Putah Creek / Dry Creek Subbasins

Drainage Report

CITY OF WINTERS CALIFORNIA

August 2005

Prepared By:

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WOOD RODGERS

September 9, 2005

Mr. Nicholas Ponticello, P.E.
City of Winters
c/o Ponticello Enterprises
1216 Fortna Avenue
Woodland, California 95776

Dear ~~Mr. Ponticello~~ Nick:

Subject: City of Winters, Moody Slough Subbasin and Putah/Dry Creek Subbasins Drainage Reports and Moody Slough and Putah Cree/Dry Creek Subbasins Drainage Allocation Report – Submittal of Final Reports

Enclosed are the final reports that were prepared by Wood Rodgers, Inc. for the City of Winters (City). These reports were prepared to guide the City in implementing drainage infrastructure improvements to accommodate planned development. The reports (10 copies each) are entitled as follows:

1. *Moody Slough Subbasin Drainage Report*, August 2005

2. *Putah Creek / Dry Creek Subbasins Drainage Report*, August 2005

3. *Moody Slough and Putah Creek / Dry Creek Subbasins Storm Drainage Cost Allocation Report*, August 2005

Please note that the models for the hydrologic and hydraulic analyses are not included in the Moody Slough and Putah Creek / Dry Creek subbasin reports. Two CD's, which contain the modeling information for each respective report, are enclosed with this transmittal for the City's use. Wood Rodgers has noted in the reports that copies of this information can be provided upon request from the City.

Wood Rodgers appreciates having the opportunity to assist the City with this assignment.

Sincerely,

Francis E. Borcalli, P.E.
Water Resources Department Manager

Enclosures: 10 Copies of Each Report
Two CD's

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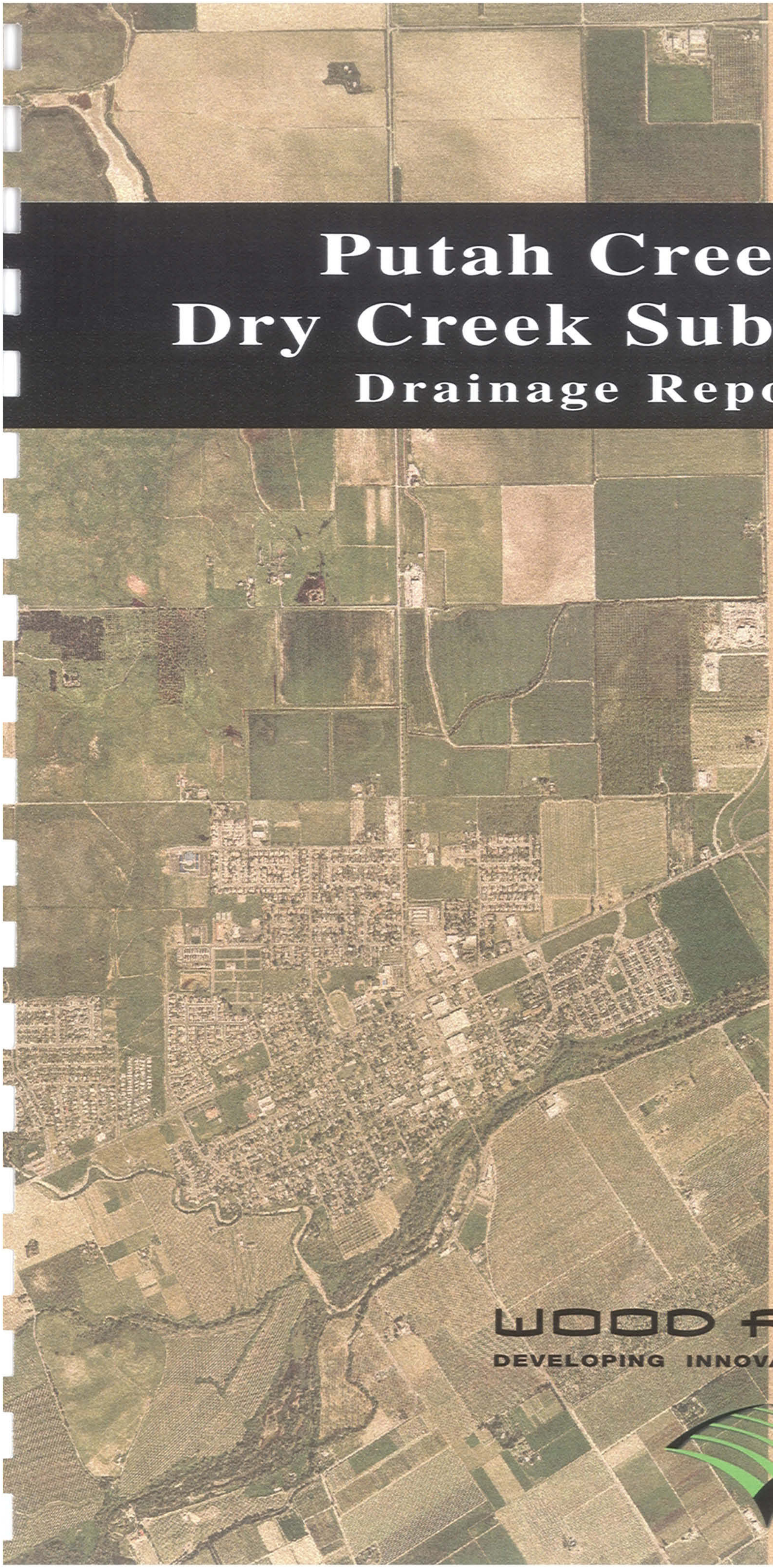
Putah Creek / Dry Creek Subbasins
Drainage Report



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INTRODUCTION

A. GENERAL

The City of Winters (City) is located north of Putah Creek and west of Interstate 505, within Yolo County, California. The City and the 2010 urban growth area are presented on Figure 1 (Plan Location).

There are three major drainage subbasins within the City. These include the Moody Slough subbasin, Putah Creek subbasin, and Dry Creek subbasin. The Putah Creek/Dry Creek subbasins consist of approximately 1.4 square miles.

The City's General Plan provides for development within the existing floodplain and across natural runoff corridors within the respective subbasins. Accordingly, several development projects have been proposed within and adjacent to the existing floodplain and runoff corridors. As part of the City's planning efforts to accommodate existing and planned growth, the City retained the services of Wood Rodgers, Inc., to develop a Drainage Report for the Moody Slough subbasin and for the Putah Creek/Dry Creek subbasins. A separate Drainage Report was prepared for the Moody Slough subbasin. This document pertains only to the Putah Creek/Dry Creek subbasins. The drainage facilities identified in the Storm Drainage Master Plan, adopted by the City on May 19, 1992, for the Putah Creek and Dry Creek subbasins, remain applicable.

B. PURPOSE

The purpose of this Drainage Report is to identify facilities to accommodate existing and planned development while mitigating the impacts to storm water runoff and flooding.

C. SCOPE

The scope of this Drainage Report includes the following tasks:

1. Evaluate existing drainage and flooding conditions within the Putah Creek/Dry Creek subbasins.

2. Identify regulatory agencies, policies, guidelines, and permitting requirements and develop storm drainage and surface water quality treatment design criteria and standards.

3. Identify cumulative drainage and flooding impacts for the Putah Creek/Dry Creek subbasins associated with ultimate development in accordance with the City's General Plan.

4. Identify phased drainage master planned facilities to mitigate increases to existing flooding problems and accommodate proposed development within the Putah Creek/Dry Creek subbasins.

D. PREVIOUS STUDIES

Several drainage studies were previously developed for Putah Creek/Dry Creek subbasins and surrounding areas. Various studies include:

- "Covell Drainage System Comprehensive Master Plan," prepared by Borcalli & Associates, Inc. 1993, for the Yolo County Flood Control & Water Conservation District (YCFC&WCD). This report includes a comprehensive evaluation of existing and proposed conditions for the Covell Drain, Willow Slough, and Dry Slough watersheds, which includes the Moody Slough subbasin.

- "Final Feasibility Report, Environmental Assessment/Initial Study, Winters and Vicinity, California," prepared by the U.S. Army Corps of Engineers (USACOE), February 1997. This study identifies improvements to reduce flood risk to existing development from Moody Slough. The proposed improvements include levees and a diversion channel from Moody Slough south to Putah Creek. The study includes a Levee/Channel Plan - Alternative 2, and a Levee/Channel Locally Preferred Plan - Alternative 3, which differ in the capacity of the diversion channel. However, due to lack of funding, the project identified by the USACOE has not been constructed.

- "Winters North Area Flood Control Study," prepared by Nolte & Associates, May 1993. This study identifies improvements to accommodate a proposed development in the City's north area within the Moody Slough subbasin. The north area development and proposed drainage improvements identified in the Nolte study also have not been constructed.

- "City of Winters Storm Drainage Master Plan," prepared by CH2M Hill, May 19, 1992, which includes drainage master planning for the City. The CH2M Hill study does not include current existing conditions or development proposals (Appendix A).

- County of Yolo, Department of Public Works and Transportation, "Davis-Winters Drainage Report, Chickahominy-Dry Slough Drainage Complex-Drainage Report," March 1986. This report identifies various projects to control flooding in the watershed area bounded by County Road 29 on the north, Putah Creek on the south, the Winters hills on the west, and the Yolo Bypass on the east.

- U.S. Department of Agriculture, Soil Conservation Service (SCS), "Chickahominy-Moody Slough Watershed - Investigation of Flood Problems," January 1982. This study evaluates several measures and projects to reduce the extent of flooding in the Chickahominy-Dry Slough watershed.

- U.S. Department of Agriculture, SCS, "Chickahominy-Moody Slough Watershed," State's Report to Steering Committee, January 17, 1980. This report delineates the estimated 100-year floodplain and identifies costs for various alternatives to reduce flooding in the watershed.

DESCRIPTION OF REGULATORY AGENCIES, POLICIES, AND GUIDELINES

The waterways discussed in this report come under the jurisdiction of federal, state, regional, and local regulatory agencies. Some of the more significant policies and guidelines related to drainage and flooding for each regulatory agency are discussed below.

A. LOCAL

City of Winters

The City is responsible for reviewing and approving development proposals within the City. The City's General Plan contains specific goals, policies, and implementation programs intended to minimize the potential impacts associated with drainage and flooding hazards. The respective goals, policies, and implementation program related to storm drainage are presented below:

Goal I.A:

To provide for orderly, well-planned, and balanced growth consistent with the limits imposed by the City's infrastructure and service capabilities and by the City's ability to assimilate new growth.

Policies:

I.A.9. *No new development may occur within the flood-overlay area shown in Figure II-1, until a feasibility and design study for a comprehensive solution to the 100-year flooding problem has been completed and a fee schedule has been established or financing program adopted which includes all affected and contributing properties for financing the comprehensive flood control solution.*

Goal IV.A:

To maintain an adequate level of service in the Winters' public facilities and services to meet the needs of existing future development.

Policies:

IV.A.1. The City shall ensure, insofar as possible, that public facilities and services are developed and operational, as they are needed to serve new development.

IV.A.2. The City shall regularly monitor current levels of service in Winters' public facilities and services.

IV.A.3. The City shall ensure through capital facility planning and budgeting and through review of private development projects that City-adopted level of service standards are maintained.

IV.A.4. The City shall ensure through a combination of development fees and other funding mechanisms that new development pays its fair share of the costs of developing new facilities and services. The City at its sole discretion may allow developers to construct needed improvements according to City specification in lieu of paying development fees for such improvements.

IV.A.5. The City shall ensure through a combination of assessment districts, utility user taxes, and other funding mechanisms that adequate funding is available for the improvement, operation, and maintenance of public facilities and services.

Goal IV.D:

To maintain an adequate level of service in the City's storm drainage system to accommodate runoff from existing and future development and to prevent property damage due to flooding.

Policies:

IV.D.1. The City shall maintain a regular program for replacing and upgrading older and undersized storm drains.

IV.D.2. The City shall expand and develop storm drainage facilities to accommodate the needs of existing and planned development.

IV.D.3. *The City shall determine the feasibility of developing a recreational lake in conjunction with development of the North Area to serve as a detention facility, designed to accommodate all storm water runoff from the North Area.*

IV.D.4. *The City, in cooperation with property owners, developers, and the Yolo County Flood Control and Water Conservation District shall undertake a feasibility and design study for a comprehensive solution to the flooding problems associated with Chickahominy and Moody Sloughs. The comprehensive solution may include such features as diversion to Putah Creek, diversion under I-505, detention ponds, changes in land use designations, elevating building pads, and structural flood proofing as deemed effective and cost effective. As a condition to any development entitlement approval, all development affected by or contributing to the 100-year flooding problem shall be required to contribute to the financing of the comprehensive flood control solution in an amount that reflects that property's relative contribution to the flooding problem or benefit from the program adopted.*

IV.D.5. *Future drainage system discharges, including discharges into Putah Creek, shall comply with applicable state and federal pollutant discharge requirements.*

In May 1992, the City adopted the Rancho Arroyo Drainage Shed Ordinance 96-02, which identifies policies, standards, and fees associated with drainage and flooding for development within the Rancho Arroyo drainage shed.

It is the policy of the City to protect all new habitable structures from the 100-year (one percent) flood event.

Where other public agencies assert jurisdiction over aspects of drainage improvements required by the City, approval would be provided by such jurisdictions prior to issuing permits or approving improvement plans.

The City is a participant in the National Flood Insurance Program (NFIP), and all development within the City would comply with the policies and guidelines of the NFIP. As the local administrator for the NFIP, the City is responsible for processing revisions to

Flood Insurance Rate Maps (FIRMs) through the Federal Emergency Management Agency (FEMA).

During the planning process of a development, phasing scenarios are developed based upon the best available information. However, actual development phasing can vary significantly due to the many factors that influence the type, rate, and location of development. The City is the entity responsible for ensuring the integrity of Specific Plans and for ensuring the integrity of the proposed drainage facilities is maintained.

Relationship of the Specific Plan to the General Plan

All state-mandated general plan elements are included in the City's General Plan, as approved by the City Council in 1992.

County of Yolo

Local Agency Formation Committee (LAFCO) – All annexations of land into the City require the approval of LAFCO.

Department of Public Works – The crossing of county roads with drainage facilities requires encroachment permits from the county, as well as design review.

B. REGIONAL

YCFC&WCD

At the request of the Yolo County Board of Supervisors, in 1951 the California Legislature created the YCFC&WCD as an independent Special District. The primary purpose of the YCFC&WCD was to seek new water sources and manage these sources efficiently. YCFCWCD's boundaries cover 195,000 acres in Yolo County, which includes the cities of Woodland, Davis, and Winters, as well as the towns of Capay, Esparto, Madison, and other small communities within the Capay Valley. Currently, the YCFC&WCD owns and operates two dams and reservoirs with hydroelectric plants, a diversion structure on Cache Creek, and more than 150 miles of canals and laterals to deliver irrigation water.

To the extent improvements or modifications are proposed that affect YCFCWCD's facilities, YCFCWCD is to be kept informed throughout the planning process, and such improvements or modifications are to be planned and designed in coordination with YCFC&WCD. YCFCWCD's approval is required in advance of implementing any modifications.

C. STATE

State Reclamation Board

The State Reclamation Board has jurisdiction over features of the Sacramento River Flood Control Project, including Putah Creek, which has a designated floodway. Thus, the State Reclamation Board may require an Encroachment Permit for projects affecting the channel or discharges into Putah Creek.

State Water Resources Control Board

The State Water Resources Control Board (SWRCB) has jurisdiction for permitting and licensing the use of surface water, as well as an enforcement responsibility. Changes to drainage patterns that may result in significant changes to existing water rights should be reviewed with the SWRCB.

Regional Water Quality Control Board

The Regional Water Quality Control Board is responsible for administering permits for discharges regulated by a Clean Water Act Permit issued under the National Pollutant Discharge Elimination System (NPDES). The City is not a medium or large municipality, and thus not included in the first phase of the U.S. EPA's storm water program. With the growth and densities proposed, the state could designate the City as a small municipality requiring a NPDES municipal storm water permit.

D. FEDERAL

FEMA

FEMA is the Federal Administrator of the NFIP. Flood Insurance Studies and FIRMs, prepared by FEMA, show inundation areas and depths for potential flooding. FEMA has published standards and criteria in a document entitled, "Guidelines and Specifications for Flood Hazard Mapping Partners," February 2002. The FEMA FIRM for the City is Community Panel Number 060425 0001 C, dated November 20, 1998. The area within Yolo County that is adjacent to the City is presented on Community Panel Number 060423 0540 C, dated March 23, 1999.

The City is a Floodplain Administrator for FEMA's National Flood Insurance Program.

As developments are proposed and constructed, FEMA is responsible for reviewing submitted changes and for issuing revisions to FIRMs, through Conditional Letters of Map Revision (CLOMRs) and Letters of Map Revision (LOMRs), as requested by the City. The City can condition any new development to provide sufficient evidence and prepare all LOMRs and CLOMRs at the discretion of the City.

IDENTIFICATION OF REGULATORY PERMITTING REQUIREMENTS

To the extent that improvements are required at existing waters of the United States, the following permits may be required:

- U.S. Army Corps of Engineers 401 and 404 Permits – 401 and 404 Permits are required to direct storm water discharge and for construction of facilities in Putah Creek.

- Clean Water Act, NPDES Permit – The NPDES storm water permitting program is administered by the SWRCB through regional water quality control boards. Municipalities with storm systems serving a population of less than 10,000 are not required to obtain a NPDES Permit. A NPDES Permit may be required for construction associated with projects that exceed five acres.

- State Reclamation Board Encroachment Permit – An Encroachment Permit from the State Reclamation Board may be required for discharge and construction of facilities in Putah Creek.

- California Department of Fish and Game Streambed Alteration Permit – A Section 1601 or Streambed Alteration Permit is required for construction-related activities affecting Putah Creek.

- Caltrans/Yolo County Encroachment Permits – The construction of new conveyance facilities requires modifying existing or constructing new structures at Highway 128 and various county roads. Accordingly, encroachment permits from Caltrans and Yolo County are required.

FORMULATION OF STORM DRAINAGE AND SURFACE WATER QUALITY TREATMENT DESIGN CRITERIA AND STANDARDS

Wood Rodgers gathered and evaluated information regarding historic and current hydrologic methodologies, data, and design standards used within the region. Based upon this review, Wood Rodgers developed design criteria and standards for flood control and surface water quality treatment for incorporation into the revised Winters Design Standards.

Since different types of drainage facilities serve various purposes that may require differing levels of flood protection, water quality treatment, and/or maintenance and operation, it is appropriate to define the various types of facilities. The definitions established for the City include the following two categories:

1. *Type 1 Drainage Facilities – Runoff corridors, channels, culverts associated with channels, bridges, detention ponds, pump stations, and levees*

2. *Type 2 Drainage Facilities – Roadside ditches, storm drainage pipe systems, and overland conveyance systems*

A minimum 100-year design storm frequency shall be used to design Type 1 facilities. A minimum 10-year design storm frequency shall be used to design Type 2 facilities. Developing drainage master plans on a drainage basin basis would ensure existing and proposed drainage facilities meet the immediate and long-term goals of the community. Drainage master plans should identify drainage facilities that accommodate existing and planned future land use within the drainage basin. Although the phasing of development is not known with certainty, it is important to maintain the integrity of the proposed drainage system presented in the Drainage Report as development occurs. Frequently, the phasing of a development is not necessarily consistent with the most economical plan for phasing drainage infrastructure; however, it may be the most financially feasible.

The design standards and criteria developed for this report are intended to be acceptable to all parties with jurisdiction over drainage and flood control for the area.

Additionally, the revised Winters Design Standards may require peer review of complex storm drain systems, implementing the recommended improvements if designed by the developer's consultant and not by the City's consultant.

A. TYPE 1 DRAINAGE FACILITIES

Type 1 drainage facilities include conveyance, flood protection, water quality treatment, and recreational, environmental, and aesthetic elements, which may consist of channels, culverts associated with channels, bridges, detention ponds, pump stations, and levees. Type 1 drainage facilities should meet objectives consistent with the City's General Plan. In most cases, an analysis of the 100-year and 10-year storm events would provide the information necessary to design and evaluate the existing and proposed drainage system. The duration of the storms used in the analysis should represent the worst-case flooding scenarios with respect to peak flow and peak volume. The facility's design shall be evaluated under a 200-year storm to determine how sensitive the level of protection is to the basic criteria.

Hydrology-Design Flow

Within the area, YCFCWCD's hydrologic model developed for the Willow Slough, Dry Slough, and Covell Drain drainages has been widely used to evaluate existing drainage and flooding patterns for flood insurance studies and to design regional drainage facilities. The model utilizes HEC-1, a computer model developed by the USACOE, which is applied throughout the United States and other countries. HEC-1 is a valuable tool used to calculate, route, and combine runoff hydrographs.

For the evaluation and design of Type 1 and Type 2 drainage facilities within the City, the modeling methods presented in Table 1 shall apply.

Synthetic Unit Hydrographs – Synthetic unit hydrographs shall be generated using the SCS dimensionless unit hydrograph method.

Antecedent Moisture Content (AMC) – The AMC is based upon the condition of the soil prior to the modeled storm event occurring. Presented in Table 2 are the ways the AMC would vary with storm frequency. These values were based upon information developed

for the "Covell Drainage System Comprehensive Drainage Plan, WMP-93-01-3," September 1993.

Soil Conservation Service Curve Numbers – The SCS Curve Number (CN) is based upon land use soil type and AMC. For CN values between an AMC I, AMC II, or AMC III, the CN would be interpolated. Based upon SCS Technical Release 55 (June 1986), presented in Table 3 are the CNs for each land use type for a 24-hour storm for AMC II. The CN shall be adjusted from AMC II values, if necessary, using Table 4. Refer to Table 2, if necessary, for the storm recurrence/AMC correlation. The CN shall be adjusted again for storm durations other than 24 hours in accordance with the National Engineering Handbook, Section 4 (NEH4) and SCS Technical Release 60 (TR60). Presented in Table 4 are the adjusted CNs for a 10-day storm. Within NEH4, Table 10-1 can be used to correlate CN values for all AMC values once one AMC condition is known.

Precipitation – As part of the "Covell Drainage System Comprehensive Drainage Plan," in 1993, Mr. James D. Goodridge prepared design storm information for Yolo and Solano counties. This information is included as Appendix B.

Base Flow – The base flow is assumed to be 1 cfs/sq/mile.

Water Quality Treatment Volume

Storm water runoff carries with it many pollutants in varying concentrations that are suspended and/or dissolved in the runoff. As property is developed, Best Management Practices (BMPs) provide an opportunity to reduce the loading of pollutants to receiving waters.

Storm water runoff would normally convey a disproportionate loading of pollutants in the initial period of runoff during a storm event. This initial period is usually the most critical and is commonly referred to as the "first flush." The "first flush" contaminants most frequently associated with storm water include sediment, nutrients, bacteria, oxygen demanding substances, oil and grease, heavy metals, other toxic chemicals, and floatables.

Detention ponds can include water quality treatment elements to minimize potential impacts to the quality of surface runoff entering receiving waters. The State of California developed a method to determine the optimum volume of storage for water quality detention ponds according to given impervious acreage of a drainage area. These methods are applicable within the City. The report entitled, "California Storm Best Management Practices Handbooks," describes the analyses that establish the methods and criteria acceptable for water quality facilities. The mean storm event for the City's area is 0.55 inch (obtained from the California Storm Water Best Management Practices Handbooks). Dry and wet ponds can be used to provide water quality treatment.

Detention Ponds

Detention ponds would have a minimum of one foot of freeboard in a 100-year storm event. Ponds would include a minimum 20-foot perimeter buffer with an all-weather access road. The access road would allow an adequate turning radius for maintenance vehicles. Ramps to the bottom of the pond with 10 percent maximum slope would be provided. The side slopes of the ponds would be 3:1, or flatter, eliminating the need for safety fencing. To the extent practical, the depths of the ponds would be designed to minimize groundwater seepage into the ponds. For wet ponds, a minimum pool depth of three feet is required to inhibit the growth of cattails, which is desirable from a maintenance standpoint. Depending upon the particular pond and groundwater levels, the summertime pond level can be allowed higher since flood control storage is not required.

For detention ponds that incorporate lake features, a lake/wetlands consultant shall be retained to provide detailed information regarding the operation and maintenance elements of the entire lake facility.

Pump Stations

To the extent possible, gravity systems are preferred over systems that rely on storm drainage pumping. Pump stations would be designed to discharge the design capacity using a minimum of two mixed-flow vertical pump and motor units. A minimum of one additional pump and motor unit of equal size would be included as a backup. An attempt would be made to control the outflow from pump stations for storm events equal to and less than the 100-year storm event by staggering the "set point" for initiating pump operation, to

provide a reasonable downstream flow pattern similar to existing conditions. For example, if a pump station needs four pumps to deliver the 100-year design flow to avoid having the 100-year discharge occurring during small storm events, each pump would be set to begin operating based upon a predetermined schedule according to pond water level.

A low-flow pump would be included in the design of the pump station to discharge runoff occurring during the summer months.

The pump station sump would be sized according to the "Hydraulic Institute Standards for Centrifugal, Rotary, and Reciprocating Pumps." Storm water would be conveyed from the detention pond into the sump through an open inlet section. Before entering the pump vault, the storm water would pass through a power-driven catenary trash rack system. The invert of each sump would be lower than the invert of the pond or intake channel so the detention ponds can be completely dewatered to facilitate maintenance.

Typically, each pump would discharge into a separate pipe that includes a combined siphon breaker and air relief valve and vault at the high point on the discharge pipe, and a flap gate with headwall at the terminal structure in the drain. Where discharge lines tend to be long (over 200 feet), or where the discharge line must cross under existing drains, roads, or railroads, the discharge line would be manifolded to discharge through a single pipeline. Electrical control equipment would be enclosed in a prefabricated metal or concrete block building on a concrete foundation with minimum outside dimensions 8 feet wide by 20 feet long. The electrical equipment would include pump controls, water level detection system, float switch for sump high-water level alarm and low-level automatic shutoff, solenoid-controlled automatic pump motor oiler, and telemetry system. The type of pump controls and telemetry system would be uniform throughout the City. In addition, the building would be equipped with two doors, wall louvers, rotary turbine roof vent, interior and exterior lighting, and a space heater.

On-site diesel generators would provide back-up power for each pump station. Each generator would be sized to supply power to the drainage pumps running at design capacity, as well as to the electrical control equipment, lighting, and electrical building space heater. The generators would be radiator-cooled and skid-mounted, and would include a heater, batteries, battery charger, control panel with auto-start, critical silencer, and generator circuit breaker. The diesel generator and fuel storage tank would be placed

on a concrete pad. The fuel storage tank would also be provided with a secondary containment structure. The pump station site would be enclosed with a 6-foot-high chain link fence topped with three strands of barbed wire. The fencing would include a 20-foot-wide, electrically operated double gate and a 4-foot-wide pedestrian gate. The pump station lot would be sized and the sump, electrical control building, diesel generator, and transformer arranged to allow adequate operating space for vehicles, pump, and motor removal equipment, and maintenance of the trash rack system. The paved access yard would be at a minimum elevation of two feet above the 100-year water surface elevation, and would be sloped to provide adequate on-site drainage.

Open Channels, Culverts Associated with Open Channels, and Bridges

Open channels, including runoff corridors, would have 3:1 side slopes, or flatter. For open channel design, a Manning's "n" roughness factor would be used to account for vegetation to minimize maintenance requirements as presented in Table 5, Roughness Coefficients (n). All-weather access roads for maintenance would be provided adjacent to open channels and would be a minimum of 15 feet wide. A minimum of one foot of freeboard for the 100-year storm event would be provided for open channels, culverts, and bridges. In areas where fill is required to provide freeboard for open channels, one foot of freeboard for the 100-year storm event would be provided.

The centerline curve radius of an open channel shall be a minimum of twice the bottom width, or 35 feet, whichever is greater.

Levees

Levees would be designed in accordance with FEMA criteria and as stipulated in the Code of Federal Regulations, Title 44, Part 65. Levees are a constructed flood control feature and must meet the FEMA requirements related to design, material, compaction, and structural/geotechnical criteria. A minimum of three feet of freeboard for the 100-year storm event would be provided. Adequate width at the top and toe of the levee would be provided for maintenance. A 15-foot all-weather maintenance road would be provided.

HEC-1 and HEC-HMS Modeling

The HEC-1 and HEC-HMS computer program may be used to compute and route runoff hydrographs. The results may be used to design open channels, major road crossings, detention ponds, etc. The criteria that would be used to develop the HEC-1 and HEC-HMS models are presented in this section.

Prepare Basic Information – Lay out the proposed storm sewer system and delineate the subbasins tributary to points of concentration for the design of inlets, junctions, pipelines, etc. Delineate the land uses and hydrologic soil groups within each subbasin.

Design Capacities – Drainage facilities shall be designed to accommodate the future development of the entire upstream watershed. The future development shall be defined as full build-out of the General Plan Land Use Designations.

The capacity design criteria for storm facilities are as follows:

Pipelines – Pipelines shall be designed to convey the 10-year, 24-hour flood event while maintaining the hydraulic grade line at least one foot below the elevation of inlet grates and manhole covers.

Open Channel – Open channels shall be designed to convey the 100-year, 24-hour flood event while maintaining at least one foot of freeboard in cut sections and FEMA freeboard in leveed sections.

Bridges – Bridges shall be designed to pass the 100-year, 24-hour flood event while maintaining a minimum of one foot of freeboard to the low chord.

Culverts – Culverts shall be designed to pass the channel design capacity while meeting freeboard requirements.

Storage Facilities – Storage facilities, where volume rather than peak flow generally governs the size, shall be designed to contain or attenuate a 100-year, 10-day storm event, while maintaining at least one foot of freeboard in the pond and without creating excessive backwater effects on the tributary storm drainage system.

Storm Frequency – The frequency of the design storm used would vary by the type and size of the facility.

Storm Duration – The storm duration shall be greater than the lag time or time of concentration for the entire watershed. Long-duration storms, 36 hours, 5- and 10-day events shall be evaluated, as appropriate, where runoff volume rather than peak discharge is of importance.

Rainfall Depth-Duration-Frequency – The depth-duration-frequency information shall be obtained using data in Appendix B, and based upon a mean annual precipitation of 21 inches.

Storm Distribution – A balanced storm distribution shall be modeled using the PH records in the HEC-1 model.

Computation Time Interval – The computation time interval, which is used in the IT records of the HEC-1 program, shall be computed by dividing the shortest subbasin lag time or time of concentration by 5.5. This calculated value should be rounded down to the closest 5, 10, 15, or 30 minutes; or 1, 2, 3, or 6 hours. If the calculated value is less than five minutes (a lag time of less than 33 minutes), it should be rounded down to the nearest minute.

HEC-1 uses a number of computation intervals in conjunction with a computation time interval to define the duration of simulation.

The number of computation intervals to use in the IT records of the HEC-1 program shall be computed as:

$$\text{Number of Computation Intervals} >= \frac{\text{Storm Duration} + \text{Basin Lag or } T_c}{\text{Computation Interval}}$$

For design considerations where runoff volume rather than peak discharge is of importance, the number of computation intervals should be large enough so the final hydrograph ordinates on the receding limb of the hydrograph are close to zero.

Initial Losses – There is a correlation between the recurrence frequency of a storm and the initial loss. Calibration modeling with HEC-1 in the Sacramento area has shown that higher initial losses were appropriate for the more frequent events. Initial losses are presented in Table 6. The correlation of AMC to storm frequency and the use of the CN method is another acceptable means of accommodating initial losses.

Constant Losses – The constant loss is an infiltration rate in inches per hour based upon the infiltration rate of saturated soil. The infiltration potential is dependent upon the soil type and land use. Average infiltration rates for combinations of hydrologic soil type and land use designations for the City are presented in Table 7.

The Synthetic Urban Unit Hydrograph – The U.S. Bureau of Reclamation's (USBR) dimensionless urban unit hydrograph would be used to calculate runoff. The urban unit hydrograph was developed based upon many urban watersheds throughout the United States. The applicability of the unit hydrograph in Sacramento County was confirmed by successful comparisons of recorded runoff for several drainage basins and storms with the runoff calculated using the urban unit hydrograph. Due to similar hydrologic conditions, it is also applicable to the City.

Lag Time – The temporal distribution of the unit hydrograph is a function of the basin lag time. The lag time would be calculated by using one of two methods. Basin "n" lag method, or travel time component method. Selecting the method depends upon the available information and the purpose of the runoff analysis.

Unit Duration – The unit duration used in the IT records of the HEC-1 program is the incremental period of time for which hydrograph ordinates are calculated. The unit duration should be approximately the lag time divided by 5.5, to provide adequate definition of the runoff hydrograph.

Calculation Procedure – The procedure below outlines the steps used to compute an urban unit hydrograph.

Computing Urban Unit Hydrographs	
Step	**Description**
1	Determine basin lag time (hrs) and area (sq mi).
2	Determine unit duration (hrs).
3	Calculate Lag Time + Unit Duration/2.
4	Calculate volume of runoff resulting from one inch of rainfall on basin areas, in one-day cfs. V = Basin area x 26.89 The conversion factor, 26.89, is used to convert one inch of rainfall excess to over one square mile in 24 hours to runoff expressed in one-day cfs.
5	Calculate unit hydrograph time steps as percent of Lag + Unit Duration/2, up to 600 percent.
6	Determine dimensionless synthetic unit hydrograph ordinates from Table 8.
7	Calculate unit hydrograph ordinates by multiplying V from Step 4 by dimensionless synthetic unit graph ordinates in Step 6. The ordinates in Step 7 are in cubic feet per second as a result of one inch of rainfall over the basin. To obtain ordinates as a result of any other rainfall depth, multiply by the rainfall depth, in inches.

The spreadsheet "uh_winter.xls" generates unit hydrographs for drainage basins based upon the urban unit hydrograph, the basin area, and the basin lag (Appendix B). The unit hydrograph ordinates are entered on the UI records. These are used as input to HEC-1, which calculates runoff hydrographs based upon the effective precipitation over the basin.

Base Flow – Base flow is considered the normal day-to-day flow from groundwater, spring contributions, or even from landscaping runoff. A study of the Sacramento area determined that base flow is not significant for most drainage studies. Base flow would be included as 1 cfs/square mile.

Basin Lag – The lag time of a basin is required to calculate runoff hydrographs. Two methods would be permitted to calculate basin lag, the Basin "n" method and the travel time component method. Both methods may be used in any given multi-basin model. This section covers the recommended applications and the equations for each method. The spreadsheet "lagwint.xls" assists the user in calculating the basin lag time (Appendix C).

Basin "n" Method – The Basin "n" method of computing lag should be used for:

- Planning level analyses.
- Basins with limited conveyance systems.

The Basin "n" lag equation, which was originally developed by Snyder and later revised by the USACOE and the USBR, is expressed as:

$$L_g = C \cdot n \left[\frac{L \cdot L_c}{S^{0.5}} \right]^{0.33}$$

Where:

C = 1560 (174);
L_g = lag time, min (sec);
L = length of longest watercourse, measured as approximately 90 percent of the distance from the point of interest to the headwater divide of the basin, miles (m);
L_c = length along the longest watercourse measured upstream from the point of interest to a point close to the centroid of the basin, miles (m);
S = overall slope of the longest watercourse between the headwaters and concentration point, ft/mile (m/m); and
n = basin "n" (Table 9).

The basin "n" value is dependent upon the basin land use and the condition of the main drainage course. For basins with mixed land use and/or varying characteristics of the main drainage course, the basin "n" should be weighted for the areas draining to each type of channel development. Presented in Table 9 are recommended basin "n" values. The shaded values in Table 9 are normally not used. However, these values may be used for planning purposes to estimate the effect of channelization, or to estimate a composite "n" for large areas with mixed land use channelization.

Travel Time Component Method – The travel time component method of computing basin lag should be used for the following applications:

- Detailed conveyance system design.
- Runoff analyses of existing conveyance systems.

The travel time is the time required for runoff to flow from the most upstream point of the drainage area through the conveyance system to the point of interest. The travel time is calculated by dividing the length of the conveyance system component by the corresponding velocity of flow. The travel time, T_c, is computed as follows:

$$T_c = T_o + T_g + T_p + T_{ch}$$

Where:

T_o = overland flow time of concentration;
T_g = gutter flow travel time;
T_p = pipe flow travel time; and
T_{ch} = channel flow travel time.

The equation used to compute the travel time for each conveyance component is described below.

Overland Flow - The developed Kinematic wave empirical equation based upon available SCS, USACOE, and Federal Highways Administration (FHA) overland flow data (Sacramento City/County, 1996) is:

$$T_o = \frac{0.66 L^{0.50} n^{0.52}}{S^{0.31} i^{0.38}}$$

Where:

T_o = overland flow time of concentration, min;
L = overland flow length, ft, should generally be in the range of those specified in Table 10;
n = roughness coefficient for overland flow (Table 10);
S = average slope of flow path, ft/ft; and
i = intensity of precipitation, in/hr (Table 11).

Use of the overland time of concentration equation requires an iterative approach: an initial estimate of time of concentration updated by successive estimates of precipitation intensity. In many cases, overland flow accounts for a large part of the lag time in a basin.

To assure that consistent and reasonable values are used to calculate the total time of concentration, the maximum times of concentration for commercial and residential areas and a range of times of concentration for open space are presented in Table 12. The land use applies only to the most upstream reach of the basin, prior to entering the gutter or street.

Gutter Flow – The Manning's equation for a triangular channel cross section is used to determine the flow velocity and travel times for street gutter flow. The average distance from the overland flow surface to the nearest inlet is divided by flow velocity to obtain street gutter flow time. The gutter flow equation was derived using the following assumptions:

- The cross slope of the street is 0.02 ft/ft.
- The flow in the gutter is six inches deep and contained by the curb.
- The street surface is smooth asphalt or concrete.

$$V_g = \frac{1.12}{n} S_x^{0.67} S^{0.50} T^{0.67}$$

Where:

V_g = velocity of flow in the gutter, ft/s;
S_x = street cross slope, ft/ft, design value = 0.02;
S = street longitudinal slope, ft/ft;
T = spread of flow in gutter = d/S_x, ft;
d = depth of flow in the gutter, ft, design value = 0.5 ft; and
n = Manning's "n" for pavement, design value = .02.

Pipe Flow – Manning's equation can also be used to determine travel time of flow through pipes. Travel time is usually calculated by assuming full pipe flow. Flow velocity is calculated with the equation:

$$V = \frac{1.49}{n} R^{0.67} S^{0.50}$$

Where:

V = velocity in pipe, ft/s;
R = hydraulic radius, D/4 for full pipe flow, ft;
D = diameter of pipe, ft;
S = slope, ft/ft; and
n = Manning's "n," design value = 0.015.

Trapezoidal Channels – A modified Manning's equation is used for open channel flow to derive the velocity for trapezoidal grass-lined channels. The following assumptions were made in the derivation of the modified equation:

- Channel side slopes are 3:1, horizontal: vertical.
- Channel bottom width equals the depth.
- Top width is seven times the bottom width.

$$V = \frac{0.995}{n} b^{0.67} S^{0.5}$$

Where:

V = velocity, in ft/s;
b = bottom width, ft;
n = Manning's "n" for channel flow (Table 5); and
S = slope, ft/ft.

Lag Frequency Factors – It is assumed much of the existing storm sewer system in the City was designed to convey runoff from the 2-year storm event. Flows exceeding the storm sewer capacity back up in the streets and either pond or, if an overland release has been provided, flow in the streets.

Lag times, regardless of the method of calculation, should be amended to account for flows exceeding pipe capacities, causing temporary flooding in the streets, and thereby increasing lag times. The multiplication factors presented in Table 13 are applied to the lag times for piped areas with overland release.

Hydrograph Routing – Hydrograph routing in HEC-1 can be used to represent hydrograph movement in a channel or through a storage facility. The hydrograph is routed based upon the characteristics of the channel or the storage-outflow characteristics of the storage facility. This section lists the routing methods that would be permitted using HEC-1. It also describes techniques for modeling two types of detention basins.

Routing Methods – The HEC-1 program contains several methods to route runoff hydrographs. Three of the methods, Modified Puls, Muskingum-Cunge, and Muskingum are recommended for use in the City. The methods, applications, and required parameters are summarized in Table 14, in order of preference. In most cases, Modified Puls routing would be required where HEC-2 models are available. Additional information on these routing methods is available in the HEC-1 User's Manual.

Modified Puls Routing – The Modified Puls routing method is used for channels with available HEC-2 storage discharge information. The number of steps (NSTPS) is calculated from reach length and velocity with the following equation:

$$NSTPS = \frac{reach/length/velocity}{2xNMIN}$$

Where: NMIN is the time interval.

The factor of 2 in the denominator was added to reflect hydrograph attenuation typical of developed channels in Sacramento County. The maximum NSTPS has been set to five.

Muskingum Routing – The Muskingum routing method is used for channels where limited cross-sectional information is available. The number of subreaches is chosen to satisfy stability criteria, as described in the HEC-1 User's Manual. The Muskingum "K" value may be approximated as the travel time in hours for the reach based upon the flow velocity at normal depth. Typical ranges for the Muskingum "X" value are given below:

Channel Description	Muskingum "X" Range
Most Channel Flow is in the Floodplain	0.0-0.15
Natural Channels	0.20-0.35
Excavated Earth or Concrete Channels	0.40-0.50

Muskingum-Cunge Routing – The Muskingum-Cunge routing method is used for channels with standard cross sections.

Reservoir Routing – Reservoir routing is used to route a hydrograph through a storage facility such as a detention basin.

Off-Channel Detention Routing – Off-channel detention basins are usually the most effective means of reducing peak flow in a channel for a given storage volume. Off-channel detention basins are located adjacent to, but separate from, a channel. Peak flows in the channel are diverted into the detention basin over a weir in the side of the channel. Off-channel detention can be conceptually modeled using the diversion option in HEC-1. The diversion option allows diverting a flow from a channel based upon the total flow in the channel. The typical steps for modeling off-channel detention are:

- Divert flow to limit flow in the channel to the desired design flow.
- Determine the required channel overflow structure and off-channel storage based upon diverted hydrograph (in some cases, the detention volume is known and the reduction of flow in the channel is determined).
- Route the diverted flow through the off-channel detention basin.
- Return the routed detention basin flow to the channel.

On-Channel Detention Routing – On-channel detention includes using the excess storage capacity of a channel by building a berm across the channel and/or expanding the storage in a reach of the channel (e.g., through excavation). Another example of on-channel detention is an "end-of-pipe" basin that collects runoff from a subdivision before entering the channel. With on-channel detention, the entire runoff hydrograph is routed through the detention facility. On-channel detention can be modeled in HEC-1 by using the Modified Puls routing methods for reservoirs. In cases where detention storage is provided predominantly by the natural floodplain of the channel, it may be more appropriate to use the Modified Puls routing method for channels.

SWMM Modeling

The EPA SWMM program may be used to route runoff hydrographs generated in HEC-1 or HEC-HMS. The results can be used to design open channels, major road crossings, detention ponds, etc. The criteria that would be used to develop the SWMM models are presented in this section.

Prepare Basic Information – Lay out the proposed sewer system and delineate points of concentration for the design of inlets, junctions, pipelines, etc. Use HEC-1 or HEC-HMS to determine design flow hydrographs for each node.

Design Capacities – Drainage facilities shall be designed to accommodate the future development of the entire upstream watershed defined as full build out of the General Plan Land Use Diagram. Design capacities for storm facilities shall be consistent with the criteria described in the HEC-1 and HEC-HMS modeling section.

Physical Parameters – Analysis of existing storm drainage facilities shall be performed using values obtained from as-built record drawings or from direct measurements observed in the field. Design of storm drainage facilities shall involve sound engineering judgment with respect to appropriate open channel and conduit dimensions.

Manning's "n" Value – Roughness coefficients for existing open channel sections shall be calculated using the "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," USGS, Water-Supply Paper 2339. Base roughness

coefficients for existing conduit sections shall be determined based upon the condition and material of the pipe using manufacturer's literature or appropriate hydraulics references.

In SWMM, energy losses through a conduit are only accounted for by specifying Manning's "n". Therefore, in order to account for minor losses, such as exit and entrance losses, Manning's "n" must be adjusted (increased) accordingly. The method to determine the amount by which the Manning's "n" should be increased to account for minor losses in the conduit is as follows:

Beginning with Manning's equation and isolating n:

$$\text{Equation 1} \quad n = \frac{1.486 \cdot r^{2/3} s^{1/2}}{v}$$

Where:

r = hydraulic radius,
s = energy slope,
v = average velocity.

The energy slope can also be expressed as:

$$\text{Equation 2} \quad s = \frac{h}{l}$$

Where:

h = head (energy) loss through the conduit, and
l = length of the conduit.

The maximum loss that would occur through the conduit occurs when the velocity through the conduit is at its maximum. If an estimate is available from another source, such as a manufacturer's rating curve (h vs. Q), of the what the head loss h (entrance or exit) is at a the maximum velocity (or discharge), h and l can be substituted into Equation 2, and Equation 2 can be substituted into Equation 1, to give the resulting *increase* in Manning's n to account for minor losses through the conduit in SWMM.

If the minor losses can be specified with a loss coefficient k such that:

$$\text{Equation 3} \quad h = k\frac{v^2}{2g},$$

The corresponding increase in Manning's n to account for the head loss h becomes:

$$\text{Equation 4} \quad n = 1.486 \cdot r^{2/3} \cdot \left(\frac{k}{2gl}\right)^{1/2} = 0.1852 \cdot r^{2/3} \cdot \left(\frac{k}{l}\right)^{1/2}.$$

For purposes of estimating losses in pipes, an entrance loss coefficient of k = 0.5 and an exit loss of k = 1.0 shall be used. Additional minor losses (such as bends, expansions, contractions, etc.) can be added as required.

B. TYPE 2 DRAINAGE FACILITIES

Type 2 drainage facilities include conveyance, flood protection, water quality treatment, and recreational, environmental, and aesthetic elements, which may consist of roadside ditches, storm drainage pipe systems, and overland conveyance systems. It is important to note that emphasis should be placed upon the appropriate design of the overland conveyance system, generally streets. If the overland conveyance system is appropriately designed, the capacity of the storm drainage pipe systems, roadside ditches, and culverts would have little effect on the risk of property damage or threat to public safety from flooding.

Design Flow

The *Modified Rational Method* shall be used to design Type 2 drainage facilities. The *Modified Rational Method* calculates flow based upon storm intensity, time of concentration, imperviousness, and basin size. The *Modified Rational Method* has been widely used and tested throughout the United States.

The *Modified Rational Method* for the 10-year storm event would be used to calculate the peak design flow for storm drainage pipe systems and roadside ditches.

When the design capacity of a storm drainage pipe system is exceeded, overland conveyance systems, generally streets, are relied upon to safely convey flow downstream to detention ponds or other receiving waters. The 100-year storm event would be used for evaluating and designing overland conveyance systems.

Rational Method

The *Rational Method* may be used for peak flow calculations to design street drainage, storm sewers, and culverts not associated with channels. The application of the *Rational Method* would be limited to areas up to 640 acres.

The *Rational Method* equation has the form:

$$Q = CiA$$

Where:

Q = rate of runoff, acre-inches per hour or cubic feet per second (acre inch per hour = 1.008 cubic feet per second, a negligible difference);

C = runoff coefficient, which is the ratio of peak runoff to average rainfall intensity;

I = average rainfall intensity, inches per hour; and

A = drainage area, acres.

The *Rational Method* shall be applied using the procedure outlined below and the sample computation form presented in Table 15. An example electronic spreadsheet file, "sample.xls," showing layout and format of the spreadsheet is available from the City (Appendix C).

Prepare Basic Information – Lay out the proposed storm sewer system and delineate the subbasins tributary to points of concentration for the design of inlets, junctions, pipelines, etc. Delineate the land uses and hydrologic soil groups within each subbasin.

Determine Runoff Coefficient – The runoff coefficients, represented as “C,” for a storm having a 10-year recurrence interval are presented in Table 16 by land use designation and hydrologic soil group. The 10-year runoff coefficients are to be used with the frequency

factors presented in Table 17 for design storm frequencies other than the 10-year. The frequency factor adjusts the 10-year C for changes in infiltration and other losses with a change in storm frequency. The C value used in Table 15 is the weighted average of the C values for the subareas within the system being designed. Presented in Table 18 is a sample calculation form for weighted average C computations for a basin. A sample electronic spreadsheet file, "c_runoff.xls," is available from the City (Appendix C).

Determine Time of Concentration – The time of concentration, or the travel time, is the time required for runoff to flow from the most upstream point of the drainage area through the conveyance system to the point of interest. The travel time is calculated by dividing the length of the conveyance system component by the corresponding velocity of flow. The travel time, T_c, is computed as follows:

$$T_c = T_o + T_g + T_p + T_{ch}$$

Where:

T_o = overland flow time of concentration;
T_g = gutter flow travel time;
T_p = pipe flow travel time; and
T_{ch} = channel flow travel time.

The equation used to compute the travel time for each conveyance component is described below.

Overland Flow – The developed Kinematic wave empirical equation based upon available SCS, USACOE, and FHA overland flow data (Sacramento City/County, 1996) is:

$$T_o = \frac{0.66 L^{0.50} n^{0.52}}{S^{0.31} i^{0.38}}$$

Where:

T_o = overland flow time of concentration, minute;

L = overland flow length, ft, should generally be in the range of those specified in Table 10;

n = roughness coefficient for overland flow (Table 10);

S = average slope of flow path, ft/ft; and

i = intensity of precipitation, in/hr (Table 11).

Use of the overland time of concentration equation requires an iterative approach: an initial estimate of the time of concentration updated by successive estimates of precipitation intensity.

Presented in Table 12 are consistent and reasonable values to use to calculate the total time of concentration, maximum times of concentration for commercial and residential areas, and a range of times of concentration for open space. The land use applies only to the most upstream reach of the basin, prior to entering the gutter or street.

Gutter Flow – Manning's equation for a triangular channel cross section is used to determine the flow velocity and travel times for street gutter flow. The average distance from the overland flow surface to the nearest inlet is divided by flow velocity to obtain street gutter flow time. The gutter flow equation was derived using the following assumptions:

- The cross slope of the street is 0.02 ft/ft.
- The flow in the gutter is six inches deep and contained by the curb.
- The street surface is smooth asphalt or concrete.

The velocity of flow in the gutter is computed by the equation:

$$V_g = \frac{1.12}{n} S_x^{0.67} S^{0.50} T^{0.67}$$

Where:

V_g = velocity of flow in the gutter, ft/s;

S_x = street cross slope, ft/ft, design value = 0.02;

S = street longitudinal slope, ft/ft;

T = spread of flow in gutter = d/S_x, ft;
d = depth of flow in the gutter, ft, design value = 0.5 ft; and
n = Manning's "n" for pavement, design value = 0.02.

Pipe Flow – Manning's equation can also be used to determine travel time of flow through pipes. Travel time is usually calculated by assuming full pipe flow. Flow velocity is calculated with the equation:

$$V = \frac{1.49}{n} R^{0.67} S^{0.50}$$

Where:

V = velocity in pipe, ft/s;
R = hydraulic radius, D/4 for full pipe flow, ft;
D = diameter of pipe, ft;
S = slope, ft/ft; and
n = Manning's "n", design value = 0.015.

Trapezoidal Channels – A modified Manning's equation is used for open channel flow to derive the velocity for trapezoidal grass-lined channels. The following assumptions were made in the derivation of the modified equation:

- Channel side slopes are 3:1, horizontal: vertical.
- Channel bottom width equals the depth.
- Top width is seven times the bottom width.

$$V = \frac{0.995.}{n} b^{0.67} S^{0.5}$$

Where:

V = velocity, in ft/s;
b = bottom width, ft;
n = Manning's "n" for channel flow (Table 2); and
S = slope, ft/ft.

Determine Intensity – The rainfall intensity shall be determined using information prepared by Mr. James D. Goodridge ("Solano & Yolo County Design Rainfall") prepared as part of the "Covell Drainage System Comprehensive Drainage Plan," in 1993.

Storm Drainage Pipe Systems

The invert of any storm drainage pipe outfall at ponds would be designed to prevent standing water within the pipe systems, which can cause sedimentation that could affect the conveyance capacity and longevity of the pipes.

The storm drainage pipe systems would be designed using the 10-year storm event design flow and the 10-year storm event peak water surface elevation in the downstream pond or other receiving water. Hydraulic grade lines would be computed using Manning's formula with an "n" value to account for friction and minor losses, in accordance with the information presented in Table 19. The minimum pipe slope would be equal to or greater than the hydraulic slope. To the extent practical, the hydraulic grade line would be within the pipe. The hydraulic grade line would be at least one-half foot below the flow line of the inlet grate. The minimum velocity in closed conduits would be two feet per second when flowing full.

The minimum drainage inlet elevation would be one foot above the 100-year water surface elevation in the downstream detention pond or other receiving water.

Once flow at a point in a storm drain system exceeds the capacity of a 72-inch pipe, the facility must be designed as a Type 1 facility and cannot be placed inside parallel pipes to avoid sizing for 100-year frequency. Additionally, downstream components within a drainage system cannot revert to Type 2 facilities once a Type 1 designation is reached (i.e., pipes draining detention ponds).

The pipe inverts would be designed to provide minimum cover at the upstream areas of the drainage. The minimum pipe diameter allowable would be 18 inches.

Manholes

Standard precast concrete or saddle-type manholes shall be used where required. Maximum spacing between manholes shall be 500 feet for pipe sizes of 48 inches and under, and 800 feet for pipes of 54 inches and larger.

Manholes shall be located at junction points, angle points greater than 20 degrees, and changes in conduit size. On curved pipes with a radius of 200 feet to 400 feet, manholes shall be placed at the beginning of curve (B.C.) and ending of curve (E.C.) and at 300 feet maximum intervals along the curve. On curves with a radius exceeding 400 feet, manholes shall be placed at the B.C. and E.C. and at 400 maximum intervals along the curve for pipes 24 inches and less in diameter and 500 feet maximum intervals along the curve for pipes greater than 24 inches in diameter.

Inlets

The spacing of storm water drainage inlets shall not exceed a maximum of 500 feet. Storm water drainage inlets shall be located to prevent surface flow through street intersections.

Pipes

Storm water drainage pipes shall be reinforced concrete pipe, nonreinforced concrete pipe, or cast-in-place concrete pipe. All pipes shall be constructed with a minimum cover of two feet, or as approved by the City's Director of Public Works.

The minimum velocity in closed conduits shall be two feet/sec when flowing full. The minimum pipe diameter shall be 18 inches.

Flowage Easements

Where the flooding of land outside the City and urban growth area serves to attenuate the peak runoff similar to a detention pond, a flowage easement shall be acquired to ensure the functional integrity of the land as a component of the City's storm drainage system is preserved over time.

Pipe Discharges into Water Quality Ponds

The location of pipe discharges at a pond would be designed to enhance water quality treatment within the pond and to prevent the "short-circuiting" flow through the pond.

Overland Conveyance Systems

All new development within the City would include the design of street systems or other suitable release paths to convey flow in excess of pipe capacity, in an unobstructed manner, to the detention pond or other receiving waters. The overland conveyance facilities would provide water surface elevations below the pad elevations in the 100-year storm event. The street system would be designed to minimize flooding depths within the street. To the extent practical, the overland flooding depths should be designed with a maximum of one foot from the gutter flow line. The street design would incorporate designated overland flow paths from the streets to the pond.

Roadside Ditches

Roadside ditches would be designed to minimize safety hazards and emphasize water quality treatment by implementing BMPs. At a minimum, roadside ditches would be designed to convey the 10-year storm event design flow.

Nonregional Water Quality Treatment

In addition to regional water quality treatment detention ponds previously discussed, other water quality treatment BMPs should be implemented. Source and treatment control BMPs may include:

- Grassy Swales.
- Filter Strips.
- Media Filters.
- Infiltration Devices.
- Storm Drain Signage.

C. FEMA CRITERIA AND CITY STANDARDS

Drainage facilities would comply with FEMA criteria and City standards. These criteria and standards include, but are not limited to:

- One foot of freeboard to existing ground in the 100-year storm event for open channels and ponds.
- Minimum three feet of freeboard in the 100-year storm event for levees. The structural integrity of levees must be certified in accordance with FEMA guidelines.
- Backup power and pump capacity for pump stations.
- Finished floor elevations one foot above the base flood elevation (100-year storm event).
- Fill within the 100-year floodplain would be compacted to 95 percent of the maximum density obtainable with the standard proctor test method issued by the American Society for Testing and Materials, or an equivalent test method acceptable to FEMA.

D. INTERIM CONDITIONS

As development progresses within the City on an incremental basis, interim drainage conditions must be evaluated. Some limited flexibility for criteria and standards may be considered for interim conditions, but in no case would the following be allowed:

- Risking property damage from flooding.
- Jeopardizing public safety.
- Increasing floodplain elevations to surrounding lands.
- Creating significant impacts to surface or groundwater quality.

DESCRIPTION OF EXISTING CONDITIONS

A. TOPOGRAPHY AND SUBBASIN BOUNDARIES

Within the Putah Creek/Dry Creek subbasins, which consist of approximately 1.4 square miles, the terrain generally slopes from the eastern bank of Dry Creek in the west to Interstate 505 in the east. The approximate ground elevations range from a maximum El. 180, in the coastal foothills to El. 122 in the vicinity of Willow Canal, National Geodetic Vertical Datum of 1929 (NGVD 29). Just upstream of Road 89 (Railroad Road), the approximate ground elevation is El. 128 (NGVD 29).

Presented on Figure 2 is the delineation of the subbasin areas draining to Putah Creek/Dry Creek. The sources of the topographic mapping presented on Figure 2 are the following U.S. Department of Interior Geologic Survey Quadrangle maps (USGS Quads) for California, 7.5 Minute Series:

- Winters Quadrangle
- Monticello Dam Quadrangle

The vertical and horizontal data are National Geodetic Vertical Datum 1929 (NGVD29) and North American Datum of 1927 (NAD27), respectively. This information was combined digitally with topography developed from a 1974 aerial survey performed by American Aerial Surveys.

Additional topographic mapping for portions of the Putah Creek/Dry Creek subbasins is presented in the Nolte study and the USACOE study.

B. LAND USE

The existing land use within the Putah Creek/Dry Creek subbasins primarily consists of agriculture, urban residential, commercial, light industrial, and open space.

C. SOILS INFORMATION

Based upon a report prepared by the U.S. Department of Agriculture, SCS, entitled, "Soil Survey of Yolo County, California," June 1972, the soils within the Putah Creek/Dry Creek subbasins have been classified as hydrologic soil types "B," "C," and "D." Refer to the referenced SCS document for specific area delineations.

D. GROUNDWATER ELEVATION DATA

Historical data for spring and fall groundwater levels, published by the California Department of Water Resources, shows the groundwater table within the low-lying areas of the Putah Creek/Dry Creek subbasins just upstream of Interstate 505, to rarely be less than approximately 15 feet below existing ground level.

E. EXISTING DRAINAGE AND IRRIGATION FACILITIES

Drainage through the city limits of Winters is conveyed through typical roadside ditches, gutters, and storm drains. During flooding conditions where the capacity of the existing storm drain system is exceeded, overland flow would be conveyed through the streets as shallow sheet flow. The principal existing storm drain facilities and contributing areas are presented in the CH2M Hill study.

The city is bounded on the west by Dry Creek, which consists of a natural drainage channel running north to south and crosses Highway 128 before forming a confluence with Putah Creek. Putah Creek bounds the city to the south and runs from west to east, crossing Interstate 505. The Willow Canal is operated and maintained by YCFC&WCD for irrigation to serve areas within and downstream of the Moody Slough subbasin. The Willow Canal extends south and east from Chapman Reservoir to the east side of Road 89. From there, it travels south where it crosses Moody Slough and continues east, crossing under Interstate 505 just north of the Highway 128 crossing. The Willow Canal has berms above natural grade to maintain a positive head to facilitate gravity irrigation deliveries. The berm along the south face of the Willow Canal represents the northern boundary of the Putah Creek/Dry Creek subbasins.

F. HYDROLOGIC MODELING

Wood Rodgers prepared hydrologic computer models to represent drainage and flooding conditions for Putah Creek and Dry Creek for storm events of various recurrence intervals and durations. The Dodson & Associates, Inc. ProHEC-1 (based upon USACOE's HEC-1 [Version 4.1e]) computer program was used in accordance with the criteria and standards previously identified in this report. The 100-year storm event was analyzed to assist with evaluating worst-case flooding in the existing city limits.

HEC-1 models of the following storm events were developed for Putah Creek and Dry Creek for the existing conditions:

- 100-year, 10-day storm event.
- 100-year, 24-hour storm event.

To reflect the capacity of the City's existing storm drain system in the 100-year HEC-1 modeling, Wood Rodgers diverted the 10-year storm design capacity of the existing system from each subbasin. As noted in the report prepared by CH2M Hill, there are several sections of pipe (particularly in older areas of the City) that are undersized for a 10-year event.

Wood Rodgers took the average storm drain capacity in the City's existing system as a reduction factor to apply to the 10-year design flows calculated using the *Rational Method*. The adjusted 10-year storm drain capacity was diverted from each shed in the HEC-1 modeling to determine the residual 100-year overland runoff.

Due to the storage volume within the hydrologic routing, the 10-day storm event results in the worst-case flooding scenarios for a 100-year recurrence interval for the existing conditions within the Putah Creek/Dry Creek subbasins.

In general, overland flow through the city limits of Winters is divided between the north and south sides of Highway 128. However, near the Winters High School, there is insufficient conveyance capacity through the roadside ditch running parallel to the north side of Highway 128 to convey the 100-year storm without overtopping the roadway. During the 100-year 24-hour duration storm and 100-year 10-day storm, approximately

24 cfs and 23 cfs spills south over the highway at this location during the respective flood events.

During large storm events, ponding would occur at the north end of the Carter Ranch property due to a depressed area that collects overland flow coming from the south and the west. East of the Carter Ranch property, overland flow on the north side of Highway 128 flows east and collects in a depressed storage area bounded by the Willow Canal to the north, Interstate 505 to the east, and Highway 128 to the south. During the 100-year event, the drainage capacities of existing culverts at this location are exceeded causing ponding to occur. Once sufficient stage elevation is reached at this location, flow would begin to overtop the berm that runs along the south bank of the Willow Canal and would flow north.

Overland flow south of the highway that occurs during large storm events flows east and collects near the intersection of Highway 128 and Interstate 505 before draining directly to Putah Creek. The majority of this flow collects and is conveyed through a drainage ditch that runs east, parallel to Highway 128. During large storm events however, the capacity of existing culverts at Morgan Street, East Main Street, and Purtell Place are exceeded, which causes ponding to occur along the south side of Highway 128 upstream of these culverts.

G. FLOODING

Within the Putah Creek/Dry Creek subbasins, the FEMA FIRM Community Panel Numbers listed under Section II.D. of this report shows the effective flood insurance zone designations.

Presented on Figure 3 are the approximate delineations of FEMA's 100-year floodplain and Wood Rodgers' revisions to the existing 100-year floodplain.

Due to the undersized capacity of the Moody Slough channel and drainage crossings at Road 89 and Interstate 505, there is a significant 100-year floodplain located upstream of these drainage crossings and backs water into the Putah Creek subbasin. Zone A is shown just upstream of Interstate 505.

The 100-year peak stages from Wood Rodgers' hydrologic models in Putah Creek and Dry Creek are roughly comparable to the FEMA floodplain maps (Figure 3).

IDENTIFICATION OF PROJECT-SPECIFIC CONSTRAINTS

A. CONSTRAINTS

Development within an existing floodplain requires significantly more mitigation than development outside of an existing floodplain. For any development, the increased peak rate of runoff, volume of surface runoff, and changes in timing of runoff would need to be mitigated. However, when development encroaches into an existing floodplain, compensating mitigation is typically required in the form of replacing floodplain storage or additional conveyance and/or pumping capacity.

Based upon the documentation contained within the USACOE's study, no constraints have been identified that would preclude increasing storm drainage discharges to Putah Creek. However, the USACOE study only analyzed impacts of a maximum discharge to Putah Creek of 1,000 cfs. In the study, this flow is characterized as additional flow in excess of existing Putah Creek flows; therefore, this 1,000 cfs is assumed to be divertable flow in addition to flows that currently contribute to Putah Creek drainage. Therefore, proposed discharges to Putah Creek that exceed 1,000 cfs may require additional analysis to determine potential impacts and mitigation measures. Evaluating impacts and mitigation measures for discharges to Putah Creek is beyond the scope of this study. Accordingly, a maximum diversion to Putah Creek of 1,000 cfs is set as a "target" flow.

B. OPPORTUNITIES

Wood Rodgers' analysis of the Moody Slough subbasin includes a proposed diversion channel that would route overflow from the proposed Moody Slough water quality ponds to Putah Creek. This channel has been designed with a maximum conveyance capacity of 1,150 cfs. This diversion channel can also serve as a flood control facility for the Putah Creek/Dry Creek subbasins by receiving overland releases as well as serving as a collection facility for overflow draining from water quality facilities.

DESCRIPTION OF PROPOSED LAND USE AND DEVELOPMENT PHASING

Proposed land uses and a road layout within the City are presented on Figure 4. Development within the City would occur over time, possible 10 to 20 years. Actual development phasing may differ from that included in this report. If significant changes to phasing occur, supplemental information may need to be developed to verify the integrity of the proposed drainage facilities for the Putah Creek/Dry Creek subbasins is maintained.

Existing and proposed land uses within the General Plan Urban Area of the City are in accordance with the City's General Plan.

Presented on Figure 4, are the land uses and the preliminary roadway layout for the City's General Plan Urban Area used in this analysis. Since this study began, the City has updated its General Plan. Wood Rodgers evaluated the amended land use changes and determined the changes were significant enough to warrant incorporation into the Rancho Arroyo Subbasin Storm Drainage Evaluation (Appendix D). A copy of the amended land use is also included in Appendix D. The amended land uses are not considered significant with respect to the design of the facilities elsewhere in the Putah Creek/Dry Creek subbasins.

FORMULATION OF DRAINAGE PLANS

Drainage plans were formulated for the Ultimate Developed Conditions. The objective was to identify cost-effective "backbone" drainage facilities that would provide protection to the proposed development and prevent adverse impacts on surrounding lands. To avoid additional analysis beyond that included in the USACOE's study regarding impacts to Putah Creek, the capacity of the diversion channel was limited to 1,150 cfs. This accounts for 150 cfs that currently spills into Putah Creek from the Willow Canal being redirected through the proposed facilities, in addition to the 1,000 cfs identified in the USACOE study.

A. HYDROLOGIC AND HYDRAULIC MODELING

Hydrologic models using HEC-1 were prepared for the Ultimate Developed Conditions. The criteria and standards described earlier in this report were used to develop models for the following storm events:

- 100-year, 10-day storm event.
- 100-year, 24-hour storm event.

Presented on Figure 2 are the subbasins used in the hydrologic analysis of the Putah Creek/Dry Creek subbasins under Ultimate Development. A description of the drainage plan is provided below.

B. ULTIMATE DEVELOPED CONDITIONS DRAINAGE PLAN

The Ultimate Developed Conditions Drainage Plan is described below. The drainage facilities and discharge locations are presented on Figure 5.

Drainage Facilities

The drainage facilities for the Ultimate Developed Conditions include the following elements:

Rancho Arroyo Regional Detention Pond

The existing pond shall be modified to accommodate a new pump station at the pond outlet. A two-stage pumping operation would be used to satisfy requirements for flood control and water quality. The water quality component shall act like a dry detention basin with a 48-hour drawdown, correlating to a 1 cfs water quality pump rate. An additional pumping rate of 2 cfs would be triggered when flood control is required to maintain one foot of freeboard in the pond. Back-up power to the pumps shall be designed in accordance with the guidelines outlined in this report. Modifications shall include removing the existing standpipe outlet and provisions to facilitate construction of the lift station and pumping operation.

Putah Creek Diversion

- A trapezoidal channel with a 40-foot bottom width and 3:1 side slopes and 15-foot access/maintenance roads along both sides would be constructed to convey floodwater from Moody Slough to Putah Creek. Safety fencing would be placed along the perimeter of the diversion channel. This facility is an element of the storm drainage facilities proposed for the Moody Slough subbasin.

  A typical drainage channel section is presented on Figure 6.

- A road crossing would be constructed under Highway 128 with five 5’x8’concrete box culverts.

- An outfall structure (concrete spillway baffled apron) would be constructed to dissipate diverted flows into Putah Creek.

<u>*Putah Creek Detention/Water Quality Pond #1*</u>

➢ An excavated pond with detention and wet water quality pond features would be constructed. A typical section for a detention/water quality pond is shown on Figure 7.

  An outlet control weir structure (10-foot weir crest width) would be constructed to control flood diversion to the Putah Creek diversion facilities above El. 116.5, through an energy dissipation structure, which would also control retention of low flow volumes of runoff (below El. 116.5).

<u>*Putah Creek Detention/Water Quality Pond #2*</u>

➢ An excavated pond with detention and dry water quality pond features would be constructed.

➢ An outlet control weir structure (20-foot weir crest width) would be constructed to control flood diversions to Putah Creek diversion facilities above elevation 118.6, through an energy dissipation structure, which would also control retention of low flow volumes of runoff (below El. 118.6).

➢ A forebay would be constructed at the outlet, which would collect flow spilling over the weir structure to convey flow up to the 10-year event through a 48-inchpipe (S = 0.0025), which would drain directly to the Putah Creek diversion channel. Excess flow from storms larger than the 10-year event that exceed the capacity of the pipe would be conveyed overland to the Putah Creek diversion channel as shallow sheet flow.

<u>*Putah Creek Detention/Water Quality Pond #3*</u>

➢ An excavated pond with detention and wet water quality pond features would be constructed.

- An outlet control weir structure (50-foot weir crest width) would be constructed to control flood control diversion to Putah Creek diversion facilities above elevation 114.5 through an energy dissipation structure, which would also control retention of low flow volumes of runoff (below El. 114.5).

<u>*Putah Creek Detention/Water Quality Pond #4*</u>

- An excavated pond with detention and wet water quality features pond features would be constructed.

- An outlet control weir structure (10-foot weir crest width) would be constructed to control flood diversion to the Putah Creek diversion facilities above El. 113.3, through an energy dissipation structure, which would also control retention of low flow volumes of runoff (below El. 113.3).

<u>*Grant Street Interceptor*</u>

- A canal with a capacity of 110 cfs would be constructed between Broadview Drive and Highway 128 along the almond orchard east property line, which would capture the overland flow north of Highway 128 and redirect it into a 60-inch storm drain that would run parallel to Highway 128 and drain to the Putah Creek diversion channel. The canal and pipe would be designed for the 100-year storm capacity and would require a drop inlet structure at the pipe inlet to accommodate the 3.1-foot difference between channel and pipe inverts.

<u>*Other Features*</u>

- A 66-inch storm drain would be constructed to capture flow that collects along the south face of Highway 128 between Morgan Street and the southwest water quality pond. The pipe would be designed with 100-year storm capacity of 184 cfs and would run parallel to the south face of

Highway 128 between Morgan Street and the southwest water quality pond.

The proposed 100-year floodplain is presented on Figure 6.

Storm Drainage, Flooding, Surface Water Quality, and Irrigation Impacts

The drainage facilities described above would protect the proposed development from the risk of flood damage and threat to public safety. Additionally, implementation of the Ultimate Developed Conditions Drainage Plan would result in no significant adverse drainage, flooding, and irrigation impacts on surrounding lands. Diverting high flows to Putah Creek mitigates for increased rate and volume of runoff as a result of development, for changes in timing, and for loss of floodplain storage. The proposed detention ponds, in conjunction with additional source and treatment control measures, would provide storm water quality treatment.

OPINION OF PROBABLE COST

Opinions of probable cost were developed for the storm drainage facilities described in this report.

The Opinion of Probable Cost for Drainage Facilities for the Ultimate Developed Conditions, is **$4,026,340**. A breakdown of the Opinion of Probable Costs is included in Appendix E.

Costs presented as part of this report do not include costs shared with the Moody Slough subbasin related to the Putah Creek diversion improvements. Refer to the report prepared by Wood Rodgers entitled, "Moody Slough and Putah Creek / Dry Creek Subbasins, Storm Drainage Cost Allocation Report," dated August 2005, for details on shared facilities and costs.

FINDINGS AND RECOMMENDATIONS

A. FINDINGS

Summarized below are the findings of Wood Rodgers relative to storm drainage and flood control within the Putah Creek/Dry Creek subbasins.

1. The existing flooding within the City is minor (average depth of less than 1.0 foot for a 100-year storm event). However, the extent and depth of flooding within the General Plan area is much greater.

2. The construction of facilities outlined in the Drainage Report would significantly reduce the extent of the 100-year floodplain within the General Plan area; however, detailed topographic mapping would be required to confirm the presence or extent of a residual 100-year floodplain.

3. Storm drainage from the majority of existing development within the City drains directly to Putah Creek and Dry Creek without water quality treatment. Due to the extent of existing development, the construction of storm water treatment facilities is problematic.

4. The construction of facilities outlined in the Drainage Report would facilitate removing the existing 100-year floodplain in the northeast portion of the General Plan area to the extent on-site storm drainage facilities are properly designed and integrated with the proposed storm drainage facilities.

B. RECOMMENDATIONS

Based upon Wood Rodgers' work in preparing the Drainage Report and findings noted above, Wood Rodgers recommends the following:

1. Implement storm drainage facilities to accommodate new development within the Putah Creek/Dry Creek subbasins in conformance with the facilities outlined in this Drainage Report. Proposals to develop drainage infrastructure different than outlined

in this Drainage Report should be evaluated to ensure the integrity of the drainage facilities is maintained as development occurs.

2. Obtain a CLOMR from FEMA prior to approval of improvement plans for new development within the existing 100-year floodplain.

3. Require a comprehensive drainage analysis from the development community consistent with the City's adopted drainage standards, which accounts for and mitigates adverse on-site and off-site drainage/flooding impacts that may be caused by proposed development.

4. Update the opinion of probable costs contained in CH2M Hill's report using a multiplication factor for unit costs based upon historical cost indexes.

REFERENCES

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Yolo, County of, Department of Public Works and Transportation, "Davis-Winters Drainage Report, Chickahominy-Dry Slough Drainage Complex-Drainage Report," March 1986.

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Tables

Tables

TABLE 1

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

METHODS FOR ESTIMATING DESIGN FLOW

Application	Method	Maximum Basin Size	Design Parameter	Reference
Design of: • Street Drainage • Storm Drains • Culverts not Associated With Channels	Rational	640 ac	Flow	Hydrology Standards, Section IV.B.
Master Plans or Designs of: • Storm Drains • Open Channels • Bridges and Culverts • Detention Basins	HEC-1	No Limit	Flow and Volume	Hydrology Standards, Section IV.A.
Water Quality Detention Basins		No Limit	Volume	California Storm Water Best Management Practices Handbook

TABLE 2

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

ADJUSTMENT RESULTS FOR HEC-1 MODELS

Recurrence Interval, yr	Antecedent Moisture Conditions
100	2.00 (II)
50	1.55
10	1.10
2	1.00 (I)

TABLE 3

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

24-HOUR RUNOFF CURVE NUMBERS BY LAND USE, AMC II

Land Use	CN			
	A	B	C	D
Fallow	69	78	83	87
Idle	39	61	74	80
Row Crop (grown in winter)	64	74	81	85
Grain	62	73	81	84
Pasture	39	61	74	80
Orchard	32	58	72	79
Lawn Areas	39	61	74	80
Farmstead	59	74	82	86
Oak Areas, Grass Understory		48	57	63
Native Grasses	49	69	79	84
Suburban Residential (acre lots)	51	68	79	84
Urban	75	83.5	88.5	91
Urban Residential (1/4 acre lots)	61	75	83	87
Urban Industrial	81	88	91	93
Urban Commercial	89	92	94	95
Paved Areas (IE Roadways)	98	98	98	98
Apartments, Duplex	77	85	90	92
Residential (6,000 ft^2 lots)	73	82.5	88.25	90.75
Residential (8,000 ft^2 lots)	65	77.5	84.75	88.25
Residential (1/2 acre lots)	54	70	80	85
School (half commercial, half open space)	64	76.5	84	87.5
Park	39	61	74	80
Vacant	77	86	91	94

Source: USDA, Soil Conservation Service, Urban Hydrology in Small Watersheds, TR-55, June 1986.

TABLE 4

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

10-DAY RUNOFF CURVE NUMBER ADJUSTMENT[1]

Runoff Curve Numbers for					
1 Day	10 Days	1 Day	10 Days	1 Day	10 Days
100	100	80	65	60	41
99	98	79	64	59	40
98	96	78	62	58	39
97	94	77	61	57	38
96	92	76	60	56	37
95	90	75	58	55	36
94	88	74	57	54	35
93	86	73	56	53	34
92	84	72	54	52	33
91	82	71	53	51	33
90	81	70	52	50	32
89	79	69	51	49	31
88	77	68	50	48	30
87	76	67	49	47	29
86	74	66	47	46	28
85	72	65	46	45	28
84	71	64	45	44	27
83	69	63	44	43	26
82	68	62	43	42	25
81	66	61	42	41	24

[1] This table is used only if the 100-year frequency 10-day point rainfall is six or more inches. If it is less, the 10-day CN is the same as that for the 1-day CN.

Source: USDA, Soil Conservation Service, Earth Dams and Reservoirs, TR-60, October 1985.

TABLE 5

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

MANNING'S "n" FOR CHANNEL FLOW

Land Use Description	Manning's "n"
Concrete Pipe	0.015
Corrugated Metal Pipe	0.024
Concrete-Lined Channels	0.015
Earth Channel -- Straight/Smooth	0.022
Earth Channel -- Dredged	0.028
Mowed Grass Lined Channel	0.035
Natural Channel -- Clean/Some Pools	0.040
Natural Channel -- Winding/Some Vegetation	0.048
Natural Channel -- Winding/Stony/Partial Vegetation	0.060
Natural Channel -- Debris/Pools/Rocks/Full Vegetation	0.070
Floodplain -- Isolated Trees/Mowed Grass	0.040
Floodplain -- Isolated Trees/High Grass	0.050
Floodplain -- Few Trees/Shrubs/Weeds	0.080
Floodplain -- Scattered Trees/Shrubs	0.120
Floodplain -- Numerous Trees/Dense Vines	0.200

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 6

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

INITIAL LOSSES

Recurrence Interval	Loss, inches
2	0.40
5	0.25
10	0.20
25	0.15
50	0.12
100	0.10
200	0.08
500	0.06

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 7

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

LAND USE VS. EFFECTIVE PERCENT IMPERVIOUS
AND INFILTRATION RATES

Land Use from Aerial Photography	General Plan Land Use Designation	Effective % Impervious	Infiltration Rates (in/hr) by Hydrologic Soil Group		
			B	C	D
Highways, Parking	Central Commercial (CC)	95	0.14	0.07	0.04
Commercial, Office	General Commercial (GC) Service Commercial (SC) Highway Commercial (HC) Business Park (BP)	90	0.06	0.08	0.05
Industrial	Industrial (I)	85	0.162	0.082	0.052
Apartments	N/A	80	0.165	0.085	0.055
Mobile Home Park	N/A	75	0.167	0.087	0.057
Condominiums	Med. Density Residential (MDR)	70	0.17	0.09	0.06
Residential: 8-10 du/acre (20-25 du/ha)	Medium/Low Density Residential (MLDR)	60	0.18	0.10	0.07
Residential: 6-8 du/acre (15-20 du/ha)	Neighborhood Preservation (NP) Planned Neighborhood (PN)	50	0.18	0.10	0.07
Residential: 4-6 du/acre (10-15 du/ha)	Low Density Residential (LDR)	40	0.18	0.10	0.07
Residential: 3-4 du/acre (7.5-10 du/ha)	N/A	30	0.18	0.10	0.07

TABLE 7

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

LAND USE VS. EFFECTIVE PERCENT IMPERVIOUS AND INFILTRATION RATES

Land Use from Aerial Photography	General Plan Land Use Designation	Effective % Impervious	Infiltration Rates (in/hr) by Hydrologic Soil Group		
			B	C	D
Residential: 2-3 du/acre (5-7.5 du/ha)	Very-Low Density Residential (VLDR)	25	0.18	0.10	0.07
Residential: 1-2 du/acre (2.5-5 du/ha)	N/A	20	0.18	0.10	0.07
Residential: .5-1 du/acre (1-2.5 du/ha)	Rural Residential (RR)	15	0.18	0.10	0.07
Residential: .2-.5 du/acre (0.5-1 du/ha)	N/A	10	0.18	0.10	0.07
Residential: < .2 du/acre (.05 du/ha)	Agricultural Residential (AR)	5	0.18	0.10	0.07
Open Space, Grassland	N/A	2	0.18	0.10	0.07
Agriculture	N/A	2	0.18	0.10	0.07

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 8

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

USBR'S DIMENSIONLESS URBAN UNIT HYDROGRAPH

Ordinate Number	Time t in % of L_g + 0.5D	q
1	0	0.00
2	5	0.64
3	10	1.56
4	15	2.52
5	20	3.57
6	25	4.36
7	30	5.80
8	35	6.95
9	40	8.38
10	45	9.87
11	50	11.52
12	55	13.19
13	60	15.18
14	65	17.32
15	70	19.27
16	75	19.74
17	80	20.00
18	85	19.74
19	90	19.27
20	95	17.72
21	100	16.12

TABLE 8

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

USBR'S DIMENSIONLESS URBAN UNIT HYDROGRAPH

Ordinate Number	Time t in % of L_g + 0.5D	q
22	105	14.50
23	110	13.08
24	115	12.19
25	120	11.31
26	125	10.27
27	130	9.63
28	135	8.96
29	140	8.27
30	145	7.75
31	150	7.22
32	155	6.75
33	160	6.27
34	165	5.94
35	170	5.55
36	175	5.24
37	180	4.92
38	185	4.63
39	190	4.39
40	195	4.18
41	200	3.93
42	205	3.73

TABLE 8

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

USBR'S DIMENSIONLESS URBAN UNIT HYDROGRAPH

Ordinate Number	Time t in % of L_g + 0.5D	q
43	210	3.55
44	215	3.37
45	220	3.24
46	225	3.04
47	230	2.93
48	235	2.75
49	240	2.67
50	245	2.53
51	250	2.47
52	255	2.37
53	260	2.30
54	265	2.21
55	270	2.12
56	275	2.04
57	280	1.98
58	285	1.90
59	290	1.83
60	295	1.78
61	300	1.71
62	305	1.64
63	310	1.60

TABLE 8

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

USBR'S DIMENSIONLESS URBAN UNIT HYDROGRAPH

Ordinate Number	Time t in % of L_g + 0.5D	q
64	315	1.53
65	320	1.49
66	325	1.42
67	330	1.39
68	335	1.32
69	340	1.28
70	345	1.23
71	350	1.21
72	355	1.15
73	360	1.11
74	365	1.07
75	370	1.03
76	375	1.00
77	380	0.97
78	385	0.93
79	390	0.90
80	395	0.87
81	400	0.84
82	405	0.81
83	410	0.78
84	415	0.75

TABLE 8

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

USBR'S DIMENSIONLESS URBAN UNIT HYDROGRAPH

Ordinate Number	Time t in % of L_g + 0.5D	q
85	420	0.73
86	425	0.69
87	430	0.67
88	435	0.64
89	440	0.62
90	445	0.60
91	450	0.58
92	455	0.56
93	460	0.54
94	465	0.52
95	470	0.50
96	475	0.49
97	480	0.48
98	485	0.46
99	490	0.45
100	495	0.43
101	500	0.41
102	505	0.40
103	510	0.39
104	515	0.37
105	520	0.36

TABLE 8

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

USBR'S DIMENSIONLESS URBAN UNIT HYDROGRAPH

Ordinate Number	Time t in % of L_g + 0.5D	q
106	525	0.34
107	530	0.33
108	535	0.32
109	540	0.31
110	545	0.30
111	550	0.29
112	555	0.28
113	560	0.27
114	565	0.26
115	570	0.25
116	575	0.24
117	580	0.24
118	585	0.23
119	590	0.22
120	595	0.21
121	600	0.21

TABLE 9

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

BASIN "n" FOR UNIT HYDROGRAPH LAG EQUATION

Basin Land Use	Percent Impervious	Channelization Description	
		Developed Pipe/Channel	Undeveloped Natural
Highways, Parking	95	0.030	0.067
Commercial, Offices	90	0.031	0.070
Intensive Industrial	85	0.032	0.071
Apartments, High Density Res.	80	0.033	0.072
Mobile Home Park	75	0.034	0.073
Condominiums, Med. Density Res.	70	0.035	0.074
Residential 8-10 du/acre (20-25 du/ha), Ext Industrial	60	0.037	0.076
Residential 6-8 du/acre (15-20 du/ha), Low Density Res., School	50	0.040	0.080
Residential 4-6 du/acre (10-15 du/ha)	40	0.042	0.084
Residential 3-4 du/acre (7.5-10 du/ha)	30	0.046	0.088
Residential 2-3 du/acre (5-7.5 du/ha)	25	0.050	0.090
Residential 1-2 du/acre (2.5-5 du/ha)	20	0.053	0.093
Residential .5-1 du/acre (1-2.5 du/ha)	15	0.056	0.096
Residential .2-.5 du/acre (0.5-1 du/ha), Ag Res.	10	0.060	0.100
Residential < .2 du/acre (0.5 du/ha), Recreation	5	0.065	0.110
Open Space, Grassland, Ag	2	0.070	0.115
Open Space, Woodland, Natural	1	0.075	0.120
Dense Oak, Shrubs, Vines	1	0.080	0.150
Shaded values are normally not used.			

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 10

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

PARAMETERS FOR OVERLAND FLOW
WITH FLOW DEPTHS LESS THAN 2 INCHES (50 mm)

Surface	Overland "n"	Distance, ft (m)
Pavement - Smooth	0.02	50 (15)
Pavement - Rough/Cracked	0.05	50 (15)
Bare Soil - Newly Graded Areas	0.10	100 (30)
Range - Heavily Grazed	0.15	100 (30)
Turf - 1-2"/Lawns/Golf Course	0.20	100 (30)
Turf - 2-4"/Parks/Medians/Pasture	0.30	200 (60)
Turf 4-6"/Natural Grassland	0.40	200 (60)
Few Trees - Grass Undergrowth	0.50	300 (90)
Scattered Trees - Weed/Shrub Undergrowth	0.60	300 (90)
Numerous Trees - Dense Undergrowth	0.80	300 (90)

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 11

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

OVERLAND FLOW PRECIPITATION INTENSITY

Design Frequency (yr)	Precipitation Intensity in/hr (mm/hr)	C	Initial Estimates	
			T_O = 5 min in/hr (mm/hr)	T_O = 10 min in/hr (mm/hr)
2	$i = CT_O^{-0.519}$	3.8 (96.5)	1.65 (41.9)	1.15 (29.2)
5	$i = CT_O^{-0.558}$	6.3 (160)	2.57 (65.3)	1.74 (44.2)
10	$i = CT_O^{-0.576}$	8.13 (206.5)	3.22 (81.8)	2.16 (54.9)
25	$i = CT_O^{-0.601}$	16 (279.4)	4.18 (106.2)	2.76 (70.1)
50	$i = CT_O^{-0.620}$	13.6 (345)	4.84 (122.9)	3.12 (79.2)
100	$i = CT_O^{-0.627}$	15.8 (401)	5.76 (146.3)	3.73 (94.7)
200	$i = CT_O^{-0.642}$	18.4 (467)	6.55 (166.4)	4.20 (106.7)
500	$i = CT_O^{-0.652}$	22.1 (561)	7.74 (196.5)	4.92 (125.0)

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 12

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

STANDARD OVERLAND FLOW PARAMETERS

Land Use	Overland Flow Time, min	Slope ft/ft (m/m)	Overland "n"	Distance, ft
Commercial	3	-	-	-
Residential	9	-	-	-
Open Space	17-44[1]	.001-.01	0.30	200

[1]Computed using overland flow equation depending upon slope.

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 13

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

LAG MULTIPLICATION FACTORS FOR OVERLAND RELEASE

Frequency (years)	2	5	10	25	50	100	200	500
Multiplication Factor	1.0	1.0	1.0	1.1	1.2	1.3	1.4	1.5

Source: Sacramento City/County Drainage Manual, Volume 2, “Hydrology Standards,"
December 1996.

TABLE 14

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

HYDROGRAPH ROUTING OPTIONS

Method	Application	Required Parameters
Modified Puls	Channels Influenced by Backwater Channels With Available HEC-2 Storage-Discharge Information	Reach Length Velocity in Reach Storage-Discharge Information
	Reservoir Routing	Storage-Elevation Information Elevation-Discharge Information or Orifice Data and Spillway Data
Muskingum-Cunge	Channels With Insignificant Backwater Effects Channels Represented by Eight-Point Cross Sections Channels With a Standard Cross Section, Trapezoidal, Rectangular or Circular	Channel Length Channel Slope Manning's Roughness for Overbanks and Channel Cross-section data
Muskingum	Channels With Limited Cross-Sectional Information	Number of subreaches Muskingum "K" coefficient, hrs Muskingum "X" attenuation coefficient

Source: Sacramento City/County Drainage Manual, Volume 2, "Hydrology Standards," December 1996.

TABLE 15

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

RATIONAL METHOD CALCULATION SHEET

Inlet		Area A (acres)	C	CA (acres)	Sum CA (acres)	Time of Concentration																		Total L_c (min)	i (in/hr)	CiA	Q_{DESIGN} (cfs)	Pipe Capacity (cfs)	Remarks
						Overland Flow				Gutter Flow			Pipe Flow					Channel Flow											
From	To					L_o	S_o	Overland "n"	T_o	L_g	S_g	T_g	S_p	D (in)	V (fps)	L (ft)	T_p (min)	S_c	Bottom Width	Manning's "n"	Velocity (fps)	L_c (ft)	T_c (min)						

TABLE 16

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

LAND USE VS. EFFECTIVE PERCENT IMPERVIOUS AND 10-YEAR RUNOFF COEFFICIENTS FOR THE RATIONAL METHOD

Land Use from Aerial Photography	General Plan Land Use Designation	Effective % Impervious	10-Year Runoff Coefficient by Hydrologic Soil Group		
			B	C	D
Highways, Parking	Central Commercial (CC)	95	0.86	0.87	0.87
Commercial, Office	General Commercial (GC) Service Commercial (SC) Highway Commercial (HC) Business Park (BP)	90	0.82	0.84	0.85
Industrial	Industrial (I)	85	0.78	0.80	0.82
Apartments	N/A	80	0.74	0.77	0.79
Mobile Home Park	N/A	75	0.70	0.74	0.76
Condominiums	Med. Density Residential (MDR)	70	0.66	0.71	0.74
Residential: 8-10 du/acre (20-25 du/ha)	Medium/Low Density Residential (MLDR)	60	0.58	0.64	0.68
Residential: 6-8 du/acre (15-20 du/ha)	Neighborhood Preservation (NP) Planned Neighborhood (PN)	50	0.50	0.58	0.63

TABLE 16

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

LAND USE VS. EFFECTIVE PERCENT IMPERVIOUS AND 10-YEAR RUNOFF COEFFICIENTS FOR THE RATIONAL METHOD

Land Use from Aerial Photography	General Plan Land Use Designation	Effective % Impervious	10-Year Runoff Coefficient by Hydrologic Soil Group		
			B	C	D
Residential: 3-4 du/acre (7.5-10 du/ha)	N/A	30	0.34	0.45	0.52
Residential: 2-3 du/acre (5-7.5 du/ha)	Very-Low Density Residential (VLDR)	25	0.30	0.41	0.49
Residential: 1-2 du/acre (2.5-5 du/ha)	N/A	20	0.26	0.38	0.46
Residential: .5-1 du/acre (1-2.5 du/ha)	Rural Residential (RR)	15	0.22	0.35	0.43
Residential: .2-.5 du/acre (0.5-1 du/ha)	N/A	10	0.18	0.32	0.41
Residential: < .2 du/acre (.05 du/ha)	Agricultural Residential (AR)	5	0.14	0.28	0.38
Open Space, Grassland	N/A	2	0.12	0.26	0.36
Agriculture	N/A	2	0.26	0.41	0.51

TABLE 17

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

RATIONAL METHOD
RUNOFF COEFFICIENT FREQUENCY FACTORS

Return Period, years	Frequency Factor "F"
2	0.83
5	0.90
10	1.00
25	1.08
50	1.15
100	1.24

TABLE 18

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

RATIONAL METHOD
SUBBASIN RUNOFF COEFFICIENT CALCULATION SHEET

Land Use	Effective Percent Impervious	Hydrologic Soil Group B			Hydrologic Soil Group C			Hydrologic Soil Group D		
		Runoff Coeff (C)	Area (acres)	F X C X Area[1]	Runoff Coeff (C)	Area (acres)	F X C X Area[1]	Runoff Coeff (C)	Area (acres)	F X C X Area[1]
Central Commercial (CC)	95	0.86			0.87			0.87		
General Commercial (GC)	90	0.82			0.84			0.85		
Service Commercial (SC)	90	0.82			0.84			0.85		
Highway Commercial (HC)	90	0.82			0.84			0.85		
Business Park (BP)	90	0.82			0.87			0.85		
Industrial (I)	85	0.78			0.80			0.82		
Apartments	80	0.74			0.77			0.79		
Mobile Home Park	75	0.70			0.74			0.76		
Medium Density Res. (MDR)	70	0.66			0.71			0.74		
Medium/Low Density Res. (MLDR)	60	0.58			0.64			0.68		
Neighborhood Preservation (NP)	50	0.50			0.58			0.63		
Planned Neighborhood (PN)	50	0.50			0.58			0.63		
Low Density Residential (LDR)	40	0.42			0.51			0.57		
Residential, 3-4 du/acre	30	0.34			0.45			0.52		
Very Low Density Residential (VLDR)	25	0.30			0.41			0.49		
Residential, 1-2 du/acre	20	0.26			0.38			0.46		
Rural Residential (RR)	15	0.22			0.35			0.43		
Residential, 0.2-.5 du/acre	10	0.18			0.32			0.41		
Agricultural Residential (AR)	5	0.14			0.28			0.38		
Open Space, Grassland	2	0.12			0.26			0.36		
Agricultural	2	0.26			0.41			0.51		
TOTALS			0.00	0.00		0.00	0.00		0.00	0.00

Total Area 0.00
Sum (Coeff X Area) 0.00
Weighted Subbasin
Runoff Coefficient Sum (Coeff x Area)/Total Area

[1]Apply Runoff Coefficient Frequency F Factor of 0.83, 0.90, 1.00, 1.08, 1.15, and 1.24 to 10-Year Runoff Coefficient for design storm return periods of 2, 5, 10, 25, 50, and 100 years, respectively.

TABLE 19

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK/DRY CREEK SUBBASINS

EQUIVALENT ROUGHNESS COEFFICIENT FOR CALCULATION OF HYDRAULIC GRADE LINE FOR STORM DRAINAGE DESIGN

Pipe Material	Base Manning's Roughness Coefficient, nbase
Corrugated Metal	0.024
Concrete	0.015

Equivalent Entrance Loss Adjustment: $n_1 = \left(\frac{0.087d^{4/3}}{lg}\right)^{1/2}$

Equivalent Exit Loss Adjustment: $n_2 = \left(\frac{0.174d^{4/3}}{lg}\right)^{1/2}$

Where:
d = pipe diameter (ft.)
l = pipe length (ft.)
g = 32.2 ft./s^2

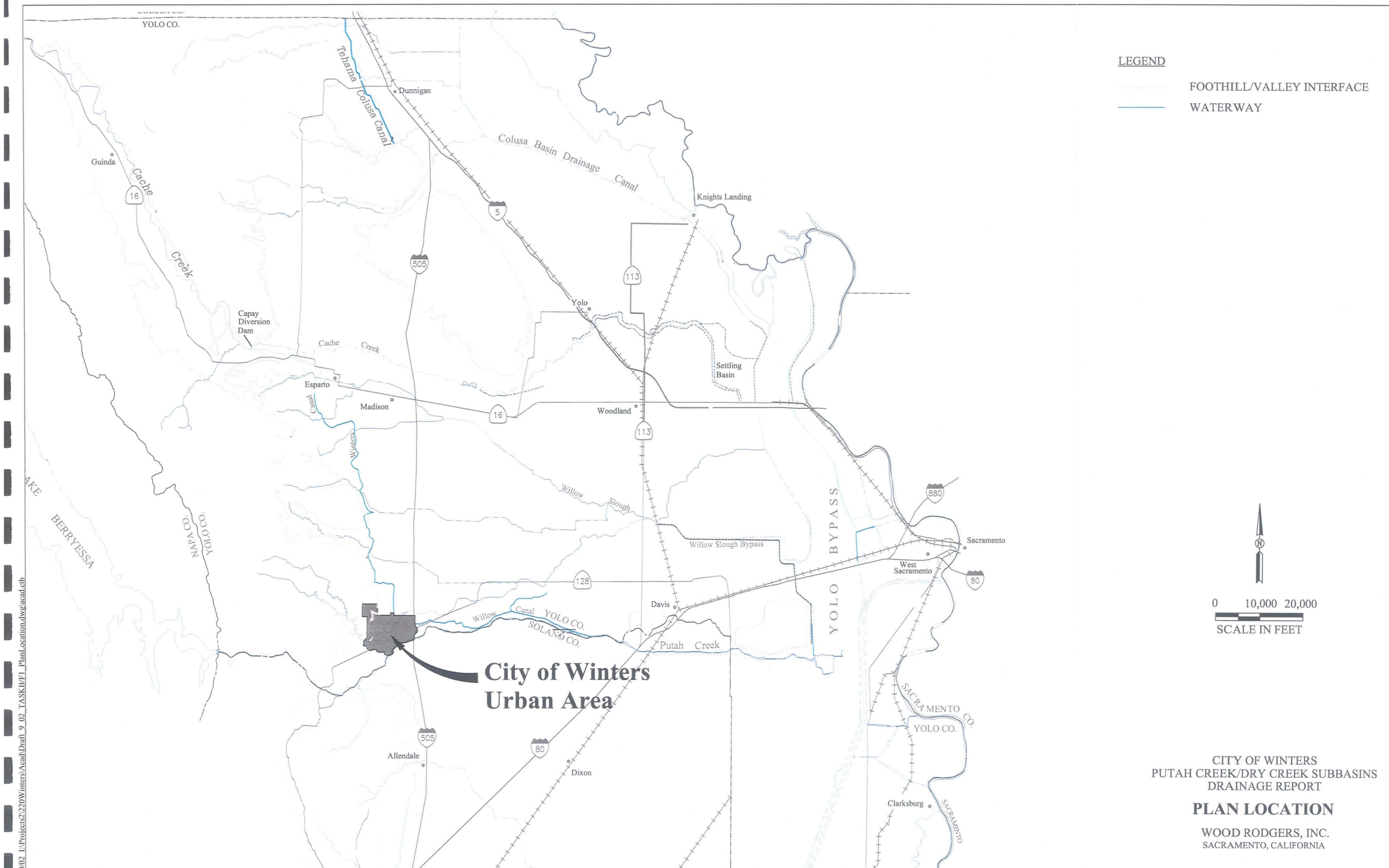
$n_{total} = n_{base} + n_1 + n_2$

Source: Chow, Ven Te, *Open Channel Hydraulics*, 1959.

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Figures

Figures

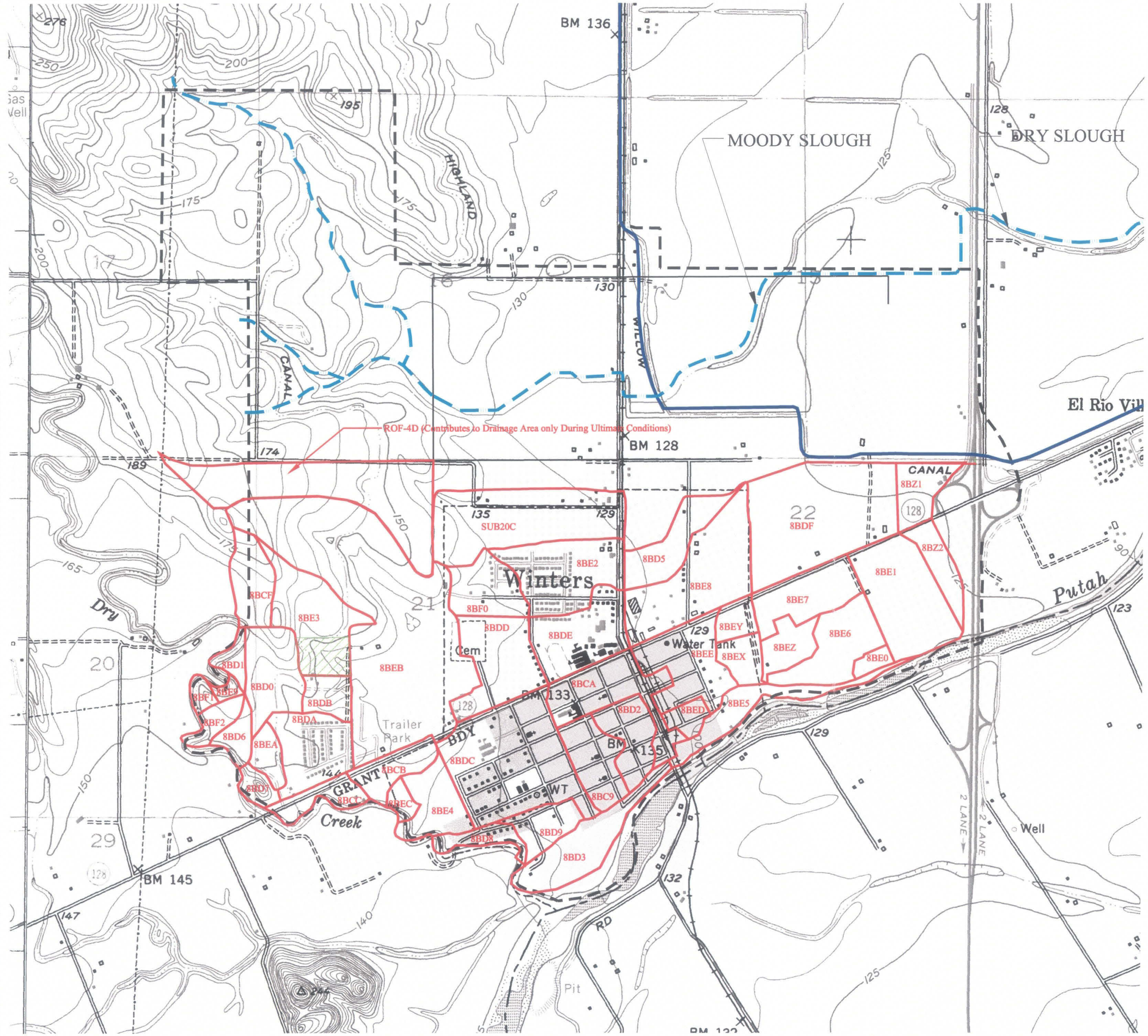


CITY OF WINTERS
PUTAH CREEK/DRY CREEK SUBBASINS
DRAINAGE REPORT

PLAN LOCATION

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 1



LEGEND

SILVER RIDGE ESTATES PROPERTY WHICH DRAINS TO "OFF-2D" IN CARTER RANCH HEC-HMS ANALYSIS

URBAN LIMIT

EXISTING CREEK

EXISTING WILLOW CANAL

PROPOSED SUBBASIN BOUNDARIES

NOTE:

The existing drainage and irrigation facilities presented on this plan are approximate for schematic purposes only.

SOURCE:

Topographic mapping is United States Geologic Survey Quadrangle Maps, National Geodetic Vertical Datum of 1929.

0 750 1500

SCALE IN FEET

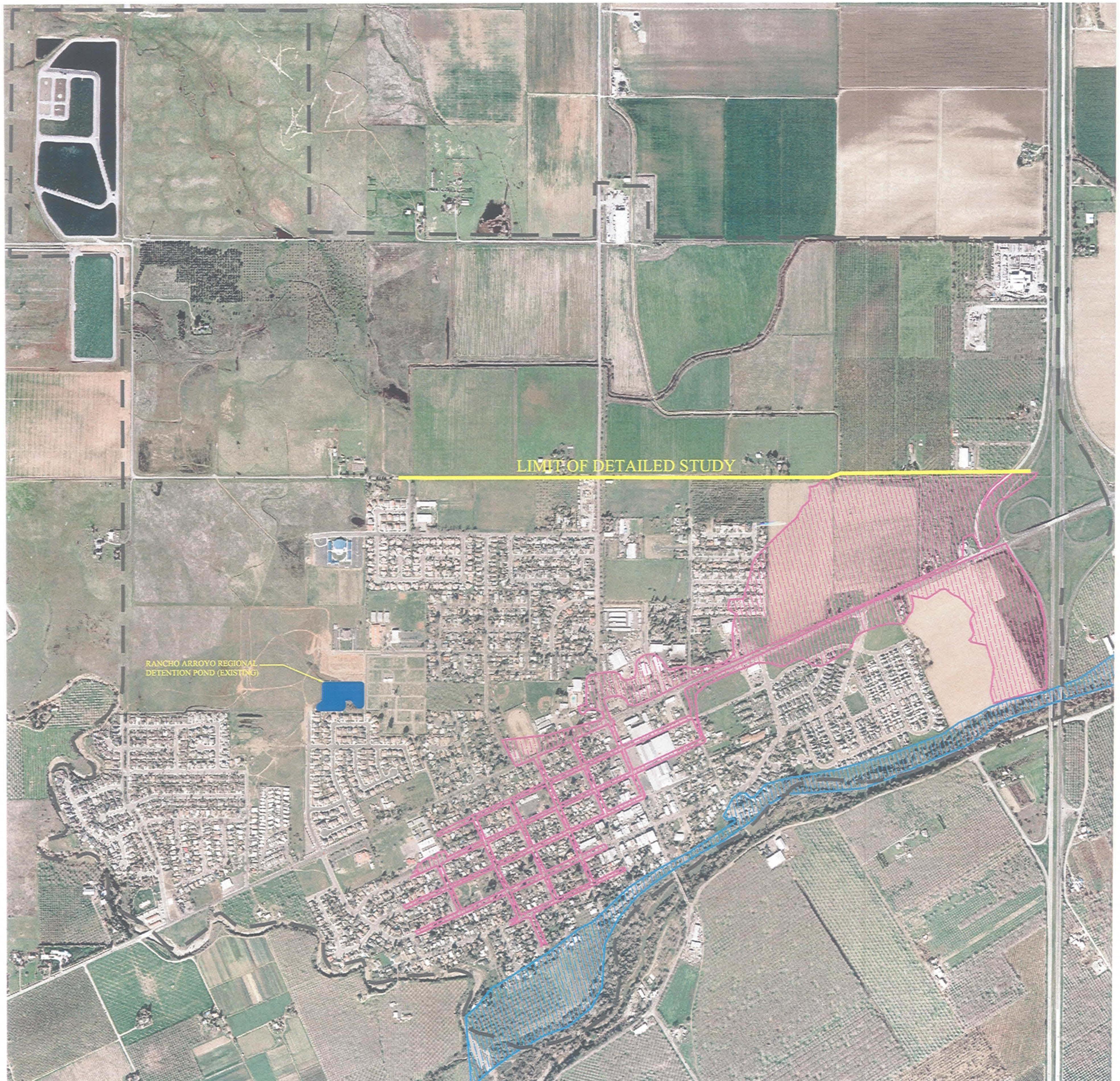
CITY OF WINTERS
PUTAH CREEK/DRY CREEK SUBBASINS
DRAINAGE REPORT

SUBBASIN BOUNDARIES

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 2

10/02 I:\Projects\220 Winters\Acad\Draft 9 02 TASKB\F5 SubBasin Boundaries.dwg\FIG5(tab)\None.ctb



LEGEND

- URBAN LIMIT
- 100-YEAR FLOODPLAIN BOUNDARY
- APPROXIMATE REVISED EXISTING 100-YEAR FLOODPLAIN
- FEMA 100-YEAR FLOODPLAIN BOUNDARY
- FEMA 100-YEAR FLOODPLAIN
- DETENTION/WATER QUALITY POND

NOTE:

The extent of 100-year floodplain boundary within the study area has been revised in accordance with the analysis performed by Wood Rodgers, Inc.

SOURCE:

Topographic mapping is United States Geologic Survey Quadrangle Maps, National Geodetic Vertical Datum of 1929.

N

0 600 1200

SCALE IN FEET

CITY OF WINTERS
PUTAH CREEK/DRY CREEK SUBBASINS
DRAINAGE REPORT

AERIAL REPRESENTATION OF EXISTING CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 3

SOURCE:
City of Winters - General Plan - May 1992

CITY OF WINTERS
PUTAH CREEK/DRY CREEK SUBBASINS
DRAINAGE REPORT

ULTIMATE LAND USE

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

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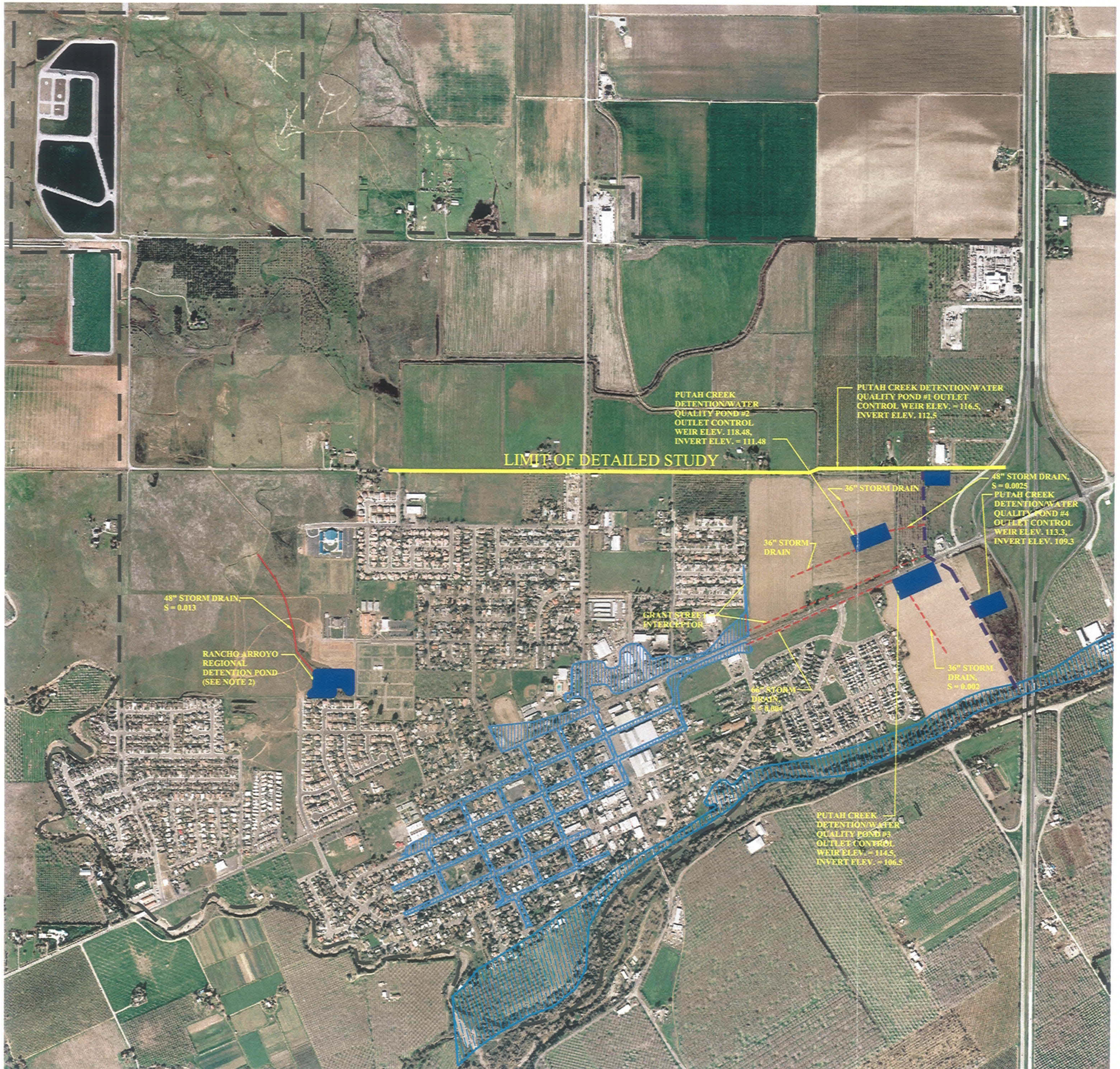
FIGURE 4



LEGEND

- DETENTION/WATER QUALITY POND
- URBAN LIMIT
- PROPOSED PIPE
- OPEN CHANNEL
- RESIDUAL 100-YEAR FLOODPLAIN 3
- FEMA 100-YEAR FLOODPLAIN BOUNDARY
- FEMA 100-YEAR FLOODPLAIN

NOTES:

1. The existing and proposed facilities presented on this plan are for illustrative purposes only.
2. A storm water quality treatment component is proposed to be added to the existing pond.
3. Residual 100-year floodplain indicated, detailed topographic mapping is required to determine the presence or extent of a residual floodplain.

SOURCE:

Topographic mapping is United States Geologic Survey Quadrangle Maps, National Geodetic Vertical Datum of 1929.

N

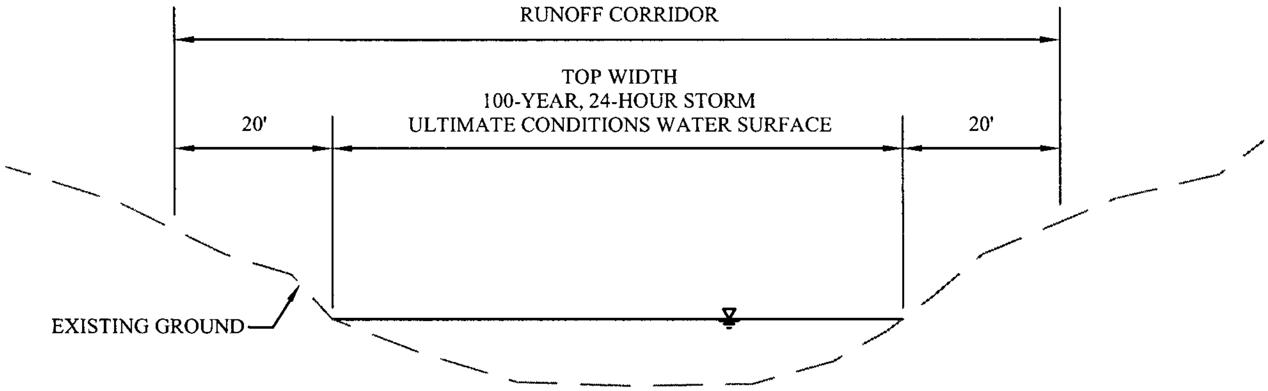
0 600 1200

SCALE IN FEET

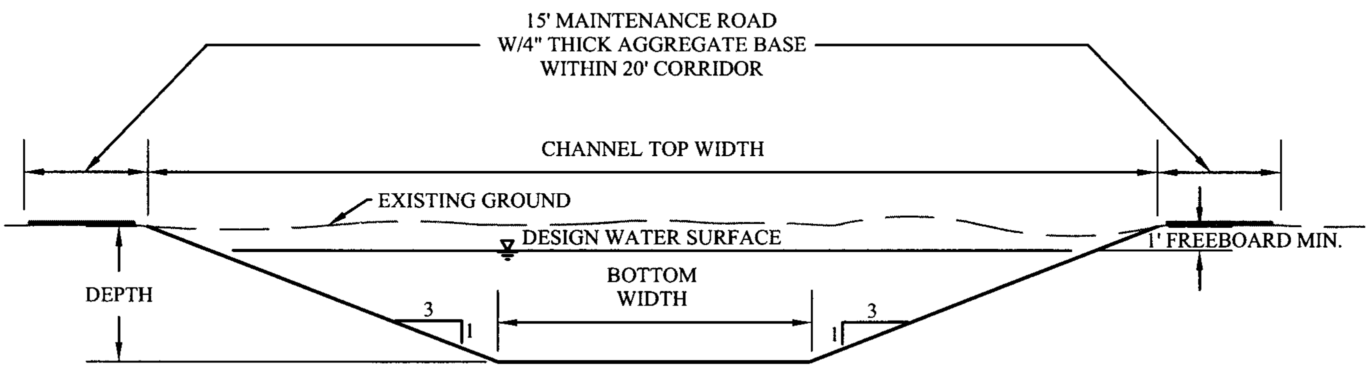
CITY OF WINTERS
PUTAH CREEK/DRY CREEK SUBBASINS
DRAINAGE REPORT

ULTIMATE CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA



RUNOFF CORRIDOR
NOT TO SCALE



NOTE:
CHANNEL DIMENSIONS WILL VARY ACCORDING TO
DESIGN HYDRAULIC CAPACITY.

TYPICAL CHANNEL CROSS SECTION
NOT TO SCALE

CITY OF WINTERS
PUTAH CREEK/DRY CREEK SUBBASINS
DRAINAGE REPORT

DRAINAGE CHANNELS
TYPICAL SECTIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

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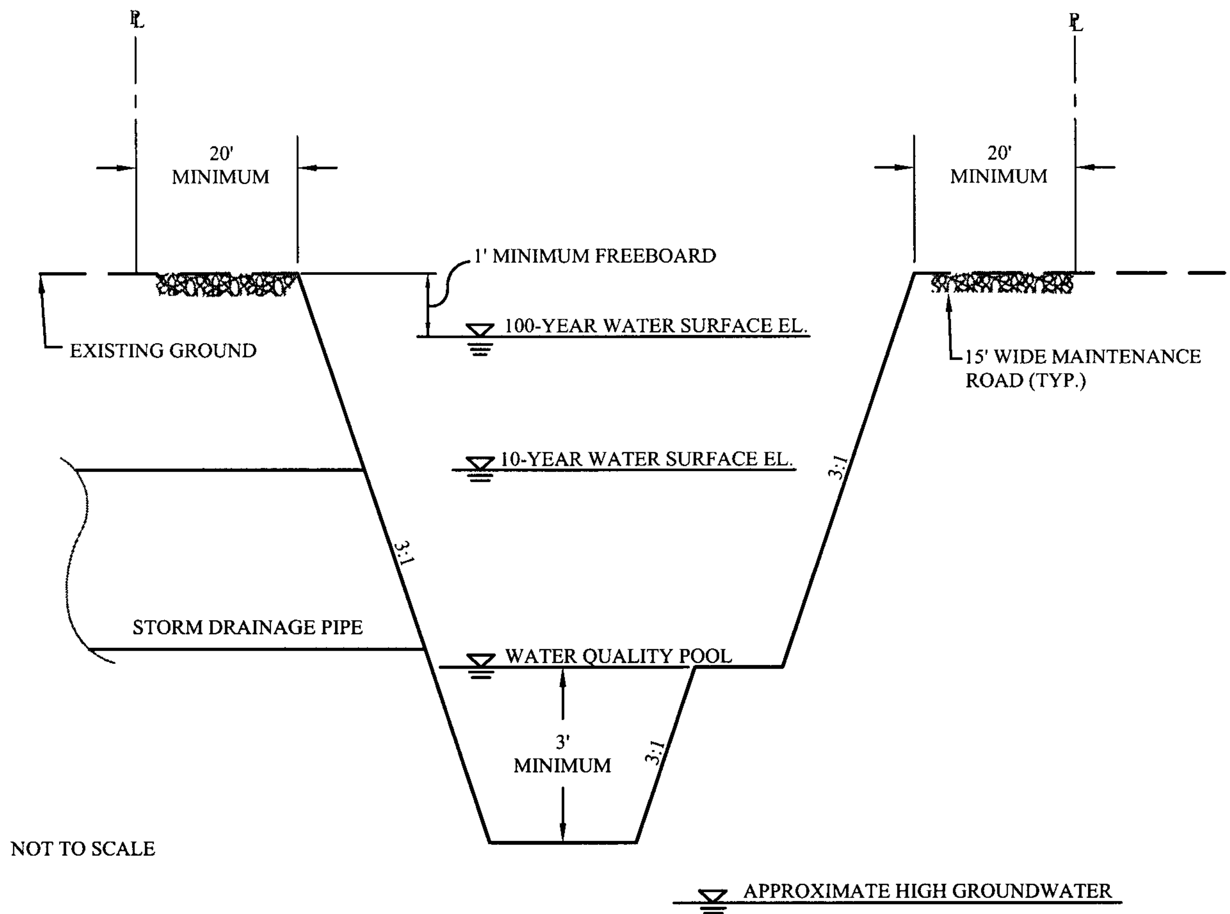

NOT TO SCALE

CITY OF WINTERS
PUTAH CREEK/DRY CREEK SUBBASINS
DRAINAGE REPORT

TYPICAL DETENTION/WATER QUALITY POND

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

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FIGURE 7

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendices

Appendices

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix A

"City of Winters Drainage Master Plan," CH2M Hill, 1992

CITY OF WINTERS

STORM DRAINAGE MASTER PLAN

Prepared by:

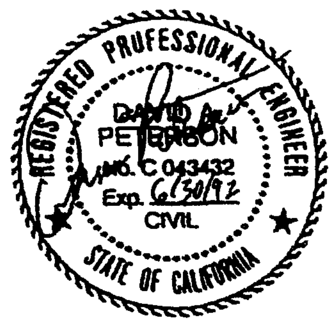
CH2M Hill

Adopted
May 19, 1992

STORM DRAINAGE MASTER PLAN

Prepared for

CITY OF WINTERS



This document has been prepared under the direction of a
Registered Professional Engineer

Prepared by

CH2M HILL

3840 Rosin Court, Suite 110
Sacramento, California 95834

May 8, 1992

SAC28849.FP.DP

INTRODUCTION

The purpose of the Storm Drain Master Plan is to identify drainage system requirements and facilities necessary to accommodate planned growth within the community of Winters until the year 2010. This report addresses only portions of the area within the Urban Limit Line draining to Dry Creek and Putah Creek. Portions of the area draining to Moody Slough have been set aside for future studies due to the identified 100-year flood plain in that area and the need for any drainage plan to be part of a comprehensive flood control solution.

This study is based on the Winters General Plan, adopted in May 1992, which provides for a population of 12,500 by the year 2010.

This master plan can be used to evaluate system requirements for development proposals and to assist in determining an equitable basis for collecting impact fees to fund improvements that have regional versus development-specific benefits. The improvements identified herein have been laid out and sized based on existing topographic data and projections of future development. System piping for the storm drainage master plan will be altered slightly with changes in road layout, actual development, and any significant land use alterations. Therefore, pipe sizing and costs as presented in the master plan are subject to further change during final design.

STUDY AREA AND LOCAL HYDROLOGY

The City of Winters is located in Yolo County approximately 30 miles west of Sacramento, California, and 7 miles east (downstream) of Monticello Dam and Lake Berryessa. Winters is on the western edge of the Sacramento River Valley against the eastern edge of the Coast Range Mountains.

Portions of the Dry Creek, Putah Creek, Chickahominy Slough, and Moody Slough drainages affect drainage in the City of Winters (see Figure 1).

Most flooding problems in the vicinity of the City have been caused in part by impeded flow in Moody and Chickahominy Sloughs. Limited channel capacity and culvert capacity at County Road 89 and I-505 are the main contributors to flooding in Moody Slough. Several reaches of Chickahominy Slough are also undersized. The channel has been straightened to a west-east path, which is not directly downslope. As a result, when the channel overtops, the flows move southeasterly away from the channel until they hit the elevated I-505. which sends the flows south toward Winters.

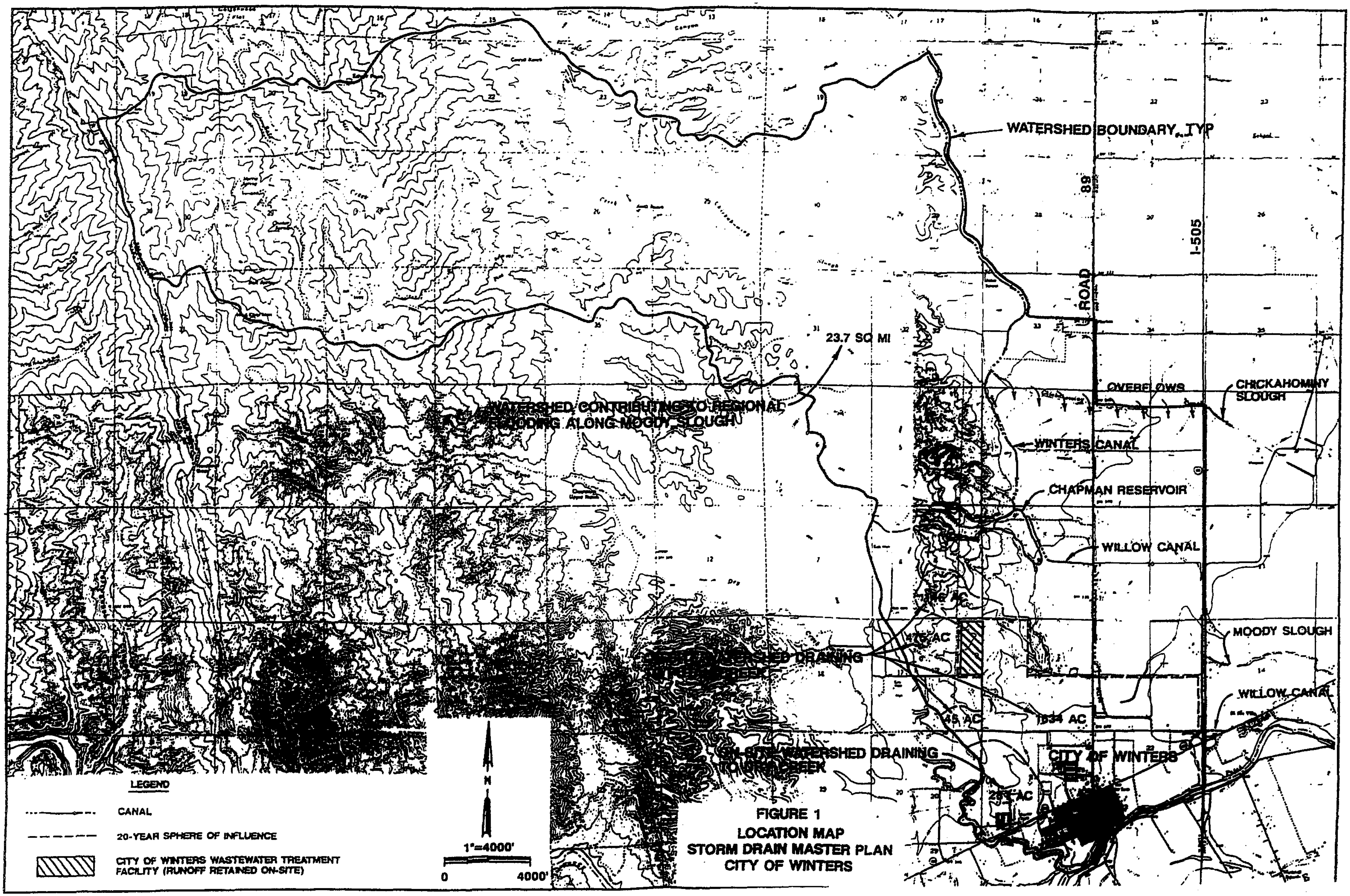


FIGURE 1
LOCATION MAP
STORM DRAIN MASTER PLAN
CITY OF WINTERS

The Winters Canal is another source of flood water to the study area. The canal, which is owned and operated by the Yolo County Flood Control and Water Conservation District, transports water from Cache Creek along the base of the Coast Range foothills for irrigation. The canal terminates at Chapman Reservoir, just north of the City of Winters. Outflows from Chapman Reservoir enter Willow Canal, flow south toward Winters, then east along Putah Creek to the City of Davis.

The canals cross a number of natural drainages. Following the irrigation season, wasteways are opened at the major drainages, and gates in the canals are closed to force winter flood waters out the wasteways into the natural drainages. One such wasteway exists about 3 miles north of Chickahominy Slough at Union School Slough. Over the next 3 miles, the canal picks up flood waters from minor drainages to the west and discharges to Chickahominy Slough. The gate at the canal siphon under Chickahominy Slough is closed in the winter so the canal is dry immediately south. It then intercepts minor drainages over the next mile until it flows into Chapman Reservoir. The Willow Canal is similarly wasted to Moody Slough in the winter.

Since the completion of Monticello Dam in 1957, flooding on Putah Creek has been virtually eliminated. The lowered water surface elevations in Putah Creek have also resulted in a lowering of flood water elevations in Dry Creek near the City of Winters. Both of these creeks have the capacity to contain the 500-year flood within their banks (SCS, 1976).

Winters is about 130 feet above mean sea level, with a general climate characterized by hot, dry summers and wet, cool winters typical of the California Central Valley. The average annual temperature is about 60°F. Temperatures in July, the warmest month, range from an average minimum of 56°F to an average maximum of 97°F. In January, the coolest month, the average minimum temperature is 36°F, and the average maximum is 55°F.

The average annual precipitation for Winters is about 20 inches (Table 1). Sixty percent of this falls during the winter flood season of December, January, and February. Only 0.2 percent of the precipitation occurs in the summer months.

Winters' population in 1990 was 4,693.

Table 1
City of Winters
Mean Annual Rainfall

Year	Total Rainfall (inches)	Year	Total Rainfall (inches)
1943	19.9	1967	35.5
1944	16.2	1968	15.1
1945	16.8	1969	29.4
1946	14.6	1970	25.8
1947	11.5	1971	18.8
1948	13.5	1972	10.9
1949	14.0	1973	33.5
1950	13.4	1974	21.4
1951	20.5	1975	22.4
1952	22.9	1976	6.2
1953	19.6	1977	9.4
1954	16.4	1978	34.4
1955	13.6	1979	19.5
1956	29.4	1980	32.8
1957	11.4	1981	15.3
1958	34.7	1982	37.0
1959	15.9	1983	43.8
1960	17.0	1984	20.3
1961	14.3	1985	17.8
1962	19.3	1986	32.8
1963	28.7	1987	12.3
1964	13.7	1988	18.4
1965	23.3	1989	15.1
1966	14.6	1990	14.4
Number of years			48
Average			20.4
Maximum			43.8
Minimum			6.2

Note: Records for Winters gage taken from National Weather Service, Department of Water Resources, and Winters Express data for water years beginning October 1 and ending September 30.

DRAINAGE FACILITIES

This portion of the master plan evaluates the adequacy of the existing storm drain system and provides a basis for sizing pipes within the City's Urban Limit Line. Storm drainage pipes will discharge to either Dry Creek or Putah Creek. Facilities have not been planned to provide storm drainage in the Moody Slough drainage basin. This area has been set aside for future study due to the identified 100-year flood plain and the need for any drainage plan in that area to be part of a comprehensive flood control solution.

SIZING CRITERIA

Sizing criteria most applicable to the City of Winters were developed from two sources: the Yolo County Basic Hydrology and Drainage Design Procedure (1965) and the City of Davis Engineering Design Standards (1990).

Land Use

For purposes of estimating runoff and determining drainage improvements, all land use was assumed to be at the ultimate level of development within the Urban Limit Line, based on the City of Winters General Plan, adopted early in May 1992.

Hydrology

A 10-year recurrence interval storm was used for analyzing the existing storm drain system and for sizing storm drains that will be needed in the future. Flows exceeding the capacity of storm drains were assumed to be carried by streets or flood easements. Finished floor elevations in new buildings must be constructed at least 1 foot above the level of the 100-year flood.

Long-term short-duration precipitation depth-duration-frequency data are unavailable for Winters. Nearby weather stations were evaluated based on years of record, annual precipitation, and topographic similarities. For this study, the rainfall data from Davis Station 2WSW were used. This station has 30 years of data for the 15-minute duration and 45 years of data for storm durations exceeding 1 hour. Table 2 presents the short-duration precipitation data, and Figure 2 presents the 10- and 100-year rainfall intensity curves.

Table 2
Short Duration Precipitation Depth-Duration-Frequency Table

Return Period In Years	Precipitation Depth (inches) for Indicated Duration (M = Minutes, H = Hours)									
	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H
2	.12	.17	.23	.31	.41	.64	.77	1.12	1.49	1.93
5	.16	.23	.31	.41	.55	.85	1.04	1.51	2.00	2.59
10	.19	.27	.36	.48	.64	.99	1.21	1.75	2.33	3.01
20	.22	.31	.41	.54	.73	1.12	1.37	1.98	2.64	3.41
25	.22	.32	.42	.56	.76	1.17	1.42	2.05	2.73	3.53
40	.23	.33	.43	.57	.77	1.18	1.44	2.09	2.78	3.59
50	.25	.35	.47	.62	.83	1.29	1.57	2.27	3.02	3.90
100	.27	.39	.51	.68	.91	1.41	1.71	2.48	3.29	4.26
200	.29	.42	.55	.73	.99	1.52	1.85	2.68	3.56	4.61
1,000	.34	.49	.64	.86	1.15	1.78	2.17	3.14	4.17	5.39
10,000	.41	.59	.77	1.03	1.39	2.14	2.60	3.77	5.01	6.47
PMP[a]	.81	1.16	1.52	2.04	2.73	4.22	5.13	7.43	9.88	12.78
No. of Years Record	5	5	30	32	45	45	45	45	45	45
Record Year	1967	1967	1958	1947	1982	1982	1982	1962	1962	1962
Record Maximum	.220	.250	.460	.580	.900	2.200	2.200	2.430	3.540	5.190

[a]PMP = Probable Maximum Precipitation. Source: Davis 2WSW Station.

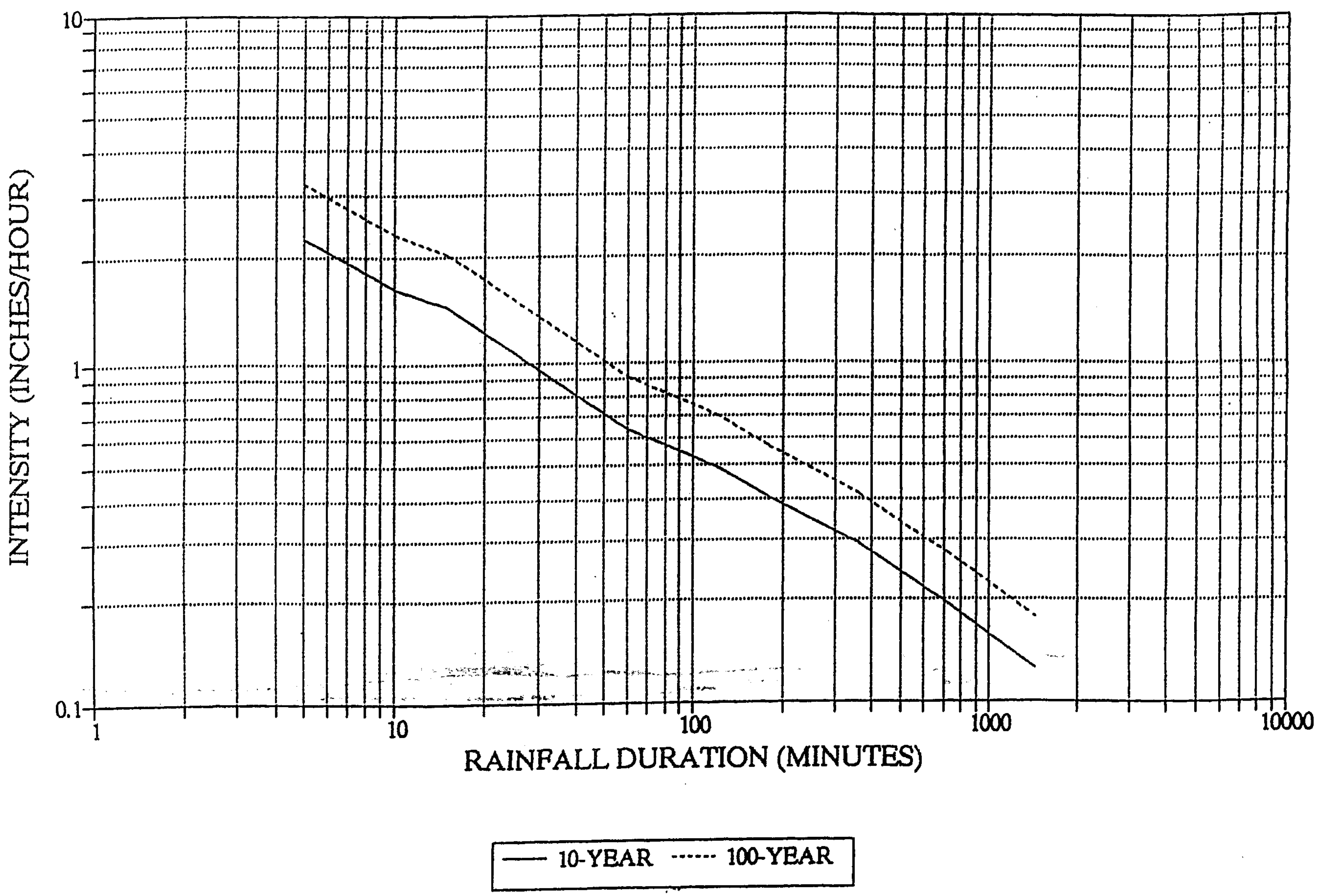


Figure 2
RAINFALL INTENSITY CURVES (DAVIS 2WSW)

The rational method was used to determine flow rates for the 10-year storm in basins of less than 640 acres (County of Yolo, 1965). The runoff flow rate was computed as:

$$Q = GCIA$$

where	Q	=	Design storm runoff (cfs).
	G	=	Geographic factor of 1.20. Used to transfer rainfall intensity data from Davis to Winters.
	C	=	Composite runoff coefficient representing the portion of rainfall that becomes runoff.
	I	=	Design storm rainfall intensity from Figure 2 (inches/hour).
	A	=	Upstream tributary drainage area (acres).

The following runoff coefficients were used in this study based on land use (City of Davis, 1990 and County of Yolo, 1965):

Land Use	**"C" for the 10-Year Storm**
Impervious	0.95
Commercial	0.80
Single-Family	0.45
Undeveloped Land	0.46
School	0.35
Park	0.25

The time of concentration (Tc) required to determine rainfall intensity (I) is based on the amount of time it takes runoff to travel from the farthest portions of a drainage basin to the facility being sized. Tc is the sum of the time components of overland flow, channel or gutter flow, and pipe flow.

For the purposes of this evaluation, Tc was determined using the following assumptions:

> Minimum time of concentration was assumed to be 10 minutes for residential and commercial areas.
>
> Time of overland flow (To) in residential areas was estimated to be 10 minutes. This was based on an average lot size of 7,500 square feet (Martin-Carpenter Associates, 1988), with 40 percent of the lot in impervious areas and 60 percent in lawn/green areas.

- To for undeveloped areas was based on the Soil Conservation Service (SCS) Method:

$$To = \frac{L^{0.8}(S_t+1)^{0.7}}{190\,S^{0.5}}$$

where
- To = Overland flow travel time in minutes
- L = Length of overland flow path in feet
- S = Slope of overland flow in feet per foot
- S_t = SCS soil water storage in inches and computed by the equation S_t = (1,000/CN) - 10 in which CN is the SCS curve number

- Time of gutter flow (Tg) was based on assuming a 4-inch depth of water in the gutter and a slope of 0.003 foot per foot (Yolo County, 1965).

- Time of channel or pipe flow was based on the Manning formula.

Pipe Flow

Analyses of the existing and future storm drain system assumed that all pipes were flowing full but not under pressure. All new pipes are assumed to be reinforced concrete pipe (RCP).

Pipe flow was estimated using Manning's formula:

$$Q = A\,\frac{1.486}{n}\,R^{2/3}\,S^{1/2}$$

where
- Q = Peak flow (cfs)
- n = Manning's n for RCP of 0.012, CMP of 0.024, and PVC of 0.010
- R = Hydraulic radius using full pipe flow is D/4 (feet)
- D = Diameter of pipe (feet)
- S = Slope (feet per foot)
- A = Area of pipe (square feet)

EXISTING SYSTEM

Capacity of the existing storm drain system was evaluated using a 10-year frequency storm. The existing system consists of 17 main lines installed at various times over the past 100 years. The majority of the storm drains consist of RCP with several short stubs of PVC and CMP. Pipe sizes range from 6 to 60 inches. All of the lines drain into Dry Creek or Putah Creek. The layout of the existing system is shown in Appendix A with estimated drainage areas provided in Appendix C.

Base mapping information for the analysis was taken from the following sources:

- Topography at 1-foot intervals—*Flood Hazard Analysis Study Area, City of Winters*. Laugenour and Meikle Civil Engineers, December 22, 1974.

- Layout and pipe sizes—*City of Winters Storm Drain System* (no date). Reviewed with City staff to determine flooding areas and pipe materials.

- Storm drain system inverts and slopes—*City of Winters Storm Drain System*. 1977.

- Regional drainage basins—*Monticello Dam Quadrangle, Photoinspected 1978*, and *Winters Quadrangle, Photoinspected 1973*. USGS 7.5-minute series maps.

As-built information does not exist for all pipes. What does exist was not field checked during this study because of time and budget limitations. Therefore, these preliminary results are of a general nature. Using criteria mentioned previously, the system was analyzed using a computer spreadsheet program. Results for the existing storm drain system appear in Appendix A. Improvements are referenced to main line numbers 900 through 5500. Each main line (i.e., 1500) is distinguished in Appendix A by circled node numbers (i.e., 1510, 1520, 1530, etc.) which are points at which flows were computed.

The analysis indicates that of the 17 main storm drain lines, 9 are undersized. Improvements required to provide 10-year storm flow capacity within the existing system are summarized in Table 3. This improvement scenario is contingent on future storm drain lines being installed west of Village Circle Street and west of the cemetery as part of future development in those areas. The total length of replacement pipe is approximately 15,000 feet of 18- to 42-inch pipe. The replacement pipes are highlighted over existing system pipes in Appendix B.

FUTURE SYSTEM

Projected growth for the City of Winters is described in the Winters General Plan, adopted in May 1992. The area requiring storm drainage facilities includes all the area within the Urban Limit Line defined in that plan.

Storm Drain Main Line[a]	Length of Pipe Required (feet)						
	18-inch	21-inch	24-inch	30-inch	36-inch	42-inch	Total
1500		250[b]	0	1,700	650		2,600
1800			450[b]				450
1900	1,000[b]						1,000
2000	400		250	800	1,650	1,650	4,750
2100	650	350	700				1,700
2200		700[b]	200[b]	500[b]			1,400
2400				400	1,150		1,550
2500	300		350	450			1,100
2600	450						450
Total	**2,800**	**1,300**	**1,950**	**3,850**	**3,450**	**1,650**	**15,000**

Table 3
Replacement of Existing Storm Drain Main Lines

[a]Main line numbers refer to those shown in Appendixes A and B.
[b]Low priority improvements due to near adequate existing capacity.

The final pipe layout is presented in Appendix B. Anticipated drainage basins are presented in Appendix C. Because of the relatively flat terrain, drainage areas may change based on final road alignments and housing development. There are three new major storm drain lines with drainage areas ranging from 16 to 132 acres.

In addition to the criteria presented in the Design Criteria section, the following assumptions were made:

- Storm drains consist of reinforced concrete pipe with a Manning's roughness value (n) of 0.012.
- Storm drains have a minimum cover of 3 feet and a maximum cover of 10 feet.
- Initial storm drain slope is set at 0.003 and adjusted to meet topography requirements.
- Pipe sizing is based on a 10-year frequency storm.
- Minimum velocity is 2.5 feet per second when flowing 50 percent full or greater (City of Davis, 1990).

- Minimum new pipe size is 18 inches.

- Manholes and catch basin spacing is 500 feet.

The analysis appearing in Appendix B indicates that new pipe sizes will vary from 18 to 54 inches. Table 4 presents a summary of the future storm drain mainlines, including the drainage areas and linear feet of pipe.

COST ESTIMATES

Basis of Cost Estimates

Costs have been estimated for the storm drain systems with order-of-magnitude accuracy. This type of estimate is expected to be accurate within +50 to -30 percent. Order-of-magnitude estimates are prepared without detailed engineering analysis of various system components or site data. The estimates are based on conceptual plans of the storm drain systems and general cost-curve information for the various components.

The cost estimates have been prepared for guidance in project evaluation and implementation from information available at the time the estimate was prepared. Final project costs and resulting feasibility will depend on a number of variable factors. As a result, final project costs will vary from the estimates presented in this section.

Future System Costs

Estimated capital costs for the future storm drain mainline system are summarized in Table 5. A description of the cost development for the various components follows. Pipeline costs were estimated using a cost estimating database prepared by CH2M HILL. Actual material costs were included for pipe and backfill materials expected to be used on the project. Standard crews and equipment were assumed based on information presented in cost estimating guides.

Sacramento area labor and equipment rates for March 1992 were used to develop crew costs for the estimate. Production rates for installing pipe and related components were determined from cost estimating guides and supplemented with field data collected for similar projects.

Also included in the pipeline costs are catch basins and manholes at 500-foot intervals and major structures at an interval of one per mile. The following items are not included as part of estimated costs: easements, clearing and grubbing, surface restoration, and disposal of excess soil. These costs are assumed to be accounted for in the overall development of the areas requiring drainage.

Table 4
Future Storm Drain Main Lines

Storm Drain Main Line[a]	Tributary Area (acres)	Length of Pipe Required (feet)												
		18-inch	21-inch	24-inch	30-inch	36-inch	42-inch	48-inch	54-inch	60-inch	72-inch	78-inch	84-inch	Total
1500				600	800									1,400
4000	115	500		2,100	1,600	750			1,000					5,950
5400	132			1,500	1,000			2,100						4,600
5500	16	700		100										800
Total		1,200		4,300	3,400	750		2,100	1,000					12,750

[a]Main line numbers refer to those shown in Appendix B.

Note: Catch basins and manholes are required at 500-foot intervals along main lines.

Table 5
Estimated Construction Costs for Future Storm Drain Main Line System

Pipe (inches)	Quantity	Unit	Unit Cost	Cost ($1,000)
18	1,200	linear ft	45	54
24	4,300	linear ft	57	245
30	3,400	linear ft	70	238
36	750	linear ft	88	66
48	2,100	linear ft	128	268
54	1,000	linear ft	146	146
Subtotal	**12,750**			**1,017**
Contingency (30%)				305
Subtotal				**1,322**
Engineering, legal, and administration (20%)				264
Total				**1,586**

Note: See Appendix B for location of improvements. Price levels are as of March 1992.

An allowance of 5 percent of the subtotal of the initial construction costs of the project components is included to account for the contractor's operational costs. These costs include mobilization and cleanup, field office, insurance, permits, overheads, and administration.

A construction cost contingency allowance of 30 percent of the subtotal estimated construction cost (including contractor's operational costs) is included. The 30 percent factor is recommended until more detailed engineering data are known. The resulting cost after application of the contingency allowance is the estimated construction cost.

Costs associated with engineering and construction management are included using an allowance of 17.5 percent of the construction cost. Administrative and legal costs associated with the project are estimated using an additional nondirect cost allowance of 2.5 percent of construction costs.

The estimated total project costs in Table 5 are the result of applying the nondirect costs to the estimated construction cost.

EXISTING SYSTEM COSTS

Estimated costs associated with replacement of 12,950 linear feet of existing storm drains on nine mains are presented in Table 6. These estimates were developed in the same manner as the future storm drain estimates, except a cost for surface restoration is included. Costs are based on the existing storm drains being removed and using the same alignment and grade for the new storm drains.

Table 6 Estimated Construction Costs for Replacement of Existing Storm Drain Main Lines				
Pipe (inches)	**Quantity**	**Unit**	**Unit Cost**	**Cost ($1,000)**
18	2,800	linear ft	67	188
21	1,300	linear ft	70	91
24	1,950	linear ft	77	150
30	3,850	linear ft	92	354
36	3,450	linear ft	109	376
42	1,650	linear ft	128	211
Subtotal	**15,000**			**1,370**
Contingency (30%)				411
Subtotal				**1,781**
Engineering, legal, and administration (20%)				357
Total				**2,138**
Note: See Appendix B for location of improvements. Price levels are for March 1992.				

REGIONAL FLOOD CONTROL

Much of the area within Winters' Urban Limit Line lies in a designated 100-year flood plain (Figure 3). This flood plain is included in the Federal Emergency Management Agency (FEMA) Flood Insurance Study (1980). Construction of structures within these areas can only be permitted if first floor elevations are at least 1 foot higher than the 100-year flood elevations, or the area must be removed from the 100-year flood plain by constructing some form of flood control project. For either scenario, construction must not raise flood elevations upstream by more than 1 foot, according to FEMA criteria. Many local agencies are requiring criteria more strict than the FEMA criteria. A more conservative criterion is avoidance of negative impacts to property owners both upstream and downstream of the project. Negative impacts could include increased ponding upstream or increased flows downstream due to elimination of flood storage. If the area is to be removed from the regulatory flood plain, a letter of map revision (LOMR) is required from FEMA.

A small area of designated flood plain occurs in a low area in and adjacent to the Winters cemetery. This area would be removed from the flood plain by grading the area to drain to the proposed storm drain system (Mainline 5400 in Appendix B).

The larger designated flood plain lies along Moody Slough and its surrounding areas. Measures required to remove this area from the flood plain will be identified in future studies.

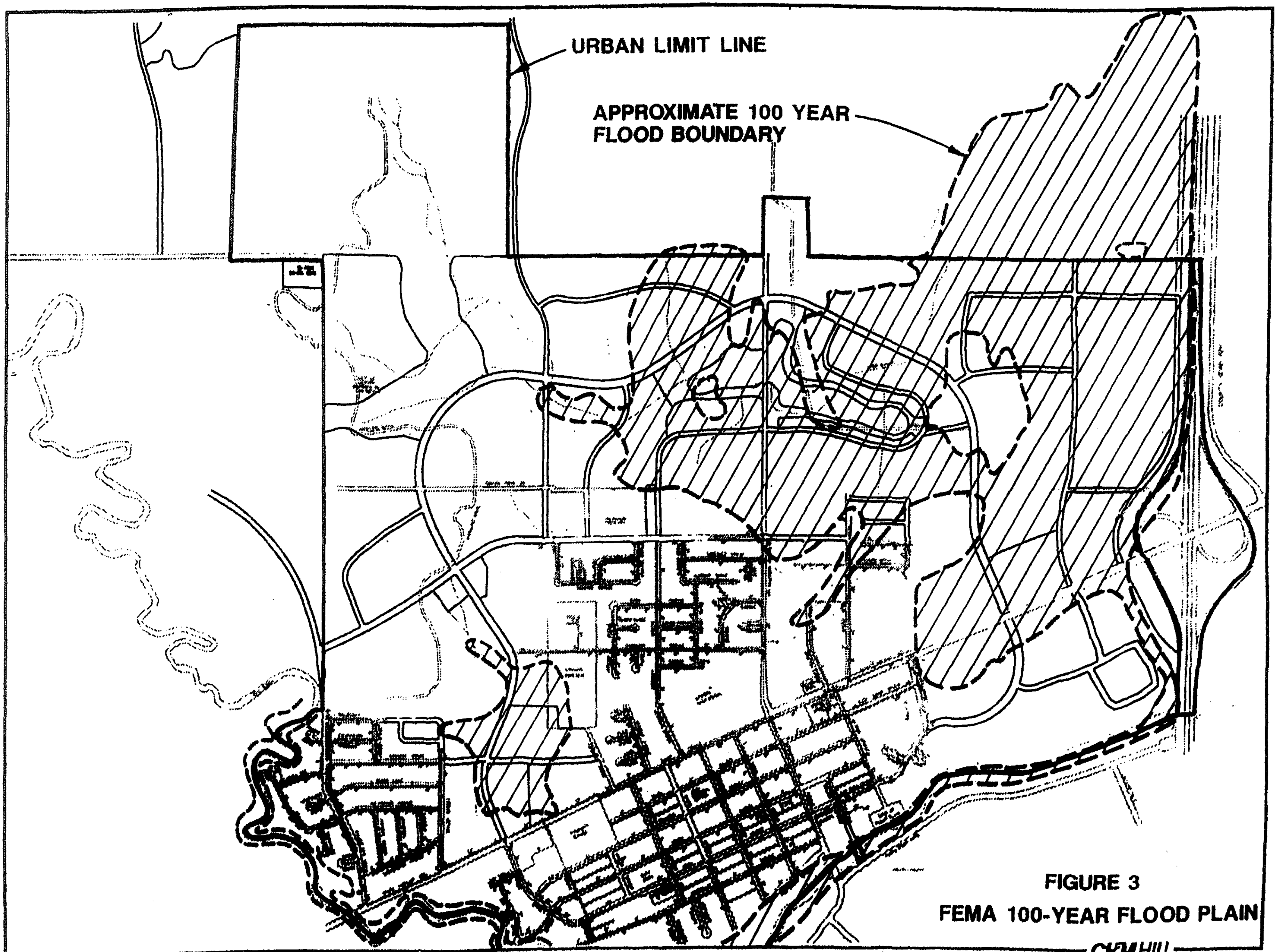


FIGURE 3
FEMA 100-YEAR FLOOD PLAIN

CH2M HILL

REFERENCES

City of Davis. *Engineering Design Standards (Preliminary).* March 22, 1990.

Federal Emergency Management Agency. *Flood Insurance Study, Yolo County, California, Unincorporated Areas.* June 1980.

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City of Winters. *General Plan.* Adopted May 1992.

County of Yolo. *Basic Hydrology and Drainage Design Procedure.* 1965.

Appendix A

EXISTING STORM DRAINS

APPENDIX A

STORM DRAIN CALCULATIONS - EXISTING SYSTEM

		Rational Equation Calculations 1/												Manning Equation Calculations For Pipe 2/						
Storm Drain Line	Upstream Node	Tributary Area (acres)	C	C*A	Summation of CA	Gutter Length (ft)	Pipe Length (ft)	To (min)	Tg (min)	Tp (min)	Tc (min)	I (in/hr)	Q (cfs)	D (in)	Slope (ft/ft)	V (ft/s)	Capacity Q (cfs)	Additional Q Req'd (cfs)	Replacement Pipe Dia. (in) 3/	Comments
						1000														Upstream Conditions
900	910	7.0	0.45	3.15	3.15	0	500	10	17	0.0	27	1.03	3.89	15	0.0033	3.3	4.03	0.00		
	920	0.0	0.00	0.00	3.15	0	0	0	0	2.5	29	0.98	3.70							@ Dry Creek
Total Acreage =		7.0																		
						800														Upstream Conditions
1000	1010	4.0	0.45	1.80	1.80	0	150	10	13	0.0	23	1.11	2.40	12	0.004	3.1	2.45	0.00		Assumed Slope
	1020	0.0	0.00	0.00	1.80	0	0	0	0	0.8	24	1.09	2.35							@ Dry Creek
Total Acreage =		4.0																		
						250														Upstream Conditions
1100	1110	1.6	0.45	0.72	0.72	0	150	10	4	0.0	15	1.44	1.24	12	0.004	3.1	2.45	0.00		Assumed Slope
	1120	0.0	0.00	0.00	0.72	0	0	0	0	0.8	15	1.44	1.24							@ Dry Creek
Total Acreage =		1.6																		
						300														Upstream Conditions
1200	1210	1.9	0.45	0.86	0.86	0	250	10	5	0.0	15	1.44	1.48	12	0.004	3.1	2.45	0.00		Assumed Slope
	1220	0.0	0.00	0.00	0.86	0	0	0	0	1.3	16	1.37	1.41							@ Dry Creek
Total Acreage =		1.9																		Upstream Conditions
						250														
1300	1310	2.8	0.45	1.26	1.26	0	200	10	4	0.0	15	1.44	2.18	12	0.004	3.1	2.45	0.00		Assumed Slope
	1320	1.4	0.45	0.63	1.89	0	250	0	0	1.1	16	1.38	3.13	12	0.004	3.1	2.45	0.68		Assumed Slope
	1330	0.0	0.00	0.00	1.89	0	0	0	0	1.3	17	1.32	2.99							@ Dry Creek
Total Acreage =		4.2																		Upstream Conditions
						500														
1400	1410	5.2	0.45	2.34	2.34	0	300	10	8	0.0	18	1.28	3.59	18	0.003	3.5	6.25	0.00		
	1420	4.1	0.45	1.85	4.19	0	250	0	0	1.4	20	1.23	6.18	24	0.001	2.5	7.77	0.00		
	1430	4.9	0.45	2.21	6.39	0	250	0	0	1.7	21	1.17	8.97	24	0.004	4.9	15.54	0.00		
	1440	7.4	0.45	3.33	9.72	0	1100	0	0	0.8	22	1.14	13.30	24	0.004	4.9	15.54	0.00		
	1450	5.5	0.45	2.48	12.20	0	350	0	0	3.7	26	1.04	15.22	30	0.004	5.7	28.17	0.00		Assumed Slope
	1460	0.0	0.00	0.00	12.20	0	0	0	0	1.0	27	1.02	14.93							@ Dry Creek
Total Acreage =		27.1																		Upstream Conditions
						550														
1500	1502	20.0	0.42	8.33	8.33	0	600	10	9	0.0	19	1.25	12.50	24	0.0027	4.1	12.77	0.00	24	New pipe
	1505	4.0	0.45	1.80	10.13	0	800	0	0	2.5	22	1.16	14.10	30	0.0027	4.7	23.15	0.00	30	New pipe
	1510	4.1	0.45	1.85	11.98	0	350	0	0	2.8	24	1.08	15.52	18	0.0027	3.4	5.93	9.59	30	
	1520	5.3	0.45	2.39	14.36	0	250	0	0	1.7	26	1.04	17.92	18	0.0025	3.2	5.70	12.22	30	
	1530	2.9	0.45	1.31	15.67	0	750	0	0	1.3	27	1.01	18.99	18	0.0024	3.2	5.59	13.40	30	
						1350														Upstream Conditions
	1542	13.0	0.45	5.85	5.85	0	250	10	23	0.0	33	0.92	6.46	15	0.002	2.6	3.14	3.32	21	
	1545	3.9	0.45	1.76	23.27	0	350	0	0	1.6	34	0.89	24.85	23	0.0044	5.0	14.55	10.30	30	Duel pipes, replace both
	1550	3.5	0.45	1.58	24.85	0	650	0	0	1.2	35	0.87	25.94	30	0.0018	3.9	18.90	7.04	36	
	1560	2.1	0.45	0.95	25.79	0	300	0	0	2.8	38	0.83	25.69	30	0.004	5.7	28.17	0.00		Assumed Slope
	1570	0.0	0.00	0.00	25.79	0	0	0	0	0.9	39	0.82	25.38							@ Dry Creek
Total Acreage =		58.8																		

APPENDIX A
STORM DRAIN CALCULATIONS - EXISTING SYSTEM

Storm Drain Line	Upstream Node	Rational Equation Calculations 1/ Tributary Area (acres)	C	C*A	Summation of CA	Gutter Length (ft)	Pipe Length (ft)	To (min)	Tg (min)	Tp (min)	Tc (min)	I (in/hr)	Q (cfs)	Manning Equation Calculations For Pipe 2/ D (in)	Slope (ft/ft)	V (ft/s)	Capacity Q (cfs)	Additional Q Req'd (cfs)	Replacement Pipe Dia. (in) 3/	Comments
						750														Upstream Conditions
1600	1610	1.0	0.45	0.45	0.45	0	150	10	13	0.0	23	1.14	0.62	12	0.004	3.1	2.45	0.00		Assumed Slope, will be handled by future system
	1620	1.5	0.45	0.68	1.13	0	400	0	0	0.8	23	1.11	1.50	48	0.004	7.9	98.68	0.00		Assumed Slope Jct. with 1640
	1630	0.0	0.45	0.00	1.13	0	0	0	0	0.8	24	1.09	1.47							
						100														Upstream Conditions
	1635	4.8	0.80	3.84	3.84	0	400	10	2	0.0	15	1.44	6.64	24	0.004	4.9	15.54	0.00		Assumed Slope
	1640	3.8	0.45	1.71	6.68	0	400	0	0	1.3	25	1.06	8.49	48	0.004	7.9	98.68	0.00		Assumed Slope
	1650	3.7	0.45	1.67	8.34	0	0	0	0	0.8	26	1.04	10.41							@ Dry Creek
Total Acreage =		14.8																		
						300														Upstream Conditions
1700	1710	1.5	0.45	0.68	0.68	0	150	10	5	0.0	15	1.44	1.17	12	0.004	3.1	2.45	0.00		Assumed Slope
	1720	0.0	0.00	0.00	0.68	0	0	0	0	0.8	16	1.40	1.13							@ Dry Creek
Total Acreage =		1.5																		
						450														Upstream Conditions
1800	1810	4.6	0.53	2.46	2.46	0	300	10	8	0.0	18	1.32	3.89	21	0.0029	3.9	9.27	0.00		Jct. with 1830
						150														Upstream Conditions
	1820	4.3	0.45	1.94	1.94	0	250	10	3	0.0	18	1.32	3.07	15	0.002	2.6	3.14	0.00		
	1830	3.5	0.45	1.58	5.97	0	300	0	0	1.6	19	1.25	8.95	21	0.0015	2.8	6.66	2.28	24	
	1840	2.9	0.45	1.31	7.27	0	150	0	0	1.8	21	1.18	10.29	21	0.002	3.2	7.69	2.60	24	
	1850	0.0	0.00	0.00	7.27	0	0	0	0	0.8	22	1.16	10.12							@ Dry Creek
Total Acreage =		15.3																		
						250														Upstream Conditions
1900	1910	4.2	0.45	1.89	1.89	0	350	10	4	0.0	15	1.44	3.27	12	0.0028	2.6	2.05	1.22	18	
	1920	3.6	0.45	1.62	3.51	0	150	0	0	2.2	17	1.33	5.60	12	0.0044	3.3	2.57	3.04	18	
	1930	0.0	0.45	0.00	3.51	0	500	0	0	0.8	18	1.29	5.43	12	0.003	2.7	2.12	3.31	18	Assumed Slope
	1940	0.0	0.00	0.00	3.51	0	0	0	0	3.1	21	1.18	4.97							@ Putah Creek
Total Acreage =		7.8																		
						400														Upstream Conditions
2000	2004	5.4	0.45	2.43	2.43	0	100	10	7	0.0	17	1.35	3.94	6	0.002	1.4	0.27	3.66	18	Replace sump w/gravity
	2005	0.0	0.45	0.00	2.43	0	0	0	0	1.2	18	1.30	3.79							
						450														Upstream Conditions
	2010	5.7	0.37	2.13	2.13	0	350	10	8	0.0	18	1.32	3.37	15	0.002	2.6	3.14	0.23		No Area West
	2015	2.6	0.45	1.17	5.73	0	250	0	0	2.3	20	1.22	8.38	15	0.002	2.6	3.14	5.24	24	
	2020	6.9	0.45	3.11	8.83	0	500	0	0	1.6	21	1.17	12.40	15	0.002	2.6	3.14	9.26	30	
	2030	7.6	0.25	1.90	10.73	0	300	0	0	3.3	25	1.08	13.91	15	0.002	2.6	3.14	10.77	30	
	2035	5.0	0.45	2.25	12.98	0	250	0	0	2.0	27	1.03	16.04	15	0.002	2.6	3.14	12.91	36	
	2040	4.7	0.45	2.12	15.10	0	300	0	0	1.6	28	0.99	17.93	15	0.003	3.1	3.84	14.09	36	
	2045	3.2	0.45	1.44	16.54	0	250	0	0	1.6	30	0.96	19.05	28.3	0.0017	3.6	15.72	3.33	36	Duel Pipes, replace 15"
						900														
	2047	8.3	0.41	3.37	3.37	0	300	10	15	0.0	30	0.96	3.88	15	0.002	2.6	3.14	0.74	18	
	2050	14.0	0.48	6.69	26.59	0	250	0	0	2.0	32	0.93	29.67	28.3	0.002	3.9	17.05	12.62	36	Duel Pipes, replace 15"
	2060	11.2	0.45	5.04	31.63	0	300	0	0	1.1	33	0.91	34.53	28.3	0.0008	2.5	10.78	23.75	36	Duel Pipes, replace 15"

APPENDIX A
STORM DRAIN CALCULATIONS · EXISTING SYSTEM

Storm Drain Line	Upstream Node	Rational Equation Calculations 1/ Tributary Area (acres)	C	C*A	Summation of CA	Gutter Length (ft)	Pipe Length (ft)	To (min)	Tg (min)	Tp (min)	Tc (min)	I (in/hr)	Q (cfs)	Manning Equation Calculations For Pipe 2/ D (in)	Slope (ft/ft)	V (ft/s)	Capacity Q (cfs)	Additional Q Req'd (cfs)	Replacement Pipe Dia. (in) 3/	Comments
	2065	14.8	0.39	5.71	37.34	0	300	0	0	2.0	35	0.88	39.43	28.3	0.002	3.9	17.05	22.37	36	Duel Pipes, replace 15"
	2070	11.4	0.45	5.13	42.47	0	300	0	0	1.3	36	0.86	43.82	28.3	0.009	8.3	36.17	7.65	42	Duel Pipes, replace 15"
	2075	12.0	0.45	5.41	47.88	0	350	0	0	0.6	37	0.85	48.83	28.3	0.003	4.8	20.88	27.95	42	Duel Pipes, replace 15"
	2080	13.2	0.45	5.94	53.82	0	600	0	0	1.2	38	0.84	54.25	36.1	0.003	5.6	39.97	14.27	42	Duel Pipes, replace 24"
	2090	0.0	0.45	0.00	53.82	0	400	0	0	1.8	40	0.81	52.31	24	0.003	4.3	13.46	38.85	42	Assumed slope @ Putah Cr
	2095	0.0	0.00	0.00	53.82	0	0	0	0	1.6	41	0.80	51.66							
Total Acreage =		126.0																		Upstream Conditions
						450														
2100	2110	1.8	0.35	0.63	0.63	0	350	10	8	0.0	18	1.32	1.00	6	0.0022	1.5	0.29	0.71	18	
	2120	6.7	0.45	3.02	3.65	0	300	0	0	4.0	22	1.17	5.12	8	0.0048	2.6	0.91	4.21	18	
	2130	6.3	0.57	3.57	7.22	0	350	0	0	1.9	23	1.11	9.61	10	0.004	2.8	1.50	8.11	21	
	2140	8.7	0.53	4.65	11.87	0	700	0	0	2.1	26	1.05	14.95	12	0.004	3.1	2.45	12.50	24	Assumed Slope
	2150	0.0	0.00	0.00	11.87	0	0	0	0	3.7	29	0.97	13.81							@ Putah Creek
Total Acreage =		23.5																		Upstream Conditions
																				No Area West
						300														
2200	2205	3.1	0.42	1.30	1.30	0	300	10	5	0.0	15	1.44	2.24	12	0.002	2.2	1.73	0.51		Assumed Slope
	2215	5.2	0.38	1.97	3.27	0	700	0	0	2.3	17	1.33	5.21	15	0.002	2.6	3.14	2.07	21	Assumed Slope
	2225	6.9	0.45	3.11	6.37	0	200	0	0	4.6	22	1.16	8.87	18	0.002	2.9	5.10	3.77	24	Assumed Slope
																				No Area West
						250														
	2231	2.5	0.45	1.13	1.13	0	400	10	4	0.0	15	1.44	1.94	15	0.002	2.6	3.14	0.00		Assumed Slope
	2232	3.3	0.45	1.48	2.61	0	550	0	0	2.6	18	1.31	4.10	18	0.002	2.9	5.10	0.00		Assumed Slope
	2230	5.2	0.40	2.06	11.04	0	500	0	0	1.2	23	1.12	14.84	24	0.002	3.5	10.99	3.85	30	Assumed Slope
	2235	6.9	0.45	3.11	14.15	0	300	0	0	2.4	25	1.06	17.99	30	0.002	4.1	19.92	0.00		Assumed Slope
	2240	4.5	0.45	2.03	16.17	0	400	0	0	1.2	27	1.03	19.99	36	0.002	4.6	32.40	0.00		Assumed Slope
	2245	7.0	0.45	3.15	19.32	0	550	0	0	1.5	28	1.00	23.18	36	0.002	4.6	32.40	0.00		Assumed Slope
	2250	7.6	0.45	3.42	22.74	0	500	0	0	2.0	30	0.96	26.20	42	0.002	5.1	48.87	0.00		Assumed Slope
	2260	6.2	0.53	3.32	26.06	0	300	0	0	1.6	32	0.93	29.08	42	0.002	5.1	48.87	0.00		
	2265	3.6	0.45	1.62	27.68	0	200	0	0	1.0	33	0.91	30.22	48	0.002	5.6	69.78	0.00		
	2270	10.7	0.49	5.27	40.91	0	550	0	0	0.6	33	0.90	44.18	48	0.0008	3.6	44.68	0.00		Added flow from 2430
	2285	19.6	0.68	13.43	54.34	0	750	0	0	2.6	36	0.86	56.07	48	0.001	3.9	49.34	6.73		Level Pipe
	2290	34.2	0.58	19.84	74.17	0	350	0	0	3.2	39	0.82	72.98	60	0.008	12.9	253.05	0.00		
	2310	20.2	0.82	16.48	90.65	0	300	0	0	0.5	39	0.82	89.19	60	0.008	12.9	253.05	0.00		
	2320	2.8	0.80	2.24	118.12	0	400	0	0	0.4	40	0.81	114.81	60	0.0038	8.9	174.40	0.00		Added flow from 2460
	2330	5.8	0.80	4.64	122.76	0	250	0	0	0.8	41	0.80	117.84	60	0.0038	8.9	174.40	0.00		
	2340	8.9	0.45	4.01	131.09	0	600	0	0	0.5	41	0.80	125.84	60	0.004	9.1	178.93	0.00		Assumed Slope
	2350	0.0	0.00	0.00	131.09	0	0	0	0	1.1	42	0.79	124.27							@ Putah Creek
Total Acreage =		164.2																		Upstream Conditions
						1100														
2400	2410	11.8	0.45	5.31	5.31	0	200	10	18	0.0	28	0.99	6.31	18	0.0015	2.5	4.42	1.89		No improvements in back
	2420	1.8	0.45	0.81	6.12	0	1350	0	0	1.3	30	0.97	7.12	18	0.0015	2.5	4.42	2.71		yards
	2430	4.1	0.45	1.85	7.97	0	50	0	0	9.0	39	0.83	7.93	15	0.001	1.8	2.22	5.72		
	2431	0.0	0.00	0.00	7.97	0	0	0	0	0.5	39	0.82	7.84							Flow to 2270
						500								15	0.001	1.8				Pipe S. of 2430
	2440	21.5	0.36	7.77	7.77	0	400	10	8	0.0	18	1.28	11.93	15	0.001	1.8	2.22	9.71	30	3 new catch basins

APPENDIX A
STORM DRAIN CALCULATIONS - EXISTING SYSTEM

		Rational Equation Calculations 1/												Manning Equation Calculations For Pipe 2/						
Storm Drain Line	Upstream Node	Tributary Area (acres)	C	C*A	Summation of CA	Gutter Length (ft)	Pipe Length (ft)	To (min)	Tg (min)	Tp (min)	Tc (min)	I (in/hr)	Q (cfs)	D (in)	Slope (ft/ft)	V (ft/s)	Capacity Q (cfs)	Additional Q Req'd (cfs)	Replacement Pipe Dia. (in) 3/	Comments
	2450	15.5	0.68	10.61	18.37	0	300	0	0	3.7	22	1.15	25.35	18	0.001	2.0	3.61	21.74	36	
	2460	9.8	0.70	6.86	25.23	0	850	0	0	2.4	24	1.08	32.70	10	0.002	2.0	1.06	31.63	36	New pipe to 2320
	2461	0.0	0.00	0.00	25.23	0	0	0	0	7.3	32	0.93	28.16							Flow to 2320
						200								18	0.004	4.1				Pipe S. of 2460
	2470	3.0	0.80	2.40	2.40	0	400	10	3	0.0	15	1.44	4.15	24	0.0008	2.2	6.95	0.00		
	2480	3.0	0.80	2.40	4.80	0	250	0	0	3.0	18	1.29	7.43	24	0.0012	2.7	8.51	0.00		
	2490	1.0	0.80	0.80	5.60	0	400	0	0	1.5	20	1.23	8.27	24	0.004	4.9	15.54	0.00		Assumed Slope
	2495	0.0	0.00	0.00	5.60	0	0	0	0	1.3	21	1.19	8.00							@ Putah Creek
Total Acreage =		71.5																		
						300														Upstream Conditions
2500	2510	1.0	0.45	0.45	0.45	0	100	10	5	0	15	1.44	0.78	12	0.011	5.2	4.06	0.00		
	2520	3.3	0.80	2.64	3.09	0	300	0	0	0.3	15	1.42	5.27	15	0.002	2.6	3.14	2.13	18	
	2530	2.2	0.80	1.76	4.85	0	200	0	0	2.0	17	1.33	7.74	15	0.002	2.6	3.14	4.60	24	
	2540	3.0	0.80	2.40	7.25	0	150	0	0	1.3	19	1.27	11.05	15	0.002	2.6	3.14	7.91	24	Assumed Slope
	2550	5.0	0.80	4.00	11.25	0	450	0	0	1.0	20	1.23	16.61	15	0.002	2.6	3.14	13.47	30	Assumed Slope & Pipe
	2560	0.0	0.00	0.00	11.25	0	0	0	0	2.9	22	1.14	15.39							@ Putah Creek
Total Acreage =		14.5																		
						450														Upstream Conditions
2600	2610	4.5	0.68	3.08	3.08	0	100	10	8	0.0	18	1.32	4.87	8	0.002	1.7	0.59	4.28		
	2620	2.0	0.63	1.25	4.33	0	450	0	0	1.0	18	1.27	6.59	8	0.002	1.7	0.59	6.00	18	New pipe to 2340
	2630	0.0	0.00	0.00	4.33	0	0	0	0	4.5	23	1.12	5.81							Assumed Slope @ Putah Cr
Total Acreage =		6.5																		

Notes:

1. Q = GCIA, where

G (Geogr. Factor) = 1.20
C factors = 0.45 (Low, medium density residential)
0.80 (Commercial, light industrial, high density residential)
0.35 (Schools)
0.25 (Parks)
0.46 (Undeveloped land)
0.95 (Impervious areas)
I (intensity) = 10-yr intensity taken from Figure 2 for the computed Tc

To = 10 minutes for residential areas.
Tg = Time of gutter flow from Yolo County, 1965 Nomograph
where Vgutter = 1 ft/sec
Tp = Estimated from pipe length and velocity
Tc = 15 minutes minimum (City of Davis)

2. Q = 1.486AR^(2/3)S^(1/2)/n for pipe flowing full, where
A = Cross sectional area of pipe (3.14159D^2/4)
R = Hydraulic Radius (D/4)
S = Invert slope of pipe
n for RCP = 0.012

3. Required Replacement Pipe assumes same alignment, length and slope as existing pipe. See Appendix B for location of required improvements.

Appendix B

FUTURE STORM DRAINS

APPENDIX

STORM DRAIN CALCULATIONS · FUTURE SYSTEM

			Rational Equation Calculations 1/												Manning Equation Calculations For Pipe 2/				
Storm Drain Line	Upstream Node	Ground Elev. (ft)	Tributary Area (acres)	C	C*A	Summation of CA	Gutter Length (ft)	Pipe Length (ft)	To (min)	Tg (min)	Tp (min)	Tc (min)	I (in/hr)	Q (cfs)	D (inch)	Invert Elev. (ft)	Slope (ft/ft)	V (ft/s)	Capacity Q (cfs)
							500												
4000	4010	126.6	5.0	0.80	4.00	4.00	0	500	10	8	0.0	18	1.28	6.15	18	121.6	0.003	3.5	6.25
											2.4								
							300												
	4020	123.2	7.0	0.80	5.60	5.60	0	1100	10	5	0.0	15	1.44	9.68	24	118.2	0.003	4.3	13.46
	4030	126.2	10.0	0.80	8.00	17.60	0	750	0	0	4.3	21	1.19	25.20	36	113.9	0.003	5.6	39.68
											2.2								
							500												
	4040	126.0	27.0	0.80	21.60	21.60	0	1000	10	8	0.0	18	1.28	33.19	30	120.5	0.008	8.1	39.84
											2.1								
							300												
	4050	130.0	20.0	0.45	9.00	9.00	0	1000	10	5	0.0	15	1.44	15.55	24	124.5	0.007	6.5	20.55
	4060	129.0	19.4	0.55	10.70	19.70	0	600	0	0	2.5	18	1.31	31.06	30	117.0	0.007	7.6	37.27
	4070	127.0	26.9	0.67	17.99	76.89	0	1000	0	0	1.3	23	1.12	103.68	54	110.2	0.003	7.4	117.00
											2.3								
Total Acreage =			115.3																
							500												
5400	5410	153.0	17.6	0.42	7.34	7.34	0	600	10	8	0.0	18	1.28	11.28	24	148.0	0.008	7.0	21.97
	5420	148.0	4.5	0.80	3.60	10.94	0	900	0	0	1.4	20	1.23	16.09	24	143.2	0.008	7.0	21.97
	5430	141.0	25.4	0.45	11.43	22.37	0	1000	0	0	2.1	22	1.15	30.97	30	135.5	0.006	7.0	34.51
	5440	135.0	30.9	0.43	13.24	35.61	0	1700	0	0	2.4	24	1.09	46.42	48	128.0	0.001	3.9	49.34
	5450	140.0	53.2	0.44	23.36	58.96	0	400	0	0	7.2	31	0.93	66.02	48	126.3	0.002	5.6	69.78
Total Acreage =			131.6																
							500												
5500	5510	165.0	8.0	0.45	3.60	3.60	0	700	10	8	0.0	18	1.28	5.53	18	160.0	0.007	5.4	9.54
	5520	160.0	8.3	0.45	3.74	7.34	0	100	0	0	2.2	20	1.20	10.56	24	154.6	0.006	6.1	19.03
Total Acreage =			16.3																

Notes:

1. Q = GCIA, where

G (Geogr. Factor) = 1.20

C factors = 0.45 (Low, medium density residential)
0.80 (Commercial, light industrial, high density residential)
0.35 (Schools)
0.25 (Parks)
0.46 (Undeveloped land)
0.95 (Impervious areas)

I (intensity) = 10-yr intensity taken from Figure 2 for the computed Tc

To = 10 minutes for residential and commercial areas.
Tg = Time of gutter flow from Yolo County, 1965 Nomagraph
where Vgutter = 1 ft/sec
Tp = Estimated from pipe length and velocity
Tc = 15 minutes minimum (City of Davis)

2. Q = 1.486AR^(2/3)S^(1/2)/n for pipe flowing full, where
A = Cross sectional area of pipe (3.14159D^2/4)
R = Hydraulic radius (D/4)
S = Invert slope of pipe, assumed > 0.003

Upstream-most node in each system invert assumed 5 ft below grade
Pipe cover assumed between 3 and 12 feet
n for RCP = 0.012

Appendix C

DRAINAGE BOUNDARIES

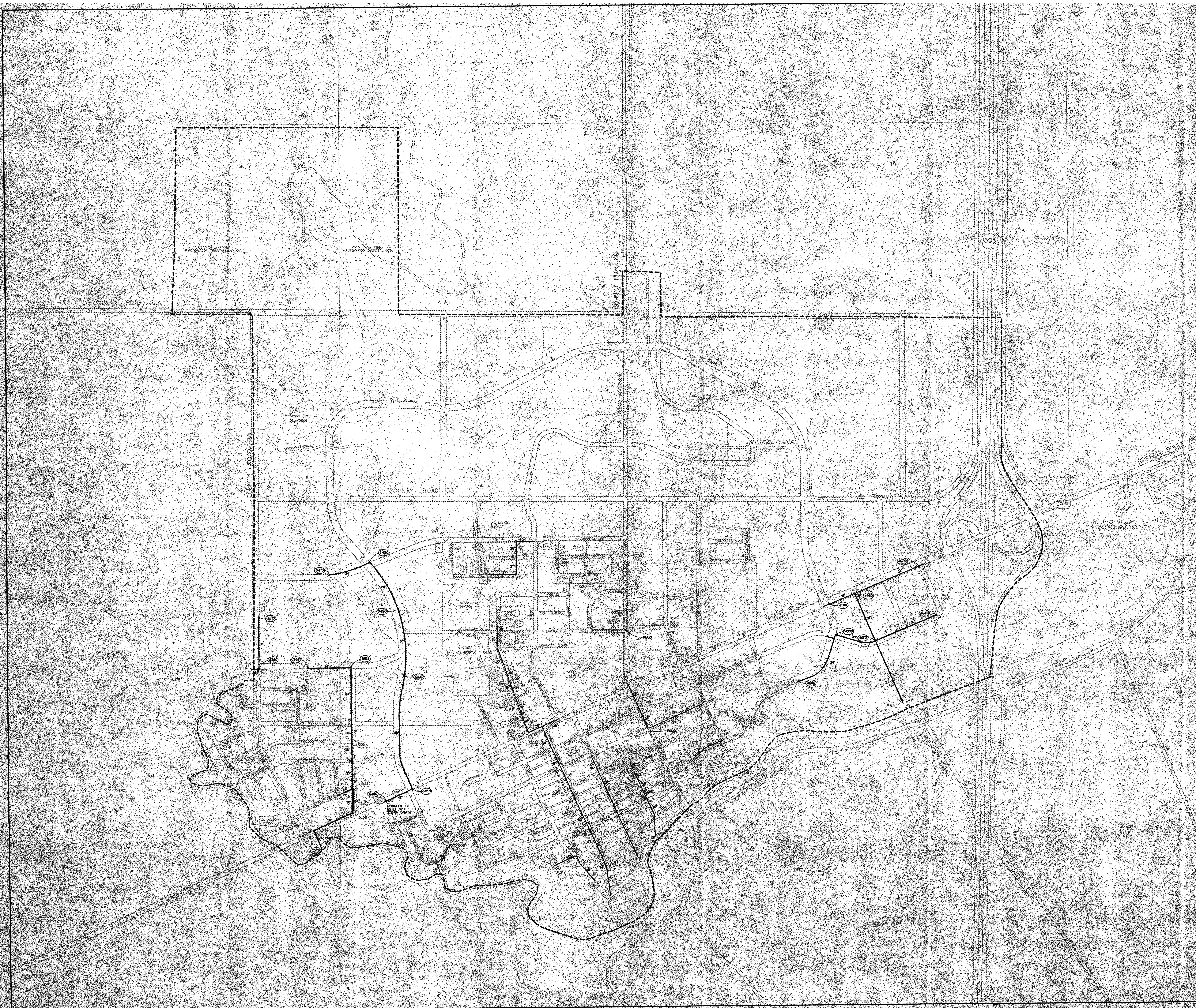
WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix B

"Solano & Yolo County Design Rainfall," James D. Goodridge

FUTURE STORM DRAIN SYSTEM



LEGEND
NODE NUMBER
PIPE DIAMETER (RCP)
20 YEAR SOI

CITY OF WINTERS

N

0' 500' 1,000'

APRIL 1992

EXISTING STORM DRAIN SYSTEM

LEGEND
NODE NUMBER
PIPE DIAMETER (RCP)
20 YEAR SOI

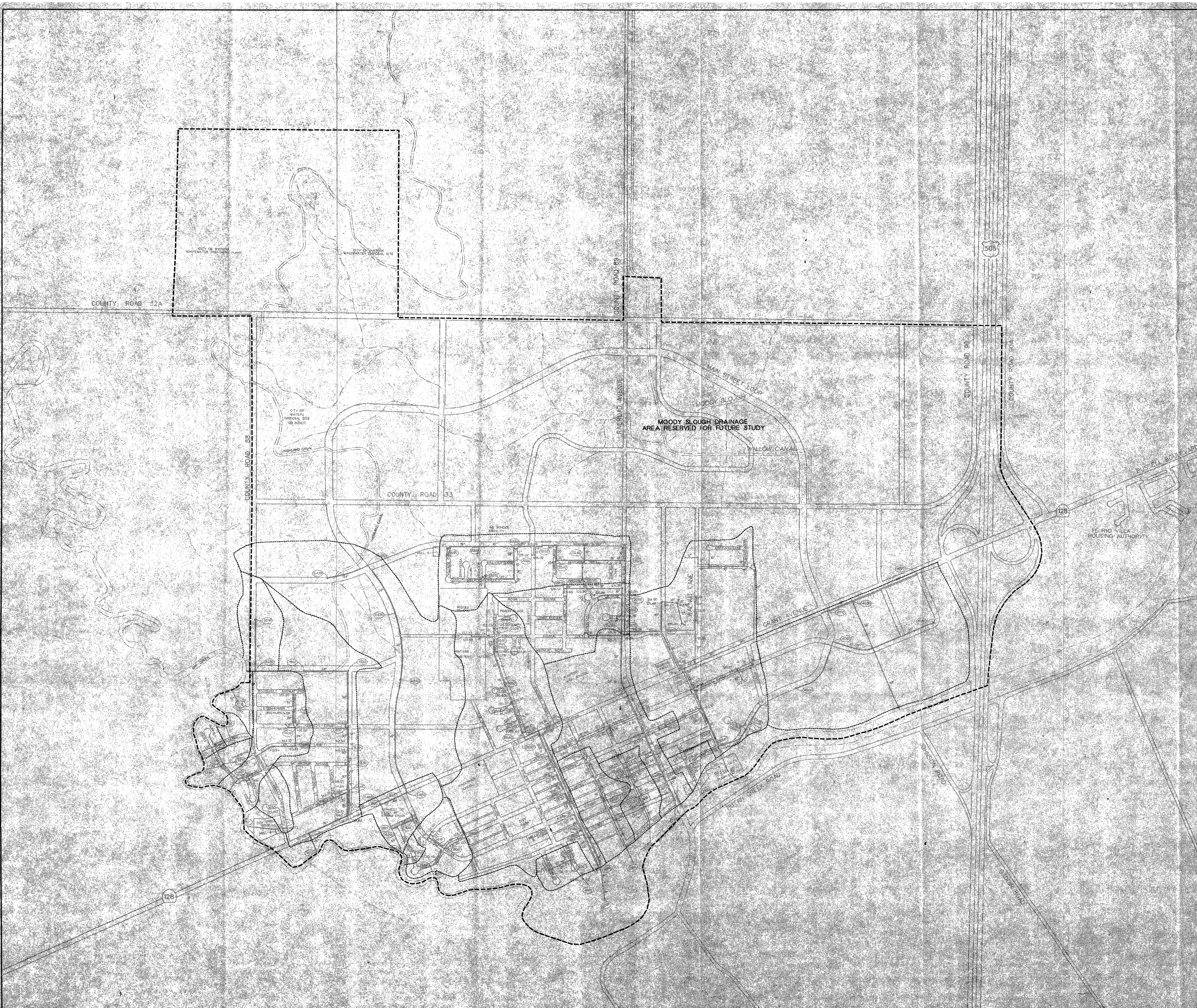
CITY OF WINTERS

N

0' 500' 1,000'

APRIL 1992

DRAINAGE BOUNDARIES



LEGEND

NODE NUMBER

PIPE DIAMETER (RCP)

20 YEAR SOI

DRAINAGE BOUNDARY

CITY OF WINTERS

N

0 500 1,000

APRIL 1992

Solano & Yolo County Design Rainfall

This study was prepared at the request of Mr. Lee Frederiksen of Borcalli and Associates of Sacramento. It is intended to be used in selecting a design storm for any location in Solano or Yolo Counties for storm duration of five minutes to ten days and for return periods of 2 to 100 years.

This revision is to modify the Design Rainfalls on Table 1 for durations of over two days. Also return periods of 500, 1000 and 10,000 years were added to make this tudy useful to a broader range of users.

To find a design storm; first look up the mean annual precipitation (MAP) on (Figure 1) and then enter the MAP column of the Tables 1 for the desired storm duration and return period. The design rainfall shown on Table 1 is in parts, one each for return periods of 2.3, 5, 10, 25, 50 and 100 years. Table 1 is in units if inches.

The data of this study were from Climatological Data for California published by the National Climatic Data Center located in Ashville, N. C. Additional data were obtained from many sources including Mr. Jim Gibboney (916) 322 7159 of the Central District Office of the Department of Water Resources in Sacramento, the Vallejo City Water Works and Contra Costa County Public Works Department.

The methods used in this study to analyze rain records are similar to those used in Rainfall for Drainage Design, Bulletin 195 of the Department of Water Resources, and in Proceedings of a Workshop on County Hydrology Manuals, August 16-17, 1990, sponsored by Water Resources Center, University of California, published by Lighthouse publications, Mission Viejo, CA 92692.

Eighty-one rain gages listed on Table 2 were used in this study. These represent 2953 station years of data. Seventeen of the 82 gages are recording rain records. They are listed on Table 2. Table 2 contains the average annual extreme rainfalls at all of the rain records of this study. Some of the individual rainfall depth duration frequency tables may differ from the design rainfalls of Table 1, because 2953 station years of data are included in Table 1 and the longest individual record of this study is Sacramento with only 120 years of daily rainfall data.

All design storms were calculated as a fraction of the mean annual precipitation (MAP). The relationship between the maximum annual 1440 minute rainfall to the mean annual

precipitation (MAP) was shown on Figure 2. The non recording rain gage records were adjusted for fixed interval corrections by a factor of 1.14 so that all maximum daily data would be comparable with the data from the recording gages. The shorter records had a higher value of the ratio of the annual maximum daily to the MAP as shown on Figure 3. The final design value of the relationship between the average maximum one day and the average total annual rainfall was based on records with 70, or more years of data.

The tabulated extreme 1 day precipitation from the recording gages are intended to represent the actual maximum 1440 consecutive minutes for the year. Recording gage extreme rains usually average 14% higher than once a day fixed time observations.

The shorter records also had a larger value of the sample value coefficient of variation as shown on Figure 4. The longer records seem to converge on the design value of .352 that has been used since 1983, by the Department of Water Resources. The coefficient of variation for storms longer than one day are listed on Tables 6, 7 and 8, along with the regional coefficients of skew and Frequency Factors.

The ratios of short duration rainfalls to the one day (or 1440 minute) storm is based on the relationship shown at the bottom of page 6 of Table 2. These values were plotted on Figure 5.

Tables of design storms are for return periods of 2, 5, 10, 25, 50, 100, 500, 1000 and 10,000 years and storm durations of 5, 10, 15, 30 minutes, 1, 2 , 3, 6, 12 hours1, 2, 3, 4, 6, 8, 10, 15, 20, 30, and 60 days and 1 year. The design storms are expressed in terms of the MAP which ranges in Solano and Yolo Counties from 14 to 40 inches. These tables were calculated for storm duration of 3 hours or less using the following relationship:

$$P_{ij} = (-.22 + .13047*MAP)*(1+K_j*CV)*T_i^{.43747}$$

where P_{ij} is the design precipitation for return period j and storm duration i.

MAP is the mean annual precipitation Figure 1

(-.22 + .13047*MAP) is the fraction of MAP occurring in the average maximum day from Figure 2.

CV is the design value of the Coefficient of Variation, specifically .352 for this region of the Sacramento Valley drainage.

T_i is the time in days (note for 5 min use 5 / 1440.)

n is .43747, the slope of the log rain vs., log minutes shown on Figure 5.

K_j is the frequency factor for the Pearsons Type III distribution (for storms of one day or less) with an of skew 1.1 as shown below:

Return Period Years	Frequency Factors
2	-.180
5	.745
10	1.341
25	2.066
50	2.420
100	3.087
200	3.575
500	4.300
1000	4.673
10000	6.185

Frequency factors represent the number of standard deviations in excess of the mean that are used to define storms of various return periods.

The mean annual precipitation (MAP) map Figure 1 is based on the 1951 to 1980 averages corresponding to the period used by the National Weather Service for their climatic normals.

The maximum rainfall for each calendar day from 1917 to 1989 at Davis was plotted on the cover of this study.

Notable large rainfalls in or near Solano and Yolo Counties during historic times include the April 20, 1880 storm at Mount Saint Helena at 4340 feet elevation, where 14.70 inches of rain fell in one day. No records of this event are available for Yolo or Solano Counties, but the largest ever daily rainfall of 5.28 inches occurred at Sacramento on this date. The return period for 5.28 inches in one day at Sacramento is over 500 years.

The December 19 to 27, 1955 deposited record high rainfalls in an area from Winters Northeastward to the Feather River Basin. The Winters-Lewis rain gage cough 14.13 inches in 8 days. The return period was over 1500 years.

The January 4 storm of the San Francisco Bay Area caused many deaths from land slides in Marin and Santa Cruz Counties. The highest rainfall reported for Solano County was 6.04 inches. This occurred at the Vallejo 4 N rain gage. The return period was about 1400 years.

In the last half century the biggest rainfall was during the Columbus Day storm of 1962. During October 12 to 14, 1962 a band of rainfalls with return periods in excess of 1000 years was scattered generally from Oakland northeastward to Marysville and to Alturas. The Solano - Yolo area was bracketed on both sides with heavy rains. The largest return period for the 3 day storm was 340 years at Mare Island, which had 8.28 inches. Davis had 7.81 inches in 3 days with a 275 year return period. It was fortunate that this storm fell on dry ground at the end of the normal summer drought, when there was a large soil moisture deficit to absorb the heavy rains.

The water year 1983 was the wettest year in the 109 years of record which were examined in Yolo and Solano Counties. There was extensive flooding in poorly drained areas due to the years having almost twice the average number of rainy days. At Sacramento where the record starts in 1850. there was 36.57 inches. The previous high year was 36.35 inches in 1853. The five wettest years in the region's history were followed by five of the driest years in the last decade.

The storm of February 11 to 20, 1986 was heaviest in the Sierra Navada and in the Napa River Basin as well as the streams draining into the Fairfield- Cordelia area. Record 10 day rainfalls occurs at Lake Curry, Green Valley and at Lake Frey. The Atlas Road rain gage reported 41.08 inches in 10 days which was 7.4 standard deviations above the mean 10 day storm total. The estimated return period is in excess of 100,000 years. Stream channels to the South East of Atlas Peak were lined with large boulders and swept clear of vegetation suggestive of a debris flow, after this storm.

The maximum 24 hour rainfall ever recorded in the San Francisco Bay drainage area was the 15.28 inches at Atlas Road on February 17, 1986. The previous maximum was the Mt. Saint Helena storm of April 1884. The highest ever one day rain in the Central Valley Drainage area was 17.60 at four Trees in the Feather River Basin also on February 17, 1986.

The 20 largest rainfalls at selected stations for each month are listed on Table 3. The maximum daily rainfall for each month at selected stations in or near Solano and Yolo counties is listed on Table 4. The maximum daily rainfall by months for all of California is listed on Table 5. Other data a on extreme rainfalls are included, as well as a plot of 109 year trends in total annual rainfall in Yolo and Solano Counties.

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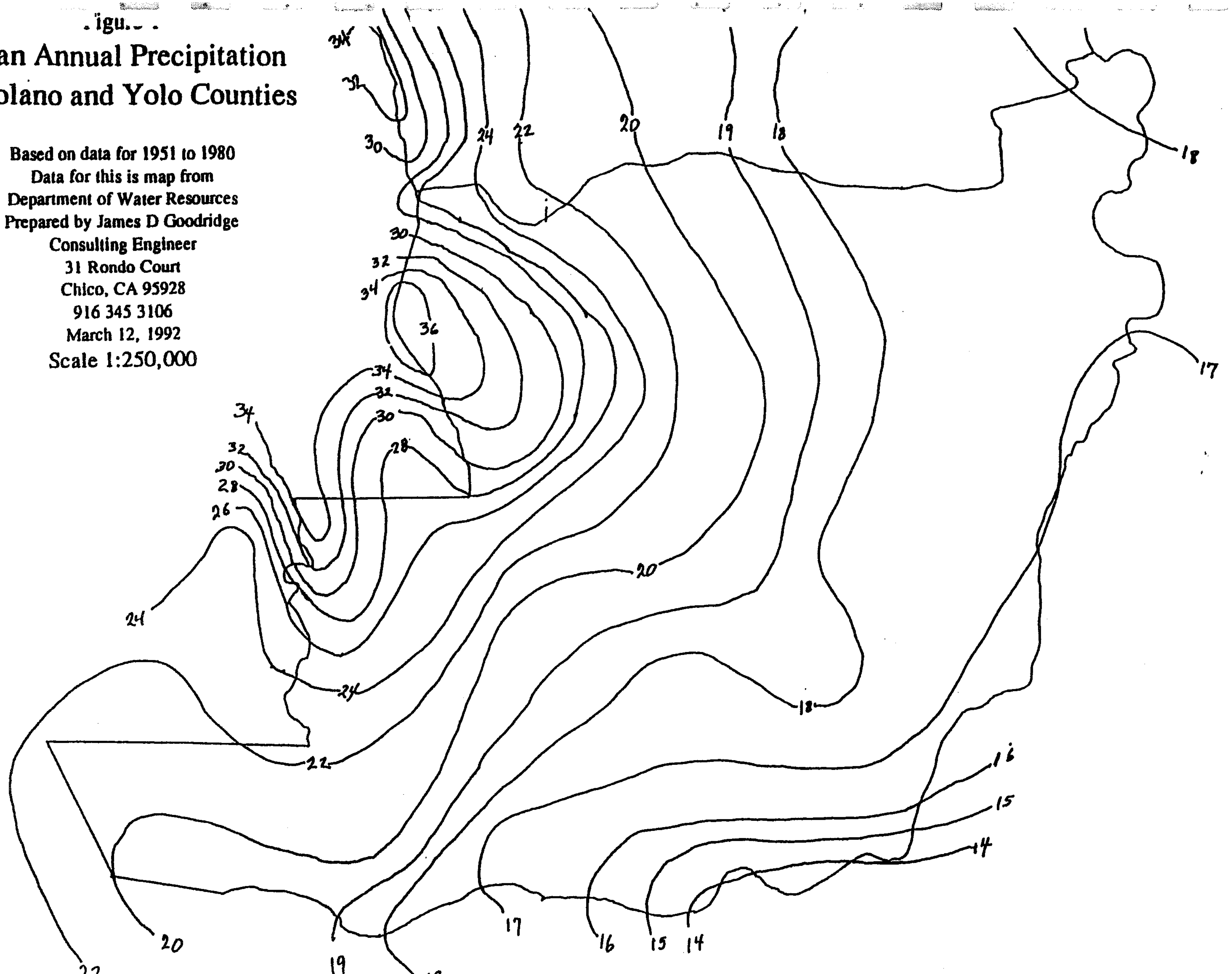
Mean Annual Precipitation in Solano and Yolo Counties

Based on data for 1951 to 1980
Data for this is map from
Department of Water Resources
Prepared by James D Goodridge
Consulting Engineer
31 Rondo Court
Chico, CA 95928
916 345 3106
March 12, 1992
Scale 1:250,000



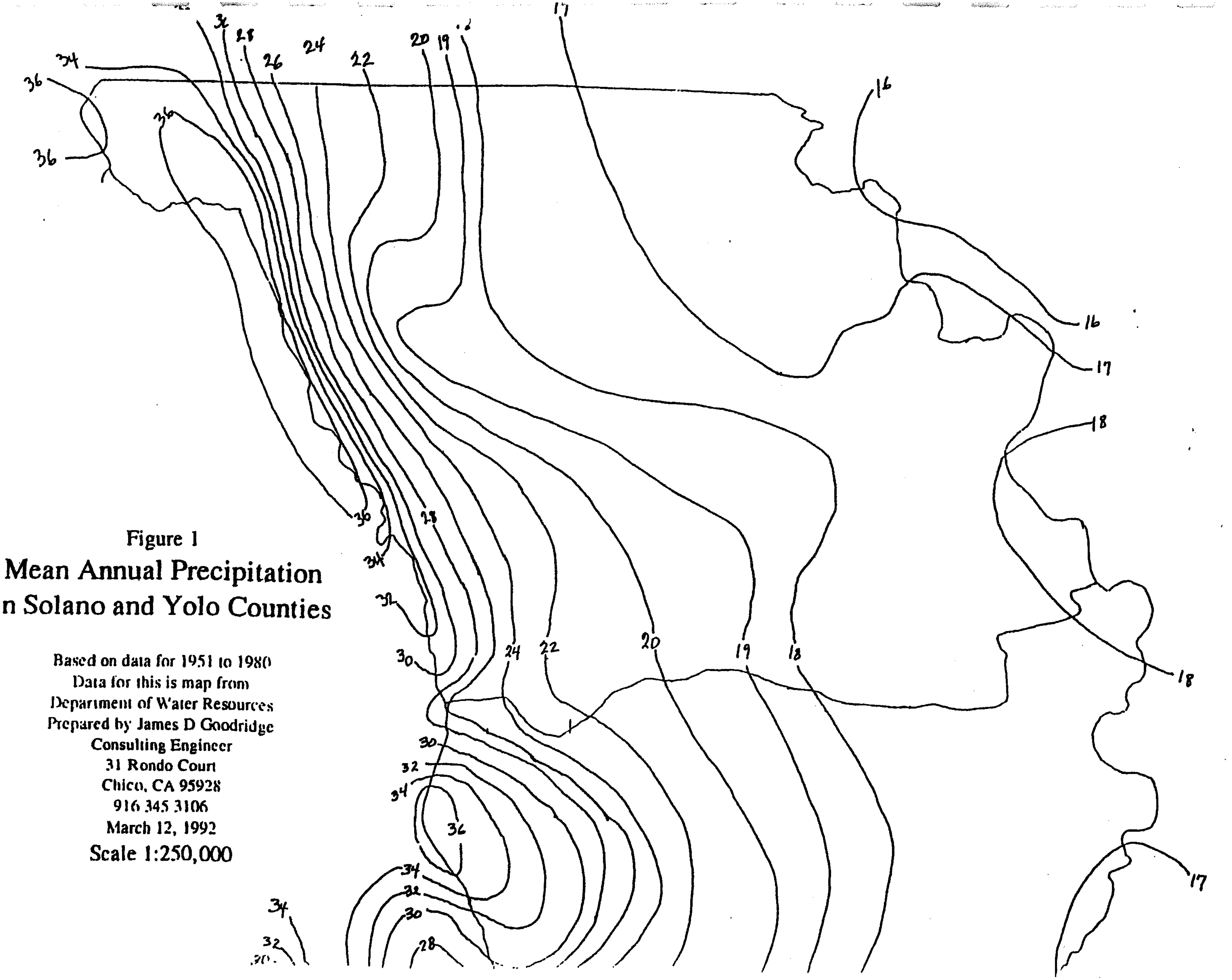


Figure 1
Mean Annual Precipitation in Solano and Yolo Counties

Based on data for 1951 to 1980
Data for this is map from
Department of Water Resources
Prepared by James D Goodridge
Consulting Engineer
31 Rondo Court
Chico, CA 95928
916 345 3106
March 12, 1992
Scale 1:250,000

Solano and Yolo Design Ranifall

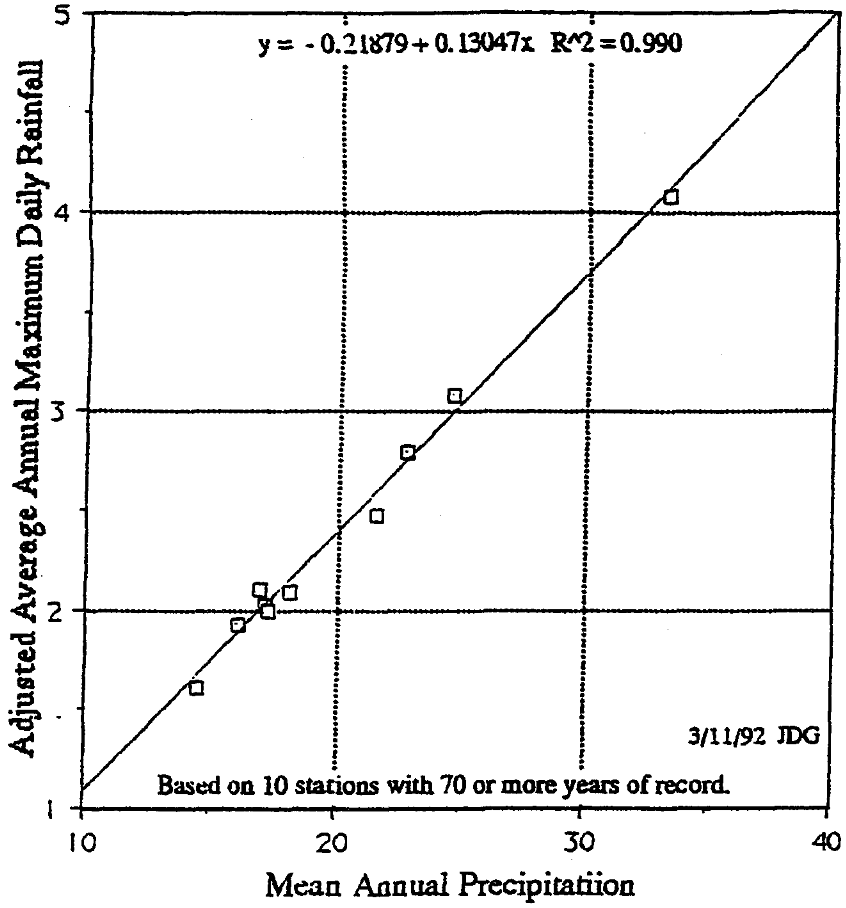


Figure 2

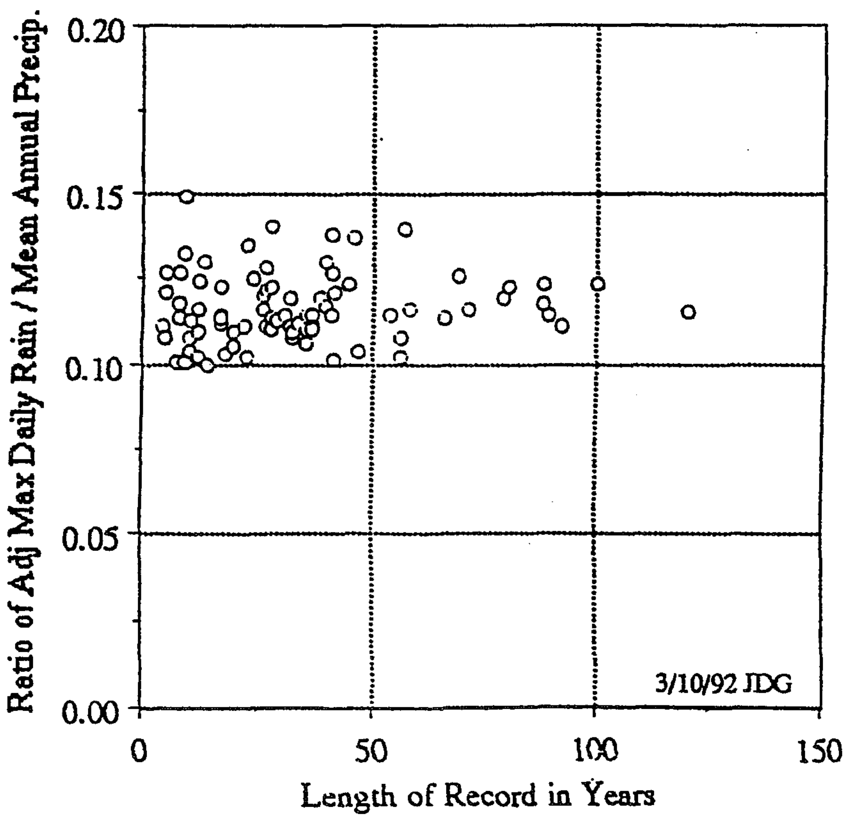


Figure 3

Solano & Yolo County Resign Rainfall

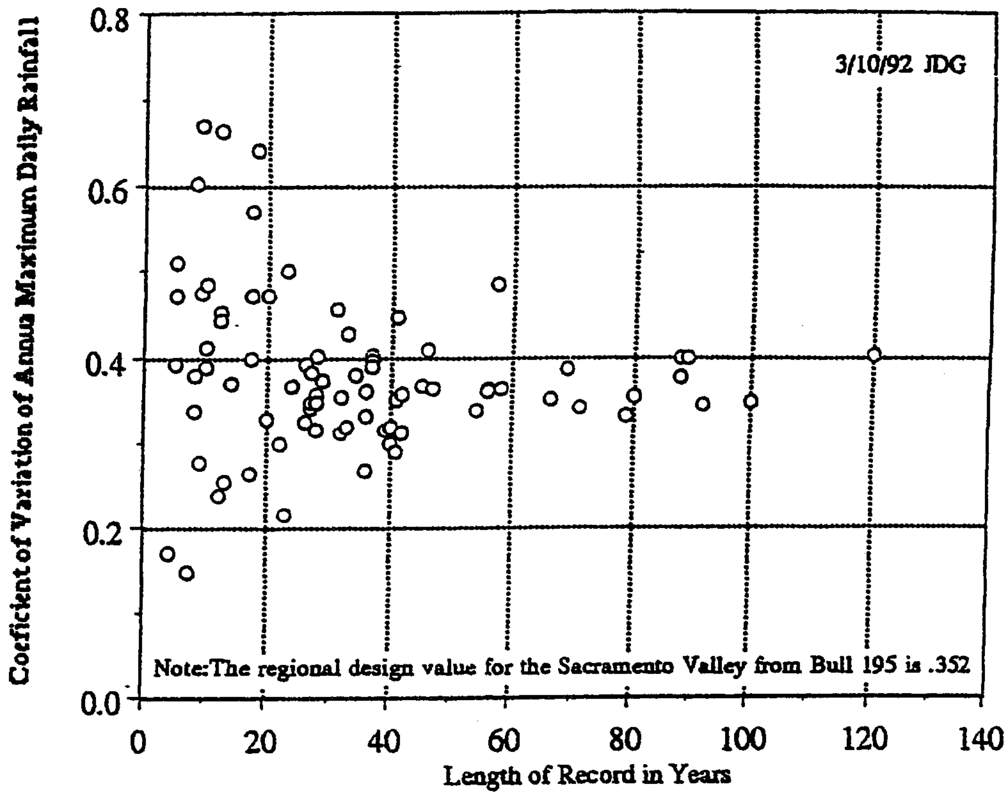


Figure 4

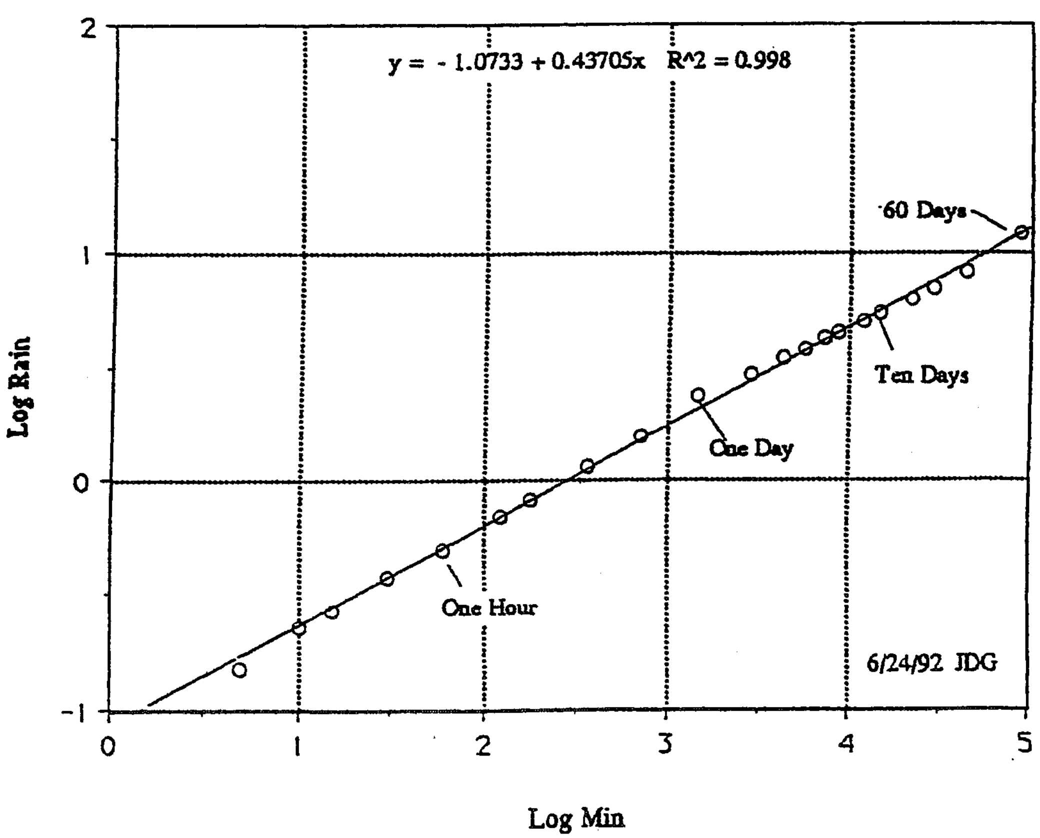


Figure 5

2 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.13	0.18	0.22	0.29	0.40	0.54	0.64	0.80	1.07	1.60	2.02	2.37	2.62	2.87	3.07	3.46	3.72	4.26	4.76	5.61	8.26	13.86
15	0.14	0.20	0.23	0.32	0.43	0.58	0.70	0.86	1.16	1.73	2.18	2.56	2.84	3.10	3.32	3.74	4.02	4.60	5.14	6.06	8.93	14.99
16	0.16	0.21	0.25	0.34	0.46	0.63	0.75	0.92	1.25	1.86	2.34	2.75	3.05	3.34	3.57	4.02	4.32	4.95	5.53	6.52	9.60	16.12
17	0.17	0.23	0.27	0.37	0.49	0.67	0.80	0.99	1.33	1.99	2.51	2.94	3.26	3.57	3.82	4.30	4.62	5.29	5.92	6.97	10.27	17.24
18	0.18	0.24	0.29	0.39	0.53	0.71	0.85	1.05	1.42	2.12	2.67	3.14	3.48	3.80	4.07	4.58	4.92	5.64	6.30	7.43	10.94	18.37
19	0.19	0.26	0.30	0.41	0.56	0.76	0.90	1.12	1.51	2.24	2.83	3.33	3.69	4.03	4.32	4.86	5.23	5.98	6.69	7.88	11.61	19.49
20	0.20	0.27	0.32	0.44	0.59	0.80	0.96	1.18	1.59	2.37	3.00	3.52	3.90	4.27	4.57	5.14	5.53	6.33	7.08	8.34	12.28	20.62
22	0.22	0.30	0.36	0.48	0.66	0.89	1.06	1.31	1.77	2.63	3.33	3.90	4.33	4.73	5.06	5.70	6.13	7.02	7.85	9.25	13.62	22.87
24	0.24	0.33	0.39	0.53	0.72	0.98	1.16	1.44	1.94	2.89	3.65	4.29	4.75	5.20	5.56	6.27	6.73	7.71	8.62	10.16	14.96	25.12
26	0.26	0.36	0.43	0.58	0.79	1.06	1.27	1.57	2.11	3.15	3.98	4.67	5.18	5.67	6.06	6.83	7.34	8.40	9.39	11.07	16.30	27.37
28	0.29	0.39	0.46	0.63	0.85	1.15	1.37	1.70	2.29	3.41	4.31	5.06	5.61	6.13	6.56	7.39	7.94	9.09	10.17	11.98	17.64	29.63
30	0.31	0.42	0.50	0.68	0.91	1.24	1.48	1.83	2.46	3.67	4.64	5.44	6.03	6.60	7.06	7.95	8.55	9.79	10.94	12.89	18.99	31.88
32	0.33	0.45	0.53	0.72	0.98	1.33	1.58	1.96	2.64	3.93	4.96	5.83	6.46	7.06	7.56	8.51	9.15	10.48	11.71	13.80	20.33	34.13
34	0.35	0.48	0.57	0.77	1.04	1.41	1.69	2.09	2.81	4.19	5.29	6.21	6.88	7.53	8.06	9.07	9.75	11.17	12.49	14.71	21.67	36.38
36	0.37	0.51	0.60	0.82	1.11	1.50	1.79	2.22	2.98	4.45	5.62	6.60	7.31	8.00	8.56	9.63	10.36	11.86	13.26	15.62	23.01	38.63
38	0.40	0.54	0.64	0.87	1.17	1.59	1.90	2.35	3.16	4.71	5.94	6.98	7.74	8.46	9.05	10.20	10.96	12.55	14.03	16.53	24.35	40.88
40	0.42	0.56	0.67	0.91	1.24	1.68	2.00	2.47	3.33	4.97	6.27	7.36	8.16	8.93	9.55	10.76	11.56	13.24	14.80	17.44	25.69	43.14

5 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.17	0.23	0.28	0.37	0.51	0.68	0.82	1.11	1.50	2.03	2.54	2.99	3.31	3.62	3.88	4.36	4.69	5.37	6.01	7.08	10.42	17.50
15	0.18	0.25	0.30	0.40	0.55	0.74	0.88	1.20	1.62	2.19	2.75	3.23	3.58	3.92	4.19	4.72	5.07	5.81	6.49	7.65	11.27	18.92
16	0.20	0.27	0.32	0.43	0.59	0.80	0.95	1.29	1.74	2.36	2.96	3.47	3.85	4.21	4.50	5.07	5.45	6.24	6.98	8.23	12.11	20.34
17	0.21	0.29	0.34	0.46	0.63	0.85	1.02	1.38	1.86	2.52	3.16	3.72	4.12	4.50	4.82	5.43	5.83	6.68	7.47	8.80	12.96	21.76
18	0.23	0.31	0.36	0.49	0.67	0.91	1.08	1.47	1.98	2.69	3.37	3.96	4.39	4.80	5.13	5.78	6.21	7.12	7.96	9.37	13.81	23.18
19	0.24	0.32	0.39	0.52	0.71	0.96	1.15	1.56	2.11	2.85	3.58	4.20	4.66	5.09	5.45	6.14	6.60	7.55	8.44	9.95	14.65	24.60
20	0.25	0.34	0.41	0.55	0.75	1.02	1.21	1.64	2.23	3.02	3.78	4.44	4.92	5.39	5.76	6.49	6.98	7.99	8.93	10.52	15.50	26.03
22	0.28	0.38	0.45	0.62	0.83	1.13	1.35	1.82	2.47	3.35	4.20	4.93	5.46	5.98	6.39	7.20	7.74	8.86	9.91	11.67	17.19	28.87
24	0.31	0.42	0.50	0.68	0.92	1.24	1.48	2.00	2.71	3.68	4.61	5.41	6.00	6.56	7.02	7.91	8.50	9.73	10.88	12.82	18.89	31.71
26	0.34	0.46	0.54	0.74	1.00	1.35	1.61	2.18	2.96	4.00	5.02	5.90	6.54	7.15	7.65	8.62	9.26	10.61	11.86	13.97	20.58	34.55
28	0.36	0.49	0.59	0.80	1.08	1.46	1.75	2.36	3.20	4.33	5.44	6.38	7.08	7.74	8.28	9.33	10.02	11.48	12.83	15.12	22.27	37.39
30	0.39	0.53	0.63	0.86	1.16	1.57	1.88	2.54	3.44	4.66	5.85	6.87	7.61	8.33	8.91	10.03	10.79	12.35	13.81	16.27	23.96	40.24
32	0.42	0.57	0.68	0.92	1.24	1.68	2.01	2.72	3.69	4.99	6.26	7.35	8.15	8.92	9.54	10.74	11.55	13.22	14.78	17.42	25.66	43.08
34	0.45	0.61	0.72	0.98	1.33	1.79	2.14	2.90	3.93	5.32	6.68	7.84	8.69	9.50	10.17	11.45	12.31	14.10	15.76	18.57	27.35	45.92
36	0.47	0.64	0.77	1.04	1.41	1.91	2.28	3.08	4.17	5.65	7.09	8.32	9.23	10.09	10.80	12.16	13.07	14.97	16.74	19.72	29.04	48.76
38	0.50	0.68	0.81	1.10	1.49	2.02	2.41	3.26	4.42	5.98	7.50	8.81	9.76	10.68	11.43	12.87	13.83	15.84	17.71	20.87	30.73	51.61
40	0.53	0.72	0.86	1.16	1.57	2.13	2.54	3.44	4.66	6.31	7.92	9.30	10.30	11.27	12.06	13.58	14.60	16.71	18.69	22.02	32.43	54.45

10 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.20	0.27	0.32	0.43	0.59	0.80	0.95	1.18	1.59	2.37	2.97	3.48	3.86	4.22	4.52	5.09	5.47	6.26	7.00	8.25	12.15	20.41
15	0.21	0.29	0.35	0.47	0.64	0.86	1.03	1.27	1.72	2.56	3.21	3.77	4.17	4.57	4.89	5.50	5.91	6.77	7.57	8.92	13.14	22.07
16	0.23	0.31	0.37	0.51	0.68	0.93	1.11	1.37	1.84	2.75	3.45	4.05	4.49	4.91	5.25	5.92	6.36	7.28	8.14	9.59	14.13	23.72
17	0.25	0.33	0.40	0.54	0.73	0.99	1.18	1.47	1.97	2.94	3.69	4.33	4.80	5.25	5.62	6.33	6.80	7.79	8.71	10.26	15.12	25.38
18	0.26	0.36	0.43	0.58	0.78	1.06	1.26	1.56	2.10	3.13	3.93	4.62	5.12	5.60	5.99	6.74	7.25	8.30	9.28	10.93	16.10	27.04
19	0.28	0.38	0.45	0.61	0.83	1.12	1.34	1.66	2.23	3.33	4.17	4.90	5.43	5.94	6.35	7.16	7.69	8.81	9.85	11.60	17.09	28.69
20	0.30	0.40	0.48	0.65	0.88	1.19	1.42	1.75	2.36	3.52	4.41	5.18	5.74	6.28	6.72	7.57	8.14	9.32	10.42	12.27	18.08	30.35
22	0.33	0.44	0.53	0.72	0.97	1.32	1.57	1.94	2.62	3.90	4.90	5.75	6.37	6.97	7.46	8.40	9.02	10.33	11.55	13.61	20.05	33.67
24	0.36	0.49	0.58	0.79	1.07	1.45	1.73	2.13	2.88	4.29	5.38	6.31	7.00	7.65	8.19	9.22	9.91	11.35	12.69	14.95	22.02	36.98
26	0.39	0.53	0.63	0.86	1.16	1.57	1.88	2.33	3.13	4.67	5.86	6.88	7.62	8.34	8.92	10.05	10.80	12.37	13.83	16.29	24.00	40.30
28	0.42	0.57	0.69	0.93	1.26	1.70	2.04	2.52	3.39	5.05	6.34	7.44	8.25	9.03	9.66	10.88	11.69	13.39	14.97	17.63	25.97	43.61
30	0.46	0.62	0.74	1.00	1.35	1.83	2.19	2.71	3.65	5.44	6.82	8.01	8.88	9.71	10.39	11.70	12.58	14.40	16.10	18.98	27.95	46.92
32	0.49	0.66	0.79	1.07	1.45	1.96	2.34	2.90	3.91	5.82	7.31	8.58	9.51	10.40	11.13	12.53	13.47	15.42	17.24	20.32	29.92	50.24
34	0.52	0.71	0.84	1.14	1.55	2.09	2.50	3.09	4.16	6.21	7.79	9.14	10.13	11.08	11.86	13.36	14.36	16.44	18.38	21.66	31.89	53.55
36	0.55	0.75	0.89	1.21	1.64	2.22	2.65	3.28	4.42	6.59	8.27	9.71	10.76	11.77	12.59	14.18	15.24	17.46	19.52	23.00	33.87	56.87
38	0.59	0.79	0.95	1.28	1.74	2.35	2.81	3.47	4.68	6.98	8.75	10.27	11.39	12.46	13.33	15.01	16.13	18.47	20.65	24.34	35.84	60.18
40	0.62	0.84	1.00	1.35	1.83	2.48	2.96	3.67	4.94	7.36	9.23	10.84	12.01	13.14	14.06	15.84	17.02	19.49	21.79	25.68	37.82	63.50

25 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.23	0.32	0.38	0.51	0.69	0.94	1.12	1.38	1.86	2.78	3.48	4.09	4.53	4.96	5.30	5.97	6.42	7.35	8.22	9.68	14.26	23.95
15	0.25	0.34	0.41	0.55	0.75	1.01	1.21	1.49	2.01	3.00	3.76	4.42	4.90	5.36	5.73	6.46	6.94	7.95	8.89	10.47	15.42	25.89
16	0.27	0.37	0.44	0.59	0.80	1.09	1.30	1.61	2.16	3.23	4.05	4.75	5.27	5.76	6.16	6.94	7.46	8.54	9.55	11.26	16.58	27.84
17	0.29	0.39	0.47	0.63	0.86	1.16	1.39	1.72	2.32	3.45	4.33	5.08	5.63	6.16	6.59	7.43	7.98	9.14	10.22	12.04	17.74	29.78
18	0.31	0.42	0.50	0.68	0.92	1.24	1.48	1.83	2.47	3.68	4.61	5.42	6.00	6.57	7.03	7.91	8.50	9.74	10.89	12.83	18.89	31.72
19	0.33	0.44	0.53	0.72	0.97	1.32	1.57	1.94	2.62	3.90	4.90	5.75	6.37	6.97	7.46	8.40	9.03	10.34	11.56	13.61	20.05	33.67
20	0.35	0.47	0.56	0.76	1.03	1.39	1.66	2.06	2.77	4.13	5.18	6.08	6.74	7.37	7.89	8.88	9.55	10.93	12.22	14.40	21.21	35.61
22	0.38	0.52	0.62	0.84	1.14	1.54	1.84	2.28	3.07	4.58	5.74	6.74	7.47	8.18	8.75	9.85	10.59	12.13	13.56	15.97	23.53	39.50
24	0.42	0.57	0.68	0.92	1.25	1.70	2.03	2.50	3.37	5.03	6.31	7.41	8.21	8.98	9.61	10.82	11.63	13.32	14.89	17.55	25.84	43.39
26	0.46	0.62	0.74	1.01	1.36	1.85	2.21	2.73	3.68	5.48	6.88	8.07	8.95	9.79	10.47	11.79	12.67	14.51	16.23	19.12	28.16	47.28
28	0.50	0.67	0.81	1.09	1.48	2.00	2.39	2.95	3.98	5.93	7.44	8.74	9.68	10.59	11.33	12.76	13.72	15.71	17.56	20.69	30.48	51.17
30	0.54	0.73	0.87	1.17	1.59	2.15	2.57	3.18	4.28	6.38	8.01	9.40	10.42	11.40	12.19	13.73	14.76	16.90	18.90	22.26	32.79	55.06
32	0.57	0.78	0.93	1.26	1.70	2.30	2.75	3.40	4.58	6.83	8.57	10.06	11.15	12.20	13.05	14.70	15.80	18.09	20.23	23.84	35.11	58.95
34	0.61	0.83	0.99	1.34	1.81	2.46	2.93	3.63	4.89	7.28	9.14	10.73	11.89	13.01	13.92	15.67	16.84	19.29	21.57	25.41	37.42	62.84
36	0.65	0.88	1.05	1.42	1.93	2.61	3.11	3.85	5.19	7.73	9.70	11.39	12.63	13.81	14.78	16.64	17.89	20.48	22.90	26.98	39.74	66.73
38	0.69	0.93	1.11	1.50	2.04	2.76	3.30	4.08	5.49	8.18	10.27	12.06	13.36	14.62	15.64	17.61	18.93	21.68	24.24	28.56	42.06	70.62
40	0.73	0.98	1.17	1.59	2.15	2.91	3.48	4.30	5.79	8.64	10.83	12.72	14.10	15.42	16.50	18.58	19.97	22.87	25.57	30.13	44.37	74.51

50 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.26	0.35	0.42	0.56	0.76	1.03	1.24	1.53	2.06	3.07	3.85	4.52	5.01	5.48	5.86	6.60	7.10	8.13	9.09	10.71	15.77	26.48
15	0.28	0.38	0.45	0.61	0.83	1.12	1.34	1.65	2.23	3.32	4.16	4.89	5.42	5.93	6.34	7.14	7.67	8.79	9.83	11.58	17.05	28.63
16	0.30	0.41	0.48	0.66	0.89	1.20	1.44	1.78	2.39	3.57	4.48	5.25	5.82	6.37	6.82	7.68	8.25	9.45	10.56	12.45	18.33	30.78
17	0.32	0.43	0.52	0.70	0.95	1.29	1.54	1.90	2.56	3.82	4.79	5.62	6.23	6.82	7.29	8.21	8.83	10.11	11.30	13.32	19.61	32.93
18	0.34	0.46	0.55	0.75	1.01	1.37	1.64	2.03	2.73	4.07	5.10	5.99	6.64	7.26	7.77	8.75	9.40	10.77	12.04	14.19	20.89	35.08
19	0.36	0.49	0.59	0.79	1.07	1.46	1.74	2.15	2.89	4.32	5.41	6.36	7.04	7.71	8.24	9.28	9.98	11.43	12.78	15.06	22.17	37.23
20	0.38	0.52	0.62	0.84	1.14	1.54	1.84	2.27	3.06	4.56	5.73	6.72	7.45	8.15	8.72	9.82	10.56	12.09	13.52	15.92	23.45	39.38
22	0.43	0.58	0.69	0.93	1.26	1.71	2.04	2.52	3.40	5.06	6.35	7.46	8.26	9.04	9.67	10.89	11.71	13.41	14.99	17.66	26.01	43.68
24	0.47	0.63	0.76	1.02	1.38	1.88	2.24	2.77	3.73	5.56	6.98	8.19	9.08	9.93	10.63	11.97	12.86	14.73	16.47	19.40	28.58	47.98
26	0.51	0.69	0.82	1.11	1.51	2.04	2.44	3.02	4.07	6.06	7.60	8.93	9.89	10.82	11.58	13.04	14.01	16.05	17.94	21.14	31.14	52.28
28	0.55	0.75	0.89	1.21	1.63	2.21	2.64	3.27	4.40	6.56	8.23	9.66	10.71	11.71	12.53	14.11	15.17	17.37	19.42	22.88	33.70	56.58
30	0.59	0.80	0.96	1.30	1.76	2.38	2.84	3.51	4.73	7.06	8.85	10.39	11.52	12.60	13.48	15.18	16.32	18.69	20.89	24.62	36.26	60.88
32	0.63	0.86	1.03	1.39	1.88	2.55	3.04	3.76	5.07	7.56	9.48	11.13	12.33	13.49	14.44	16.26	17.47	20.01	22.37	26.36	38.82	65.18
34	0.68	0.92	1.09	1.48	2.01	2.72	3.24	4.01	5.40	8.05	10.10	11.86	13.15	14.38	15.39	17.33	18.63	21.33	23.85	28.10	41.38	69.48
36	0.72	0.97	1.16	1.57	2.13	2.88	3.44	4.26	5.74	8.55	10.73	12.60	13.96	15.27	16.34	18.40	19.78	22.65	25.32	29.84	43.94	73.78
38	0.76	1.03	1.23	1.66	2.25	3.05	3.64	4.51	6.07	9.05	11.35	13.33	14.77	16.16	17.29	19.47	20.93	23.97	26.80	31.58	46.51	78.09
40	0.80	1.09	1.30	1.76	2.38	3.22	3.84	4.76	6.41	9.55	11.98	14.06	15.59	17.05	18.24	20.55	22.08	25.29	28.27	33.32	49.07	82.39

100 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.28	0.38	0.46	0.62	0.83	1.13	1.35	1.67	2.25	3.35	4.21	4.94	5.47	5.99	6.41	7.21	7.75	8.88	9.93	11.70	17.23	28.93
15	0.30	0.41	0.49	0.67	0.90	1.22	1.46	1.81	2.43	3.63	4.55	5.34	5.92	6.47	6.93	7.80	8.38	9.60	10.73	12.65	18.63	31.28
16	0.33	0.44	0.53	0.72	0.97	1.31	1.57	1.94	2.61	3.90	4.89	5.74	6.36	6.96	7.45	8.39	9.01	10.32	11.54	13.60	20.03	33.63
17	0.35	0.47	0.57	0.77	1.04	1.41	1.68	2.08	2.80	4.17	5.23	6.14	6.81	7.45	7.97	8.97	9.64	11.04	12.35	14.55	21.43	35.98
18	0.37	0.51	0.60	0.82	1.11	1.50	1.79	2.21	2.98	4.44	5.57	6.54	7.25	7.93	8.49	9.56	10.27	11.76	13.15	15.50	22.83	38.33
19	0.40	0.54	0.64	0.87	1.17	1.59	1.90	2.35	3.16	4.71	5.91	6.94	7.70	8.42	9.01	10.14	10.90	12.49	13.96	16.45	24.22	40.67
20	0.42	0.57	0.68	0.92	1.24	1.68	2.01	2.48	3.35	4.99	6.26	7.34	8.14	8.91	9.53	10.73	11.53	13.21	14.77	17.40	25.62	43.02
22	0.46	0.63	0.75	1.02	1.38	1.87	2.23	2.75	3.71	5.53	6.94	8.15	9.03	9.88	10.57	11.90	12.79	14.65	16.38	19.30	28.42	47.72
24	0.51	0.69	0.82	1.12	1.51	2.05	2.45	3.03	4.08	6.08	7.62	8.95	9.92	10.85	11.61	13.07	14.05	16.09	17.99	21.20	31.22	52.42
26	0.56	0.75	0.90	1.22	1.65	2.23	2.67	3.30	4.44	6.62	8.31	9.75	10.81	11.82	12.65	14.25	15.31	17.53	19.60	23.10	34.02	57.12
28	0.60	0.81	0.97	1.32	1.78	2.42	2.88	3.57	4.81	7.16	8.99	10.55	11.70	12.80	13.69	15.42	16.57	18.98	21.22	25.00	36.82	61.82
30	0.65	0.88	1.05	1.42	1.92	2.60	3.10	3.84	5.17	7.71	9.67	11.36	12.59	13.77	14.73	16.59	17.83	20.42	22.83	26.90	39.61	66.52
32	0.69	0.94	1.12	1.52	2.06	2.78	3.32	4.11	5.54	8.25	10.36	12.16	13.47	14.74	15.77	17.76	19.09	21.86	24.44	28.80	42.41	71.21
34	0.74	1.00	1.19	1.62	2.19	2.97	3.54	4.38	5.90	8.80	11.04	12.96	14.36	15.71	16.81	18.93	20.35	23.30	26.05	30.70	45.21	75.91
36	0.78	1.06	1.27	1.72	2.33	3.15	3.76	4.65	6.27	9.34	11.72	13.76	15.25	16.69	17.85	20.10	21.61	24.74	27.67	32.60	48.01	80.61
38	0.83	1.12	1.34	1.82	2.46	3.33	3.98	4.92	6.63	9.89	12.40	14.56	16.14	17.66	18.89	21.28	22.87	26.19	29.28	34.50	50.81	85.31
40	0.88	1.19	1.42	1.92	2.60	3.52	4.20	5.20	7.00	10.43	13.09	15.37	17.03	18.63	19.93	22.45	24.13	27.63	30.89	36.40	53.61	90.01

500 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.34	0.46	0.54	0.74	1.00	1.35	1.62	2.00	2.69	4.01	5.04	5.91	6.55	7.17	7.67	8.64	9.28	10.63	11.88	14.00	20.62	34.6
15	0.36	0.49	0.59	0.80	1.08	1.46	1.75	2.16	2.91	4.34	5.44	6.39	7.08	7.75	8.29	9.34	10.04	11.49	12.85	15.14	22.30	37.4
16	0.39	0.53	0.63	0.86	1.16	1.57	1.88	2.32	3.13	4.67	5.85	6.87	7.62	8.33	8.91	10.04	10.79	12.36	13.81	16.28	23.97	40.3
17	0.42	0.57	0.68	0.92	1.24	1.68	2.01	2.49	3.35	4.99	6.26	7.35	8.15	8.91	9.54	10.74	11.54	13.22	14.78	17.41	25.65	43.1
18	0.45	0.60	0.72	0.98	1.32	1.79	2.14	2.65	3.57	5.32	6.67	7.83	8.68	9.50	10.16	11.44	12.30	14.08	15.74	18.55	27.32	45.9
19	0.47	0.64	0.77	1.04	1.41	1.90	2.27	2.81	3.79	5.64	7.08	8.31	9.21	10.08	10.78	12.14	13.05	14.95	16.71	19.69	29.00	48.7
20	0.50	0.68	0.81	1.10	1.49	2.01	2.40	2.97	4.00	5.97	7.49	8.79	9.74	10.66	11.40	12.84	13.81	15.81	17.67	20.83	30.67	51.5
22	0.56	0.75	0.90	1.22	1.65	2.23	2.67	3.30	4.44	6.62	8.31	9.75	10.81	11.82	12.65	14.25	15.31	17.53	19.60	23.10	34.02	57.1
24	0.61	0.83	0.99	1.34	1.81	2.45	2.93	3.62	4.88	7.27	9.12	10.71	11.87	12.99	13.90	15.65	16.82	19.26	21.53	25.37	37.37	62.7
26	0.67	0.90	1.08	1.46	1.97	2.67	3.19	3.95	5.32	7.92	9.94	11.67	12.94	14.15	15.14	17.05	18.33	20.99	23.46	27.65	40.72	68.4
28	0.72	0.98	1.16	1.58	2.14	2.89	3.45	4.27	5.75	8.58	10.76	12.63	14.00	15.32	16.39	18.45	19.84	22.71	25.39	29.92	44.07	74.0
30	0.77	1.05	1.25	1.70	2.30	3.11	3.72	4.60	6.19	9.23	11.58	13.59	15.06	16.48	17.63	19.86	21.34	24.44	27.33	32.20	47.42	79.6
32	0.83	1.12	1.34	1.82	2.46	3.33	3.98	4.92	6.63	9.88	12.40	14.55	16.13	17.64	18.88	21.26	22.85	26.17	29.26	34.47	50.77	85.2
34	0.88	1.20	1.43	1.94	2.62	3.55	4.24	5.25	7.07	10.53	13.21	15.51	17.19	18.81	20.12	22.66	24.36	27.89	31.19	36.74	54.12	90.9
36	0.94	1.27	1.52	2.06	2.78	3.77	4.50	5.57	7.50	11.18	14.03	16.47	18.26	19.97	21.37	24.06	25.87	29.62	33.12	39.02	57.47	96.5
38	0.99	1.35	1.61	2.18	2.95	3.99	4.77	5.89	7.94	11.84	14.85	17.43	19.32	21.14	22.61	25.47	27.37	31.35	35.05	41.29	60.82	102.1
40	1.05	1.42	1.70	2.30	3.11	4.21	5.03	6.22	8.38	12.49	15.67	18.39	20.39	22.30	23.86	26.87	28.88	33.07	36.98	43.57	64.17	107.7

1000 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.45	0.61	0.73	0.99	1.34	1.81	2.16	2.67	3.60	5.36	6.73	7.90	8.76	9.58	10.25	11.54	12.41	14.21	15.88	18.72	27.57	46.3
15	0.49	0.66	0.79	1.07	1.44	1.96	2.34	2.89	3.89	5.80	7.28	8.54	9.47	10.36	11.08	12.48	13.41	15.36	17.17	20.24	29.80	50.0
16	0.52	0.71	0.85	1.15	1.55	2.10	2.51	3.11	4.18	6.24	7.82	9.18	10.18	11.14	11.91	13.42	14.42	16.51	18.46	21.76	32.04	53.8
17	0.56	0.76	0.91	1.23	1.66	2.25	2.69	3.32	4.48	6.67	8.37	9.83	10.89	11.91	12.75	14.35	15.43	17.67	19.75	23.28	34.28	57.6
18	0.60	0.81	0.96	1.31	1.77	2.40	2.86	3.54	4.77	7.11	8.92	10.47	11.60	12.69	13.58	15.29	16.44	18.82	21.04	24.80	36.52	61.3
19	0.63	0.86	1.02	1.39	1.88	2.54	3.04	3.76	5.06	7.54	9.46	11.11	12.31	13.47	14.41	16.23	17.44	19.98	22.33	26.32	38.76	65.1
20	0.67	0.91	1.08	1.47	1.99	2.69	3.21	3.97	5.35	7.98	10.01	11.75	13.02	14.25	15.24	17.17	18.45	21.13	23.62	27.84	41.00	68.8
22	0.74	1.01	1.20	1.63	2.20	2.98	3.56	4.41	5.94	8.85	11.10	13.03	14.45	15.80	16.91	19.04	20.47	23.44	26.20	30.88	45.47	76.4
24	0.82	1.11	1.32	1.79	2.42	3.28	3.91	4.84	6.52	9.72	12.20	14.32	15.87	17.36	18.57	20.92	22.48	25.74	28.78	33.92	49.95	83.9
26	0.89	1.20	1.44	1.95	2.64	3.57	4.26	5.28	7.11	10.59	13.29	15.60	17.29	18.92	20.24	22.79	24.50	28.05	31.36	36.96	54.43	91.4
28	0.96	1.30	1.56	2.11	2.85	3.87	4.62	5.71	7.69	11.46	14.38	16.88	18.71	20.47	21.90	24.67	26.51	30.36	33.94	39.99	58.90	98.9
30	1.04	1.40	1.67	2.27	3.07	4.16	4.97	6.14	8.27	12.33	15.47	18.17	20.14	22.03	23.57	26.54	28.53	32.67	36.52	43.03	63.38	106.4
32	1.11	1.50	1.79	2.43	3.29	4.45	5.32	6.58	8.86	13.21	16.57	19.45	21.56	23.58	25.23	28.42	30.54	34.97	39.10	46.07	67.86	113.9
34	1.18	1.60	1.91	2.59	3.51	4.75	5.67	7.01	9.44	14.08	17.66	20.73	22.98	25.14	26.90	30.29	32.56	37.28	41.68	49.11	72.34	121.5
36	1.26	1.70	2.03	2.75	3.72	5.04	6.02	7.45	10.03	14.95	18.75	22.02	24.40	26.70	28.56	32.17	34.57	39.59	44.26	52.15	76.81	129.0
38	1.33	1.80	2.15	2.91	3.94	5.33	6.37	7.88	10.61	15.82	19.85	23.30	25.83	28.25	30.23	34.04	36.59	41.90	46.84	55.19	81.29	136.5
40	1.40	1.90	2.27	3.07	4.16	5.63	6.72	8.31	11.20	16.69	20.94	24.58	27.25	29.81	31.89	35.91	38.60	44.20	49.42	58.23	85.77	144.0

10,000 Year Storm for Solano and Yolo Counties

Mean Ann Precip	5M	10M	15M	30M	1H	2H	3H	6H	12H	24H	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	Year
14	0.63	0.85	1.02	1.38	1.87	2.53	3.03	3.74	5.04	7.51	9.43	11.07	12.27	13.42	14.36	16.17	17.38	19.90	22.25	26.22	38.6	64.8
15	0.68	0.92	1.10	1.49	2.02	2.74	3.27	4.05	5.45	8.13	10.19	11.97	13.26	14.51	15.52	17.48	18.79	21.52	24.06	28.35	41.8	70.1
16	0.73	0.99	1.19	1.61	2.18	2.95	3.52	4.35	5.86	8.74	10.96	12.87	14.26	15.60	16.69	18.80	20.20	23.14	25.87	30.48	44.9	75.4
17	0.78	1.06	1.27	1.72	2.33	3.15	3.76	4.65	6.27	9.35	11.72	13.77	15.26	16.69	17.86	20.11	21.62	24.75	27.67	32.61	48.0	80.6
18	0.84	1.13	1.35	1.83	2.48	3.36	4.01	4.96	6.68	9.96	12.49	14.66	16.25	17.78	19.02	21.42	23.03	26.37	29.48	34.74	51.2	85.9
19	0.89	1.20	1.43	1.94	2.63	3.56	4.25	5.26	7.09	10.57	13.26	15.56	17.25	18.87	20.19	22.74	24.44	27.98	31.29	36.87	54.3	91.2
20	0.94	1.27	1.52	2.06	2.78	3.77	4.50	5.57	7.50	11.18	14.02	16.46	18.25	19.96	21.36	24.05	25.85	29.60	33.09	38.99	57.4	96.4
22	1.04	1.41	1.68	2.28	3.09	4.18	4.99	6.17	8.32	12.40	15.55	18.26	20.24	22.14	23.69	26.68	28.67	32.83	36.71	43.25	63.7	107.0
24	1.14	1.55	1.85	2.50	3.39	4.59	5.48	6.78	9.14	13.62	17.08	20.06	22.23	24.32	26.02	29.30	31.50	36.07	40.32	47.51	70.0	117.5
26	1.25	1.69	2.01	2.73	3.69	5.00	5.97	7.39	9.95	14.84	18.62	21.86	24.22	26.50	28.35	31.93	34.32	39.30	43.94	51.77	76.2	128.0
28	1.35	1.83	2.18	2.95	4.00	5.42	6.47	8.00	10.77	16.06	20.15	23.65	26.22	28.68	30.68	34.55	37.14	42.53	47.55	56.03	82.5	138.6
30	1.45	1.96	2.35	3.18	4.30	5.83	6.96	8.61	11.59	17.28	21.68	25.45	28.21	30.86	33.02	37.18	39.96	45.76	51.17	60.29	88.8	149.1
32	1.55	2.10	2.51	3.40	4.61	6.24	7.45	9.21	12.41	18.50	23.21	27.25	30.20	33.04	35.35	39.81	42.79	49.00	54.78	64.54	95.1	159.6
34	1.66	2.24	2.68	3.63	4.91	6.65	7.94	9.82	13.23	19.72	24.74	29.05	32.19	35.22	37.68	42.43	45.61	52.23	58.39	68.80	101.3	170.1
36	1.76	2.38	2.84	3.85	5.21	7.06	8.43	10.43	14.05	20.94	26.27	30.84	34.19	37.40	40.01	45.06	48.43	55.46	62.01	73.06	107.6	180.7
38	1.86	2.52	3.01	4.07	5.52	7.47	8.92	11.04	14.87	22.16	27.80	32.64	36.18	39.58	42.34	47.69	51.26	58.69	65.62	77.32	113.9	191.2
40	1.96	2.66	3.17	4.30	5.82	7.88	9.41	11.65	15.69	23.38	29.33	34.44	38.17	41.76	44.68	50.31	54.08	61.93	69.24	81.58	120.1	201.7

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix C

Spreadsheets for Hydrologic and Hydraulic Calculations (Digital Files Available from City Upon Request)

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix D

Rancho Arroyo Subbasin Storm Drainage Evaluation

CITY OF WINTERS

RANCHO ARROYO SUBBASIN

STORM DRAINAGE EVALUATION

April 2004

Prepared By:

WOOD RODGERS
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INTRODUCTION

The existing Rancho Arroyo Regional Detention Pond (Pond) was constructed as part of the Carter Ranch Subdivision, which is located in the Rancho Arroyo drainage basin (Figure 1). The Rancho Arroyo basin is located in the City of Winters, California, north of Grant Avenue/Highway 128 between Taylor Street and Cemetery Drive, and bounded on the north by County Road 33. North of Roosevelt Avenue, the Rancho Arroyo basin extends west to the City's urban limit.

The Pond is accessible via the network of city roads. The area in and around the Rancho Arroyo basin consists of existing residential areas, public land (cemetery, schools, etc.), and undeveloped areas zoned for future residential development. A map of land uses that was revised in 2003 as part of the City's General Plan amendment is included in this report (Map 1).

The Pond is designed to replace storage volume naturally available within topographically low ground, which formed a seasonal pond in the Rancho Arroyo basin and attenuated storm runoff prior to development. The Pond would also provide required water quality treatment for portions of the basin undergoing new development (with existing developed areas draining directly to pipes and discharge into Dry Creek) as well as add sufficient volume to mitigate the impact of increased runoff from proposed development. The primary features of the Pond include an outlet structure, outlet conduit, pump station, water quality volume, and flood control volume. The layout of features comprising the proposed Pond design is illustrated on Exhibit C of the Morton & Pitalo report entitled, "Drainage Study for Carter Ranch," July 12, 2002.

The City is interested in determining the appropriate configuration and operation of a regional detention pond to serve the Rancho Arroyo basin while conforming to the City's overall drainage report for the subbasin. In preparing the City of Winters' drainage report for the Putah Creek/Dry Creek subbasins, Wood Rodgers was requested to evaluate the existing design and determine modifications, if needed, in order that storm drainage within the Rancho Arroyo basin is dealt with in a manner consistent with the master plan for the subbasin.

The City also requested an evaluation of the runoff that is currently draining to the Rancho Arroyo basin from the Silver Ridge Estates property. Silver Ridge Estates (Figure 1) has been graded to drain overland into the Rancho Arroyo basin, contrary to what was specified in the Storm Drainage Master Plan developed by CH2M Hill in 1992 for the City. In preparing the City of Winters' drainage report for the Putah Creek/Dry Creek subbasins, Wood Rodgers was requested to evaluate the affects of this runoff on the performance of existing storm drainage facilities in the Rancho Arroyo basin, and to determine modifications, if needed, in order that storm drainage within the Rancho Arroyo basin is dealt with in a manner consistent with the master plan for the subbasin. The property surrounding the existing Pond includes north Main Street on the west, land zoned for low-income housing on the north, a cemetery on the east, and Carter Ranch development on the south. Accordingly, increasing the "footprint" of the Pond may be possible but is not desirable.

Two drainage studies were previously developed to analyze the affects of drainage in the Rancho Arroyo basin. These studies include:

1. "Storm Drainage Study for Winters Highlands Subdivision Unit 1," prepared by Laugenour and Meikle in 1994, for Triad Homes, Inc. This report includes a comprehensive examination of existing conditions as well as ultimate drainage requirements for the Winters Highlands Subdivision and the Rancho Arroyo drainage shed designated to drain into the North Main Street Extension described in the "West Central Master Plan" (WCMP).

2. "Drainage Study for Carter Ranch," prepared by Morton & Pitalo, Inc., updated July 12, 2002 for the City of Winters. This report includes an examination of existing conditions as well as ultimate drainage requirements for the Carter Ranch Subdivision designated to drain into the North Main Street trunk.

PURPOSE

The purpose of this analysis is to include:

1. Evaluating the design of the Rancho Arroyo storm drainage facilities prepared by others and developing, if deemed necessary, measures to modify the facilities to deal with storm drainage consistent with the criteria formulated for the subbasin.

2. Evaluating the affects of draining the Silver Ridge Estates property to the 48-inch storm drain that runs along north Main Street, to determine if additional facilities are required to convey this runoff in a manner consistent with the criteria set forth in the drainage report.

3. Evaluating the suitability of using the Rancho Arroyo storm drainage facilities as interim drainage facilities for a portion of the Winters Highlands Subdivision included Subbasin ROF-4D as shown on Figure 1. Prior to the Moody Slough basin drainage facilities being constructed, ultimately this subbasin would be integrated into the Moody Slough basin master plan.

HYDROLOGIC MODELING/DESIGN PARAMETERS

The existing Pond was designed and constructed on behalf of the developers as part of the Carter Ranch Subdivision. As part of the design process, several HEC-HMS hydrologic models were prepared. These models do not reflect off-site runoff from Silver Ridge Estates, which has been graded to drain to the North Main Street trunk. In addition, these models do not account for runoff from Subbasin ROF-4D, which the City wants to drain to the Rancho Arroyo Detention Pond during an interim period until drainage facilities for the Moody Slough subbasin are constructed. The drainage subbasins for the purposes of hydrologic modeling for the Rancho Arroyo basin are shown on Figure 1.

The existing Pond was designed with provision for an overland release with a trapezoidal weir for storm events exceeding the 100-year storm. As-built drawings received from the City indicate the crest of the weir is at El. 137.6. The maximum water surface in the Pond should not exceed El. 136.6 to maintain a one-foot of freeboard, which is deemed a prudent grade line for non-leveed drainage facilities. The original HEC-HMS modeling provided in Reference 2 above indicated the Pond would be allowed to fill to El. 138.1 during an 8-day storm event, with flow overtopping the weir and spilling overland. The portion of this overland release (if any) collected by the Carter Ranch storm drain system is not known, and is beyond the scope of this study. However, the City's intent is to ensure the Pond would provide flood control necessary for the 100-year event and to minimize flooding during the 100-year event wherever possible. Accordingly, the target water surface elevation in the Pond is set at El. 136.6 for the 100-year event.

The original Carter Ranch HEC-HMS modeling used an initial loss value of 0.25 inch for post-development conditions. This value is higher than the initial loss used in the drainage report hydrology and does not appear to account for variation in storm exceedence frequency or variations in soils and land use within the basin. Two sets of models were run independently for purposes of comparing these loss rates for the 100-year storm to evaluate the worst-case scenario. Using initial loss rates consistent with the drainage report resulted in larger runoff volumes and peak flows.

To calculate Lag time for HEC-HMS input, the *Snyder Method* was applied to the drainage areas being added to the model in the analysis to reflect drainage from the Silver Ridge Estates and Winters Highlands subdivisions. This is consistent with the modeling for the drainage report.

The original modeling does not reflect the storage available below El. 130.8 in the Pond, or the effects of pumping water out of the Pond during flood events. To best analyze the effects of adding storage volume, and the interaction of pumping this volume during storm events, it is most appropriately modeled using a hydraulic modeling program with HEC-HMS hydrographs.

For the long-duration storm analysis, the original modeling used 8-day storm data from a storm observed in February 1986, at the City of Roseville Filter Station (the precipitation distribution was adjusted for depth in accordance with the City's 8-day storm depth provided by DWR, Sacramento; however, no frequency is referenced to this event). As part of the study for the Covell Drain performed on behalf of the Yolo County Flood Control & Water Conservation District by Borcalli & Associates, Inc., Mr. James D. Goodridge prepared an appendix entitled, "Solano & Yolo County Design Rainfall." The 100-year, 10-day storm depth for Winters as provided in the Goodridge document was used to develop a model that could be used to compare the 8-day storm developed for the original HEC-HMS models, and to determine the worst-case long duration storm event. The 100-year, 10-day storm pattern proved to be the "worst case" and was selected as the long duration design event.

HYDRAULIC MODELING

To analyze the performance of the existing Pond and simultaneously evaluate the performance of the North Main Street trunk, a series of unsteady state hydraulic models were developed using MIKE SWMM (EPA SWMM with graphical interface). SWMM was selected to evaluate the effects of hydraulically derived discharge parameters effecting Pond operation that cannot be adequately modeled in HEC-HMS. The SWMM models were developed using the HEC-HMS models' routing input as a template, and utilized the same parameters for slopes, pipe lengths, pipe diameters, and Pond volumes, which were verified with the facilities drawings in the developer's report. Runoff hydrographs produced by HEC-HMS for 24-hour and 10-day events were run through the SWMM model at the appropriate nodes. Several model configurations were used to evaluate draining the Pond by gravity, draining the Pond by pumping, with and without the Subbasin ROF-4D draining to the Pond, as well as with and without the drainage from Silver Ridge Estates. In all cases, the maximum allowable water surface in the Pond was El. 136.6.

DRAINAGE ANALYSES

A part of this analysis is to evaluate the performance of the existing outlet configuration, and to determine if a more suitable outlet configuration exists. The existing Pond outlet consists of a 24-inch perforated riser, which drains the Pond by gravity to El. 130.8. A manually operated 0.8 cfs pump has been installed to serve as backup to drain the Pond below El. 130.8 in case the Pond does not drain within 15 days of the last storm.

In addition, the following drainage scenarios were analyzed for the Rancho Arroyo Basin:

<u>Drainage Scenario 1</u>

Under Drainage Scenario 1, the Pond drains by gravity to the North Main trunk through a pipe. The existing standpipe is removed, and the existing 0.8 cfs pump is not operated during the storm. Storage below El. 130.8 is not pumped and is considered ineffective for flood control, and no drainage from Subbasin ROF-4D is routed to the Pond. Drainage from Silver Ridge Estates is routed through the North Main Street trunk.

<u>Drainage Scenario 2</u>

Under Drainage Scenario 2, the Pond drains by gravity to the North Main trunk through a pipe. The existing standpipe is removed, and the existing 0.8 cfs pump is not operated during the storm. Storage below El. 130.8 is pumped between storms, and no drainage from Subbasin ROF-4D is routed to the Pond. Drainage from Silver Ridge Estates is routed through the North Main Street trunk.

Drainage Scenario 3

Under Drainage Scenario 3, the Pond is drained by pumping. The existing standpipe is removed, and the existing 0.8 cfs pump is not operated during the storm. Storage below El. 130.8 is pumped between storms, and drainage from Subbasin ROF-4D is not routed to the Pond. Drainage from Silver Ridge Estates is routed through the North Main Street trunk.

Drainage Scenario 4

Under Drainage Scenario 4, the Pond is drained by gravity through a 24-inch pipe. The existing standpipe is removed, and the existing 0.8 cfs pump is not operated during the storm. Storage below El. 130.8 is pumped between storms, and drainage from Subbasin ROF-4D is routed to the Pond. Drainage from Silver Ridge Estates is routed through the North Main Street trunk.

Drainage Scenario 5

Under Drainage Scenario 5, the Pond drains by pumping. The existing standpipe and the existing 0.8 cfs pump are removed. Storage below El. 130.8 is pumped between storms, and drainage from Subbasin ROF-4D is routed to the Pond. Drainage from Silver Ridge Estates is routed through the North Main Street trunk. Pumping is configured to be consistent with the guidelines for water quality detention and flood control set forth in the drainage report.

RESULTS OF ANALYSES

The results of the analyses of the existing Pond configuration and the various drainage scenarios described above, are summarized below.

Existing Pond Configuration

The existing design utilized a perforated standpipe to limit flow draining from the Pond and was intended to ensure the capacity of the outfall pipe draining to Dry Creek would not be exceeded. The results of the SWMM analysis indicate that such limitation is unnecessary, as there is capacity available in the North Main Street trunk even when flow from Silver Ridge Estates is added. The system functions adequately under a 100-year, 24-hour storm, however, the target water surface elevation of El. 136.6 in the Pond is exceeded under a 10-day storm. The existing Pond configuration is shown on Figure 2.

Introducing runoff from Silver Ridge Estates into the North Main Street trunk during the peak 100-year storm event can be accommodated without adverse impacts. The MIKE SWMM modeling shows maximum values in the range of El. 136 to El. 137 in the vicinity of the 36-inch portion of the North Main Street trunk, which is comparable to existing ground and is expected to be lower than the adjacent lots indicated on the Carter Ranch Grading Plan. As such, runoff from Silver Ridge Estates does not present a risk of flood damage to homes in the Carter Ranch Subdivision during the 100-year event. Existing topographic mapping indicates there is low ground west of North Main Street, which would require a small amount of fill to be brought to sufficient grade to avoid flooding.

Drainage Scenario 1

Under Drainage Scenario 1, it is necessary to expand the Pond footprint with a 40-foot bench at El. 131 along the north side of the Pond (increasing the Pond footprint by 0.44 acres). The revised Pond layout for this design configuration covers

approximately 3.85 acres and is shown on Figure 3. Under gravity drainage, the 100-year, 24-hour event is the worst-case scenario for the Pond. Under this scenario, the maximum 100-year, 24-hour water surface is El. 136.6.

Drainage Scenario 2

Under Drainage Scenario 2, it is necessary to expand the Pond footprint with a 15-foot-wide bench at El. 131 along the north side of the Pond (increasing the footprint by 0.16 acres). The revised Pond layout for this design configuration covers approximately 3.57 acres and is shown on Figure 4. Under gravity drainage, the 100-year, 24-hour event is the worst-case scenario for the Pond. Under this scenario, the maximum 100-year, 24-hour water surface is 136.52.

Drainage Scenario 3

Under Drainage Scenario 3, the footprint of the Pond does not change. The 100-year, 24-hour event is the worst-case scenario for the Pond. A 1 cfs pump rate is required to manage the water quality functions of the Pond. An additional 2 cfs pump rate is required for flood control, for a total pumping rate of 3 cfs. The 0.8 cfs unit would be removed. Under this scenario, the maximum 100-year, 24-hour water surface is El. 136.0. The revised Pond layout for this design configuration covers approximately 3.41 acres and is shown on Figure 5.

Drainage Scenario 4

Drainage Scenario 4 requires a substantial increase in the Pond footprint to keep the water surface under El. 136.6. The revised Pond layout for this design configuration covers approximately 7.69 acres (increasing the footprint by 4.28 acres) and is shown on Figure 6. Under gravity drainage, the 100-year, 24-hour event is the worst-case

scenario for the Pond. Under this configuration, the maximum 100-year, 24-hour water surface is 136.6.

Drainage Scenario 5

Drainage Scenario 5 does not require modification to the existing Pond footprint (Figure 7). Under a pumping operation, the 100-year, 24-hour event is the worst-case scenario for the Pond. A pump station with two pump stages is required to manage the water quality and flood control functions of the Pond. One pump stage would be 1 cfs, and would ensure appropriate water quality detention time, and a second flood control pump stage of 13.5 cfs would be required for a total pumping capacity of 14.5 cfs. The 0.8 cfs unit would be removed. Under this configuration, the maximum 100-year, 24-hour water surface is El. 136.2.

FINDINGS AND RECOMMENDATION

Findings

Summarized below are the findings resulting from the analyses performed for this work.

1. The existing Pond configuration does not fully mitigate the worst-case scenario and allows flow to be released from the Pond, creating street flooding.

2. Storm drainage from Silver Ridge Estates can be discharged to the North Main Street trunk without adversely affecting drainage in other areas of the system.

3. To avoid enlarging the footprint of the existing Pond, additional pumping capacity is required depending upon whether or not runoff from Subbasin ROF-4D is included.

Recommendation

Based upon the results of the analyses performed and understanding the City prefers not to enlarge the footprint of the existing Pond, it is recommended the City consider implementing Drainage Scenario 3 or Drainage Scenario 5, depending upon the timing for development of the Winters Highland Subdivision outside the Rancho Arroyo basin.

REFERENCES

Borcalli & Associate, Inc.; "Covell Drainage System Comprehensive Drainage Plan," WMP-93-01-03, prepared for the Yolo County Flood Control and Water Conservation district, September 1993.

CH2M Hill; "Storm Drainage Master Plan, City of Winters," May 1992.

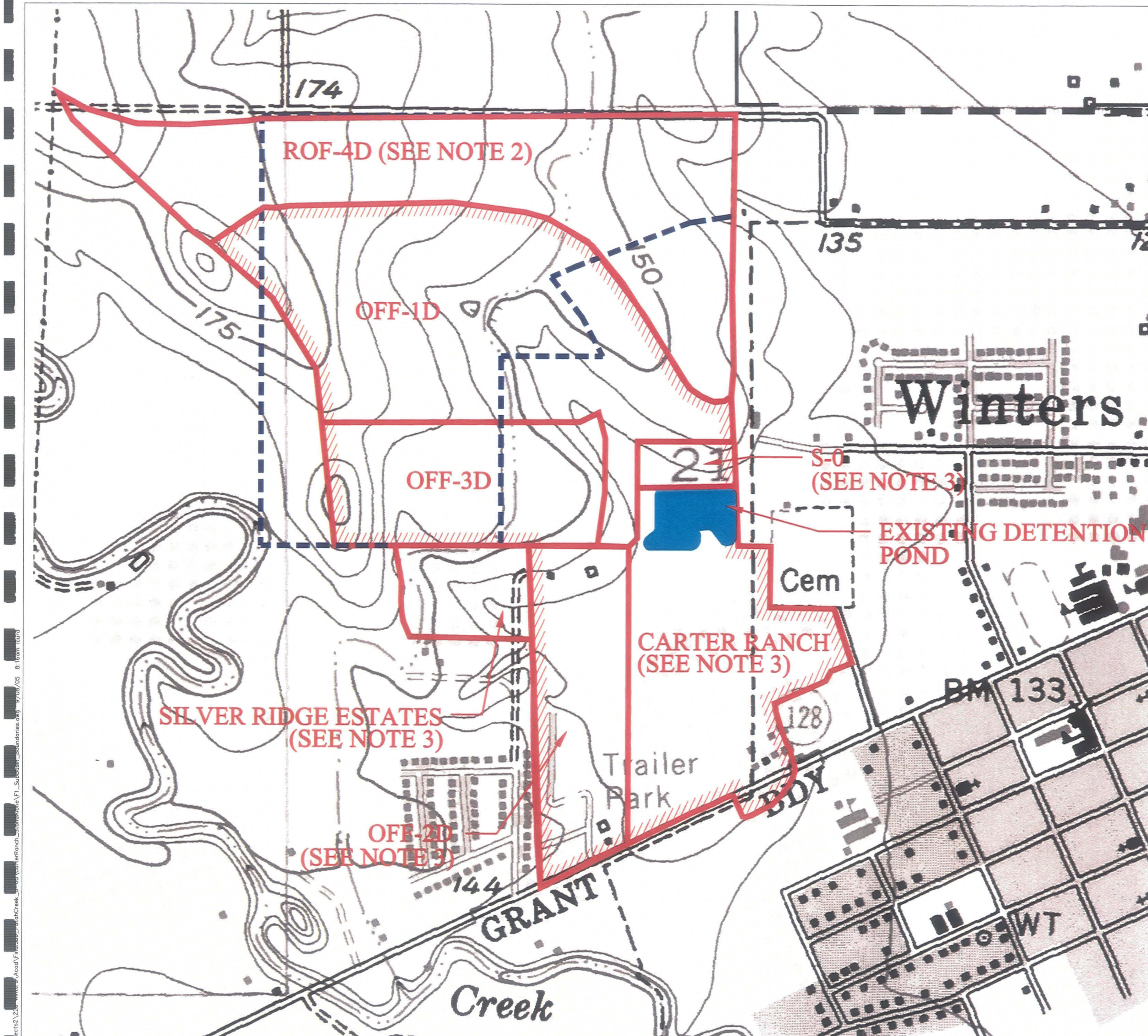
Goodridge, James D.; Paper entitled, "Solano and Yolo County Design Rainfall," 1993.

Laugenour and Meikle; "Storm Drainage Study for Winters Highlands Subdivision Unit," 1994.

Morton and Pitalo, Inc.; "Drainage Study for Carter Ranch," July 2002.

Sacramento City/County; "Sacramento City/County Drainage Manual, Vol. 2: Hydrology Standards," 1996.

FIGURES



LEGEND

PROPOSED SUBBASIN BOUNDARIES

RANCHO ARROYO DRAINAGE BASIN BOUNDARY

WINTERS HIGHLANDS SUBDIVISION BOUNDARY

NOTE:

1. On-site shed boundaries for the Carter Ranch Subdivision can be found in Morton and Pitalo's report.
2. Subbasin ROF-4D which includes a portion of the Winters Highlands Subdivision included in the Moody Slough Subbasin. Interim drainage from this subbasin may be proposed to be part of the Rancho Arroyo Basin.
3. Runoff from the Carter Ranch, OFF-2D, and Silver Ridge Estates subbasins drains to a 48-inch pipe and does not drain to the detention pond.

SOURCES

1. Topographic mapping is United States Geologic Survey Quadrangle Maps, National Geodetic Vertical Datum of 1929.
2. "Drainage Study for Carter Ranch," Morton & Pitalo, Inc. July 2002.
3. "Storm Drainage Study for Winters Highlands Subdivision Unit 1," Langenour & Meikle, June 1994.

N

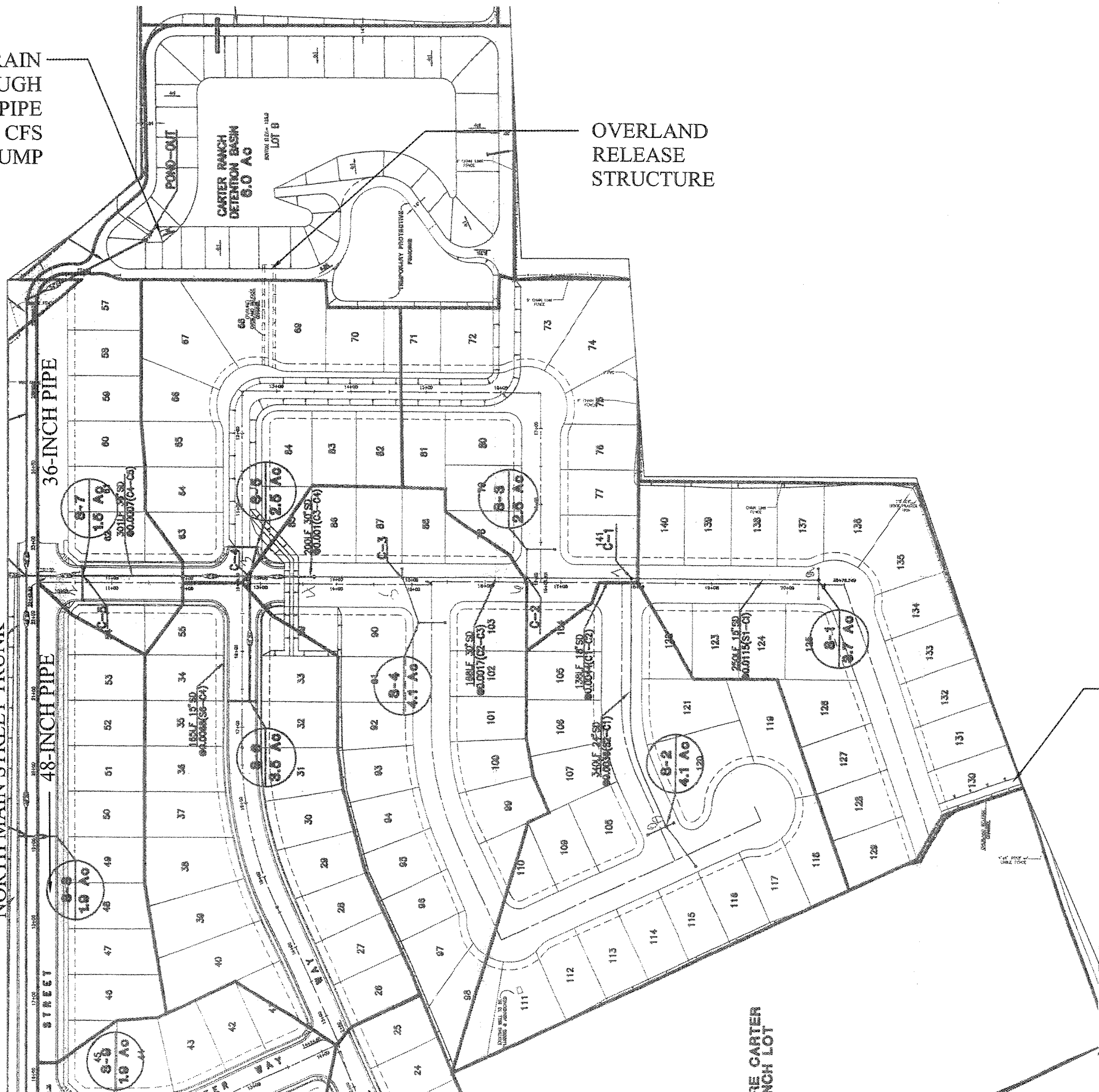
0 150 300
SCALE IN FEET

CITY OF WINTERS
RANCHO ARROYO SUBBASIN
STORM DRAINAGE EVALUATION

SUBBASIN BOUNDARIES

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 1



SOURCE

"Drainage Study for Carter Ranch," Morton & Pitalo, July 2002.

CITY OF WINTERS
RANCHO ARROYO SUBBASIN
STORM DRAINAGE EVALUATION

DETENTION POND CONFIGURATION EXISTING CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 2

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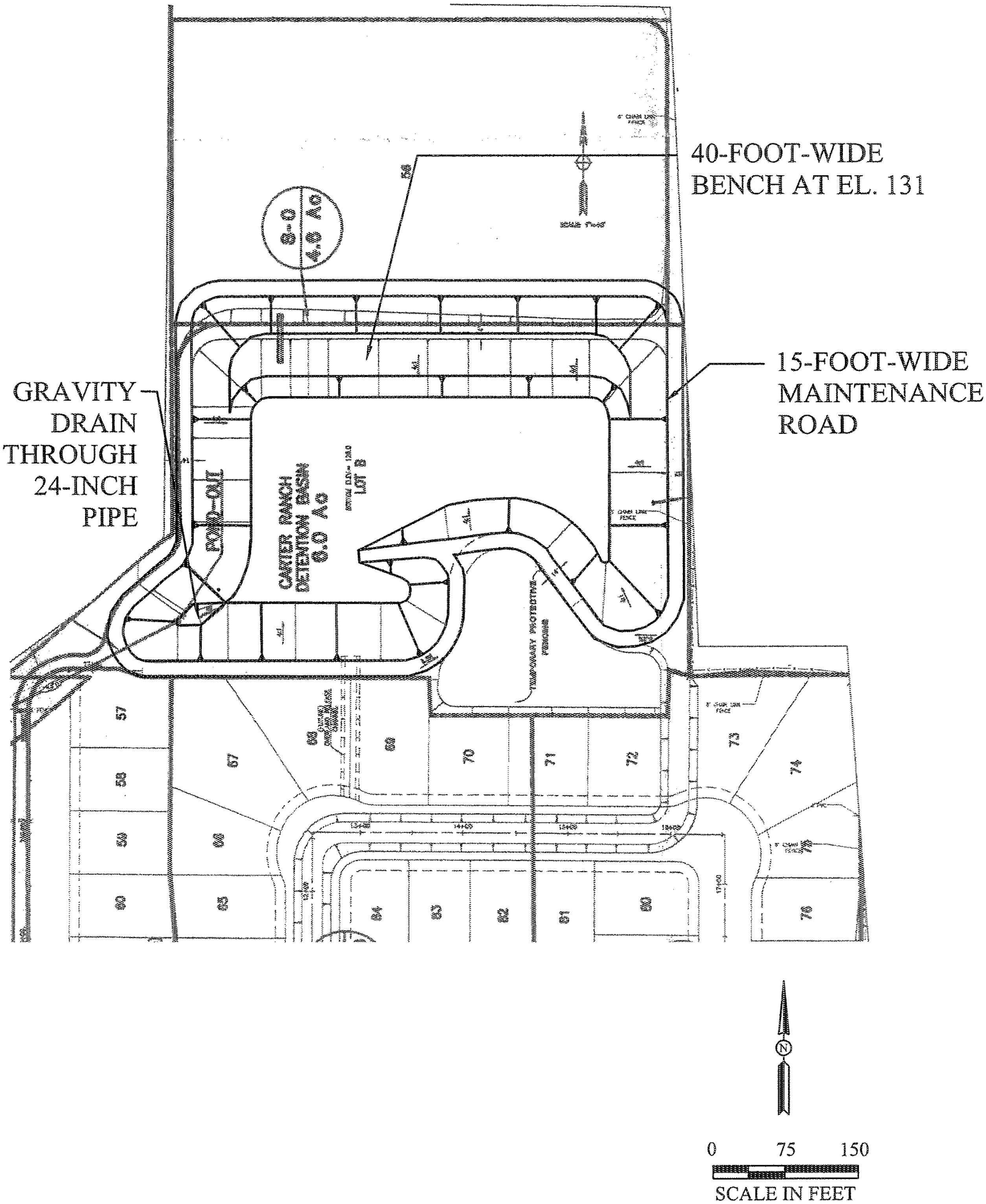

CITY OF WINTERS
RANCHO ARROYO SUBBASIN
STORM DRAINAGE EVALUATION

DETENTION POND CONFIGURATION
DRAINAGE SCENARIO 1

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

SOURCE

"Drainage Study for Carter Ranch," Morton & Pitalo, July 2002.

FIGURE 3

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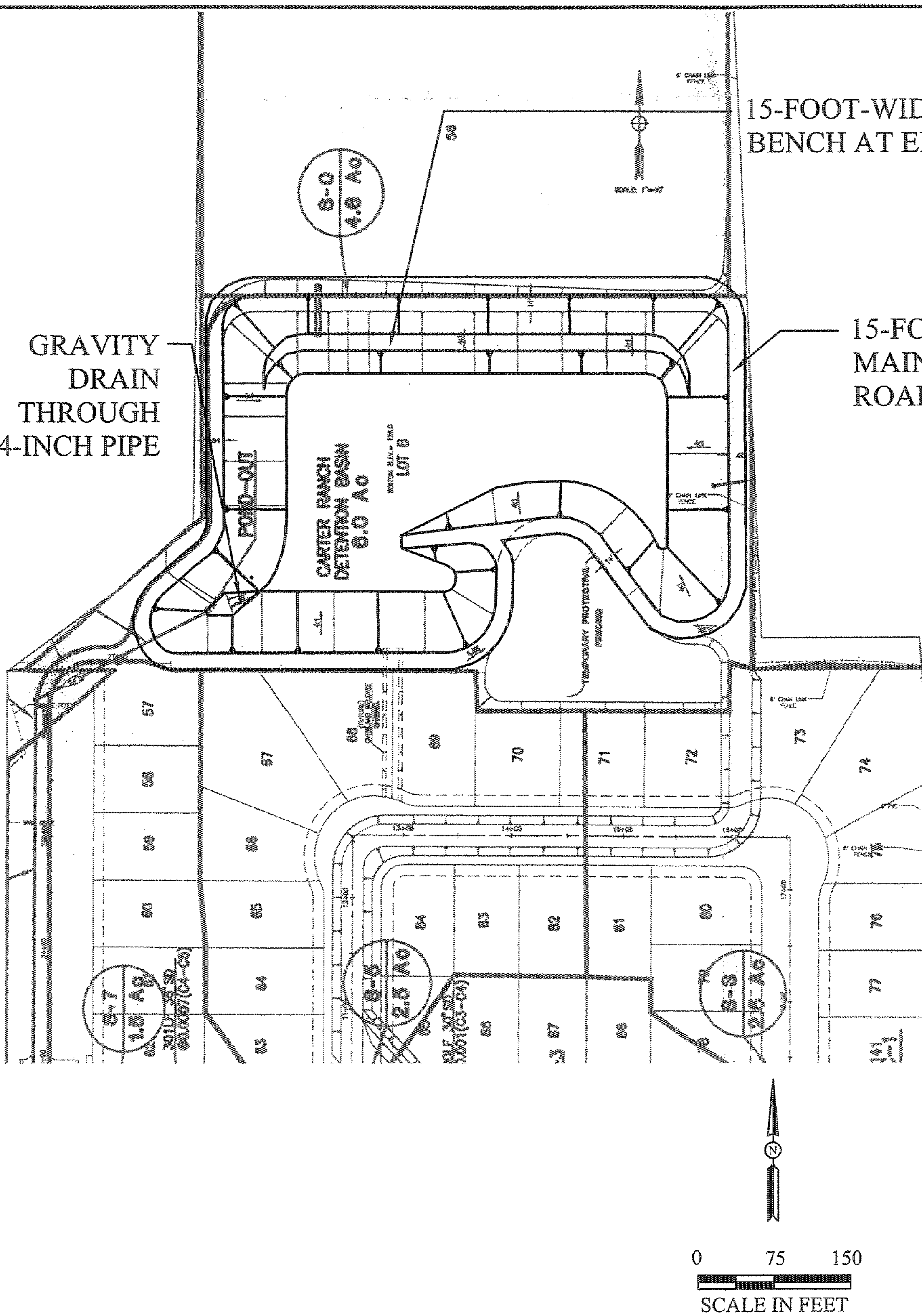


CITY OF WINTERS
RANCHO ARROYO SUBBASIN
STORM DRAINAGE EVALUATION

DETENTION POND CONFIGURATION
DRAINAGE SCENARIO 2

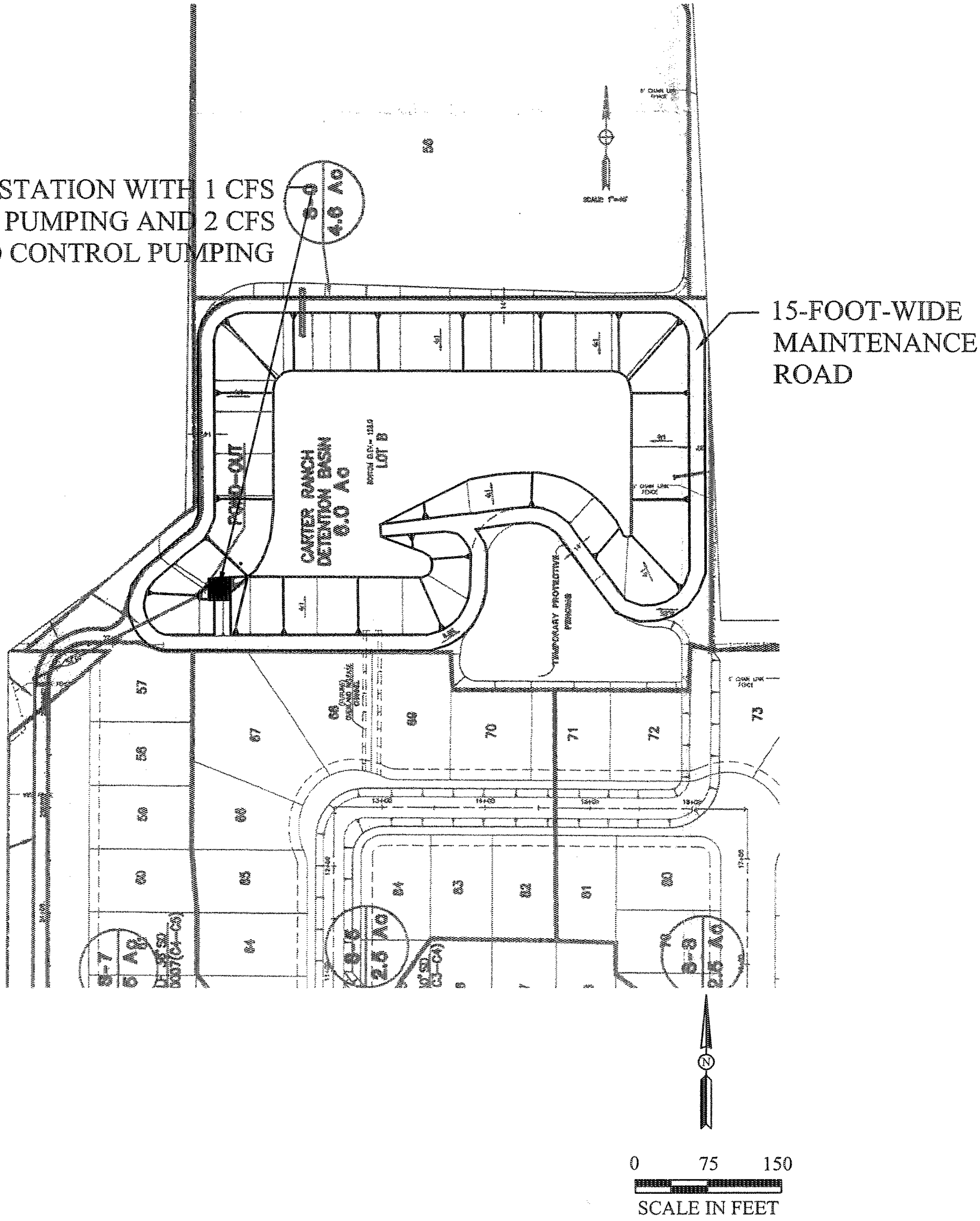
WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

SOURCE

"Drainage Study for Carter Ranch," Morton & Pitalo, July 2002.

I:\Projects2\220 Winters\Acad\FinalSub_PutahCreek_8-05\CarterRanch_StandAlone\F4_CarterRanch_Rev_3-03.dwg 9/08/05 8:18am lcurd

FIGURE 4



CITY OF WINTERS
RANCHO ARROYO SUBBASIN
STORM DRAINAGE EVALUATION

DETENTION POND CONFIGURATION
DRAINAGE SCENARIO 3

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

SOURCE

"Drainage Study for Carter Ranch," Morton & Pitalo, July 2002.

FIGURE 5

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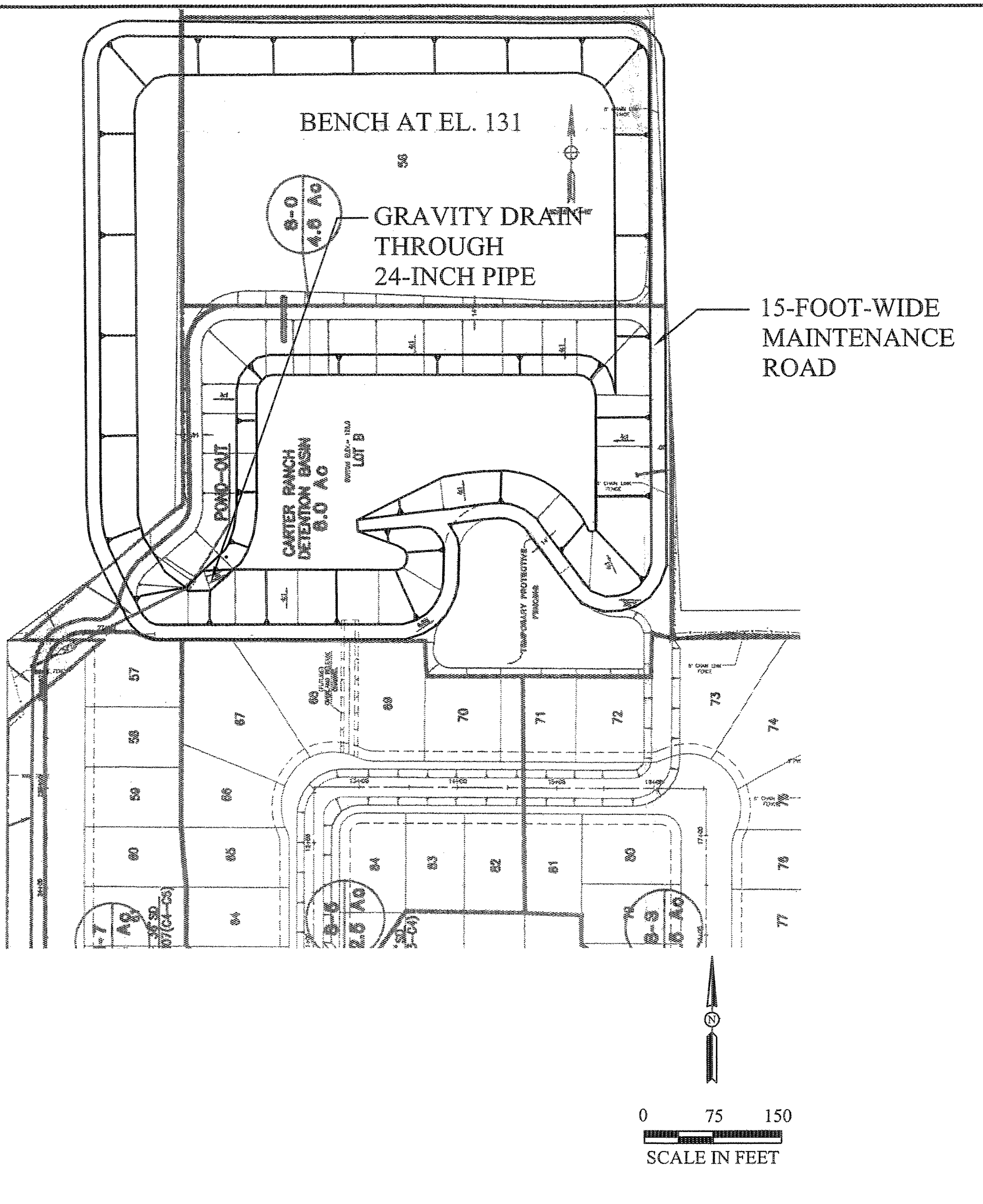

SOURCE

"Drainage Study for Carter Ranch," Morton & Pitalo, July 2002.

CITY OF WINTERS
RANCHO ARROYO SUBBASIN
STORM DRAINAGE EVALUATION

DETENTION POND CONFIGURATION
DRAINAGE SCENARIO 4

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 6

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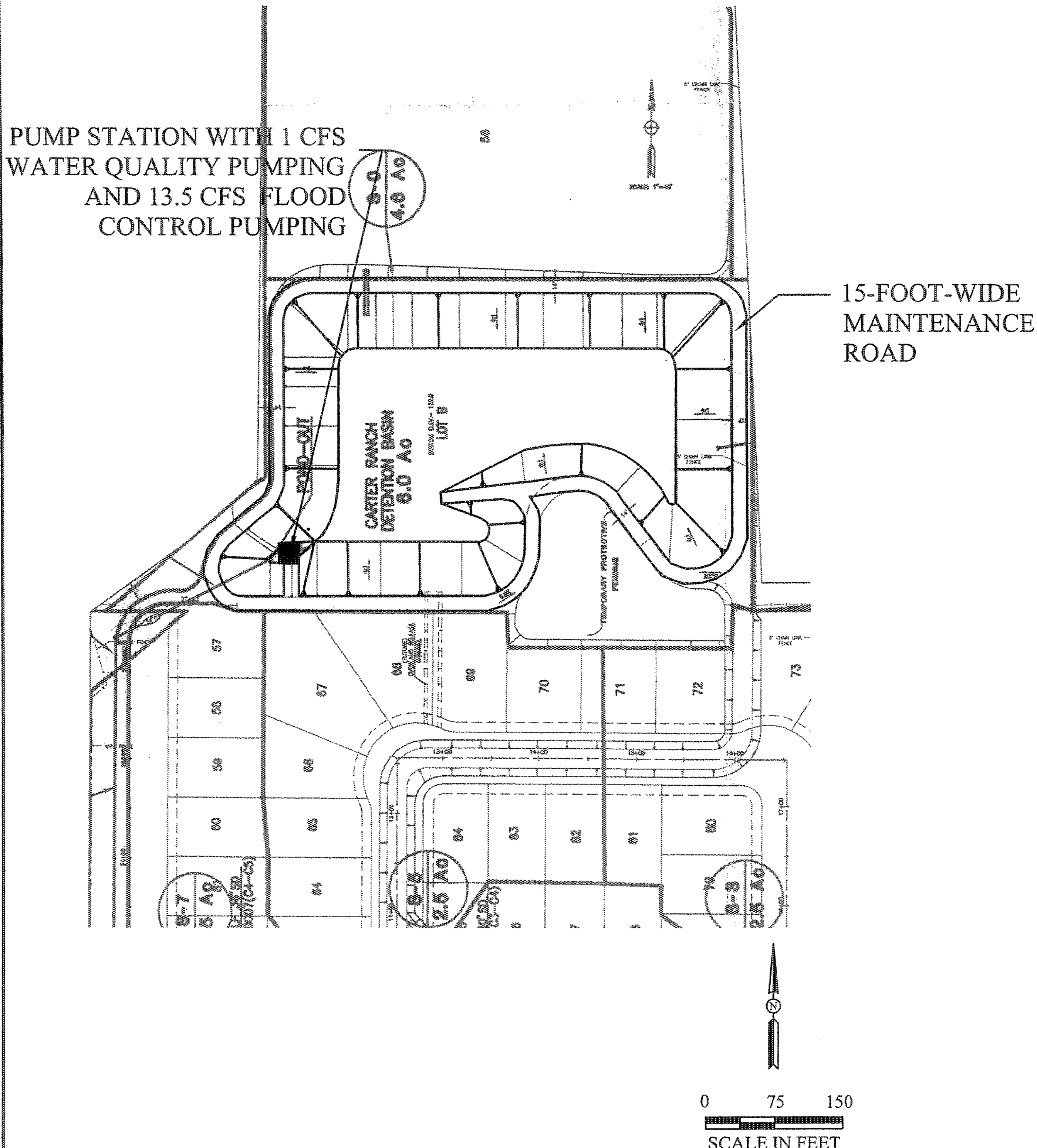

CITY OF WINTERS
RANCHO ARROYO SUBBASIN
STORM DRAINAGE EVALUATION

DETENTION POND CONFIGURATION
DRAINAGE SCENARIO 5

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

SOURCE

"Drainage Study for Carter Ranch," Morton & Pitalo, July 2002.

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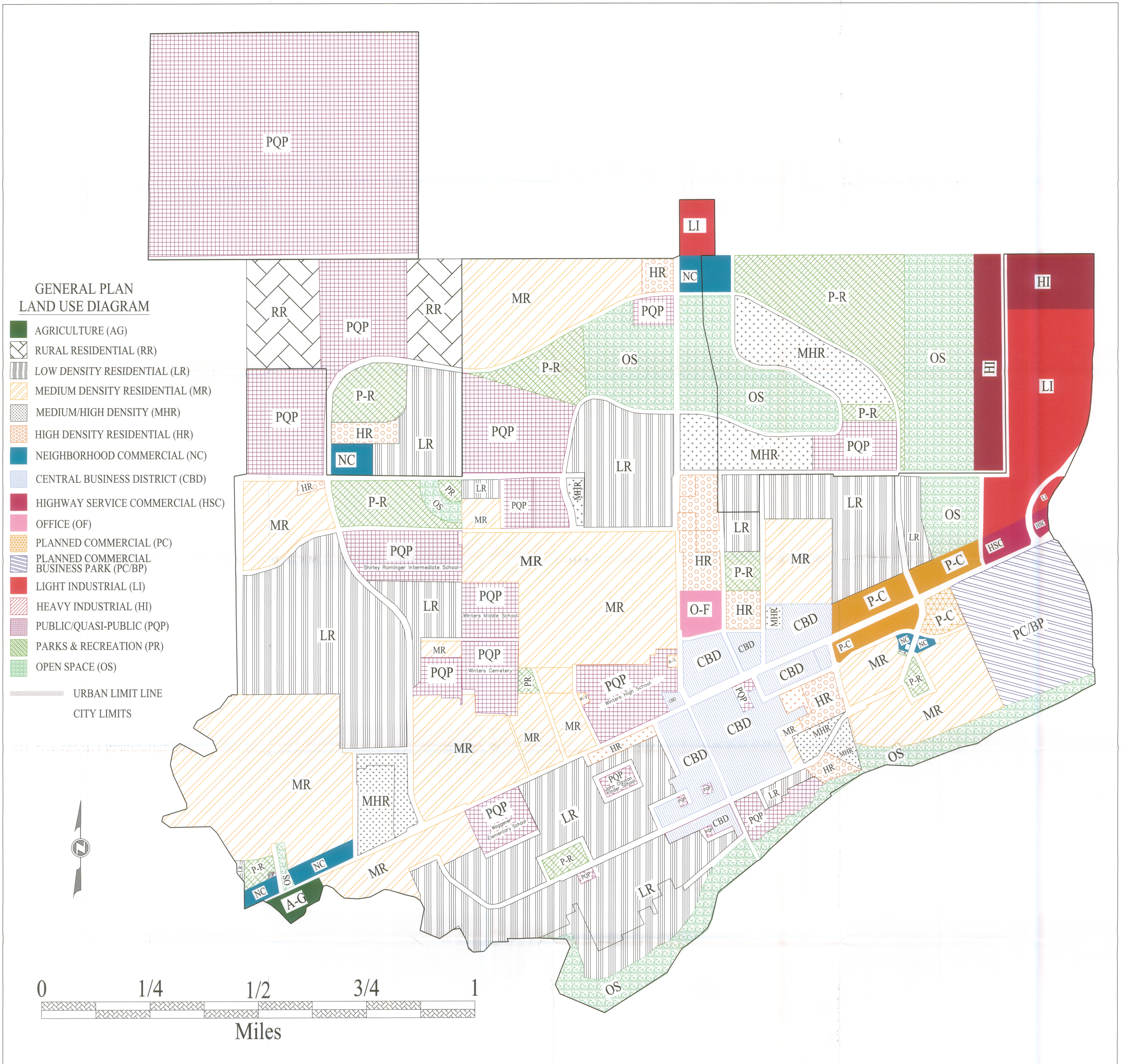
FIGURE 7



GENERAL PLAN AMENDMENT MAP
JUNE 2003
CITY OF WINTERS

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WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix E

Opinion of Probable Costs

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK / DRY CREEK SUBBASINS

OPINION OF PROBABLE COSTS[1]
ULTIMATE CONDITIONS

Description			Quantity	Unit	Unit Cost, $	Total Cost, $
1.	Rancho Arroyo Detention/Water Quality Pond Improvement Costs					
	a.	Pump Station (Includes Back-up Pumps)	5	cfs	20,150.00	90,675
	b.	48" Diameter RCP Trunk Pipe to Rancho Arroyo Detention/Water Quality Pond	1,515	lf	180.22	273,000
		Manholes				
		· 72" Diameter	3	ea	2,923.36	8,800
Subtotal Rancho Arroyo Detention/Water Quality Pond Improvements						372,475
2.	Putah Creek Detention/Water Quality Pond No. 1					
	a.	Land Acquistion				
		· Fee	1	ac	10,075.00	9,400
		· Acquisition Allowance	1	ls	25%	2,350
	b.	Pond Construction				
		· Excavate and Load Into Trucks	5,347	cy	1.78	9,500
		· Haul and Dump Excess Material	5,347	cy	1.15	6,200
		· Spread, Compact, and Shape Excess Material	5,347	cy	1.47	7,800
		· Construct Perimeter Road	343	tn	15.19	5,200
	c.	Outlet Control Weir Structure				
		· Excavate and Load Into Trucks	39	cy	1.78	100
		· Haul and Dump Excess Material	39	cy	1.15	0
		· Spread, Compact, and Shape Excess Material	39	cy	1.47	100
		· Riprap - Weir Construction	35	tn	41.91	1,500
		· Grout - Weir Construction	4	cy	366.73	1,400
Subtotal Northeast Detention/Water Quality Pond Improvements						43,550
3.	Putah Creek Detention/Water Quality Pond No. 2					
	a.	Land Acquistion				
		· Fee	2	ac	10,075.00	18,700
		· Acquisition Allowance	1	ls	25%	4,675
	b.	Pond Construction				
		· Excavate and Load Into Trucks	17,671	cy	1.78	31,500
		· Haul and Dump Excess Material	17,671	cy	1.15	20,400
		· Spread, Compact, and Shape Excess Material	17,671	cy	1.47	25,900
		· Construct Perimeter Road	516	tn	15.19	7,800
	c.	36" Diameter RCP Trunk Pipes	1,321	lf	121.54	160,600
		Manholes				
		· 60" Diameter	4	ea	2,923.36	11,700
	d.	Outlet Structure at Pond From 36" Trunk Pipes	2	ea	5,239.00	10,500
	e.	Outlet Control Weir Structure			0.00	
		· Excavate and Load Into Trucks	111	cy	1.78	200
		· Haul and Dump Excess Material	111	cy	1.15	100
		· Spread, Compact, and Shape Excess Material	111	cy	1.47	200
		· Riprap - Weir Construction	83	tn	41.91	3,500
		· Grout - Weir Construction	9	cy	366.73	3,300
	f.	48" Pipe Inlet Structure				
		· Excavate and Load Into Trucks	41	cy	1.78	100
		· Haul and Dump Excess Material	41	cy	1.15	0
		· Spread, Compact, and Shape Excess Material	41	cy	1.47	100
		· Reinforced Concrete Structure	1	ea	9,472.11	9,500
	g.	48" Diameter RCP Outlet Pipe to Putah Creek Diversion	426	lf	180.22	76,800
		Manholes				
		· 72" Diameter	1	ea	2,923.36	2,900
Subtotal Northwest Detention/Water Quality Pond Improvements						388,475
4.	Grant Street Interceptor					
	a.	Open Channel				
		· Land Acquisition Fee	1	ac	10,075.00	12,400
		· Acquisition Allowance	1	ls	25%	3,100
	b.	Channel Construction				
		· Excavate and Load Into Trucks	1,700	cy	1.78	3,000
		· Haul and Dump Excess Material	1,700	cy	1.15	2,000
		· Spread, Compact, and Shape Excess Material	1,700	cy	1.47	2,500
		· Construct Patrol/Access Roadways	766	tn	15.19	11,600
	c.	60" Diameter RCP	2,269	lf	249.38	565,800
		Manholes				
		· Saddle	8	ea	5,857.20	46,900

CITY OF WINTERS
DRAINAGE REPORT - PUTAH CREEK / DRY CREEK SUBBASINS

OPINION OF PROBABLE COSTS[1]
ULTIMATE CONDITIONS

Description	Quantity	Unit	Unit Cost, $	Total Cost, $
d. Construction of Inlet Structure at 60" Pipe	1	ea	8,749.13	8,700
e. Outlet Structure at Putah Creek Diversion	1	ea	5,239.00	5,200
Subtotal Grant Street Interceptor				661,200
5. Putah Creek Detention/Water Quality Pond No. 3				
a. Land Acquistion				
· Fee	3	ac	10,075.00	29,000
· Acquisition Allowance	1	ls	25%	7,250
b. Pond Construction				
· Excavate and Load Into Trucks	43,761	cy	1.78	77,900
· Haul and Dump Excess Material	43,761	cy	1.15	50,400
· Spread, Compact, and Shape Excess Material	43,761	cy	1.47	64,200
· Construct Perimeter Road	582	tn	15.19	8,800
c. Storm Drain Pipes				
36" Diameter RCP Trunk Pipes	795	lf	121.54	96,600
66" Diameter RCP	1,858	lf	288.15	535,400
Manholes				
· 60" Diameter	6	ea	2,923.36	17,500
· Saddle	3	ea	5,857.20	17,600
d. Outlet Structure at Pond From Trunk Pipes	2	ea	5,239.00	10,500
e. Outlet Control Weir Structure				
· Excavate and Load Into Trucks	1,054	cy	1.78	1,900
· Haul and Dump Excess Material	1,054	cy	1.15	1,200
· Spread, Compact, and Shape Excess Material	1,054	cy	1.47	1,500
· Riprap - Weir Construction	385	tn	41.91	16,100
· Grout - Weir Construction	42	cy	366.73	15,400
f. 66" Pipe Inlet Structure				
· Excavate and Load Into Trucks	39	cy	1.78	100
· Haul and Dump Excess Material	39	cy	1.15	0
· Spread, Compact, and Shape Excess Material	39	cy	1.47	100
· Reinforced Concrete Structure	1	ea	10,373.22	10,400
Subtotal Southwest Detention/Water Quality Pond Improvements				961,850
6. Putah Creek Detention/Water Quality Pond No. 4				
a. Land Acquistion				
· Fee	2	ac	10,075.00	18,700
· Acquisition Allowance	1	ls	25%	4,675
b. Pond Construction				
· Excavate and Load Into Trucks	21,147	cy	1.78	37,700
· Haul and Dump Excess Material	21,147	cy	1.15	24,400
· Spread, Compact, and Shape Excess Material	21,147	cy	1.47	31,000
· Construct Perimeter Road	421	tn	15.19	6,400
c. Outlet Control Weir Structure				
· Excavate and Load Into Trucks	205	cy	1.78	400
· Haul and Dump Excess Material	205	cy	1.15	200
· Spread, Compact, and Shape Excess Material	205	cy	1.47	300
· Riprap - Weir Construction	76	tn	41.91	3,200
· Grout - Weir Construction	8	cy	366.73	3,000
Subtotal Southeast Detention/Water Quality Pond Improvements				129,975
Subtotal Ultimate Facilities (Includes Land Acquisition Costs)				2,557,525
Land Acquisition Costs				109,500
Subtotal Ultimate Facilities (Does Not Include Land Acquisition Costs)				2,448,025
Construction Contingencies (25%)				612,006
Administration and Engineering (35%)				856,809
TOTAL ULTIMATE FACILITIES COST (Includes Land Acquisition Costs)[2]				4,026,340

[1]Unit costs are based upon 2004 price levels.

[2]Putah Creek diversion improvements, totaling $2,775,410, are shared by land in the Moody Slough subbasin. Refer to the report prepared by Wood Rodgers, Inc., entitled, "Moody Slough and Putah Creek / Dry Creek Subbasins Storm Drainage Cost Allocation Report," dated August 2005, for cost-sharing details.

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