as improvements needed to address deficiencies in the existing drainage system.

ORGANIZATION OF STORM DRAINAGE MASTER PLAN

To simplify the use of the SDMP, it is organized into two volumes and a Program Environmental Impact Report (EIR) as follows:

Volume I describes the development of this SDMP; guiding principles; regulatory framework; background and key concepts; existing and proposed program activities; candidate watershed projects; partnerships, funding, and implementation of the SDMP. Volume II provides the technical analysis for the SDMP.

The EIR evaluates the broad environmental effects of future improvements and new development to comply with California Environmental Quality Act (CEQA) Guidelines Section 15168 of Title 14 of the California Code of Regulations addressing Program EIRs.

ORGANIZATION OF STORM DRAINAGE MASTER PLAN (SDMP) VOLUME II

The City drains within thirteen watersheds and Volume II of the SDMP technical analysis is organized by these thirteen watersheds as shown in Figure 1-3 and as follows:

- Laguna Creek Watershed;
- Elk Grove Creek Watershed;
- Whitehouse Creek Watershed;
- Strawberry Creek Watershed;
- Laguna West Channel Watershed;
- Laguna West Lakes Watershed;
- Lakeside Watershed;
- Laguna Stonelake Watershed;
- Shed A Watershed;
- Shed B Watershed;
- Shed C Watershed;
- Grant Line Channel Watershed; and
- Deer Creek Watershed.

In addition, there is a separate Chapter for the East Elk Grove area/rural region which includes the watersheds of Laguna Creek, Deer Creek and Elk Grove Creek. The East Elk Grove area/rural region have unique characteristics and were separated from the other watersheds to recognize the distinct nature of this region.

OVERVIEW OF MAJOR WATERSHEDS

Within the watersheds there are ten major natural creeks or open channels that convey runoff within the City. These creeks are as follows:

- Elk Grove Creek;
- Laguna Creek;
- Strawberry Creek;
- Whitehouse Creek
- Deer Creek;
- Shed A Channel;
- Shed B Channel;
- Shed C Channel;
- Grant Line Channel; and
- Laguna West Channel.

Four of the creeks convey runoff that originates outside the City limits: Elk Grove Creek, Laguna Creek, Strawberry Creek, and Deer Creek. All of the watersheds and channels located within the City, ultimately drain into the Stone Lakes National Wildlife Refuge floodplain with the exception of Deer Creek and Grant Line Channel watersheds, which drain to Deer Creek and ultimately to the Cosumnes and Mokelumne Rivers.

DEFINITIONS

There are a number of acronyms used in this report and they are defined in Table 1-1 below.

Table 1-1. Definitions

Acronym	Definition
AC	Acre
CBC	Concrete Box Culvert
CCI	Construction Cost Index
CIP	Capital Improvement Plan
CY	Cubic Yard
DET	Detention
EGC	Elk Grove Creek
ENR	Engineering News Record
LF	Lineal Feet
LID	Low Impact Development
LRSP	Laguna Ridge Specific Plan
LS	Lump Sum
LT1	Laguna Creek Tributary No. 1
LT2	Laguna Creek Tributary No. 2
LT3	Laguna Creek Tributary No. 3
LT4	Laguna Creek Tributary No. 4
RCB	Reinforced Concrete Box
RCP	Reinforced Concrete Pipe
SDMP	Storm Drainage Master Plan
SPA	Specific Plan Area
TW	Top Width
UPRR	Union Pacific Railroad
West Yost	West Yost Associates

REFERENCES

There are a large number of prior technical studies and documents that were reviewed and used as appropriate during the preparation of the SDMP. These studies and documents are organized by watershed and listed below. Other specific references are listed within the appropriate chapter.

Laguna Creek and Tributaries (Elk Grove Creek and Whitehouse Creek)

1. Drainage Study for Elk Grove Creek, MacKay & Somps, August 28, 2006.
2. Drainage Study for Vintara Park, MacKay & Somps, December 5, 2005.
3. East Area Storm Drainage Master Plan, Revised Draft Version, Harris & Associates, November 18, 2005.
4. Sacramento County Laguna Creek LOMR Hydrologic Data, July 2005.
5. Laguna Creek Feasibility Study Final Report, Quincy engineering, Inc., June 13, 2005.
6. Laguna Creek Hydrologic and Hydraulic Analysis, David Ford Consulting Engineers, March 2005.
7. Technical Memorandum, Drainage Analysis for Fieldstone Unit 3 and Waterman Ranch Detention Basin within East Elk Grove Specific Plan, Watermark Engineering, Inc., February 10, 2006.
8. LOMR for Lower Laguna Creek Bypass, Borcalli & Associates, May 26, 1999 (Including Technical Support Documentation Notebook, December 21, 1999).
9. Vineyard Springs Comprehensive Plan, Drainage Master Plan – Final, The Spink Corporation, March 10, 1998.
10. Upper Laguna Creek Drainage Master Plan, Status Report, Sacramento County Water Resources Division, September 1997.
11. Lower Laguna Creek Drainage Master Plan, Sacramento County Water Resources Division, May 1996.
12. Lower Laguna Creek Drainage Master Plan, Appendix A, Hydrology Technical Report, Sacramento County Water Resources Division, October 1995.
13. Lower Laguna Creek Drainage Master Plan, Appendix B, Hydraulics Technical Report, Sacramento County Water Resources Division, October 1995.
14. East Elk Grove Specific Plan, Preliminary Technical Studies Report, MacKay & Somps, March 1994.

Grant Line Channel

15. Elk Grove Regional Park and Emerald Lakes Golf Course Storage Capacities, Letter from Psomas to City of Elk Grove, June 2005.
16. Grantline Channel and Pump Station D-39 Hydrologic and Hydraulic Analysis, PSOMAS, March 2005.

Laguna Stonelake

17. Laguna Stone Lake Detention Basin & Pump Station Report, 100% Submittal, Wood Rodgers, March 2000.
18. Public Facilities Report, Laguna Stonelake, Wood Rodgers, June 1999.
19. Elliot Ranch South Drainage Study, Wood Rodgers, March 1999.

Laguna West Lakes

20. Design Report, Laguna Creek Unit No. 4 Hydrology Study, The Spink Corporation, July 1990.
21. Sacramento County Water Agency's Guide to Operation and Maintenance of the Laguna West Levees and Pump Station, The Spink Corporation, Undated.

Lakeside

22. Design Report, Lakeside Development Hydrology Study, The Spink Corporation, July 1991.
23. Pump Calculations and Sump Design for the Lakeside Storm Drainage Pump Station, The Spink Corporation, July 1992.

Sheds A & B

24. Laguna Ridge Specific Plan Supplemental Drainage Plan for Local Drainage Shed B, Wood Rodgers, May 2005.
25. Laguna Ridge Specific Plan Storm Drainage CIP, Wood Rodgers, February 2005.
26. East Franklin Interim Drainage Facility Analysis, Wood Rodgers, August 20, 2003.
27. Sacramento Method 100-Year Drainage Shed Map for Laguna Ridge Major Roads, Wood Rodgers, Inc., May 2005.
28. Drainage Shed Map for Laguna Ridge – The Grove North, Wood Rodgers, Inc., September 2005.
29. Drainage Shed Map for Laguna Ridge – The Grove South, Wood Rodgers, Inc., September 2005.
30. Preliminary Drain Study for Elk Grove Auto Mall Phase III, Wood Rodgers, Inc., January 2006.
31. Drainage Shed Map for Del Webb @ Laguna Ridge, Wood Rodgers, Inc., December 2005.

Shed C

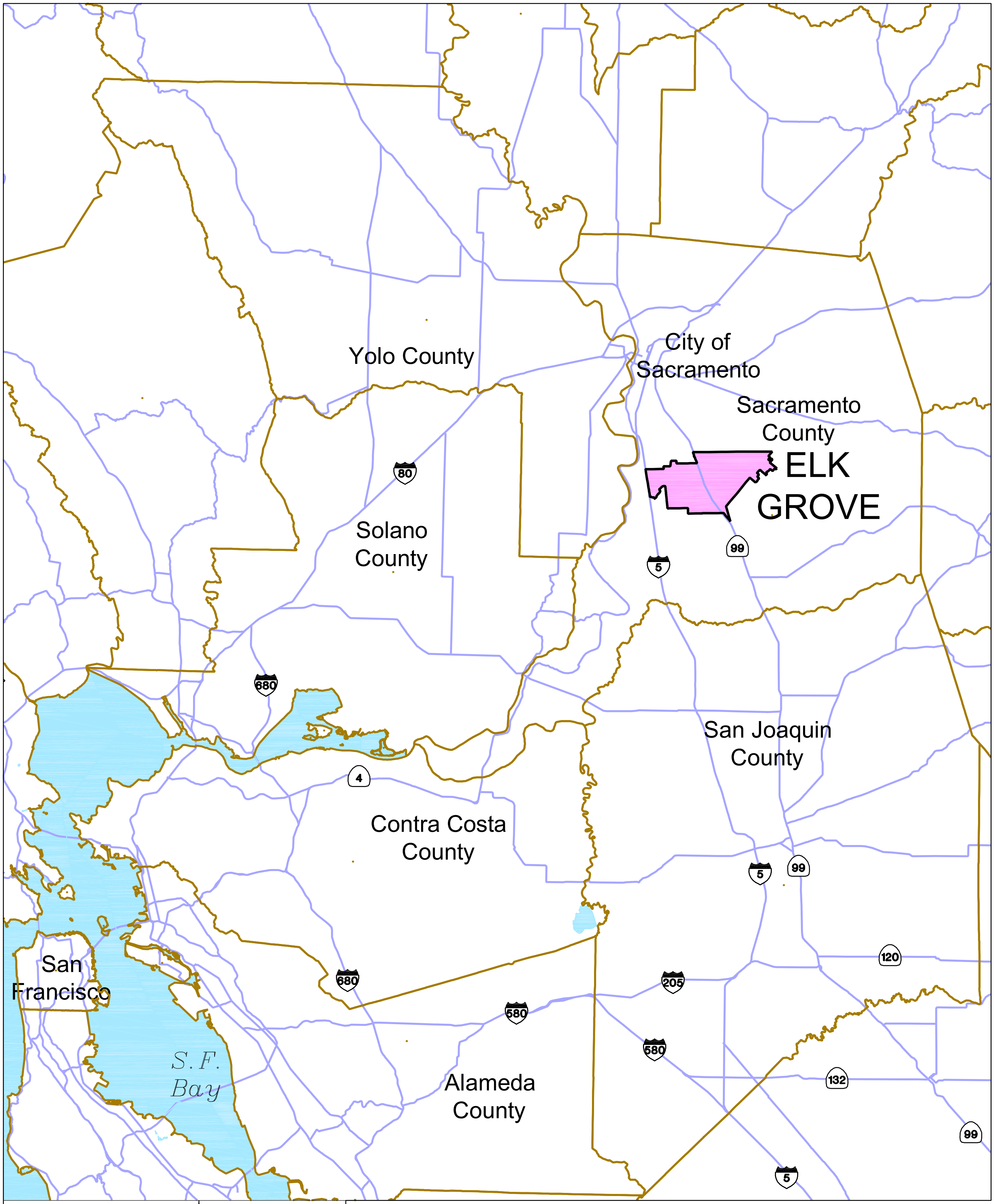
32. Laguna Ridge Specific Plan Supplemental Drainage Plan for Local Drainage Shed C, Wood Rodgers, October 2005.
33. Master Drainage Plan for Elk Grove Promenade, Local Drainage Area Shed C, Wood Rodgers, October 2005.

Strawberry Creek

34. Strawberry and Jacinto Creeks, Drainage Master Plan, Draft Report, County of Sacramento Water Resources Division, July 1993.
35. Middle Branch Strawberry Creek Drainage Master Plan DEIR, ESA, January 22, 1993.
36. Storm Drainage Master Plan Report, Upper Reach of Middle Branch of Strawberry Creek, Elk Grove/West Vineyard Area, MacKay & Somps, February 5, 1992.

Miscellaneous

37. Elk Grove General Plan adopted by the City Council November 19, 2003 and reflecting Amendments through January 5, 2005.
38. Sacramento County's General Plan Land Use Diagram dated December 15, 1993.
39. Stormwater Quality Improvement Plan (SQIP) for City of Elk Grove, Chapter 6, July 2003.
40. Sacramento City/County Drainage Manual, Volume 2, Hydrology Standards, December 1996 and Supplements.



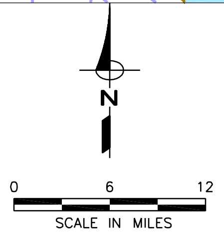


Figure 1-1

City of Elk Grove
Storm Drainage
Master Plan Volume II
VICINITY MAP

WEST YOST
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Consulting Engineers

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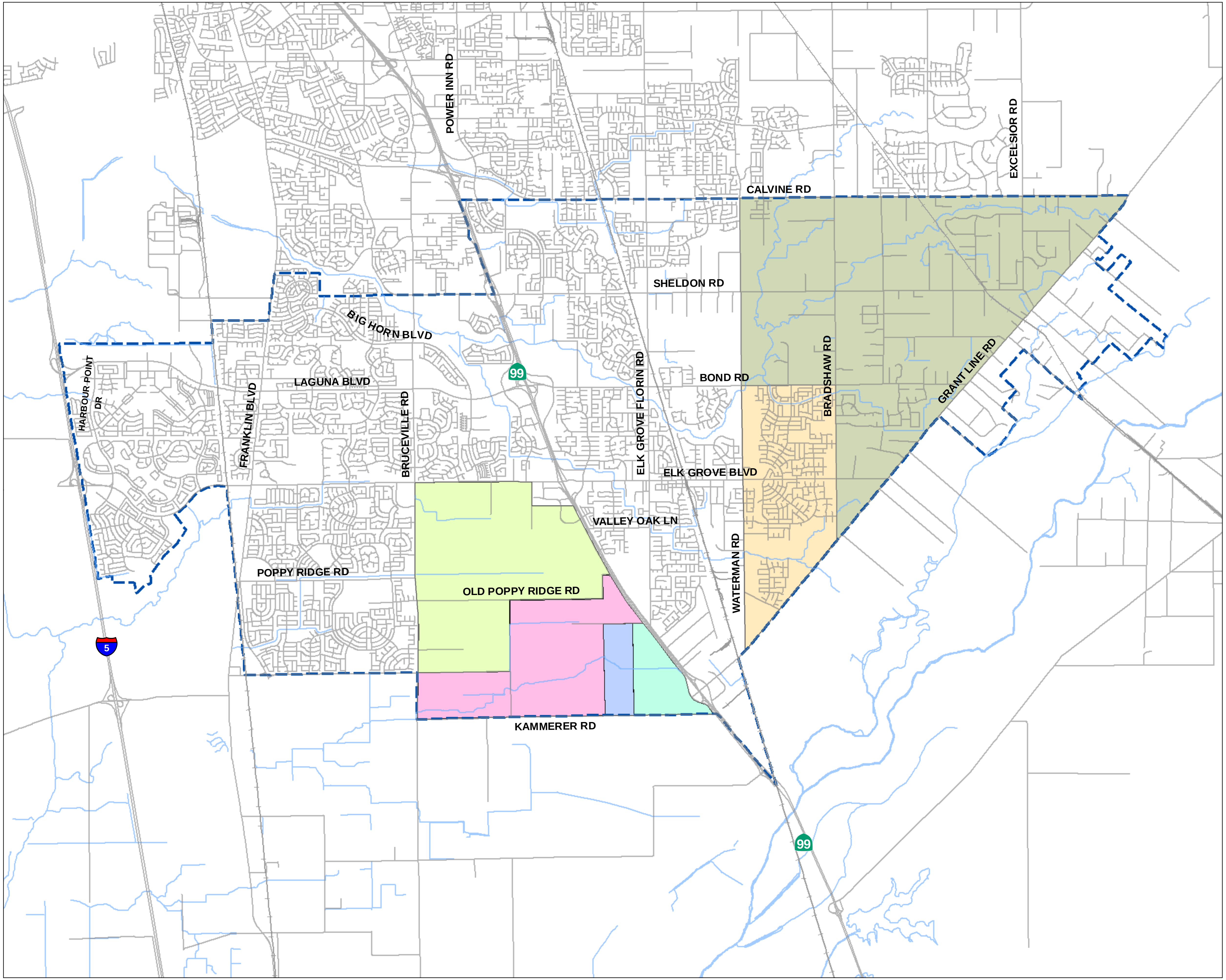

FIGURE 1-2

City of Elk Grove
Storm Drainage Master Plan
Volume II

MAJOR DEVELOPMENT AREAS

N

0 0.5 1 2

Scale in Miles

NOTES:

LEGEND:

- ▭ City Limit
- ▨ Laguna Ridge Specific Plan Area
- ▨ East Elk Grove Specific Plan Area
- ▨ Southeast Policy Area
- ▨ Sterling Meadows
- ▨ Elk Grove Promenade (Lent Ranch)
- ▨ East Elk Grove Area

CITY OF ELK GROVE CALIFORNIA

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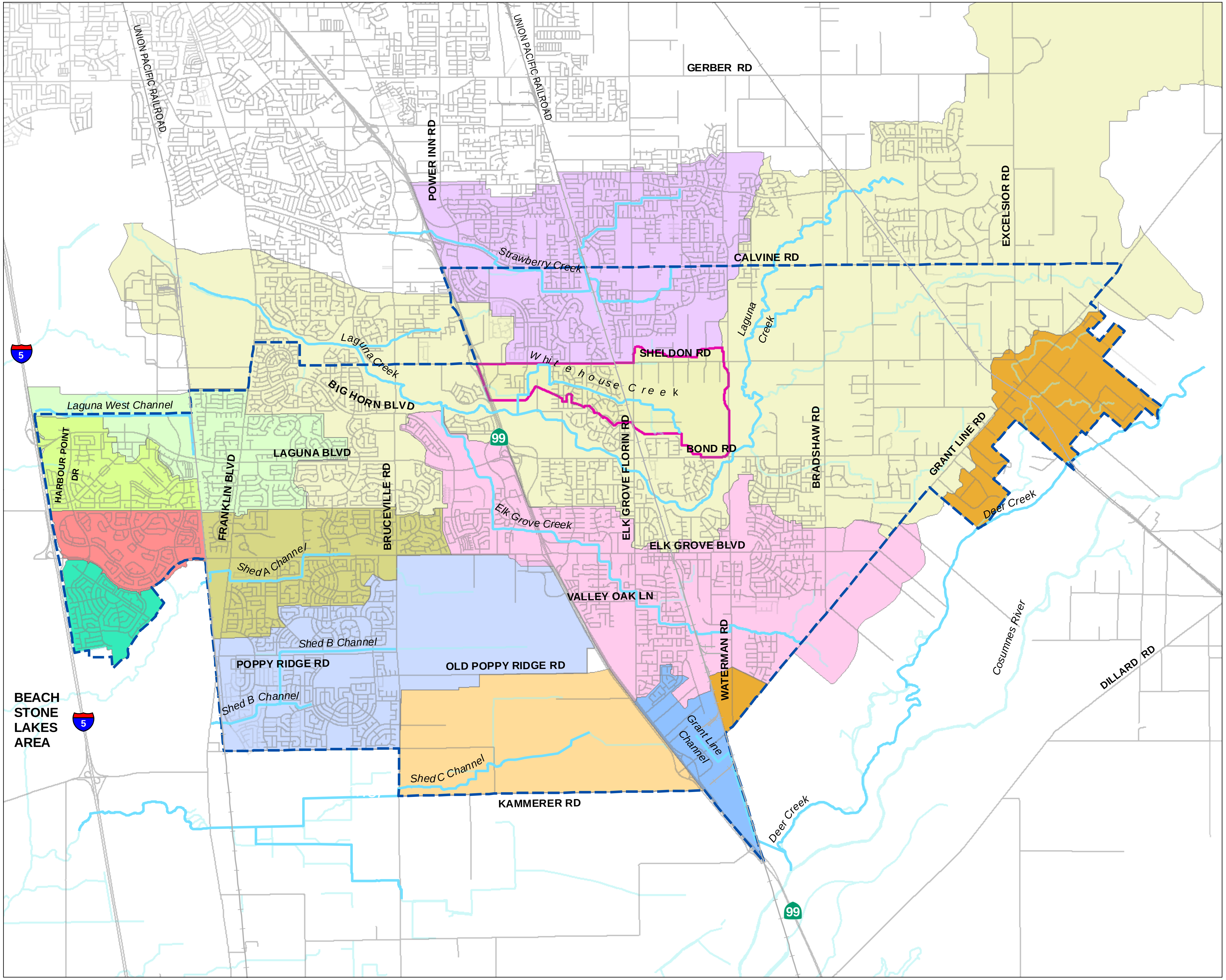

FIGURE 1-3

City of Elk Grove
Storm Drainage Master Plan
Volume II

MAJOR WATERSHEDS

NOTES:

LEGEND:

- ⬚ City Limit
- ┼ Railroad
- 🟧 Deer Creek
- ▨ Elk Grove Creek
- 🟦 Grant Line Channel
- 🟨 Laguna Creek
- 🟩 Laguna Stonelake
- 🟩 Laguna West Channel
- 🟩 Laguna West Lakes
- 🟥 Lakeside
- 🟫 Shed A
- 🟦 Shed B
- 🟧 Shed C
- 🟪 Strawberry Creek
- ☐ Whitehouse Creek

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ASSOCIATES
Consulting Engineers

CHAPTER 2. DRAINAGE PLANNING CRITERIA

This chapter describes the drainage planning criteria used during preparation of the SDMP. The planning criteria establish the basis for the hydrologic and hydraulic evaluations and planning of drainage facilities. Storm drainage planning criteria typically include descriptions of the methods to be used in calculating runoff (or design flows), the level of protection to be provided by storm drainage facilities (*i.e.*, 10-years, 100-years), and the hydraulic factors to be used in sizing drainage facilities, such as channels, pipelines, and detention basins.

After incorporation, the City adopted the County of Sacramento Public Works Agency Improvements Standards and the Sacramento City/County Drainage Manual. Those two documents originally provided the design criteria for new drainage facilities that were constructed within the City. Since that time the City has adopted its own set of improvement standards. The City's storm drainage standards are very similar to County's. One exception is the runoff calculation methodology for pipeline design. The County standards allow use of the "Nolte Method" to determine design flows for new pipelines. The Nolte Method was developed in the 1960s and yields design flow rates that typically have recurrence intervals between 2-years to 5-years. The City standards adopted the 10-year storm for sizing new pipelines.

For the analysis of existing pipes, the evaluation criteria were selected with the recognition that most existing pipes were designed using different criteria than those used today. Specifically, the level of performance for existing pipes were evaluated using current runoff methodology (10-year storm), but with a less stringent freeboard criteria. Most of the existing pipelines and some of the existing drainage channels in the City, were designed using the Nolte Method, which produces flow rates that are smaller than the 10-year flow. There is value in evaluating the performance of the existing facilities using the current design standard (i.e. the 10-year storm), but it is impractical to expect these facilities to provide the same level of protection required for a new system. Doing so would cause a large percentage of the existing facilities to be identified as inadequate. Upgrading or replacing a large percentage of the existing facilities in the City would be financially infeasible; therefore, the evaluation criteria for existing facilities are relaxed as described below.

DRAINAGE PLANNING CRITERIA FOR NEW FACILITIES

Design Runoff

- All new drainage systems were preliminarily sized to accommodate the expected runoff from the entire upstream watershed under buildout land use conditions. Buildout condition land use is based on the City General Plan.
- For storm drain pipe sizing, the runoff for drainage areas of 160 acres and smaller was computed from the Sacramento City/County Drainage Manual, Hydrology Standards – Volume 2 (December 1996) using the Sacramento Method.
- The runoff rates used for sizing of drainage channels, detention facilities, and bridges, were determined using HEC-1 and the Sacramento Method.

Hydraulic Design and Level of Protection

- The level of protection for future structures was evaluated using the 100-year flood event. Finished floor elevations were set at least 1.0 foot above all sources of 100-year flooding or 1.5 feet above the controlling overland release point.
- Future pipes were preliminarily sized so that the hydraulic grade line for a 10-year storm would be a minimum 0.5 feet below the elevation of all inlet grates and a minimum of 1.0 foot below the elevation of all manhole covers.
- It was assumed that the risk of flood damage would be reduced by ensuring the 100-year storm runoff ponds and flows through the proposed development with appropriate freeboard protecting existing and proposed structures.

Culvert Design

- Where roads are not to be overtopped, it was assumed that 1.0 foot of freeboard would be provided between the 100-year water surface elevation and the culvert soffit.

Open Channel Design

- Proposed channels were configured to have 3 horizontal to 1 vertical (3:1) or flatter side slopes.
- All proposed open channel drainage systems were sized to carry the 100-year frequency design storm with a minimum of 1.0 foot of freeboard.
- Although not specifically mentioned in the standards, per FEMA requirements, riverine levees must provide a minimum freeboard of three feet above the 100-year water surface elevation. An additional 1-foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.

Detention Basin Design

- A 100-year, 24-hour storm was used for sizing detention storage facilities used for flood control. Detention basin facilities were sized to provide a minimum freeboard of 1.0 foot above the emergency spillway with the emergency spillway located 0.5 feet above the 100-year water surface elevation within the basin. If a detention basin was proposed to be used for both stormwater quality and flood control purposes, the stormwater quality volume was not considered as part of the required flood control storage volume.

EVALUATION CRITERIA FOR EXISTING FACILITIES

Design Runoff Criteria

- Same as the criteria for new facilities.

Hydraulic Design Criteria and Level of Protection

- The hydraulic grade line for a 10-year storm should not exceed the top of curb.
- Structures should be protected from the 100-year storm. The hydraulic grade line for the 100-year storm should not exceed the elevation of building pads.

Culvert Evaluation Criteria

- Culverts flowing submerged were considered acceptable, as long as the upstream backwater did not exceed the allowable levels of protection described above.

Open Channel Evaluation Criteria

- Open channels should provide sufficient capacity to prevent the flooding of building pads during a 100-year storm event.

Detention Basin Evaluation Criteria

- Detention facilities should provide sufficient detention volume to prevent flooding of building pads during a 100-year storm event.

CHAPTER 3. APPROACH

FACILITIES EVALUATED DURING SDMP

For this SDMP, major drainage facilities currently serving the City were evaluated. This included existing natural channels, constructed channels, pipelines, pump stations, and detention basins. In addition, potential future drainage facilities were evaluated, including channels, pipelines, and detention basins. Due to the large number of facilities required to serve the City, it was not feasible to evaluate them all. Descriptions of the facilities evaluated during this SDMP are provided below.

Existing Natural and Man-Made Channels

Most of the major channels serving the City were evaluated including Laguna Creek, Whitehouse Creek, Elk Grove Creek, Strawberry Creek, the Laguna West Channel, and the Grant Line Channel. The associated bridge and culverts structures were also evaluated. Two major channels were not evaluated: the Shed A Channel and the Shed B Channel. These two channels were designed and constructed relatively recently and are believed to meet the City's flood control performance criteria.

Existing Pipelines

Existing pipelines analyzed include those pipelines 27-inches in diameter and larger that lie within, or cross under, the City's arterial roadways. Additionally, existing pipelines serving areas with known flooding problems were evaluated. Many of the known problem areas lie within the Elk Grove "Old Town" area, which is the area bounded by Waterman Road, Bond Road, Highway 99, and Grant Line Road. All of the pipelines in the Old Town area with diameters of 27 inches and larger were evaluated regardless of roadway size.

Existing Pump Stations

For this SDMP, four major pump stations were evaluated in conjunction with the associated trunk system draining to it. The pump stations serving the Laguna West, Lakeside, Laguna Stonelake, and Grant Line Channel watersheds were evaluated.

Existing Detention Basins

Existing detention basins were evaluated if they have a significant effect on the flood flow rates in the other facilities being evaluated during this SDMP.

New Facilities for Future Development

For future development areas, the new channel, detention, culvert, and major pipeline improvements required for on-site flood protection and for mitigation of downstream impacts were evaluated. For the on-site pipeline improvements, the major trunk pipelines down to a minimum size between 36 to 42 inches were defined. The evaluation of facilities required for future development focused on the following large undeveloped areas within the City:

- All of Drainage Shed C within the City.
- The “East Elk Grove area/rural region” that is bounded by Waterman Road and Laguna Creek on the west, Calvine Road on the north and Grant Line Road on the southeast.
- The area southeast of the intersection of Calvine Road and Waterman Road. This area covers roughly 1,000 acres within the Laguna Creek watershed and is anticipated for Rural Residential and Estate Residential development.
- The area northwest of the intersection of Bond Road and Waterman Road in the upper Whitehouse Creek watershed. This area covers roughly 400 acres and is anticipated for Low Density Residential and Rural Residential development.

HYDROLOGIC ANALYSES – CALCULATION OF FLOOD FLOWS

Hydrologic modeling was performed to calculate design flows for the 2-, 10-, and 100-year storm events. The modeling was performed with Sacramento County’s HEC-1 preprocessor program, SacCalc. The SacCalc model was used to calculate design flows using the Sacramento Method as described in the Sacramento City/County Drainage Manual, Volume 2 - Hydrology Standards. The calculated flow hydrographs were used during the hydraulic analysis of major creeks and channels, detention basins, and pipelines.

For watersheds that are completely built-out, or nearly so, flow hydrographs were calculated for buildout conditions. For watersheds where existing and buildout flows are anticipated to be significantly different, both existing and buildout flow hydrographs were calculated. Existing land-uses were determined from subdivision maps and aerial photographs provided by the City. Buildout condition land-use was determined from current General Plan for the City.

Watershed boundaries were determined from a variety of sources including 2-foot contour topographic mapping, drainage facility maps, as-built plans, previous studies, aerial photographs, and field visits.

Soil types were determined from a soil survey prepared by the U.S. Department of Agriculture Natural Resources Conservation Service. The soil survey provides a classification of soils based on their capacity to infiltrate rainfall. The majority of the soils within the City are classified as Type D soils that have a low capacity to infiltrate storm water. In watersheds that have only a small percentage of other soil types, it was assumed that the entire watershed consists of Type D soils.

HYDRAULIC ANALYSES – CALCULATION OF WATER SURFACE ELEVATIONS

Hydraulic models were prepared to evaluate the flood control performance of the major existing drainage facilities serving the City. The results of the hydraulic analyses were used to identify existing performance deficiencies and, when necessary, to evaluate the improvements required to bring the existing system into compliance with the drainage criteria. Hydraulic models were also prepared to evaluate and size the facilities required to serve the three future development areas described above. The approach to the hydraulic modeling varied depending upon the facility being analyzed.

Storm Drainage Pipelines: Existing and future storm drainage pipelines were evaluated using the XPSWMM modeling software. The XPSWMM program was configured to perform unsteady-state calculations, which allowed the effects of surface storage and overland flow to be considered.

Creeks, Channels, Culverts, and Flood Control Detention Basins: Most major creeks, channels, culverts, and detention basins were evaluated using the HEC-RAS program. For many of the major creeks, the HEC-RAS models were configured to run in an unsteady-state mode. This allowed the effects of channel storage on the flows in the creeks to be evaluated more effectively. The unsteady-state mode also allowed off-channel detention basins to be included in the hydraulic model.

For the Laguna West Channel and Grant Line Channel watersheds, the hydraulic analyses were performed using XPSWMM. Much of the existing pipe system in these watersheds were evaluated during this SDMP and use of the XPSWMM model allowed the pipe and channel systems to be linked together in a single model.

In the Laguna West Lakes and Lakeside watersheds, the modeled drainage system consists of a underground pipe network that delivers runoff to a lake system that is evacuated by a pump station. There are no open channels within these watersheds. Again, XPSWMM was used for these watersheds since it allowed the pipe, lake, and pump systems to be linked together in a single model.

For the Laguna Stonelake watershed, the modeled facilities include a detention basin and a pump station. Because pipeline and channel modeling was not required, the modeling for this watershed was performed entirely with the SacCalc program.

STORMWATER QUALITY

New development projects within the City are required to provide facilities to reduce the pollutant discharges associated with stormwater. It was beyond the scope of this SDMP to define the specific stormwater quality treatment facilities required within individual development projects. However, within Shed C, a preliminary regional drainage improvement plan was developed that will provide, among other things, stormwater quality treatment. In the other major future development areas, it was assumed that stormwater quality treatment would be provided with dry detention basins. The basins were preliminarily sized to provide enough information to develop a preliminary cost for inclusion in a future Capital Improvement Plan.

CHAPTER 4. LAGUNA CREEK

WATERSHED DESCRIPTION

Laguna Creek is the largest stream within the City of Elk Grove. The location of the Laguna Creek watershed is shown on Figure 4-1. Runoff in the watershed flows generally to the southwest until the creek reaches Waterman Road. There, the creek bends, flowing to the northwest, towards its confluence with Morrison Creek. The total watershed area, at the confluence of Morrison Creek, is approximately 48 square miles (mi^2). Laguna Creek is the most southern stream of the larger Morrison Creek stream group.

The headwaters of Laguna Creek begin in the City of Rancho Cordova to the northeast. Laguna Creek flows into the City at Calvine Road, picking up Whitehouse Creek and Elk Grove Creek before leaving the City boundaries near Sheldon Road. The creek then picks up flows from Jacinto Creek within the City of Sacramento limits and joins Morrison Creek just east of Interstate 5.

The largest tributaries to Laguna Creek include:

- Elk Grove Creek, with a watershed area of 6.5 mi^2.
- Tributary No. 1, with a watershed area of 5.0 mi^2.
- Whitehouse Creek, with a watershed area of 1.8 mi^2.
- Jacinto Creek, with a watershed area of 1.0 mi^2.

EVALUATION OF LAGUNA CREEK (INCLUDING WHITEHOUSE CREEK)

The evaluation of Laguna Creek also included a concurrent evaluation of Whitehouse Creek. A single hydraulic model was created for the two creeks since the water surface elevations in Laguna Creek are felt to have a significant effect on those in Whitehouse Creek. Therefore, it was beneficial to link the modeling of the two creeks together. The Laguna Creek analysis also required inflows from the other major tributaries to be calculated (e.g. Elk Grove Creek and Tributary No. 1), but the hydraulic modeling for the other major tributaries was performed separately and the discussion of that modeling occurs in other chapters of this report.

Subsequent to the preparation of the Laguna Creek/Whitehouse Creek modeling, a more refined hydrologic and hydraulic model for the upper reach of Whitehouse Creek was prepared David Ford Consulting. This modeling was used by Ford to define the existing 100-year water surface elevations and floodplain limits in Whitehouse Creek upstream of the UPRR (formerly the SPRR). The City submitted a Letter of Map Revision to FEMA requesting that they recognize the revised floodplain limits in this area. For this SDMP, the results from the refined modeling have been used as the basis of the 100-year water surface elevations and floodplain limits along Whitehouse Creek upstream of the UPRR. A report describing the development of the upper Whitehouse Creek modeling is provided as Attachment 4A.

Hydrologic Analysis for Laguna Creek

For the hydrologic analysis, a watershed model was used to transform design rainfall (of specified probability) over a given area to runoff hydrographs. The watershed model provided the inflow hydrographs needed for the channel hydraulic model, which was used route the flows through Laguna Creek. See Figures 4-2a and 4-2b for the subshed boundaries and designations.

For the Laguna Creek analysis, the design flows at several key locations are needed, so the design storms for the subsheds contributing at those points must be correctly defined. The Laguna Creek Watershed is over 48 square miles and requires multiple SacCalc models to be used, each with appropriate aerial adjustment factors for the design storms.

The computed runoff hydrographs for four storm centerings (contributing basin areas) were used for this analysis. These are listed in Table 4-1. Each of these storm centerings was routed though the Laguna Creek watershed using the HEC-RAS model.

Table 4-1. Storm Centering Locations

Storm Area, mi$^{(2)}$	Approximate Centering Location$^{(1)}$
22	Laguna Creek at upstream City limit at Calvine Road.
32	Laguna Creek near intersection of Waterman Road and Bond Road.
40	Laguna Creek near the confluence with Elk Grove Creek.
48	Laguna Creek near the confluence with Morrison Creek, entire Laguna Creek watershed.

(1) Approximate location represents the point in the watershed with the specified contributing area.

Hydrology for Existing Conditions

The existing conditions hydrology for Laguna and Whitehouse Creeks was developed using the most recent topographic data, storm drain system maps, land use designations, soil maps and aerial photography information. The information developed from these sources was input into SacCalc (Sacramento County's HEC-1 preprocessor, 2004) for development of existing storm runoff conditions. The 2-year, 10-year and 100-year events were evaluated.

The SacCalc model was originally created by David Ford Consulting in 2005 for a hydrologic study of the Laguna Creek watershed for the City of Elk Grove. For this SDMP the subshed boundaries were modified based on the most current topographic information. The model was also updated to be consistent with the modeling prepared for the East Elk Grove area/rural region, as discussed in Chapter 6.

The flow hydrographs from the East Elk Grove area/rural region are for buildout conditions as that is the only scenario evaluated for that area. Therefore, the existing conditions analysis for Laguna Creek actually uses buildout condition flows for Tributary 1. However, as land use changes from existing to buildout will not significantly change runoff patterns in this portion of the watershed, buildout condition flows coming in from Tributary 1 will not greatly overestimate the existing condition flows for the entire watershed. The hydrologic data for each subshed is shown in Table 4-2. The SacCalc layout configuration is shown in Figure 4-3a and 4-3b.

Table 4-2. Hydrologic Parameters for Laguna Creek Subsheds

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope,ft/ft	Land Use (acres) and Percent Impervious																	Avgerage % Imp
						Highway, Parking	Comm./ Office	Ind.	HDR	Mobile	MDR	Resd, 8 to 10 du/ac	Resd, 6 to 8 du/ac	Resd, 4 to 6 du/ac	Resd, 3 to 4 du/ac	Resd, 2 to 3 du/ac	Resd, 1 to 2 du/ac	Resd, .05 to 1 du/ac	Resd, .02 to .05 du/ac	< .02 du/ac, recreation	OpenSpace, grassland/ag	Vacant	
						95%	90%	85%	80%	75%	70%	60%	50%	40%	30%	25%	20%	15%	10%	5%	2%	1%	
Existing Conditions																							
LC01	828.7	241	18,813	9,198	0.0040	21.0		0.0													645.3	162.4	4
LC02	559.0	226	9,900	5,702	0.0053	5.6															65.3	488.2	2
LC04	849.0	179	11,588	5,571	0.0032	26.9														33.5	788.6		5
LC10	854.1	153	11,140	7,764	0.0049	9.6	132.8														711.7		17
LC12	1,020.7	186	16,897	8,102	0.0060	19.4														46.7	876.1	78.5	4
LC18	276.9	125	6,222	2,861	0.0050	20.0															256.9		9
LC21	545.3	121	8,207	5,554	0.0036	10.9															535.3		4
LC19	832.1	152	16,048	9,842	0.0031	43.5	57.0		2.0	2.3	5.5		9.7					13.1	61.5	76.7	537.2	111.5	15
LC23	637.7	130	6,289	3,451	0.0061	35.7	1.8							2.0					125.2	1.0	471.6		9
LC22	177.8	113	3,634	2,412	0.0039	5.6															172.3		5
LC24	630.4	102	10,683	2,554	0.0026	17.2	0.4														612.9		5
LC26	271.9	96	5,723	2,872	0.0028	1.4	38.9														231.2		15
LC30	1,100.0	127	22,864	12,133	0.0029	25.5	60.8	193.7			7.5							0.6	0.8	1.1	735.5	88.5	24
LC35	1,177.5	100	18,531	7,349	0.0028	11.4	95.6		1.6	4.0			14.9		0.9			6.7	39.4	31.8	934.3	37.6	11
LC40	513.7	87	11,962	7,161	0.0022	18.1	28.8		8.6		5.7		5.8		8.8	2.2		12.6	99.9	95.9	127.6	109.2	15
LC41	812.4	66	11,798	6,131	0.0029	107.3	2.4		383.7	1.2	18.9	0.9	2.2					19.3	21.8	19.6	66.3	168.8	54
LC14	1,077.2	159	17,386	5,929	0.0035	370.1	6.9	7.4													692.8		35
LC31	627.2	107	13,794	8,747	0.0030	4.1		15.1													608.3		5
LC42A	846.0	53	9,640	3,656	0.0044	83.2	42.8		126.2	0.1	5.9		8.3	0.2				7.5	90.2	129.0	97.6	255.2	29
LC45	279.9	46	6,362	3,512	0.0041	14.0												5.5	21.2	90.6	123.6	25.2	8
LC60	83.8	58	13,711	6,874	0.0019	2.8				5.9									0.1	28.2	10.5	36.3	11
LC61	1,499.0	56	22,972	10,178	0.0014		14.0						114.0	175.0		19.0	56.0	352.0		620.0	149.0		16
LC42B	628.1	52	9,667	4,188	0.0017	19.4	8.6		7.9	20.3	16.4		1.2					17.1	86.6	208.4	59.6	182.5	13
LC18B	71.5	125	2,980	1,280	0.0070			3.3													68.2		6
L21680	208.0	50	3,696	1,637	0.0005	26.5	3.1		85.2		1.2	0.5	1.4		0.5			2.0		10.6	70.6	6.5	48
L21650	74.9	43	2,492	1,299	0.0027	9.5			32.4				0.6							17.0	13.0	2.4	49
L21640	173.3	46	4,314	2,709	0.0021	29.2	5.0	3.2	92.5		8.2	0.7	1.6	0.1	0.8			6.1			21.5	4.5	68
L21625	89.3	41	2,550	1,447	0.0054	26.0	6.5	0.8	25.4		0.9		0.3					1.0	1.8	11.0	13.1	2.6	60
L21610	88.2	40	3,237	1,478	0.0013	18.4	5.6		36.7		18.3		0.3	3.8							3.2	2.0	75
L21670	100.1	40	3,268	2,292	0.0029	15.5			55.1		0.3		0.7					0.1	0.3	16.4	10.7	1.1	60
L21590	117.5	38	4,435	3,078	0.0024	21.6	8.6		53.4		0.7	0.2	0.4								24.4	8.3	62
L21580	121.1	39	4,034	2,371	0.0020	24.7	0.1		53.1				1.8								33.7	7.6	56
L21560	152.5	34	5,438	1,869	0.0016	21.2	38.1		73.4		0.5	0.2	0.1								18.3	0.7	75
L21520	88.3	30	3,849	2,091	0.0005	7.6	48.7		10.5				0.6	0.2					7.5	2.9	10.3		69
L21510	370.1	34	7,999	3,849	0.0012	54.2	201.3		75.8		7.4			0.2							14.6	16.5	81
L21490	175.6	25	3,511	2,154	0.0005	26.3	0.0		63.1		3.5	0.1	0.4					0.5	4.9		71.5	5.0	46
L51690	120.7	65	7,102	4,002	0.0035	4.7					2.1							23.0	9.1		82.0		10
L51689	54.4	55	1,980	1,352	0.0045	2.2													8.1		44.1		7
L51680	121.8	55	4,599	2,598	0.0033	6.4					2.4							25.9	20.0		67.0	0.3	12
L51670	91.4	40	2,160	1,045	0.0013	5.2					1.1							3.8	49.9	4.0	1.4	26.2	13
L51660	214.9	45	7,830	5,797	0.0031	16.8	1.7	0.0	3.9		1.9							65.7	10.1	5.9	32.9	76.1	16
L51590	41.4	38	2,297	1,399	0.0020	1.8		1.6	0.2											4.8	6.6	26.5	9
L51580	146.2	38	4,034	2,709	0.0011	26.4	0.2		101.9		0.2										17.5		73
L51570	74.5	32	2,160	1,162	0.0031	15.3	4.4		44.0				0.6								7.5	2.7	73
L51550	52.1	30	2,519	1,098	0.0016	4.0	0.1		4.7				0.4						22.0	6.3	10.2	4.5	20
L51540	116.0	30	3,802	2,001	0.0008	23.9	18.8		24.4			0.1	0.2					6.2	8.7		14.1	19.5	53
L51490	55.3	27	1,800	998	0.0008	12.0	2.5		9.5		2.8		0.1							0.8	1.1	26.5	43
L51480	86.2	25	3,300	2,001	0.0006	6.8	8.9		5.2												0.6	64.7	22
L21460	96.0	24	2,619	1,389	0.0015	9.1	3.8		2.3										1.5		44.1	35.3	16
L21410	93.5	27	3,907	2,587	0.0008	11.2	12.1	1.0	6.3	1.1								1.2	5.2		17.8	37.7	32
L21390	19.4	27	422	211	0.0011	2.7	14.3		0.9													1.4	84
L21370	908.5	26	13,490	8,041	0.0011	153.1	80.8		438.1		6.0		1.9	0.3				4.9	3.6		172.0	48.0	64
L21320	585.2	19	9,673	4,800	0.0014	113.3			320.7		0.4	0.2	0.5					0.5			61.6	88.0	63
L21310	553.3	20	8,501	3,802	0.0005	120.6	1.3		336.3		0.3			3.5				1.0			15.3	75.1	70
L21250	691.1	13	5,174	2,429	0.0003	11.2	318.8		0.2												360.6	0.3	44
L21150	397.4	10	4,013	3,432	0.0012		336.5														60.9		77
L201	39.0	33	1,550	780	0.0010	6.2	2.6		1.2		0.4							2.8	3.5	1.4	2.5	18.4	27
L31420	158.4	30	4,118	1,795	0.0012	25.0	4.8	1.4	46.5	4.2	0.6		1.2					0.7	9.6		28.5	36.0	46
L31410	117.8	24	3,326	2,112	0.0018	24.4	8.0		37.2	0.1	0.7	1.5		0.1				3.6	9.4		2.2	30.7	54
L31350	50.8	22	1,795	898	0.0017	9.1	1.7		26.0												9.3	4.8	61
L202	62.2	34	1,600	950	0.0010	9.9	4.2		1.8		0.7							4.4	5.6	2.3	4.1	29.4	27
L203	70.3	36	2,182	1,048	0.0010	11.2	4.7		2.1		0.8							5.0	6.3	2.6	4.6	33.2	27
Buildout Conditions																							
LC01	828.7	241	18,813	9,198	0.004	21.0		0.0													645.3	162.4	4
LC02	559.0	226	9,900	5,702	0.0053	5.6															65.3	488.2	2
LC04	849.0	179	11,588	5,571	0.00315	26.9														33.5	788.6		5
LC10	854.1	153	11,140	7,764	0.0049	9.6	132.8														711.7		17
LC12	NaN	186	16,897	8,102	0.006	19.4														46.7	876.1	78.5	4
LC18	276.9	125	6,222	2,861	0.005	20.0															256.9		9
LC21	545.3	121	8,207	5,554	0.0036	10.9															535.3		4
LC19	832.1	152	16,048	9,842	0.0031	43.5	57.0		2.0	2.3	5.5		9.7					13.1	61.5	76.7	537.2	111.5	15
LC23	637.7	130	6,289	3,451	0.0061	35.7	1.8							2.0					125.2	1.0	471.6		9
LC22	177.8	113	3,634	2,412	0.0039	5.6															172.3		5
LC24	630.4	102	10,683	2,554	0.0026	17.2	0.4														612.9		5
LC26	271.9	96	5,723	2,872	0.0028	1.4	38.9														231.2		15
LC30	NaN	127	22,864	12,133	0.0029	25.5	60.8	193.7			7.5							0.6	0.8	1.1	735.5	88.5	24
LC35	NaN	100	18,531	7,349	0.00281	11.4	95.6		1.6	4.0			14.9		0.9			6.7	39.4	31.8	934.3	37.6	11
LC40	513.7	87	11,962	7,161	0.00217	18.1	28.8		8.6		5.7		5.8		8.8	2.2		12.6	99.9	95.9	127.6	109.2	15
LC41	812.4	66	11,798	6,131	0.0029	107.3	2.4		383.7	1.2	18.9	0.9	2.2					19.3	21.8	19.6	66.3	168.8	54
LC14	NaN	159	17,386	5,929	0.0035	370.1	6.9	7.4													692.8		35
LC31	627.2	107	13,794	8,747	0.003	4.1		15.1													608.3		5
LC42A	846.0	53	9,640	3,656	0.0044	83.2	42.8		126.2	0.1	5.9		8.3	0.2				7.5	90.2	129.0	97.6	255.2	29
LC45	265.5	46	6,362	3,512	0.0041	14.5												5.5	20.7	11.4	123.6	104.5	2
LC60	81.0	58	13,711	6,874	0.0019	3.0				5.9									0.1	27.9	10.5	36.3	8
LC61	1,499.0	56	22,972	10,178	0.0014		14.0						114.0	175.0		19.0	56.0	352.0		620.0	149.0		16
LC42B	609.8	52	9,667	4,188	0.0017	19.4	8.6		19.4	20.3	16.8		5.0					17.1	86.6	206.0	59.6	169.3	12
LC18B	71.5	125	2,980	1,280	0.007			3.3													68.2		6
L21680	208.0	50	3,696	1,637	0.0005	26.6	16.4		86.1		1.2	0.5	0.5		0.5			2.0		10.5	57.3	6.5	54
L21650	74.9	43	2,492	1,299	0.0027	8.0			32.8				1.6							17.0	13.0	2.6	48
L21640	173.3	46	4,314	2,709	0.0021	29.2	5.0	3.2	95.1		6.5	0.7	0.5	0.1				7.1			21.5	4.5	68
L21625	89.3	41	2,550	1,447	0.0054	13.3	6.4	0.8	33.0		1.7		4.2					1.0	1.8	11.0	13.5	2.6	56
L21610	88.2	40	3,237	1,478	0.0013	18.4	5.6		36.9		18.4		0.1	3.8							3.2	1.9	75
L21670	100.1	40	3,268	2,292	0.0029	15.5			55.3		0.6		0.2					0.1	0.3	16.4	10.7	1.1	60
L21590	117.5	38	4,435	3,078	0.0024	21.6	8.6		54.0		0.5		0.2								24.4	8.3	62
L21580	121.1	39	4,034	2,371	0.002	24.7	0.1		61.5		0.6		0.0								33.7	0.5	61
L21560	152.5	34	5,438	1,869	0.0016	21.2	38.1		73.8		0.6		0.2								18.3	0.4	75
L21520	88.3	30	3,849	2,091	0.0005	7.6	48.7		10.3		0.2		0.6	0.2					7.5	2.9	10.3		69
L21510	370.1	34	7,999	3,849	0.0012	54.3	201.3		76.6		7.2		0.5	0.2							14.6	15.5	81
L21490	175.6	25	3,511	2,154	0.0005	26.1	0.0		63.1		3.5	0.1	0.4					0.5			71.8	9.9	45
L51690	120.7	65	7,102	4,002	0.0035	4.7					2.1							23.0	9.1		82.0		10
L51689	54.4	55	1,980	1,352	0.0045	2.6	2.1		2.0										4.1		43.7		13
L51680	121.8	55	4,599	2,598	0.0033	6.4					2.4							25.9	20.0		67.0	0.3	12
L51670	91.4	40	2,160	1,045	0.0013	5.2					1.1							3.8	49.9	4.0	1.4	26.2	13
L51660	214.9	45	7,830	5,797	0.0031	30.9	1.7	0.0	3.9		3.8		1.9					65.3	10.1	5.9	18.9	72.7	23
L51590	41.4	38	2,297	1,399	0.002	1.8	14.7	1.6	0.2											4.8	6.6	11.8	41
L51580	146.2	38	4,034	2,709	0.0011	26.4	0.2		101.9		0.2										17.5		73
L51570	74.5	32	2,160	1,162	0.0031	15.3	4.4		42.7		1.8		0.1								7.5	2.7	73
L51550	52.1	30	2,519	1,098	0.0016	4.0	0.1		6.0		1.0								22.0	6.3	10.2	2.6	23
L51540	116.0	30	3,802	2,001	0.0008	23.9	28.5		24.6			0.1						6.2	8.7		14.1	9.8	61
L51490	55.3	27	1,800	998	0.0008	12.0	2.5		11.0												1.1	28.8	41
L51480	86.2	25	3,300	2,001	0.0006	6.8	8.9		5.2												0.6	64.7	22
L21460	100.1	24	2,619	1,389	0.0015	13.1	3.8		2.3		2.3		1.5								44.1	33.0	22
L21410	93.5	27	3,907	2,587	0.0008	11.2	12.1	1.0	6.3	1.1								1.2	5.2		17.8	37.7	32
L21390	19.4	27	422	211	0.0011	2.7	14.3		0.9													1.4	84
L21370	908.5	26	13,490	8,041	0.0011	155.4	82.2		438.3		5.9		1.2	0.3				4.9	3.5		173.9	42.8	64
L21320	585.2	19	9,673	4,800	0.0014	263.3			263.5		0.2	0.2	0.5								57.4	0.0	79
L21310	553.3	20	8,501	3,802	0.0005	120.6	1.3		336.3		0.3			3.5				1.0			15.3	75.1	70
L21250	691.1	13	5,174	2,429	0.0003	11.2	318.8		0.2												360.6	0.3	44
L21150	397.4	10	4,013	3,432	0.0012		336.5														60.9		77
L31420	158.4	30	4,118	1,795	0.0012	25.0	4.8	1.4	46.5	4.2	0.6		1.2					0.7	9.6		28.5	36.0	186
L31410	117.8	24	3,326	2,112	0.0018	24.4	8.0		37.2	0.1	0.7	1.5		0.1				3.6	9.4		2.2	30.7	40
L31350	50.8	22	1,795	898	0.0017	9.1	1.7		26.0												9.3	4.8	26
L202	62.2	34	1,600	950	0.001	9.9	4.2		1.8		0.7							4.4	5.6	2.3	4.1	29.4	33
L201	39.0	33	1,550	780	0.001	6.2	2.6		1.2		0.4							2.8	3.5	1.4	2.5	18.4	17
L203	70.3	36	2,182	1,048	0.001	11.2	4.7		2.1		0.8							5.0	6.3	2.6	4.6	33.2	27

The SacCalc model was used to generate inflow hydrographs for the HEC-RAS hydraulic model. Table 4-3 lists the hydrographs calculated with SacCalc and the location where they are input info the HEC-RAS model.

Table 4-3. HEC-RAS Inflow Locations – Existing Conditions

Creek	SacCalc Location	SacCalc Node Type	HEC-RAS Boundary Type	HEC-RAS Inflow Station
Laguna	LCC2A	Junction	Flow	65509
	LC19	Subbasin	Lateral	64410
	LC21	Subbasin	Uniform	63584 to 61053
	LCR23	Routing	Uniform	61053 to 60773
	LC22	Subbasin	Uniform	60584 to 57584
	LC24	Subbasin	Uniform	57584 to 56584
	LC26	Subbasin	Uniform	53669 to 48924
	LC30	Subbasin	Lateral	48349
	LC31	Subbasin	Uniform	47954 to 43824
	LC35	Subbasin	Uniform	43824 to 33787
	LC40	Subbasin	Lateral	33587
	LC41	Subbasin	Uniform	33357 to 23674
	LC42A	Subbasin	Uniform	23663 to 15915
	LC42B	Subbasin	Uniform	15793 to 10509
	LC45	Subbasin	Uniform	10315 to 5203
Tributary 1	JNC19 (Trib 1)	Junction	Lateral	4251
Tributary 2	JNC 1 (Trib 2)	Junction	Lateral	3942
Tributary 3	JNC 4 (Trib 3)	Junction	Lateral	2631
Tributary 4	JNC 015 (Trib 4)	Junction	Lateral	1446
Laguna	L21680	Subbasin	Lateral	7.622
	L21640	Subbasin	Lateral	7.025
	L21650	Subbasin	Lateral	7.006
	L21610	Subbasin	Lateral	6.762
	L21625	Subbasin	Lateral	6.642
	L21670	Subbasin	Lateral	6.424
	L21580	Subbasin	Lateral	6.146

Table 4-3. HEC-RAS Inflow Locations – Existing Conditions, Cont'd…

Creek	SacCalc Location	SacCalc Node Type	HEC-RAS Boundary Type	HEC-RAS Inflow Station
	L21590	Subbasin	Lateral	6.072
	L21560	Subbasin	Lateral	5.55
	Bypass	Hand Input	Flow Hydro.	1.515
	L51540	Subbasin	Lateral	1.345
	L51490	Subbasin	Lateral	0.989
	L21490	Subbasin	Lateral	0.863
	L51480	Subbasin	Lateral	0.385
	L21460	Subbasin	Lateral	0.341
	L21390	Subbasin	Lateral	0.038
	L21410	Subbasin	Lateral	0.019
	L21510	Subbasin	Lateral	4.831
	L21520	Subbasin	Lateral	4.785
	L43415	Subbasin	Uniform	4.137 to 4.116
	L21370	Subbasin	Lateral	3.203
	C-JC3	Subbasin	Lateral	2.942
	L21310	Subbasin	Lateral	1.78
	L21320	Subbasin	Lateral	1.769
	L21250	Subbasin	Lateral	0.865
	L21150	Subbasin	Lateral	0.402
	DS confluence	Hand Input	Stage	0.195
	Transfer Reach	Hand Input	Flow Hydro.	6000
Transfer Reach	Transfer Reach	DS Boundary	Normal Depth	600
	L53675	Junction	Flow Hydro.	4.323
Whitehouse	L51670	Subbasin	Lateral	3.684
	L51660	Subbasin	Lateral	3.489
	L51590	Subbasin	Lateral	3.393
	L51580	Subbasin	Lateral	3.028
	L51570	Subbasin	Lateral	2.638
	L51550	Subbasin	Lateral	2.087

Hydrology for Buildout Conditions

The hydrologic model for buildout conditions was developed by updating the existing conditions model with buildout land use information. The schematic of buildout conditions is the same as the existing conditions schematic shown on Figure 4-3a and 4-3b. The input for the buildout conditions model can also be found in Table 4-2.

Other than land-use changes, there is one significant change between existing and buildout conditions. This is the addition of detention basins by Sacramento County in the upper portion of the watershed. Detention basins are planned for construction in the Laguna Creek watershed upstream of the City within the County of Sacramento. These basins are intended to limit the increase in flows due to development in the County. The exact sizes and locations of these basins are not known. However, planning information was provided by Sacramento County for use with this SDMP.

Table 4-4 lists the hydrographs calculated with SacCalc and the location where they are input into the HEC-RAS model.

Table 4-4. HEC-RAS Inflow Locations - Buildout Conditions

Creek	SacCalc Location	SacCalc Node Type	HEC-RAS Boundary Type	HEC-RAS Inflow Station
Laguna	LCC2A	Junction	Flow	65509
	LC19	Subbasin	Lateral	64410
	LC21	Subbasin	Uniform	63584 to 61053
	LCR23	Routing	Uniform	61053 to 60773
	LC22	Subbasin	Uniform	60584 to 57584
	LC24	Subbasin	Uniform	57584 to 56584
	LC26	Subbasin	Uniform	53669 to 48924
	LC30	Subbasin	Lateral	48349
	LC31	Subbasin	Uniform	47954 to 43824
	LC35	Subbasin	Uniform	43824 to 33787
	LC40	Subbasin	Lateral	33587
	LC41	Subbasin	Uniform	33357 to 23674
	LC42A	Subbasin	Uniform	23663 to 15915
	LC42B	Subbasin	Uniform	15793 to 10509
	LC45	Subbasin	Uniform	10315 to 5203
Tributary 1	JNC19 (Trib 1)	Junction	Lateral	4251
Tributary 2	JNC 1 (Trib 2)	Junction	Lateral	3942

Table 4-4. HEC-RAS Inflow Locations - Buildout Conditions, Cont'd…

Creek	SacCalc Location	SacCalc Node Type	HEC-RAS Boundary Type	HEC-RAS Inflow Station
Tributary 3	JNC 4 (Trib 3)	Junction	Lateral	2631
	JNC 015 (Trib 4)	Junction	Lateral	1446
Laguna	L21680	Subbasin	Lateral	7.622
	L21640	Subbasin	Lateral	7.025
	L21650	Subbasin	Lateral	7.006
	L21610	Subbasin	Lateral	6.762
	L21625	Subbasin	Lateral	6.642
	L21670	Subbasin	Lateral	6.424
	L21580	Subbasin	Lateral	6.146
	L21590	Subbasin	Lateral	6.072
	L21560	Subbasin	Lateral	5.55
	Bypass	Hand Input	Flow Hydro.	1.515
	L51540	Subbasin	Lateral	1.345
	L51490	Subbasin	Lateral	0.989
	L21490	Subbasin	Lateral	0.863
	L51480	Subbasin	Lateral	0.385
	L21460	Subbasin	Lateral	0.341
	L21390	Subbasin	Lateral	0.038
	L21410	Subbasin	Lateral	0.019
	L21510	Subbasin	Lateral	4.831
	L21520	Subbasin	Lateral	4.785
	L43415	Subbasin	Uniform	4.137 to 4.116
	L21370	Subbasin	Lateral	3.203
	C-JC3	Subbasin	Lateral	2.942
	L21310	Subbasin	Lateral	1.78
	L21320	Subbasin	Lateral	1.769
	L21250	Subbasin	Lateral	0.865
	L21150	Subbasin	Lateral	0.402
	DS Confluence	Hand Input	Stage	0.195

Table 4-4. HEC-RAS Inflow Locations - Buildout Conditions, Cont'd…

Creek	SacCalc Location	SacCalc Node Type	HEC-RAS Boundary Type	HEC-RAS Inflow Station
Transfer Reach	Transfer Reach	Hand Input	Flow Hydro.	6000
	Transfer Reach	DS Boundary	Normal Depth	600
Whitehouse	L53675	Junction	Flow hydro.	4.323
	L51670	Subbasin	Lateral	3.684
	L51660	Subbasin	Lateral	3.489
	L51590	Subbasin	Lateral	3.400
	L51580	Subbasin	Lateral	3.028
	L51570	Subbasin	Lateral	2.638
	L51550	Subbasin	Lateral	2.087

Hydraulic Analysis for Laguna Creek and Whitehouse Creek

An unsteady-flow hydraulic model was prepared for Laguna Creek and Whitehouse Creek using HEC-RAS. The unsteady-flow model, using input runoff hydrographs, considers backwater, ponding, and channel storage effects as the flow is routed through the open channel system. Also, the unsteady-flow model provides an effective tool to evaluate the effects of Laguna Creek water surface elevations on those in Whitehouse Creek. See Figures 4-4a and 4-4b for the layout of the hydraulic model. More detailed maps showing the model cross section locations are provided on Figures 4-6a through 4-6j.

Existing Conditions Hydraulic Analysis

An unsteady state hydraulic model for existing conditions was developed using the inflow hydrographs from the SacCalc model developed using existing land use conditions. The downstream boundary condition, a stage hydrograph in Morrison Creek, is based on hydraulic modeling prepared for the County of Sacramento.

Table 4-5 shows the peak flows and water surface elevations at key locations as calculated in the HEC-RAS model for existing conditions. Figures 4-5a through 4-5i show the water surface profiles along Laguna and Whitehouse Creeks.

Table 4-5. Existing Condition Flows and Water Surface Elevations for Laguna and Whitehouse Creeks

HEC-RAS River Station	Creek	Location	2-year max water surface, ft	2-year max flow, cfs	10-year max water surface, ft	10-year max flow, cfs	100-year max water surface, ft	100-year max flow, cfs
1.94	Whitehouse	Low flow weir	30.1	81	31.0	147	32.1	242
2.2455	Whitehouse	Ski Lake Dr	32.4	99	33.0	170	33.6	254
2.5	Whitehouse	Harding Hall Drive	32.5	101	33.2	172	34.0	257
2.6165	Whitehouse	Bike Crossing	32.6	106	33.3	181	34.0	267
2.755	Whitehouse	Sheldon Creek Drive	32.6	95	33.3	164	34.0	246
3.159	Whitehouse	Camden Parkway	32.8	68	33.5	123	34.5	210
3.3595	Whitehouse	UPRR	33.2	69	33.8	119	34.7	217
3.459	Whitehouse	Elk Grove Florin Road	33.7	62	34.4	107	36.8	200
3.662	Whitehouse	Borrow Pit Road	38.4	43	38.7	64	41.1	134
4.067	Whitehouse	Campbell Road	43.4	28	43.6	37	43.8	200
3.5615	Laguna Creek	Bruceville Road	23.0	285	23.8	401	24.0	522
4.054	Laguna Creek	Lewis Stein	25.8	296	26.2	416	26.4	523
4.711	Laguna Creek	West Stockton	29.1	931	29.8	1,652	30.7	2,752
4.727	Laguna Creek	Hwy 99	29.2	931	30.0	1,652	30.9	2,752
4.745	Laguna Creek	East Stockton	29.2	931	30.1	1,652	31.3	2,752
6.32	Laguna Creek	Bond Road	34.8	895	35.8	1,583	36.8	2,591
6.57	Laguna Creek	Elk Grove Florin Road	36.1	892	37.6	1,579	39.0	2,580
6.94	Laguna Creek	SPRR Crossing	39.5	890	40.6	1,572	41.6	2,565
7.082	Laguna Creek	DS of Waterman, US of SPRR	39.8	880	40.9	1,557	42.1	2,538
7.561	Laguna Creek	Low water crossing DS of Waterman	41.1	882	41.9	1,564	43.2	2,550
8.1175	Laguna Creek	Waterman Road	43.5	883	45.0	1,563	46.6	2,546
51	Laguna Creek	Bond Rd	43.9	884	45.4	1,563	47.1	2,546
8734	Laguna Creek	Sheldon Road	49.7	637	50.2	898	50.8	1,235
15854	Laguna Creek	Calvine Road	54.9	640	55.5	893	56.1	1,241
0	Bypass Channel	Downstream End	14.8	1057	16.5	2063	18.3	3521
0.3575	Bypass Channel	Bruceville Road	17.4	1055	19.1	2052	20.8	3507
1.446	Bypass Channel	Upstream End	21.7	865	23.7	1538	26.0	2532

Note: The 100-Year data on Whitehouse Creek from station 3.659 upstream to station 4.067 is based on modeling by David Ford Consulting from a LOMR submitted to FEMA in 2011.

The results from the HEC-RAS model indicate that there are some areas of over bank flooding in the watershed during the 100-year event. However, there are no structures that are threatened during the 100-year event. There are some backwater effects into Whitehouse Creek at the confluence with Laguna, but these do not result in predicted structure flooding.

Hydraulic Analysis for Buildout Conditions

For buildout conditions, the hydraulic model was revised to include flow hydrographs representing buildout land uses. Table 4-6 summarizes the results from the HEC-RAS model for buildout conditions. Comparisons of existing and buildout condition peak flows and water surface elevations are presented on Table 4-7. Buildout condition water surface profiles produced from the HEC-RAS model are shown on Figures 4-7a through 4-7i.

Improvements to Creek System

Because the Laguna Creek system provides adequate flood protection for both existing and buildout conditions, no improvements are recommended.

EVALUATION OF EXISTING PIPELINES

As indicated in Chapter 3, existing pipelines within the City's arterial roadways with diameters 27 inches and greater were evaluated during this SDMP. Pipelines meeting that size criterion within the "Old Town" area were also evaluated. The Old Town area is bounded by Waterman Road, Bond Road, Highway 99, and Grant Line Road and a portion of this area lies within the Laguna Creek watershed. Nine existing trunk pipelines in the Laguna Creek watershed meet the criterion for a detailed evaluation. Figures 4-8 through 4-12 show the sizes and limits of the existing pipelines that were evaluated.

Hydrologic Analysis of Existing Pipelines

SacCalc models were prepared to calculate the 2-year, 10-year, and 100-year flows into the existing pipe systems. The SacCalc models used to calculate flows in the existing pipelines are more refined than the model used to calculate flows in Laguna Creek. Smaller subsheds were used in the pipeline models to better define the variation in flow along the pipeline.

All of the subsheds served by the existing pipelines are completely developed, or nearly so. Therefore, flow rates were calculated for buildout conditions only. Existing condition flows are not expected to differ significantly.

Figures 4-8 through 4-12 present the subshed boundaries used for the flow calculations. Table 4-8 presents the key hydrologic parameters for each subshed under buildout land use conditions. Table 4-9 presents the calculated peak flows from each subshed for the three storm events.

Table 4-6. Buildout Condition Flows and Water Surface Elevations for Laguna and Whitehouse Creeks

HEC-RAS River Station	Creek	Location	2-year max water surface, ft	2-year max flow, cfs	10-year max water surface, ft	10-year max flow, cfs	100-year max water surface, ft	100-year max flow, cfs
1.94	Whitehouse	Low flow weir	30.3	98	31.1	167	32.1	258
2.2455	Whitehouse	Ski Lake Dr	32.6	121	33.1	183	33.7	271
2.5	Whitehouse	Harding Hall Drive	32.7	141	33.3	184	34.1	272
2.6165	Whitehouse	Bike Crossing	32.8	126	33.4	190	34.1	279
2.755	Whitehouse	Sheldon Creek Drive	32.8	116	33.4	174	34.2	258
3.159	Whitehouse	Camden Parkway	32.8	68	33.5	123	34.5	210
3.3595	Whitehouse	UPRR	33.2	69	33.8	119	34.7	217
3.459	Whitehouse	Elk Grove Florin Road	33.7	62	34.4	107	36.8	200
3.662	Whitehouse	Borrow Pit Road	38.4	43	38.7	64	41.1	134
4.067	Whitehouse	Campbell Road	43.4	28	43.6	37	43.8	200
3.5615	Laguna Creek	Bruceville Road	23.1	332	23.8	454	24.1	570
4.054	Laguna Creek	Lewis Stein	26.0	345	26.3	459	26.6	573
4.711	Laguna Creek	West Stockton	29.1	912	29.7	1,554	30.5	2,540
4.727	Laguna Creek	Hwy 99	29.2	912	29.9	1,554	30.8	2,540
4.745	Laguna Creek	East Stockton	29.2	912	30.0	1,554	32.1	2,540
6.32	Laguna Creek	Bond Road	34.7	837	35.7	1,453	36.7	2,360
6.57	Laguna Creek	Elk Grove Florin Road	36.0	834	37.2	1,448	38.7	2,349
6.94	Laguna Creek	SPRR Crossing	39.5	828	40.4	1,438	41.4	2,375
7.082	Laguna Creek	DS of Waterman, US of SPRR	39.7	818	40.7	1,421	41.8	2,308
7.561	Laguna Creek	Low water crossing DS of Waterman	41.1	820	41.7	1,430	42.9	2,320
8.1175	Laguna Creek	Waterman Road	43.4	817	44.7	1,427	46.3	2,317
51	Laguna Creek	Bond Rd	43.7	817	45.1	1,427	46.7	2,318
8734	Laguna Creek	Sheldon Road	49.5	585	50.3	927	50.9	1,303
15854	Laguna Creek	Calvine Road	54.8	578	55.5	927	56.2	1,322
0	Bypass Channel	Downstream End	15.0	1,143	16.7	2,196	18.3	3,509
0.3575	Bypass Channel	Bruceville Road	17.6	1,147	19.3	2,187	20.8	3,496
1.446	Bypass Channel	Upstream End	21.8	848	23.8	1,446	25.8	2,348

Table 4-7. Comparison of Existing and Buildout Condition Water Surface Elevations

HEC-RAS River Station	Creek	Location	2-year max water surface existing, ft	2-year max water surface buildout, ft	10-year max water surface existing, ft	10-year max water surface buildout, ft	100-year max water surface existing, ft	100-year max water surface buildout, ft
1.94	Whitehouse	Low flow weir	30.1	30.3	31.0	31.1	32.1	32.1
2.2455	Whitehouse	Ski Lake Dr	32.4	32.6	33.0	33.1	33.6	33.7
2.5	Whitehouse	Harding Hall Drive	32.5	32.7	33.2	33.3	34.0	34.1
2.6165	Whitehouse	Bike Crossing	32.6	32.8	33.3	33.4	34.0	34.1
2.755	Whitehouse	Sheldon Creek Drive	32.6	32.8	33.3	33.4	34.0	34.2
3.159	Whitehouse	Camden Parkway	32.8	33.8	33.3	33.7	34.5	34.5
3.3595	Whitehouse	UPRR	33.2	33.2	33.8	33.8	34.7	34.7
3.459	Whitehouse	Elk Grove Florin Road	33.7	34.7	34.4	34.4	36.8	36.8
3.662	Whitehouse	Borrow Pit Road	38.4	38.4	38.7	38.7	41.1	41.1
4.067	Whitehouse	Campbell Road	43.4	43.4	43.6	43.6	43.8	43.8
3.5615	Laguna Creek	Bruceville Road	23.0	23.1	23.8	23.8	24.0	24.1
4.054	Laguna Creek	Lewis Stein	25.8	26.0	26.2	26.3	26.4	26.6
4.711	Laguna Creek	West Stockton	29.1	29.1	29.8	29.7	30.7	30.5
4.727	Laguna Creek	Hwy 99	29.2	29.2	30.0	29.9	30.9	30.8
4.745	Laguna Creek	East Stockton	29.2	29.2	30.1	30.0	31.3	31.1
6.32	Laguna Creek	Bond Road	34.8	34.7	35.8	35.7	36.8	36.7
6.57	Laguna Creek	Elk Grove Florin Road	36.1	36.0	37.6	37.2	39.0	38.7
6.94	Laguna Creek	SPRR Crossing	39.5	39.5	40.6	40.4	41.6	41.4
7.082	Laguna Creek	DS of Waterman, US of UPRR	39.8	39.7	40.9	40.7	42.1	41.8
7.561	Laguna Creek	Low water crossing DS of Waterman	41.1	41.1	41.9	41.7	43.2	42.8
8.1175	Laguna Creek	Waterman Road	43.5	43.4	45.0	44.7	46.6	46.3
51	Laguna Creek	Bond Rd	43.9	43.7	45.4	45.1	47.1	46.7
8734	Laguna Creek	Sheldon Road	49.7	49.5	50.2	50.3	50.8	50.9
15854	Laguna Creek	Calvine Road	54.9	54.8	55.5	55.5	56.1	56.2
0	Bypass Channel	Downstream End	14.8	15.0	16.5	16.7	18.3	18.3
0.3575	Bypass Channel	Bruceville Road	17.4	17.6	19.1	19.3	20.8	20.8
1.446	Bypass Channel	Downstream End	21.7	21.8	23.7	23.8	26.0	25.8

Table 4-8. Hydrologic Parameters for Existing Pipeline Models LC1-LC6

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land-use, acres and Percent Impervious									Average % Imp.
						Comm./ Office	HDR	MDR	Resd, 6-8 du/ac, School	Resd, 4-6 du/ac	Resd, 3-4 du/ac	Rural Res.	Park	Open	
						90%	80%	70%	50%	40%	30%	10%	5%	2%	
Buildout Conditions															
BH110	18.3	22	880	282	0.0020					18.3					40
BH115	99.0	22	5,827	2,901	0.0005					91.3			3.8	3.8	37
BH120	11.4	22	1,162	393	0.0014					11.4					40
BH125	59.0	22	3,037	837	0.0016		7.4			44.2			7.4		41
BH130	72.5	22	2,642	780	0.0012				0.7	65.3			5.1	1.5	37
BH205	41.9	22	2,031	719	0.0016			18.9		23.0					54
BH210	23.1	22	1,682	786	0.0012					23.1					40
BH215	8.5	22	830	358	0.0014					8.5					40
BH220	15.2	22	1,479	595	0.0020					15.2					40
BH230	53.2	22	1,922	260	0.0008					53.2					40
LB105	68.0	28	3,859	1,359	0.0012	1.4	8.8			44.2			13.6		39
LB110	44.9	28	1,891	765	0.0030					42.7			2.2		38
LB115	38.0	28	993	174	0.0038				7.6	30.4					42
LB120	112.6	28	2,466	1,403	0.0013	2.3	2.3		33.8	73.2			1.1		44
LB125	317.0	28	5,275	1,719	0.0005	3.2	15.8	7.9	12.7	237.7			39.6		39
LB130	14.2	28	1,345	291	0.0007	12.0	0.7			1.4					85
LB135	17.8	28	1,057	343	0.0009	11.9	5.9								87
LB140	24.8	28	2,160	907	0.0019	1.2				23.6					43
LB145	129.0	28	3,820	1,350	0.0010		3.9		25.8	96.7			2.6		43
LC3101	4.3	26	900	450	0.0006							4.3			10
LC3105	17.0	29	1,330	660	0.0015	11.7				3.2			2.0		71
LC3110	61.8	29	3,060	1,530	0.0007	61.8									90
LC4110	93.1	36	4,610	2,350	0.0009	85.9	7.2								89
LC4120	19.1	36	2,040	1,000	0.0010	19.1									90
LC4130	15.0	36	1,460	700	0.0014	9.5			5.5						75
LC4150	37.6	36	2,140	1,100	0.0019				37.6						50
LC4160	39.9	36	2,930	1,460	0.0017				35.7				4.2		45
LC4170	37.4	36	1,710	1,400	0.0023				37.4						50
LC4200	6.7	36	900	500	0.0022	6.7									90
LC5110	6.5	39	760	380	0.0030	6.5									90
LC5120	9.7	39	1,020	510	0.0013				9.7						50
LC5130	34.8	39	2,480	1,240	0.0006				34.8						50
LC6110	6.0	39	570	280	0.0007					6.0					40
LC6120	16.4	39	1,110	550	0.0008				5.1	11.3					43
LC6130	39.1	39	1,940	970	0.0010				7.2	31.9					42
LC7110	21.2	40	2,430	1,200	0.0012					18.6			2.6		36
LC7120	16.0	40	1,420	700	0.0028					16.0					40
LC8110	9.3	40	1,180	600	0.0017	9.3									90
LC8120	9.5	40	850	430	0.0024	9.5									90
LC8130	36.9	40	1,600	800	0.0013					24.9	12.0				37
LC8140	52.4	42	2,360	1,100	0.0017					52.4					40
LC8150	36.9	43	2,320	1,160	0.0017					35.8			1.1		39
LC910	30.1	59	1,100	500	0.0009	0.0				30.1					40
LC920	19.6	61	850	386	0.0012	13.6				6.0					75
LC930	27.5	61	1,750	795	0.0006					27.5					40
LC940	107.2	64	4,100	1,864	0.0005					107.2					40

Table 4-9. Calculated Subshed Flows for Existing Pipelines LC1-LC9

Subshed	Area, acres	Buildout Condition Flows, cfs		
		2-Year	10-Year	100-Year
BH110	18.3	15	29	44
BH115	99.0	35	63	89
BH120	11.4	8	16	24
BH125	59.0	32	61	90
BH130	72.5	40	76	111
BH205	41.9	27	52	75
BH210	23.1	14	27	39
BH215	8.5	7	13	19
BH220	15.2	10	20	29
BH230	53.2	37	70	104
LB105	68.0	30	59	87
LB110	44.9	27	55	81
LB115	38.0	31	66	103
LB120	112.6	56	112	161
LB125	317.0	117	231	331
LB130	14.2	11	22	33
LB135	17.8	14	29	44
LB140	24.8	14	28	41
LB145	129.0	58	115	164
LC3101	4.3	3	5	8
LC3105	17.0	13	23	35
LC3110	61.8	37	67	95
LC4110	93.1	50	88	124
LC4120	19.1	13	24	35
LC4130	15.0	11	21	31
LC4150	37.6	23	42	61
LC4160	39.9	21	38	56
LC4170	37.4	23	43	62
LC4200	6.7	6	11	17
LC5110	6.5	6	12	18
LC5120	9.7	7	14	20
LC5130	34.8	18	33	46
LC6110	6.0	5	10	15
LC6120	16.4	11	21	31
LC6130	39.1	23	42	61
LC7110	21.2	11	21	30
LC7120	16.0	11	21	31
LC8110	9.3	8	15	22
LC8120	9.5	9	17	26
LC8130	36.9	23	42	62
LC8140	52.4	30	56	80
LC8150	36.9	21	39	56
LC910	30.1	21	40	58
LC920	19.6	17	32	48
LC930	27.5	16	30	43
LC940	107.2	46	83	116

Hydraulic Analysis of Existing Pipelines

Hydraulic models of the nine existing pipe systems were created using XPSWMM. Calculated water surface elevations for the 2-year, 10-year, and 100-year storm events are summarized on Table 4-10. Calculated peak flows are summarized in Table 4-11. As Table 4-10 shows, the City's performance criteria for existing pipelines are met for all but two pipelines. Existing Pipeline LC4 has two locations where the 10-year water surface elevation is above the curb and one location where the 100-year water surface elevation is predicted to inundate a building pad. Existing Pipeline LC8 has one location with 10-year flooding above the curb.

Improvements to Existing Pipelines

Pipe improvements are necessary to eliminate the predicted flooding along Existing Pipelines LC4 and LC8. For Existing Pipeline LC4, upsizing of 2,460 feet of pipeline is required to eliminate the street and building pad flooding. For Existing Pipeline LC8, upsizing of 3,080 feet of pipeline is required to eliminate the excessive street flooding. The pipe improvements required to bring the pipe systems in compliance with the performance criteria are shown on Figures 4-13 and 4-14.

EVALUATION OF FUTURE PIPELINES AND STORMWATER QUALITY FACILITIES

In the Laguna Creek watershed, future pipeline improvements are expected in the area southeast of the intersection of Calvine Road and Waterman Road. This area is largely undeveloped and is anticipated for future Rural Residential and Estate Residential development. Potential future trunk pipeline alignments were estimated and sized. In addition to the pipeline improvements, facilities will be required to provide stormwater quality treatment for runoff from development areas. It was assumed that stormwater quality treatment would be provided with dry detention basins located near the end of each trunk pipeline.

Because the future development patterns and roadway layouts in this area are unknown, it is anticipated that the actual layout of the trunk pipe system will be different from that proposed in this SDMP. However, the trunk pipelines and stormwater quality facilities defined in this SDMP are adequate for the development of a Capital Improvement Plan in the future.

Hydrologic Analysis of Future Pipelines

The future trunk pipes were analyzed for the 10-year storm only. Evaluation of the 2-year storm is unnecessary since the water surface will be contained within the pipe. The water surface elevations for the 100-year storm will be dependent on the layout and grading of the future streets, which are unknown at this time.

SacCalc models were prepared to calculate the buildout 10-year flows into the future trunk pipe systems. Figure 4-15 presents the subshed boundaries used for the flow calculations. Table 4-12 presents the key hydrologic parameters for each subshed for buildout conditions. Table 4-13 presents the calculated 10-year peak flows from each subshed.

Hydraulic Analysis of Future Pipelines

Hydraulic models of six trunk pipe systems were created using XPSWMM. Pipelines were sized to provide a 10-year water surface elevation that is a minimum of 0.5 feet below the anticipated gutter elevations in the future streets. The estimated pipe lengths, sizes, and slopes are presented on Table 4-14 along with the calculated water surface elevations for the 10-year storm event. The pipe alignments and sizes are shown on Figure 4-15.

Table 4-10. Calculated Water Surface Elevations for Existing Pipelines LC1-LC9 (NGVD29)

Node Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline No. LC1							
BH100	n/a	20.0	8.7	14.1	16.1	-	-
BH110	20.1	20.0	16.8	17.2	18.0	-	-
BH115	21.8	22.0	17.3	17.9	19.1	-	-
BH120	22.3	23.0	18.0	18.7	19.8	-	-
BH125	20.8	23.0	18.4	19.2	20.2	-	-
BH127	22.3	21.0	18.7	19.3	19.9		-
BH130	21.0	22.0	18.9	19.5	20.0	-	-
BH205	19.7	21.0	17.4	18.8	19.9	-	-
BH210	21.1	22.0	17.7	19.3	20.6	-	-
BH215	23.4	22.0	17.9	19.6	20.8	-	-
BH220	21.3	23.0	18.5	20.1	21.1	-	-
BH225	20.7	23.7	19.0	20.1	20.9	-	-
BH230	21.0	21.7	19.9	20.4	20.9	-	-
Pipeline No. LC2							
LB100	n/a	n/a	12.9	17.1	19.3	-	-
LB105	22.9	24.0	17.5	18.1	19.6	-	-
LB110	23.3	24.9	18.5	20.1	21.0	-	-
LB115	24.0	26.0	20.4	22.8	23.6	-	-
LB120	24.6	27.0	21.6	23.9	24.8	-	-
LB125	25.9	27.0	24.3	25.1	25.7	-	-
LB130	28.6	30.0	27.1	27.7	28.0	-	-
LB135	30.0	32.0	27.9	29.6	29.5	-	-
LB140	32.1	32.0	28.2	29.0	29.4	-	-
LB145	31.0	32.0	27.5	28.5	29.0	-	-
Pipeline No. LC3							
LC3101	26.8	27.0	21.8	23.8	24.0	-	-
LC3105	26.6	26.8	22.4	23.9	25.3	-	-
LC3110	27.3	28.9	22.8	24.8	26.9	-	-
Pipeline No. LC4							
LC4100	34.6	36.0	29.2	30.1	31.4	-	-
LC4110	34.6	36.0	30.1	31.6	32.9	-	-
LC4120	37.4	35.8	30.5	32.4	33.7	-	-
LC4130	35.6	35.5	30.7	32.8	34.1	-	-
LC4150	35.2	35.5	31.2	33.6	34.7	-	-
LC4160	34.3	35.2	32.8	34.8	35.7	Yes	Yes
LC4170	35.8	36.7	34.6	36.2	36.6	Yes	-
LC4200	35.6	36.0	31.2	32.9	34.6	-	-
Pipeline No. LC5							
LC5100	35.8	n/a	31.0	35.0	36.0	-	-
LC5110	37.4	38.0	31.8	35.0	36.0	-	-
LC5120	37.5	38.5	33.4	36.2	37.4	-	-
LC5130	37.7	38.7	35.2	37.7	38.4	-	-
Pipeline No. LC6							
LC6100	35.7	n/a	35.1	35.2	36.2	-	-
LC6110	38.7	40.1	36.6	37.3	37.9	-	-
LC6120	39.2	40.4	38.1	39.2	39.7	-	-
LC6130	39.9	41.0	39.7	39.9	40.1	-	-
Pipeline No. LC7							
LC7100	40.3	n/a	31.2	35.7	36.8	-	-
LC7110	39.8	39.2	31.5	35.7	36.8	-	-
LC7115	37.8	39.2	32.5	35.7	36.8	-	-
LC7120	38.2	39.5	33.1	35.7	37.3	-	-

Table 4-10. Calculated Water Surface Elevations and Flows for Existing Pipelines LC1-LC9 (NGVD29), cont'd…

Node Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline No. LC8							
LC8100	40.5	n/a	33.3	35.7	36.8	-	-
LC8110	41.0	40.0	33.8	35.7	36.9	-	-
LC8120	41.5	40.0	34.4	36.2	37.7	-	-
LC8130	44.5	46.0	36.6	39.6	42.0	-	-
LC8140	42.5	44.0	39.3	41.3	43.4	-	-
LC8145	40.1	44.0	39.9	40.7	41.4	Yes	-
LC8150	43.3	44.0	42.8	43.3	43.5	-	-
Pipeline No. LC9							
LC900	n/a	60.0	51.0	55.3	55.9	-	-
LC910	58.5	59.0	52.5	58.0	58.5	-	-
LC920	60.8	61.3	53.5	60.5	60.8	-	-
LC930	61.3	61.8	54.7	61.0	61.5	-	-
LC940	63.5	64.0	57.0	62.3	62.9	-	-

Table 4-11. Calculated Peak Flows for Existing Pipelines LC1-LC9

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline No. LC1						
BH110R	BH110	BH100	Pipe	158	228	287
BH115R	BH115	BH110	Pipe	87	123	151
BH120R	BH120	BH115	Pipe	60	66	74
BH125R	BH125	BH120	Pipe	55	57	62
BH127R	BH127	BH125	Pipe	33	41	44
BH130R	BH130	BH127	Pipe	33	41	44
BH205R	BH205	BH110	Pipe	68	108	135
BH210R	BH210	BH205	Pipe	41	56	73
BH215R	BH215	BH210	Pipe	28	34	36
BH220R	BH220	BH215	Pipe	22	23	27
BH225R	BH225	BH220	Pipe	17	19	20
BH230R	BH230	BH225	Pipe	17	19	20
Pipeline No. LC2						
LB105A.1	LB105	LB102	Pipe	111	152	177
LB105B	LB105	LB102	Pipe	111	152	177
LB110R	LB110	LB105	Pipe	197	259	286
LB115R	LB115	LB110	Pipe	184	213	230
237.1	LB120	LB115	Pipe	181	181	189
236.1	LB125	LB120	Pipe	146	159	148
231.1	LB130	LB125	Pipe	77	90	92
LB135R	LB135	LB130	Pipe	22	38	28
LB140R	LB140	LB135	Pipe	14	15	15
232.1	LB145	LB130	Pipe	58	65	60
OLR120	LB120	LB115	Overland	0	0	7
OLR125	LB125	LB120	Overland	0	0	1
Pipeline No. LC3						
206.1	LC3105	LC3101	Pipe	43	84	118
205.1	LC3110	LC3105	Pipe	36	67	95
Pipeline No. LC4						
220.1	LC4110	LC4100	Pipe	104	182	212
219.1	LC4120	LC4110	Pipe	64	96	111
218.1	LC4130	LC4120	Pipe	56	82	94
216.1	LC4150	LC4130	Pipe	54	70	81
215.1	LC4160	LC4150	Pipe	38	50	51
214.1	LC4170	LC4160	Pipe	19	24	24
LC4_200C	LC4200	LC4130	Pipe	6	10	15
215.2	LC4160	LC4150	Overland	0	0	3
214.2	LC4170	LC4160	Overland	0	1	9
Pipeline No. LC5						
208.1	LC5110	LC5100	Pipe	23	34	40
206.1	LC5120	LC5110	Pipe	21	25	26
204.1	LC5130	LC5120	Pipe	17	18	16
204.2	LC5130	LC5120	Overland	0	0	8
Pipeline No. LC6						
110.1	LC6110	LC6100	Pipe	29	38	42
120.1	LC6120	LC6110	Pipe	26	30	31
130.1	LC6130	LC6120	Pipe	19	20	20
120.2	LC6120	LC6110	Overland	0	0	5
130.2	LC6130	LC6120	Overland	0	0	2
Pipeline No. LC7						
LC7110C	LC7110	LC7100	Pipe	21	37	75
203.1	LC7120	LC7115	Pipe	11	21	31
205.1	LC7115	LC7110	Pipe	11	21	31

Table 4-11. Calculated Peak Flows for Existing Pipelines LC1-LC9, cont'd…

Conduit Name	Upstream Node	Downstream Node	Type of Flow	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline No. LC8						
LC8110C	LC8110	LC8100	Pipe	71	97	118
LC8120C	LC8120	LC8110	Pipe	68	86	99
LC8130C	LC8130	LC8120	Pipe	65	77	86
LC8140C	LC8140	LC8130	Pipe	46	42	42
LC8145C	LC8145	LC8140	Pipe	23	28	26
203.1	LC8150	LC8145	Pipe	20	20	20
203.2	LC8150	LC8145	Overland	0	18	36
Pipeline No. LC9						
RLC910	LC910	LC900	Pipe	67	126	134
RLC920	LC920	LC910	Pipe	58	98	97
RLC930	LC930	LC920	Pipe	53	95	112
RLC940	LC940	LC930	Pipe	44	70	81
OLRLC920	LC920	LC910	Overland	0	1	8
OLRLC930	LC930	LC920	Overland	0	1	9

Table 4-12. Hydrologic Parameters for Future Pipeline Models LCN1-LCN6

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land Use, acres and Percent Impervious		Average % Imp.	Stormwater Quality Detention Volume, ac-ft
						Resd, 3-4 du/ac	Rural Res.		
						30%	10%		
Buildout Conditions									
LCN110	19.4	67	1,290	586	0.0155	0.0	19.4	10	0.29
LCN120	28.3	67	780	354	0.0269	12.4	15.9	19	0.42
LCN130	32.3	70	1,380	625	0.0145	32.3	0.0	30	0.67
LCN210	78.0	57	1,790	971	0.0017	0.0	78.0	10	1.17
LCN310	6.5	57	420	201	0.0024	6.5	0.0	30	0.14
LCN320	28.9	59	1,530	698	0.0013	28.9	0.0	30	0.60
LCN410	8.6	57	450	212	0.0022	8.6	0.0	30	0.18
LCN420	29.1	59	1,930	878	0.0010	29.1	0.0	30	0.60
LCN510	29.4	55	870	395	0.0023	0.0	29.4	10	0.44
LCN520	50.1	58	1,440	744	0.0028	0.0	50.1	10	0.75
LCN610	37.6	60	1,140	700	0.0010	0.0	37.6	10	0.56
LCN620	54.1	62	1,360	680	0.0007	0.0	54.1	10	0.81
LCN630	26.9	64	1,720	900	0.0006	0.0	26.9	10	0.40
LCN640	39.4	60	1,520	800	0.0010	0.0	39.4	10	0.59

Table 4-13. Calculated Subshed Flows for Future Pipelines LCN1-LCN6

Subshed	Area, acres	10-Year Flow, cfs
LCN110	19.4	25
LCN120	28.3	47
LCN130	32.3	47
LCN210	78.0	71
LCN310	6.5	12
LCN320	28.9	33
LCN410	8.6	15
LCN420	29.1	30
LCN510	29.4	37
LCN520	50.1	53
LCN610	37.7	39
LCN620	54.1	51
LCN630	26.9	23
LCN640	39.4	37

Table 4-14. Hydraulic Design Data for Future Pipelines LCN1-LCN6

Upstream Node	Dowstream Node	Diameter, in	Length	Upstream Invert Elevation, ft	Downstream Invert Elevation, ft	Design Slope, ft/ft	10-Year Peak Flow, cfs	Est. Future Gutter Elev., ft	Upstream 10-Year hgl, ft
Future Pipeline LCN1									
LCN110	LCN100	60	670	45.7	45.0	0.0010	117	54.0	51.7
LCN120	LCN110	60	870	46.5	45.7	0.0010	93	54.5	53.4
LCN130	LCN120	36	580	52.0	46.5	0.0094	47	58.0	57.1
Future Pipeline LCN2									
LCN210	LCN200	48	1060	46.0	44.0	0.0019	71	53.0	52.3
Future Pipeline LCN3									
LCN310	LCN300	48	250	49.0	47.0	0.0080	40	56.0	55.2
LCN320	LCN310	36	620	50.0	49.0	0.0016	33	56.5	55.9
Future Pipeline LCN4									
LCN410	LCN400	42	490	47.4	46.9	0.0010	37	54.5	53.8
LCN420	LCN410	36	620	48.0	47.4	0.0010	30	56.0	54.9
Future Pipeline LCN5									
LCN510	LCN500	54	500	46.0	45.5	0.0010	86	54.0	52.0
LCN520	LCN510	42	710	46.7	46.0	0.0010	52	56.0	54.1
Future Pipeline LCN6									
LCN610	LCN600	48 (2 pipes)	690	54.5	54.0	0.0007	103	60.0	59.1
LCN620	LCN610	54	1360	56.0	54.5	0.0011	72	64.0	61.4
LCN630	LCN620	42	1110	58.0	56.0	0.0018	37	64.0	62.3
LCN640	LCN600	48	760	54.5	54.0	0.0007	24	64.0	58.7

1. All elevations are based on the National Geodetic Vertical Datum of 1929.
2. Pipe data is conceptual. More detailed analyses will be required with development projects as they occur.

Analysis of Stormwater Quality Facilities

Stormwater quality treatment was assumed to be provided with dry detention basins near the end of each trunk pipeline. The stormwater quality storage volumes required for the trunk pipeline watersheds were estimated based on the method presented in Appendix E of the Stormwater Quality Design Manual for the Sacramento and South Placer Regions, May 2007. The volumes required for each subshed are presented in the last column of Table 4-12. The volumes for each subshed serving a pipeline were added together to determine the total volume required at the end of the pipeline. Table 4-15 summarizes the total water quality volume required within each pipe shed.

Table 4-15. Summary of Required Stormwater Quality Volumes

Future Pipeline/Watershed	Required Storage Volume, ac-ft
LCN1	1.38
LCN2	1.7
LCN3	0.74
LCN4	0.78
LCN5	1.19
LCN6	2.36

PRELIMINARY IMPROVEMENTS

As discussed above, future drainage improvements are anticipated in the Laguna Creek watershed. These improvements are summarized below and on Table 4-16. These improvements are considered preliminary. They are adequate for development of a Capital Improvement Plan, but the ultimate improvements will be defined from a more detailed design study and could vary from those recommended in this SDMP.

- Upsizing of Existing Pipelines LC4 and LC8 (See Figures 4-13 and 4-14).
- Construction of six new trunk pipelines to serve future development in the area southeast of the intersection of Calvine and Waterman Roads (See Figure 4-15).
- Construction of six stormwater quality detention basins to serve the watersheds drained by each of the new trunk lines (See Table 4-15). The total required storage volume is estimated to be 7.6 acre-feet.

Table 4-16. Preliminary Improvements in Laguna Creek Watershed

Item	Quantity	Unit
Existing Pipeline Upgrades		
36" RCP	995	LF
60" RCP	2775	LF
66" RCP	1773	LF
Manholes	16	EA
Outfall Structures	1	EA
Future Pipelines		
36" RCP	1820	LF
42" RCP	2310	LF
48" RCP	3450	LF
54" RCP	1860	LF
60" RCP	1540	LF
Manholes	31	EA
Outfall Structures	6	EA
Stormwater Quality Detention Basins		
SWQ Detention for Sheds LCN1-LCN6	29,400	CY

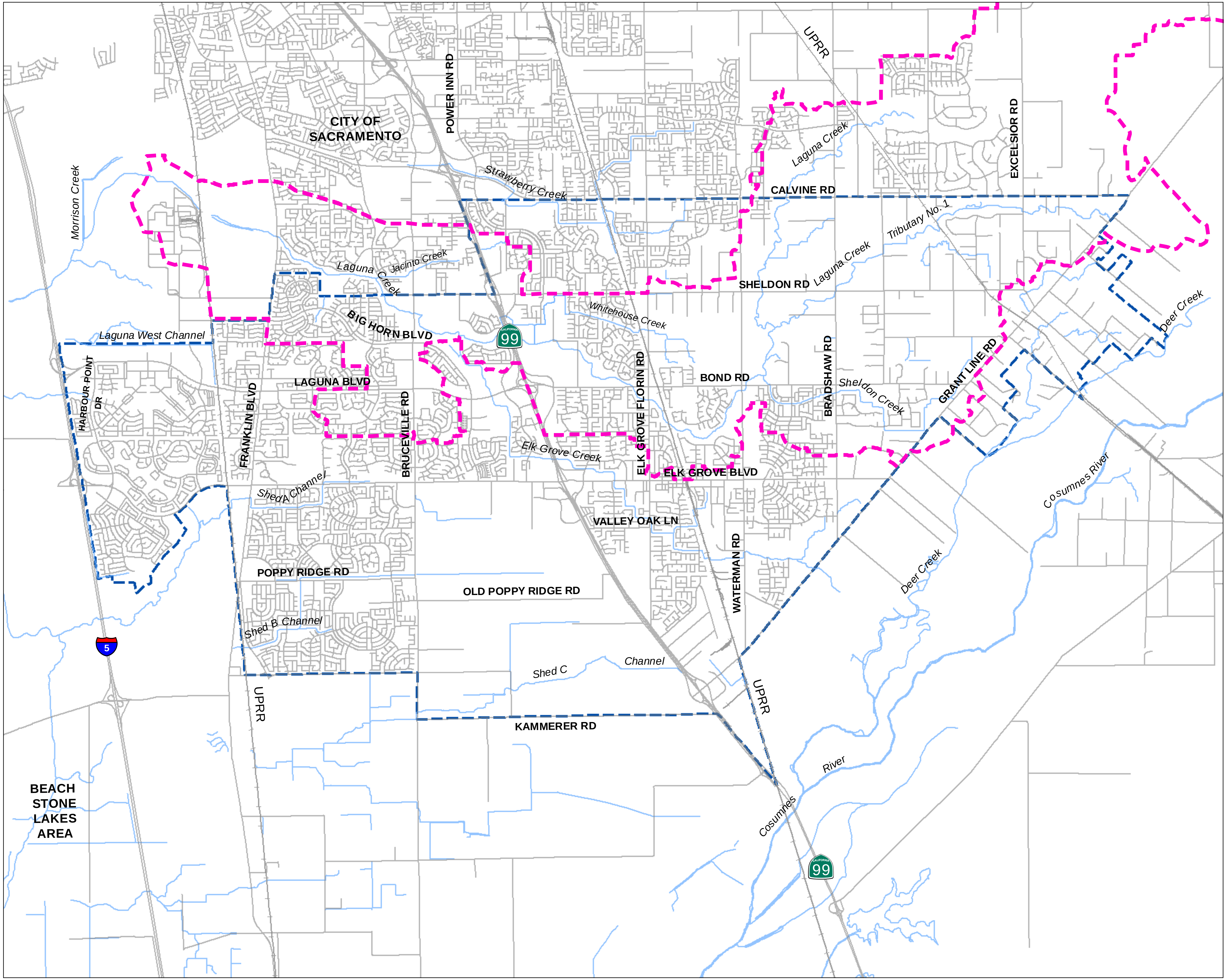
Notes:

1. Facilities in East Area are not included.

FIGURE 4-1

City of Elk Grove
Storm Drainage Master Plan Study
Volume II

LAGUNA CREEK
LOCATION MAP



N

0 0.5 1 2
Scale in Miles

NOTES:

LEGEND:

City Limit

Laguna Creek Watershed

WEST YOST
ASSOCIATES
Consulting Engineers

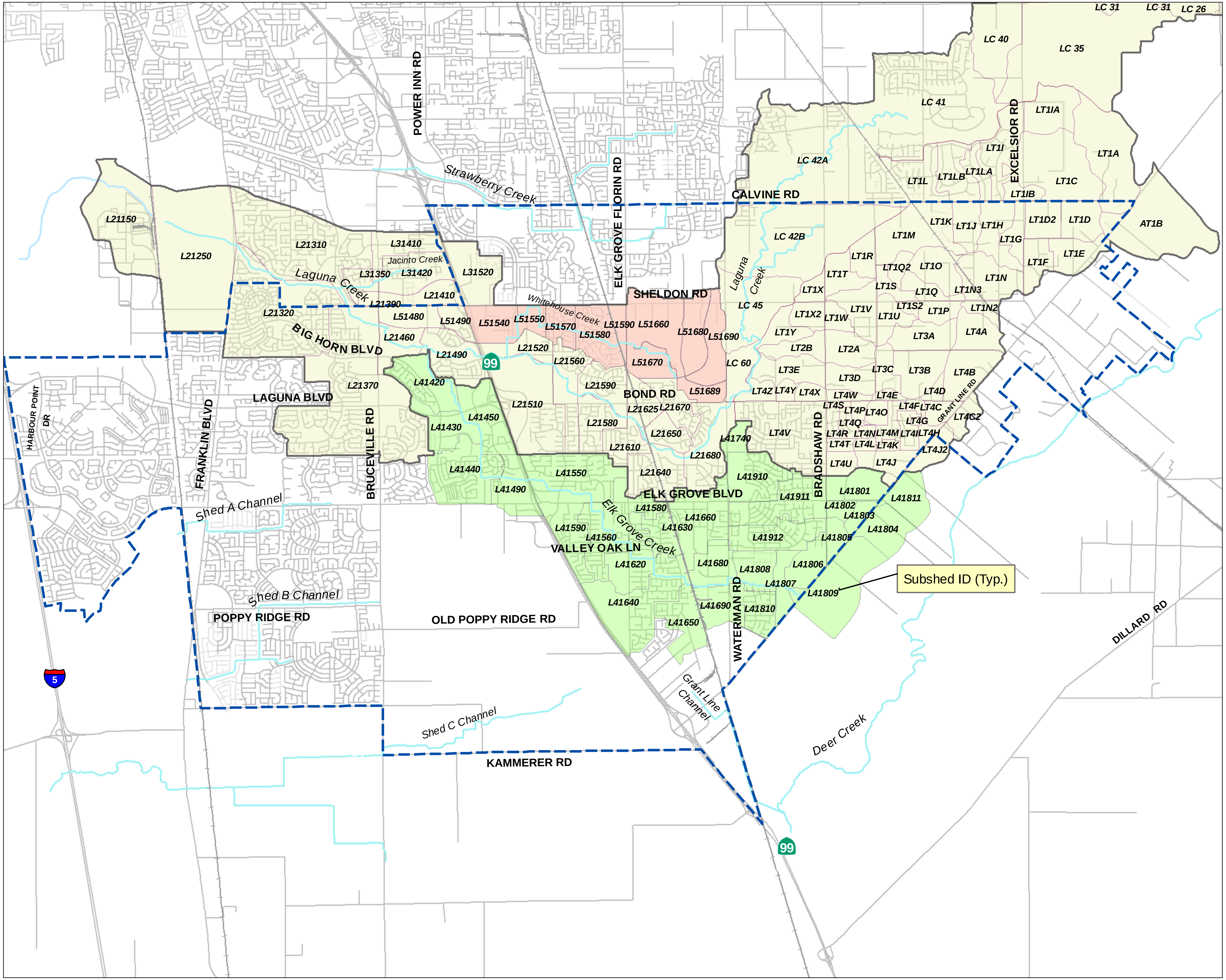
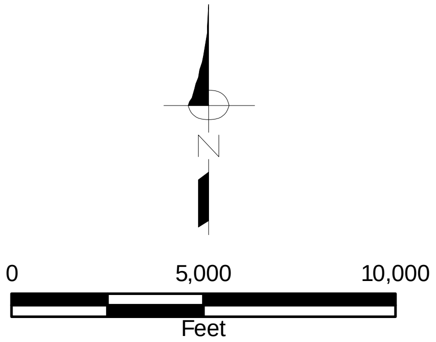


FIGURE 4-2A

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK AND TRIBUTARIES SUBSHEDS (LOWER)



NOTES:

LEGEND:

- City Limit
- channel-laguna-updated
- 🟥 Whitehouse Creek Subsheds
- 🟨 Laguna Creek Subsheds
- 🟩 Elk Grove Creek Subsheds

WEST YOST ASSOCIATES
Consulting Engineers

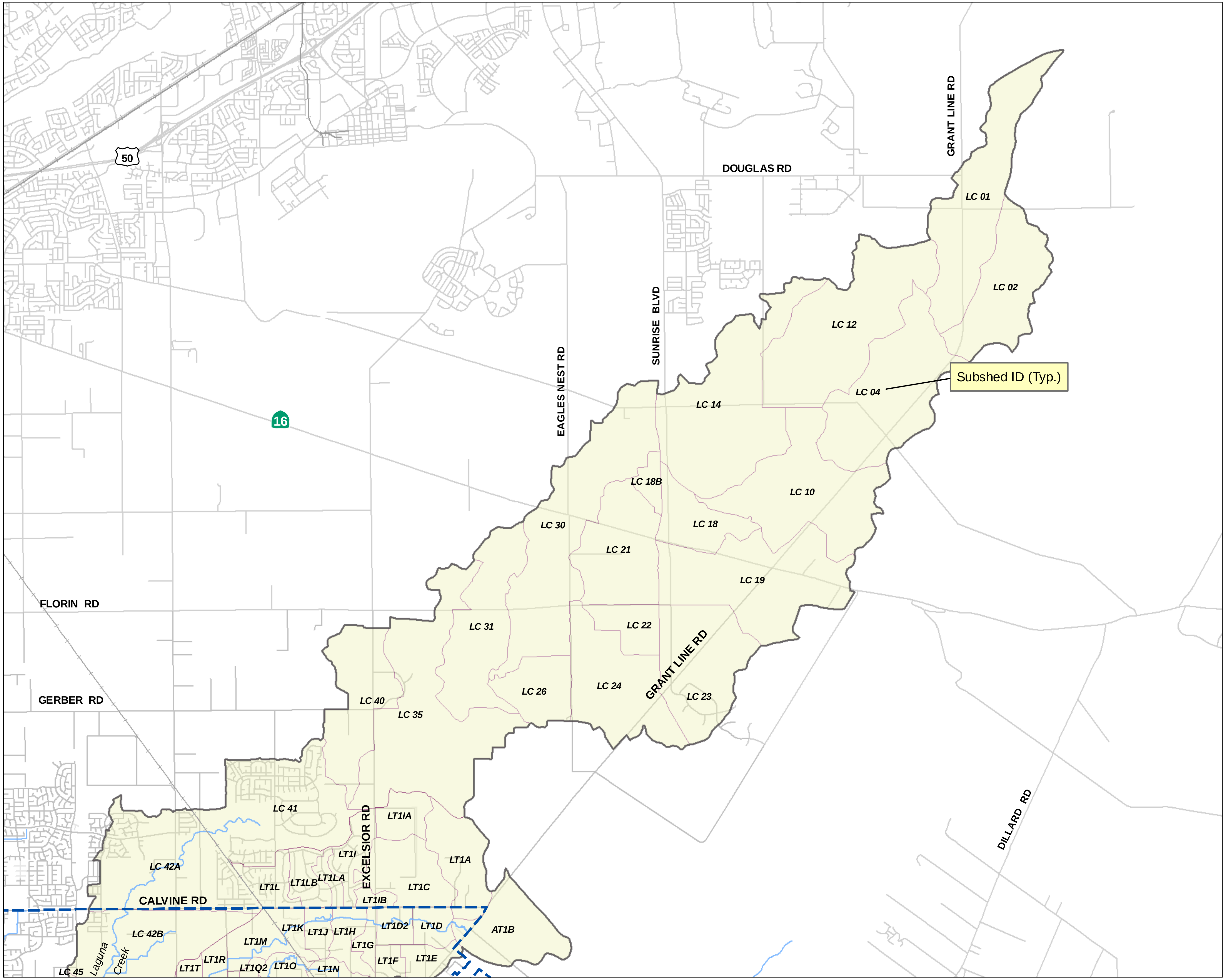


FIGURE 4-2B

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK AND TRIBUTARIES SUBSHEDS (UPPER)

N

0 5,000 10,000
Feet

NOTES:

LEGEND:

🟨 Laguna Creek Subsheds

▭ City Limit

WEST YOST
ASSOCIATES
Consulting Engineers

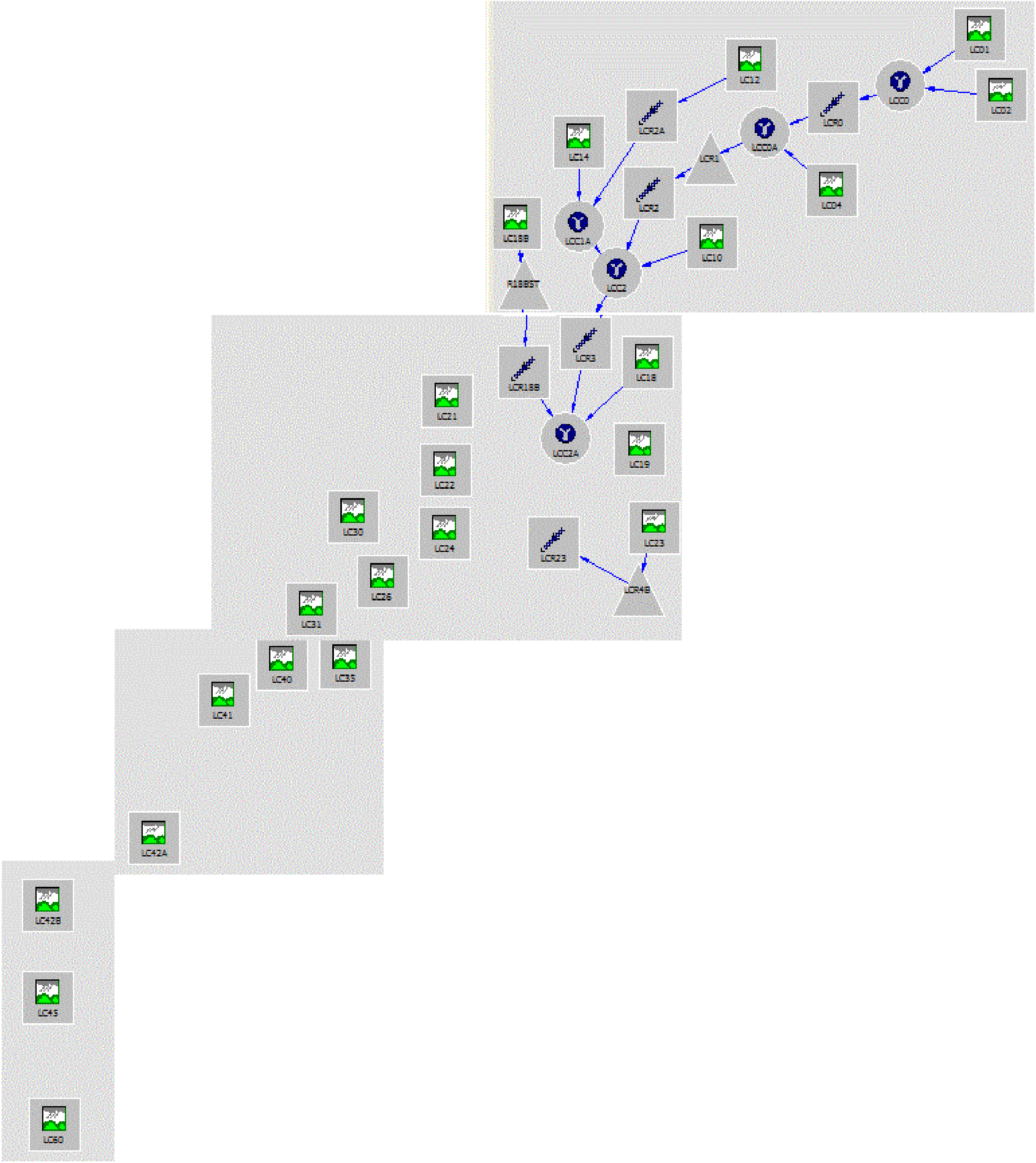
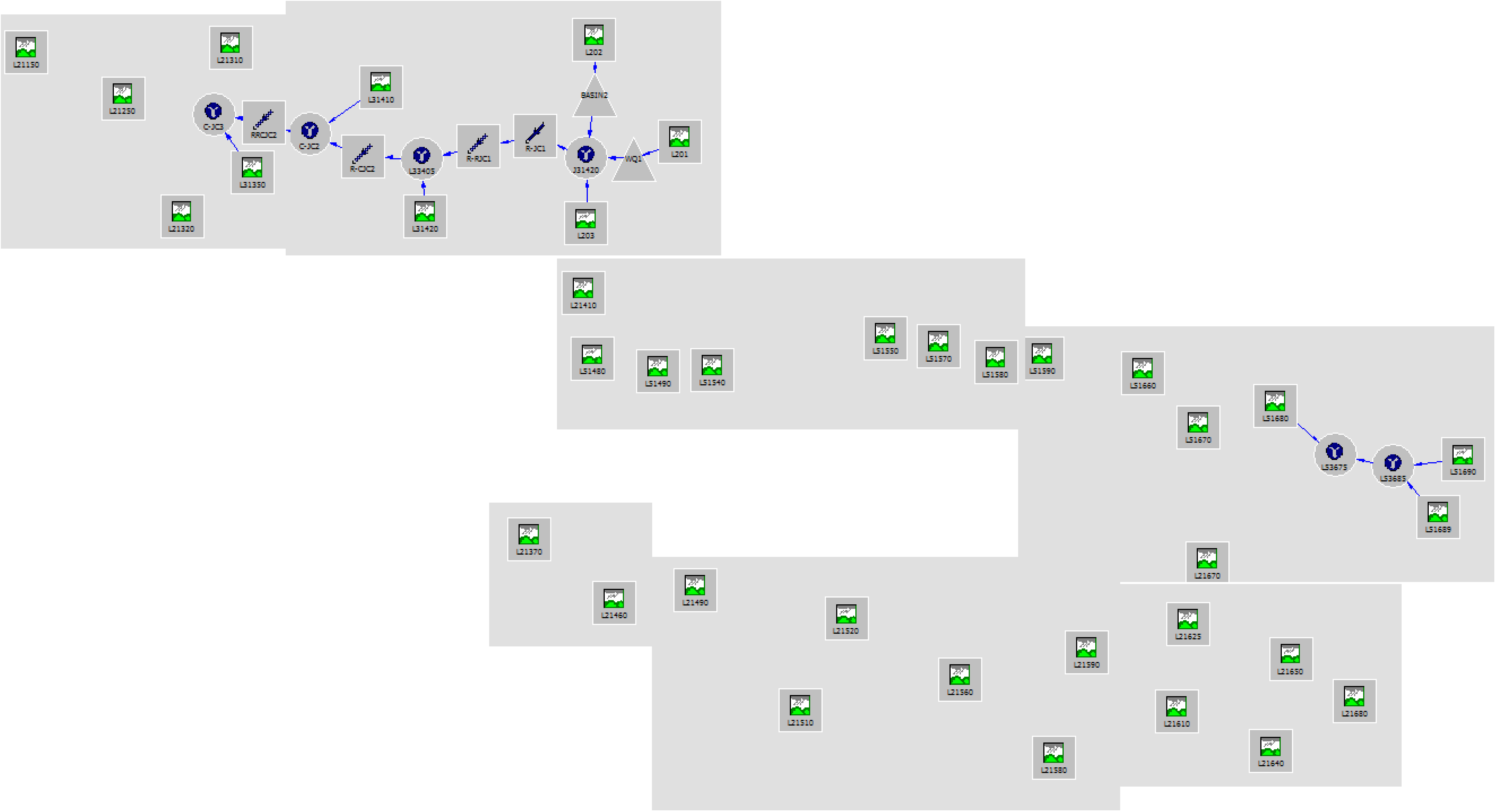


Figure 4-3a. Upper Laguna Creek SacCalc Layout

Figure 4-3b. Lower Laguna and Whitehouse Creeks SacCalc Layout



Lower Laguna and Whitehouse Creeks
HEC-RAS Layout

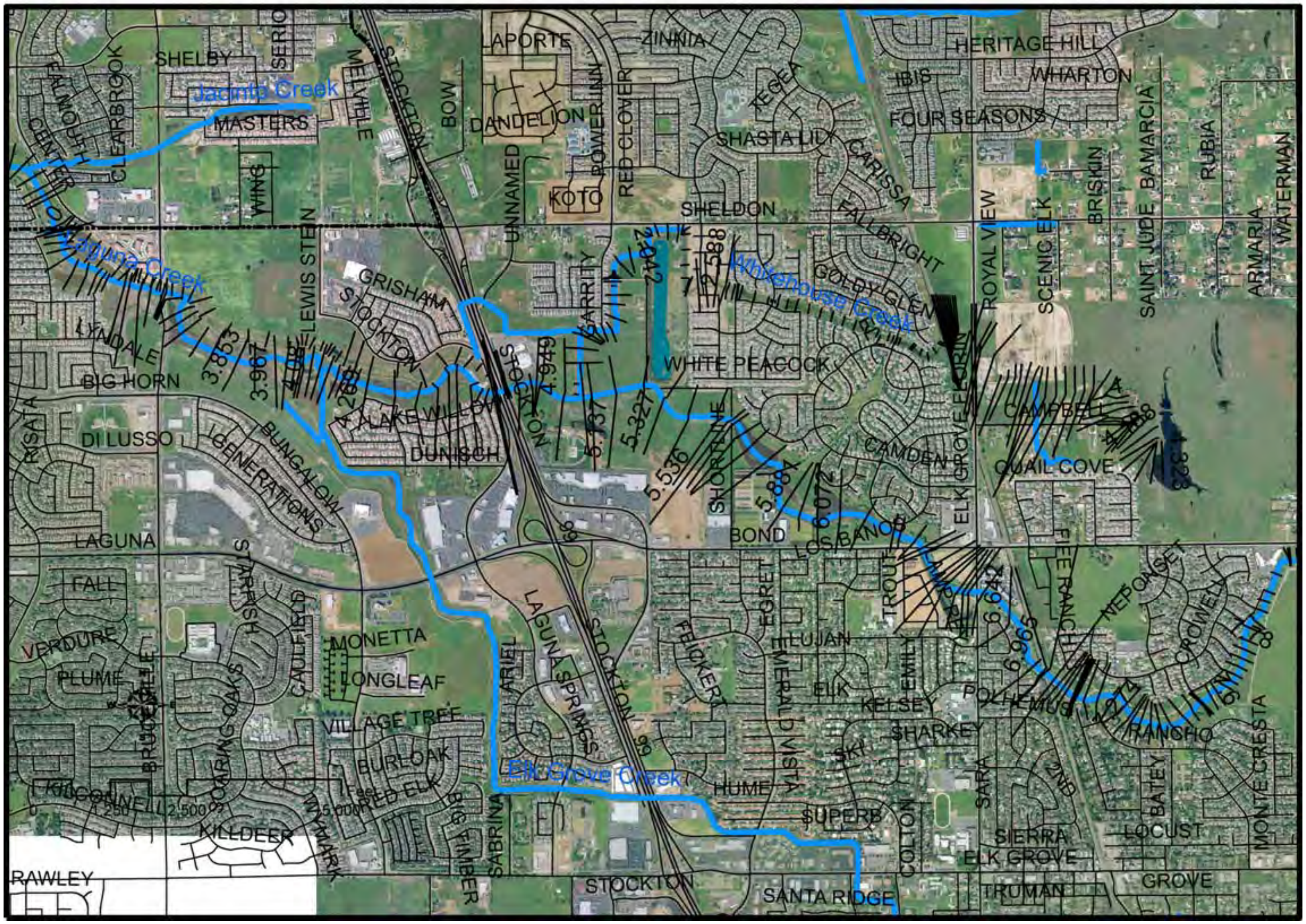


Figure 4-4a. Lower Laguna Creek and Whitehouse Creek HEC-RAS Layout

This shows general location of cross sections, this is not a georeferenced model.

Upper Laguna Creek
HEC-RAS Layout

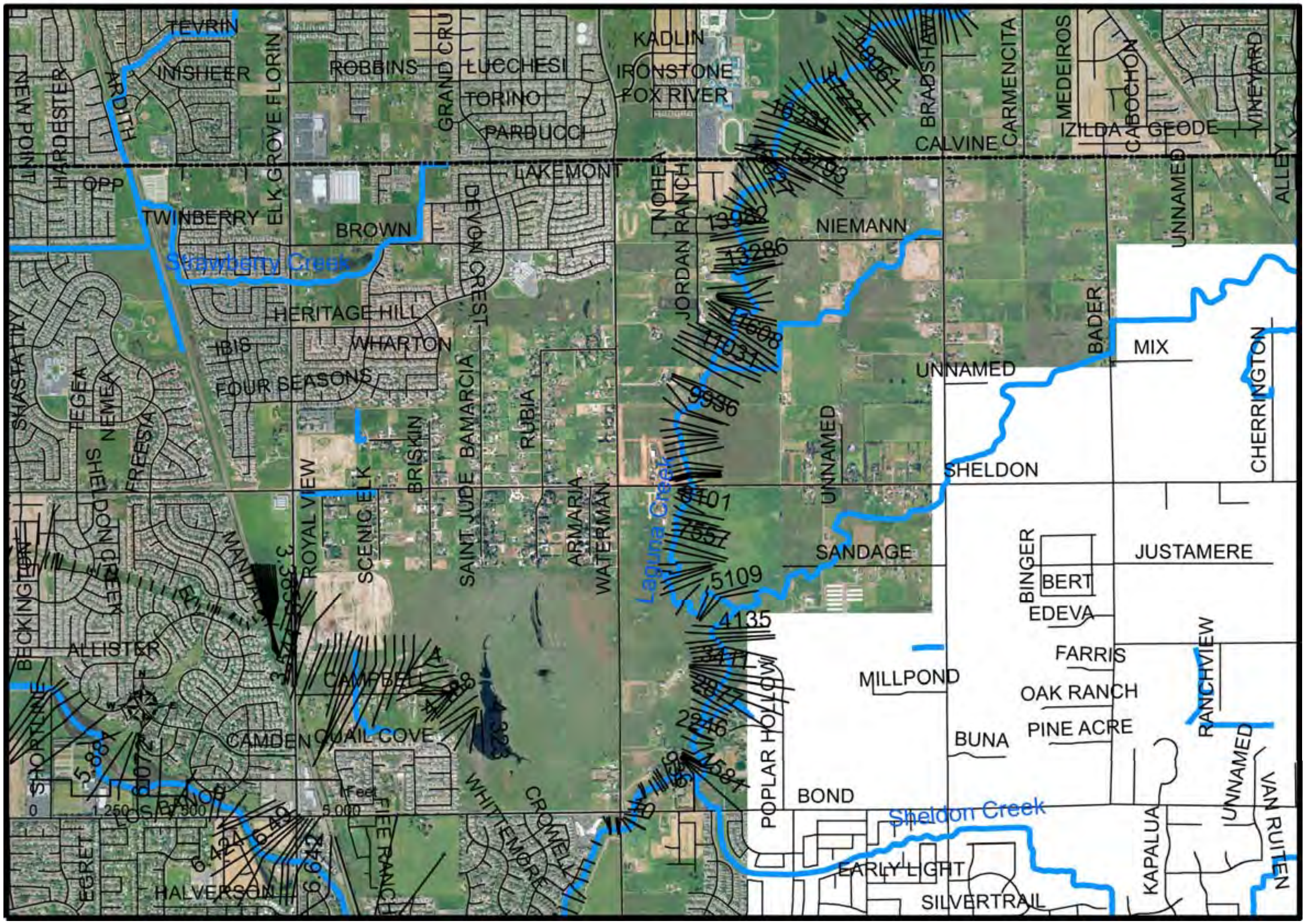


Figure 4-4b.Upper Laguna Creek HEC-RAS Layout

This shows general location of cross sections, this is not a georeferenced model.

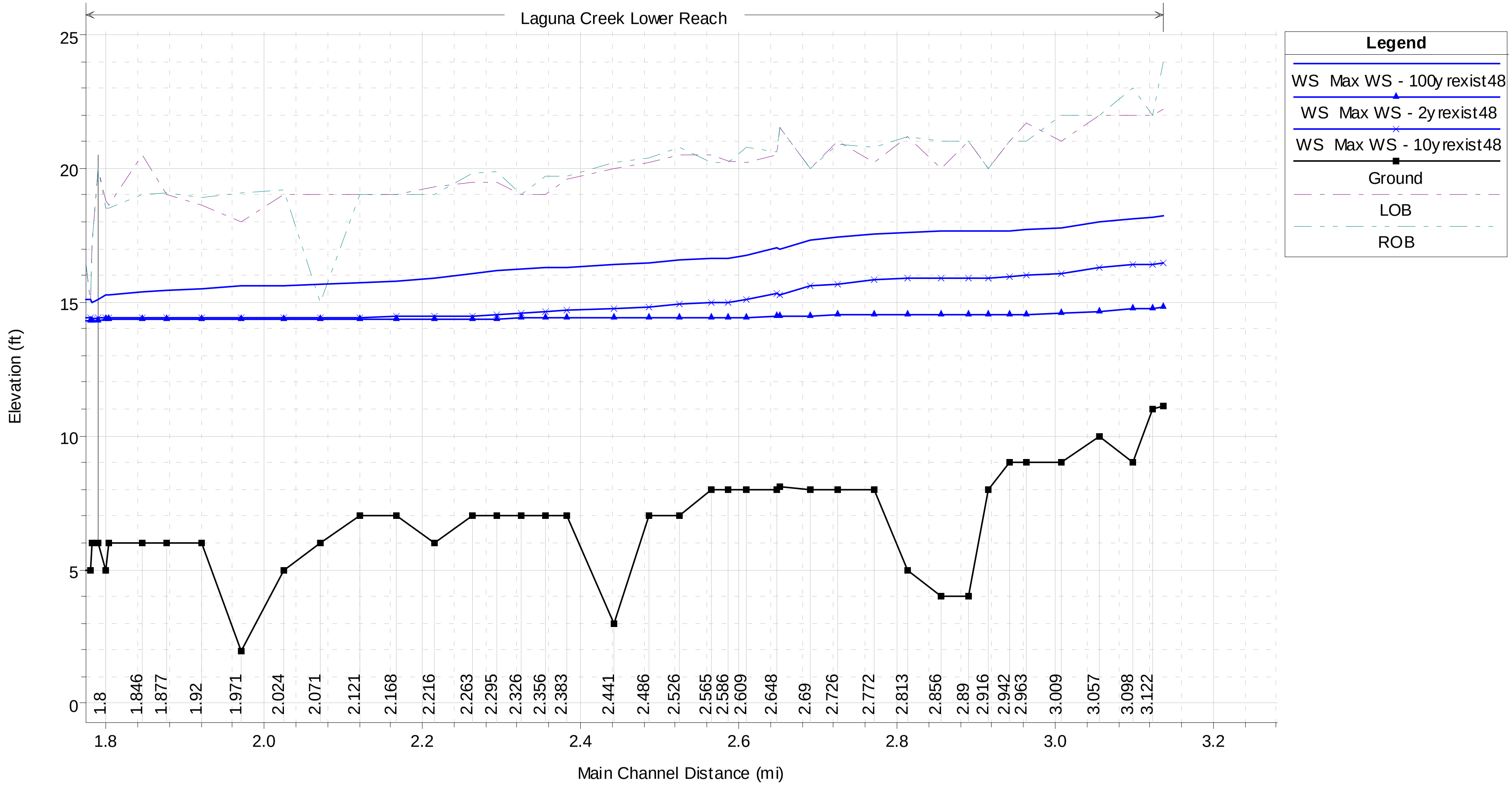


Figure 4-5a. Laguna Creek Lower Reach Sta. 1.8 to 3.12

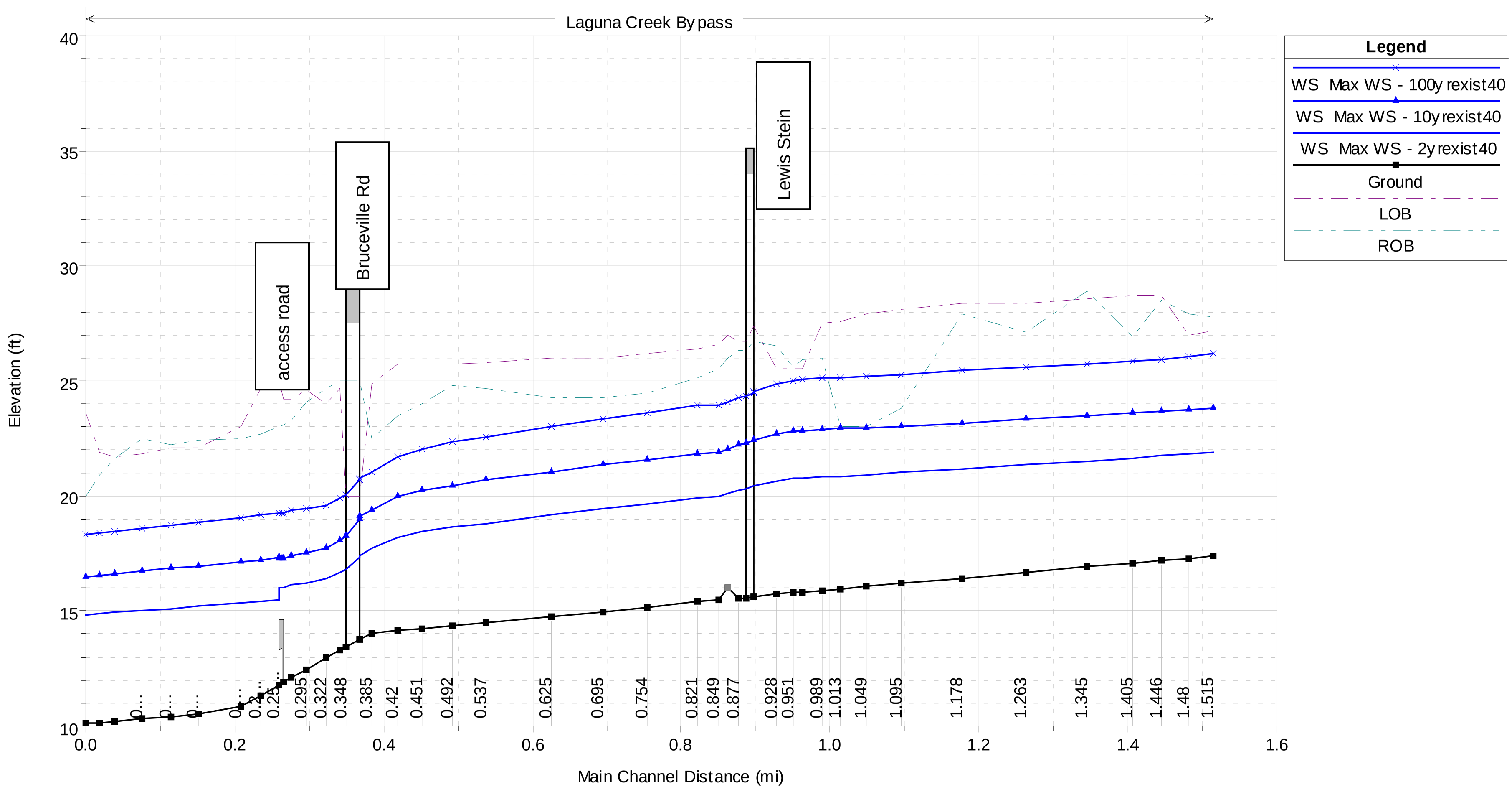


Figure 4-5b. Laguna Creek Bypass Sta. 0.0 to 1.515

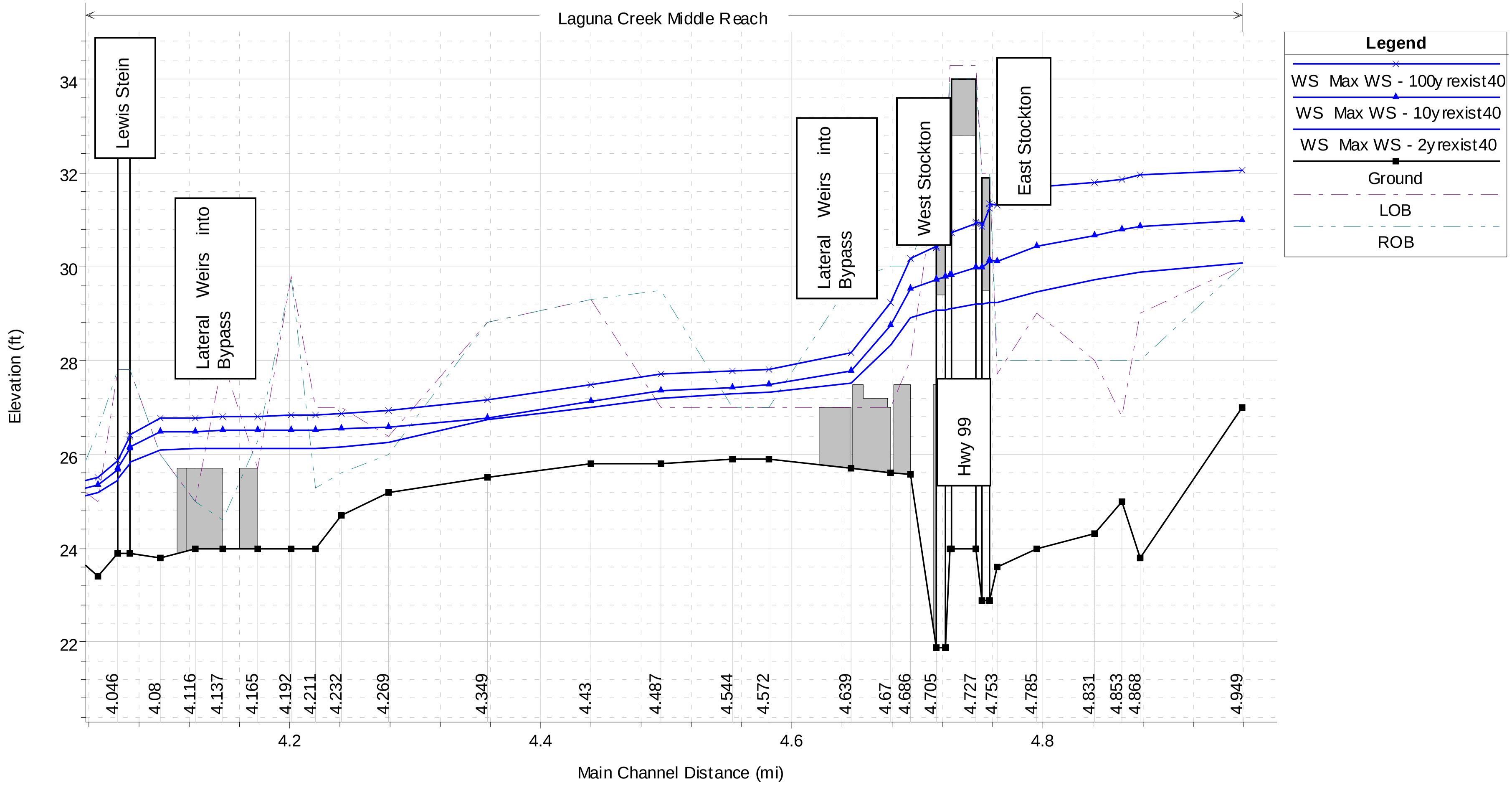


Figure 4-5c. Laguna Creek Middle Reach Sta. 4.406 to 4.948

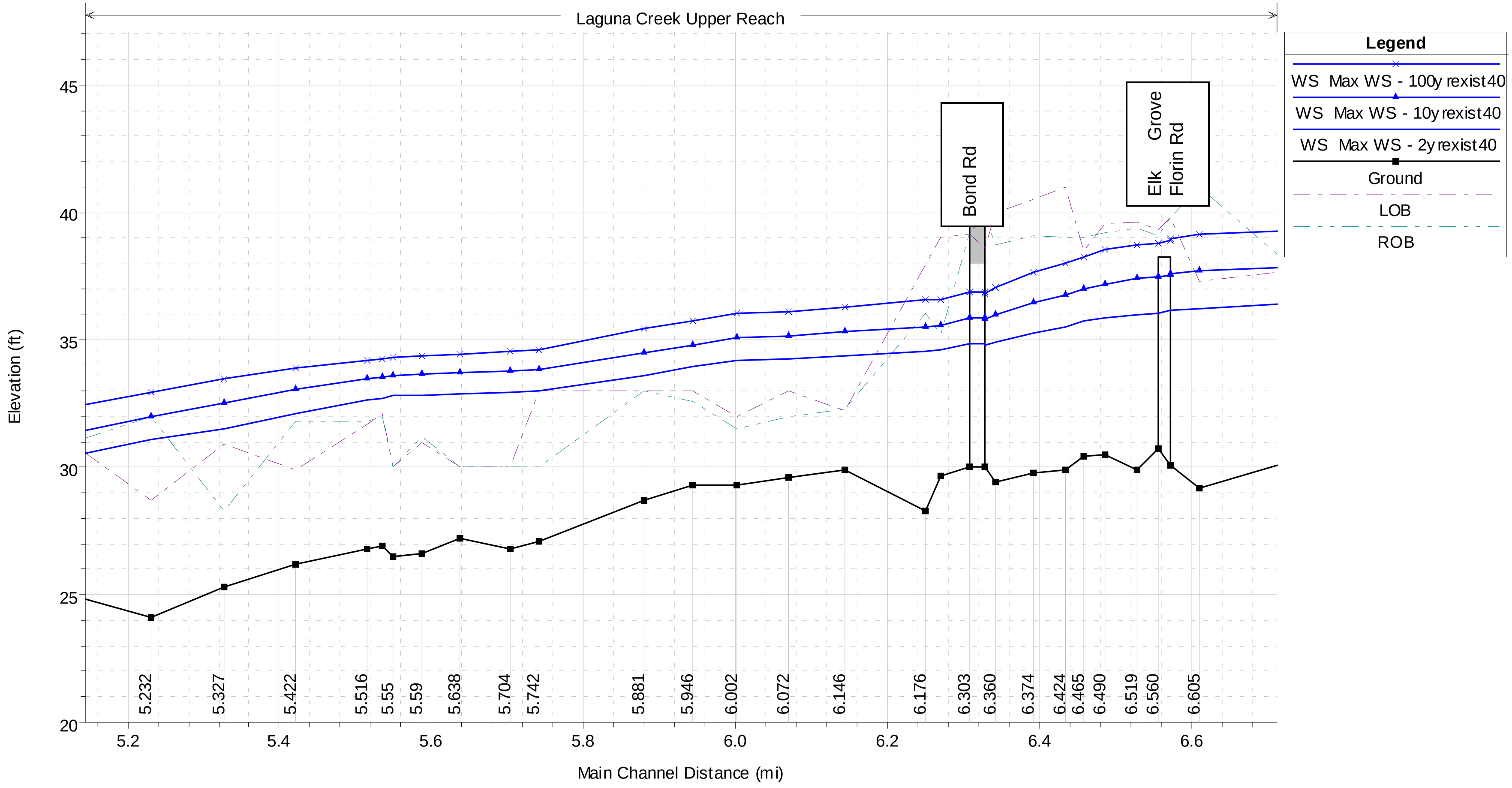


Figure 4-5d. Laguna Creek Upper Reach Sta. 5.232 to 6.605

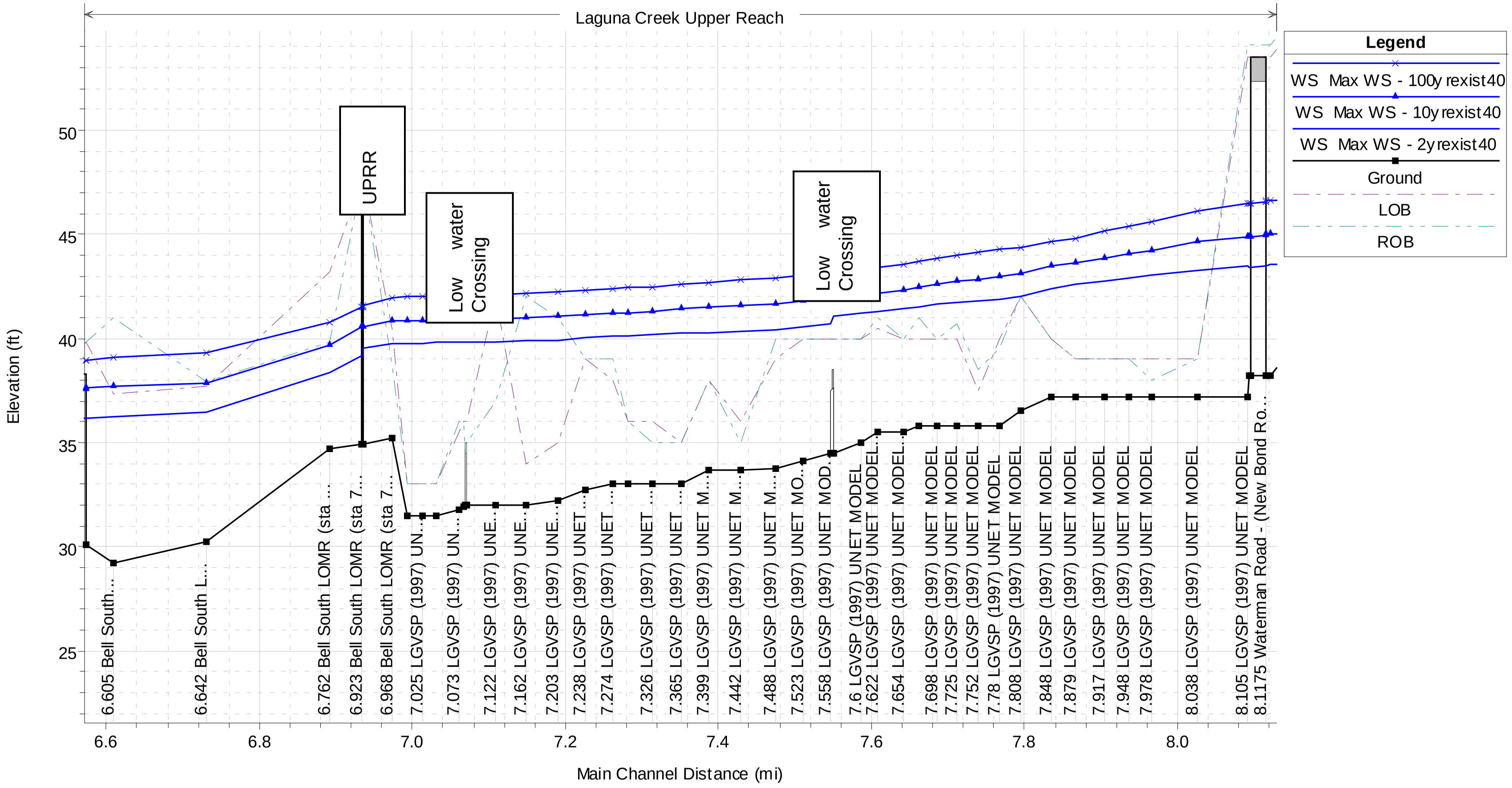


Figure 4-5e. Laguna Creek Upper Reach Sta. 6.605 to 8.1175

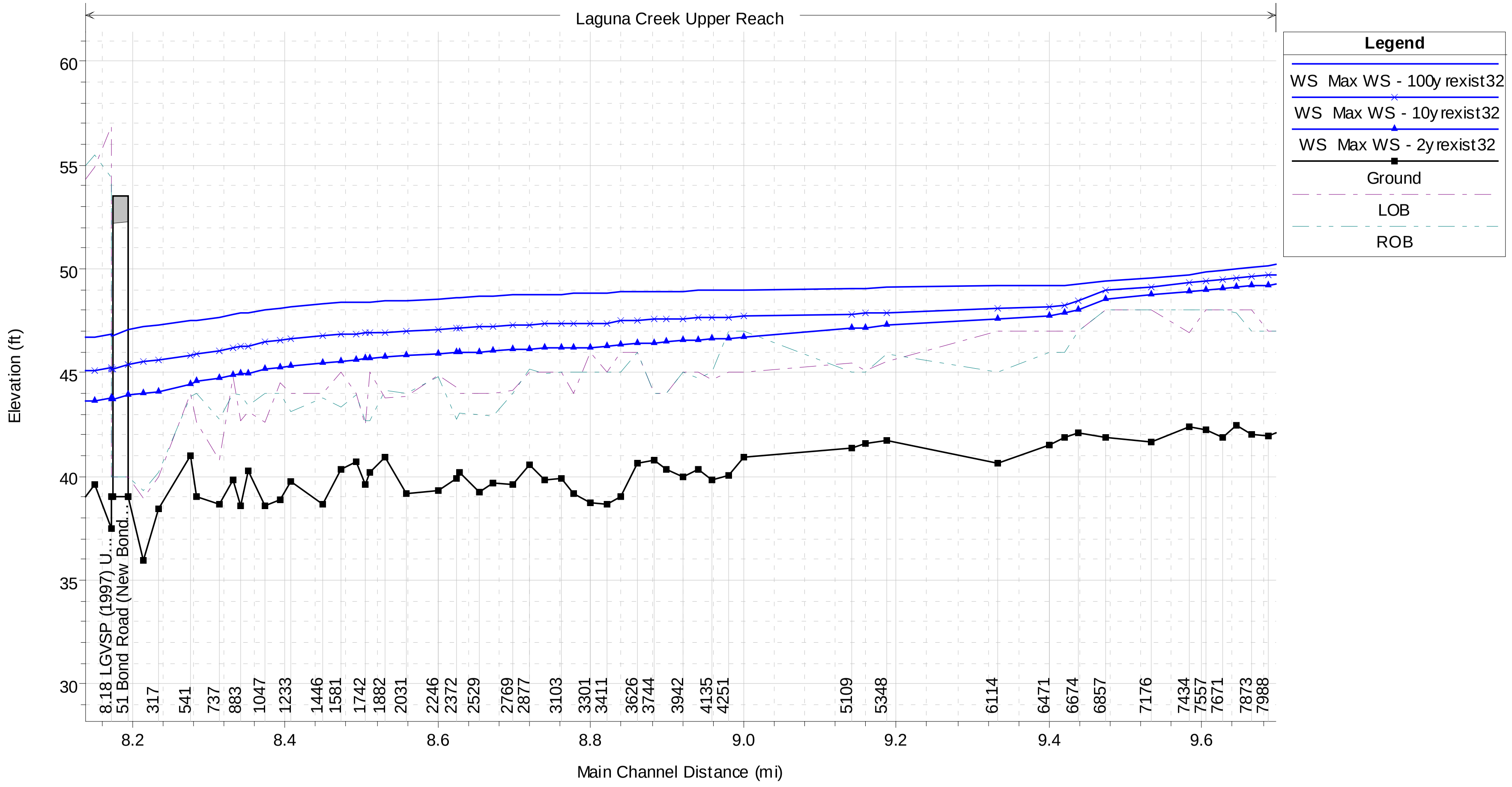


Figure 4-5f. Laguna Creek Upper Reach Sta. 8.181 to 7988

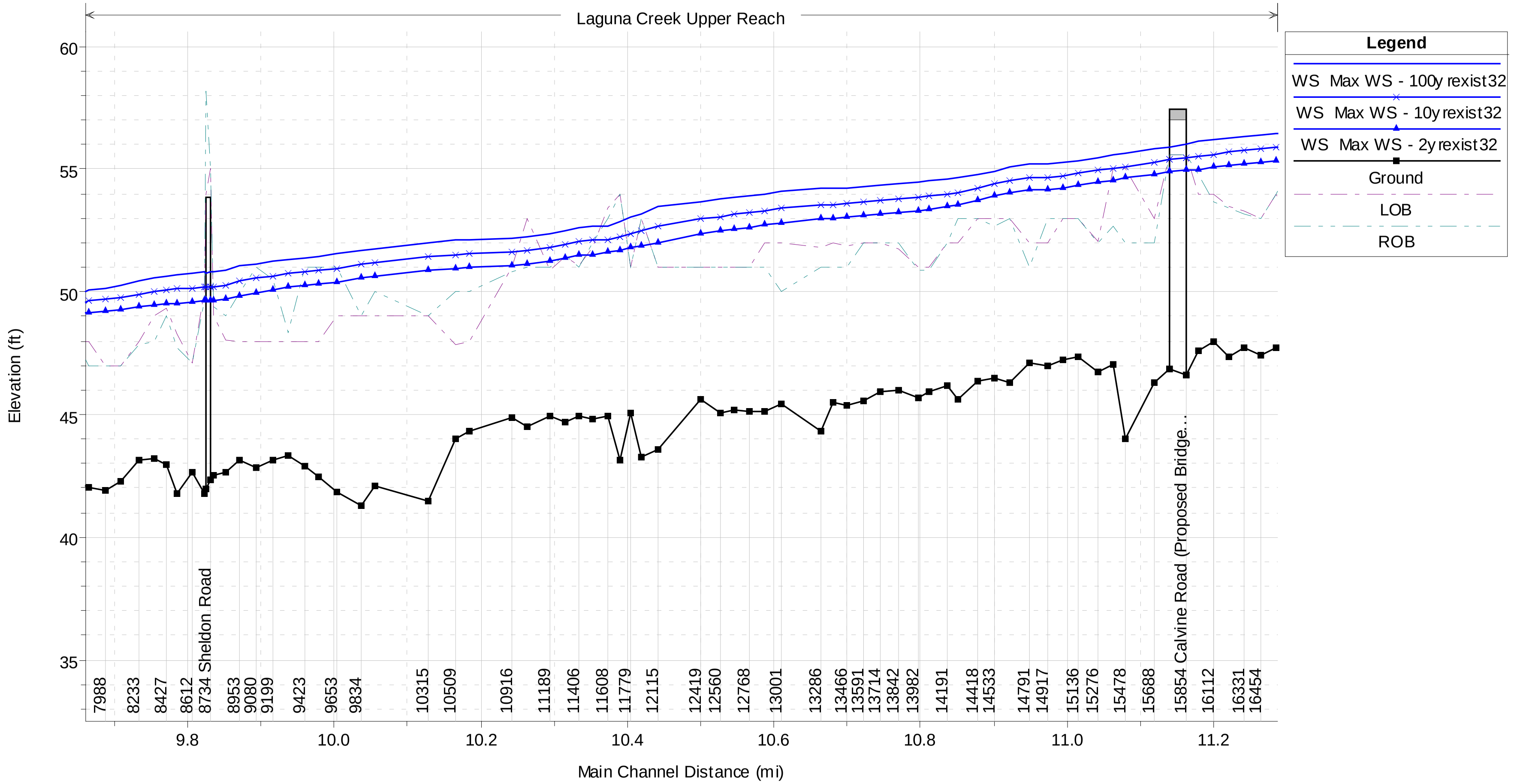


Figure 4-5g. Laguna Creek Upper Reach Sta. 7988 to 16454

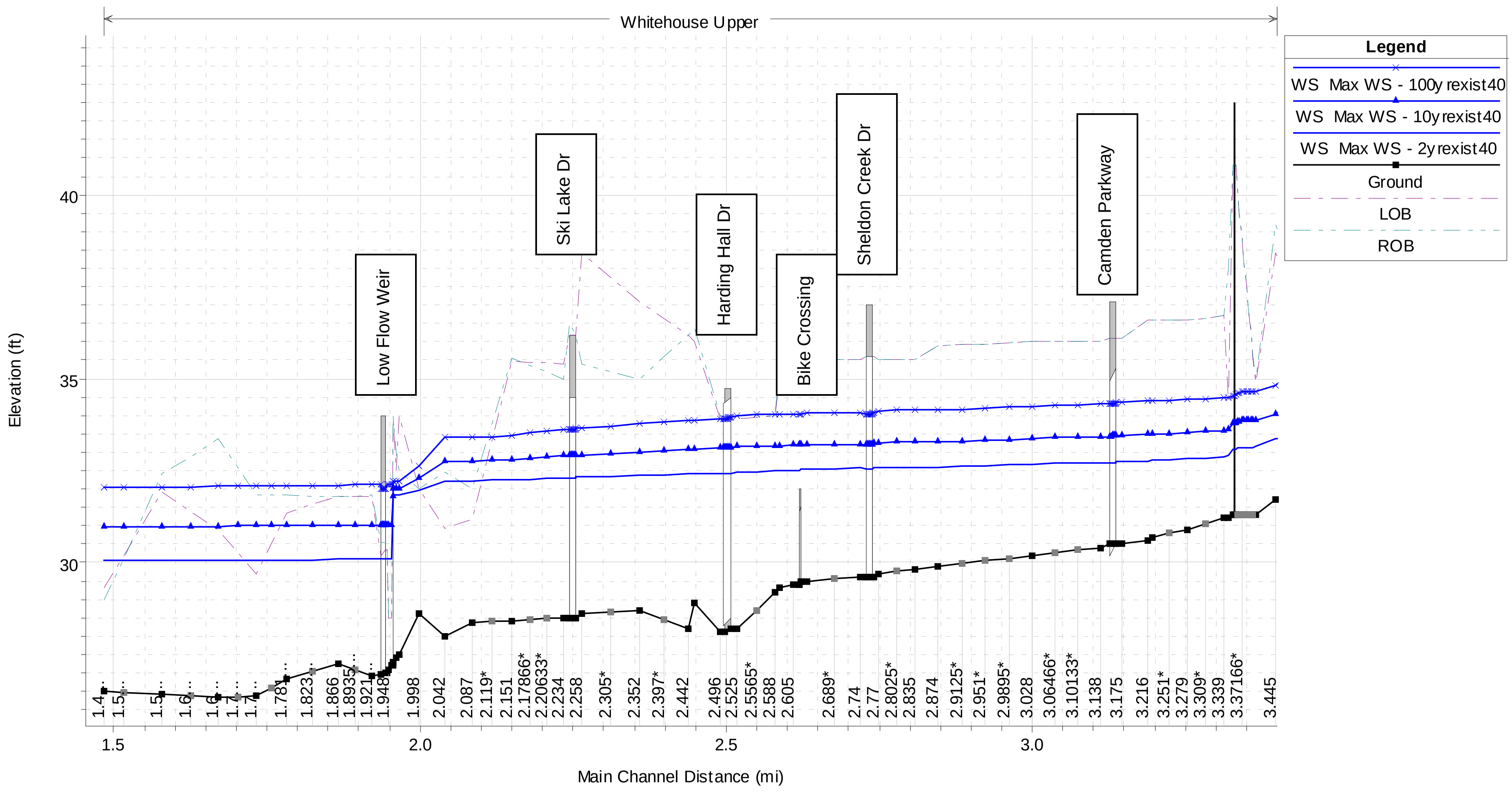


Figure 4-5h. Whitehouse Creek Sta. 1.4 to 3.445

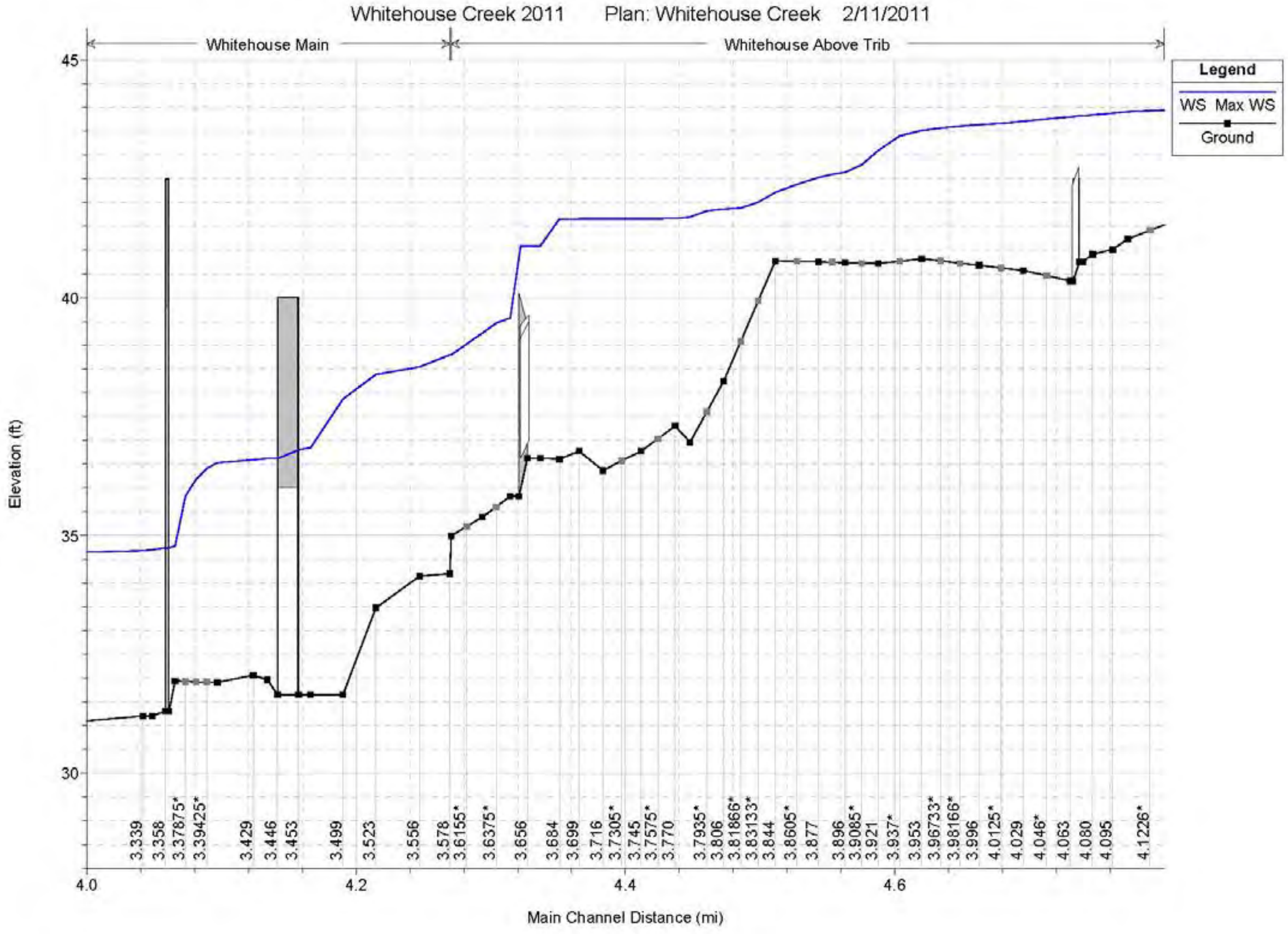


Figure 4-5i. Whitehouse Creek Sta. 3.339 to 4.1226

Bypass Channel Peak Flows
10-Year Exist. = 2,063 cfs
10-Year Ult.= 2,196 cfs
100-Year Exist. = 3,521 cfs
100-Year Ult.= 3,509 cfs

Laguna Creek Peak Flows
10-Year Exist. = 525 cfs
10-Year Ult.= 551 cfs
100-Year Exist. = 763 cfs

FIGURE 4-6a

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800
Feet

NOTES:

Legend

- HEC-RAS Cross Section
- Elevation Contour (NGVD 29)
- ▨ Laguna Buildout 100 Year fp
- ▨ Laguna Exist 100 Year fp
- ☐ City Limit

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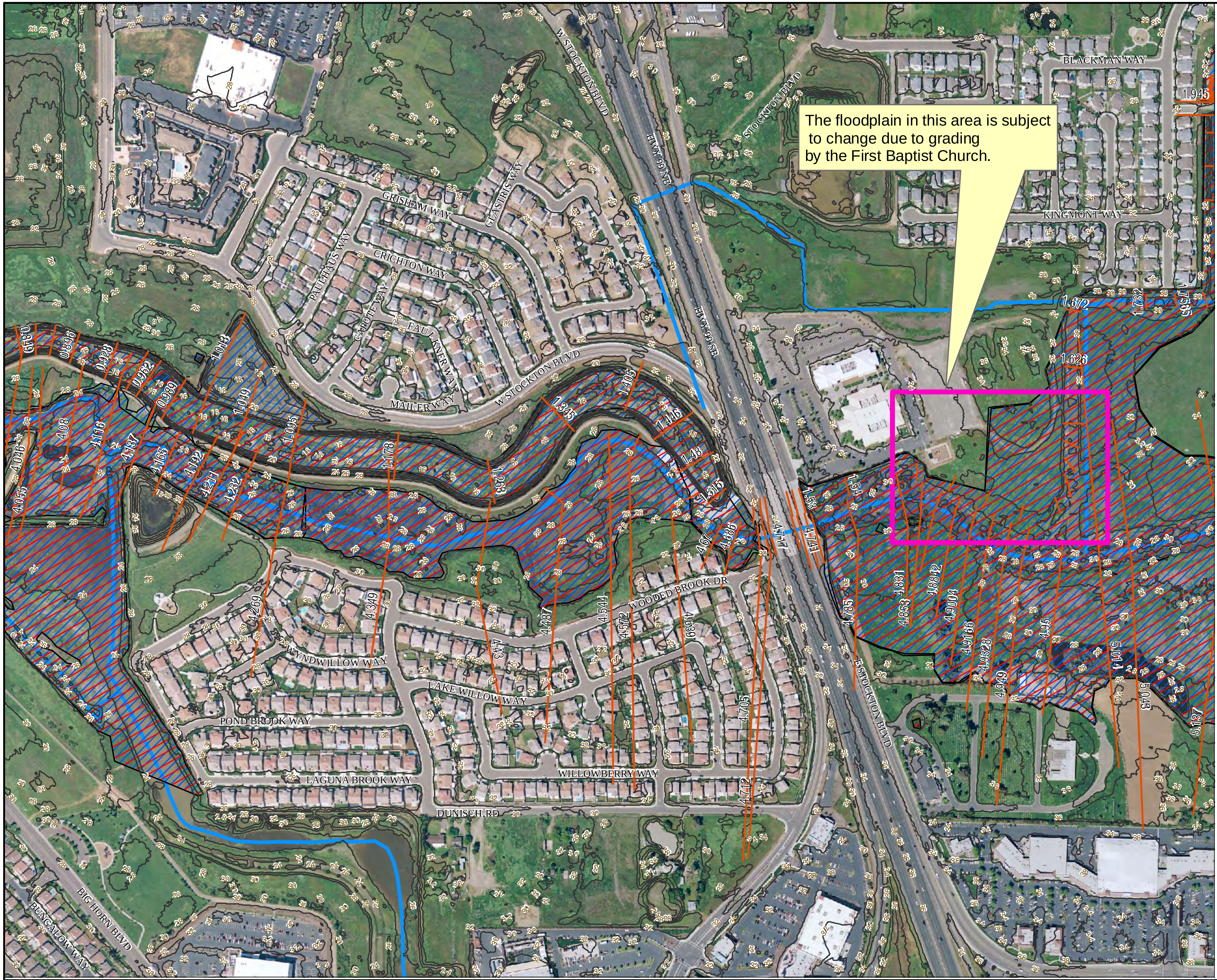

The floodplain in this area is subject to change due to grading by the First Baptist Church.

FIGURE 4-6b

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800
Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Laguna Buildout 100 Year fp
▨	Laguna Exist 100 Year fp
☐	City Limit

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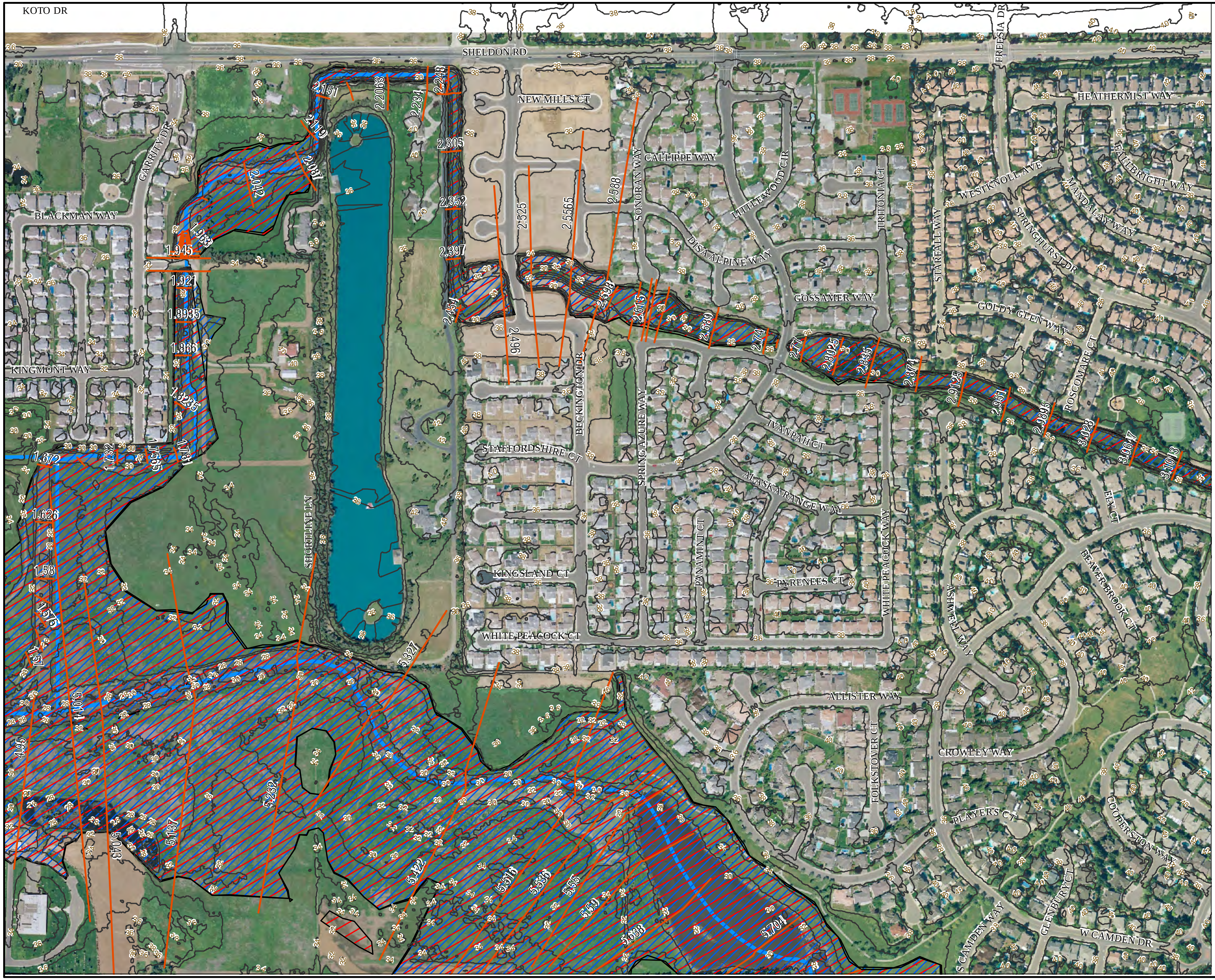

FIGURE 4-6c

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800

Feet

NOTES:

Legend

- HEC-RAS Cross Section
- Elevation Contour (NGVD 29)
- ▨ Laguna Buildout 100 Year fp
- ▨ Laguna Exist 100 Year fp
- ☐ City Limit

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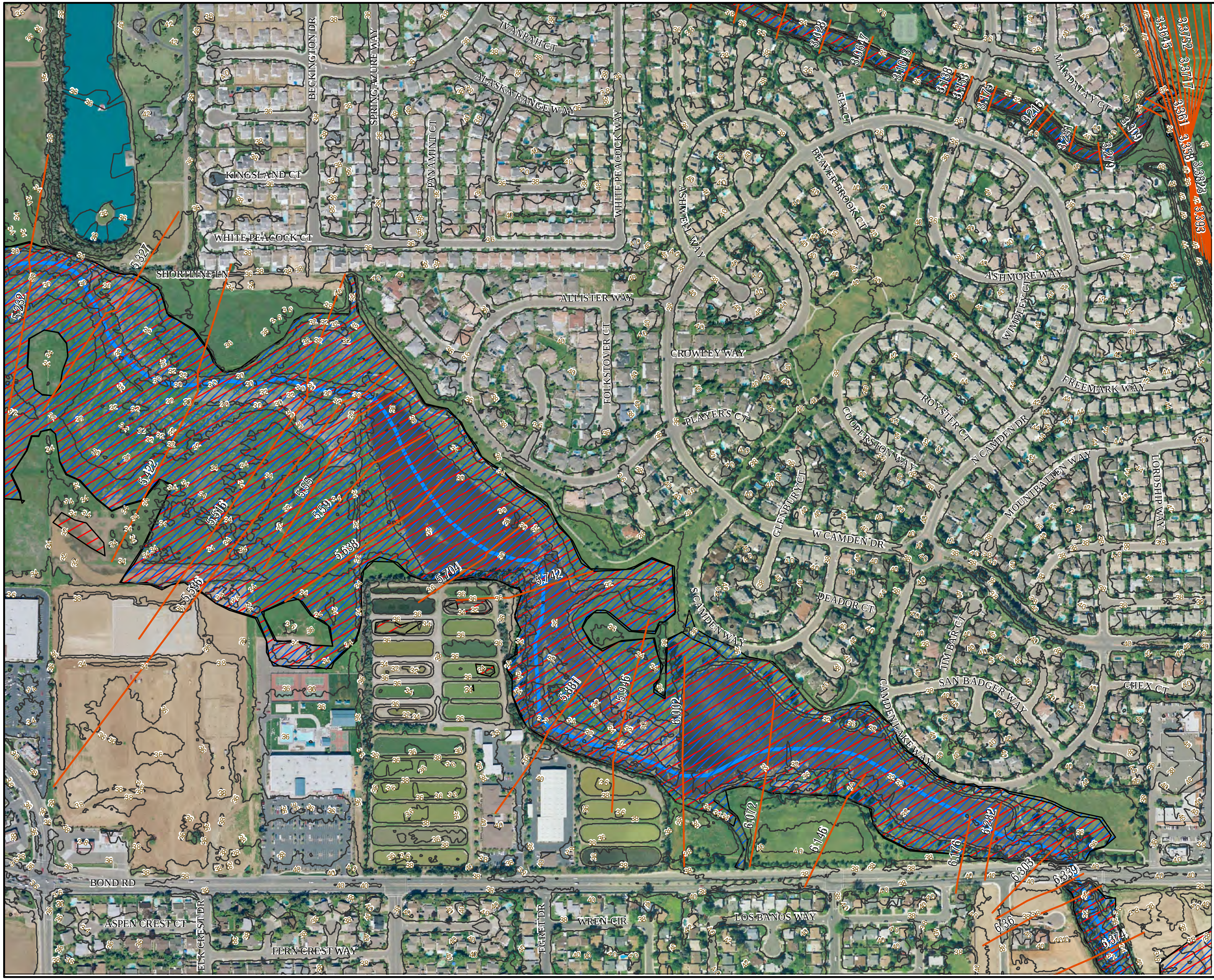

FIGURE 4-6d

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800
Feet

NOTES:

Legend

- HEC-RAS Cross Section
- Elevation Contour (NGVD 29)
- ▨ Laguna Buildout 100 Year fp
- ▨ Laguna Exist 100 Year fp
- ☐ City Limit

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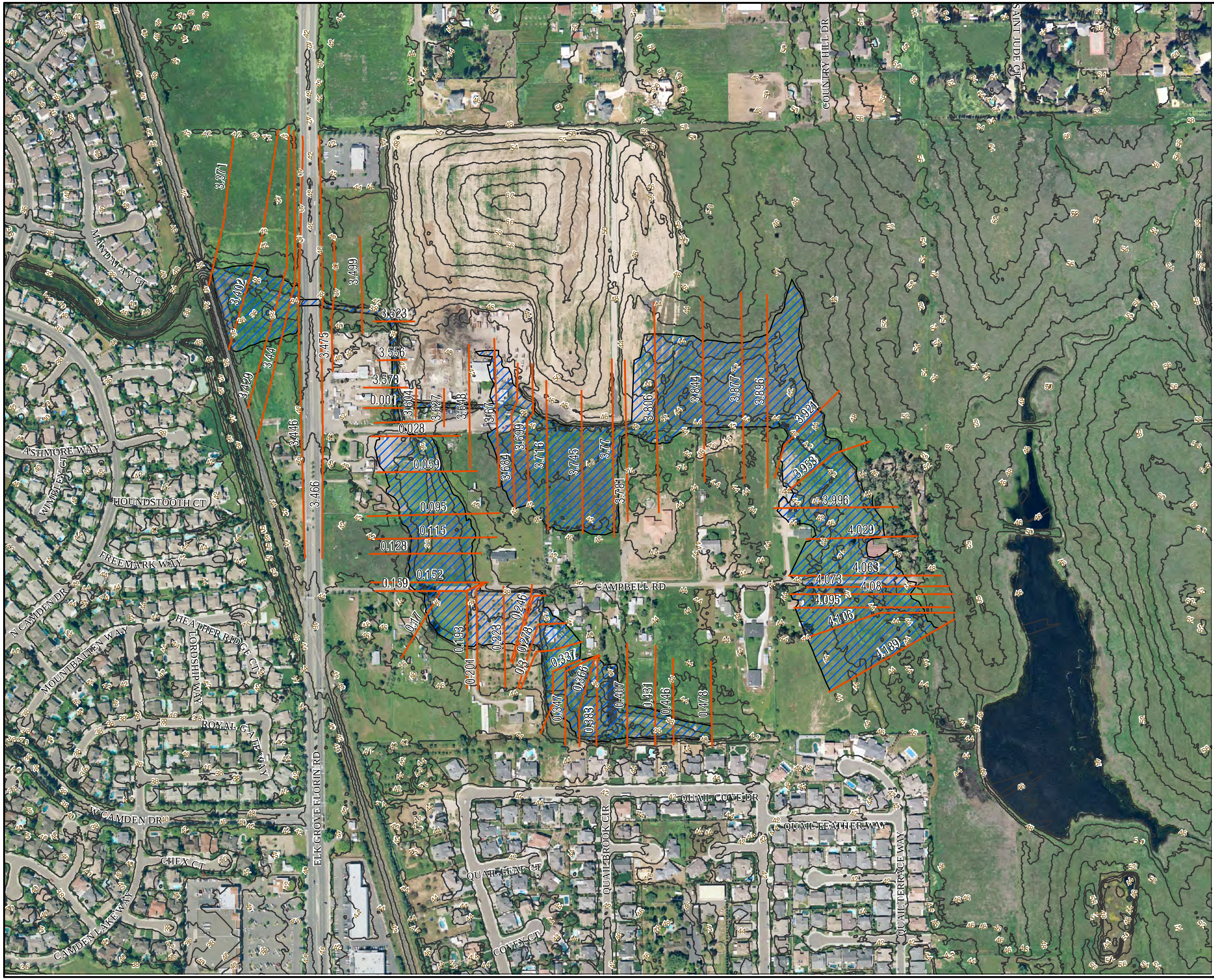

FIGURE 4-6e

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800
Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Laguna Exist 100 Year fp
☐	City Limit

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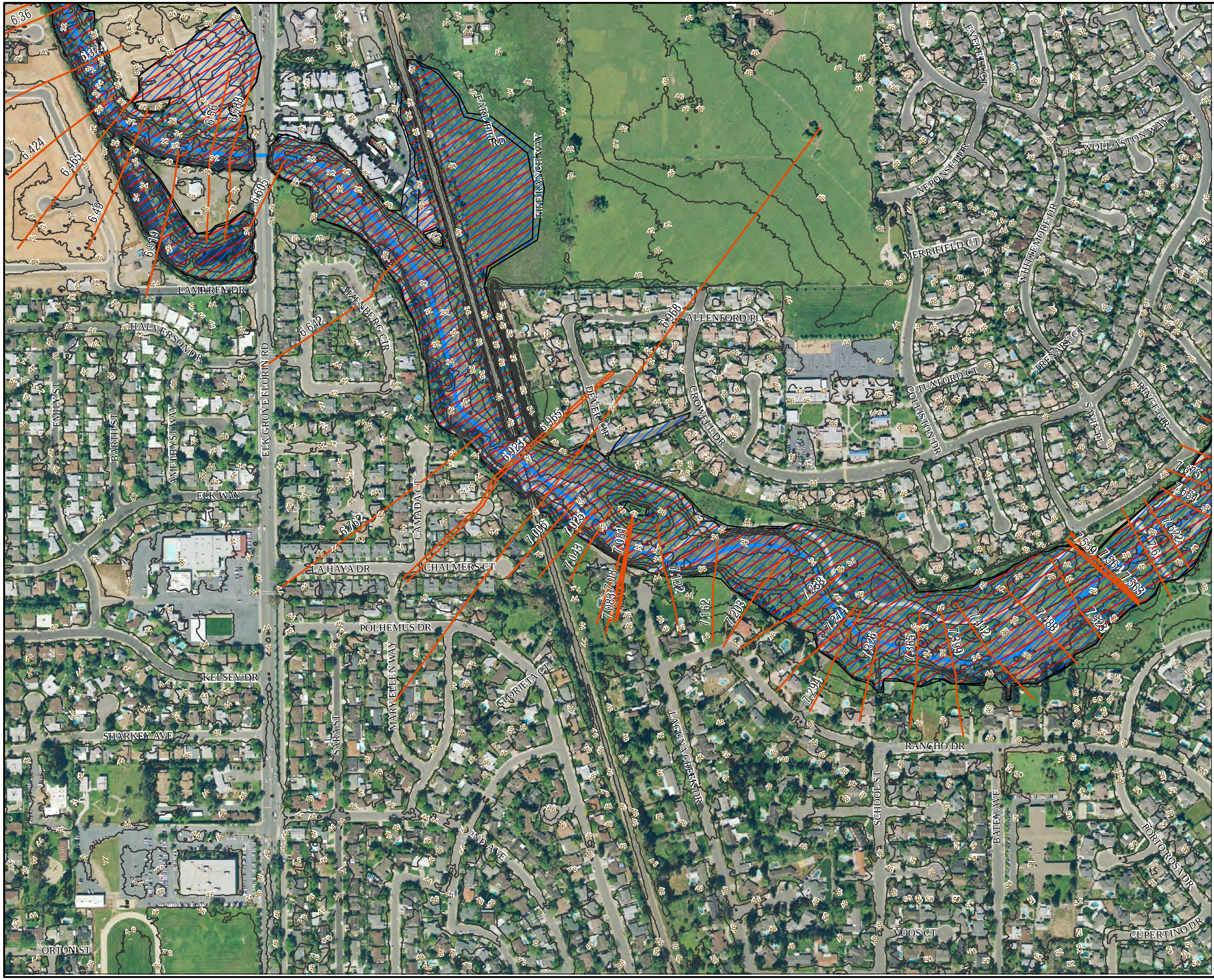

FIGURE 4-6f

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800

Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Laguna Buildout 100 Year fp
▨	Laguna Exist 100 Year fp
☐	City Limit

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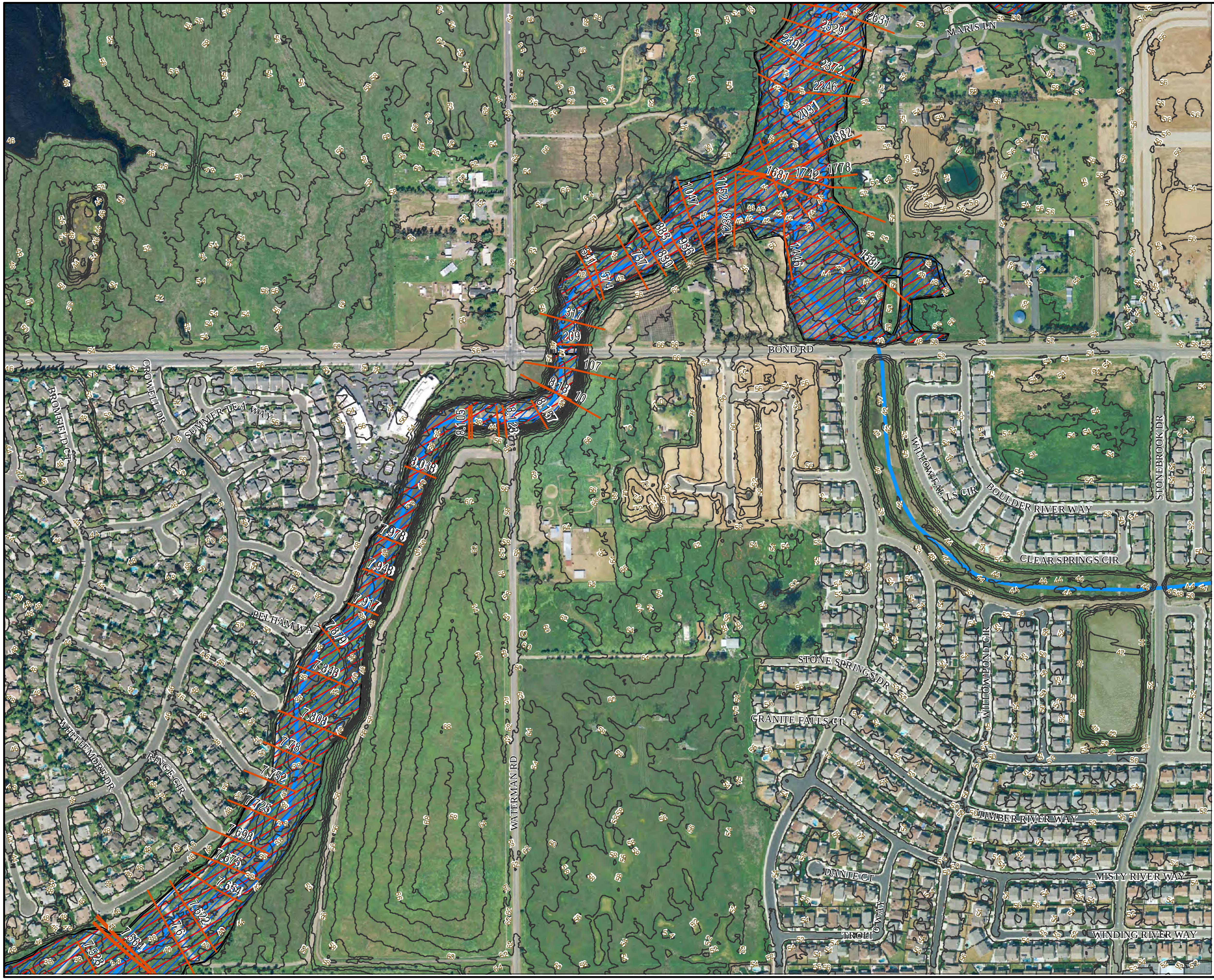

FIGURE 4-6g

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800

Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Laguna Buildout 100 Year fp
▨	Laguna Exist 100 Year fp
☐	City Limit

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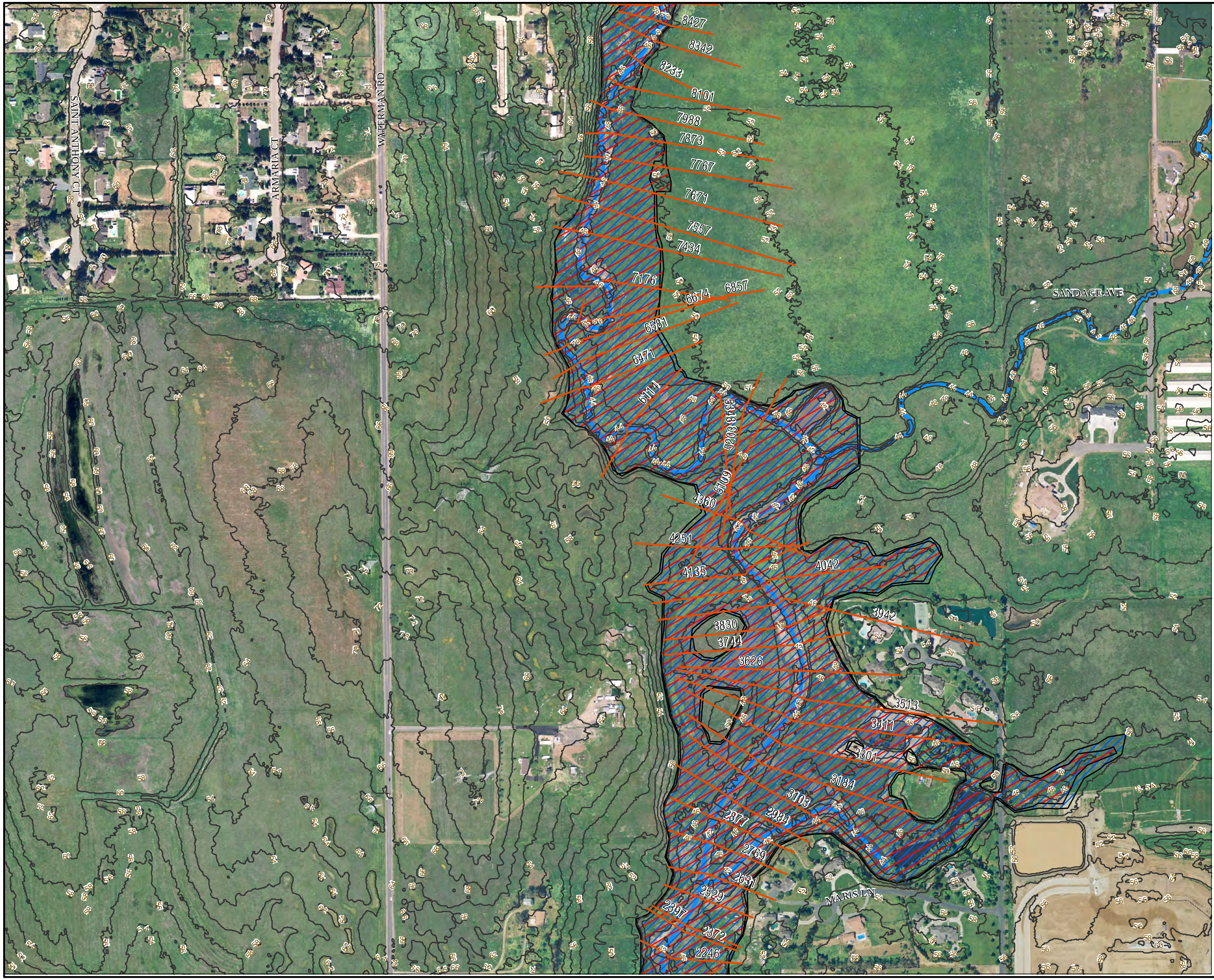

FIGURE 4-6h

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800

Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Laguna Buildout 100 Year fp
▨	Laguna Exist 100 Year fp
⬚	City Limit

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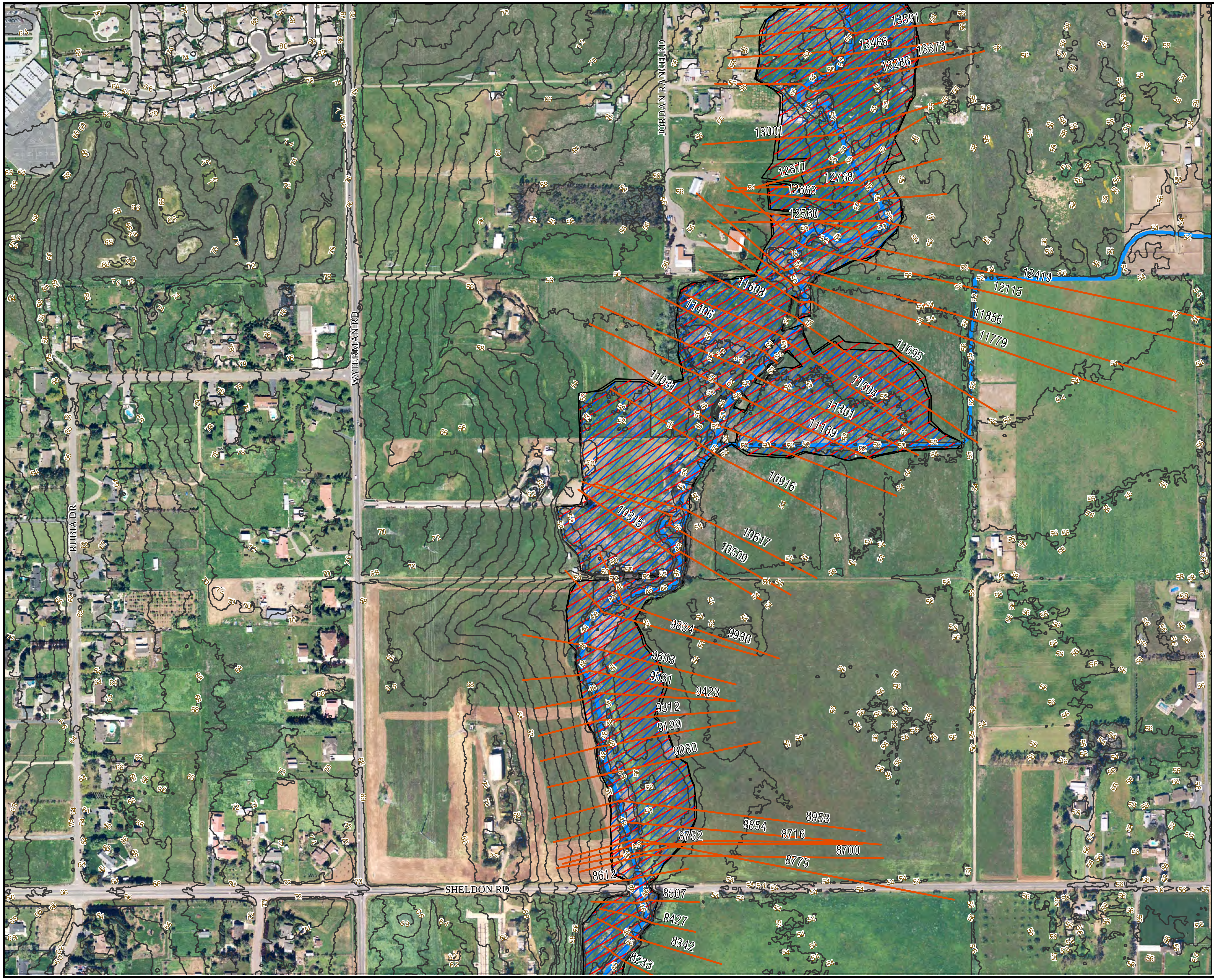

FIGURE 4-6i

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800

Feet

NOTES:

Legend

- HEC-RAS Cross Section
- Elevation Contour (NGVD 29)
- ▨ Laguna Buildout 100 Year fp
- ▨ Laguna Exist 100 Year fp
- ☐ City Limit

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Offsite Peak Flows
10-Year Exist. = 893 cfs
10-Year Ult.= 928 cfs
100-Year Exist. = 1,271 cfs
100-Year Ult.= 1,322 cfs

FIGURE 4-6j

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800
Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Laguna Buildout 100 Year fp
▨	Laguna Exist 100 Year fp
▭	City Limit

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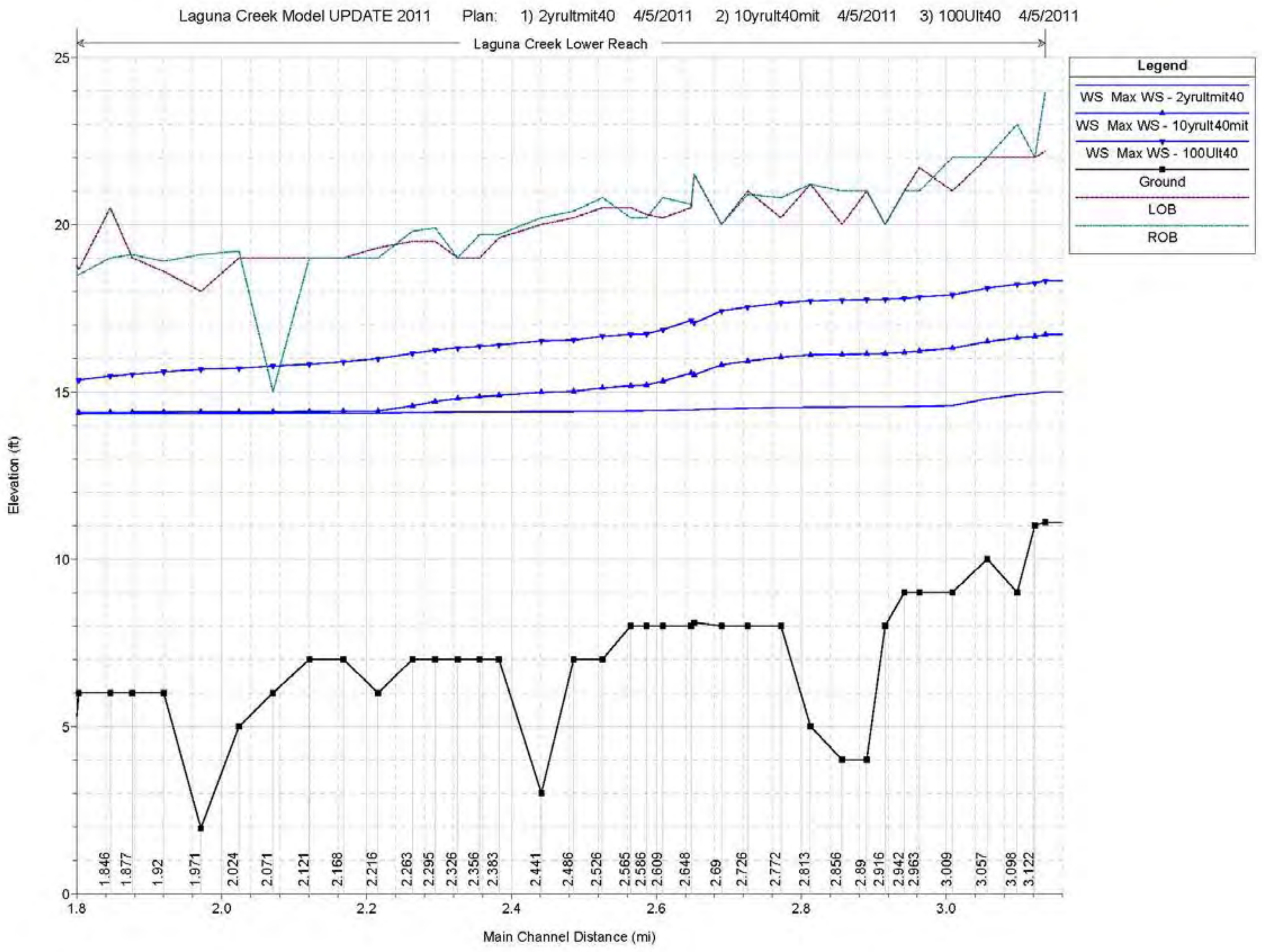

Figure 4-7a. Laguna Creek Lower Reach Sta. 1.846 to 3.122

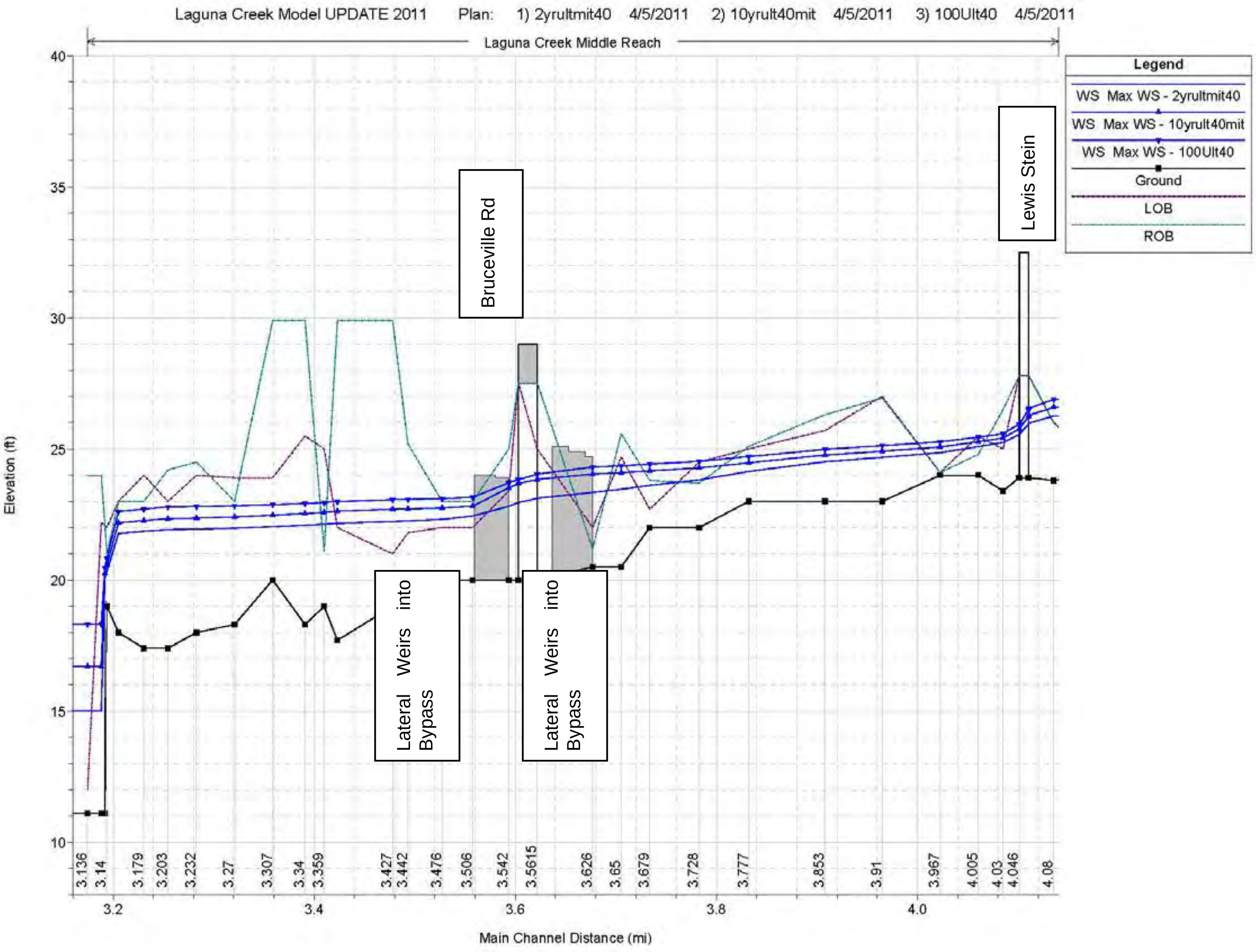

Figure 4-7b. Laguna Creek Upper Reach Sta. 3.136 to 4.08

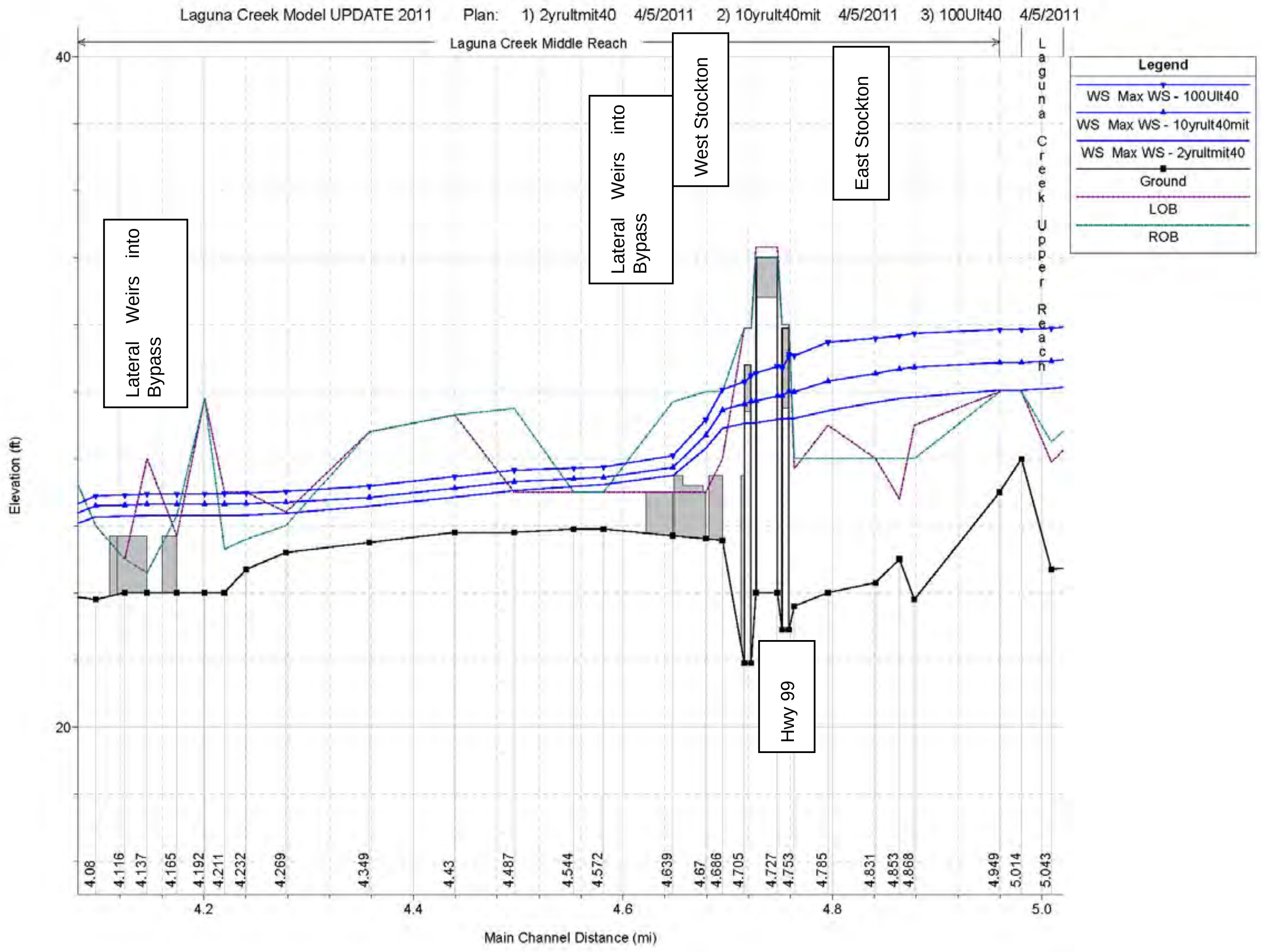

Figure 4-7c. Laguna Creek Upper Reach Sta. 4.03 to 5.014

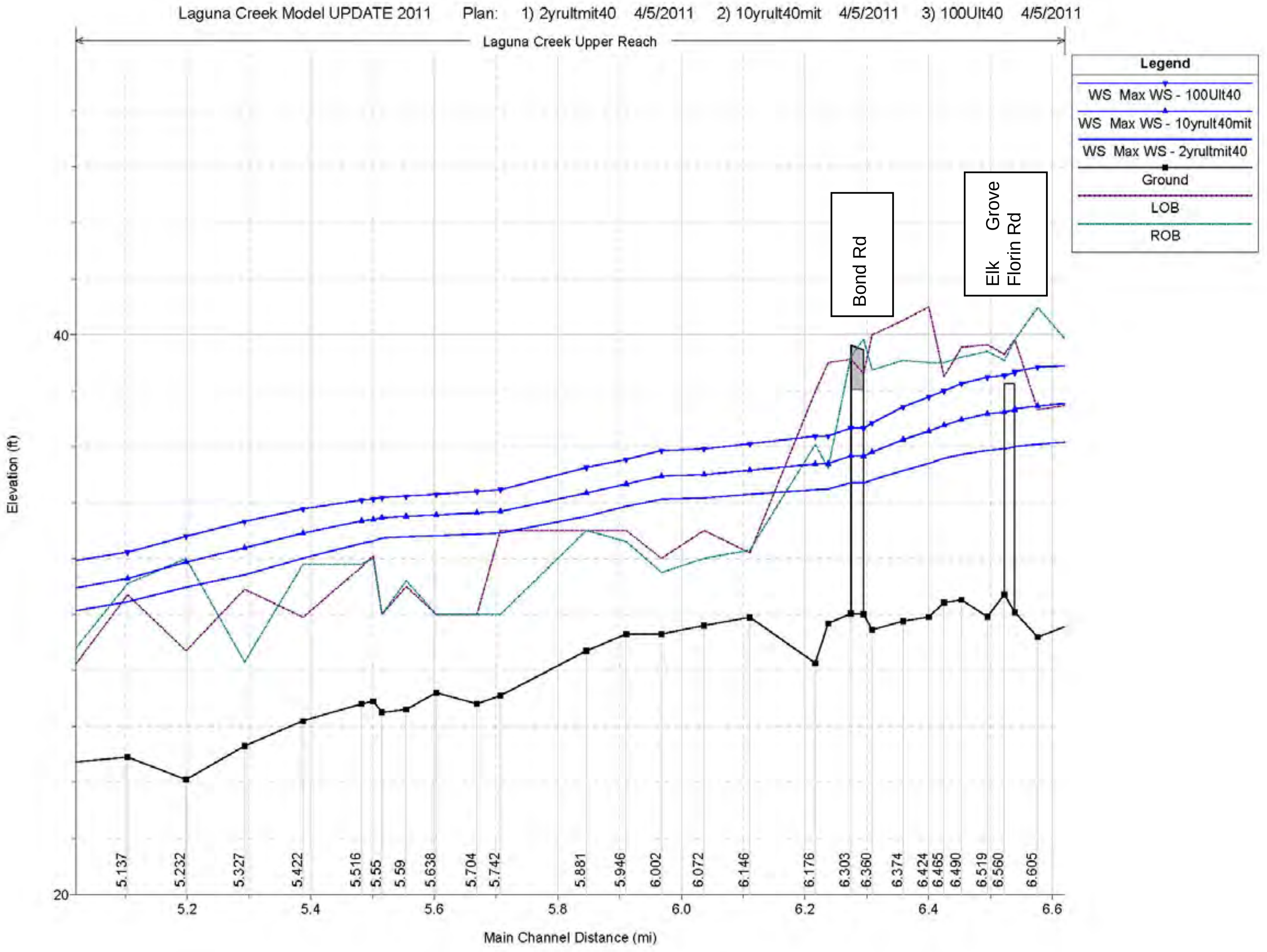

Figure 4-7d. Laguna Creek Upper Reach Sta. 5.014 to 6.605

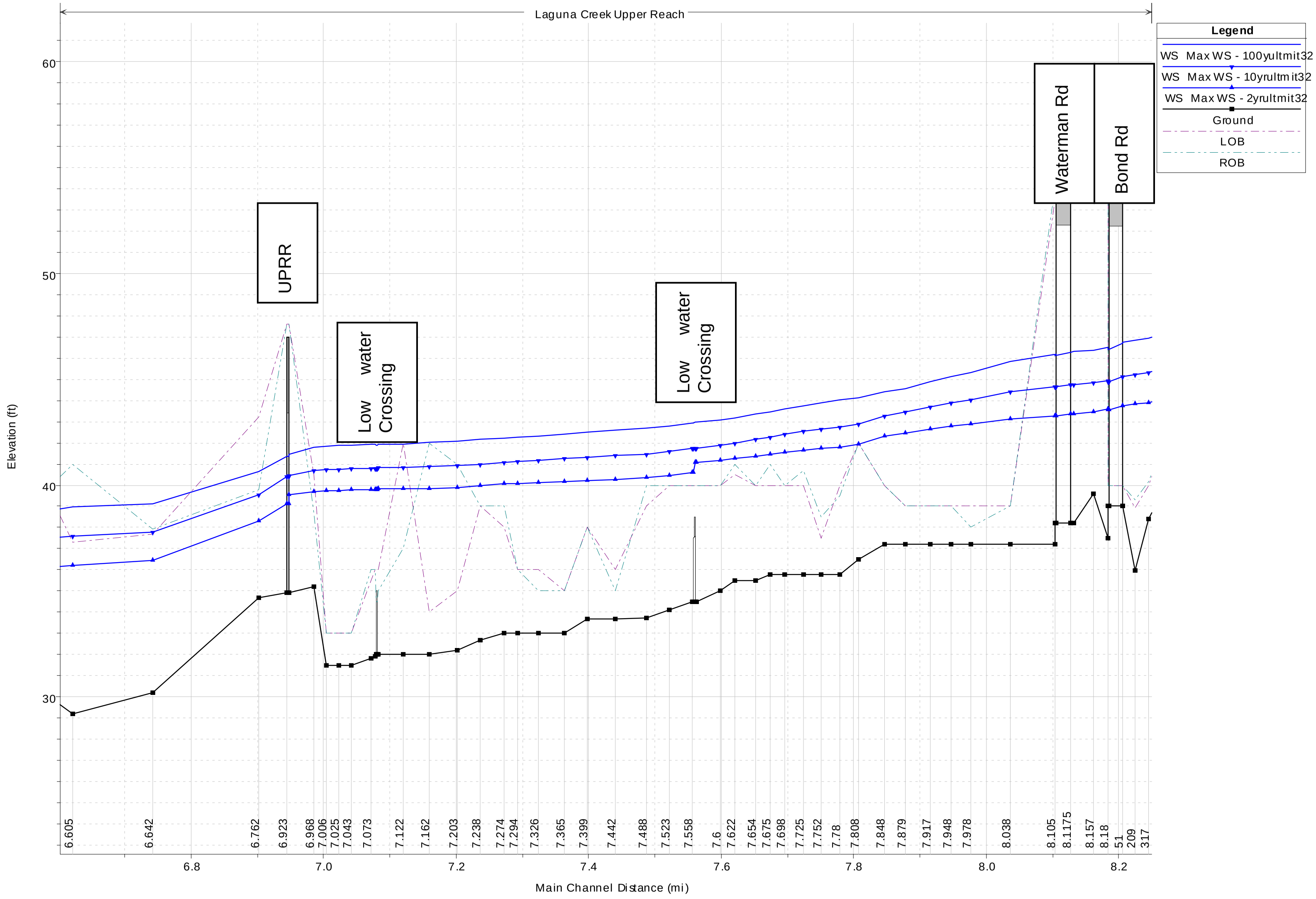

Figure 4-7e. Laguna Creek Upper Reach Sta. 6.605 to 317

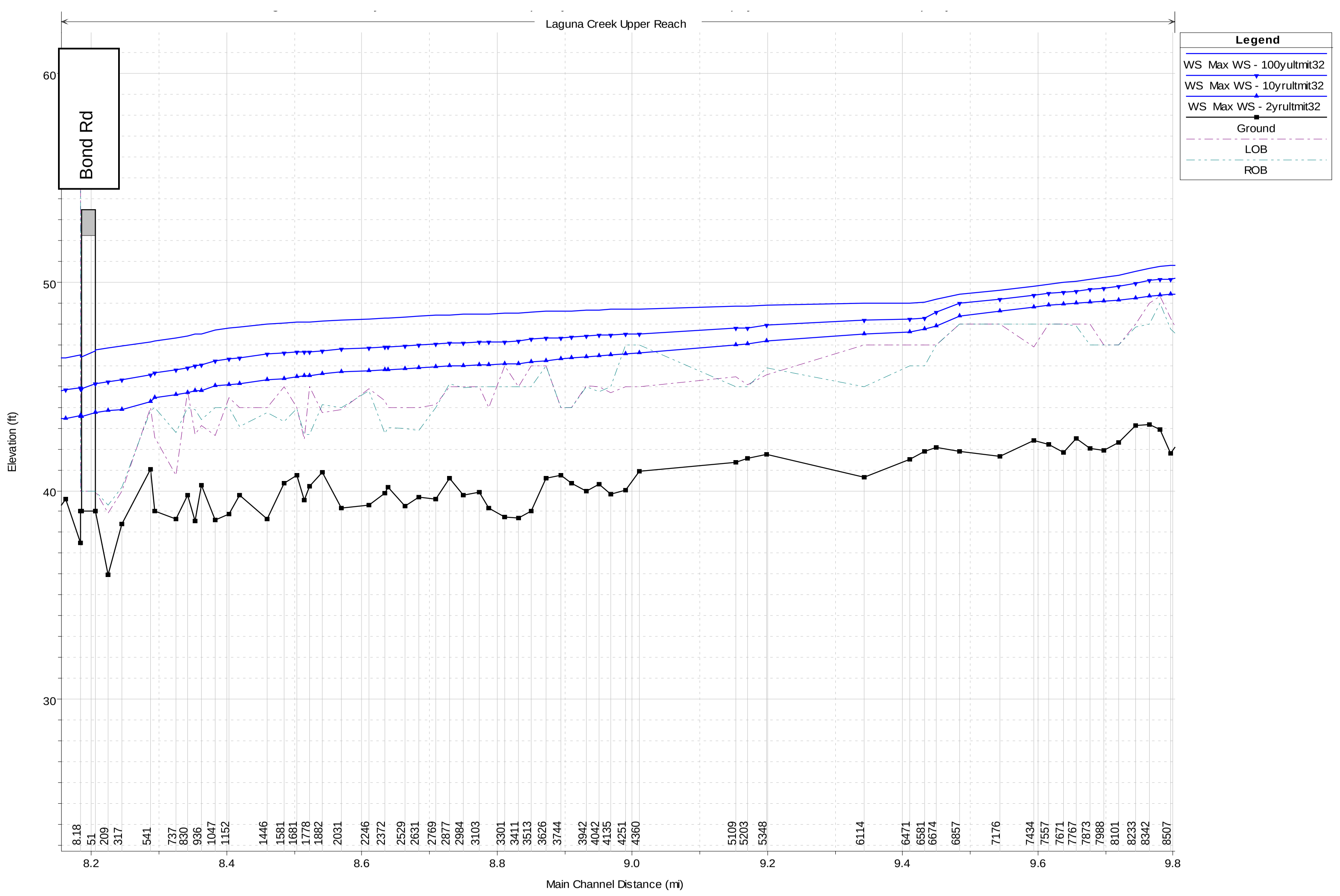

Figure 4-7f. Laguna Creek Upper Reach Sta. 8.18 to 8507

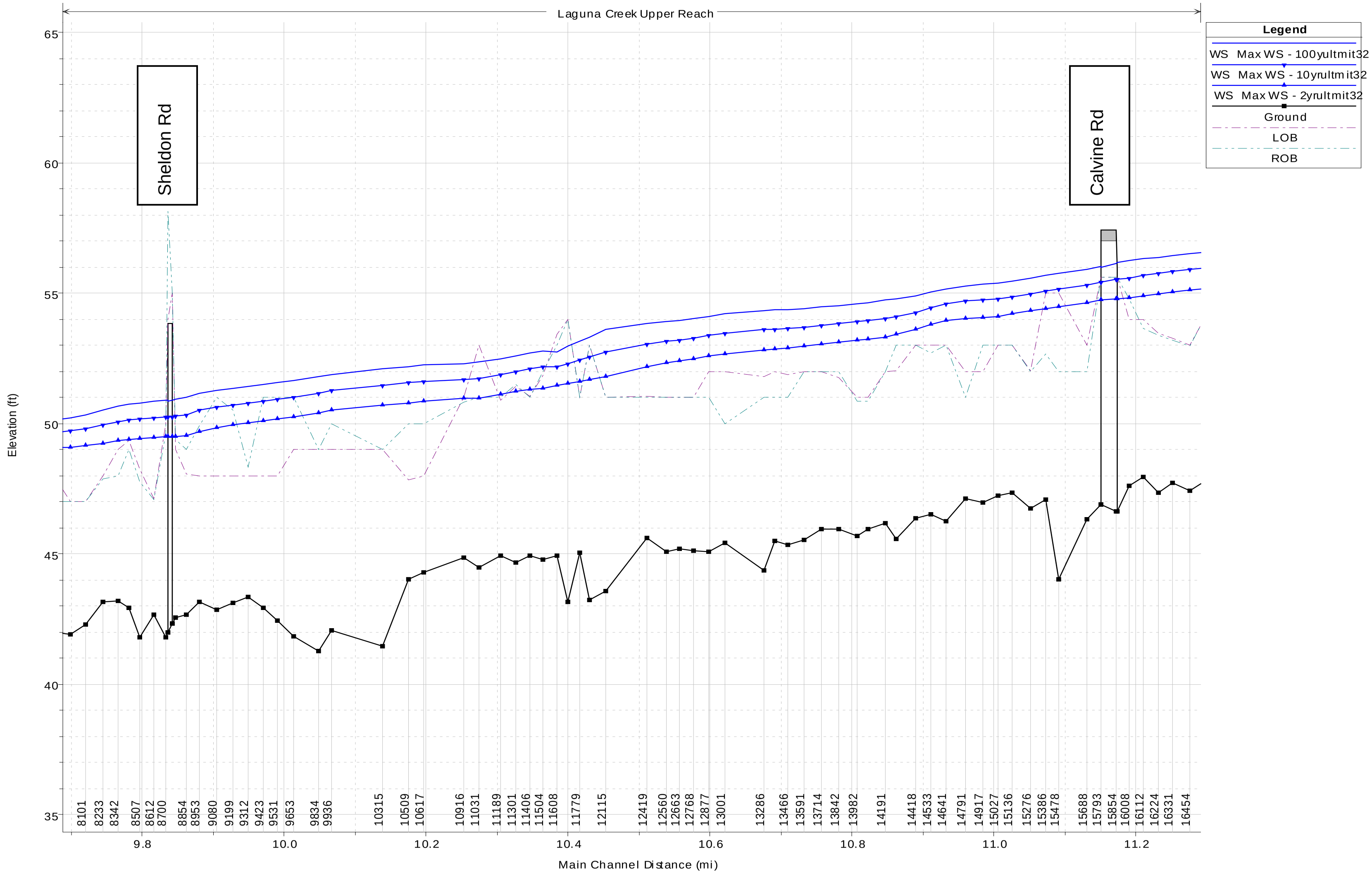

Figure 4-7g. Laguna Creek Upper Reach Sta. 8101 to 16454

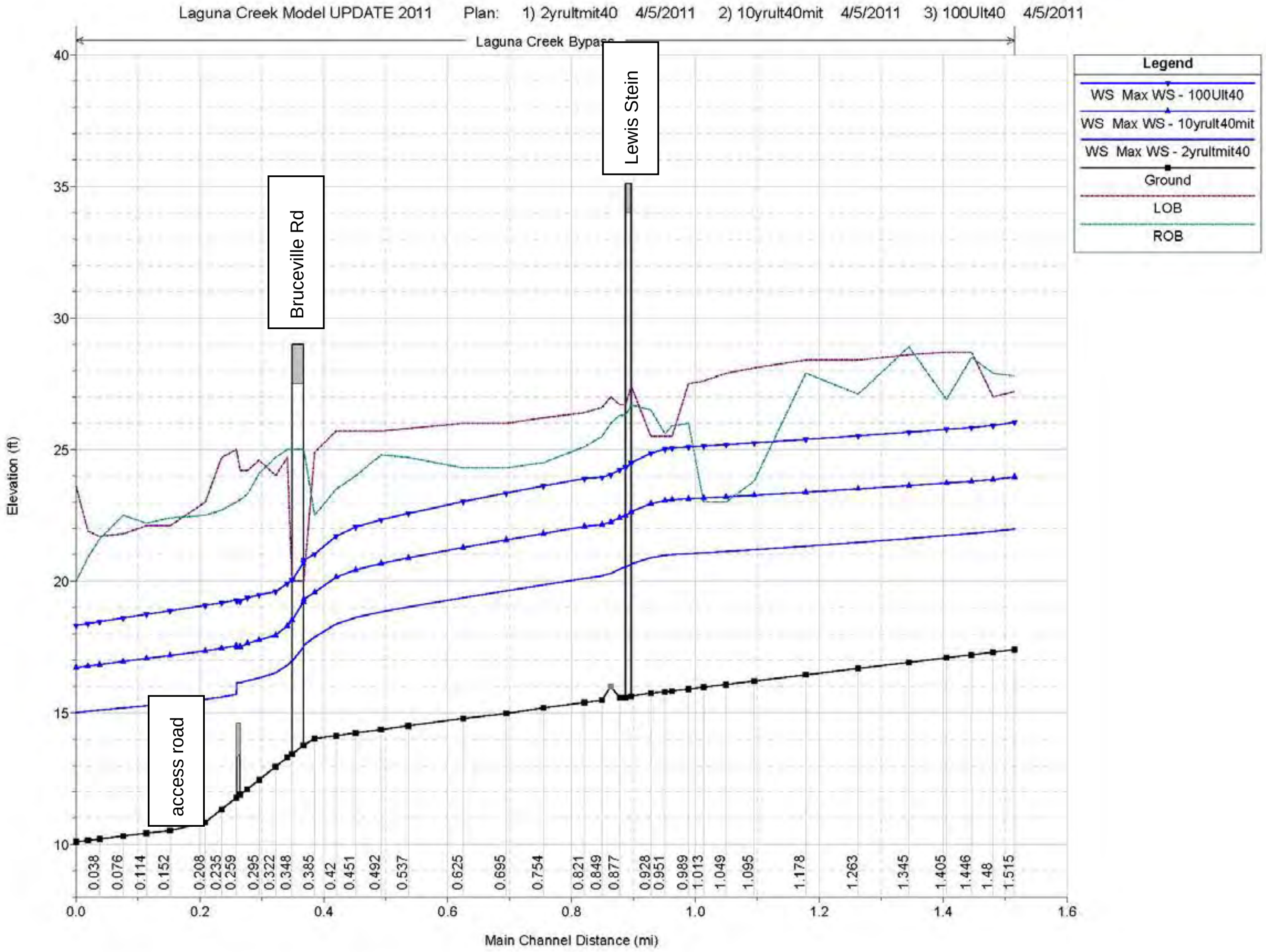

Figure 4-7h. Laguna Creek Bypass Channel Sta. 0 to 1.515

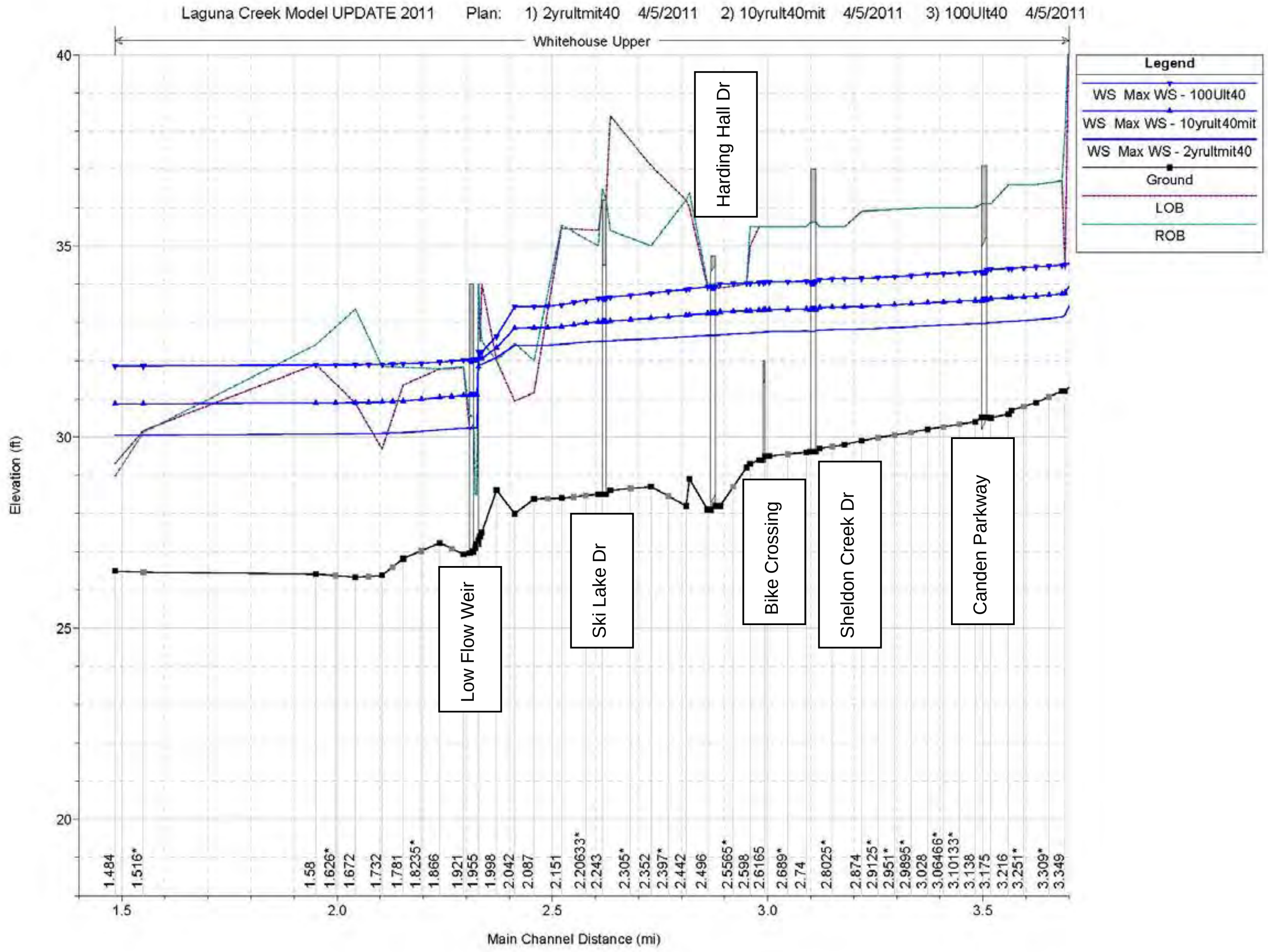

Figure 4-7i. Whitehouse Creek Sta. 1.484 to 3.349

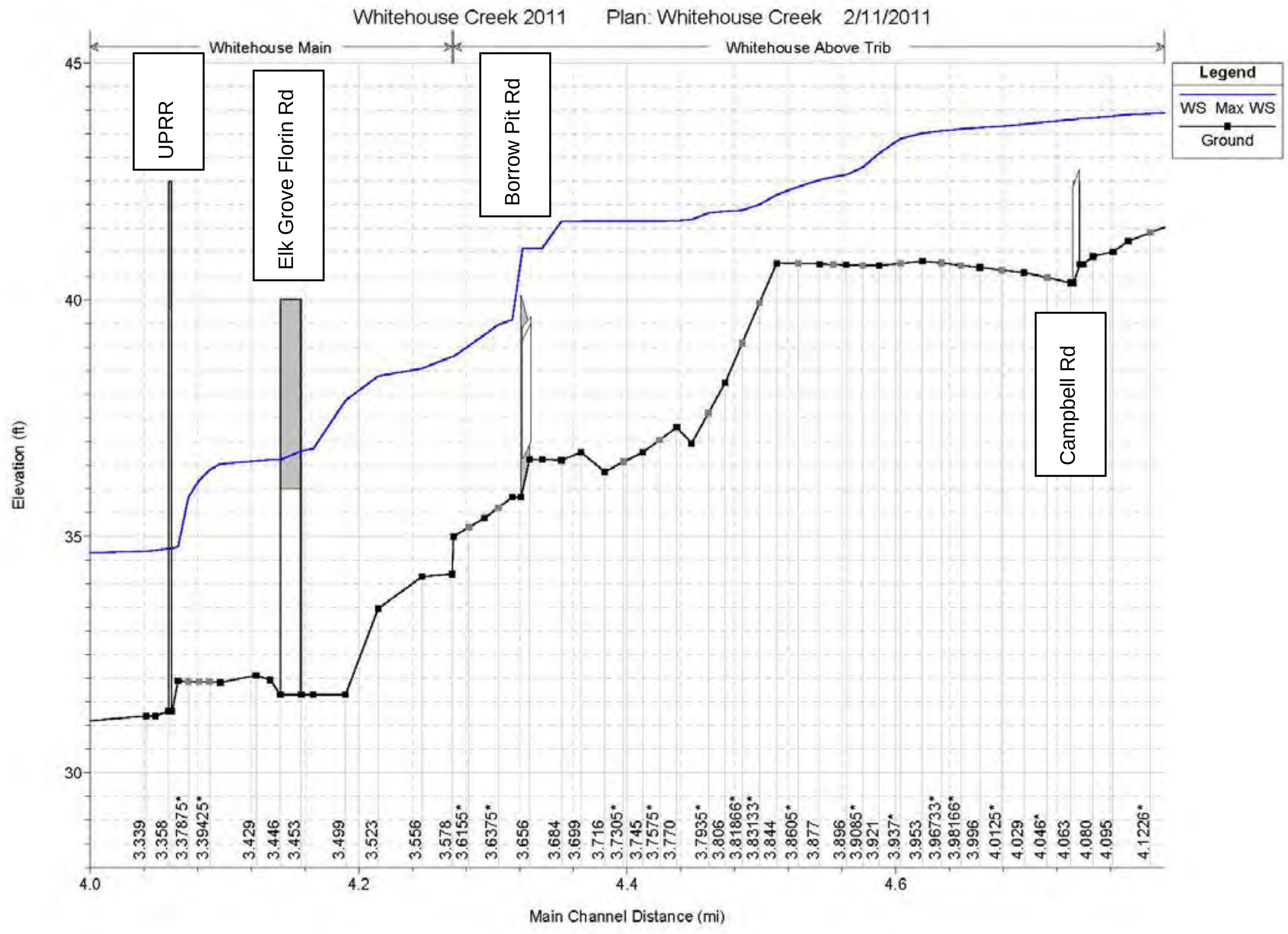

Figure 4-7j. Whitehouse Creek Sta. 3.339 to 4.1226

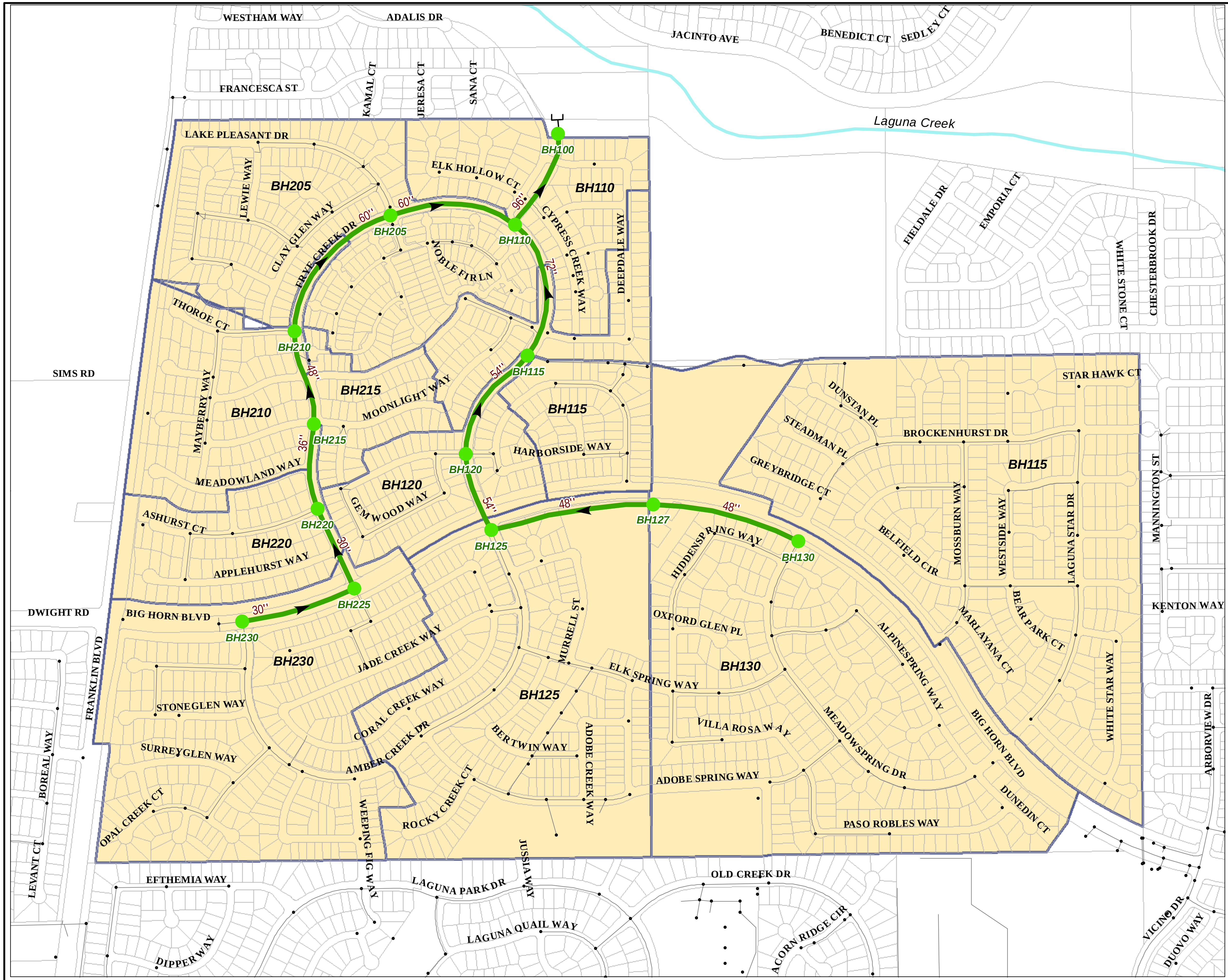

FIGURE 4-8

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
EXISTING PIPELINE LC1
SUBSHEDS AND MODELED FACILITIES

N

0 500 1,000
Feet

LEGEND:

BH125

- Modeled Pipeline and Node
- Existing Pipeline LC1 Subshed

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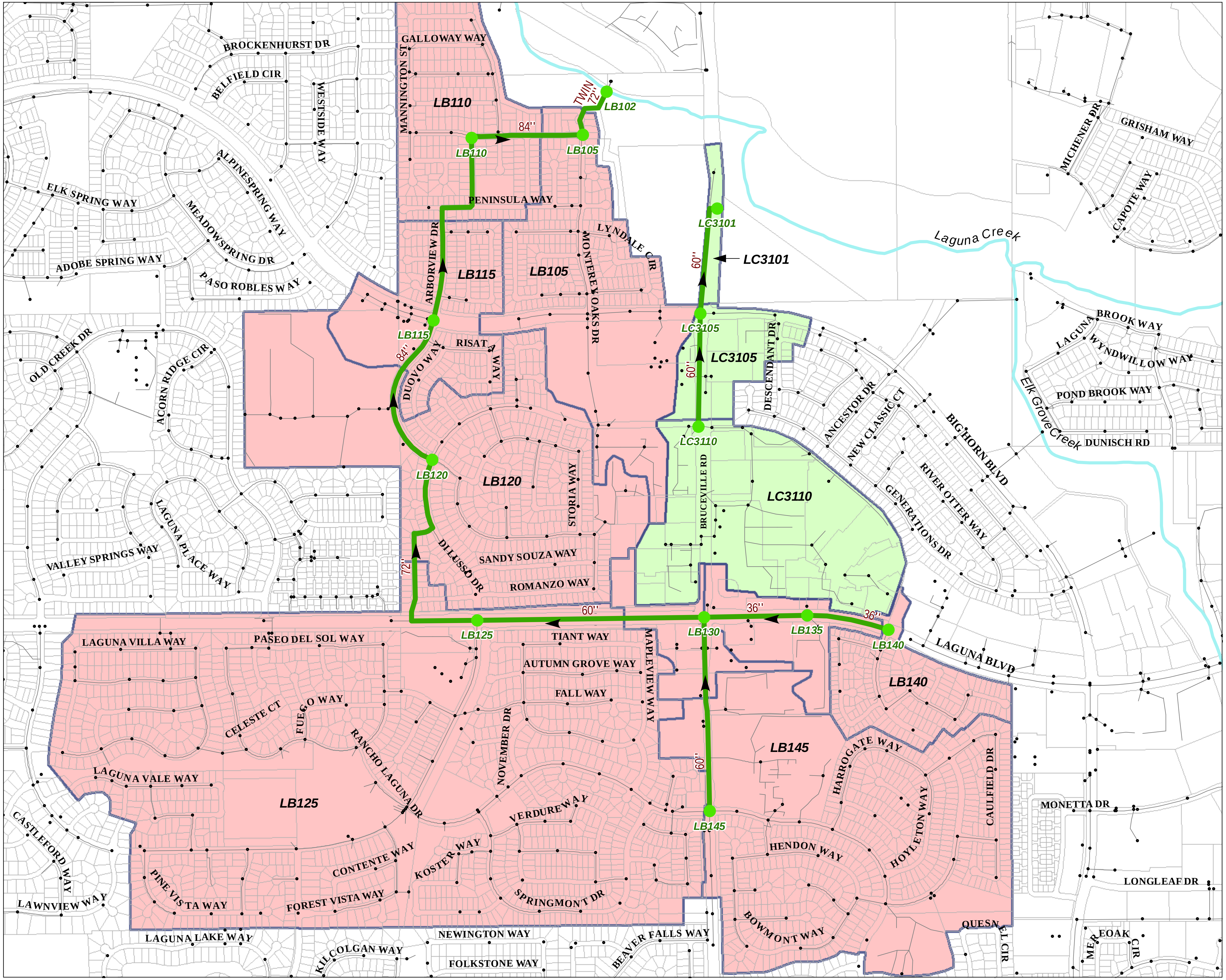
FIGURE 4-9

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
EXISTING PIPELINES LC2 & LC3
SUBSHEDS & MODELED FACILITIES



N

0 800 1,600
Feet

LEGEND:

LC3110

- Modeled Pipeline and Node
- 🟥 Existing Pipeline LC2 Subshed
- 🟩 Existing Pipeline LC3 Subshed

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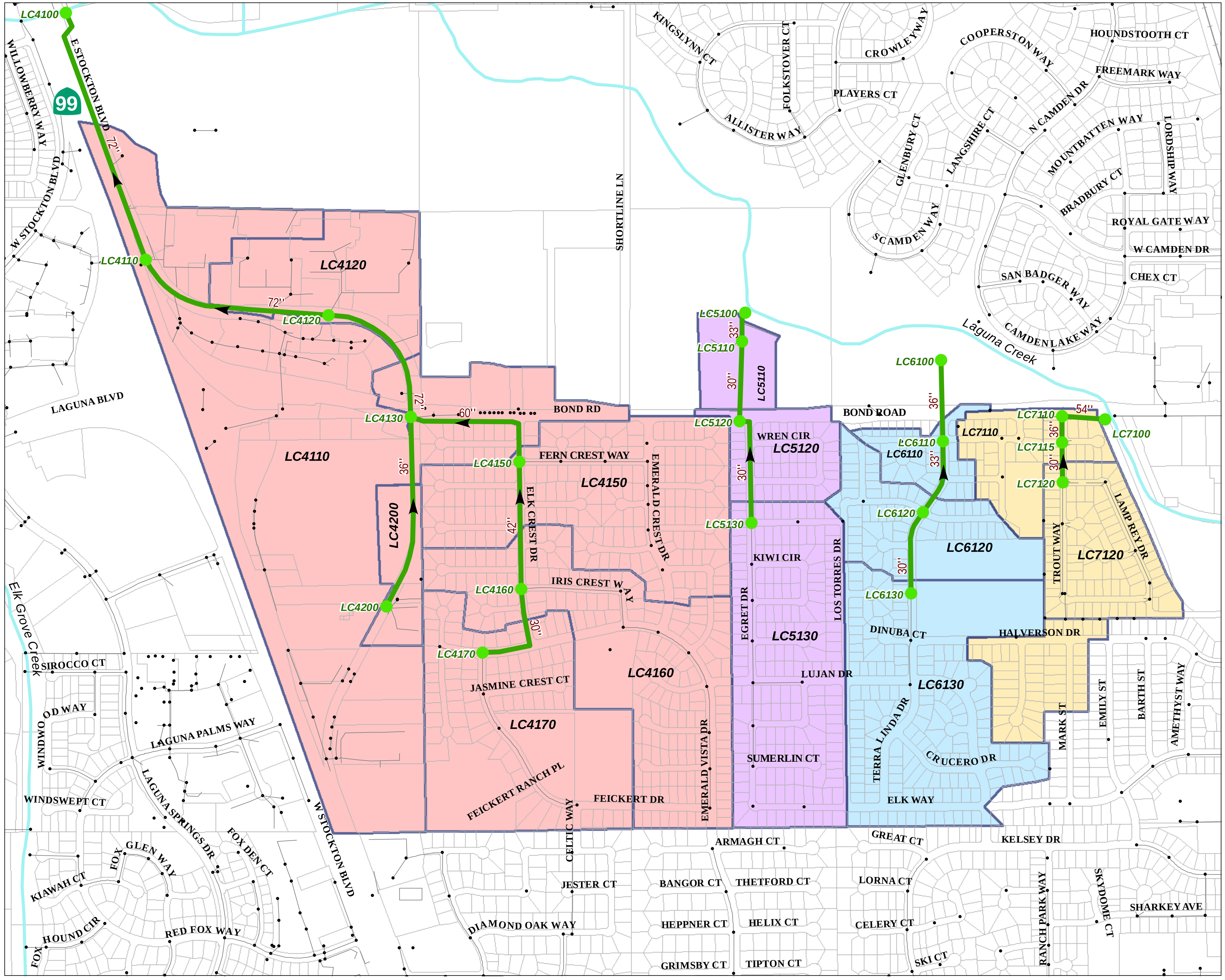

FIGURE 4-10

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
EXISTING PIPELINES LC4, LC5, LC6, LC7
SUBSHEDS & MODELED FACILITIES

N

0 600 1,200
Feet

LEGEND:

LC5130
- Modeled Pipeline and Node
- 🟥 Existing Pipeline LC4 Subshed
- 🟪 Existing Pipeline LC5 Subshed
- 🟦 Existing Pipeline LC6 Subshed
- 🟨 Existing Pipeline LC7 Subshed

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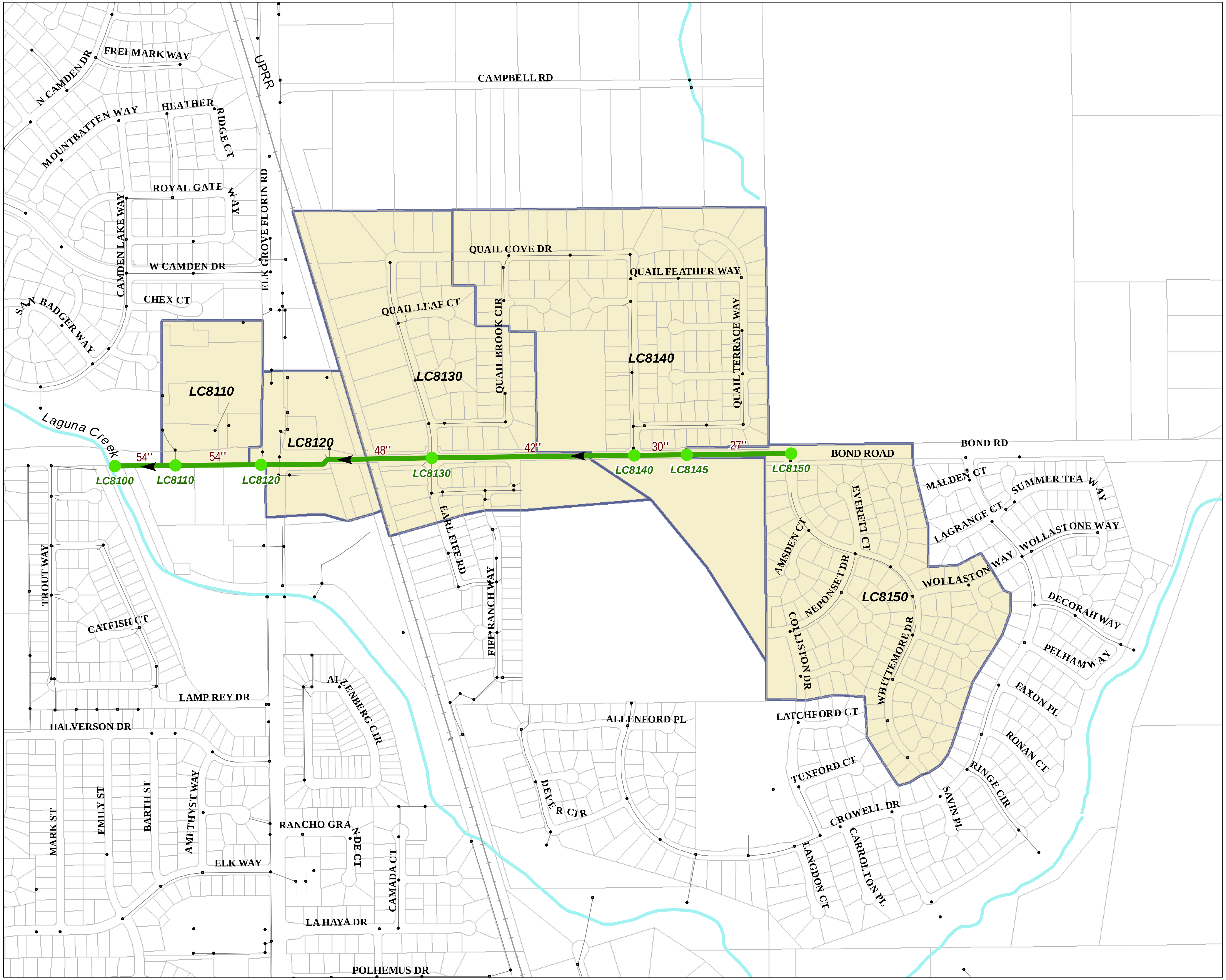

FIGURE 4-11

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
EXISTING PIPELINE LC8
SUBSHEDS & MODELED FACILITIES

N

0 600 1,200
Feet

LEGEND:

LC8130

●━	Modeled Pipeline and Node
▭	Existing Pipeline LC8 Subshed

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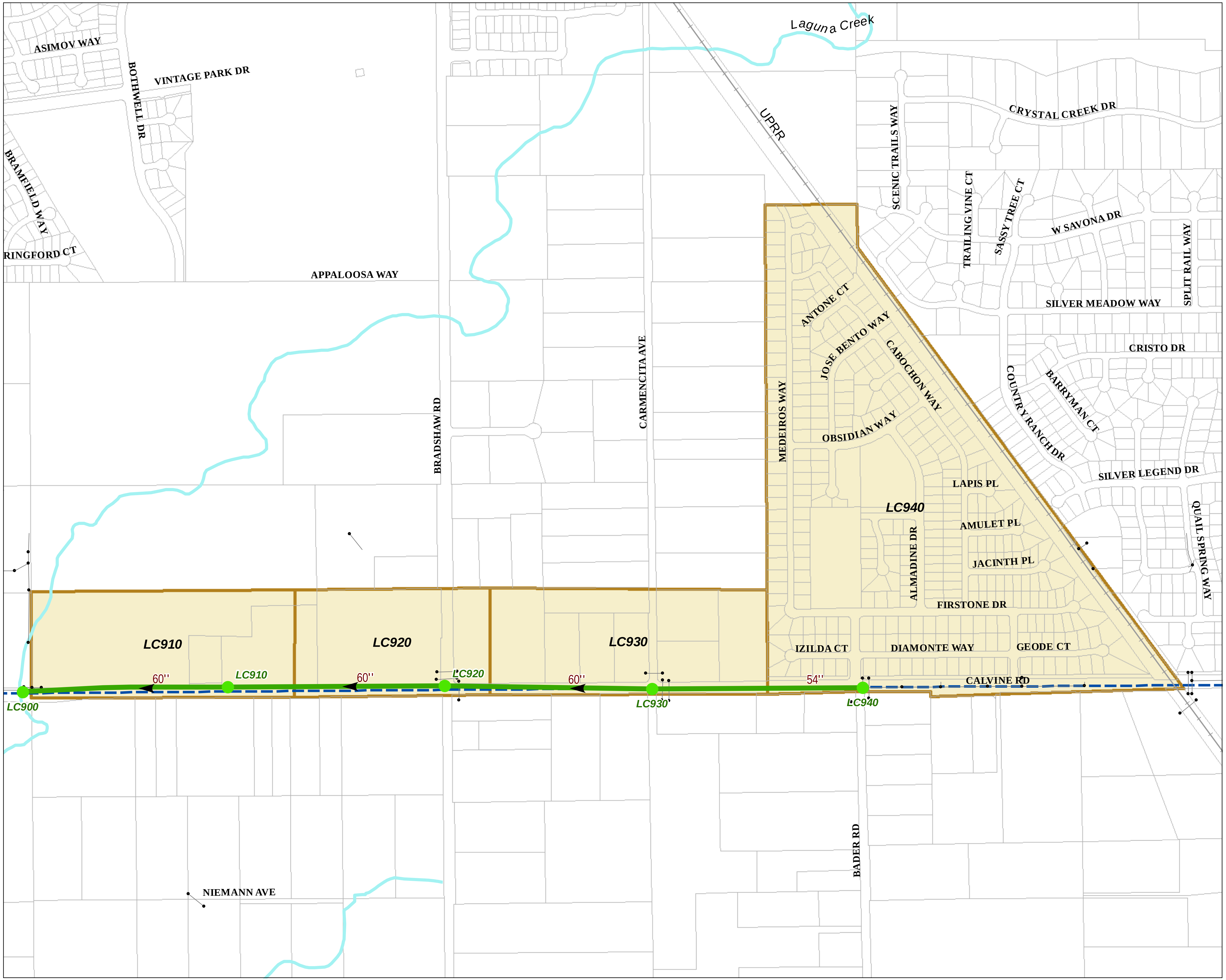
FIGURE 4-12
City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
EXISTING PIPELINE LC9
SUBSHEDS & MODELED FACILITIES

N

0 600 1,200
Feet



LEGEND:

LC910
━●━ Modeled Pipeline and Node
▭ Existing Pipeline LC9 Subshed
┅ City Limit

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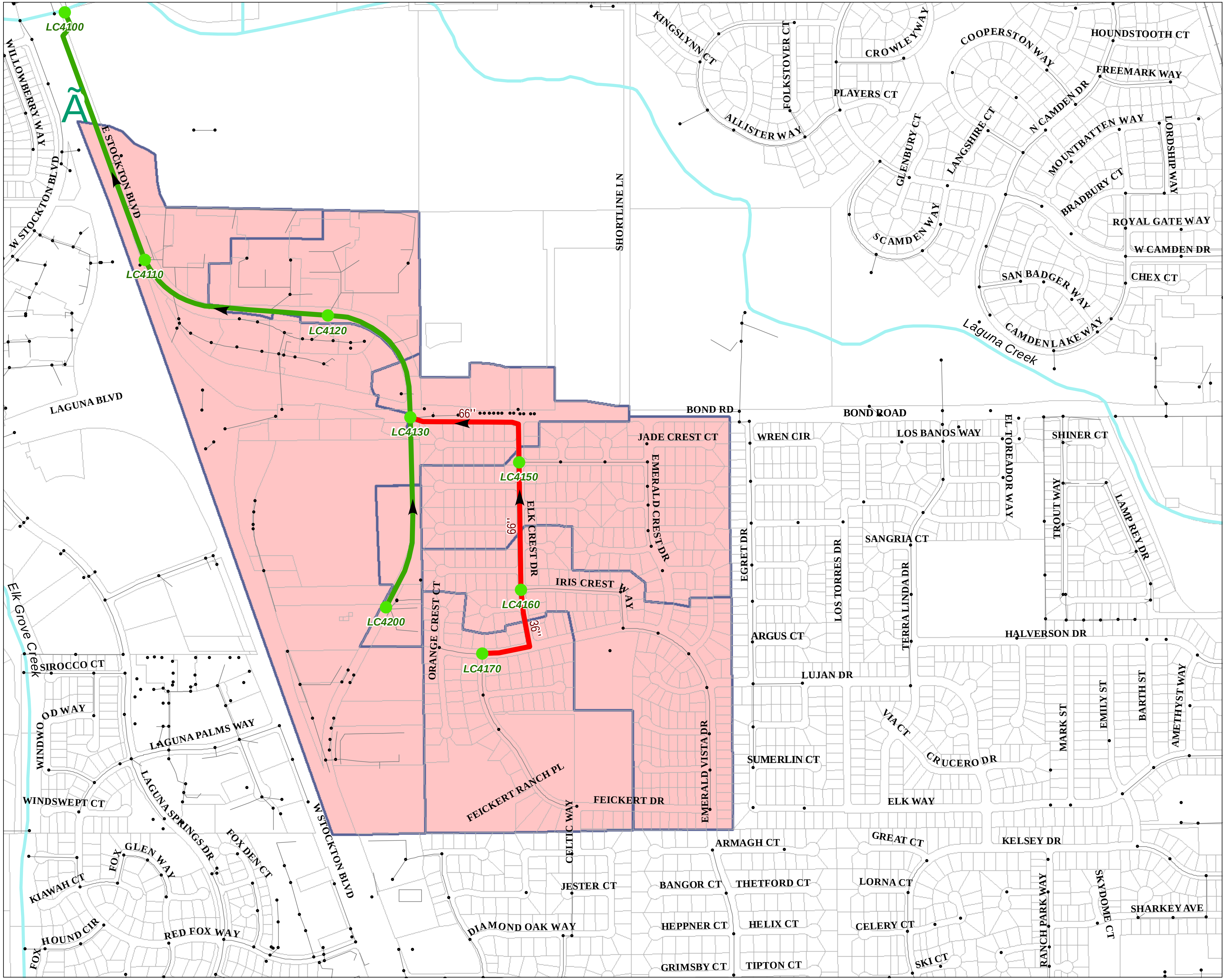
FIGURE 4-13
City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
EXISTING PIPELINE LC4
IMPROVEMENTS



LEGEND:

LC4150

- 🟩 Modeled Pipeline and Node
- 🟥 Upsized Pipeline
- ▨ Existing Pipeline LC4 Subshed

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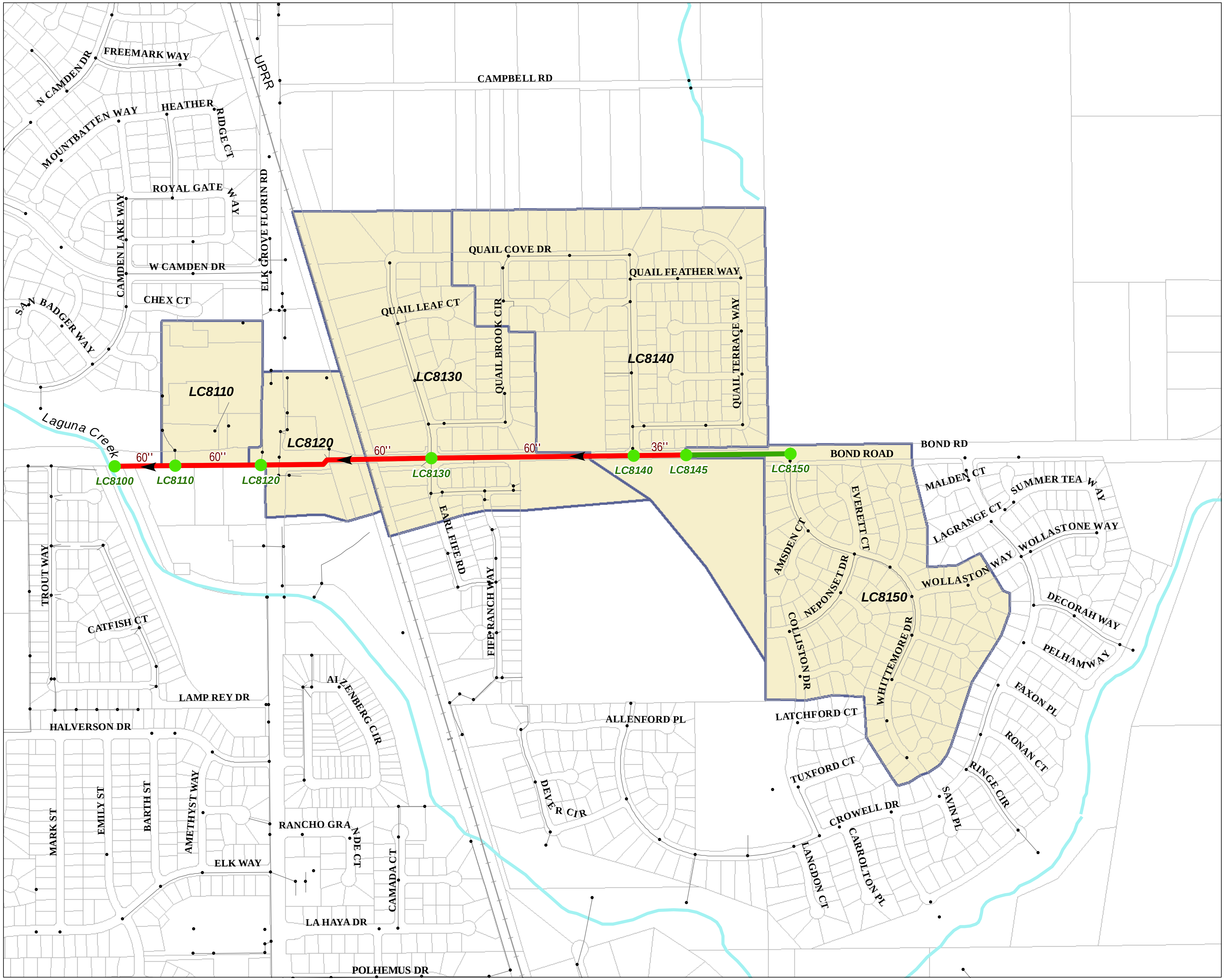
FIGURE 4-14
City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA CREEK
EXISTING PIPELINE LC8
IMPROVEMENTS



N

0 600 1,200
Feet

LEGEND:

LC8130	
🟩●	Modeled Pipeline and Node
🟥	Upsized Pipeline
▭	Existing Pipeline LC8 Subshed

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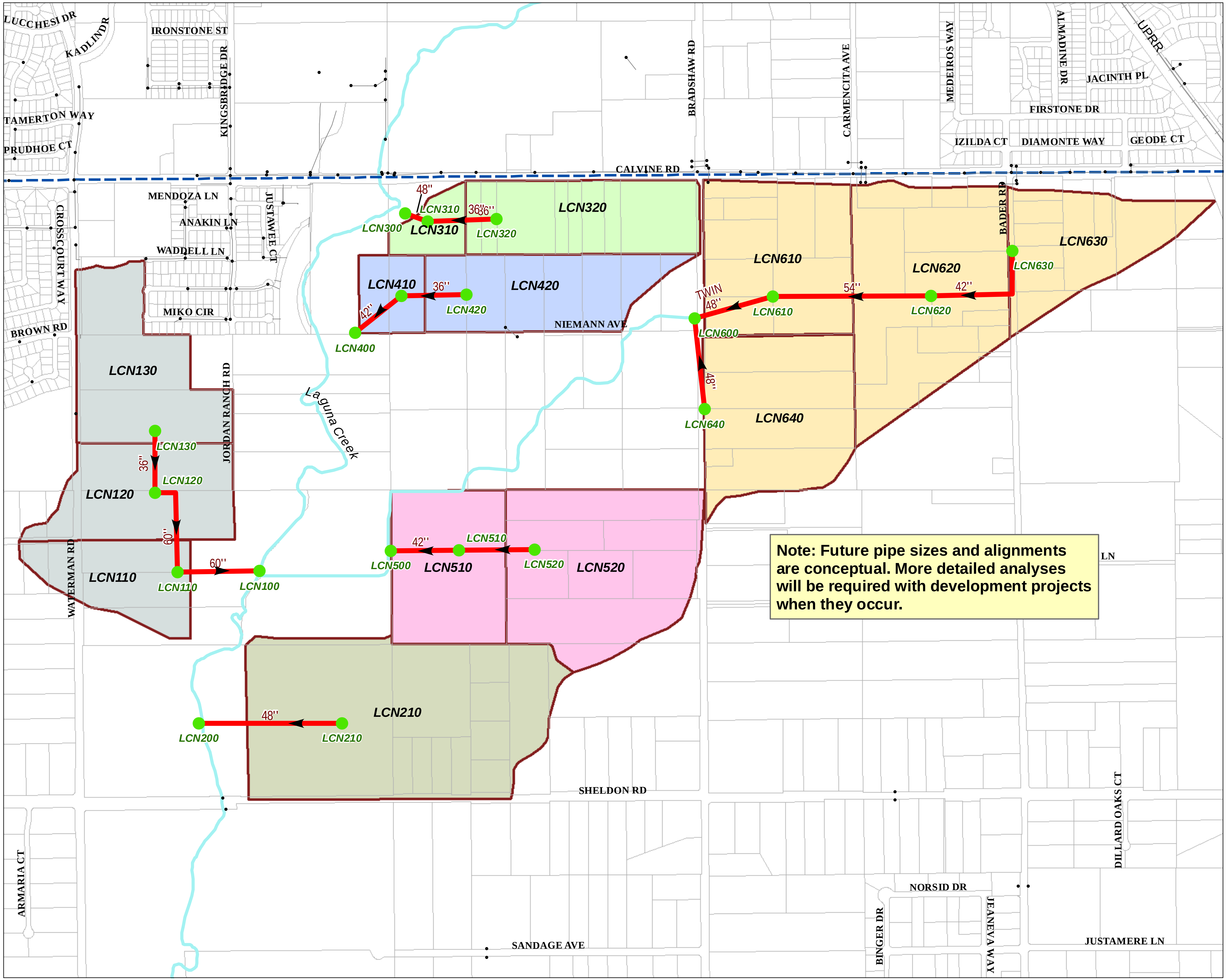
FIGURE 4-15
City of Elk Grove
Storm Drainage Master Plan
Volume II
LAGUNA CREEK
FUTURE TRUNK PIPELINES LCN1-LCN6
SUBSHEDS & FACILITIES



Note: Future pipe sizes and alignments are conceptual. More detailed analyses will be required with development projects when they occur.

LEGEND:

LCN420 Modeled Pipeline and Node
▨ Future Trunk Pipeline LCN1 Subshed
▨ Future Trunk Pipeline LCN2 Subshed
▨ Future Trunk Pipeline LCN3 Subshed
▨ Future Trunk Pipeline LCN4 Subshed
▨ Future Trunk Pipeline LCN5 Subshed
▨ Future Trunk Pipeline LCN6 Subshed
City Limit

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ATTACHMENT 4A

Upper Whitehouse Floodplain Study

Attachment 4A

Hydrologic and hydraulic analysis to assess existing condition floodplain extents for Whitehouse Creek and the unnamed tributary to Whitehouse Creek

October 2009

City of Elk Grove

DAVID FORD CONSULTING ENGINEERS

David Ford Consulting Engineers, Inc.
2015 J Street, Suite 200
Sacramento, CA 95811
Ph. 916.447.8779
Fx. 916.447.8780

Engineer's certification

I hereby certify that I am a professional engineer licensed in the state of California and that the accompanying report was prepared by me or under my supervision.

BRIAN A. BROWN
No. 69338
Exp. 6/30/10
CIVIL

10/6/09

Brian A. Brown

Table of contents

Overview

Situation

In 2006, the City of Elk Grove completed the Draft Flood Control and Storm Drainage Master Plan (DMP). As part of this analysis, hydrologic and hydraulic models were developed and floodplains delineated for the lower portion of Whitehouse Creek from the confluence with Laguna Creek upstream to Elk Grove Florin Road. The upper portion of Whitehouse Creek was not analyzed for this study.

Panel 340 of the Sacramento County FEMA Flood Insurance Rate Map (FIRM) dated July 6, 1998, shows a stretch of floodplain connecting the Laguna Creek and Whitehouse Creek floodplains east of Elk Grove Florin Road and crossing Bond Road as shown in Figure 1. This stretch of floodplain is referred to herein as the cross-basin transfer area. Recent development in the cross-basin transfer area near Bond Road has changed the topography, potentially preventing the cross-basin transfer of flood flows.

Tasks

The purpose of this analysis was to evaluate the unstudied upper portion of Whitehouse Creek and its unnamed tributary and determine the current floodplain extent. We also investigated the cross-basin transfer between Laguna and Whitehouse creeks shown on the FIRM panel.

Action

To analyze the upper portion of Whitehouse Creek, its unnamed tributary, the potential cross-basin transfer, and determine the floodplain extent we completed the following:

1. Reviewed available information from FEMA documents and previous modeling efforts.
2. Updated the existing hydrologic model as follows:
    - Included recent development on Whitehouse Creek.
    - Re-delineated the watershed for the upper portion of Whitehouse Creek.
    - Delineated a new basin for the unnamed tributary for inclusion in the hydrologic model.
3. Updated the existing hydraulic model as follows:
    - Updated Whitehouse Creek channel geometry in the hydraulic model to reflect a new field survey.
    - Added the unnamed tributary to the hydraulic model based on a new survey.
4. Delineated new floodplains for Whitehouse Creek and the unnamed tributary.

Results

We determined that the cross-basin transfer during the p=0.01 event shown on Panel 340 of the Sacramento County FEMA FIRM no longer exists. Stage in Laguna Creek does not exceed the elevation of the ridgeline that divides the Laguna and Whitehouse Creek watersheds. Delineating new watersheds for the upper portion of Whitehouse Creek and the unnamed tributary redistributed flow into the system, but the value of flow did not change significantly. Updating the hydraulic model caused a slight increase in water surface elevation (WSEL) in Whitehouse Creek, a rise of approximately 0.2 feet, but no change in WSEL in Laguna Creek downstream of the confluence with Whitehouse Creek. The newly delineated floodplain for the study area is shown in Figure 8. The sections following provide further detail about the information we reviewed, our analysis, and our findings.

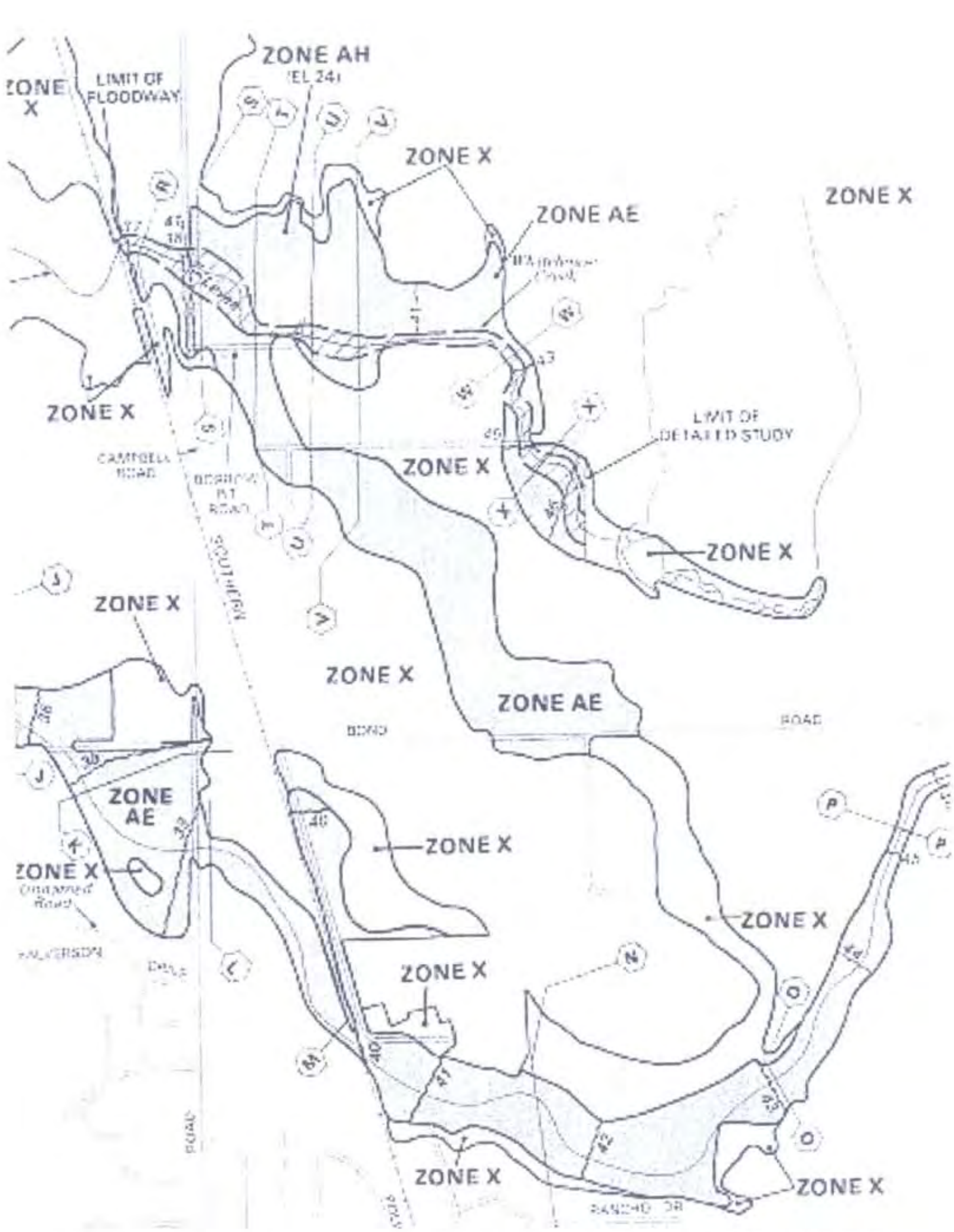

Figure 1. Area of concern from Sacramento County FIRM, Panel 340

Review of available information

To assess existing conditions, we reviewed readily available information consisting of FEMA documents, existing topographic data, DMP hydrologic and hydraulic models, and floodplain mapping.

Review of FEMA documents

The FEMA Map Service Center provides Letters of Map Change (LOMC) for specified FIRM panels issued after June 1, 1997. Panel 340 of the Sacramento County FIRM (Panel ID 0602620340D) contains 12 LOMCs, filed between 1998 and 2006. LOMCs were downloaded and reviewed to determine any changes to the FIRM in the area of concern. The LOMCs pertaining to the area of concern were either Letters of Map Amendment or Letters of Map Revision Based on Fill (LOMR-F). The documents removed portions of parcels and/or structures from the Special Flood Hazard Area (SFHA) based on their elevation being above the p=0.01 flood elevation.

FEMA's process of LOMCs and LOMR-Fs has allowed development in the cross-basin transfer area. Figure 2 shows a current aerial photo with the approximate FEMA floodplain and structures removed from the SFHA.

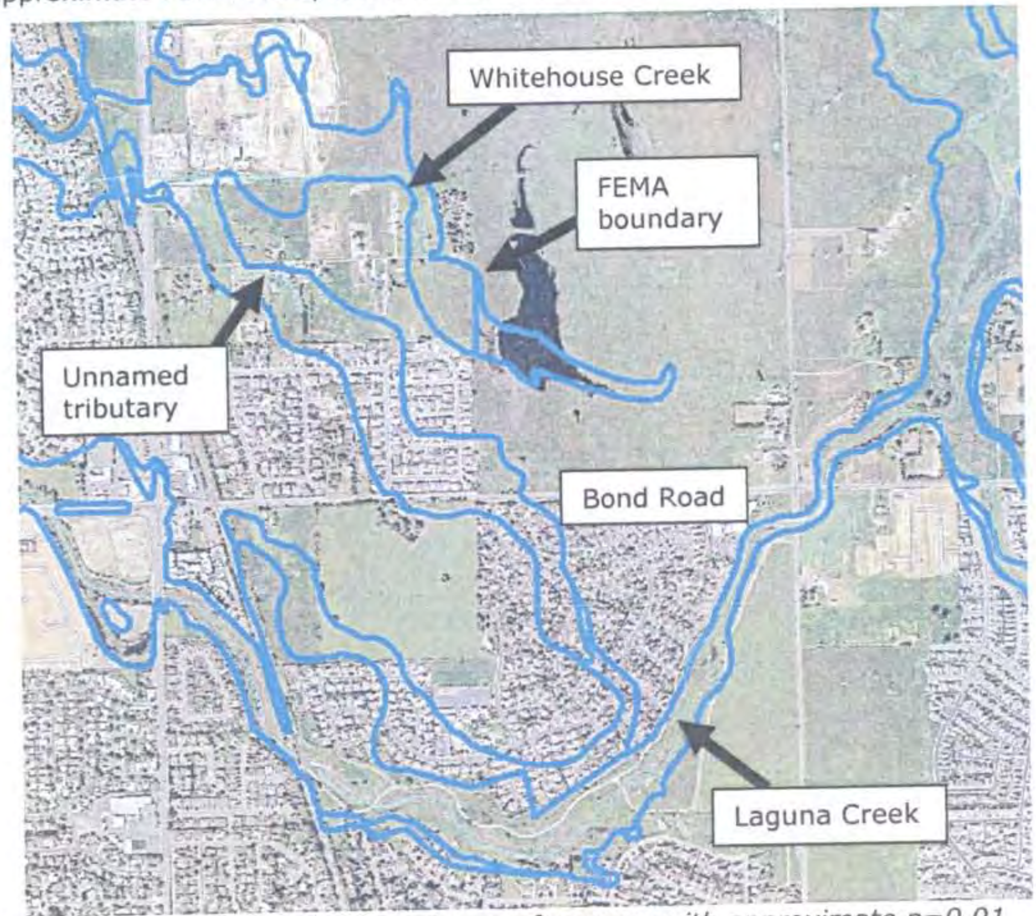

Figure 2. Current aerial photo of area of concern with approximate p=0.01 event FEMA floodplain boundary

Review of available topographic data

Water surface profiles from the DMP and topographic information were reviewed to assess existing conditions. By comparing the Laguna Creek water surface profile to topographic data, we investigated whether a cross-basin transfer exists. An approximate ridgeline was drawn considering high ground elevation and development features. Figure 3 shows the ridgeline and ground elevation contours in the area of concern.

Figure 3. Ridgeline in area of concern with ground elevation contours

We extended the cross sections from the DMP Laguna Creek hydraulic model to the ridgeline. This allowed us to compare the ridgeline and computed p=0.01 water surface elevations. Figure 4 shows this comparison. The cross-basin transfer no longer exists because stage due to flow in Laguna Creek does not exceed the elevation of the ridgeline and enter Whitehouse Creek.

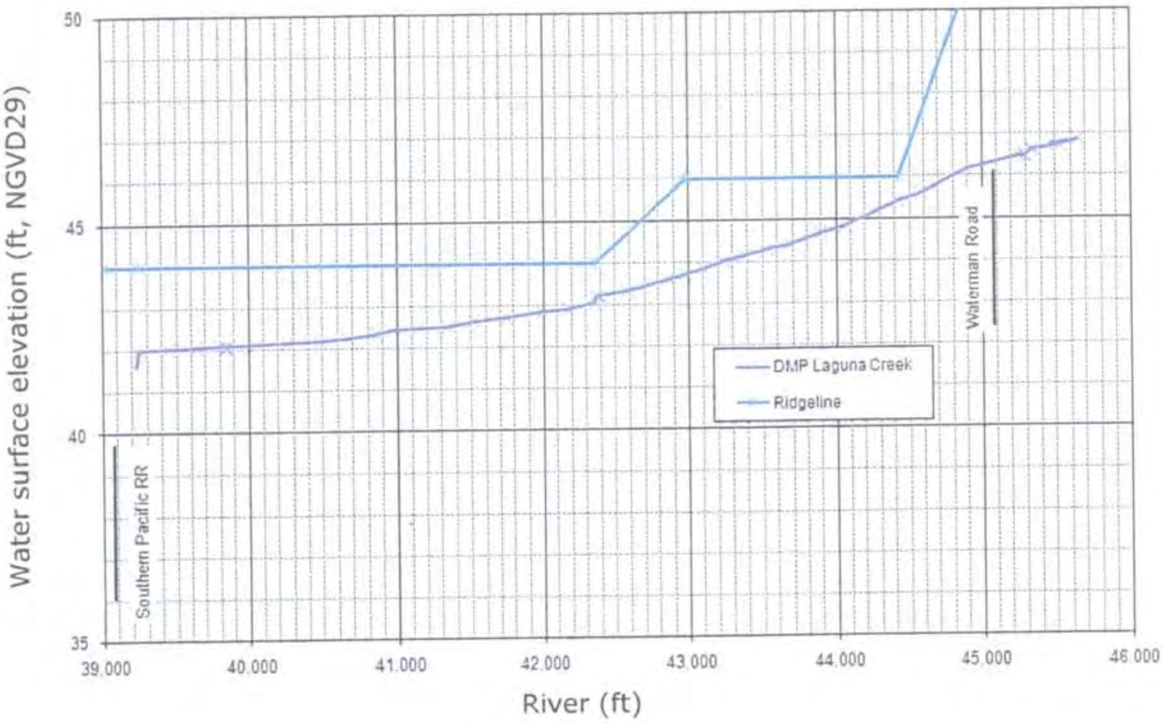

Figure 4. Profile for Laguna Creek (p=0.01 event) and ridgeline in area of concern

Hydrologic analysis

Information with which we started

We reviewed 3 hydrologic models of Whitehouse Creek for use in this analysis: an HEC-1 model developed by Sacramento County (County) in 2003, the DMP SacCalc model, and a 2007 Whitehouse Creek update SacCalc model. The County's model was developed to investigate Whitehouse Creek's hydrologic response to the recent capping of a nearby landfill. The was supplied by Sacramento County in 2005 as background information for the DMP. The DMP model, developed in 2006, used watershed delineations from the 2003 Laguna Creek analysis. The 2007 Whitehouse Creek update added impervious area to the DMP model to reflect recent paving for a tow-yard. This update was documented in a memo to the City titled "Whitehouse Creek Update", dated May 7, 2007.

Changes we made

The DMP and County models had similar watershed delineations. However, due to subtle differences between watershed boundaries and their correlation to the alignment of Whitehouse Creek, we delineated new watersheds. New watersheds generated for the upper portion of Whitehouse Creek upstream of Elk Grove Florin Road were based on topographic data from the DMP. Slight adjustments were made to the adjacent Laguna Creek watersheds (specifically watersheds L21625, L21670, and L21680) to ensure all boundaries were adjacent. Figure 5 shows the updated watershed boundaries contributing to the upper portion of Whitehouse Creek.

The new delineations considered the unnamed tributary and the Quail Ranch Estates drainage areas. Watershed 1 contributes to the unnamed tributary and watershed 2 represents the drainage area for Quail Ranch Estates. Quail Ranch Estates drainage is conveyed through an underground pipe system, which outlets south to the Laguna Creek watershed. However, overland release for the development drains to the Whitehouse Creek watershed, specifically the unnamed tributary.

For the overland contribution from Quail Ranch Estates, we assumed the pipe system has capacity to convey a p=0.10 event. This assumption is consistent with historic minimum design standards for storm drain systems in Sacramento County. Based on this assumption, flow contribution to Whitehouse Creek is the volumetric difference between the p=0.01 and p=0.10 events (approximately 33 cfs). The entire Quail Ranch Estates drainage area drains south to Laguna Creek, consistent with the DMP. This is a conservative approach due to the uncertainty of the pipe system and overland drainage.

Once the watersheds were delineated, we identified impervious areas through a GIS intersection of available data sources. The data sources used in the intersection include the newly delineated watersheds along with soils data and General Plan land use data (both data sources used in the DMP). The DMP established a relationship between General Plan land use and percent impervious. This relationship along with the results of the intersection provided the watershed impervious area as a function of soil and land use. The updated watershed parameters were configured in SacCalc. Per the 2007

Whitehouse Creek update, the impervious area change due to recent development in watershed 51660 was also configured in SacCalc. The runoff hydrographs for the p=0.10 and p=0.01 events were computed. When computing the runoff hydrographs, a 1.44-square-mile storm area was used to represent the upper Whitehouse basin, consistent with the previous modeling efforts.

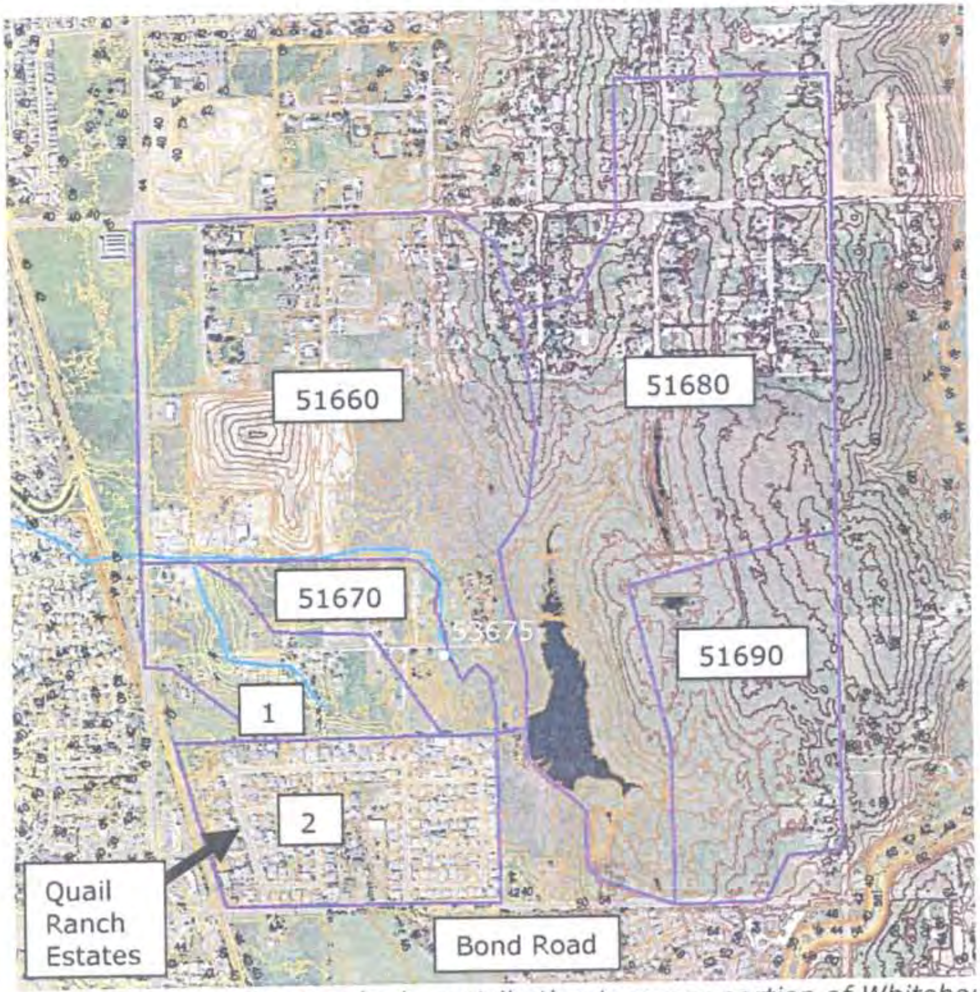

Figure 5. Updated watersheds contributing to upper portion of Whitehouse Creek. Note that watersheds 51680 and 51690 were combined at junction 53675 in SacCalc in order to have a direct handoff in HEC-RAS.

Results of the analysis

SacCalc results were compared to the DMP results to ensure similar flows were entering the system using the new watershed delineations and impervious areas. The newly delineated watershed parameters are listed in Table 1. Table 2 shows a comparison of DMP and newly calculated peak flows. Note that a direct watershed to watershed comparison cannot be made due to significant area differences between the newly delineated sheds and the DMP sheds. The purpose of this table is to show the peak flow values and compare the total flow entering the system.

Table 1. Newly delineated watershed parameters

Watershed (1)	Area (ac) (2)	Mean elevation (ft) (3)	Channel length (ft) (4)	Centroid length (ft) (5)	Slope (ft/ft) (6)	Lag (min) (7)	Initial loss (in) (8)	Constant loss rate (in/hr) (9)	Impervious area (%) (10)
1	38.9	40	1587	1305	.0027	24.6	0.10	0.067	18.2
2	66.3	40	2532	1314	.0029	37.8	0.10	0.055	71.7
51660	187.8	45	7830	5797	.0031	98.0	0.10	0.082	17.5
51670	28.1	40	2160	1045	.0013	33.9	0.10	0.069	9.7
51680	224.4	55	4599	2598	.0033	66.7	0.10	0.102	11.5
51590	41.4	38	7102	4002	.0035	108.6	0.10	0.107	6.4

Table 2. Whitehouse Creek peak flows from newly delineated watersheds and DMP

Watershed designation (1)	New delineation peak flow p=0.01 event (cfs) (2)	DMP peak flow p=0.01 event (cfs) (3)[1]
51660	121.9	118.9
51670	36.9	93.9
51680	183.9	87.9
51690	45.2	68.8
51689[2]	-	50.2
1	61.8	-
2[3]	84.4	-
Total flow entering system[4]	449.7[4]	419.7

1. Note that a direct watershed to watershed comparison cannot be made due to significant area differences between the newly delineated sheds and the DMP sheds. The purpose of this table is to show the peak flow values and compare the total flow entering the system.
2. Watershed 51689 was removed, with the contributing area distributed to watersheds 51680 and 51690.
3. The difference between the p=0.01 and p=0.10 events enters Whitehouse Creek. The p=0.10 event peak flow for watershed 2 is 51.9 cfs.
4. To make a direct comparison, the newly delineated watersheds omit watershed 2, as watershed 2 represents a significant addition of contributing area to the system.

Hydraulic analysis

Information with which we started

We reviewed 2 hydraulic models of Whitehouse Creek for use in this analysis: the DMP model, and the 2007 Whitehouse Creek update. Both models were configured in the HEC-RAS computer program using an unsteady flow analysis. The 2007 Whitehouse Creek update added new cross sections from updated topographic information and updated a bridge definition directly downstream of Elk Grove Florin Road.

Changes we made

We started with the DMP model, which includes Laguna and Whitehouse creeks. The updated cross sections from the 2007 Whitehouse Creek update were added to the model, creating a base model for the new analysis. The DMP water surface profiles were computed using the HEC-RAS computer program, version 3.1.3. The current version of HEC-RAS is version 4.0. We ran the DMP model in both versions to determine differences in computed water surface elevations. Differences are negligible (between 0.00 - 0.04 feet), therefore we proceeded using version 4.0.

Channel surveys of the upper portion of Whitehouse Creek and its unnamed tributary were recently conducted. All new survey data is referenced to NGVD 29, the same datum used for the DMP. West Yost Associates (WYA) surveyed cross sections in April 2009 starting directly below the confluence of the unnamed tributary, continuing upstream to the intersection of Whitehouse Creek and Campbell Road. New cross section data start at River Station (RS) 3.544 in the DMP model. From the survey we added 14 cross sections, 2 culverts, and 2 bridges upstream of RS 3.544 to Whitehouse Creek. In 2008 WYA surveyed 3 cross sections on Whitehouse Creek to determine if the current topography matched the topography used in the DMP for this area. We also added these 3 cross sections to the hydraulic model.

The DMP did not include a hydraulic model of Whitehouse Creek's unnamed tributary. The unnamed tributary was surveyed by German Engineering/S360 Development Services in May 2009. WYA supplemented the survey by German Engineering/S360 Development Services in August 2009. From the survey we added 15 cross sections, 3 culverts, and 3 bridges to the hydraulic model to define the unnamed tributary.

A few cross sections of Whitehouse Creek and the unnamed tributary did not fully contain the flow. Sections were laterally extended based on Sacramento County LiDAR data. The same LiDAR data were also used for floodplain mapping in the DMP. A schematic of the HEC-RAS model is shown in Figure 6.

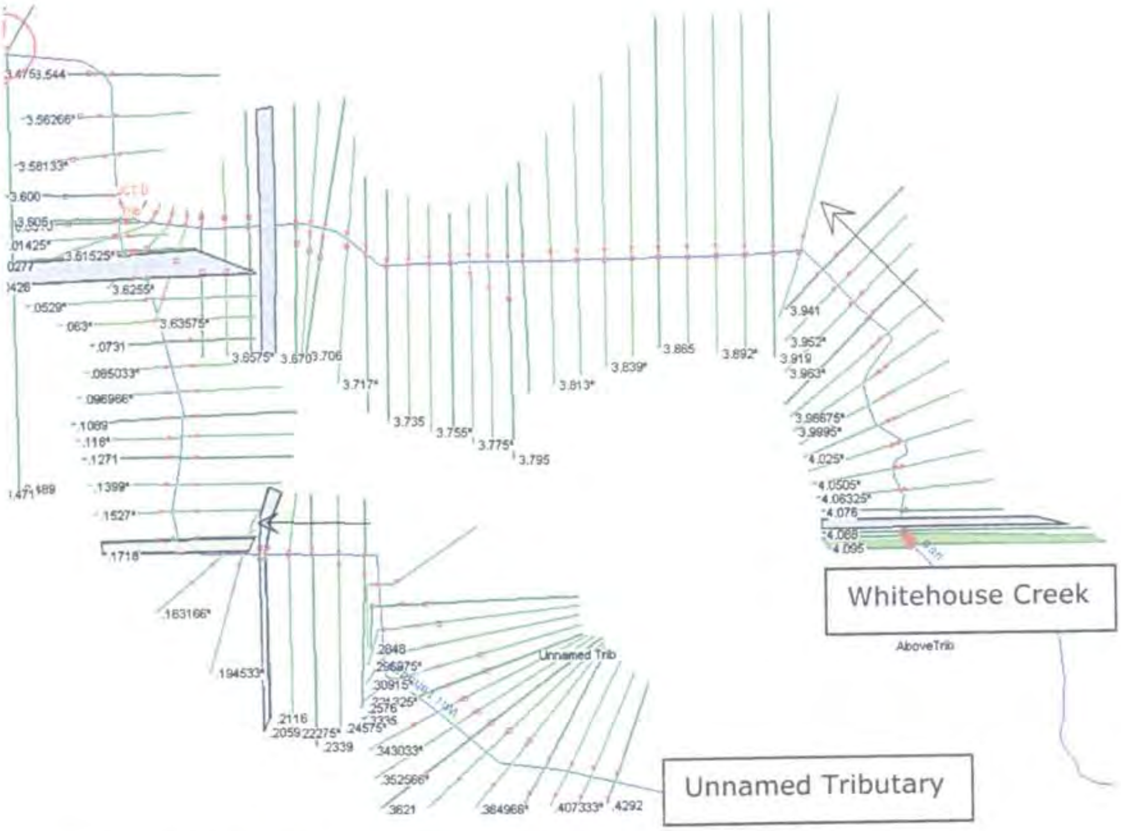

Figure 6. Schematic of HEC-RAS model

Once the geometric data for the hydraulic model were defined, we determined the SacCalc to HEC-RAS handoff locations for flow hydrographs. Table 3 shows the SacCalc to HEC-RAS handoff locations.

Table 3. HEC-RAS handoff locations

Creek (1)	Watershed designation (2)	HEC-RAS river station (3)
Whitehouse	51660	3.489
Whitehouse	51680	4.095[1]
Whitehouse	51690	4.095[1]
Unnamed tributary	51670	0.3964
Unnamed tributary	1	0.3621
Unnamed tributary	2	0.4292

1. Watersheds were combined at junction 53675 in SacCalc to have a direct handoff in HEC-RAS.

Results of the analysis

The HEC-RAS flow and WSEL results for Whitehouse Creek and its unnamed tributary are listed in Table 4. The peak flow decreases as it travels downstream due to attenuation in the overbank areas. On the unnamed tributary water spreads out and overland flow occurs due to a poorly defined channel. This causes the reduction in peak flow on the unnamed tributary.

Table 4. HEC-RAS results for study area

Creek (1)	**River station** (2)	**WSEL (ft)** (3)	**Flow (cfs)** (4)
Whitehouse	4.095	43.85	214
Whitehouse	3.865	43.10	189
Whitehouse	3.800	41.85	186
Whitehouse	3.669	39.32	150
Whitehouse	3.605	38.38	150
Unnamed tributary	0.4292	38.56	33
Unnamed tributary	0.2576	38.45	96
Unnamed tributary	0.1655	38.43	90
Unnamed tributary	0.0731	38.43	44
Unnamed tributary	0.0010	38.38	41

The profile plot for Whitehouse Creek above the unnamed tributary is shown in Figure 7, and the profile plot for the unnamed tributary is shown in Figure 8.

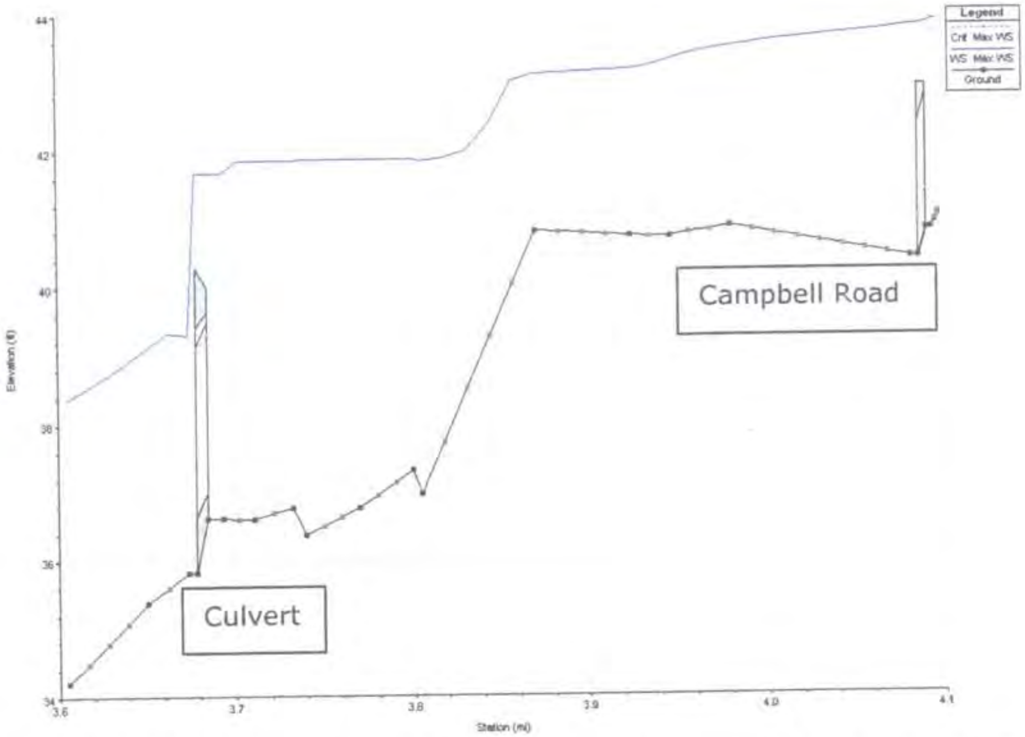

Figure 7. Whitehouse Creek profile upstream of unnamed tributary, p=0.01 event

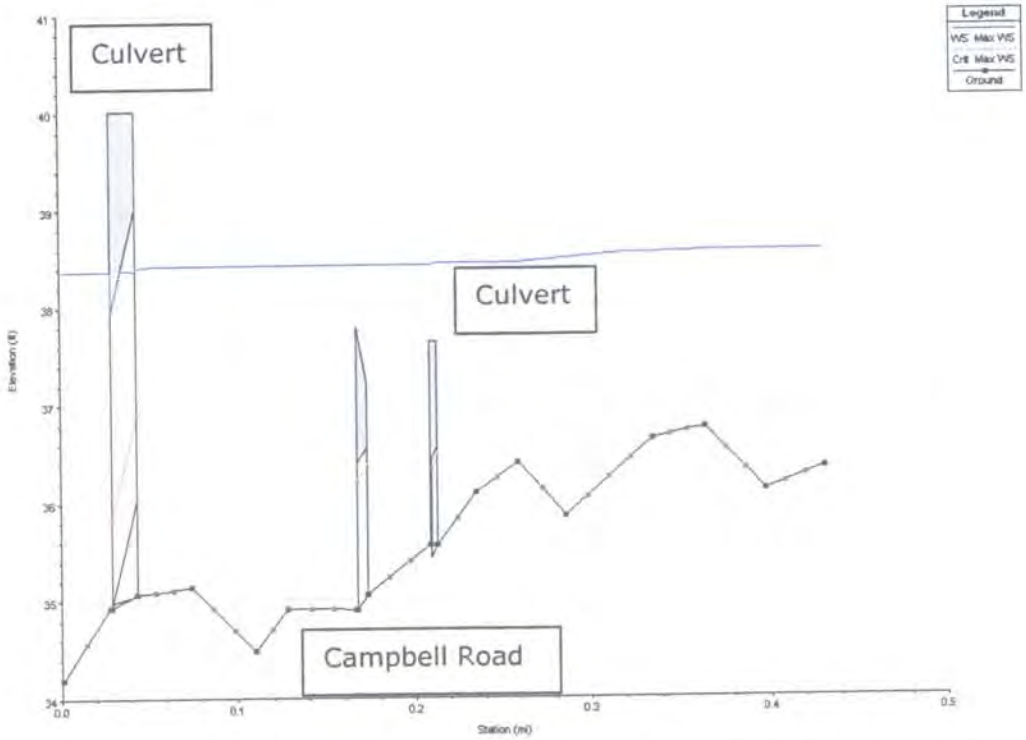

Figure 8. Unnamed tributary to Whitehouse Creek profile, p=0.01 event

The stage and flow hydrographs for Whitehouse Creek above the confluence with the unnamed tributary (RS 3.605) are shown in Figure 9. The stage and flow hydrographs for the unknown tributary above the confluence with the Whitehouse Creek (RS 0.001) are shown in Figure 10. The flow hydrograph for the unnamed tributary decreases at the same time the flow hydrograph for Whitehouse Creek reaches its peak value. This causes a backwater effect in the unnamed tributary.

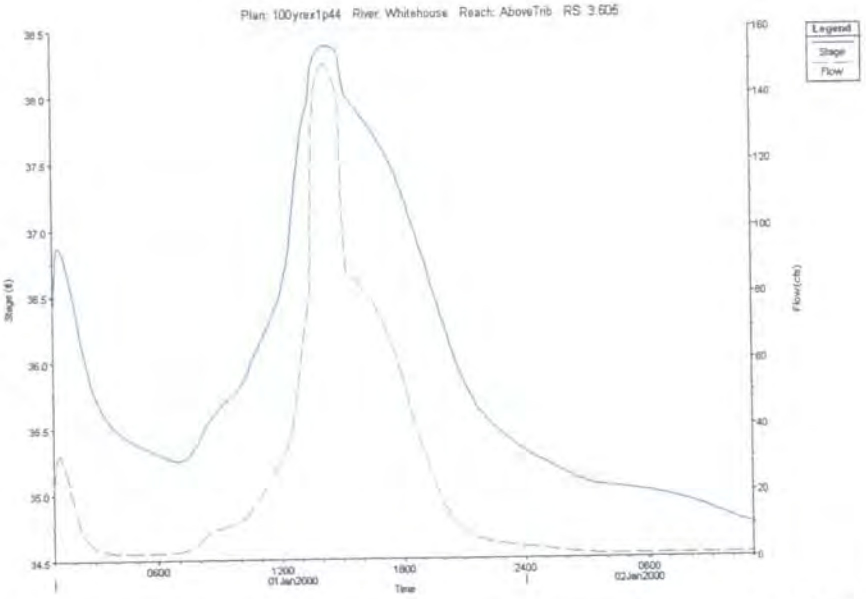

Figure 9. Stage and flow hydrographs for Whitehouse Creek at RS 3.605

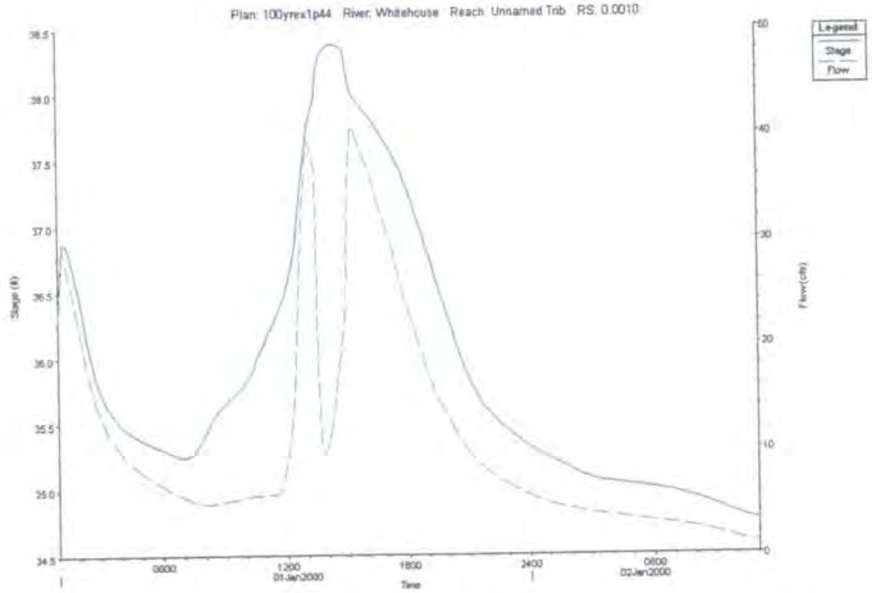

Figure 10. Stage and flow hydrographs for unnamed tributary at RS 0.001

Changes made to the upstream portion of the Whitehouse Creek hydraulic model may impact previously computed water surface elevations downstream of Elk Grove Florin Road. To evaluate this, we compared the p=0.01 event computed WSELs to those from the DMP. WSELs at bridge crossings are listed in Table 5.

Table 5. WSELs at bridge crossings

Creek (1)	River Station (2)	Location (3)	New WSEL (ft) (4)	DMP WSEL (ft) (5)	Difference (ft) (6)
Whitehouse	3.459	Elk Grove Florin Road	37.4	35.4[1]	2.0
Whitehouse	3.3595	UPRR	35.0	34.7	0.3
Whitehouse	3.159	Camden Parkway	34.8	34.6	0.2
Whitehouse	2.755	Sheldon Creek Drive	34.5	34.3	0.2
Whitehouse	2.6165	Bike Crossing	34.4	34.2	0.2
Whitehouse	2.5	Harding Hall Drive	34.4	34.2	0.2
Whitehouse	2.2455	Ski Lake Drive	33.9	33.8	0.1
Whitehouse	1.94	Low Flow Weir	32.3	32.2	0.1
Laguna	4.745	East Stockton	31.4	31.4	0.1
Laguna	3.5615	Bruceville Road	24.0	24.0	0.0

1. The 2007 Whitehouse Creek update computed WSEL was 37.0 based on updated topographic information downstream of RS 3.459.

The Whitehouse Creek WSEL from the DMP at RS 3.459 was updated based on topographic information used in the 2007 Whitehouse Creek update study. The 2007 calculated WSEL was 37.0 feet. Therefore the difference between the 2007 study and our calculated water surface elevation is 0.4 feet. In Whitehouse Creek the WSEL increases by a maximum of 0.4 feet downstream to the confluence with Laguna Creek. There is no noticeable increase in WSEL by the time Whitehouse Creek reaches the confluence with Laguna Creek.

Floodplain delineation

Floodplains for Whitehouse Creek and the unnamed tributary were delineated using HEC-GeoRAS and based on results from the hydraulic model. We built a ground surface triangulated irregular network (TIN) for floodplain mapping by combining the new survey data and contours used in the DMP. The HEC-GeoRAS delineated floodplain required post processing due to our limited survey data. The newly delineated floodplain for Whitehouse Creek and the unnamed tributary is shown in Figure 11.

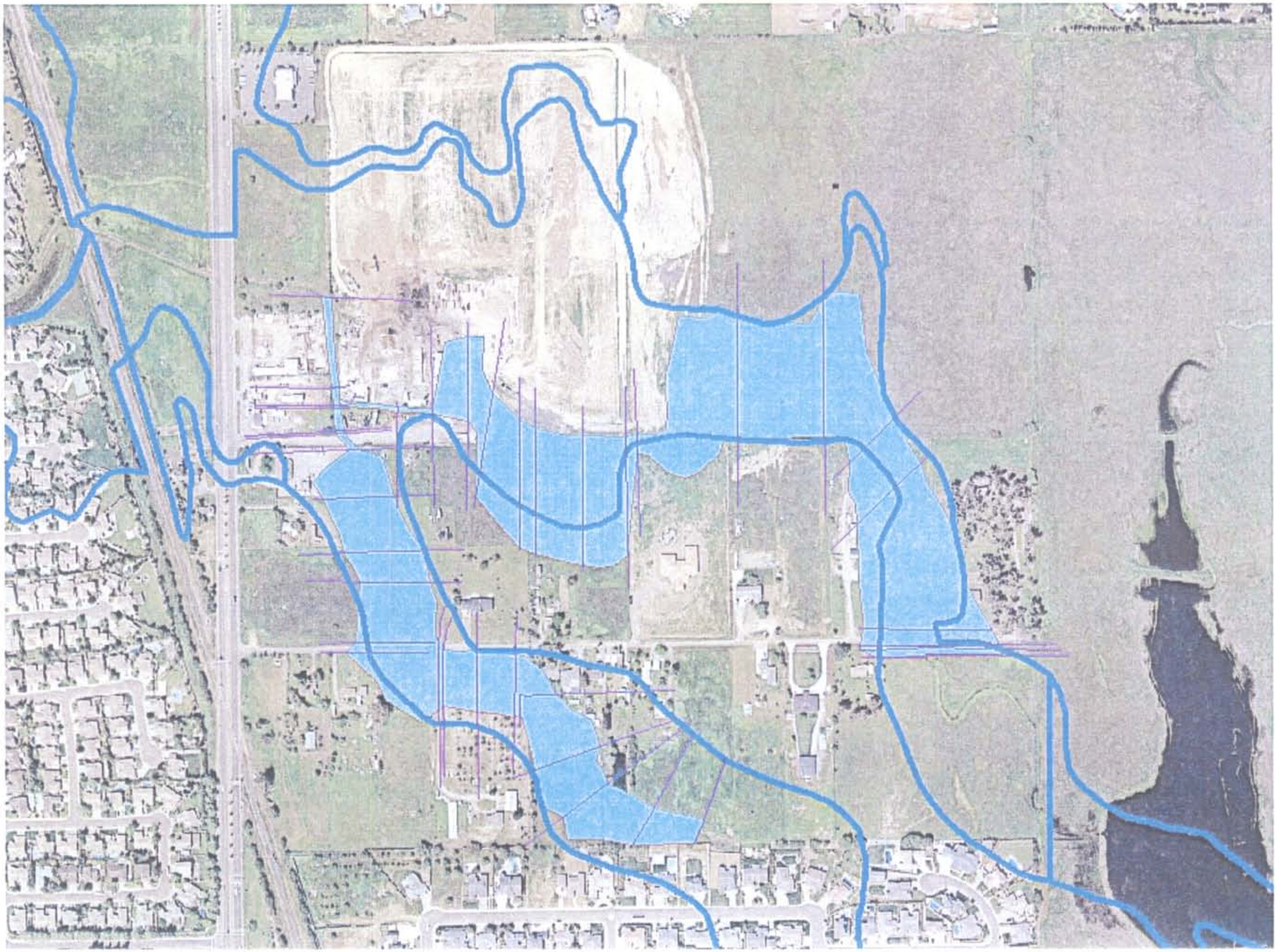

Figure 11. Floodplain for p=0.01 event, using a 1.44-square-mile storm centering, with approximate p=0.01 event FEMA floodplain boundary

CHAPTER 5. ELK GROVE CREEK

WATERSHED DESCRIPTION

Elk Grove Creek is located in the southeast portion of the City and flows generally from east to west, beginning just east of Grant Line Road and joining Laguna Creek just west of Highway 99 (See Figure 5-1). The contributing area for this creek is approximately 4,300 acres with elevations ranging between 66 feet in the east to 35 feet in the west.

EVALUATION OF ELK GROVE CREEK

Hydrologic Analysis for Elk Grove Creek

For the hydrologic analysis of Elk Grove Creek, a watershed model was used to transform design rainfall (of specified probability) over a given area to runoff hydrographs. The watershed model was configured to route the flows from the subsheds through Elk Grove Creek and to calculate the peak flows within the creek and key locations. The calculated peak flows are used in the hydraulic model of the channel as discussed below. Figure 5-2 presents the subshed locations used for the hydrologic model of the creek.

Hydrology for Existing Conditions

The existing conditions hydrology for Elk Grove Creek was developed using the most recent topographic data, storm drain system maps, land use designations, soil maps and aerial photographs. These sources were used to prepare the input data for a SacCalc hydrologic model for existing runoff conditions. The 2-year, 10-year and 100-year events were evaluated.

The area east of Waterman Road was originally modeled by Harris and Associates for the East Area Storm Drain Master Plan (EASDMP). This model was input directly into the SacCalc model that was developed for the remaining watershed. However, only buildout conditions were modeled for the EASDMP. To simulate existing flows from the East Elk Grove Area/rural region, a detention basin was added to the model just upstream of Waterman Road to reduce the peak 100-year flow to 513 cfs, which was estimated by others to be the existing peak 100-year flow at that location. See Figure 5-3 for a schematic of the SacCalc model. The subsheds that were copied from the EASDMP are L41801 through L41811. The hydrologic parameters for each subshed are shown in Table 5-1.

Table 5-1. Hydrologic Parameters for Elk Grove Creek Subsheds

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land Use, acres and Percent Impervious																	Average % Imp
						Highway, Parking	Comm./ Office	Ind.	HDR	Mobile	MDR	Resd, 8 to 10 du/ac	Resd, 6 to 8 du/ac	Resd, 4 to 6 du/ac	Resd, 3 to 4 du/ac	Resd, 2 to 3 du/ac	Resd, 1 to 2 du/ac	Resd, .05 to 1 du/ac	Resd, .02 to .05 du/ac	< .02 du/ac, recreation	OpenSpace, grassland/ag	Vacant	
						95%	90%	85%	80%	75%	70%	60%	50%	40%	30%	25%	20%	15%	10%	5%	2%	1%	
Existing Conditions																							
L41680	111.2	50	2,643	1,392	0.0028	10.0	8.9	65.7						0.0					0.5	5.2	0.9	20.1	66
L41690	104.2	47	2,791	1,783	0.0025	3.4		9.8						0.0						3.2	7.0	80.7	12
L41650	143.5	47	4,051	2,087	0.0012	15.1	0.3	25.8					33.6	9.1					0.3		23.7	35.7	40
L41640	317.8	45	4,593	1,813	0.0003	50.3	3.0				0.7			135.8					0.4		126.8	0.9	34
L41740	43.7	52	2,318	1,176	0.0016	1.6							0.0							7.3	26.3	8.5	6
L41660	232.7	50	5,213	3,614	0.0011	38.4	22.8	33.4	5.8		0.9	2.8	12.7	77.0		7.0		0.3	2.8	0.1	18.3	10.4	57
L41630	78.2	45	2,348	1,842	0.0030	12.2			5.5		3.8		1.7	43.8	0.2				0.4		10.3	0.3	48
L41620	150.6	46	2,258	1,835	0.0017	20.7	9.7		3.1		2.6		2.2	44.4					4.9		62.9	0.1	35
L41580	136.4	42	4,480	2,246	0.0028	22.3	24.7		2.6		4.2	0.4	0.2	26.2					0.2		53.6	2.1	44
L41560	108.7	40	2,657	2,042	0.0010	22.3	3.4				0.5			78.4							3.7	0.4	52
L41590	199.1	40	6,625	3,573	0.0007	35.6	44.5	2.1	8.2					61.1							37.9	9.8	54
L41550	228.3	38	5,198	2,657	0.0006	39.1	14.9	2.4	3.8		0.5		0.3	143.5					2.4		15.3	6.2	50
L41490	132.2	38	2,923	2,095	0.0017	33.7	61.1	4.3	6.8		8.3			8.0			0.8	0.2	0.3		4.8	3.9	80
L41440	243.6	35	3,374	2,139	0.0002	43.3	10.6		8.7				84.0	19.1				7.8	8.7		23.4	38.1	45
L41450	168.2	37	3,853	2,385	0.0020	39.4	64.6	0.6	3.2		0.0			11.1			0.8		1.4		10.1	37.1	62
L41430	120.5	35	2,084	1,402	0.0009	16.0	43.7		0.3		0.4										7.8	52.4	46
L41420	158.6	30	3,536	1,863	0.0008	20.0	11.8				45.1		1.0						1.3	1.0	37.5	40.8	40
L41801	111.3	64	3,359	1,720	0.0014	Specified Basin "n" = 0.056, Loss Computed, % Imp. Computed																	13
L41802	31.9	60	1,182	650	0.0017	Specified Basin "n" = 0.056, Loss Computed																	15
L41803	37.2	62	2,612	1,310	0.0023	Specified Basin "n" = 0.056, Loss Computed																	15
L41804	119.3	62	3,675	1,900	0.0016																100.0		2
L41805	47.6	58	2,777	1,420	0.0023	Specified Basin "n" = 0.056, Loss Computed																	15
L41806	96.6	54	4,188	2,150	0.0014								7.0		50.0				20.0		23.0		22
L41807	64.6	52	2,636	1,360	0.0015								37.0		46.0				10.0		7.0		52
L41808	102.5	52	3,781	1,920	0.0014			15.0					60.0								25.0		42
L41809	295.3	52	5,250	2,700	0.0015										4.0						96.0		1
L41810	94.2	52	4,423	2,250	0.0009								87.0								13.0		46
L41811	109.2	66	3,150	2,200	0.0019																100.0		2
L41910	166.6	50	2,856	1,461	0.0020	24.9	7.0						31.7	44.9					7.5	25.7	19.7	5.3	40
L41911	211.9	50	5,582	2,489	0.0004	41.0					0.7		11.3	79.9		37.2		2.0	14.8		20.6	4.5	42
L41912	162.1	50	4,729	2,951	0.0008	29.3							14.5	26.8	0.1	66.2			0.2		17.3	7.7	39
Buildout Conditions																							
L41680	111.2	50	2,643	1,392	0.0028	28.6	8.9	54.5	0.3											1.8	1.8	15.3	73.7
L41690	104.2	47	2,791	1,783	0.0025	3.4		9.8	0.0											3.2	7.0	80.7	12.2
L41650	143.5	47	4,051	2,087	0.0012	28.0	0.3	19.5	41.1		1.8										23.7	29.0	54.7
L41640	317.8	45	4,593	1,813	0.0003	50.3	3.0		130.8		0.7		3.0	0.2				2.2	0.1		126.8	0.9	50.3
L41740	43.6	52	2,318	1,176	0.0016	1.6			0.1											7.3	26.3	8.5	5.8
L41660	232.7	50	5,213	3,614	0.0011	42.8	14.8	33.4	100.5		4.0	0.2	3.0	0.3				1.5	2.5		13.9	15.6	72.3
L41630	78.2	45	2,348	1,842	0.0030	14.8			46.4		5.5		0.6	0.5					0.3		9.6	0.5	71.3
L41620	150.6	46	2,258	1,835	0.0017	22.9	9.7		44.5		5.2			0.5					4.9		62.9	0.1	47.5
L41580	136.4	42	4,480	2,246	0.0028	23.5	24.8		25.8		6.2	0.4	0.5	0.7							53.1	1.5	52.4
L41560	108.7	40	2,657	2,042	0.0010	22.3	3.4		74.2		4.2		0.7								3.7	0.2	80.0
L41590	199.0	40	6,625	3,573	0.0007	47.6	39.0	2.1	70.0			0.2		0.3							29.5	10.4	69.8
L41550	228.4	38	5,198	2,657	0.0006	44.3	12.8		133.3		14.4		0.7	0.5					2.2		15.3	4.9	75.1
L41490	132.2	38	2,923	2,095	0.0017	35.5	61.1	4.3	22.7		0.4							0.9	0.3		4.8	2.2	84.1
L41440	243.6	35	3,374	2,139	0.0002	46.6	7.5		111.3		1.9		0.5					5.9	8.7		23.5	37.8	59.2
L41450	168.2	37	3,853	2,385	0.0020	65.4	61.1	0.6	13.9		0.9							0.8			12.6	13.0	77.2
L41430	120.5	35	2,084	1,402	0.0009	28.7	34.5		2.1		1.2		2.0								8.7	43.4	51.8
L41420	158.6	30	3,536	1,863	0.0008	19.9	19.7		47.2		27.5							1.3			37.6	5.5	59.7
L41801	111.3	64	3,359	1,720	0.0014	Specified Basin "n" = 0.056, Loss Computed, % Imp. Computed																	13.5
L41802	31.9	60	1,182	650	0.0017	Specified Basin "n" = 0.056, Loss Computed																	15.0
L41803	37.2	62	2,612	1,310	0.0023	Specified Basin "n" = 0.056, Loss Computed																	15.0
L41804	119.3	62	3,675	1,900	0.0016																100.0		1.7
L41805	47.6	58	2,777	1,420	0.0023	Specified Basin "n" = 0.056, Loss Computed																	15.0
L41806	96.6	54	4,188	2,150	0.0014								7.0		50.0				20.0		23.0		21.7
L41807	64.6	52	2,636	1,360	0.0015								37.0		46.0				10.0		7.0		51.7
L41808	102.5	52	3,781	1,920	0.0014			15.0					60.0								25.0		42.2
L41809	295.3	52	5,250	2,700	0.0015										4.0						96.0		1.1
L41810	94.2	52	4,423	2,250	0.0009								87.0								13.0		46.5
L41811	109.2	66	3,150	2,200	0.0019																100.0		1.8
L41910	166.7	50	2,856	1,461	0.0020	25.4	5.6		86.2				0.2					1.6	6.7		19.7	21.4	59.8
L41911	211.8	50	5,582	2,489	0.0004	40.3			118.4				0.7	10.6				2.0	10.3		20.6	8.9	65.8
L41912	162.1	50	4,729	2,951	0.0008	29.3			104.9		0.5	1.6	0.3	0.6							17.3	7.7	70.2

An aerial reduction factor was applied to the rainfall totals based on storm area to properly depict the rainfall that will generate the peak flows coming into Elk Grove Creek. The computed runoff hydrographs for seven storm centerings (contributing basin areas) were used for this analysis. This analysis yielded the peak flows that were then entered into HEC-RAS for the water surface profile calculations. Table 5-2 presents the calculated peak flows in Elk Grove Creek for existing conditions.

Table 5-2. Elk Grove Creek SacCalc Peak Flows – Existing Conditions

SacCalc Node	HEC-RAS Creek Station	Total Contributing Area, sq mi	2-year Peak Flow, cfs	10-year Peak Flow, cfs	100-year Peak Flow, cfs
R4184	5.5243	1.59	183	356	604
L43745	5.229	1.73	161	309	512
L43635	4.147	4.19	325	688	1,147
L43565	3.188	4.81	370	771	1,194
L43545	2.434	5.48	406	805	1,179
L43435	1.148	6.52	453	901	1,269
L43415	0.34	6.76	458	909	1,271

Hydrology for Buildout Conditions

The hydrologic model for buildout conditions was developed by updating the existing conditions model with buildout land use information. The area east of Waterman Road was kept the same as the existing conditions model since future development in that area will be required to mitigate for potential flow increases. The SacCalc schematic for buildout conditions is the same as the existing conditions schematic shown on Figure 5-3. The input for the buildout conditions model can also be found in Table 5-1.

The peak flow results for buildout conditions are shown in Table 5-3. These are the values that were used to calculate water surface profiles in the creek for buildout conditions. The 10-year and 100-year peak flows are predicted to increase by less than ten percent above existing condition rates.

Table 5-3. Elk Grove Creek SacCalc Peak Flows – Buildout Conditions

SacCalc Location	HEC-RAS Creek Station	Total Contributing Area, sq mi	2-year Peak Flow, cfs	10-year Peak Flow, cfs	100-year Peak Flow, cfs
R4184	5.5243	1.59	183	356	604
L43745	5.229	1.73	161	309	512
L43635	4.147	4.19	376	741	1,206
L43565	3.188	4.81	435	826	1,245
L43545	2.434	5.48	479	849	1,215
L43435	1.148	6.52	533	936	1,332
L43415	0.34	6.76	532	950	1,309

Hydraulic Analysis

A steady-state hydraulic model was prepared for Elk Grove Creek using HEC-RAS. A steady-state analysis is adequate for Elk Grove Creek since there are no off-channel detention basins within the modeled reach and the flow attenuation from overbank flooding can be adequately represented using hydrologic routing. This approach yielded good results based on a comparison with observed high water marks from the storm of December 2005.

The hydraulic data for this project was developed using a combination of existing HEC-2 and HEC-RAS models from previous studies, 2-foot contour topographic mapping provided by the City, and field surveys of several bridges and culverts. Peak flow data from the SacCalc models were used to calculate water surface elevations for the 2-year, 10-year, and 100-year events. Figure 5-4 presents the overall layout of the hydraulic model. More detailed maps showing the model cross section locations are provided on Figures 5-6a through 5-6e.

The starting water surface elevation at the downstream boundary of the HEC-RAS model, just upstream of Laguna Creek, is based on normal depth calculations. The peak flows in Elk Grove Creek are not expected to occur concurrently with the peak flows in Laguna Creek. Therefore, setting the starting water surface elevation equal to the peak water surface elevation in Laguna Creek is considered to be overly conservative. As a sensitivity analysis, hydraulic calculations were performed with varying starting water surface elevations and the water surface profiles for the various runs converged quickly. Therefore, using normal depth to determine the starting water surface elevation is considered to be reasonable.

Existing Conditions Hydraulic Analysis

Table 5-4 shows the calculated flows and water surface elevations at key locations for existing conditions. Figures 5-5a through 5-5d show the water surface profiles along Elk Grove Creek. Included on the profiles are the observed water surface elevations that were surveyed after the December 2005 storm event. The City estimated that the December 2005 storm was approximately a 45-year event. The observed water elevations from that storm are between the calculated water surface elevations for the 10-year and 100-year storms, indicating that the calculated water surface elevations are reasonable.

Table 5-4. Elk Grove Creek Existing Conditions Hydraulic Results

HEC-RAS Creek Station	Location	2-year peak flow, cfs	2-year max water surface, ft	10-year peak flow, cfs	10-year max water surface, ft	100-year peak flow, cfs	100-year max water surface, ft
1.1035	Laguna Boulevard	453	26.6	901	27.7	1,269	28.4
2.078	Laguna Springs Drive	406	29.7	805	31.2	1,179	32.1
2.366	Highway 99	406	31.7	805	32.8	1,179	33.7
2.74749	Emerald Vista	370	35.2	771	36.7	1,194	38.0
3.133	Elk Grove Blvd	370	36.3	771	38.2	1,194	39.5
3.8505	Elk Grove Florin	325	36.9	688	39.2	1,147	40.6
4.28649	Markofer School Rd	161	38.2	309	40.5	313	42.2
4.67	Falcon Meadow Dr	161	39.2	309	41.4	512	43.7
4.8495	UPRR	161	41.0	309	42.3	512	44.3
5.3195	Waterman Road	161	43.1	309	44.6	512	46.5

Note: Water surface elevations are based on NGVD29.

The results from the HEC-RAS model indicate that significant overbank flooding may occur along the creek between Highway 99 and Falcon Meadow Way (see Figures 5-5c and 5-5d). Because the accuracy of the available topographic mapping is limited (2-foot contours), the extent of the overbank flooding is approximate.

Buildout Conditions Hydraulic Analysis

For buildout conditions, there were no changes to the geometry of the river system. The only difference from existing conditions is that buildout condition flows are used in the analysis. The area east of Waterman Road includes large areas anticipated for future development. The City is requiring development projects in that area to mitigate for potential flow increases due to development. Therefore, the maximum flow from the east at Waterman road is unchanged from existing conditions.

West of Waterman Road, there are smaller undeveloped areas scattered through the watershed and infill development is expected. Runoff from this area was based on full buildout without mitigation measures for future development.

Table 5-5 summarizes the results from HEC-RAS model for buildout conditions. Comparisons of existing and buildout condition flows and water surface elevations are presented on Tables 5-6 and 5-7. For the 10-year storm, buildout condition peak flows are predicted to increase by 4 percent to 8 percent between Laguna Creek and Elk Grove Boulevard. For the 100-year storm, buildout condition peak flows are predicted to increase by 3 percent to 5 percent in the same reach. These flow increases result in an increase in water surface elevations between 0.1 to 0.2 feet.

Table 5-5. Elk Grove Creek Buildout Conditions Hydraulic Results

HEC-RAS Creek Station	Location	2-year peak flow, cfs	2-year max water surface ft	10-year peak flow cfs	10-year max water surface ft	100-year peak flow cfs	100-year max water surface ft
1.1035	Laguna Boulevard	533	26.8	936	27.8	1,332	28.5
2.078	Laguna Springs Drive	479	30.1	849	31.3	1,215	32.2
2.366	Highway 99	479	31.9	849	32.9	1,215	33.8
2.74749	Emerald Vista	435	35.5	826	36.8	1,245	38.1
3.133	Elk Grove Blvd	435	36.7	826	38.4	1,245	39.7
3.8505	Elk Grove Florin	376	37.3	741	39.2	1,206	40.8
4.28649	Markofer School Rd	161	38.5	309	40.6	512	42.4
4.67	Falcon Meadow Dr	161	39.4	309	41.5	512	43.9
4.8495	UPRR	161	41.0	309	42.3	512	44.4
5.3195	Waterman Road	161	43.1	309	44.6	512	46.5

Note: Water surface elevations are based on NGVD29.

Table 5-6. Comparison of Flows between Existing and Buildout Conditions

HEC-RAS Creek Station	Location	2-year peak flow existing, cfs	2-year peak flow buildout, cfs	10-year peak flow existing, cfs	10-year peak flow buildout, cfs	100-year peak flow existing, cfs	100-year peak flow buildout, cfs
1.1035	Laguna Boulevard	453	533	901	936	1,269	1,332
2.078	Laguna Springs Drive	406	479	805	849	1,179	1,215
2.366	Highway 99	406	479	805	849	1,179	1,215
2.74749	Emerald Vista	370	435	771	826	1,194	1,245
3.133	Elk Grove Blvd	370	435	771	826	1,194	1,245
3.8505	Elk Grove Florin	325	376	688	741	1,147	1,206
4.28649	Markofer School Rd	161	161	309	309	512	512
4.67	Falcon Meadow Dr	161	161	309	309	512	512
4.8495	UPRR	161	161	309	309	512	512
5.3195	Waterman Road	161	161	309	309	512	512

Table 5-7. Comparison of Water Surface Elevations between Existing and Buildout

HEC-RAS River Station	Location	2-year max water surface existing, ft	2-year max water surface buildout, ft	10-year max water surface existing, ft	10-year max water surface buildout, ft	100-year max water surface existing, ft	100-year max water surface buildout, ft
5.3195	Waterman Road	43.1	43.1	44.6	44.6	46.5	46.5
4.8495	SPRR	41.0	41.0	42.3	42.3	44.3	44.4
4.67	Falcon Meadow Dr	39.2	39.4	41.4	41.5	43.7	43.9
4.28649	Markofer School Rd	38.2	38.5	40.5	40.6	42.2	42.4
3.8505	Elk Grove Florin	36.9	37.3	39.2	39.2	40.6	40.8
3.133	Elk Grove Blvd	36.3	36.7	38.2	38.4	39.5	39.7
2.74749	Emerald Vista	35.2	35.5	36.7	36.8	38.0	38.1
2.366	Highway 99	31.7	31.9	32.8	32.9	33.7	33.8
2.078	Laguna Springs Drive	29.7	30.1	31.2	31.3	32.1	32.2
1.1035	Laguna Boulevard	26.6	26.8	27.7	27.8	28.4	28.5

Other Hydrologic and Hydraulic Studies of Elk Grove Creek

A number of additional studies of the Elk Grove Creek system were performed after the preparation of the hydrologic and hydraulic modeling described above. The studies were focused on refining the hydraulic analysis to better understand the potential flooding along the creek and on evaluating solutions to reduce or eliminate the flooding. These studies are described below.

Analysis of North and South Branch Elk Grove Creek Detention Basins

David Ford Consulting Engineers evaluated the potential benefits of modifying four existing detention basins in the upper Elk Grove Creek watershed. Each of the four existing detention basins are located just upstream of Waterman Road. The goal of the study was to determine whether modifications to the existing detention basins could reduce flood flows in Elk Grove Creek for frequent events (10-year storm and smaller) without causing adverse impacts to flood flows during infrequent events. Three alternatives were considered that were based on modifying the detention basin outlets to produce increased storage for the more frequent storm events. After evaluating the three alternatives with SacCalc and HEC-RAS modeling, Ford determined that the potential reductions to peak flows in Elk Grove Creek during frequent storm events were very small (< 3%) and these reductions did not translate into meaningful reductions in water surface elevations in the creek. Therefore, these detention alternatives have been dropped from further consideration. A report that describes the Ford study in detail is provided as Attachment 5A.

Refined Elk Grove Creek Modeling and Preliminary Evaluation of Improvements

For this study, West Yost refined the HEC-RAS hydraulic model for Elk Grove Creek using field survey data collected by Psomas in 2008. The field survey produced cross sections along the creek at approximately 200 foot intervals between Laguna Boulevard and Elk Grove Florin Road. With the refined model, West Yost assessed the potential flood control benefits of the following improvements:

- Adding another box culvert under Highway 99.
- Constructing a flood control detention basin just upstream of Highway 99.
- Constructing a two flood control detention basins upstream of Elk Grove Florin Road.
- Constructing a flood control detention basin just downstream of the UPRR.

The results of the study revealed that none of the individual flood control improvements alone would significantly reduce flooding. In combination, the benefits would increase, but significant flood risk would still remain. These findings led the City to commission a study to evaluate a multi-objective solution to the flooding problem that considered the improvements listed above in addition to other improvements. This study is described below.

Elk Grove Creek Conceptual Hydrodynamic Modeling

For this study, cbec eco-engineering assisted the City with the evaluation of a "no constraints" alternative for multi-objective channel management of Elk Grove Creek. The purpose of this "no constraints" alternative was to investigate opportunities for flood attenuation, ecosystem and water quality enhancement. Using the previously developed hydrologic and hydraulic modeling as a starting point, cbec developed a 2-dimensional hydrodynamic model of the creek using MIKE21FM. The model was used to evaluate the 2-year and 100-year storms for both existing conditions and for an alternative to reduce flooding and provide ecosystem improvements.

For existing conditions, the 2D model results showed reasonable agreement with those from the prior HEC-RAS modeling. However, some differences were noted. In general, the 2D model predicted a larger head loss across the bridges and culverts than those predicted by HEC-RAS. This resulted in higher water surface elevations upstream of the bridges and greater backwater conditions upstream. A maximum difference of approximately 1.5 feet is predicted upstream of Elk Grove–Florin Road. On average, the water surface elevations predicted by the 2D model are about 0.3 feet higher on average compared to those produced by the HEC-RAS model. The floodplain limits predicted by the 2D model can be compared to those predicted by HEC-RAS on Figures 5-6a through 5-6e.

The "no constraints" alternative consisted of the following elements:

- 10 multi-objective detention basins
- Channel modifications to develop a defined low flow channel and a lowered maintenance path
- Channel realignment to provide a more sinuous path
- Lower of adjacent floodplain areas along the creek

Modeling results indicate the "no-constraints" alternative could produce significant reductions in the 100-year water surface elevations along the creek. A maximum reduction of 4.2 feet is predicted between Waterman Road and the SPRR Bridge. Downstream from this reach, 3.4 feet of reduction is the maximum predicted benefit, decreasing to 2 feet and less downstream of Falcon Meadow Road. On average, the alternative would produce a water surface elevation reduction of 1.6 feet. These reductions would eliminate most of the predicted 100-year flooding, although some residual flooding would remain.

An estimate of the cost for excavation and land acquisition was included in the study. The total cost for these items was estimated to be approximately $70.8 million. A report that fully describes the study is included as Attachment 5B.

Recommendations for Elk Grove Creek System

Although the remaining infill development anticipated west of Waterman Road is not predicted to cause large increases to the peak flows and water surface elevations in Elk Grove Creek, because there are existing flooding problems along the creek so it is important to not increase the problem. Therefore, it is recommended that any potential increase in peak runoff due to future development be fully mitigated.

The storage volume required to mitigate for potential increases in runoff due to future infill development west of Waterman Road was estimated by first calculating the volume in the buildout condition hydrograph above the existing condition peak flow rate for a 100-year storm. This volume was calculated to be approximately 12 acre-feet. The total required storage volume will be more than this value due to the inherent inefficiencies in the operation of a detention basin. Therefore, the total required storage volume was assumed to be 24 acre-feet. To estimate the excavation quantity for the detention storage, it was assumed that the required storage volume will be spread between two or three detention basins and that the basins will have 1.5 feet of freeboard during a 100-year storm. Note that this estimated detention volume would only provide mitigation for the infill development and would not reduce the existing flooding problem.

The evaluations of drainage system improvements as described above revealed that there are opportunities to reduce the existing flooding problem along Elk Grove Creek, but the solutions are costly. As indicated by the differences in the results between the HEC-RAS and 2D modeling, there is some uncertainty in the water surface elevations predicted by the different models developed for the creek system. To help reduce this uncertainty, it is recommended that future modeling efforts include calibration to a historic storm event, such as the December 2005 event. Surveyed high water marks at various points along the creek are available for that storm. A calibrated model would provide some assurance that the flood risk along the creek is characterized accurately and that any facilities proposed to reduce the flood risk are not oversized (or undersized).

The “no constraints” alternative shows promising results for flood protection and ecosystem enhancement. However, implementation of the full alternative will be difficult due to its cost and other constraints. Additional analyses should be performed to test the benefits of individual elements and the facilities that provide the most significant benefits at a reasonable cost (i.e. low cost-benefit ratio) should be given priority for implementation. Improvements that provide multiple benefits such as flood control, stormwater quality treatment, and ecosystem enhancement are desirable because these improvements may provide opportunities to obtain grant funding.

EVALUATION OF EXISTING PIPELINES

Existing pipelines within the City’s arterial roadways with diameters 27 inches and greater were evaluated during this study. In addition, all of the pipelines meeting that size criterion within the “Old Town” area were also evaluated. The Old Town area is bounded by Waterman Road, Bond Road, Highway 99, and Grant Line Road and much of this area lies within the Elk Grove Creek watershed. Seventeen existing trunk pipelines in the Elk Grove Creek watershed met the criterion that triggered a detailed evaluation. Figures 5-7 through 5-12 show the existing pipelines that were evaluated.

Hydrologic Analysis of Existing Pipelines

SacCalc models were prepared to calculate the 2-year, 10-year, and 100-year flows into the pipe systems. The SacCalc models for the existing pipelines are separate from the model used to calculate flows in the creek. The pipeline models are more detailed to better define the variation in flow along the pipeline.

Many of the subsheds served by the existing pipelines are completely developed or nearly so. For those cases, flows were only calculated for buildout conditions. A few subsheds have significant areas of undeveloped land. The pipe systems serving those watersheds were evaluated and were found to meet the performance criteria under buildout conditions. Therefore, existing flow rates were not calculated for any subsheds.

Figures 5-7 through 5-12 present the subshed boundaries used for the flow calculations. Table 5-8 presents the key hydrologic parameters for each subshed under buildout land use conditions. Table 5-9 presents the calculated peak flows from each subshed for the three storm events.

Hydraulic Analysis of Existing Pipelines

Hydraulic models of the nine pipe systems were created using XPSWMM. Calculated flows and water surface elevations for the 2-year, 10-year, and 100-year storm events are summarized on Table 5-10. Calculated peak flows are presented on Table 5-11. As Table 5-10 shows, the City's performance criteria for existing pipelines are met for all but two pipelines. Existing Pipeline EGC9 has one location where the 10-year water surface elevation is above the curb and one location where the 100-year water surface elevation is just above a building pad. Existing Pipeline EGC15 has three locations where the 10-year water surface elevations are above the curb.

Improvements to Existing Pipelines

Pipe improvements are necessary to eliminate the predicted flooding along Existing Pipelines EGC9 and EGC15. To eliminate the predicted flooding, 2,400 feet of pipeline will need to be upsized. The pipe improvements required to bring the existing pipe systems in compliance with the performance criteria are shown on Figures 5-13 and 5-14. These improvements are considered preliminary. They are adequate for development of a Capital Improvement Plan, but the ultimate improvements will be defined from a more detailed design study and could vary from those recommended in this study.

EVALUATION OF FUTURE PIPELINES

West of Waterman Road, the Elk Grove Creek watershed is mostly developed and no major trunk pipelines are anticipated in the future to serve new development. East of Waterman Road, significant development is anticipated and future trunk pipelines will be required. The discussion of the anticipated future facilities in the area east of Waterman Road is included with the discussion of the East Elk Grove area/rural region in Chapter 6.

PRELIMINARY IMPROVEMENTS

As discussed above, improvements are required in the Elk Grove Creek watershed west of Waterman Road. These improvements are summarized below and on Table 5-12. These improvements are considered preliminary. They are adequate for development of a Capital Improvement Plan, but the ultimate improvements will be defined from a more detailed design study and could vary from those recommended in this study.

- Improvements to Existing Pipeline EGC9 (See Figure 5-13).

Table 5-8. Hydrologic Parameters for Existing Pipeline Models EGC1 - EGC17

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land Use, acres and Percent Impervious													Average % Imp.
						Highway, Parking	Comm./ Office	Indust.	HDR	Mobile Home Park	MDR	Ext. Ind., Resd, 8-10 du/ac	Resd, 6-8 du/ac, School	Resd, 4-6 du/ac	Resd, 3-4 du/ac	Rural Res.	Park	Open	
						95%	90%	85%	80%	75%	70%	60%	50%	40%	30%	10%	5%	2%	
Buildout Conditions																			
EG120	7.5	28	600	270	0.0003		4.1											3.4	50.4
EG130	38.7	30	2,290	1,400	0.0017									38.7					40.0
EG150	5.7	28	480	270	0.0010		0.6											5.1	10.8
EG160	3.7	29	520	280	0.0039		3.7												90.0
EG210	5.3	28	660	290	0.0031		0.3										5.0		9.3
EG220	22.1	30	2,070	870	0.0019									22.1					40.0
EG230	3.5	28	340	170	0.0015		2.1							1.4					70.0
EG240	1.2	30	660	290	0.0008	1.1											0.1		86.0
EG310	14.9	30	970	460	0.0005		14.9												90.0
EG320	11.1	31	920	500	0.0022		8.9							2.2					80.0
EG330	3.5	30	540	300	0.0009		3.5												90.0
EG420	16.9	32	870	360	0.0011		16.9												90.0
EG430	37.7	32	2,420	1,090	0.0008		37.7												90.0
EG440	15.8	32	1,600	760	0.0006		15.8												90.0
EG460	40.5	32	2,380	1,080	0.0008		40.5												90.0
EG520	43.2	34	1,490	840	0.0013		43.2												90.0
EG540	11.4	34	950	430	0.0042		11.4												90.0
EG550	10.1	34	1,110	580	0.0018		10.1												90.0
EG560	9.5	34	1,420	850	0.0014		9.5												90.0
EGC6110	13.6	33	1,230	620	0.0016		7.3							5.5			0.8		64.8
EGC6120	16.0	35	1,050	520	0.0057		14.3							1.7			0.0		84.8
EGC6130	31.7	34	1,440	720	0.0014		0.0							26.6			5.1		34.4
EGC7110	17.4	33	1,470	730	0.0029		0.0				0.0			17.4				0.0	40.0
EGC7120	23.4	35	1,500	750	0.0024		0.0				0.0			23.4				0.0	40.0
EGC7130	11.2	34	1,200	530	0.0025		0.0				8.2			3.0				0.0	62.0
EGC7140	14.2	34	1,140	550	0.0011		0.0				0.0			14.2				0.0	40.0
EGC7150	10.1	35	880	590	0.0052		5.9				4.1			0.0				0.0	81.8
EGC7160	7.1	36	790	390	0.0004		3.5				0.0			0.0				3.6	45.3
EG810	12.4	35	550	249	0.0018		12.4												90.0
EG820	6.4	39	780	355	0.0128		6.4												90.0
EG830	16.3	39	1,700	904	0.0006		16.3												90.0
EG840	3.1	39	670	371	0.0015		3.1												90.0
EG0910	6.3	36	960	430	0.0021		6.3												90.0
EG0920	15.4	38	1,240	580	0.0009	2.2	7.7						5.5						76.3
EG0930	20.2	40	1,320	540	0.0015	4.4	10.1						5.7						79.9
EG0940	19.6	39	1,320	630	0.0033									19.6					40.0
EG0950	31.9	40	1,670	820	0.0028									31.9					40.0
EG1010	28.3	40	1,550	760	0.0013		13.6		0.5					14.2					64.7
EG1020	30.8	40	2,380	1,220	0.0008									30.8					40.0
EG1025	24.9	40	1,540	770	0.0013					15.6				4.3			4.9		55.1
EG1040	41.7	40	3,000	1,480	0.0013	6.7				3.3				31.8					51.6
EG1105	2.6	40	460	250	0.0087		2.2											0.5	74.2
EG1110	3.8	40	490	270	0.0020		3.8												90.0
EG1120	15.7	42	1,360	790	0.0029		8.2							7.5					66.1
EG1140	32.6	40	2,220	1,170	0.0009		1.0						1.5	26.8			3.3		38.4
EG1160	9.1	44	1,270	690	0.0008				1.8				7.3						56.0
EG1210	7.6	44	1,370	759	0.0044		2.8					0.0	4.8	0.0					64.6
EG1220	17.6	47	1,280	583	0.0016		9.6					0.7	0.0	7.2					68.2
EG1230	34.8	47	1,290	664	0.0008		11.0					1.7	0.0	22.1					56.8
EG1310	8.6	41	1,130	514	0.0009									8.6					40.0
EG1320	8.3	42	890	404	0.0011									8.3					40.0
EG1330	37.2	43	1,740	792	0.0007									37.2					40.0
EG1510	14.2	43	720	326	0.0036		0.0							14.2					40.0
EG1520	20.3	43	1,120	507	0.0032		0.0							20.3					40.0
EG1530	29.3	45	1,400	636	0.0014		4.2							25.1					47.2
EG1605	29.6	46	2,000	910	0.0020									29.6					40.0
EG1610	22.0	47	1,080	489	0.0014									22.0					40.0
EG1615	10.5	45	1,120	510	0.0018								3.3	7.2					43.1
EG1620	8.3	46	760	347	0.0050									8.3					40.0
EG1625	11.9	48	1,070	487	0.0009									11.9					40.0
EG1635	20.1	45	1,070	486	0.0003									20.1					40.0
EG1645	12.9	45	1,260	574	0.0020									12.9					40.0
EG1650	124.7	45	4,570	2,075	0.0006									5.4			119.4		6.5
EG1715	18.0	47	790	359	0.0022							6.6		6.8				4.6	37.5
EG1720	37.7	47	1,780	811	0.0008							21.8		9.2			6.8		45.3
EG1725	13.9	49	950	434	0.0021							13.9		0.0					60.0
EG1735	36.9	48	2,130	969	0.0016			32.7						0.0				4.2	75.5
EG1740	29.6	47	1,600	728	0.0013							29.6		0.0					60.0
EG1765	11.0	47	560	254	0.0036									11.0					40.0
EG1770	27.4	48	1,870	849	0.0005									27.4					40.0

Table 5-9. Calculated Subshed Flows for Existing Pipelines EGC1 - EGC17

Subshed	Area, acres	Buildout Condition Flows, cfs		
		2-Year	10-Year	100-Year
EG120	7.5	6	10	17
EG130	38.7	21	40	57
EG150	5.7	4	8	13
EG160	3.7	4	8	12
EG210	5.3	4	7	13
EG220	22.1	14	25	37
EG230	3.5	4	7	11
EG240	1.2	1	2	3
EG310	14.9	12	23	34
EG320	11.1	10	18	28
EG330	3.5	3	6	10
EG420	16.9	15	29	44
EG430	37.7	25	45	65
EG440	15.8	11	21	31
EG460	40.5	27	49	70
EG520	43.2	33	61	90
EG540	11.4	11	21	31
EG550	10.1	7	14	20
EG560	9.5	9	16	24
EGC6110	13.6	10	19	29
EGC6120	16.0	15	28	42
EGC6130	31.7	20	37	56
EGC7110	17.4	12	23	33
EGC7120	23.4	16	30	44
EGC7130	11.2	9	17	25
EGC7140	14.2	10	19	27
EGC7150	10.1	9	17	27
EGC7160	7.1	5	9	15
EG810	12.4	13	24	38
EG820	6.4	7	13	20
EG830	16.3	11	21	30
EG840	3.1	3	6	9
EG0910	6.3	6	11	16
EG0920	15.4	12	22	32
EG0930	20.2	16	30	45
EG0940	19.6	14	27	40
EG0950	31.9	21	40	58
EG1010	28.3	20	37	54
EG1020	30.8	16	30	43
EG1025	24.9	22	41	58
EG1040	41.7	16	30	46
EG1105	2.6	3	5	9
EG1110	3.8	4	8	12
EG1120	15.7	12	22	33
EG1140	32.6	17	32	47
EG1160	9.1	6	12	17
EG1210	7.6	6	11	17
EG1220	17.6	14	25	38
EG1230	34.8	24	45	65
EG1310	8.6	6	11	17
EG1320	8.3	6	12	18
EG1330	37.2	22	41	59
EG1510	14.2	12	24	36
EG1520	20.3	16	29	44
EG1530	29.3	20	38	55
EG1605	29.6	18	34	49
EG1610	22.0	16	30	45
EG1615	10.5	8	15	22
EG1620	8.3	7	14	21
EG1625	11.9	8	16	23
EG1635	20.1	13	24	35
EG1645	12.9	9	17	26
EG1650	124.7	28	51	83
EG1715	18.0	14	27	42
EG1720	37.7	22	41	62
EG1725	13.9	12	22	33
EG1735	36.9	23	42	63
EG1740	29.6	20	38	56
EG1765	11.0	10	20	30
EG1770	27.4	15	29	41

Table 5-10. Calculated Water Surface Elevations for Existing Pipelines EGC1 - EGC17 (NGVD29)

Node & Conduit Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline No. EGC1							
EG100	N/A	N/A	26.2	26.2	27.2	-	-
EG110	N/A	N/A	26.5	27.0	28.0	-	-
EG120	28.5	30.0	26.7	27.7	28.7	-	-
EG130	29.6	30.0	26.9	28.2	29.2	-	-
EG140	28.5	30.0	26.9	28.3	29.1	-	-
EG150	29.3	30.0	27.1	28.9	29.2	-	-
EG160	28.5	30.0	26.9	28.3	29.1	-	-
Pipeline No. EGC2							
EG200	N/A	N/A	26.0	26.4	27.0	-	-
EG210	28.9	28.0	26.0	26.6	27.3	-	-
EG220	28.5	29.0	26.2	27.0	28.0	-	-
EG230	29.6	29.0	26.2	27.2	28.3	-	-
EG240	29.5	30.0	26.2	27.1	28.0	-	-
Pipeline No. EGC3							
EG300	N/A	N/A	27.0	27.0	28.0	-	-
EG310	29.3	30.0	27.3	28.2	29.3	-	-
EG320	29.4	30.0	27.6	28.9	29.9	-	-
EG330	30.3	30.5	27.7	29.2	30.1	-	-
Pipeline No. EGC4							
EG400	33.0	N/A	28.2	28.2	28.3	-	-
EG410	33.5	30.5	28.3	28.5	28.8	-	-
EG420	32.3	30.5	28.7	29.2	29.9	-	-
EG430	30.8	32.0	28.9	29.9	30.9	-	-
EG440	32.7	30.5	28.2	28.3	28.6	-	-
EG450	31.8	30.5	28.4	28.7	29.4	-	-
EG460	30.5	32.0	28.7	29.4	30.5	-	-
Pipeline No. EGC5							
EG500	N/A	32.3	28.7	28.7	29.2	-	-
EG520	33.8	32.3	29.2	30.0	31.3	-	-
EG530	33.7	32.7	29.5	30.4	31.7	-	-
EG540	34.1	33.0	29.8	31.1	32.2	-	-
EG550	33.2	32.8	30.2	31.7	32.5	-	-
EG560	32.0	33.1	30.4	32.0	32.7	-	-
Pipeline No. EGC6							
EG6100	32.5	32.0	29.5	29.5	30.5	-	-
EG6105	34.5	32.0	29.5	29.5	30.7	-	-
EG6110	34.5	32.5	29.7	29.9	31.4	-	-
EG6115	36.5	34.0	30.1	30.4	31.5	-	-
EG6120	33.5	34.0	31.1	31.7	32.4	-	-
EG6125	32.4	34.0	31.3	31.8	32.3	-	-
EG6130	32.7	34.2	31.7	32.1	32.6	-	-
Pipeline No. EGC7							
EGC7100	34.5	33.7	30.0	30.0	31.0	-	-
EGC7110	33.1	34.5	30.6	31.1	32.3	-	-
EGC7120	33.8	35.5	31.3	32.0	33.0	-	-
EGC7130	33.2	34.9	31.8	32.4	33.0	-	-
EGC7140	34.5	36.0	32.0	32.5	33.1	-	-
EGC7150	33.0	35.0	32.2	32.8	33.3	-	-
EGC7160	35.6	35.9	32.3	32.9	33.3	-	-
Pipeline No. EGC8							
EG800	n/a	34.2	31.5	31.6	32.4	-	-
EG810	34.0	34.2	32.0	33.1	33.5	-	-
EG820	34.0	34.2	32.1	33.6	34.1	-	-
EG830	39.0	38.0	32.4	34.6	36.2	-	-
EG840	38.4	38.0	32.4	35.0	36.3	-	-

Table 5-10. Calculated Water Surface Elevations for Existing Pipelines EGC1 - EGC17 (NGVD29), Cont'd…

Node Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline No. EGC9							
EG0900	38.3	N/A	35.4	35.5	36.6	-	-
EG0910	37.4	38.0	35.6	36.0	37.4	-	-
EG0920	37.5	38.0	35.7	36.5	37.8	-	-
EG0925	37.7	38.0	35.8	36.7	37.9	-	-
EG0930	39.6	40.0	36.6	39.2	39.6	-	-
EG0940	37.0	38.2	36.5	36.9	37.4	-	-
EG0950	37.5	38.3	37.5	37.9	38.3	Yes	Yes
Pipeline No. EGC10							
EG1000	N/A	N/A	35.8	35.8	35.8	-	-
EG1010	37.5	40.0	36.9	37.4	37.9	-	-
EG1015	42.9	40.0	37.6	38.0	38.3	-	-
EG1020	42.3	40.0	38.4	38.9	39.2	-	-
EG1025	41.7	42.0	38.6	39.1	39.5	-	-
EG1040	41.0	42.0	39.6	40.0	40.5	-	-
Pipeline No. EGC11							
EG1100	N/A	N/A	36.0	36.0	37.2	-	-
EG1105	40.8	40.0	36.1	36.4	37.6	-	-
EG1110	40.6	40.0	36.6	37.5	38.7	-	-
EG1120	40.2	42.0	36.9	38.4	39.4	-	-
EG1140	40.2	40.5	37.5	39.5	40.1	-	-
EG1145	41.7	42.0	37.5	39.5	40.2	-	-
EG1160	44.1	44.6	37.8	40.6	42.4	-	-
Pipeline No. EGC12							
EG1200	n/a	40.0	36.4	36.4	37.5	-	-
EG1210	41.6	41.6	38.4	39.7	41.2	-	-
EG1220	46.5	47.0	42.5	45.3	46.0	-	-
EG1230	47.0	47.0	45.4	45.8	46.2	-	-
Pipeline No. EGC13							
EG1300	n/a	42.0	38.8	38.8	40.2	-	-
EG1310	40.5	42.0	39.1	39.6	40.9	-	-
EG1320	41.5	43.0	39.8	41.4	42.7	-	-
EG1330	42.8	44.0	40.2	42.6	44.0	-	-
Pipeline No. EGC15							
EG1500	n/a	43.0	38.2	38.2	38.2	-	-
EG1510	41.5	44.0	41.0	41.7	42.2	Yes	-
EG1520	41.9	44.0	41.8	42.2	42.6	Yes	-
EG1530	44.8	46.0	44.5	45.0	45.2	Yes	-
Pipeline No. EGC16							
EG1600	n/a	43.7	37.7	39.3	40.5	-	-
EG1605	44.0	45.3	38.6	40.2	43.5	-	-
EG1610	47.3	47.4	38.9	40.4	44.2	-	-
EG1615	44.3	45.7	38.7	40.4	43.8	-	-
EG1620	45.1	46.3	39.0	40.6	44.7	-	-
EG1625	48.8	48.8	39.3	40.7	44.5	-	-
EG1630	44.2	45.5	39.4	40.7	44.0	-	-
EG1632	45.3	45.7	39.6	40.8	44.0	-	-
EG1635	45.1	45.8	39.7	40.8	44.0	-	-
EG1640	43.5	45.0	39.7	40.8	43.5	-	-
EG1645	44.3	44.7	40.3	41.1	44.1	-	-
EG1650	43.7	44.9	40.0	40.6	41.3	-	-

Table 5-10. Calculated Water Surface Elevations for Existing Pipelines EGC1 - EGC17 (NGVD29), Cont'd…

Node Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline No. EGC17							
EG1700	n/a	45.0	39.6	41.3	41.3	-	-
EG1705	n/a	47.0	40.4	43.0	44.0	-	-
EG1710	n/a	47.5	40.8	43.8	44.9	-	-
EG1715	46.6	47.6	41.3	44.7	45.9	-	-
EG1720	46.8	48.5	41.6	45.3	46.2	-	-
EG1725	46.8	48.5	42.0	45.6	46.5	-	-
EG1730	46.1	48.0	42.1	45.7	46.6	-	-
EG1735	n/a	49.0	42.7	47.5	48.7	-	-
EG1740	46.3	49.0	42.8	46.3	46.9	-	-
EG1760	45.0	45.0	39.7	42.7	43.4	-	-
EG1765	47.0	47.0	39.7	43.8	44.6	-	-
EG1770	46.9	47.5	39.8	44.5	45.2	-	-

Table 5-11. Calculated Peak Flows for Existing Pipelines EGC1 - EGC17

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline No. EGC1						
REG110.1	EG110	EG100	Pipe	31	53	53
REG120.1	EG120	EG110	Pipe	31	53	53
REG130.1	EG130	EG120	Pipe	26	46	44
REG140.1	EG140	EG130	Pipe	4	8	10
REG150.1	EG150	EG140	Pipe	4	7	7
REG160.1	EG160	EG130	Pipe	4	7	8
OLR150	EG150	EG140	Overland	0	0	6
Pipeline No. EGC2						
REG210.1	EG210	EG200	Pipe	19	36	51
REG220.1	EG220	EG210	Pipe	16	30	42
REG230.1	EG230	EG220	Pipe	4	7	11
REG240.1	EG240	EG220	Pipe	1	2	3
Pipeline No. EGC3						
REG310.1	EG310	EG300	Pipe	25	46	49
REG320.1	EG320	EG310	Pipe	13	24	24
REG330.1	EG330	EG320	Pipe	3	6	9
Pipeline No. EGC4						
REG410.1	EG410	EG400	Pipe	71	100	118
REG420.1	EG420	EG410	Pipe	38	65	83
REG430.1	EG430	EG420	Pipe	25	46	62
REG440.1	EG440	EG400	Pipe	38	69	91
REG450.1	EG450	EG440	Pipe	27	49	65
REG460.1	EG460	EG450	Pipe	27	49	65
Pipeline No. EGC5						
REG520.1	EG520	EG500	Pipe	59	97	122
REG530.1	EG530	EG520	Pipe	26	37	37
REG540.1	EG540	EG530	Pipe	25	37	37
REG550.1	EG550	EG540	Pipe	16	27	29
REG560.1	EG560	EG550	Pipe	8	17	19
Pipeline No. EGC6						
EGC6 105C	EG6105	EG6100	Pipe	38	51	55
EGC6 110C	EG6110	EG6105	Pipe	38	50	55
EGC6 115C	EG6115	EG6110	Pipe	28	32	28
EGC6 120C	EG6120	EG6115	Pipe	28	32	28
EGC6 125C	EG6125	EG6120	Pipe	19	22	19
EGC6 130C	EG6130	EG6125	Pipe	19	22	19
Pipeline No. EGC7						
EGC7 110C	EG7110	EG7100	Pipe	58	76	79
R7120	EG7120	EG7110	Pipe	46	54	49
239.1	EG7130	EG7120	Pipe	30	36	33
237.1	EG7140	EG7130	Pipe	23	32	30
235.1	EG7150	EG7140	Pipe	13	19	17
233.1	EG7160	EG7150	Pipe	5	9	14
Pipeline No. EGC8						
REG810	EG810	EG800	Pipe	29	52	45
REG820	EG820	EG810	Pipe	19	33	35
REG830	EG830	EG820	Pipe	14	26	35
REG840	EG840	EG830	Pipe	3	11	8
OLR820	EG820	EG810	Overland	0	0	8

Table 5-11. Calculated Peak Flows for Existing Pipelines EGC1 - EGC17, Cont'd…

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline No. EGC9						
REG0910.1	EG0910	EG0900	Pipe	34	61	77
REG0920.1	EG0920	EG0910	Pipe	28	52	50
REG0925.1	EG0925	EG0920	Pipe	16	30	24
REG0930.1	EG0930	EG0925	Pipe	16	29	26
REG0940.1	EG0940	EG0900	Pipe	25	28	22
REG0950.1	EG0950	EG0940	Pipe	14	15	12
OLR0920	EG0920	EG0910	Overland	0	0	20
OLR0925	EG0925	EG0920	Overland	0	0	20
OLR0930	EG0930	EG0925	Overland	0	0	19
OLR0950	EG0950	EG0940	Overland	0	3	6
Pipeline No. EGC10						
REG1010.1	EG1010	EG1000	Pipe	58	71	81
REG1015.1	EG1015	EG1010	Pipe	42	46	49
REG1020.1	EG1020	EG1015	Pipe	42	46	49
REG1025.1	EG1025	EG1020	Pipe	32	41	42
REG1040.1	EG1040	EG1025	Pipe	22	26	27
Pipeline No. EGC11						
REG1105.1	EG1105	EG1100	Pipe	35	58	58
REG1110.1	EG1110	EG1105	Pipe	34	54	53
REG1120.1	EG1120	EG1110	Pipe	32	50	47
REG1140.1	EG1140	EG1120	Pipe	22	37	34
REG1145.1	EG1145	EG1140	Pipe	6	12	17
REG1160.1	EG1160	EG1145	Pipe	6	12	17
OLR2	EG1140	EG1100	Overland	0	0	3
Pipeline No. EGC12						
REG1210	EG1210	EG1200	Pipe	36	46	49
REG1220	EG1220	EG1210	Pipe	31	35	33
REG1230	EG1230	EG1220	Pipe	21	23	23
Pipeline No. EGC13						
REG1310	EG1310	EG1300	Pipe	33	52	51
REG1320	EG1320	EG1310	Pipe	27	44	43
REG1330	EG1330	EG1320	Pipe	22	37	39
Pipeline No. EGC15						
REG1510	EG1510	EG1500	Pipe	31	35	38
REG1520	EG1520	EG1510	Pipe	22	20	21
REG1530	EG1530	EG1520	Pipe	17	18	18
OLREG1520	EG1520	EG1510	Overland	7	28	41
OLREG1530	EG1530	EG1520	Overland	2	19	37
Pipeline No. EGC16						
REG1605	EG1605	EG1600	Pipe	57	108	170
REG1610	EG1610	EG1605	Pipe	35	70	104
REG1615	EG1615	EG1605	Pipe	7	15	27
REG1620	EG1620	EG1610	Pipe	24	50	59
REG1625	EG1625	EG1620	Pipe	21	47	45
REG1630	EG1630	EG1625	Pipe	15	36	25
REG1632	EG1632	EG1630	Pipe	15	38	26
REG1635	EG1635	EG1632	Pipe	15	39	28
REG1640	EG1640	EG1635	Pipe	9	15	21
REG1645	EG1645	EG1640	Pipe	9	17	27
REG1650	EG1650	EG1640	Pipe	10	14	21

Table 5-11. Calculated Peak Flows for Existing Pipelines EGC1 - EGC17, Cont'd…

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline No. EGC17						
REG1705	EG1705	EG1700	Pipe	69	126	148
REG1710	EG1710	EG1705	Pipe	70	125	139
REG1715	EG1715	EG1710	Pipe	72	128	155
REG1720	EG1720	EG1715	Pipe	65	113	121
REG1725	EG1725	EG1720	Pipe	45	83	88
REG1730	EG1730	EG1725	Pipe	18	37	57
REG1735	EG1735	EG1725	Pipe	23	42	48
REG1740	EG1740	EG1730	Pipe	20	35	34
REG1760	EG1760	EG1700	Pipe	17	53	65
REG1765	EG1765	EG1760	Pipe	18	47	60
REG1770	EG1770	EG1765	Pipe	15	32	41
OLREG1730	EG1730	EG1725	Overland	0	0	5
OLREG1735	EG1735	EG1725	Overland	0	0	9
OLREG1740	EG1740	EG1730	Overland	0	3	19

Table 5-12. Preliminary Improvements for Elk Grove Creek Watershed

Item	Quantity	Unit
Existing Pipeline Upgrades		
36" RCP	1323	LF
42" RCP	1077	LF
Manholes	7	EA
Outfall Structures	2	EA
Detention Basins		
Undefined Buildout Mitigation Basins	62,000	CY

Notes:

1. Facilities in the East Area are not included.
2. Improvements are required to eliminate or reduce existing flooding along the creek. Additional studies are required to define the necessary improvements

- Improvements to Existing Pipeline EGC15 (See Figure 5-14).
- Provide 24 acre-feet of detention storage to mitigate for potential impacts to flows in Elk Grove Creek due to future development.

Improvements are also necessary to eliminate the existing flood risk predicted along the creek. Additional studies are required to define the specific improvements for creek flooding.

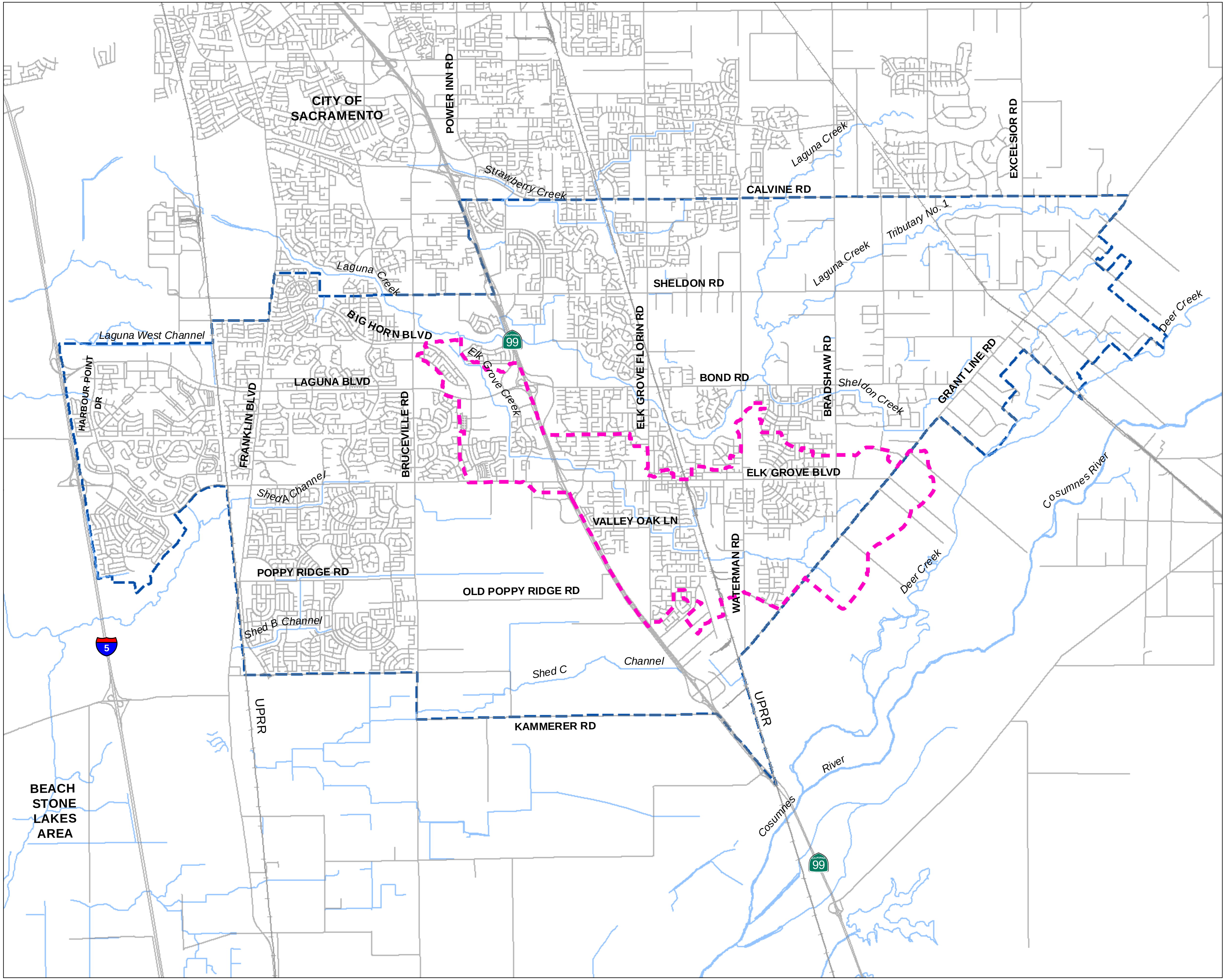

FIGURE 5-1

City of Elk Grove
Storm Drainage Master Plan Study
Volume II

ELK GROVE CREEK LOCATION MAP

NOTES:

LEGEND:

- City Limit
- Elk Grove Creek Watershed

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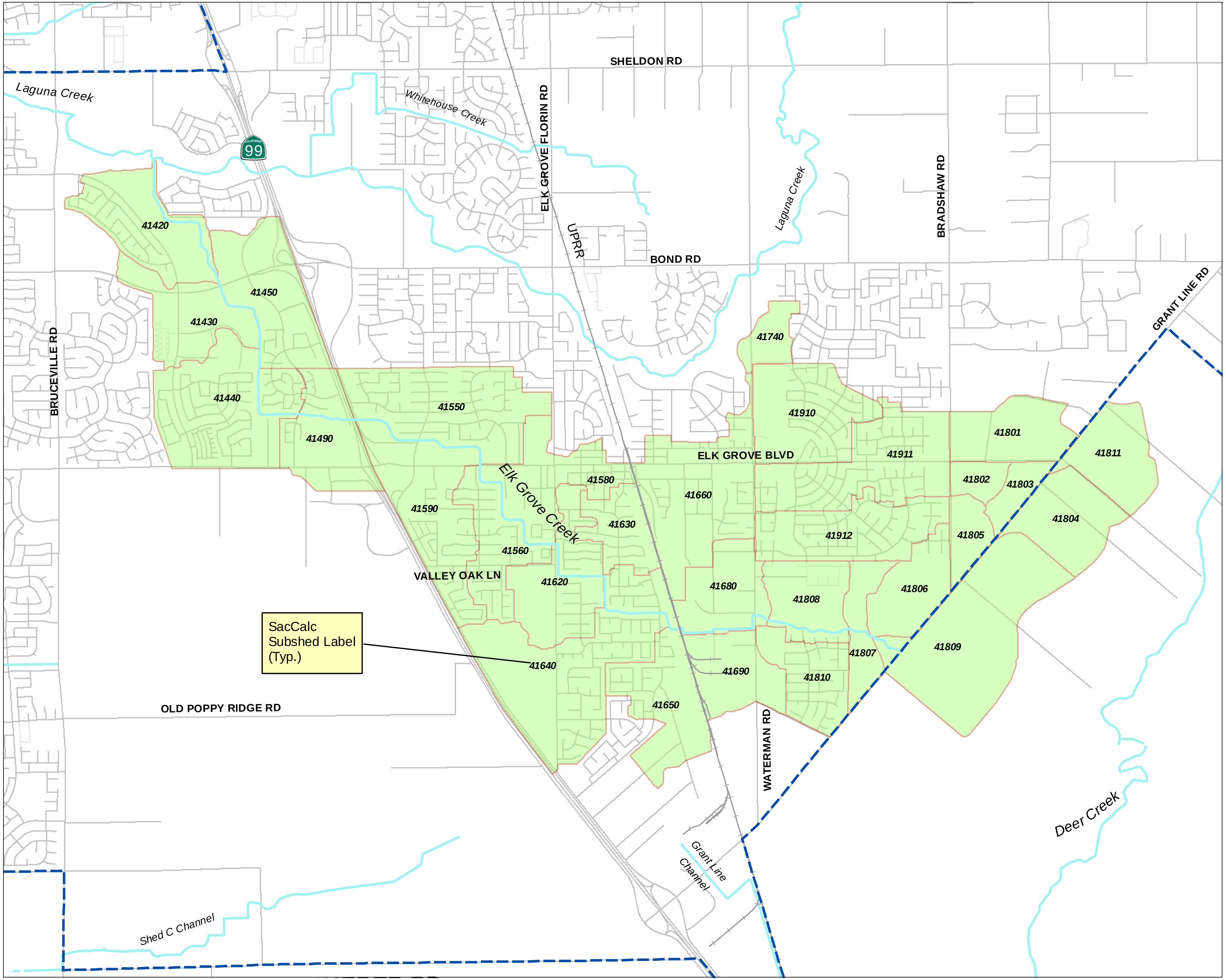

FIGURE 5-2

City of Elk Grove
Storm Drainage Master Plan Study
Volume II

ELK GROVE CREEK SUBSHEDS

NOTES:

LEGEND:

- ⬚ City Limit
- 🟩 Elk Grove Creek Subsheds

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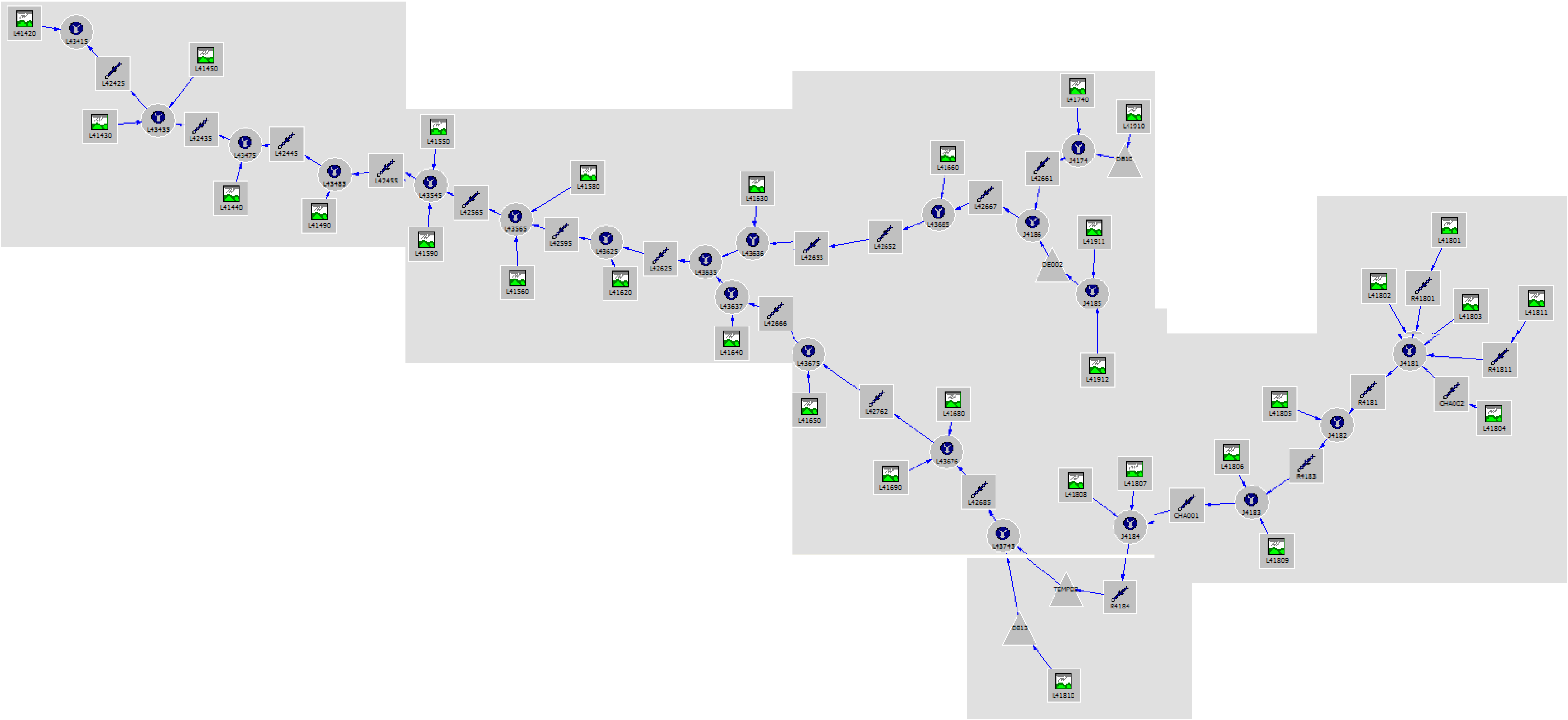
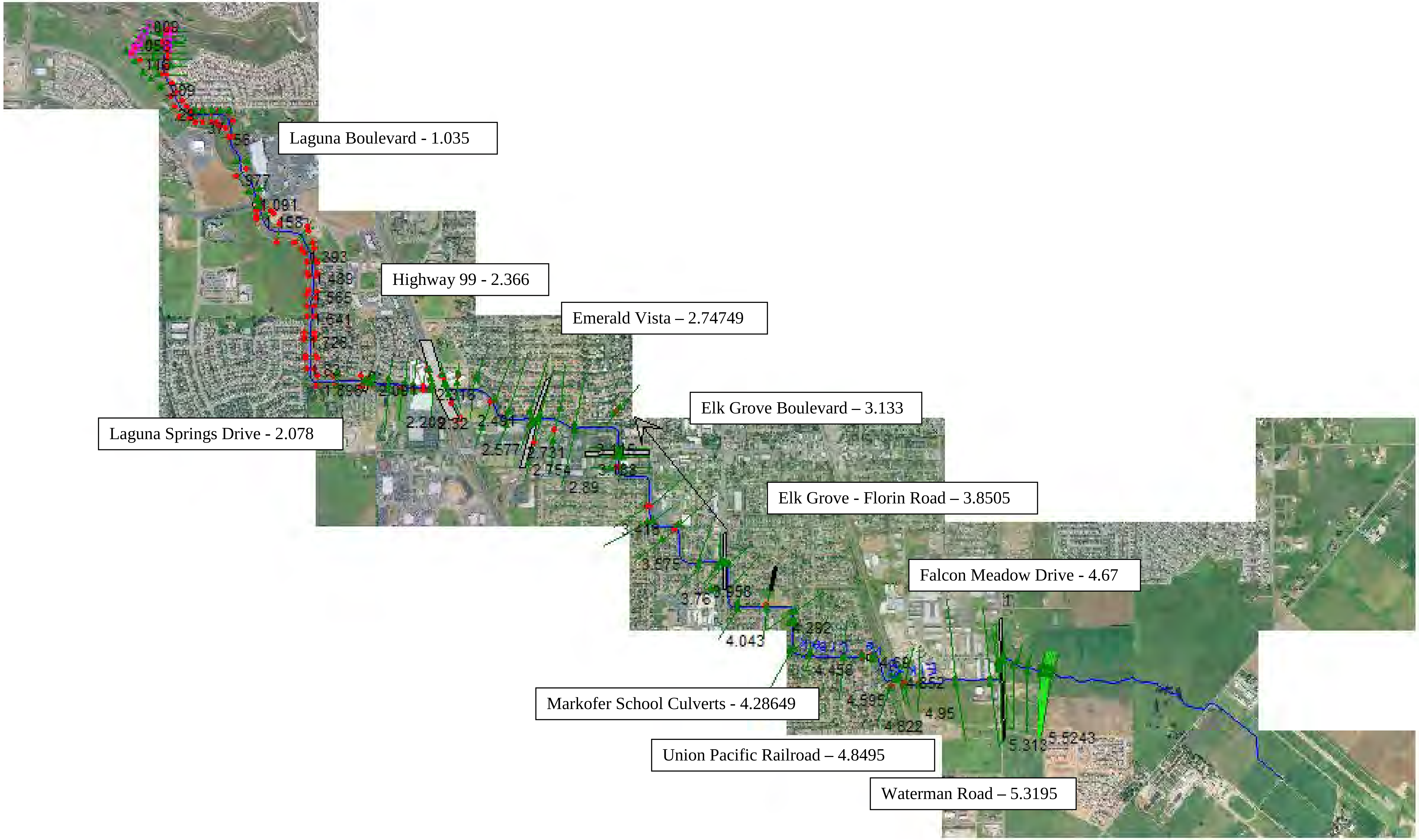
Figure 5-3. Elk Grove Creek SacCalc Schematic




Laguna Boulevard - 1.035

Highway 99 - 2.366

Emerald Vista – 2.74749

Elk Grove Boulevard – 3.133

Laguna Springs Drive - 2.078

Elk Grove - Florin Road – 3.8505

Falcon Meadow Drive - 4.67

Markofer School Culverts - 4.28649

Union Pacific Railroad – 4.8495

Waterman Road – 5.3195

Figure 5-4. Elk Grove Creek HEC-RAS Model Layout

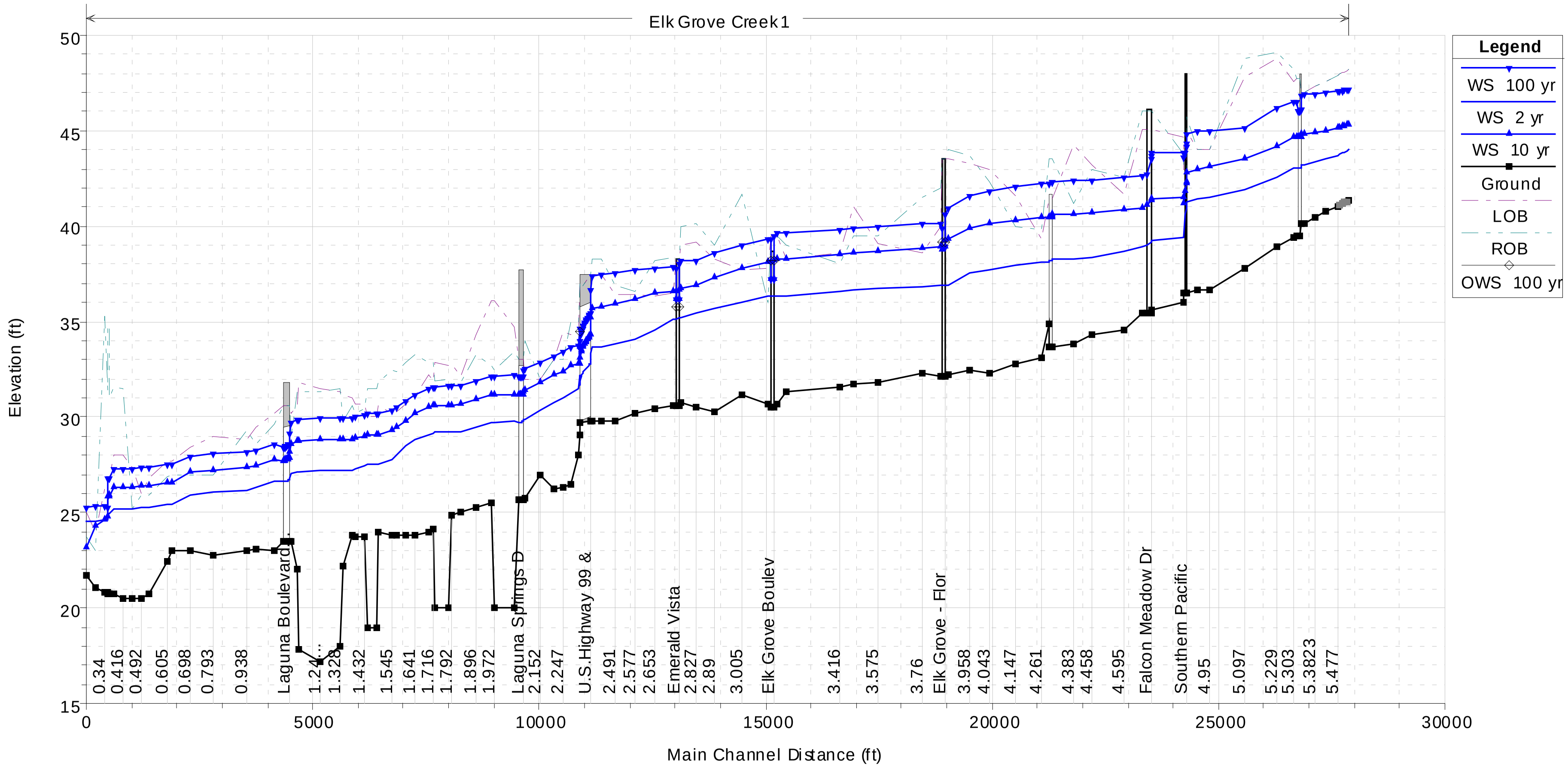


Figure 5-5a. Elk Grove Creek Existing Conditions Water Surface Profiles

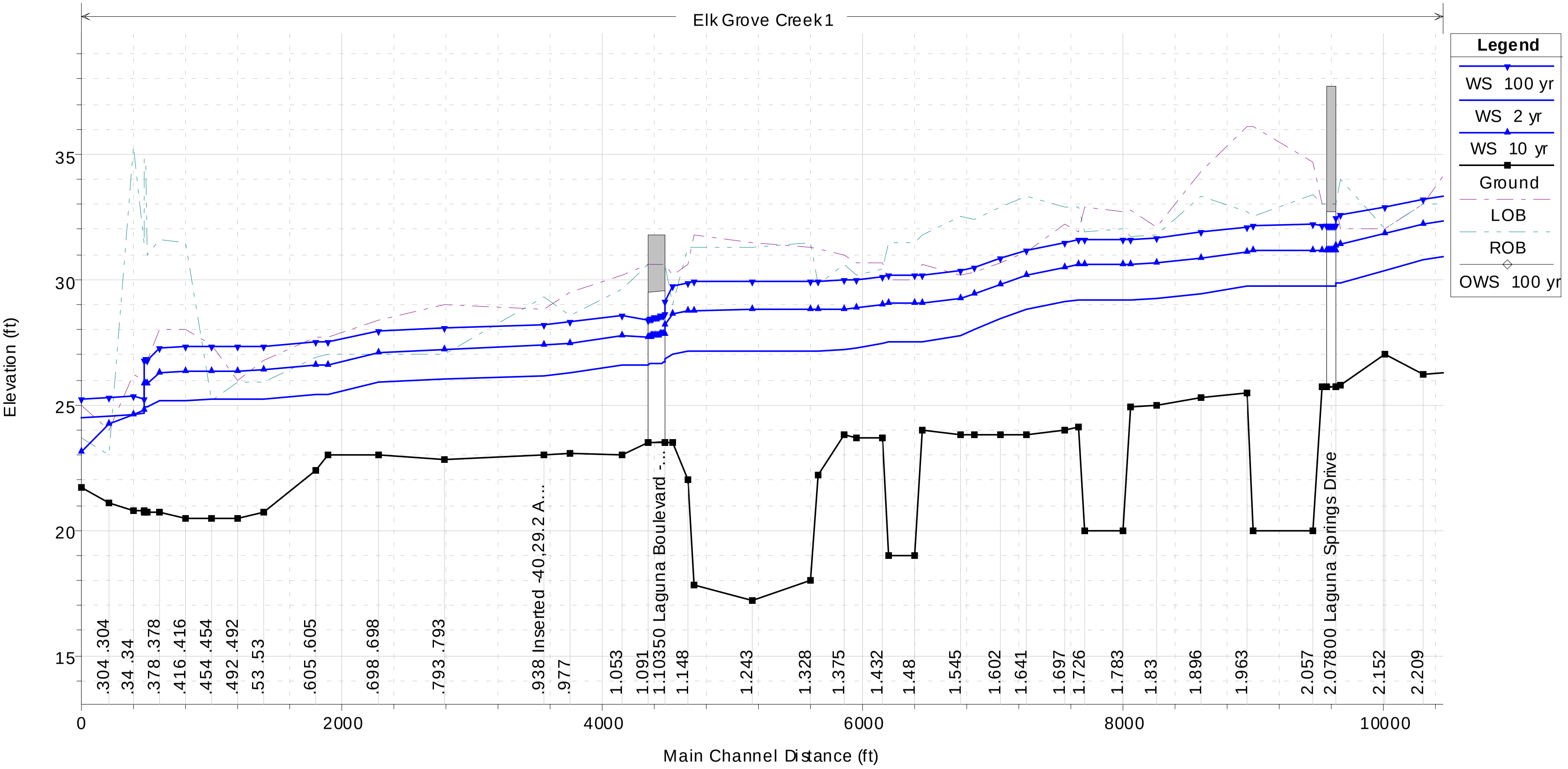


Figure 5-5b. Elk Grove Creek Existing Conditions Sta 0.304 to 2.209

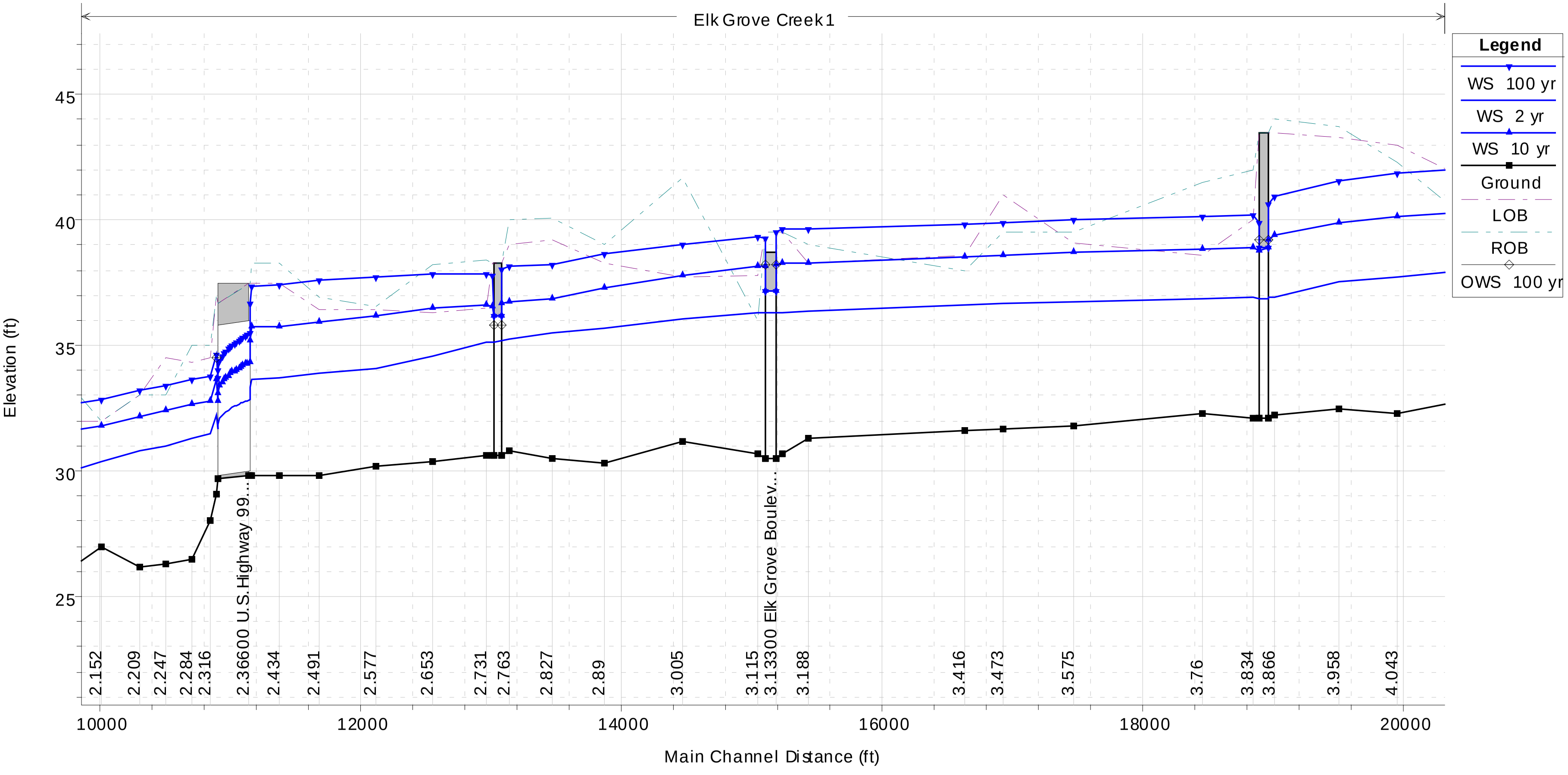


Figure 5-5c. Elk Grove Creek Existing Conditions Sta 2.152 to 4.403

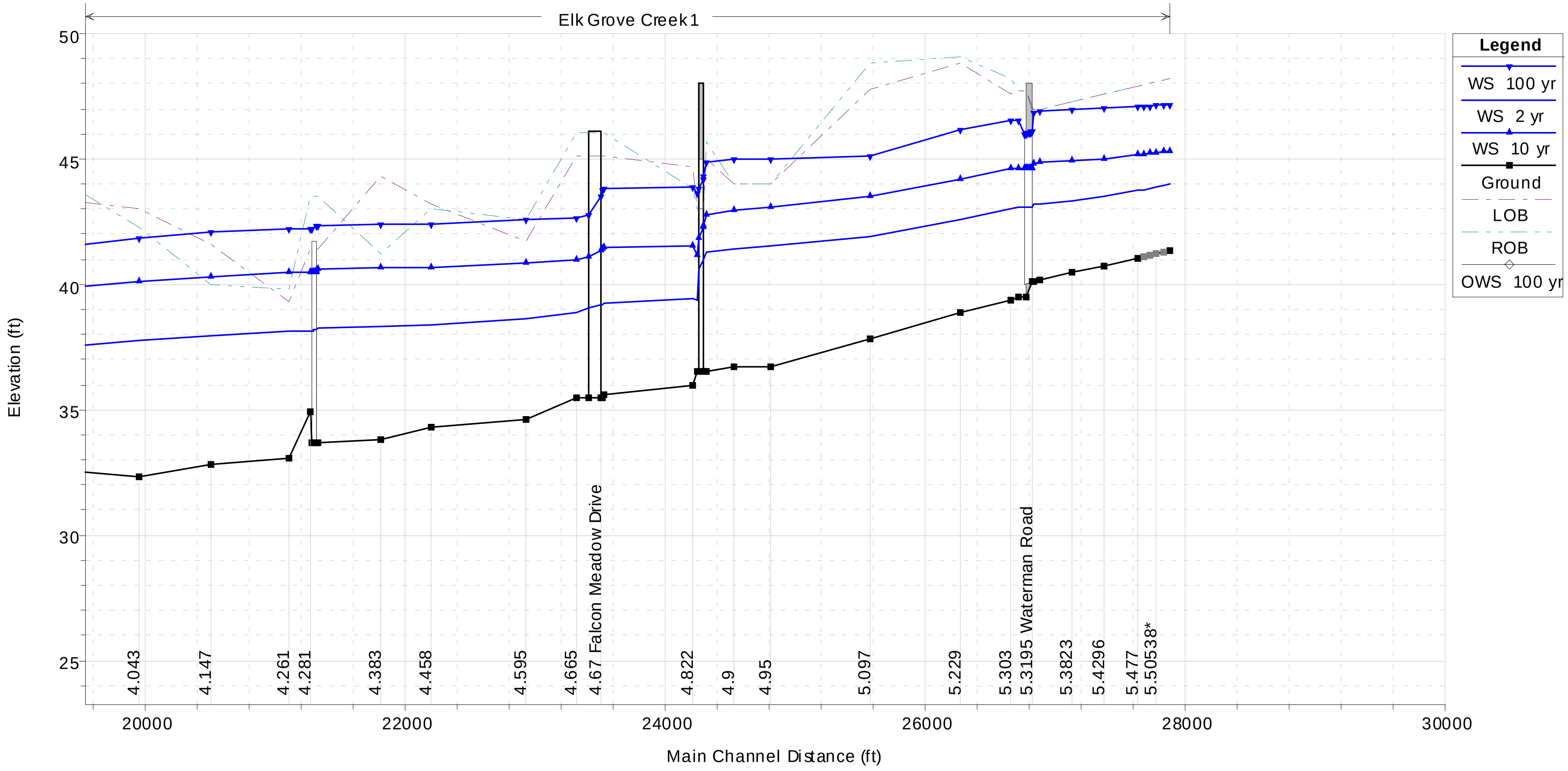


Figure 5-5d.Elk Grove Creek Existing Conditions Sta 4.043 to 5.50

FIGURE 5-6a

City of Elk Grove
Storm Drainage Master Plan
Volume II

Elk Grove Creek
Approximate 100-Year Floodplain

N

0 400 800
Feet

NOTES:

LEGEND:

- — HEC-RAS Cross Section
- — Elevation Contour (NGVD 29)
- ☐ 100-Year Floodplain (HEC-RAS)
- ▨ 100-Year Floodplain (2D Model)
- — 100-Year Floodplain (FEMA)

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Note: FEMA floodplain maps do not show a floodplain in this area. The FEMA maps indicate that the 100-year floodplain is contained in the channel.

FIGURE 5-6b

City of Elk Grove
Storm Drainage Master Plan
Volume II

Elk Grove Creek
Approximate 100-Year Floodplain

N

0 400 800
Feet

NOTES:

LEGEND:

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
☐	100-Year Floodplain (HEC-RAS)
▨	100-Year Floodplain (2D Model)
—	100-Year Floodplain (FEMA)

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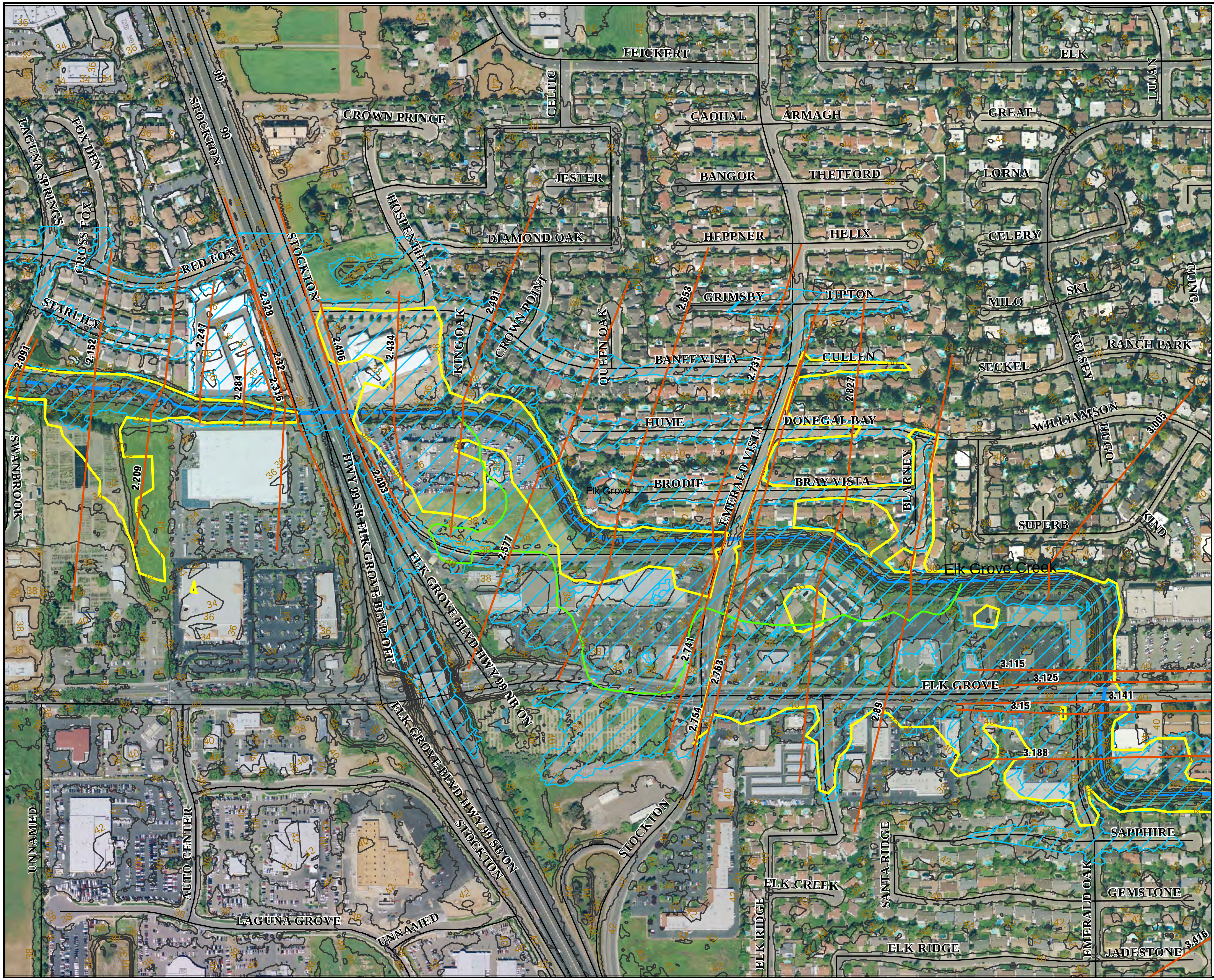

FIGURE 5-6c

City of Elk Grove
Storm Drainage Master Plan
Volume II

Elk Grove Creek
Approximate 100-Year Floodplain

NOTES:

LEGEND:

— HEC-RAS Cross Section

— Elevation Contour (NGVD 29)

▨ 100-Year Floodplain (2D Model)

☐ 100-Year Floodplain (HEC-RAS)

— 100-Year Floodplain (FEMA)

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FIGURE 5-6d

City of Elk Grove
Storm Drainage Master Plan
Volume II

Elk Grove Creek
Approximate 100-Year Floodplain

N

0 400 800
Feet

NOTES:

LEGEND:

- HEC-RAS Cross Section
- Elevation Contour (NGVD 29)
- ▨ 100-Year Floodplain (City)
- ☐ 100-Year Floodplain (HEC-RAS)
- 100-Year Floodplain (FEMA)

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FIGURE 5-6e

City of Elk Grove
Storm Drainage Master Plan
Volume II

Elk Grove Creek
Approximate 100-Year Floodplain

N

0 400 800
Feet

NOTES:

LEGEND:

- HEC-RAS Cross Section
- Elevation Contour (NGVD 29)
- ☐ 100-Year Floodplain (HEC-RAS)
- ▨ 100-Year Floodplain (2D Model)
- 100-Year Floodplain (FEMA)

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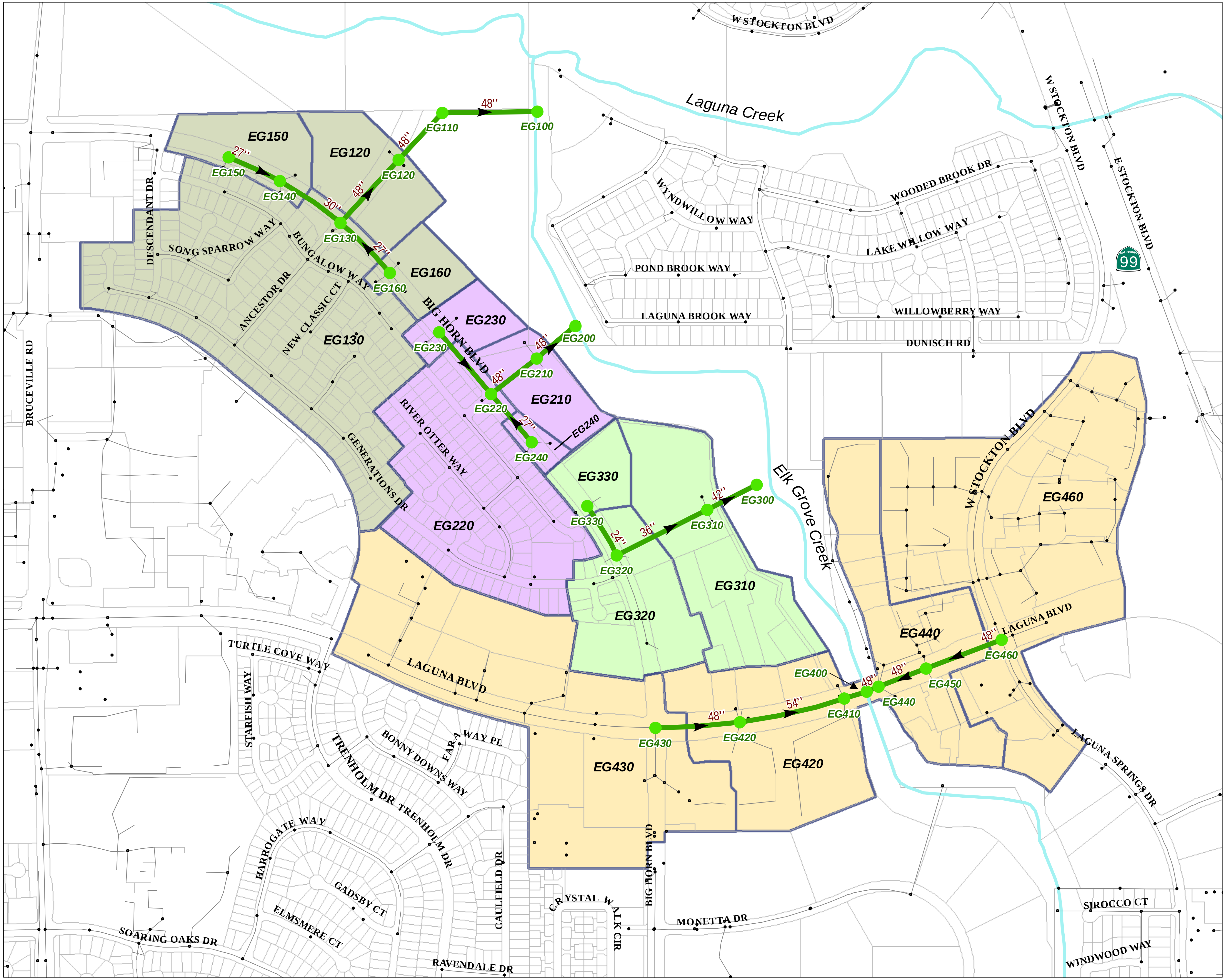

FIGURE 5-7

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK
EXISTING PIPELINES EGC1, EGC2, EGC3, EGC4
SUBSHEDS & MODELED FACILITIES

N

0 500 1,000
Feet

LEGEND:

EG410 ● Modeled Pipeline and Node	
▨	Elk Grove Creek Pipeline EGC1 Subshed
🟪	Elk Grove Creek Pipeline EGC2 Subshed
🟩	Elk Grove Creek Pipeline EGC3 Subshed
🟨	Elk Grove Creek Pipeline EGC4 Subshed

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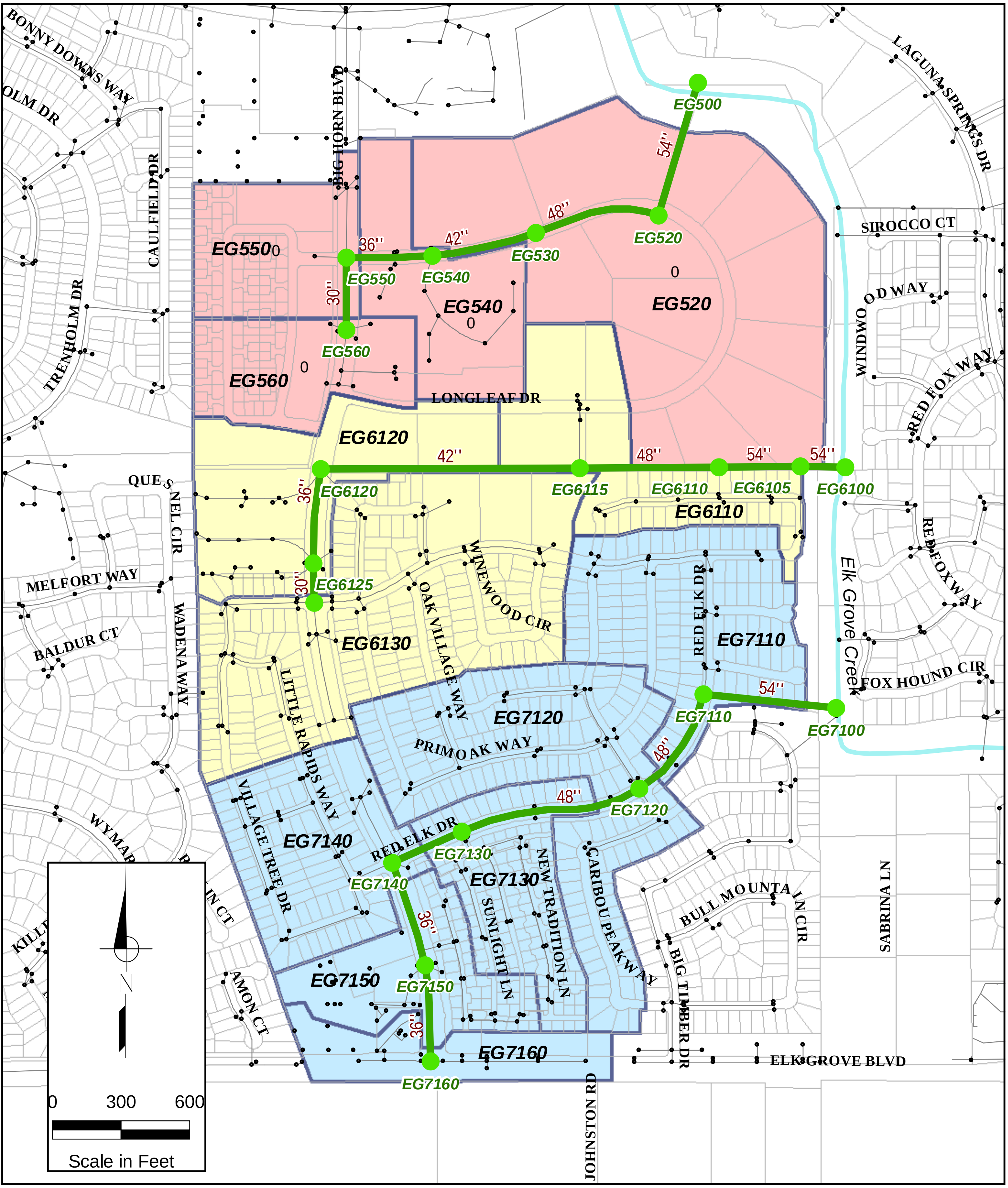

LEGEND:

EG6110

Modeled Pipeline and Node

🟥 Elk Grove Creek Pipeline EGC5 Subshed

🟨 Elk Grove Creek Pipeline EGC6 Subshed

🟦 Elk Grove Creek Pipeline EGC7 Subshed

FIGURE 5-8

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK
EXISTING PIPELINES EGC5, EGC6, EGC7
SUBSHEDS & MODELED FACILITIES

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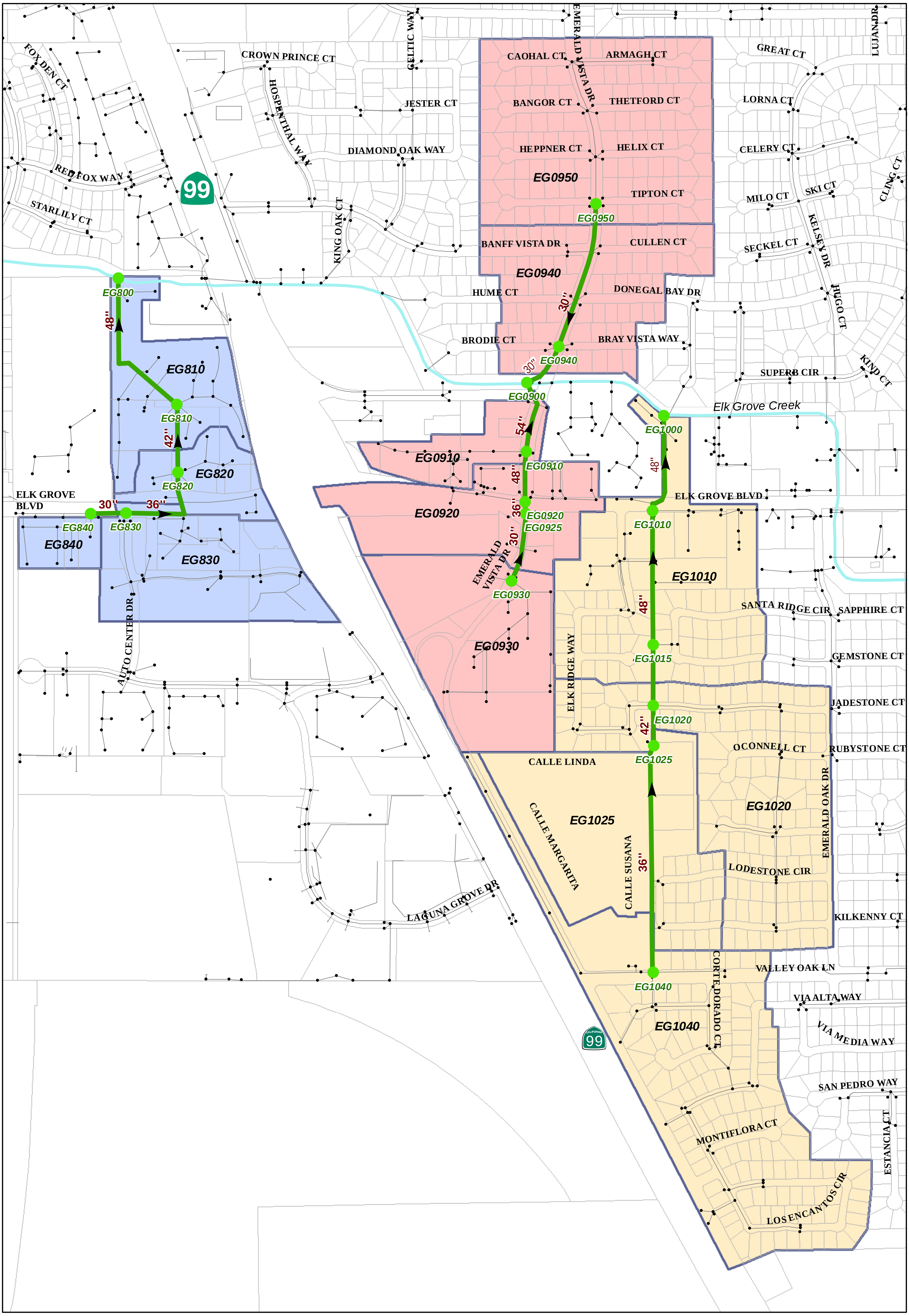

LEGEND:

EG1020 Modeled Pipeline and Node

🟦 Elk Grove Creek Pipeline EGC8 Subshed

🟥 Elk Grove Creek Pipeline EGC9 Subshed

🟨 Elk Grove Creek Pipeline EGC10 Subshed

N

0 500 1,000

Feet

FIGURE 5-9

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK EXISTING PIPELINES
EGC8 - EGC10 SUBSHEDS & MODELED FACILITIES

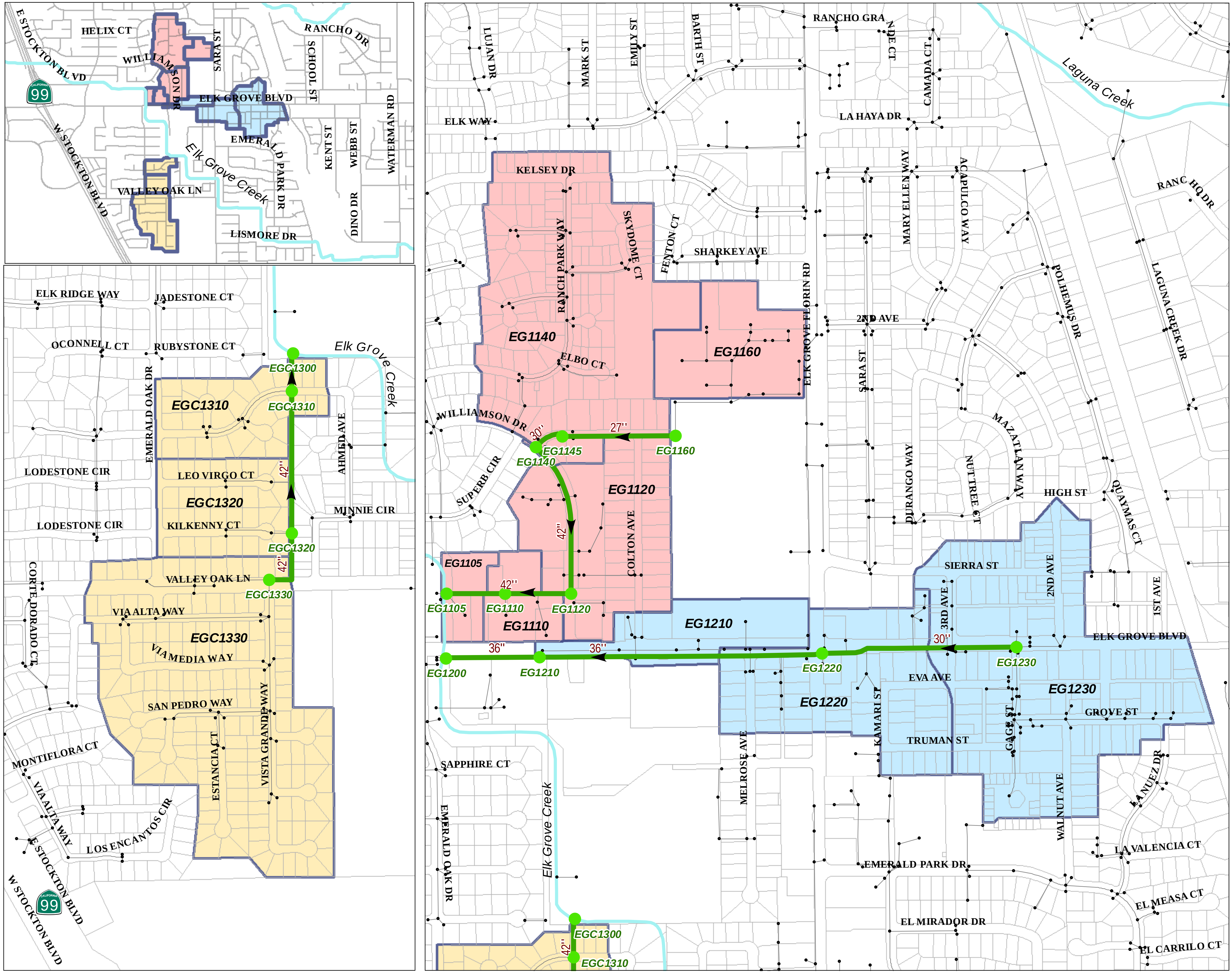

FIGURE 5-10

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK
EXISTING PIPELINES EGC11, EGC12, EGC13
SUBSHEDS & MODELED FACILITIES

N

0 500 1,000
Feet

NOTES:

LEGEND:

EG1230 ●━● Modeled Pipeline
🟥 Elk Grove Creek Pipeline EGC11 Subshed
🟦 Elk Grove Creek Pipeline EGC12 Subshed
🟨 Elk Grove Creek Pipeline EGC13 Subshed

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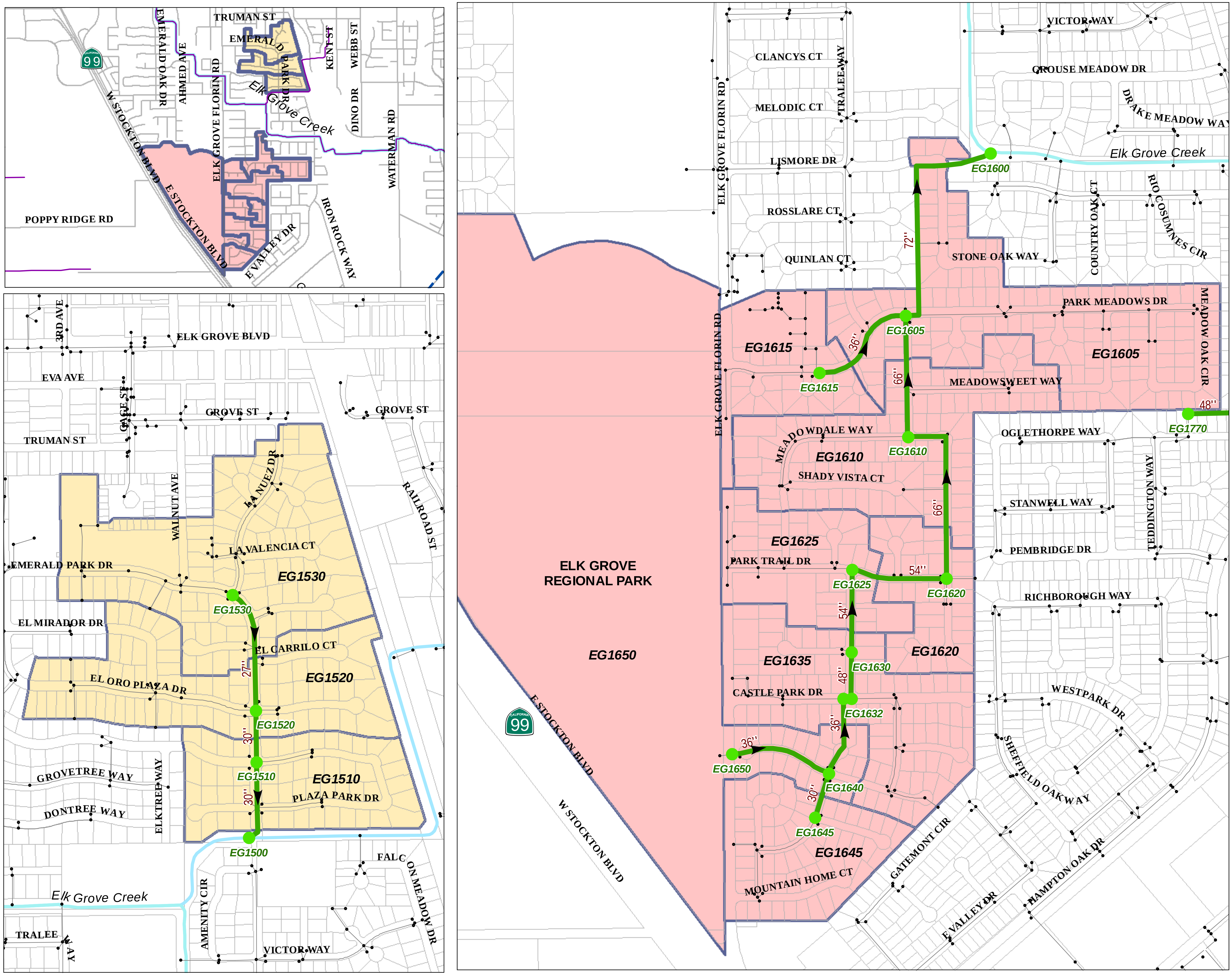

FIGURE 5-11

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK
EXISTING PIPELINES EGC15 AND EGC 16
SUBSHEDS & MODELED FACILITIES

N

0 500 1,000
Feet

NOTES:

LEGEND:

EG1610	Modeled Pipeline and Node
🟨	Elk Grove Creek Pipeline EGC15 Subshed
🟥	Elk Grove Creek Pipeline EGC16 Subshed

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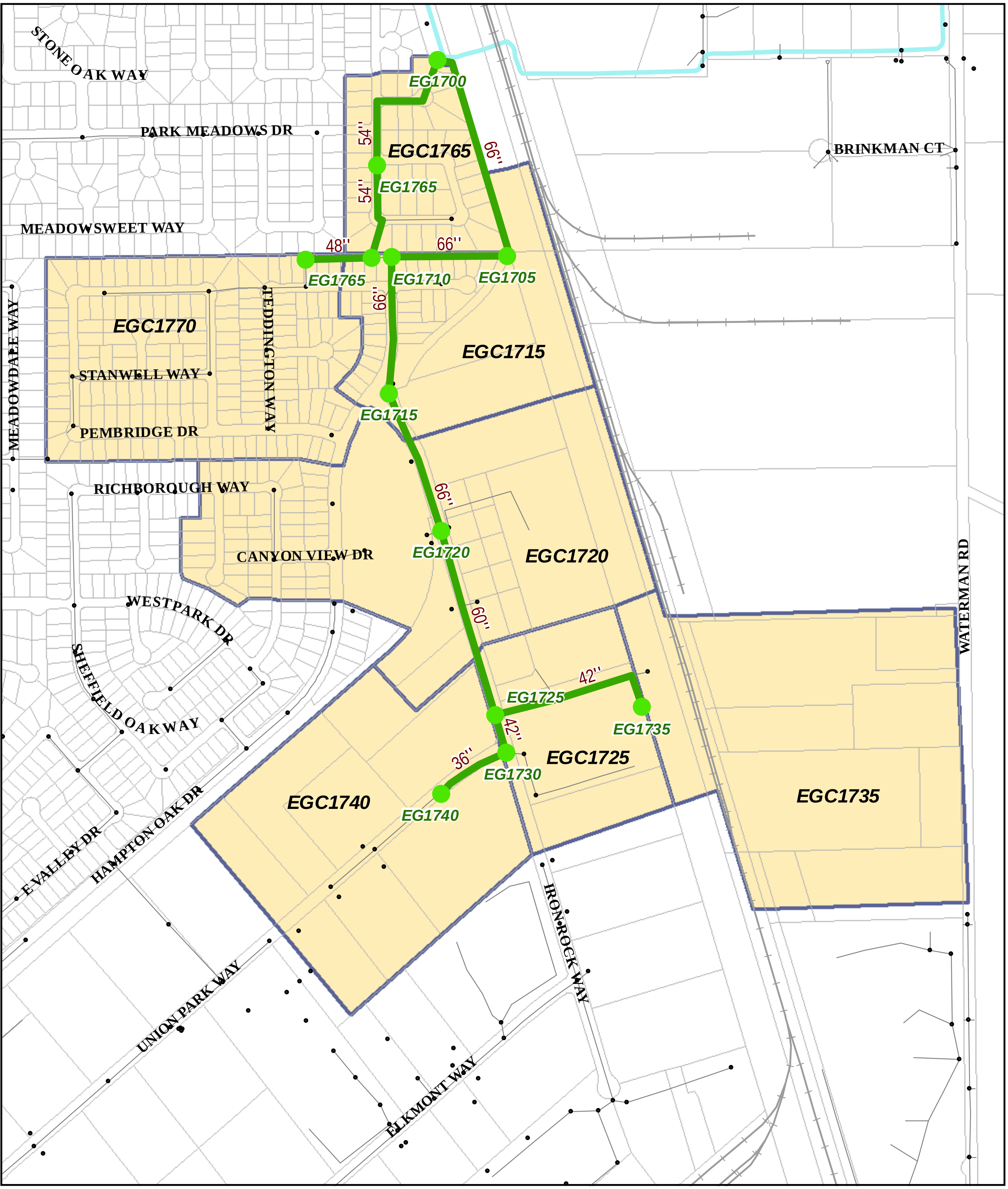

LEGEND:

EG1725

Modeled Pipeline and Node

▨ Elk Grove Creek Pipeline EGC17 Subshed

N

0 300 600

Scale in Feet

FIGURE 5-12

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK
EXISTING PIPELINE EGC17
SUBSHEDS & MODELED FACILITIES

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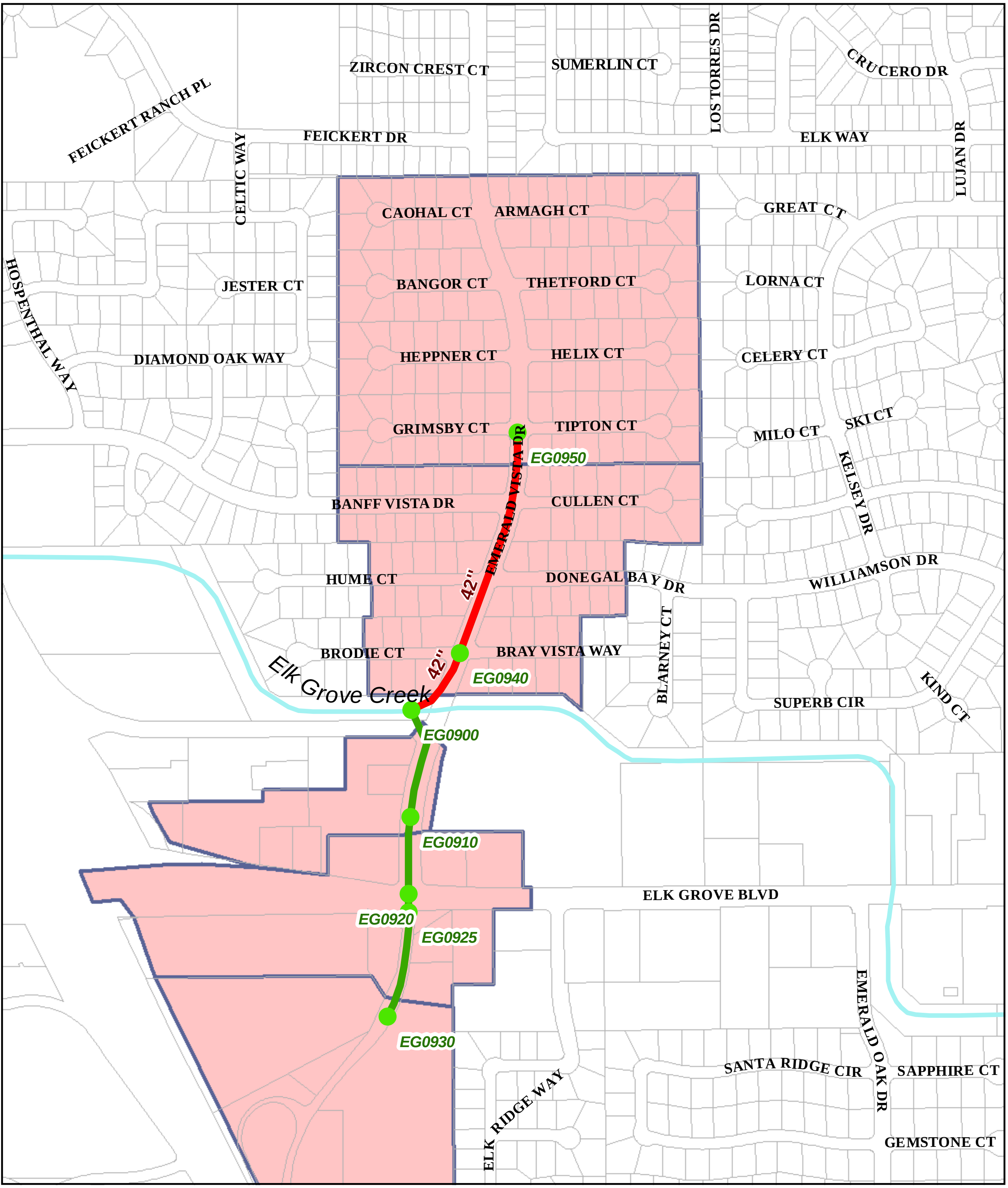

LEGEND:

EG0920 ● Modeled Pipeline and Node	
🟥 Upsized Pipeline	
▨ Elk Grove Creek Pipeline EGC9 Subshed	

N

0 250 500

Scale in Feet

FIGURE 5-13

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK EXISTING
PIPELINE EGC9 IMPROVEMENTS

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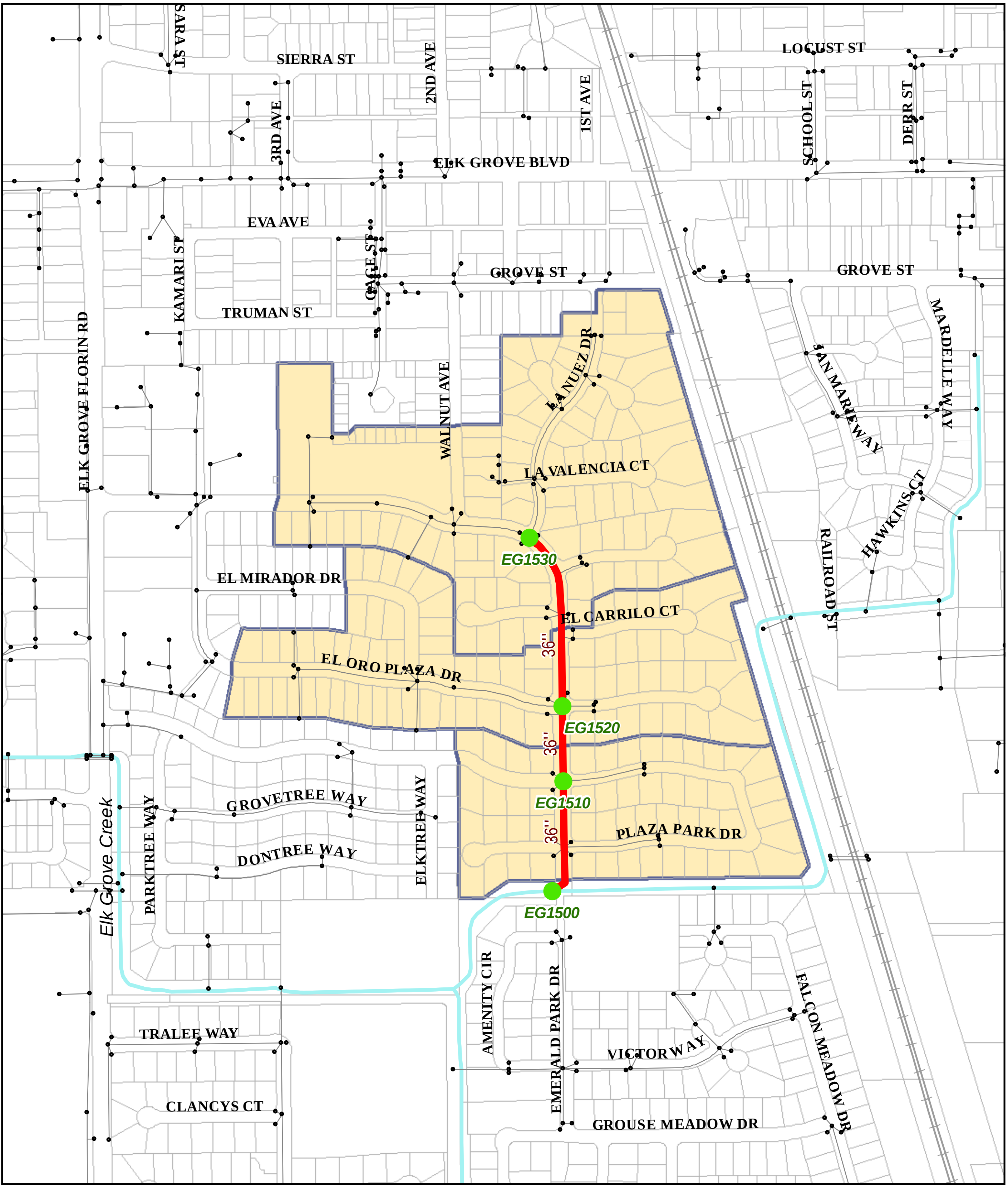

LEGEND:

EG1610

🟩 Modeled Pipeline and Node

🟥 Upsized Pipeline

▨ Elk Grove Creek Pipeline EGC15 Subshed

N

0 250 500

Scale in Feet

FIGURE 5-14

City of Elk Grove
Storm Drainage Master Plan
Volume II

ELK GROVE CREEK
EXISTING PIPELINE EGC15
IMPROVEMENTS

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ATTACHMENT 5A

Elk Grove Creek Detention Analysis

12/13/2010

Attachment 5A

Mark
Fr yours To keep.
Clarence K.

DAVID FORD
CONSULTING
ENGINEERS

David Ford Consulting Engineers, Inc.

2015 J Street, Suite 200
Sacramento, CA 95811

Ph. 916.447.8779
Fx. 916.447.8780

JOHANNES J. DeVRIES
RCE 014195
exp 3/09

MEMORANDUM

To: Clarence Korhonen, PE
City of Elk Grove

From: Joe DeVries, PhD, PE; Brian Brown, PE J DeVries

Date: September 15, 2008

Subject: Analysis of north and south branch Elk Grove Creek detention basins

Background

Detention basins have been constructed on the north and south branches of Elk Grove Creek. The 3 detention basins located on the north branch are referred to as detention basins (DB) 10, 11, and 12. The detention basin on the south branch is referred to as DB 13 (also known as the Hudson basin). Attachment 1 shows the locations of DBs 10, 11, 12, and 13. The detention basins were constructed to reduce downstream flood flows on Elk Grove Creek. However, the downstream portion of Elk Grove Creek, specifically downstream of Elk Grove-Florin Road, has caused flooding of nearby homes.

Other recent studies assessed the effect increased channel roughness has on water surface elevations in Elk Grove Creek downstream of Elk Grove-Florin Road. The increased channel roughness is due to planted vegetation in support of environmental restoration efforts; however it is thought that this may increase flood potential on Elk Grove Creek. This study does not address this issue. Rather, this study addresses performance of the basins.

Study goal

The goal of this study is to answer the question: Can modifications to the existing DB 10, 11, 12, and 13 reduce flows in Elk Grove Creek for frequent events, yet not result in adverse impacts for the infrequent events?

For purposes of this analysis, the frequent events are those with frequencies up to the p=0.10 (10-year) event. The infrequent events are those which exceed the p=0.10 (10-year) flows.

What we did in this study

1. We reviewed reports provided by the City on the physical properties of DB 10, 11, 12, and 13. We also reviewed the representation of these basins in the existing Elk Grove Creek Drainage Master Plan (DMP) computer models. The DMP was designed using computer programs SacCalc and

HEC-RAS. The models from the DMP were used as starting point models for this analysis.

2. We assessed the range of potential downstream flow reduction from DB 10, 11, 12, and 13 for 3 design events: p=0.50 (2-year), p=0.10 (10-year), and p=0.01 (100-year).
3. We reviewed the City's drainage facility maps and improvement plans to obtain information on existing underground piping systems, area contributing runoff to the basins, street layouts, locations of open channels, and connection points to Elk Grove Creek.
4. We investigated field conditions at each DB.
5. We established the baseline modeling condition using models from the DMP and our findings from the field investigation and supplied reports.
6. We formulated alternatives that might reduce downstream stages.
7. We compared the hydrologic and hydraulic effects of each alternative to the baseline condition and their differences are reported herein. We compared the peak flow and stage in Elk Grove Creek at the Elk Grove-Florin Road crossing as well as the peak stage or volume in each detention basin.

A detailed description of these steps is described below.

Range of potential downstream flow reduction

We assessed the range of potential downstream flow reduction from modifications to DB 10, 11, 12, and 13. For 3 design events, p=0.50 (2-year), p=0.10 (10-year), and p=0.01 (100-year), we used the models originally developed for the DMP and assessed the maximum potential downstream flow reduction from the detention basins. For this, we removed all runoff contribution from the area upstream of each of the detention basins. This reflects the maximum potential downstream flow reduction at the index point (the location where the creek crosses Elk Grove-Florin Road).

Baseline modeling condition and potential maximum flow reduction

We established the baseline modeling condition using models from the DMP and our findings from the field investigation and supplied reports. Because this analysis was more focused on specific watershed features and area than the DMP analysis, model changes were required for this alternative analysis. These model changes included updating the storage-discharge relationship for some of the basins to reflect the current basin configuration. This was based on our review of reports and drawings provided by the City and a field visit to the basins on July 29, 2008.

Modeling of the catchment areas contributing flow to the index point on Elk Grove Creek at Elk Grove-Florin Road was done using SacCalc for the condition with DBs 10-13 contributing flow with their updated representation in the model and with the area contributing flow to basins 10-13 removed from the model. A significant potential reduction in peak discharge can be obtained if the basins are modified to increase basin storage at the lower basin elevations, as shown in Table 1.

Table 1. Peak discharges for baseline modeling condition for 3 design events

Condition (1)	**2-yr Q_{peak} cfs** (2)	**10-yr Q_{peak} cfs** (3)	**100-yr Q_{peak} cfs** (4)
With DBs 10-13 (baseline, i.e., with current configuration)	370	699	1111
Drainage area contributing to DBs 10-13 removed	265	524	883
Potential maximum flow reduction	105	175	228

Row 1 in Table 1 shows the baseline flows for the specified events at the point in the SacCalc model where the flow downstream of the basins is modeled in the HEC-RAS model as tributary inflow. This location is where the northern tributary joins the main branch of Elk Grove Creek, just downstream from Emerald Park Drive on the northern tributary (River Station 3.844). The second row shows the flows with the area contributing to DBs 10-13 removed (approximately 635 acres). The difference is the potential maximum flow reduction. Table 2 shows the reduction in water surface elevation associated with removing the drainage area upstream of DBs 10-13 for the 3 design events.

Table 2. Maximum water surface elevation (WSEL) for potential benefit estimate at River Station 3.844, in feet

Condition (1)	**WSEL p=0.50** (2)	**WSEL p=0.10** (3)	**WSEL p=0.01** (4)
With DBs 10-13 defined	37.24	38.87	39.86
Drainage area contributing to DBs 10-13 removed	36.53	38.28	39.45
Potential maximum stage reduction	0.71	0.59	0.41

Modeling considerations

When we evaluated the outlet hydraulics for the basins we found that all DBs are under outlet control. For DB 10 and 11 the outlets are relatively large, long pipes connected to larger pipes. These are connected further downstream to an open channel that carries the flow to Elk Grove Creek. The slopes of these pipes and channels are quite flat (averaging approximately 0.0011 ft/ft). For flows that do not fill the pipes, the flow through the system is subcritical open-channel flow and the control for the profiles is downstream control.

The discharge capacity of DB 10 and 11 outlet pipes is limited by friction loss once they flow full. The outlet pipe for DB 10 is a 60 in. diameter pipe, which begins to flow full at 75 cfs. The pipe is 920 ft long and is connected downstream to an 84 in. diameter pipe west of Waterman Road. The pipe from DB 11 also connects to the 84 in. diameter pipe. The outlet from DB 11 is a 66 in. diameter pipe, and it is 700 ft long. It begins to flow full at a discharge of 96 cfs.

The just full flow capacity of the 84 in. pipe is slightly greater than the combined capacities of the 60 in. and 66 in. pipes. Our hydraulic analysis showed that 60 in. and 66 in. pipes will experience full pipe flow before the 84 in. pipe begins flowing full. At high flows from DBs 10 and 11, the controlling factor governing outflow from the basin is friction loss in these outlet pipes (that is, outlet control).

DB 12 is connected to DB 11 by a 66-in. diameter pipe. Flow from DB 12 into DB 11 is directly governed by storage in DB 11. We represent DB 11 and 12 in SacCalc as a single storage element.

DB 13 has a 3 ft diameter outlet pipe and connects directly to Elk Grove Creek some distance upstream from the connection point of the other basins. The discharge from DB 13 is also under outlet control.

We used HEC-RAS to develop discharge rating curves, which show the relationship between discharge and water surface elevation for each detention basin. A separate HEC-RAS model was developed for each detention basin. Each of these models represented the conveyance features from the detention basin to its confluence with Elk Grove Creek. The closed conduit portions of the system were modeled using HEC-RAS culvert routines with appropriate loss coefficients at pipe junctions and the open channels.

The upper limit for the curves was set as the discharge when the water surface elevation was at the rim of the basin. If this elevation is exceeded, water will spill out of the basins and flow toward Waterman Road. Overland flow will reach Elk Grove Creek much later than the peak flow in the creek and should not influence downstream peak flood elevations. The discharge rating curves were then converted to storage-discharge relationships for use in the SacCalc model.

Basin modifications

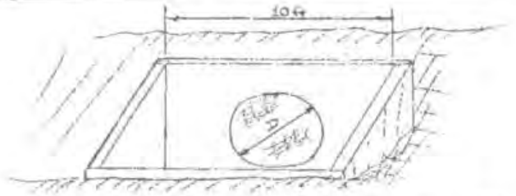
We identified and evaluated 3 alternative basin modifications for DB 10, 11 plus 12, and 13. These alternatives are conceptual modifications at a reconnaissance level, not detailed plans and specifications. Each alternative consists of modifications to all 4 of the detention basins. We did not consider changes to channel roughness, as our preliminary calculations showed this to have a minor effect on peak discharges. When the roughness parameters for the routing reaches in the SacCalc model downstream from the detention basins were increased to 0.050 from 0.015, the peak discharges were decreased by about 6%. Because some of these routing reaches are closed conduits, it is not feasible to increase their roughness and decrease their capacity.

The criteria we used to establish appropriate basin modification alternatives were: 1) the p=0.01 (100-yr) water surface in the basin should be at least 1.0 ft below the basin rim elevation, and 2) the basin must drain within 48 hours of the peak storage.

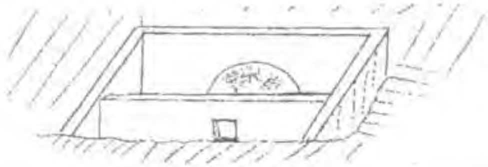
Our initial modification was the use of a weir with the existing 12 in. diameter low-flow outlet pipe at each basin outlet. This did not meet the criterion of draining the pond in 48 hours and this alternative was not pursued further.

The existing outlet for each of the basins has a 10 ft wide inlet section with sloping side walls, as shown in Figure 2(a). The inlet pipes are set in a vertical headwall in this section.

Alternatives that met the criteria involved putting a weir section at the basin outlet with a crest elevation above the outlet to produce increased storage for the more frequent flood events. The weir section included a 1.5 ft square opening with its elevation at the bottom of the basin, allowing water to pond below the crest of the weir. For Alternative 1, the weir crest elevation in each basin was set at 43 ft and for Alternative 2, the weir crest elevation in each basin was set at 44 ft. (The datum is the same as used in other Elk Grove models.) Figure 2(b) shows a sketch of the weir and low level outlet. Since inlet and basin bottom elevations and outlet width for all of the basins are nearly the same, we used the same weir elevations for each of the basins.



(a) This sketch shows the existing outlet configuration. (Note: the trashrack is not shown.)



(b) This sketch shows the weir with a low level outlet.

Figure 2. Sketches of existing and alternative outlet configurations

Once the modifications for the detention basins were formulated, we configured the SacCalc and HEC-RAS models to reflect each alternative. For each, we evaluated the 3 design events (p=0.50, p=0.10, and p=0.01 events).

Rating curves for the individual detention basins are shown in Attachment 2.

Comparison of alternatives

We compared the effect of the 2 alternatives to the baseline condition, and the differences in outlet peak discharge and maximum detention pond storage. These results are reported in Table 3. We compared the peak flow and stage in Elk Grove Creek at the Elk Grove-Florin Road crossing as well as the peak stage or volume in each detention basin (Table 3 and Table 4). Table 5 and Table 6 show the reduction in peak water surface elevation for the two alternatives.

Table 3. Alternative 1: Weir at elevation 43 ft, comparison of peak discharges to baseline modeling condition for 3 design events

Condition (1)	**2-yr Q_{peak} cfs (2)**	**10-yr Q_{peak} cfs (3)**	**100-yr Q_{peak} cfs (4)**
Baseline for DBs 10-13	370	700	1110
Alternative 1: Weir at elevation 43 ft	370	690	1110
Computed maximum flow reduction	0	10	0

Table 4. Alternative 1: Weir at elevation 43 ft, comparison of peak water surface elevations (WSEL) to baseline modeling condition for 3 design events

Condition (1)	**2-yr WSEL (2)**	**10-yr WSEL (3)**	**100-yr WSEL (4)**
Baseline for DBs 10-13	37.2	38.9	39.9
Alternative 1: Weir at elevation 43 ft	37.2	38.9	39.9
Computed maximum WSEL reduction	0.0	0.0	0.0

Table 5. Alternative 2: Weir at elevation 44 ft, comparison of peak discharges to baseline modeling condition for 3 design events

Condition (1)	**2-yr Q_{peak} cfs (2)**	**10-yr Q_{peak} cfs (3)**	**100-yr Q_{peak} cfs (4)**
Baseline for DBs 10-13	370	700	1110
Alternative 2: Weir at elevation 44 ft	360	680	1110
Computed maximum flow reduction	10	20	0

Table 6. Alternative 2: Weir at elevation 44 ft, comparison of peak water surface elevations (WSEL) to baseline modeling condition for 3 design events

Condition (1)	**2-yr WSEL (2)**	**10-yr WSEL (3)**	**100-yr WSEL (4)**
Baseline for DBs 10-13	37.2	38.8	39.9
Alternative 2: Weir at elevation 44 ft	37.2	38.8	39.9
Computed maximum WSEL reduction	0.0	0.0	0.0

Conclusions

Modifying the outlets of the detention basins does not reduce the peak discharge or water surface elevation at the confluence with Elk Grove Creek for the p=0.50 (2-year) or p=0.10 (10-year) design events, even though the outlet discharge at the basins is reduced. For the p=0.01 (100-year) design event, the weir does not influence the basin outflow and the peak discharge is the same as the baseline flow for the modeled alternatives.

Because there is significant additional drainage area contributing below the detention basins, substantial amounts of flow are added to the drainage system by the time flow reaches Elk Grove Creek. This results in no reduction of the peak discharges in Elk Grove Creek, and consequently there will be no decrease in peak water surface elevation at this location resulting from detention basin modification.

An additional finding from this study is that DB 11 and 12 are not adequate to contain the p=0.01 (100-year) design discharges as computed by SacCalc. However, these discharges may not be as large as calculated by SacCalc because the program assumes that all calculated surface runoff can be conveyed directly to a downstream point. For the subdivisions contributing flow to DB 11 and 12, the drain inlets and below ground drainage pipes are sized to accommodate flows calculated by the Nolte method. The Nolte method typically provides peak discharges for a 2-yr to 5-yr magnitude. Flows greater than a p=0.10 (10-year) event cannot freely enter the drainage system leading to the detention basins, causing street ponding. This will significantly attenuate flow and reduce peak inflows to the detention basins. We did not quantify this, however, because this is beyond the scope of this study.

An initial finding showed that increasing the detention basin capacity could potentially reduce flows in Elk Grove Creek. However, the major conclusion from this study is that modification of the detention basin outlet design will not produce a reduction in peak discharges in the downstream reaches of Elk Grove Creek.

Attachment 1: Project location

Attachment 2: Rating curves developed for DB 10, 11 and 12, and 13

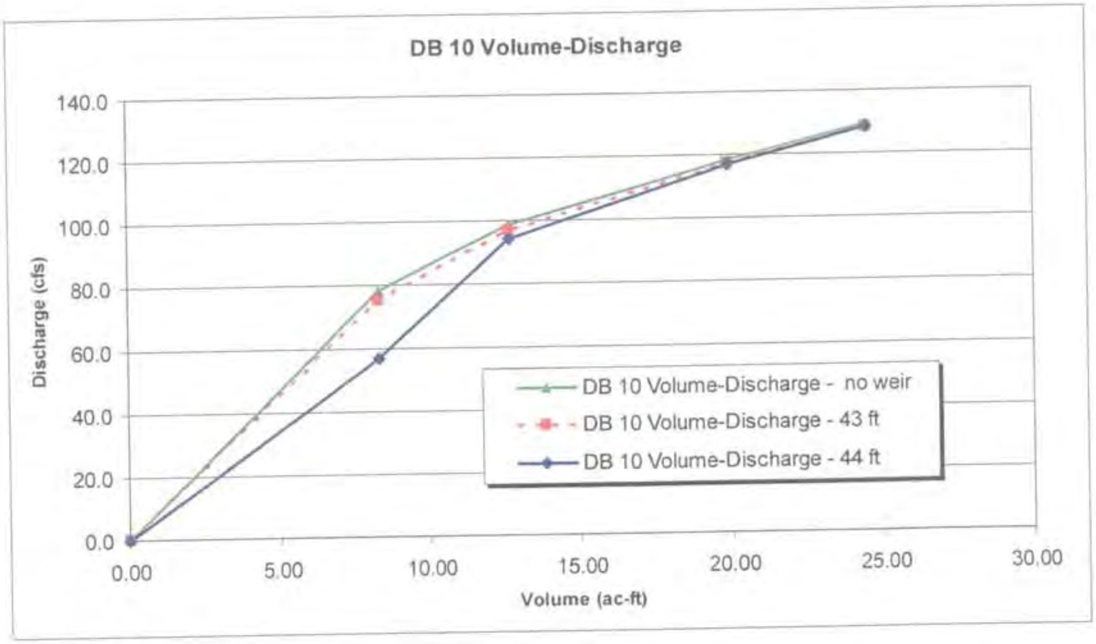


Figure A1. DB 10 rating curve

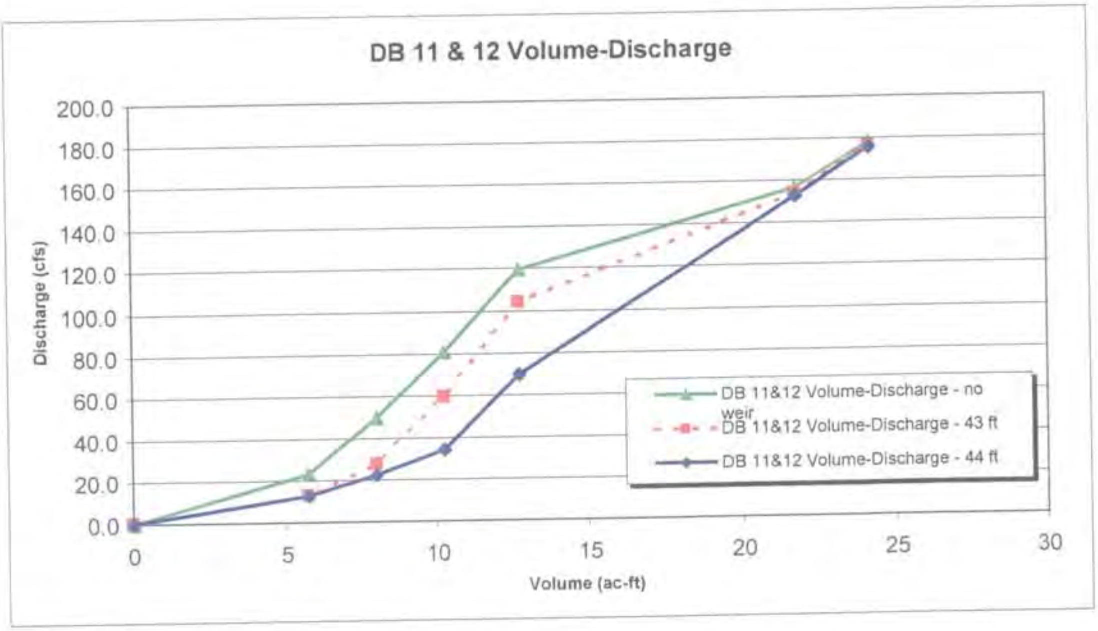


Figure A2. DB 11 and 12 rating curve

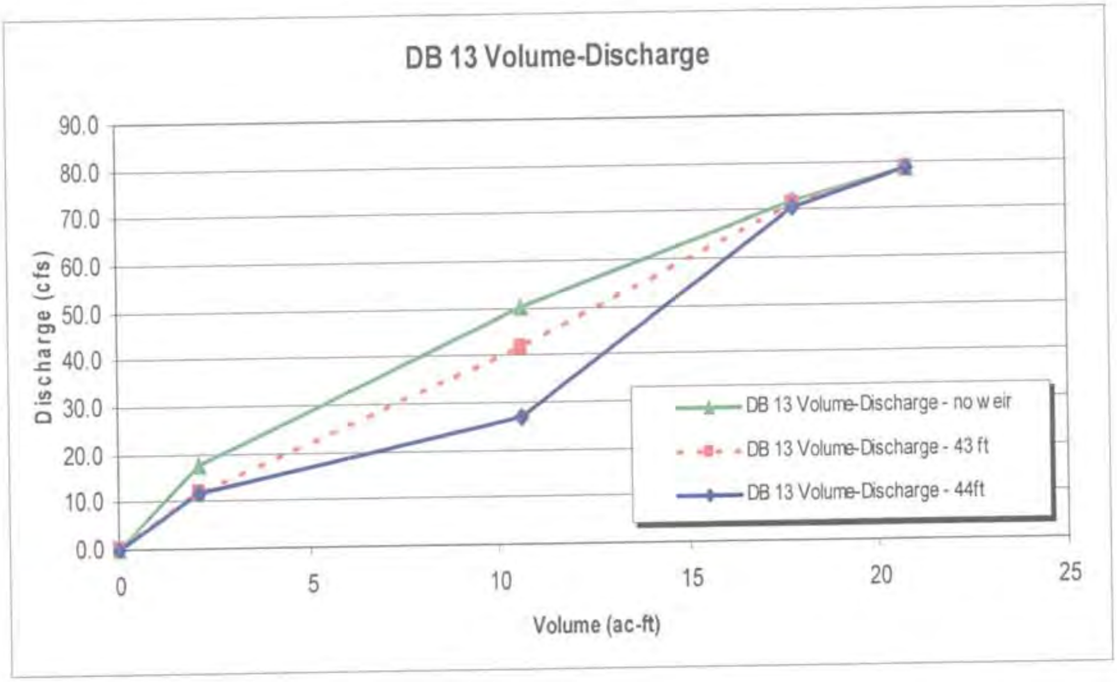


Figure A3. DB 13 rating curve

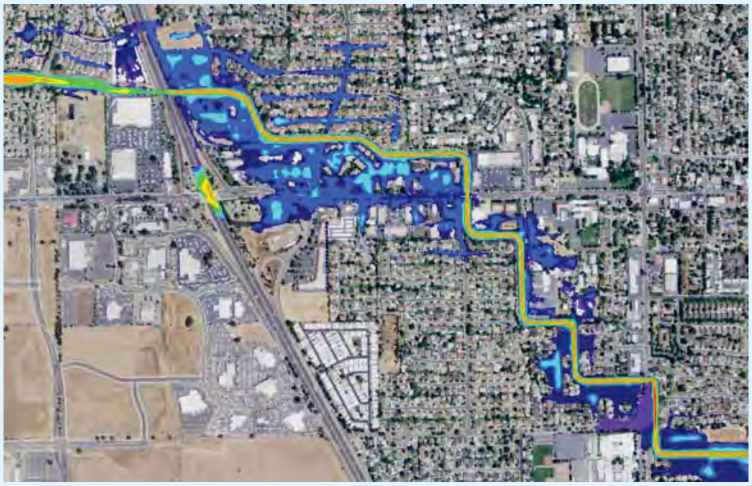
ATTACHMENT 5B

Elk Grove Creek Conceptual Hydrodynamic Modeling – Final Report

Attachment 5B

eco engineering

Hydrology | Hydraulics | Geomorphology | Design | Field Services



Elk Grove Creek Conceptual Hydrodynamic Modeling Final Report

Prepared for the
City of Elk Grove

March 2011
Project Number 10-1008

This report is intended solely for the use and benefit of the City of Elk Grove. No other person or entity shall be entitled to rely on the details contained herein without the express written consent of cbec, inc., eco engineering, 1255 Starboard Drive, Suite B, West Sacramento, CA 95691.

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GLOSSARY OF ACRONYMS

Acronym	Meaning
DB	Multi-Objective Detention Basin Project Site
DFCE	David Ford Consulting Engineers
EAC	Elk Grove Expert Advisory Committee
EGC	Elk Grove Creek
FP	Floodplain Area Project Site
HEC-RAS	Army Corps of Engineers – Hydrologic Engineering Center – River Analysis System
HWY	Highway
NCA	No Constraints Alternative
LiDAR	Light Detection and Ranging
WSE	Water Surface Elevation
WYA	West Yost Associates

1 INTRODUCTION

1.1 BACKGROUND

At the request of Fernando Dueñas of the City of Elk Grove (the City), cbec, inc., eco engineering (cbec) assisted in the development of a "no constraints" alternative for multi-objective channel management of Elk Grove Creek (EGC). The "no constraints" alternative that was developed is the product of field investigations by cbec as well as through collaboration with the Elk Grove Expert Advisory Committee (EAC) and the City. The purpose of this "no constraints" alternative was to investigate if any opportunities for flood attenuation, ecosystem and water quality enhancement have been missed through previous planning studies which feed into the Storm Drainage Master Plan for the City.

In order to evaluate the opportunities provided by the no constraints alternative (NCA), a two dimensional (2D) hydrodynamic model, MIKE21FM, was developed for the EGC corridor. This model was then used to simulate existing channel and floodplain conditions for EGC as well as for the NCA. In addition, the results of the existing condition simulation were compared to the results of the existing one dimensional HEC-RAS model of the creek, developed by West Yost Associates (WYA 2007). In addition, the existing HEC-RAS model was used as a data source for channel topography and boundary conditions.

1.2 STUDY OBJECTIVES

The goals and objectives of this conceptual hydrodynamic modeling effort are:

- Explore the maximum flood reduction benefit to EGC by utilizing floodplain areas and multi-objective detention basins to store floodwaters during a 100-yr 24 hr precipitation event.
- Explore multi-objective channel management opportunities.
- Build upon the existing 1D HEC-RAS model and its results with a 2D hydrodynamic model.
- Collaborate with the EAC in development of the NCA.

2 ALTERNATIVE DEVELOPMENT

2.1 NO CONSTRAINTS ALTERNATIVE

The development of the NCA was a collaborative process utilizing input from cbec staff, West Yost staff, City staff and certain members of the EAC. The first step was to analyze the EGC corridor on recent aerial photography in order to identify areas where channel/ floodplain expansion and multi-objective detention basin creation was possible due to a lack of existing infrastructure. Following this initial identification of possible areas, cbec staff walked the EGC corridor from intersection of Waterman Blvd and EGC to the confluence of Laguna Creek and EGC to further investigate the feasibility of various options, as well as identify others. After this field reconnaissance, cbec met with the City and West Yost staff to discuss the proposed project elements. During this meeting several alterations were suggested, which included the removal of some proposed elements, the inclusion of some new elements, and the expansion of others. The final NCA consists of five floodplain areas and ten multi-objective detention basins as shown in Figure 1.

2.2 EXISTING CONDITIONS

In order to develop a hydrodynamic model which accurately depicts the existing topographic conditions within and adjacent to EGC several data sources were utilized. These sources include Sacramento County LiDAR data collected in 2007 and cross sections extracted from the WYA HEC-RAS model for the reach between Elk Grove Florin Rd and Laguna Blvd some of which were collected in 2008. These data sources combined provide the best available topographic data for EGC and its floodplain. In AutoCAD, the geo-referenced channel cross section data and creek centerline were used to create a three dimensional surface representing EGC by interpolating between the individual cross sections. Given the homogeneity of the shape of the EGC channel this approach provides a realistic three dimensional representation of the channel derived from 2D cross section data. This surface was pasted into the LiDAR surface to provide the best representation of the channel bottom, banks and adjacent floodplains. LiDAR data could not be used solely, as the technology does not allow for an accurate representation of underwater areas, instead recording the water surface as the ground surface.

3 HYDRODYNAMIC MODEL DEVELOPMENT

3.1 EXISTING CONDITIONS MODEL

A 2D hydrodynamic model was developed for existing topographic conditions using DHI's MIKE21FM model (http://www.dhisoftware.com/). The modeling system is based on the numerical solution of 2D incompressible Reynolds averaged Navier-Stokes equations, consisting of continuity and momentum equations closed by a turbulent closure scheme. The spatial discretization of the equations is performed using a cell-centered finite volume method whereby the unstructured mesh provides the flexibility to adapt the mesh resolution to relevant physical scales. A combination of quadrilateral and triangular elements were used.

To facilitate comparison between the HEC-RAS results and the MIKE21FM results, effort was made to maintain consistency between the inputs. As discussed previously identical channel geometry and inflow boundary conditions were utilized. In addition, similar loss coefficients were used for the numerous hydraulic structures (i.e., culverts and bridges); however given the difference between the two models exactly identical methods were not possible in every instance. Three values for channel roughness (i.e., Manning's n) were used: n=0.035 for the reach upstream of Waterman Blvd and the tributary, n=0.045 for the reach between Waterman Blvd and Laguna Springs Dr., and n=0.055 for the reach downstream of Laguna Springs Dr. These roughness values are similar to the values used in the HEC-RAS model.

Figure 2 shows the model bathymetry and a sample of the model mesh used for the existing conditions simulations.

The model was run with flow input hydrographs for the 24-hr design storms for 2- and 100-yr recurrence intervals. These inflow hydrographs were taken from the HEC-RAS model, and were developed by WYA using SacCalc (DFCE 2001). As the events simulated were design storms, and not actual runoff events, calibration and validation of the MIKE21FM model was not possible. In addition, modification of the model was not undertaken to improve agreement between the HEC-RAS model results and the MIKE21FM model results for the existing conditions simulations.

3.2 ENLARGED HWY 99 BRIDGE MODEL

Previous hydraulic modeling of EGC has documented a significant backwater condition that results upstream of the HWY 99 Bridge. In order to investigate the possibility for flood stage reduction by increasing the capacity of the HWY 99 Bridge, the existing conditions model was slightly modified. In the existing condition there are two box culverts (12 ft wide X 6 ft high X 244 ft long) which pass water beneath HWY 99. For this alternative one additional box culvert with identical geometry was added to this location, per the direction of the City staff. No changes were made to the channel cross sectional geometry upstream or downstream of the bridge, or any other part of the model inputs, thus none of the components comprising the NCA were included.

3.3 NO CONSTRAINTS ALTERNATIVE MODEL

Following the development of the existing topographic conditions model, the topography and model mesh were altered to include the 15 NCA elements, which included channel modifications, channel realignments, floodplains and multi-objective detention basins. Channel modifications included the creation of a narrow low-flow channel, and a lowering of the adjacent floodplain areas/access road to allow for more channel capacity (Figure 3). All channel modifications were made within the existing creek corridor and did not extend into adjacent developed lands. In two areas, FP-1 and FP-3 (Figures 1 and 4), the channel alignment was realigned to a more sinuous path. In five locations areas adjacent to the channel were lowered to create floodplains which were inundated by flows less than the 2-yr flood level. At ten additional locations, multi-objective detention basins were created by excavating areas to no lower than the deepest point of the adjacent stream channel in order to facilitate drainage by gravity flow. Basins were separated from adjacent channels/floodplains by entrance sills which limited basin inundation to flows at or above the ~2-yr flood level. The areal extent and excavated volume of each of the NCA floodplains and multi-objective detention basins are provided in Table 1. Aside from the topographic modifications described above, identical boundary conditions, hydraulic structures and roughness values were used in the NCA simulations as the existing conditions simulations.

Figure 4 shows the model bathymetry and a sample of the model mesh used for the NCA simulations.

Table 1. No Constraints Alternative elements, size, excavated volume, simulated storage and approximate earthwork and land acquisition costs

Element	Description	Area (ac)	Excavated Volume (ac-ft)	Simulated Storage (ac-ft)	Earthwork and Land Acquisition Costs[1] (million USD)
Floodplain Areas					
FP-1	B/W Waterman and SPRR	22.9	85	23	$10.5
FP-2	North of Markofer School	1.9	9	6	$0.91
FP-3	US of HWY 99 (carlot)	4.3	15	14	$2.0
FP-4	At Sutter Expansion	9.5	58	25	$4.7
FP-5	Channel US of Elk Grove Blvd	0.2	0.6	0.6	$0.09
Multi-Objective Detention Basins					
DB-1	US Waterman	19.6	163	100	$10.5
DB-2	B/W Waterman and SPRR	34.6	400	131	$20.3
DB-3	DS SPPR river left	16.4	157	37	$9.1
DB-4	DS SPRR river right	5.3	47	23	$2.9
DB-5	Ranchette on EGC tributary	2.6	31	21	$1.5
DB-6	Markofer School Playground	3.2	16	19	$1.5
DB-7	North of FP-2	1.9	19	13	$1.1
DB-8	Park East of Markofer School	1.6	8	9	$0.77
DB-9	US of Elk Grove Blvd	6.8	28	25	$3.2
DB-10	West of FP-3	3.5	21	14	$1.7

Notes:

1. Earthwork and land acquisition costs calculated with the following values: $10/yd^3 for excavation and $400,000/ac for land acquisition, per the direction of the City. Values reported do not include project planning, design, environmental review, construction management, or the construction cost associated with drainage facilities (e.g., weirs, headwalls, culverts, bridges, etc.)
2. For location of the various NCA elements, please refer to Figure 1
3. US = upstream, DS = downstream
4. SPRR = Southern Pacific Railroad

4 RESULTS AND DISCUSSION

4.1 EXISTING CONDITIONS

Results from the existing conditions MIKE21FM model simulations are provided in Figures 5 to 7. A comparison of the 100-yr simulation WSE profiles for the HEC-RAS model and the MIKE21FM model under existing conditions shows reasonable agreement between the two models considering the numerical formulation differences between the two models (1D and 2D, respectively), as shown by Figure 5. While the results are not identical, they do show similar trends. In general the existing condition MIKE21FM model simulates a larger amount of head loss across the hydraulic structures (i.e., bridges) than shown by the HEC-RAS model results. This larger head loss, in general translates to greater backwater conditions upstream of the bridges and subsequently higher water surface elevations. A maximum difference of 1.52 ft is observed upstream of the Elk Grove – Florin Rd Bridge. Throughout the model domain, the MIKE model results were 0.31 ft higher on average when compared to the HEC-RAS model results.

Maximum inundation extents for the 2-yr design flood and 100-yr design flood are provided in Figures 6 and 7, respectively. Results from the 2-yr design flood simulation show that this level of flooding is contained within the existing creek channel. In addition, Figure 6 shows that the floodplain areas inundated at this flow level are negligible, with the one exception being the area downstream of Laguna Blvd where shallow floodplain inundation occurs. Maximum inundation extents for the 100-yr design flood are provided in Figure 7, along with the inundation extents predicted with HEC-RAS model. Results from this simulation confirm the need for flood management along the EGC corridor. Large areas of residential neighborhoods are inundated for this event, with the majority occurring downstream of Southern Pacific Railroad, and upstream of HWY 99. In these areas, inundation levels approach 3 ft in many areas. The inundation extents simulated with the MIKE21FM show remarkable agreement with the HEC-RAS simulated floodplain extents. Notable differences are observed upstream of HWY 99, where the MIKE results show floodwaters flowing north and then east and west down residential streets, where the RAS results only show inundation down the streets to the east. Table 2 shows the number of buildings which were affected in some way by this design flood flow simulation.

4.2 EXPANDED HWY 99 BRIDGE

A comparison of existing condition 100-yr simulation results with the expanded HWY 99 Bridge simulation results indicate that the addition of a third identical box culvert at HWY 99 did not have a large impact on 100-yr flood water surface elevations. While minor amounts of stage reduction were observed upstream of the bridge, they were generally small and did not persist very far upstream. A maximum WSE reduction of 0.4 ft was simulated just upstream of the HWY 99 Bridge, with 0.2 ft of reduction ~1,600 ft upstream of the bridge, and 0.1 ft of reduction ~7,150 ft upstream of the bridge. Upstream of this point notable WSE reductions were not detected. The additional box culvert increased WSE downstream by up to 0.2 ft for the first ~200ft downstream of the bridge, and 0.1 ft on average for the remainder of the downstream channel.

4.3 NO CONSTRAINTS ALTERNATIVE

Results from the NCA MIKE21FM model simulations are provided in Figures 8 to 12. A comparison of the MIKE21FM 100-yr simulation WSE profiles for the existing conditions and NCA demonstrate the magnitude of flood reduction that is possible through the combination of the various NCA elements. A comparison of 100-yr simulated WSE profiles for the two alternatives demonstrates the magnitude of reduction which is possible by utilizing multi-objective flood management techniques (i.e., channel modifications, floodplains and multi-objective detention basins). A maximum benefit of 4.19 ft of WSE reduction is achieved between Waterman Rd and the SPRR Bridge. Downstream from this reach, 3.35 ft of reduction is the maximum benefit, reducing to 2 ft and less below Falcon Meadow Rd. On average, there is 1.55 ft WSE reduction achieved throughout the model domain.

A majority of the flood benefit is attributed to the large floodplain (FP-1) and multi-objective detention basins (DB-1 to DB-4) which are located in the upper reach of the creek, which store a large volume of flood water, drastically reducing peak flow magnitude to the downstream reaches. Figure 10 shows a comparison of existing condition and NCA 100-yr hydrographs for three locations within the model domain. The upper most panel, which shows a hydrographic comparison just downstream of the confluence of the northern branch (or tributary) of EGC (near Markofer Elementary School), indicates that upstream storage reduced the peak flow magnitude from 1,023 cfs to 608 cfs.

Maximum inundation extents for the 2-yr design flood and 100-yr design flood are provided in Figures 11 and 12, respectively. Results from the 2-yr design flood simulation show that with the NCA modifications, large areas are inundated by flows equaling or less than the 2-yr design flood. These areas provide the opportunity for the creation of riparian forests and natural spaces within the city limits. These floodplains and multi-objective detention basins would provide open spaces and natural habitats for native plants and animals, which could be enjoyed by humans as well. In addition, these areas present opportunities for water quality improvement due to the deposition of fine sediment and particulate matter and the uptake of nutrients by vegetation.

Maximum inundation extents for the 100-yr design flood with NCA modifications are provided in Figure 12. This figure when compared to Figure 7 demonstrates that substantial flood reduction could be achieved through multi-objective flood management strategies. Approximately 459 ac-ft of flood waters are detained in the various NCA elements resulting in peak flow magnitude, WSE and inundation extent reductions during the 100-year design flood, as shown. A small amount of inundation still occurs in the area south of EGC near Emerald Vista Rd, however the widespread inundation simulated in the existing conditions simulations is mostly alleviated. As shown in Table 2, 5 structures are affected by floodwaters in the NCA as compared to 136 in the existing conditions simulation.

Table 2. Number of structures, by type, inundated during the 100-yr event

Type of Structure	Existing Conditions	No Constraints Alternative
Home	104	4
Commercial / Public	21	1
Apartment	11	0
Total	136	5

Note:

1. Structures were counted in any part of the main structure was covered by the 100-yr inundation predicted by the MIKE21FM models as shown in Figures 7 and 12.
2. Inundated structures in the NCA do not include structures which would be removed in the construction of the NCA project elements.

5 CONCLUSION AND RECCOMMENDED NEXT STEPS

This conceptual hydrodynamic modeling exercise demonstrates the substantial opportunities for concomitant flood attenuation, ecosystem and water quality enhancement through multi-objective flood management strategies on Elk Grove Creek through the City of Elk Grove. NCA simulations demonstrate an increase in channel floodplain connectivity for frequently occurring floods as well as substantial flood risk reduction for larger magnitude flood events. In summary, 134 acres of land are proposed for enhancement providing flood risk reduction, water quality improvements, ecosystem enhancements, aesthetic enhancement as well as recreational opportunities.

While the various project elements were iteratively designed in order to achieve inundation during the 2-yr flow, and maximum flood reduction benefits for the 100-yr flow, further optimization is possible though not pursued at this time. These results could provide additional improvements to both ecosystem function and flood control. In addition, the role of the individual NCA elements should be investigated in order to determine the cost:benefit ratio of each element. Approximate earthwork and land acquisition costs for each NCA elements are provided in Table 1 in order to allow for an initial consideration of costs vs. benefits for the NCA. In addition to the investigation of individual elements, future analysis should include simulation of a subset of the original 15 NCA elements which may be more feasible as actual constraints are recognized. For example one possible future scenario that has been identified includes DB-1, DB-2, FP-1, FP-5 and DB-8, as shown in Figure 1 and Table 1.

While reasonable agreement was observed between the HEC-RAS model results and those simulated with the 2D MIKE21FM hydrodynamic model, caution should be used when translating the model results to reality, as the MIKE21FM model was not calibrated or validated. Future efforts should include model calibration and validation for a real flood event (e.g., ~45-yr 12-hr event which occurred in December 2005) in order to improve the predictive ability of the model. Furthermore, we recommend that the incremental benefit of flood reduction elements be assessed individually, rather than cumulatively, as has been done through this study.

6 REFERENCES

West Yost Associates (WYA). 2007. City of Elk Grove – Flood Control and Storm Drainage Master Plan Study.

DHI. 2009. MIKE 21 Flow Model: Hydrodynamic Module Scientific Documentation. Danish Hydraulics Institute.

David Ford Consulting Engineers (DFCE). 2001. SacCalc software version 1.1. http://www.msa2.saccounty.net/dwr/Pages/SacCalc.aspx.

7 LIST OF PREPARERS

Chris Hammersmark, Ph.D., P.E., Project Manager
April Sawyer, B. S., Lead Modeler
Chris Bowles, Ph.D., P.E., Project Director

8 ACKNOWLEDGMENTS

Fernando Duenas, Clarence Korhonen, Darren Wilson, City of Elk Grove
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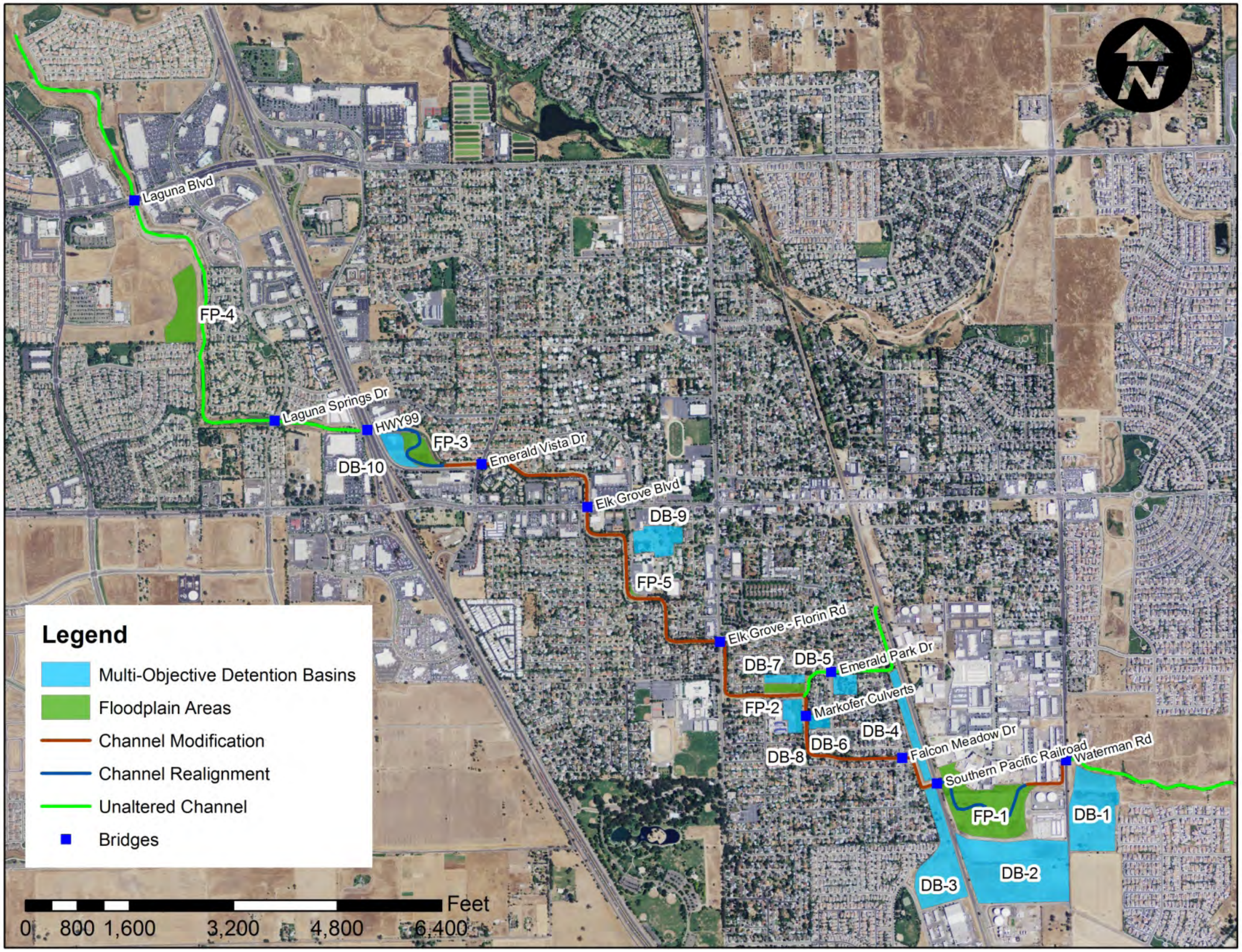
Sam Diaz, P.E., ACAD assistance



Notes:

Elk Grove Creek Conceptual Hydrodynamic Modeling

Elements of the No Constraints Alternative Design

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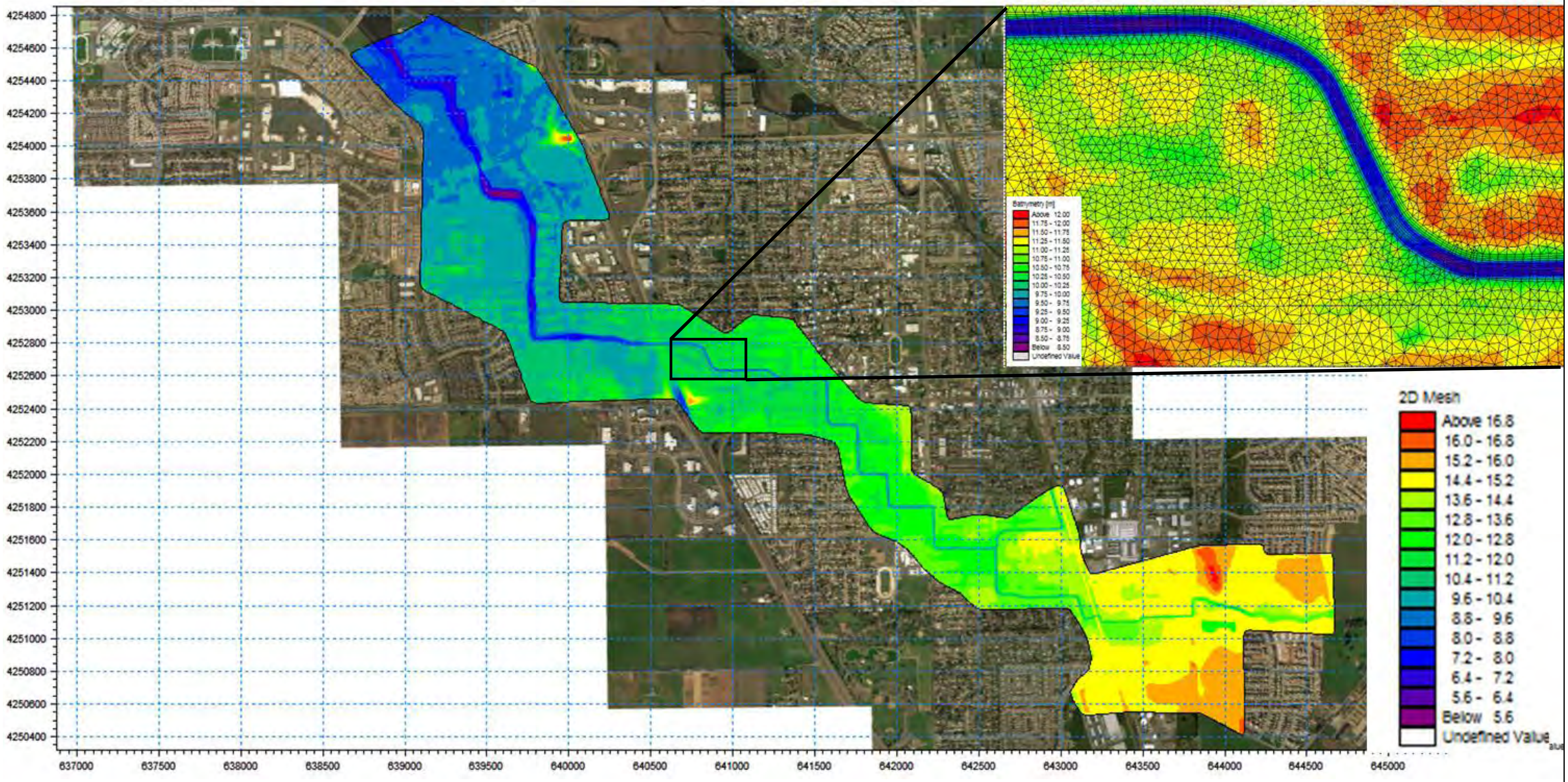
Figure 1



Notes: Inset plot shows mesh at the car lot upstream of Highway 99. Elevations in NGVD29, meters.

Elk Grove Creek Conceptual Hydrodynamic Modeling

Existing Conditions MIKE21FM Model Mesh

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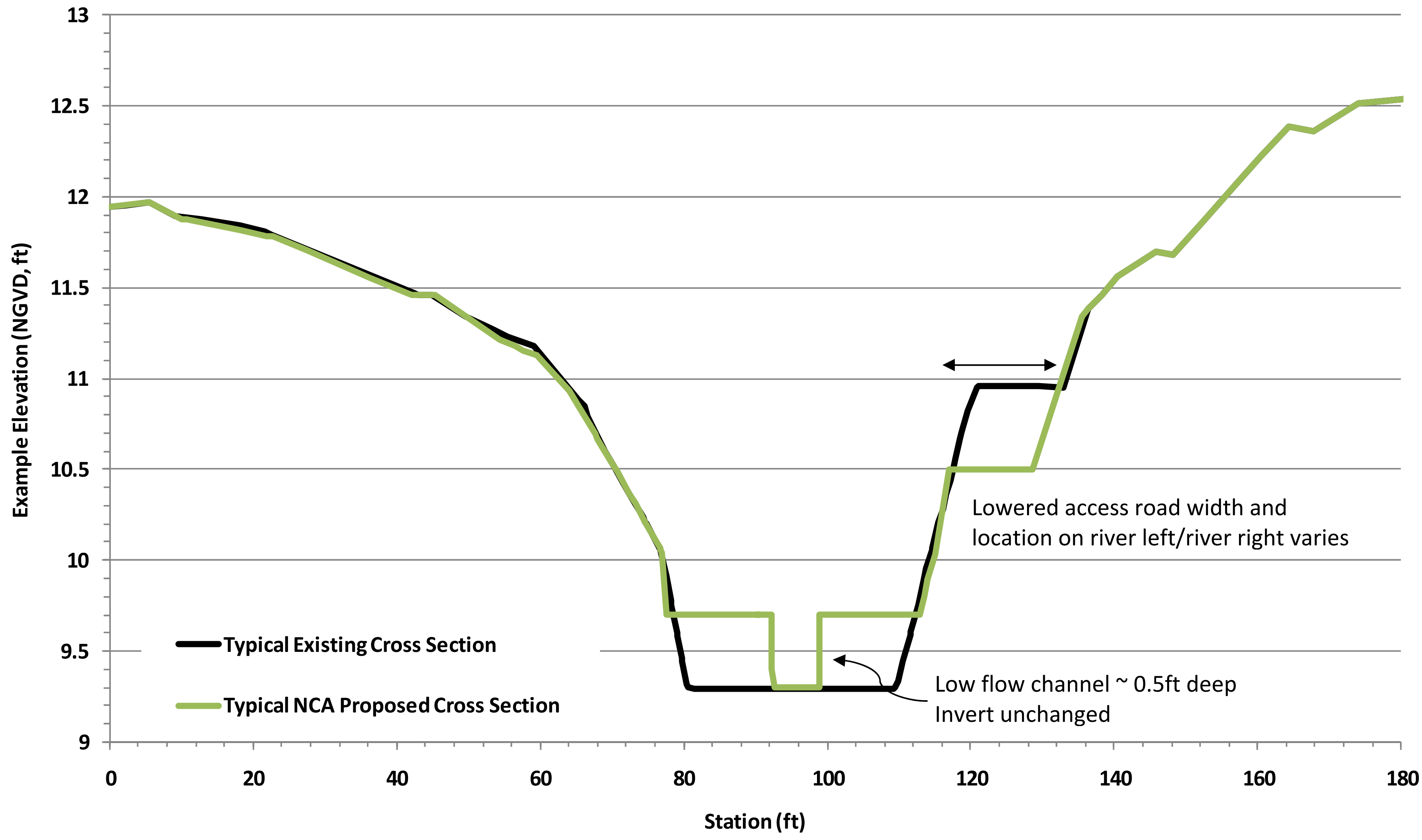
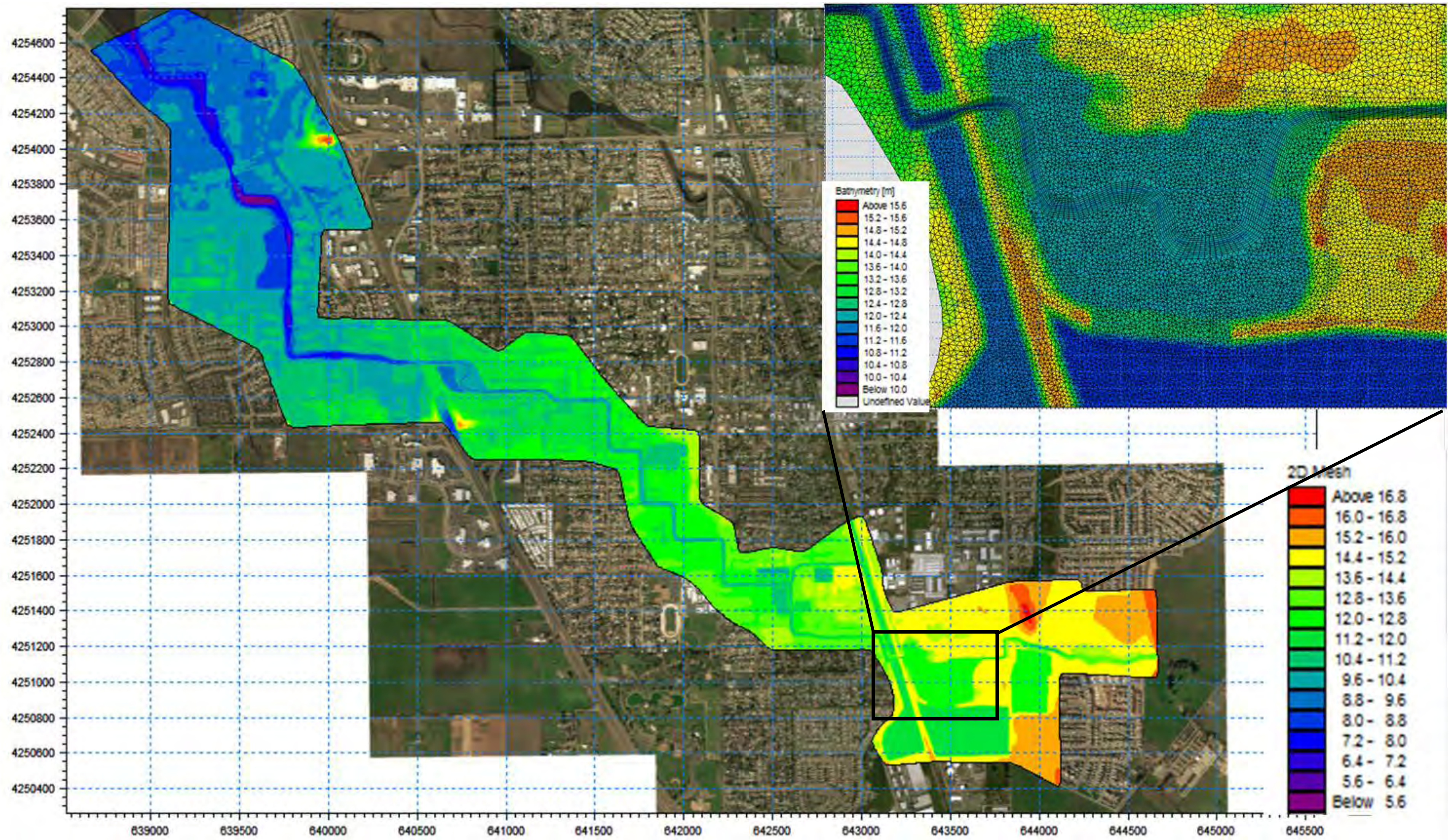
Figure 2



Elk Grove Creek Conceptual Hydrodynamic Modeling

Typical Existing Conditions and NCA Cross Sections

Project No. 10-1008	Created By: AMS	**Figure 3**

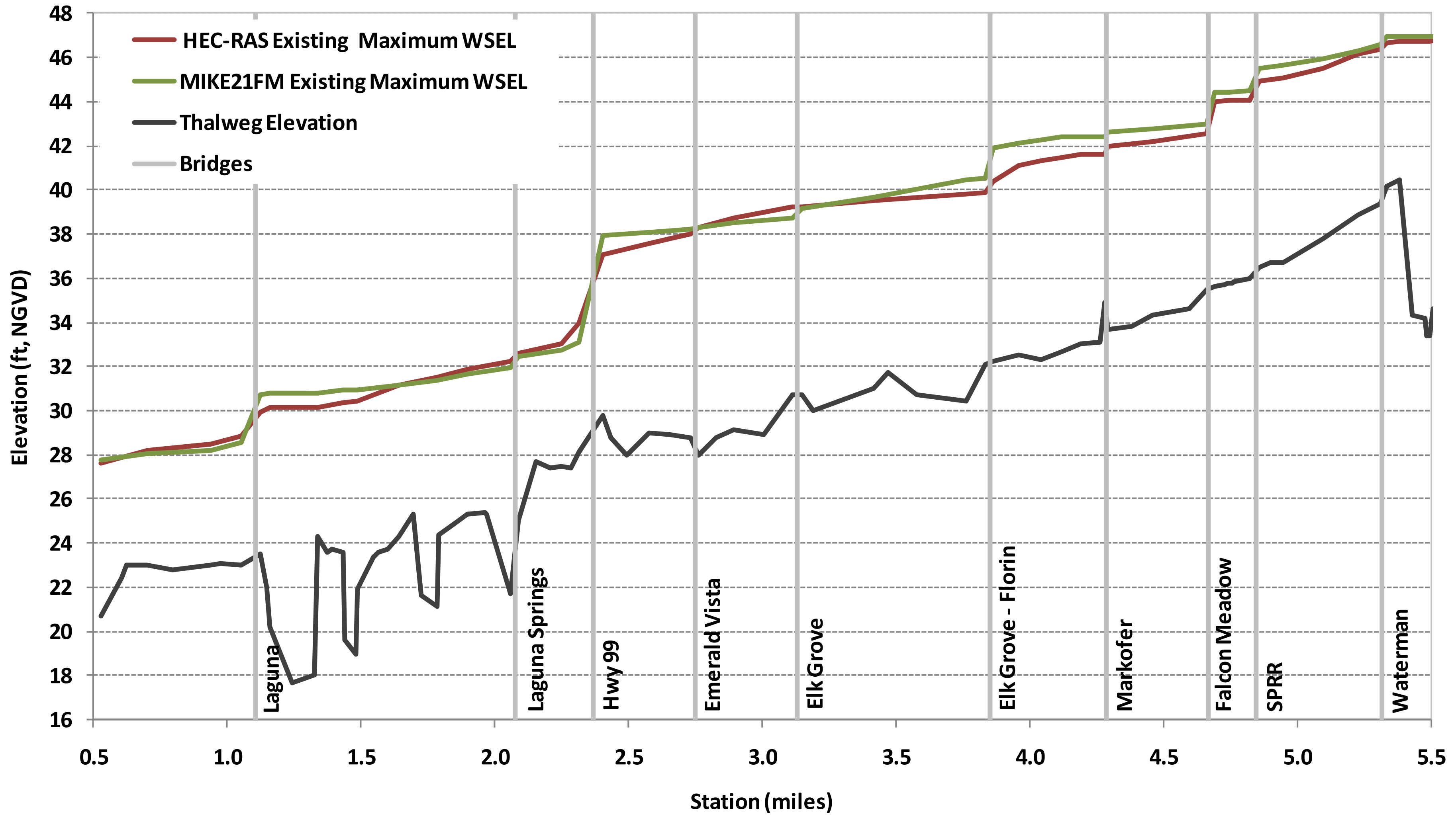


Notes: Inset plot at lot upstream of rail road tracks. Elevations in NGVD29, meters.

Elk Grove Creek Conceptual Hydrodynamic Modeling

No Constraints Alternative MIKE21FM Model Mesh

Project No. 10-1008	Created By: AMS	**Figure 4**

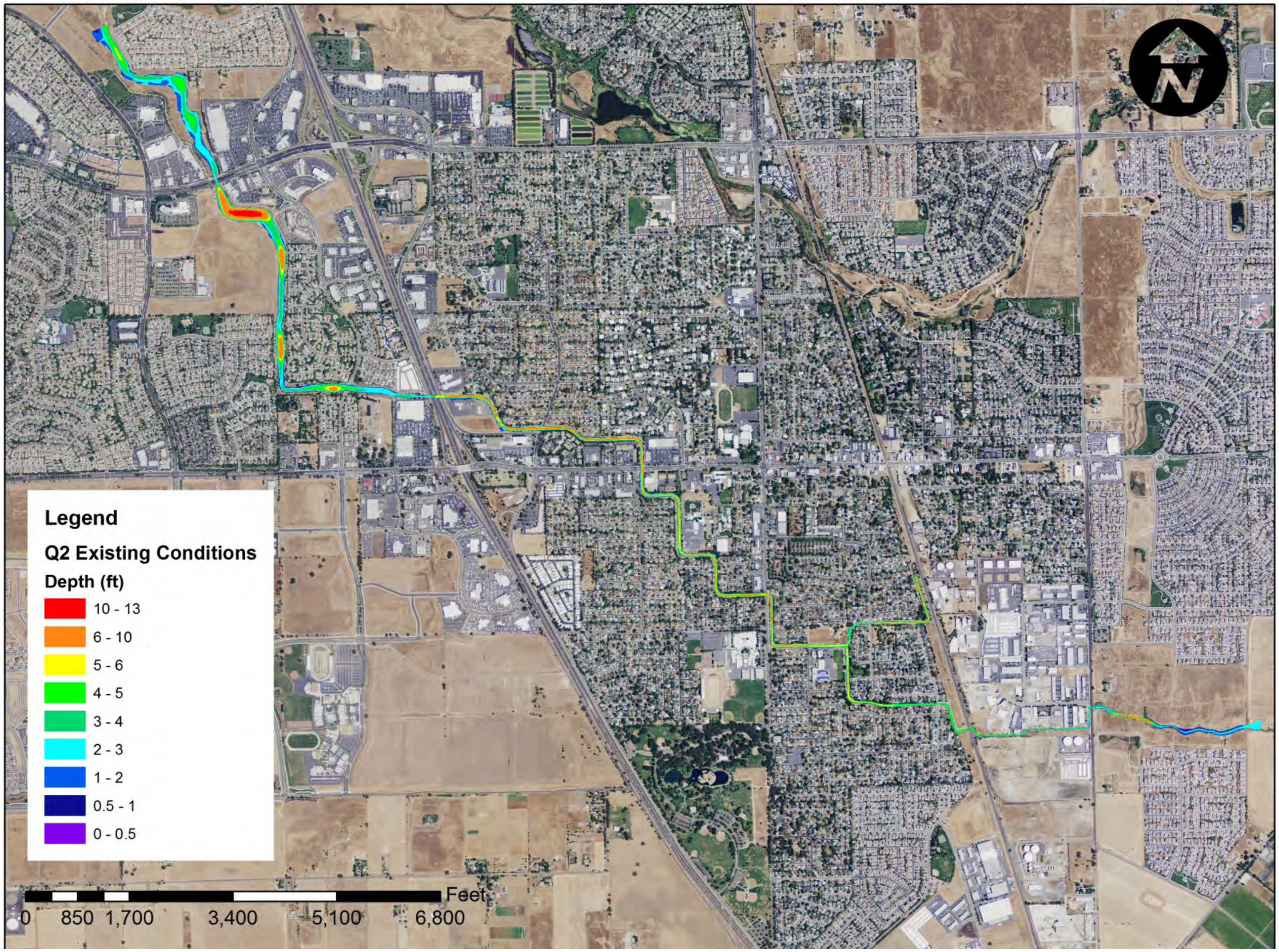


Notes: (WSEL) water surface elevation

Elk Grove Creek Conceptual Hydrodynamic Modeling

Existing Conditions 100-Yr Long Profile Comparison

Project No. 10-1008	Created By: AMS	**Figure 5**



Notes: Q2 is the 2-Yr flow event for Elk Grove Creek, or the flow event that has a 50% chance of occurrence in any given year.

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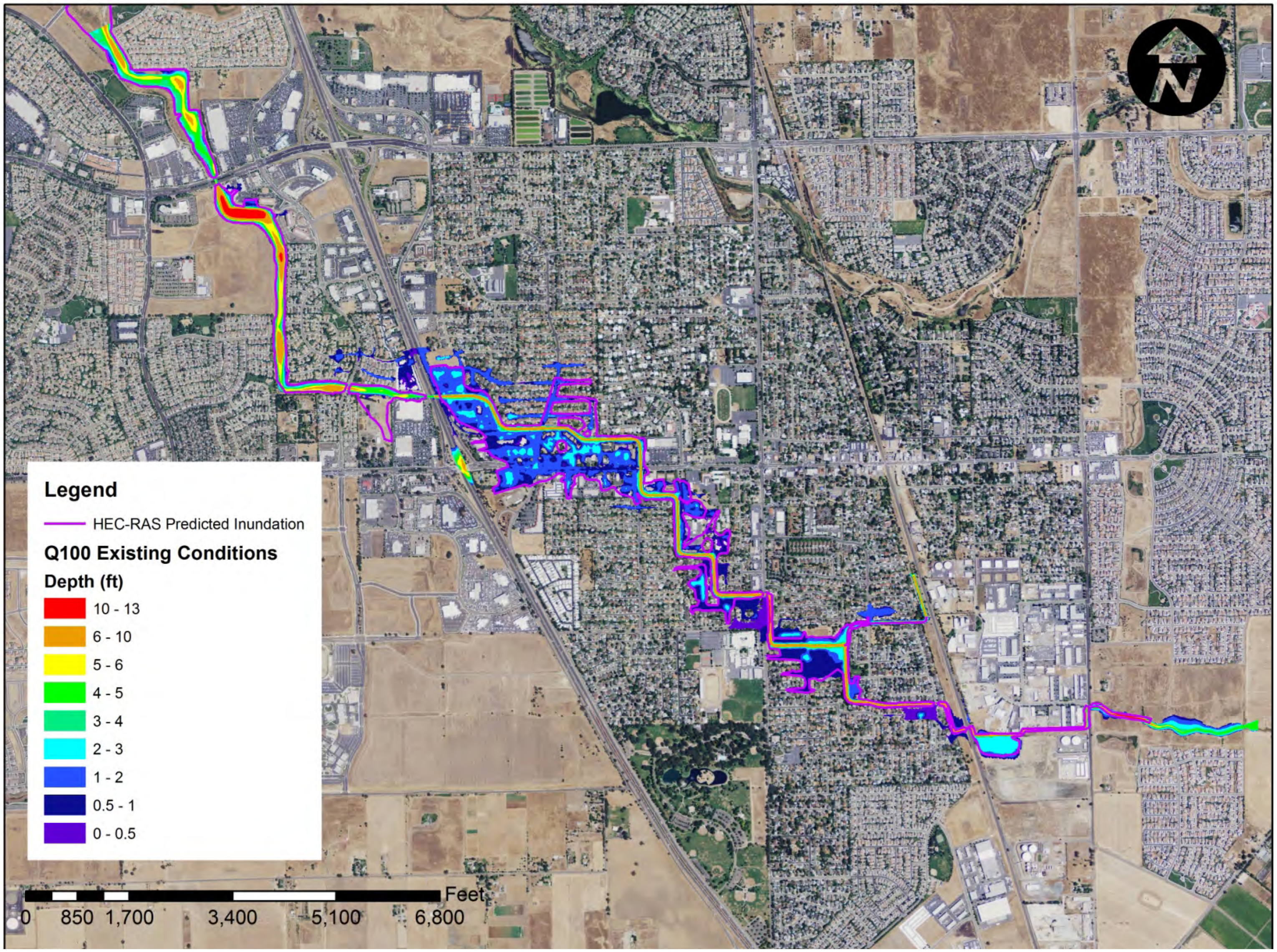
Elk Grove Creek Conceptual Hydrodynamic Modeling

Existing Conditions 2-Yr Max Inundation Extents

Project No. 10-1008	Created By: AMS	**Figure 6**

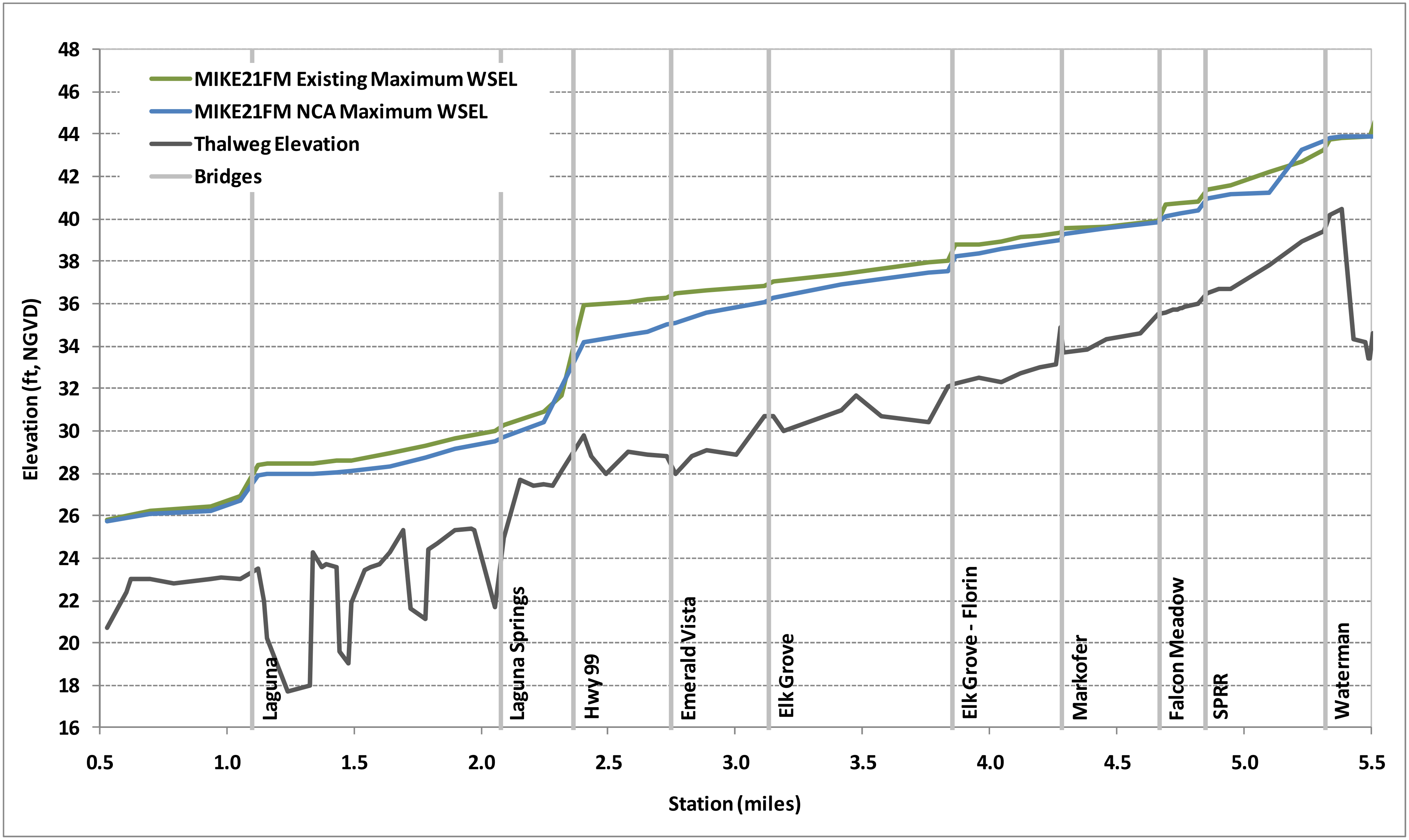


Notes: Q100 is the 100-Yr flow event for Elk Grove Creek, or the flow event that has a 1% chance of occurrence in any given year.

Elk Grove Creek Conceptual Hydrodynamic Modeling

Existing Conditions 100-Yr Max Inundation Extents

Project No. 10-1008 | Created By: AMS | **Figure 7**



Notes: Q2 is the 2-Yr flow event for Elk Grove Creek, or the flow event that has a 50% chance of occurrence in any given year.

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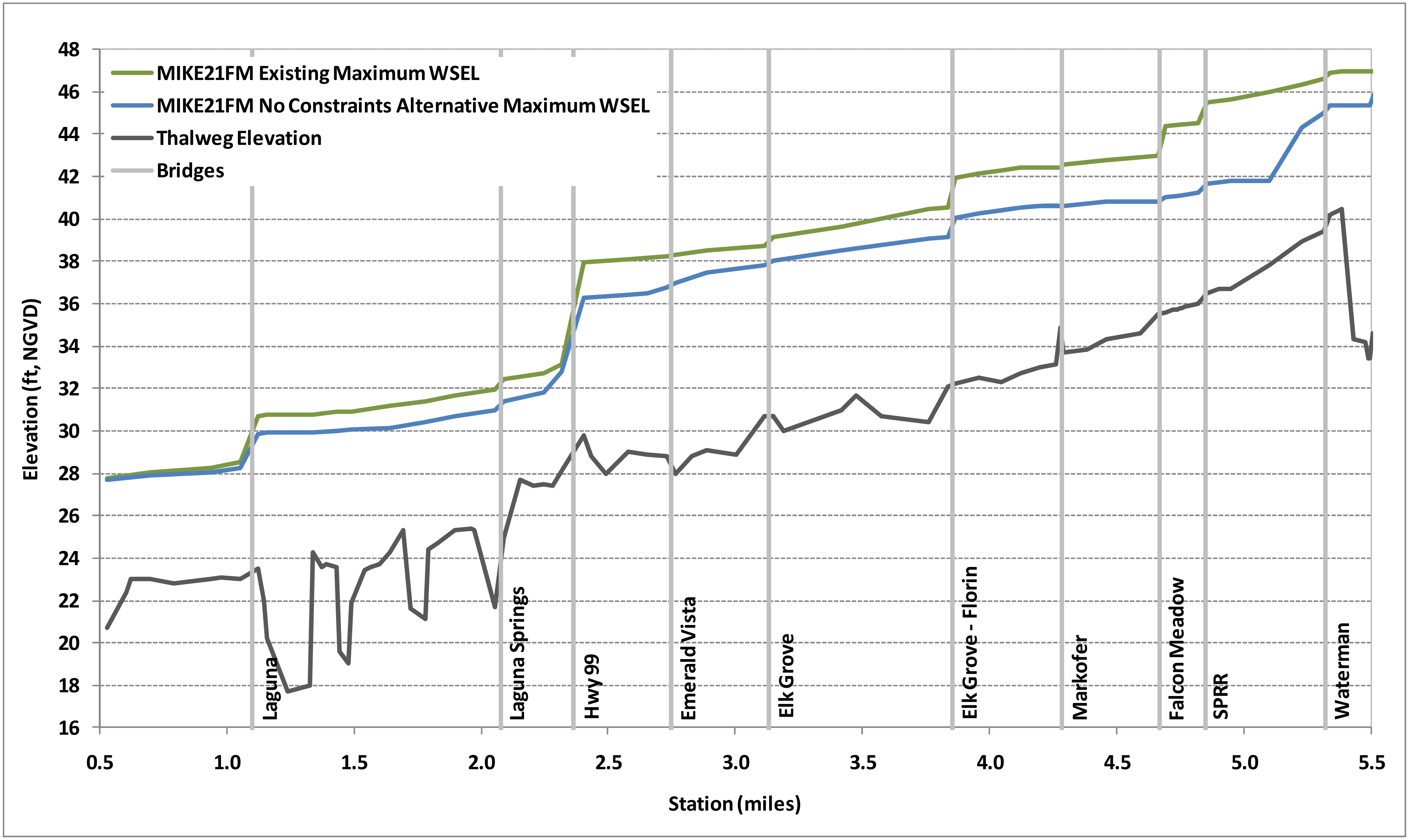
Elk Grove Creek Conceptual Hydrodynamic Modeling

No Constraints Alternative 2-Yr Long Profile Comparison

Project No. 10-1008 | Created By: AMS | **Figure 8**

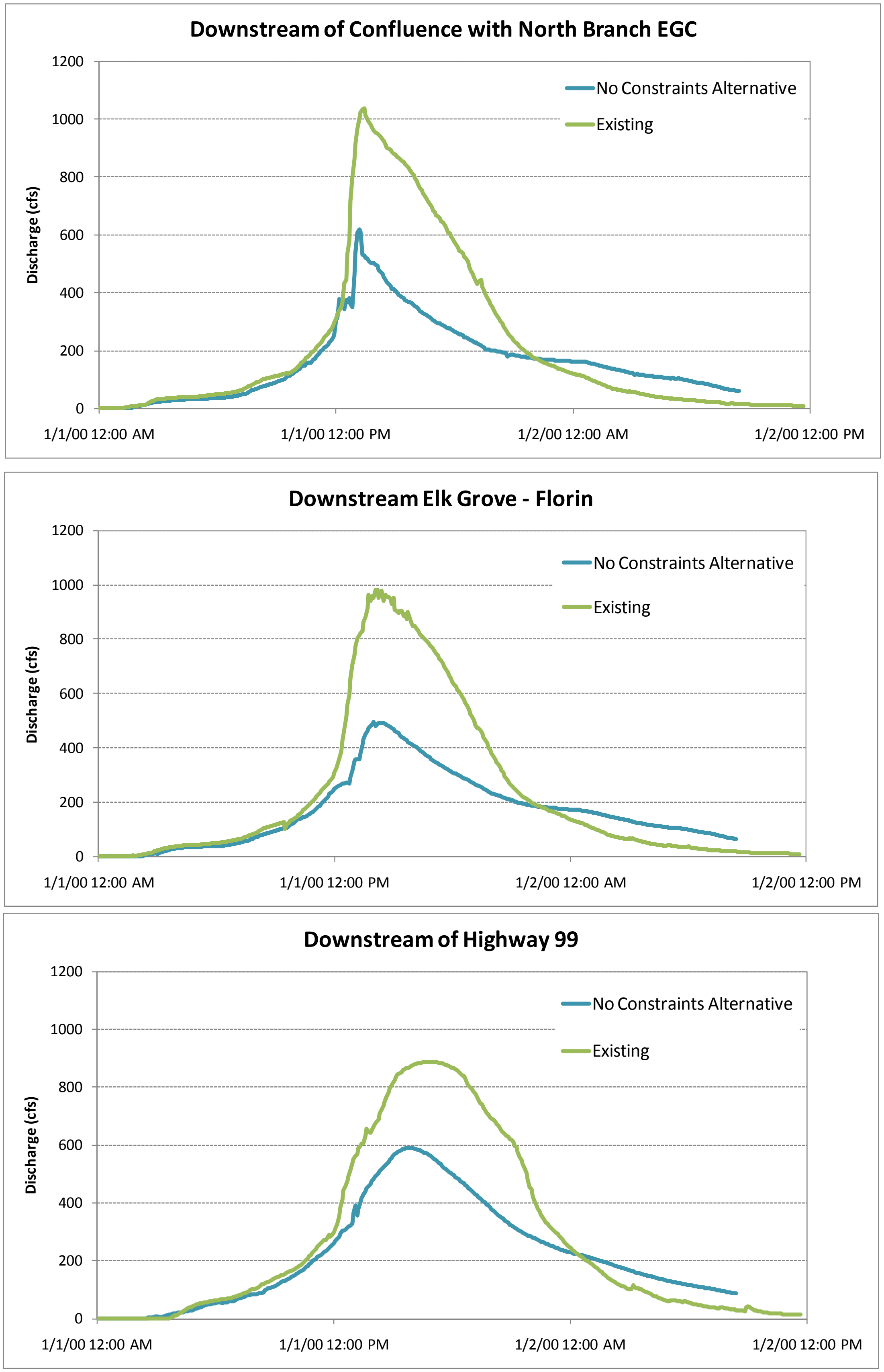


Notes: Q100 is the 100-Yr flow event for Elk Grove Creek, or the flow event that has a 1% chance of occurrence in any given year.

Elk Grove Creek Conceptual Hydrodynamic Modeling

No Constraints Alternative 100-Yr Long Profile Comparison

Project No. 10-1008	Created By: AMS	**Figure 9**



Notes:

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Elk Grove Creek Conceptual Hydrodynamic Modeling

100-Yr Hydrograph Comparisons

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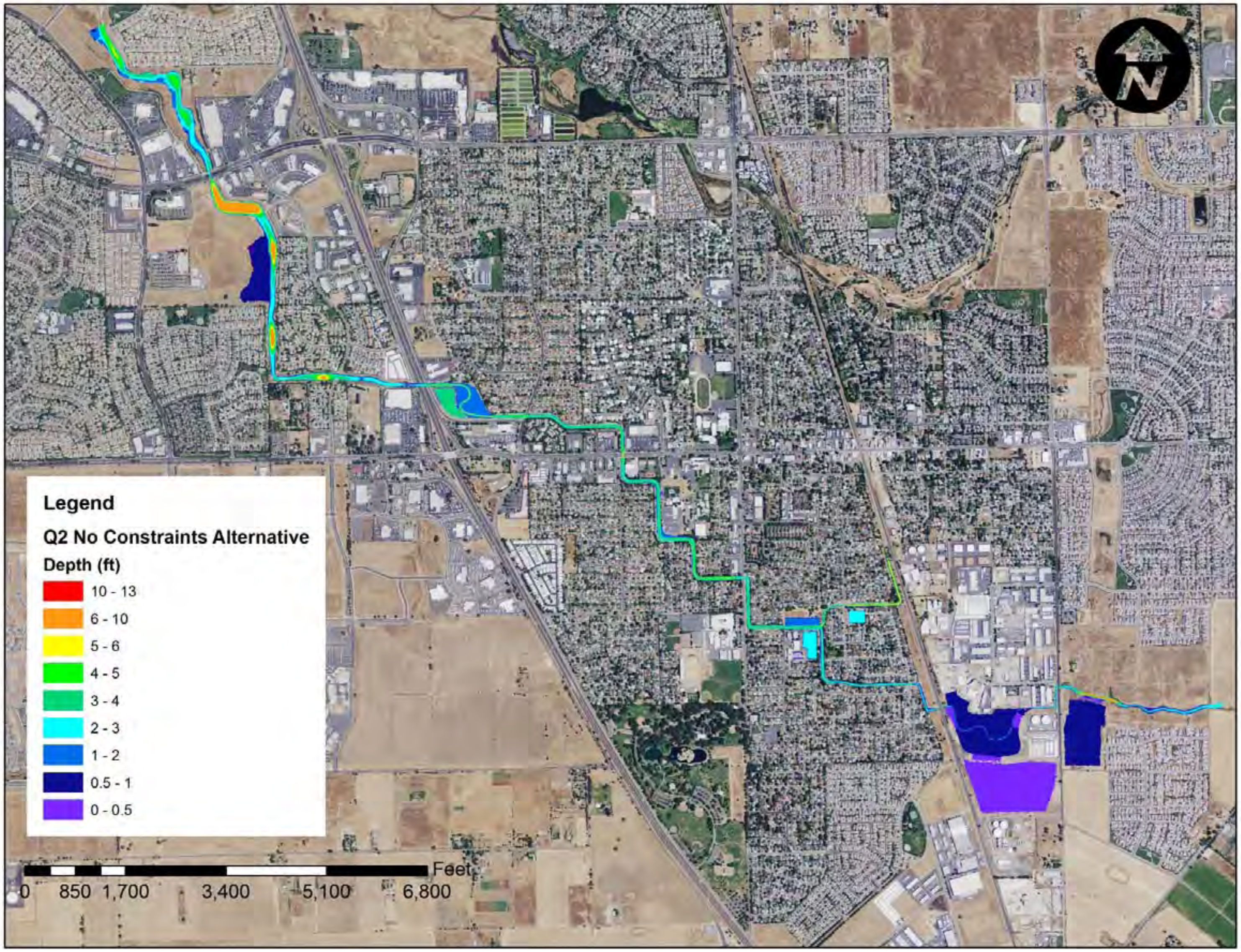
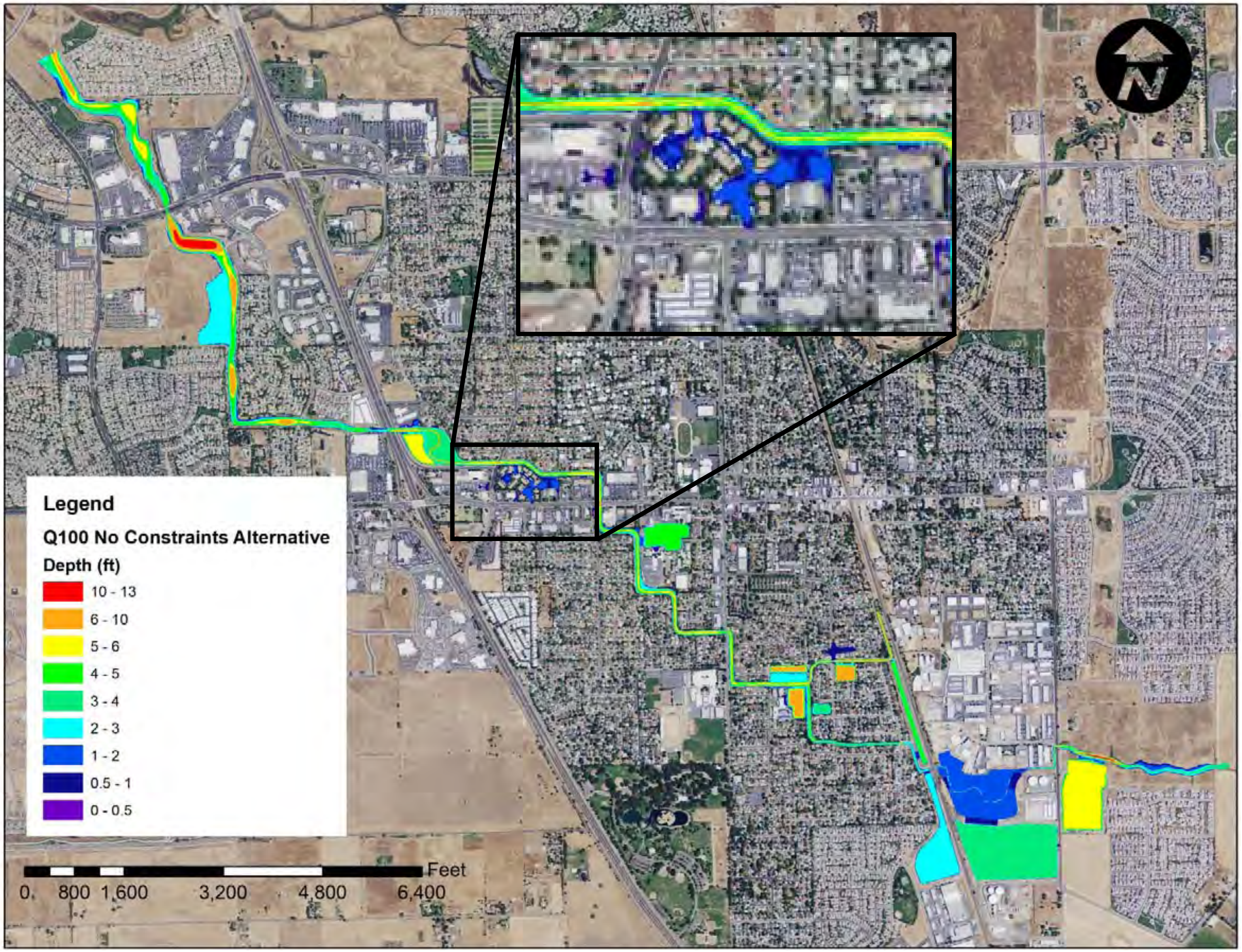
Figure 10



Notes: Q2 is the 2-Yr flow event for Elk Grove Creek, or the flow event that has a 50% chance of occurrence in any given year.	cbec eco engineering	*Elk Grove Creek Conceptual Hydrodynamic Modeling* **No Constraints Alternative 2-Yr Max Inundation Extents**	
		Project No. 10-1008	Created By: AMS	**Figure 11**



Notes: Q100 is the 100-Yr flow event for Elk Grove Creek, or the flow event that has a 1% chance of occurrence in any given year.

Elk Grove Creek Conceptual Hydrodynamic Modeling

No Constraints Alternative 100-Yr Max Inundation Extents

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Figure 12

CHAPTER 6. EAST ELK GROVE AREA/RURAL REGION

WATERSHED DESCRIPTION

The East Elk Grove area/rural region lies in the eastern part of the City and covers approximately 6,800 acres. The area is generally bounded by Waterman Road and Laguna Creek on the west, Calvine Road on the north, and Grant Line Road on the southeast (See Figure 6-1). This area covers portions of both the Laguna Creek and Elk Grove Creek watersheds. The evaluation of the drainage facilities in this area was previously performed by Harris & Associates and their analyses and results are incorporated into this chapter. In addition, the drainage improvements required in the portion of the Elk Grove Creek watershed that lies within the East Elk Grove Specific Plan area were evaluated by MacKay & Somps Civil Engineers, Inc. The recommended facilities from the MacKay & Somps study are also incorporated into this chapter.

EXISTING DRAINAGE FACILITIES

Existing storm drainage features and facilities in the study area include the following:

- Natural and semi-natural creeks and channels
- Excavated channels and roadside ditches
- Local detention basins serving low density development projects and individual properties
- Storm drains in streets and selected subdivisions
- Cross-drainage structures at streets, driveways, railroads, and facilities on private property

The more prominent creeks and channels, the larger detention basins, existing underground storm drains, and cross drainage structures are represented on Figure 6-2. A field inventory and measurement process was completed to determine the location, size and characteristics of existing cross-drainage structures at arterial, collector, and local streets. The data that was collected included measurement of cross-drainage structure dimensions, cross-drainage structure lengths, height dimensions from invert to top of roadway, and photographs of the drainage structure inlets and outlets. This has been provided to the City as a separate document in hard copy and electronic format.

A HEC-RAS hydraulic model was formulated for each cross-drainage structure and was run iteratively for different discharges to determine the capacities of each of them prior to flow overtopping the roadway. The existing capacity of each of these cross-drainage structures is listed on Table 6-1. As shown on Table 6-1, there are numerous cross-drainage structures that currently have a very limited capacity when compared against the potential runoff rates that will contribute to them. As a result, there will be flooding of several streets in the study area during major storm events.

Table 6-1. Capacities of Existing Cross-Drainage Structures(1)

No.	Culvert I.D.	Location	Type of Structure	Size(2)	Existing Capacity, cfs	Upgrade 10-year Q, cfs	Upgrade 100-year Q, cfs
Laguna Creek Tributary # 1							
1	LT1-C01	Calvine Road/Grant Line Road	Circular Culvert	2 Cells -18"	24	130	223
2	LT1-C02	Grant Line Rd. South of Calvine Rd.	Circular Culvert	1 Cell - 18"	9	18	31
3	LT1-C03	Kinder Ln South of Calvine Rd.	Circular Culvert	4 Cells - 36"	128	266	444
4	LT1-C04	Excelsior Rd. South of Calvine Rd.	Box Culvert	2 Cells - 10'x5'	444	78	155
5	LT1-C05	Excelsior Rd. @ Halfway Rd 40' West	Circular Culvert	1 Cell - 48"	*	119	213
6	LT1-C06	Corfu Dr. 400' W of Excelsior Rd.	Circular Culvert	3 Cells - 48"	261	153	274
7	LT1-C07	Calvine Rd. 1050' W of Excelsior Rd.	Circular Culvert	1 Cell - 36"	61	36	66
8	LT1-C07B	Calvine Rd. 1700' W of Excelsior Rd.	Box Culvert	2 Cells - 7'x4'	334	275	487
9	LT1-C08	Calvine Rd. 2500' W of Excelsior Rd.	Circular Culvert	1 Cell - 40"	*	71	125
10	LT1-C10	Mecca Rd. 500' S of Cul- de Sac	Circular Culvert	3 Cells - 24"	60	20	36
11	LT1-C11	Lincott Ct 500' E of Sleepy Hollow Ln	Circular Culvert	3 Cells - 30"	122	27	47
12	LT1-C12	Corfu Dr. 400' E of Sleepy Hollow Ln	Circular Culvert	3 Cells - 30"	100	53	95
13	LT1-C13	Corfu Dr. 450' w of Sleepy Hollow Ln	Circular Culvert	1 Cell - 18"	9	7	13
14	LT1-C14	Atlantis Dr. @ Sleepy Hollow Unit 2	Circular Culvert	1 Cell - 15"	6	6	11
15	LT1-C15	Corfu Dr. @ Sleepy Hollow Unit 2	Circular Culvert	1 Cell - 24"	19	19	32
16	LT1-C17	Chambray Ln 5' S of Denim Rd	Circular Culvert	1 Cell - 12"	4	3	5
17	LT1-C18	Mecca Rd. 900' N of Chambray Ln	Arch Culvert	1 Cell 18"x12"	3	27	50
18	LT1-C19	Sleepy Hollow Ln 1000' S of Linscott Ct	Circular Culvert	1 Cell - 12"	4	72	131
19	LT1-C20	CCT 900' W of Sleepy Hollow Ln	Circular Culvert	1 Cell - 24"	20	15	24
20	LT1-C21	Cherrington Ln 2400' N of Sheldon Rd	Circular Culvert	1 Cell -30"	31	62	122
21	LT1-C21A	Mix Ln 1300' E of Bader Rd	Circular Culvert	1 Cell - 20"	15	129	229
22	LT1-C22	CCT 1900' SE of Calvine Rd	Bridge	2 Cells - 13'x11'	2000	649	1135
23	LT1-C23	Bader Rd 2000' N of Sheldon Rd	Bridge	2 Cells - 30'x 7.5'	990	674	1169
24	LT1-C24	Sheldon Rd 300' E of Cherrington Ln	Circular Culvert	1 Cell - 18"	12	54	100
25	LT1-C25	Bader Rd 600' S of Sheldon Rd	Circular Culvert	1 Cell - 15"	6	43	79
26	LT1-C26	Sheldon Rd 1100' W of Bader Rd	Circular Culvert	2 Cells - 24"	38	92	167
27	LT1-C27	Sheldon Rd at Bradshaw Rd	Bridge	2 Cells - 15'X7'	900	637	1167
28	LT1-C27A	Sheldon Rd 135' E of Bradshaw Rd	Circular Culvert	1 Cell - 30"	30	36	66
29	LT1-C28	Sandage Ave 1975' W of Bradshaw Rd	Circular Culvert	1 Cell - 18"	9	14	25
30	LT1-C29	Denim Rd @ Excelsior Rd	Circular Culvert	1 Cell - 12"	4	3	5
31	LT1-C29A	Mecca Rd 175' S of Cul- de Sac	Circular Culvert	2 Cells - 18"	14	3	5
32	LT1-C30	Grant Line Rd. South of Calvine Rd.	Bridge			115	195
Laguna Creek Tributary # 2							
1	LT2-C01	Bradshaw Rd 550' N of Millpond Ct	Circuar Culvert	1 Cell - 30"	31	83	138
Laguna Creek Tributary # 3							
1	LT3-C01	Pleasant Grove School Rd 1200' E of Bader Rd	Arch Culvert	1 Cell - 22"x15"	7	74	123
2	LT3-C02	Ranch View Ct 380' S of Pleasant Grove School Rd	Arch Culvert	1 Cell - 22"x15"	7	9	15
3	LT3-C03	Bader Rd 700' N of Bond Rd	Arch Culvert	3 Cells 50"x 33"	101	170	286
4	LT3-C04	Pine Acre Ct 800' W of Bader Rd	Circular Culvert	1 Cell - 21"	13	9	15
5	LT3-C05	Bradshaw Rd 450' S of Millpond Ct	Arch Culvert	2 Cells - 26"x24"	37	194	328
6	LT3-C06	Millpond Ct 700' W of Bradshaw Rd	Arch Culvert	3 Cells 60"x36"	245	213	354
7	LT3-C07	Millpond Ct 100' S of Cul-de Sac	Arch Culvert	4 Cells 60"x36"	246	226	373
Laguna Creek Tributary # 4							
1	LT4-C01	Pleasant Grove School Rd 950' E of Mickey Rd	Arch Culvert	1 Cell 36"x24"	24	86	145
2	LT4-C02	Bond Rd 500' W of Grant Line Rd	Arch Culvert	2 Cells 36"x24"	52	73	118
3	LT4-C03	Grant Line Rd 1100' S of Bond Rd	Arch Culvert	1 Cell 48"x 33"	66	27	55
5	LT4-C05	Secretariat Ln	Circular Culvert	1 Cell 36"	*	*	100
6	LT4-C06	Kapalua Ln 290' W of Mango Ln	Arch Culvert	1 Cell 25.7'x8.7'	1170	225	391
7	LT4-C07	Bradshaw Rd 400' s of Bond Rd	Box Culvert	3 Cells 7'x5'	520	256	506
8	LT4-C08	Bond Rd. 1300' E of Bader Rd	Circular Culvert	1 Cell 18"	9	26	44
9	LT4-C09	Salmon Creek Rd 700' S of Bond Rd	Box Culvert	3 Cells 8'x5'	520	333	578
10	LT4-C10	Stonebrook Rd 1000' S of Bond Rd	Box Culvert	3 Cells 9'x5'	520	333	578
11	LT4-C11	Bond Road 1600' E of Waterman Rd	Box Culvert	4 Cells 7'x5'	520	461	847
Elk Grove Creek							
1	EGC-C01	Elk Grove Blvd 1900' West of Grant Line Rd	Arch Culvert	1 Cell 22"x12"	7	83	138
2	EGC-C02	Grant Line Rd 1800' S of Elk Grove Blvd	Circular Culvert	1 Cell 15"	6	54	89
3	EGC-C03	Bradshaw Rd 120' N of Ridgerock Dr	Circular Culvert	1 Cell 42"	**	183	301
4	EGC-C04	Grant Line Rd 2000' S of Bradshaw Rd	Arch Culvert	2 Cells 56"x38"	104	117	191
5	EGC-C05	Waterman Rd @ Kent Street	Arch Culvert	3 Cells 56"x36"	254	373	637

* Existing storm drain outlet
** Existing storm drain inlet

(1) From Harris Associates November 2005.
(2) A cell is a single culvert.

HEC-RAS evaluations were also performed along selected reaches of Laguna Creek Tributary No.'s 1 and 4 to aid in the evaluation of the capacity of these existing channels, their cross-drainage structures, and infrastructure improvement options, and to assist in the hydrologic modeling work. In general, there are several reaches of Laguna Creek Tributary No. 1 that have flood prone areas that extend well outside of the main channel area under existing conditions. The effective flood zones mapped by FEMA along Laguna Creek Tributary No. 1 have been overlaid onto Figure 6-2. Laguna Creek Tributary No. 4 contains a number of cross-drainage structures at streets that will be surcharged during a major storm in the absence of remedial measures.

HYDROLOGIC ANALYSIS FOR THE EAST ELK GROVE AREA/RURAL REGION

The East Elk Grove area/rural region was subdivided into primary watersheds and subsheds as shown on Figure 6-2. The primary watersheds have been defined to be:

- Laguna Creek Tributary No. 1
- Laguna Creek Tributary No. 2
- Laguna Creek Tributary No. 3
- Laguna Creek Tributary No. 4
- Elk Grove Creek

Upstream portions of several of the primary watersheds extend into areas within Sacramento County outside of the City corporate limits. The most significant of these upstream watershed areas are the subsheds that contribute runoff to Laguna Creek Tributary No. 1, north of Calvine Road and east of Grant Line Road (See Figure 6-2).

Hydrologic modeling was performed for the primary watersheds and subsheds to determine discharges produced at key points of concentration during the 10-year and 100-year storm events. The hydrologic modeling was prepared using SacCalc and HEC-1. Because future land use densities in the East Elk Grove area/rural region are relatively low, the difference in peak flows between existing and buildout conditions is not expected to be large. Because of this, flows were calculated for buildout conditions only. The key subshed data used to prepare the hydrologic models are presented on Table 6-2. Schematic layouts of the SacCalc Models for Laguna Creek Tributaries 1 through 4 are presented on Figures 6-3 through 6-6.

The Modified Pul's (Storage) method was used to route the computed subshed hydrographs through Laguna Creek Tributary No. 1. Storage-outflow rating curves were developed for several channel reaches from its confluence with Laguna Creek to Excelsior Road using the HEC-RAS computer model, based on Sacramento County provided data and applicable subdivision topographic and hydraulic data. Upstream of Excelsior Road and along other tributary routing reaches where HEC-RAS data was not available, the Muskingum-Cunge method was used, assuming a trapezoidal channel having a 5-foot bottom width, 10:1 side slopes and a Manning's roughness coefficient of 0.08. For all other watersheds, the Muskingum-Cunge method was used to perform the routing calculations for subsheds. Proposed detention basins that are a part of the upgrade infrastructure discussed in a later section of this report were evaluated using the Modified Puls method for reservoir routing.

Table 6-2. Hydrologic Parameters for East Elk Grove Area/Rural Region Subsheds

						Land Use (acres) and Percent Impervious								
						Comm./ Office	Ind.	MDR	Resd, 6 to 8 du/ac	Resd, 3 to 4 du/ac	Resd, .05 to 1 du/ac	Resd, .02 to .05 du/ac	Open Space, grassland/ag	
Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope,ft/ft	90%	85%	70%	50%	30%	15%	10%	2%	Average % Imp
Laguna Creek Tributary No. 1														
LT1A	311.7	90	5,954	3,100	0.0017							146.5	165.2	6
LT1B	388.4	85	5,410	2,900	0.0009								388.4	2
LT1E	105.9	80	3,738	1,900	0.0016							105.9		10
LT1F	82.7	80	2,677	1,450	0.0030							82.7		10
LT1G	57.5	74	2,733	1,410	0.0004							57.5		10
LT1C	313.3	85	6,847	3,420	0.0015							313.3		10
LT1I	159.4	80	6,000	3,000	0.0004						159.4			15
LT1H	125.1	72	4,453	2,230	0.0029							125.1		10
LT1J	82.4	70	2,513	1,300	0.0036							82.4		10
LT1L	135.4	68	4,800	2,500	0.0015				81.8		53.6			36
LT1K	83.8	66	2,523	1,300	0.0036							83.8		10
LT1N	195.1	74	4,096	2,150	0.0034							195.1		10
LT1P	55.1	70	1,642	950	0.0073							55.1		10
LT1Q	57.4	64	3,772	1,950	0.0042							57.4		10
LT1Q2	34.0	64	1,850	960	0.0043							34.0		10
LT1M	151.2	64	3,297	1,770	0.0012							151.2		10
LT1R	64.6	62	2,586	1,350	0.0016							64.6		10
LT1S	61.3	66	3,553	1,800	0.0013							61.3		10
LT1U	73.8	68	1,718	940	0.0035							73.8		10
LT1V	111.9	62	4,245	2,170	0.0024							111.9		10
LT1T	82.8	60	2,016	1,110	0.0040							82.8		10
LT1W	42.6	62	2,275	1,170	0.0044							42.6		10
LT1X	39.4	60	1,899	1,050	0.0053							39.4		10
LT1X2	59.2	54	2,962	1,500	0.0034							59.2		10
LT1Y	69.8	52	4,111	2,100	0.0034							69.8		10
LT1D	188.9	80	7,084	3,600	0.0082							188.9		10
LT1O	118.6	68	5,075	2,900	0.0032							118.6		10
TL1IB	42.9	72	2,600	1,300	0.0069							42.9		10
LT1IA	145.0	90	3,600	2,000	0.0056						145.0			15
LT1LA	51.3	70	3,600	1,800	0.0018						51.3			15
LT1LB	125.4	70	4,600	2,300	0.0006						125.4			15
Laguna Creek Tributary No. 2														
LT2A	102.7	64	2,956	1,550	0.0030					1.0		101.6		10
LT2B	75.3	58	5,293	2,810	0.0037				4.6	17.3		52.0		17
Laguna Creek Tributary No. 3														
LT3A	114.8	72	3,921	2,000	0.0010							114.8		10
LT3B	131.2	68	3,121	1,600	0.0014				6.6			124.6		12
LT3C	59.8	68	3,131	1,600	0.0019							59.8		10
LT3D	86.4	64	3,734	1,900	0.0001				1.7	1.7		82.9		11
LT3E	117.2	54	5,622	2,930	0.0029				43.4	73.8				37
Laguna Creek Tributary No. 4														
LT4A	102.5	76	2,581	1,350	0.0028	2.1						100.5		12
LT4B	139.3	70	4,032	2,050	0.0020				1.1			138.2		10
LT4D	22.6	72	2,218	1,150	0.0009							22.6		10
LT4E	19.8	68	1,826	910	0.0010							19.8		10
LT4F	17.2	68	1,122	710	0.0022						14.3	2.9		14
LT4C	44.6	66	2,203	1,300	0.0021						43.7	0.9		15
LT4G	6.3	68	606	350	0.0084						6.3			15
LT4I	46.9	64	2,428	1,360	0.0019						46.9			15
LT4H	24.6	66	1,927	1,160	0.0016						24.6			15
LT4K	15.6	60	1,206	610	0.0012						15.6			15
LT4J	86.5	62	2,601	1,310	0.0023						86.5			15
LT4L	18.6	60	1,175	590	0.0025						18.6			15
LT4N	13.0	64	859	430	0.0020						13.0			15
LT4M	45.2	62	3,082	1,550	0.0043						45.2			15
LT4O	33.7	62	1,509	755	0.0040						33.7			15
LT4W	29.2	60	2,296	1,150	0.0034						1.7	27.4		10
LT4P	20.0	60	2,061	1,035	0.0001						20.0			15
LT4Q	33.4	60	1,982	995	0.0040						33.4			15
LT4R	28.7	60	2,112	1,060	0.0045						28.7			15
LT4S	8.9	58	1,306	655	0.0008						8.9			15
LT4T	22.7	64	1,453	730	0.0014						22.7			15
LT4U	46.2	62	1,795	900	0.0017						46.2			15
LT4V	349.5	66	4,725	2,450	0.0017	10.5		45.4	213.2	38.4			41.9	46
LT4X	33.8	60	2,090	1,050	0.0013				33.8					50
LT4Y	26.1	58	1,719	860	0.0022				25.4	0.8				49
LT4Z	45.3	52	1,637	820	0.0073					45.3				30
LT4C2	49.3	68	1,750	900	0.0023							49.3		10
LT4J2	67.3	68	3,150	1,900	0.0025							14.1	53.2	4
Elk Grove Creek														
EGC1	111.3	64	3,359	1,720	0.0014						111.3			15
EGC2	31.9	60	1,182	650	0.0017						31.9			15
EGC3	37.2	62	2,612	1,310	0.0023						37.2			15
EGC4	119.3	62	3,675	1,900	0.0016								119.3	2
EGC5	47.6	58	2,777	1,420	0.0023						47.6			15
EGC9	295.3	54	5,250	2,700	0.0015					11.8			283.5	3
EGC6	96.6	54	4,188	2,150	0.0014				6.8	48.3		19.3	22.2	21
EGC7	64.6	52	2,636	1,360	0.0015				23.9	29.7		6.5	4.5	33
EGC8	102.5	52	3,781	1,920	0.0014		15.4		61.5				25.6	43
EGC10	94.2	52	4,423	2,250	0.0009				81.9				12.2	44
EGC11	109.2	66	3,150	2,200	0.0019								109.2	2

Calculated peak flows from each subshed are presented in Table 6-3. The calculated flows at key points along the creeks are presented on Table 6-4 and Figure 6-2. The listed peak flows are for buildout conditions assuming no modifications to the existing drainage system.

As discussed in the next section, detention basins are recommended for construction within the watersheds for Laguna Creek Tributary No.'s 1 and 4. Hydrologic models were prepared to evaluate the effects of these detention basins. Table 6-4 and Figure 6-7 present the peak flows at key locations with the detention basins included.

PRELIMINARY IMPROVEMENTS FOR THE EAST ELK GROVE AREA/RURAL REGION

This section presents the preliminary improvements that were considered for the East Elk Grove area/rural region at buildout. The City's General Plan contains a large number of goals, policies and actions that are intended to provide guidance and direction with respect to future development, the character of the City, and facilities. There are numerous references to the "Sheldon" area and the importance of preserving its unique ecological and rural characteristics. It is not practical to list all of the goals, policies, and actions that may be considered applicable to the development of the recommended improvements for the East Elk Grove area/rural region. However, the following paraphrased guidelines were strongly considered in the development of the improvements recommended for the East Elk Grove area/rural region:

- Naturally vegetated stream corridors are of value in assisting in the removal of pollutants and in the providing of habitat and community amenities. The retention of natural stream corridors is encouraged.
- The City will permit stream channel realignment only when necessary to eliminate flood hazards, to protect and preserve natural features and vegetation which would otherwise be removed, or if the existing channel has been significantly disrupted by agricultural improvements or other man-made changes.
- Development adjacent to a natural stream shall provide a stream buffer zone along the stream. The list of natural streams includes Laguna Creek and its tributaries and Elk Grove Creek.
- To the extent possible, retain natural drainage courses in all cases where preservation of natural drainage is physically feasible and consistent with the need to provide flood protection.
- Discourage the number of crossings of natural creeks in order to reduce potential flooding and access problems.
- Vehicular access to the buildable area of all parcels must be at or above the 10-year flood elevation.

Table 6-3. Calculated Subshed Flows for East Elk Grove Area/Rural Region

Subshed	Area, acres	Buildout Condition Flows, cfs	
		10-Year	100-Year
Laguna Creek Tributary No. 1			
LT1A	311.7	130	223
LT1B	388.4	133	226
LT1E	105.9	68	121
LT1F	82.7	63	114
LT1G	57.5	36	64
LT1C	313.3	157	274
LT1I	159.4	77	134
LT1H	125.1	80	142
LT1J	82.4	66	120
LT1L	135.4	93	150
LT1K	83.8	67	122
LT1N	195.1	129	231
LT1P	55.1	54	100
LT1Q	57.4	40	72
LT1Q2	34.0	31	57
LT1M	151.2	98	175
LT1R	64.6	47	85
LT1S	61.3	39	70
LT1U	73.8	67	124
LT1V	111.9	71	126
LT1T	82.8	72	132
LT1W	42.6	36	66
LT1X	39.4	36	66
LT1X2	59.2	45	81
LT1Y	69.8	46	83
LT1D	188.9	110	196
LT1O	118.6	70	125
TL1IB	42.9	36	66
LT1IA	145.0	109	197
LT1LA	51.3	35	64
LT1LB	125.4	71	125
Laguna Creek Tributary No. 2			
LT2A	102.7	83	138
LT2B	75.3	52	82
Laguna Creek Tributary No. 3			
LT3A	114.8	74	123
LT3B	131.2	97	161
LT3C	59.8	45	75
LT3D	86.4	46	76
LT3E	117.2	90	126
Laguna Creek Tributary No. 4			
LT4A	102.5	86	145
LT4B	139.3	95	158
LT4D	22.6	18	30
LT4E	19.8	17	29
LT4F	17.2	19	33

Table 6-3. Calculated Subshed Flows for East Elk Grove Area/Rural Region, Cont'd...

Subshed	Area, acres	Buildout Condition Flows, cfs	
		10-Year	100-Year
LT4C	44.6	39	66
LT4G	6.3	10	17
LT4I	46.9	40	67
LT4H	24.6	22	38
LT4K	15.6	17	29
LT4J	86.5	75	126
LT4L	18.6	22	37
LT4N	13.0	17	28
LT4M	45.2	39	65
LT4O	33.7	38	64
LT4W	29.2	26	44
LT4P	20.0	14	24
LT4Q	33.4	34	57
LT4R	28.7	28	48
LT4S	8.9	9	15
LT4T	22.7	23	39
LT4U	46.2	45	75
LT4V	349.5	258	376
LT4X	33.8	37	54
LT4Y	26.1	32	47
LT4Z	45.3	58	99
LT4C2	49.3	47	80
LT4J2	67.3	35	59
Elk Grove Creek			
EGC1	111.3	83	138
EGC2	31.9	35	60
EGC3	37.2	32	54
EGC4	119.3	54	89
EGC5	47.6	40	67
EGC9	295.3	117	191
EGC6	96.6	61	93
EGC7	64.6	58	85
EGC8	102.5	74	111
EGC10	94.2	67	97
EGC11	109.2	51	84

Table 6-4. Calculated Creek Flows for East Elk Grove Area/Rural Region

Location	Buildout Condition Flows Without Improvements, cfs		Buildout Condition Flows With Improvements, cfs	
	10-Year	100-Year	10-Year	100-Year
Laguna Creek Tributary No. 1				
Excelsior Road	265	449	78	155
CCTR	752	1,280	649	1,135
Bader Road	841	1,444	674	1,169
Bradshaw Road	804	1,470	637	1,167
Confluence with Laguna Creek	797	1,449	630	1,149
Laguna Creek Tributary No. 2				
Bradshaw Road	83	138	83	138
Confluence with Laguna Creek	113	193	113	193
Laguna Creek Tributary No. 3				
Bader Road	170	286	170	286
Bradshaw Road	194	328	194	328
Confluence with Laguna Creek	205	351	205	351
Laguna Creek Tributary No. 4				
Bond Road (East Crossing)	149	255	73	118
Bradshaw Road	333	578	288	506
Bond Road (West Crossing)	478	886	461	847
Elk Grove Creek				
Grant Line Road	54	89	54	89
Waterman Road	373	637	309	512

Descriptions of Preliminary Improvements

The preliminary storm drainage improvements that are considered for the East Elk Grove area/rural region are shown on Figures 6-7 and 6-8. A list of the preliminary facilities is provided in Table 6-5. These improvements include:

- Upgrading of many of the existing drainage structures that cross the primary and secondary stream channels at roadways to improve capacity in conformance with the design standards.
- Installation of underground storm drain systems along Bond Road and Sheldon Road to intercept and convey contributing flows derived from local offsite subsheds and accommodate future widening of these major arterial streets.
- Construction of four detention basins to lower downstream discharges along Laguna Creek Tributary No.'s 1 and 4 to more manageable rates and reduce downstream flood potential.
- Emphasis on retention of the natural and existing channels, except where augmentation or reshaping of local channels is needed to improve capacity or provide a local outfall for arterial street drainage.
- Within the East Elk Grove Specific Plan Area in the Elk Grove Creek watershed, the recommended improvements include construction of a new detention basin, a new pump station, and realignment of a short reach of Elk Grove Creek.

Cross-Drainage Structures

Many of the existing drainage structures that cross primary and secondary stream channels at streets within the study area have a very limited capacity and need to be upgraded. An evaluation of each of the cross-drainage structures in the study area was performed. This evaluation produced a list of cross-drainage structures that need to be upgraded and a determination of the proposed replacement structure. Replacement structures were sized to pass the 100-year discharge without flow overtopping the roadway and are listed on Table 6-5. Hydraulic evaluations of each new structure were performed using the HEC-RAS computer model

Storm Drains

It is not the intent of this master plan to produce an integrated network of underground storm drains to serve the East Elk Grove area/rural region. Instead, this master plan emphasizes the use of existing streams and channels as the primary flow conveyance mechanisms in this area. Most arterial streets in the East Elk Grove area/rural region traverse somewhat perpendicularly across the direction of drainage flow, and the use of cross-drainage structures and underground storm drains to convey street drainage, only, are the preferred solution for these streets.

Table 6-5. Preliminary Improvements for the East Elk Grove Area/Rural Region$^{(1)}$

Item	Linear Feet	Cubic Yards/ Linear Foot	Quantity	Unit
Culverts				
LT1-C01 (2 - 6' x 4' CBC)	100	0.800	80	CY
LT1-C07 (1 - 36" RCP)			100	LF
LT1-C07B (add 1 - 7' x 4' cell to existing CBC)	100	0.467	47	CY
LT1-C20 (1 - 24" RCP, Materials & Bore)			30	LF
LT1-C23 (80' Long Bridge)			4,800	SF
LT1-C25 (1 - 6' x 3' CBC)	60	0.433	26	CY
LT1-C26 (2 - 5' x 3' CBC)	100	0.659	66	CY
LT1-C27 (Bridge TBD - 80' Length Assumed)			8,000	SF
LT2-C01 (2 - 5' x 3' CBC)	100	0.659	66	CY
LT3-C01 (2 - 6' x 5' CBC)	60	0.874	52	CY
LT3-C03 (2 - 10' x 3.5' CBC)	60	1.470	88	CY
LT3-C05 (3 - 10' x 3' CBC)	100	2.164	216	CY
LT4-C01 (3 - 6' x 2.5' CBC)	60	1.019	61	CY
LT4-C02 (2 - 6' x 2.5' CBC)	100	0.717	72	CY
EGC-C01 (3 - 6' x 2.5' CBC)	100	1.019	102	CY
EGC-C02 (1 - 7' x 2.5' CBC)	100	0.489	49	CY
EGC-C03 (2 - 7' x 4' CBC), Bradshaw & D/Ws	120	0.933	112	CY
EGC-C04 (2 - 4' x 4' CBC)	100	0.630	63	CY
Storm Drains				
18" RCP (Bond Road)			3,100	LF
24" RCP (Bond Road)			1,800	LF
30" RCP (Bond Road)			700	LF
36" RCP (Bond Road)			1,800	LF
42" RCP (Bond Road)			1,000	LF
48" RCP (Bond Road)			2,700	LF
18" RCP (Sheldon Road)			1,300	LF
24" RCP (Sheldon Road)			2,000	LF
30" RCP (Sheldon Road)			2,100	LF
36" RCP (Sheldon Road)			3,000	LF
42" RCP (Sheldon Road)			1,800	LF
48" RCP (Sheldon Road)			1,300	LF
Detention Basins				
DET 1 (Laguna Creek Tributary # 1)			110	AF
DET 2 (Laguna Creek Tributary # 1)			20	AF
DET 3 (Laguna Creek Tributary # 1)			21	AF
DET 3 Pump Station (Capacity = 3 cfs)			1	LS
DET 4 (Laguna Creek Tributary # 4)			17	AF
Open Channels				
#1 - Bond Road to Trib. # 4 (Near Grant Line)	800	2.67	2,136	CY
#2 - Bond Road to Trib. # 4 (Near Bradshaw)	250	2.67	668	CY
#3 - Trib 3 Grading (Bader to Past Bradshaw)	3520	1.48	5,210	CY
#4 - EG Creek Overflow Channel	2300	4.74	10,902	CY
East Elk Grove SPA - Elk Grove Creek$^{(2)}$				
10'x6' Box Culvert			1	LS
Channel Realignment	700	21.43	15,000	CY
Pappas Detention Basin			56	AF
Hudson Detention Expansion			12	AF

Source of items and quantities:

(1) City of Elk Grove East Area Storm Drainage Master Plan, Harris & Associates, November 18, 2005.

(2) Preliminary Costs for Reconstruction of Elk Grove Creek, Alternative 4C - Off Stream Detention with Concrete Culverts, Mackay & Somps Civil Engineers, Inc., February 2006.

There are two major arterial streets, Bond Road and Sheldon Road, which extend westerly in the direction of drainage flow within their respective watersheds. These arterial streets are proposed for future widening and traffic capacity improvements. To provide a storm drainage system that will prevent these streets from being flooded during a 100-year storm event, the interception and conveyance systems need to not only consider street drainage, but also consider flows that encroach upon the street from several local offsite subsheds. As described in Volume I of this SDMP, the City intends to test a variety of approaches for controlling runoff along these roadways including dry wells, bottomless arch pipes, and detention basins. Should those approaches not provide the intended performance, it may be necessary to construct storm drains along these streets to intercept and convey the 10-year runoff. These potential storm drains are represented at a planning level on Figure 6-7. The Bond Road storm drainage facilities are proposed to utilize existing channelized sections of Laguna Creek Tributary No. 4 as points of outfall. The Sheldon Road storm drainage facilities are proposed to utilize Laguna Creek Tributary No. 1 at Bradshaw Road as their point of outfall.

The only other storm drain recommended for construction in the East Elk Grove area/rural region consists of a pipe that is proposed to provide drainage relief to a low lying segment of Sleepy Hollow Lane on the east side of the Central California Traction Railroad, north of Sheldon Road. This segment of roadway is subjected to frequent ponding and flooding during the rainy season due to the absence of a positive outlet. It is proposed that a storm drain inlet be installed at the flooded area, and a pipe be extended to the north along Sleepy Hollow Lane to discharge into future Detention Basin 2, which is proposed within a tributary to Laguna Creek Tributary No. 1.

Detention Basins

Construction of four detention basins is recommended as a part of the overall approach to future development in the East Elk Grove area/rural region. These detention basins are proposed to serve the primary watersheds for Laguna Creek Tributary No.'s 1 and 4 and are represented on Figure 6-7. They have also been incorporated into the SacCalc/HEC-1 hydrologic models that reflect upgraded conditions. The following are descriptions of each proposed detention basin and its intended benefits:

Detention Basin 1 (DET 1) – DET 1 is a proposed detention basin located east of Excelsior Road that is intended to capture, store, and attenuate runoff generated within the upstream headwaters for Laguna Creek Tributary No. 1 and provide a reduction in the 100-year peak flow rates experienced along downstream reaches. It is proposed to be roughly 8.5 feet deep, have a storage volume of 110 acre-feet and reduce the 100-year discharge from 444 cfs to 105 cfs at the detention basin site, as well as reduce downstream discharges. This detention basin will reduce the 100-year floodplain area for downstream channel reaches and reduce the capacity requirements for existing and future downstream culverts and bridges.

Detention Basin 2 (DET 2) – DET 2 is proposed to capture and attenuate flow generated within the upstream portions of a minor tributary to Laguna Creek Tributary No. 1, on the east side of the Central California Traction Railroad. The existing minor tributary downstream of proposed DET 2 consists of a generally low capacity, meandering channel or ditch that eventually joins Laguna Creek Tributary No. 1 upstream of Bader Road. Major channelization work would be required to provide capacity along the downstream reach of this minor tributary and this remedy was considered to be undesirable and inconsistent with the General Plan. DET 2 is proposed to have a depth of approximately four feet, a storage volume of 20 acre-feet and reduce the 100-year outflow from 160 cfs to 24 cfs at the detention basin site. This attenuation will lower the flood hazard along the downstream segment of this minor tributary. Also, the presence of DET 2 will serve to provide an outfall for the minor storm drain improvement previously described along Sleepy Hollow Lane and capacity improvements made to the upstream reach of this minor tributary, if any, in conjunction with future land development activities in the area.

Detention Basin 3 (DET 3) – DET 3 is proposed to be located on the north side of Sheldon Road, east of Bader Road. It is intended to function in conjunction with storm drain system installations that have been recommended to accompany the future widening of Sheldon Road. The accumulated rates of runoff that are proposed to be intercepted by Sheldon Road storm drains become increasingly higher as the system extends westerly, and the proposed storm drains reach a size of 2 – 48-inch pipes at the detention basin site. DET 3 is needed to reduce the size of storm drains required to serve Sheldon Road from the detention basin site downstream to the outfall location at Laguna Creek Tributary No. 1 (Bradshaw Road). It will also facilitate the construction of the larger proposed storm drains that enter it from the east by providing available depth, gradient and cover. DET 3 is proposed to have a capacity of 21 acre-feet and would reduce the incoming 100-year discharge from 155 cfs to 3 cfs. This detention basin will be roughly 10 feet deep and will be drained by a pump station.

Detention Basin 4 (DET 4) – DET 4 is proposed to be located on the north side of Bond Road near its intersection with Grant Line Road and is proposed to store and attenuate runoff generated within the most upstream subsheds within the primary watershed for Laguna Creek Tributary No. 4. The detention basin will reduce the flows in the downstream channelized sections of Laguna Creek Tributary No. 4 as it passes through existing subdivisions, which is particularly important at the drainage structure crossings (which would currently be surcharged during a 100-year event at watershed buildout). Further, the reduction in downstream flow rates will lower channel water surface elevations in a manner that will lower the tailwater elevation and improve hydraulic capacity for the future Bond Road drainage system points of outfall into the channel between Bradshaw Road and Grant Line Road. DET 4 is proposed to be approximately 4 feet deep, have a storage volume of 17 acre-feet and would reduce the incoming 100-year discharge from 255 cfs to 118 cfs.

Open Channels

In keeping with the goals, policies and actions contained in the General Plan, the natural and existing stream channels within the East Elk Grove area/rural region are proposed to remain substantially intact. The proposed augmentations to existing channels and construction of new channels are described below and are shown on Figure 6-7 and 6-8.

- Enlargement of an existing channel that extends from Bond Road south to a channelized segment of Laguna Creek Tributary No. 4 about 1,300 feet east of Bradshaw Road. This enlarged channel is proposed to be a point of outfall for segments of the proposed storm drain serving the widening of Bond Road.
- Enlargement of an existing channel that extends from Bond Road south to a channelized segment of Laguna Creek Tributary No. 4 about 2,000 feet west of Grant Line Road. This enlarged channel is proposed to be a point of outfall for segments of the proposed storm drain serving the widening of Bond Road.

- Clearing and reshaping of the existing manmade segment of Laguna Creek Tributary No. 3 between Bader Road and a location just downstream of Bradshaw to eliminate irregularities and create a continuous positive grade. There is also a 90 degree bend in the existing channel that contains a low area on the south bank, roughly 1,300 feet east of Bradshaw Road that allows higher stage flows to begin to spill south from the Laguna Creek Tributary No. 3 primary watershed toward Bond Road and into the Laguna Creek Tributary No. 4 primary watershed. It is proposed that this low segment of the south bank be filled to eliminate this watershed spillover.
- Construction of a new channel along the east side of Bradshaw Road at Elk Grove Creek. This channel is needed to cost-effectively provide a 100-year return period capacity to Bradshaw Road at this location. There is an existing drainage system serving Elk Grove Creek that is inadequate at this location. The new channel will be an “overflow” channel that will convey excess flow south along the east side of the roadway and then west to rejoin Elk Grove Creek about 1,200 feet west of Bradshaw Road. This improvement will require the installation of culvert crossings of the channel extension along the east side of Bradshaw Road to facilitate continued driveway access to private properties.

East Elk Grove Specific Plan Improvements

MacKay & Somps Civil Engineers, Inc. has prepared a preliminary drainage study that identifies the required drainage improvements in the Elk Grove Creek watershed within the East Elk Grove Specific Plan Area (See Reference No. 1). The required improvements are shown on Figure 6-8 and include:

- Pappas Detention Basin – A new detention basin, referred to as the Pappas Basin, is proposed within the East Elk Grove Specific Plan area. The basin will be located on the north side of Elk Grove Creek approximately 3,000 feet upstream from Waterman Road. The basin is intended to reduce the buildout condition flows to eliminate the need to increase the capacity of the creek. The peak storage volume in the basin for a 100-year event is 56 acre-feet. A pump station with a capacity of 20 cfs will be provided to evacuate runoff from the pond.
- Hudson Detention Basin – The Hudson Detention Basin is an existing detention basin location south of Elk Grove Creek, just east of Waterman Road. The bottom of the existing basin is proposed to be lowered to increase the storage capacity of the basin by about 12 acre-feet.
- Elk Grove Creek is proposed to be realigned from Waterman Road to a point about 700 feet upstream. The realignment is necessary to improve the operation of an existing detention basin location just upstream of Waterman Road (Hudson Basin).
- A new set of culverts will be required where the realigned channel crosses under Waterman Road. Two 10 feet by 6 feet box culverts are proposed.

Other East Elk Grove Area/Rural Region Studies

Hydraulic Analysis of Channel Along Sheldon Road

In 2010, David Ford Consulting performed a hydrologic and hydraulic analysis for one of the tributary channels that drains to Laguna Creek Tributary No. 1. The channel begins approximately 200 feet upstream of Bader Road and continues north and west for approximately 1,700 feet to Tributary 1, crossing Sheldon Road on the way. The limits of the creek system can be seen in the report prepared by Ford, which is included at the end of this Chapter as Attachment 6A.

This watershed was evaluated by Harris and Associates as a part of their East Area Storm Drainage Master Plan, but not to the same level of detail as Ford. Harris calculated flood flows and evaluated the hydraulic capacity of the culverts at Bader Road and Sheldon Road, but they did not evaluate the hydraulic capacity of the channel itself or consider the potential effects of backwater from Tributary No. 1. Because of this, there was some concern that the culvert improvements recommended by Harris may not be effective if the channel capacity was insufficient to convey the flow or backwater impeded the flow. To address the issue, the City asked Ford to prepare a refined hydraulic analysis of the system.

The Ford study concluded, among other things, that 10-year flood protection for Bader Road and Sheldon Road can be achieved by upsizing the existing culverts to double 2'x4' boxes. For the 100-year storm, backwater from Tributary No. 1 can cause flooding Sheldon Road to be overtopped regardless of the culvert capacity. The Harris study recommended that the Bader Road and Sheldon Road culverts be upsized to a 3'x6' box, and double 3'x5' boxes, respectively. The Harris recommendations would provide a slightly higher capacity and could provide 100-year protection to the roads in the absence of backwater from Tributary No. 1. Therefore, the culvert improvements from the Harris study are included as the recommended improvements in this master plan.

Laguna Creek Tributary 1 Hydraulic Analysis

In 2011, David Ford Consulting performed a hydraulic analysis for Laguna Creek Tributary 1. For this analysis, an updated hydraulic model was prepared for the creek using HEC-RAS. The geometry of the creek was defined using LIDAR topographic data prepared for the California Department of Water Resources Central Valley floodplain mapping effort. Steady-state hydraulic models were prepared using the flood flows calculated by Harris & Associates for the City of Elk Grove East Area Storm Drainage Master Plan and also the flows developed for the FEMA Flood Insurance Study. The flood flows developed by Harris & Associates are significantly larger than those developed by FEMA. As a result, the 100-year water surface elevations calculated using the Harris & Associates flows are significantly higher than those predicted with the FEMA flows. An unsteady-state hydraulic model was also prepared using the Harris & Associates flows to assess the effects of hydraulic routing on the flood flows. The water surface elevations calculated with the unsteady-state model are slightly lower than those predicted with the steady-state model. Floodplain maps are provided in the Ford study, which is included as Attachment 6B.

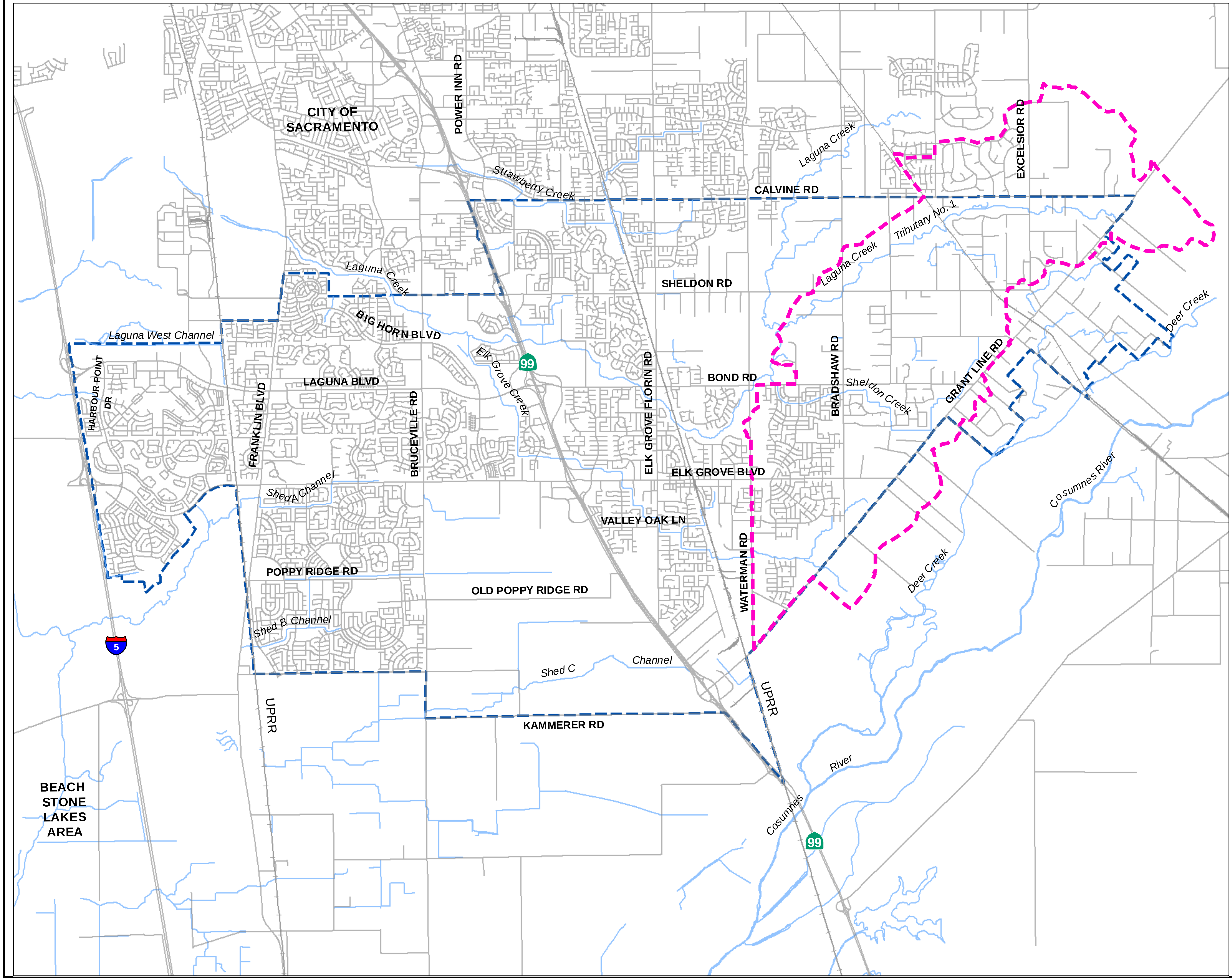


FIGURE 6-1

City of Elk Grove Storm Drainage Master Plan Volume II

EAST AREA LOCATION MAP

N

0 0.5 1 2

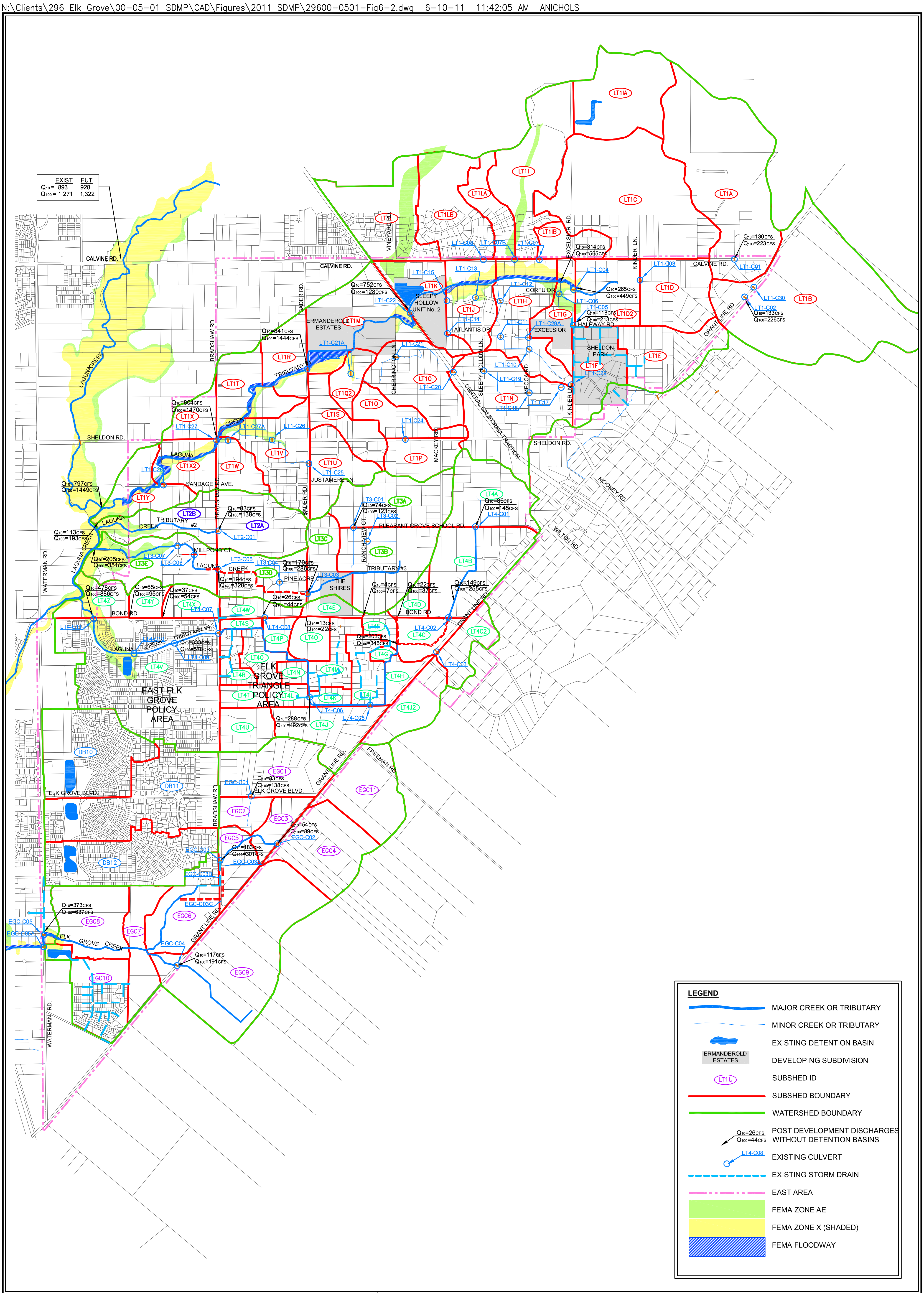
Scale in Miles

NOTES:

LEGEND:

⬚ City Limit

⬚ East Area Watershed



SOURCE OF GRAPHIC:
HARRIS & ASSOCIATES

0 1300 2600
SCALE IN FEET

Figure 6-2
City of Elk Grove
Storm Drainage
Master Plan Volume II
EAST AREA SUBSHEDS & DRAINAGE FEATURES

Figure 6-3. Laguna Creek Tributary No. 1 SacCalc Layout

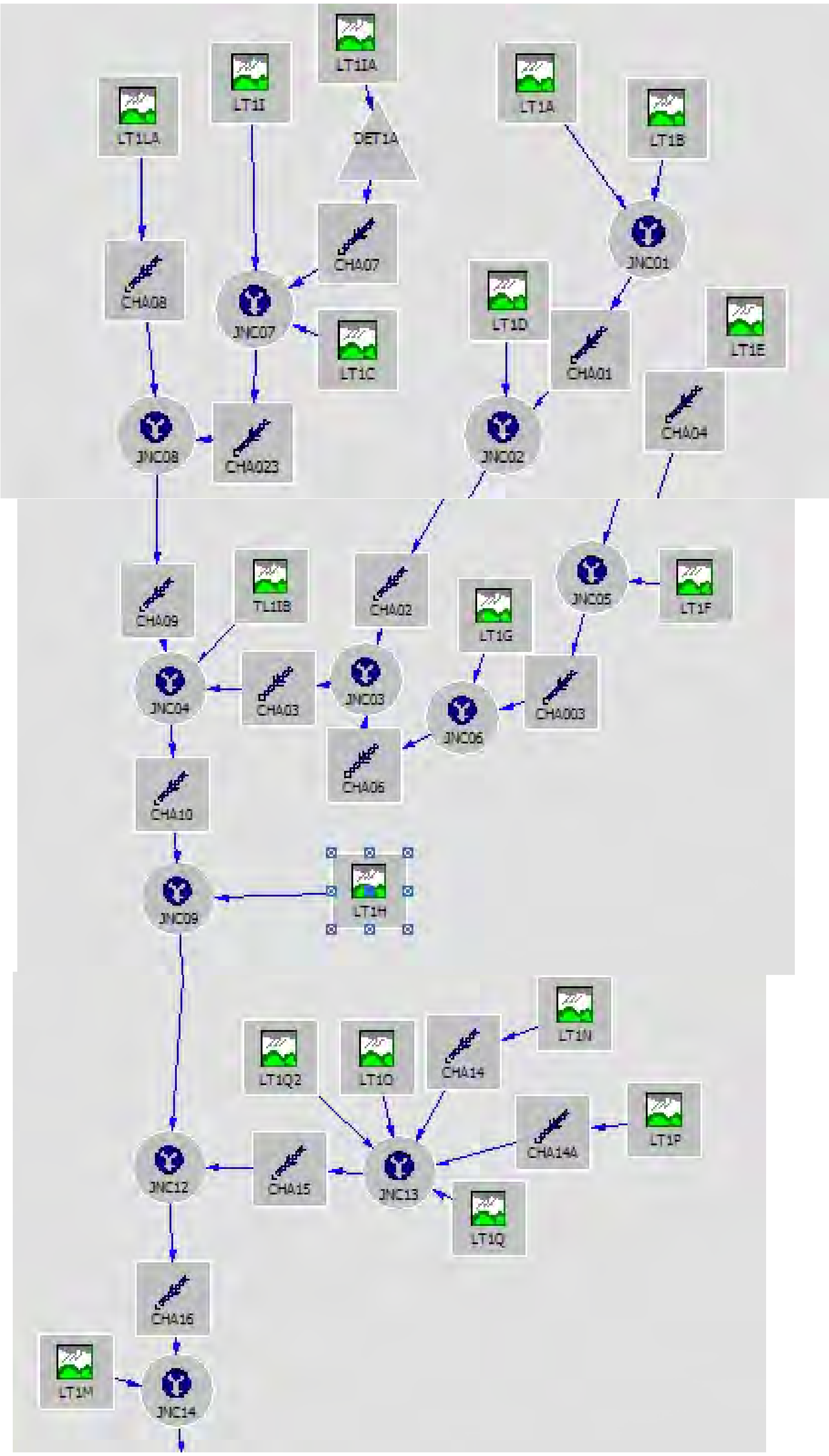


Figure 6-3. Laguna Creek Tributary No. 1 SacCalc Layout, cont.

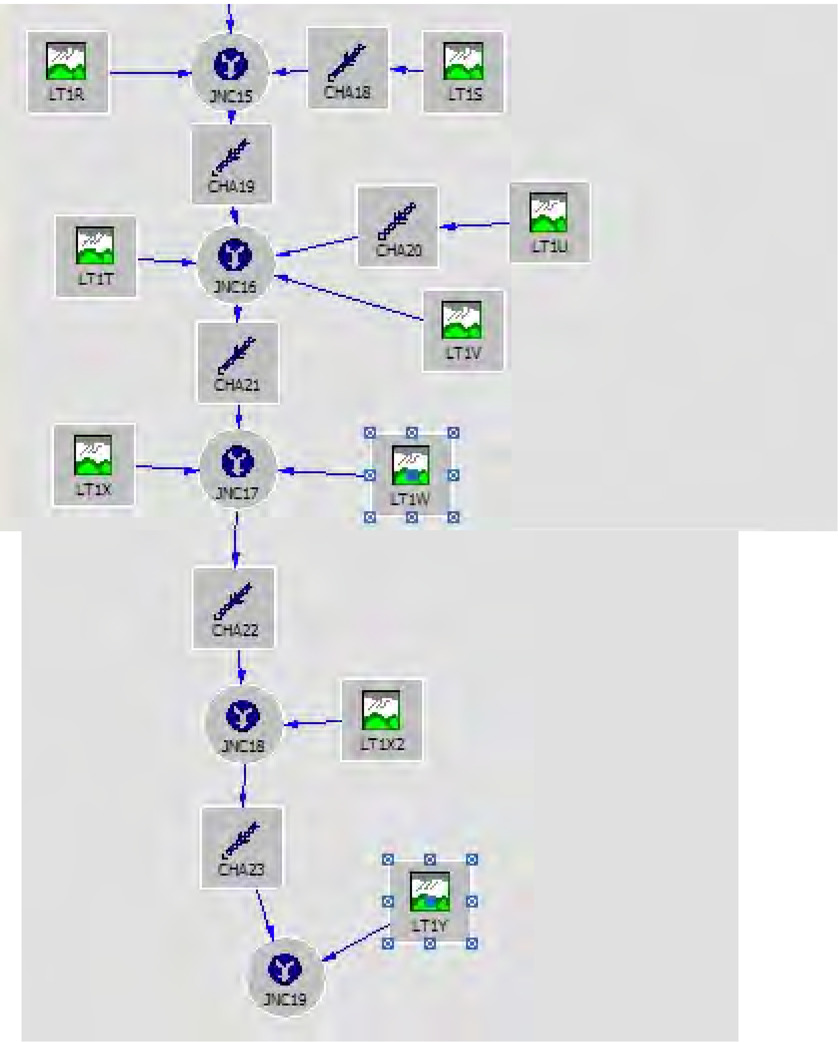


Figure 6-4. Laguna Creek Tributary No. 2 SacCalc Layout

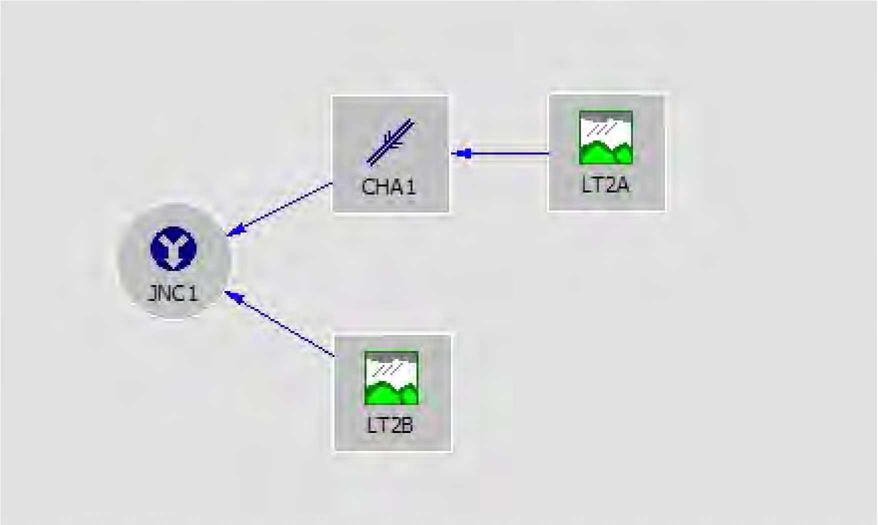


Figure 6-5. Laguna Creek Tributary No. 3 SacCalc Layout

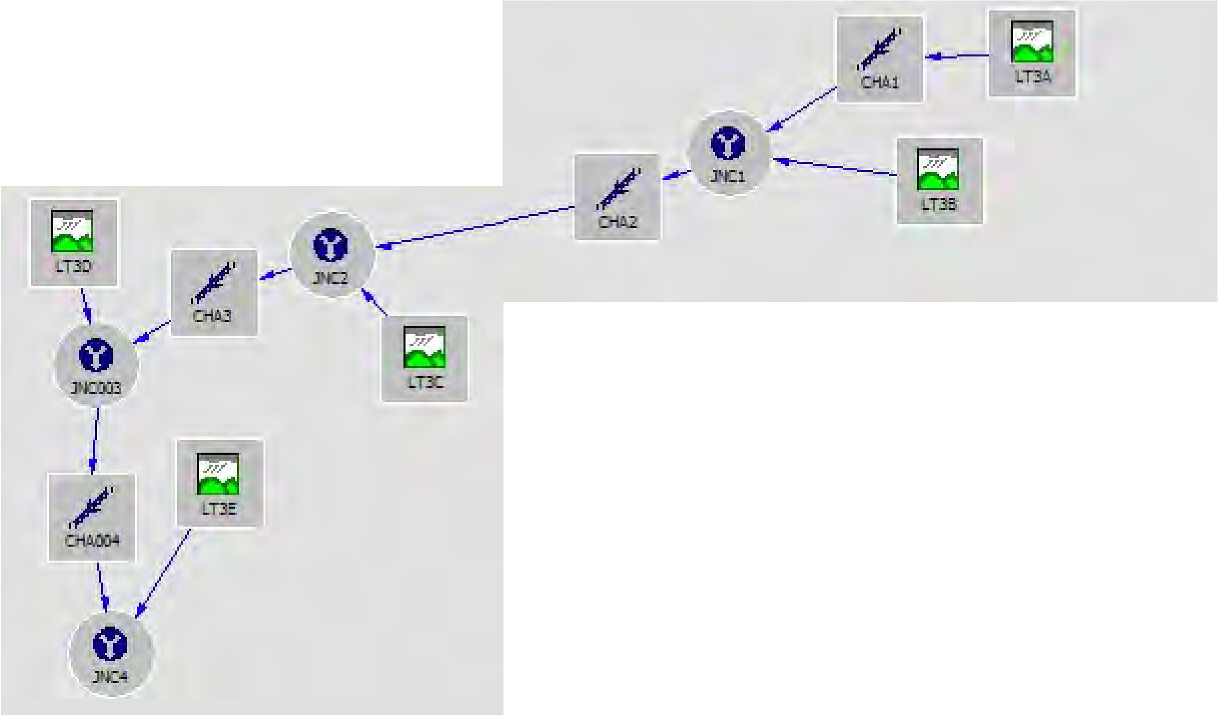


Figure 6-6. Laguna Creek Tributary No. 4 SacCalc Layout

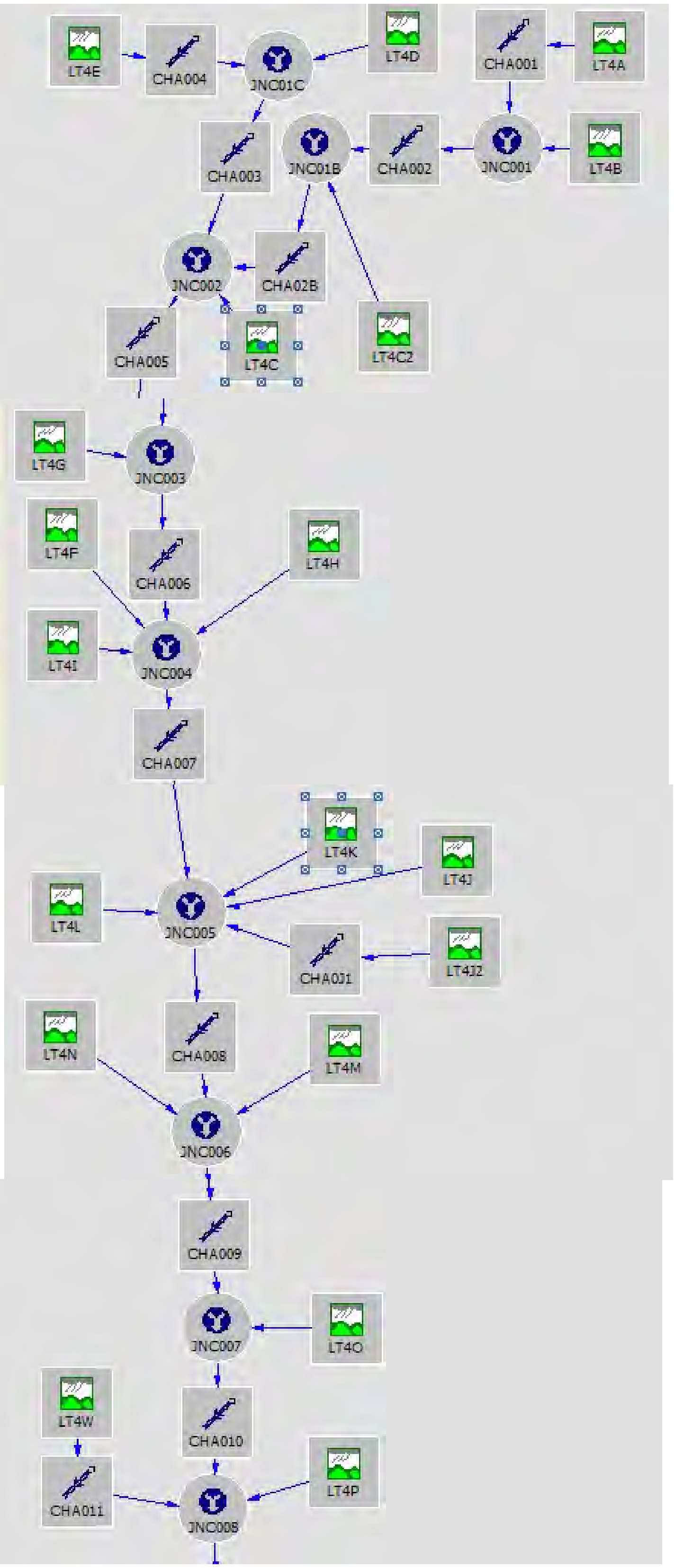
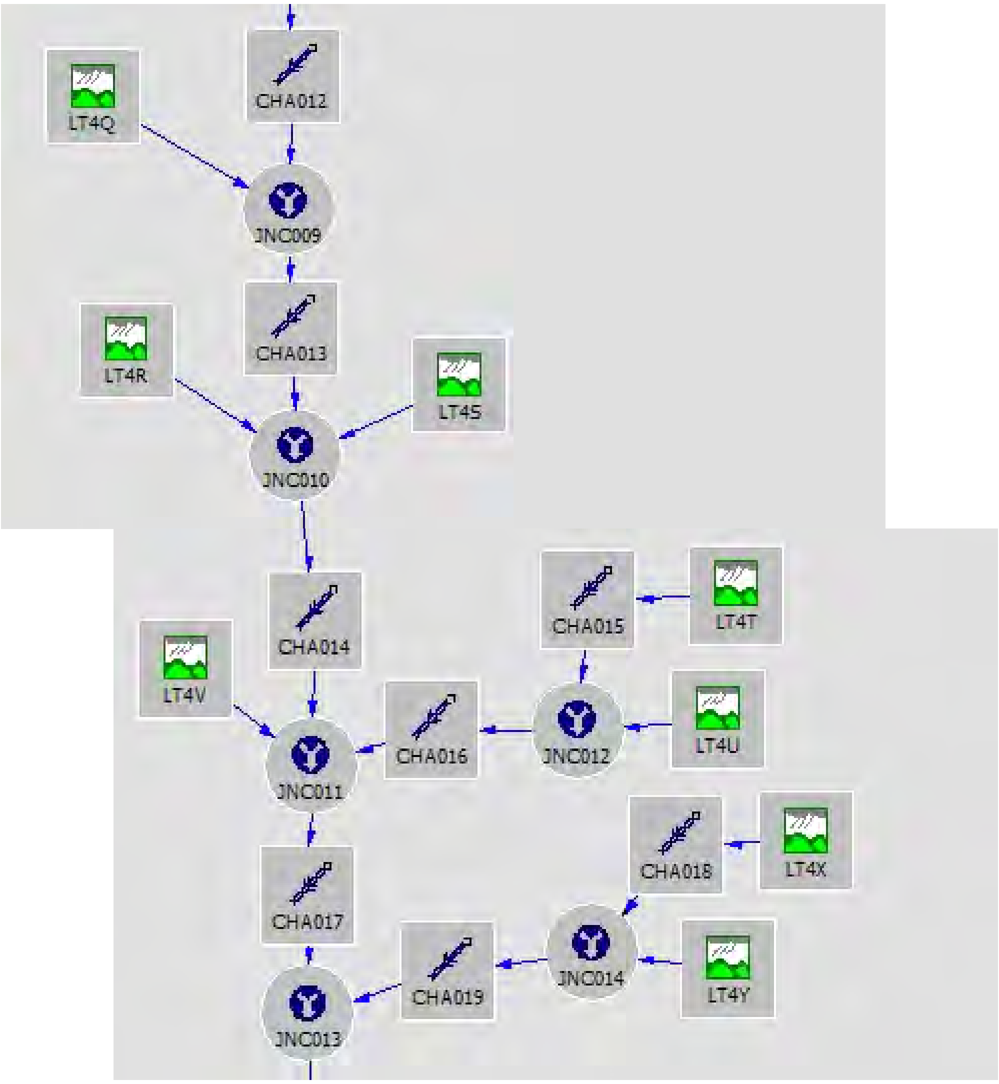


Figure 6-6. Laguna Creek Tributary No. 4 SacCalc Layout, cont.



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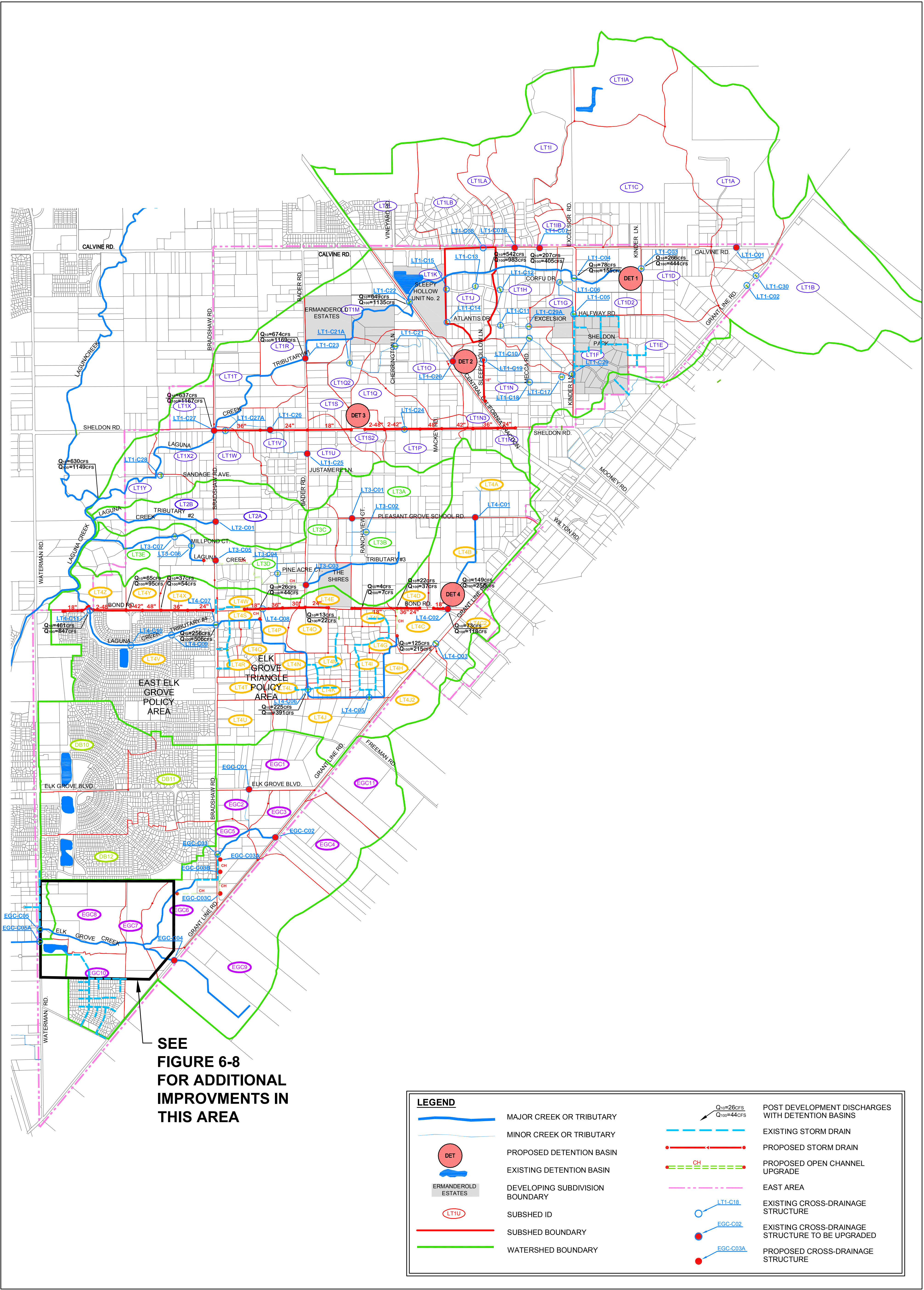

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SCALE IN FEET

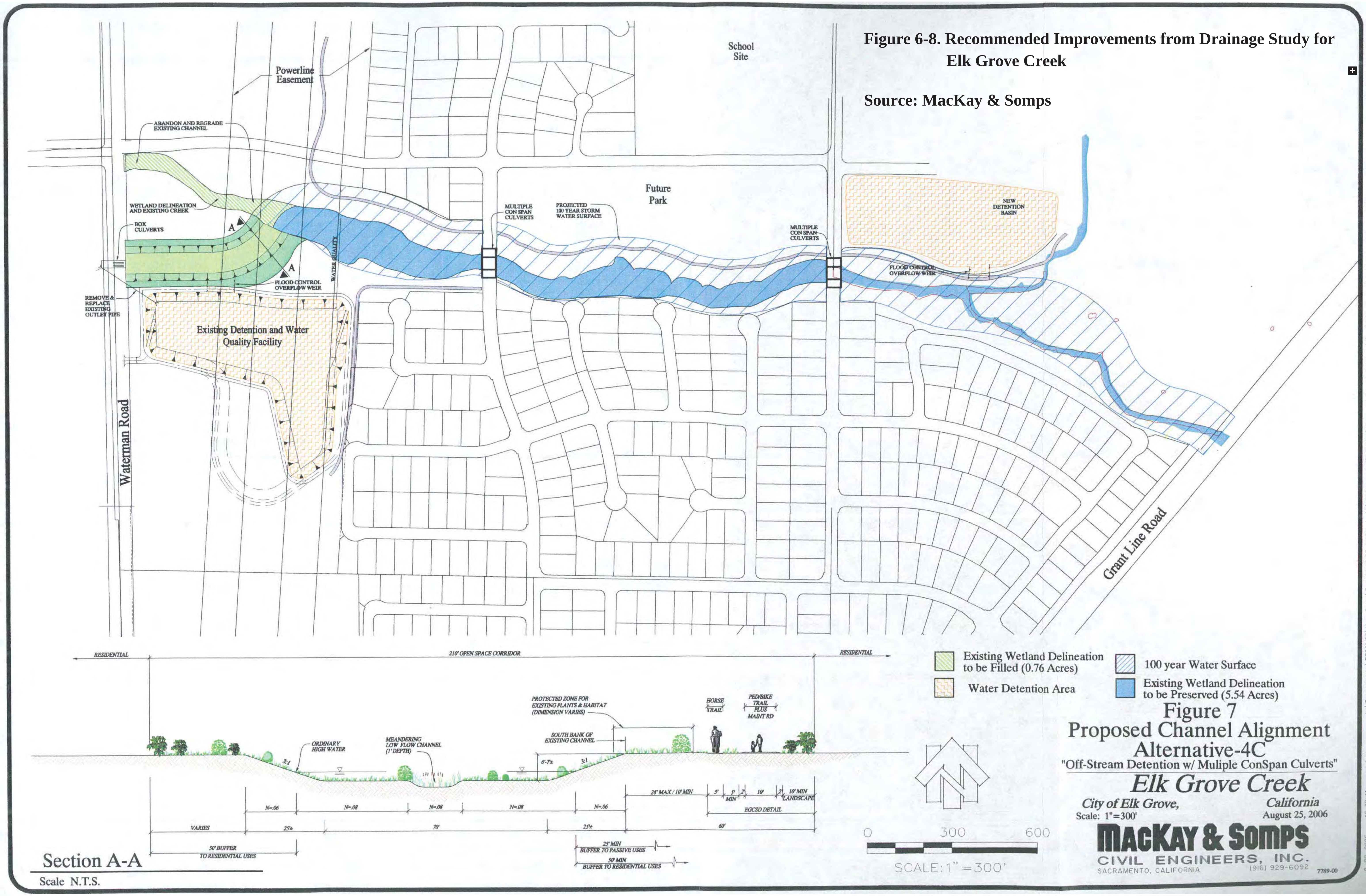
SOURCE OF GRAPHIC:
HARRIS & ASSOCIATES

Figure 6-7
City of Elk Grove
Storm Drainage
Master Plan Volume II
EAST AREA STUDY
RECOMMENDED IMPROVEMENTS

WEST YOST ASSOCIATES
Consulting Engineers

Figure 6-8. Recommended Improvements from Drainage Study for Elk Grove Creek

Source: MacKay & Somps



ATTACHMENT 6A

Analysis of Channel Along Sheldon Road

12/13/2010

Mark.
For yours To keep
Clarence K.

DAVID FORD
CONSULTING
ENGINEERS

David Ford Consulting Engineers, Inc.
2015 J Street, Suite 200
Sacramento, CA 95811
Ph. 916.447.8779
Fx. 916.447.8780

BRIAN A. BROWN
No. 69338
Exp. 6/30/12

MEMORANDUM

To: Clarence E. Korhonen, PE

From: Brian A. Brown, PE and Tom Molls, PhD, PE

Date: July 14, 2010

Subject: Hydrologic and hydraulic analysis for channel along Sheldon Road

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Situation

The City of Elk Grove (City) is refining the drainage analysis along Sheldon Road and lands adjacent to Sheldon Road that contribute runoff to the existing open channel system. Specifically, the channel begins approximately 200 feet upstream of Bader Road and terminates at its confluence with Tributary 1 to Laguna Creek as shown in red on Figure 1.

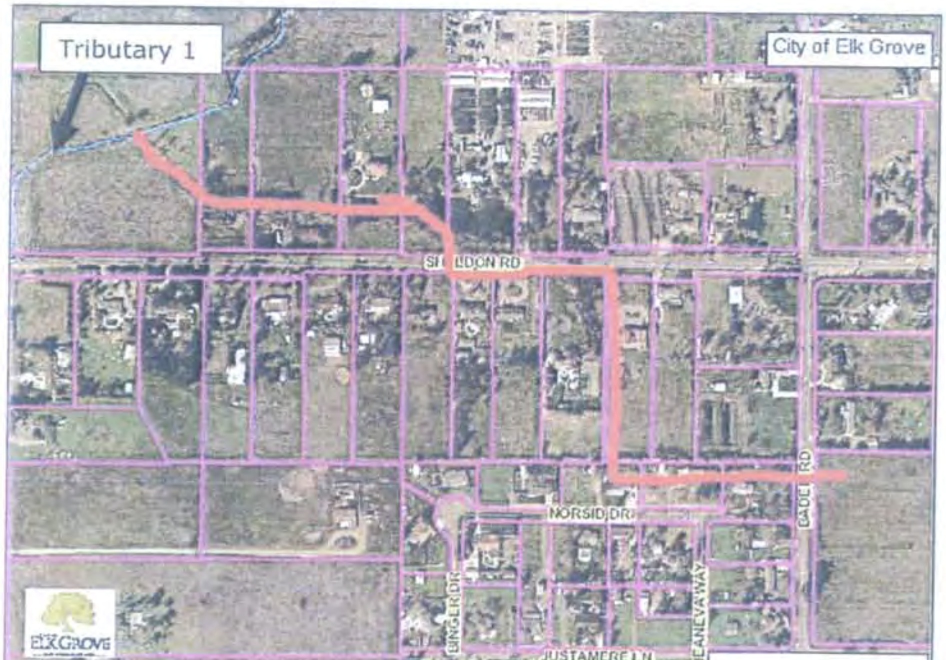


Figure 1. Channel along Sheldon Road to be studied (provided by City staff)

The current study area was included in the City of Elk Grove Storm Drainage Master Plan for East Elk Grove Area by Harris and Associates in 2005 (Harris study). According to the Harris study, the existing pipe crossings at Bader Road and Sheldon Road are undersized. The upsizing of these pipes may not completely alleviate the potential for flooding as the backwater from Tributary 1 could contribute to flooding in this area. In 2006, the City of Elk Grove completed the Draft Flood Control and Storm Drainage Master Plan (DMP). The DMP developed hydrologic and hydraulic models for the City of Elk Grove using the Harris study as the upstream boundary conditions. The City believes a detailed study is needed to understand better the behavior of the runoff from this area.

Tasks

The purpose of this analysis is to evaluate hydraulically the drainage channel based on new topographic data. Specifically, it is to determine the water surface elevation (WSEL) in the channel for the p=0.10 and p=0.01 events and to determine the effect of the WSEL in Tributary 1 on the WSEL in the drainage channel.

Actions

To analyze the drainage channel and determine the effect of WSELs in Tributary 1 on the WSEL in the drainage channel, we completed the following:

1. Reviewed available information from existing drainage studies and topographic information to assess the runoff timing between the channel and Tributary 1.
2. Created a hydrologic model to compute the runoff for the area contributing to the drainage channel based on LiDAR provided by the City.
3. Created a steady flow hydraulic model of the drainage channel to compute the WSEL for the p=0.10 and p=0.01 events based on cross section data provided by the City.
4. Assessed the hydraulic model results to evaluate the p=0.10 and p=0.01 events and determine the effects of the WSEL in Tributary 1 on the WSEL in the drainage channel.

Results

The results of our analysis are described below.

Review of available information

To assess existing conditions, we reviewed the Harris study and the existing topographic data.

The Harris study delineated watersheds, computed runoff, and routed flows for the eastern portion of Elk Grove, which includes Tributary 1. The Harris study used the computer program SacCalc to compute the runoff and route the system flows. The land use parameters defined in SacCalc were based on the City's General Plan Land Use Policy Map, as the study assumed future buildout of the study area.

The existing topographic information for the area was supplied as a GIS shapefile depicting contour lines. We reviewed the topography to ensure the contour lines properly matched the development features in the area. Additionally, we compared the contours to the Harris study watersheds in the project area. The watersheds for the Harris study followed the ridgelines from the contours and are appropriate for the Sheldon study area.

Determining hydrologic parameters

We used the watersheds from the Harris study as the starting point in determining the hydrologic parameters for the Sheldon study area. Watersheds LT1U (73.816 acres) and LT1V (111.875 acres) are the contributing watersheds to the open channel for the Sheldon study area. We subdivided watershed LT1V to quantify properly the runoff conveyed by the culvert under Sheldon Road. Watershed LT1V1 is approximately 69.510 acres, while watershed LT1V2 is approximately 42.365 acres. Figure 2 shows the watersheds contributing to the open channel for the Sheldon study area.

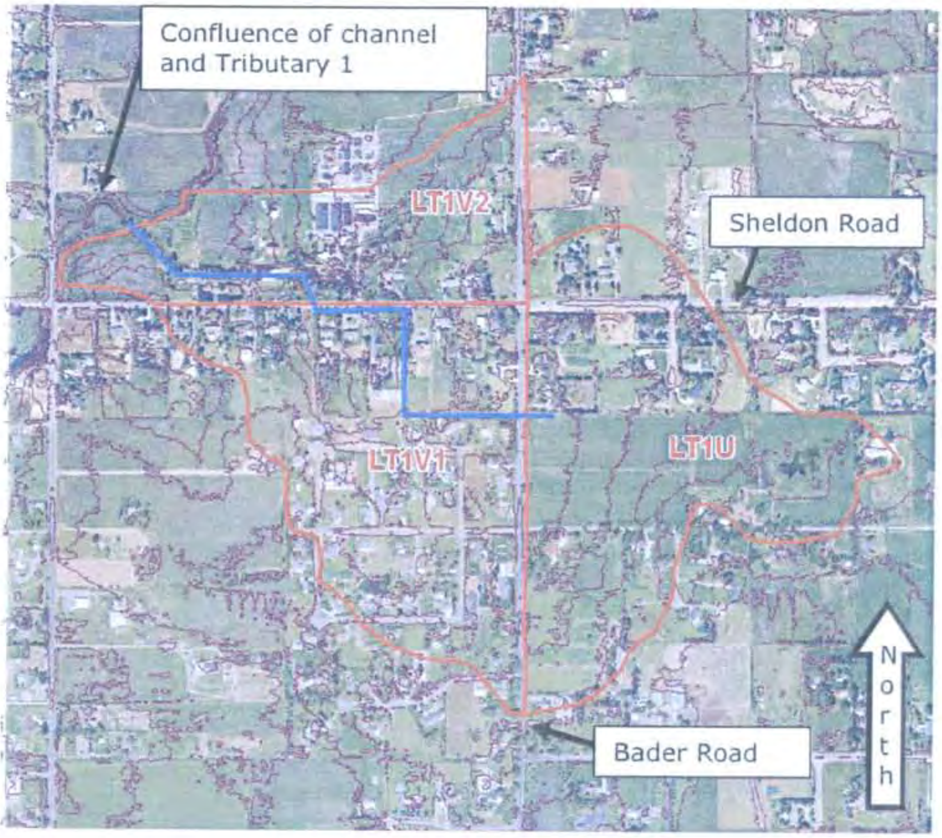


Figure 2. Watersheds contributing to runoff for Sheldon study area

In the Harris study, both watersheds were modeled with 100% of their area being 10% impervious. The modeling parameters are consistent with the General Plan. The General Plan land use policy designation for the area is Rural Residential, which is 10% impervious. However, modeling 100% of the watershed as Rural Residential does not account for the roadway within the watershed. Additionally, the Harris study modeled the drainage areas as only Type D soil. The current Sheldon study analyzes existing conditions and the land use codes from the general plan are used to determine the impervious areas.

Using the watersheds as the base, a GIS intersection was configured and executed to determine the impervious areas based on soil type. The data sources used in the intersection include the watersheds along with soils data and the City's General Plan land use data. The soils data and General Plan land use data were also used in the 2006 DMP. The DMP established a relationship between General Plan land use and percent impervious. This relationship, along with the results of the GIS intersection, provided the watershed impervious area as a function of soil and land use. The area of percent impervious for each watershed is listed in Table 1.

Table 1. New percent impervious for watersheds

	Watershed LT1U		**Watershed LT1V1**		**Watershed LT1V2**	
Percent Impervious (1)	**Soil Type C acreage (2)**	**Soil Type D acreage (3)**	**Soil Type C acreage (4)**	**Soil Type D acreage (5)**	**Soil Type C acreage (4)**	**Soil Type D acreage (5)**
1	28.266	2.130	0.711	-	3.080	8.265
2	5.705	2.259	0.140	0.030	-	-
5	0.102	-	0.844	-	0.836	0.225
10	6.872	2.071	13.984	11.172	16.008	8.155
15	15.887	-	10.148	17.002	2.150	-
50	2.065	-	-	-	-	-
70	2.997	0.399	-	0.074	-	-
80	0.142	-	3.570	1.312	-	-
95	-	4.921	-	10.523	-	3.646

The updated watershed parameters produced a composite percent impervious value of 16.18% for watershed LT1U, 29.63% for watershed LT1V1, and 15.03% for watershed LT1V2. As stated previously, the Harris Study assumed 10% impervious for these watersheds. The composite values agree with the development pattern observed on the aerial photographs. The percent impervious values from Table 1 were configured in the Sacramento County SacCalc model (SacCalc) with the watershed slope and the length of longest watercourse.

Hydrologic model results

We computed the runoff hydrographs for the p=0.10 and p=0.01 events. When computing the runoff hydrographs, the total watershed area was used for the storm area, consistent with the Harris study. SacCalc computes the runoff hydrographs, though we are interested in peak flows for the Sheldon study area. The peak flows are necessary to assess properly potential backwater effects from Tributary 1. Using a steady-state hydraulic model with the peak flows removes attenuation and any time dependency in addition to producing a conservative water surface profile (WSP). The peak flows computed with the current SacCalc model are slightly higher than the flows computed in the Harris study. The peak flows computed with the current SacCalc model and from the Harris study are listed in Table 2.

Table 2. Peak flow from SacCalc model

	Current peak flows (cfs)		**Harris study peak flows (cfs)**	
Watershed (1)	**p=0.10 (2)**	**p=0.01 (3)**	**p=0.10 (4)**	**p=0.01 (5)**
LT1U	72	123	67	124
LT1V1	52	87	71[1]	126[1]
LT1V2	36	61		

1. Note that the current study divided watershed LT1V into 2 watersheds, LT1V1 and LT1V2.

Hydraulic model configuration

A hydraulic model was configured based on survey data provided by PSOMAS. The survey data extended from approximately 90 feet upstream of Bader Road to the channel's termination at Tributary 1. From the survey data, we configured 8 culvert crossings and 69 geometric cross sections in an HEC-RAS model using HEC-RAS version 4.1.0. The survey data indicated 2 channel outflow locations to Tributary 1, which were added to the hydraulic model. Figure 3 shows the HEC-RAS schematic.

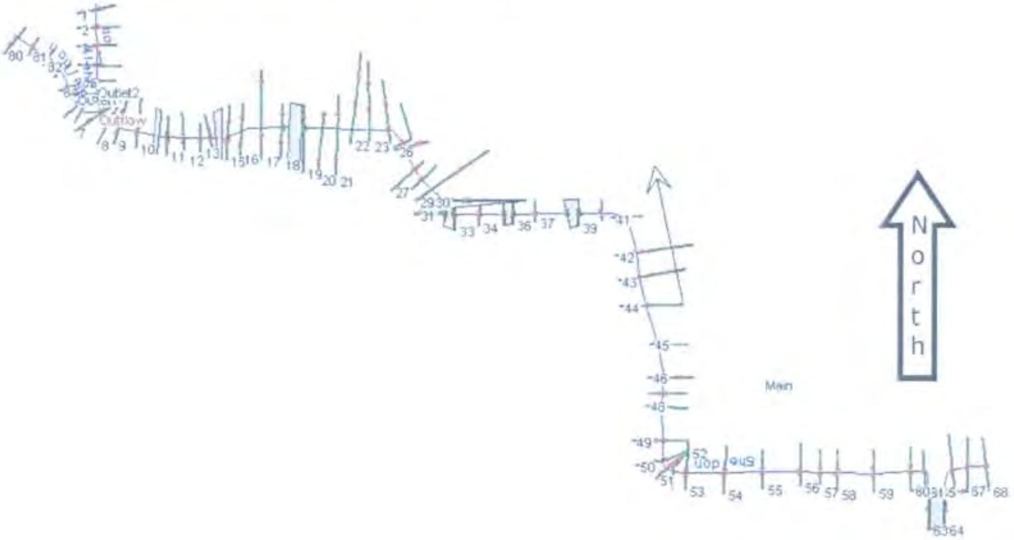


Figure 3. HEC-RAS model schematic

The peak flows computed from SacCalc were used for the upstream model boundary and the flow change locations that were established based on watershed boundaries. Table 3 shows the flow change locations in the HEC-RAS model.

Table 3. Peak flow from SacCalc model

Watershed (1)	**HEC-RAS river station (2)**	**Flows for p=0.10 event (cfs) (3)**	**Flows for p=0.01 event (cfs) (4)**
LT1U	68	72	123
LT1V1	34	124[1]	210[1]
LT1V2	16	160[1]	271[1]

1. Upstream flow plus watershed inflow.

We used the stage hydrograph for Tributary 1 at the confluence with the channel to establish the downstream boundary condition. The Laguna Creek Watershed Analysis by Ford, dated July 25, 2005 (2005 Ford Study) provided the appropriate stage hydrograph in Tributary 1. The stage hydrograph from river station 4936 in Tributary 1 was used to determine the boundary condition, as this is the approximate location of the channel confluence with Tributary 1. Figure 4 shows the stage hydrograph used to determine the downstream boundary condition.

We determined the roughness values for the hydraulic model based on a field visit. The channel from Bader to Sheldon Road is heavily vegetated while the overbank contains natural grasses with minor vegetation. For this reach we assigned a Manning's *n* value of 0.07 to the channel and 0.05 to the overbanks. These Manning's *n* values were also used for the channel outlet, as the same characteristics were observed in that area. For the stretch along Sheldon Road and through the residential ponds, we used a Manning's *n* value of 0.04 for the channel and 0.05 for the overbanks. This area has less vegetation in the channel and has similar overbank characteristics to the other channel segments.

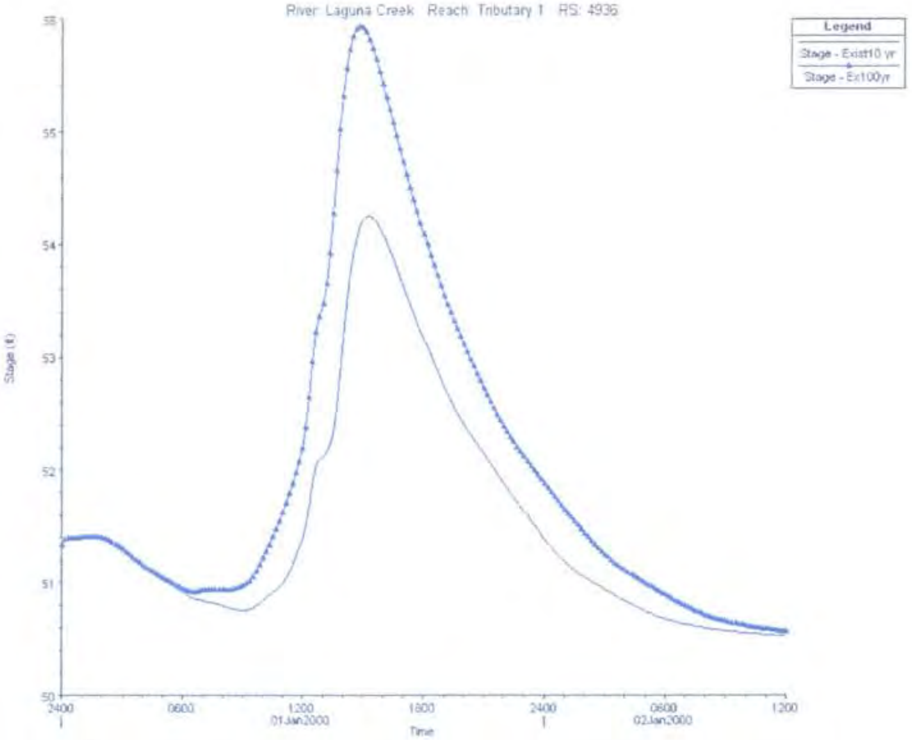


Figure 4. Stage hydrograph at river station 4936 in Tributary 1

The Tributary 1 and drainage channel models are not coupled because the original county UNET model (the model used in the 2005 Ford study) over accounted for floodplain storage. The 2005 Ford study added encroachments that were added to the Tributary 1 model based on the build-out assumption that floodplain storage would be removed. Due to the uncertainty with the County model development and attenuation in this reach, our ability to assess the timing of the channel discharge with the timing of Tributary 1 is limited and prevents the coupling of the models. In addition, the hydrologic (SacCalc) and hydraulic (HEC-RAS) models are not coupled.

For this analysis, we used the stage hydrograph from the Tributary 1 model to produce a range of downstream boundary conditions for the drainage channel HEC-RAS model. We chose a minimum, medium, and maximum stage from the stage hydrographs for each probabilistic event. The stage hydrograph is shown in Figure 4 and the minimum, medium, and maximum stages are listed in Table 4.

Table 4. Range of downstream boundary conditions

Tail-water scenario (1)	**p=0.10 (ft)** (2)	**p=0.01(ft)** (3)
Minimum	51.34	51.34
Medium	52.80	53.64
Maximum	54.25	55.93

Hydraulic model results

We used the hydraulic model to compute the WSEL for the p=0.10 and p=0.01 events. The maximum WSEL profiles for the p=0.10 and p=0.01 events are shown in Figure 5 and Figure 6, respectively.

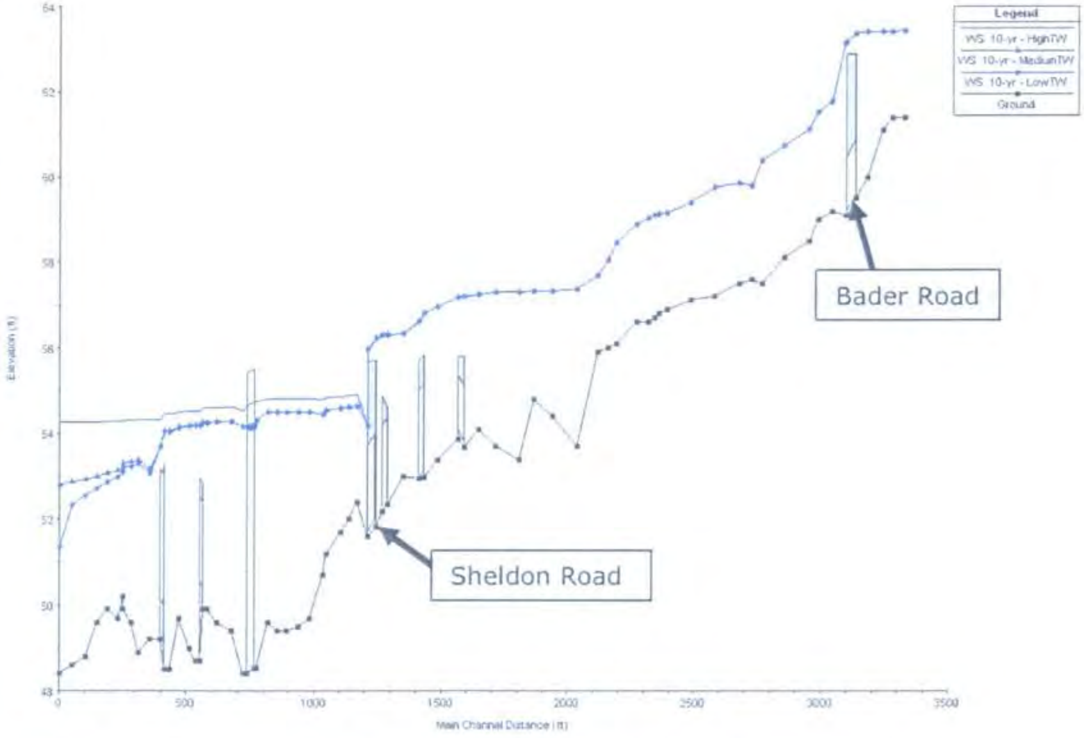


Figure 5. HEC-RAS maximum WSP in the drainage channel for the p=0.10 event

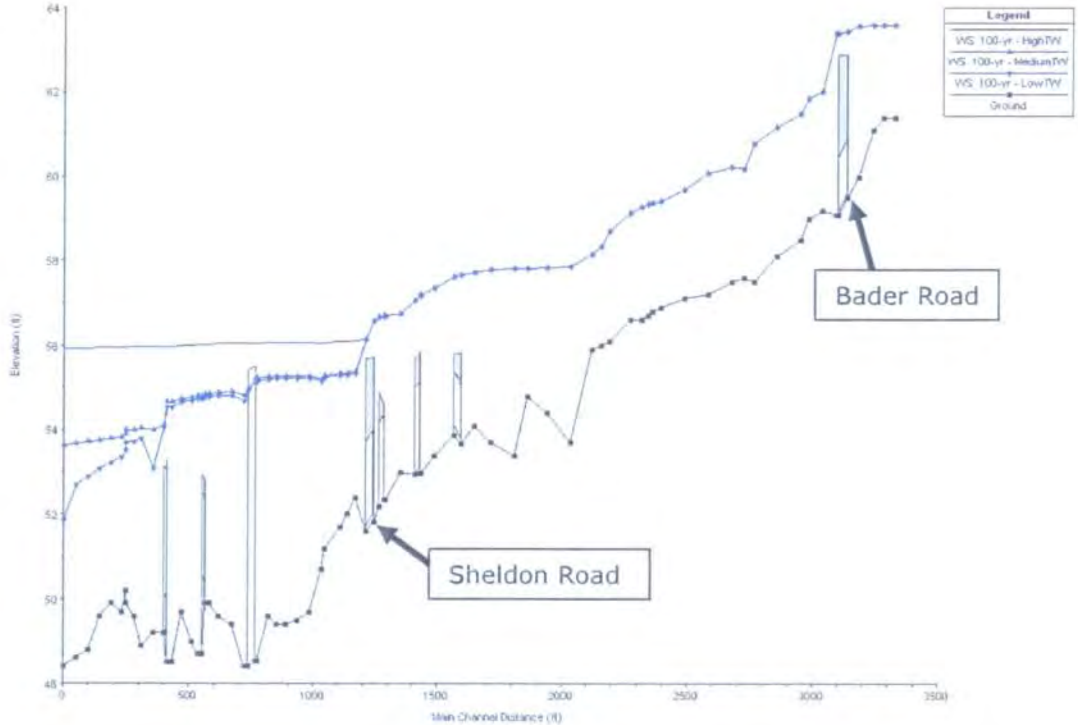


Figure 6. HEC-RAS maximum WSP in the drainage channel for the p=0.01 event

The hydraulic model shows the overtopping of both Sheldon and Bader roads in the p=0.10 and p=0.01 events. Specifically, the City is interested in quantifying the overtopping in the p=0.10 event. In the p=0.10 event the tail-water elevation does not have an effect on the WSEL upstream of Sheldon Road. Additionally, Sheldon Road overtops for the 3 tail-water conditions that were evaluated. The approximate elevation of Sheldon Road is 55.7 feet, while the water surface profile for the low-, mid-, and high tail-water conditions are approximately 56.2 feet.

Evaluation of hydraulic alternatives for the p=0.10 event

We analyzed 3 hydraulic alternatives at the request of the City with the objective of preventing Sheldon Road (RS 30.5 in the HEC-RAS model) from overtopping, with a tolerance of 0.01 feet, in the p=0.10 event. The alternatives were sized with the objective of reducing the channel WSEL to at or below the overtopping elevation of Sheldon Road, 55.7 feet.

The 3 hydraulic alternatives are described below:

- Alternative 1 involved widening the channel between Bader and Sheldon roads and upsizing the Sheldon Road crossing to 2 box culverts, each 2 feet by 4 feet. In widening the channel, the bank stations were not altered for the cross sections and 2:1 horizontal to vertical side slopes were configured within the bank stations to reach the channel invert elevation. There were approximately 2 locations where a 2:1 side slope could not be configured due to channel width constraints. At these locations we did not alter the existing channel cross section.

- Alternative 2 involved detaining the water from watershed LT1U to reduce flow into the channel. We configured the model to pass a steady flow through the model that would prevent overtopping, and then computed the volume above that steady flow from the watershed hydrograph that would need to be detained. We determined from model runs that reducing the flow from watershed LT1U to a 3 cfs steady flow would not overtop Sheldon Road and Bader Road, requiring approximately 5.59 acre-feet of water to be detained upstream of Bader Road.
- Alternative 3 involved upsizing the Sheldon Road crossing to 2 box culverts, each 2 feet by 4 feet, without any modifications to the channel. We analyzed this alternative to determine if widening the channel, as done in the first alternative, had a significant effect on the WSEL at Sheldon Road.

The three alternatives have similar WSEL values as the alternatives were sized with the objective of keeping the WSEL at an elevation of 55.7 feet, the overtopping elevation of Sheldon Road. The WSEL for the 3 alternatives are shown in Table 5.

Table 5. WSEL at Sheldon Road for hydraulic alternatives

Tail-water scenario (1)	Existing WSEL (ft) (2)	Alternative 1 WSEL (ft) (3)	Alternative 2 WSEL (ft) (4)	Alternative 3 WSEL (ft) (5)
Minimum	56.2	55.7	55.7	55.7
Medium	56.2	55.7	55.7	55.7
Maximum	56.2	55.7	55.8	55.7

In addition the City requested that we investigate Alternative 3a to determine if adding culvert capacity at Bader Road would prevent Bader Road from overtopping in the p=0.10 event. We augmented Alternative 3 by upsizing the existing Bader Road culvert from a 1.25-foot diameter circular pipe to 2 box culverts, each 2 feet by 4 feet. Upsizing the Bader Road culverts prevents Bader Road from overtopping. The profile for Alternative 3a, showing no overtopping at Sheldon Road and Bader Road, is shown in Figure 7.

Figure 7. HEC-RAS maximum WSP for the p=0.10 event with upsized culvert conveyance at Bader Road and Sheldon Road, Alternative 3a

Conclusion

Based on our analysis, we found the following:

- For the p=0.10 event, the WSEL in Tributary 1 does not cause a backwater effect above Sheldon Road.
- For the p=0.01 event, there is a backwater effect above Sheldon Road when Tributary 1 reaches its peak stage.
- Based on the steady flow model Sheldon Road overtops by 0.5 feet in the current conditions for the p=0.10 event. The overtopping value is a conservative value as a steady flow model has been used for the analysis.
- Alternative 1 includes expanding culvert conveyance at Sheldon Road and channel capacity between Bader Road and Sheldon Road to prevent the overtopping of Sheldon Road in the p=0.10 event. The existing 2 circular culverts at Sheldon Road, 2 feet in diameter, could be replaced by 2 box culverts, each 2 feet by 4 feet, to prevent overtopping.
- Alternative 2 includes constructing a detention basin upstream of Bader Road to reduce the contributing runoff from watershed LT1U to prevent the overtopping of Sheldon Road and Bader Road in the p=0.10 event. Approximately 5.59 acre-feet of detention would be needed upstream of Bader Road to prevent Sheldon Road from overtopping, with an allowable tolerance of 0.1 feet.
- Alternative 3 includes expanding culvert conveyance to prevent the overtopping of Sheldon Road in the p=0.10 event. The existing 2 circular

culverts at Sheldon Road, 2 feet in diameter, could be replaced by 2 box culverts, each 2 feet by 4 feet, to prevent overtopping.

- Alternative 3a augments alternative 3 and includes expanding culvert conveyance to prevent the overtopping of Bader Road in the p=0.10 event. The existing 1.25-foot diameter circular culvert at Bader Road can be replaced by 2 box culverts, each 2 feet by 4 feet, to prevent overtopping.

ATTACHMENT 6B

Laguna Creek Tributary 1 Hydraulic Analysis

DAVID FORD
CONSULTING
ENGINEERS

David Ford Consulting Engineers, Inc.
2015 J Street, Suite 200
Sacramento, CA 95811
Ph. 916.447.8779
Fx. 916.447.8780

BRIAN A. BROWN
No. 69338
Exp. 6/30/12

MEMORANDUM

To: Fernando Duenas, PE

From: Brian A. Brown, PE

Date: May 26, 2011

Subject: Laguna Creek Tributary 1 hydraulic analysis

Situation

The City of Elk Grove (City) is investigating whether the p=0.01 event floodplain changes along a portion of Laguna Creek Tributary 1 as a result of new surveyed cross sections. The specific location of the new cross sections is from Sleepy Hollow Unit 2 to Excelsior Road, as shown in Figure 1.

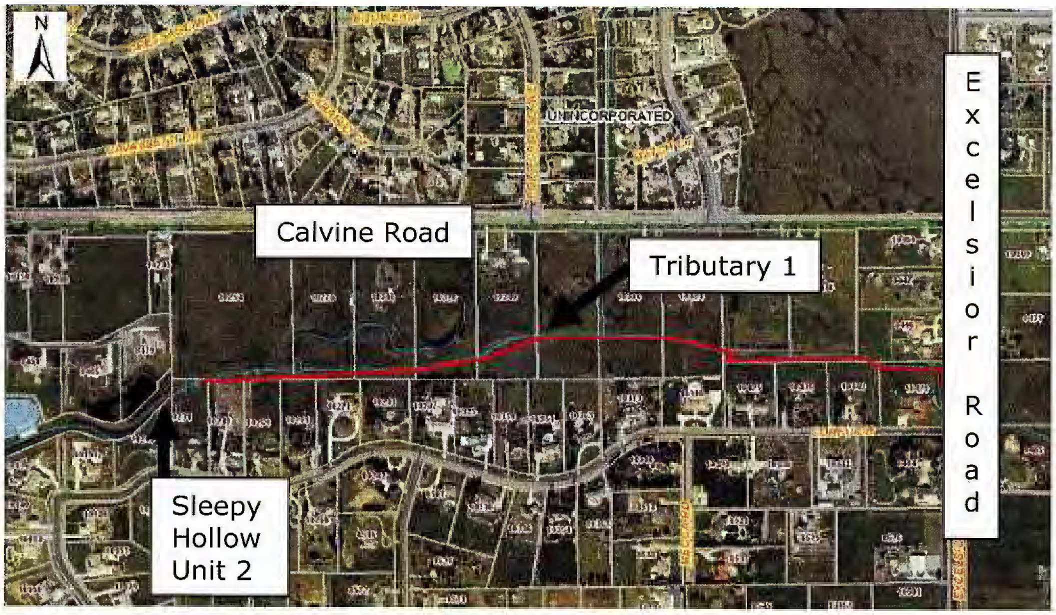


Figure 1. Laguna Creek Tributary 1 from Sleepy Hollow Unit 2 to Excelsior Road (map provided by City staff)

The hydrology and hydraulics of Tributary 1 were analyzed in several previous studies:

- The existing model for Tributary 1 was configured by Sacramento County (County model) at an unknown date, sometime prior to 1997, with the topographic data available at the time of study.

Less than ½" to Reinforce Berm

- The "2005 Laguna Creek watershed analysis" (2005 Laguna) used the geometry from the County model for Tributary 1 when updating the Laguna Creek watershed model.
- The Tributary 1 hydrology was analyzed by Harris and Associates in the "2005 City of Elk Grove East Area Storm Drainage Master Plan Revised Draft" (Harris DMP).
- The 2005 Draft City of Elk Grove Flood Control and Storm Drainage Master Plan by West Yost and Associates (WYA Master Plan) used the Harris DMP flow from Tributary 1 as a lateral inflow into Laguna Creek. The WYA Master Plan included Laguna Creek but did not include Tributary 1.

Newly surveyed cross sections and LiDAR data for the area are now available which provide a more accurate indication of the hydraulics in Tributary 1. The Tributary 1 hydrologic analysis from the Harris DMP was used as the baseline hydrology for this analysis.

Task

We determined whether the p=0.01 event floodplain changes along a portion of Laguna Creek Tributary 1 based on new surveyed cross sections.

Actions

To evaluate the hydraulic characteristics of Tributary 1 based on updated topographic data, we answered the following questions:

- What is the p=0.01 water surface elevation (WSEL) in Tributary 1 using the Harris DMP hydrology and new hydraulic cross sections?
- How sensitive is the WSEL in Tributary 1 to different Manning's *n* values?
- How does the computed WSEL in Tributary 1 using the Harris DMP hydrology compare to the computed WSEL using approximate FEMA flows?
- Does running the model in unsteady mode, using runoff hydrographs instead of peak flows, reduce the WSEL in Tributary 1?
- What are the new floodplain extents for the p=0.01 event in Tributary 1 based on the Harris DMP hydrology?

Thus, we:

1. Developed a digital terrain model (DTM) based on LiDAR topographic information from the California Department of Water Resources (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program.
2. Compared cross sections cut from the DTM to the newly surveyed cross sections.
3. Added field survey points to the DTM to produce an updated base topography dataset.
4. Created and executed a hydraulic model in steady state mode based on new topography and the Harris DMP hydrology.
5. Executed the hydraulic model with FEMA hydrology for Tributary 1 for comparison purposes.

6. Completed a Manning's *n* value sensitivity analysis for the hydraulic model.
7. Assessed the hydraulic model results to evaluate p=0.01 event.
8. Configured and ran an unsteady plan for the p=0.01 event.
9. Delineated the extents of the p=0.01 floodplain for Tributary 1.

Development of DTM

As part of the CVFED program, the DWR acquired detailed LiDAR data for the Lower Sacramento River Basin in February and March 2010. For the Laguna Creek Tributary 1 study area, we requested and received from DWR the Task Order (TO) 13 CVFED LiDAR data. The CVFED LiDAR data are referenced to the NAD 83 horizontal datum and NAVD 88 vertical datum.

To match the field survey data and previous modeling efforts for this area, we converted the CVFED LiDAR data from the NAVD 88 datum to the NGVD 29 datum. We used a conversion factor of −2.38 feet to convert the data from NAVD 88 to NGVD 29. We obtained the −2.38 foot conversion factor using the US Army Corps of Engineers computer program Corpscon, version 6.0. The point LiDAR data received from the DWR were used to build a triangulated irregular network (TIN) surface using the 3D Analyst extension in ArcGIS.

Comparison of cross sections

The point field survey data follow approximate cross section lines that were provided in AutoCAD format. For this comparison of cross sections we:

1. Imported these planimetric cross section lines into GIS and adjusted them to fit best the field survey points.
2. Used HEC-GeoRAS to assign georeferenced elevation data to these cross sections based on the TIN.
3. Compared the resulting georeferenced cross sections to the field surveyed cross sections.
4. Determined whether the LiDAR data captured the low flow portion of the channel and whether the survey data correlate to the LiDAR data in the channel and overbanks.

In all, 17 sections were compared to see how the LiDAR data compared to the field survey data. The locations of these sections are shown in Figure 2.

Figure 3 through Figure 7 are screenshots of the HEC-RAS graphic cross-section data editor showing the section comparison. The sections cut from the LiDAR data are shown in black and the field surveyed sections are shown in magenta.

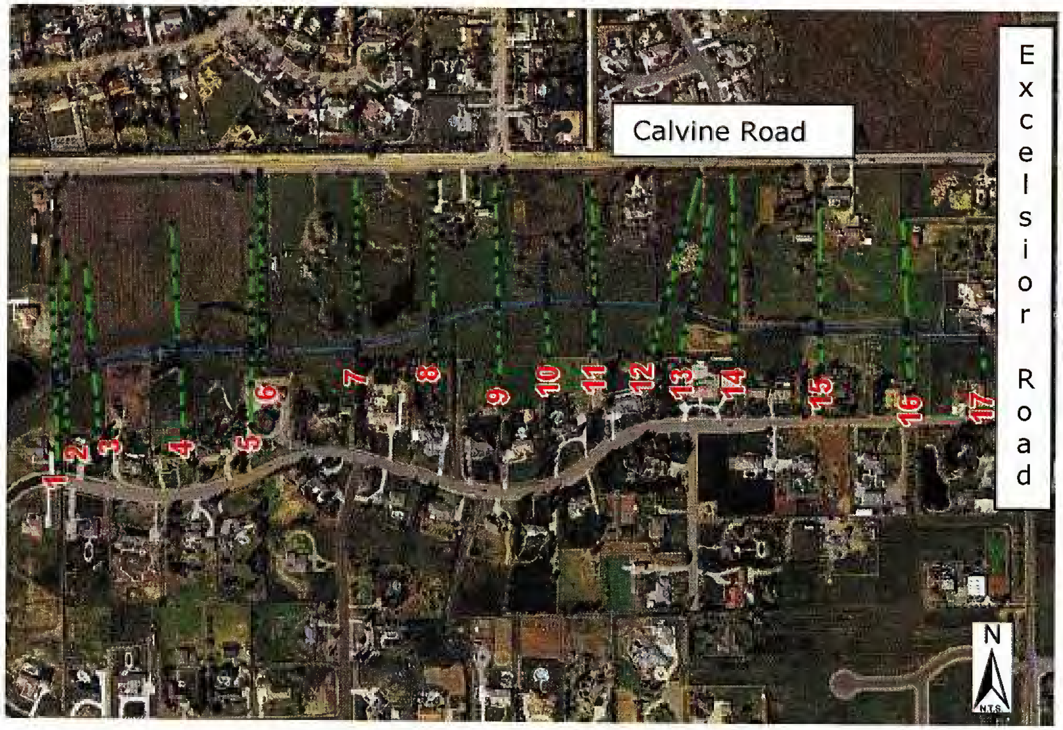
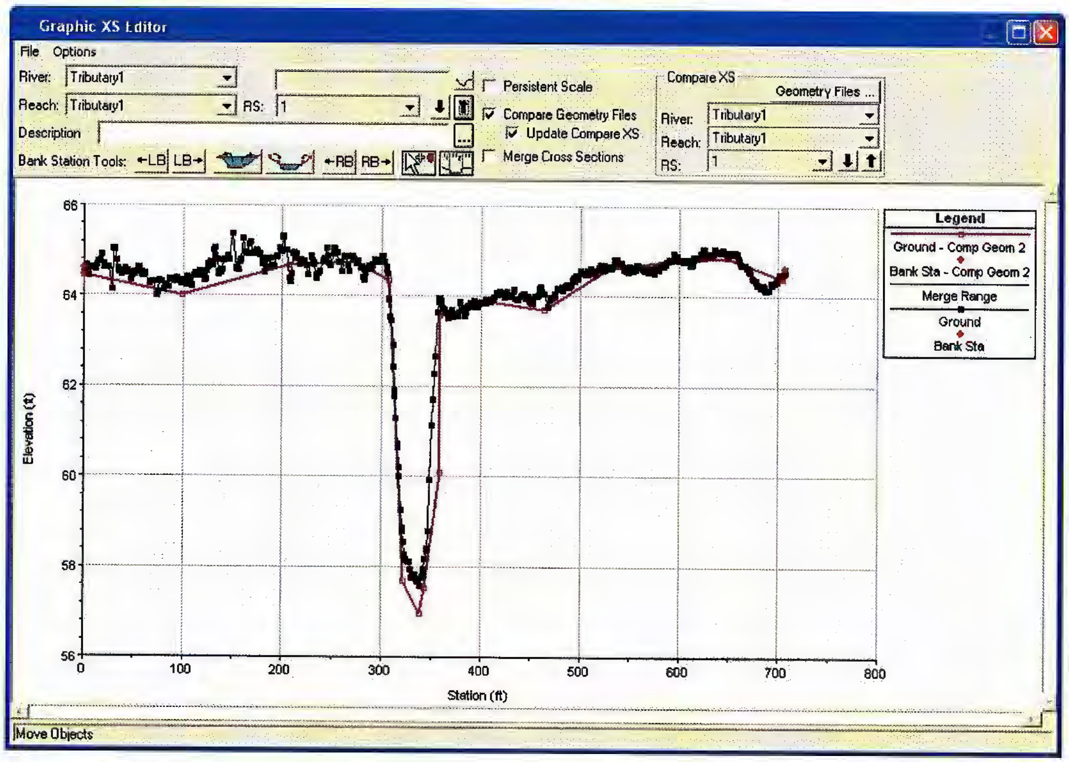


Figure 2. Locations where field-surveyed cross sections were compared to cross sections cut from LiDAR data


Graphic XS Editor

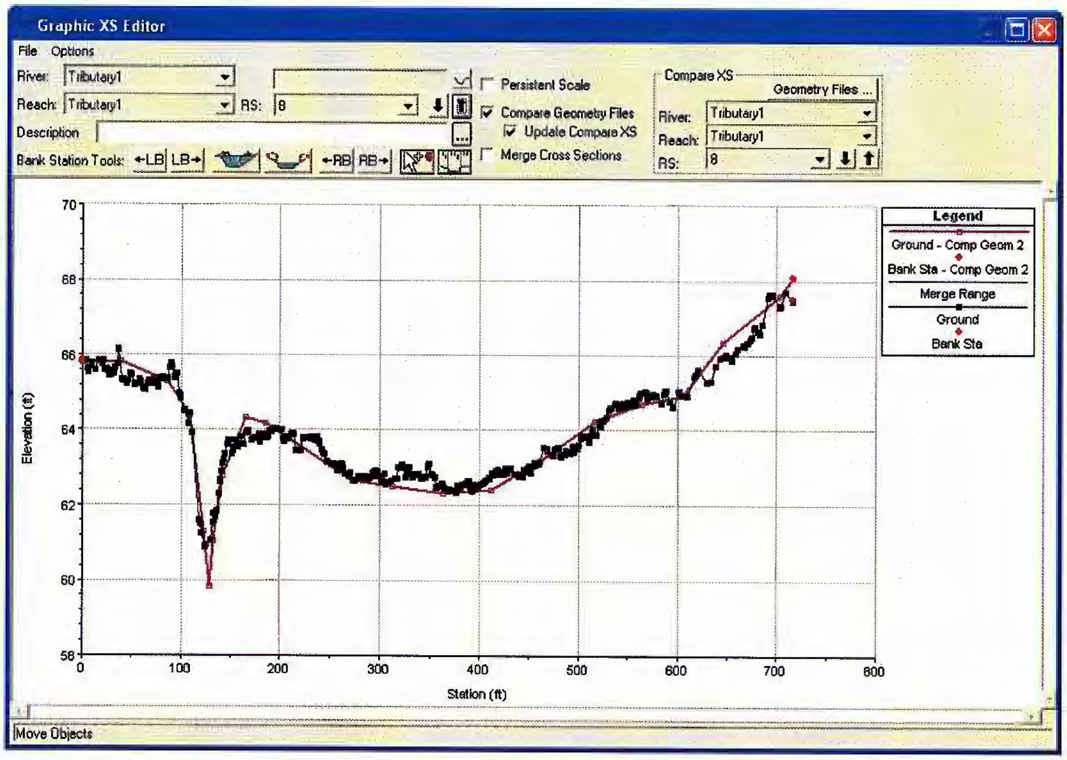
River: Tributary1
Reach: Tributary1
RS: 1
Description
☐ Persistent Scale
☑ Compare Geometry Files
☑ Update Compare XS
☐ Merge Cross Sections

Compare XS
River: Tributary1
Reach: Tributary1
RS: 1

Figure 3. Cross section comparison at location 1

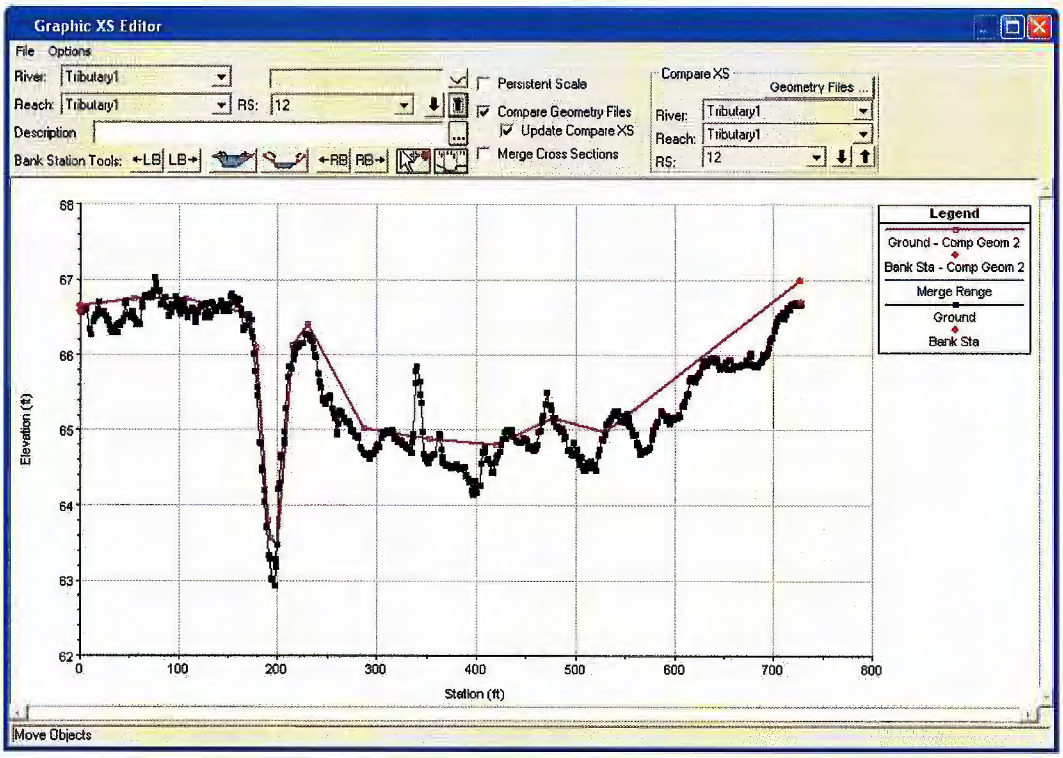
River: Tributary1
Reach: Tributary1 RS: 6
274.93, 64.92
☐ Persistent Scale
☑ Compare Geometry Files
☑ Update Compare XS
☐ Merge Cross Sections
Compare XS River: Tributary1
Reach: Tributary1
RS: 6

Figure 4. Cross section comparison at location 6



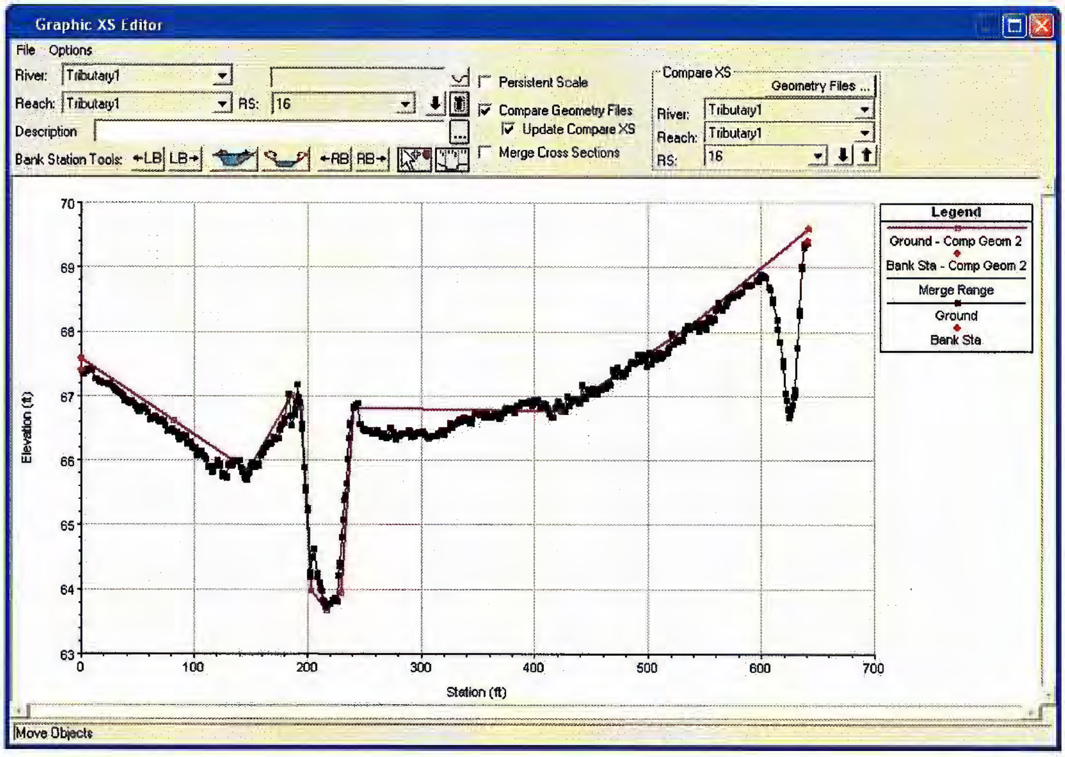
River: Tributary1
Reach: Tributary1 RS: 8
☐ Persistent Scale
☑ Compare Geometry Files
☑ Update Compare XS
☐ Merge Cross Sections
Compare XS River: Tributary1
Reach: Tributary1
RS: 8

Figure 5. Cross section comparison at location 8



River: Tributary1
Reach: Tributary1
RS: 12
Description:
☐ Persistent Scale
☑ Compare Geometry Files
☑ Update Compare XS
☐ Merge Cross Sections
Compare XS River: Tributary1
Reach: Tributary1
RS: 12
Move Objects

Figure 6. Cross section comparison at location 12



River: Tributary1
Reach: Tributary1
RS: 16
Description:
☐ Persistent Scale
☑ Compare Geometry Files
☑ Update Compare XS
☐ Merge Cross Sections
Compare XS River: Tributary1
Reach: Tributary1
RS: 16
Move Objects

Figure 7. Cross section comparison at location 16

The main difference between the LiDAR and field survey datasets is that the LiDAR data have a higher resolution than the field survey data, particularly in the overbanks. For example, the right side of location 16 (Figure 7) shows a dip in the LiDAR that is not present in the field survey data. (No field data were acquired in this area.) Figure 7 illustrates the limitation of the field survey data: there are not enough points to capture fully all of the elevation changes in the overbanks.

In the comparison between the sections, the LiDAR data adequately capture the low flow portion of the channel in this area. Had the field survey data captured a low flow portion of the channel that was not present in the LiDAR data, we would have needed to develop an approximate low flow channel. This approximate low flow channel would then have been stitched into the LiDAR sections where there was no field survey data. This process would have been required to represent field conditions better, as the LiDAR could potentially miss the low flow portion of the channel if water was present during data acquisition. Here, all 17 of the comparison cross sections were provided to the City and discussed in a conference call. Since the LiDAR sections adequately captured the low flow channel, there was no need to stitch in an approximate low flow channel for Tributary 1.

After we compared the cross sections, we updated the TIN by adding the field survey data. In doing so, we removed 4 of the field survey points. These points were removed because the field survey showed elevations that were approximately 3 feet lower than the LiDAR data in this area. The other field survey points near these 4 points agreed with the LiDAR data, prompting the removal of these points. We added the field survey data to the TIN to enhance the surface, providing additional detail to the LiDAR data.

Hydraulic model configuration

We laid out new cross sections for Tributary 1 using the approximate stream centerline distances from the 2005 Laguna model. Sections were also added along bends and at locations where the overbank flow area changed to define the flow path and flow area better. We developed the new cross-section geometry using HEC-GeoRAS to assign georeferenced elevation data to the model cross sections. The updated TIN provided the elevation source for the cross sections. The hydraulic model cross section locations are shown in Figure 8.

The 2005 Laguna model contains 2 hydraulic structures in the Tributary 1 reach. Both structures are bridges, 1 representing Bradshaw Road and the other representing Bader Road. These bridges were previously modeled as clear span bridges with less than 10 roadway deck points. In updating the model, we anticipated using the bridge definitions from the previous model. However, in the new sections, the elevations of the bridge decks in the previous model are approximately 2 feet higher than the current topography. Due to this discrepancy and the lack of definition in the previous roadway decks, we updated the bridges for this model. We updated the bridges by cutting new cross sections over the roadway and approximating a 1-foot deck thickness for the bridges. We used the same procedure to add approximate bridge definitions at the Central California Traction Railroad tracks (CCTRR) and at Excelsior Road. The bridge definitions in this model are approximate and can be refined by the City in future studies based on field survey data or as-built plans.

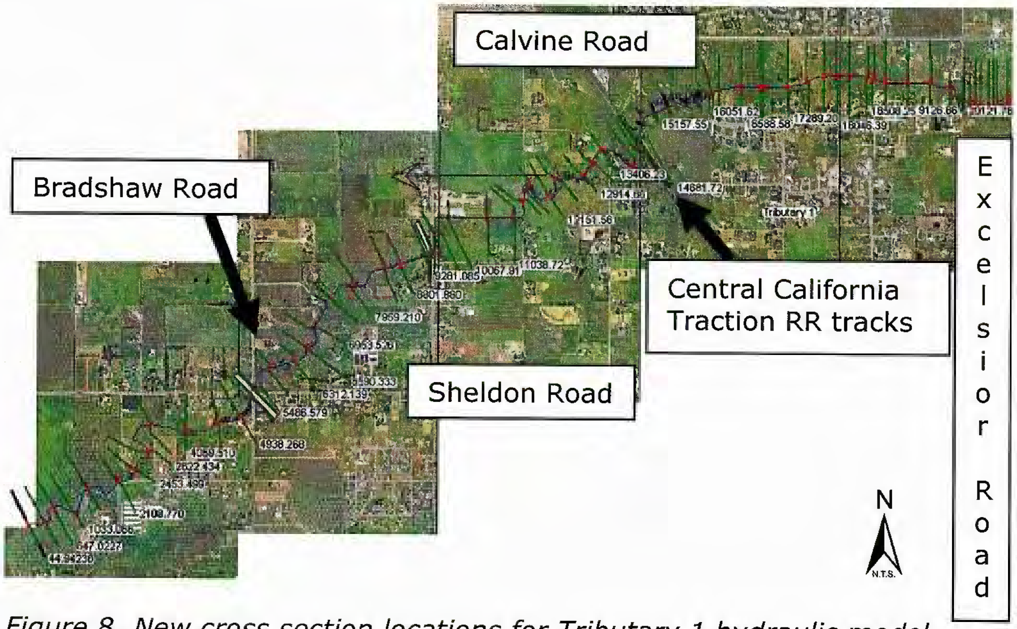


Figure 8. New cross section locations for Tributary 1 hydraulic model

Other parameters we configured in the new hydraulic model include Manning's *n* values, ineffective flow areas, and blocked obstructions. Per the City's instruction, we used Manning's *n* values of 0.06 for the channel and 0.08 for the overbanks. The City based these values on previous modeling efforts along Tributary 1. We configured ineffective flow areas in the model at locations where the water is not being actively conveyed by the channel, where we anticipate little or no velocity in the channel. We added blocked obstructions in 2 areas of the model to remove area that is accounted for in other models. The blocked obstructions at RS 44.94238 block out flow area that is accounted for in the Laguna Creek model. The other blocked obstructions in the model are located at RS 14897.08 through RS 15381.83 and represent storage accounted for in the Sleepy Hollow detention basin. The routing effects of this detention basin are presumed to be included in the storage-discharge relationship in the SacCalc hydrologic model.

Hydraulic model boundary conditions

The boundary conditions for the hydraulic model are from previous studies. The downstream boundary for the model is the WSEL in Laguna Creek from the WYA Master Plan model for the p=0.01 event. We removed the Tributary 1 lateral inflow and ran this model to establish the WSELs in Laguna Creek. Tributary 1 enters Laguna Creek between RS 5109 and RS 4360. The average WSEL between these 2 sections is 48.03 feet, which is used as the downstream stage boundary condition.

The flows into the model are taken from the Harris DMP SacCalc model. The flow change locations where the cumulative peak flows from the SacCalc routing reaches enter the HEC-RAS model are listed in Table 1.

Table 1. Tributary 1 HEC-RAS flow change locations and cumulative peak flows from routing reaches in Harris DMP model

HEC-RAS river station (1)	**SacCalc node (2)**	**Peak flow[1] (cfs) (3)**
21619.32	CHA02	448
20174.00	JNC03	565
19836.53	JNC04	1075
18303.64	JNC09	1161
16845.68	JNC10	1179
14681.69	JNC11	1280
11038.70	JNC12	1422
9671.98	JNC14	1444
7959.19	JNC15	1455
6312.12	JNC16	1461
5486.56	JNC17	1470
2453.49	JNC18	1450
647.02	JNC19	1449

950

1. HEC-RAS steady model flow change locations.

Use of hydraulic model to compute WSEL

We used the hydraulic model to compute the steady-state WSEL for the p=0.01 event. The profile computed using the Harris DMP flows shown in Table 1 is shown in Figure 9.

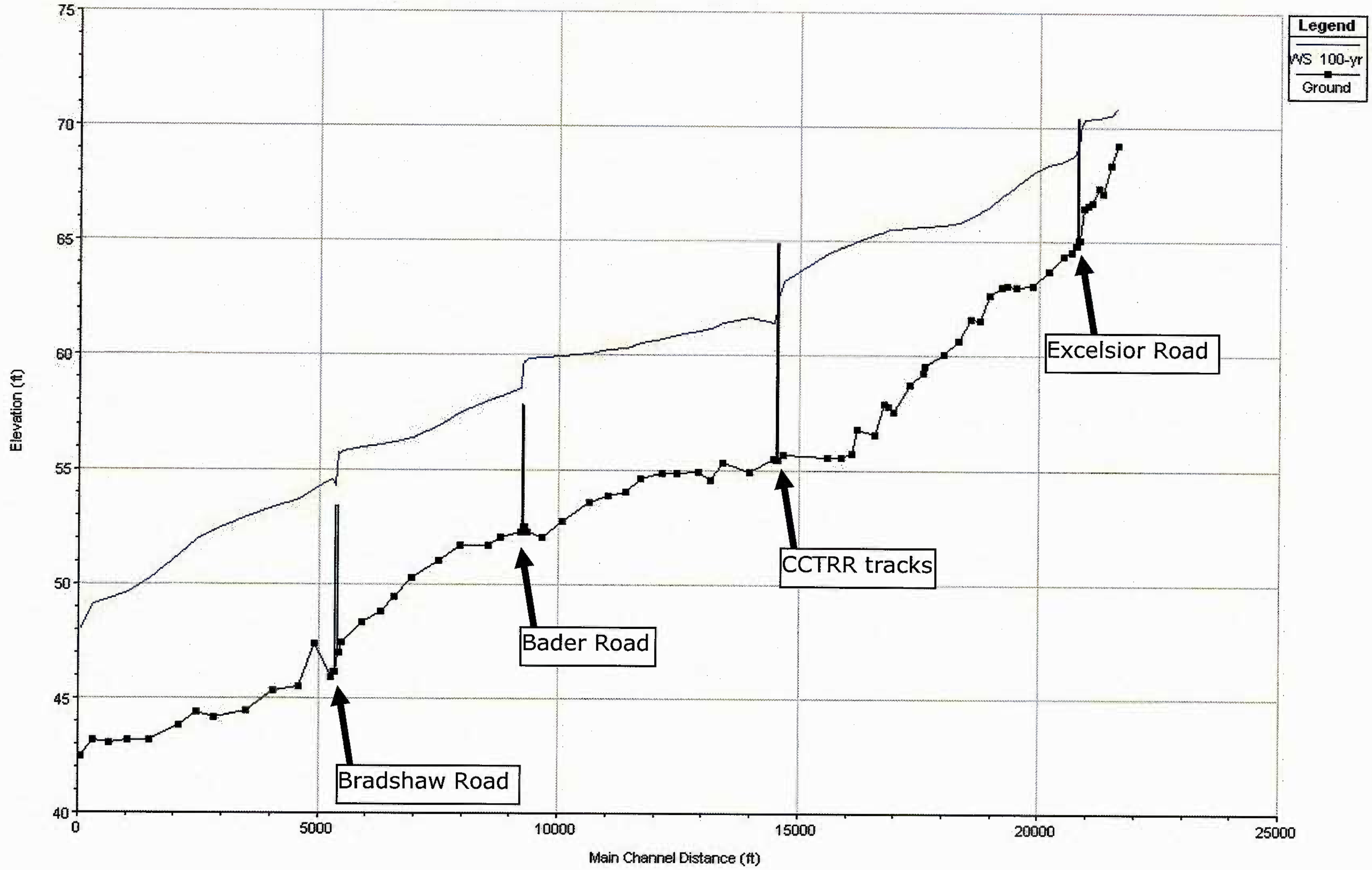


Figure 9. Tributary 1 hydraulic model profile for the p=0.01 event, using Harris DMP flows

Comparison using FEMA hydrology

For comparison purposes we ran the regulatory FEMA flows through the updated hydraulic model. The Sacramento County Flood Insurance Study (FIS) contains 1 flow for Laguna Creek Tributary 1 for the p=0.01 event. This flow, a peak discharge of 950 cubic feet per second (cfs), is the total flow in Tributary 1 for the p=0.01 event. This is the only flow provided for the p=0.01 event and there is uncertainty as to where the flow reaches this value in the system. Using this flow as an upstream boundary condition provides a worst-case scenario, as it overestimates the upstream flows and water surface elevations. The FIS showed a downstream WSEL of 49.5 feet in Tributary 1 for the p=0.01 event.

We used a model provided by the City for additional FIS flows in Tributary 1. The City provided previous studies for Tributary 1 which included a portion of the Tributary 1 FIS model (partial FIS model). This model contained flows for the p=0.01 event in the upstream portion of Tributary 1. These flows can be used in addition to the single flow of 950 cfs from the Sacramento County FIS. To determine where these flows enter the updated model, we used the distances from the partial FIS model flow change locations to the modeled bridges. These were applied to the new model to obtain the approximate river station for the flow change location in the new model. An estimated upstream flow of 300 cfs is used to approximate the upstream boundary condition because the upstream portion of the new model is outside the limits of the partial FIS model. The location where the 950 cfs enters the model is a conservative approximation because the location is closer to the upstream flow change location than the downstream end of the model. These flow change locations are listed in Table 2.

Table 2. Tributary 1 HEC-RAS locations where FIS flows are applied

HEC-RAS river station (1)	FEMA p=0.01 flow[1] (cfs) (2)
21619.32	300[2]
20918.76	380
18303.64	525
14681.69	665
9281.06	760
8551.72[3]	950

1. HEC-RAS steady model flow change locations.

2. Upstream flow value is estimated due to lack of data.

3. Downstream flow location is estimated due to lack of data.

We computed the water surface profile using the approximate FIS flows and the new geometric data. This profile is then compared to the profile produced from the Harris DMP to see the relationship between the 2 profiles. The profile computed from the approximate FIS flows and the profile computed using the Harris DMP hydrology for the p=0.01 event are shown in Figure 10. The profile computed with the approximate FIS flows is lower than the profile computed using the Harris DMP hydrology. As indicated in Table 1 and Table 2, the approximate FIS flows are significantly lower than the Harris DMP flows. For quantitative comparison, WSELs for the Harris DMP flows and the approximate FIS flows are listed in Table 3.

Table 3. Tributary 1 HEC-RAS WSELs for Harris DMP flows and approximate FIS flows

River station (1)	**WSEL for initial plan[1] (ft) (2)**	**WSEL using approximate FIS flows[1] (ft) (4)**
21619.32	70.88	70.62
20995.39	70.38	70.11
19836.53	68.01	66.96
17298.25	65.62	63.60
15606.75	64.47	62.47
13406.21	61.42	60.34
11711.45	60.58	59.54
10067.89	59.94	58.84
8551.72	58.05	57.30
6953.50	56.37	55.59
5898.28	55.95	55.19
3505.44	52.90	52.17
1490.47	50.24	50.05

1. Manning's *n* values of 0.06 for the channel and 0.08 for the overbanks were used.

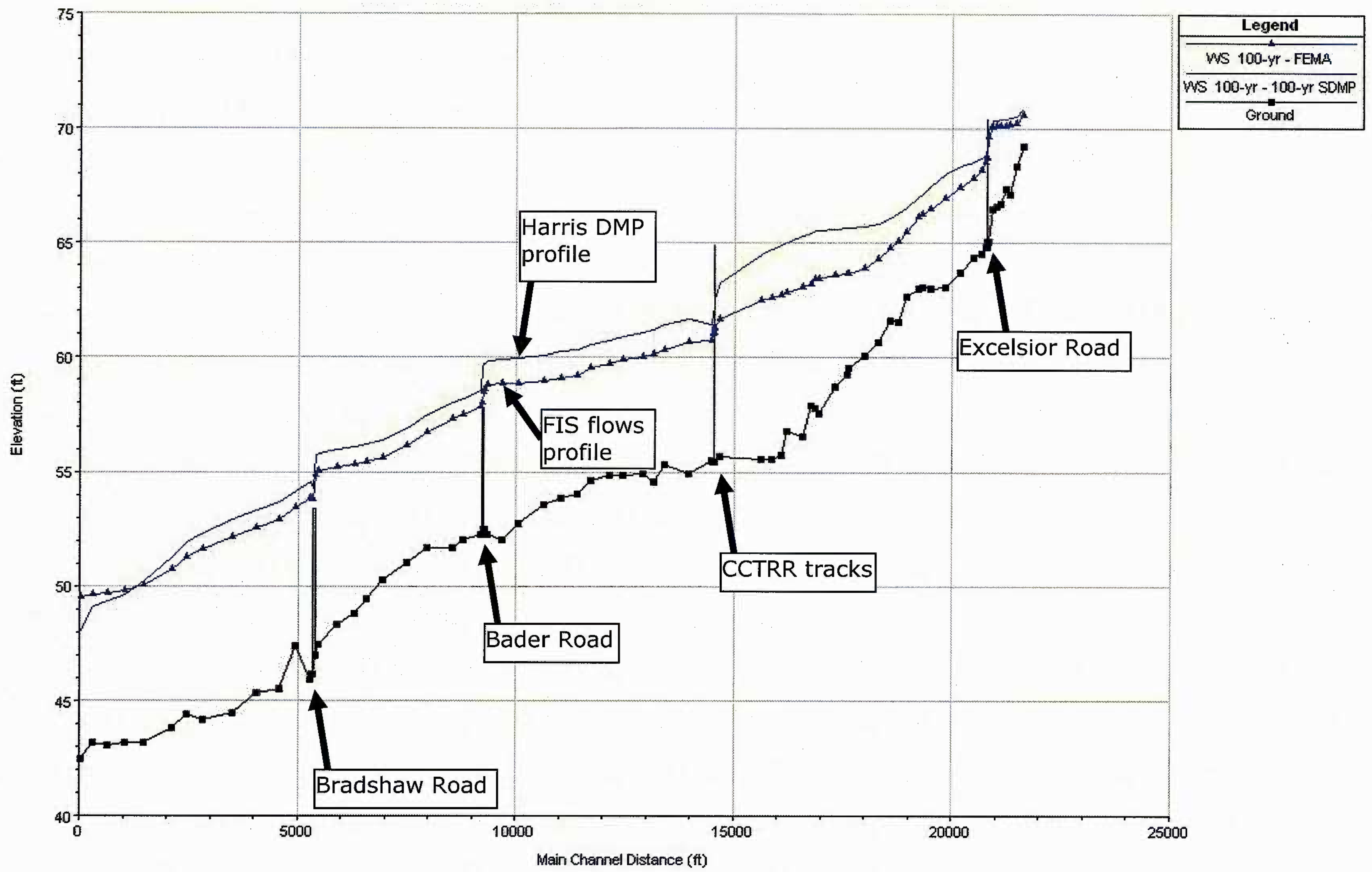


Figure 10. Tributary 1 hydraulic model profile for the p=0.01 event, comparison of the approximate FEMA flows to the Harris DMP flow

Manning's *n* value sensitivity

The City is interested in the sensitivity of the model to adjustments in the Manning's *n* value. Per the City's instruction, we used Manning's *n* values of 0.05 for the channel and 0.06 for the overbanks in the sensitivity analysis. The initial model plan used Manning's *n* values of 0.06 for the channel and 0.08 for the overbanks. The profile for the sensitivity model plan compared to the initial profile is shown in Figure 11.

Reducing the Manning's *n* values in the channel from 0.06 to 0.05 and in the overbank from 0.08 to 0.06 reduces the WSEL in Tributary 1. The water surface profile is reduced on average by 0.35 feet, with 0.00 being the minimum reduction (RS 44.94, the downstream section) and 0.78 being the maximum reduction (RS 14481.05, downstream of the CCTRR crossing). For quantitative comparison, WSELs for the initial model plan and Manning's *n* sensitivity plan are listed in Table 4.

Table 4. Tributary 1 HEC-RAS WSELs for initial model plan and Manning's n sensitivity plan

River station (1)	WSEL for initial plan[1] (ft) (2)	WSEL for *n* value sensitivity plan[2] (ft) (3)
21619.32	70.88	70.72
20995.39	70.38	70.24
19836.53	68.01	67.71
17298.25	65.62	65.14
15606.75	64.47	64.02
13406.21	61.42	60.98
11711.45	60.58	60.27
10067.89	59.94	59.74
8551.72	58.05	57.61
6953.50	56.37	55.96
5898.28	55.95	55.59
3505.44	52.90	52.46
1490.47	50.24	49.88

1. Manning's *n* values of 0.06 for the channel and 0.08 for the overbanks were used.

2. Manning's *n* values of 0.05 for the channel and 0.06 for the overbanks were used.

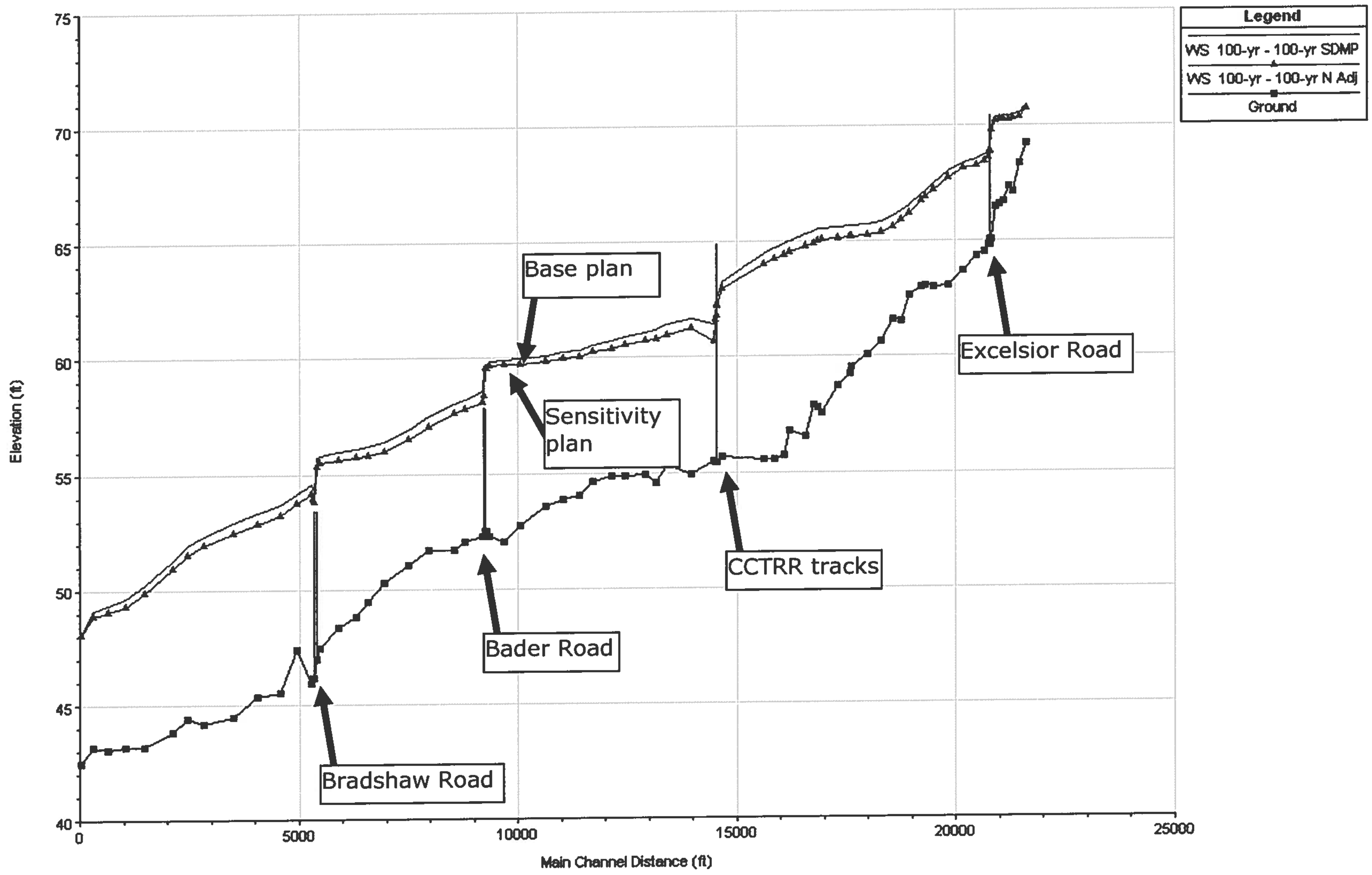


Figure 11. Tributary 1 hydraulic model profile for the p=0.01 event, testing the profile sensitivity to the Manning's n values

Development of unsteady HEC-RAS model

The City requested an unsteady HEC-RAS model for Tributary 1 to determine if running the model in unsteady mode, using runoff hydrographs instead of peak flows, reduces the WSEL in Tributary 1. Per the City's instruction, we used Manning's *n* values of 0.05 for the channel and 0.06 for the overbanks. We used information provided by West Yost and Associates (WYA) to configure the unsteady model plan. Information provided by WYA included a new hydrologic model to reflect the tributary area draining to a detention basin and the hydraulic definition of the detention basin.

We updated the geometry for the unsteady plan to include the Sleepy Hollow Unit No. 2 detention basin based on the definition provided by WYA. The basin is defined in the HEC-RAS model using the elevation versus volume curve, lateral weir, and culvert definitions provided in a spreadsheet by WYA on April 29, 2011.

The unsteady flow file for the model was configured using runoff hydrographs from the updated SacCalc model provided by WYA on May 4, 2011. The SacCalc model revised 2 subsheds so that all of the area that drains to the Sleepy Hollow Unit No. 2 detention basin is covered by a single subshed. The handoff locations where the hydrographs from the SacCalc model enter the HEC-RAS model are listed in Table 5.

The downstream boundary for the model is the stage hydrograph from the Laguna Creek WYA Master Plan model for the p=0.01 event. We removed the Tributary 1 lateral inflow and ran this model to establish the stage hydrograph in Laguna Creek for the p=0.01 event. The stage hydrograph for RS 3460 is used as the downstream boundary condition and has a maximum stage of 47.85 feet. This is the first RS downstream of where Tributary 1 enters Laguna Creek.

We used the unsteady HEC-RAS hydraulic model to compute the WSEL for the p=0.01 event. The maximum WSEL profile for the unsteady HEC-RAS model is shown in Figure 12.

The profile for the unsteady HEC-RAS model compared to the profile for the steady model (both models having Manning's *n* values of 0.05 for the channel and 0.06 for the overbanks) is shown in Figure 13. The unsteady plan produces a slightly lower WSEL than the steady plan for Tributary 1. This is due to the steady model using flows based on the routing methods specified in SacCalc while the unsteady model uses HEC-RAS to route the runoff hydrographs. The unsteady HEC-RAS model routes the flows based on the channel geometry and accounts for attenuation in the reach.

The unsteady HEC-RAS model profile and profile using the approximate FIS flows (using Manning's *n* values of 0.05 for the channel and 0.06 for the overbanks) for the p=0.01 event are shown in Figure 14. The unsteady HEC-RAS model profile is generally higher than the profile computed using the approximate FIS flows. The exceptions to this trend are the 5 most downstream cross sections, where the boundary condition influences the flow profile. For quantitative comparison, WSELs for the steady model plan, unsteady model plan, and the approximate FIS plan are listed in Table 6. All of the WSEL values in the table are based on the models having Manning's *n* values of 0.05 for the channel and 0.06 for the overbanks. The differences between the HEC-RAS computed WSELs are also listed in Table 6. A comparison between the approximate FIS flow WSELs and the unsteady plan WSELs shows higher WSELs for the unsteady plan upstream of Bader Road.

Table 5. Tributary 1 handoff locations where computed hydrographs from SacCalc (Harris DMP model) enter HEC-RAS model

HEC-RAS river station (1)	**SacCalc node (2)**	**Peak hydrograph flow (cfs) (3)**
21619.32	JNC02	449
20481.38	CHA09	500
20174.00	CHA06	273
19499.75	TL1IB	66
18303.64	LT1H	142
16845.68	LT1J	120
16761.29	LT1LB	125
15606.75	LT1K	83
11038.70	CHA15	351
10067.89	LT1M	175
9671.98	CHA18	65
7959.19	LT1R	85
7510.15	CHA20	88
6590.31	LT1T	132
6312.12	LT1V	126
4938.25	LT1W	66
4059.49	LT1X	66
3505.44	LT1X2	81
647.02	LT1Y	83
Basin1	LT1L	169

Table 6. Tributary 1 HEC-RAS WSEL comparisons

River station (1)	**WSEL for steady plan[1](ft) (2)**	**Max WSEL for unsteady plan[1](ft) (3)**	**WSEL using approximate FIS flows[1] (ft) (4)**	**WSEL difference (steady – unsteady)[2] (ft) (5)**	**WSEL difference (unsteady – approximate FIS)[3] (ft) (6)**
21619.32	70.72	70.91	70.60	-0.19	0.31
20995.39	70.24	70.16	69.97	0.08	0.19
19836.53	67.71	67.61	66.74	0.10	0.87
17298.25	65.14	64.56	63.21	0.58	1.35
15606.75	64.02	63.47	62.06	0.55	1.41
13406.21	60.98	60.74	59.95	0.24	0.79
11711.45	60.27	60.03	59.15	0.24	0.88
10067.89	59.74	59.36	58.24	0.38	1.12
8551.72	57.61	57.17	56.96	0.44	0.21
6953.50	55.96	55.53	55.16	0.43	0.37
5898.28	55.59	55.19	54.79	0.40	0.40
3505.44	52.46	51.95	51.82	0.51	0.13
1490.47	49.88	49.55	49.87	0.33	-0.32[4]

1. Manning's *n* values of 0.05 for the channel and 0.06 for the overbanks were used.

2. WSEL difference = (WSEL for steady plan) – (WSEL for unsteady plan).

3. WSEL difference = (WSEL for unsteady plan) – (WSEL using approximate FIS flows).

4. Negative value due to different downstream boundary conditions in model plans.

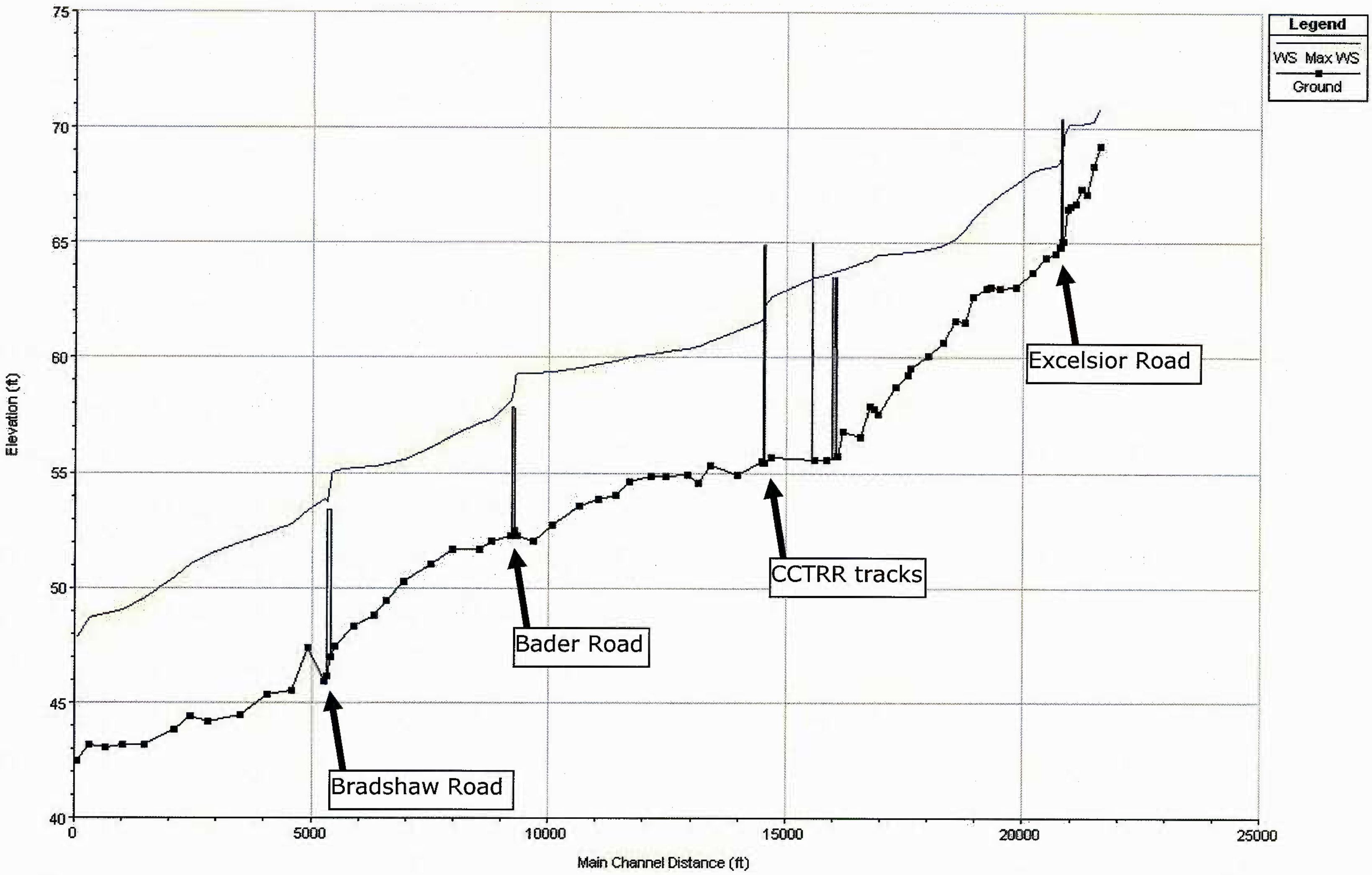


Figure 12. Tributary 1 unsteady HEC-RAS hydraulic model profile for the p=0.01 event

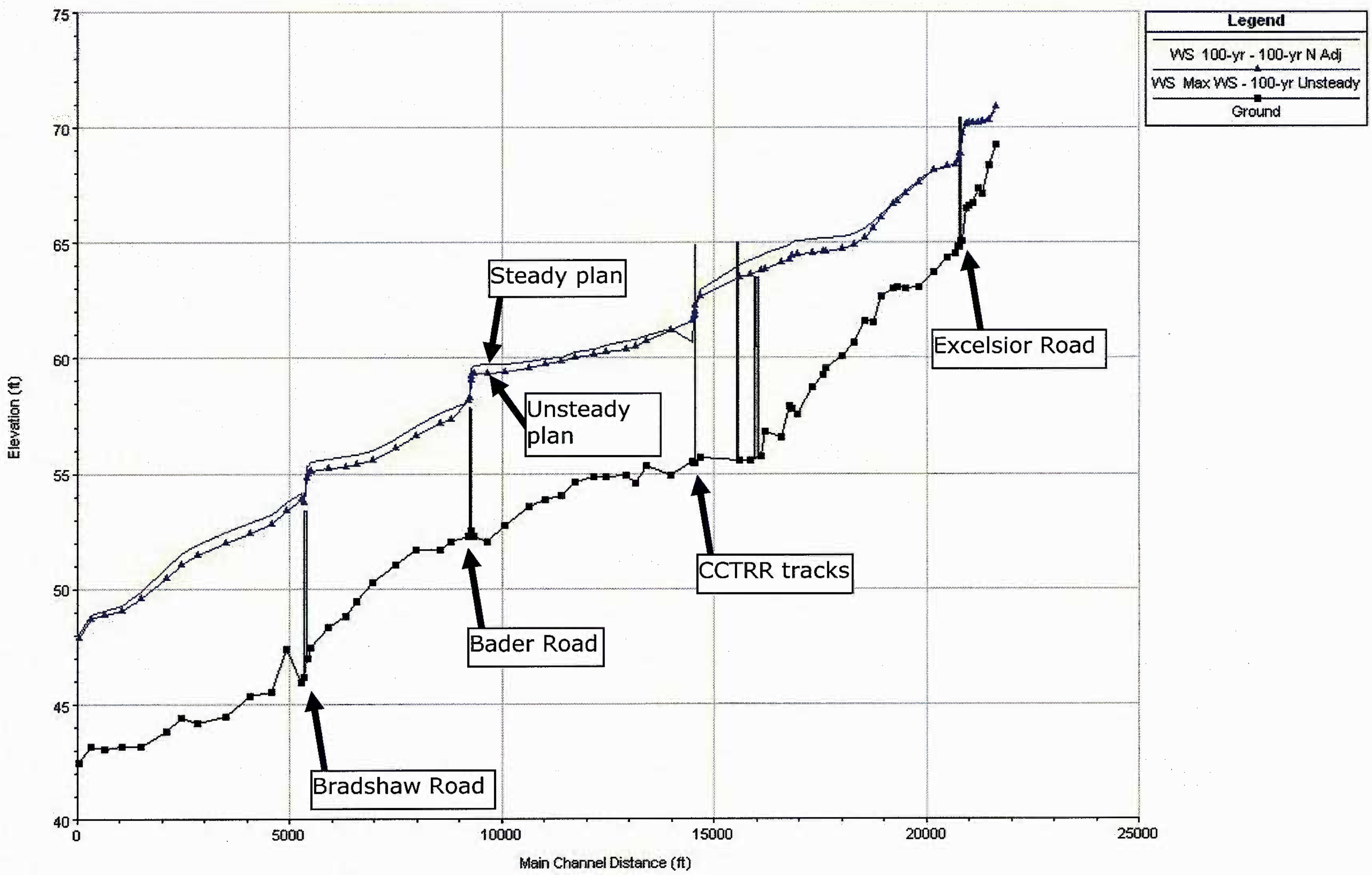
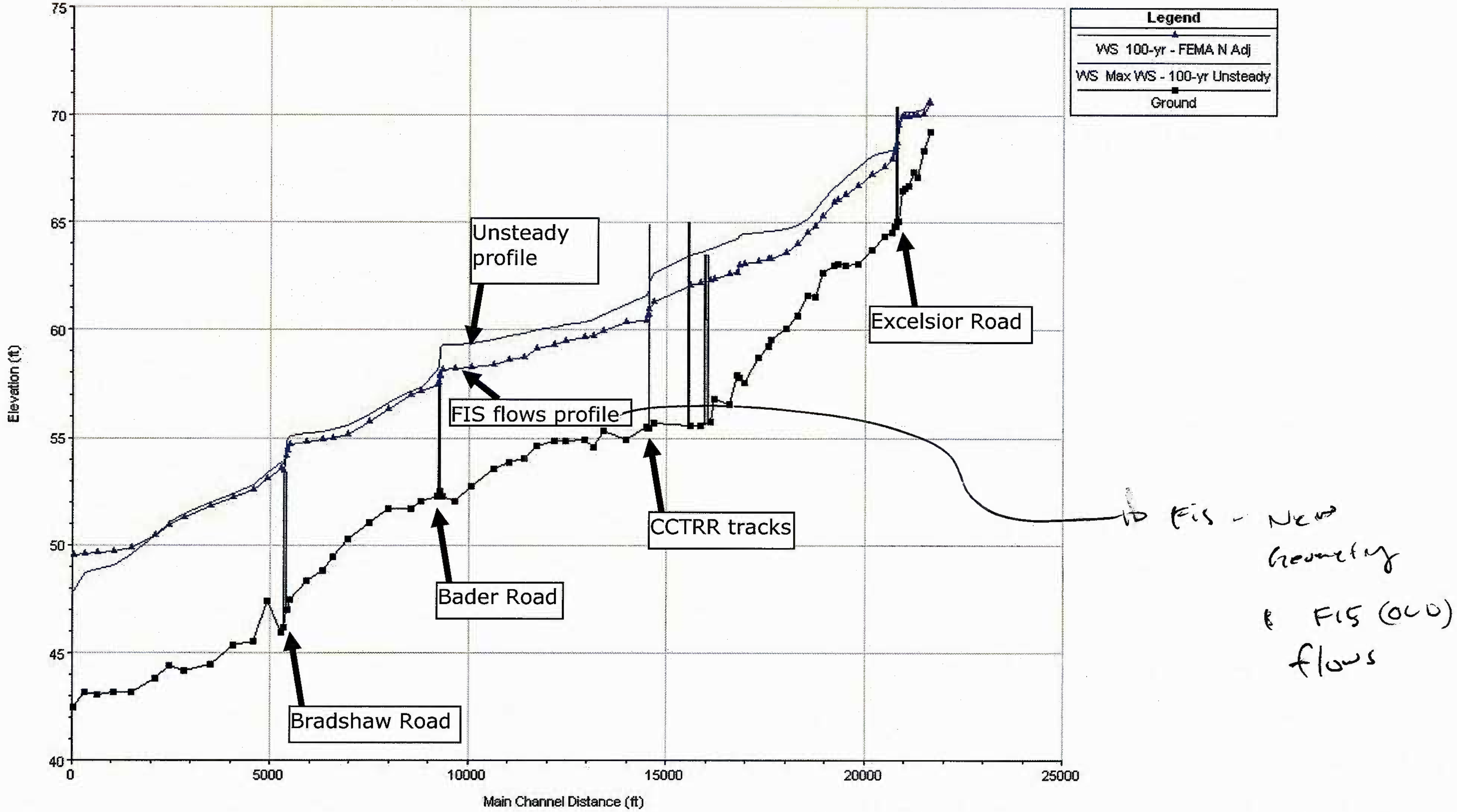


Figure 13. Comparison of unsteady HEC-RAS hydraulic model and steady model profile for the p=0.01 event (both models having Manning's n values of 0.05 for the channel and 0.06 for the overbanks)



to FIS - New Geometry & FIS (OLD) flows

Figure 14. Tributary 1 unsteady HEC-RAS hydraulic model profile compared to the approximate FEMA FIS flows profile for the p=0.01 event (both models having Manning's n values of 0.05 for the channel and 0.06 for the overbanks)

Results

Based on our analysis, we found the following:

- The CVFED LiDAR data capture the low flow channel in the area where survey data are available for comparison.
- The bridge definitions in the 2005 Laguna model (taken from the County model for Tributary 1) were approximate and the deck elevations did not reflect the elevations from the LiDAR data. Bridge decks were defined from the LiDAR data and deck thicknesses were estimated.
- The hydraulic profile for the p=0.01 event using the Harris DMP flows show bridge overtopping at Bader and Bradshaw Road.
- The WSELs computed for the p=0.01 event using the Harris DMP flows are higher than the WSELs computed using approximate FIS flows.
- Reducing the Manning's *n* values in the channel from 0.06 to 0.05 and in the overbank from 0.08 to 0.06 reduces the WSEL in Tributary 1. The average WSEL reduction is 0.35 feet.
- The unsteady model plan for Tributary 1 produces WSELs which are slightly lower than the WSELs produced using the steady flow plan.
- The unsteady model plan for Tributary 1 produces WSELs which are generally higher than the WSELs produced using the approximate FIS flows. The exceptions to this trend are the 5 most downstream cross sections, where the boundary condition influences the flow profile.

We used the computed WSELs to develop floodplain limits for the p=0.01 event. The floodplain limits were developed from within HEC-GeoRAS using updated cross-section cut lines, computed WSELs, and topographic data. We delineated 2 floodplains: the floodplain for the unsteady model plan and the floodplain for the approximate FIS flows. The floodplain for the unsteady model provides the best indication of existing conditions, using available hydrology and the updated hydraulic model. The floodplain for the approximate FIS flows provides a benchmark for comparison to the unsteady model plan. The floodplain for the unsteady model plan is shown in Figure 15. The floodplain for the approximate FIS flows is shown in Figure 16. Both floodplains and the FEMA Q3 data for the area bounded by the CCTRR and Excelsior Road are shown in Figure 17.

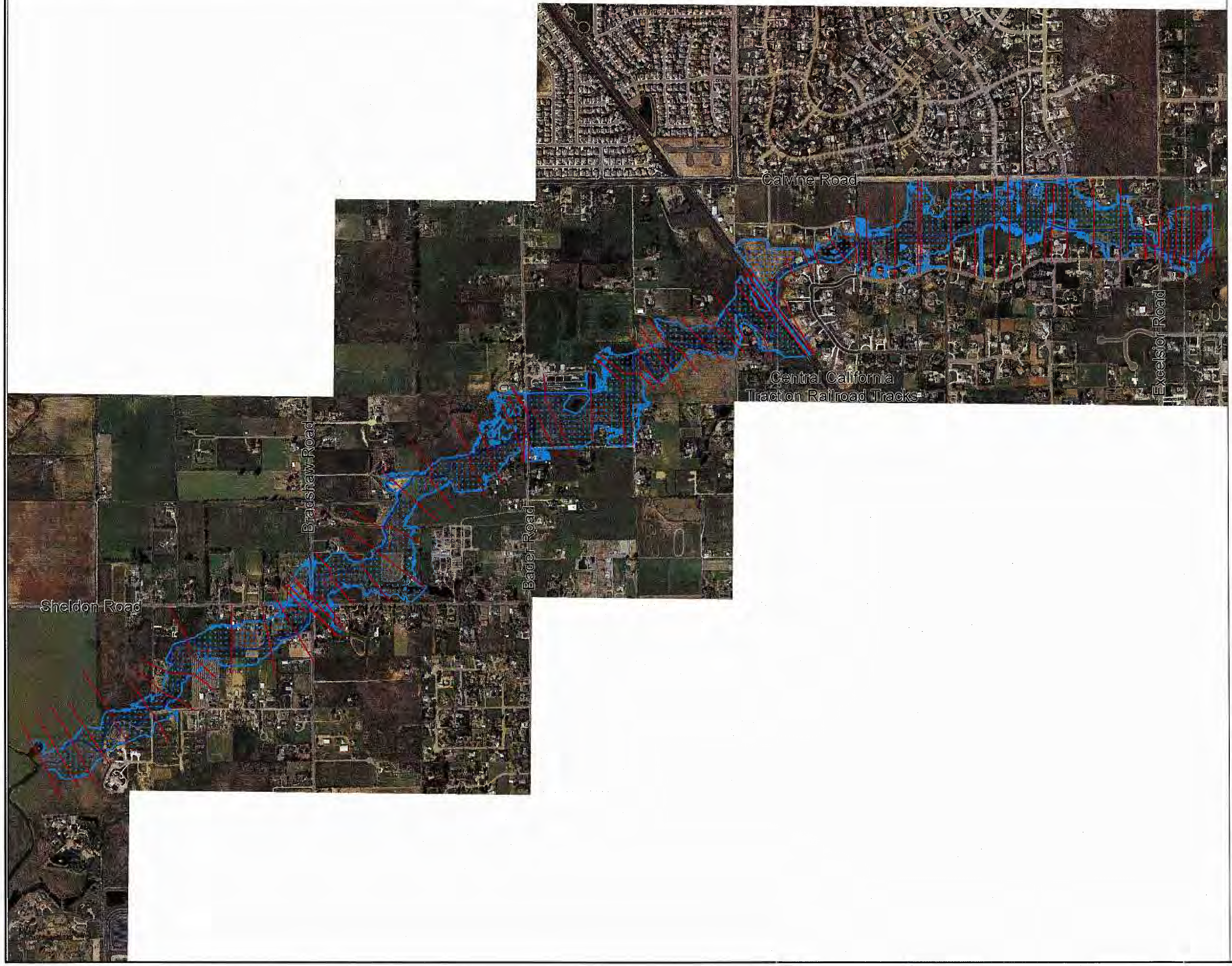


Figure 15

Tributary 1

Approximate 100-yr floodplain based on unsteady HEC-RAS model plan

NOTES:

Vertical Datum: NGVD29

Floodplain based on model results from 2011 Tributary 1 HEC-RAS model.

Model and floodplain based on LiDAR topographic information from the California Department of Water Resources (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program.

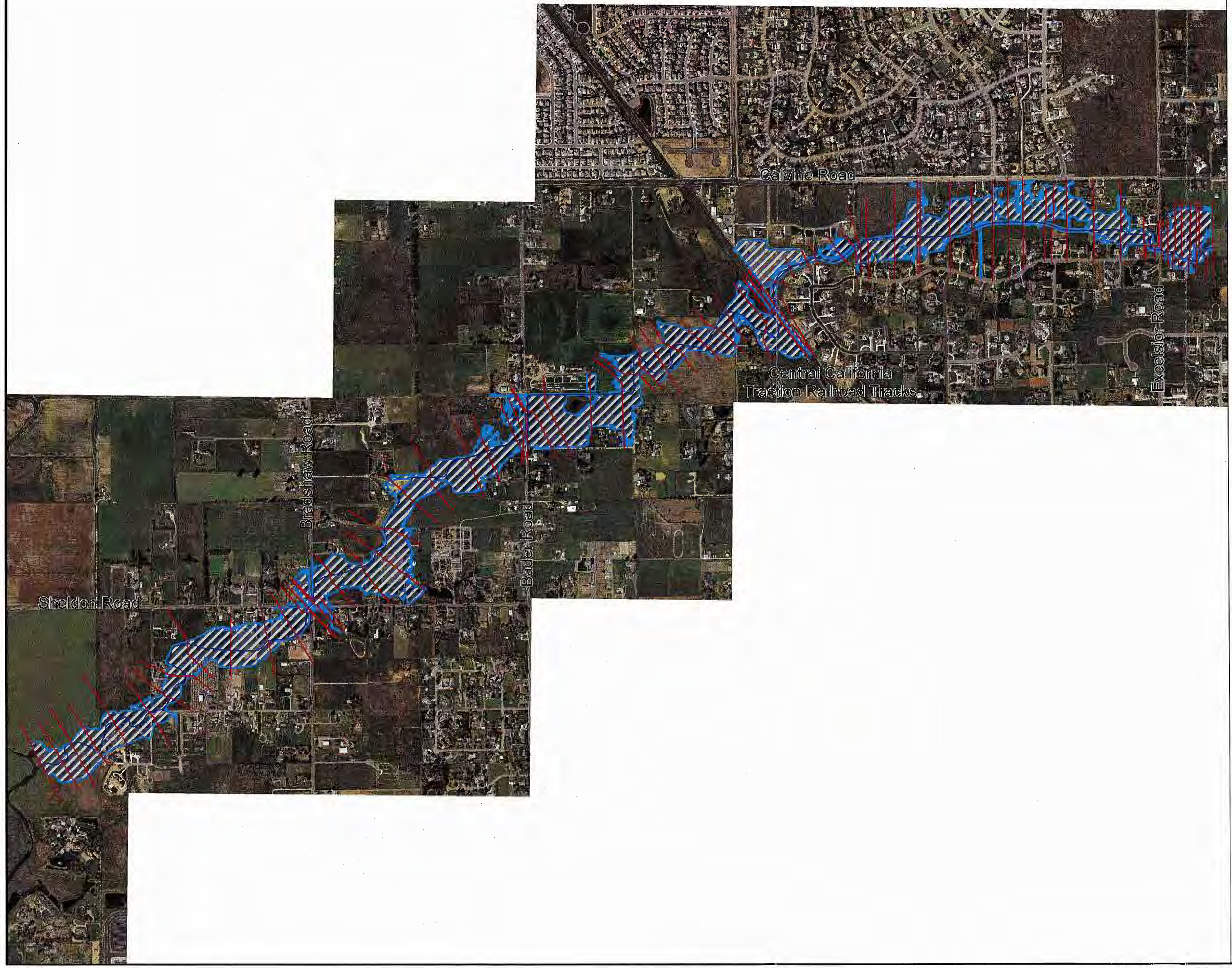
LEGEND:

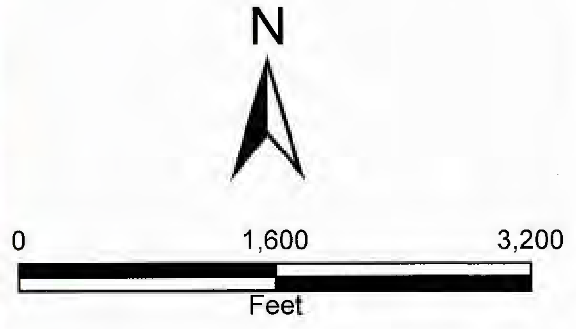
—	HEC-RAS cross sections
—	Tributary 1 centerline
▨	Floodplain from unsteady model plan

Figure 16

Tributary 1

Approximate 100-yr floodplain based on approximate FIS flows





NOTES:

Vertical Datum: NGVD29

Floodplain based on model results from 2011 Tributary 1 HEC-RAS model.

Model and floodplain based on LiDAR topographic information from the California Department of Water Resources (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program.

LEGEND:

— HEC-RAS cross sections

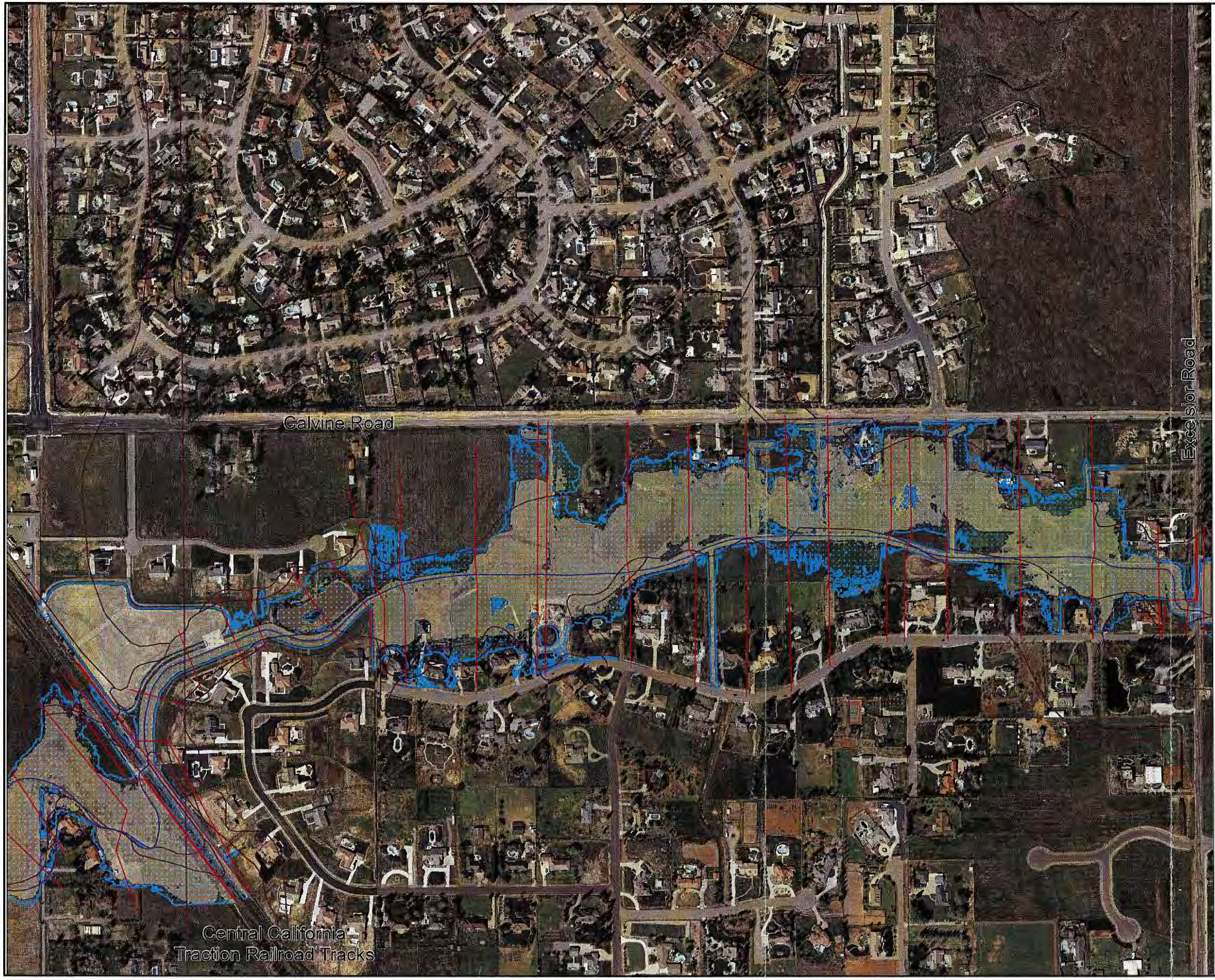
— Tributary 1 centerline

▨ Floodplain from approximate FIS flows

Figure 17

Tributary 1

Approximate 100-yr floodplains



N

0 575 1,150
Feet

NOTES:

Vertical Datum: NGVD29

Floodplain based on model results from 2011 Tributary 1 HEC-RAS model.

Model and floodplain based on LiDAR topographic information from the California Department of Water Resources (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program.

LEGEND:

— HEC-RAS cross sections

— Tributary 1 centerline

☐ FEMA Q3 data

🟨 Floodplain from approximate FIS flows

▨ Floodplain from unsteady model plan

CHAPTER 7. WHITEHOUSE CREEK

WATERSHED DESCRIPTION

Whitehouse Creek drains a watershed with an area of approximately 1,030 acres and is a major tributary to Laguna Creek. The watershed is approximately bounded by Highway 99 to the west, Sheldon Road to the north, Waterman Road to the east, and Bond Road to the south (See Figure 7-1). The creek begins northeast of the intersection of Bond and Waterman Roads and flows to the west for approximately 1.5 miles then turns south and continues for approximately 0.5 miles before joining Laguna Creek 1,200 feet upstream of Highway 99.

EVALUATION OF WHITEHOUSE CREEK

The evaluation of Whitehouse Creek was included with the evaluation of Laguna Creek as described in Chapter 4.

EVALUATION OF EXISTING PIPELINES

One existing pipeline in the Whitehouse Creek watershed was evaluated. According to City staff, excessive street flooding has been reported in the area near the intersection of North Camden Way and Springhurst Drive. Figure 7-2 shows the sizes and locations of the existing pipes that were evaluated in this area.

Hydrologic Analysis of Existing Pipeline

A SacCalc model was prepared to calculate the 2-year, 10-year, and 100-year flows into the one existing pipe system. The watershed served by the existing pipeline is completely developed; therefore, flows were calculated for buildout conditions only.

Figure 7-2 presents the subshed boundaries used for the flow calculations for the existing pipeline. Table 7-1 presents the key hydrologic parameters for these subsheds. Table 7-2 presents the calculated peak flows from each subshed for the three storm events.

Hydraulic Analysis of Existing Pipeline

A hydraulic model of the existing pipe system was created using XPSWMM. Calculated water surface elevations for the 2-year, 10-year, and 100-year storm events are summarized on Table 7-3. Calculated peak flows are presented on Table 7-4. As Table 7-3 shows, the City’s performance criterion for the 10-year storm is not met at two locations along the pipeline. These results are not surprising since this area was reported to be an area with frequent street flooding.

Improvements to Existing Pipelines

Pipe improvements are necessary to eliminate the excessive street flooding along the existing pipeline. Approximately 750 feet of pipeline in North Camden Way and Springhurst Drive will need to be upsized to reduce the street flooding to acceptable levels. The required pipe improvements are shown on Figure 7-3.

Table 7-1. Hydrologic Parameters for Existing Pipeline WHC1

Subshed	Area, acres	Mean Elev., ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land Use, acres and Percent Impervious		Average % Imp.
						Resd., 4-6 du/ac	Park	
						40%	5%	
WHC110	19.6	37	1,330	606	0.0015	18.3	1.4	38
WHC120	7.9	38	520	288	0.0077	7.9	0.0	40
WHC130	19.9	39	1,400	699	0.0014	19.9	0.0	40

Table 7-2. Calculated Subshed Flows for Existing Pipeline WHC1

Subshed	Area, acres	Buildout Condition Flows, cfs		
		2-Year	10-Year	100-Year
WHC110	19.6	13	24	35
WHC120	7.9	8	15	23
WHC130	19.9	13	25	36

Table 7-3. Calculated Water Surface Elevations for Existing Pipeline WHC1 (NGVD29)

Node Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline No. WHC1							
WHC100	N/A	38.7	33.5	33.5	33.7	—	—
WHC110	37.1	38.3	34.3	35.2	36.3	—	—
WHC120	37.0	38.2	36.5	37.2	37.9	Yes	—
WHC130	38.4	39.1	37.6	38.4	38.6	Yes	—

Table 7-4. Calculated Peak Flows for Existing Pipeline WHC1

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline No. WHC1						
RWHC110	WHC110	WHC100	Pipe	25	35	43
RWHC120	WHC120	WHC110	Pipe	13	14	15
RWHC130	WHC130	WHC120	Pipe	13	15	14
OLRWHC120	WHC120	WHC110	Overland	0	0	3
OLRWHC130	WHC130	WHC120	Overland	0	10	23

EVALUATION OF FUTURE PIPELINES AND STORMWATER QUALITY FACILITIES

Future trunk pipelines are expected to be required in the upper portion of the Whitehouse Creek watershed to the east of the Union Pacific Railroad and north of Bond Road. This area includes significant undeveloped areas that are anticipated for Rural Residential, Low Density Residential, Commercial, and Office development in the future. Potential future trunk pipeline alignments were estimated and sized. In addition to the pipeline improvements, facilities will be required to provide stormwater quality treatment for runoff from development areas. It was assumed that stormwater quality treatment would be provided with dry detention basins located near the downstream end of each trunk pipeline.

Because the future development patterns and roadway layouts in this area are unknown, it is anticipated that the actual layout of the trunk pipe system will be different from that proposed with this SDMP. However, the trunk pipelines and stormwater quality facilities defined in this SDMP are adequate for the development of a Capital Improvement Plan in the future.

Hydrologic Analysis of Future Pipelines

The future trunk pipes were analyzed for the 10-year storm only. Evaluation of the 2-year storm is unnecessary since the water surface will be contained within the pipe. The allowed maximum water surface elevations for the 100-year storm will be dependent on the layout and grading of the future streets and overland release points, which are unknown at this time.

SacCalc models were prepared to calculate the buildout 10-year flows into the future trunk pipe systems. Figure 7-4 presents the subshed boundaries within the anticipated development areas used for the flow calculations. Table 7-5 presents the key hydrologic parameters for each subshed for buildout conditions. Table 7-6 presents the calculated 10-year peak flows from each subshed.

Table 7-5. Hydrologic Parameters for Future Pipeline Models WHC2-WHC6

Subshed	Area, acres	Mean Elev., ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land Use, acres and Percent Impervious						Average % Imp.	Stormwater Quality Detention Volume, ac-ft
						Comm., Office	Ext Indust., Resd., 8-10 du/ac	Resd, 4-6 du/ac	Resd, 3-4 du/ac	Rural Res.	Park		
						90%	60%	40%	30%	10%	5%		
WHC210	9.5	41	660	300	0.0030	5.5	4.0					77	n/a
WHC220	29.8	45	1,270	688	0.0047	14.2				15.6		48	n/a
WHC230	64.7	53	3,610	1,642	0.0061	2.0				62.7		12	n/a
WHC310	33.2	48	1,180	608	0.0068					33.2		10	0.50
WHC320	21.9	55	1,520	841	0.0118					21.9		10	0.33
WHC410	6.3	58	930	465	0.0075			6.3				40	0.16
WHC420	25.2	62	2,030	1,015	0.0049			25.2				40	0.63
WHC510	13.4	60	1,670	835	0.0072		3.0	10.4				44	0.36
WHC520	25.5	60	1,970	985	0.0061		10.4	15.1				48	0.74
WHC610	4.8	50	490	245	0.0020			2.1			2.7	20	0.07
WHC620	27.8	55	1,660	830	0.0060	8.0	13.0	4.9			1.9	61	1.04

Table 7-6. Calculated Subshed Flows for Future Pipelines WHC2-WHC6

Subshed	Area, acres	10-Year Flow, cfs
WHC210	9.5	25
WHC220	29.8	47
WHC230	64.7	47
WHC310	33.2	71
WHC320	21.9	12
WHC410	6.3	10
WHC420	25.2	31
WHC510	13.4	18
WHC520	25.5	76
WHC610	4.8	8
WHC620	27.8	39

Hydraulic Analysis of Future Pipelines

Hydraulic models of five future trunk pipe systems were created using XPSWMM. Pipelines were sized to provide a 10-year water surface elevation that is a minimum of 0.5 feet below the anticipated gutter elevations in the future streets. The estimated pipe lengths, sizes, and slopes are presented on Table 7-7 along with the calculated water surface elevations for the 10-year storm event. The pipe alignments and sizes are shown on Figure 7-4.

Analysis of Stormwater Quality Facilities

Stormwater quality treatment was assumed to be provided with dry detention basins near the downstream end of each trunk pipeline with the exception of Pipeline WHC2. The watershed for Pipeline WHC2 is mostly developed except for some relatively small commercial properties adjacent to Elk Grove Florin Road. Because of the relatively small sizes of the undeveloped properties, it is assumed that these properties will provide on-site stormwater quality facilities that will not be part of the future Capital Improvement Plan. The stormwater quality storage volumes required for the other four trunk pipe watersheds were estimated based on the method presented in Appendix E of the Stormwter Quality Design Manual for the Sacramento and south Placer Regions, May 2007. The storage volumes required for each subshed are presented in the last column of Table 7-5. The volumes required for each subshed were added together to determine the total storage volume required at the downstream end of the pipeline. Table 7-8 summarizes the total water quality storage volume required within each pipe shed.

Table 7-7. Design Data and Water Surface Elevations for Future Pipelines WHC2-WHC6

Upstream Node	Downstream Node	Diameter, in	Length	Upstream Invert Elevation, ft	Downstream Invert Elevation, ft	Design Slope, ft/ft	10-Year Peak Flow, cfs	Est. Upstream Ground Elev., ft	Upstream 10-Year hgl, ft
Future Pipeline WHC2									
WHC210	WHC200	48	447	32.8	32.2	0.0013	77	41.0	38.1
WHC220	WHC210	48	930	34.1	32.8	0.0014	64	41.0	40.2
Future Pipeline WHC3									
WHC310	WHC300	54	510	36.4	36.0	0.0008	66	43.0	41.4
WHC320	WHC310	36	636	36.9	36.4	0.0008	25	44.0	42.6
Future Pipeline WHC4									
WHC410	WHC400	42	280	45.9	45.7	0.0007	38	50.6	49.5
WHC420	WHC410	36	470	50.0	45.9	0.0087	31	55.0	51.8
Future Pipeline WHC5									
WHC510	WHC500	48	355	46.0	45.7	0.0008	85	52.0	50.6
WHC520	WHC510	48	490	46.4	46.0	0.0008	69	53.0	52.1
Future Pipeline WHC6									
WHC610	WHC600	48	360	46.0	45.7	0.0008	46	51.0	49.5
WHC620	WHC610	42	330	46.3	46.0	0.0009	39	51.0	49.6

Table 7-8. Summary of Required Stormwater Quality Detention Storage Volumes

Pipeline/Watershed	Storage Volume , ac-ft
WHC3	0.83
WHC4	0.79
WHC5	1.10
WHC6	1.11

PRELIMINARY IMPROVEMENTS

As discussed above, drainage improvements to eliminate existing deficiencies and serve anticipated development are required in the Whitehouse Creek watershed. These improvements are summarized below and on Table 7-9. These improvements are considered preliminary. They are adequate for development of a Capital Improvement Plan, but the ultimate improvements will be defined from a more detailed design SDMP and could vary from those recommended in this SDMP.

- Upsizing of Existing Pipeline WHC1 (See Figure 7-3).
- Construction of five new pipelines to serve future development in the area east of the UPRR and north of Bond Road (See Figure 7-4).
- Construction of four stormwater quality detention basins to serve the watersheds drained by the new trunk lines. The total required detention storage volume is estimated to be 3.83 acre-feet.

Table 7-9. Preliminary Improvements for Whitehouse Creek

Item	Quantity	Unit
Existing Pipe Upgrades		
30" RCP	180	LF
36" RCP	569	LF
Manholes	3	EA
Outfall Structures	1	EA
New Pipelines		
36" RCP	1,106	LF
42" RCP	610	LF
48" RCP	2,585	LF
54" RCP	510	LF
Manholes	14	EA
Outfall Structures	5	EA
Stormwater Quality Detention Basins		
SWQ Detention for Sheds WHC3-WHCN6	15,000	CY

Other Whitehouse Creek Studies

St. Anthony Court Flooding Problem

A flooding problem in the upper portion of the Whitehouse Creek has been reported on St. Anthony Court, which is a cul-de-sac off of Sheldon Road, just west of Waterman Road. A resident that lives at the south end of the cul-de-sac has reported flooding in the past, most recently in December 2005.

St. Anthony Court is drained by roadside ditches that convey runoff from north to south. At the sound end of the cul-de-sac, runoff is drained to a culvert that conveys runoff under a low berm and discharges into a drainage swale that continues south and west though an open field. This swale eventually joins Whitehouse Creek near Campbell Road. The total watershed area draining to the south end of the court is approximately 54 acres.

City staff have discussed the problem with the resident and have performed a site review of the area. City staff believe that the problem could potentially be relieved with improvements to the drainage system downstream from the cul-de-sac. Improvements could consist of enlarging the culvert and the drainage swale downstream of the culvert. However, these facilities are located on private property and the swale appears to contain wetland habitat. Because of these constraints, the City requested that West Yost evaluate an alternative to divert flow away from the problem area.

West Yost evaluated construction of a pipeline in Sheldon Road beginning at the intersection of St. Anthony Court and Sheldon Road and continuing east for approximately 2,400 feet to Laguna Creek. The pipeline would divert all runoff from the tributary area north of Sheldon Road for storms events up to the 100-year storm. This would reduce the tributary area draining to the south end of St. Anthony Court from 54 acres to approximately 35 acres, a 35 percent reduction. The specific benefits to the St. Anthony Court area were not quantified as a part of the study because the alternative is too costly to be feasible. The alternative would require approximately 1,690 feet of 36-inch pipe, 740 feet of 42-inch pipe, 6 manholes, and an outfall structure. The cost to implement the alternative was estimated to be roughly $1 million. Because of the high cost of this alternative, it is recommended that any future efforts to solve this problem be directed toward improvements at the downstream end of the system.

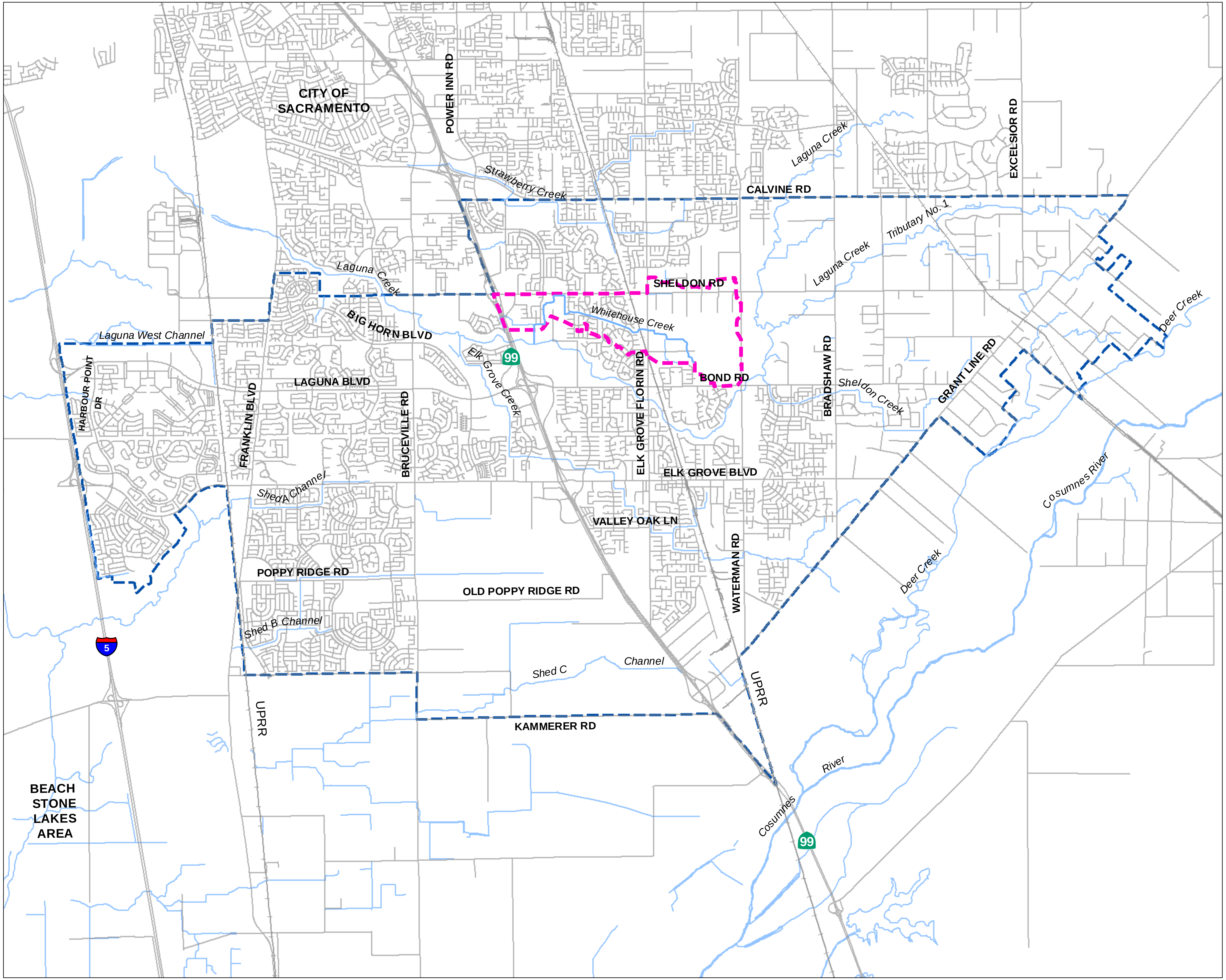


FIGURE 7-1

City of Elk Grove Storm Drainage Master Plan Volume II

WHITEHOUSE CREEK LOCATION MAP

N

0 0.5 1 2

Scale in Miles

NOTES:

LEGEND:

City Limit

Whitehouse Creek Watershed

WEST YOST ASSOCIATES
Consulting Engineers

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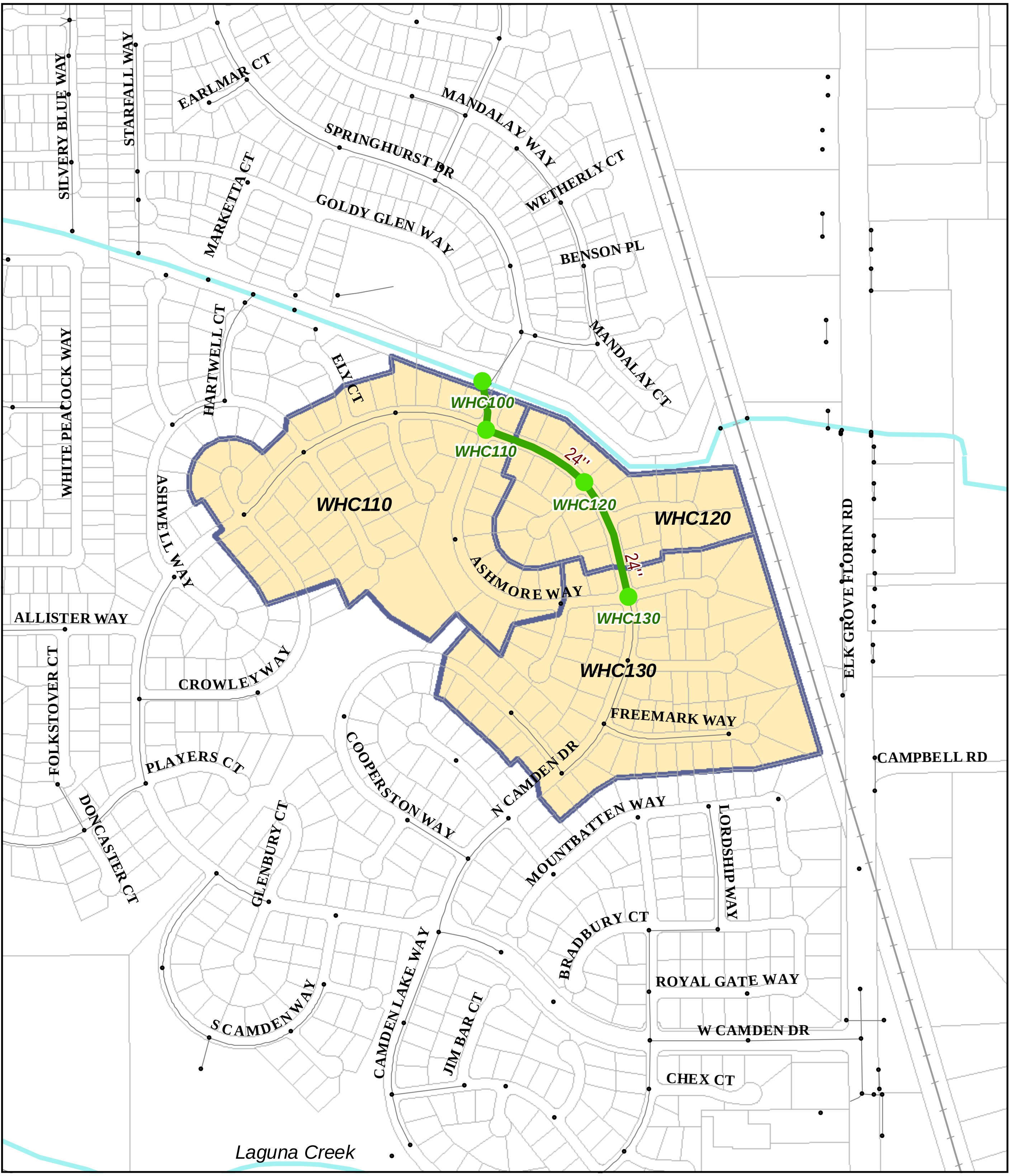

LEGEND:

WHC120

Modeled Pipeline and Node

▨ Existing Pipeline WHC1 Subshed

N

0 250 500

Scale in Feet

FIGURE 7-2
City of Elk Grove
Storm Drainage Master Plan
Volume II

WHITEHOUSE CREEK
EXISTING PIPELINE WHC1
SUBSHEDS AND MODELED FACILITIES

WEST YOST
ASSOCIATES
Consulting Engineers

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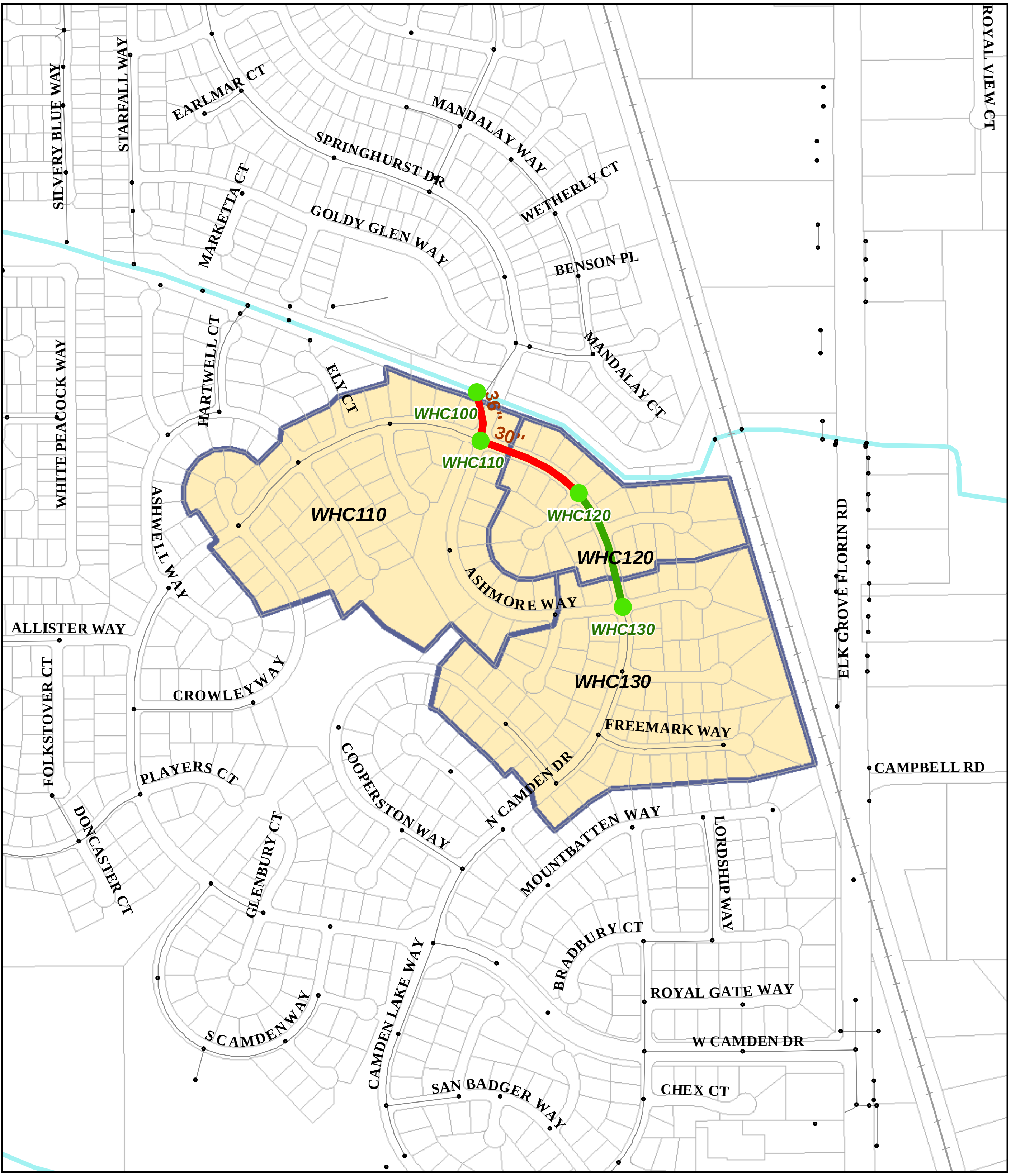


LEGEND:

WHC120 🟩 Modeled Pipeline and Node

▨ Existing Pipeline WHC1 Subshed

36" 🟥 Upsized Pipe

N

0 250 500

Scale in Feet

FIGURE 7-3
City of Elk Grove
Storm Drainage Master Plan
Volume II

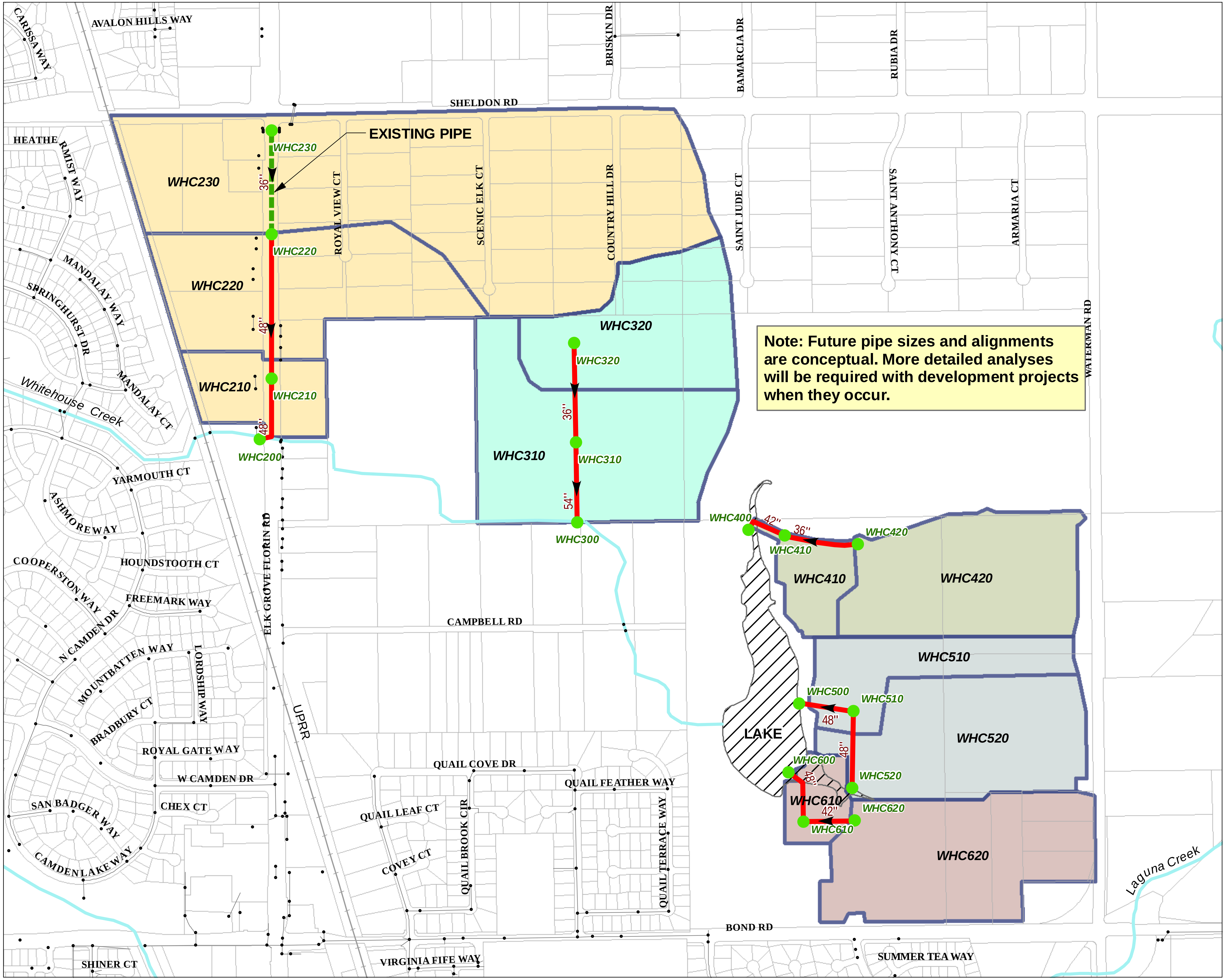
WHITEHOUSE CREEK
EXISTING PIPELINE WHC1
RECOMMENDED IMPROVEMENTS

WEST YOST ASSOCIATES
Consulting Engineers

FIGURE 7-4

City of Elk Grove
Storm Drainage Master Plan
Volume II

WHITEHOUSE CREEK
FUTURE TRUNK PIPELINES WHC2 - WHC6
SUBSHEDS AND FACILITIES



Note: Future pipe sizes and alignments are conceptual. More detailed analyses will be required with development projects when they occur.

LEGEND:

WHC220

- 🟥● Modeled Pipeline and Node
- ▭ Future Pipeline WHC2 Subshed
- ▭ Future Pipeline WHC3 Subshed
- ▭ Future Pipeline WHC4 Subshed
- ▭ Future Pipeline WHC5 Subshed
- ▭ Future Pipeline WHC6 Subshed

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 8. STRAWBERRY CREEK

WATERSHED DESCRIPTION

Strawberry Creek is located in the northern portion of the City (See Figure 8-1) and flows generally from east to west, beginning in the County of Sacramento, flowing through the City of Elk Grove, and then into the City of Sacramento before joining Unionhouse Creek. The watershed covers approximately 3,700 acres with elevations ranging from elevation 30 feet in the west to 72 feet in the east. There are two upper branches, the North Fork and the Middle Branch. The North Fork begins in Sacramento County and flows into the City at Calvine Road, just west of the Union Pacific Railroad (UPRR). The North Branch travels south along the UPRR for approximately 650 feet before joining the Middle Branch. The Middle Branch begins south of Calvine Road and west of Waterman Road. The channel turns south for a short distance before turning west and generally continuing through the City in a westerly direction before leaving the City and crossing Calvine Road about 2,300 feet east of Highway 99. See Figures 8-1 and 8-2 for maps of the watershed.

EVALUATION OF STRAWBERRY CREEK SYSTEM

Hydrologic Analysis for Strawberry Creek

For the hydrologic analysis, a watershed model was used to transform design rainfall (of specified probability) over a given area to runoff hydrographs. The hydrologic data was developed using the most recent topographic data, storm drain system maps, land use designations, soil maps and aerial photography information. The information developed from these sources was input into SacCalc for development of existing and buildout storm runoff conditions. See Figure 8-2 for the subshed map. The SacCalc model provided the inflow hydrographs needed for the channel hydraulic model, which is used to route the flows through the Strawberry Creek system.

Hydrology for Existing and Buildout Conditions

When beginning the hydrologic analysis for Strawberry Creek watershed, the existing land use was compared to the buildout land use and found that the land use is unchanged for the buildout conditions. For this reason, only one hydrologic model was prepared for Strawberry Creek.

The buildout conditions hydrologic SacCalc model was developed to calculate the flows for the 2-year, 10-year, and 100-year storm events. Figure 8-3 shows the configuration of the SacCalc model. The hydrologic data for subsheds SC-10 through SC-20 were taken from Sacramento County’s Strawberry and Jacinto Creeks Drainage Master Plan Draft Report dated July 1993. Data for subsheds STR-01 through STR-13 were developed using 2-foot contour topographic mapping, storm drain facility maps, City General Plan land use data, and detention basin data from the City. The goal of the hydrologic analysis was to determine the flow hydrographs to be used in the unsteady hydraulic analysis of Strawberry Creek to establish maximum water surface profiles for the three flood events. The hydrologic data for each subshed is shown in Table 8-1.

Table 8-1. Hydrologic Parameters for Strawberry Creek

Subshed	Area (acres)	Mean Elevation (ft, NGVD29)	Basin Length (ft)	Basin Centroid Length (ft)	Basin Slope (ft/ft)	Land Use (acres) and Percent Impervious															Avg % Imp
						Highway, Parking	Comm./ Office	Ind.	HDR	Mobile	MDR	Resd, 8 to 10 du/ac	Resd, 6 to 8 du/ac	Resd, 4 to 6 du/ac	Resd, 2 to 3 du/ac	Resd, .05 to 1 du/ac	Resd, .02 to .05 du/ac	< .02 du/ac, recreation	OpenSpace, grassland/ag	Vacant	
						95%	90%	85%	80%	75%	70%	60%	50%	40%	25%	15%	10%	5%	2%	1%	
SC-10	286.6	72	7830	5998	0.0058	55.1	2.1		154.0		1.1			0.2			0.5	21.0	17.2	35.4	63
SC-11	158.6	45				Specified Lag Time = 26.6 minutes, Loss Computed, % Imp. Computed															74
SC-13A	393.6	40	3611	3211	0.0060	61.5	12.8		179.8		9.6		0.1	0.3		7.3	43.9	0.2	48.2	30.0	58
STR-01	268.4	72	5280	2500	0.0060	56.6	0.4		155.8		0.8		4.2			0.0	0.1		47.5	3.1	68
STR-03	39.9	54	1272	730	0.0070	3.3	7.2		3.2							3.9	15.1			7.2	36
STR-02	335.0	64	4900	2300	0.0070	45.8			122.4		2.1		1.7	1.5		3.0	67.4	15.3	31.4	44.3	46
STR-04	55.8	48	2700	1790	0.0020	5.0	12.9	6.9	0.3								12.5	5.5	5.5	7.3	43
STR-05	36.8	44	1870	1100	0.0050	5.2			11.6		0.3		0.5				3.0		16.2	0.0	42
STR-06	215.3	50	5235	2500	0.0060	28.4	9.3		74.8		1.0	0.1	0.2		0.2	24.3	50.9	2.8	13.6	9.8	49
STR-07	96.5	40	3500	1640	0.0020	13.8	14.0	5.5	17.0							3.5	21.1	9.6	3.1	9.0	49
STR-08	101.9	38	2035	1225	0.0040	18.0	0.1		47.9		0.1		0.1			0.9	4.5		28.1	2.3	56
STR-09	315.5	40	4700	3000	0.0010	55.1		0.1	176.7		0.3	0.2	1.5	0.2		1.5	4.7	17.9	54.7	2.6	63
STR-10	127.7	38	3500	1900	0.0020	24.7	1.3		73.6		6.1		0.6	1.2		3.3	2.7	1.7	6.3	6.2	70
STR-11	38.3	36	2000	1300	0.0020	7.0			26.9										4.0	0.4	74
STR-12	213.2	36	6000	3200	0.0010	33.5			70.0		5.2		1.7			0.8	2.1		16.4	83.6	44
STR-13	136.2	34	4000	2600	0.0010	20.0	12.3		18.1				7.9						9.0	68.9	36
SC-17	127.8	37				Specified Lag Time = 63.6 minutes, Loss = .08 in/hr															44
SC-18	305.3	32				Specified Lag Time = 60.0 minutes, Loss = .08 in/hr															35
SC-19	312.3	30	4140	2202	0.0008	Specified Basin "n" = 0.115, Loss = .08 in/hr															22
SC-20	133.4	32	3300	2200	0.0020	40.1	28.5	6.4	35.2		0.1								11.0	12.1	73

Table 8-2 lists the peak flows from the hydrographs produced from the SacCalc model. These are peak flows from the subsheds, not the routed peak flows within the creek. See the discussion on the hydraulics analysis of Strawberry Creek for a table showing the peak flows in the creek.

Table 8-2. Peak Flows from SacCalc for Strawberry Creek

SacCalc Node	HEC-RAS Creek Station	Location Where Flows Enter Creek	2-year peak flow, cfs	10-year peak flow, cfs	100-year peak flow, cfs
C-ST02	4.176	Calvine Road	110	229	369
STR-02	3.967	Brown Rd	108	229	388
C-ST05	3.57	Upstream from Elk Grove Florin Road	32	67	113
STR-08	3.142	Downstream from Black Kite Road	43	92	156
STR-07	2.871	Upstream from UPRR	34	71	119
RCSC13	2.836	Downstream from UPRR	248	492	813
RST06A	2.52	Upstream from Sheldon North	71	148	249
STR-09	2.203	Downstream from Sheldon North	98	196	311
STR-11	2.044	Upstream from DB 1B	18	39	63
STR-10	1.804	Upstream of DB 1B	52	108	173
STR-12	1.721	At DB 1B	52	100	162
STR-13	1.304	Upstream of Calvine Rd near High School	34	67	110
CSC18	1.194	Downstream of Calvine Rd near High School	37	73	326
SC-20	0.815	Upstream of Power Inn Rd near High School	52	107	173
SC-19	0.512 to 0.776	Between Power Inn Rd and Hwy 99	60	120	204

Hydraulic Analysis of Strawberry Creek

An unsteady-flow hydraulic model was prepared for Strawberry Creek using HEC-RAS. The unsteady-flow model, using input runoff hydrographs, considers backwater, ponding, and channel storage effects as the flow is routed. Also, the unsteady-flow model was used to evaluate the performance of the detention basins in the watershed.

The hydraulic data for this project was developed using a combination of existing HEC-2 and HEC-RAS models from previous studies, detention basin information and topographic mapping provided by the City, and field verification of several bridges and culverts. Once the geometry files were established, flow data from the SacCalc models was input into the hydraulic model.

There are two upper branches of Strawberry Creek, the North Fork and the Middle Branch. The North Fork is primarily in the County of Sacramento and was not included in this hydraulic analysis. The Middle Branch begins south of Calvine and west of Waterman and flows together with the North Branch just downstream and west of the UPRR, south of Calvine Road. The combined North Fork and Middle Branch form the main branch of Strawberry Creek. A schematic of the Strawberry Creek HEC-RAS model is provided in Figure 8-4. More detailed maps showing the model cross section locations are provided on Figures 8-6a through 8-6c.

Existing Conditions Hydraulic Analysis

Existing conditions of Strawberry Creek were modeled with the most current geometric data available from previous HEC-2 models and several recently surveyed cross-sections. The vertical datum for the geometric data is NGVD29. The inflows to the model are the hydrographs created from the SacCalc model. As mentioned earlier in the hydrologic analyses section, the buildout flows are the same as the existing conditions flows. Therefore, the existing conditions hydraulic model uses existing conditions geometry with buildout flow rates. The hydraulic model was developed using HEC-RAS and was used to calculate water surface elevations for the 2-year, 10-year, and 100-year storm events.

The peak flow and maximum water surface results from HEC-RAS for existing conditions are shown in Table 8-3. Figures 8-5a through 8-5f present the calculated water surface profiles from HEC-RAS for existing conditions. The 100-year floodplain is contained within the channels and detention basins and there is no flooding of structures in the 100-year event for existing conditions in Strawberry Creek. The approximate limits of the 100-year floodplain are presented on Figures 8-6a through 8-6c.

Buildout Conditions Hydraulic Analysis

For the buildout conditions analysis, the geometric data was updated to reflect the addition of a proposed road crossing along the Middle Branch at Lakemont Drive. A 32-foot wide by 7-foot high Con-Span bridge has been proposed for this crossing as a part of the Northwest Unit No. 1 project. Since buildout flow rates were used for the existing conditions analysis, no revisions to the flow rates were required.

The peak flow and maximum water surface results from HEC-RAS for buildout conditions are shown in Table 8-4. Again, there are no flooded structures in the 100-year event for buildout conditions in Strawberry Creek. Because the water surface elevations between existing and buildout conditions only differ slightly, the water surface profiles and floodplain boundaries for buildout conditions were not produced.

Table 8-3. Strawberry Creek HEC-RAS Results – Existing Conditions

HEC-RAS River Station	Location	2-year peak flow, cfs	2-year max water surface, ft	10-year peak flow, cfs	10-year max water surface, ft	100-year peak flow, cfs	100-year max water surface, ft
4.176	Calvine Road	110	42.0	228	43.0	366	44.0
3.967	Brown Rd	106	38.8	219	40.4	354	41.8
3.57	Upstream from Elk Grove Florin Rd	200	35.0	345	36.5	399	37.2
3.142	Downstream from Black Kite Rd	202	31.6	360	33.3	390	34.4
2.871	Upstream from UPRR	224	31.0	354	32.9	424	34.1
2.836	Downstream from UPRR	247	30.8	394	32.7	445	33.8
2.52	Upstream from Sheldon North	462	30.2	821	32.0	1045	33.1
2.203	Downstream from Sheldon North	510	29.2	920	30.8	818	32.4
2.044	Upstream from DB 1B	586	28.6	1069	29.9	901	32.1
1.804	Upstream of DB 1B	589	28.0	597	29.3	894	31.9
1.721	At DB 1B	587	27.8	534	29.2	830	31.9
1.304	Upstream of Calvine Rd near HS	500	26.7	634	28.5	809	31.0
1.194	Downstream of Calvine Rd near HS	517	26.4	657	28.1	1053	29.9
0.815	Upstream of Power Inn Rd near HS	542	26.1	708	27.9	1199	29.5
0.776	Power Inn Rd	558	25.8	727	27.3	1219	28.4
0.512	Hwy 99	605	25.7	790	27.2	1365	28.3

Note: Water surface elevations are based on NGVD29.

Table 8-4. Strawberry Creek HEC-RAS Results – Buildout Conditions

HEC-RAS River Station	Location	2-year peak flow, cfs	2-year max water surface, ft	10-year peak flow, cfs	10-year max water surface, ft	100-year peak flow, cfs	100-year max water surface, ft
4.176	Calvine Road	110	42.0	228	43.0	366	44.0
3.967	Brown Rd	107	39.0	219	40.5	355	42.0
3.57	Upstream from Elk Grove Florin Rd	201	35.0	348	36.5	412	37.3
3.142	Downstream from Black Kite Rd	202	31.6	363	33.3	393	34.3
2.871	Upstream from UPRR	224	31.0	357	32.9	427	34.1
2.836	Downstream from UPRR	247	30.8	397	32.7	450	33.8
2.52	Upstream from Sheldon North	463	30.2	824	32.0	1041	33.1
2.203	Downstream from Sheldon North	513	29.2	925	30.7	839	32.3
2.044	Upstream from DB 1B	589	28.6	1075	29.8	919	32.0
1.804	Upstream of DB 1B	593	27.9	635	29.1	910	31.8
1.721	At DB 1B	547	27.8	332	29.1	550	31.8
1.304	Upstream of Calvine Rd near HS	502	26.6	690	28.1	842	30.9
1.194	Downstream of Calvine Rd near HS	519	26.3	712	27.7	1072	29.8
0.815	Upstream of Power Inn Rd near HS	545	26.1	763	27.4	1228	29.5
0.776	Power Inn Rd	560	25.8	781	26.9	1247	28.3
0.512	Hwy 99	608	25.8	847	26.8	1417	28.2

Note: Water surface elevations are based on NGVD29.

EVALUATION OF EXISTING PIPELINES

As indicated in Chapter 3, existing pipelines within the City’s arterial roadways with diameters 27 inches and greater were subject to evaluation during this SDMP. Eight existing trunk pipelines in the Strawberry Creek watershed met that criterion. Figures 8-7, 8-8, 8-9, and 8-10 show the existing pipelines that were evaluated.

Hydrologic Analysis of Existing Pipelines

SacCalc models were prepared to calculate the 2-year, 10-year, and 100-year flows entering the six pipe systems. The SacCalc models used to calculate flows in the existing pipelines are different from the models used to calculate creek flows. Smaller subsheds are necessary in the pipeline models to define the flows at various points along the pipelines. Because each pipeline serves a watershed that is nearly completely developed, flows were only calculated for buildout conditions. Figures 8-7, 8-8, 8-9 and 8-10 present the subshed boundaries used for the flow calculations. Table 8-5 presents the key hydrologic parameters for each subshed for buildout conditions. Table 8-6 presents the calculated peak flows from each subshed for the three storm events.

Table 8-5. Hydrologic Parameters for Existing Pipeline Models SBC1-SBC8

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land Use, acres and Percent Impervious							Average % Imp.
						Comm./ Office 90%	HDR 80%	Resd, 6-8 du/ac, School 50%	Resd, 4-6 du/ac 40%	Resd, 3-4 du/ac 30%	Rural Res. 10%	Park 5%	
Buildout Conditions													
SBC110	8.6	30	780	352	0.0051	8.6							36
SBC120	27.0	29	1,400	636	0.0014	27.0							33
SBC130	15.8	32	2,610	1,186	0.0011	15.8							36
SBC140	51.8	31	2,340	1,064	0.0012			16.0	34.7			1.1	40
SBC210	20.4	34	2,120	1,060	0.0019				20.4				40
SBC220	35.8	35	2,476	1,238	0.0004				35.8				40
SBC230	51.5	35	2,750	1,375	0.0009				51.5				40
SBC310	3.2	36	900	450	0.0025	3.2							90
SBC320	8.4	37	920	400	0.0023	3.2			5.2				59
SBC330	32.8	39	1,970	985	0.0020	8.8			24.0				53
SBC340	41.4	48	2,113	1,057	0.0076	10.3			31.0				53
SBC410	9.1	41	854	427	0.0023		6.9		2.3				70
SBC420	28.3	50	1,730	571	0.0116	9.3			18.9				57
SBC430	23.6	50	2,963	1,482	0.0067	11.8			11.8				65
SBC510	44.5	40	2,111	697	0.0023	4.0			37.8	2.7			44
SBC520	8.5	40	682	341	0.0069				8.5				40
SBC530	42.8	50	2,990	1,495	0.0067				32.9	7.3	2.6		37
SBC540	143.1	56	3,925	1,963	0.0082				8.6		111.7	22.9	11
SBC610	17.2	51	1,737	869	0.0058				17.2				40
SBC620	16.5	62	2,474	816	0.0097				1.5	15.0			31
SBC630	237.8	69	6,109	3,054	0.0056				178.3			59.4	31
SBC720	31.6	34	1,440	700	0.0014			4.8	26.8				42
SBC730	56.6	34	1,740	900	0.0012			12.5	44.1				42
SBC740	30.8	38	2,040	900	0.0020			24.0	6.8				48
SBC810	5.3	33	550	250	0.0050				5.3				40
SBC820	6.3	33	770	225	0.0038				6.3				40
SBC830	7.6	33	725	450	0.0040				7.3				40
SBC840	5.7	33	1,100	495	0.0025				5.7				40
SBC850	2.4	35	400	50	0.0030				2.4				40
SBC855	21.8	35	1,570	460	0.0030				20.3			1.5	38

Table 8-6. Calculated Subshed Flows for Existing Pipelines SBC1-SBC8

Subshed	Area (acres)	Buildout Condition Flows (cfs)		
		2-Year	10-Year	100-Year
SBC110	8.6	9	16	25
SBC120	27.0	22	41	60
SBC130	15.8	10	19	27
SBC140	51.8	29	54	78
SBC210	20.4	12	22	32
SBC220	35.8	18	32	46
SBC230	51.5	26	48	69
SBC310	3.2	3	6	9
SBC320	8.4	7	13	20
SBC330	32.8	21	39	57
SBC340	41.4	29	54	79
SBC410	9.1	8	15	23
SBC420	28.3	23	44	66
SBC430	23.6	15	28	41
SBC510	44.5	29	54	79
SBC520	8.5	8	15	23
SBC530	42.8	24	46	66
SBC540	143.1	62	115	191
SBC610	17.2	12	22	33
SBC620	16.5	11	20	30
SBC630	237.8	98	180	266
SBC720	31.6	21	39	57
SBC730	56.6	34	64	93
SBC740	30.8	19	36	53
SBC810	5.3	6	11	19
SBC820	6.3	6	11	17
SBC830	7.6	6	12	18
SBC840	5.7	4	8	12
SBC850	2.4	3	5	8
SBC855	21.8	16	30	44

Hydraulic Analysis of Existing Pipelines

Hydraulic models of the eight pipe systems were created using XPSWMM. Calculated water surface elevations and flows for the 2-year, 10-year, and 100-year storm events are summarized on Tables 8-7 and 8-8. Portions of pipelines SBC1 and SBC6 are outside of the City limits. Table 8-7 lists the results for only those nodes within the City. As Table 8-7 shows, the City's performance criteria for the seven existing pipelines are met at all but one location. The 10-year water surface elevation is below the top of curb at all locations except one, and the 100-year water surface elevation is below building pads.

In pipeline SBC7, there is one node (SBC740) where the 10-year water surface is above the curb. This node is at a local low point in the roadway and the extent of the above curb flooding is relatively small. To correct the deficiency would require replacement of over 3,700 feet of pipe and would be very costly. Because of this, and because the 100-year criterion at this location is met, there are no improvements recommended for pipeline SBC7.

The results for Pipeline SBC8 are similar as those for SBC7. In the upper part of the watershed there are three nodes where the 10-year water surface is above the curb. These nodes are at local low points in the roadway and the extent of the above curb flooding is relatively small. To correct the deficiency would require replacement of a large reach of pipe and would be very costly. Because of this, and because the 100-year criterion at this location is easily met, there are no improvements recommended for pipeline SBC8.

EVALUATION OF FUTURE PIPELINES

The Strawberry Creek watershed is mostly developed and no future major trunk pipelines are anticipated to serve new development.

PRELIMINARY IMPROVEMENTS

The existing creek, detention basins, and trunk pipelines in the Strawberry Creek watershed currently provide adequate flood protection within the City's portion of the watershed. A new creek crossing is anticipated in the near future for the extension of Lakemont Drive. A 32-foot wide by 7-foot high Con-Span bridge has been proposed for this crossing as a part of the Northwest Unit No. 1 project. This bridge was evaluated during this SDMP and found to provide adequate conveyance for buildout condition flows. Because bridge and culvert structures for new roads are considered roadway costs, it is anticipated that the costs for the Lakemont Drive bridge will not be included in the future Drainage Capital Improvement Plan. No other drainage improvements are recommended for Strawberry Creek watershed.

Table 8-7. Calculated Water Surface Elevations for Existing Pipelines SBC1-SBC8 (NGVD29)

Node Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline No. SBC1							
SBC130	30.3	31.6	23.0	28.2	29.1	-	-
SBC140	29.3	30.3	23.7	28.7	29.3	-	-
Pipeline No. SBC2							
SBC200	n/a	32.0	24.7	29.1	31.8	-	-
SBC210	32.0	33.0	26.7	29.6	32.2	-	
SBC220	33.7	34.0	27.6	31.1	33.4	-	-
SBC230	34.4	36.0	30.2	34.1	34.8	-	-
Pipeline No. SBC3							
SBC300	n/a	n/a	32.3	32.9	34.1	-	-
SBC310	39.7	40.0	33.9	34.1	34.7	-	
SBC320	37.0	37.2	36.3	36.7	37.1	-	-
SBC330	39.5	40.0	37.7	38.2	38.7	-	-
SBC340	40.5	41.0	37.9	38.7	39.4	-	-
Pipeline No. SBC4							
SBC410	39.5	40.5	34.0	36.3	37.1	-	-
SBC420	40.4	41.5	34.2	36.3	37.5	-	-
SBC430	40.1	41.0	34.4	36.3	37.8	-	-
Pipeline No. SBC5							
SBC500	36.7	38.0	33.9	33.9	35.2	-	-
SBC510	37.5	38.7	35.6	36.4	37.5	-	-
SBC520	37.6	38.4	36.2	37.4	38.3	-	-
SBC525	38.8	39.4	36.8	38.1	39.0		
SBC530	40.0	42.0	37.1	39.2	40.4	-	-
SBC540	40.1	42.0	37.5	40.1	41.2	-	-
Pipeline No. SBC6							
SBC610	47.0	48.0	42.2	43.0	44.0	-	-
SBC620	48.2	50.0	43.3	44.9	48.0	-	-
Pipeline No. SBC7							
SBC710	N/A	34.2	30.0	30.1	31.9	-	-
SBC720a	31.8	34.1	31.0	31.5	32.9	-	-
SBC720	32.9	34.2	31.6	32.2	33.5	-	-
SBC730	34.1	35.1	32.6	33.8	34.6	-	-
SBC740	33.0	36.0	32.9	34.1	34.8	Yes	-
Pipeline No. SBC8							
SBC800	N/A	N/A	29.9	30.0	30.0	-	-
SBC810	32.0	34.1	30.6	30.9	31.3	-	-
SBC820	32.2	34.1	31.2	31.5	31.9	-	-
SBC830	32.1	34.1	31.7	32.1	32.3	-	-
SBC840	32.2	34.2	32.2	32.5	32.9	Yes	-
SBC850	32.5	35.5	32.7	33.1	33.5	Yes	-
SBC855	33.2	35.5	33.4	33.5	33.8	Yes	-

Note: All elevations are based on NGVD29.

Table 8-8. Calculated Peak Flows for Existing Pipelines SBC1 - SBC8

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline No. SBC1						
RSBC140	SBC140	SBC130	Pipe	30	31	33
Pipeline No. SBC2						
RSBC210	SBC210	SBC200	Pipe	53	87	123
RSBC220	SBC220	SBC210	Pipe	43	70	100
RSBC230	SBC230	SBC220	Pipe	26	35	43
OLRSBC230	SBC230	SBC220	Overland	0	8	23
Pipeline No. SBC3						
RSC310.1	SBC310	SBC300	Pipe	37	39	42
RSC320.1	SBC320	SBC310	Pipe	36	38	39
RSC330.1	SBC330	SBC320	Pipe	40	39	40
RSC340.1	SBC340	SBC330	Pipe	29	54	67
OLRSC330	SBC330	SBC320	Overland	0	0	9
Pipeline No. SBC4						
RSBC420	SBC420	SBC410	Pipe	33	67	97
RSBC430	SBC430	SBC420	Pipe	16	29	41
Pipeline No. SBC5						
RSBC510	SBC510	SBC500	Pipe	100	118	114
RSBC520	SBC520	SBC510	Pipe	84	104	100
RSBC525	SBC525	SBC520	Pipe	106	129	145
RSBC530	SBC530	SBC525	Pipe	95	129	146
RSBC540	SBC540	SBC530	Pipe	63	94	88
OLRSBC510	SBC510	SBC500	Overland	0	0	12
OLRSBC520	SBC520	SBC510	Overland	0	1	31
OLRSBC525	SBC525	SBC520	Overland	0	1	56
OLRSBC530	SBC530	SBC525	Overland	0	0	57
OLRSBC540	SBC540	SBC530	Overland	0	6	65
Pipeline No. SBC6						
RSBC610	SBC620	SBC610	Pipe	104	186	270
RSBC620	SBC630	SBC620	Pipe	98	170	171
OLRSBC610	SBC620	SBC610	Overland	0	0	2
OLRSBC620	SBC630	SBC620	Overland	0	6	132
Pipeline No. SBC7						
710C	SBC710	SBC800	Pipe	67	85	77
720C	SBC720	SBC710	Pipe	67	87	76
730C	SBC730	SBC720	Pipe	47	61	57
740C	SBC740	SBC730	Pipe	22	31	28
730OLR	SBC730	SBC720	Overland	0	0	20
740OLR	SBC740	SBC730	Overland	0	0	13
Pipeline No. SBC8						
SBC810C	SBC810	SBC800	Pipe	24	28	32
SBC820P1	SBC820	SBC810	Pipe	20	22	25
SBC830P1	SBC830	SBC820	Pipe	16	18	19
SBC840aP1	SBC835	SBC830	Pipe	13	14	14
SBC840P	SBC840	SBC835	Pipe	13	14	14
SBC851P1	SBC850	SBC840	Pipe	11	12	12
SBC850P.1	SBC855	SBC850	Pipe	10	11	8
SBC830C1	SBC830	SBC820	Overland	0	0	1
SBC840C.1	SBC840	SBC835	Overland	0	0	4
SBC851C1	SBC850	SBC840	Overland	0	0	4
SBC850C.1	SBC855	SBC850	Overland	1	2	12

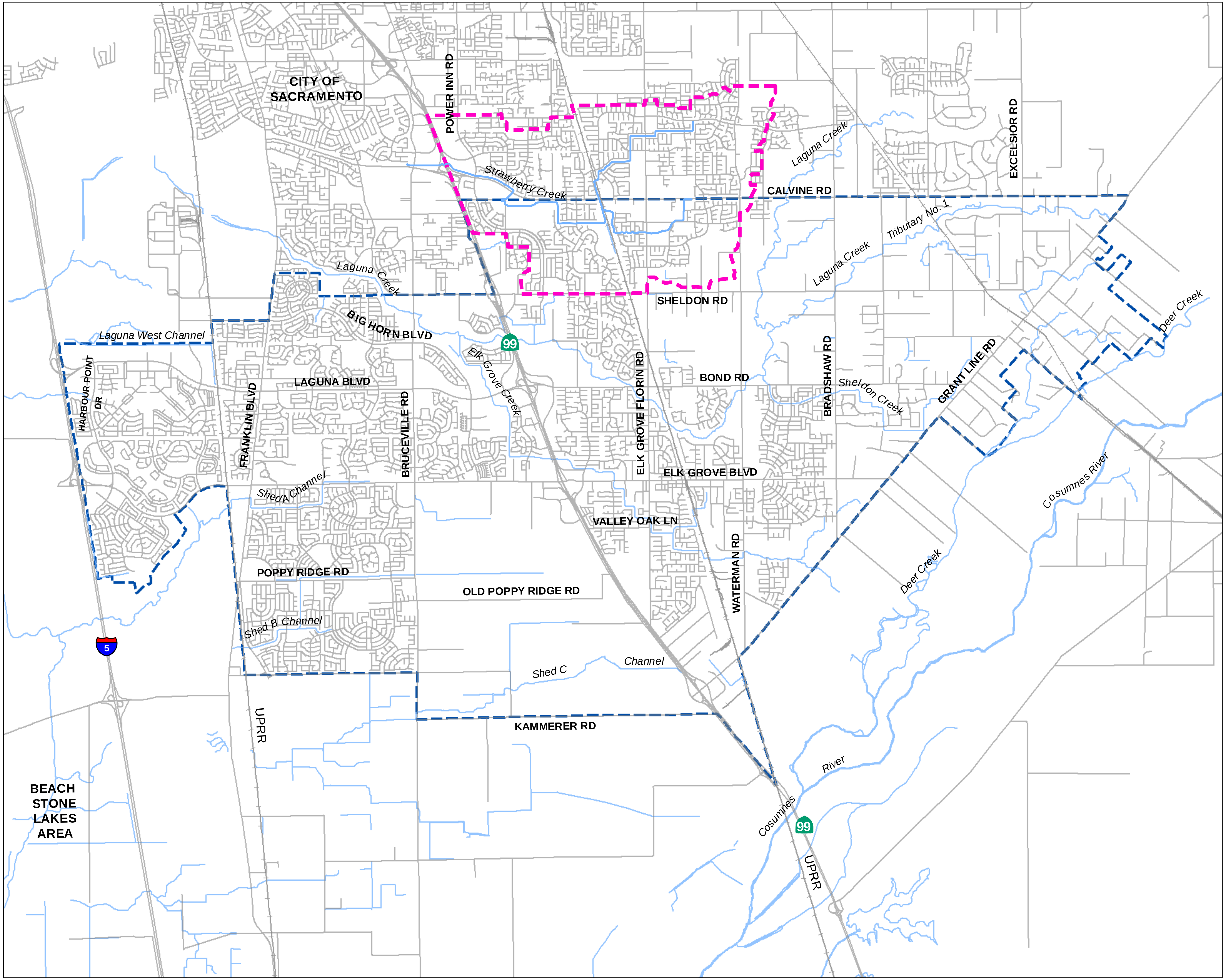


FIGURE 8-1

City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
LOCATION MAP

N

0 0.5 1 2
Scale in Miles

NOTES:

LEGEND:

- City Limit
- Strawberry Creek Watershed

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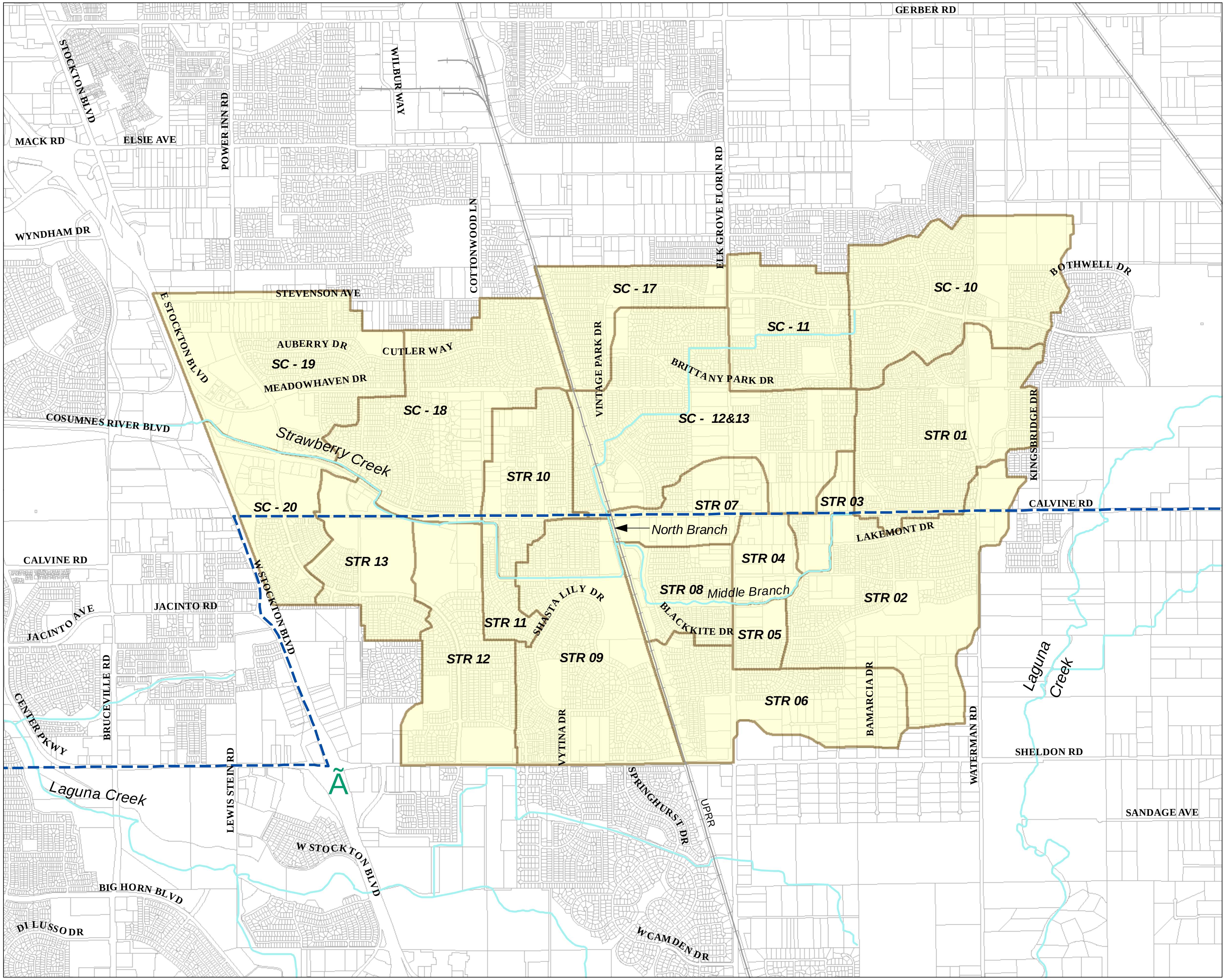

FIGURE 8-2

City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
SUBSHEDS FOR CREEK MODELING

NOTES:

LEGEND:

▭	City Limit
🟨	Strawberry Creek Subsheds

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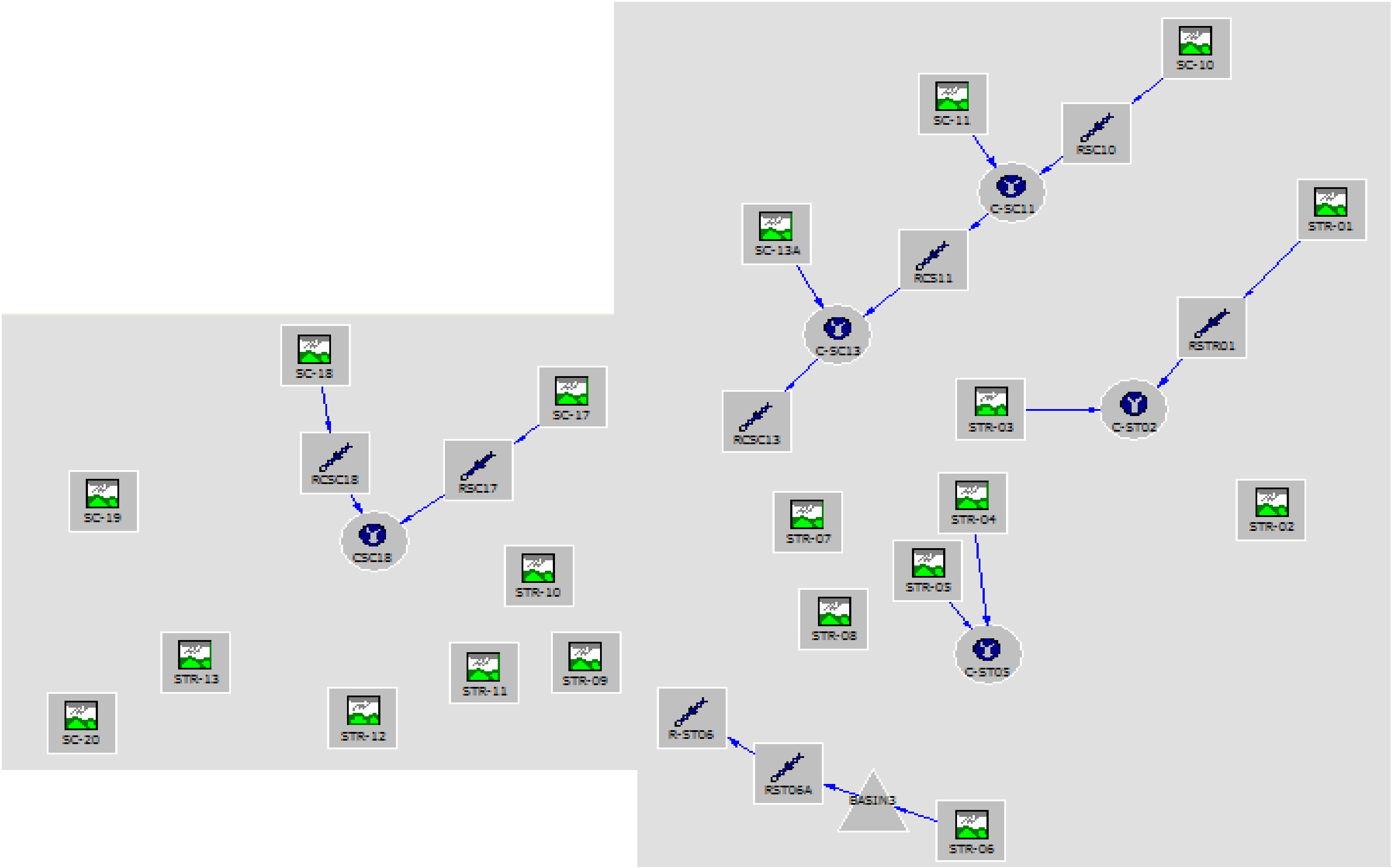


Figure 8-3. Strawberry Creek SacCalc Configuration

Note: SacCalc was used to create the hydrology for input into the unsteady HEC-RAS model for routing; therefore, not all of the elements are connected.

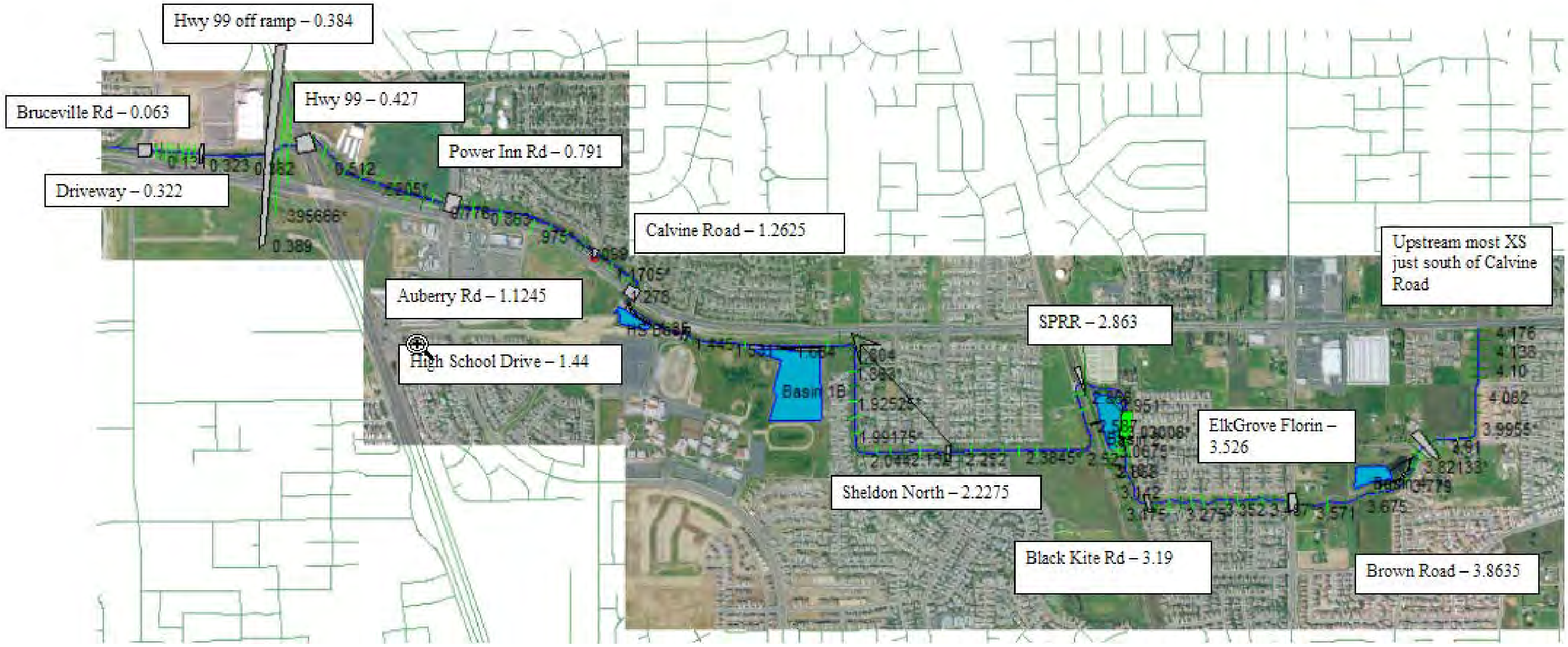


Figure 8-4. Strawberry Creek HEC-RAS Model Configuration

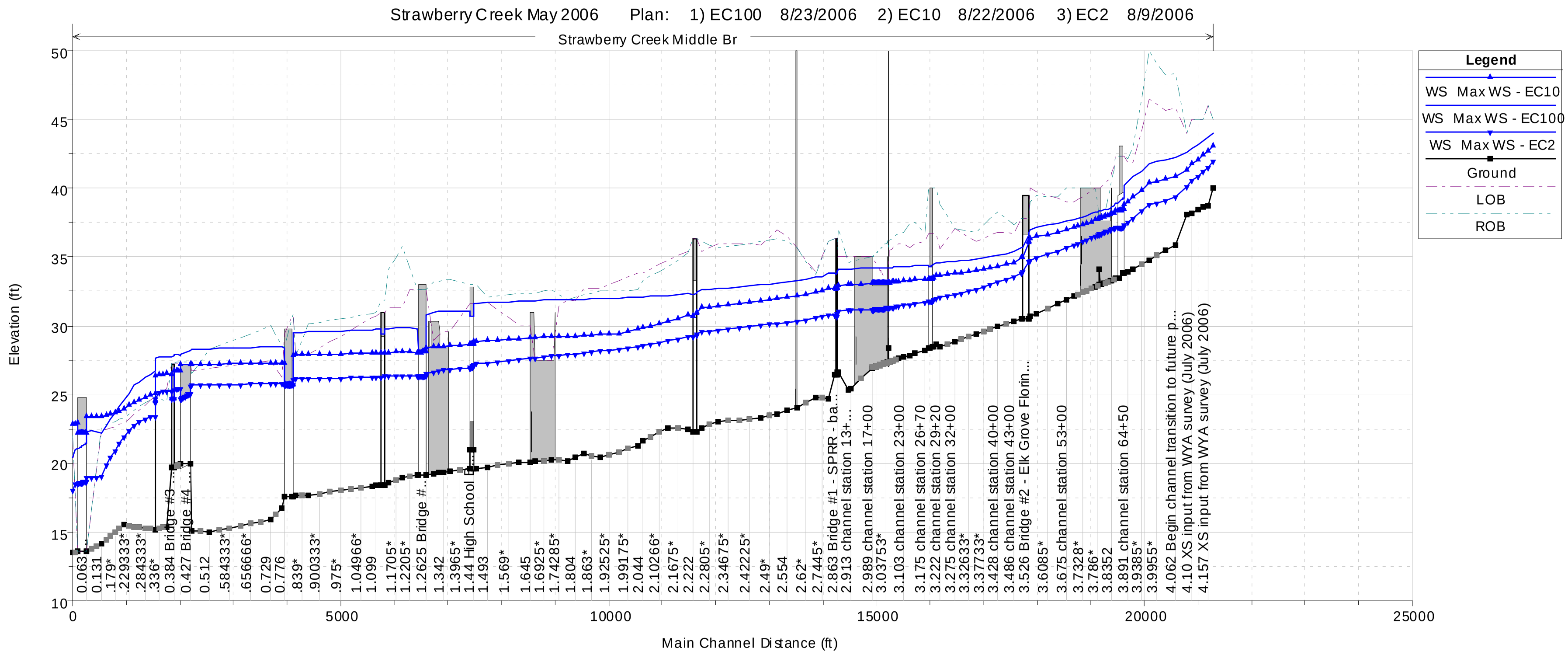


Figure 8-5a. Strawberry Creek RAS Results

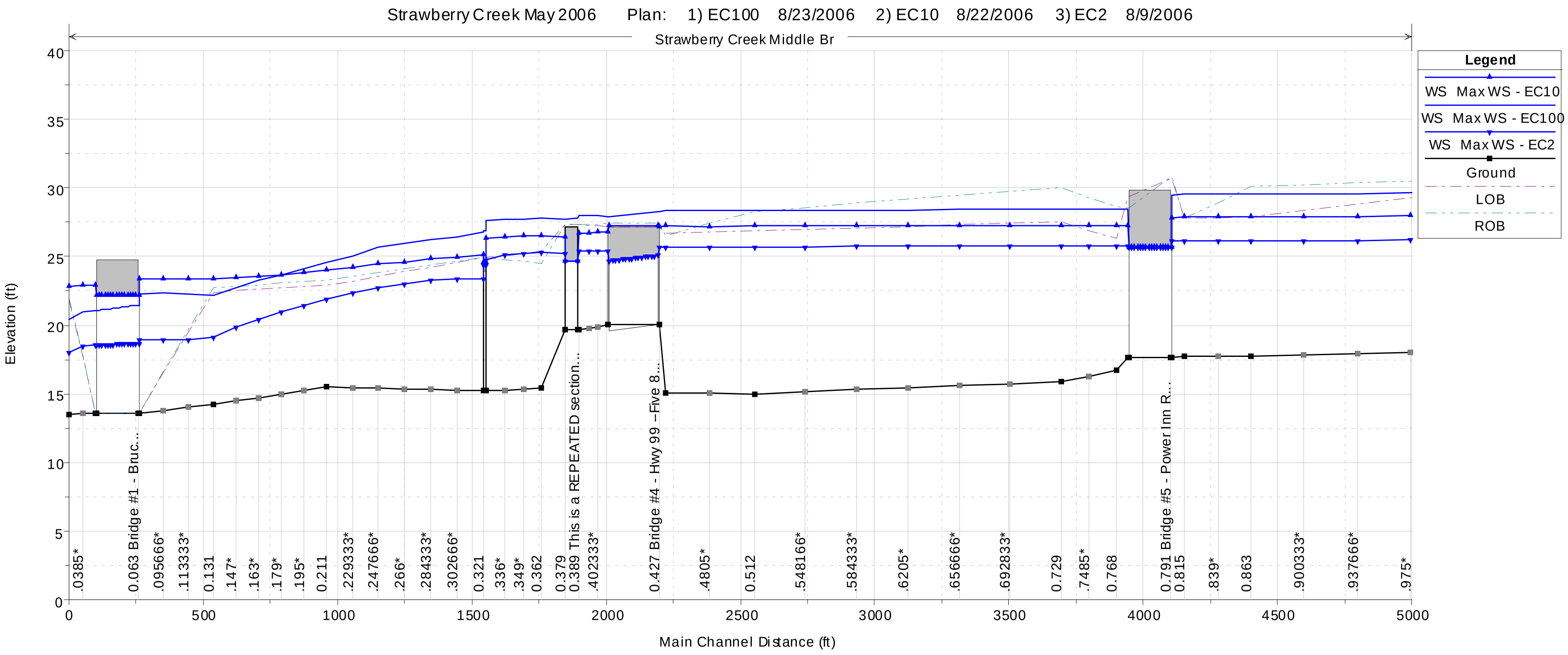


Figure 8-5b. Strawberry Creek Sta 0.385 to 0.975

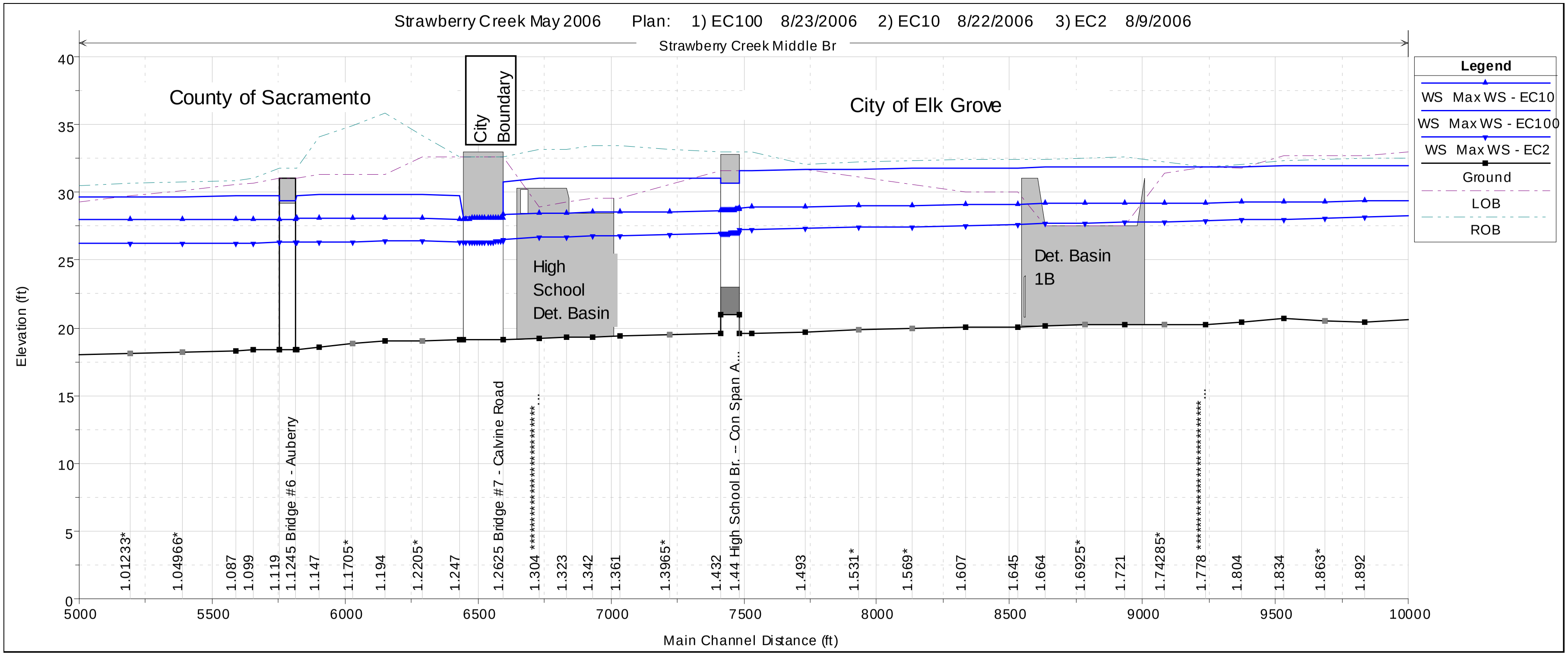


Figure 8-5c. Strawberry Creek Sta 1.01233 to 1.82

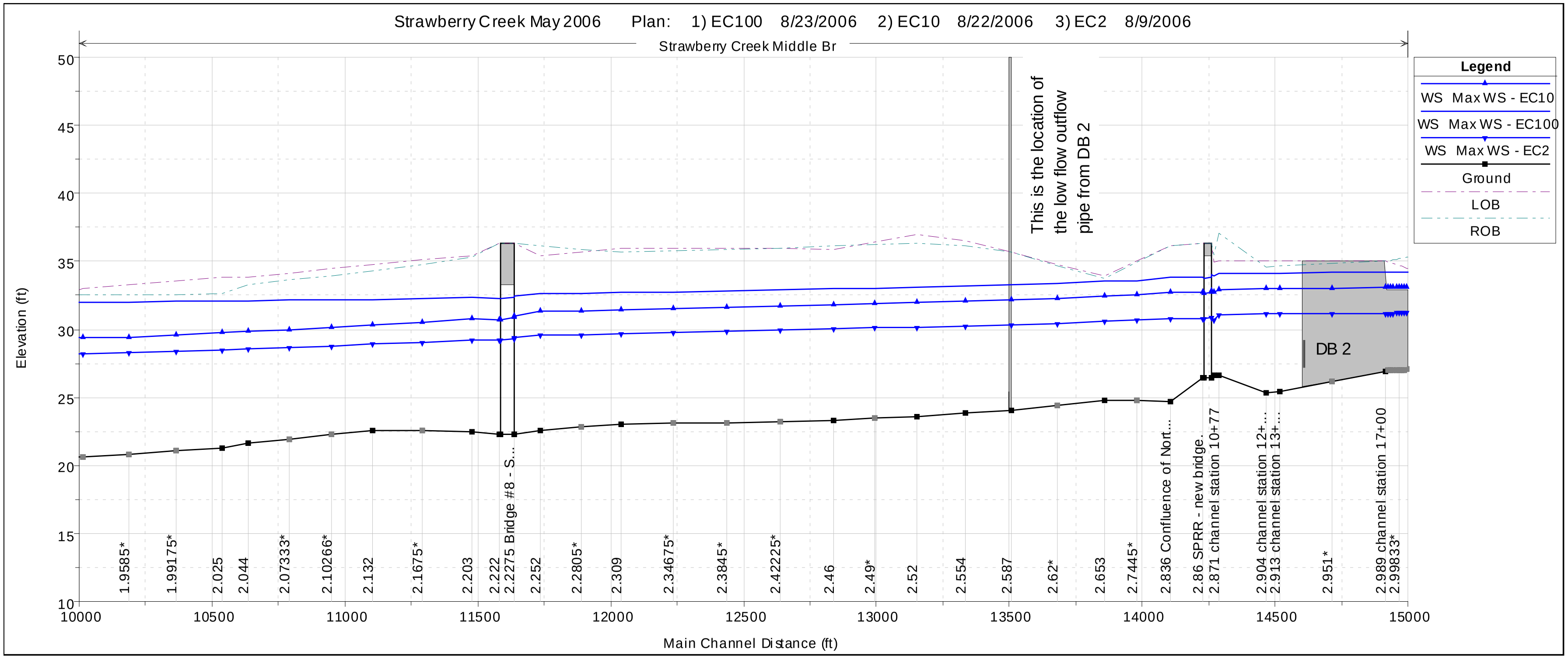


Figure 8-5d. Strawberry Creek Sta 1.9585 to 2.99833

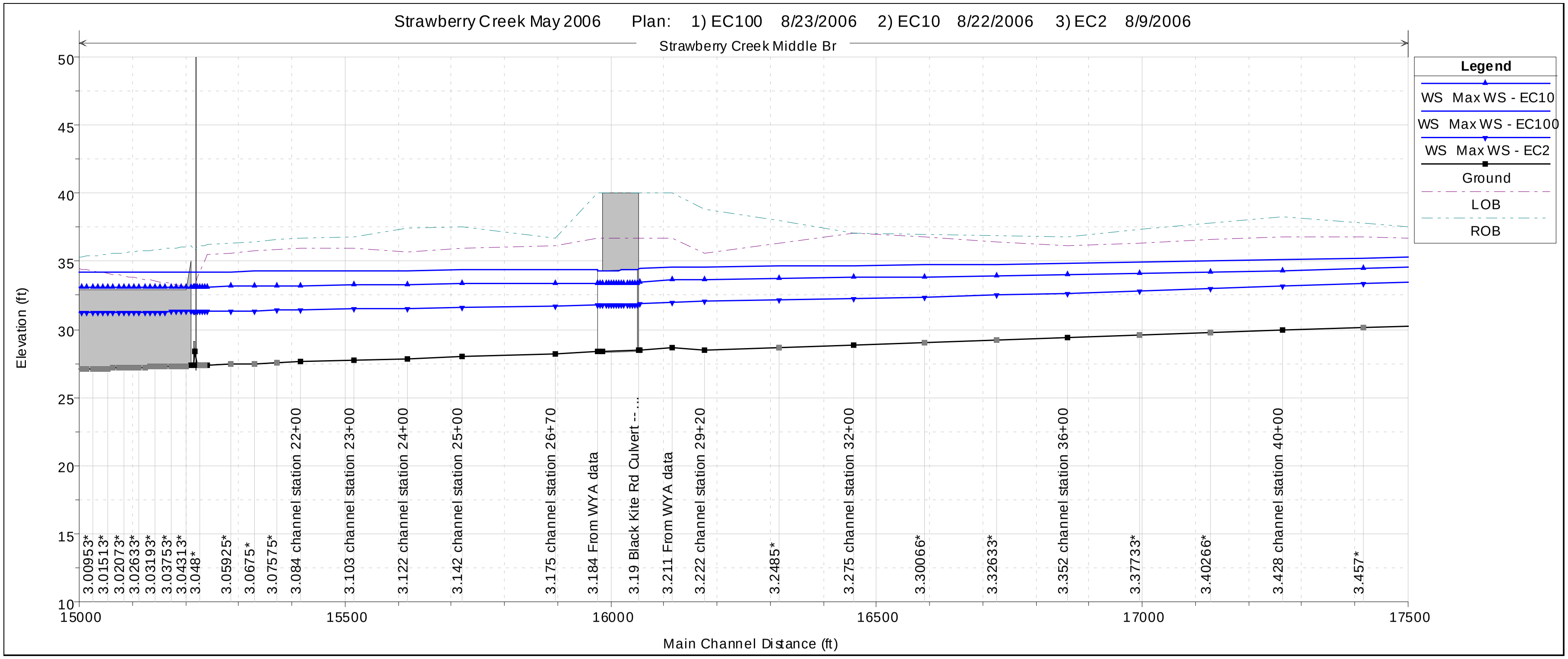


Figure 8-5e. Strawberry Creek Sta 3.00953 to 3.457

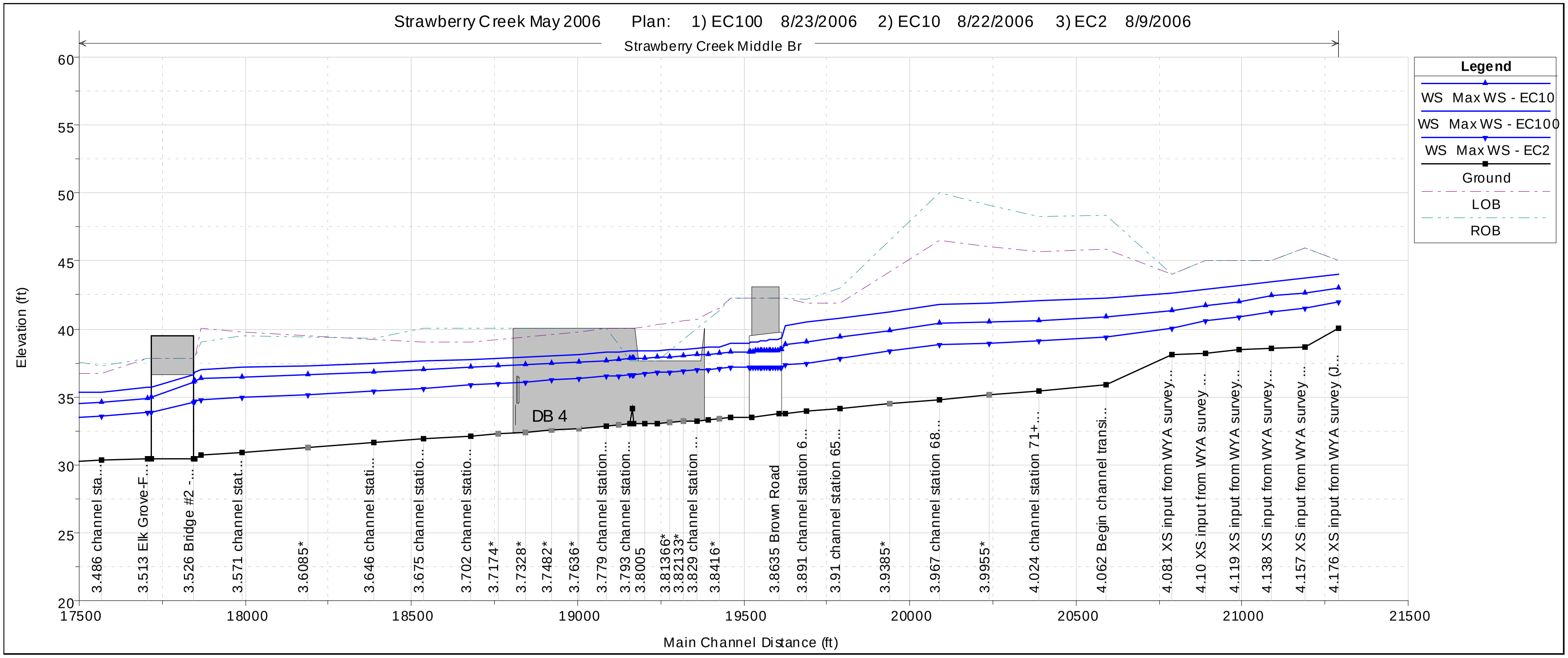
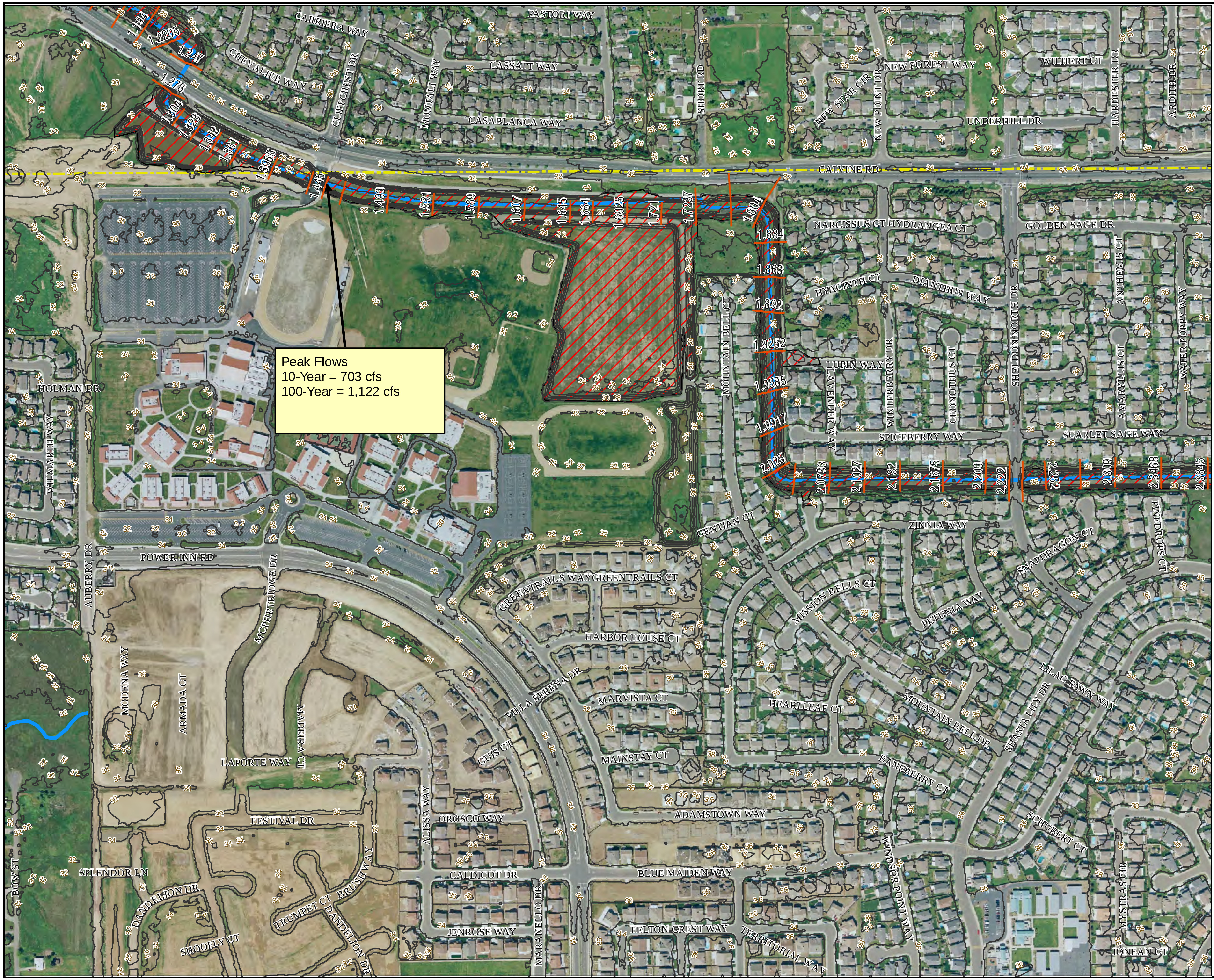


Figure 8-5f. Strawberry Creek Sta 3.486 to 4.176



Peak Flows
10-Year = 703 cfs
100-Year = 1,122 cfs

FIGURE 8-6a

City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800
Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Existing 100 Year Floodplain
☐	City Limit

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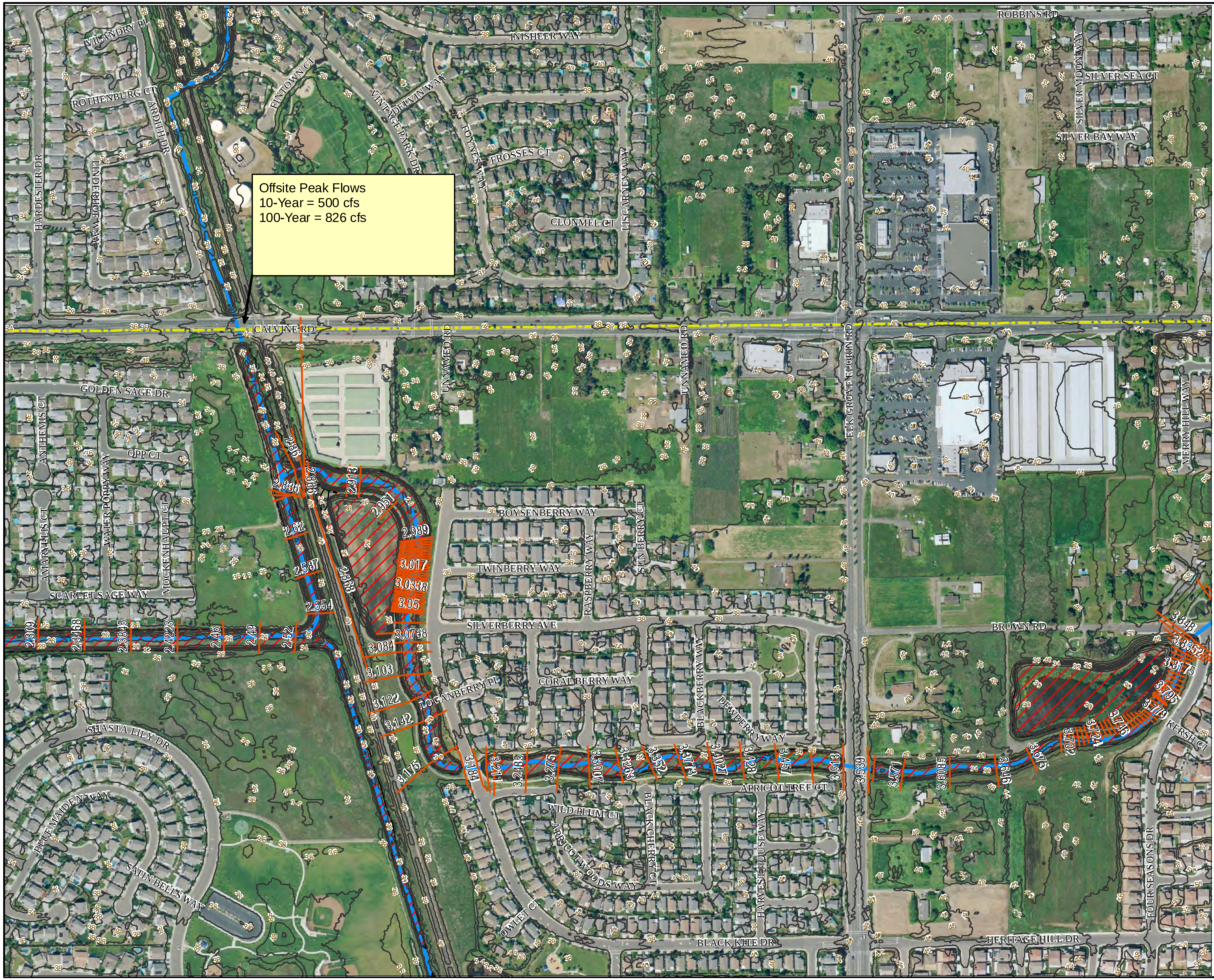

Offsite Peak Flows
10-Year = 500 cfs
100-Year = 826 cfs

FIGURE 8-6b

City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800
Feet

NOTES:

Legend

—	HEC-RAS Cross Section
—	Elevation Contour (NGVD 29)
▨	Existing 100 Year Floodplain
▭	City Limit

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FIGURE 8-6c

City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
APPROXIMATE 100-YEAR FLOODPLAIN

N

0 400 800

Feet

NOTES:

Legend

- HEC-RAS Cross Section
- Elevation Contour (NGVD 29)
- ▨ Existing 100 Year Floodplain
- City Limit

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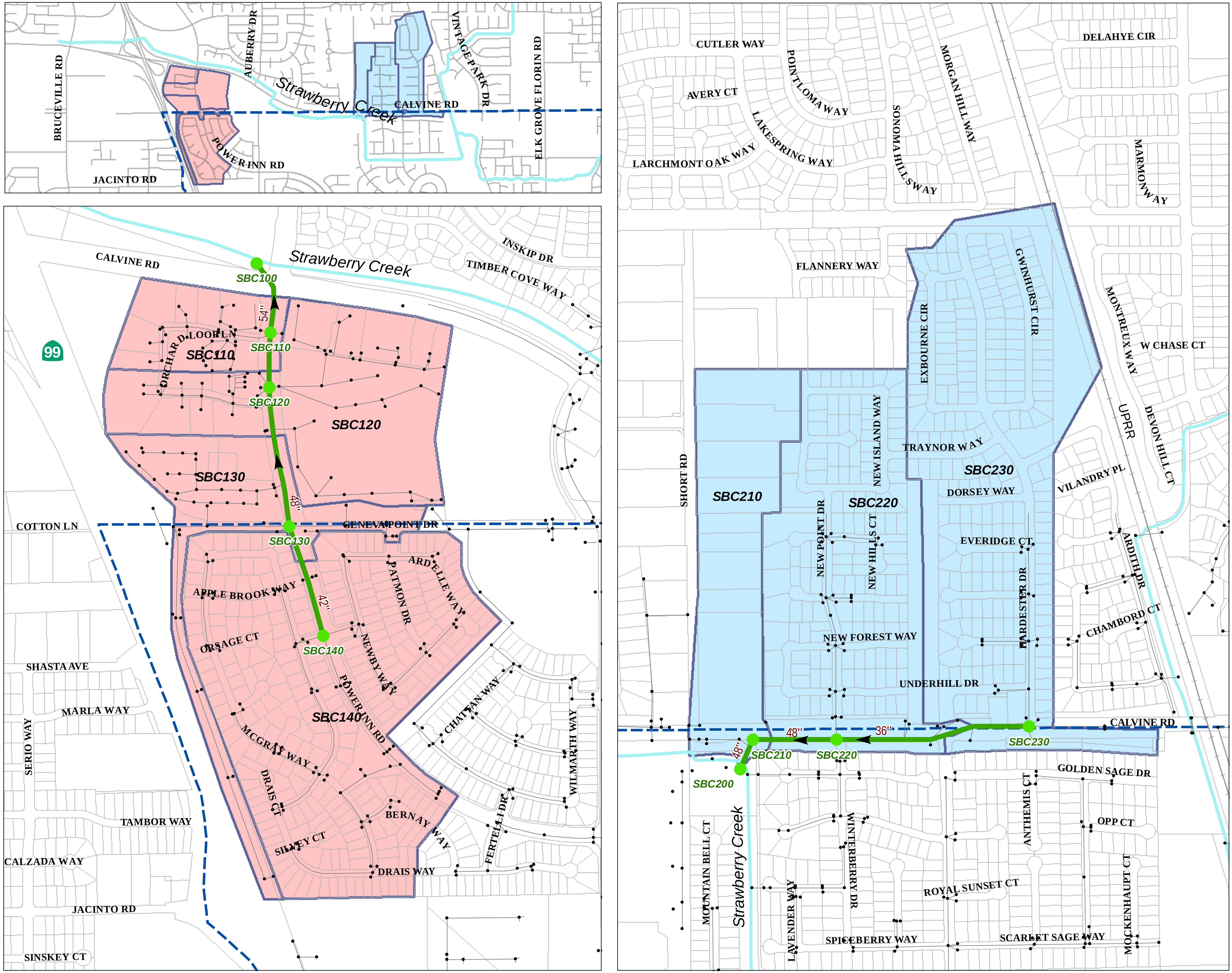
FIGURE 8-7

City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
EXISTING PIPELINES SBC1 & SBC2
SUBSHEDS & MODELED FACILITIES



N

0 500 1,000
Feet

NOTES:

LEGEND:

- SBC140 Modeled Pipeline and Node
- 🟥 Strawberry Ck Pipeline SBC1 Subshed
- 🟦 Strawberry Ck Pipeline SBC2 Subshed
- City Limit

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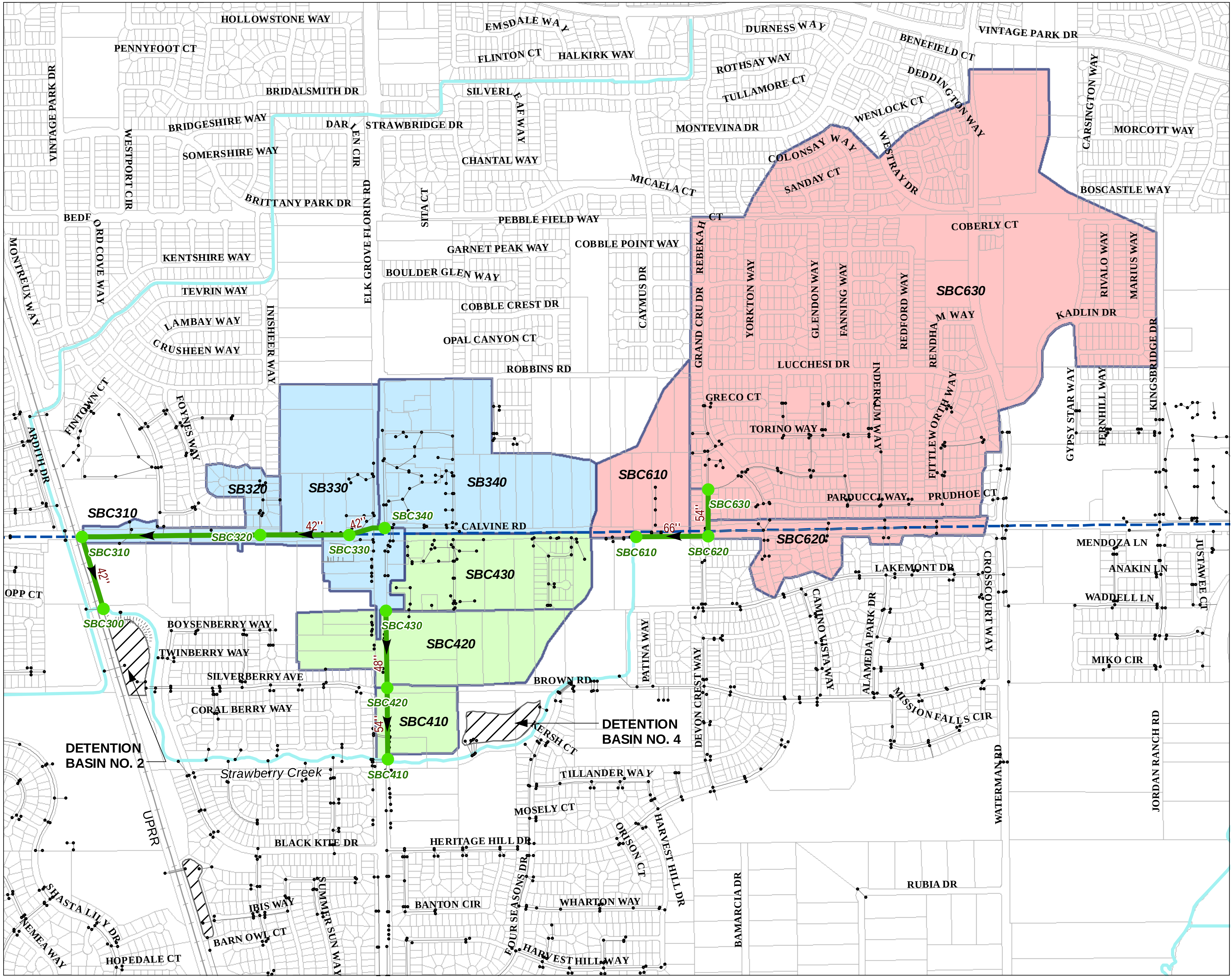

FIGURE 8-8
City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
EXISTING PIPELINES SBC3, SBC4, SBC6
SUBSHEDS & MODELED FACILITIES

N

0 800 1,600
Feet

NOTES:

LEGEND:

SBC610	
●━●	Modeled Pipeline and Node
🟦	Strawberry Ck Pipeline SBC3 Subshed
🟩	Strawberry Ck Pipeline SBC4 Subshed
🟥	Strawberry Ck Pipeline SBC6 Subshed
▭	City Limit

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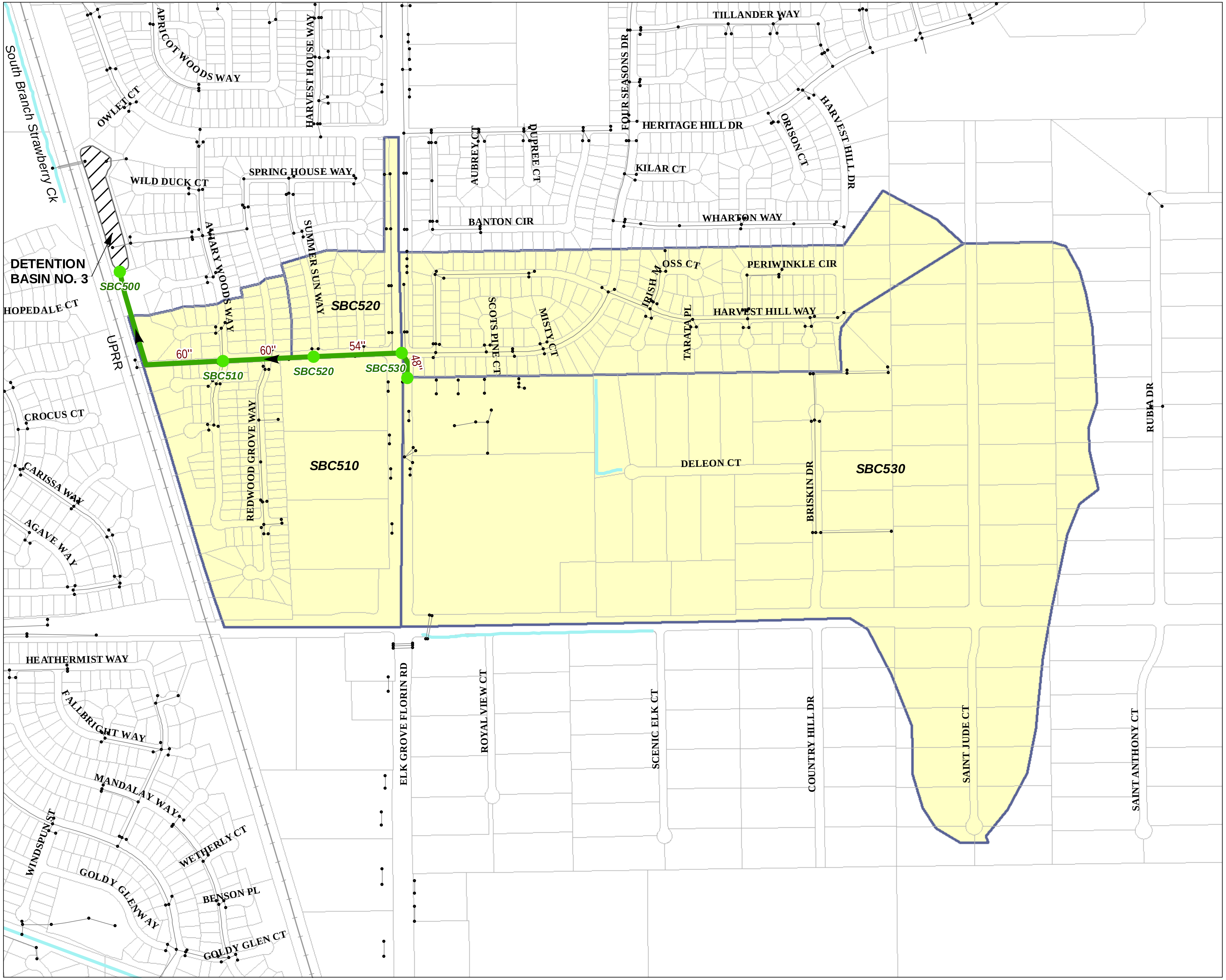

FIGURE 8-9

City of Elk Grove Storm Drainage Master Plan Volume II

STRAWBERRY CREEK
EXISTING PIPELINE SBC5
SUBSHEDS & MODELED FACILITIES

0 500 1,000
Feet

NOTES:

LEGEND:

SBC520

🟩 Modeled Pipeline and Node

▭ Strawberry Ck Pipeline SBC5 Subshed

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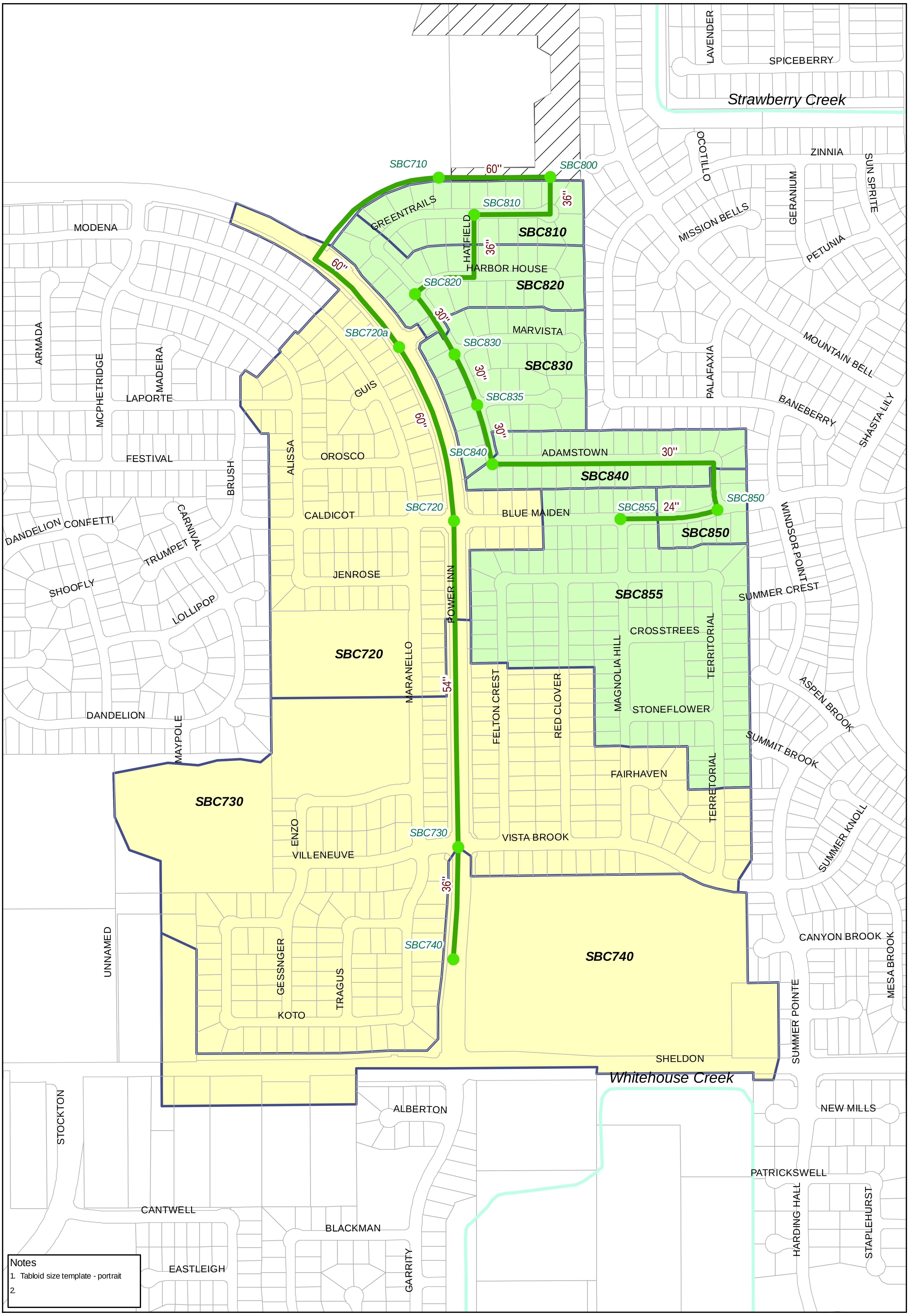

Notes
1. Tabloid size template - portrait
2.

LEGEND
- ● Modeled Node
- ▬ Modeled Pipeline
- 🟨 Pipeline SBC7 SubShed
- 🟩 Pipeline SBC8 SubShed

N

0 190 380
Scale in Feet

FIGURE 8-10
City of Elk Grove
Storm Drainage Master Plan
Volume II

STRAWBERRY CREEK
EXISTING PIPELINES SBC7 & SBC8
SUBSHEDS & MODELED FACILITIES

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CHAPTER 9. LAGUNA WEST CHANNEL

WATERSHED DESCRIPTION

The Laguna West Channel watershed lies in the western part of the City and covers approximately 1,500 acres. The area is generally bounded by Interstate 5 to the west, the Laguna West Lakes watershed to the south, DiLusso Drive to the east, and Dwight Road to the north (see Figure 9-1). The eastern part of the watershed is drained by an underground pipe system that generally conveys runoff to the west and to the Laguna West Channel. The Laguna West Channel is a constructed channel that begins just north of Laguna Boulevard and west of the Union Pacific Railroad. From that point, the channel travels north for approximately 2,400 feet to the City limits where it turns to the west and continues to Interstate 5. It crosses under the freeway and exits the City in three 8 feet by 12 feet box culverts and discharges into the Beach Stone Lakes area west of the freeway.

EVALUATION OF EXISTING FACILITIES

The existing facilities that were evaluated in this watershed include the Laguna West Channel, two detention basins, and a number of trunk pipelines. A description of the evaluation is provided below.

Hydrologic Analysis for Existing Facilities

A hydrologic analysis was performed to determine the 2-year, 10-year and 100-year flows for buildout conditions. Because the watershed is almost completely developed, existing condition flows are expected to be nearly equal to buildout flows and, therefore, only buildout condition flows were calculated. For the hydrologic modeling, the watershed was divided into 28 subsheds as shown on Figures 9-2a and 9-2b. Table 9-1 presents the key hydrologic parameters for each subshed for buildout conditions. Table 9-2 presents the resultant peak flows from each subshed for the 2-year, 10-year, and 100-year storms. Note that the SacCalc models were only used to calculate the flows from each subshed before they enter the main drainage system. These inflows were combined and routed within the drainage system using a hydraulic model as discussed below.

Hydraulic Analysis of Existing Facilities

A hydraulic analysis was performed to determine the peak flows and water surface elevations within key portions of the existing drainage system for the 2-year, 10-year, and 100-year storm events. An unsteady-state analysis was performed using the XPSWMM modeling software. Figures 9-2a and 9-2b show the major drainage facilities included in the model. Descriptions of these facilities are provided below.

Trunk Pipelines

The eastern portion of the watershed is served by an underground pipe network that includes large trunk pipes up to 108-inches in diameter. Pipelines with diameters of 27 inches or larger that lie within or cross under an arterial roadway were included in the hydraulic model. The pipeline sizes, lengths, and invert elevations were determined from as-built plans.

Table 9-1. Hydrologic Parameters for Laguna West Channel Watershed

						Land-Use (acres) and Percent Imperviousness									
Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Highway, Parking	Comm./ Office	Ind.	HDR	MDR	Resd, 6-8 du/ac	Resd, 3-4 du/ac	Park	Open	Average % Imp.
						95%	90%	85%	80%	70%	50%	30%	5%	2%	
Buildout Conditions															
LW200	63.8	19.0	2,736	2,358	0.0007		5.9	57.9							85
LW205	18.2	17.0	1,687	1,137	0.0018		0.1	18.2							85
LW210	12.1	18.0	1,109	680	0.0003			12.1							85
LW220	43.3	17.5	1,921	1,281	0.0008		25.3		15.5		1.7		0.8		83
LW225	7.0	16.0	1,082	505	0.0014		1.8	5.3							86
LW230	22.6	18.0	2,697	1,601	0.0013		4.4	17.5					0.6		84
LW300	25.2	19.0	1,810	1,282	0.0011		2.2	23.0							85
LW305	6.5	30.0	1,060	517	0.0330			0.5					6.0		11
LW310	20.0	19.0	3,115	1,472	0.0006			20.0							85
LW315	8.9	20.0	1,417	641	0.0007			7.1			0.0	0.0	1.8		69
LW320	49.6	22.5	2,049	1,737	0.0024						35.6	13.0	0.9		44
LW410	117.8	22.0	5,466	3,315	0.0007	1.2	0.6		9.6	5.6	83.3	10.2	7.3		50
LW415	18.3	23.0	2,238	2,168	0.0009		15.8		0.6		0.0		1.8		81
LW425	109.3	23.0	5,402	2,797	0.0011		1.5		13.2	13.8	40.2	27.2	13.4		46
LW430	95.5	23.5	5,001	2,727	0.0002							94.4	1.1		30
LW450	11.7	21.5	1,034	665	0.0029	0.2	9.6			1.1	0.1		0.7		83
LW455	37.6	22.0	2,588	1,350	0.0015	0.2				2.2	22.3	3.0	9.8		38
LW460	41.9	21.5	2,350	1,547	0.0021		2.3				35.9	3.7			50
LW465	16.1	21.0	1,657	1,019	0.0012		0.6				15.5				51
LW475	22.9	22.5	1,947	1,406	0.0005		11.5				8.9	1.4	1.2		67
LW480	50.8	23.0	2,788	1,994	0.0007						44.3	6.0	0.4		47
LW505	72.2	21.5	2,878	2,275	0.0003		0.2				62.5	1.8	7.6		45
LW515	60.0	21.5	2,833	1,962	0.0004		3.0				54.9		2.2		50
LW530	249.3	25.5	8,255	4,915	0.0006	4.6	7.1			13.6	201.1		23.0		49
LWC120	90.2	12.0	1,475	738	0.0081		4.0	3.7						82.4	9
LWC170	129.0	18.0	1,105	553	0.0090		8.3	2.9						117.8	10
LWC230	59.2	18.0	1,095	547	0.0018		4.2	0.1						54.9	8
LWC250	22.5	22.5	2,351	1,150	0.0013						13.9	8.0	0.6		42

Table 9-2. Calculated Subshed Flows - Laguna West Channel Watershed

Subshed	Area, acres	Buildout Condition Flows, cfs		
		2-Year	10-Year	100-Year
LW200	63.8	31	63	90
LW205	18.2	11	24	35
LW210	12.1	7	16	23
LW220	43.3	24	51	74
LW225	7.0	5	11	17
LW230	22.6	12	25	36
LW300	25.2	15	31	45
LW305	6.5	4	9	16
LW310	20.0	10	21	30
LW315	8.9	5	11	17
LW320	49.6	24	51	74
LW410	117.8	40	81	114
LW415	18.3	9	19	27
LW425	109.3	38	79	114
LW430	95.5	27	56	78
LW450	11.7	8	18	28
LW455	37.6	16	35	53
LW460	41.9	20	44	63
LW465	16.1	9	18	27
LW475	22.9	11	23	34
LW480	50.8	21	44	62
LW505	72.2	26	54	78
LW515	60.0	24	49	70
LW530	249.3	72	143	201
LWC120	90.2	44	98	166
LWC170	129.0	69	156	266
LWC230	59.2	28	62	105
LWC250	22.5	11	23	33

Laguna West Channel

The Laguna West Channel is a man-made, earthen, trapezoidal channel that generally conveys runoff from east to west. The channel cross section geometry used in the model was based on improvement plans.

Detention Basins

There are two detention basins in the watershed. Detention Basin L-1 is located in the north end of watershed adjacent to the Laguna West Channel (see Figure 9-2a). Basin L-1 provides approximately 3.9 acre-feet of stormwater quality treatment volume and 30.1 acre-feet of flood control storage for tributary area of approximately 167 acres. Detention Basin L-2 is located at the upstream end of the Laguna West Channel (see Figure 9-2a). Basin L-2 provides approximately 3.8 acre-feet stormwater quality treatment volume and 18.0 acre-feet of flood control storage for a tributary area of approximately 110 acres.

Results of Hydraulic Analysis

For all storm events, the starting water surface elevation at the outfall on the west side of Interstate 5 was based on normal depth in the culvert. This results in starting water surface elevations that are lower than the corresponding peak water surface elevations in the Beach Stone Lakes area at the downstream end of the model (FEMA 100-year water surface elevation is 16.0 in the Beach Stone Lakes area). However, they are considered reasonable for this SDMP since the peak flows from the watershed are expected to occur well before the peak stage occurs in the Beach Stone Lakes area. This is because the peak stages in the Beach Stone Lakes area are controlled by flows from the Cosumnes River and Mokelumne River watersheds that back up into the Beach Stone Lakes area. Due to the large size of the Cosumnes and Mokelumne River watersheds, the peak flows from these rivers occur well after the peak flows from the Laguna West Channel watershed.

Calculated water surface elevations and peak flows for the 2-year, 10-year, and 100-year storm events are summarized on Tables 9-3 and 9-4. Water surface profiles for the Laguna West Channel are presented on Figure 9-3. As Table 9-3 shows, the performance criteria for existing drainage systems are met at all locations. Street flooding is predicted at a few locations during a 10-year storm, but the depths are below the top of curb at all locations. No building pads are predicted to flood during a 100-year storm.

EVALUATION OF FUTURE FACILITIES

The Laguna West Channel watershed is almost completely developed and no future major drainage facilities are anticipated to serve new development.

PRELIMINARY IMPROVEMENTS

The results of the hydrologic analysis indicate that the existing major drainage facilities serving this watershed provide adequate flood protection. No major new facilities are required since the watershed is nearly built-out. As a result, no drainage improvements are recommended for the Laguna West Channel watershed.

Table 9-3. Calculated Water Surface Elevations - Laguna West Channel Watershed (NGVD29)

Node Name	Notes	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
LWDC100	Laguna West Channel	n/a	n/a	2.4	3.4	4.2	-	-
LWC120		n/a	16.0	2.9	4.1	5.1	-	-
LWDC140		n/a	16.0	6.8	8.4	9.5	-	-
LWC170		n/a	16.0	8.1	9.8	11.0	-	-
LWDC200		n/a	n/a	9.3	11.1	12.3	-	-
LWC230		n/a	18.0	10.4	12.1	13.2	-	-
LWC250		n/a	18.0	11.1	12.8	13.8	-	-
LWDC270		n/a	20.0	11.2	13.0	14.0	-	-
LWDC300		n/a	n/a	11.4	13.2	14.2	-	-
LWDC400		n/a	n/a	11.5	13.3	14.2	-	-
LWL1GH	Pipe System	n/a	n/a	9.3	11.1	12.4	-	-
LW200		n/a	n/a	12.7	13.2	14.0	-	-
LW205		17.7	20.0	12.7	13.3	14.3	-	-
LW210		18.5	20.0	12.7	13.3	14.6	-	-
LW215		16.3	18.0	12.7	13.5	15.4	-	-
LW220		15.5	18.0	12.7	14.4	16.4	-	-
LW225		14.9	18.0	12.7	13.8	15.5	-	-
LW230		16.7	18.0	12.7	15.4	16.1	-	-
LWL2GH		n/a	n/a	11.4	13.2	14.5	-	-
LW300		n/a	20.0	13.6	14.1	14.7	-	-
LW305		19.7	18.0	13.6	14.6	15.8	-	-
LW310		18.5	20.0	13.6	15.4	18.4	-	-
LW315		15.7	18.0	13.6	14.6	16.0	-	-
LW320		18.5	18.0	13.6	15.6	16.6	-	-
LW403		n/a	n/a	11.8	13.6	14.6	-	-
LW404		n/a	18.0	11.9	13.7	14.7	-	-
LW405		n/a	20.0	12.2	14.1	15.2	-	-
LW410		19.7	19.8	13.1	15.8	17.3	-	-
LW415		22.4	22.0	14.0	17.4	18.9	-	-
LW420		22.9	22.0	14.5	18.0	19.4	-	-
LW425		19.3	22.0	15.6	19.0	19.8	-	-
LW430		23.5	24.0	14.8	18.4	20.2	-	-
LW440		20.3	22.0	12.7	14.9	16.5	-	-
LW445		18.3	20.0	12.8	15.0	17.6	-	-
LW450		20.1	22.0	12.8	15.7	19.7	-	-
LW455		20.6	22.0	13.0	15.4	17.5	-	-
LW460		19.9	22.0	13.4	16.3	18.4	-	-
LW465		19.0	21.6	14.2	17.9	19.4	-	-
LW470		19.8	22.0	14.7	18.8	19.9	-	-
LW475		23.0	23.5	15.2	19.6	20.4	-	-
LW480		22.6	22.0	15.6	20.1	20.6	-	-
LW500		n/a	n/a	11.3	13.1	14.1	-	-
LW505		18.8	20.0	12.0	13.8	14.7	-	-
LW510		19.1	20.0	12.6	14.6	15.5	-	-
LW515		20.6	22.0	13.1	15.4	16.2	-	-
LW520		25.1	24.0	13.4	15.9	16.6	-	-
LW525		23.1	24.0	13.6	16.0	16.8	-	-
LW530		23.1	22.0	13.7	16.1	16.9	-	-

Table 9-4. Calculated Peak Flows - Laguna West Channel Watershed

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
RLWDC120	LWC120	LWDC100	Channel	84	168	252
RLWDC140	LWDC140	LWC120	Channel	244	488	729
RLWDC170	LWC170	LWDC140	Channel	247	492	732
RLWDC200	LWDC200	LWC170	Channel	237	477	699
RLWDC230	LWC230	LWDC200	Channel	244	467	605
RLWDC250	LWC250	LWC230	Channel	239	464	595
RLWDC270	LWDC270	LWC250	Channel	136	287	396
RLWDC300	LWDC300	LWDC270	Channel	146	294	408
RLWDC400	LWDC400	LWDC300	Channel	156	304	355
RLWL1GH	LWL1GH	LWDC200	Pipe	5	36	110
RLW200	LW200	LWL1GH	Weir	5	36	110
RLW205	LW205	LW200	Pipe	53	110	119
RLW210	LW210	LW205	Pipe	43	87	84
RLW215	LW215	LW210	Pipe	37	72	62
RLW220	LW220	LW215	Pipe	24	51	53
RLW225	LW225	LW215	Pipe	5	11	10
RLW230	LW230	LW215	Pipe	12	16	14
RLWL2HG	LWL2GH	LWDC300	Pipe	2	30	72
RLW300	LW300	LWL2GH	Weir	2	30	72
RLW305	LW305	LW300	Pipe	40	78	70
RLW310	LW310	LW305	Pipe	10	21	28
RLW315	LW315	LW305	Pipe	5	11	13
RLW320	LW320	LW305	Pipe	24	41	34
RLW403A	LW403	LWDC400	Pipe	153	271	306
RLW403B	LW403	LWDC400	Pipe	10	47	65
RLW404	LW404	LW403	Pipe	163	317	368
RLW405	LW405	LW404	Pipe	163	318	368
RLW410	LW410	LW405	Pipe	106	204	228
RLW415	LW415	LW410	Pipe	68	127	127
RLW420	LW420	LW415	Pipe	63	118	124
RLW425	LW425	LW420	Pipe	38	76	87
RLW430	LW430	LW420	Pipe	27	56	78
RLW440	LW440	LW405	Pipe	72	150	173
RLW445	LW445	LW440	Pipe	7	19	27
RLW450	LW450	LW445	Pipe	8	18	27
RLW455	LW455	LW440	Pipe	69	141	159
RLW460	LW460	LW455	Pipe	54	110	109
RLW465	LW465	LW460	Pipe	37	67	64
RLW470	LW470	LW465	Pipe	31	52	54
RLW475	LW475	LW470	Pipe	31	52	54
RLW480	LW480	LW475	Pipe	21	43	48
RLW500	LW500	LWC250	Pipe	103	187	207
RLW505	LW505	LW500	Pipe	104	189	209
RLW510	LW510	LW505	Pipe	86	146	157
RLW515	LW515	LW510	Pipe	86	146	156
RLW520	LW520	LW515	Pipe	71	125	143
RLW525	LW525	LW520	Pipe	72	124	142
RLW530	LW530	LW525	Pipe	72	124	142

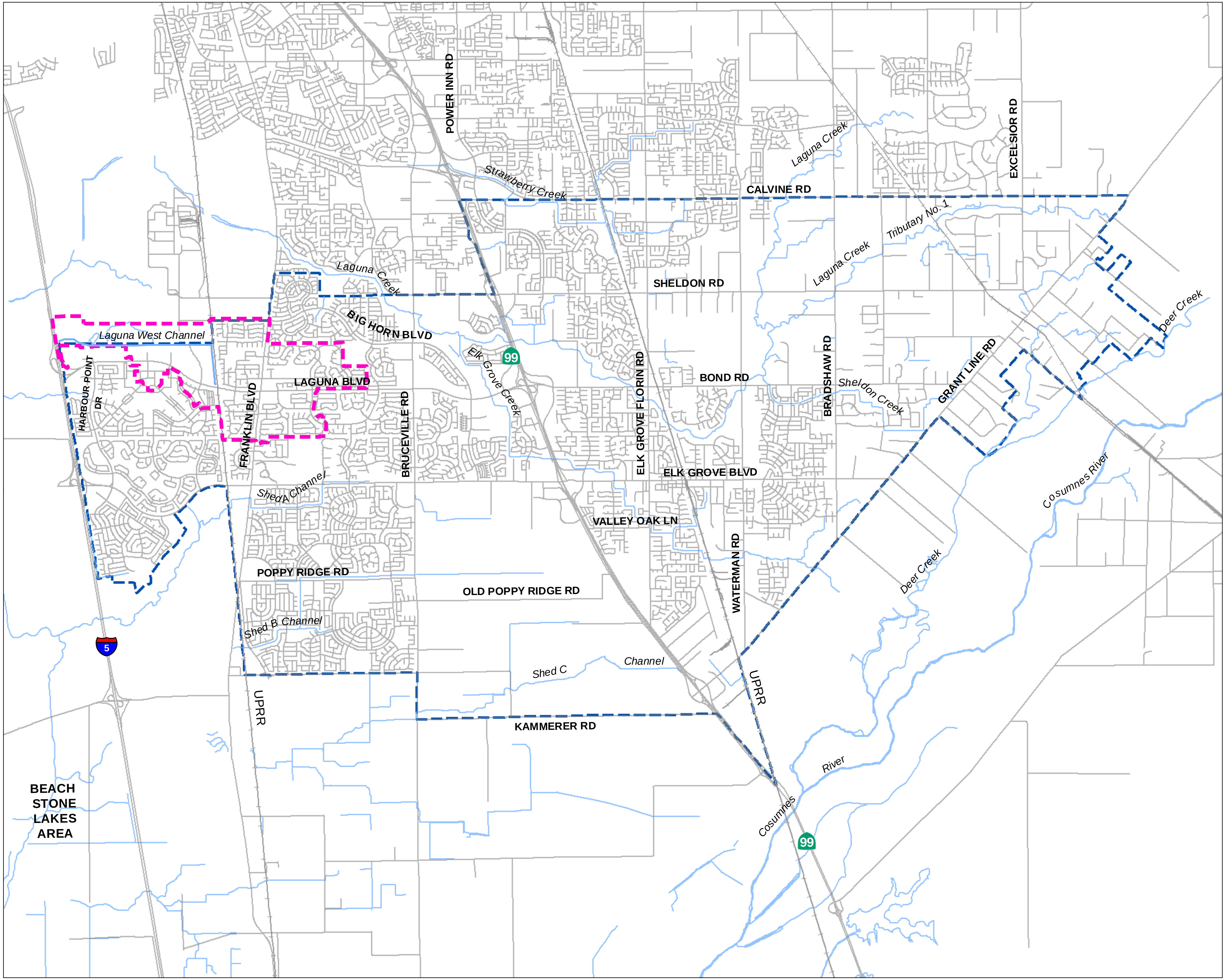


FIGURE 9-1

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA WEST CHANNEL LOCATION MAP

N

0 0.5 1 2

Scale in Miles

NOTES:

LEGEND:

- City Limit
- Laguna West Channel Watershed

WEST YOST ASSOCIATES
Consulting Engineers

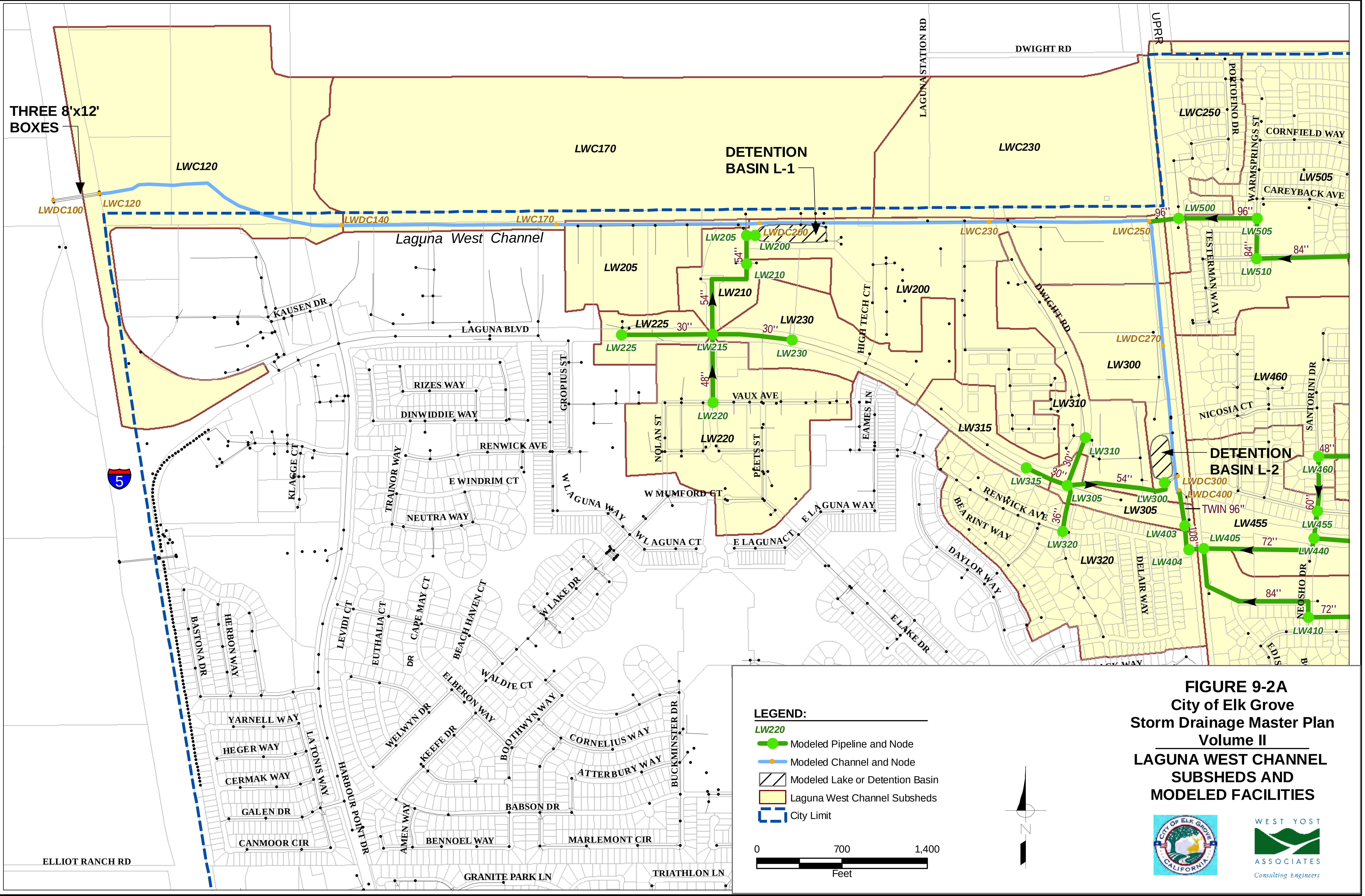


FIGURE 9-2A
City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA WEST CHANNEL SUBSHEDS AND MODELED FACILITIES

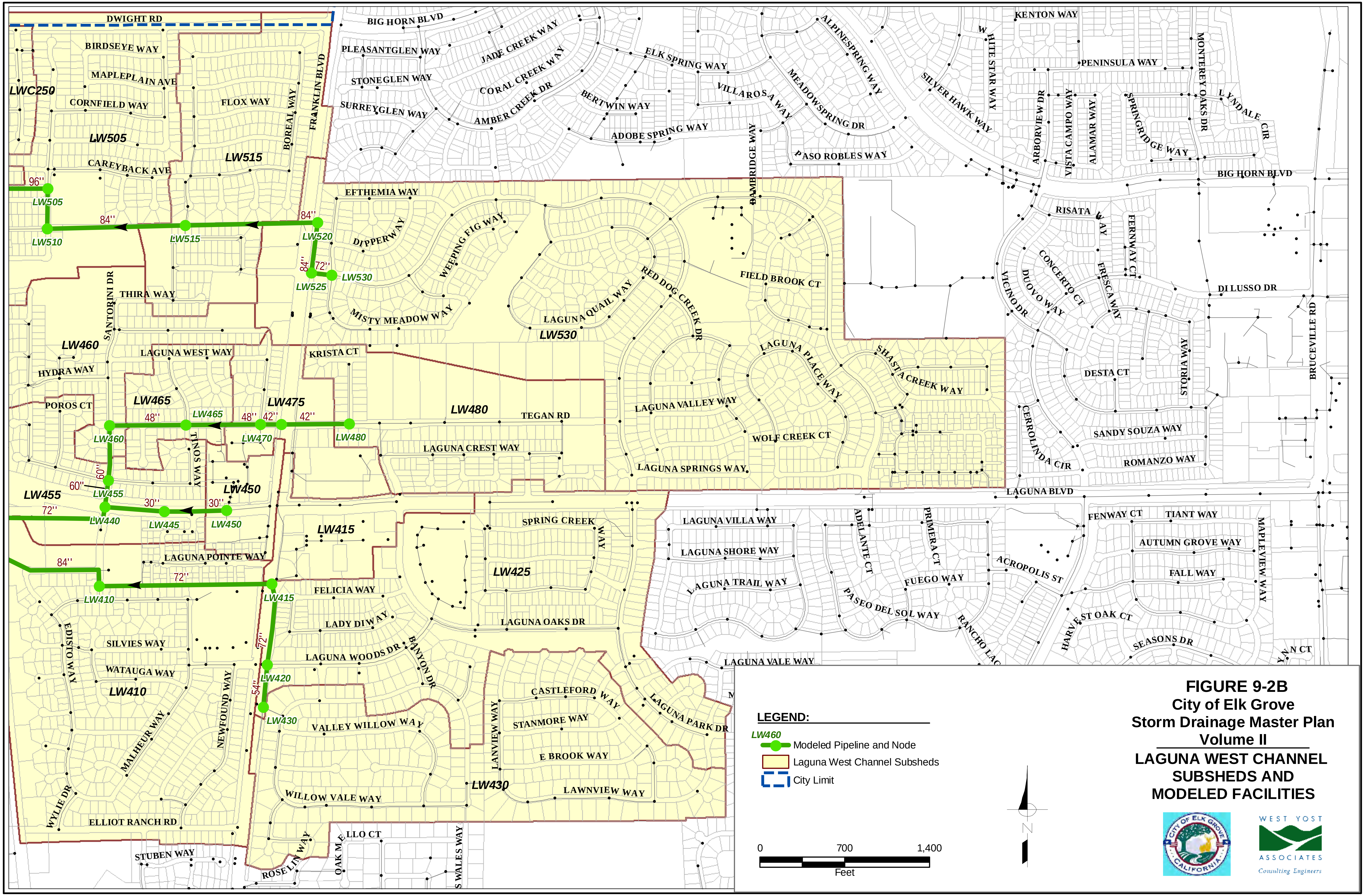
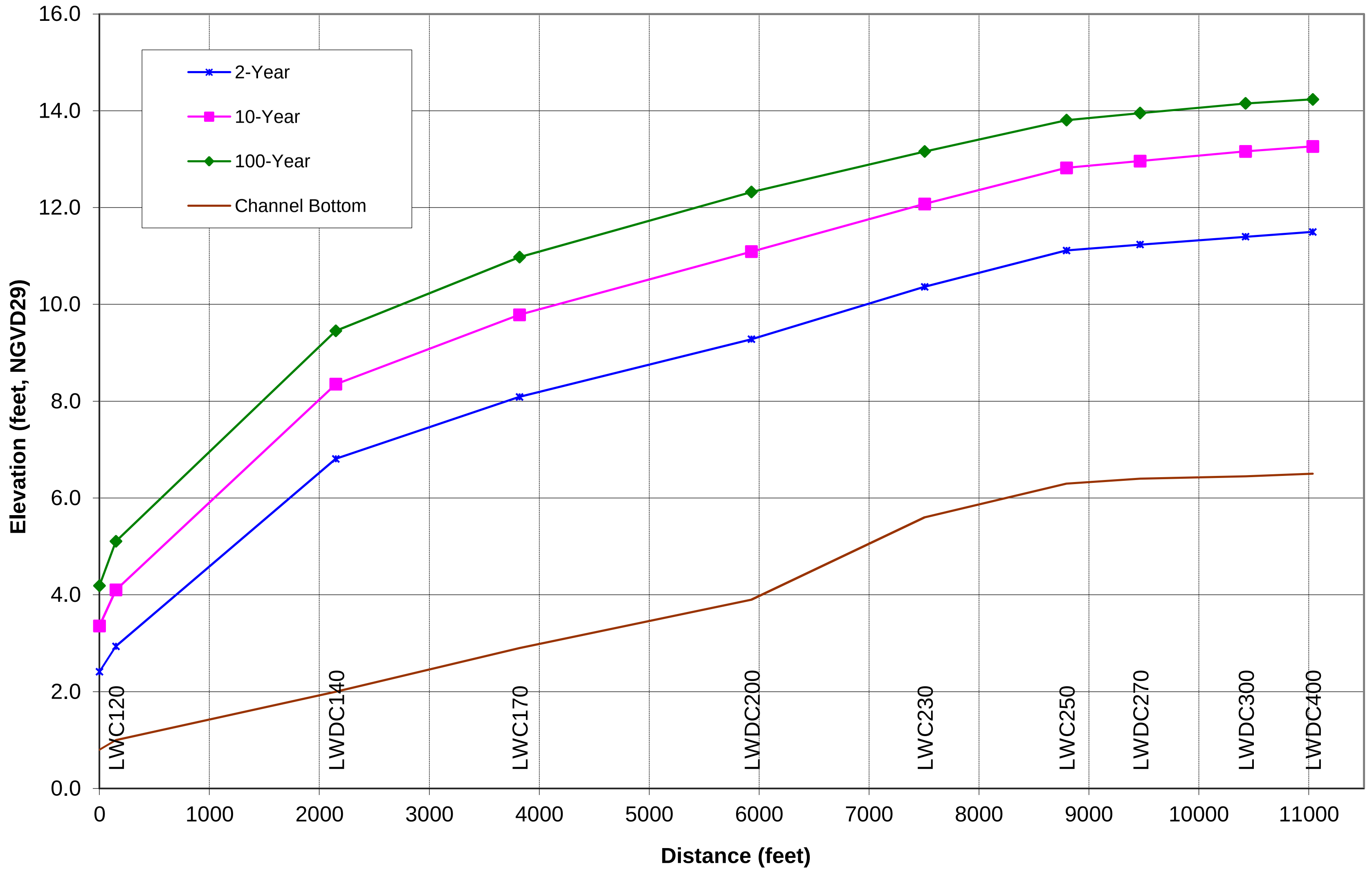


FIGURE 9-2B
City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA WEST CHANNEL SUBSHEDS AND MODELED FACILITIES

WEST YOST ASSOCIATES
Consulting Engineers

FIGURE 9-3
Laguna West Channel Water Surface Profiles



CHAPTER 10. LAGUNA WEST LAKES

WATERSHED DESCRIPTION

The Laguna West Lakes watershed lies in the western part of the City and covers approximately 740 acres. The area is generally bounded by Interstate 5 to the west, the Lakeside development to the south, the Western Pacific Railroad to the east, and the Laguna West Channel watershed to the north (see Figure 10-1). The watershed is drained by an underground pipe network that conveys runoff to a system of lakes, a detention basin, and a pump station. Runoff within the watershed is generally conveyed from east to west and is ultimately discharged through a levee along the west side of the watershed that protects the area from the floodplain in the Beach Stone Lakes area on the west side of Interstate 5.

EVALUATION OF EXISTING FACILITIES

The existing facilities that were evaluated in this watershed include the lake and detention basin system, the pump station, and a number of trunk pipelines. A description of the evaluation is provided below.

Hydrologic Analysis for Existing Facilities

A hydrologic analysis was performed to determine the 2-year, 10-year and 100-year flows for buildout conditions. Because the watershed is nearly entirely built out, only the buildout condition flows were calculated. For the hydrologic modeling, the watershed was divided into 24 subsheds as shown on Figure 10-2. Table 10-1 presents the key hydrologic parameters for each subshed for buildout conditions. Table 10-2 presents the resultant peak flows from each subshed for the 2-year, 10-year, and 100-year storms. Note that the SacCalc models were only used to calculate the flows from each subshed before they enter the main drainage system. These inflows were combined and routed within the drainage system using a hydraulic model as discussed below.

Hydraulic Analysis of Existing Facilities

A hydraulic analysis was performed to determine the peak flows and water surface elevations within key portions of the drainage system for the 2-year, 10-year, and 100-year storm events. An unsteady-state analysis was performed using the XPSWMM modeling software. Figure 10-2 provides a plan showing the major drainage facilities included in the model. Descriptions of these facilities are provided below.

Trunk Pipelines

The watershed is served by an underground pipe network that delivers runoff to one of the lakes or the detention basin. Pipelines with diameters of 27 inches or larger that lie within or cross under an arterial roadway were included in the hydraulic model. This included trunk pipelines in Harbour Point Drive and Laguna Boulevard. The pipeline sizes, lengths, and invert elevations were determined from as-built plans.

Table 10-1. Hydrologic Parameters for Laguna West Lakes Watershed

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Slope, ft/ft	Comm./ Office	HDR	MDR	Resd, 6-8 du/ac	Resd, 3-4 du/ac	Park	Open	Average % Imp.
					90%	80%	70%	50%	30%	5%	2%	
Buildout Conditions												
LW100	17.5	15.5	1,000	0.0010	0.7	0.0	0.0	0.0	0.0	16.8	0.0	8
LW110	28.7	13.0	3,173	0.0006	0.0	0.0	0.0	20.1	0.0	8.6	0.0	37
LW135	36.4	15.0	2,395	0.0008	0.0	0.0	0.0	33.1	0.0	3.3	0.0	46
LW140	41.5	15.0	2,409	0.0008	34.2	0.0	0.0	0.0	1.6	5.6	0.0	76
LW150	9.4	16.0	960	0.0006	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0
LW155	46.6	16.0	2,209	0.0003	3.8	0.0	0.0	0.0	0.0	0.7	0.0	7
LW160	21.8	17.0	1,958	0.0013	0.0	0.0	0.0	0.0	0.0	0.9	0.0	0
LW170	15.6	17.0	1,798	0.0011	0.0	0.0	0.0	0.0	0.0	0.6	0.0	0
EL	32.3	13.0	2,726	0.0008	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0
LWLE10	18.5	15.0	1,014	0.0020	0.2	6.3	6.2	0.1	2.7	3.1	0.0	57
LWLE20	22.7	15.0	1,014	0.0020	0.5	16.1	3.2	0.0	0.0	2.1	0.0	69
LWLE30	9.1	15.0	817	0.0024	0.1	8.6	0.0	0.0	0.0	0.2	0.0	76
LWLE40	43.1	16.0	1,983	0.0020	0.5	0.0	0.0	29.0	5.2	8.2	0.0	39
LWLE50	34.6	16.0	1,877	0.0021	0.0	0.0	0.0	28.8	5.1	0.6	0.0	46
LWLE60	12.9	15.5	1,038	0.0029	0.0	0.0	0.0	1.7	9.9	1.2	0.0	30
LWLE80	78.1	18.0	3,351	0.0024	0.0	0.0	0.0	51.8	15.1	10.9	0.0	40
WL	27.3	13.0	3,523	0.0008	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0
LWLW10	59.3	15.0	2,794	0.0021	1.9	9.9	8.8	21.1	5.8	6.6	0.0	48
LWLW20	22.9	13.0	1,197	0.0017	0.0	0.0	0.0	0.0	22.8	0.1	0.0	30
LWLW30	42.7	14.0	2,414	0.0017	0.0	0.0	0.0	30.3	11.8	0.6	0.0	44
LWLW40	11.8	15.0	1,082	0.0018	0.0	10.8	0.0	0.0	0.0	1.0	0.0	74
LWLW50	16.3	15.0	1,049	0.0019	2.2	9.7	2.8	0.0	0.0	1.6	0.0	72
LWLW60	60.6	14.0	3,470	0.0012	0.0	0.0	0.0	49.4	4.5	6.7	0.0	44
LWLW70	30.0	13.0	2,374	0.0008	0.0	0.0	0.0	19.4	5.7	4.8	0.0	39

Table 10-2. Calculated Subshed Flows - Laguna West Lakes Watershed

Subshed	Area, acres	Buildout Condition Flows, cfs	
		2-Year	10-Year
LW100	17.5	9	19
LW110	28.7	12	23
LW135	36.4	17	34
LW140	41.5	23	45
LW150	9.4	7	14
LW155	46.6	26	52
LW160	21.8	14	29
LW170	15.6	10	19
EL	32.3	19	37
LWLE10	18.5	12	25
LWLE20	22.7	18	37
LWLE30	9.1	8	15
LWLE40	43.1	22	45
LWLE50	34.6	19	38
LWLE60	12.9	8	16
LWLE80	78.1	34	68
WL	27.3	15	29
LWLW10	59.3	29	58
LWLW20	22.9	13	26
LWLW30	42.7	21	42
LWLW40	11.8	9	18
LWLW50	16.3	12	24
LWLW60	60.6	25	49
LWLW70	30.0	14	27

Lakes and Detention Basin

The watershed contains two large man-made lakes that act as stormwater storage and conveyance facilities. In addition, the lakes provide stormwater quality. There is a third lake situated between the main lakes, but this lake is small and does not provide a significant volume of storage. During storm events, runoff flows from the East Lake to the Middle Lake and then to the West Lake via weirs between the lakes. Runoff from the West Lake is then discharged over a weir and into a set of culverts under Harbour Point Drive. These culverts discharge into Detention Basin L-3 on the west side of the road. Runoff that is not discharged directly into one of the lakes is discharged into a sedimentation and water quality pond adjacent to Harbour Point Drive and ultimately into Detention Basin L-3. A schematic of the lake system is shown on Figure 10-3.

Pump Station D-53

Runoff entering Detention Basin L-3 is discharged out of the watershed by Pump Station D-53. This pump station contains three 100 horsepower pumps, each with a rated capacity of 35 cfs. The pump stations discharge runoff through a levee to the east of the Interstate 5. At that point, runoff is conveyed under Interstate 5 through two 72-inch pipes to the Beach Stone Lakes area, which has a 100-year water surface elevation of 16.0 feet. There are also gravity outlets that can convey runoff from Detention Basin L-3 to the west side of the levee when the tailwater elevation is low enough. For this analysis, it was conservatively assumed that the tailwater elevation would be too high to allow gravity outflow.

Calculated water surface elevations and peak flows for the 2-year, 10-year, and 100-year storm events are summarized on Tables 10-3 and 10-4. As Table 10-3 shows, the performance criteria for existing drainage systems are met at all locations. The 10-year water surface elevation is below the top of curb at all locations and the 100-year water surface elevation is below building pads.

EVALUATION OF FUTURE FACILITIES

The Laguna West Lakes watershed is almost completely developed and no future major drainage facilities are anticipated to serve new development.

PRELIMINARY IMPROVEMENTS

The results of the hydrologic analysis indicate that the existing major drainage facilities serving this watershed provide adequate flood protection. No major new facilities are required since the watershed is nearly built-out. As a result, no drainage improvements are recommended for the Laguna West Lakes watershed.

Table 10-3. Calculated Water Surface Elevations - Laguna West Lakes Watershed (NGVD29)

Node Name	Notes	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
LW100	Detention Basin L-3	n/a	14.0	8.6	9.0	11.5	-	-
LWLW00	West Lake	n/a	15.0	12.1	12.5	13.0	-	-
LWML00	Middle Lake	n/a	16.0	12.9	13.2	13.5	-	-
LWLE00	East Lake	n/a	16.0	13.6	13.9	14.2	-	-
LW105	Pipe System	17.2	16.0	8.6	9.0	11.5	-	-
LW110		14.1	16.0	8.6	9.5	11.5	-	-
LW115		13.3	16.0	8.6	9.7	11.5	-	-
LW120		12.7	14.0	8.6	9.9	11.5	-	-
LW125		13.2	14.0	8.6	10.4	12.1	-	-
LW130		13.5	16.0	8.6	12.2	13.5	-	-
LW135		14.1	16.0	8.8	14.0	15.1	-	-
LW140		14.7	16.0	8.6	9.0	11.5	-	-
LW145		15.7	16.0	8.6	9.6	12.1	-	-
LW150		12.8	16.0	8.6	9.9	12.6	-	-
LW155		16.3	16.0	8.6	10.7	14.3	-	-
LW160		13.3	16.0	8.6	10.8	13.3	-	-
LW165		14.7	16.0	9.0	11.3	13.4	-	-
LW170		14.3	14.5	9.9	12.9	14.0	-	-

Table 10-4. Calculated Peak Flows - Laguna West Lakes Watershed

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
RLW105	LW105	LW100	Pipe	100	195	255
RLW110	LW110	LW105	Pipe	28	54	63
RLW115	LW115	LW110	Pipe	17	31	33
RLW120	LW120	LW115	Pipe	17	31	33
RLW125	LW125	LW120	Pipe	17	31	33
RLW130	LW130	LW125	Pipe	17	31	29
RLW135	LW135	LW130	Pipe	17	32	29
RLW140	LW140	LW105	Pipe	74	150	200
RLW145	LW145	LW140	Pipe	52	106	134
RLW150	LW150	LW145	Pipe	7	13	19
RLW155	LW155	LW145	Pipe	26	52	74
RLW160	LW160	LW145	Pipe	22	45	49
RLW165	LW165	LW160	Pipe	9	19	20
RLW170	LW170	LW165	Pipe	9	18	20
OLRLW130	LW130	LW125	Overland	0	0	5
OLRLW135	LW135	LW130	Overland	0	3	12

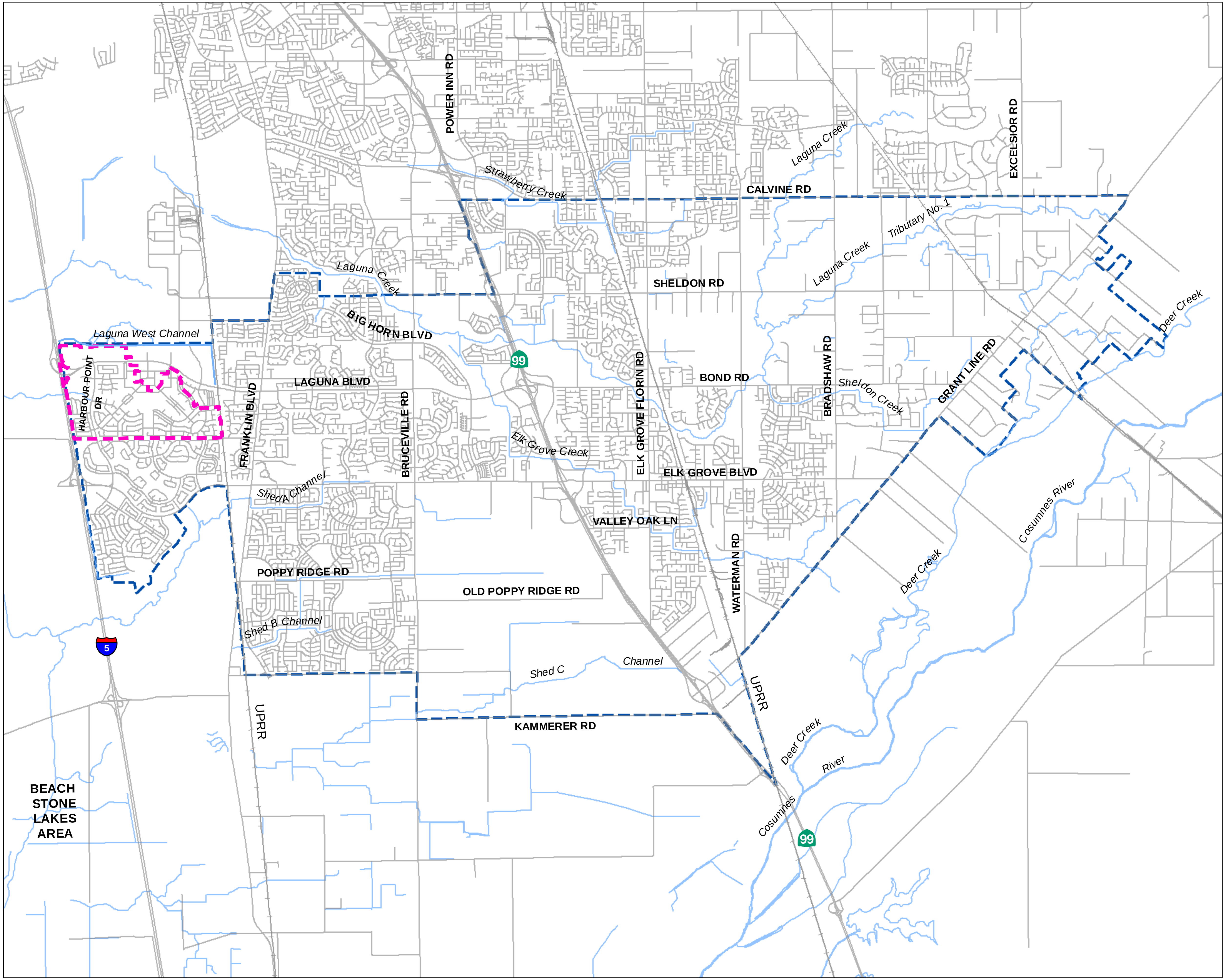


FIGURE 10-1

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA WEST LAKES LOCATION MAP

N

0 0.5 1 2
Scale in Miles

NOTES:

LEGEND:

City Limit

Laguna West Lakes Watershed

WEST YOST ASSOCIATES
Consulting Engineers

FIGURE 10-2

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA WEST LAKES SUBSHEDS AND MODELED FACILITIES

N

0 800 1,600
Feet

NOTES:

LEGEND:

LW165 ●━●	Modeled Node and Pipeline
▨	Modeled Lake or Detention Basin
▭	Laguna West Lakes Subsheds
⌜⌝	City Limit
□	Modeled Pump Station

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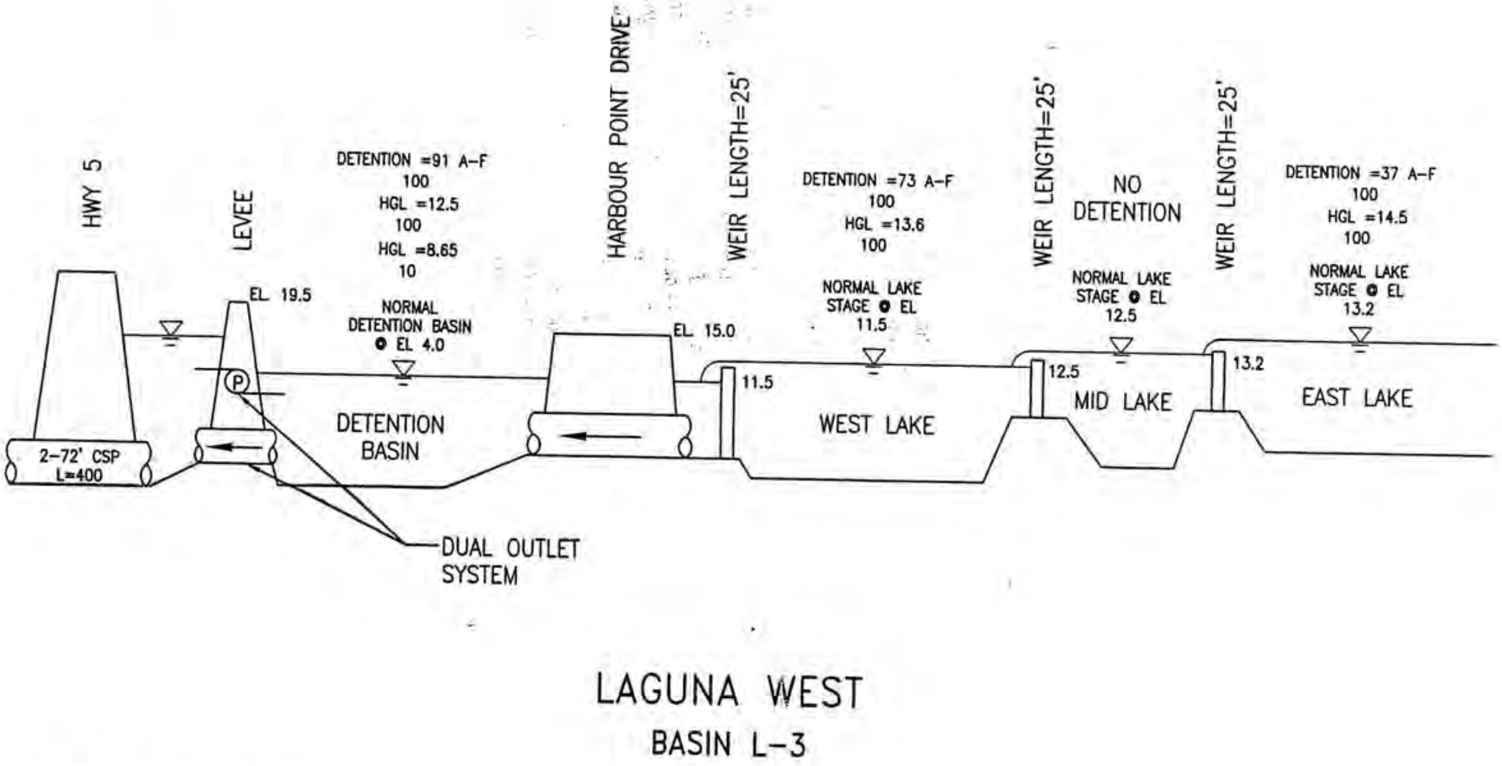

Note: Graphic from PSOMAS, Inc.

Figure 10-3

City of Elk Grove
Storm Drainage Master Plan Volume II

LAGUNA WEST LAKES
LAKE SYSTEM SCHEMATIC

WEST YOST
ASSOCIATES
Consulting Engineers

CHAPTER 11. LAKESIDE

WATERSHED DESCRIPTION

The Lakeside watershed lies in the western part of the City and covers approximately 660 acres. The area is generally bounded by Interstate 5 to the west, Elk Grove Boulevard to the south, the Union Pacific Railroad to the east, and the Laguna West Lakes watershed to the north (see Figure 11-1). The watershed is drained by an underground pipe network that conveys runoff to a man-made lake and a pump station. Runoff within the watershed is generally conveyed from the northwest to the southeast and is ultimately discharged out of the watershed and into the Shed A Channel south of the watershed. The watershed is protected from the Beach Stone Lakes floodplain by a levee along it western boundary.

EVALUATION OF EXISTING FACILITIES

The existing facilities that were evaluated in this watershed include the lake, a trunk pipeline, and a pump station. The evaluation is described below.

Hydrologic Analysis for Existing Facilities

A hydrologic analysis was performed to determine the 2-year, 10-year and 100-year flows for buildout conditions. Because the watershed is nearly completely developed, only the buildout condition flows were calculated. For the hydrologic modeling, the watershed was divided into 8 subsheds as shown on Figure 11-2. Table 11-1 presents the key hydrologic parameters for each subshed for buildout conditions. Table 11-2 presents the resultant peak flows from each subshed for the 2-year, 10-year, and 100-year storms. Note that the SacCalc models were only used to calculate the flows from each subshed before they enter the main drainage system. These inflows were combined and routed within the drainage system using a hydraulic model as discussed below.

Hydraulic Analysis of Existing Facilities

A hydraulic analysis was performed to determine the peak flows and water surface elevations within key portions of the drainage system for the 2-year, 10-year, and 100-year storm events. An unsteady-state analysis was performed using the XPSWMM modeling software. Figure 11-2 provides a plan showing the major drainage facilities included in the model. Descriptions of these facilities are provided below.

Trunk Pipelines

For the SDMP, pipelines with diameters of 27 inches or larger that lie within or cross under an arterial roadways were included modeled. Within this watershed, one pipe exists that meets that criterion, a 72-inch pipe that conveys runoff from the Lakeside Lake to City Pump Station D-50 (See Figure 11-2). The pipeline size, length, and invert elevations were determined from as-built plans.

Table 11-1. Hydrologic Parameters for Lakeside Watershed

Subshed	Area, acres	Mean Elev., ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land-Use, acres and Percent Imperviousness							Average % Imp.
						Highway, Parking	Comm./ Office	Ind.	HDR	Resd, 6-8 du/ac	Resd, 3-4 du/ac	Park	
						95%	90%	85%	80%	50%	30%	5%	
Buildout Conditions													
Lake	47.0	12	4,294	2,070	0.0008	47.0	0.0	0.0	0.0	0.0	0.0	0.0	95
LS100	135.6	14	3,438	1,563	0.0023	3.4	31.8	40.0	17.7	38.9	0.0	3.9	73
LS200	23.4	14	1,549	704	0.0019	3.9	0.0	0.0	0.0	0.0	17.6	1.8	39
LS300	60.0	14	2,126	966	0.0019	6.9	0.0	0.0	0.0	21.9	31.0	0.2	45
LS400	84.9	14	5,069	2,304	0.0012	0.0	0.0	0.0	0.0	84.2	0.0	0.8	50
LS500	184.3	16	5,310	2,413	0.0015	23.2	0.0	0.0	14.7	95.3	26.3	24.8	49
LS600	82.7	13	2,889	1,313	0.0021	6.7	0.0	0.0	0.0	73.9	0.0	2.2	52
LSPS	38.5	16	2,699	1,227	0.0082	0.0	0.6	0.0	5.5	0.1	0.0	32.3	17

Table 11-2. Calculated Subshed Flows – Lakeside Watershed

Subshed	Area, acres	Buildout Condition Flows, cfs		
		2-Year	10-Year	100-Year
Lake	47.0	24	47	66
LS100	135.6	67	129	184
LS200	23.4	13	26	38
LS300	60.0	30	60	86
LS400	84.9	33	64	90
LS500	184.3	70	135	194
LS600	82.7	41	80	114
LSPS	38.5	16	32	53

Lakeside Lake

The watershed contains a man-made lake that provides stormwater storage and conveyance, as well as stormwater quality treatment. Based on 2-foot contour mapping, the lake covers approximately 32 acres at its normal pool elevation of 10.0 feet. At the 100-year water surface elevation of 12.4 feet, the lake provides over 85 acre-feet of storage above the normal pool elevation.

Runoff from the lake is controlled by a weir near the intersection of Southlake Drive and Grand Point Lane. Water flowing over the weir enters the 72-inch pipeline described above, which conveys runoff to the D-50 Pump Station.

Pump Station D-50

Pump Station D-50 is located at the southeast end of the watershed adjacent to Elk Grove Boulevard. The pump station contains three 125 horsepower pumps, each with a rated capacity of 43 cfs. The pumps discharge runoff into an outlet channel that carries runoff for approximately 1,200 feet before entering the Shed A Channel.

A schematic profile of the lake system is shown on Figure 11-3.

Results of Hydraulic Analysis

Calculated water surface elevations and flows for the 2-year, 10-year, and 100-year storm events are summarized on Tables 11-3 and 11-4. As Table 11-3 shows, the performance criteria for existing drainage systems are met at all modeled locations. The 10-year water surface elevations in the lake and the 72-inch trunk pipe are below street elevation and the 100-year water surface elevations are below building pads.

EVALUATION OF FUTURE FACILITIES

The Lakeside watershed is almost completely developed and no future major drainage facilities are anticipated to serve new development.

PRELIMINARY IMPROVEMENTS

The results of the hydrologic and hydraulic analyses indicate that the existing major drainage facilities serving this watershed provide adequate flood protection. No major new facilities are required since the watershed is nearly built-out. Based on this, no drainage improvements are recommended for the Lakeside watershed.

Table 11-3. Calculated Water Surface Elevations for Lakeside Watershed (NGVD29)

Node Name	Notes	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
LSL00	Lake	n/a	14.0	10.9	11.5	12.4	—	—
LS900	Grand Pt. Ln./Shoreline Dr.	13.7	14.0	7.4	11.2	12.3	—	—
LSPS	Elk Grove Blvd./Pump Sta.	16.3	16.0	7.0	10.8	10.9	—	—

Table 11-4. Calculated Peak Flows - Lakeside Watershed

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
RLS900	LS900	LSPS	Pipe	85	134	136

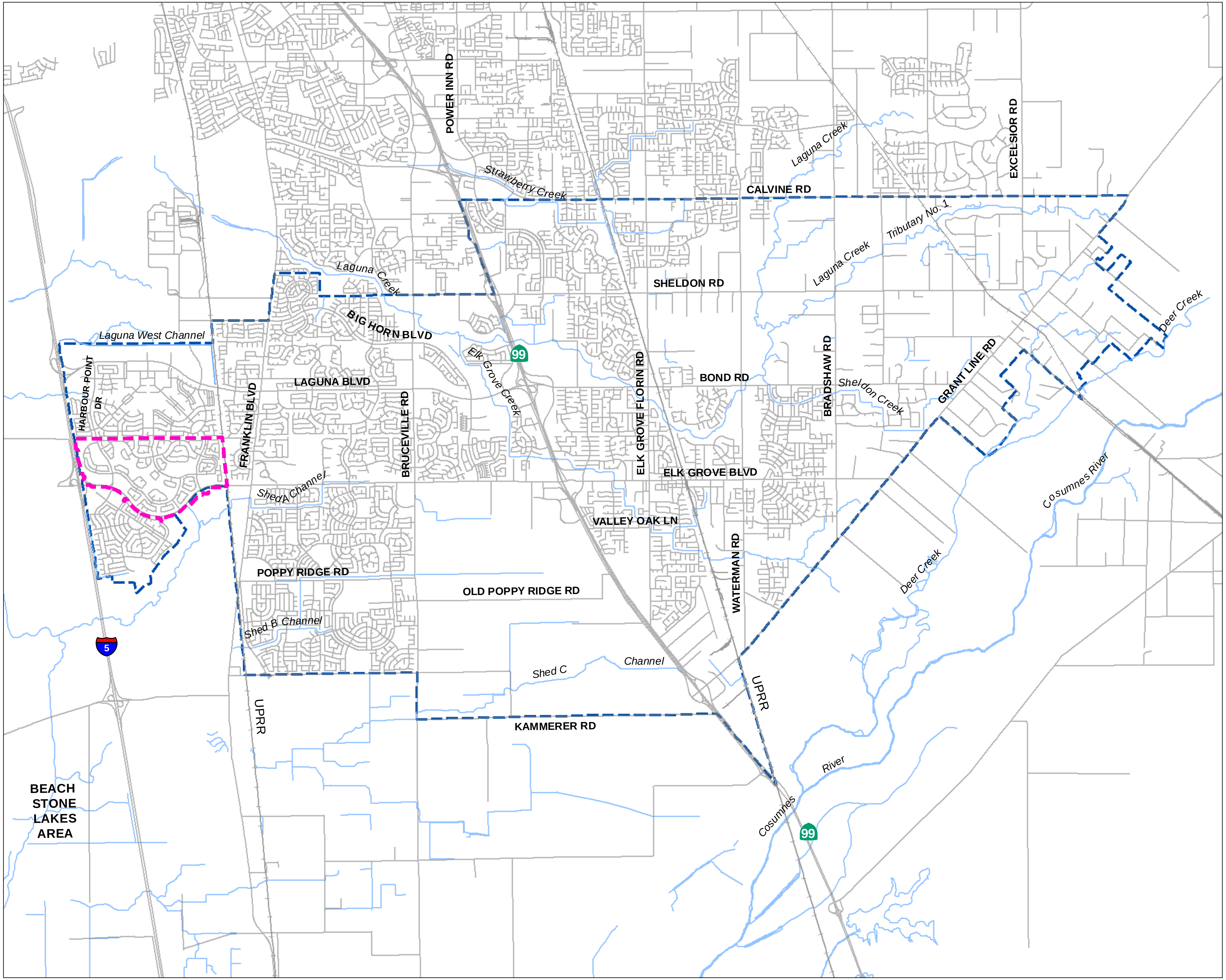

FIGURE 11-1

City of Elk Grove Storm Drainage Master Plan Volume II

LAKESIDE LOCATION MAP

N

0 0.5 1 2

Scale in Miles

NOTES:

LEGEND:

City Limit

Lakeside Watershed

WEST YOST ASSOCIATES

Consulting Engineers

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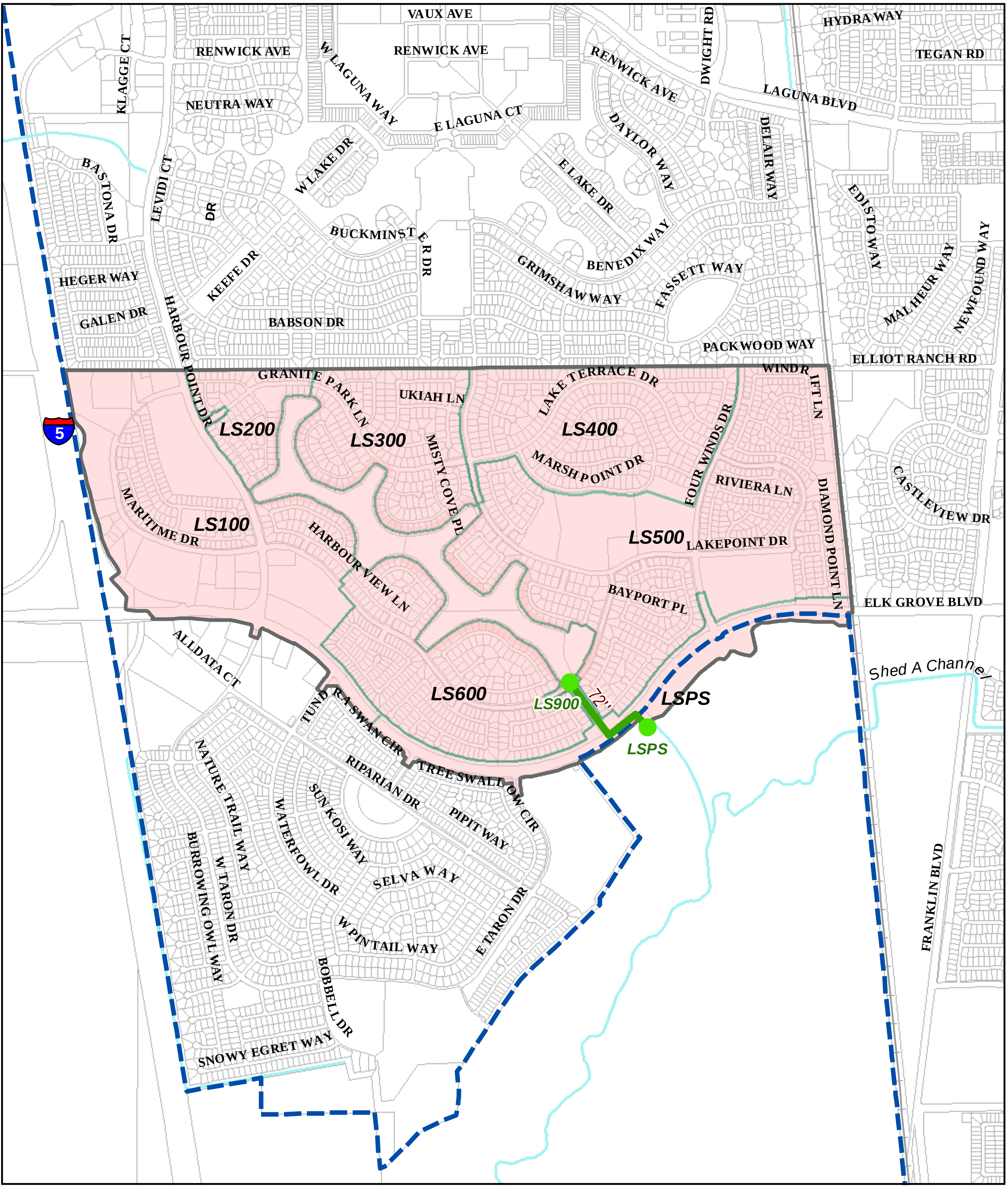

LEGEND:

LS900 Modeled Node and Pipeline

🟥 Lakeside Subsheds

City Limit

N

0 750 1,500

Scale in Feet

FIGURE 11-2

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAKESIDE SUBSHEDS AND MODELED DRAINAGE FACILITIES

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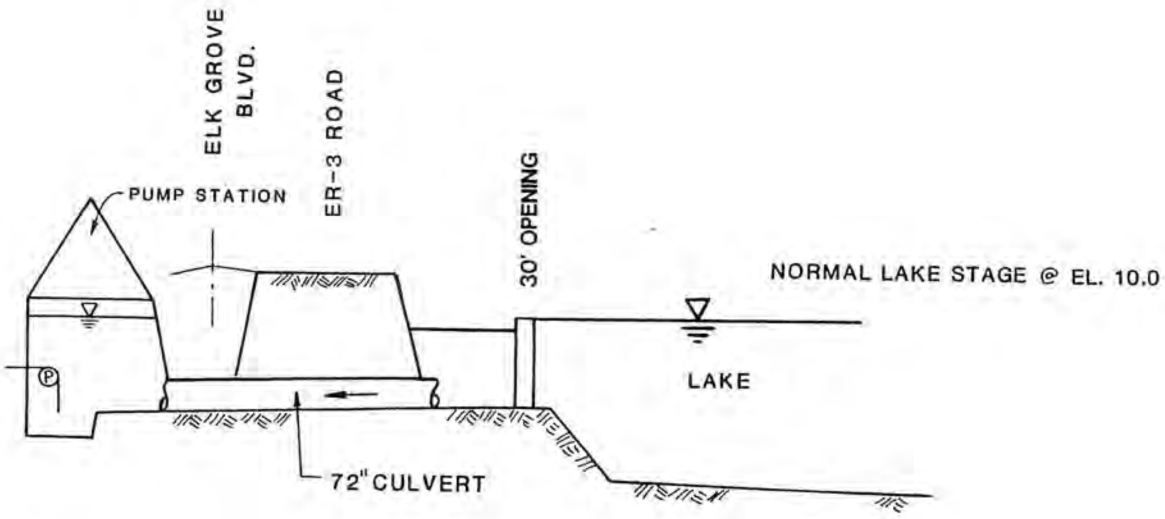
Consulting Engineers

CITY OF ELK GROVE CALIFORNIA



LAKESIDE

SCHEMATIC OF SYSTEM HYDRAULICS

Note: Figure from Reference 21.

Figure 11-3

City of Elk Grove
Storm Drainage Master Plan Volume II

LAKESIDE
LAKE SYSTEM SCHEMATIC

WEST YOST
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CHAPTER 12. LAGUNA STONELAKE

WATERSHED DESCRIPTION

The Laguna Stonelake watershed lies in the western part of the City and covers approximately 430 acres. The area is generally bounded by Interstate 5 to the west, Elk Grove Boulevard to the north, the Shed A Channel to the south and east (see Figure 12-1). Levees have been constructed along the west and south sides of the watershed to protect the interior from the Beach Stone Lakes floodplain. The interior watershed is drained by an underground pipe network that conveys runoff to a detention basin at the south end. Runoff entering the detention basin is pumped to an outfall channel that conveys runoff to the Shed A channel.

EVALUATION OF EXISTING FACILITIES

The existing facilities that were evaluated in this watershed include the detention basin and pump station. A description of the evaluation is provided below.

Hydrologic Analysis for Existing Facilities

A hydrologic model was prepared using SacCalc to evaluate the detention basin and pump station system serving the watershed. A hydraulic model was not prepared for this watershed since there are no open channels serving the watershed and no trunk pipelines meeting the evaluation criterion. The hydrologic models were used to calculate the 2-year, 10-year and 100-year flows for buildout conditions. Because the watershed is essentially built-out, only the buildout condition flows were calculated. For the hydrologic modeling, the Laguna Stonelake watershed was divided into 2 subsheds as shown on Figure 12-2. Table 12-1 presents the key hydrologic parameters for each subshed for buildout conditions.

The hydrologic models were used to route the flows through the detention basin and pump station system and to calculate water surface elevations in the detention basin during the three storm events. Descriptions of the detention basin and pump station are provided below.

Detention Basin

The Laguna Stonelake watershed drains to City Detention Basin L-4. This basin provides flood control storage and stormwater quality treatment. The basin is designed with a permanent pool at elevation -8.0 feet. Above that elevation, the basin provides approximately 78 acre-feet of flood control storage volume at the 100-year pool elevation of 5.1 feet. The elevation-volume relationship for the detention basin is provided on Table 12-2.

Table 12-1. Hydrologic Parameters for Laguna Stonelake Watershed

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land Use, acres and Percent Imperviousness					Avg. % Imp.
						Comm./ Office	Ind.	HDR	Resd, 6-8 du/ac	Park	
						90%	85%	80%	50%	5%	
Buildout Conditions											
L1	264.5	11.0	5,664	3,147	0.0013	37.0	18.5	0.0	196.5	12.5	56
L2	163.8	13.0	3,619	2,011	0.0020	1.8	0.0	26.9	102.5	32.6	46

Table 12-2. Detention Basin L-4 Storage Data

Elevation, feet (NGVD29)	Depth, feet	Storage, acre-feet	Storage above Permanent Pool, acre-feet
-13.5	0	0	0
-8	5.5	9.6	0
-6	7.5	17	7.4
-4	9.5	27	17.4
-2	11.5	39	29.4
0	13.5	52	42.4
2	15.5	65	55.4
4	17.5	79	69.4
7	20.5	102.9	93.3

Pump Station D-51

Pump Station D-51 is located at the southeast end of the watershed adjacent to the detention basin. The pump station contains four 100 horsepower pumps, each with a rated capacity of 20 cfs. There is also a 30 horsepower low flow pump with a rated capacity of 7.8 cfs. Three of the pumps are operated during a storm event and the fourth pump is used as a stand-by unit. The low flow pump does not operate during a storm event. The pump station discharges into an outlet channel that carries the runoff for approximately 1,000 feet before entering the Shed A Channel. Table 12-3 presents the operating elevations of the pump station.

Table 12-3. Pump Station D-51 Operating Elevations

Water Surface Elevation, feet (NGVD29)	Action
Rising Water Surface Elevation	
-7.8	Low flow pump on.
-4.0	Low flow pump off.
-4.0	Pump 1 on.
1.0	Pump 2 on.
2.5	Pump 3 on.
Falling Water Surface Elevation	
-6.0	Pumps 2 and 3 off.
-8.0	Pump1 off.
-8.2	Low flow pump off.

Results of the Hydrologic Analysis

Table 12-4 presents the resultant peak flows from each subshed for the 2-year, 10-year, and 100-year storms. The calculated water surface elevations in the detention basin for the three storm events are presented on Table 12-5. The results indicate that the detention basin and pump station effectively protect the Laguna Stonelake area from flooding from runoff generated in the watershed. The 100-year water surface elevation is at least 5 feet below the lowest pad elevation nearby and is also well below the low ground elevation of the adjacent Laguna Stonelake Community Park.

As indicated above, levees protect the Laguna Stonelake watershed from flood waters in the Beach Stone Lakes Area. Evaluation of the levees was not part of this SDMP. However, according to design documents, the levees surrounding the watershed were constructed to a minimum elevation of 19.5 feet, which provides over three feet of freeboard from FEMA 100-year water surface elevation of 16.0 feet.

EVALUATION OF FUTURE FACILITIES

The Laguna Stonelake watershed is almost completely developed and no major new drainage facilities are anticipated to serve new development.

PRELIMINARY IMPROVEMENTS

The results of the hydrologic analysis indicate that the existing detention and pumping facilities serving this watershed provide adequate flood protection and no major drainage facilities are anticipated in the future. Based on these findings, no drainage improvements are recommended for the Laguna Stonelake watershed.

Table 12-4. Calculated Subshed Flows for Laguna Stonelake Watershed

Subshed	Area, acres	Buildout Condition Flows, cfs		
		2-Year	10-Year	100-Year
L1	264.5	106	196	275
L2	163.8	75	141	208
L1L2(1)	428.3	176	325	459

(1) Combined flow into Detention Basin L-4.

Table 12-5. Calculated Water Surface Elevations in Detention Basin L3

Storm Event	Water Surface Elev., feet (NGVD29)
2-Year	-1.8
10-Year	1.5
100-Year	5.1

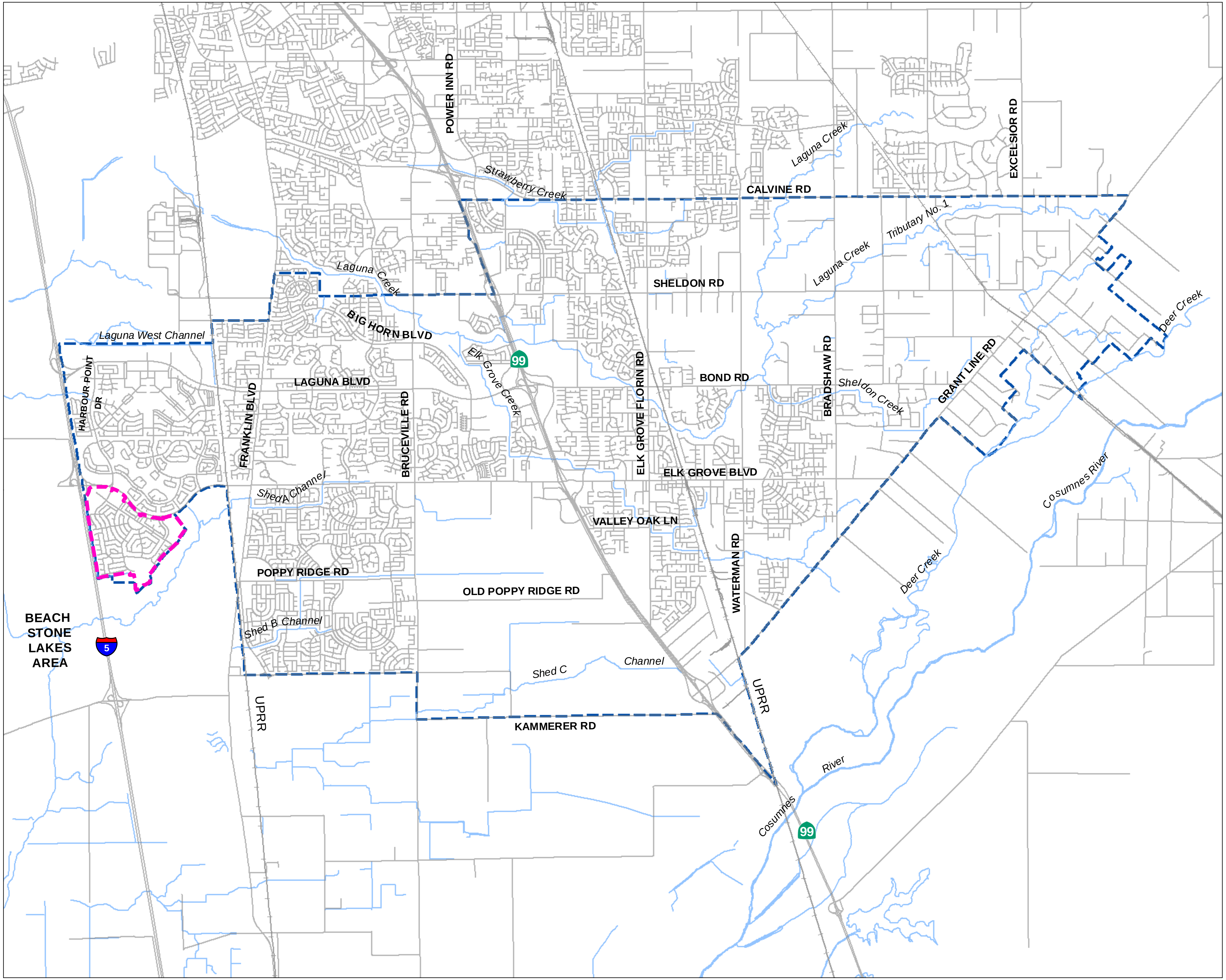

FIGURE 12-1

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA STONELAKE
LOCATION MAP

N

0 0.5 1 2
Scale in Miles

NOTES:

LEGEND:

- City Limit
- Laguna Stonelake Watershed

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Consulting Engineers

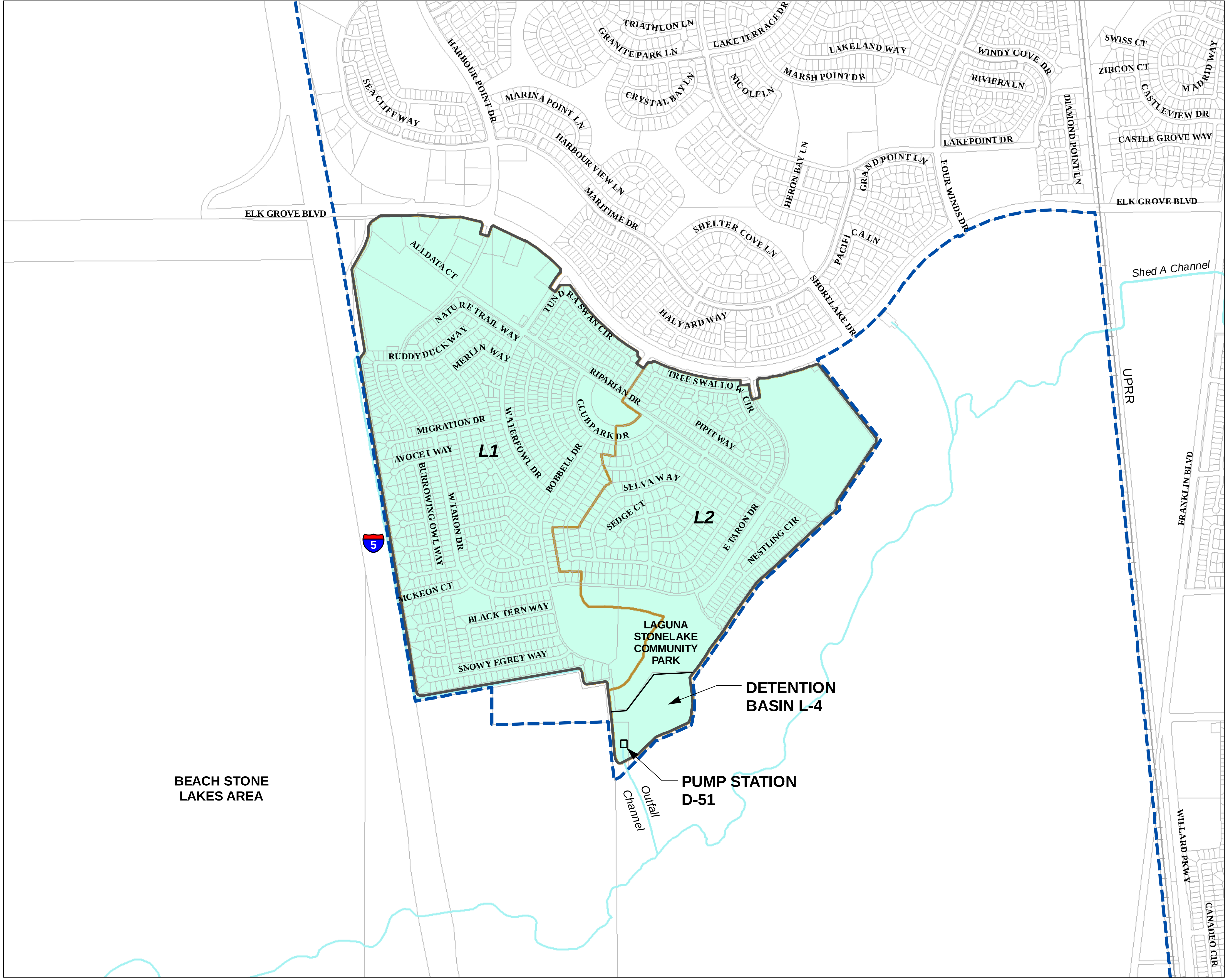


FIGURE 12-2

City of Elk Grove
Storm Drainage Master Plan
Volume II

LAGUNA STONELAKE SUBSHEDS

N

0 1,000 2,000
Feet

NOTES:

LEGEND:

🟩 Laguna Stonelake Subsheds

⬚ City Limit

CITY OF ELK GROVE CALIFORNIA

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 13. SHED A

WATERSHED DESCRIPTION

The Shed A watershed lies in the southwest part of the City and covers approximately 1,530 acres. The watershed lies east of the Union Pacific Railroad (UPRR) and straddles Elk Grove Boulevard (see Figure 13-1). The watershed includes the northern portion of the East Franklin Specific Plan area. The watershed is drained by an underground pipe system that conveys runoff to the Shed A Channel. The Shed A Channel carries runoff to the west, exits the City at the UPRR, and continues southwest to the Beach Stone Lakes area west of Interstate 5.

EVALUATION OF SHED A CHANNEL

No hydrologic or hydraulic analyses were performed for the Shed A Channel. The recent master plan data prepared for the East Franklin area is considered by the City to provide an adequate analysis of the channel and the channel has been constructed to its ultimate configuration. The original design profile for the channel is provided as Attachment 13A.

EVALUATION OF EXISTING PIPELINES

As indicated in Chapter 3, existing pipelines within the City's arterial roadways with diameters 27 inches and larger were subject to evaluation during this SDMP. Three existing pipelines in the Shed A watershed met this criterion. Figures 13-2 and 13-3 show the sizes and locations of the 3 pipelines.

Hydrologic Analysis of Existing Pipelines

SacCalc models were prepared to calculate the 2-year, 10-year, and 100-year flows into the pipe systems. Because the watersheds served by the 3 pipelines are nearly completely developed, flows were only calculated for buildout conditions. Figures 13-2 and 13-3 present the subshed boundaries used for the flow calculations. Table 13-1 presents the key hydrologic parameters for each subshed for buildout conditions. Table 13-2 presents the calculated peak flows from each subshed for the three storm events.

Hydraulic Analysis of Existing Pipelines

Hydraulic models were prepared to determine the flows and water surface elevations within the 3 existing pipelines for the 2-year, 10-year, and 100-year storm events. An unsteady-state model was prepared using the XPSWMM modeling software. Figures 13-2 and 13-3 show the pipelines included in the models. Calculated water surface elevations for the 2-year, 10-year, and 100-year storm events are summarized on Table 13-3. Calculated peak flows in the pipe systems are presented on Table 13-4. As Table 13-3 shows, the performance criteria for existing drainage systems are met at all locations. The 10-year water surface elevations are below the top of curb at all locations and the 100-year water surface elevations are below building pads.

Table 13-1. Hydrologic Parameters for Existing Pipeline Models A1-A3

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land-Use, acres and Percent Imperviousness								Average % Imp.
						Highway, Parking	Comm./Office	HDR	Resd, 8-10 du/ac	Resd, 6-8 du/ac	Resd, 4-6 du/ac	Resd, 3-4 du/ac	Park	
						95%	90%	80%	60%	50%	40%	30%	5%	
Buildout Conditions														
SA110	78.3	20	3,150	1,280	0.0004		0.0				54.8	19.6	3.9	36
SA115	82.5	20	3,420	1,100	0.0006		0.0	3.3			12.4	66.0	0.8	33
SA205	208.4	26	5,170	2,250	0.0012		0.0			20.8	156.3		31.3	36
SA220	22.0	24	1,100	500	0.0036		0.0				22.0			40
SA230	15.4	28	1,450	680	0.0015		0.0				15.4			40
SA235	54.2	30	3,120	960	0.0006		40.6	6.5			4.3		2.7	81
SA305	98.7	30	3,930	1,430	0.0010		19.7		4.9		74.0			51
SA315	35.4	30	3,440	1,440	0.0012		3.5				31.8			45
SA320	109.7	30	3,770	1,350	0.0011		0.0				109.7			40

Table 13-2. Calculated Subshed Flows for Existing Pipelines A1-A3

Subshed	Area, acres	Buildout Condition Flows, cfs		
		2-Year	10-Year	100-Year
SA110	78.3	33	65	93
SA115	82.5	36	71	101
SA205	208.4	79	154	223
SA220	22.0	16	32	48
SA230	15.4	9	19	28
SA235	54.2	30	59	86
SA305	98.7	46	91	128
SA315	35.4	17	33	47
SA320	109.7	50	99	140

EVALUATION OF FUTURE FACILITIES

The Shed A watershed is mostly developed and no new major drainage facilities are anticipated to serve new development.

PRELIMINARY IMPROVEMENTS

The existing channel and trunk pipelines in the Shed A watershed are considered to provide adequate flood protection to the watershed and no major facilities are anticipated for future development. Based on this, no drainage improvements are recommended for the Shed A watershed.

Table 13-3. Calculated Water Surface Elevations for Existing Pipelines A1-A3 (NGVD29)

Node Name	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	2-Year Water Surface Elevation, feet	10-Year Water Surface Elevation, feet	100-Year Water Surface Elevation, feet	10-Year Flooding Above Curb?	100-Year Pad Flooding?
Pipeline A1							
SA100	20.3	n/a	15.1	16.7	17.1	—	—
SA110	23.4	22.0	15.7	17.9	18.7	—	—
SA115	22.5	22.6	16.1	18.4	19.3	—	—
Pipeline A2							
SA200	20.4	21.0	17.3	17.3	18.4	—	—
SA205	20.0	21.7	17.7	18.8	20.7	—	—
SA220	22.2	24.0	18.4	21.8	22.7	—	—
SA225	25.4	26.0	18.6	22.9	23.1	—	—
SA227	24.0	26.0	19.0	23.6	23.7	—	—
SA230	25.3	28.0	19.3	25.0	25.0	—	—
SA235	27.2	29.4	20.5	26.5	27.1	—	—
Pipeline A3							
SA300	24.5	26.0	18.5	20.5	21.5	—	—
SA305	25.5	26.0	19.7	23.0	24.9	—	—
SA310	29.2	30.0	21.2	25.8	27.5	—	—
SA315	30.2	30.0	22.1	28.4	29.8	—	—
SA320	30.5	31.5	22.7	30.1	31.0	—	—

Table 13-4. Calculated Peak Flows - Shed A Pipelines A1-A3

Conduit Name	Upstream Node	Downstream Node	Conduit Type	2-Year Peak Flow, cfs	10-Year Peak Flow, cfs	100-Year Peak Flow, cfs
Pipeline A1						
RSA110.1	SA110	SA100	Pipe	65	101	119
RSA115.1	SA115	SA110	Pipe	35	51	59
Pipeline A2						
RSA205.1	SA205	SA200	Pipe	57	108	138
RSA220.1	SA220	SA205	Pipe	36	81	75
RSA225.1	SA225	SA220	Pipe	30	60	51
RSA227.1	SA227	SA225	Pipe	33	56	51
RSA230.2	SA230	SA227	Pipe	34	58	58
RSA235.1	SA235	SA230	Pipe	29	45	44
Link13	SA227	SA200	Overland	0	2	21
OLR235	SA235	SA230	Overland	0	0	5
Pipeline A3						
RSA305.1	SA305	SA300	Pipe	106	201	233
RSA310.1	SA310	SA305	Pipe	63	113	114
RSA315.1	SA315	SA310	Pipe	64	113	114
RSA320.1	SA320	SA315	Pipe	49	90	93
Link12	SA320	SA300	Overland	0	0	6
OLR320	SA320	SA315	Overland	0	0	6

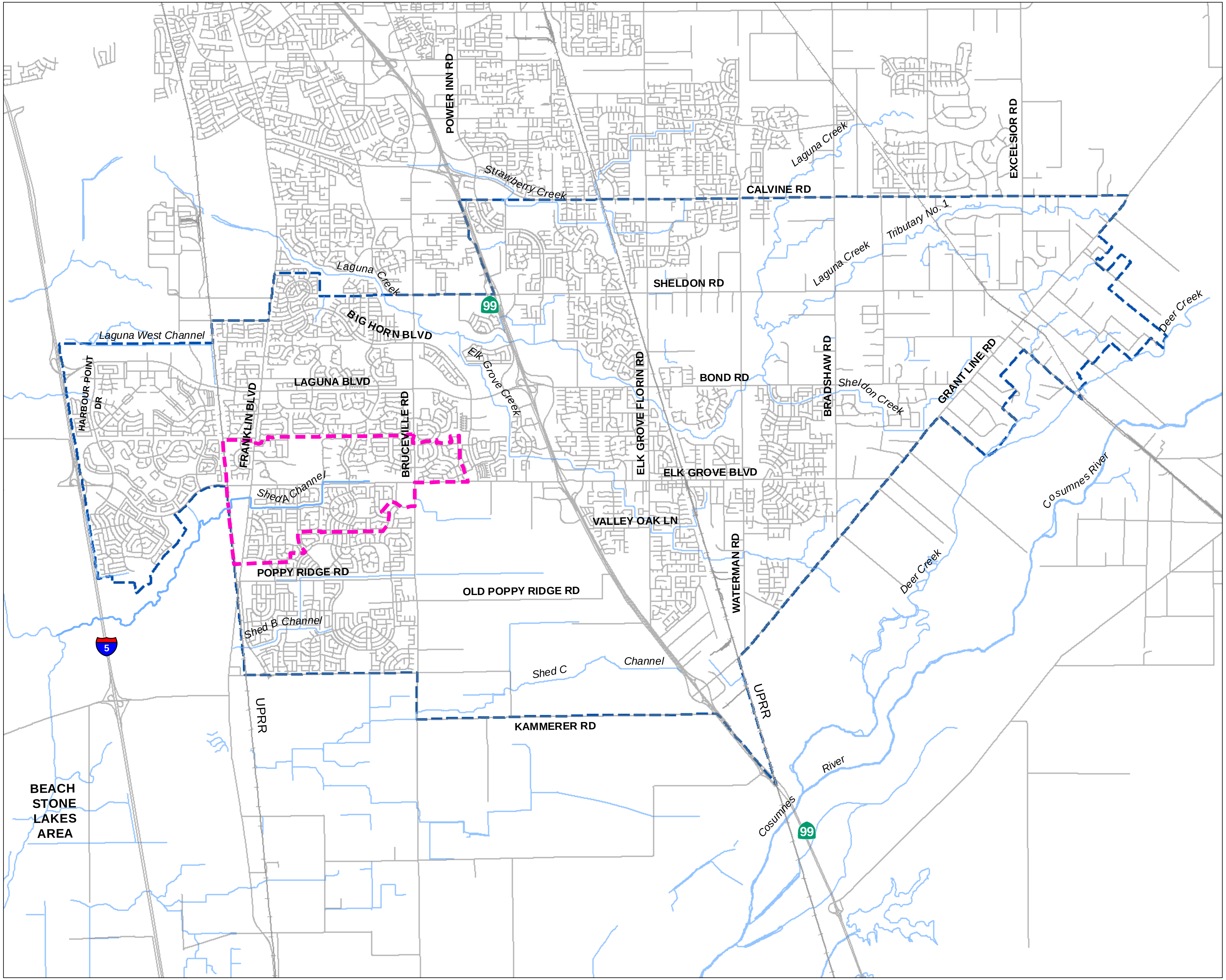


FIGURE 13-1

City of Elk Grove
Storm Drainage Master Plan
Volume II

SHED A
LOCATION MAP

N

0 0.5 1 2
Scale in Miles

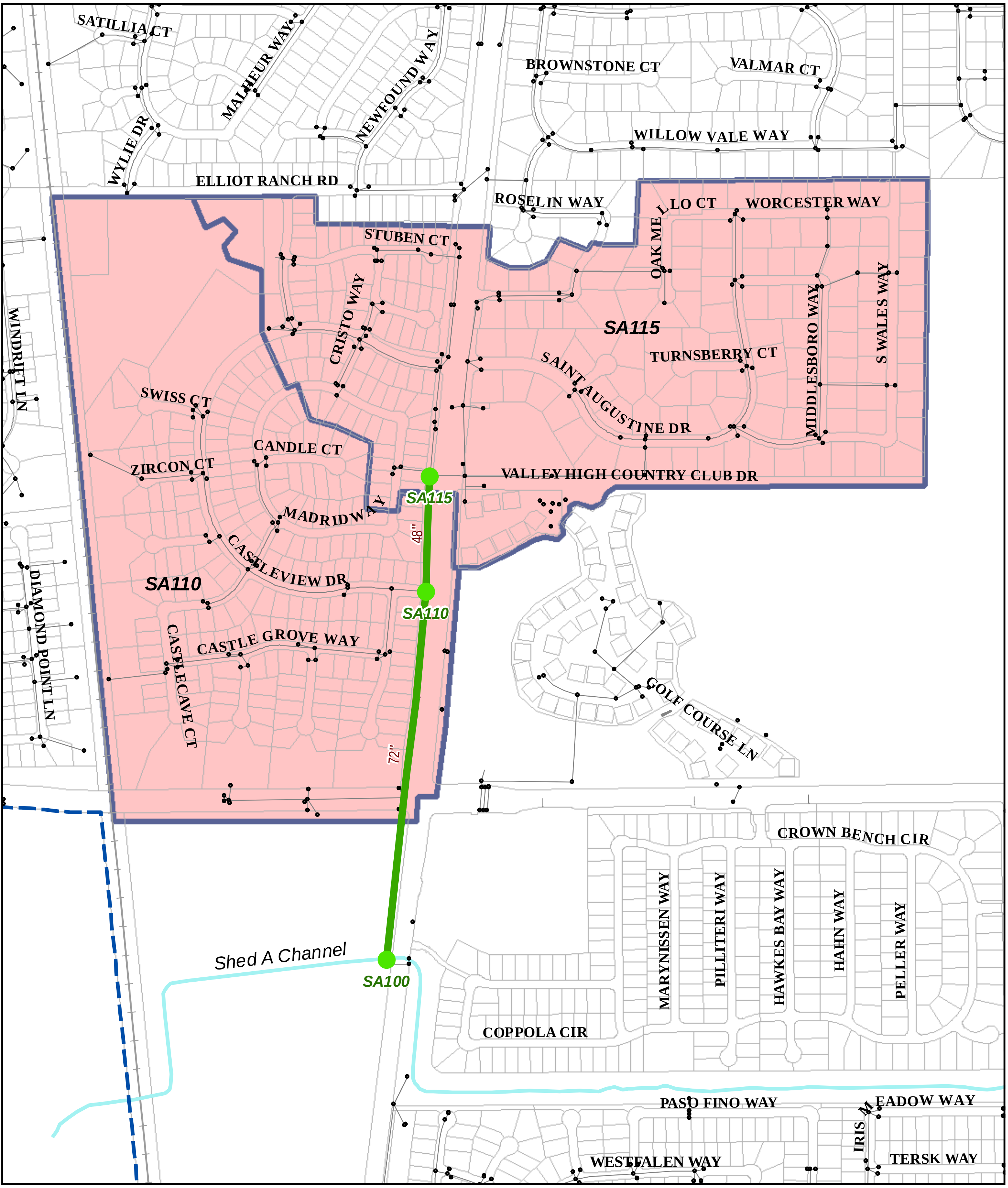
NOTES:

LEGEND:

- City Limit
- Shed A Watershed

WEST YOST ASSOCIATES
Consulting Engineers

N:\Clients\296 Elk Grove\00-05-01 SDMP\GIS\Figures\2011 SDMP\Fig13-2-ShedA-West_2011_letter.mxd 4/14/2011



LEGEND:

SA110 — Modeled Node and Pipeline

▨ Shed A Pipeline A1 Subsheds

City Limit

N

0 300 600
Scale in Feet

FIGURE 13-2

City of Elk Grove
Storm Drainage Master Plan
Volume II

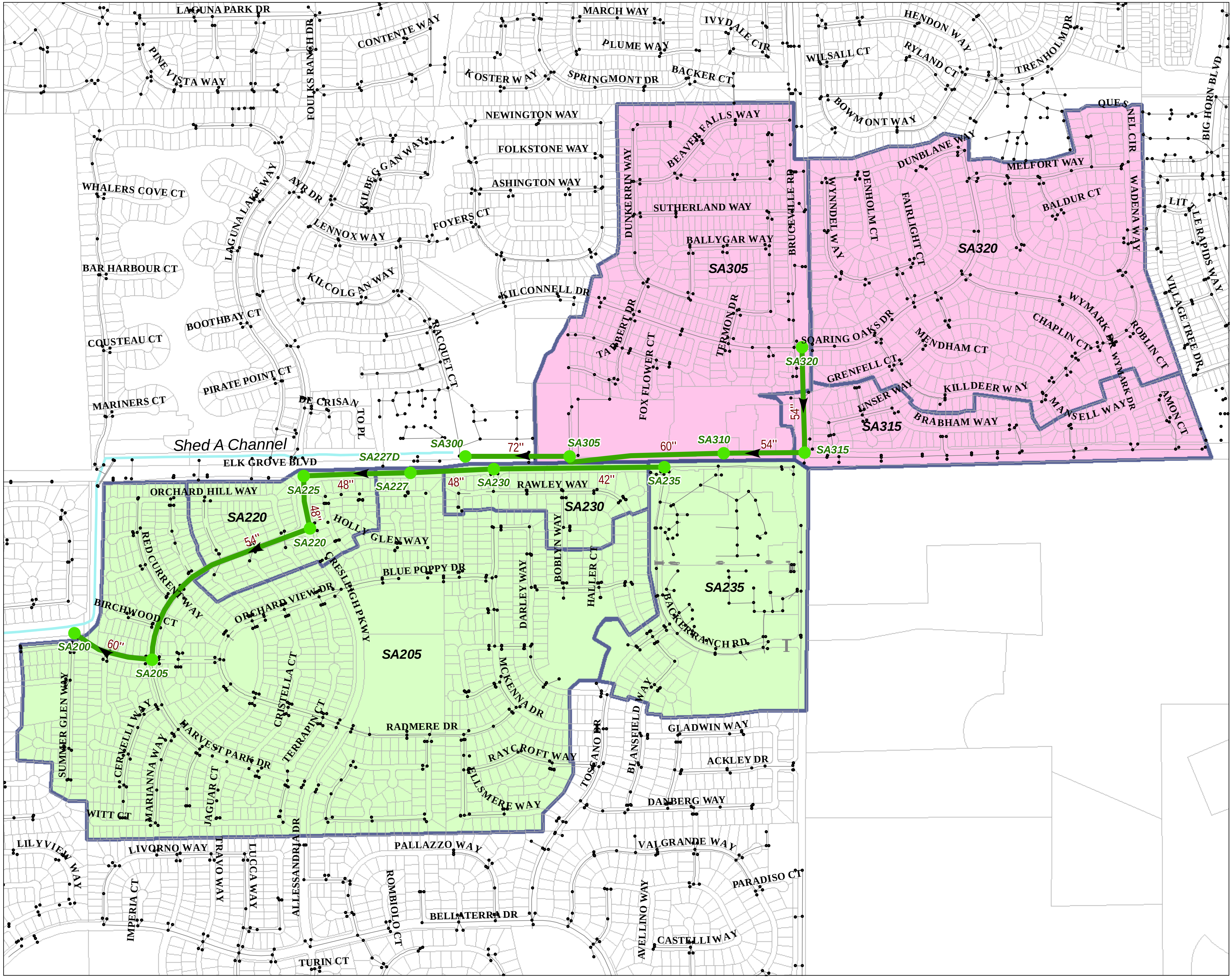
SHED A
EXISTING PIPELINE A1
SUBSHEDS & MODELED FACILITIES

WEST YOST ASSOCIATES
Consulting Engineers

FIGURE 13-3

City of Elk Grove Storm Drainage Master Plan Volume II

SHED A
EXISTING PIPELINES A2 & A3
SUBSHEDS & MODELED FACILITIES



N

0 700 1,400
Feet

NOTES:

LEGEND:

SA300 ●	Modeled Node and Pipeline
🟩	Shed A Pipeline A2 Subshed
🟪	Shed A Pipeline A3 Subshed
⬚	City Limit

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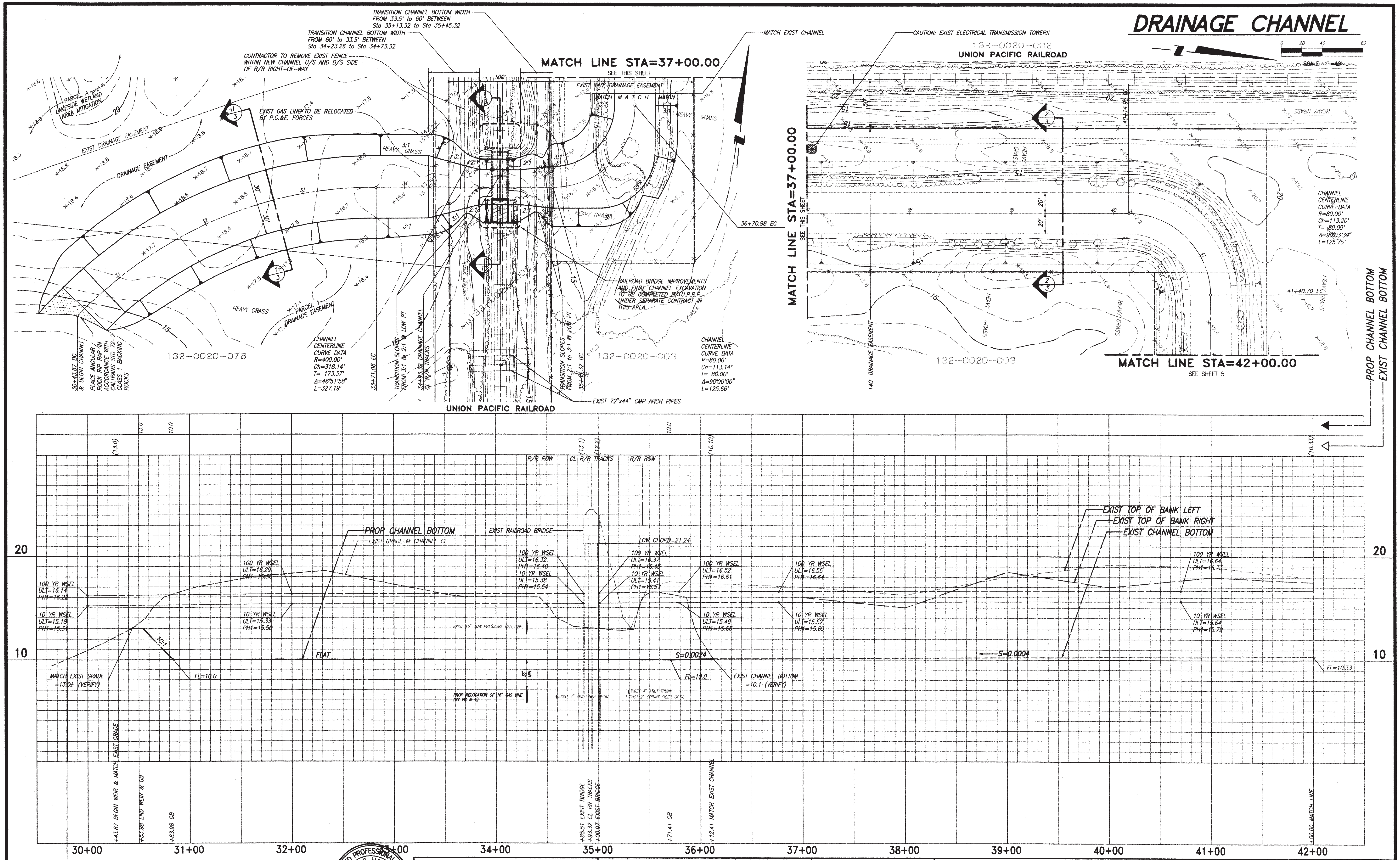
Consulting Engineers

ATTACHMENT 13A

Shed B Channel Profiles

ATTACHMENT 13A

DRAINAGE CHANNEL



EAST FRANKLIN – SHED A – PHASE 1

RICK C. HANSEN
No. 44748
Exp. 3-31-06

SUBMITTED BY:
Murray Smith & ASSOCIATES, ENGINEERING
3110 GOLD CANAL DRIVE (916) 635-1511
RANCHO CORDOVA, CA 95670

RICK C. HANSEN R.C.E. NO. 44748 8-8-02 DATE

REVISIONS

NO.	DESCRIPTION	ENGR INIT	COUNTY APPROVAL BY	COUNTY APPROVAL DATE

BENCH MARK

SEE SHEET 1

CONSTRUCTION PLANS
CITY OF ELK GROVE, CALIFORNIA

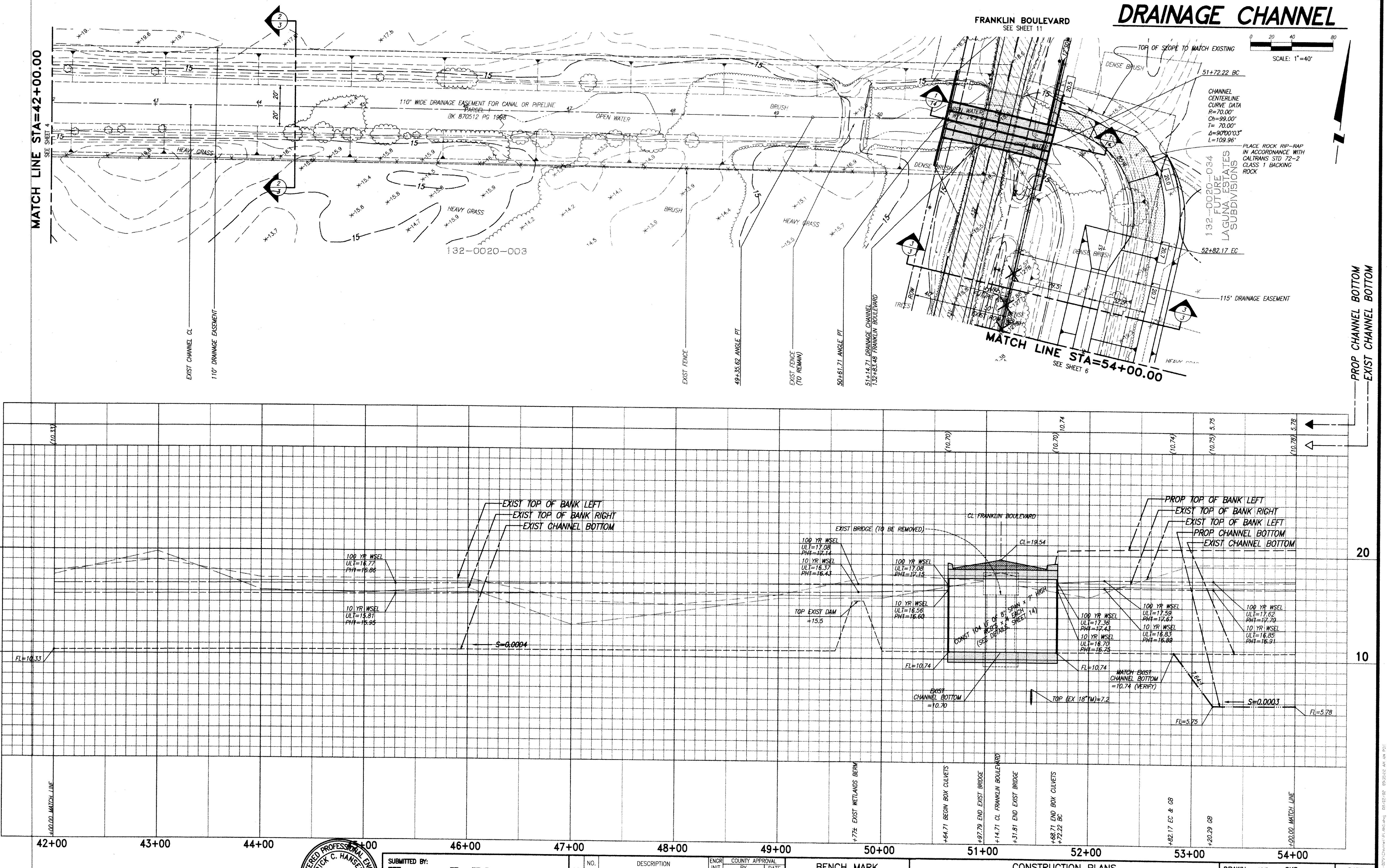
EAST FRANKLIN-SHED A-PHASE 1

DRAINAGE CHANNEL Sta 30+29.20 to Sta 42+00.00

DRAWN: MVP CKD. RCH
F.B. REF: SEE SHEET 1
SCALE: 1"=40' 1"=4'
PROJECT No. 01-029
DATE AUG 2002
SHEET 4 OF 17

RECORD DRAWING

DRAINAGE CHANNEL



EAST FRANKLIN - SHED A - PHASE 1

REGISTERED PROFESSIONAL ENGINEER
RICK C. HANSEN
No. 44748
Exp. 3-31-06
CIVIL
STATE OF CALIFORNIA

SUBMITTED BY:
Murray Smith & ASSOCIATES, ENGINEERING
3110 GOLD CANAL DRIVE (916) 635-1511
RANCHO CORDOVA, CA 95670

RHansen 8-8-02
RICK C. HANSEN R.C.E. NO. 44748 DATE

REVISIONS

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BENCH MARK

SEE SHEET 1

CONSTRUCTION PLANS
CITY OF ELK GROVE, CALIFORNIA

EAST FRANKLIN-SHED A-PHASE 1

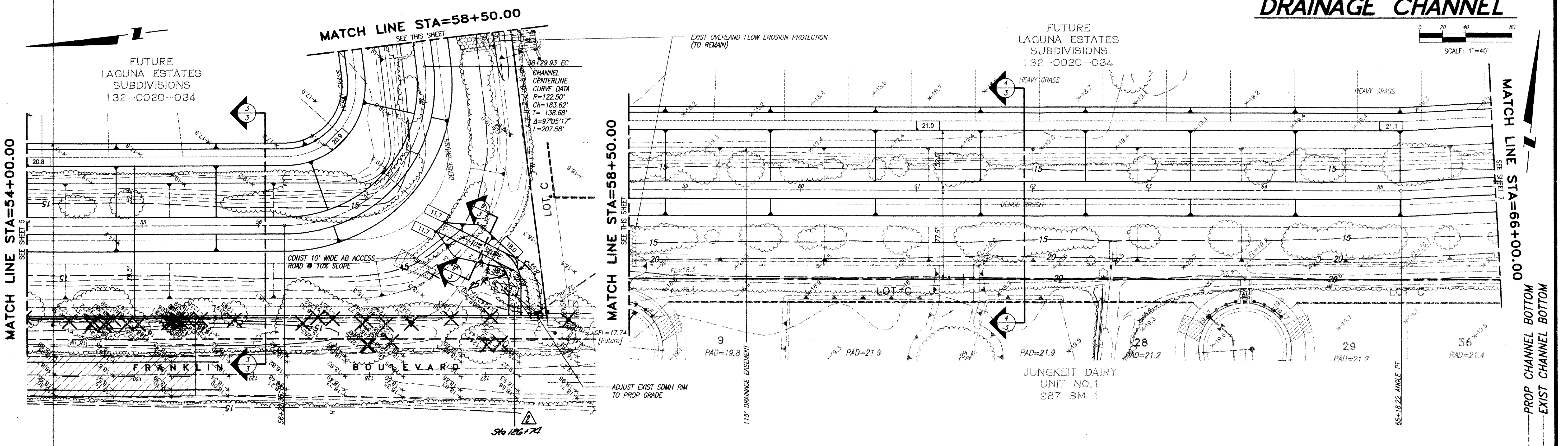
DRAINAGE CHANNEL Sta 42+00.00 to Sta 54+00.00

DRAWN: MVP CKD. RCH
F.B. REF: SEE SHEET 1
SCALE: 1"=40' 1"=4'
PROJECT No. 01-029

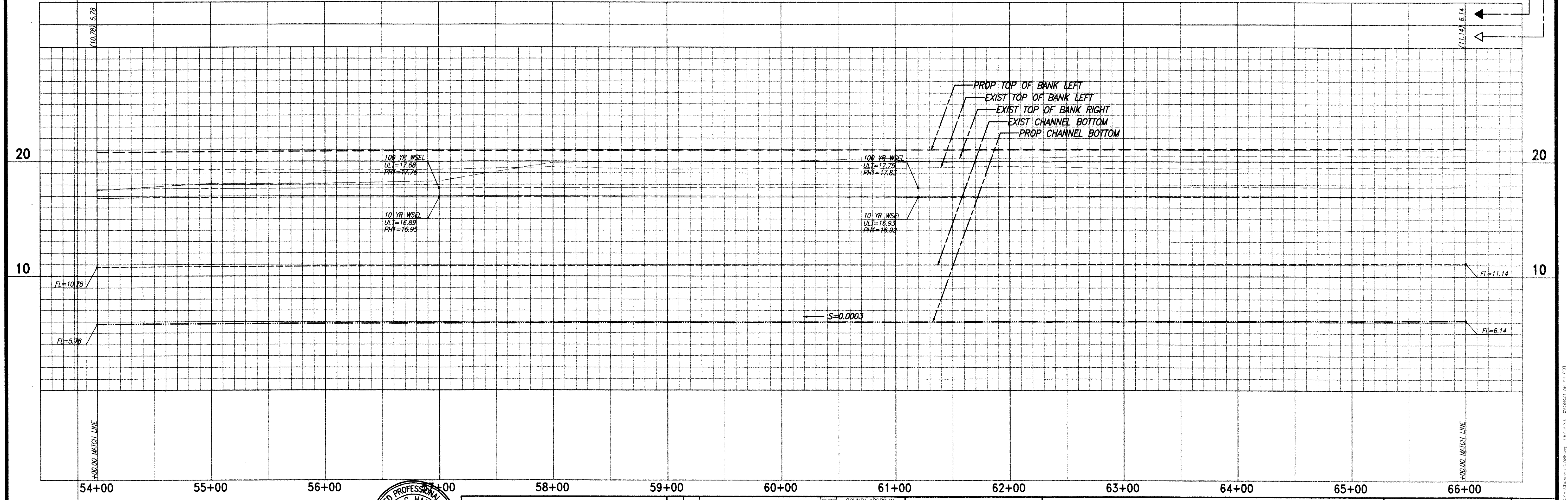
DATE AUG 2002
SHEET 5 OF 17

RECORD DRAWING

DRAINAGE CHANNEL



PROP CHANNEL BOTTOM
EXIST CHANNEL BOTTOM



EAST FRANKLIN - SHED A - PHASE 1

No. 44748
Exp. 3-31-06
CIVIL

SUBMITTED BY:
Murray Smith & ASSOCIATES, ENGINEERING
3110 GOLD CANAL DRIVE (916) 635-1511
RANCHO CORDOVA, CA 95670

Hansen 8-8-02
RICK C. HANSEN R.C.E. NO. 44748 DATE

REVISIONS

NO.	DESCRIPTION	ENGR INIT	COUNTY APPROVAL BY	COUNTY APPROVAL DATE
2	REVISE ACCESS RAMP LOCATION			7/30/03

BENCH MARK

SEE SHEET 1

CONSTRUCTION PLANS
CITY OF ELK GROVE, CALIFORNIA

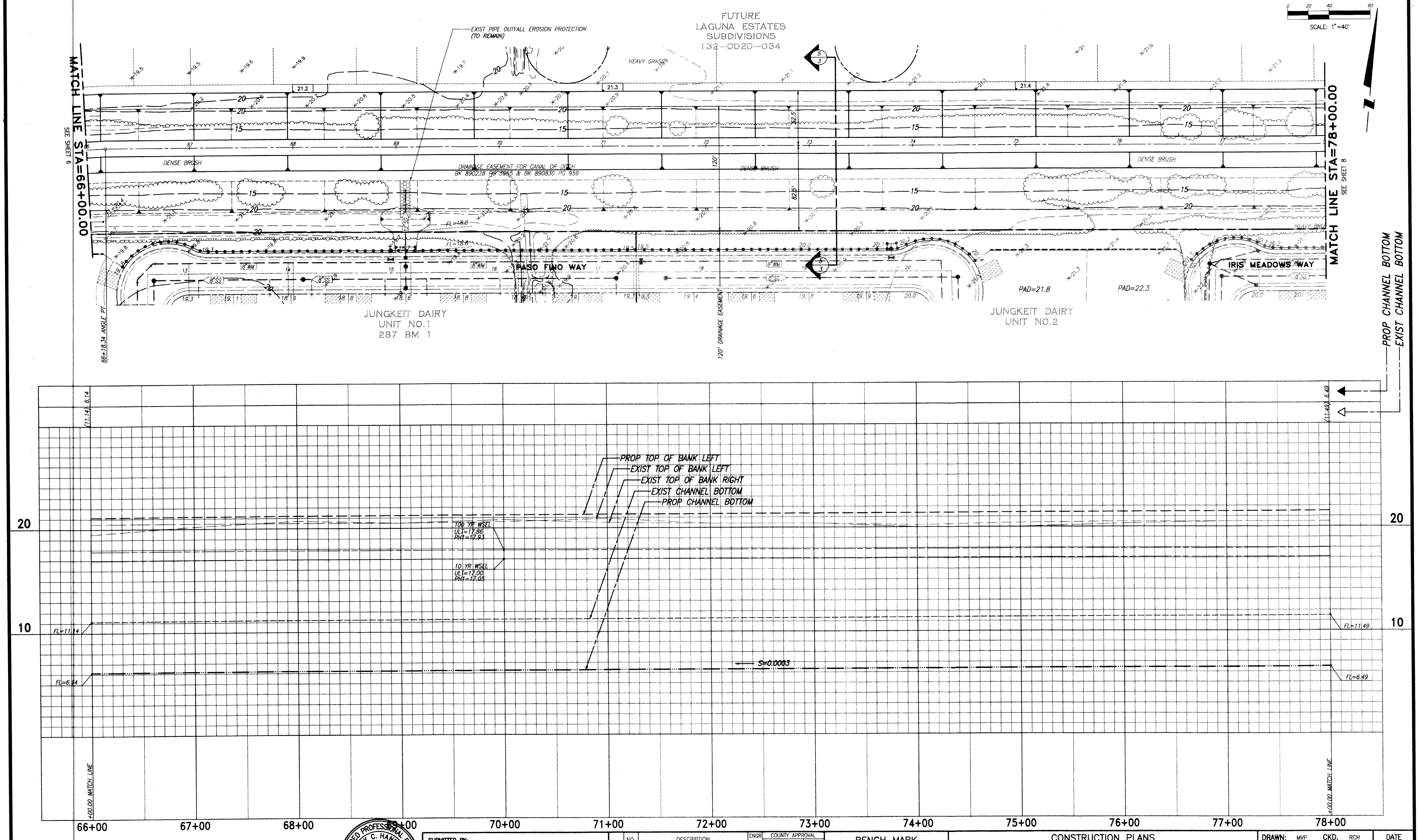
EAST FRANKLIN-SHED A-PHASE 1

DRAINAGE CHANNEL Sta 54+00.00 to Sta 90+00.00

DRAWN: MVP CKD. RCH
F.B. REF: SEE SHEET 1
SCALE: 1"=40' 1"=4'
PROJECT No. 01-029
DATE AUG 2002
SHEET 6 OF 17

RECORD DRAWING

DRAINAGE CHANNEL



EAST FRANKLIN – SHED A – PHASE 1

REGISTERED PROFESSIONAL ENGINEER
RICK C. HANSEN
No. 44748
Exp. 3-31-06
CIVIL
STATE OF CALIFORNIA

SUBMITTED BY:
Murray Smith & ASSOCIATES, ENGINEERING
3110 GOLD CANAL DRIVE (916) 635-1511
RANCHO CORDOVA, CA 95670
CHansen
RICK C. HANSEN R.C.E. NO. 44748
8-8-02
DATE

REVISIONS NO.	DESCRIPTION	ENGR INIT	COUNTY APPROVAL BY	DATE

BENCH MARK

SEE SHEET 1

CONSTRUCTION PLANS
CITY OF ELK GROVE, CALIFORNIA

EAST FRANKLIN-SHED A-PHASE 1

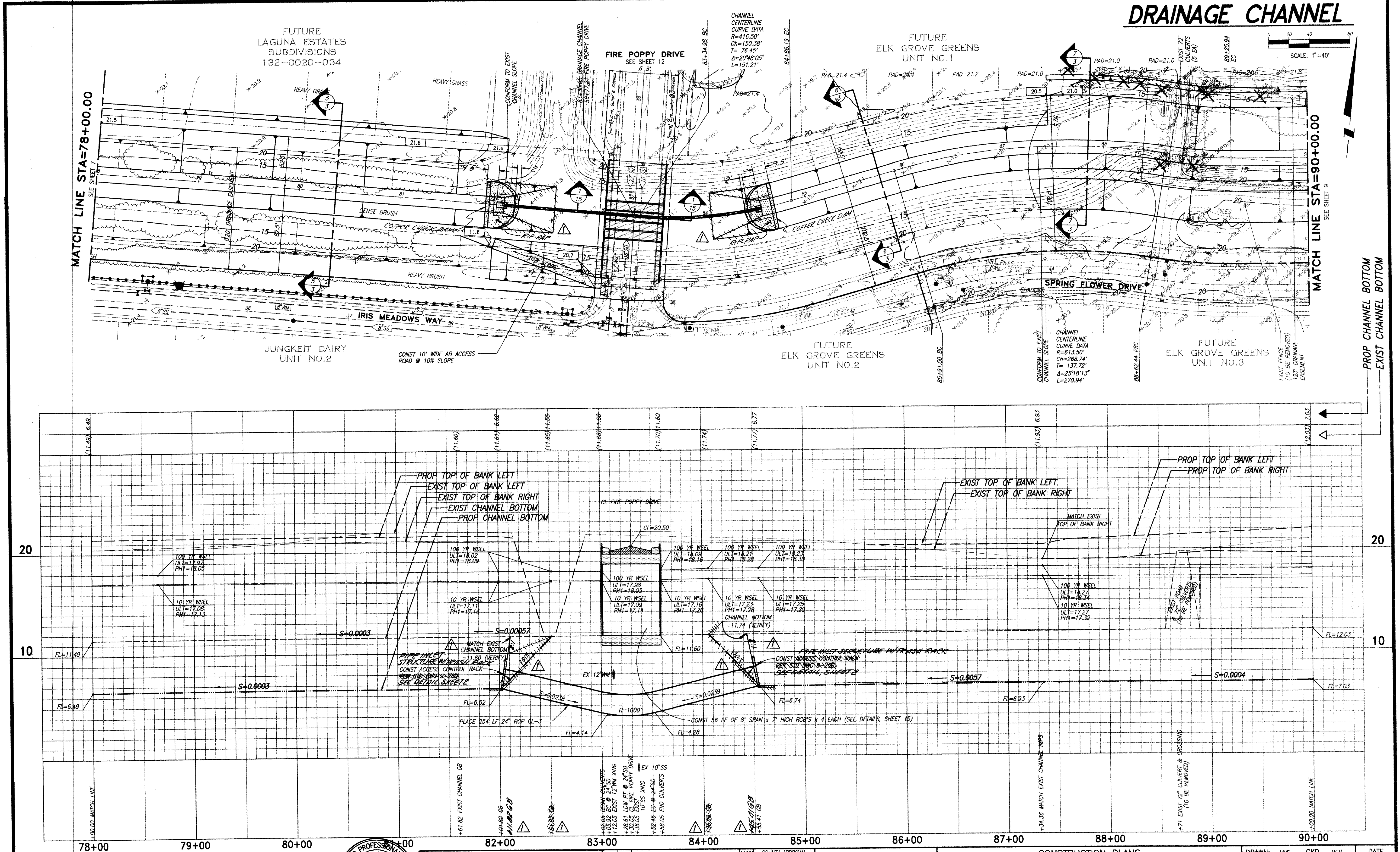
DRAINAGE CHANNEL Sta 66+00.00 to Sta 78+00.00

DRAWN: MVP CKD. RCH
F.B. REF: SEE SHEET 1
SCALE: 1"=40' 1"=4'
PROJECT No. 01-029

DATE AUG 2002
SHEET 7 OF 17

RECORD DRAWING

DRAINAGE CHANNEL



REGISTERED PROFESSIONAL ENGINEER
RICK C. HANSEN
No. 44748
Exp. 3-31-06
CIVIL
STATE OF CALIFORNIA

SUBMITTED BY:
Murray Smith & ASSOCIATES, ENGINEERING
3110 GOLD CANAL DRIVE (916) 635-1511
RANCHO CORDOVA, CA 95670
RICK C. HANSEN R.C.E. NO. 44748
8-8-02
DATE

REVISIONS

NO.	DESCRIPTION	ENGR INIT	COUNTY APPROVAL BY	COUNTY APPROVAL DATE
1	REVISE SIPHON INLET & OUTLET STRUCTURES		Cd	7/10/03

BENCH MARK

SEE SHEET 1

CONSTRUCTION PLANS
CITY OF ELK GROVE, CALIFORNIA

EAST FRANKLIN-SHED A-PHASE 1

DRAINAGE CHANNEL Sta 78+00.00 to Sta 90+00.00

DRAWN: MVP CKD. RCH
F.B. REF: SEE SHEET 1
SCALE: 1"=40' 1"=4'
PROJECT No. 01-029

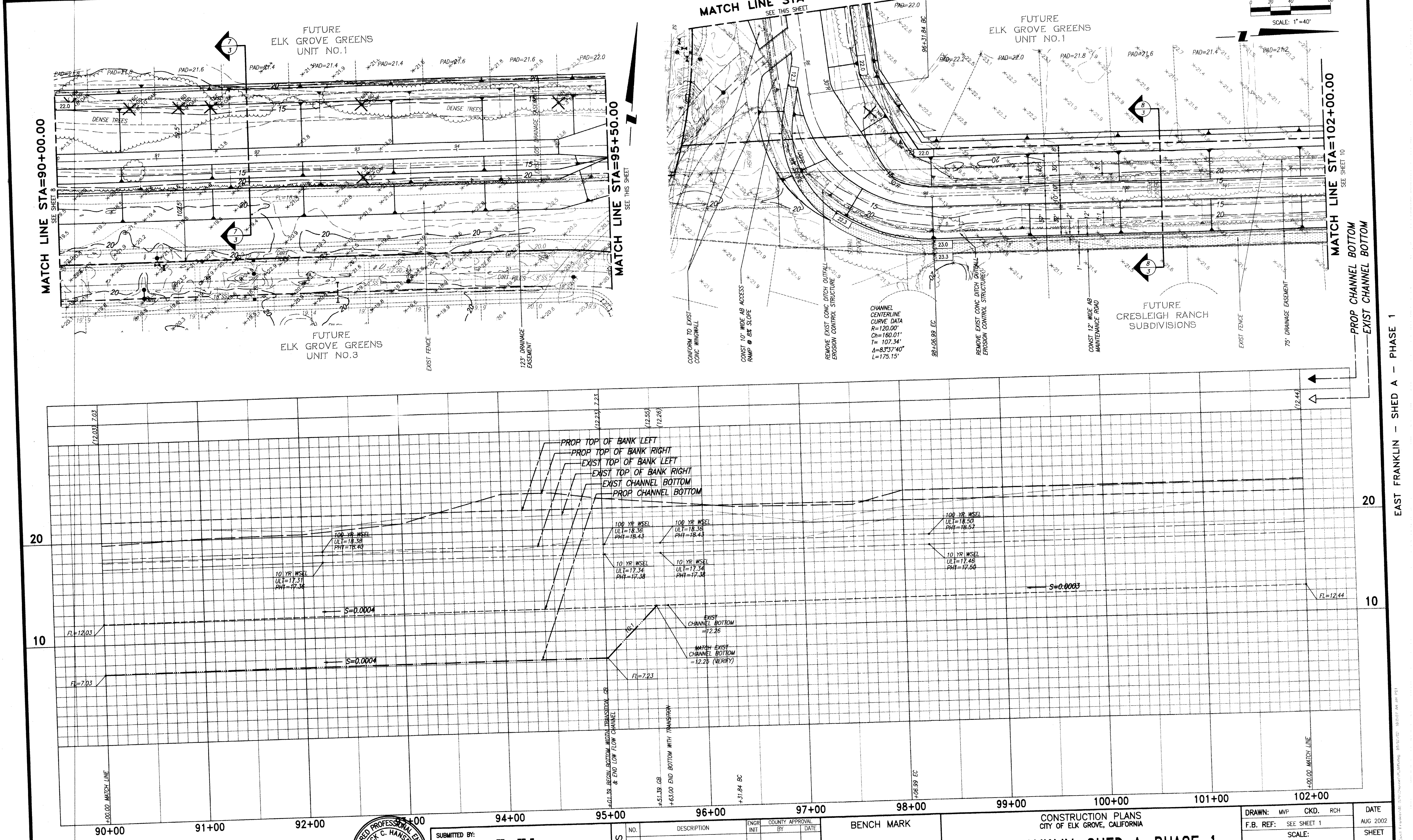
DATE AUG 2002
SHEET 8 OF 17

EAST FRANKLIN – SHED A – PHASE 1

RECORD DRAWING

DRAINAGE CHANNEL

SCALE: 1"=40'



REGISTERED PROFESSIONAL ENGINEER
RICK C. HANSEN
No. 44748
Exp. 3-31-06
CIVIL
STATE OF CALIFORNIA

SUBMITTED BY:
Murray Smith & ASSOCIATES, ENGINEERING
3110 GOLD CANAL DRIVE (916) 635-1511
RANCHO CORDOVA, CA 95670

RICK C. HANSEN R.C.E. NO. 44748 8-8-02 DATE

REVISIONS

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BENCH MARK

SEE SHEET 1

CONSTRUCTION PLANS
CITY OF ELK GROVE, CALIFORNIA

EAST FRANKLIN-SHED A-PHASE 1

DRAINAGE CHANNEL Sta 90+00.00 to Sta 102+00.00

DRAWN: MVP CKD. RCH
F.B. REF: SEE SHEET 1
SCALE: 1"=40' 1"=4'
PROJECT No. 01-029

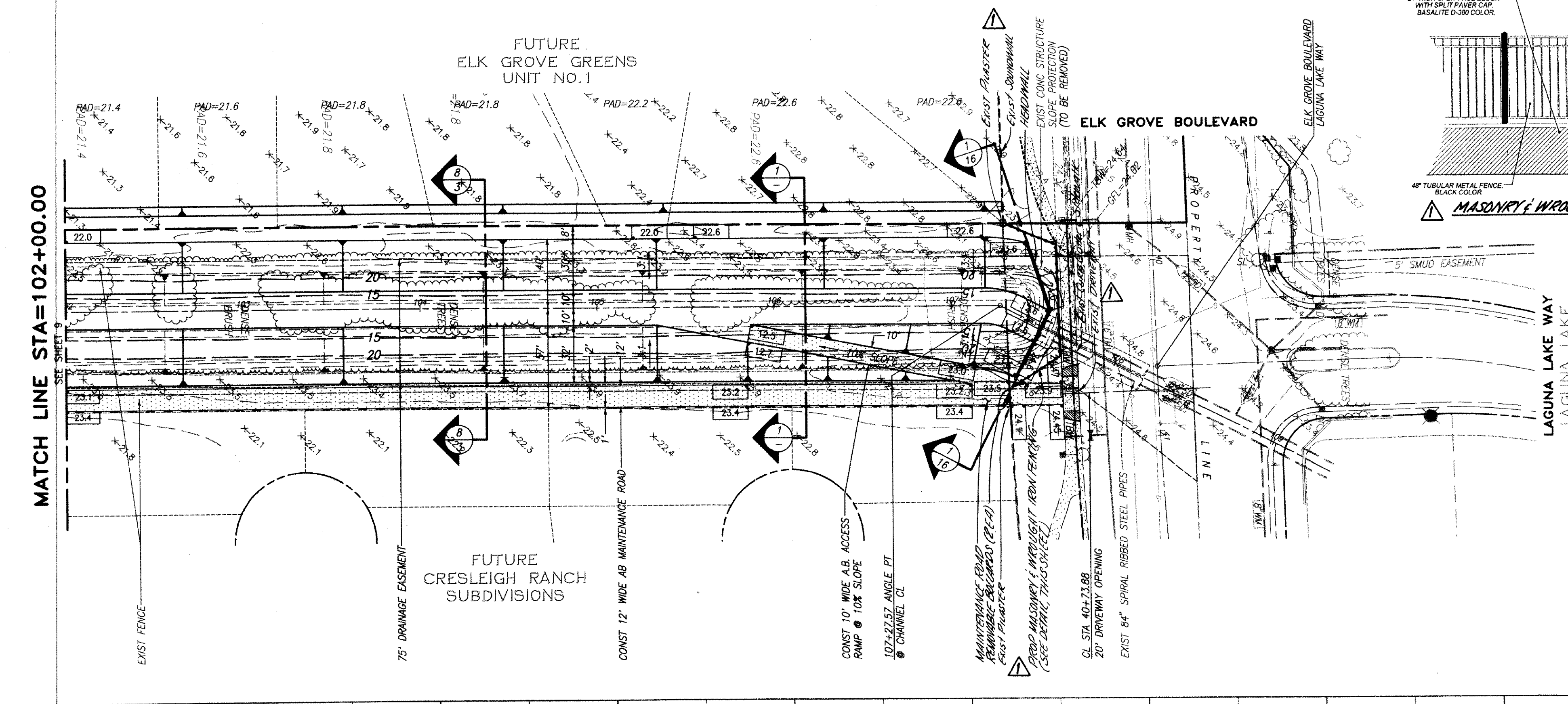
DATE AUG 2002
SHEET 9 OF 17

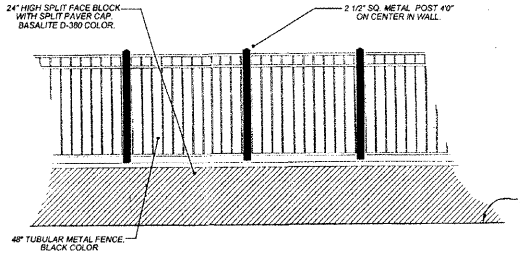
EAST FRANKLIN - SHED A - PHASE 1

RECORD DRAWING

DRAINAGE CHANNEL

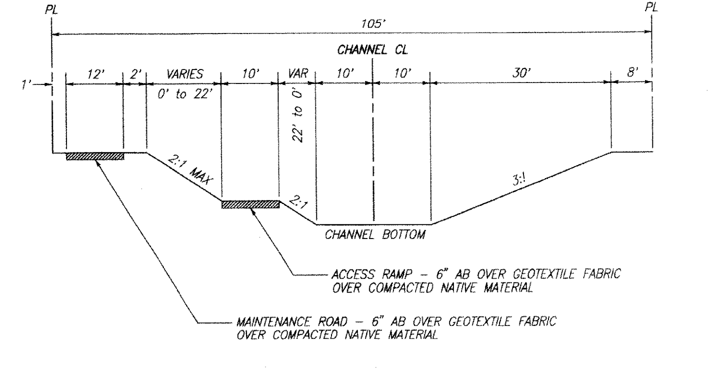
SCALE: 1"=40'





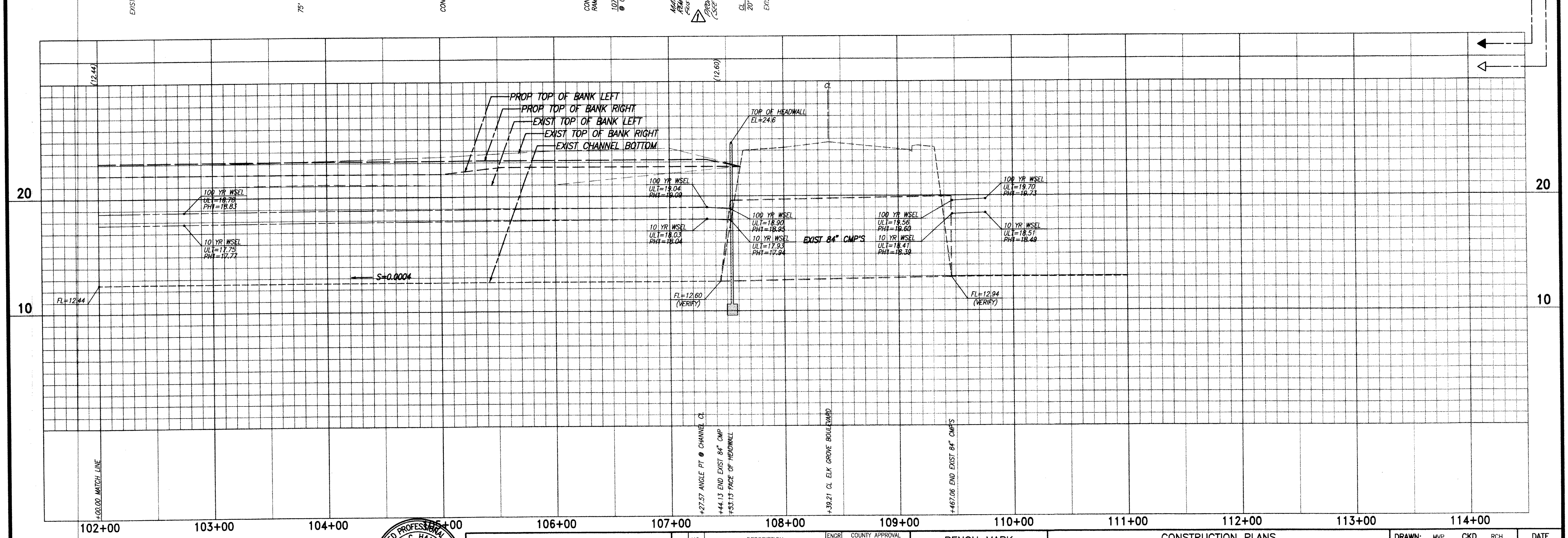
⚠ MASONRY & WROUGHT IRON FENCING DETAIL

NO SCALE



1/10 MAINTENANCE ROAD/ACCESS RAMP SECTION

NO SCALE



EAST FRANKLIN – SHED A – PHASE 1

REGISTERED PROFESSIONAL ENGINEER
RICK C. HANSEN
No. 44748
Exp. 3-31-06
CIVIL
STATE OF CALIFORNIA

SUBMITTED BY:
Murray Smith & ASSOCIATES, ENGINEERING
3110 GOLD CANAL DRIVE (916) 635-1511
RANCHO CORDOVA, CA 95670

RICK C. HANSEN R.C.E. NO. 44748 8-8-02 DATE

REVISIONS NO.	DESCRIPTION	ENGR INIT	COUNTY APPROVAL BY	DATE
⚠ 1	ADD FENCING DETAIL			7/...

BENCH MARK

SEE SHEET 1

CONSTRUCTION PLANS
CITY OF ELK GROVE, CALIFORNIA

EAST FRANKLIN-SHED A-PHASE 1

DRAINAGE CHANNEL Sta 102+00.00 to Sta X

DRAWN: MVP CKD. RCH
F.B. REF: SEE SHEET 1
SCALE: 1"=40' 1"=4'
PROJECT No. 01-029
DATE AUG 2002
SHEET 10 OF 17

RECORD DRAWING

CHAPTER 14. SHED B

WATERSHED DESCRIPTION

The Shed B watershed lies in the southwest part of the City and covers approximately 3,320 acres. The watershed lies east of the Union Pacific Railroad (UPRR), west of Highway 99, and straddles Poppy Ridge Road (see Figure 14-1). The watershed includes the southern portion of the East Franklin Specific Plan area and the northern portion of the Laguna Ridge Specific Plan area. The drainage infrastructure for this area is almost entirely complete and consists of an underground pipe system that conveys runoff to the Shed B Channel. The Shed B Channel begins at Big Horn Boulevard and carries runoff to the west, exits the City at the UPRR, and continues to the Beach Stone Lakes area west of Interstate 5.

EVALUATION OF SHED B CHANNEL

No hydrologic or hydraulic analyses were performed for the Shed B Channel. The recent master plan data prepared for the East Franklin and Laguna Ridge Specific Plan areas are considered by the City to provide an adequate analysis of the existing and future channel. Water surface profiles for Shed B are presented on Figure 14-2.

EVALUATION OF PIPELINES

The Shed B watershed is drained by an underground pipe network that delivers runoff to the Shed B Channel. Detailed drainage analyses of the pipe system for the Laguna Ridge Specific Plan area have been performed by Wood Rodgers. These analyses were reviewed and found to be adequate.

PRELIMINARY IMPROVEMENTS

Any additional pipe improvements required in the watershed should be constructed in accordance with the planning or design studies prepared for the Laguna Ridge Specific Plan area.

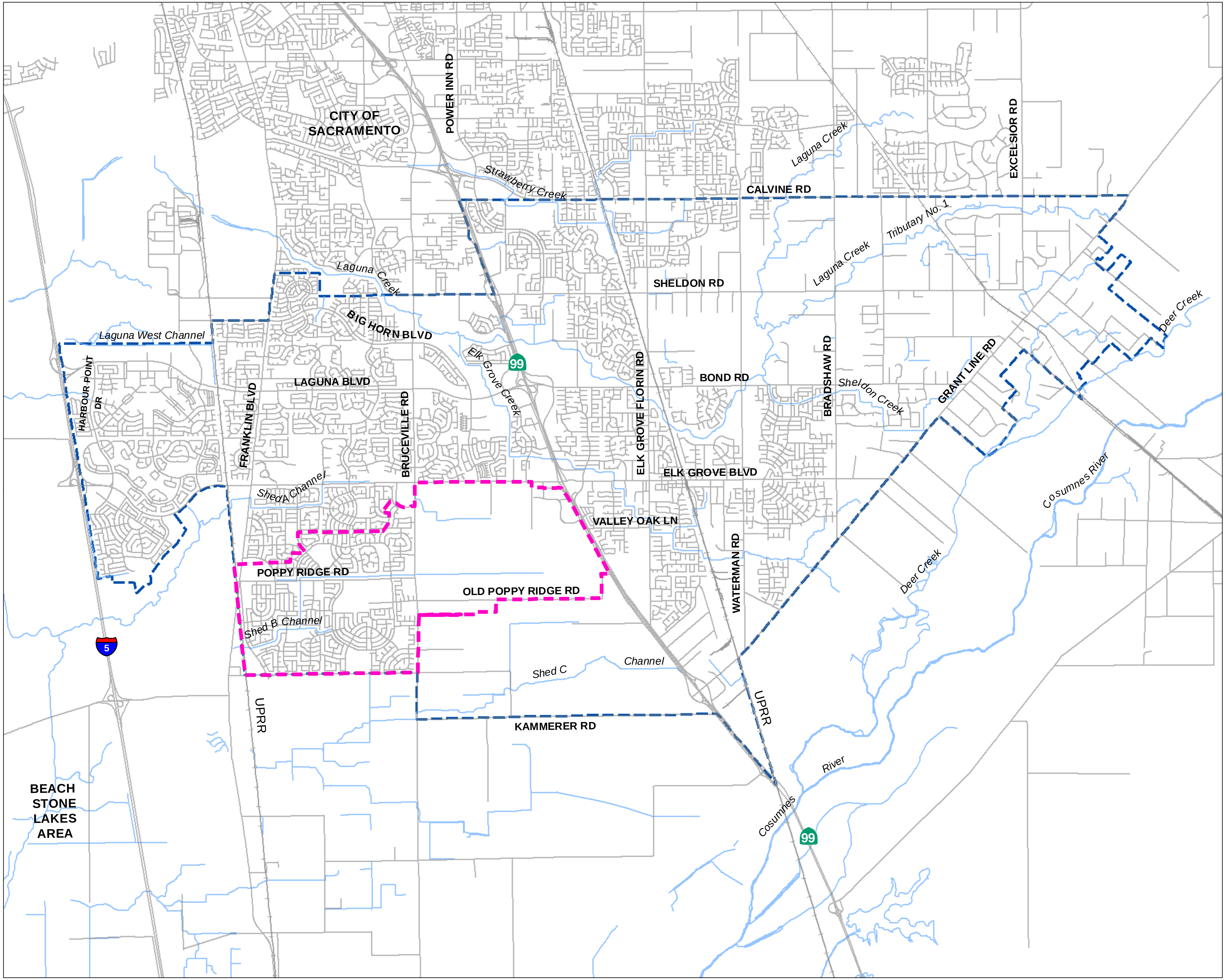


FIGURE 14-1

City of Elk Grove Storm Drainage Master Plan Volume II

SHED B LOCATION MAP

N

0 0.5 1 2
Scale in Miles

NOTES:

LEGEND:

▭ City Limit
▭ Shed B Watershed

WEST YOST ASSOCIATES
Consulting Engineers

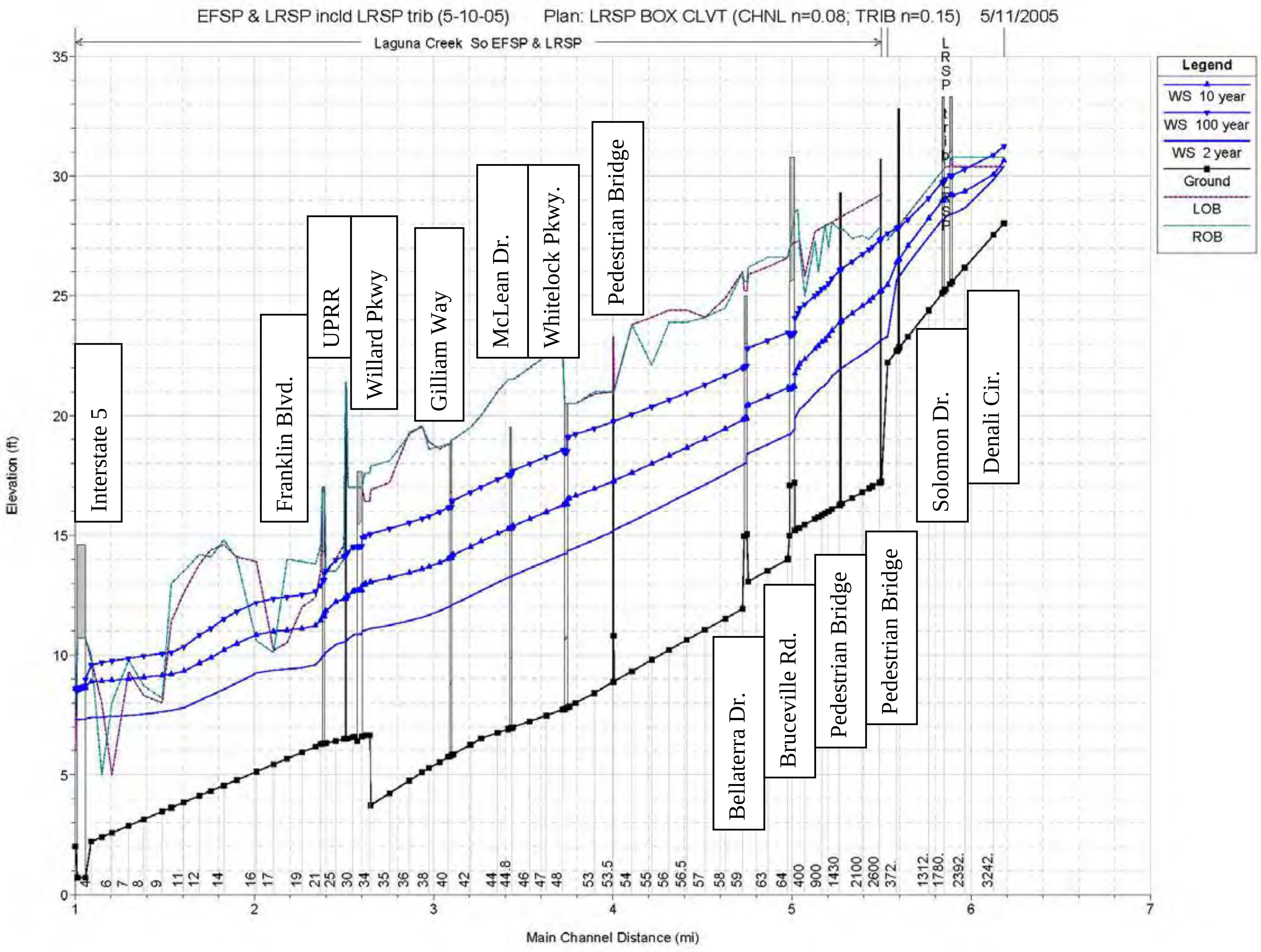


Figure 14-2 Shed B Channel Water Surface Profiles

CHAPTER 15. SHED C

WATERSHED DESCRIPTION

Drainage Shed C lies in southern Sacramento County and covers nearly 7,900 acres (See Figure 15-1). Of that total, approximately 2,100 acres lie within the City. The watershed generally slopes from east to west with an average slope of about 0.10 percent. The existing land use within the watershed is agricultural with the exception of the Elk Grove Promenade site, which covers 525 acres in the upstream (eastern) portion of the watershed. Although the Promenade project has stalled before completion, much of the site improvements were completed including the construction of roads and parking lots, buildings, and underground utilities, including a storm drainage pipe system. The pipe system collects runoff from the Promenade site and delivers it to a detention basin that was constructed with the project.

Downstream of the existing detention basin, runoff is conveyed in a well defined agricultural drainage channel, which is referred to as the Shed C Channel in this report. The Shed C Channel begins near the western boundary of the future Sterling Meadows project and conveys runoff to the southwest for approximately 12,600 feet until it reaches Bruceville Road. At that point, the channel exits the City and continues west for approximately 22,000 feet where it crosses under Interstate 5 and enters the Stone Lakes National Wildlife Refuge.

DRAINAGE PLAN CONCEPT

As development occurs in the watershed, drainage system improvements will be required to provide flood protection and mitigation, stormwater quality treatment, and hydromodification mitigation. The preliminary drainage plan for Shed C was developed with input from the Expert Advisory Committee (EAC) that was formed by the City to help guide the development of the master plan. The drainage concept for Shed C was developed with consideration of the guiding principles that were developed by the EAC for the drainage master plan:

1. Storm water management systems shall be designed to take maximum advantage of the natural hydrological processes of the existing landscape.
2. Alternative storm water management approaches shall be adopted, wherever and whenever feasible, to complement approaches to traditional storm water management systems. Alternative approaches may include distributed systems (e.g. low impact development systems), flow duration control basins, and/or instream rehabilitation.
3. Design of storm water management projects shall balance considerations related to environmental effects, capital and operating costs, property rights, economic development impacts, and recreational opportunities without compromising public safety and/or property.
4. Storm water management systems shall be designed so that the volume, quality, and timing of downstream discharges will minimize impacts to downstream resources, such as the Stone Lakes National Wildlife Refuge.
5. The Storm Drainage Master Plan shall comply with applicable local, state, and federal laws and regulations.

With these guiding principles in mind, the drainage concept for Shed C includes a multi-functional drainage corridor that will restore some of the natural stream and habitat values that have been lost over time. The multi-functional corridor will include a low flow channel that is stable and self sustaining and will be designed based on natural processes. The low flow channel will meander within a larger floodplain corridor that will provide flood storage and conveyance as well as an opportunity for the creation of wetlands habitat. It is recommended that the corridor also include a path that will provide recreational and educational opportunities for the City's residents.

Additional key components of the drainage concept are detention basins to be included at major inflow points to the drainage corridor. These detention basins will provide flood storage and flow duration control to mitigate for potential flood flow increases and hydromodification effects due to the proposed urban development in the watershed. The detention basins will also provide an opportunity for wetlands creation. Figure 15-2 presents a plan showing the drainage concept.

ANALYSIS APPROACH

The Shed C analysis consisted of two major components: 1) a continuous hydrologic analysis; and 2) an event based analysis as described below.

Continuous Hydrologic Analysis

An important consideration in the Shed C analysis is the potential hydromodification effects of development in the watershed. Hydromodification is the change in runoff characteristics within a watershed caused by land use changes. These altered runoff characteristics can result in increased erosion and sedimentation, degradation of stream habitat, increased flood flows, and other negative impacts. Research has shown that a large percentage of the sediment transport and erosion in a stream system occurs at flow rates less than generated by the 2-year storm (Geosyntec, 2007). Because of this, traditional hydrologic analyses that focus on individual design storms (e.g. 2-year, 10-year, etc.) are not suitable for hydromodification analyses. To insure that the cumulative effects of all potentially erosive flows are considered, a continuous hydrologic model is required. For the SDMP, a continuous hydrologic simulation was performed using the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) software. The model was used to evaluate the long-term rainfall-runoff response for the Shed C watershed for two land-use conditions:

- Base Conditions
- Buildout Conditions

The development of the continuous simulation model is described in more detail later in this report.

Event Based Analysis

A traditional event based analysis was also performed to assess the flood control performance of the proposed system. Single event hydrologic and hydraulic models were prepared for the 10-year and 100-year storms for both base conditions and for the mitigated buildout condition. The results were used to confirm that the proposed project would adequately mitigate for potential impacts to flood flows and to confirm the required size of the flood control channel.

CONTINUOUS SIMULATION MODEL - BASE CONDITIONS

A continuous simulation model was developed for the Base Conditions using HEC-HMS. The starting point for the model was the previously created SacCalc/HEC-1 hydrologic models that were used for the City's draft master plan published in 2006. The SacCalc/HEC-1 models were imported into HEC-HMS and certain model input parameters were revised to be consistent with the continuous simulation approach. The model input data is described below.

Watershed Boundaries

For the hydrologic modeling, Shed C was divided into the subsheds shown on Figure 15-3. Watershed areas and other model parameters are listed on Table 15-1.

Land Use

For Base Conditions, the majority of the watershed was assumed to be undeveloped agricultural land. However, there are some exceptions including the Elk Grove Promenade and Sterling Meadows properties at the upstream end of Shed C (subsheds A1 and A2 on Figure 15-3). The Elk Grove Promenade project was previously approved by the City and the site improvements were largely completed prior to the project being stalled during the recent economic recession. The project construction included a large detention basin to serve both the Promenade and Sterling Meadows sites. Therefore, for the base condition model, full buildout was assumed for the Promenade and Sterling Meadows projects and the existing detention basin that serves these sites was also included.

The other exception is the Laguna Ridge Specific Plan area. Tentative maps and drainage studies have already been approved for the projects within that specific plan. The development of this area will include construction of a detention basin for stormwater quality treatment and flood control and will also include a constructed channel that will convey flows from the project area to the Shed C Channel. Because the proposed drainage system has already been approved, buildout conditions were assumed for the Laguna Ridge Specific Plan area.

Unit Hydrographs

Unit hydrographs for the continuous simulation model were imported directly from the SacCalc/HEC-1 models. The input parameters for the calculation of unit hydrographs in SacCalc are presented on Table 15-1. As discussed later in this report, some modification to the unit hydrographs were made to account for the Low Impact Development techniques that are anticipated to be included with future development projects.

Table 15-1. Hydrologic Parameters

						Base Condition Land-use (acres)								Buildout Land-use (acres)							
Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Ag.	Park	Res. 4-6 du/ac	Res. 6-8 du/ac	Res. 8-10 du/ac	MDR	Comm.	Avg. Percent Imp.	Ag.	Park	Res. 4-6 du/ac	Res. 6-8 du/ac	Res. 8-10 du/ac	MDR	Comm.	Avg. Percent Imp.
A1	319.2	42	5,547	1,324	0.0008	0	0	0	0	0	0	319.2	90.0	0	0	0	0	0	0	319.2	90.0
A2	202.4	40	5,800	1,600	0.0008	0	0	202.4	0	0	0	0	40.0	0	0	0	202.4	0	0	0	40.0
A4	99.1	39	2,900	700	0.0008	99.1	0	0	0	0	0	0	2.0	0	0	0	99.1	0	0	0	50.0
A4A	290.7	39	4,700	1,500	0.0008	290.7	0	0	0	0	0	0	2.0	0	0	0	290.7	0	0	0	50.0
A4B	215.5	39	4,800	1,400	0.0008	215.5	0	0	0	0	0	0	2.0	0	0	0	215.5	0	0	0	50.0
A4C	93.6	39	2,200	1,100	0.0008	93.6	0	0	0	0	0	0	2.0	0	0	0	93.6	0	0	0	50.0
A5	247.7	36	4,800	2,000	0.0012	247.7	0	0	0	0	0	0	2.0	0	0	0	247.7	0	0	0	50.0
A5B	91.6	30	4,000	1,700	0.0008	91.6	0	0	0	0	0	0	2.0	0	0	0	91.6	0	0	0	50.0
LRS14	125.6	32	3,400	2,000	0.0003	0	6.3	71.9	15.0	20.0	0	12.4	48.0	0	6.3	71.9	15.0	20.0	0	12.4	48.0
LRS15	184.3	30	4,000	2,000	0.0003	0	28.4	130.7	0	0	21.2	4.0	39.2	28.4	130.7	0	0	21.2	4.0	39.2	28.4
LRS19	54.5	32	2,200	1,100	0.0003	0	0	39.5	0	0	15.0	0	48.3	0	0	39.5	0	0	15.0	0	48.3
LRS20	42.5	30	1,200	600	0.0003	0	0	22.1	15.0	0	0	5.4	50.6	0	0	22.1	15.0	0	0	5.4	50.6
MA5C	40.5	28	1,200	500	0.0017	40.5	0	0	0	0	0	0	2.0	0	0	0	40.5	0	0	0	50.0
A6	95.0	27	3,500	1,700	0.0008	95.0	0	0	0	0	0	0	2.0	95.0	0	0	0	0	0	0	2.0
A8	216.4	27	5,200	2,600	0.0008	216.4	0	0	0	0	0	0	2.0	216.4	0	0	0	0	0	0	2.0
A10	557.0	20	6,400	1,900	0.0008	557.0	0	0	0	0	0	0	2.0	557.0	0	0	0	0	0	0	2.0
A11	213.2	19	5,300	1,000	0.0008	213.2	0	0	0	0	0	0	2.0	213.2	0	0	0	0	0	0	2.0
A12	470.8	42	7,400	3,500	0.0008	470.8	0	0	0	0	0	0	2.0	470.8	0	0	0	0	0	0	2.0
A13	257.9	38	5,400	2,700	0.0008	257.9	0	0	0	0	0	0	2.0	257.9	0	0	0	0	0	0	2.0
A14	481.7	38	7,500	1,400	0.0008	481.7	0	0	0	0	0	0	2.0	481.7	0	0	0	0	0	0	2.0
A15	487.3	35	6,900	1,400	0.0008	487.3	0	0	0	0	0	0	2.0	487.3	0	0	0	0	0	0	2.0
A16	723.2	32	10,000	1,300	0.0008	723.2	0	0	0	0	0	0	2.0	723.2	0	0	0	0	0	0	2.0
A17	722.6	28	9,000	1,200	0.0008	722.6	0	0	0	0	0	0	2.0	722.6	0	0	0	0	0	0	2.0
A18	699.3	20	12,000	6,300	0.0008	699.3	0	0	0	0	0	0	2.0	699.3	0	0	0	0	0	0	2.0
A19	223.6	18	5,300	1,200	0.0008	223.6	0	0	0	0	0	0	2.0	223.6	0	0	0	0	0	0	2.0
A20	80.9	16	2,800	600	0.0008	80.9	0	0	0	0	0	0	2.0	80.9	0	0	0	0	0	0	2.0
A21	156.4	14	5,000	2,600	0.0008	156.4	0	0	0	0	0	0	2.0	156.4	0	0	0	0	0	0	2.0
A22	96.7	18	3,600	900	0.0008	96.7	0	0	0	0	0	0	2.0	96.7	0	0	0	0	0	0	2.0
A23	66.6	16	2,900	600	0.0008	66.6	0	0	0	0	0	0	2.0	66.6	0	0	0	0	0	0	2.0
A24	88.5	13	3,700	2,000	0.0008	88.5	0	0	0	0	0	0	2.0	88.5	0	0	0	0	0	0	2.0
A25	219.5	12	5,000	1,900	0.0008	219.5	0	0	0	0	0	0	2.0	219.5	0	0	0	0	0	0	2.0

Precipitation Data

For the continuous simulation analysis, 73 years of hourly precipitation for water years 1937 through 2009 was obtained from various gages in the area as summarized on Table 15-2.

Table 15-2. Summary of Precipitation Sources

Gage ID	Gage Description	Date Range
HPD047630	Sacramento Post Office National Weather Service (Adjusted)	10/1/1936 to 12/3/1962 and 05/9/1974 to 8/4/1974
ElkGroveFD	The Elk Grove Fire Station on Elk Grove Boulevard	12/04/1962 to 5/8/1974
ElkGroveFH	The Elk Grove Fish Hatchery on Bond Road	8/5/1975 to 6/5/1985
ElkGroveFH ALERT	ALERT gage at the Elk Grove Fish Hatchery on Bond Road	6/6/1985 to 11/6/2002
0270td3240	ALERT gage Laguna Creek at Waterman Road	11/7/2002 to 9/30/2009

To better represent precipitation in Elk Grove, the rainfall data from the Sacramento Post Office gage was adjusted using a ratio of the average annual rainfall between the Post Office and Elk Grove rain gages. Based on this approach, a factor of 0.94 was applied to the Sacramento Post Office hourly rainfall values.

Soil Moisture Accounting Parameters

For the SDMP, the rainfall loss method was changed from the event based approach used in the original SacCalc/HEC-1 models to the Soil Moisture Accounting method, which was incorporated into HEC-HMS specifically for continuous simulations. This method allows for a continuous accounting of rainfall losses including evapotranspiration, surface storage, infiltration, and interflow. Ideally, the model parameters assigned to represent the various processes would be determined from a calibration analysis based on measured stream flow data. Unfortunately, stream flow records for the Shed C watershed are not available. Therefore, the model input from a calibrated HEC-HMS model for Laguna Creek was used to guide the input choices for the SDMP. The Laguna Creek model was prepared by Geosyntech (Geosyntec, 2007) and the information developed for that study was applied to this one. The soils types within the Shed C watershed were determined using the latest soil survey data from the Natural Resources Conservation Service. Subsheds in the Laguna Creek model with the same soil types as those within Shed C were identified and the Soil Moisture Accounting parameters those subsheds were applied to the Shed C model. Table 15-3 presents the values used for the SDMP.

Table 15-3. Soil Moisture Accounting Parameters

Subshed	Canopy Storage (in)	Surface Storage (in)	Maximum Infiltration (in/hr)	Imp. (%)	Soil Storage (in)	Tension Storage (in)	Soil Percolation (in/hr)	Gw 1 Storage (in)	Gw 1 Percolation (in)	Groundwater 1 Storage Coeff.
A01	0.08	0.3	0.07	90	6	4.8	0.07	10	0.07	200
A02	0.08	0.3	0.07	40	6	4.8	0.07	10	0.07	200
A04	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A04A	0.08	0.3	0.07	2	6	4.8	0.07	10	0.08	200
A04B	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A04C	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A05	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A05B	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
LRS14	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
LRS15	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
LRS19	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
LRS20	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
MA5C	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A06	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A08	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A10	0.08	0.3	0.07	2	-6	4.8	0.07	10	0.07	200
A11	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A12	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A13	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A14	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A15	0.08	0.3	0.07	2	6	4.8	0.08	10	0.07	200
A16	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A17	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A18	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A19	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A20	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A21	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A22	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A23	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A24	0.08	0.3	0.07	2	6	4.8	0.07	10	0.07	200
A25	0.08	0.3	0.06	2	6	4.8	0.06	10	0.06	200

CONTINUOUS SIMULATION MODEL - BUILDOUT CONDITIONS

For buildout conditions, the continuous simulation model parameters were updated to represent full buildout within the City limits. The specific buildout assumptions for the continuous simulation model are discussed below.

Land Use

For the buildout conditions model, the area within the City limits was assumed to be fully developed, while land-use for the area outside of the City was unchanged from the base conditions. Buildout land-uses within the City were determined from the prior planning work performed for the Laguna Ridge Specific Plan Area (Wood Rodgers, 2005) and for the Southeast Policy Area (approximate land-use mix provided by MacKay & Somps). Specific land-use types within the Southeast Policy Area are not yet defined and a preliminary land-use mix provided by MacKay and Somps was used to estimate an average land-use density and this was applied to each subshed within the Southeast Policy Area. Based on this approach, the average buildout land-use density was estimated to be equivalent to residential land-use at a density of 6 to 8 units per acre. This results in an average imperviousness of 50 percent. Table 15-1 presents the land-use assumed for each subshed for both base and buildout conditions. As discussed below, some adjustment to the average imperviousness was made based on the estimated effects of low impact development.

Unit Hydrographs

Unit hydrographs were calculated using the buildout condition SacCalc models that were prepared for the draft 2006 SDMP. The input parameters for the calculation of unit hydrographs in SacCalc are presented on Table 15-1. Some adjustment to the lag times were made based on the estimated effects of low impact development. These adjustments are discussed below.

Low Impact Development

Based on the requirements of the City’s NPDES stormwater permit, low impact development (LID) must be incorporated into future development projects in the City. LID is a storm water management approach that emphasizes the use of on-site natural features integrated with engineered hydrologic controls distributed throughout a watershed that promote infiltration, filtration, storage, and evaporation of runoff close to the source. Examples of LID techniques include grassy swales, permeable pavement, disconnected roof drains, and rain gardens. It was beyond the scope of the SDMP to define specific LID techniques to be implemented within each development area. However, the generalized benefits of LID on infiltration rates, lag times, and stormwater quality treatment were assessed and factored into the analysis.

In traditional development, runoff from impervious areas is typically directed into a pipe and conveyed directly to the receiving water without flowing across pervious areas. LID promotes retaining pervious areas and routing runoff from impervious areas across pervious areas before directing it to the receiving water. This can increase the volume of runoff that infiltrates into the soil, which reduces the *effective* impervious area of a development. However, due to limitations in infiltration capacity of the soil, not all of the increased runoff produced from the impervious area can be infiltrated. To preliminarily quantify the potential benefits of LID on runoff volumes, a test was conducted based on the Soil Conservation

(SCS) Curve Number method (SCS, 1986), which is a method that relates runoff volume to rainfall total and soil infiltration capacity. For this test, we evaluated the potential runoff for two different land-use scenarios: 1) fully developed conditions (assuming 50 percent imperviousness) with runoff from the impervious area directly connected to the storm drain system; and 2) fully developed conditions (assuming 50 percent imperviousness) with LID techniques implemented that fully redirect runoff from the impervious areas across the pervious areas. It seems unlikely that LID techniques can accomplish a uniform spreading of runoff from the impervious areas across the pervious area, therefore a third scenario was also considered: 3) fully developed conditions with LID techniques implemented that redirect runoff from the impervious area across half of the pervious area.

The testing approach used for the SDMP is similar to the approach used by Holman-Dodds, et al to assess the benefits of infiltration based urban stormwater management (Holman-Dodds, 2003). For the SDMP the specific inputs were adjusted for local precipitation and soil infiltration rates and the scenario three was added. The SCS Curve Number method uses the following equations to calculate runoff volume:

$$Y = \frac{\left(P_{24} - I_a\right)^2}{\left(P_{24} - I_a + S\right)P_{24}}$$

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

$$I_a = 0.2S$$

Where:

Y	is the runoff depth from a 24-hour storm as a proportion of the precipitation depth,
P_{24}	is the 24-hour precipitation depth [inches],
S	is the currently available soil-moisture deficit [inches],
I_a	is the initial abstraction [inches], and
CN	is the SCS Curve Number for the soil.

For the SDMP, the 2-year, 5-year, 10-year, and 100-year storms were evaluated with the total precipitation for a 24-hour duration (P_{24}) determined from the Sacramento City/County Drainage Manual. Based on the predominant soil type in the watershed (San Joaquin), the Curve Number for the bare soil was determined to be 85 based on published data by the SCS (SCS, 1986). Impervious surfaces were assumed to have a Curve Number of 98.

The results of the testing are presented in Table 15-4. As the table shows, directing runoff from impervious areas across pervious areas can reduce the total volume of runoff from a storm event. The benefits vary depending on the storm event, with relatively significant volume reductions, on a percentage basis, predicted for small storms and much smaller reductions for large storms. Directing runoff from impervious areas across half of the impervious area rather than all of the pervious area decreases the volume reduction by only 1 to 2 percent.

Table 15-4 Results of LID Curve Number Tests

Storm Return Period (years)	24-Hour Rainfall Total (in)	Scenario 1 - Developed with Impervious Area Directly Connected		Scenario 2 - Developed with Impervious Area Fully Directed Across Pervious Area		Scenario 3 - Developed with Impervious Area Directed Across Half of Pervious Area	
		Runoff Volume (in)	Reduction (%)	Runoff Volume (in)	Reduction (%)	Runoff Volume (in)	Reduction (%)
2	1.90	1.20	n/a	1.04	13.3	1.08	11.1
5	2.50	1.72	n/a	1.58	8.1	1.62	6.2
10	2.98	2.16	n/a	2.02	6.5	2.06	4.9
100	4.25	3.35	n/a	3.23	3.6	3.27	2.4

It is unreasonable to expect that runoff from the impervious areas can be uniformly distributed over all of the pervious areas in a watershed (Scenario 2). It is more reasonable to expect that the runoff from the previous areas could be directed over half of the pervious area in a watershed (Scenario 3). Even so, it is likely that runoff from some portion of the impervious area will continue to flow directly to a storm drain system. This may be offset by the fact that the Curve Number testing was based on the relatively low infiltration capacity of the existing soils. Soil amendments associated with LID techniques may increase the ability of the soil to hold and infiltrate runoff. Also, the Curve Number testing did not account for any storage of runoff in bio-retention type LID techniques such as rain gardens. These techniques could significantly increase infiltration, especially for small, frequent storm events. Therefore, for the SDMP, it was assumed that LID techniques will achieve runoff reductions similar to those calculated for the Scenario 3. As planning and design of projects in the watershed progresses, this assumption should be evaluated in more detail and model assumptions and facility sizing should be adjusted as necessary.

The results of the Curve Number infiltration tests were applied to the continuous simulation modeling with HEC-HMS by reducing the effective imperviousness of the proposed development areas. The appropriate value for the adjusted effective imperviousness was determined by testing a range of imperviousness values for a sample watershed with HEC-1 and selecting the value that most closely matched the runoff volume reductions determined from the Curve Number tests. Using that approach, the average imperviousness of the development area was reduced from 50 percent to 42 percent. Table 15-5 shows the calculated runoff volumes for a sample watershed assuming 50 percent imperviousness and 42 percent imperviousness. The reduction in runoff volume for the lower imperviousness value is presented and compared to the predicted runoff reduction developed with the Curve Number Tests described above. As the table shows, the volume reduction is under predicted for the 2-year event, but is reasonably close for the 5-year, 10-year, and 100-year events. Overall, it appears that that reducing the effective imperviousness from 50 to 42 percent provides a reasonable estimate of the potential reduction in runoff volume due to the implementation of LID measures.

Table 15-5. Runoff Volume Test Results

Storm Return Period (years)	Total Runoff Volume Subshed with 50% impervious cover (in)	Total Runoff Volume Subshed with 42% impervious cover (in)	Reduction in Runoff Volume (%)	Target Reduction in Runoff Volume from Curve Number Tests (%)
2	1.29	1.20	7.0	11.1
5	1.83	1.72	6.0	6.2
10	2.24	2.12	5.4	4.9
100	3.49	3.36	3.7	2.4

Directing runoff across pervious areas can also increase the lag time in a watershed, which can reduce the peak flow. A study of lag times for watersheds with traditional development and with low impact development (Hood, 2007) indicated that the lag time for the LID watershed was three times longer on average than for the traditional development watershed. For the Shed C study, it was assumed that LID would increase the lag time by a factor of two. The accuracy of this assumption is uncertain, but the results of the study are not significantly affected by the lag time because the peak flows into the Shed C Channel area will be controlled by detention basin storage. Modest differences in the predicted peak flows into the detention storage areas will not significantly affect the required storage volume or the peak discharge from the detention areas into the channel.

Detention Basins

Other key features of the drainage concept are the detention basins to be included at inflow points to the drainage corridor. These detention basins will provide runoff storage volume that will mitigate for potential increases in peak flood flows and will provide flow duration control to mitigate for the potential hydromodification effects. The basins will also provide stormwater quality treatment and the opportunity to create wetlands to mitigate for potential impacts to existing wetland features in the watershed. The general location of the detention basins that were assumed for the SDMP are presented on Figure 15-4.

For stormwater quality treatment purposes, the detention basins were assumed to be configured as Constructed Wetland Basins per the Sacramento Stormwater Quality Manual (Sacramento Stormwater Quality Partnership, 2007). This configuration assumes that each basin will include a permanent pool of water and will include four zones: a forebay, an open water zone, a wetland zone with aquatic plants, and an outlet zone. An area above the permanent pool will be provided to detain the stormwater quality treatment volume and slowly release it after a storm. For Shed C, additional storage volume is provided above what is required for stormwater quality treatment to mitigate hydromodification and flood control impacts. A typical detention basin layout is presented on Figure 15-5.

Wetland detention basins can be community amenities that provide multiple benefits including wildlife habitat, stormwater quality treatment, flood control, and flow duration control. Along with these benefits comes a higher level of maintenance to insure proper function and also the need to provide a supplemental water supply to maintain the permanent pool. It may not be necessary, or desirable, to configure each detention basin as a constructed wetland area. The wetland area required to mitigate for impacts will be determined after a more detailed study is performed that defines the existing habitat in the watershed and after discussions with the appropriate permitting agencies are held. At that time, a more informed decision can be made on the exact configuration of each of the proposed detention basins.

Detention basin sizes were determined in two steps. The permanent pool and stormwater quality volumes were determined based on the tributary area and imperviousness of the contributing watershed in accordance with the Stormwater Quality Design Manual (Sacramento Stormwater Quality Partnership, 2007). Runoff reduction credits to account for the implementation of basic LID features such as disconnected roof drains, disconnected pavement, and interceptor trees were determined from Appendix D of the design manual. The storage volumes required for flood and hydromodification control were determined through a series of model runs using the continuous simulation hydrologic model. Combinations of detention basin volumes and outlet configurations were iteratively tested with the model until the desired results were achieved. Tables 15-6 through 15-11 provide summaries of the detention basin volumes and outlet sizes. More discussion of the results from the modeling and the effectiveness of the detention basins in providing mitigation is presented later in this chapter.

The locations and sizes of the detention basins identified in the SDMP are preliminary. As more detailed land-use planning is performed, a more refined analysis should be performed to determine the appropriate number, locations, and sizes of the required detention basins. All runoff from developed areas should be directed into a detention basin. The detention basins, in conjunction with LID, will provide all the necessary stormwater quality treatment and flood flow mitigation for the developing areas within the watershed. As more refined studies are prepared, if it is found that runoff from some small, isolated areas cannot be feasibly directed to a detention basin, some direct discharge into the channel may be allowed if separate stormwater quality treatment is provided and a detailed study shows that the overall flood control and hydromodification goals for the watershed are still met.

Stable Channel Design

The existing Shed C Channel is essentially a man-made agricultural ditch that has been highly altered from its natural form. Its original alignment has been straightened and it has numerous ninety degree bends. The channel side slopes are uniform and steep and vegetation has been removed from many reaches. It is desired to create a more naturalized multi-functional channel corridor that will include a low flow channel designed to be stable based on the anticipated flow regime and natural processes. The low flow channel will meander within a larger floodplain corridor that will provide flood storage and conveyance, wetlands habitat, and passive recreation opportunities. The sizing of the channel involved the following steps:

- Develop a preliminary alignment for the channel.
- Determine the channel forming discharge and low flow geometry.
- Determine the channel meander dimensions.
- Check to insure that the geometry provides adequate flood conveyance capacity.

Table 15-6. Detention Basin Data for DET44A

Outlet Data		
Tributary Area	390	acres
Outlet Orifice Size	12	inches
Outlet Orifice Elevation	29.0	feet
Main Spillway Width	7.5	feet
Main Spillway Elevation	30.0	feet

Elevation-Volume-Flow Data										
Description	Elevation (ft)	Depth (ft)	Width (ft)	Length (ft)	Area (sf)	Area (ac)	Volume (ac-ft)	Outlet Orifice Flow(b) (cfs)	Spill Flow(b) (cfs)	Total Flow(b) (cfs)
Permanent Pool	29.0	0.0	400	800	320,000	7.3	0.0	0.0	0.0	0.0
Water Quality Pool	30.0	1.0	408	816	332,928	7.6	7.5	3.8	0.0	3.8
	31.0	2.0	416	832	346,112	7.9	15.3	5.4	21.0	26.4
	32.0	3.0	424	848	359,552	8.3	23.4	6.7	59.4	66.1
	33.0	4.0	432	864	373,248	8.6	31.8	7.7	109.1	116.8
	34.0	5.0	440	880	387,200	8.9	40.5	8.6	168.0	176.6
Top of Basin	35.0	6.0	448	896	401,408	9.2	49.6	9.4	234.8	244.2

Notes:

(a) All elevations are based on NGVD29.

(b) Flow data assumes no backwater effects from the Shed C Channel. This assumption was tested with event modeling using HEC-RAS and found to be reasonable.

Table 15-7. Detention Basin Data for DETA4B

Outlet Data		
Tributary Area =	216	acres
Outlet Orifice Size	9	inches
Outlet Orifice Elevation	25.6	feet
Main Spillway Width	4.2	feet
Main Spillway Elevation	27.1	feet

Elevation-Volume-Flow Data										
Description	Elevation (ft)	Depth (ft)	Width (ft)	Length (ft)	Area (sf)	Area (ac)	Volume (ac-ft)	Outlet Orifice Flow(b) (cfs)	Spill Flow(b) (cfs)	Total Flow(b)(cfs)
Permanent Pool	25.6	0.0	295	590	174,050	4.0	0.0	0.0	0.0	0.0
Water Quality Pool	26.4	0.8	301	603	181,684	4.2	3.3	1.9	0.0	1.9
	27.4	1.8	309	619	191,457	4.4	7.5	2.9	1.9	4.8
	27.6	2.0	311	622	193,442	4.4	8.4	3.1	4.2	7.2
	28.0	2.4	314	628	197,443	4.5	10.2	3.4	10.0	13.4
	29.0	3.4	322	644	207,626	4.8	14.9	4.0	30.8	34.8
	30.0	4.4	330	660	218,064	5.0	19.8	4.5	58.1	62.6
	31.0	5.4	338	676	228,758	5.3	24.9	5.0	90.6	95.6
Top of Basin	32.0	6.4	346	692	239,709	5.5	30.3	5.5	127.6	133.0

Notes:

(a) All elevations are based on NGVD29.

(b) Flow data assumes no backwater effects from the Shed C Channel. This assumption was tested with event modeling using HEC-RAS and found to be reasonable.

Table 15-8. Detention Basin Data for DETA4C

Outlet Data		
Tributary Area =	94	acres
Outlet Orifice Size	6	inches
Outlet Orifice Elevation	23.5	feet
Main Spillway Width	10	feet
Main Spillway Elevation	25.5	feet

Elevation-Volume-Flow Data										
Description	Elevation (ft)	Depth (ft)	Width (ft)	Length (ft)	Area (sf)	Area (ac)	Volume (ac-ft)	Outlet Orifice Flow(b) (cfs)	Spill Flow(b) (cfs)	Total Flow(b) (cfs)
Permanent Pool	23.5	0.0	195	390	76,050	1.7	0.0	0.0	0.0	0.0
Water Quality Pool	24.5	1.0	203	406	82,418	1.9	1.8	1.0	0.0	1.0
	25.5	2.0	211	422	89,042	2.0	3.8	1.4	0.0	1.4
	25.6	2.1	212	424	89,718	2.1	4.0	1.4	0.9	2.3
	26.0	2.5	215	430	92,450	2.1	4.8	1.5	9.9	11.4
	27.0	3.5	223	446	99,458	2.3	7.0	1.8	51.4	53.2
	27.5	4.0	227	454	103,058	2.4	8.2	1.9	79.2	81.1
Top of Basin	28.5	5.0	235	470	110,450	2.5	10.6	2.1	145.5	147.6

Notes:

(a) All elevations are based on NGVD29.

(b) Flow data assumes no backwater effects from the Shed C Channel. This assumption was tested with event modeling using HEC-RAS and found to be reasonable.

Table 15-9. Detention Basin Data for DETA5

Outlet Data		
Tributary Area =	248	acres
Outlet Orifice Size	10	inches
Outlet Orifice Elevation	28.0	feet
Main Spillway Width	5.8	feet
Main Spillway Elevation	29.2	feet

Elevation-Volume-Flow Data										
Description	Elevation (ft)	Depth (ft)	Width (ft)	Length (ft)	Area (sf)	Area (ac)	Volume (ac-ft)	Outlet Orifice Flow[b] (cfs)	Spill Flow[b] (cfs)	Total Flow[b] (cfs)
Permanent Pool	28.0	0.0	310	620	192,200	4.4	0.0	0.0	0.0	0.0
Water Quality Pool	29.0	1.0	318	636	202,248	4.6	4.5	2.7	0.0	2.7
	30.0	2.0	326	652	212,552	4.9	9.3	3.8	11.6	15.4
	30.1	2.1	327	654	213,596	4.9	9.8	3.9	13.9	17.7
	31.0	3.0	334	668	223,112	5.1	14.3	4.6	39.2	43.8
	32.0	4.0	342	684	233,928	5.4	19.5	5.3	76.1	81.4
	33.0	5.0	350	700	245,000	5.6	25.0	6.0	120.3	126.3
Top of Basin	34.0	6.0	358	716	256,328	5.9	30.8	6.5	170.8	177.3

Notes:

(a) All elevations are based on NGVD29.

(b) Flow data assumes no backwater effects from the Shed C Channel. This assumption was tested with event modeling using HEC-RAS and found to be reasonable.

Table 15-10. Detention Basin Data for DETA5B

Outlet Data		
Tributary Area =	92	acres
Outlet Orifice Size	6	inches
Outlet Orifice Elevation	22.0	feet
Main Spillway Width	12	feet
Main Spillway Elevation	23.8	feet

Elevation-Volume-Flow Data										
Description	Elevation (ft)	Depth (ft)	Width (ft)	Length (ft)	Area (sf)	Area (ac)	Volume (ac-ft)	Outlet Orifice Flow[b] (cfs)	Spill Flow[b] (cfs)	Total Flow[b](cfs)
Permanent Pool	22.0	0.0	190	380	72,200	1.7	0.0	0.0	0.0	0.0
Water Quality Pool	23.0	1.0	198	396	78,408	1.8	1.7	1.0	0.0	1.0
	24.0	2.0	206	412	84,872	1.9	3.6	1.4	3.0	4.4
	24.1	2.1	207	414	85,532	2.0	3.8	1.4	5.5	6.9
	25.0	3.0	214	428	91,592	2.1	5.6	1.7	44.2	45.8
	25.5	3.5	218	436	95,048	2.2	6.7	1.8	74.5	76.3
Top of Basin	26.0	4.0	222	444	98,568	2.3	7.8	1.9	109.6	111.6

Notes:

[a] All elevations are based on NGVD29.

[b] Flow data assumes no backwater effects from the Shed C Channel. This assumption was tested with event modeling using HEC-RAS and found to be reasonable.

Table 15-11. Detention Basin Data for DETA6M

Outlet Data		
Tributary Area =	136	acres
Outlet Orifice Size	7	inches
Outlet Orifice Elevation	21.0	feet
Main Spillway Width	8.2	feet
Main Spillway Elevation	22.0	feet

Elevation-Volume-Flow Data										
Description	Elevation (ft)	Depth (ft)	Width (ft)	Length (ft)	Area (sf)	Area (ac)	Volume (ac-ft)	Outlet Orifice Flow[b] (cfs)	Spill Flow[b] (cfs)	Total Flow[b] (cfs)
Permanent Pool	21.0	0.0	235	470	110,450	2.5	0.0	0.0	0.0	0.0
Water Quality Pool	22.0	1.0	243	486	118,098	2.7	2.6	1.3	0.0	1.3
	23.0	2.0	251	502	126,002	2.9	5.4	1.9	23.0	24.8
	23.1	2.1	252	504	126,806	2.9	5.7	1.9	26.5	28.4
	24.0	3.0	259	518	134,162	3.1	8.4	2.3	64.9	67.2
	24.5	3.5	263	526	138,338	3.2	10.0	2.4	90.8	93.2
Top of Basin	25.5	4.5	271	542	146,882	3.4	13.2	2.8	150.3	153.1

Notes:

(a) All elevations are based on NGVD29.

(b) Flow data assumes no backwater effects from the Shed C Channel. This assumption was tested with event modeling using HEC-RAS and found to be reasonable.

Channel Alignment

A preliminary channel alignment was developed that generally follows the existing channel alignment but provides a more natural, meandering path that eliminates the sharp bends. The channel ties into the fixed points at the upstream end near the existing detention basin and at the downstream end at Bruceville Road. The preliminary alignment is shown on Figure 15-4. It is anticipated that the ultimate alignment of the channel could be different based on land planning constraints that are identified as a Specific Plan for the Southeast Policy Area is developed. However, the general concept of creating a more naturalized alignment should be retained.

Channel Forming Discharge

The channel forming discharge is the flow rate that is most effective in shaping a stream channel. For the SDMP, the channel forming discharge was estimated using the effective work method, which provides a way to estimate the flow magnitude associated with the maximum potential erosion over a long period. First, a histogram was used to create a flow frequency distribution of hourly peak flows (in 10 cfs intervals) from the continuous simulation model results. The potential erosion was determined using the Andrew Simon’s effective work equation for consolidated materials:

$$W = \sum_{i=1}^{n} k(\tau_i - \tau_c)^{1.5}\Delta t$$

Where:

W = the total work performed in dimensionless units

k = erodibility coefficient

τ_i = the applied hydraulic shear stress, lbs/sf

τ_c = the critical shear stress that initiates erosion, lbs/sf

For the SDMP, k was ignored (or assumed to be 1.0) because it is the same for base conditions and buildout conditions and does not affect the results. The applied shear stress was based on the following equation:

$$\tau_i = \gamma DS$$

Where:

γ = the unit weight of water (62.4 lbs/sf)

D = the depth of flow, ft

S = the slope of the channel, ft/ft

The critical shear stress was determined based on Figure 3-1 from Guidance Manual for Design of Multi-Functional Drainage Corridors, County of Sacramento, 2003. That figure is shown in Figure 15-6 below. Based on that information, the critical shear stress was estimated to be 0.10 lbs/sf, which is an appropriate value for fairly compact to loose clay soil.

To perform the work calculations, it was necessary to make an initial estimate of the channel forming flow and channel geometry. The channel forming flow was first estimated by determining the flow-frequency relationship in the channel for mitigated buildout conditions. Channel forming discharges typically vary between a 1-year to 2-year event, with a 1.5-year event being a reasonable average (Leopold, 1964). Therefore, the 1.5-year event was used as a starting point to estimate the channel forming discharge.

Using the estimated channel forming discharge, the average width and depth of the low flow channel was determined using the Manning's Equation:

$$d = \left[\frac{Q \times n}{1.49(W / D)\sqrt{S}} \right]^{3/8}$$

Where:

d = the average depth of the low flow channel, ft

Q = the channel forming discharge, cfs

n = Manning's roughness coefficient

W/D = the width the depth ratio of the low flow channel

S = the slope of the channel, ft/ft

To use the equation it is necessary to estimate the width to depth ratio (W/D) for the channel. This ratio is dependent on the ability of the channel to resist erosion, which is a function of soil characteristics and vegetation. Measurements of width to depth ratios for existing creeks in the Sacramento area were performed by Zentner and Zentner and are published in the Guidance Manual for Design of Multi-Functional Drainage Corridors, County of Sacramento, 2003. Laguna Creek near Bradshaw Road, which has the same soil type as those along the Shed C Channel, had a measured W/D ratio between 12 and 14. Therefore, a W/D ratio of 12 was selected for the SDMP.

Using the initial channel dimensions, the effective work method was applied and the channel forming discharge was calculated. If the calculated discharge was different than the original estimate, the new value was used to re-size the channel and the process continued iteratively until the flow value used to size the channel matched the channel forming flow calculated by the effective work method. The reasonableness of the channel forming flow was then checked against the flood frequency curve.

Using the process described above, the preliminary channel forming discharge and low flow channel geometry was determined for three reaches along the channel. The reaches are shown on Figure 15-4 and are described below.

- Reach 1 – From the Elk Grove Promenade detention basin to McMillan Road.
- Reach 2 – From McMillan Road to the confluence with the channel from the Laguna Ridge Specific Plan area.
- Reach 3 – From the confluence with the Laguna Ridge Specific Plan channel to Bruceville Road.

Figures 15-7 through 15-9 present the results from the effective work method for the three reaches. As shown on Figure 15-7, in Reach 1, the large majority of peak flows over the 73 year period of record are 65 cfs or less. However, flows in that range are too small to produce shear stresses above the critical shear stress and therefore those flows do not perform work on the channel. Flows above 65 cfs do cause work to be performed on the channel and the flow rate that produces the most work over the modeled period is 115 cfs. Therefore 115 cfs is selected as the channel forming discharge for Reach 1. The results for Reaches 2 and 3 are shown on Figures 15-8 and 15-9, respectively. As shown on those figures, the channel forming discharge is approximately 165 cfs for Reach 2 and 185 cfs for Reach 3. Figure 15-10 presents the flow frequency curves for the three reaches. As can be seen on that figure, the return periods of the channel forming flows for the three reaches varies between 0.9 and 1.7 years. The values for Reaches 1 and 2 fall within the typical range of 1 to 2 years as discussed above. The value for Reach 3 is just below this range.

Using these flows along with Manning’s equation and the assumed width to depth ratio as discussed above, the average dimensions of the low flow channel were calculated using a Manning’s n of 0.04 and a slope of 0.0008. Because the equation provides the average dimensions based on a rectangular channel, the resultant dimensions were converted to an equivalent trapezoidal shape based on a side slope of 3 to 1 (horizontal to vertical). Table 15-12 presents the calculated dimensions of the low flow channel for each reach.

Table 15-12. Shed C Low Flow Channel Geometry

Reach	Est. Channel Forming Flow (cfs)	Approx. Return Period (years)	Depth (ft)	Average Width w (ft)	Trapezoidal Bottom Width (ft)	Trapezoidal Top Width (ft)	Wave Length L (ft)	Belt Width B (ft)	Radius of Curvature r_c (ft)
1. Promenade to McMillan Road	115	1.7	2.3	27	21	34	266	151	53
2. McMillan Road to LRSP Channel	165	1.4	2.6	31	24	39	309	176	62
3. LRSP Channel to Bruceville Road	185	0.9	2.8	33	25	42	332	189	66

Channel Meander Dimensions

After determining average low flow channel sizes, the meander dimensions can be estimated. The meander dimensions are based on equations developed from empirical observations. For the SDMP, the meander dimensions were estimated using the equations presented in the Stream Corridor Restoration, Principles, Processes, and Practices, Federal Interagency Stream Restoration Group, USDA, 2001. These equations are as follows:

$B = 3.7w^{1.12}$

$M_L = 4.4w^{1.12}$

$L = 6.5w^{1.12}$

$r_c = 1.3w^{1.12}$

The variables in the above equation are shown in Figure 15-11 below. For the SDMP, because detailed channel design was not performed, the main variable of interest was the meander amplitude. This variable establishes the minimum width of the floodway corridor (i.e. the bottom width of the flood control channel). The calculated meander dimensions for the channel are presented on Table 15-12.

Other Drainage System Features

An additional mitigation measure is proposed for the Shed C drainage system related to the existing detention basin constructed for the Elk Grove Promenade project. The channel rehabilitation measures proposed with the SDMP will increase the channel flow capacity above that for the existing channel. As a result, the proposed channel will carry more flow at lower depths. This will change the stage-discharge relationship at the existing Promenade detention basin. For the same water surface elevation in the detention basin, the discharge would be larger after construction of the channel improvements than under current conditions. This would not reduce the flood control effectiveness of the basin, but it would induce hydromodification impacts. Modifications to the existing outlet were tested to determine a potential mitigation for this effect. From the testing, it was determined that eliminating one of the three 36-inch pipe outlets from the detention basin will mitigate the hydromodification impacts without compromising the flood control performance. Therefore, it is recommended that one of the 36-inch outlets be blocked when the channel improvements are constructed.

The proposed drainage system corridor will also provide the opportunity to construct a trail system that will provide recreation opportunities. The location and alignment of the trail system should be established as part of a more detailed study prior to development in the watershed.

Effectiveness of Mitigation Measures for Hydromodification

The effectiveness of the mitigation measures for hydromodification were assessed by comparing the results for base conditions and buildout conditions at the downstream boundary of the City (Bruceville Road). The cumulative effective work and flow duration results were compared.

Figure 15-12 presents the comparison of the cumulative effective work performed in the channel based on Simon’s effective work equation presented earlier in this chapter. For the comparison, the erosion potential due to buildout was measured as the ratio of the cumulative effective work for Buildout Conditions versus Base Conditions as follows:

$E_{p =} W_{post}/W_{base,}$ where:

E_p = the erosion potential

W_{post} = the cumulative work performed for post project conditions (buildout conditions)

W_{base} = the cumulative work performed for base conditions

As shown on Figure 15-12, the proposed project could increase the erosion potential at the downstream boundary by approximately 8 percent. For comparison purposes, the erosion potential for buildout conditions without the proposed detention basins was also determined. For that condition, the potential increase in erosion is approximate by 76 percent.

Figure 15-13 presents a comparison of the flow duration results. As indicated on the figure, buildout within Shed C without detention would significantly increase flow durations within the watershed. With detention included, the flow durations are comparable to those for Base Conditions.

Specific criteria for the City of Elk Grove have not yet been established related to hydromodification mitigation. In a recent study of Laguna Creek (Geosyntec, 2007), it was proposed that development projects should not cause an increase in potential erosion of more than 20 percent. Based on that standard, the proposed mitigation measures for Shed C appear to be appropriate and they will effectively mitigate for the potential hydromodification impacts in the watershed. The effectiveness of the facilities for mitigating potential flood impacts is evaluated in detail in the next section.

EVENT BASED ANALYSIS

A traditional event based analysis was performed to assess the flood control performance of the proposed facilities. Flood flow hydrographs were calculated with SacCalc/HEC-1 for pre-development and buildout conditions for the 10-year and 100-year storm events. A storm duration of 24-hours was used for both events. Water surface profiles were calculated for pre-project and buildout conditions using an unsteady HEC-RAS model.

Hydrologic Analysis

A hydrologic analysis was performed to determine the 10-year and 100-year flows entering the Shed C Channel. The flood flows were calculated using the same SacCalc/HEC-1 models that were used as the basis of the continuous simulation model described previously. The main difference between the event based model and the continuous simulation model is that the event based model uses a simpler method to account for rainfall losses. The event based model uses an initial loss rate to account for rainfall that is intercepted by vegetation or stored in surface depressions and a constant loss rate to account for infiltration into the soil. The loss rates used for the SDMP are based on those presented in the Sacramento City/County Hydrology Manual.

For the SacCalc modeling, Shed C was divided into 29 subsheds as shown on Figure 15-3. For the pre-development conditions model, it was assumed that no development had occurred in the watershed (i.e. the Promenade and Laguna Ridge projects were not developed). For buildout conditions, it was assumed that the entire area within the City limits was developed. The buildout land-use conditions for the event based analysis are exactly the same as those used for the continuous simulation modeling. The SacCalc models were used to calculate the flood flows entering the Shed C Channel. The model was not used to route flows within the channel; this was done using an unsteady HEC-RAS hydraulic model as described below.

Hydraulic Analysis

A hydraulic analysis was performed using HEC-RAS to determine the flows and water surface elevations within the Shed C Channel for the 10-year and 100-year storm events. Descriptions of the various features of the HEC-RAS model are provided below.

Channel Geometry

The hydraulic model of the Shed C Channel begins just downstream of the existing Promenade detention basin at the west boundary of Subshed A2. The model extends downstream to the west side of Interstate 5. The channel geometry was defined using approximately 150 cross sections.

The cross section locations within the City limits are shown on Figure 15-14. For pre-development conditions, the cross sections from the upstream end of the model to approximately 1,000 feet downstream of McMillan Road are based on a field survey performed by West Yost in 2009. The remaining cross sections are based on a combination of field survey data collected by Murray Smith & Associates in the late 1990's and LIDAR generated topographic mapping. All elevations in the SDMP are based on the National Geodetic Vertical Datum of 1929. The original Murray Smith survey data was unavailable for review. However, a comparison of the cross section data with LIDAR topographic data provided by the City indicated that there is general agreement between the elevations of the cross sections and the LIDAR topographic mapping in the overbank areas.

For buildout conditions, the cross sections within the City limits were replaced to represent the proposed channel geometry based on the results from the continuous simulation model. Separate cross sections were developed for each three reaches of the Shed C Channel that are shown on Figure 15-4. The proposed low flow channel geometry for each of the three reaches is listed on Table 15-12. The preliminary bottom widths of the floodway portions of the channel were based on the Belt Width values on Table 15-12. The average side slopes of the low flow and flood control channel were set at 3:1 and 4:1, respectively. These are average values and the expectation is that the side slopes will vary to provide a more natural appearance. A typical cross section is shown on Figure 15-15.

Manning's Roughness Coefficients

For pre-development conditions, the Manning's roughness coefficients range from 0.04 to 0.06 within the main channel and 0.04 to 0.05 in the overbank areas. For buildout conditions, the roughness coefficients for the proposed channel were set at 0.04 within the low flow channel and 0.08 within the overbank areas. The relatively large value used in the overbank area for buildout conditions is intended to allow for the establishment of significant riparian vegetation and to reduce maintenance requirements.

Bridges and Culverts

There are nine existing bridge or culvert crossings included in the model. Within the City limits, there are six culvert crossings. Five of these culverts are small pipe culverts used for farm roads that cross the channel. The other set of culverts within the City is located at Bruceville Road, where two 48-inch concrete pipelines cross under the roadway. Downstream of the City there are bridge structures at the Union Pacific Railroad and Interstate 5. At Franklin Boulevard, there are four 15 feet x 4.5 feet concrete box culverts.

For buildout conditions, the existing small pipe culverts within the City were removed from the model and it was assumed that no road crossings would be constructed at those locations. It was assumed that new culverts would be required at Bruceville Road and McMillan Road. If additional road crossings are proposed as more detailed land-use planning occurs in the watershed, the culverts or bridges will need to be appropriately sized using a hydraulic model to insure adequate capacity.

Detention Basins

For predevelopment conditions, no detention basins were included in the model. For buildout conditions, detention basins were added to the model based on the results from the continuous simulation modeling as presented on Tables 15-6 through 15-11. One exception is the detention basin proposed for Subshed A5. This detention basin is not anticipated to be constructed immediately adjacent to the channel and backwater from the channel is not expected to affect the outflow characteristics. Therefore, this detention basin was not added to the model. Outflow from the detention basin was calculated with the SacCalc/HEC-1 model and input into the HEC-RAS model.

Boundary Conditions

For the 10-year and 100-year water surface calculations, the water surface elevations at the downstream end of the model were set at constant elevations of 7.3 feet and 8.6 feet, respectively. These values are lower than the corresponding peak water surface elevations in the Beach Stone Lakes area at the downstream end of the model. However, they are considered reasonable for the SDMP because the peak flows from Shed C are expected to occur well before the peak stage occurs in the Beach Stone Lakes area. This is because the peak stages in the Beach Stone Lakes area are controlled by flows from the Cosumnes River and Mokelumne River watersheds that back up into the Beach Stone Lakes area. Due to the large size of the Cosumnes and Mokelumne River watersheds, the peak flows from these rivers occur well after the peak flows from Shed C. As a sensitivity test, the downstream stage for the 100-year storm event was increased from 8.6 feet to 12.0 feet. Even with the large increase in the starting downstream water surface elevation, the water surface elevations from the test model and pre-development model merge at Franklin Boulevard, which is well downstream of the City limits. Therefore, the results of the SDMP are not sensitive to variations in the starting water surface elevation at the downstream end of the hydraulic model.

Results from the Hydrologic and Hydraulic Analyses

The HEC-RAS model was used to route the inflows from the tributary subsheds through the Shed C Channel and to calculate water surface elevations in the channel using an unsteady-state analysis. For pre-development conditions, the channel capacity is insufficient to contain the 10-year or the 100-year flows within the channel and significant overbank flooding is predicted. Figure 15-16 presents the calculated water surface profiles for pre-development conditions within the City limits. Figure 15-14 shows the pre-development floodplain limits for the 100-year event. None of the culverts within the City have capacity to pass event the 10-year flow without overtopping. Also, it appears that structure flooding may occur during a 100-year storm near cross sections 5685, 7040, and 9730. However, because the assessment of potential flooding in that area is based on 2-foot contour LIDAR topographic mapping, which has limited accuracy, the structures may actually lie outside of the 100-year floodplain. Detailed output tables from the HEC-RAS model for pre-development conditions are provided in Attachment 15A.

As mentioned above, the width of the flood control portion of the proposed channel within the City limits for buildout conditions was based on the Belt Width value determined during the stable channel analysis. The results of the event based hydraulic modeling indicated that additional flood control width is required within lower end of Reach 2 and all of Reach 3. These reaches of the channel are relatively shallow compared to the other reaches because of the need to tie-in to the existing channel at the downstream end of the City. Therefore, an additional 40 feet of channel width was required for these reaches. The proposed channel dimensions are shown on Table 15-13. With these channel improvements, the 100-year water flood flows will be contained within the channel with approximately 1 foot of freeboard. Figure 15-17 presents the calculated water surface profiles for buildout conditions within the City limits. Detailed output tables from the HEC-RAS model for buildout conditions are provided in Attachment 15B.

Table 15-13. Proposed Channel Dimensions

Reach	Reach Length (ft)	Low Flow Depth (ft)	Low Flow Bottom Width (ft)	Low Flow Top Width (ft)	Flood Control Bottom Width (ft)	Approx. Flood Control Top Width (ft)
1. Promenade to McMillan Road	5,700	2.3	21	34	151	190
2a. McMillan Road to 2,900 feet Downstream	2,200	2.6	24	39	176	204
2a. 2,900 feet Downstream of McMillan Road to LRSP Channel	2,145	2.6	24	39	207	230
3. LRSP Channel to Bruceville Road	1,610	2.8	25	42	230	250

Note: LRSP = Laguna Ridge Specific Plan

The modeling results indicate that the proposed drainage system including LID, detention, and channel improvements will adequately mitigate for potential flood flow increases downstream of the City. Table 15-14 lists the calculated peak flood flows at the downstream end of the City (Bruceville Road). As shown in the table, the peak flood flows for the 10-year and 100-year storms are predicted to be reduced slightly.

Table 15-14. Comparison of Flood Flows in cfs

Location	10-Year		100-Year	
	Pre-Development	Buildout	Pre-Development	Buildout
Bruceville Road	471	434	816	798

RECOMMENDATIONS

It is recommended that a multi-functional drainage system be constructed in the Shed C watershed to accommodate future development in the watershed and to restore some of the natural function of the watershed that has been lost. The multi-functional corridor should include a low flow channel that is stable and self-sustaining, and meanders within a larger floodway corridor that will provide flood conveyance as well as wetlands habitat. At key points along the corridor, detention basins should be constructed to provide storage volume to mitigate for potential flood flow and hydromodification impacts. The channel and detention basins will also provide the opportunity to establish riparian habitat. It is recommended that the corridor also include a path that will provide recreational and educational opportunities for the City’s residents.

The SDMP demonstrates that the proposed concept is hydraulically feasible and presents a preliminary configuration of the required facilities and sizes. As land-use planning progresses in the watershed the SDMP should be refined. It is anticipated that the overall land-use density within the watershed could change and the land-use density within individual subsheds will almost certainly change. After there is more certainty in the land-plan the hydrologic modeling should be updated to better represent the land-use densities and subshed delineations. The alignment of the proposed channel corridor should be refined to fit with the proposed land plan and the channel size should be refined based on the updated hydrology. The locations and sizes of the detention basins should also be refined.

REFERENCES

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Wood Rodgers, 2005. Drainage Document for Laguna Ridge Specific Plan, Supplemental Master Drainage Plan for Local Drainage Area Shed C.

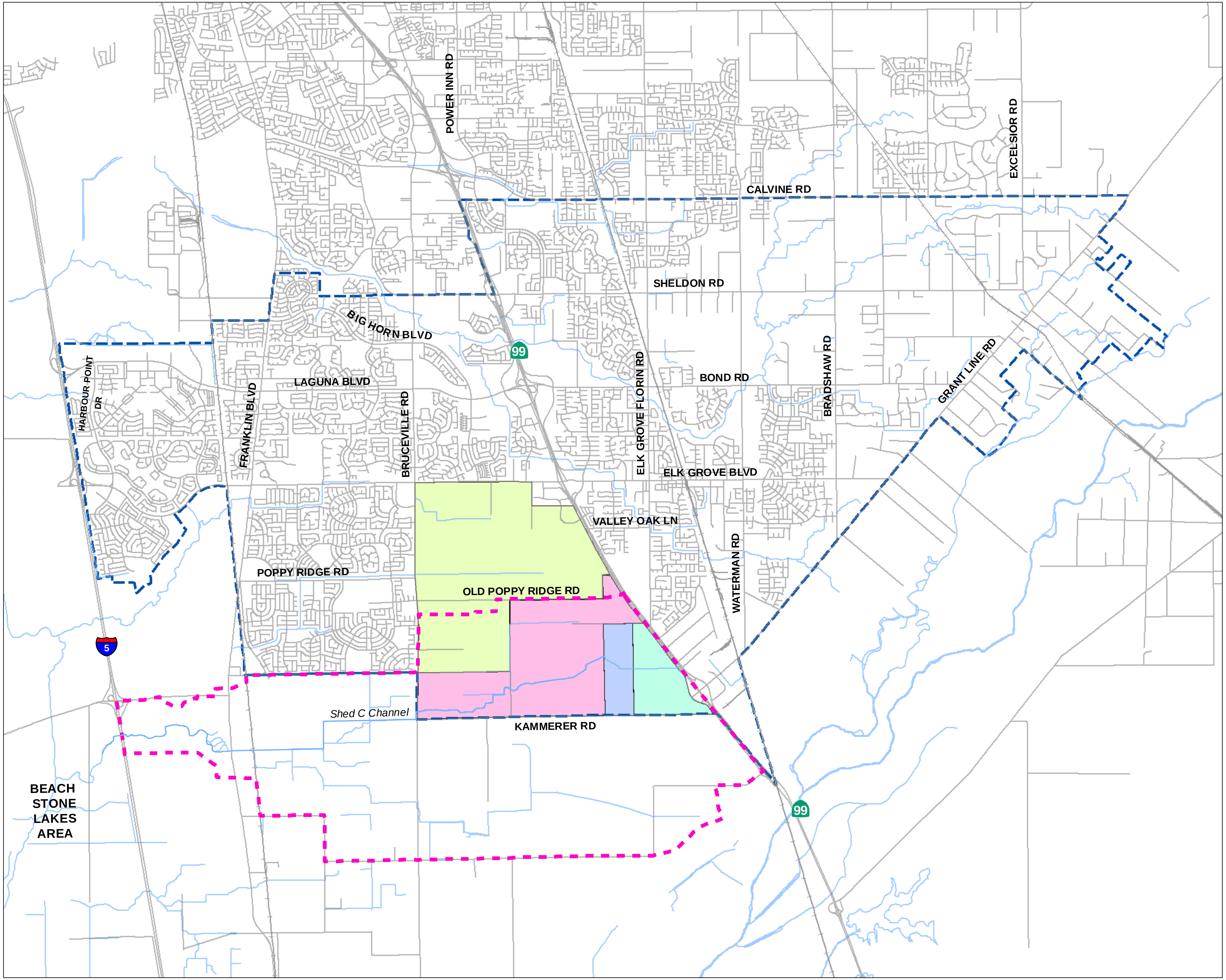


FIGURE 15-1

City of Elk Grove Storm Drainage Master Plan Volume II

SHED C LOCATION MAP

NOTES:

LEGEND:

- - - City Limit
- - - Shed C Watershed
🟩 Laguna Ridge Specific Plan Area
🟪 Southeast Policy Area
🟦 Sterling Meadows
🟩 Elk Grove Promenade (Lent Ranch)

WEST YOST
ASSOCIATES
Consulting Engineers

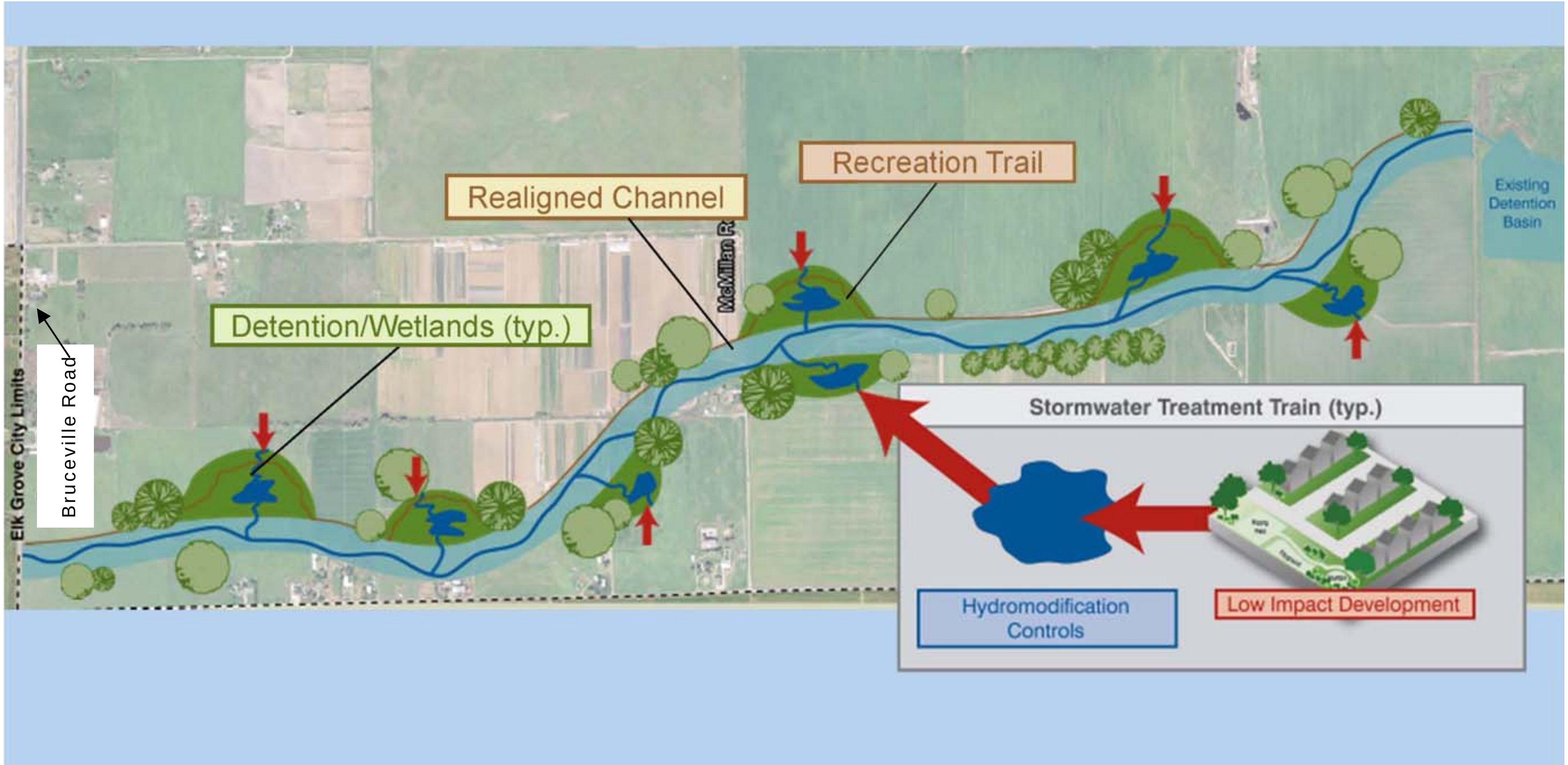
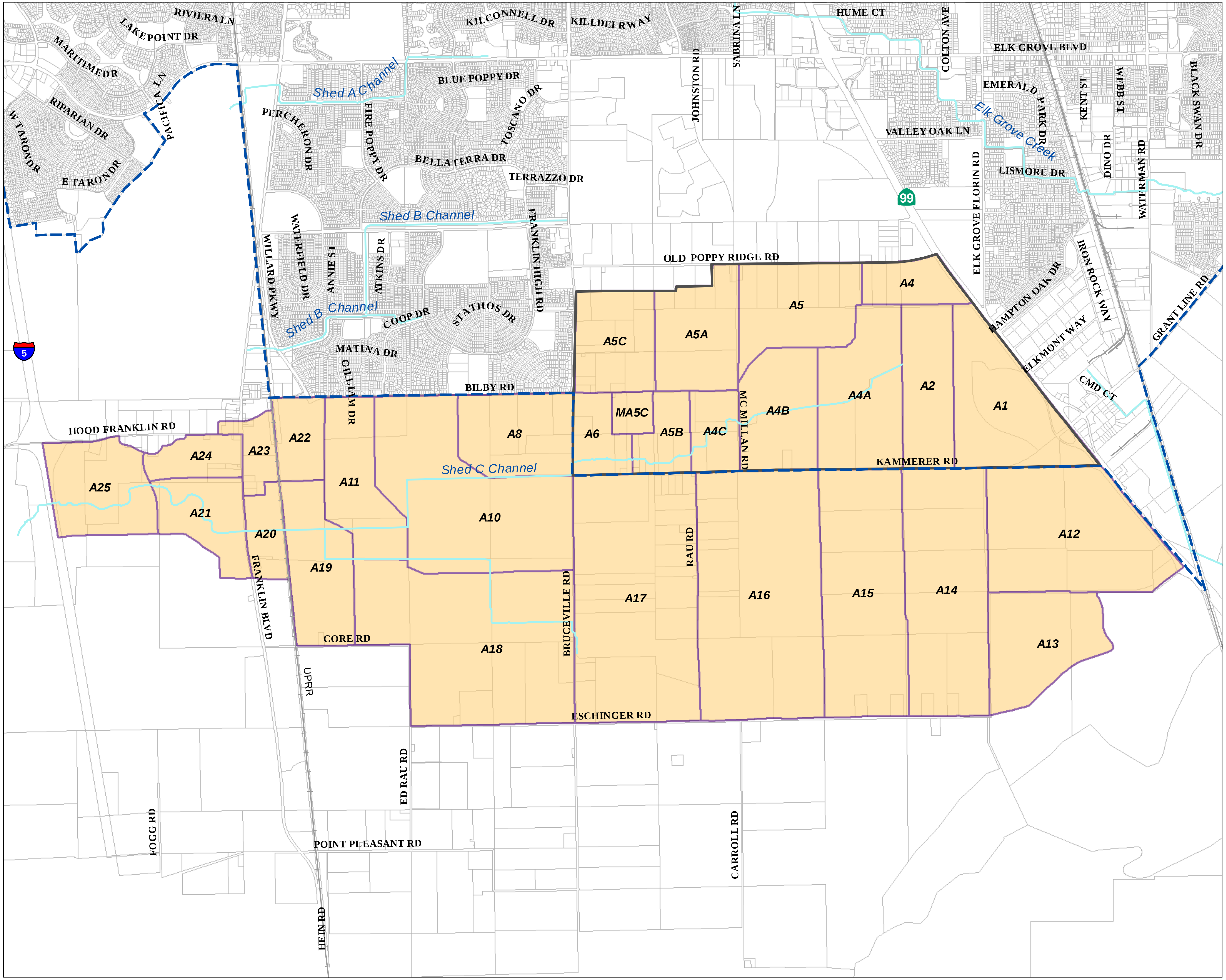


Figure 15-2. Shed C Drainage Corridor Concept

FIGURE 15-3

City of Elk Grove
Storm Drainage Master Plan
Volume II

SHED C SUBSHED MAP



N

0 3,000 6,000
Feet

LEGEND:

▭ City Limit

▨ Shed C Subsheds

— Existing Channel

CITY OF ELK GROVE CALIFORNIA

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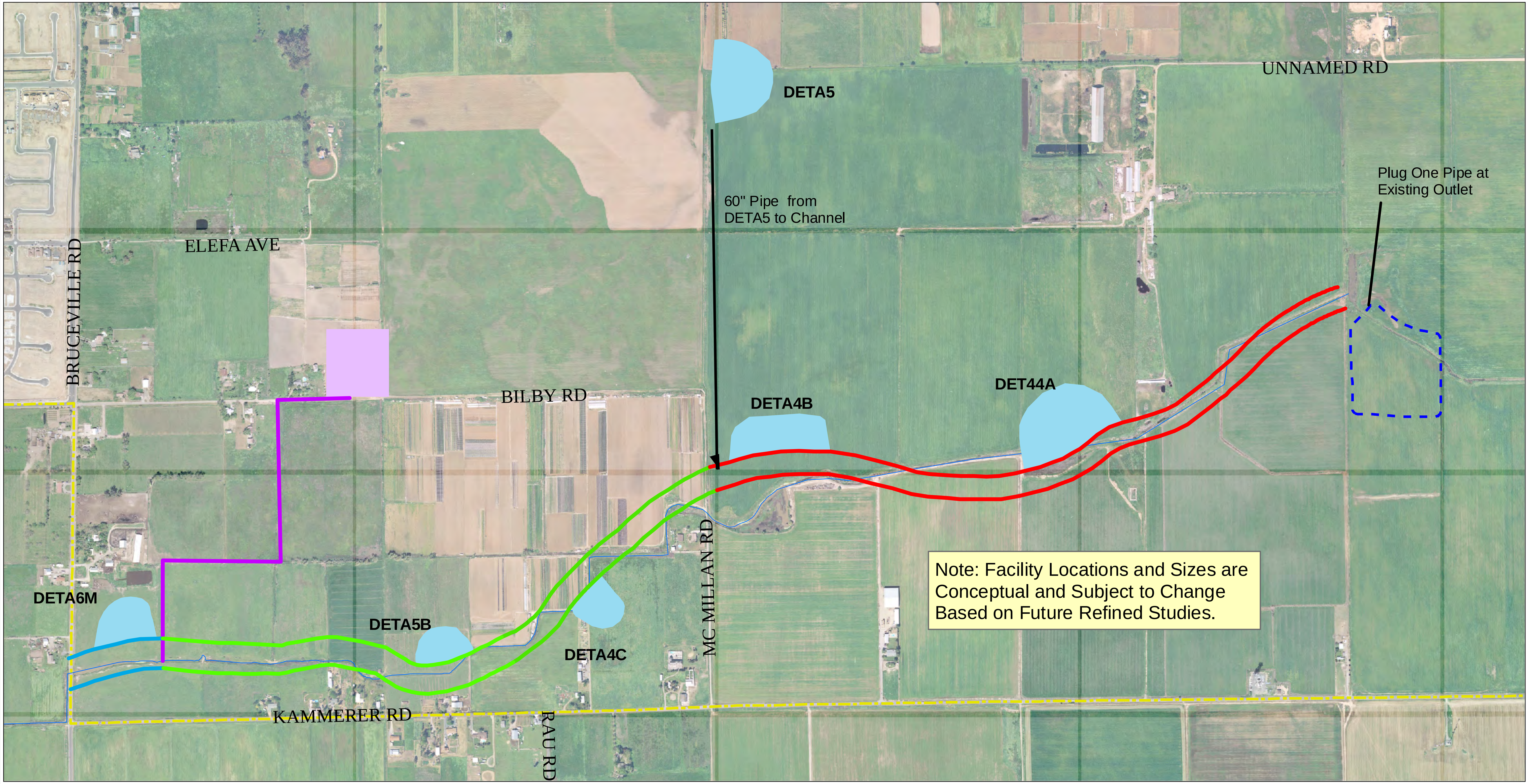
Consulting Engineers



Note: Facility Locations and Sizes are Conceptual and Subject to Change Based on Future Refined Studies.

Legend

- ― Existing Channel
- ― Proposed Channel Reach 1
- ― Proposed Channel Reach 2
- ― Proposed Channel Reach 3
- - - Promenade Detention (Existing) *Existing Detention Basin*
- ― Channel Proposed with LRSP
- ▨ Detention Proposed with LRSP
- ▨ Proposed Detention Basin
- ▭ City Limits

N

0 800 1,600
Feet

FIGURE 15-4

City of Elk Grove
Storm Drainage Master Plan
Volume II

PROPOSED DRAINAGE FACILITIES

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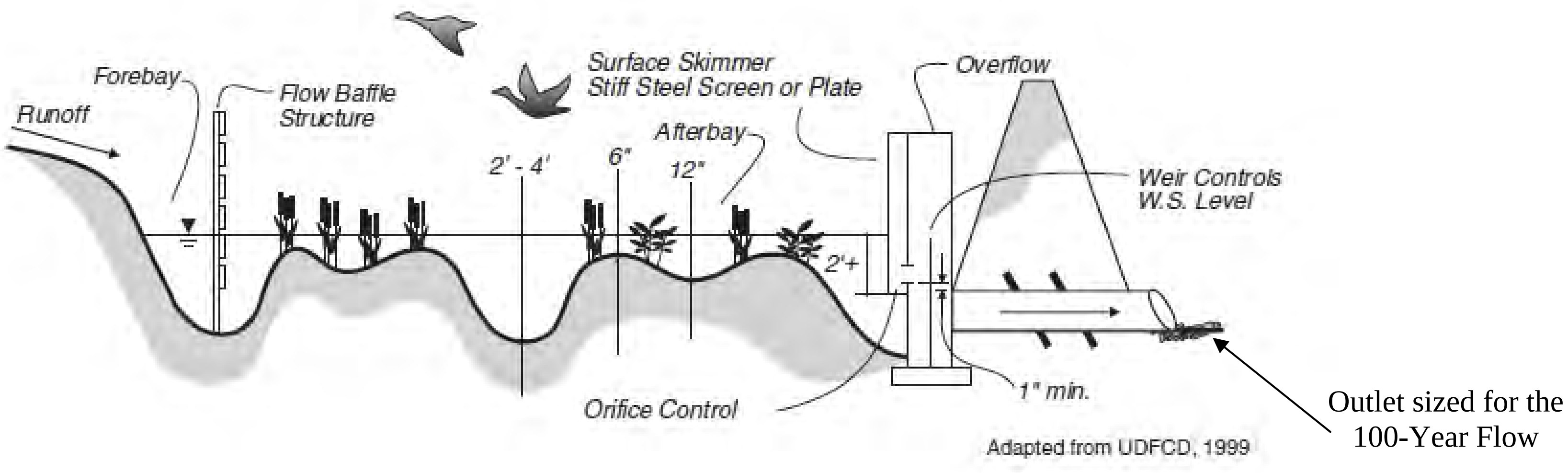


Adapted from UDFCD, 1999

Figure 15-5. Typical Detention Basin Layout

Note: Adapted from Figure CWB-1 from Stormwater Quality Design Manual for the Sacramento and south Placer Region.

(ASCE Manual No. 77, pg 329)

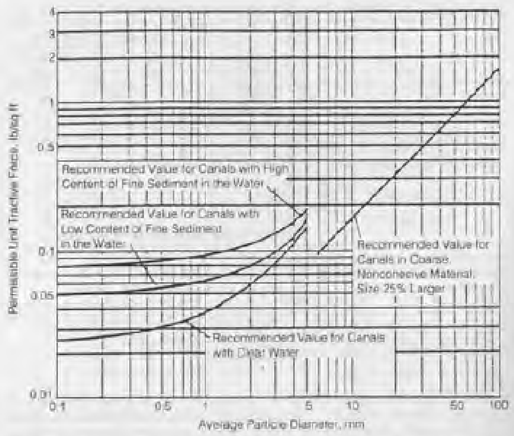


Figure 9.5—Recommended permissible unit tractive forces for canals in noncohesive material (lb/sq ft × 47.88 = Pa) (Chow, 1959).

bridge piers is caused by the vortex resulting from water piling up on the upstream edge and subsequent acceleration of flow around the nose of the pier.

Local scour is a function of a combination of several of the following factors:

(a) Slope of the channel

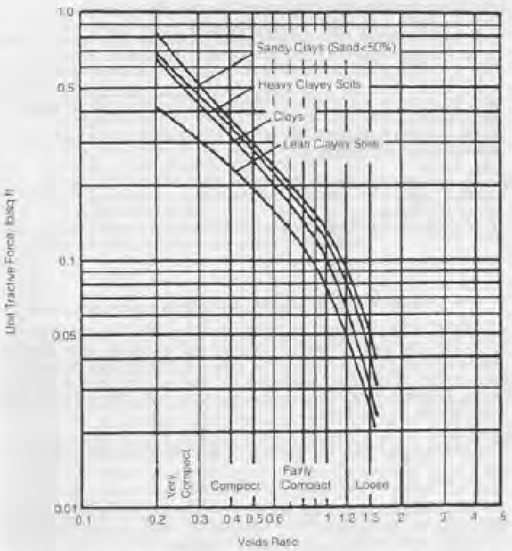


Figure 9.6—Permissible unit tractive forces for canals in cohesive material as converted from data on permissible velocities (lb/sq ft × 47.88 = Pa) (USBR and Hydrotech. Const., 1936)

Figure 15-6. Determination of Critical Channel Shear Stress

FIGURE 15-7
Shed C Channel Reach 1 - Channel Forming Flow Calculations

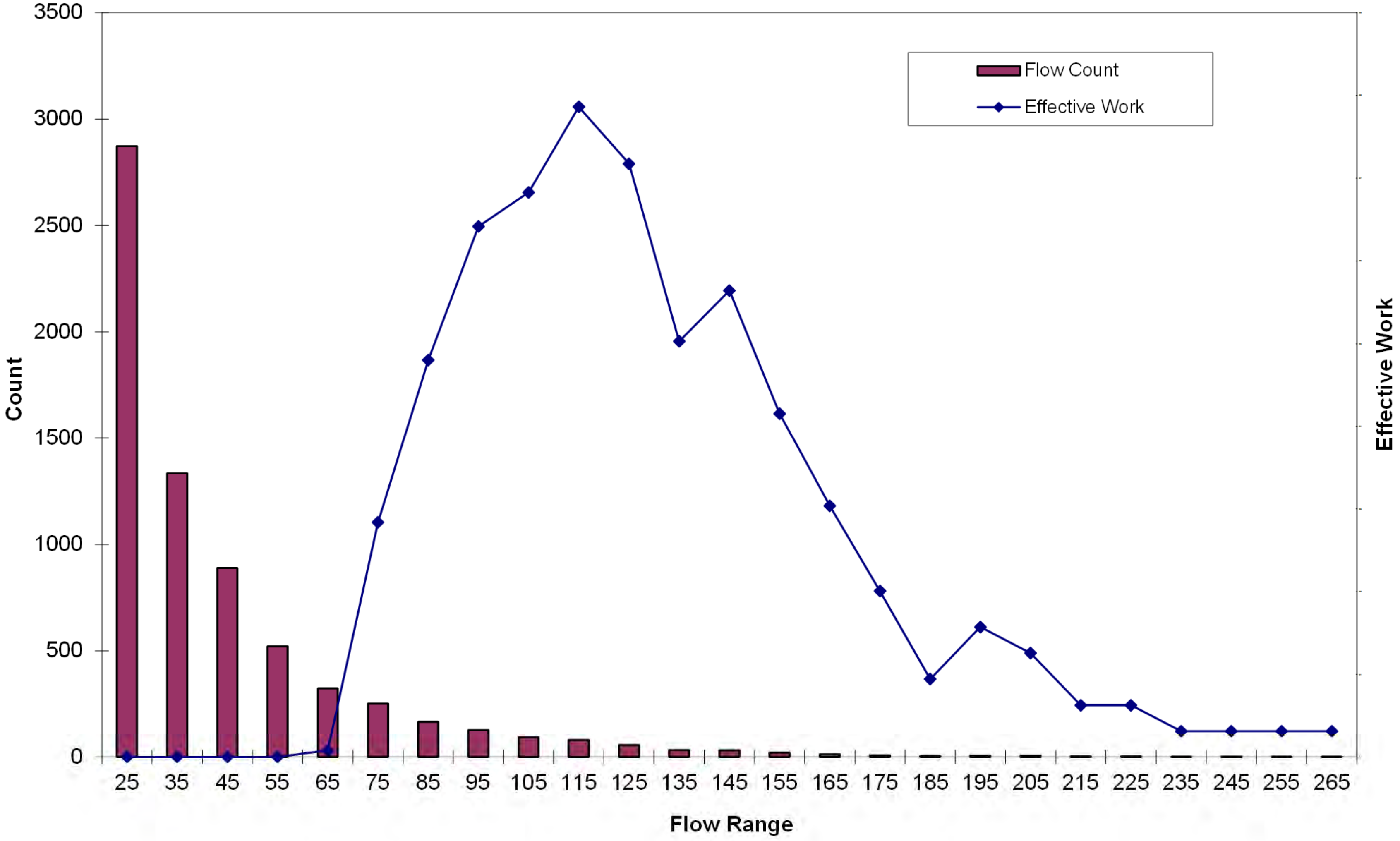


FIGURE 15-8
Shed C Channel Reach 2 - Channel Forming Flow Calculations

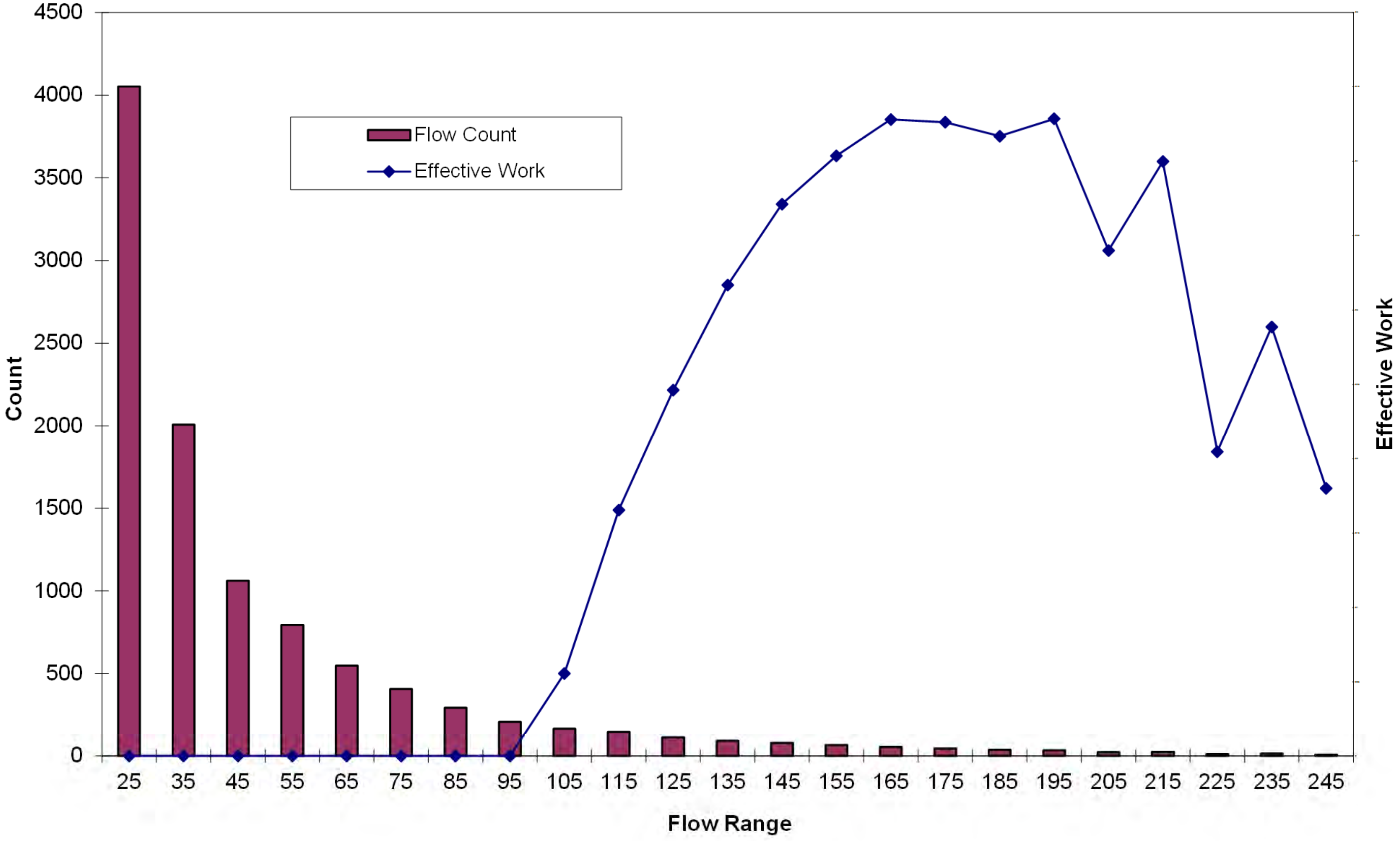


FIGURE 15-9
Shed C Channel Reach 3 - Channel Forming Flow Calculations

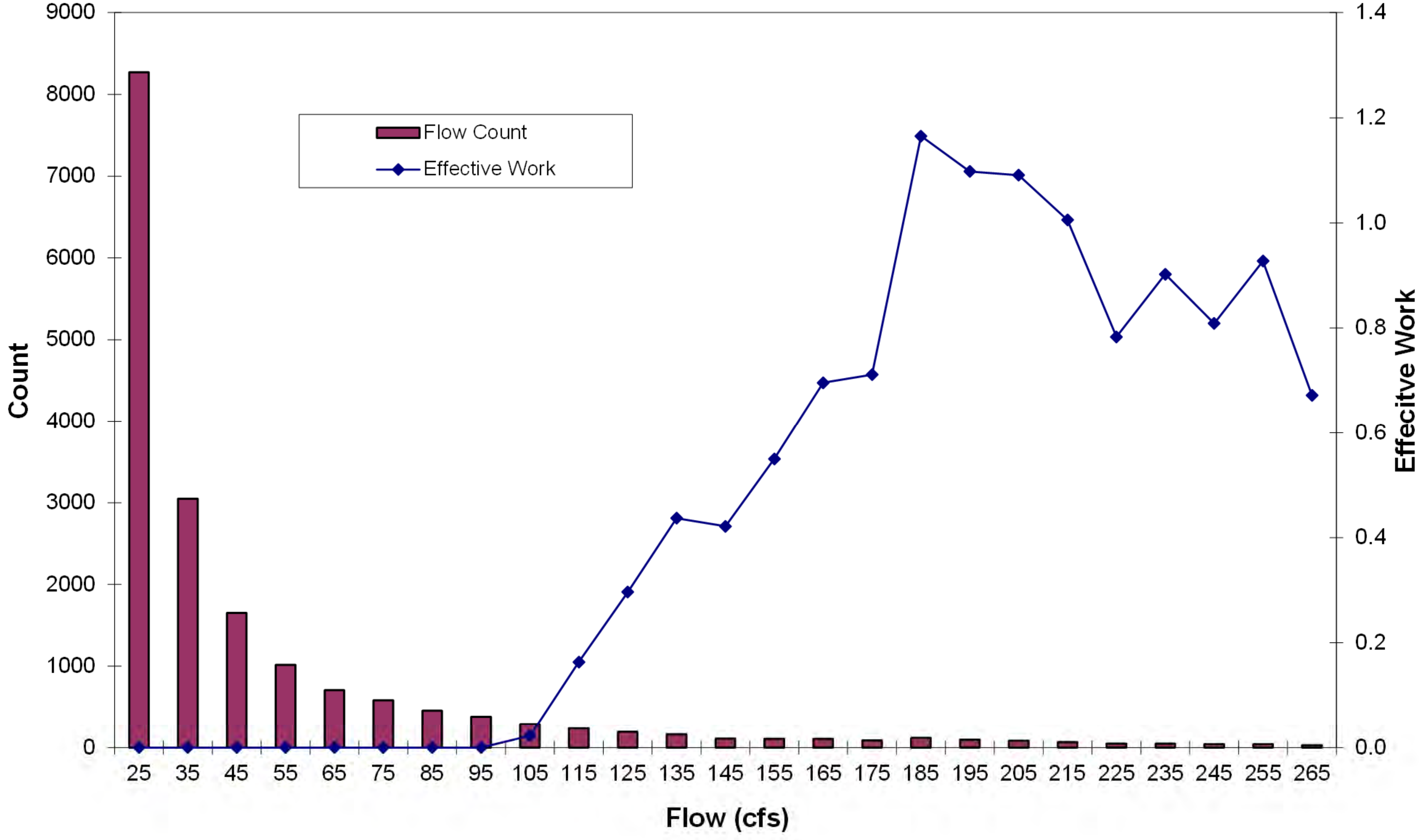
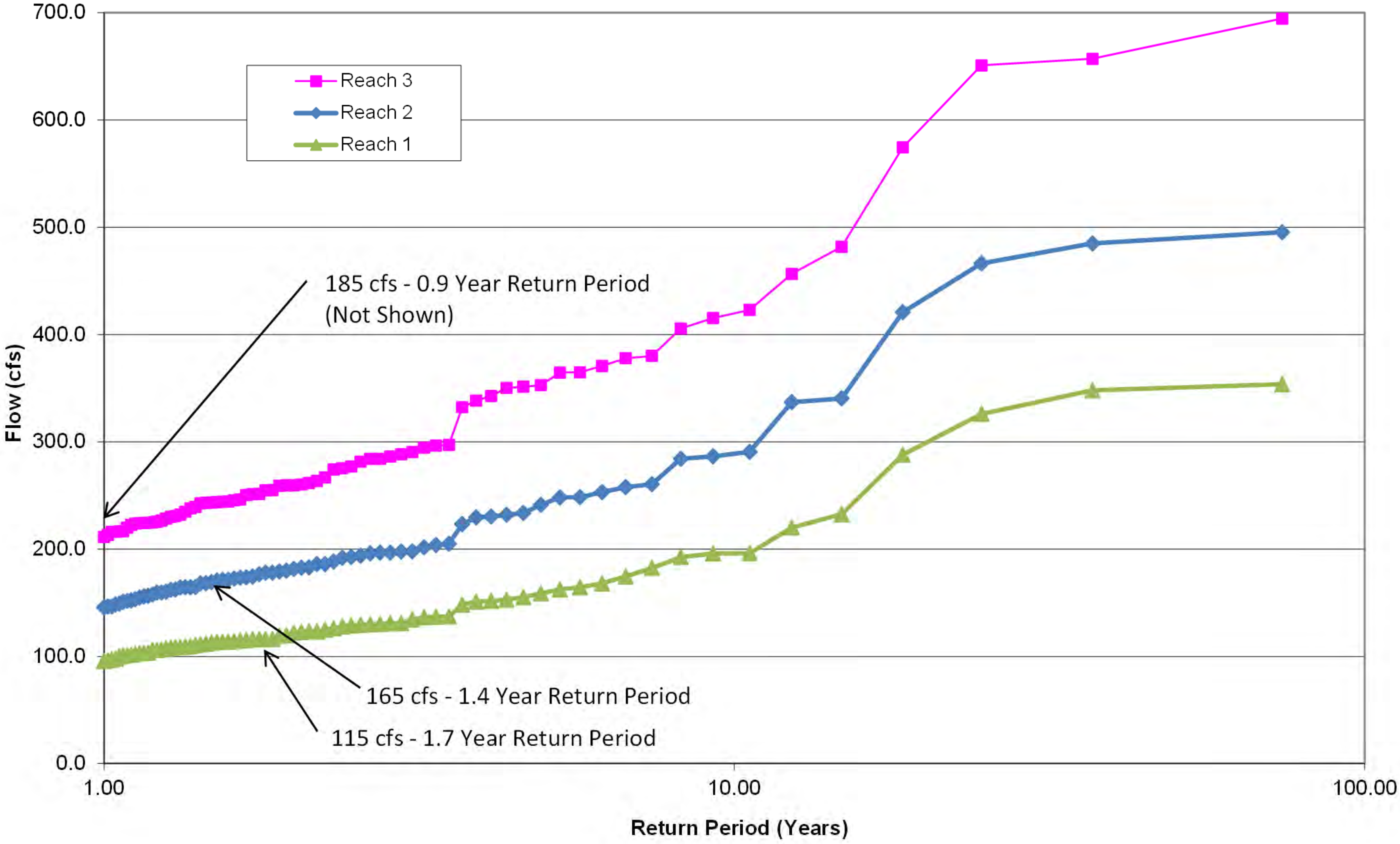


FIGURE 15-10
Flow Frequency for Shed C Channel for Mitigated Buildout Conditions



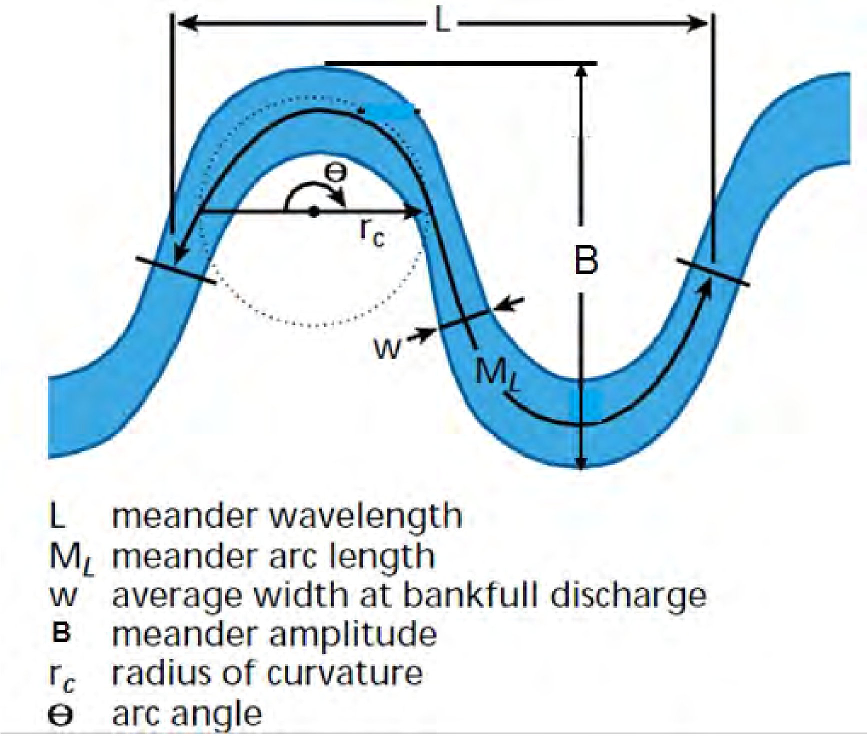


Figure 15-11 Typical Low Flow Channel Meander Dimensions

FIGURE 15-12
Cumulative Effective Work at Bruceville Road

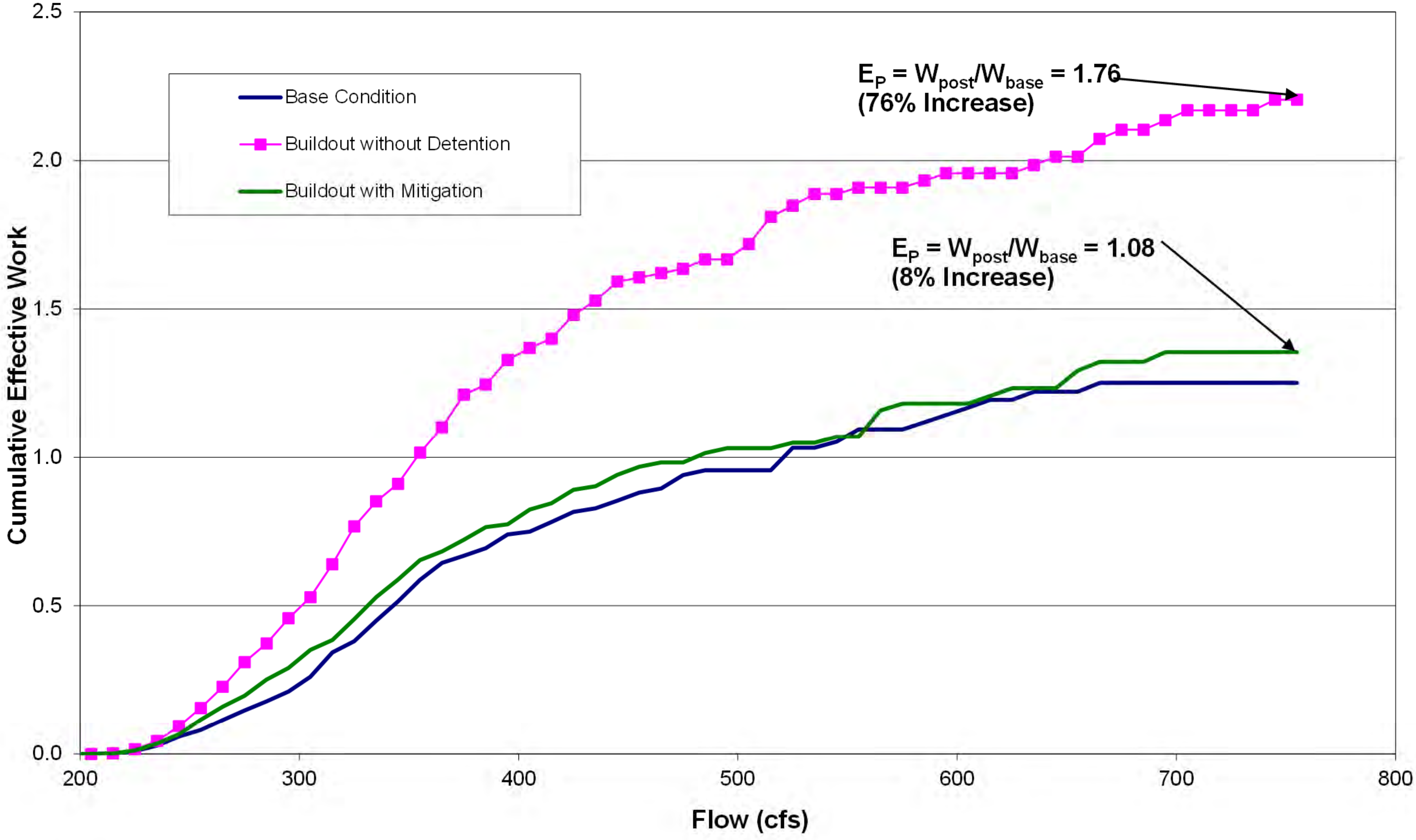
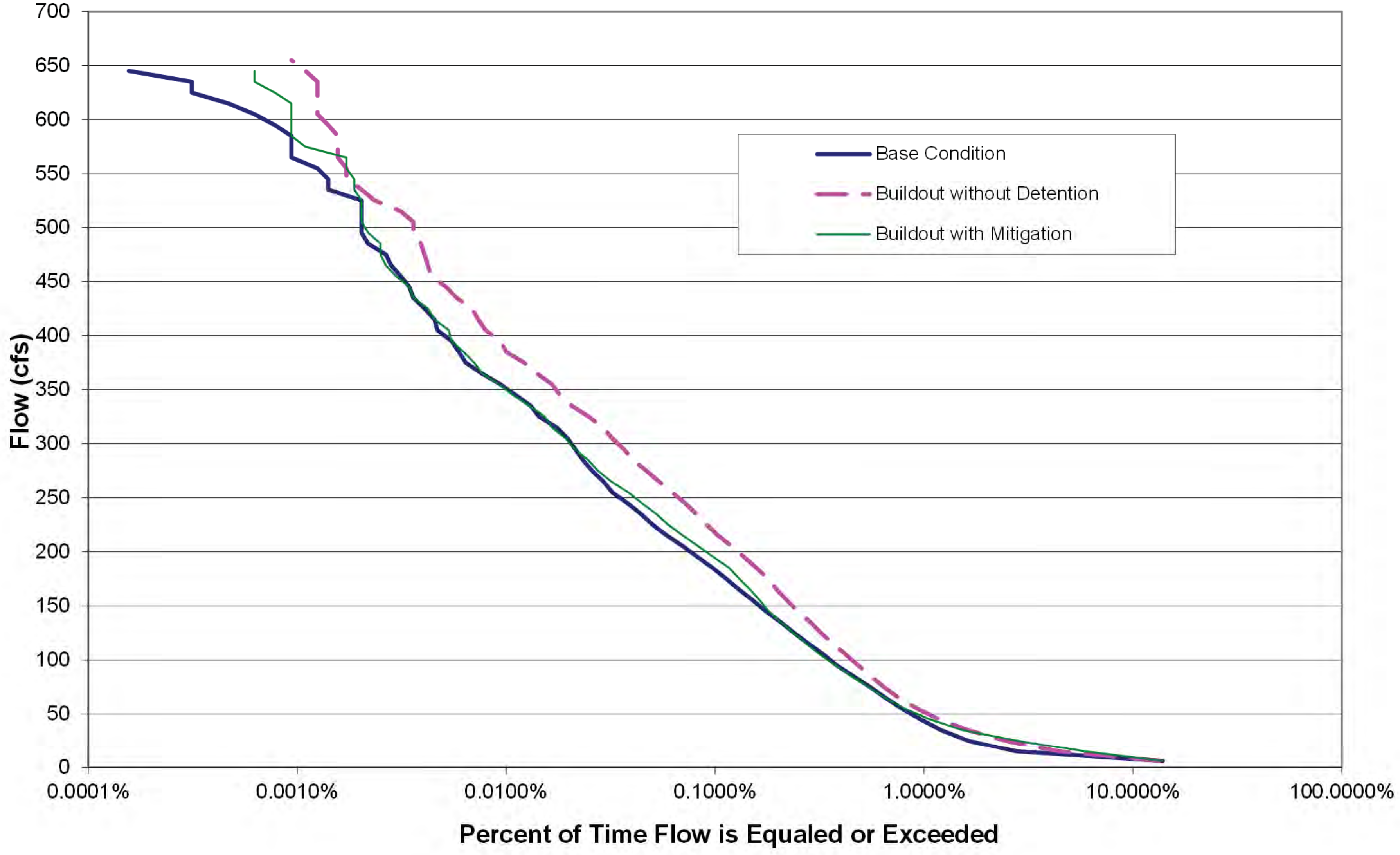


FIGURE 15-13
Flow Duration Comparison at Bruceville Road



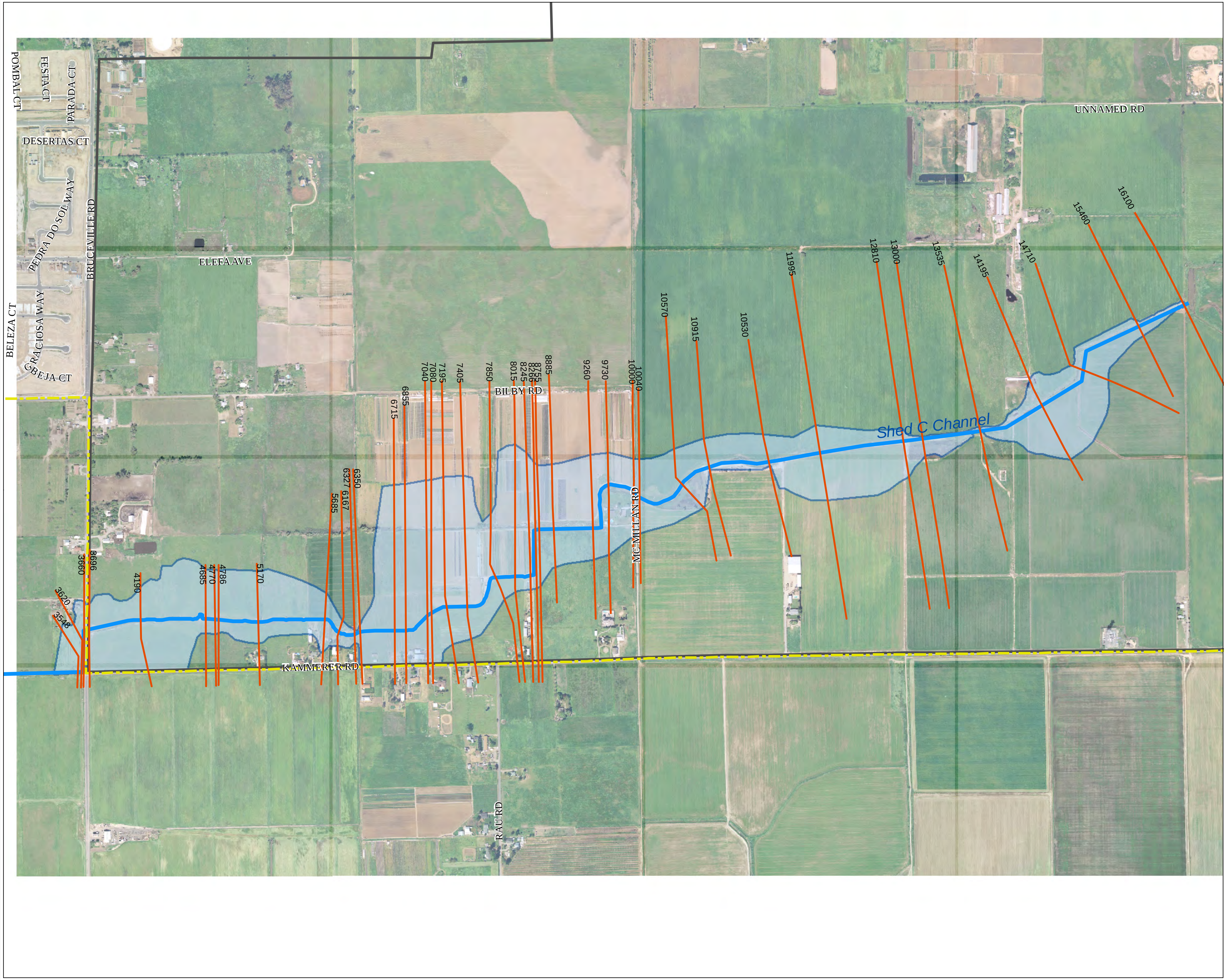


FIGURE 15-14

City of Elk Grove
Storm Drainage Master Plan
Volume II

PRE-DEVELOPMENT
APPROXIMATE 100-YEAR FLOODPLAIN

N

0
600
1,200
Feet

NOTES:

Legend

━	Existing Channel
━	HEC-RAS Cross Section
▭	Approximate 100-Year Floodplain
▭	City Limit

WEST YOST ASSOCIATES
Consulting Engineers

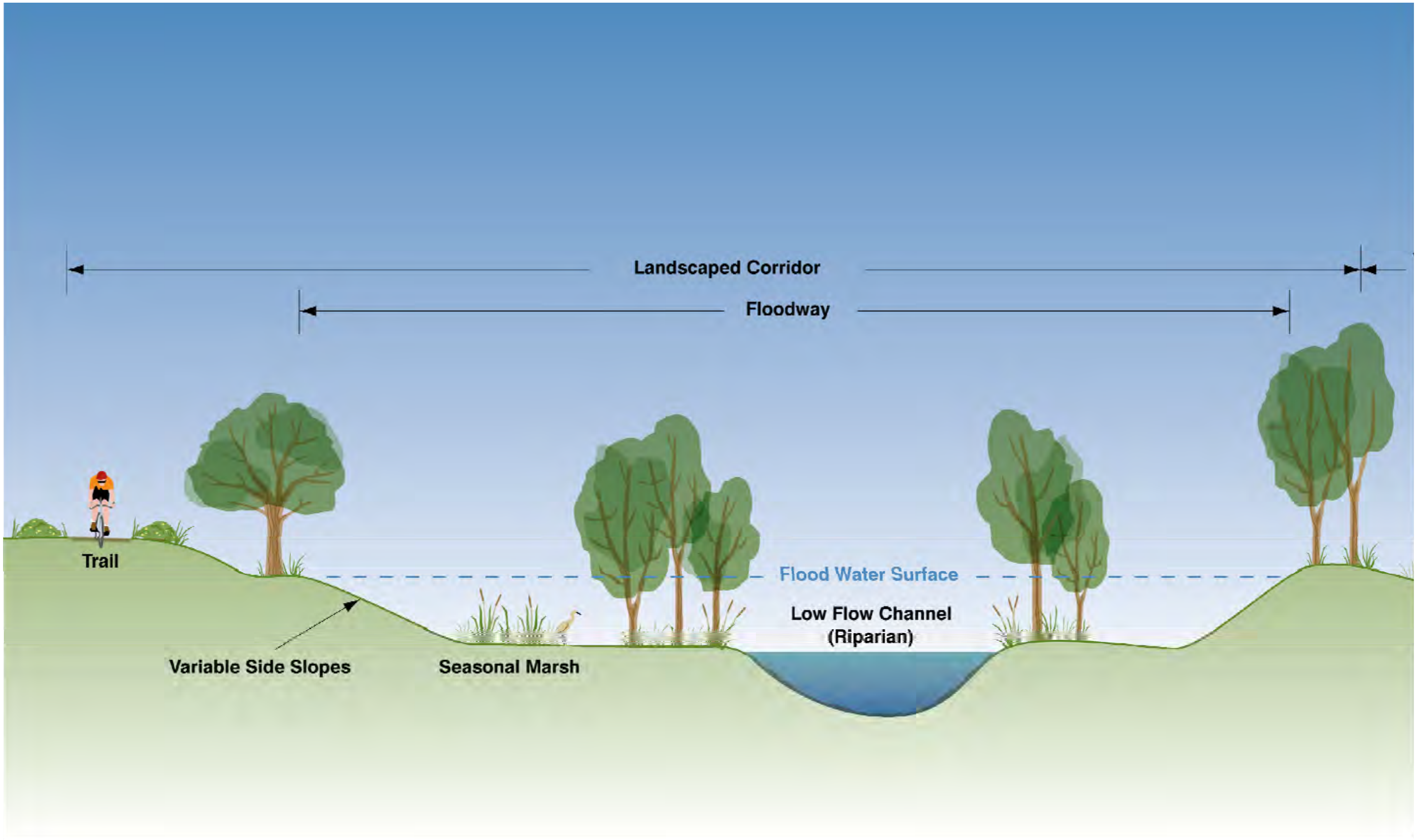


Figure 15-15. Shed C Channel – Proposed Cross Section

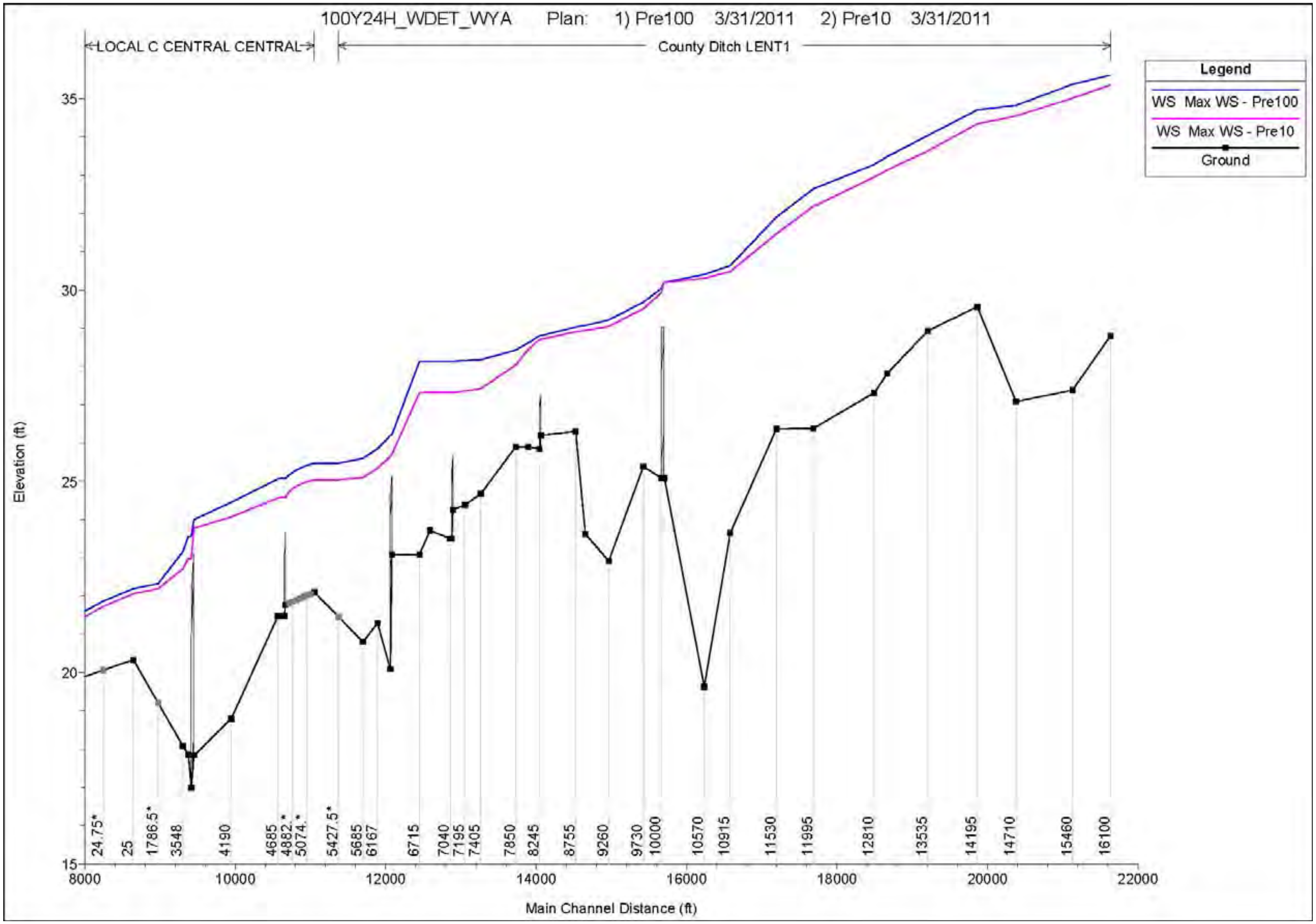


Figure 15-16. Pre-Development Water Surface Profiles

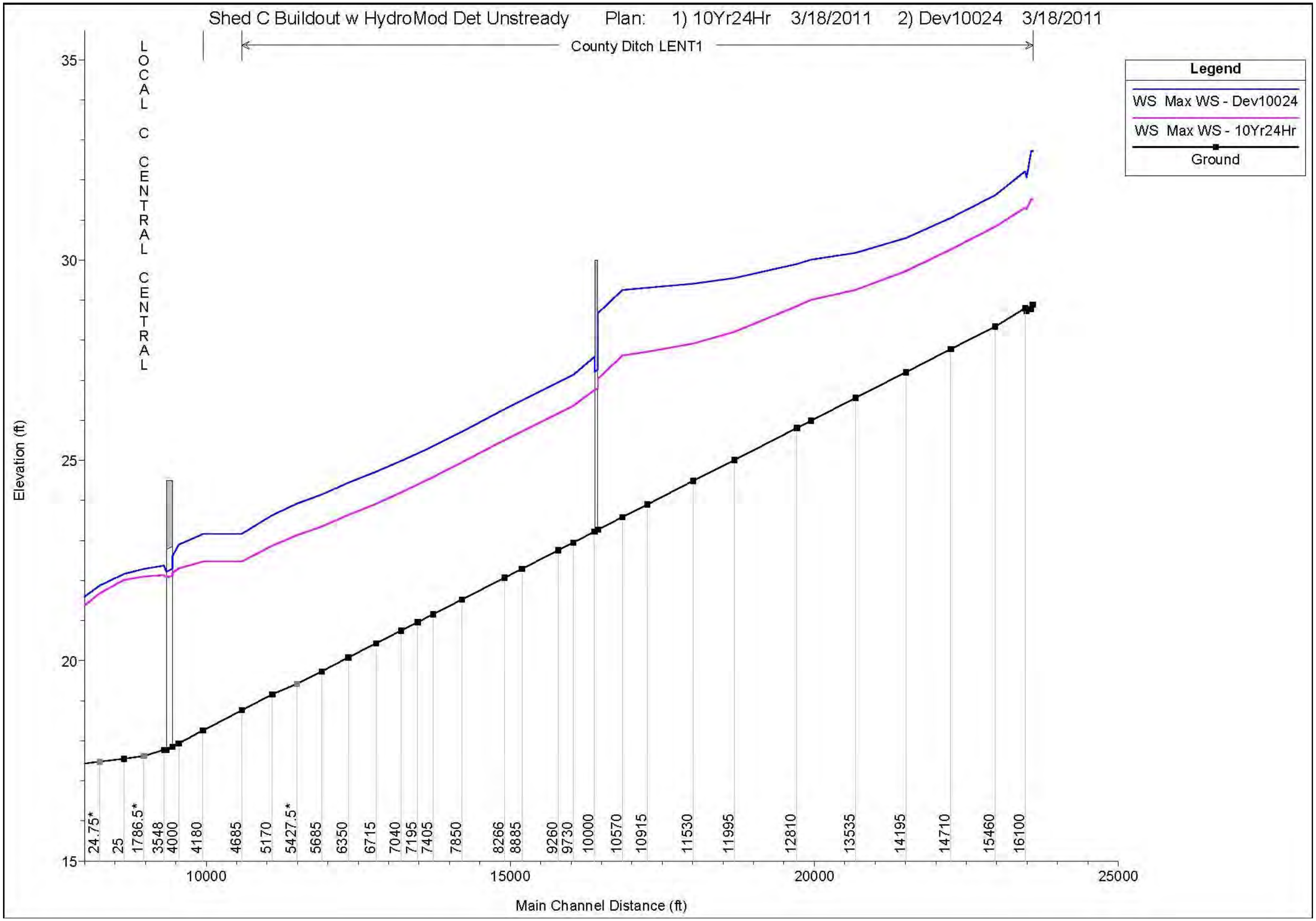


Figure 15-17. Buildout Water Surface Profiles

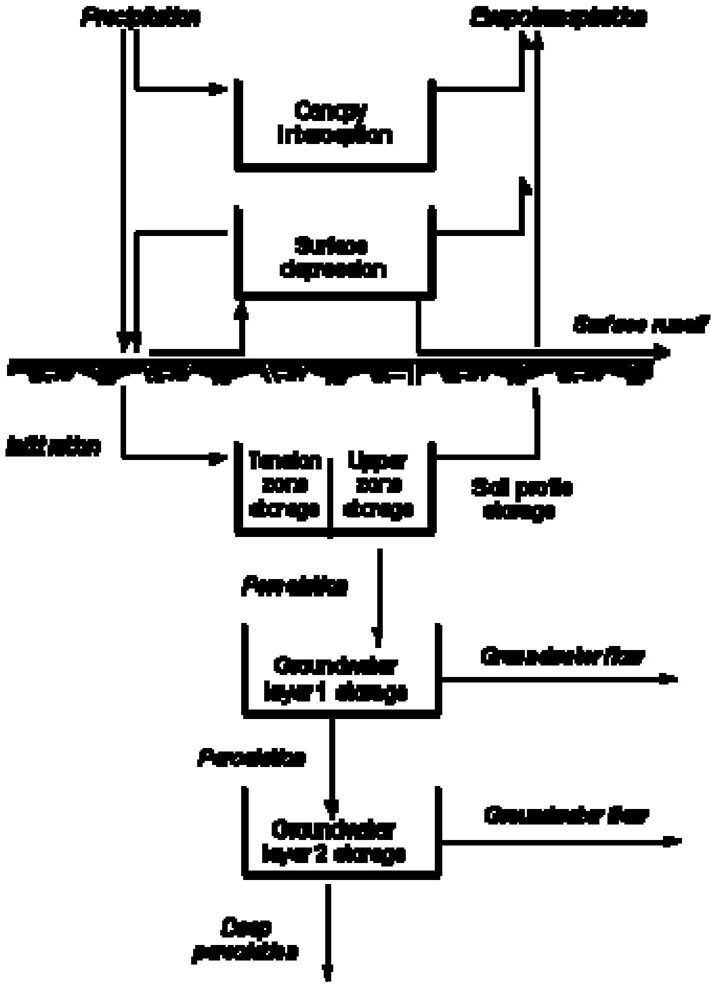


Figure 15-18. Continuous Soil Moisture Accounting Schematic

(Source: HEC-HMS Technical Reference Manual, March 2000)

ATTACHMENT 15A

HEC-RAS Output – Pre-Development Conditions

Attachment 15A

HEC-RAS Profile: Max WS

River	Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
					(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
LOCAL C CENTRAL	CENTRAL	5170	Max WS	Pre100	926.29	22.10	25.47		25.48	0.000530	1.75	1366.08	1505.49	0.18
LOCAL C CENTRAL	CENTRAL	5170	Max WS	Pre10	459.97	22.10	25.04		25.05	0.000526	1.57	791.85	1157.10	0.17
LOCAL C CENTRAL	CENTRAL	5122.*	Max WS	Pre100	906.91	22.06	25.44		25.46	0.000581	1.82	1278.46	1415.83	0.19
LOCAL C CENTRAL	CENTRAL	5122.*	Max WS	Pre10	459.68	22.06	25.01		25.03	0.000593	1.65	742.37	1083.36	0.18
LOCAL C CENTRAL	CENTRAL	5074.*	Max WS	Pre100	899.46	22.02	25.41		25.43	0.000653	1.91	1195.45	1330.69	0.20
LOCAL C CENTRAL	CENTRAL	5074.*	Max WS	Pre10	459.42	22.02	24.99		25.00	0.000651	1.72	694.69	986.59	0.19
LOCAL C CENTRAL	CENTRAL	5026.*	Max WS	Pre100	895.64	21.98	25.38		25.39	0.000757	2.04	1104.53	1237.47	0.21
LOCAL C CENTRAL	CENTRAL	5026.*	Max WS	Pre10	459.16	21.98	24.95		24.97	0.000742	1.82	642.56	902.77	0.20
LOCAL C CENTRAL	CENTRAL	4978.*	Max WS	Pre100	891.70	21.94	25.34		25.36	0.000869	2.18	1024.81	1154.42	0.23
LOCAL C CENTRAL	CENTRAL	4978.*	Max WS	Pre10	458.87	21.94	24.91		24.93	0.000845	1.93	597.80	837.44	0.22
LOCAL C CENTRAL	CENTRAL	4930.*	Max WS	Pre100	884.96	21.89	25.29		25.31	0.001016	2.34	940.18	1067.13	0.24
LOCAL C CENTRAL	CENTRAL	4930.*	Max WS	Pre10	458.53	21.89	24.87		24.89	0.000999	2.08	548.65	777.67	0.24
LOCAL C CENTRAL	CENTRAL	4882.*	Max WS	Pre100	875.65	21.85	25.23		25.26	0.001281	2.60	831.69	963.32	0.27
LOCAL C CENTRAL	CENTRAL	4882.*	Max WS	Pre10	458.10	21.85	24.81		24.84	0.001327	2.38	479.91	708.03	0.27
LOCAL C CENTRAL	CENTRAL	4834.*	Max WS	Pre100	859.41	21.81	25.16		25.19	0.001541	2.82	749.03	888.29	0.30
LOCAL C CENTRAL	CENTRAL	4834.*	Max WS	Pre10	457.44	21.81	24.72		24.77	0.001844	2.76	415.90	657.85	0.32
LOCAL C CENTRAL	CENTRAL	4786	Max WS	Pre100	815.00	21.77	25.06		25.11	0.001988	3.16	634.54	787.39	0.34
LOCAL C CENTRAL	CENTRAL	4786	Max WS	Pre10	456.05	21.77	24.57		24.66	0.003564	3.71	307.19	556.07	0.44
LOCAL C CENTRAL	CENTRAL	4785			Culvert									
LOCAL C CENTRAL	CENTRAL	4770	Max WS	Pre100	821.88	21.47	25.09		25.10	0.000321	1.41	1128.09	835.71	0.14
LOCAL C CENTRAL	CENTRAL	4770	Max WS	Pre10	456.35	21.47	24.59		24.60	0.000265	1.14	756.90	666.99	0.12
LOCAL C CENTRAL	CENTRAL	4685	Max WS	Pre100	815.07	21.47	25.06		25.07	0.000273	0.80	1152.31	1178.06	0.12
LOCAL C CENTRAL	CENTRAL	4685	Max WS	Pre10	456.11	21.47	24.56		24.57	0.000326	0.70	681.20	725.47	0.12
LOCAL C CENTRAL	CENTRAL	4190	Max WS	Pre100	786.51	18.79	24.44		24.53	0.001620	3.49	509.26	609.73	0.32
LOCAL C CENTRAL	CENTRAL	4190	Max WS	Pre10	455.96	18.79	24.06		24.13	0.001201	2.80	328.14	346.22	0.27
LOCAL C CENTRAL	CENTRAL	4000	Max WS	Pre100	786.35	17.84	24.00		24.04	0.000265	1.72	691.63	694.24	0.14
LOCAL C CENTRAL	CENTRAL	4000	Max WS	Pre10	455.95	17.84	23.77		23.79	0.000117	1.11	548.37	566.02	0.09
LOCAL C CENTRAL	CENTRAL	3696	Max WS	Pre100	806.16	17.84	24.00		24.04	0.000280	1.77	688.61	691.78	0.14
LOCAL C CENTRAL	CENTRAL	3696	Max WS	Pre10	470.83	17.84	23.77		23.79	0.000125	1.15	547.01	564.66	0.09
LOCAL C CENTRAL	CENTRAL	3695			Culvert									
LOCAL C CENTRAL	CENTRAL	3660	Max WS	Pre100	809.82	16.98	23.55		23.60	0.000442	2.07	699.07	661.96	0.17
LOCAL C CENTRAL	CENTRAL	3660	Max WS	Pre10	470.66	16.98	22.97		23.01	0.000359	1.70	382.87	389.75	0.15
LOCAL C CENTRAL	CENTRAL	3620	Max WS	Pre100	809.57	17.85	23.56		23.57	0.000228	1.40	1231.60	876.72	0.12
LOCAL C CENTRAL	CENTRAL	3620	Max WS	Pre10	470.67	17.85	22.98		22.99	0.000224	1.25	779.86	681.11	0.12
LOCAL C CENTRAL	CENTRAL	3548	Max WS	Pre100	808.51	18.08	23.16	22.09	23.79	0.005839	6.71	193.29	356.66	0.59
LOCAL C CENTRAL	CENTRAL	3548	Max WS	Pre10	470.63	18.08	22.71		23.08	0.003567	4.85	100.13	63.05	0.45
LOCAL C CENTRAL	CENTRAL	1786.5*	Max WS	Pre100	776.34	19.20	22.32		22.32	0.000031	0.31	2524.99	1566.15	0.04
LOCAL C CENTRAL	CENTRAL	1786.5*	Max WS	Pre10	469.63	19.20	22.19		22.19	0.000015	0.20	2322.14	1510.87	0.03
LOCAL C CENTRAL	CENTRAL	25	Max WS	Pre100	756.35	20.33	22.19		22.20	0.000755	0.53	1300.02	3068.17	0.15
LOCAL C CENTRAL	CENTRAL	25	Max WS	Pre10	468.74	20.33	22.06		22.06	0.000808	0.42	901.70	2890.64	0.15
LOCAL C CENTRAL	CENTRAL	24.75*	Max WS	Pre100	743.97	20.06	21.86		21.87	0.000922	0.64	1136.91	2626.70	0.17
LOCAL C CENTRAL	CENTRAL	24.75*	Max WS	Pre10	466.70	20.06	21.72		21.73	0.001084	0.56	779.47	2422.76	0.18
LOCAL C CENTRAL	CENTRAL	24.5*	Max WS	Pre100	735.86	19.78	21.43		21.45	0.001472	0.81	888.79	2120.21	0.22
LOCAL C CENTRAL	CENTRAL	24.5*	Max WS	Pre10	466.55	19.78	21.29		21.30	0.001874	0.73	597.71	1887.00	0.23
LOCAL C CENTRAL	CENTRAL	24.25*	Max WS	Pre100	731.27	19.51	20.82		20.84	0.002758	1.05	723.72	1974.21	0.29
LOCAL C CENTRAL	CENTRAL	24.25*	Max WS	Pre10	465.96	19.51	20.65		20.67	0.002524	1.10	458.17	1215.82	0.29
LOCAL C CENTRAL	CENTRAL	24	Max WS	Pre100	725.99	19.24	20.39		20.39	0.000555	0.60	1325.68	2336.82	0.14
LOCAL C CENTRAL	CENTRAL	24	Max WS	Pre10	465.52	19.24	20.19		20.19	0.000514	0.58	918.16	1850.04	0.14
LOCAL C CENTRAL	CENTRAL	23.5*	Max WS	Pre100	763.05	18.15	19.90		19.92	0.001628	0.99	773.00	1444.18	0.24
LOCAL C CENTRAL	CENTRAL	23.5*	Max WS	Pre10	493.39	18.15	19.70		19.72	0.001912	0.95	520.01	1162.36	0.25
LOCAL C CENTRAL	CENTRAL	23	Max WS	Pre100	755.82	17.05	19.34		19.35	0.000798	0.78	1058.81	1839.03	0.17
LOCAL C CENTRAL	CENTRAL	23	Max WS	Pre10	491.47	17.05	19.15		19.15	0.000932	0.71	721.50	1597.10	0.18
LOCAL C CENTRAL	CENTRAL	22.75*	Max WS	Pre100	752.85	16.91	19.02		19.03	0.000816	0.85	906.38	1296.78	0.18
LOCAL C CENTRAL	CENTRAL	22.75*	Max WS	Pre10	489.11	16.91	18.81		18.82	0.000878	0.77	647.38	1127.83	0.18
LOCAL C CENTRAL	CENTRAL	22.5*	Max WS	Pre100	749.77	16.78	18.72		18.73	0.000776	0.81	929.17	1373.68	0.17
LOCAL C CENTRAL	CENTRAL	22.5*	Max WS	Pre10	487.14	16.78	18.49		18.50	0.000707	0.74	663.33	1047.80	0.16
LOCAL C CENTRAL	CENTRAL	22.25*	Max WS	Pre100	747.65	16.65	18.45		18.46	0.000610	0.78	970.94	1373.16	0.16
LOCAL C CENTRAL	CENTRAL	22.25*	Max WS	Pre10	483.87	16.65	18.23		18.24	0.000609	0.69	702.66	1113.04	0.15
LOCAL C CENTRAL	CENTRAL	22	Max WS	Pre100	745.55	16.51	18.27		18.27	0.000305	0.61	1446.34	1993.58	0.11
LOCAL C CENTRAL	CENTRAL	22	Max WS	Pre10	481.34	16.51	18.04		18.04	0.000354	0.55	1009.80	1779.05	0.12
LOCAL C CENTRAL	CENTRAL	21.8571*	Max WS	Pre100	826.59	16.20	18.12		18.12	0.000396	0.71	1348.59	1843.50	0.13
LOCAL C CENTRAL	CENTRAL	21.8571*	Max WS	Pre10	529.06	16.20	17.87		17.87	0.000465	0.65	915.66	1601.52	0.13

HEC-RAS Profile: Max WS (Continued)

River	Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
					(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
LOCAL C CENTRAL	CENTRAL	21.7142*	Max WS	Pre100	823.18	15.89	17.94		17.94	0.000452	0.76	1225.60	1667.49	0.14
LOCAL C CENTRAL	CENTRAL	21.7142*	Max WS	Pre10	525.84	15.89	17.66		17.67	0.000536	0.71	801.67	1374.77	0.14
LOCAL C CENTRAL	CENTRAL	21.5714*	Max WS	Pre100	816.79	15.58	17.73		17.74	0.000518	0.80	1105.15	1532.52	0.15
LOCAL C CENTRAL	CENTRAL	21.5714*	Max WS	Pre10	523.98	15.58	17.42		17.43	0.000600	0.77	706.41	1044.88	0.15
LOCAL C CENTRAL	CENTRAL	21.4285*	Max WS	Pre100	803.62	15.26	17.51		17.52	0.000527	0.82	1020.07	1310.58	0.15
LOCAL C CENTRAL	CENTRAL	21.4285*	Max WS	Pre10	521.16	15.26	17.14		17.15	0.000682	0.83	632.84	861.27	0.16
LOCAL C CENTRAL	CENTRAL	21.2857*	Max WS	Pre100	745.32	14.95	17.31		17.31	0.000429	0.76	1000.27	1158.47	0.14
LOCAL C CENTRAL	CENTRAL	21.2857*	Max WS	Pre10	515.18	14.95	16.84		16.85	0.000720	0.89	581.56	693.97	0.17
LOCAL C CENTRAL	CENTRAL	21.1428*	Max WS	Pre100	665.82	14.64	17.16		17.17	0.000268	0.63	1069.70	1126.52	0.11
LOCAL C CENTRAL	CENTRAL	21.1428*	Max WS	Pre10	447.42	14.64	16.57		16.58	0.000471	0.79	568.13	588.22	0.14
LOCAL C CENTRAL	CENTRAL	21	Max WS	Pre100	605.62	14.33	17.07		17.08	0.000154	0.50	1236.99	1265.33	0.08
LOCAL C CENTRAL	CENTRAL	21	Max WS	Pre10	397.34	14.33	16.41		16.41	0.000266	0.63	628.45	589.37	0.11
LOCAL C CENTRAL	CENTRAL	20.6666*	Max WS	Pre100	594.60	13.21	17.03		17.03	0.000044	0.28	2121.01	1797.89	0.04
LOCAL C CENTRAL	CENTRAL	20.6666*	Max WS	Pre10	382.58	13.21	16.33		16.33	0.000093	0.36	1063.93	1059.39	0.06
LOCAL C CENTRAL	CENTRAL	20.3333*	Max WS	Pre100	590.36	12.09	17.02		17.02	0.000013	0.17	3441.24	2497.27	0.03
LOCAL C CENTRAL	CENTRAL	20.3333*	Max WS	Pre10	377.47	12.09	16.30		16.30	0.000033	0.20	1856.90	1975.94	0.04
LOCAL C CENTRAL	CENTRAL	20	Max WS	Pre100	589.68	10.97	17.02		17.02	0.000005	0.11	5270.46	3392.72	0.02
LOCAL C CENTRAL	CENTRAL	20	Max WS	Pre10	376.57	10.97	16.29		16.29	0.000008	0.12	3138.51	2571.67	0.02
County Ditch	LENT1	16100	Max WS	Pre100	338.98	28.80	35.61		35.64	0.000150	1.32	257.38	61.37	0.11
County Ditch	LENT1	16100	Max WS	Pre10	203.21	28.80	35.36		35.37	0.000064	0.84	241.85	59.58	0.07
County Ditch	LENT1	15460	Max WS	Pre100	336.33	27.38	35.37		35.42	0.000779	2.15	256.42	248.77	0.20
County Ditch	LENT1	15460	Max WS	Pre10	200.43	27.38	35.01		35.10	0.001082	2.43	82.48	24.12	0.23
County Ditch	LENT1	14710	Max WS	Pre100	299.73	27.08	34.82		34.89	0.000606	2.24	165.37	95.33	0.19
County Ditch	LENT1	14710	Max WS	Pre10	195.79	27.08	34.55		34.59	0.000344	1.64	140.24	87.28	0.14
County Ditch	LENT1	14195	Max WS	Pre100	262.15	29.56	34.71		34.71	0.000023	0.34	1076.96	683.20	0.04
County Ditch	LENT1	14195	Max WS	Pre10	195.25	29.56	34.34		34.38	0.000471	1.51	129.28	48.57	0.16
County Ditch	LENT1	13535	Max WS	Pre100	261.88	28.93	34.04		34.14	0.001931	2.58	101.47	34.21	0.26
County Ditch	LENT1	13535	Max WS	Pre10	194.91	28.93	33.63		33.71	0.001571	2.21	88.02	32.03	0.24
County Ditch	LENT1	13000	Max WS	Pre100	261.71	27.82	33.49		33.52	0.000509	1.57	203.60	113.60	0.14
County Ditch	LENT1	13000	Max WS	Pre10	194.78	27.82	33.14		33.16	0.000469	1.42	167.30	93.08	0.13
County Ditch	LENT1	12810	Max WS	Pre100	261.38	27.31	33.28		33.38	0.001524	2.66	112.84	86.69	0.23
County Ditch	LENT1	12810	Max WS	Pre10	194.68	27.31	32.95		33.03	0.001188	2.25	90.03	52.28	0.20
County Ditch	LENT1	11995	Max WS	Pre100	433.82	26.38	32.64		32.64	0.000230	0.81	754.34	773.33	0.08
County Ditch	LENT1	11995	Max WS	Pre10	326.74	26.38	32.18		32.20	0.000758	1.37	304.22	268.16	0.14
County Ditch	LENT1	11530	Max WS	Pre100	433.65	26.37	31.90		32.04	0.002490	2.95	147.00	37.01	0.26
County Ditch	LENT1	11530	Max WS	Pre10	326.56	26.37	31.47		31.56	0.001918	2.49	131.36	35.21	0.23
County Ditch	LENT1	10915	Max WS	Pre100	430.25	23.64	30.63		30.69	0.001964	2.13	256.48	271.91	0.22
County Ditch	LENT1	10915	Max WS	Pre10	326.51	23.64	30.48		30.53	0.001555	1.88	216.63	252.77	0.20
County Ditch	LENT1	10570	Max WS	Pre100	417.74	19.63	30.40		30.41	0.000009	0.32	1542.51	545.75	0.02
County Ditch	LENT1	10570	Max WS	Pre10	324.28	19.63	30.30		30.30	0.000006	0.26	1487.24	527.44	0.02
County Ditch	LENT1	10040	Max WS	Pre100	326.05	25.08	30.20		30.20	0.000278	0.87	589.05	762.73	0.09
County Ditch	LENT1	10040	Max WS	Pre10	355.61	25.08	30.19		30.20	0.000332	0.95	588.45	762.39	0.09
County Ditch	LENT1	10030			Culvert									
County Ditch	LENT1	10000	Max WS	Pre100	549.33	25.08	30.04		30.07	0.001329	1.83	474.03	681.98	0.19
County Ditch	LENT1	10000	Max WS	Pre10	399.44	25.08	29.93		30.00	0.002792	2.60	240.97	424.86	0.27
County Ditch	LENT1	9730	Max WS	Pre100	547.05	25.38	29.68		29.70	0.001676	1.62	504.60	878.30	0.20
County Ditch	LENT1	9730	Max WS	Pre10	396.62	25.38	29.51		29.53	0.001970	1.66	365.83	758.89	0.21
County Ditch	LENT1	9260	Max WS	Pre100	544.33	22.91	29.21		29.26	0.000922	1.91	424.18	540.97	0.16
County Ditch	LENT1	9260	Max WS	Pre10	391.20	22.91	29.04		29.08	0.000656	1.58	341.31	428.26	0.14
County Ditch	LENT1	8885	Max WS	Pre100	542.30	23.62	29.07		29.08	0.000188	0.74	1030.25	1042.46	0.07
County Ditch	LENT1	8885	Max WS	Pre10	388.74	23.62	28.94		28.94	0.000133	0.61	899.75	965.14	0.06
County Ditch	LENT1	8755	Max WS	Pre100	541.81	26.30	29.03		29.03	0.000466	0.74	909.89	1340.11	0.10
County Ditch	LENT1	8755	Max WS	Pre10	388.29	26.30	28.91		28.91	0.000386	0.64	755.18	1204.81	0.09
County Ditch	LENT1	8266	Max WS	Pre100	540.60	26.20	28.81		28.81	0.000553	0.73	1023.73	2170.41	0.11
County Ditch	LENT1	8266	Max WS	Pre10	388.14	26.20	28.71		28.71	0.000501	0.66	821.96	1937.55	0.10
County Ditch	LENT1	8265			Culvert									
County Ditch	LENT1	8245	Max WS	Pre100	540.60	25.85	28.80		28.81	0.000397	1.38	712.83	560.48	0.15
County Ditch	LENT1	8245	Max WS	Pre10	388.06	25.85	28.71		28.71	0.000257	1.08	659.90	553.54	0.12
County Ditch	LENT1	8015	Max WS	Pre100	461.83	25.90	28.62		28.64	0.001485	1.82	604.57	1673.23	0.27
County Ditch	LENT1	8015	Max WS	Pre10	387.03	25.90	28.44		28.50	0.003123	2.42	346.18	1210.04	0.38
County Ditch	LENT1	7850	Max WS	Pre100	448.67	25.90	28.43		28.43	0.000400	1.21	1117.95	2440.00	0.15
County Ditch	LENT1	7850	Max WS	Pre10	386.43	25.90	28.04		28.08	0.002189	2.46	374.55	848.81	0.33

HEC-RAS Profile: Max WS (Continued)

River	Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
					(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
County Ditch	LENT1	7405	Max WS	Pre100	424.63	24.67	28.17		28.17	0.000034	0.44	2263.86	2100.00	0.05
County Ditch	LENT1	7405	Max WS	Pre10	276.11	24.67	27.42		27.42	0.000303	1.08	769.27	1542.83	0.13
County Ditch	LENT1	7195	Max WS	Pre100	392.91	24.38	28.15		28.15	0.000047	0.55	1430.05	980.00	0.05
County Ditch	LENT1	7195	Max WS	Pre10	269.12	24.38	27.36		27.36	0.000226	1.01	652.75	945.68	0.11
County Ditch	LENT1	7080	Max WS	Pre100	311.54	24.25	28.14		28.14	0.000043	0.54	1062.30	616.00	0.05
County Ditch	LENT1	7080	Max WS	Pre10	265.38	24.25	27.32		27.33	0.000225	1.04	558.96	616.00	0.11
County Ditch	LENT1	7079			Culvert									
County Ditch	LENT1	7040	Max WS	Pre100	311.54	23.50	28.14		28.14	0.000002	0.12	5356.29	2900.00	0.01
County Ditch	LENT1	7040	Max WS	Pre10	265.53	23.50	27.32		27.32	0.000008	0.22	2990.56	2900.00	0.02
County Ditch	LENT1	6855	Max WS	Pre100	316.16	23.71	28.13		28.13	0.000003	0.17	3589.77	1750.00	0.01
County Ditch	LENT1	6855	Max WS	Pre10	265.34	23.71	27.32		27.32	0.000012	0.27	2165.26	1750.00	0.03
County Ditch	LENT1	6715	Max WS	Pre100	312.39	23.08	28.13		28.13	0.000016	0.38	1932.46	1350.00	0.03
County Ditch	LENT1	6715	Max WS	Pre10	265.27	23.08	27.31		27.31	0.000061	0.66	983.58	896.55	0.06
County Ditch	LENT1	6350	Max WS	Pre100	870.80	23.08	26.22		26.49	0.009822	6.63	345.67	587.36	0.73
County Ditch	LENT1	6350	Max WS	Pre10	173.96	23.08	25.71		25.83	0.003354	3.34	115.06	311.05	0.41
County Ditch	LENT1	6349			Culvert									
County Ditch	LENT1	6327	Max WS	Pre100	805.47	20.08	26.19		26.24	0.000363	1.95	657.85	476.50	0.16
County Ditch	LENT1	6327	Max WS	Pre10	263.43	20.08	25.64		25.65	0.000076	0.83	449.35	337.88	0.07
County Ditch	LENT1	6167	Max WS	Pre100	729.06	21.29	25.85		25.93	0.002797	3.79	553.84	1026.89	0.40
County Ditch	LENT1	6167	Max WS	Pre10	261.45	21.29	25.34		25.46	0.002928	3.40	176.95	449.42	0.39
County Ditch	LENT1	5685	Max WS	Pre100	745.88	20.80	25.59		25.61	0.000320	1.52	1111.41	1095.33	0.14
County Ditch	LENT1	5685	Max WS	Pre10	257.20	20.80	25.10		25.10	0.000108	0.80	659.15	718.74	0.08
County Ditch	LENT1	5427.5*	Max WS	Pre100	711.97	21.45	25.47		25.48	0.000481	1.77	1079.13	1277.07	0.17
County Ditch	LENT1	5427.5*	Max WS	Pre10	269.96	21.45	25.04		25.05	0.000231	1.12	626.42	877.28	0.12

ATTACHMENT 15B

HEC-RAS Output – Buildout Conditions

Attachment 15B

HEC-RAS Profile: Max WS

River	Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
					(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
FLOOD PLAIN	NORTH 1	19.25	Max WS	10Yr24Hr	913.49	12.40	16.23		16.23	0.000042	0.28	3754.92	3284.33	0.04
FLOOD PLAIN	NORTH 1	19.25	Max WS	Dev10024	1513.09	12.40	17.04		17.04	0.000026	0.22	6934.83	4995.11	0.03
FLOOD PLAIN	NORTH 1	19	Max WS	10Yr24Hr	911.49	11.89	16.22		16.22	0.000018	0.22	4826.74	3541.40	0.03
FLOOD PLAIN	NORTH 1	19	Max WS	Dev10024	1510.35	11.89	17.03		17.03	0.000016	0.19	8453.13	5564.70	0.03
FLOOD PLAIN	NORTH 1	18.8333*	Max WS	10Yr24Hr	910.37	11.83	16.19		16.19	0.000097	0.43	2504.36	2534.89	0.07
FLOOD PLAIN	NORTH 1	18.8333*	Max WS	Dev10024	1508.57	11.83	17.01		17.01	0.000049	0.31	5311.42	4077.74	0.05
FLOOD PLAIN	NORTH 1	18.6666*	Max WS	10Yr24Hr	909.72	11.76	15.91		15.95	0.001203	1.73	638.45	820.09	0.24
FLOOD PLAIN	NORTH 1	18.6666*	Max WS	Dev10024	1506.34	11.76	16.90		16.90	0.000481	0.70	2264.65	2744.51	0.14
FLOOD PLAIN	NORTH 1	18.5	Max WS	10Yr24Hr	902.36	11.70	14.82		15.05	0.003003	3.87	233.25	88.13	0.42
FLOOD PLAIN	NORTH 1	18.5	Max WS	Dev10024	1384.12	11.70	15.40		15.77	0.003799	4.84	286.11	92.05	0.48
FLOOD PLAIN	NORTH 1	18	Max WS	10Yr24Hr	904.18	11.70	14.77	13.54	15.01	0.003169	3.94	229.50	87.86	0.43
FLOOD PLAIN	NORTH 1	18	Max WS	Dev10024	1360.46	11.70	15.34	14.05	15.70	0.003903	4.85	280.26	91.56	0.49
FLOOD PLAIN	NORTH 1	17.5			Bridge									
FLOOD PLAIN	NORTH 1	17	Max WS	10Yr24Hr	907.14	11.15	14.85		14.85	0.000324	0.69	1551.75	2017.23	0.12
FLOOD PLAIN	NORTH 1	17	Max WS	Dev10024	1402.99	11.15	15.48		15.49	0.000132	0.54	3041.54	2595.48	0.08
FLOOD PLAIN	NORTH 1	16.5*	Max WS	10Yr24Hr	901.95	11.04	14.69		14.70	0.000771	0.83	1087.94	1509.07	0.17
FLOOD PLAIN	NORTH 1	16.5*	Max WS	Dev10024	1393.15	11.04	15.44		15.44	0.000185	0.47	2985.21	3360.25	0.09
FLOOD PLAIN	NORTH 1	16	Max WS	10Yr24Hr	896.69	10.93	14.53		14.54	0.000464	0.57	1560.58	2562.10	0.13
FLOOD PLAIN	NORTH 1	16	Max WS	Dev10024	1388.23	10.93	15.40		15.40	0.000062	0.30	4581.89	4354.47	0.05
FLOOD PLAIN	NORTH 1	15.5*	Max WS	10Yr24Hr	898.84	10.78	14.36		14.37	0.000291	0.66	1351.93	1255.70	0.11
FLOOD PLAIN	NORTH 1	15.5*	Max WS	Dev10024	1390.32	10.78	15.37		15.37	0.000084	0.34	4122.86	4162.67	0.06
FLOOD PLAIN	NORTH 1	15	Max WS	10Yr24Hr	898.62	10.64	14.26		14.27	0.000145	0.52	1808.77	1824.33	0.08
FLOOD PLAIN	NORTH 1	15	Max WS	Dev10024	1390.08	10.64	15.34		15.34	0.000040	0.37	4378.67	3272.97	0.05
FLOOD PLAIN	NORTH 1	14.5			Culvert									
FLOOD PLAIN	NORTH 1	14	Max WS	10Yr24Hr	898.08	11.27	13.09		13.10	0.000317	0.65	1389.85	1440.14	0.12
FLOOD PLAIN	NORTH 1	14	Max WS	Dev10024	1389.49	11.27	13.37		13.38	0.000326	0.77	1802.13	1461.34	0.12
FLOOD PLAIN	NORTH 1	13.6666*	Max WS	10Yr24Hr	897.03	9.60	12.70		12.73	0.002041	1.30	690.07	1011.55	0.28
FLOOD PLAIN	NORTH 1	13.6666*	Max WS	Dev10024	1387.09	9.60	13.10		13.12	0.001195	1.16	1195.94	1392.09	0.22
FLOOD PLAIN	NORTH 1	13.3333*	Max WS	10Yr24Hr	894.84	7.92	12.28		12.30	0.000658	1.10	810.64	648.70	0.17
FLOOD PLAIN	NORTH 1	13.3333*	Max WS	Dev10024	1383.93	7.92	12.78		12.81	0.000606	1.18	1174.35	800.56	0.17
FLOOD PLAIN	NORTH 1	13	Max WS	10Yr24Hr	893.73	6.25	12.15		12.16	0.000157	0.70	1282.77	697.27	0.09
FLOOD PLAIN	NORTH 1	13	Max WS	Dev10024	1382.10	6.25	12.66		12.67	0.000193	0.83	1662.38	808.13	0.10
FLOOD PLAIN	NORTH 1	12.8333*	Max WS	10Yr24Hr	892.98	6.54	12.08		12.08	0.000200	0.72	1238.96	768.08	0.10
FLOOD PLAIN	NORTH 1	12.8333*	Max WS	Dev10024	1380.54	6.54	12.57		12.58	0.000230	0.84	1649.88	908.75	0.11
FLOOD PLAIN	NORTH 1	12.6666*	Max WS	10Yr24Hr	892.19	6.83	11.98		11.98	0.000251	0.75	1182.61	814.16	0.11
FLOOD PLAIN	NORTH 1	12.6666*	Max WS	Dev10024	1378.46	6.83	12.45		12.46	0.000312	0.85	1621.09	1098.18	0.12
FLOOD PLAIN	NORTH 1	12.5*	Max WS	10Yr24Hr	891.17	7.12	11.83		11.84	0.000416	0.81	1106.75	1008.65	0.14
FLOOD PLAIN	NORTH 1	12.5*	Max WS	Dev10024	1375.59	7.12	12.30		12.31	0.000391	0.84	1647.14	1357.61	0.13
FLOOD PLAIN	NORTH 1	12.3333*	Max WS	10Yr24Hr	889.97	7.42	11.64		11.65	0.000524	0.84	1064.78	1092.16	0.15
FLOOD PLAIN	NORTH 1	12.3333*	Max WS	Dev10024	1372.58	7.42	12.14		12.15	0.000394	0.82	1679.08	1437.63	0.13
FLOOD PLAIN	NORTH 1	12.1666*	Max WS	10Yr24Hr	889.07	7.71	11.45		11.46	0.000330	0.80	1117.07	871.23	0.12
FLOOD PLAIN	NORTH 1	12.1666*	Max WS	Dev10024	1370.58	7.71	11.98		11.99	0.000326	0.79	1741.01	1370.02	0.12
FLOOD PLAIN	NORTH 1	12	Max WS	10Yr24Hr	888.63	8.00	11.33		11.34	0.000204	0.67	1330.32	942.52	0.10
FLOOD PLAIN	NORTH 1	12	Max WS	Dev10024	1369.62	8.00	11.86		11.87	0.000211	0.72	1904.81	1238.54	0.10
FLOOD PLAIN	NORTH 1	11.6666*	Max WS	10Yr24Hr	892.37	7.83	11.17		11.18	0.000498	0.90	994.80	883.72	0.15
FLOOD PLAIN	NORTH 1	11.6666*	Max WS	Dev10024	1373.13	7.83	11.72		11.74	0.000362	0.90	1531.18	1072.19	0.13
FLOOD PLAIN	NORTH 1	11.3333*	Max WS	10Yr24Hr	892.28	7.67	10.90		10.92	0.000691	1.19	750.42	558.30	0.18
FLOOD PLAIN	NORTH 1	11.3333*	Max WS	Dev10024	1372.82	7.67	11.50		11.52	0.000584	1.21	1136.32	727.88	0.17
FLOOD PLAIN	NORTH 1	11	Max WS	10Yr24Hr	892.24	7.50	10.51		10.56	0.000930	1.74	512.04	267.85	0.22
FLOOD PLAIN	NORTH 1	11	Max WS	Dev10024	1372.53	7.50	11.07		11.14	0.001048	2.04	672.53	303.62	0.24
FLOOD PLAIN	NORTH 1	10.6666*	Max WS	10Yr24Hr	893.31	7.25	10.22		10.25	0.000733	1.40	637.40	387.24	0.19
FLOOD PLAIN	NORTH 1	10.6666*	Max WS	Dev10024	1375.50	7.25	10.75		10.79	0.000760	1.61	853.55	431.97	0.20
FLOOD PLAIN	NORTH 1	10.3333*	Max WS	10Yr24Hr	893.11	7.00	9.78		9.82	0.001622	1.65	541.51	467.70	0.27
FLOOD PLAIN	NORTH 1	10.3333*	Max WS	Dev10024	1374.81	7.00	10.44		10.48	0.000971	1.57	875.36	553.88	0.22
FLOOD PLAIN	NORTH 1	10	Max WS	10Yr24Hr	892.82	6.75	9.27		9.31	0.001139	1.71	522.05	327.62	0.24
FLOOD PLAIN	NORTH 1	10	Max WS	Dev10024	1374.11	6.75	10.12		10.16	0.000764	1.66	826.94	401.39	0.20
FLOOD PLAIN	NORTH 1	9.5*	Max WS	10Yr24Hr	892.56	6.10	8.94		8.97	0.000608	1.37	652.42	357.30	0.18
FLOOD PLAIN	NORTH 1	9.5*	Max WS	Dev10024	1373.51	6.10	9.89		9.92	0.000446	1.26	1093.85	539.68	0.16
FLOOD PLAIN	NORTH 1	9	Max WS	10Yr24Hr	892.44	5.45	8.80		8.81	0.000214	0.93	957.97	425.90	0.11
FLOOD PLAIN	NORTH 1	9	Max WS	Dev10024	1373.32	5.45	9.78		9.79	0.000172	0.98	1399.28	489.21	0.10

HEC-RAS Profile: Max WS (Continued)

River	Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
					(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
FLOOD PLAIN	NORTH 1	8.5*	Max WS	10Yr24Hr	892.36	4.97	8.60		8.64	0.000774	1.52	585.48	326.64	0.20
FLOOD PLAIN	NORTH 1	8.5*	Max WS	Dev10024	1373.12	4.97	9.64		9.67	0.000534	1.29	1068.18	582.70	0.17
FLOOD PLAIN	NORTH 1	8	Max WS	10Yr24Hr	892.19	4.48	8.23		8.29	0.001146	2.07	431.96	204.95	0.25
FLOOD PLAIN	NORTH 1	8	Max WS	Dev10024	1372.93	4.48	9.37		9.43	0.000778	1.95	704.28	272.52	0.21
FLOOD PLAIN	NORTH 1	7.5*	Max WS	10Yr24Hr	892.08	4.24	8.07		8.09	0.000383	0.95	937.59	625.89	0.14
FLOOD PLAIN	NORTH 1	7.5*	Max WS	Dev10024	1372.81	4.24	9.28		9.29	0.000201	0.75	1825.99	1069.28	0.10
FLOOD PLAIN	NORTH 1	7	Max WS	10Yr24Hr	892.06	4.00	8.03		8.03	0.000049	0.45	1982.91	866.82	0.05
FLOOD PLAIN	NORTH 1	7	Max WS	Dev10024	1372.77	4.00	9.25		9.25	0.000033	0.41	3334.21	1274.03	0.04
FLOOD PLAIN	NORTH 1	6	Max WS	10Yr24Hr	896.05	2.98	7.65	5.48	7.80	0.001379	3.15	284.54	79.57	0.29
FLOOD PLAIN	NORTH 1	6	Max WS	Dev10024	1376.75	2.98	8.89	6.05	9.09	0.001278	3.56	386.32	84.27	0.29
FLOOD PLAIN	NORTH 1	5.5			Bridge									
FLOOD PLAIN	NORTH 1	5	Max WS	10Yr24Hr	896.05	2.98	7.51		7.69	0.001648	3.38	265.09	76.00	0.32
FLOOD PLAIN	NORTH 1	5	Max WS	Dev10024	1376.75	2.98	8.75		8.98	0.001491	3.80	362.33	80.36	0.32
FLOOD PLAIN	NORTH 1	4.9	Max WS	10Yr24Hr	896.05	2.98	7.50	5.51	7.68	0.001661	3.39	264.40	75.97	0.32
FLOOD PLAIN	NORTH 1	4.9	Max WS	Dev10024	1376.75	2.98	8.75	6.10	8.97	0.001499	3.81	361.67	80.33	0.32
FLOOD PLAIN	NORTH 1	4.5			Bridge									
FLOOD PLAIN	NORTH 1	4	Max WS	10Yr24Hr	896.02	2.98	7.36		7.52	0.001528	3.28	273.49	77.64	0.31
FLOOD PLAIN	NORTH 1	4	Max WS	Dev10024	1376.49	2.98	8.62		8.83	0.001373	3.68	374.09	81.79	0.30
FLOOD PLAIN	NORTH 1	3	Max WS	10Yr24Hr	896.02	2.98	7.34		7.34	0.000049	0.52	1737.58	615.40	0.05
FLOOD PLAIN	NORTH 1	3	Max WS	Dev10024	1376.57	2.98	8.62		8.63	0.000046	0.52	2638.91	881.35	0.05
FLOOD PLAIN	NORTH 1	2.5*	Max WS	10Yr24Hr	895.98	3.18	7.32		7.33	0.000082	0.46	1959.04	1232.36	0.06
FLOOD PLAIN	NORTH 1	2.5*	Max WS	Dev10024	1376.52	3.18	8.62		8.62	0.000029	0.36	3773.95	1537.82	0.04
FLOOD PLAIN	NORTH 1	2	Max WS	10Yr24Hr	895.95	3.38	7.31		7.31	0.000017	0.30	3027.16	1112.67	0.03
FLOOD PLAIN	NORTH 1	2	Max WS	Dev10024	1376.49	3.38	8.61		8.61	0.000012	0.30	4567.43	1289.33	0.03
FLOOD PLAIN	NORTH 1	1.5*	Max WS	10Yr24Hr	895.99	2.55	7.31		7.31	0.000016	0.30	2981.13	1016.68	0.03
FLOOD PLAIN	NORTH 1	1.5*	Max WS	Dev10024	1376.64	2.55	8.61		8.61	0.000014	0.30	4563.22	1463.60	0.03
FLOOD PLAIN	NORTH 1	1	Max WS	10Yr24Hr	398.18	1.71	7.30	2.53	7.30	0.000007	0.21	1905.49	612.67	0.02
FLOOD PLAIN	NORTH 1	1	Max WS	Dev10024	398.71	1.71	8.60	2.53	8.60	0.000003	0.13	3090.63	1143.19	0.01
County Ditch	LENT1	16230	Max WS	10Yr24Hr	0.00	28.89	31.52		31.52	0.000000	0.00	47.46	18.07	0.00
County Ditch	LENT1	16230	Max WS	Dev10024	0.00	28.89	32.72		32.72	0.000000	0.00	69.22	18.11	0.00
County Ditch	LENT1	16225	Max WS	10Yr24Hr	0.00	28.89	31.52		31.52	0.000000	0.00	47.46	18.07	0.00
County Ditch	LENT1	16225	Max WS	Dev10024	0.00	28.89	32.72		32.72	0.000000	0.00	69.22	18.11	0.00
County Ditch	LENT1	16220			Lat Struct									
County Ditch	LENT1	16200	Max WS	10Yr24Hr	0.00	28.78	31.52		31.52	0.000000	0.00	49.44	18.08	0.00
County Ditch	LENT1	16200	Max WS	Dev10024	0.00	28.78	32.72		32.72	0.000000	0.00	71.21	18.11	0.00
County Ditch	LENT1	16125	Max WS	10Yr24Hr	126.04	28.73	31.29		31.40	0.000304	2.74	46.08	18.07	0.30
County Ditch	LENT1	16125	Max WS	Dev10024	281.75	28.73	32.07		32.41	0.000677	4.67	60.27	18.09	0.45
County Ditch	LENT1	16100	Max WS	10Yr24Hr	126.08	28.80	31.31		31.38	0.001232	2.19	57.56	31.43	0.29
County Ditch	LENT1	16100	Max WS	Dev10024	282.84	28.80	32.22		32.37	0.001861	3.18	89.00	37.75	0.36
County Ditch	LENT1	15460	Max WS	10Yr24Hr	124.83	28.34	30.83		30.88	0.000835	1.70	93.75	152.56	0.21
County Ditch	LENT1	15460	Max WS	Dev10024	278.43	28.34	31.62		31.68	0.000892	2.19	216.43	158.86	0.23
County Ditch	LENT1	14710	Max WS	10Yr24Hr	123.58	27.78	30.27		30.31	0.000831	1.69	92.62	152.50	0.21
County Ditch	LENT1	14710	Max WS	Dev10024	271.16	27.78	31.05		31.11	0.000857	2.14	215.19	158.80	0.22
County Ditch	LENT1	14195	Max WS	10Yr24Hr	122.01	27.20	29.73		29.76	0.000747	1.63	98.41	152.80	0.20
County Ditch	LENT1	14195	Max WS	Dev10024	258.21	27.20	30.55		30.60	0.000685	1.95	227.32	159.41	0.20
County Ditch	LENT1	13535	Max WS	10Yr24Hr	120.96	26.56	29.25		29.28	0.000512	1.42	124.43	154.16	0.17
County Ditch	LENT1	13535	Max WS	Dev10024	231.97	26.56	30.18		30.21	0.000363	1.50	270.48	161.56	0.15
County Ditch	LENT1	13000	Max WS	10Yr24Hr	120.91	25.99	29.01		29.02	0.000269	1.13	174.70	156.75	0.12
County Ditch	LENT1	13000	Max WS	Dev10024	224.66	25.99	30.01		30.03	0.000196	1.20	335.91	164.77	0.11
County Ditch	LENT1	12850			Lat Struct									
County Ditch	LENT1	12810	Max WS	10Yr24Hr	216.22	25.81	28.85		28.90	0.000826	1.98	178.22	156.93	0.22
County Ditch	LENT1	12810	Max WS	Dev10024	384.15	25.81	29.90		29.95	0.000523	1.98	347.93	165.35	0.18
County Ditch	LENT1	11995	Max WS	10Yr24Hr	212.81	25.01	28.21		28.25	0.000600	1.76	203.39	158.20	0.19
County Ditch	LENT1	11995	Max WS	Dev10024	376.89	25.01	29.55		29.58	0.000298	1.62	422.83	168.94	0.14
County Ditch	LENT1	11530	Max WS	10Yr24Hr	212.17	24.49	27.92		27.94	0.000410	1.53	239.12	160.00	0.16
County Ditch	LENT1	11530	Max WS	Dev10024	375.73	24.49	29.41		29.43	0.000201	1.41	487.35	171.96	0.12
County Ditch	LENT1	10915	Max WS	10Yr24Hr	212.16	23.90	27.71		27.73	0.000231	1.25	301.51	163.09	0.12
County Ditch	LENT1	10915	Max WS	Dev10024	375.58	23.90	29.31		29.32	0.000128	1.21	572.16	175.87	0.10
County Ditch	LENT1	10600			Lat Struct									

HEC-RAS Profile: Max WS (Continued)

River	Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
					(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
County Ditch	LENT1	10570	Max WS	10Yr24Hr	259.40	23.58	27.62		27.64	0.000256	1.37	338.47	164.89	0.13
County Ditch	LENT1	10570	Max WS	Dev10024	459.09	23.58	29.25		29.27	0.000154	1.37	618.79	177.97	0.11
County Ditch	LENT1	10055	Max WS	10Yr24Hr	318.18	23.27	27.04		27.30	0.002100	4.12	77.21	200.83	0.37
County Ditch	LENT1	10055	Max WS	Dev10024	561.59	23.27	28.67		29.07	0.001967	5.07	110.73	214.02	0.38
County Ditch	LENT1	10030			Culvert									
County Ditch	LENT1	10000	Max WS	10Yr24Hr	318.05	23.22	26.76		27.05	0.002591	4.39	72.48	198.97	0.41
County Ditch	LENT1	10000	Max WS	Dev10024	561.21	23.22	27.59		28.20	0.003995	6.27	89.49	205.66	0.53
County Ditch	LENT1	9730	Max WS	10Yr24Hr	317.61	22.95	26.36		26.42	0.000866	2.21	242.91	183.31	0.23
County Ditch	LENT1	9730	Max WS	Dev10024	560.46	22.95	27.14		27.21	0.000882	2.61	387.21	189.50	0.24
County Ditch	LENT1	9260	Max WS	10Yr24Hr	317.39	22.76	26.17		26.23	0.000866	2.21	242.71	183.30	0.23
County Ditch	LENT1	9260	Max WS	Dev10024	560.13	22.76	26.95		27.02	0.000881	2.61	387.15	189.50	0.24
County Ditch	LENT1	8885	Max WS	10Yr24Hr	316.99	22.29	25.72		25.78	0.000840	2.18	245.40	182.51	0.22
County Ditch	LENT1	8885	Max WS	Dev10024	559.51	22.29	26.49		26.57	0.000862	2.58	388.97	187.93	0.24
County Ditch	LENT1	8266	Max WS	10Yr24Hr	316.90	22.07	25.51		25.57	0.000825	2.17	247.78	183.52	0.22
County Ditch	LENT1	8266	Max WS	Dev10024	559.34	22.07	26.28		26.36	0.000850	2.57	392.10	189.71	0.23
County Ditch	LENT1	7900			Lat Struct									
County Ditch	LENT1	7850	Max WS	10Yr24Hr	332.21	21.53	24.95		25.02	0.000931	2.30	244.82	183.39	0.24
County Ditch	LENT1	7850	Max WS	Dev10024	586.63	21.53	25.72		25.80	0.000968	2.73	386.98	189.49	0.25
County Ditch	LENT1	7405	Max WS	10Yr24Hr	331.49	21.16	24.58		24.64	0.000853	2.20	271.98	213.37	0.23
County Ditch	LENT1	7405	Max WS	Dev10024	585.67	21.16	25.35		25.42	0.000836	2.54	438.75	219.54	0.23
County Ditch	LENT1	7195	Max WS	10Yr24Hr	331.13	20.96	24.40		24.45	0.000829	2.17	275.15	213.49	0.22
County Ditch	LENT1	7195	Max WS	Dev10024	585.32	20.96	25.17		25.24	0.000811	2.51	443.61	219.71	0.23
County Ditch	LENT1	7040	Max WS	10Yr24Hr	330.81	20.75	24.20		24.25	0.000812	2.16	277.51	213.58	0.22
County Ditch	LENT1	7040	Max WS	Dev10024	585.02	20.75	24.98		25.05	0.000790	2.49	447.93	219.87	0.23
County Ditch	LENT1	6715	Max WS	10Yr24Hr	330.50	20.43	23.92		23.97	0.000759	2.10	285.76	213.89	0.21
County Ditch	LENT1	6715	Max WS	Dev10024	584.81	20.43	24.72		24.77	0.000739	2.43	459.24	220.28	0.22
County Ditch	LENT1	6350	Max WS	10Yr24Hr	330.37	20.08	23.64		23.68	0.000658	2.00	302.26	214.46	0.20
County Ditch	LENT1	6350	Max WS	Dev10024	584.76	20.08	24.44		24.50	0.000660	2.34	477.90	220.92	0.21
County Ditch	LENT1	5700			Lat Struct									
County Ditch	LENT1	5685	Max WS	10Yr24Hr	345.55	19.73	23.35		23.39	0.000658	2.01	319.13	219.54	0.20
County Ditch	LENT1	5685	Max WS	Dev10024	612.97	19.73	24.15		24.20	0.000675	2.37	497.20	225.94	0.21
County Ditch	LENT1	5427.5*	Max WS	10Yr24Hr	345.53	19.42	23.13		23.17	0.000575	1.92	334.39	215.70	0.19
County Ditch	LENT1	5427.5*	Max WS	Dev10024	612.88	19.42	23.92		23.97	0.000623	2.31	507.24	222.02	0.20
County Ditch	LENT1	5170	Max WS	10Yr24Hr	345.40	19.16	22.87		22.93	0.000803	2.22	253.95	182.37	0.22
County Ditch	LENT1	5170	Max WS	Dev10024	611.94	19.16	23.63		23.72	0.000898	2.72	395.08	187.71	0.24
County Ditch	LENT1	4685	Max WS	10Yr24Hr	341.58	18.76	22.48		22.54	0.000779	2.19	254.85	182.41	0.22
County Ditch	LENT1	4685	Max WS	Dev10024	603.46	18.76	23.17		23.25	0.000948	2.76	382.44	187.24	0.25

CHAPTER 16. GRANT LINE CHANNEL

WATERSHED DESCRIPTION

The Grant Line Channel watershed lies in southern part of the City and covers nearly 550 acres. The watershed is generally bound by Highway 99 to the south and west, the Union Pacific Railroad to the east, and the Elk Grove Regional Park to the north (see Figure 16-1). The northern part of the watershed is drained by an underground pipe system that delivers runoff to the Grant Line Channel. The Grant Line Channel is a man-made trapezoidal channel that begins north of Grant Line Road and carries runoff for approximately 7,800 feet to the south end of the watershed. The channel exits the City in a 60-inch reinforced concrete culvert that carries runoff southeast under the UPRR to an outlet channel that continues to the east for approximately 1,500 feet before joining with Deer Creek.

During storm events, the water level in Deer Creek can be high enough to prevent water from passing through the 60-inch culvert. In such cases, the water in the channel will spill over a weir to a detention basin. If the water level in the detention basin exceeds a depth of approximately eight feet, water will spill into the Emerald Lakes Golf Course to the north. Runoff that enters the detention basin is pumped across the UPRR into the outlet channel by City Pump Station D-39.

Most of the land-use within the watershed is industrial and over half of the available industrial land is already developed. Some residential land-use exists in the northern end of the watershed and it is completely developed. There are a few commercial properties adjacent to Highway 99 that are mostly undeveloped. As previously mentioned, there is a golf course at the southern end of the watershed.

HYDROLOGIC ANALYSIS

A hydrologic analysis was performed to determine the 2-year, 10-year and 100-year flows for both existing and buildout conditions. The flood flows were calculated using the SacCalc modeling software and the Sacramento Method. For the hydrologic modeling, the watershed was divided into 20 subsheds as shown on Figure 16-2. Existing land use data was determined from aerial photographs and the City’s General Plan Land Use Policy Map. Table 16-1 presents the key hydrologic parameters for each subshed for both existing and buildout conditions. Table 16-2 presents the resultant peak flows from each subshed for the 10-year and 100-year storms. Note that the SacCalc models were only used to calculate the flows from each subshed before they enter the main drainage system. These inflows were combined and routed within the drainage system using a hydraulic model as discussed below.

HYDRAULIC ANALYSIS

A hydraulic analysis was performed to determine the existing flows and water surface elevations within key portions of the drainage system for the 2-year, 10-year, and 100-year storm events. An unsteady-state analysis was performed using the XPSWMM modeling software. Figure 16-2 provides a schematic showing the major drainage facilities included in the model. Figure 16-3 provides a more detailed plan showing the pump station and detention basin system at the south end of the watershed. Descriptions of these facilities are provided below.

Table 16-1. Hydrologic Parameters

Subshed	Area, acres	Mean Elevation, ft, NGVD29	Basin Length, ft	Basin Centroid Length, ft	Basin Slope, ft/ft	Land-Use (acres)						Average % Imp.
						Highway, Parking	Comm./ Office	Ind.	Resd, 4-6 du/ac	Park	Open	
						95%	90%	85%	40%	5%	2%	
Existing Conditions												
GL105	41.1	49	1210	600	0.0008			18.9			22.2	40.2
GL110	33.5	48	1890	900	0.0005	5.0		26.8			1.7	82.4
GLC575	19.7	46	1670	850	0.0024			13.8			5.9	60.1
GL305	18.1	46	1800	700	0.0022	0.9		11.7			5.4	60.6
GL405	16.5	44	1230	790	0.0016			11.5			4.9	60.1
GL510	31.4	44	1460	860	0.0014	3.1		20.6			7.6	65.8
GL515	11.3	45	790	480	0.0051			11.3				85.0
GL520	15.6	47	1370	630	0.0015			11.7			3.9	64.3
GL525	13.7	50	1360	700	0.0029						13.7	2.0
GL535	22.1	46	1200	610	0.0020			8.9			13.3	35.2
GL540	27.7	44	1590	830	0.0004	2.8	4.1				20.7	24.5
GL550	19.6	46	1000	390	0.0010			7.9			11.8	35.2
GL555	18.2	46	2350	1400	0.0004				17.3		0.9	38.1
GL560	18.9	44	1180	600	0.0017	3.8					15.1	20.6
GL570	31.7	44	2860	1080	0.0019				31.7			40.0
GLC500	40.5	48	1260	500	0.0063			16.2			24.3	35.2
GLC700	30.4	44	1670	780	0.0022	3.9	3.9	1.2			21.3	28.9
GLC630	39.8	46	2470	1200	0.0016			35.8			4.0	76.7
GLC650	21.9	44	1830	1020	0.0022			8.8			13.2	35.2
GLB2	75.5	42	3230	1610	0.0037	7.5				45.3	22.6	13.1
Buildout Conditions												
GL105	41.1	49	1,210	600	0.0008			37.0			4.1	76.7
GL110	33.5	48	1,890	900	0.0005	5.0		26.8			1.7	82.4
GLC575	19.7	46	1,670	850	0.0024			18.8			1.0	80.9
GL305	18.1	46	1,800	700	0.0022	0.9		17.2				85.5
GL405	16.5	44	1,230	790	0.0016			16.5				85.0
GL510	31.4	44	1,460	860	0.0014	3.1		28.3				86.0
GL515	11.3	45	790	480	0.0051			11.3				85.0
GL520	15.6	47	1,370	630	0.0015			15.6				85.0
GL525	13.7	50	1,360	700	0.0029			10.3			3.4	64.3
GL535	22.1	46	1,200	610	0.0020			19.9			2.2	76.7
GL540	27.7	44	1,590	830	0.0004	2.8	2.8	22.1				86.5
GL550	19.6	46	1,000	390	0.0010			19.6				85.0
GL555	18.2	46	2,350	1,400	0.0004			0.9	17.3			42.3
GL560	18.9	44	1,180	600	0.0017	3.8	13.3	1.9				90.5
GL570	31.7	44	2,860	1,080	0.0019				31.7			40.0
GLC500	40.5	48	1,260	500	0.0063			38.4			2.0	80.9
GLC700	30.4	44	1,670	780	0.0022	3.0	9.1	18.2				87.5
GLC630	39.8	46	2,470	1,200	0.0016			35.8			4.0	76.7
GLC650	21.9	44	1,830	1,020	0.0022			21.9				85.0
GLB2	75.5	42	3,230	1,610	0.0037	7.5		3.8		41.5	22.6	17.1

Table 16-2 Calculated Subshed Flows

		Existing Conditions				Buildout Conditions			
Subshed	Area	2-Year	10-Year	100-Year, 24-Hour	100-Year, 10-Day	2-Year	10-Year	100-Year, 24-Hour	100-Year, 10-Day
GL105	41.1	23	46	73	27	29	57	85	29
GL110	33.5	21	40	58	23	21	40	58	23
GLC575	19.7	12	24	37	14	14	27	41	14
GL305	18.1	11	22	35	13	13	26	39	13
GL405	16.5	11	21	32	11	12	24	36	12
GL510	31.4	20	38	59	22	23	44	65	22
GL515	11.3	10	21	31	8	10	21	31	8
GL520	15.6	10	20	31	11	12	23	34	11
GL525	13.7	7	13	23	9	10	19	29	10
GL535	22.1	13	26	42	15	17	33	50	16
GL540	27.7	12	24	39	17	18	35	51	19
GL550	19.6	12	24	39	13	16	32	48	14
GL555	18.2	8	16	23	11	9	16	23	11
GL560	18.9	10	20	34	12	15	30	44	14
GL570	31.7	17	33	47	20	17	33	47	20
GLC500	40.5	27	54	88	28	34	68	105	29
GLC700	30.4	16	31	51	20	23	44	65	22
GLC630	39.8	24	46	68	27	24	46	68	27
GLC650	21.9	11	22	35	14	15	30	43	16
GLB2	75.5	30	59	98	45	31	60	98	46

Facilities Descriptions

Trunk Pipelines

The northern portion of the watershed is served by an underground pipe network. Pipelines with diameters of 27 inches or larger were included in the hydraulic model. The pipeline invert elevations were determined from as-built plans.

Grant Line Channel

The Grant Line Channel is a man-made, earthen, trapezoidal channel that conveys runoff from north to south. The channel is mostly vegetated with grass, although there are some locations with cattails and a few small trees. The channel cross section geometry used in the model was based on as-built plans. Cross sections were surveyed at some locations and were found to be in general agreement with the as-built plans.

Culverts

There are three sets of culverts along the channel. At Grant Line Road, there are two 66-inch concrete pipe culverts. At a driveway entrance downstream, there are two 66-inch corrugated metal pipe culverts. At the downstream end of the watershed, there is a single 60-inch concrete pipe culvert under the UPRR that acts as the main outlet from the watershed when the water surface elevation in Deer Creek is not too high. This culvert has a flap gate on the downstream side to prevent backflow from Deer Creek. For all of the culverts in the watershed, the invert elevations, pipe sizes, and pipe materials were determined from a field survey.

Detention Basin

A detention basin lies at the southern end of the watershed near the 60-inch outfall culvert (see Figure 16-3). The detention basin serves as a storage area for runoff that exceeds the capacity of the 60-inch outfall. If the flow depth at the upstream end of the outlet culvert reaches approximately 6.7 feet, elevation 36.1 feet, water will flow from the channel into the detention basin over a concrete spillway. The detention basin is separated from the Emerald Lakes Golf Course by a dike along the northern perimeter of the basin. If the depth in the detention basin exceeds elevation 36.0 feet, water will spill into the golf course over a concrete spillway. The elevation-volume relationship for the basin is listed in Table 16-3 below.

Table 16-3. Grant Line Channel – Detention Basin Storage

Elevation, feet	Depth, feet	Storage, acre-feet
28.0	0.0	0.0
30.0	2.0	1.1
32.0	4.0	3.8
34.0	6.0	7.3
36.0	8.0	11.2
38.0	10.0	15.7
40.0	12.0	20.6

Pump Station

City Pump Station D-39 is located at the south end of the watershed adjacent to the detention basin. The pump station evacuates runoff that spills into the detention basin. The pump station was constructed in 1967 and has a full rated capacity of 10 cfs. The pump station lies in a 5-foot diameter concrete wet well. The intake into the wet well is 36-inch concrete pipe with an invert elevation of approximately 30.8 feet. Because this elevation is 2.8 feet above the bottom of the detention basin, some of the detention storage is unavailable after it fills with early season runoff. There is also a 36-inch pipe between the pump station wet well and the Grant Line Channel. This pipe allows gravity flow from the detention basin to the channel if the water level in the channel is low enough. A flap gate on the channel end of the pipe prevents reverse flow through this pipe.

Boundary Conditions

During storm events, high water elevations in Deer Creek can restrict gravity flow though the 60-inch outlet culvert under the UPRR. This is particularly true for large, multi-day storm events that can produce high water surface elevations in Deer Creek at the same time that significant flows are present in the Grant Line Channel system. A 100-year stage hydrograph was available for Deer Creek from a hydraulic model prepared by the County of Sacramento. The County model simulates a 100-year, 10-day storm. Therefore, for SDMP, the Grant Line Channel watershed was evaluated for a 100-year, 10-day storm event and the stage hydrograph from the Sacramento County model was used as the downstream boundary condition. Because the 100-year, 24-hour storm can produce higher peak flows for relatively small watersheds, the Grant Line Channel watershed was also evaluated using a 100-year, 24-hour storm. No Deer Creek modeling was available for the 100-year, 24-hour storm, nor the 2-year, 10-year storm events. However, for these storm events, the tailwater in Deer Creek is not anticipated to have a significant effect on the Grant Line Channel system, therefore, a free outfall condition with no restriction to gravity outflow was used for those storms.

Evaluation Criteria

Because the Grant Line Channel drainage system is an existing system that was designed using outdated hydrology, it is not expected to provide the same level of protection as a system designed using current standards. As discussed in Chapter 2, the evaluation criteria for an existing drainage system are as follows:

1. The hydraulic grade line for a 10-year storm must not exceed the top of curb elevation.
2. Structures must be protected from the 100-year storms (24-hour and 10-day). The hydraulic grade line for the 100-year storms must not exceed the elevation of building pads.

In addition to the evaluation criteria listed above, there are two other issues of concern in the watershed that were considered in this evaluation. The operators of the Emerald Lakes Golf Course have expressed concern over the frequency of flooding of the golf course. Therefore, although the elimination of flooding in the golf course was not the main objective of the SDMP, the potential effect of future development on flooding in the golf course was evaluated and mitigation measures were developed.

Bank sloughing has been a historic problem along the Grant Line Channel. A significant contributor to the problem is the fact that the channel can sit full of water for long periods of time causing the relatively steep banks to become saturated. When the water in the channel subsides, the saturated banks are subject to sloughing. This scenario occurs when the water surface in Deer Creek is high enough to prevent gravity outflow through the 60-inch outlet culvert. Because the depth of water in the channel must reach approximately 6.8 feet before water can spill into the detention basin and be pumped out of the watershed, there is no way for the water in the channel to be emptied until the water in Deer Creek subsides.

Results of Hydraulic Analysis

Results for the 2-year Storm

Calculated water surface elevations for the 2-year storm are summarized on Table 16-4. The existing storm drainage system is capable of handling both the existing and buildout 2-year flows without significant street flooding.

Results for the 10-year Storm

Calculated water surface elevations for the 10-year storm are summarized in Table 16-5. As the table shows, for existing conditions there are three locations where excessive street flooding is predicted during a 10-year storm event. For buildout conditions, the 10-year water surface elevations within the watershed are predicted to increase by an average of 0.3 feet. Two additional locations are predicted to have street flooding above the top of curb. Figure 16-4 shows the locations of predicted street flooding during the 10-year storm.

Results for the 100-year Storm

Calculated water surface elevations for the 100-year, 24-hour and 100-year, 10-day storms are summarized on Tables 16-6 and 16-7. As the tables show, no building pads are predicted to flood under either existing or buildout conditions. However, flooding in the golf course is predicted to increase by 0.7 feet under buildout conditions during a 100-year, 10-day storm with high stages in Deer Creek (see Table 16-7).

Water surface profiles for the 2-, 10-, and 100-year storms are presented on Figure 16-5. The 100-year storm profile is based on a 10-day duration with a high tailwater in Deer Creek. This represents the worst case 100-year scenario.

EVALUATION OF ALTERNATIVE IMPROVEMENTS

Two alternative flood control solutions were developed to eliminate the predicted street flooding under buildout conditions, eliminate the potential increase in golf course flooding due to buildout, and reduce the duration of standing water in the channel during periods of high flow in Deer Creek. Some of the recommended facilities were previously evaluated by PSOMAS for their *Grantline Channel and Pump Station D-39 Hydrologic and Hydraulic Analysis Report, March 2005.*

Table 16-4. Calculated 2-Year Water Surface Elevations (NGVD29) and Flows

Node Name	Notes	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	Existing 2-Year Peak Flow, cfs	Existing 2-Year Water Surface Elevation, feet	Buildout 2-Year Peak Flow, cfs	Buildout 2-Year Water Surface Elevation, feet	Existing Street Flooding	Future Street Flooding
D39PS	Pump Station	40.5	n/a	n/a	32.3	n/a	32.3	-	-
GLB1	Detention Basin	40.5	44.0	n/a	32.3	n/a	32.3	-	-
GLB2	Golf Course	n/a	44.0	n/a	32.6	n/a	32.6	-	-
GLC100	Grant Line Channel	n/a	n/a	n/a	33.6	n/a	33.9	-	-
GLC150		40.3	n/a	125	34.0	138	34.3	-	-
GLC175		40.2	n/a	126	34.4	139	34.7	-	-
GLC200		40.7	n/a	127	34.6	140	34.9	-	-
GLC250		46.0	44.0	127	35.0	141	35.2	-	-
GLC280		44.2	48.0	129	35.3	143	35.5	-	-
GLC300		45.4	48.0	130	35.4	144	35.7	-	-
GLC340		45.6	48.0	131	35.7	146	36.0	-	-
GLC360		45.5	48.0	133	36.0	149	36.3	-	-
GLC390		46.3	48.0	135	36.2	151	36.5	-	-
GLC400		46.4	48.0	136	36.2	153	36.5	-	-
GLC410		46.1	48.0	115	36.3	128	36.5	-	-
GLC440		47.7	48.0	115	36.4	129	36.7	-	-
GLC460		47.2	46.0	116	36.5	129	36.9	-	-
GLC480		45.6	46.0	117	36.7	130	37.0	-	-
GLC500		45.1	46.0	118	36.7	132	37.0	-	-
GLC525		46.0	46.0	110	36.8	123	37.1	-	-
GLC550		46.5	46.0	112	36.9	125	37.2	-	-
GLC575		44.9	46.0	115	37.1	130	37.4	-	-
GLC600		45.6	46.0	112	37.2	127	37.5	-	-
GLC610		45.1	46.0	108	37.3	122	37.6	-	-
GLC620		44.5	46.0	103	37.3	116	37.7	-	-
GLC630		44.0	44.0	105	37.5	119	37.9	-	-
GLC650		43.8	44.0	94	37.7	106	38.0	-	-
GLC675		44.5	44.0	88	37.9	99	38.2	-	-
GLC700		43.5	43.0	88	38.1	100	38.5	-	-
GLC725		42.8	43.0	79	38.2	90	38.6	-	-
GLC750		43.3	43.0	83	38.3	95	38.7	-	-
GL105	Pipe System	48.9	48.0	42	38.2	45	38.3	-	-
GL110		48.5	48.0	20	39.8	20	39.9	-	-
GL305		44.6	45.0	11	37.3	13	37.6	-	-
GL405		41.6	45.0	10	37.4	12	37.7	-	-
GL500		44.3	n/a	85	38.3	99	38.7	-	-
GL505		42.5	43.0	86	38.6	99	39.0	-	-
GL510		43.2	44.0	42	38.8	51	39.2	-	-
GL513		43.7	44.0	28	38.9	33	39.4	-	-
GL515		42.8	45.0	29	39.1	35	39.7	-	-
GL520		44.5	45.0	26	39.4	30	40.1	-	-
GL525		45.9	46.0	18	40.0	22	40.7	-	-
GL530		44.9	47.0	n/a	40.0	n/a	40.7	-	-
GL535		44.9	46.8	13	40.6	16	41.2	-	-
GL540		43.0	44.0	46	39.0	54	39.5	-	-
GL544		42.4	44.0	16	39.3	17	39.7	-	-
GL545		43.5	44.0	17	39.6	19	40.0	-	-
GL550		44.9	46.0	18	40.1	20	40.7	-	-
GL555		46.0	46.0	8	40.8	9	41.0	-	-
GL560		42.7	44.0	25	39.6	25	40.0	-	-
GL565		43.5	44.7	16	39.9	17	40.4	-	-
GL570		43.1	44.6	17	41.2	17	41.7	-	-

Table 16-5. Calculated 10-Year Water Surface Elevations (NGVD29) and Flows

Node Name	Notes	Estimated Top of Curb Elevation, feet	Estimated Pad Elevation, feet	Existing 10-Year Peak Flow, cfs	Existing 10-Year Water Surface Elevation, feet	Buildout 10-Year Peak Flow, cfs	Buildout 10-Year Water Surface Elevation, feet	Existing Street Flooding	Future Street Flooding
D39PS	Pump Station	40.5	n/a	n/a	33.0	n/a	33.0	-	-
GLB1	Detention Basin	40.5	44.0	n/a	32.8	n/a	32.8	-	-
GLB2	Golf Course	n/a	44.0	n/a	33.0	n/a	33.1	-	-
GLC100	Grant Line Channel	n/a	n/a	n/a	35.9	n/a	36.3	-	-
GLC150		40.3	n/a	202	36.2	212	36.5	-	-
GLC175		40.2	n/a	204	36.4	213	36.7	-	-
GLC200		40.7	n/a	207	36.5	217	36.8	-	-
GLC250		46.0	44.0	211	36.8	222	37.0	-	-
GLC280		44.2	48.0	217	37.0	230	37.2	-	-
GLC300		45.4	48.0	221	37.1	234	37.3	-	-
GLC340		45.6	48.0	224	37.3	239	37.6	-	-
GLC360		45.5	48.0	230	37.6	245	37.8	-	-
GLC390		46.3	48.0	234	37.8	251	38.0	-	-
GLC400		46.4	48.0	237	37.8	225	38.0	-	-
GLC410		46.1	48.0	196	37.8	206	38.0	-	-
GLC440		47.7	48.0	197	38.0	207	38.2	-	-
GLC460		47.2	46.0	197	38.4	208	38.7	-	-
GLC480		45.6	46.0	198	38.5	209	38.7	-	-
GLC500		45.1	46.0	201	38.6	211	38.8	-	-
GLC525		46.0	46.0	185	38.7	195	38.9	-	-
GLC550		46.5	46.0	188	38.8	198	39.0	-	-
GLC575		44.9	46.0	194	38.9	209	39.1	-	-
GLC600		45.6	46.0	189	39.0	204	39.2	-	-
GLC610		45.1	46.0	179	39.4	193	39.6	-	-
GLC620		44.5	46.0	170	39.4	180	39.6	-	-
GLC630		44.0	44.0	173	39.5	184	39.8	-	-
GLC650		43.8	44.0	152	39.6	157	39.9	-	-
GLC675		44.5	44.0	140	40.1	143	40.4	-	-
GLC700		43.5	43.0	140	40.6	144	40.9	-	-
GLC725		42.8	43.0	122	40.7	124	41.0	-	-
GLC750		43.3	43.0	139	40.7	149	41.0	-	-
GL105	Pipe System	48.9	48.0	80	39.8	86	40.1	-	-
GL110		48.5	48.0	37	41.1	39	41.4	-	-
GL305		44.6	45.0	22	40.2	26	41.5	-	-
GL405		41.6	45.0	21	39.7	24	40.6	-	-
GL500		44.3	n/a	152	40.7	164	41.0	-	-
GL505		42.5	43.0	155	41.2	167	41.5	-	-
GL510		43.2	44.0	86	41.6	89	42.0	-	-
GL513		43.7	44.0	50	42.1	49	42.5	-	-
GL515		42.8	45.0	50	42.6	49	***43.0***	-	Yes
GL520		44.5	45.0	41	43.5	43	43.9	-	-
GL525		45.9	46.0	25	44.4	26	44.8	-	-
GL530		44.9	47.0	n/a	44.5	n/a	44.8	-	-
GL535		44.9	46.8	19	***45.1***	20	***45.3***	Yes	Yes
GL540		43.0	44.0	75	42.0	83	42.3	-	-
GL544		42.4	44.0	28	***42.5***	28	***42.8***	Yes	Yes
GL545		43.5	44.0	30	43.3	31	43.4	-	-
GL550		44.9	46.0	31	44.7	33	44.8	-	-
GL555		46.0	46.0	15	45.3	16	45.4	-	-
GL560		42.7	44.0	34	42.6	36	***42.8***	-	Yes
GL565		43.5	44.7	21	42.9	22	43.1	-	-
GL570		43.1	44.6	21	***43.7***	21	***43.8***	Yes	Yes

Table 16-6. Calculated 100-Year, 24-Hour Water Surface Elevations (NGVD29) and Flows

Node Name	Notes	Est. Top of Curb Elev., feet	Est. Pad Elev., feet	Existing 100-Year Peak Flow, cfs	Existing 100-Year Water Surface Elevation, feet	Buildout 100-Year Peak Flow, cfs	Buildout 100-Year Water Surface Elevation, feet	Existing Pad Flooding	Future Pad Flooding
D39PS	Pump Station	n/a	n/a	n/a	34.7	n/a	35.2	-	-
GLB1	Detention Basin	n/a	n/a	n/a	34.7	n/a	35.3	-	-
GLB2	Golf Course	n/a	n/a	n/a	34.3	n/a	34.4	-	-
GLC100	Grant Line Channel	n/a	n/a	n/a	37.2	n/a	37.3	-	-
GLC150		n/a	n/a	281	37.4	294	37.5	-	-
GLC175		n/a	n/a	281	37.6	294	37.7	-	-
GLC200		n/a	n/a	282	37.7	295	37.9	-	-
GLC250		n/a	44.0	283	38.0	297	38.1	-	-
GLC280		n/a	48.0	290	38.2	306	38.3	-	-
GLC300		n/a	48.0	299	38.3	314	38.4	-	-
GLC340		n/a	48.0	308	38.5	322	38.7	-	-
GLC360		n/a	48.0	319	38.8	334	38.9	-	-
GLC390		n/a	48.0	328	39.0	343	39.1	-	-
GLC400		n/a	48.0	332	39.0	348	39.1	-	-
GLC410		n/a	48.0	247	39.0	255	39.1	-	-
GLC440		n/a	48.0	249	39.1	256	39.2	-	-
GLC460		n/a	46.0	250	39.8	258	40.0	-	-
GLC480		n/a	46.0	252	39.9	261	40.1	-	-
GLC500		n/a	46.0	255	39.9	266	40.1	-	-
GLC525		n/a	46.0	227	40.0	238	40.2	-	-
GLC550		n/a	46.0	231	40.1	245	40.3	-	-
GLC575		n/a	46.0	244	40.1	259	40.3	-	-
GLC600		n/a	46.0	233	40.2	245	40.4	-	-
GLC610		n/a	46.0	213	40.8	222	41.0	-	-
GLC620		n/a	46.0	195	40.8	203	41.0	-	-
GLC630		n/a	44.0	202	40.9	212	41.1	-	-
GLC650		n/a	44.0	162	40.9	169	41.1	-	-
GLC675		n/a	44.0	150	41.5	153	41.7	-	-
GLC700		n/a	43.0	150	42.1	153	42.3	-	-
GLC725		n/a	43.0	138	42.1	143	42.3	-	-
GLC750		n/a	43.0	145	42.1	142	42.3	-	-
GL105	Pipe System	48.9	48.0	118	42.3	123	42.7	-	-
GL110		48.5	48.0	54	44.4	54	44.6	-	-
GL305		44.6	45.0	32	44.3	32	44.6	-	-
GL405		41.6	45.0	26	41.9	25	42.2	-	-
GL500		44.3	n/a	160	42.1	158	42.3	-	-
GL505		42.5	43.0	162	42.7	160	42.9	-	-
GL510		43.2	44.0	92	43.0	87	43.2	-	-
GL513		43.7	44.0	53	43.4	49	43.6	-	-
GL515		42.8	45.0	53	43.7	49	43.8	-	-
GL520		44.5	45.0	42	44.4	42	44.5	-	-
GL525		45.9	46.0	27	45.4	28	45.6	-	-
GL530		44.9	47.0	6	45.4	7	45.6	-	-
GL535		44.9	46.8	16	45.7	17	45.8	-	-
GL540		43.0	44.0	74	43.3	78	43.4	-	-
GL544		42.4	44.0	34	43.4	34	43.5	-	-
GL545		43.5	44.0	29	43.7	28	43.7	-	-
GL550		44.9	46.0	31	45.0	31	45.1	-	-
GL555		46.0	46.0	18	45.8	18	45.8	-	-
GL560		42.7	44.0	34	43.4	36	43.5	-	-
GL565		43.5	44.7	21	43.6	21	43.7	-	-
GL570		43.1	44.6	21	44.1	21	44.1	-	-

Table 16-7. Calculated 100-Year, 10-Day Water Surface Elevations (NGVD29) and Flows

Node Name	Notes	Est. Top of Curb Elev., feet	Est. Pad Elev., feet	Existing 100-Year Peak Flow, cfs	Existing 100-Year Water Surface Elevation, feet	Buildout 100-Year Peak Flow, cfs	Buildout 100-Year Water Surface Elevation, feet	Existing Pad Flooding	Future Pad Flooding
D39PS	Pump Station	n/a	n/a	n/a	39.9	n/a	40.5	-	-
GLB1	Detention Basin	n/a	n/a	n/a	39.9	n/a	40.5	-	-
GLB2	Golf Course	n/a	n/a	n/a	39.7	n/a	40.4	-	-
GLC100	Grant Line Channel	n/a	n/a	n/a	39.9	n/a	40.5	-	-
GLC150		n/a	n/a	197	39.9	205	40.6	-	-
GLC175		n/a	n/a	200	40.0	211	40.6	-	-
GLC200		n/a	n/a	203	40.0	215	40.6	-	-
GLC250		n/a	44.0	205	40.0	218	40.6	-	-
GLC280		n/a	48.0	207	40.1	222	40.6	-	-
GLC300		n/a	48.0	211	40.1	226	40.6	-	-
GLC340		n/a	48.0	215	40.1	228	40.6	-	-
GLC360		n/a	48.0	219	40.2	232	40.6	-	-
GLC390		n/a	48.0	224	40.2	236	40.7	-	-
GLC400		n/a	48.0	226	40.2	238	40.7	-	-
GLC410		n/a	48.0	181	40.2	189	40.7	-	-
GLC440		n/a	48.0	181	40.3	190	40.7	-	-
GLC460		n/a	46.0	183	40.6	192	40.9	-	-
GLC480		n/a	46.0	184	40.6	203	40.9	-	-
GLC500		n/a	46.0	183	40.6	209	40.9	-	-
GLC525		n/a	46.0	166	40.7	179	40.9	-	-
GLC550		n/a	46.0	167	40.7	174	41.0	-	-
GLC575		n/a	46.0	173	40.7	177	41.0	-	-
GLC600		n/a	46.0	163	40.7	167	41.0	-	-
GLC610		n/a	46.0	152	40.8	156	41.2	-	-
GLC620		n/a	46.0	142	41.1	147	41.3	-	-
GLC630		n/a	44.0	145	41.1	148	41.3	-	-
GLC650		n/a	44.0	123	41.2	125	41.3	-	-
GLC675		n/a	44.0	112	41.5	113	41.7	-	-
GLC700		n/a	43.0	112	41.9	113	42.0	-	-
GLC725		n/a	43.0	99	41.9	100	42.1	-	-
GLC750		n/a	43.0	103	41.9	102	42.1	-	-
GL105	Pipe System	48.9	48.0	49	40.7	52	40.9	-	-
GL110		48.5	48.0	22	41.1	23	41.3	-	-
GL305		44.6	45.0	12	41.4	13	41.7	-	-
GL405		41.6	45.0	14	41.4	12	41.6	-	-
GL500		44.3	n/a	107	41.9	106	42.1	-	-
GL505		42.5	43.0	107	42.2	106	42.4	-	-
GL510		43.2	44.0	54	42.5	53	42.7	-	-
GL513		43.7	44.0	35	42.8	35	43.0	-	-
GL515		42.8	45.0	35	43.1	35	43.3	-	-
GL520		44.5	45.0	31	43.7	32	43.9	-	-
GL525		45.9	46.0	21	44.3	22	44.5	-	-
GL530		44.9	47.0	n/a	44.3	n/a	44.5	-	-
GL535		44.9	46.8	13	44.8	14	45.0	-	-
GL540		43.0	44.0	54	42.7	56	42.9	-	-
GL544		42.4	44.0	22	43.0	23	43.1	-	-
GL545		43.5	44.0	21	43.3	20	43.4	-	-
GL550		44.9	46.0	23	44.5	24	44.6	-	-
GL555		46.0	46.0	10	44.9	17	45.0	-	-
GL560		42.7	44.0	26	43.0	26	43.1	-	-
GL565		43.5	44.7	15	43.3	18	43.4	-	-
GL570		43.1	44.6	15	43.8	18	43.8	-	-

Descriptions of Alternatives

Descriptions of the alternatives are provided below.

Alternative 1

Alternative 1 includes improvements to the existing pipe system serving the watershed and modifications to the existing pump station.

Pipeline Improvements – Alternative 1

Pipeline improvements would be constructed in the northern portion of the watershed to eliminate the predicted street flooding under both existing and buildout conditions. Approximately 6,700 feet of existing pipeline would require upsizing. The required pipeline improvements are shown on Figure 16-6.

Pump Station and Detention Basin Improvements – Alternative 1

Alternative 1 would include improvements to the existing pump station, detention basin, and the associated piping to eliminate the potential increased flooding in the golf course due to development and to help reduce the duration of standing water in the channel. These improvements are shown on Figure 16-7 and are described below:

1. Increase the capacity of the pump station from 10 cfs to 14 cfs.
2. Construct improvements to the pump station and inlet piping. This consists of deepening the wet well to allow the intake pipe to be lowered to allow the entire volume of the detention basin to be evacuated by the pump station, re-grading of the detention basin to efficiently drain to the lowered intake pipe, and replacing the existing wooden catwalk.
3. Allow flow from the channel directly to the pump station. This consists of replacing the flap gate on the 36-inch pipe between the channel and the pump station with a float activated control valve. A float controlled valve would also be installed on the 36-inch inlet from the detention basin to the pump station. The valves would be operated to allow water from the channel to flow to the pump station early in a storm event before the detention basin begins to fill. Once the basin starts filling, the flow from the channel to the pump station would be stopped and pumping from the basin would begin.
4. Replace the existing 12-inch pipes between the detention basin and the golf course with 24-inch pipes.

Alternative 2

Alternative 2 also includes improvements to the existing pipe system serving the watershed and some modifications to the existing pump station. In addition, the storage volume of the existing detention basin would be expanded. The improvements proposed with this alternative are shown on Figures 16-6 and 16-8.

Pipeline Improvements

The same pipeline improvements included with Alternative are included with this alternative (see Figure 16-6).

Pump Station Improvements – Alternative 2

As with Alternative 1, pump station and inlet improvements would be included with Alternative 2. However, the capacity of the pump station would not be increased.

Detention Basin Improvements – Alternative 2

For Alternative 2, the bottom of the existing detention basin would be lowered to elevation 16 feet to increase the total storage volume from 20.6 acre-feet to 32.4 acre-feet. In addition, the existing 12-inch pipes between the golf course and the detention basin would be replaced with 24-inch pipes.

Evaluation of Alternatives

Flood Control Performance

Both alternatives would eliminate the existing street flooding in the watershed, eliminate potential development impacts at the golf course, and would improve the City's ability to dewater the channel. Alternative 2 would produce slightly lower water surface elevations in the golf course during a 10-year storm event. In addition, the duration of flooding in the golf course would be less with the Alternative 2 improvements.

Implementation Costs

The estimated implementation costs for Alternatives 1 and 2 are approximately $4.5 million and $4.6 million, respectively. Detailed implementation costs estimates for the two alternatives are presented on Tables 16-8 and 16-9, respectively. The cost for Alternative 2 is about 2 percent higher due to the cost of excavating the detention basin. Although the pumping capacity is higher for Alternative 1, the pump station costs for the two alternatives are estimated to be the same. This is because the pump station for Alternative 2 will pump against a higher head due to the lowering of the detention basin. For the purposes of this master plan, the costs of these two alternatives are deemed to be equivalent.

Selection of the Preferred Alternative

Alternative 2 is selected as the preferred alternative because it provides more flood protection. The additional detention storage included with Alternative 2 provides more effective flood protection at the golf course. The additional detention storage would also be beneficial in the event of a pump failure.

Table 16-8. Estimated Storm Drainage Improvements Costs
Grant Line Channel Alternative 1

Item	Quantity	Unit	Unit Cost, dollars	Total Cost, dollars	% Allocated to Fut. Dev.	Total Allocated to Fut. Dev., dollars
Existing Pipeline Upgrades						
36-inch RCP	413	LF	230	94,990	70%	66,493
42-inch RCP	1,110	LF	269	298,590	70%	209,013
48-inch RCP	500	LF	309	154,500	70%	108,150
54-inch RCP	981	LF	346	339,426	70%	237,598
60-inch RCP	2,354	LF	386	908,644	70%	636,051
66-inch RCP	380	LF	400	152,000	70%	106,400
72-inch RCP	975	LF	409	398,775	70%	279,143
Manholes	19	EA	6,500	123,500	70%	86,450
Pump Station And Detention Basin Improvements						
24-inch RCP	60	LF	154	9,240	0%	0
Detention Basin Grading	8,000	CY	10	80,000	100%	80,000
Reconstruct Pump Station	1	LS	200,000	200,000	100%	200,000
Replace Catwalk	1	LS	12,000	12,000	0%	0
Install Control Valves	1	LS	150,000	150,000	50%	75,000
Subtotal				2,921,665		2,084,298
Construction Contingency @ 20%				**584,333**		**416,860**
Subtotal Construction				**3,505,998**		**2,501,157**
Design & Planning @ 10%				350,600		250,116
Construction Management @ 10%				350,600		250,116
Environmental Review and Mitigation @ 5%				175,300		125,058
Program Implementation @ 5%				175,300		125,058
ROUNDED TOTAL ESTIMATED COST				**4,560,000**		**3,250,000**

Table 16-9. Estimated Storm Drainage Improvements Costs
Grant Line Channel Alternative 2

Item	Quantity	Unit	Unit Cost, dollars	Total Cost, dollars	% Allocated to Fut. Dev.	Total Allocated to Fut. Dev., dollars
Existing Pipeline Upgrades						
36-inch RCP	413	LF	230	94,990	70%	66,493
42-inch RCP	1,110	LF	269	298,590	70%	209,013
48-inch RCP	500	LF	309	154,500	70%	108,150
54-inch RCP	981	LF	346	339,426	70%	237,598
60-inch RCP	2,354	LF	386	908,644	70%	636,051
66-inch RCP	380	LF	400	152,000	70%	106,400
72-inch RCP	975	LF	409	398,775	70%	279,143
Manholes	19	EA	6,500	123,500	70%	86,450
Pump Station And Detention Basin Improvements						
24-inch RCP	60	LF	154	9,240	0%	0
Expand Detention Basin	19,000	CY	10	190,000	100%	190,000
Reconstruct Pump Station	1	LS	200,000	200,000	100%	200,000
Replace Catwalk	1	LS	12,000	12,000	0%	0
Install Control Valves	1	LS	150,000	150,000	50%	75,000
Subtotal				3,031,665		2,194,298
Construction Contingency @ 20%				**606,333**		**438,860**
Subtotal Construction				**3,637,998**		**2,633,157**
Design & Planning @ 10%				363,800		263,316
Construction Management @ 10%				363,800		263,316
Environmental Review and Mitigation @ 5%				181,900		131,658
Program Implementation @ 5%				181,900		131,658
ROUNDED TOTAL ESTIMATED COST				**4,730,000**		**3,420,000**

PRELIMINARY IMPROVEMENTS

As discussed above, the preliminary improvements recommended for the Grant Line Channel watershed are those improvements included with Alternative 2. The improvements included with Alternative 2 are shown on Figures 16-6 and 16-8 and are also listed on Table 16-9. These improvements are considered preliminary. They are adequate for development of a Capital Improvement Plan, but the ultimate improvements will be defined from a more detailed design study and could vary from those recommended in SDMP.

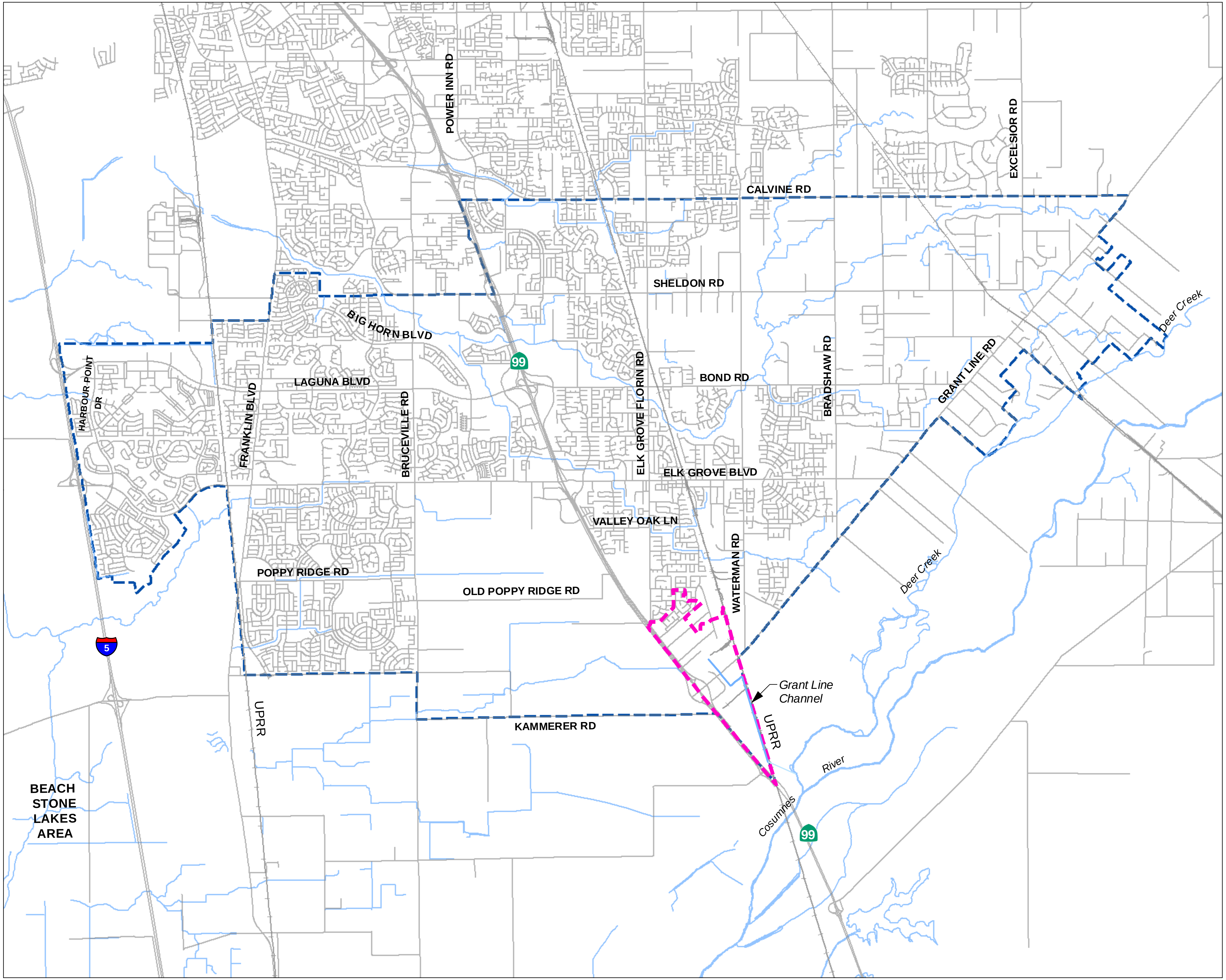


FIGURE 16-1

City of Elk Grove
Storm Drainage Master Plan
Volume II

GRANT LINE CHANNEL LOCATION MAP

N

0 0.5 1 2
Scale in Miles

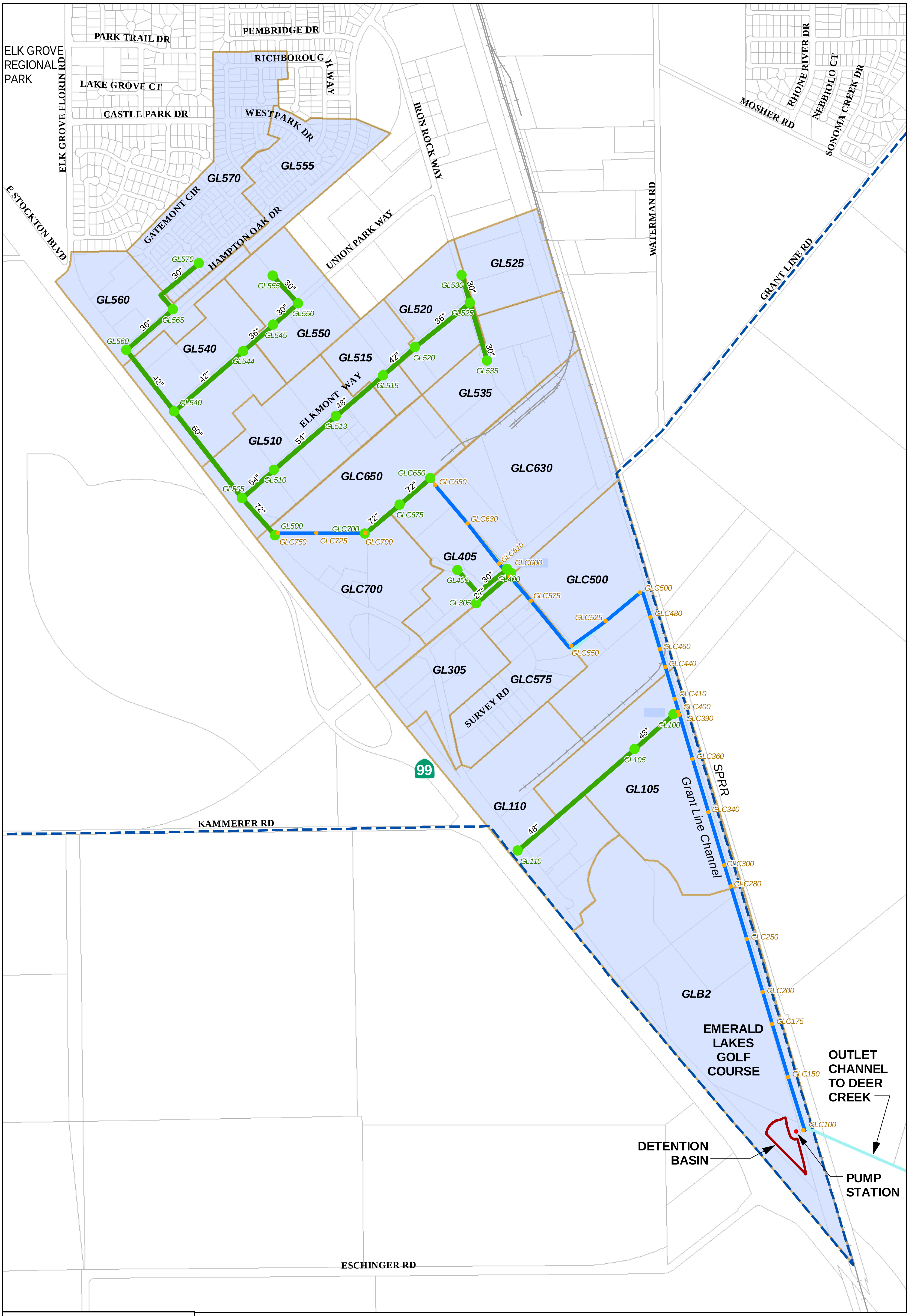
NOTES:

LEGEND:

City Limit

Grant Line Channel Watershed

WEST YOST ASSOCIATES
Consulting Engineers



LEGEND:

⬚	City Limit
36" ●	Modeled Pipeline and Node
●	Modeled Channel and Node
GL105	Grant Line Channel Subsheds

FIGURE 16-2
City of Elk Grove
Storm Drainage Master Plan
Volume II

GRANT LINE CHANNEL SUBSHEDS & MODELED FACILITIES

Figure 16-3
City of Elk Grove
Storm Drainage Master Plan Volume II
GRANT LINE CHANNEL
EXISTING DETENTION BASIN
& PUMP STATION

WEST YOST ASSOCIATES
Consulting Engineers

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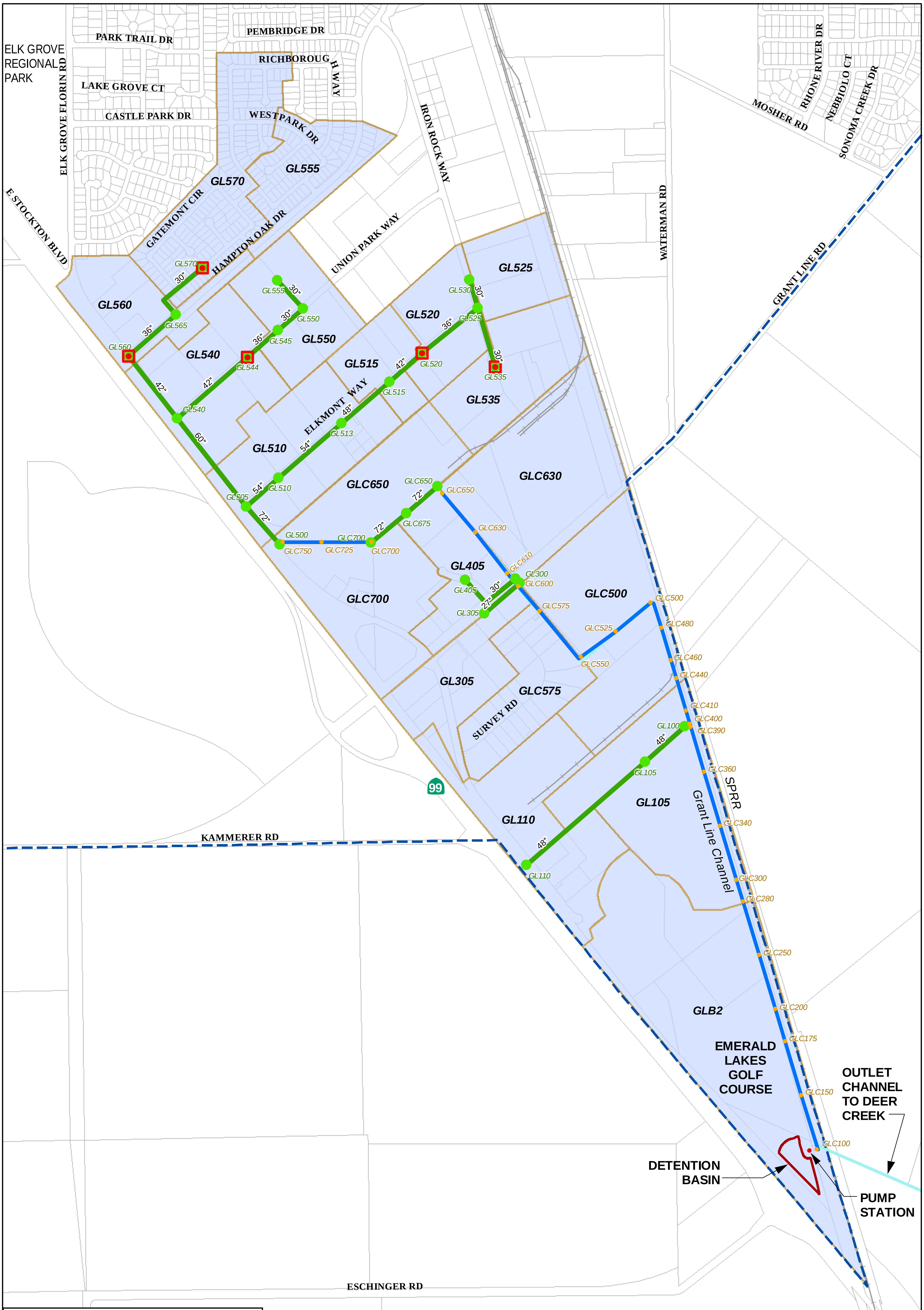

LEGEND:

City Limit

Node with Predicted 10-Year Street Flooding

36" Modeled Pipeline and Node

Modeled Channel and Node

GL105 Grant Line Channel Subsheds

N

0 800 1,600

Feet

FIGURE 16-4
City of Elk Grove
Storm Drainage Master Plan
Volume II

GRANT LINE CHANNEL PREDICTED STREET FLOODING LOCATIONS

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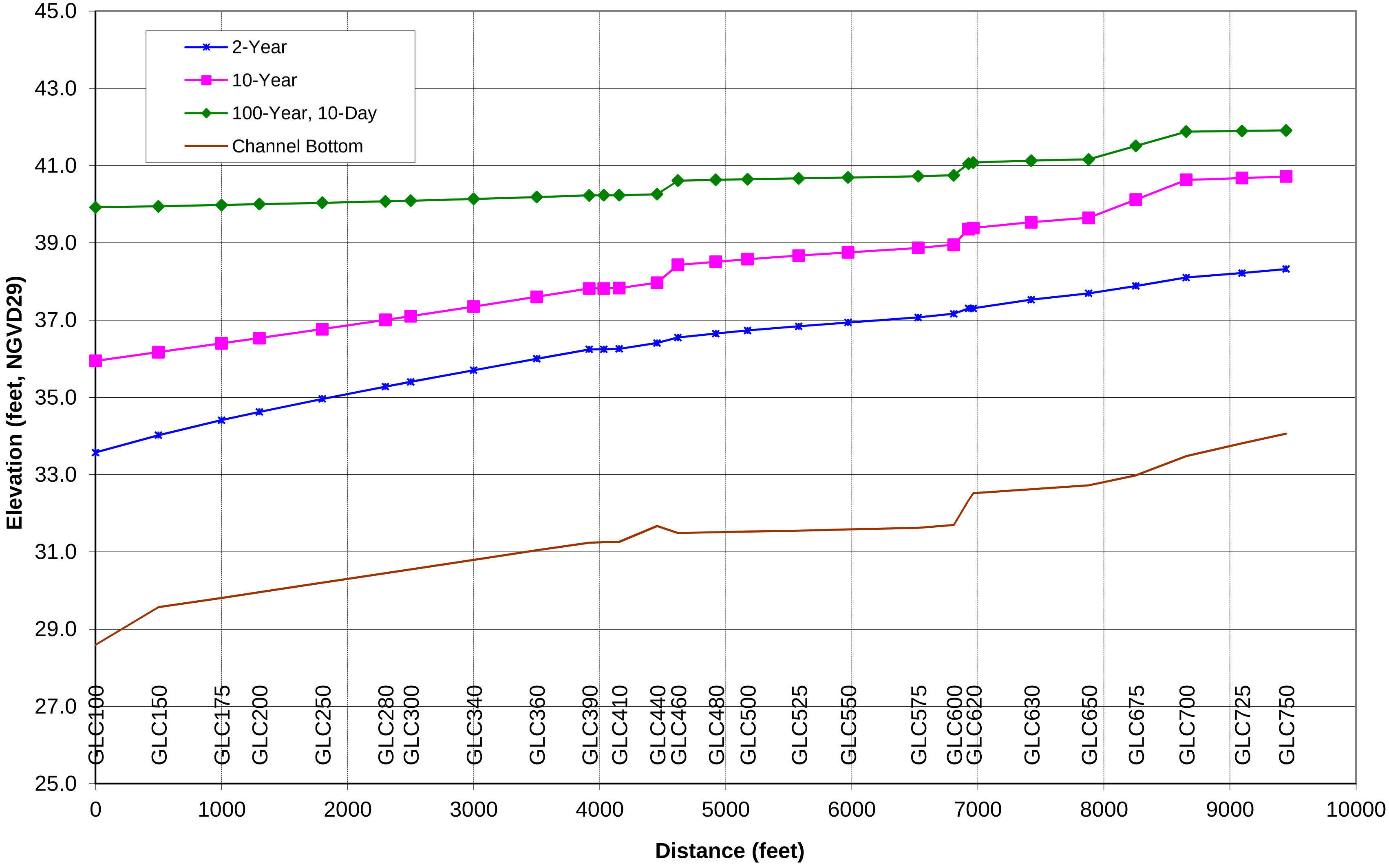
FIGURE 16-5
Grant Line Channel Existing Water Surface Profiles



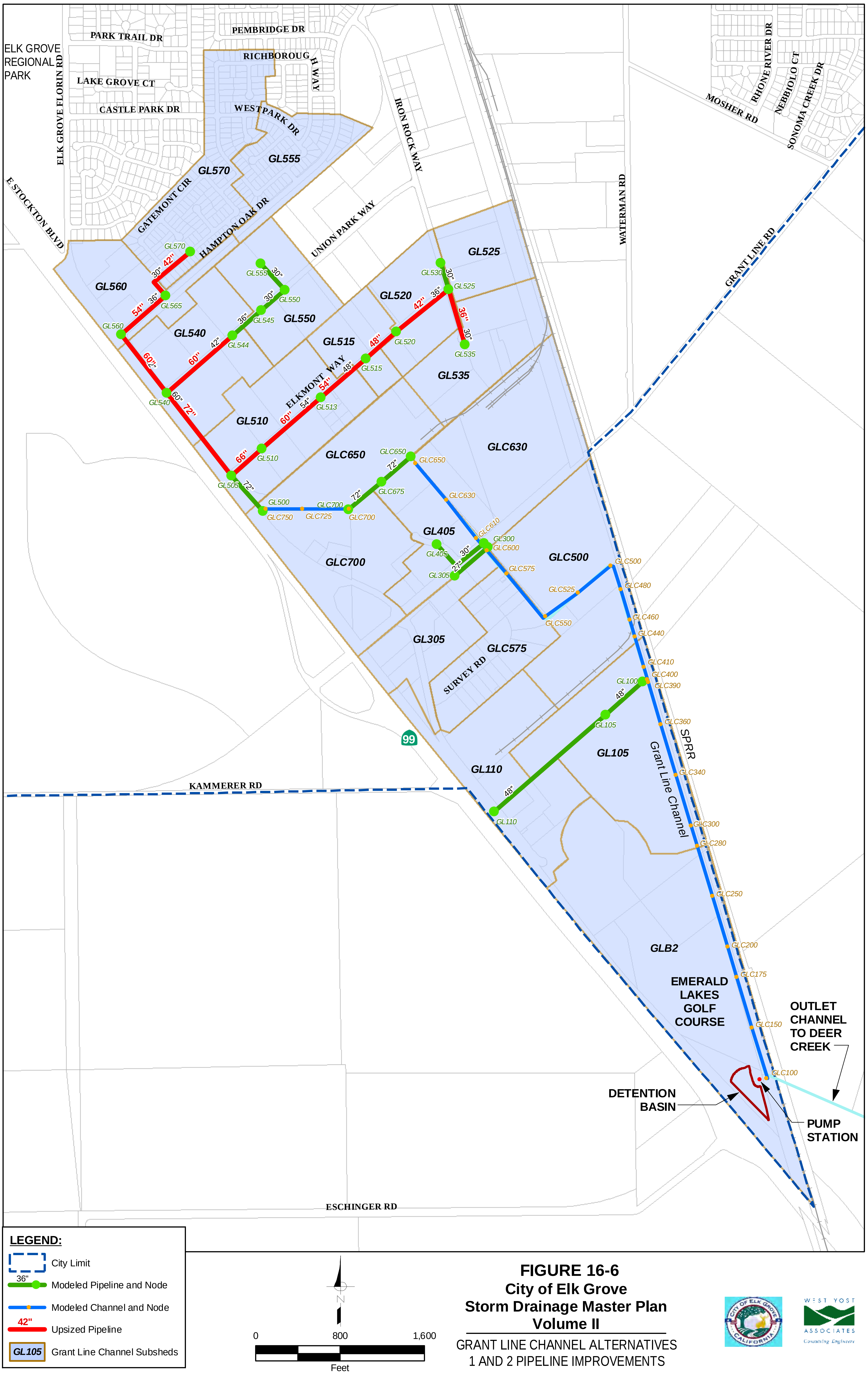


FIGURE 16-6

City of Elk Grove
Storm Drainage Master Plan
Volume II

GRANT LINE CHANNEL ALTERNATIVES
1 AND 2 PIPELINE IMPROVEMENTS

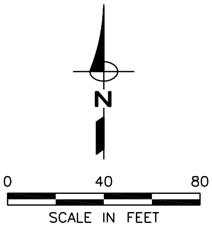


Figure 16-7
City of Elk Grove
Storm Drainage Master Plan Volume II
GRANT LINE CHANNEL
ALTERNATIVE 1 DETENTION BASIN
& PUMP STATION IMPROVEMENTS

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ASSOCIATES
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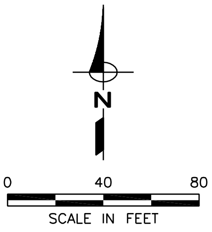

Figure 16-8
City of Elk Grove
Storm Drainage Master Plan Volume II
GRANT LINE CHANNEL ALTERNATIVE 2 DETENTION BASIN & PUMP STATION IMPROVEMENTS

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CHAPTER 17. DEER CREEK

WATERSHED DESCRIPTION

The Deer Creek watershed lies in the eastern part of the City and covers approximately 1,200 acres. The watershed includes most of the City that lies east of Grant Line Road (see Figure 17-1). This area is rural in nature and runoff is conveyed via small ditches and swales. Rural Residential land uses are anticipated for the entire watershed according to the General Plan.

WATERSHED ANALYSIS

A detailed analysis was not performed for this watershed. City staff reviewed the potential for development in this watershed and determined that approximately 160 acres may be developed in the future. City staff estimates that the future development areas will require two detention basins that will provide a total of 5 acre-feet of storage volume. The basins will be required to mitigate the potential increase in peak flows due to development. The locations of the two basins are not yet defined.

PRELIMINARY IMPROVEMENTS

The preliminary improvements recommended in the Deer Creek watershed are estimated to consist of the following:

- Provide two detention basins that provide a total storage volume of 5 acre-feet.

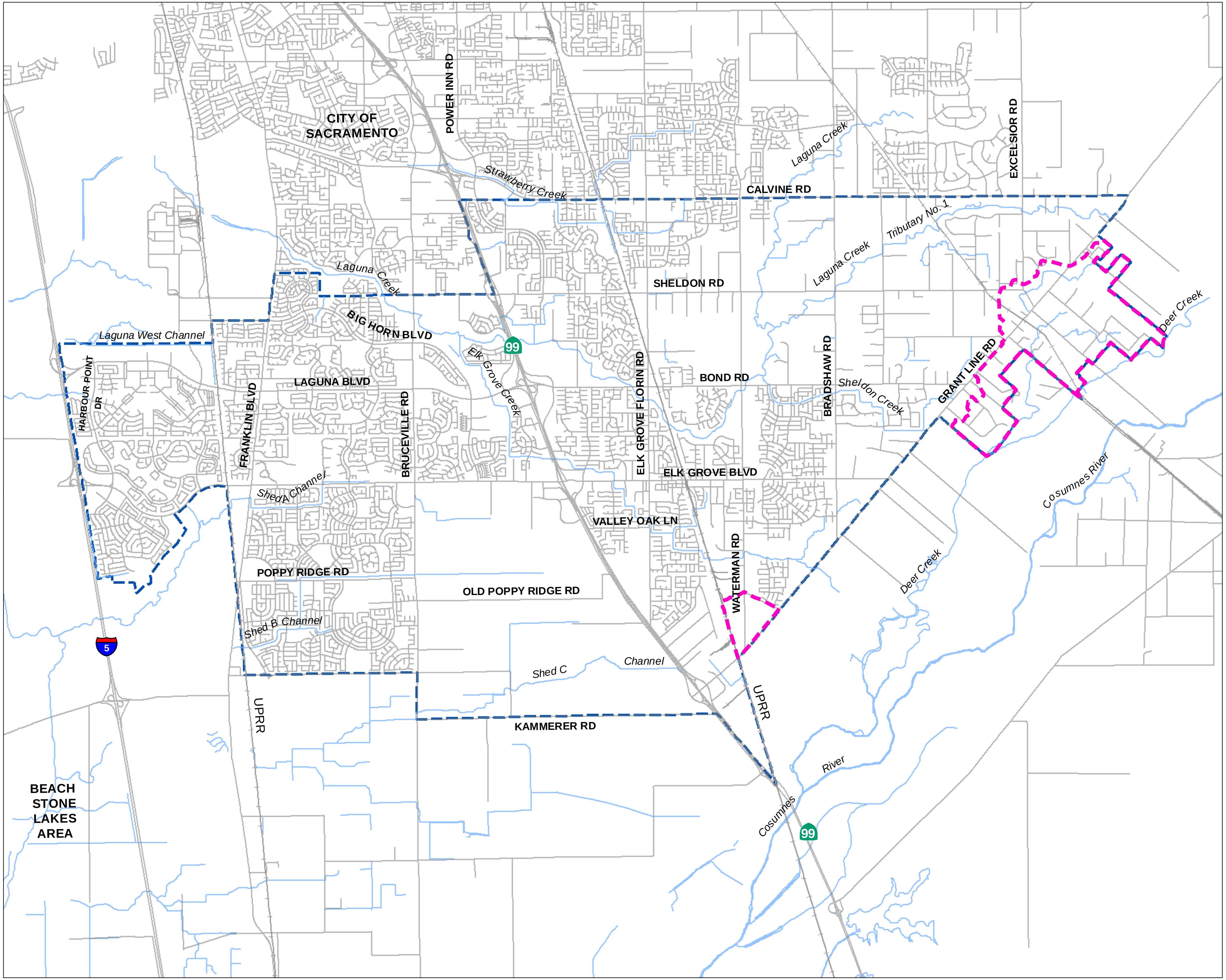
These improvements are considered preliminary. They are adequate for development of a Capital Improvement Plan, but the ultimate improvements will be defined from a more detailed design study and could vary from those recommended in SDMP.

FIGURE 17-1

City of Elk Grove
Storm Drainage Master Plan
Volume II

DEER CREEK
LOCATION MAP

0 0.5 1 2
Scale in Miles



NOTES:

LEGEND:

City Limit
Deer Creek Watershed

WEST YOST
ASSOCIATES
Consulting Engineers