Moody Slough Subbasin

Drainage Report

CITY OF WINTERS CALIFORNIA

August 2005

Prepared By:

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

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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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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 area are presented on Figure 1.

There are three major drainage subbasins within the City. These include the Moody Slough subbasin, Putah Creek subbasin, and Dry Creek subbasin.

The Moody Slough subbasin consists of approximately 5.8 square miles. Downstream of Interstate 505, Moody Slough is referred to as Dry Slough. A portion of the Moody Slough subbasin is within the City. The Chickahominy Slough subbasin is located north of the Moody Slough subbasin and, during periods of high runoff, spills into the Moody Slough subbasin upstream of Interstate 505. The Putah Creek and Dry Creek subbasins are located south and west of the Moody Slough subbasin.

The City's General Plan proposes development within the existing floodplain and across natural runoff corridors. 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 and Putah Creek / Dry Creek subbasins. This document pertains to the Moody Slough subbasin only. A separate document was prepared to address development in 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 adverse 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 Moody Slough subbasin.

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 Moody Slough subbasin 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 Moody Slough subbasin.

D. BACKGROUND

Several drainage studies were previously developed for Moody Slough 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.

- 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.

Implementation Program:

IV.4. The City shall pursue the acquisition of surface water rights.

Responsibility:

City Council
City Manager
City Engineer
Public Works Department

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. The City, as the local administrator for the NFIP, 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 that 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 require 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.

YCFC&WCD'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 YCFC&WCD's facilities, YCFC&WCD 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. YCFC&WCD'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 Study Contractors," January 1995. 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 within the Moody Slough subbasin 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 only new development to provide sufficient evidence and prepare any and all CLOMRs and LOMRs at the discretion of the City.

USACOE

As previously discussed, the USACOE has studied existing flooding associated with the Moody Slough and completed the environmental analysis for the preferred project. Since federal funding is not anticipated at this time, the USACOE's involvement with the proposed improvements would be associated with environmental permitting. Compliance with NPDES regulations would be administered through the Regional Water Quality Control Board, as noted previously.

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 404 Permit - A 404 Permit is required to direct storm water discharge into 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 Moody Slough and 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 to incorporate 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.

Evaluating and developing storm drainage facilities on a drainage basin basis would ensure existing and proposed drainage facilities meet the immediate and long-term goals of the community. The analysis 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 facilities 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 a 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, including runoff corridors,[1] 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, YCFC&WCD'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.

[1]Runoff Corridor is a term adopted for the natural waterways within the Moody Slough subbasin that originates upstream of the City's General Plan area. Special attention is given to handling runoff originating outside the City's General Plan area and conveying the runoff safely through the City's General Plan area.

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 A.

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 would be provided for the 100-year storm event. 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 Modeling

The HEC-1 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 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 A, 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:

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

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 B).

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.

<u>Travel Time Component Method</u> – 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.

<u>Overland Flow</u> – 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.

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

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:

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

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 is 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
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 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 B).

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 B).

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.

<u>Overland Flow</u> – 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 3V:1H.
- 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 in a report entitled, "Solano and Yolo County Design Rainfall," which was included in the "Covell Drainage System Comprehensive Drainage Plan," 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.

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.

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 a 100-year frequency. Additionally, downstream components within a drainage system cannot revert back to a Type 2 facility once a Type 1 designation is reached (i.e., pipes draining detention ponds).

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.
- Impacting drainage and irrigation operations for surrounding lands. (This would require close coordination with the YCFC&WCD and landowners.)

DESCRIPTION OF EXISTING CONDITIONS

A. TOPOGRAPHY AND SUBBASIN BOUNDARIES

Within the Moody Slough subbasin, which consists of approximately 5.8 square miles, the terrain generally slopes from the eastern side of the coastal foothills in the west to the valley in the east. The approximate ground elevations range from a maximum El. 313, in the coastal foothills to El. 124 where the Moody Slough crosses Interstate 505, National Geodetic Vertical Datum of 1929 (NGVD 29). Just upstream of Road 89 (Railroad Road), the approximate ground elevation is El. 128 (NGVD 29).

The Chickahominy Slough subbasin is located north of the Moody Slough subbasin, and the Putah Creek and Dry Creek subbasins are located south and west of Moody Slough.

Presented on Figure 2 is the existing Moody Slough subbasin boundary and topographic mapping for the Moody Slough subbasin and adjacent areas. 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 scanned topography from a 1974 aerial survey performed by American Aerial Surveys for the SCS report cited in Section I.D.

Additional topographic mapping for portions of the Moody Slough subbasin is presented in the Nolte study and the USACOE study.

B. LAND USE

The existing land use within the Moody Slough subbasin primarily consists of agricultural, rural residential, and open space. A small amount of urban residential development exists within the portion of the Moody Slough subbasin that lies within the existing City limits.

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 Moody Slough subbasin 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 Moody Slough subbasin just upstream of Interstate 505 to rarely be less than approximately 15 feet below existing ground level.

E. EXISTING DRAINAGE AND IRRIGATION FACILITIES

Moody Slough, presented on Figure 2, consists of a natural drainage channel located north of the City, which runs from west to east and crosses County Road 89 (Railroad Road). Just upstream of Interstate 505, Moody Slough has been realigned along property lines. The existing crossing at Moody Slough and Interstate 505 consists of two 5'x10' box culverts. East of Interstate 505, Moody Slough is named Dry Slough.

In addition to Moody Slough, drainage within the area is conveyed within typical roadside ditches and field drains.

Chapman Reservoir is located to the north of Moody Slough and west of County Road 89. YCFC&WCD's Winters Canal flows into Chapman Reservoir. The canal downstream of Chapman Reservoir is the Willow Canal. 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 principal existing drainage, flood control, and irrigation facilities are presented on Figure 2.

F. DOWNSTREAM WATER RIGHTS

Presented on Figure 3 are locations and numbers for water rights applications, permits, licenses, and decisions identified along waterways downstream of Moody Slough. Summarized below is a brief description of certain water rights filings.

Application No.	Status	Use	Amount/Period
12398	Decision 665	Irrigation	6.17 cfs/Apr 1-Oct 15
12637	Decision 665	Irrigation	7.42 cfs/Apr 1-Oct 15
030198	License	Domestic	4.02 cfs/Nov 1-Jun 10
030467	License	Fish and Wildlife Protection	10 af/Nov 1-Mar 31
19221	Decision 998	Denying Application	

As noted above, licensed water rights users exist on Dry Slough east of Interstate 505. In addition, riparian water rights probably are utilized along the waterways as well. These water rights and considerations of water for wildlife, habitat, and groundwater recharge would need to be respected.

G. HYDROLOGIC MODELING

Wood Rodgers prepared hydrologic computer models to represent drainage and flooding conditions for Moody Slough for storm events of various recurrence intervals and durations. The USACOE's HEC-1 (Version 4.1e) computer program was used in accordance with the criteria and standards previously identified in this report. In addition to the 100-year and 10-year storm events, the 2-year, 24-hour storm event was analyzed to assist with evaluating low flow conditions to address existing downstream water rights issues.

For the Moody Slough subbasin, including a portion of the Chickahominy Slough subbasin, HEC-1 models of the following storm events were developed for the existing conditions:

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

Due to the undersized culverts at road crossings, the 10-day storm event results in the worst-case flooding scenarios for a 100-year recurrence interval for the existing conditions for Moody Slough.

The Chickahominy Slough subbasin is located north of the Moody Slough subbasin. In large storm events, the capacity of Chickahominy Slough is exceeded, and flows spill from Chickahominy Slough south along Interstate 505 into the Moody Slough subbasin. In the 100-year, 10-day and 10-year, 24-hour storm events, approximately 1,869 cfs and 80 cfs spill from Chickahominy Slough into Moody Slough, respectively. In the 2-year, 24-hour storm event, 23 cfs spills from Chickahominy Slough into Moody Slough.

During large storm events, the capacity of the existing culverts at Road 89 (Railroad Road) and Moody Slough is exceeded, and ponding occurs upstream. Similarly, during large storm events, the capacity of the existing Moody Slough channel and the existing box culverts at Interstate 505 are exceeded causing ponding upstream of the crossing that results in flow spilling south and eventually reaching Putah Creek. In the 100-year, 10-day event approximately 8 cfs spills from Moody Slough into Putah Creek.

H. FLOODING

Within the Moody Slough subbasin, the FEMA FIRM Community Panel Numbers listed under Section II.D. of this report shows the effective flood insurance zone designations.

Presented on Figure 2, are FEMA's 100-year floodplain approximate delineation and the revised 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. Zone A is shown just upstream of Interstate 505.

The 100-year peak stages from Wood Rodgers' hydrologic models in the Moody Slough basin are higher than stages delineated on the FEMA floodplain maps. Since FEMA's study, there have been several improvement projects by Caltrans along Interstate 505, which have raised the profile of the highway (AC pavement overlays, etc.). Additionally, as detailed topographic mapping becomes available for portions of Moody Slough, a greater understanding of overland flow patterns has developed. Accordingly, water surfaces in the flooded areas adjacent to Interstate 505 have increased as the spill over the highway remains the only drainage outlet once the capacities of the culverts are exceeded. In large storm events, flows from the Moody Slough drainage overtop County Road 89 and Interstate 505 in the existing conditions.

IDENTIFICATION OF PROJECT-SPECIFIC CONSTRAINTS

A. CONSTRAINTS

Development within the existing floodplain requires significantly more mitigation than development outside the existing floodplain. For any development, the increased peak rate, volume of surface runoff, and changes in timing of runoff would need to be mitigated. However, when a 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 USACOE report, 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.

Although increasing storm drainage discharges downstream in Dry Slough during high flow storm events may be problematic, significantly decreasing discharges downstream in Dry Slough during low flow conditions may also be problematic. As previously discussed, downstream water rights are in place; therefore, low flow discharges must be maintained.

DESCRIPTION OF PROPOSED LAND USE AND DEVELOPMENT PHASING

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 that the integrity of the drainage facilities proposed for the Moody Slough subbasin 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 the General Plan. The revised land uses are included in City of Winters' General Plan Amendment, 2003 (Appendix C) and are assumed to not affect the sizing of the facilities for Moody Slough.

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 diversion channel, in addition to the 1,000 cfs identified in the USACOE study. The feasibility of phasing the improvements was also considered. To the extent possible interim facilities were minimized.

A. HYDROLOGIC AND HYDRAULIC MODELING

Hydrologic models using HEC-1 were prepared for the Phased Conditions With Existing Land Uses Elsewhere and 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
- 10-year, 24-hour storm event
- 2-year, 24-hour storm event

Presented on Figure 5 are the subbasins used in the hydrologic analysis of the Moody Slough subbasin under Phased Conditions, and presented on Figure 6 are the subbasins for the Ultimate Developed Conditions. Descriptions of the drainage plans are 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 7.

Drainage Facilities

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

Putah Creek Diversion

- A trapezoidal channel with a 15-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.
- 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.

Detention/Water Quality Pond No. 1

- An excavated pond with detention and wet water quality pond features would be constructed. A typical section for a detention/water quality pond can be found on Figure 8.
- An inlet structure consisting of five 10'x5' box culverts would be constructed to allow flood waters to spill into Pond No. 1 from north of the Winters urban limit to replace overland conveyance and floodplain storage displaced by development. In addition to increased runoff from development, this structure would also allow low flow to drain to the north through Winters North Drain.
- An outlet control structure (30-foot weir crest width) would be constructed to control flood diversion to the Putah Creek diversion facilities during large storm events through an Obermeyer-style variable gate structure, which would also control detention and diversion of low flow volumes of runoff to the Winters North Drain. The gate would be activated to release

flow into the Putah Creek diversion when the pond stage reaches El. 119.5. A downstream flow monitor would be installed to ensure the capacity of the diversion is not exceeded and could be linked to the gate controls for a fully automated system. During the summer the gate could be taken off line by providing brace provisions, which would allow low flow service to Moody Slough to continue.

Detention/Water Quality Pond No. 2

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

- An outlet control weir structure (9-foot weir crest width) would be constructed to control flood control diversion to Pond No. 1 facilities above El. 121 through an energy dissipation structure and five 6'x10' box culverts.

Detention/Water Quality Pond No. 3

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

- Two outlet box culverts (8'x10') would be constructed to allow flood waters to carry through to Pond No. 2, minimizing head losses and allowing the flood control volumes of Pond No. 2 and Pond No. 3 to act as one pond, thereby maximizing storage in Pond No. 2.

- Pond No. 3 is proposed to have a sedimentation basin at the upper end of the pond to provide pretreatment of flows before reaching the main water quality portion of the pond. This sedimentation basin would be obsolete once upstream water quality facilities are constructed.

Water Quality Pond No. 4

- A regional water quality pond would be constructed. The runoff corridors presented on Figure 7 shows the anticipated runoff contributing to volume in Pond No. 4. The overland flows (flood flows) from the same upstream areas must be directed via roadways or other means through the Pond No. 4 location and into Pond No. 3 for attenuation.

- Two 5'x10' box culverts would be constructed to allow flood flows to pass through the pond under the proposed road (Main Street) and downstream.

Water Quality Pond No. 5

- A regional water quality pond would be constructed to treat low flows from ultimate developed areas (within urban limits) to the east of the Putah Creek diversion and to the north of the existing Willow Canal.

- A 54-inch-diameter outlet siphon pipeline would be constructed to convey storm water flows, from contributing areas under the proposed Willow Canal levee and existing Willow Canal, to discharge into the Putah Creek diversion facilities.

Runoff Corridor No. 1

- Maintain conveyance capacity consistent with criteria established for the ultimate conditions peak flow resulting from a 100-year, 24-hour storm for draining Moody Slough subbasin areas outside the urban limit boundary that drain through the plan area. Account for increased capacity downstream, as drainage areas within the urban limit contribute to peak flow. The 100-year, 24-hour peak flow for this corridor (determined from the HEC-1 modeling) is 34 cfs at the urban limit boundary. The 100-year, 24-hour peak flow for this corridor is 58 cfs where Runoff Corridor No. 1 drains into Runoff Corridor No. 2. A typical section for the Runoff Corridor is presented on Figure 9.

Runoff Corridor No. 2

- Maintain conveyance capacity consistent with the criteria established for the ultimate conditions peak flow resulting from a 100-year, 24-hour storm for draining Moody Slough subbasin areas outside the urban limit boundary that drain through the plan area. Account for increased capacity downstream, as drainage areas within the urban limit contribute to peak flow. The 100-year, 24-hour peak flow for this corridor (determined from the HEC-1 modeling) is 216 cfs at the urban limit boundary. The 100-year, 24-hour peak flow for this corridor is 250 cfs where Runoff Corridor No. 2 drains into Runoff Corridor No. 3.

Runoff Corridor No. 3

- Maintain conveyance capacity consistent with criteria established for the ultimate conditions peak flow resulting from a 100-year, 24-hour storm for draining Moody Slough subbasin areas outside the urban limit boundary that drain through the plan area. Account for increased capacity downstream, as drainage areas within the urban limit contribute to peak flow. The 100-year, 24-hour peak flow for this corridor (determined from the HEC-1 modeling) is 93 cfs at the urban limit boundary. The 100-year, 24-hour peak flow for this corridor is 262 cfs upstream of the confluence of Runoff Corridor No. 3 and Runoff Corridor No. 2. The 100-year, 24-hour peak flow for this corridor is 468 cfs downstream of this confluence until this flow reaches Water Quality Pond No. 4.

Winters North Drain

- A 1,000 cfs open channel would be constructed along the north boundary of the urban limit to direct floodwater originating from the north toward the east to the existing culverts under Interstate 505 currently known as Moody Slough. Presented on Figure 9 is a typical cross section for the proposed channel.

- A levee would be constructed along the northern urban limit boundary on the south side of the Winters North Drain to isolate development areas

within the urban limit boundary. The levee would have a minimum top width of 15 feet, maximum side slopes of 3:1, and a minimum top at El. 130.46 (NGVD 29). A minimum of 15 feet would be provided along each side of the levee outside of the toe of the slope for maintenance and inspection. In addition, a new road is required parallel to the North Drain consistent with the City's Ultimate Land Use Plan (Figure 4). Additionally, a provision for a removable flood barrier would be installed where the proposed levee alignment crosses the existing frontage road. This structure would ensure that service along the frontage road is not interrupted, except during flood events, when the barrier could be installed to protect the City from inundation. A floodwall would also be installed between the frontage road and Interstate 505 to ensure that overtopping of the highway north of the City limits does not back flow into the City. The proposed floodwall alignment is presented on Figure 7.

- Four 8'x6' concrete box culverts would be constructed to allow drainage along the Winters North Drain to pass under County Road 89. Grading within Moody Slough to the west of County Road 89, as proposed, would be raised to prevent storm water runoff within the Winters North Drain from spilling southward into the urban limit.

Relocated Willow Canal

The canal would have a capacity of 65 cfs. YCFC&WCD (which owns, operates, and maintains the Willow Canal) prefers to have the canal relocated as a buried pipeline in the vicinity of urban areas. If the canal is constructed, the entire reach of the relocation would require fencing on both sides with gates at the north and south ends. A cursory review of costs shows the canal to be less expensive although it requires more land. Based upon costs alone, the pipeline could be constructed with the roadway, thereby reducing the land needed for public infrastructure.

- A 54-inch pipeline would be constructed between the Winters North Drain and Pond No. 1 along the north urban limit, turning south along the eastern perimeter of Pond No. 1. The pipeline would transition to a canal section, which would run south, parallel to the perimeter of Pond No. 1

and join the existing Willow Canal alignment near the southeastern corner of Pond No. 1. The relocated Willow Canal would redirect irrigation flow currently conveyed along the existing Willow Canal alignment. From the southeast corner of Pond No. 1 to Interstate 505, the relocated Willow Canal would be a 54-inch-diameter reinforced concrete pipe constructed within County Road 33. The canal would be fenced on both sides and have gates at the north and south ends of the relocation.

- Levees would be constructed along the eastern and western sides of the relocated Willow Canal along the north/south reach, as well as along the north side of the existing east/west Willow Canal west of Interstate 505. In addition, a new road is required parallel to the relocated Willow Canal and levee consistent with the City's Ultimate Land Use Plan (Figure 4).

- A flow control structure would be constructed to allow flows in the Willow Canal to pass under the Winters North Drain near County Road 89. This structure would allow shutting off flows to the downstream portion of the Willow Canal during nonirrigating periods, with a small spill structure into the Winters North Drain for controlling any flows that are inadvertently drained into the Willow Canal north of the Winters urban limit. These flows are considered part of the design channel capacity of the Winters North Drain.

  A 54-inch-diameter reinforced concrete pipe siphon would be constructed to convey flow in the Willow Canal beneath the proposed inlet box culvert structure at the north end of Pond No. 1.

- A 54-inch-diameter reinforced concrete pipe siphon would be constructed to convey flows in the Willow Canal beneath a proposed roadway at the northeast corner of Phase 1 development.

Other Features

- Conveyance of overland flow from land south of Phase 1 through Phase 2 is planned to flow along County Road 33 and discharge directly into the Putah Creek diversion (Figure 5).

For the Ultimate Developed Conditions, the 100-year floodplain in Moody Slough upstream of Interstate 505 is El. 127.42 (NGVD 29). The proposed 100-year floodplain is presented on Figure 7.

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, for loss of floodplain storage, and overland conveyance. The proposed detention ponds, in conjunction with additional source and treatment control measures, would provide surface water quality treatment. Since the Willow Canal would be realigned, significant irrigation impacts would not occur. No significant impacts to downstream low flows in Moody Slough and Dry Slough would occur.

There is sufficient water quality treatment volume within the proposed Moody Slough ponds to accommodate treatment for areas developed upstream under the City's General Plan. The placement of a sedimentation basin (Water Quality Pond No. 4) upstream of the Phase 2 development is proposed as part of the proposed drainage facilities to ensure preservation of the existing wetlands along the western border of the Phase 2 development. The existing increased sediment loads resulting from ultimate development upstream could otherwise fill the existing wetlands. Existing sediment loads are assumed to be low enough to preserve the existing wetland.

FORMULATION OF DRAINAGE FACILITY PHASING

At the time each development phase occurs, drainage facilities are required to protect the proposed development and to mitigate any adverse impacts. Drainage plans were formulated for each proposed development phase and are described below. To the extent practical, interim facilities or "throwaway" costs were minimized.

A. HYDROLOGIC MODELING

Hydrologic models using HEC-1 were prepared for each proposed phase. 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
- 10-year, 24-hour storm event
- 2-year, 24-hour storm event

B. PHASE 1 CONDITIONS

A significant amount of drainage, flood control, irrigation, and water quality treatment improvements would be required with the first phase of development. The Phase 1 Conditions Drainage Plan is described below. To the extent practical, interim facilities were minimized. The drainage facilities are presented on Figure 10.

<u>Drainage Facilities</u>

The drainage facilities for Phase 1 Conditions include the following elements:

<u>Putah Creek Diversion</u>

- A trapezoidal channel with a 15-foot bottom width, 3:1 side slopes, and access/maintenance roads would be constructed to convey floodwater from Moody Slough to Putah Creek.

- 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.

Detention/Water Quality Pond No. 1

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

- An inlet structure consisting of five 10'x5' box culverts would be constructed to allow floodwater to spill into Pond No. 1 from north of the Winters urban limit to replace overland conveyance and floodplain storage displaced by development, in addition to increased runoff from development.

- An outlet control weir structure (30-foot weir crest width) would be constructed as a flood control diversion to the Putah Creek diversion facilities during large storm events through an Obermeyer-type variable gate structure, which would also control detention and diversion of low flow volumes of runoff to the Winters North Drain. The gate would be activated to release flow into he Putah Creek diversion when the pond stage reaches El. 119.5. A downstream flow monitor would be installed to ensure that the capacity of the diversion is not exceeded and can be linked to the gate controls for a fully automated system. During the summer the gate could be taken off line by providing brace provisions, which would allow low flow service to Moody Slough to continue.

Detention/Water Quality Pond No. 2

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

- An outlet control weir structure (9-foot weir crest width) would be constructed as a flood control diversion to Pond No. 1 facilities above

El. 121 through an energy dissipation structure and five 6'x10' box culverts.

Winters North Drain

- A 1,000 cfs open channel would be constructed along the north boundary of the urban limit to direct floodwater originating from the north toward the east to Moody Slough, thence to the existing culverts under Interstate 505. Presented on Figure 9 is a typical cross section for the proposed channel.

- A levee would be constructed along the northern urban limit boundary on the south side of the Winters North Drain to isolate development areas within the urban limit boundary. The levee would have a minimum top width of 15 feet, maximum side slopes of 3:1, and a minimum top at El. 130.46 (NGVD 29). A minimum of 15 feet would be provided along each side of the levee outside of the toe of the slope for maintenance and inspection.

- Four 8'x6' concrete box culverts would be constructed to allow drainage along the Winters North Drain to pass under County Road 89. Grading within Moody Slough to the west of County Road 89 is proposed to be raised to prevent storm water runoff within the Winters North Drain from spilling southward into the urban limit.

Relocated Willow Canal

The canal would have a capacity of 65 cfs. YCFC&WCD (which owns, operates, and maintains the Willow Canal) prefers to have the canal relocated as a buried pipeline in the vicinity of urban areas. If the canal is constructed, the entire reach of the relocation would require fencing on both sides with gates at the north and south ends. A cursory review of costs shows the canal to be less expensive although it requires more land. Based upon costs alone, the pipeline could be constructed with the roadway, thereby reducing the land needed for public infrastructure.

- A 54-inch pipeline would be constructed between the Winters North Drain and Pond No. 1 along the north urban limit. It would turn south along the eastern perimeter of Pond No. 1 and transition to a canal section, which would run south parallel to the perimeter of Pond No. 1 and join the existing Willow Canal alignment near the southeastern corner of Pond No. 1. The relocated Willow Canal would redirect irrigation flow currently conveyed along the existing Willow Canal alignment.

- Levees would be constructed along the eastern and western sides of the relocated Willow Canal along the north/south reach, as well as along the north side of the existing east/west Willow Canal west of Interstate 505. In lieu of constructing the levee noted herein, the levee along the North Drain from the northeast corner of Phase 1 to County Road 90 could be constructed. Ultimately, the levee along the east boundary Phase 1 would be a "throw away" cost in that under ultimate conditions, it would not be required.

- A flow control structure would be constructed to allow flows in the Willow Canal to pass under the Winters North Drain near County Road 89. This structure would allow shutting off flows to the downstream portion of the Willow Canal during flood season, with a small spill structure into the Winters North Drain for controlling any flows that are inadvertently drained into the Willow Canal north of the Winters urban limit. These flows are considered part of the design channel capacity of the Winters North Drain.

- A 54-inch-diameter reinforced concrete pipe siphon would be constructed to convey flow in the Willow Canal beneath the proposed inlet box culvert structure at the north end of Pond No. 1.

- A 54-inch-diameter reinforced concrete pipe siphon would be constructed to convey flows in the Willow Canal beneath a proposed roadway at the northeast corner of Phase 1.

For the Phase 1 Conditions, the 100-year floodplain in Moody Slough upstream of Interstate 505 is El. 127.17 (NGVD29). The residual 100-year floodplain is presented on

Figure 10. As noted on Figure 10, the extent of the residual floodplain south of Moody Slough cannot be determined until detailed topographic mapping is available. County Road 33, which is proposed to function as a runoff corridor also, should have capacity to convey runoff in excess of the pipe system and thus eliminate any residual floodplain. The design of County Road 33 and grading the area would be critical for this item.

<u>Drainage, Flooding, Surface Water Quality, and Irrigation Impacts</u>

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 Phase 1 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 and overland conveyance. The proposed detention ponds, in conjunction with additional source and treatment control measures, would provide surface water quality treatment. Since the Willow Canal would be realigned, significant irrigation impacts would not occur. No significant impacts to downstream low flows in Moody Slough and Dry Slough would occur.

C. PHASE 2 CONDITIONS

The Phase 2 Conditions are described below. To the extent practical, interim facilities were minimized. The drainage facilities to be provided under Phase 2 are presented on Figure 11. These facilities are in addition to the improvements presented for Phase 1.

<u>Drainage Facilities</u>

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

<u>Detention/Water Quality Pond No. 3</u>

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

- Two outlet box culverts (8'x10') would be constructed to allow floodwater to carry through to Pond No. 2, minimizing head losses and allowing the flood control volumes of Pond No. 2 and Pond No. 3 to act as one pond, thereby maximizing storage in Pond No. 2.

- Pond No. 3 would have a constructed sedimentation basin at the upper end of the pond to provide pretreatment of flows before reaching the main water quality portion of the pond. This sedimentation basin would be obsolete once upstream water quality facilities are constructed.

For the Phase 2 Developed Conditions, the 100-year floodplain in Moody Slough upstream of Interstate 505 is El. 127.23 (NGVD 29). The proposed residual 100-year floodplain is presented on Figure 11.

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 and overland conveyance. The proposed detention ponds, in conjunction with additional source and treatment control measures, would provide surface water quality treatment. Since the Willow Canal would be realigned, significant irrigation impacts would not occur. No significant impacts to downstream low flows in Moody Slough and Dry Slough would occur.

OPINION OF PROBABLE COSTS

Opinions of probable cost were developed for the improvements identified in the Drainage Plans described in this report.

Presented below is a summary of the Opinion of Probable Costs for Drainage Facilities for the Ultimate Developed Conditions and each proposed phase. A breakdown of the Opinion of Probable Costs is included in Appendix D.

SUMMARY OF COSTS

Description	Total
Phase 1 Conditions	$15,307,535
Phase 2 Conditions	2,795,790
Ultimate (Increment) Conditions	1,645,260
TOTAL (Ultimate Conditions [Buildout])	**$19,748,585**

Costs presented as part of this report do not reflect cost-sharing details 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 Moody Slough subbasin.

1. The Moody Slough subbasin area has significant flooding problems resulting directly from undersized existing conveyance capacity in channels and culverts and from additional water volume from spilling flows from adjacent subbasins. These factors are further compounded by the road profile of Interstate 505, which serves to impound floodwater west of the highway when the capacities of culverts along the highway are exceeded. The areas within the City's General Plan planned for development and that currently provide floodplain storage, would have to be replaced by either additional conveyance capacity and/or replacement storage.

2. Particular attention needs to be given to ensuring the functional hydraulic performance of natural corridors originating outside but passing through the urban growth areas.

3. The diversion of storm water runoff to Putah Creek is an effective means of mitigating encroachment into the existing floodplain upstream of Interstate 505, and is a critical component to the proposed facilities. The volume of storm water currently inundating the areas west of Interstate 505 is otherwise very difficult to remove or redirect.

4. The construction of facilities outlined for the Moody Slough subbasin 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 facilities.

5. A portion of the Moody Slough basin is currently developed with storm drains directing runoff from smaller storms southward toward Putah Creek. Overland conveyance for larger events is directed to the east and would be conveyed through the Putah Creek diversion to Putah Creek. There currently is no water quality treatment for existing developed areas draining southward before reaching Putah Creek. Water quality

improvements would be evaluated for these areas during the evaluations of areas within the City, but outside of Moody Slough, that currently drain to Putah Creek as well.

6. The peak flow and volume of storm runoff flowing into Dry Slough east of Interstate 505 would be reduced with implementation of the facilities proposed for the Moody Slough subbasin.

B. RECOMMENDATIONS

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

1. Implement storm drainage facilities to accommodate new development within the Moody Slough subbasin in conformance with the facilities outlined in this report. Phasing of development different than investigated in this report should be evaluated to ensure the functional 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 100-year floodplain.

3. Obtain updated topographic mapping to verify runoff patterns and refine the analysis performed for the Drainage Report.

4. 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.

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WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Tables

Tables

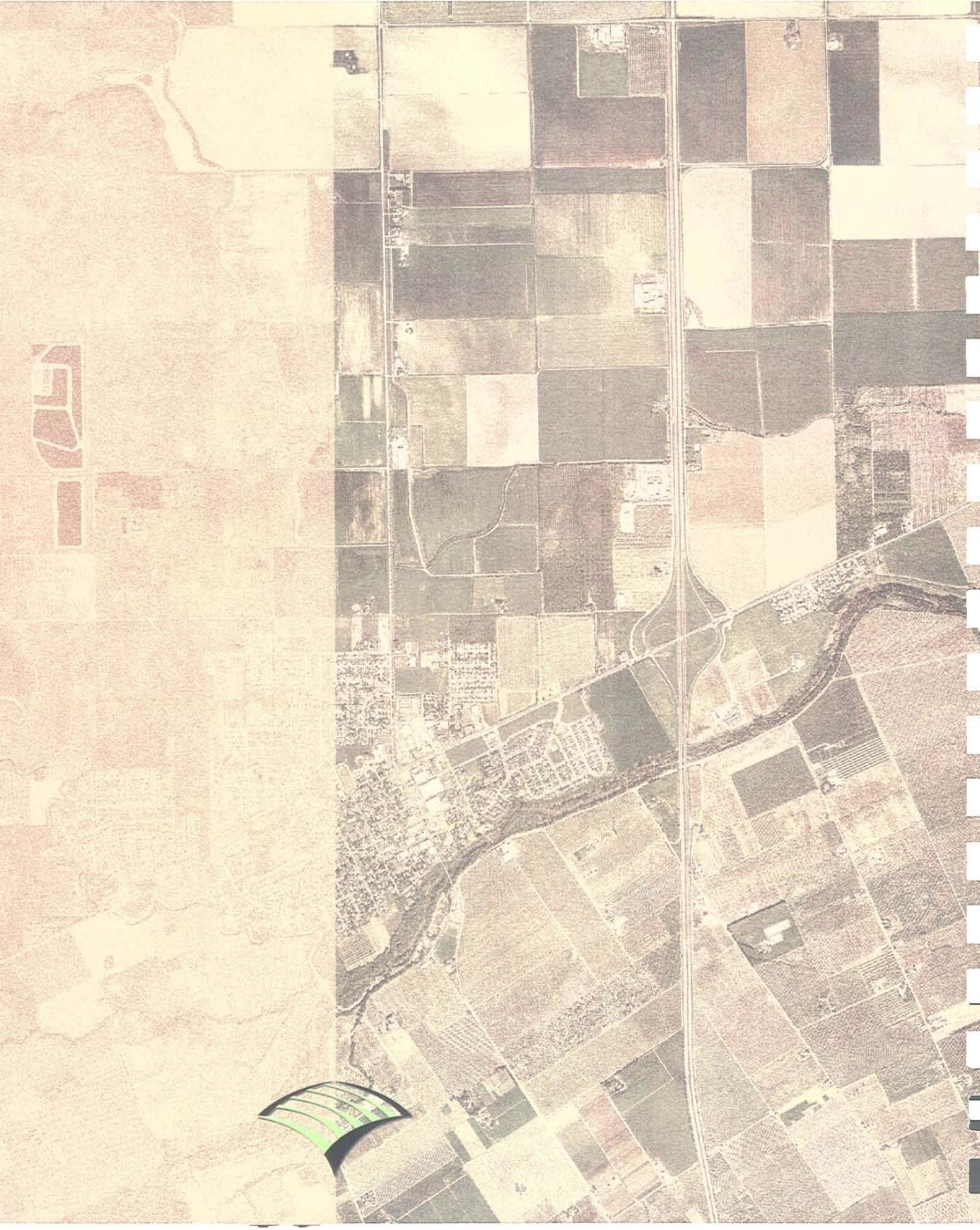


TABLE 1

CITY OF WINTERS
DRAINAGE REPORT - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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 - MOODY SLOUGH SUBBASIN

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

Pipe Material	Base Manning's Roughness Coefficient, n_{base}
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

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

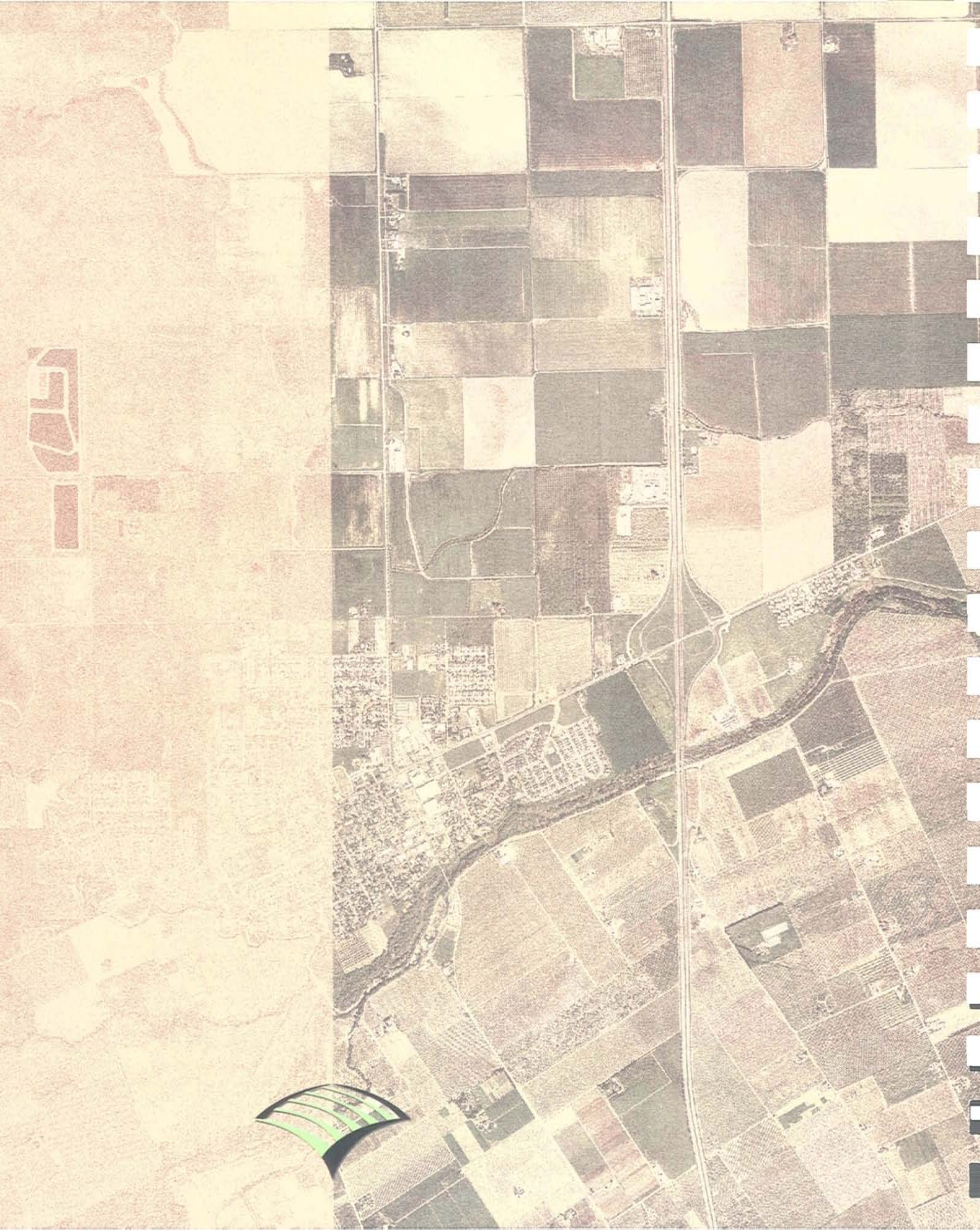
Source: Chow, Ven Te, *Open Channel Hydraulics*, 1959.

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Figures

Figures



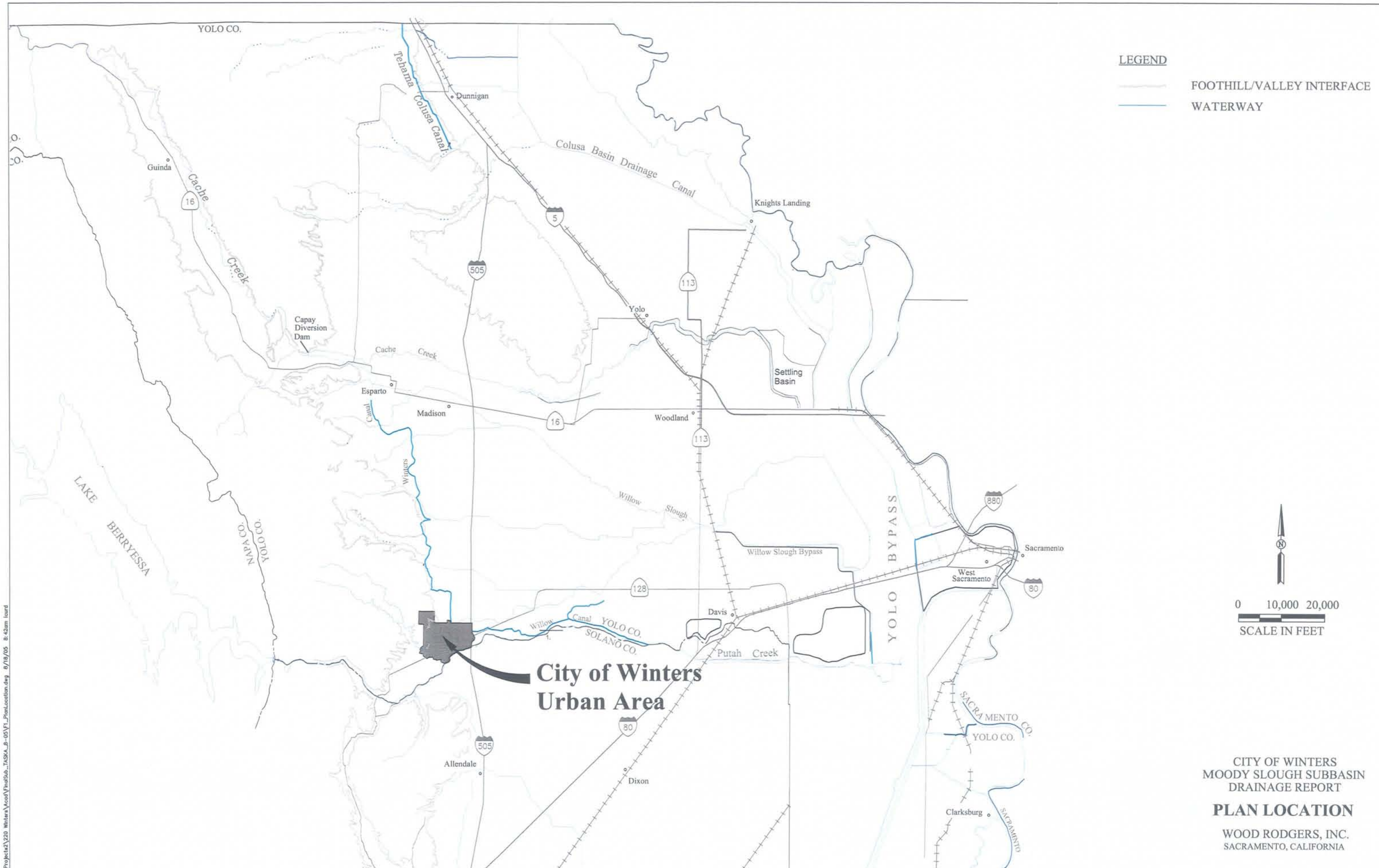
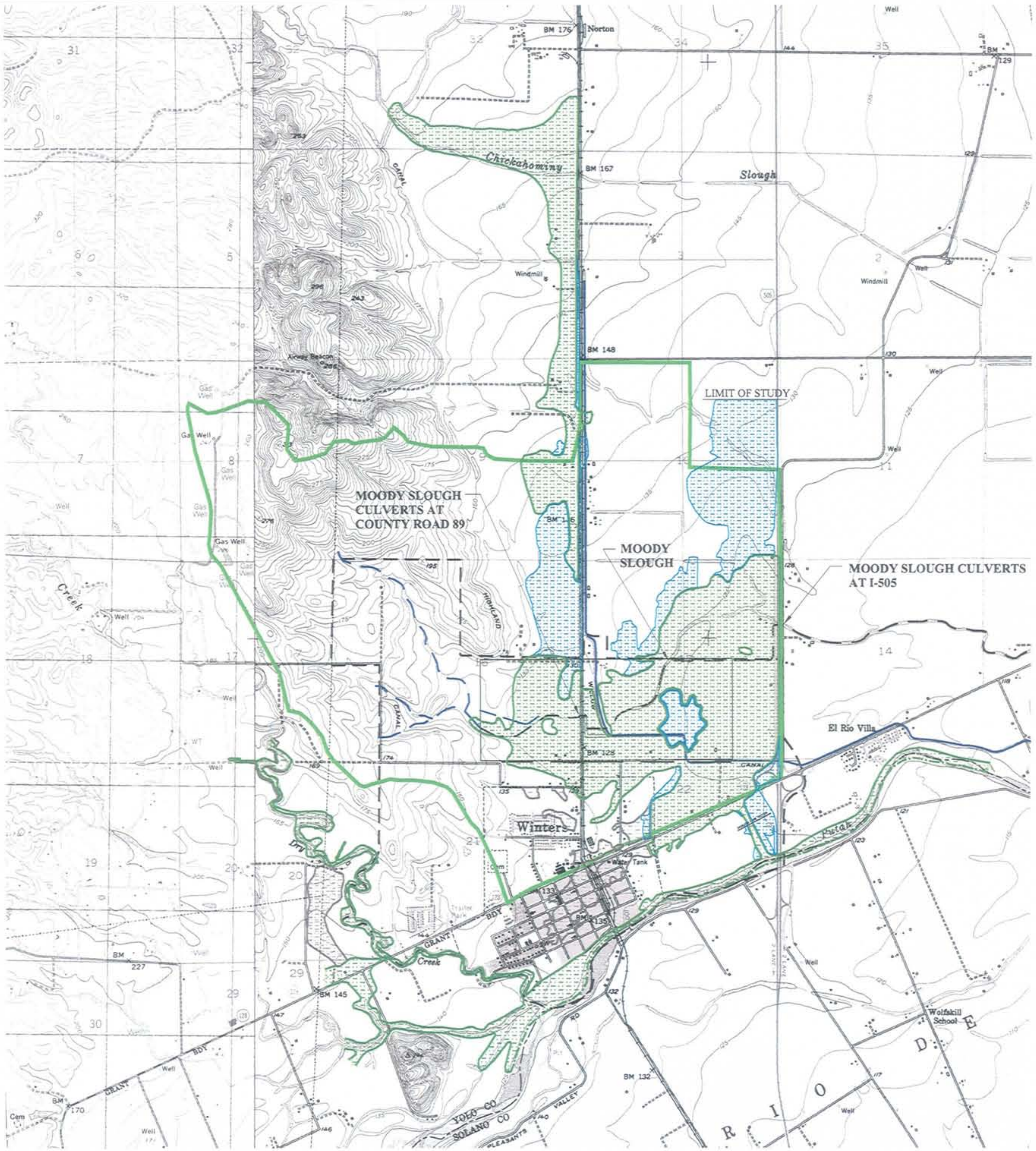

LEGEND
FOOTHILL/VALLEY INTERFACE
WATERWAY
YOLO CO.
Tehama Colusa Canal
Dunnigan
Colusa Basin Drainage Canal
Knights Landing
Guinda
Cache Creek
16
5
505
113
Yolo
Capay Diversion Dam
Cache Creek
Settling Basin
Esparto
Madison
Woodland
Canal
Winters
Willow Slough
LAKE BERRYESSA
NAPA CO.
YOLO CO.
880
Sacramento
Willow Slough Bypass
YOLO BYPASS
West Sacramento
80
128
Davis
Willow Canal
YOLO CO.
SOLANO CO.
Putah Creek
City of Winters Urban Area
SACRAMENTO CO.
YOLO CO.
Allendale
Dixon
Clarksburg
SACRAMENTO
0 10,000 20,000
SCALE IN FEET
CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT
PLAN LOCATION
WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 1



LEGEND

- APPROXIMATE REVISED 100-YEAR FLOODPLAIN (SEE NOTE 2)
- ZONE A; ZONE AE, ZONE AH; ZONE AO (SEE NOTE 3)
- ZONE B; ZONE X (SEE NOTE 3)
- URBAN LIMIT
- EXISTING CREEK
- EXISTING WILLOW CANAL
- MOODY SLOUGH SUBBASIN BOUNDARY

NOTES

1. The existing drainage and irrigation facilities presented on this plan are approximate for schematic purposes only.
2. Revised floodplain is based on the use of detailed topographic mapping. North of the I-505 interchange.
3. Approximate floodplain boundaries are shown.

SOURCES

1. Topographic mapping is United States Geologic Survey Quadrangle Maps, National Geodetic Vertical Datum of 1929.
2. Sources are effective FEMA Flood Insurance Rate Maps, Panel Numbers 060425 0001 C, 11/20/98, and 060423 0540 C, 3/23/99.

N

0 1500 3000

SCALE IN FEET

CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

EXISTING CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

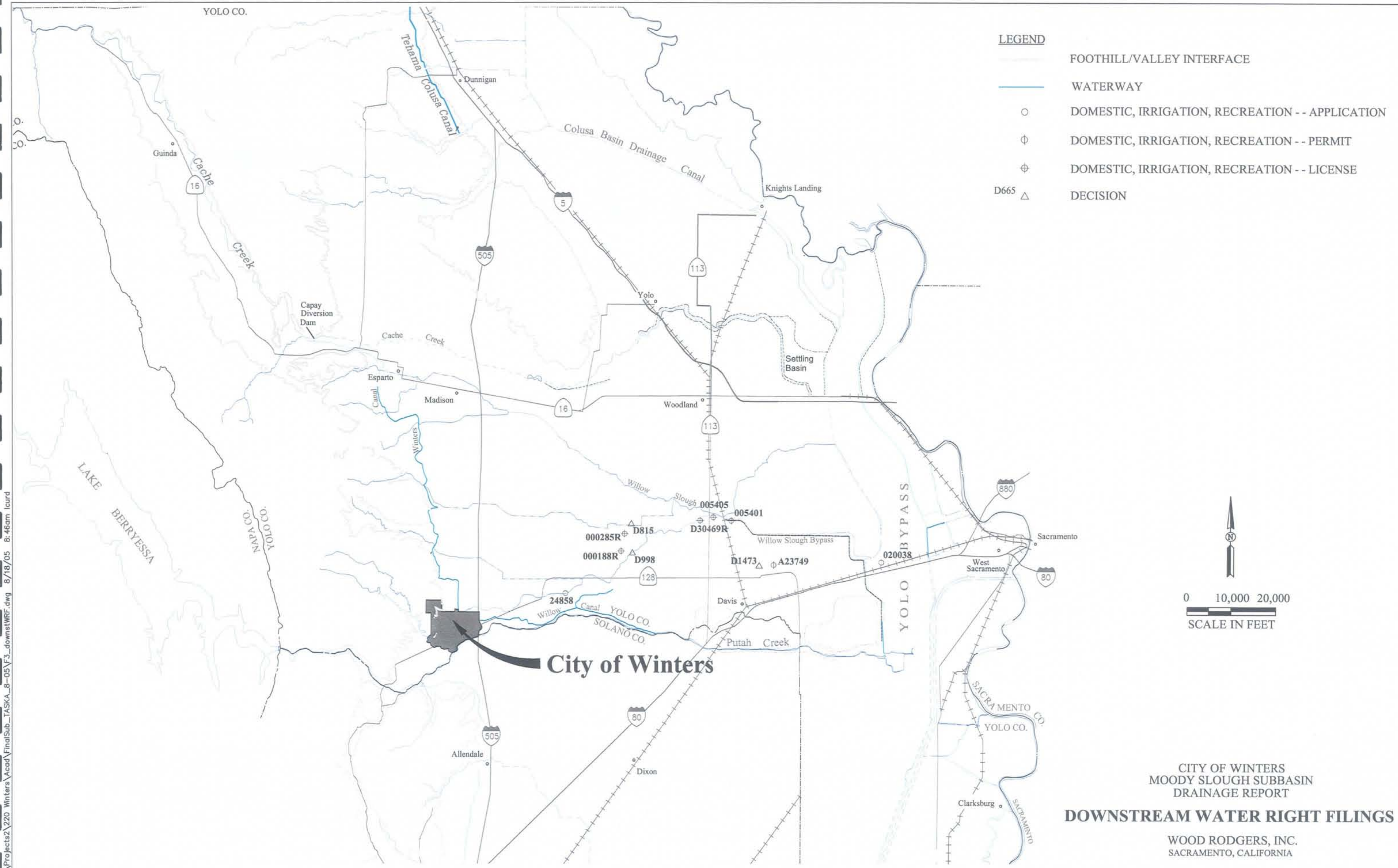


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FIGURE 3

SOURCE:
City of Winters - General Plan - May 1992

CITY OF WINTERS
MODY SLOUGH SUBBASIN
DRAINAGE REPORT

ULTIMATE LAND USE

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 4

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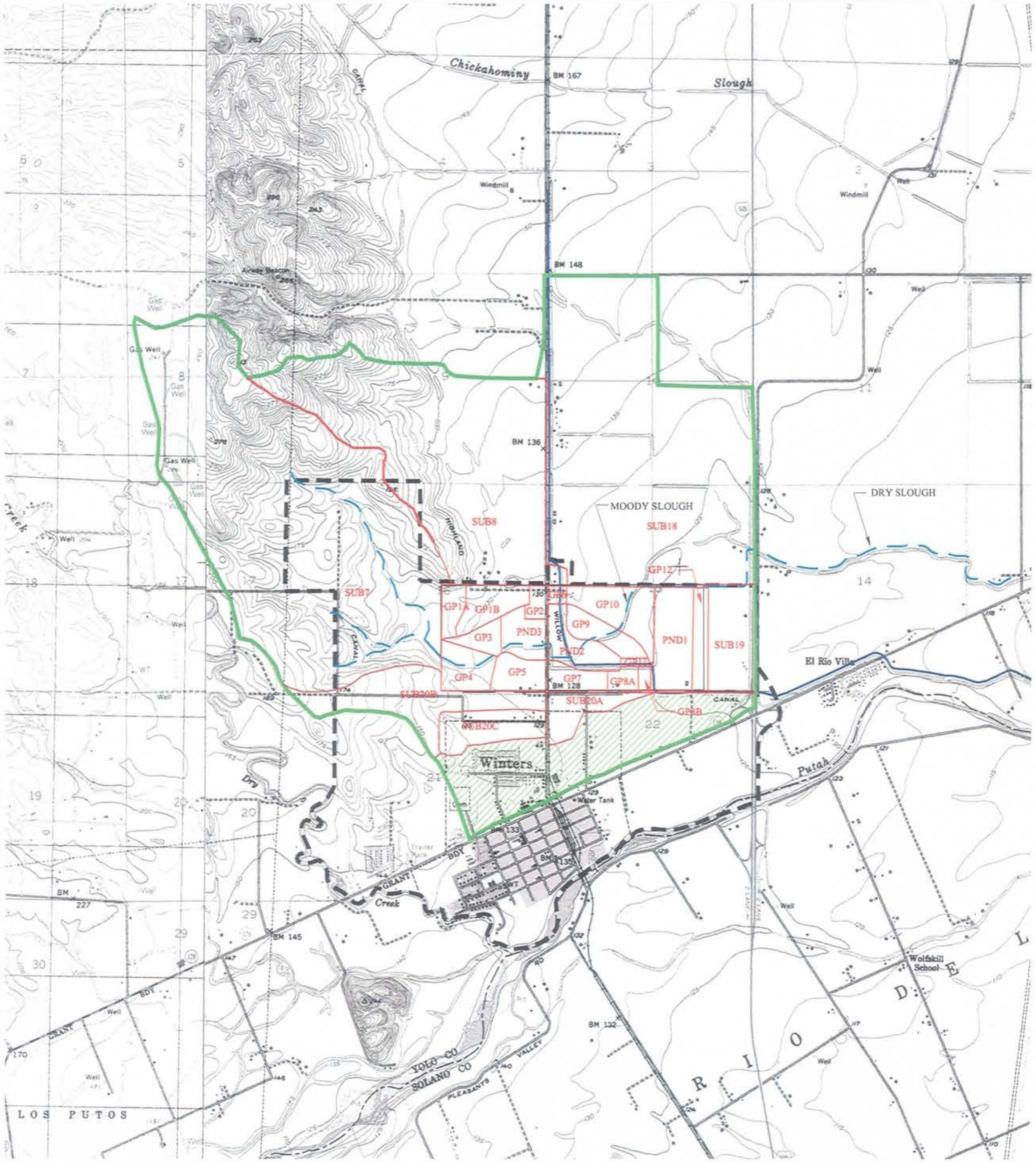


LEGEND

AREA DRAINING TO PUTAH CREEK

URBAN LIMIT

EXISTING CREEK

EXISTING WILLOW CANAL

MOODY SLOUGH SUBBASIN BOUNDARY

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 1500 3000

SCALE IN FEET

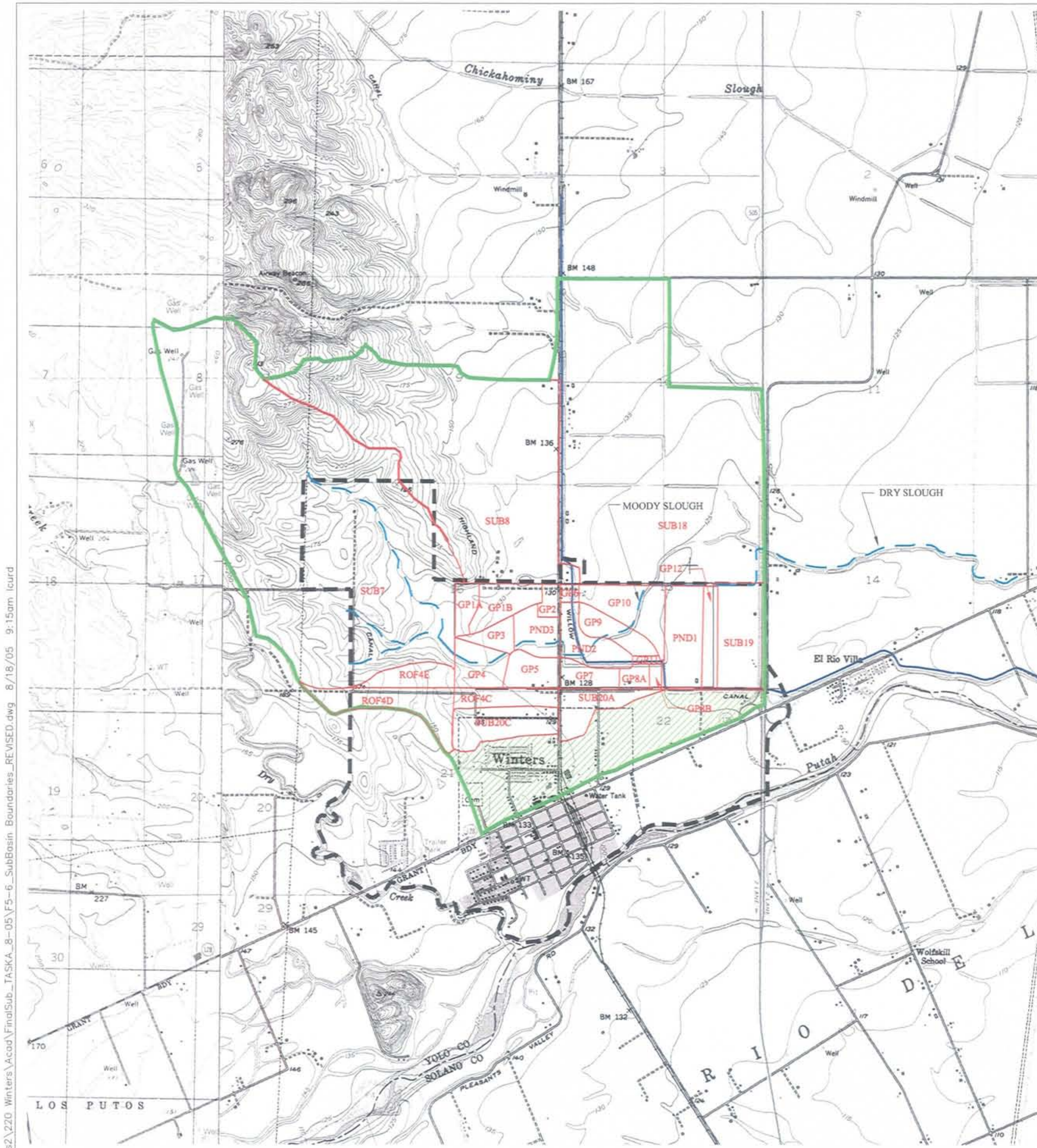
CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

SUBBASIN BOUNDARIES FOR PHASED CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 5

I:\Projects2\220 Winters\Acad\FinalSub_TASKA_8-05\F5-6_SubBasin Boundaries_REVISED.dwg 8/18/05 9:16am lcurd



LEGEND

AREA DRAINING TO PUTAH CREEK

URBAN LIMIT

EXISTING CREEK

EXISTING WILLOW CANAL

MOODY SLOUGH SUBBASIN BOUNDARY

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.

N

0 1500 3000

SCALE IN FEET

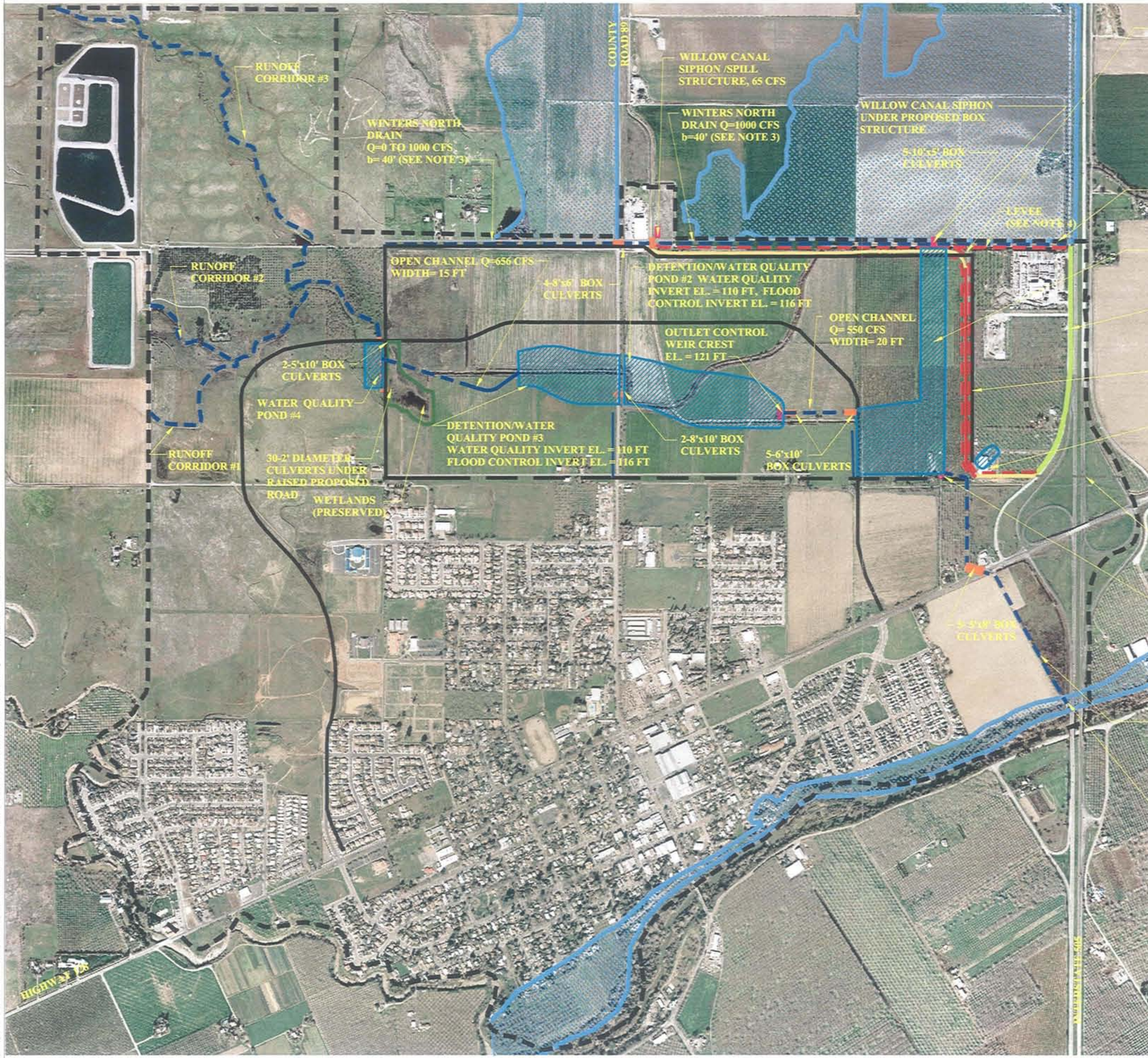
CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

SUBBASIN BOUNDARIES FOR ULTIMATE CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 6

I:\Projects2\220 Winters\Acad\FinalSub_TASKA_8-05\F5-6_SubBasin Boundaries_REVISED.dwg 8/18/05 9:15am lcurd



LEGEND

- RESIDUAL 100-YEAR FLOODPLAIN
- DETENTION/WATER QUALITY POND
- PROPOSED ROAD
- URBAN LIMIT
- EXISTING WETLANDS BOUNDARY
- OPEN CHANNEL
- RELOCATED WILLOW CANAL (OPEN CHANNEL)
- RELOCATED WILLOW CANAL IN PIPELINE
- LEVEE
- RUNOFF CORRIDOR
- PROPOSED ROAD AS RUNOFF CORRIDOR
- FLOODWALL
- STORM DRAIN PIPE
- WEIR
- CULVERTS

NOTES:

1. The existing and proposed facilities presented on this plan are for illustrative purposes only.
2. Winters North Drain capacity is based upon flood conveyance as a natural channel (possible mitigation for Moody Slough).
3. Levee east of Pond #1 running east to I-505, can be constructed in Phase 1 as an option, thereby eliminating the need for the levee shown on the east side of the relocated Willow Canal (costs do not reflect option).

SOURCE:

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

N

0 600 1200

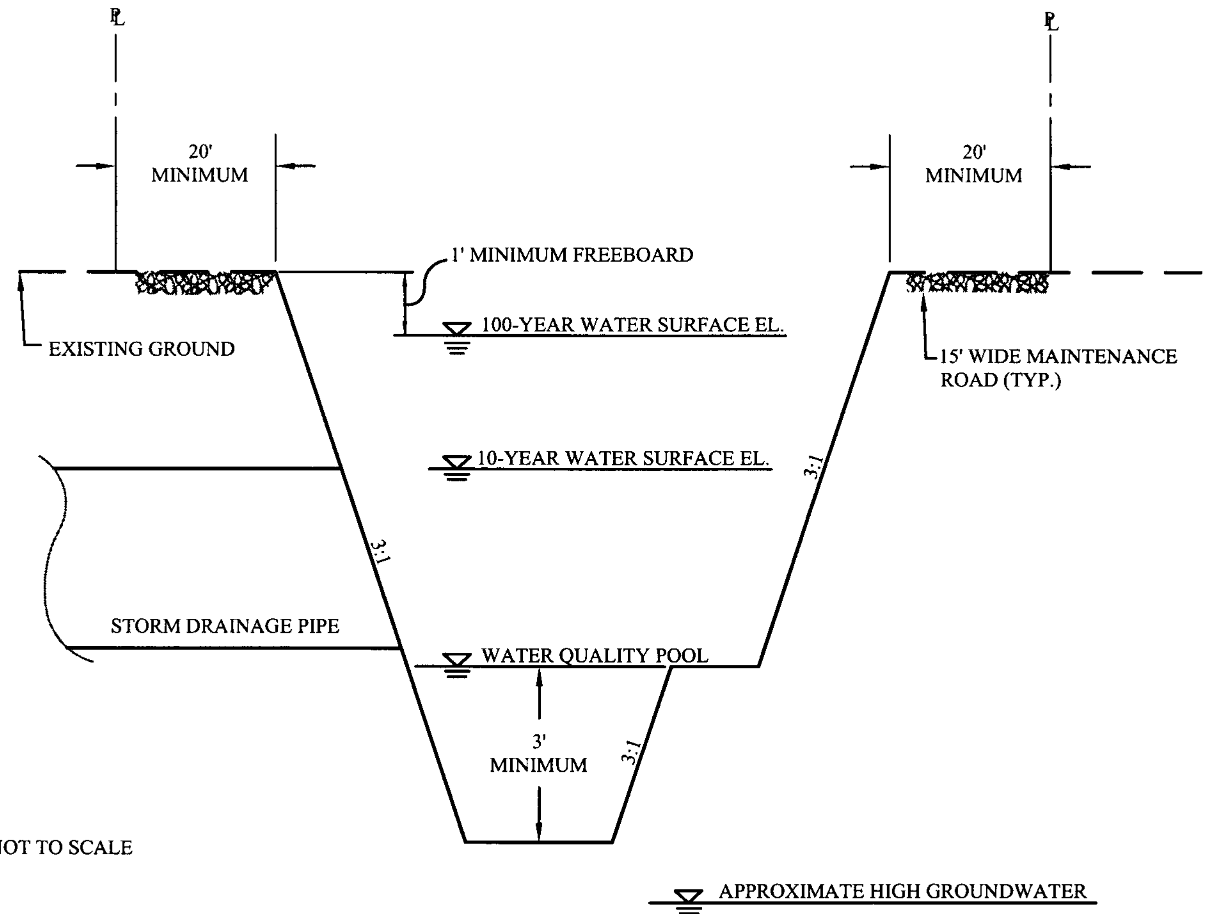
SCALE IN FEET

CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

ULTIMATE CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 7



NOT TO SCALE

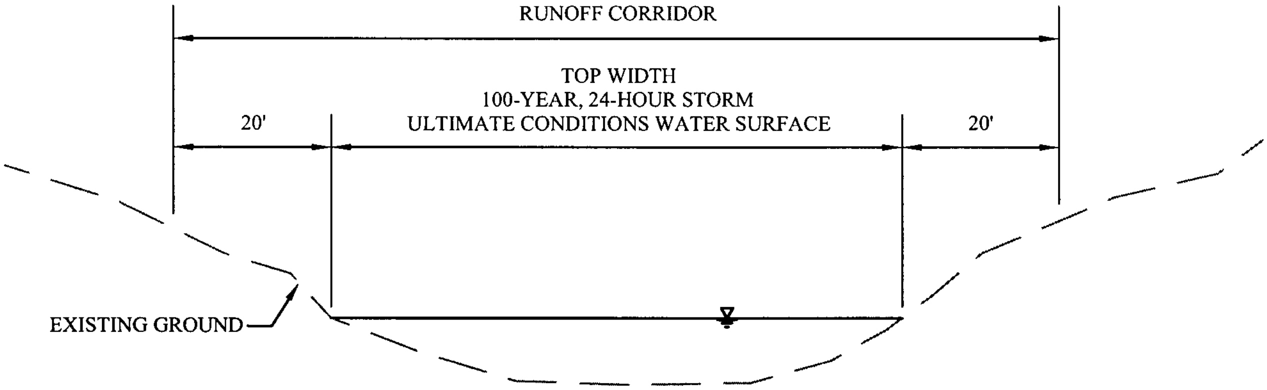
CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

TYPICAL DETENTION/WATER QUALITY POND

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

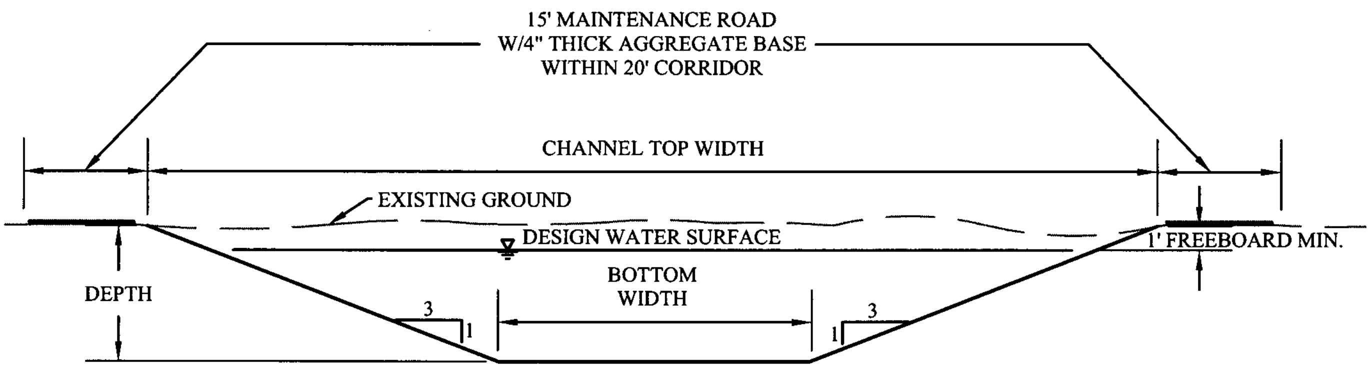
FIGURE 8

I:\Projects2\220 Winters\Acad\FinalSub_TASKA_8-05\F8_Pond Cross Section.dwg 8/18/05 9:42am lcurd



RUNOFF CORRIDOR

NOT TO SCALE



NOTE:

CHANNEL DIMENSIONS WILL VARY ACCORDING TO DESIGN HYDRAULIC CAPACITY.

TYPICAL CHANNEL CROSS SECTION

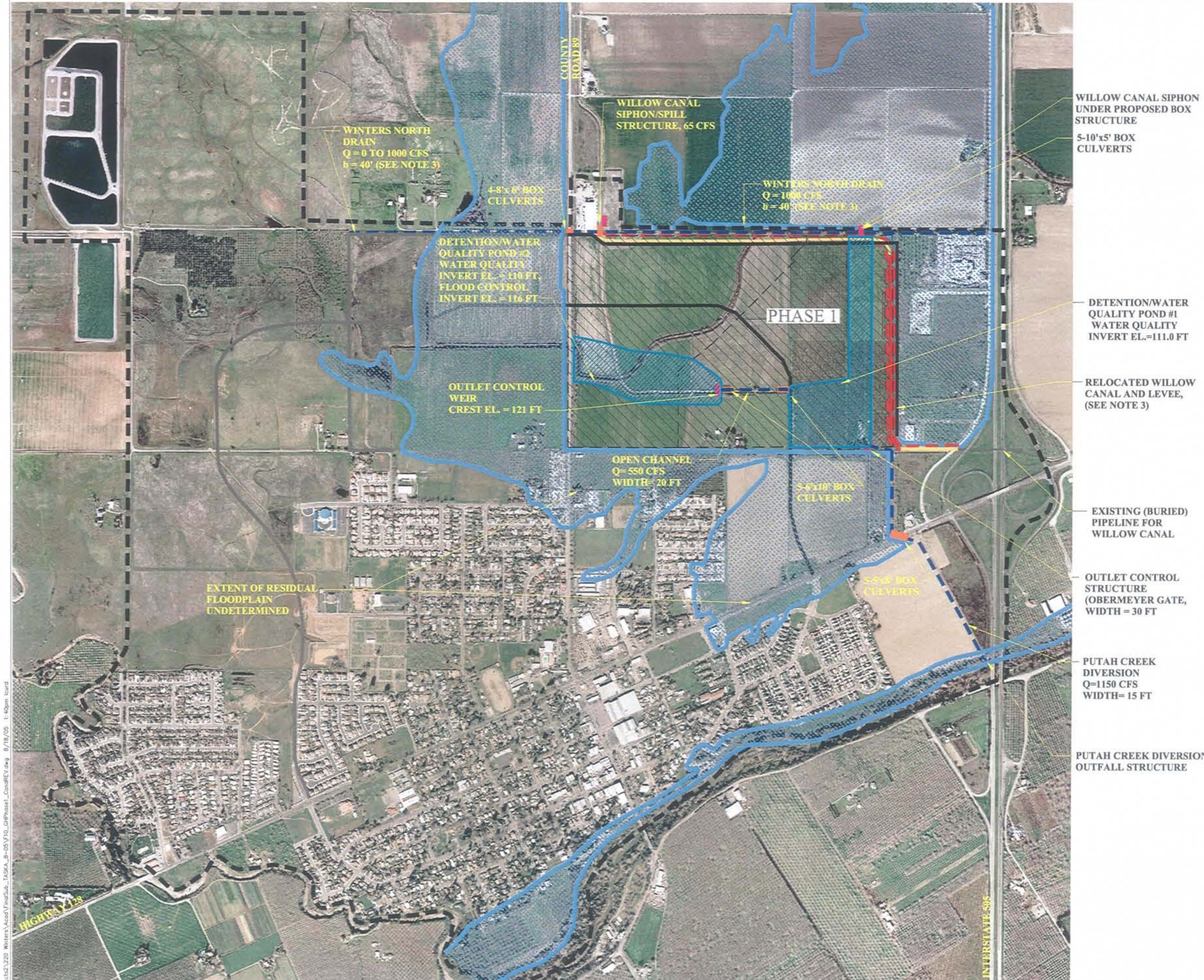
NOT TO SCALE

CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

DRAINAGE CHANNELS TYPICAL SECTIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 9



LEGEND

- PROPOSED PHASED AREAS
- RESIDUAL FEMA 100-YEAR FLOODPLAIN
- DETENTION/WATER QUALITY POND
- PROPOSED ROAD
- ULTIMATE ROAD
- URBAN LIMIT
- OPEN CHANNEL
- RELOCATED WILLOW CANAL (OPEN CHANNEL)
- RELOCATED WILLOW CANAL IN PIPELINE
- LEVEE
- RUNOFF CORRIDOR
- PROPOSED ROAD AS RUNOFF CORRIDOR
- STORM DRAIN PIPE
- WEIR
- CULVERTS

NOTES:

1. The existing and proposed facilities presented on this plan are for illustrative purposes only.
2. Winters North Drain capacity is based upon flood conveyance as a natural channel (possible mitigation for Moody Slough).
3. Levee east of Pond #1 running east to I-505, can be constructed in Phase 1 as an option, thereby eliminating the need for the levee shown on the east side of the relocated Willow Canal (costs do not reflect option).

SOURCE:

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

0 600 1200

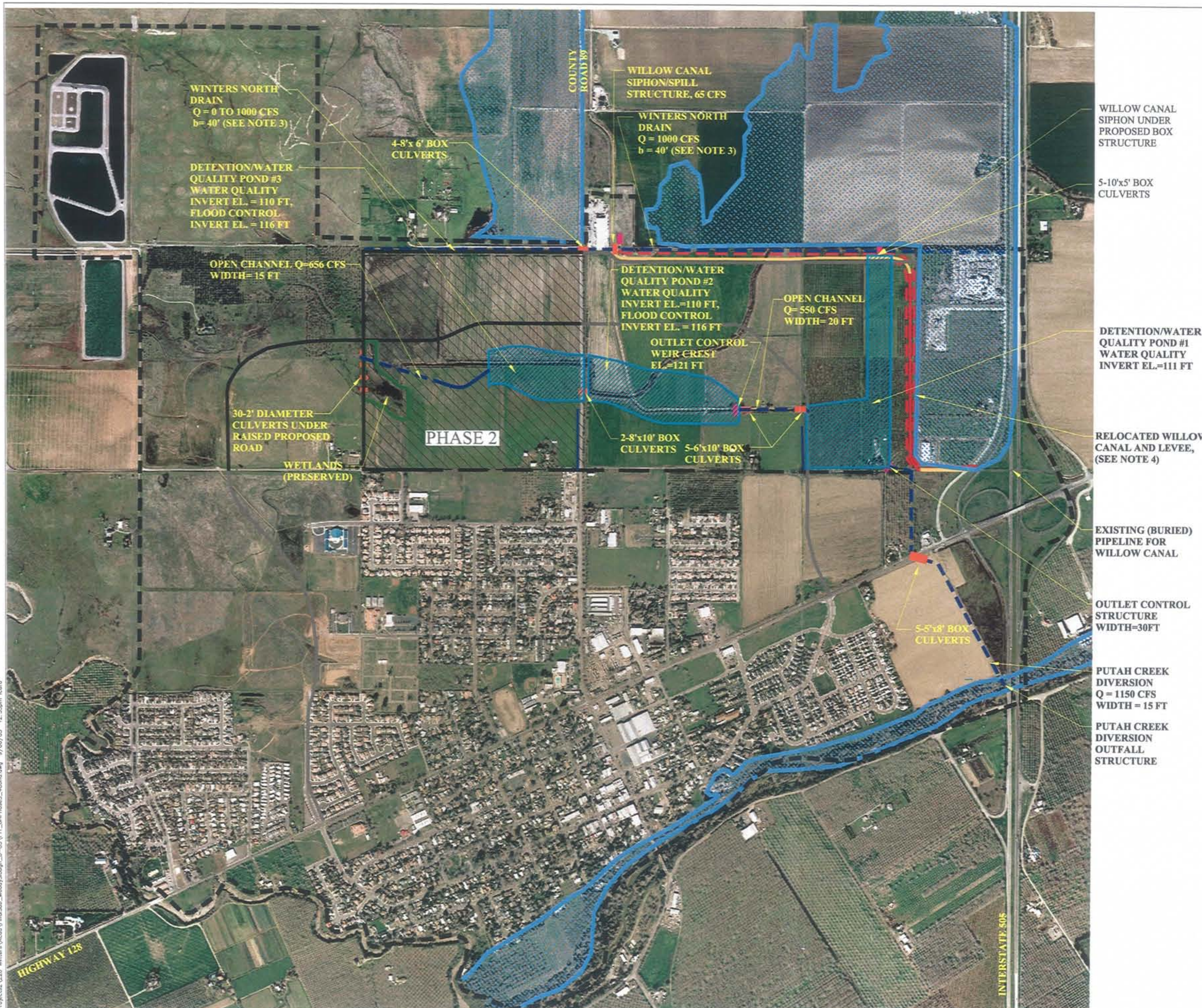
SCALE IN FEET

CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

PHASE 1 CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

FIGURE 10



LEGEND

- PROPOSED PHASED AREAS
- RESIDUAL FEMA 100-YEAR FLOODPLAIN
- DETENTION/WATER QUALITY POND
- PROPOSED ROAD
- ULTIMATE ROAD
- URBAN LIMIT
- EXISTING WETLANDS BOUNDARY
- OPEN CHANNEL
- RELOCATED WILLOW CANAL (OPEN CHANNEL)
- RELOCATED WILLOW CANAL IN PIPELINE
- LEVEE
- RUNOFF CORRIDOR
- RUNOFF CORRIDOR AS PROPOSED ROAD
- STORM DRAIN PIPE
- WEIR
- CULVERTS

NOTES:

1. The existing and proposed facilities presented on this plan are for illustrative purposes only.
2. Winters North Drain capacity is based upon flood conveyance as a natural channel (possible mitigation for Moody Slough).
3. Levee east of Pond #1 running east to I-505, can be constructed in Phase 1 as an option, thereby eliminating the need for the levee shown on the east side of the relocated Willow Canal (costs do not reflect option).

SOURCE:

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

0 600 1200

SCALE IN FEET

CITY OF WINTERS
MOODY SLOUGH SUBBASIN
DRAINAGE REPORT

PHASE 2 CONDITIONS

WOOD RODGERS, INC.
SACRAMENTO, CALIFORNIA

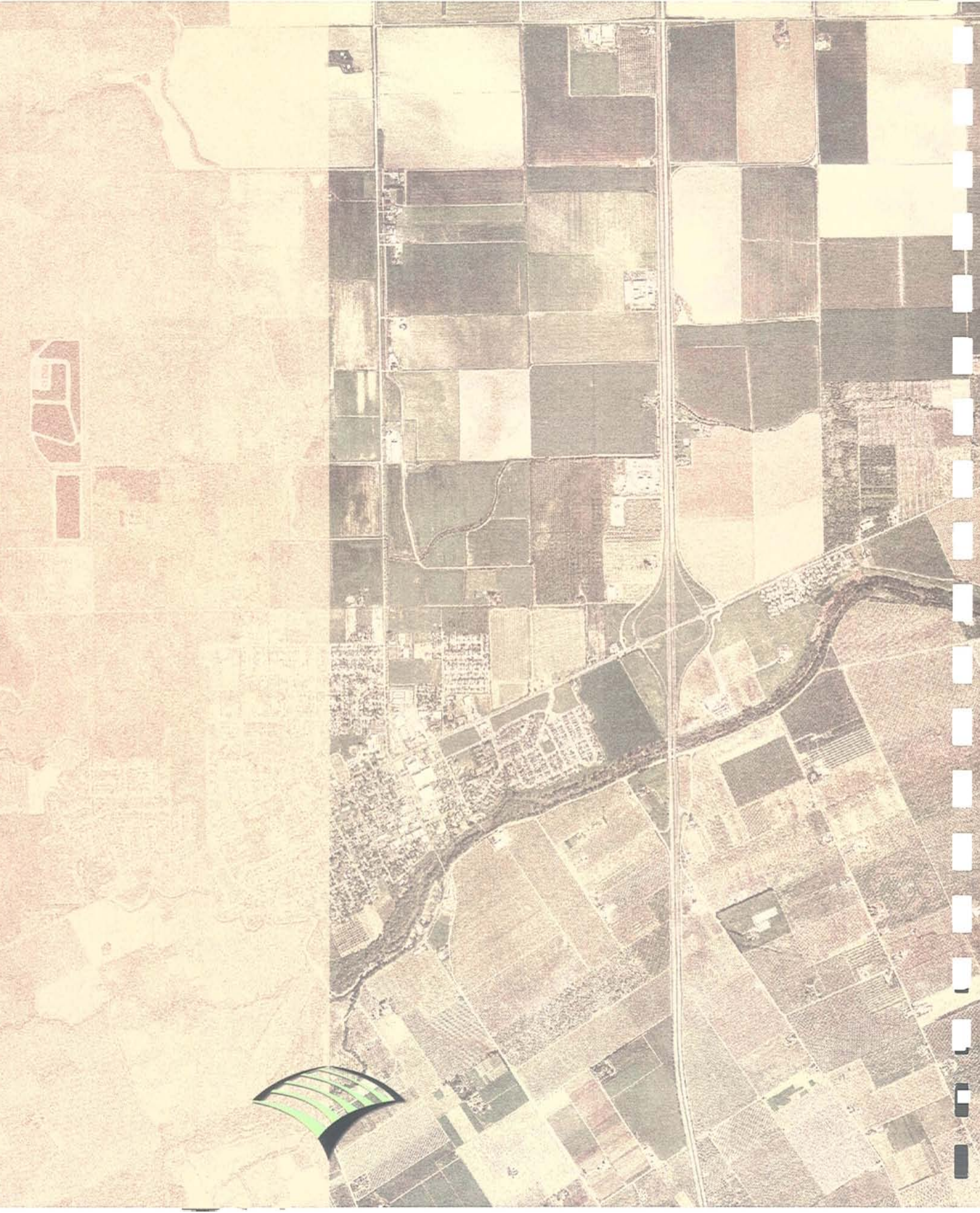
FIGURE 11

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendices

Appendices

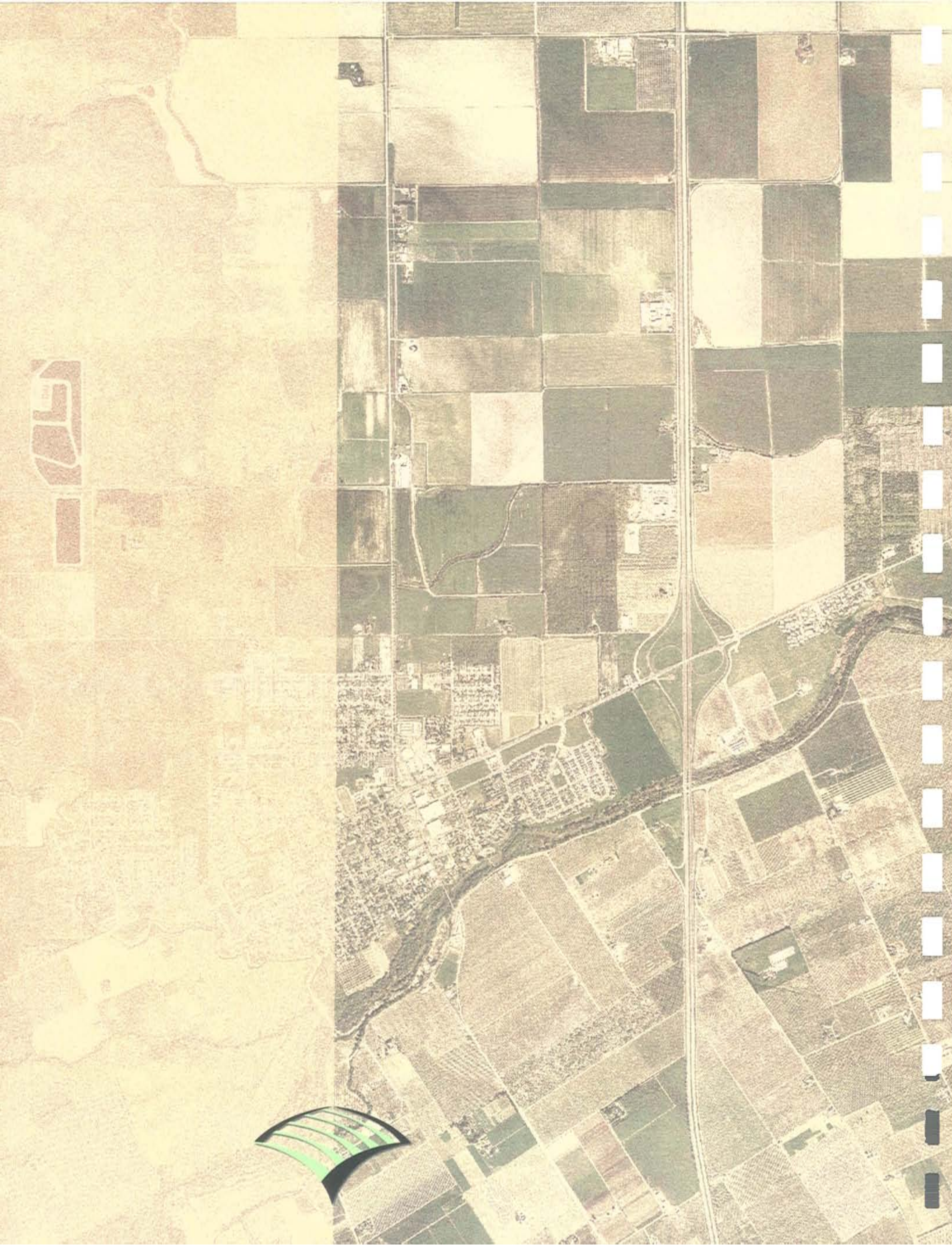


WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix A

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



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.

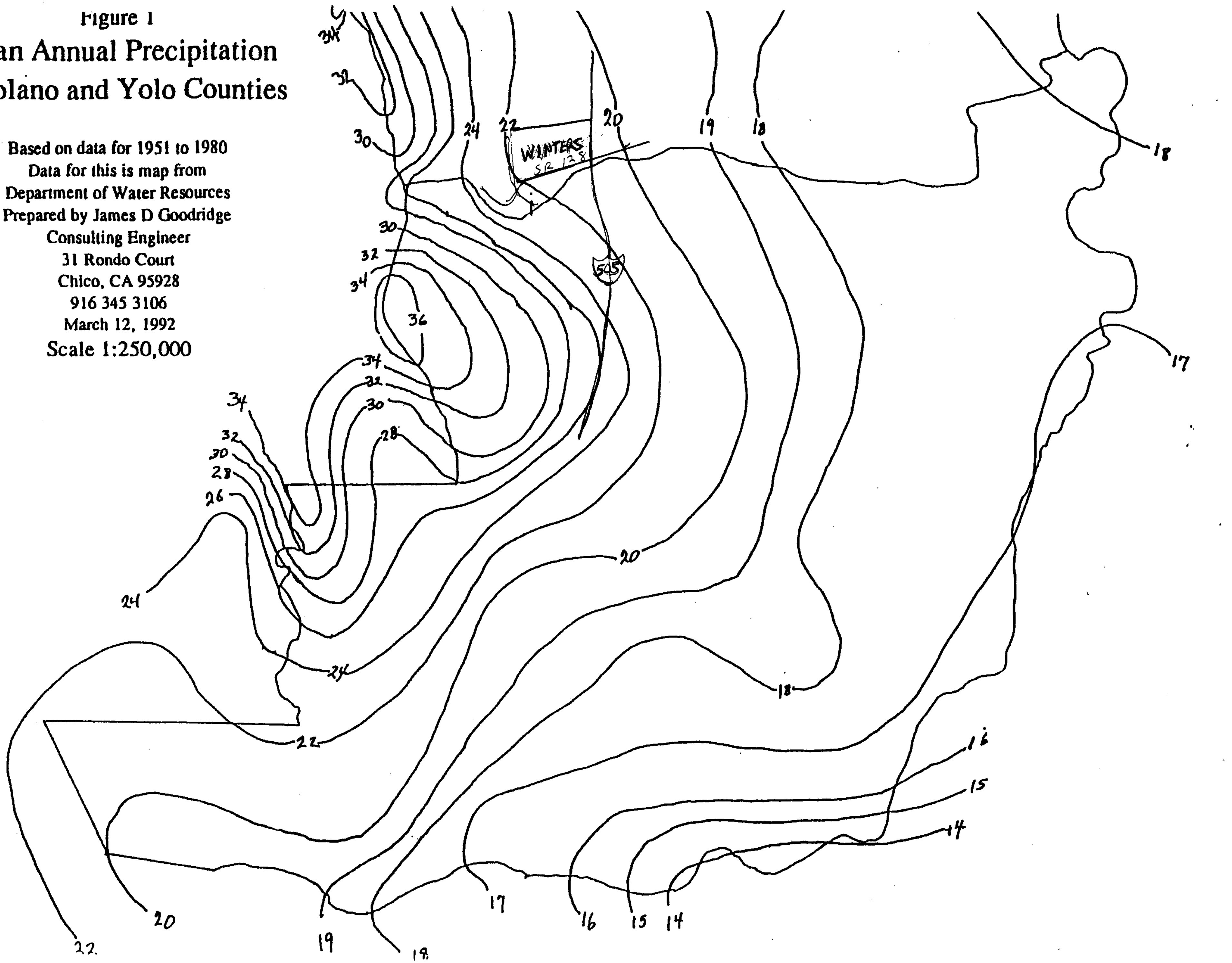
The maximum 24 hour rainfall ever recorded in the San Francisco Bay drainage area was the 15.28 inches at Atlas Road on February 17, 1986. The previous maximum was the Mt. Saint Helena storm of April 1884. The highest ever one day rain in the Central Valley Drainage area was 17.60 at four Trees in the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

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



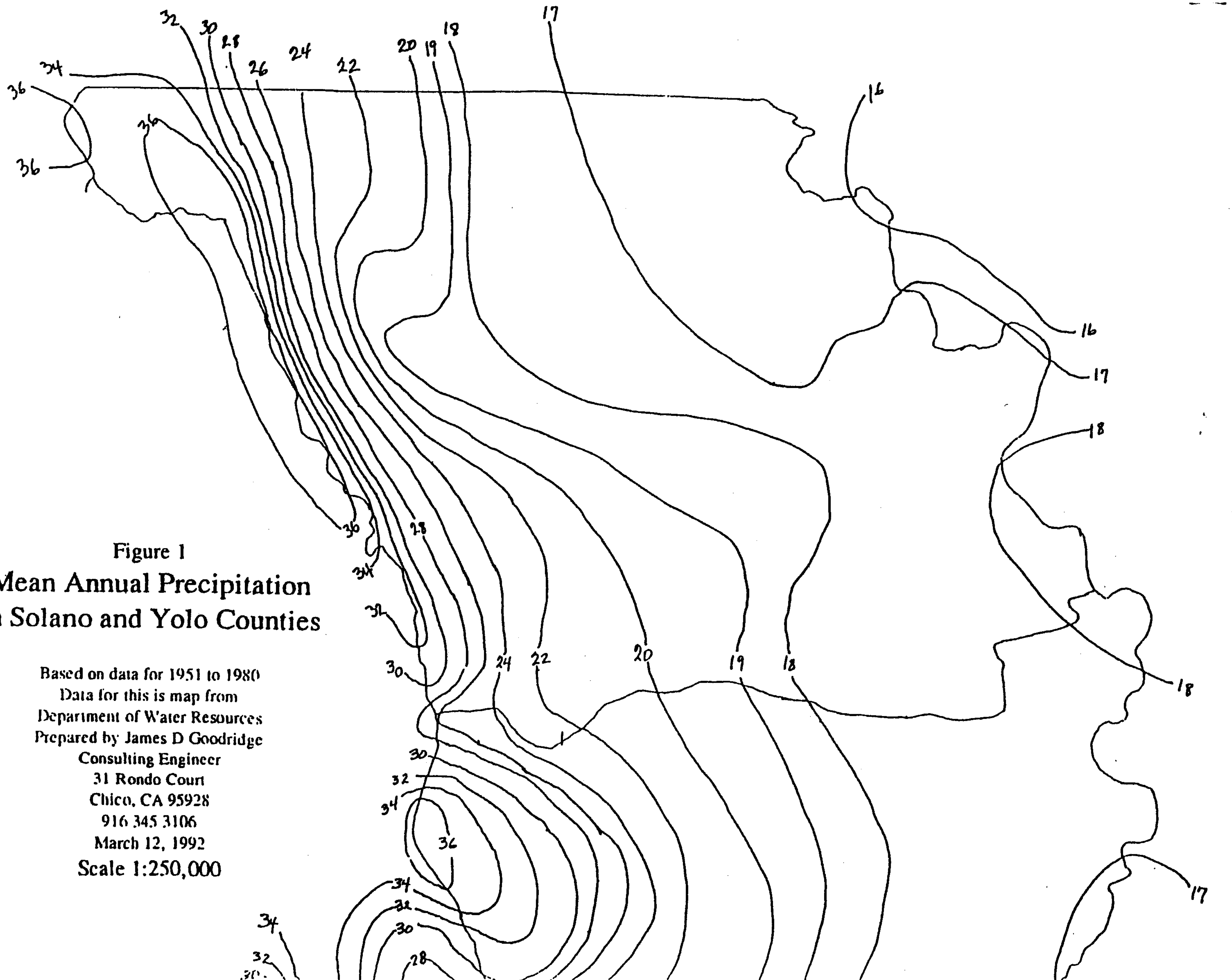


Figure 1
Mean Annual Precipitation in Solano and Yolo Counties

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

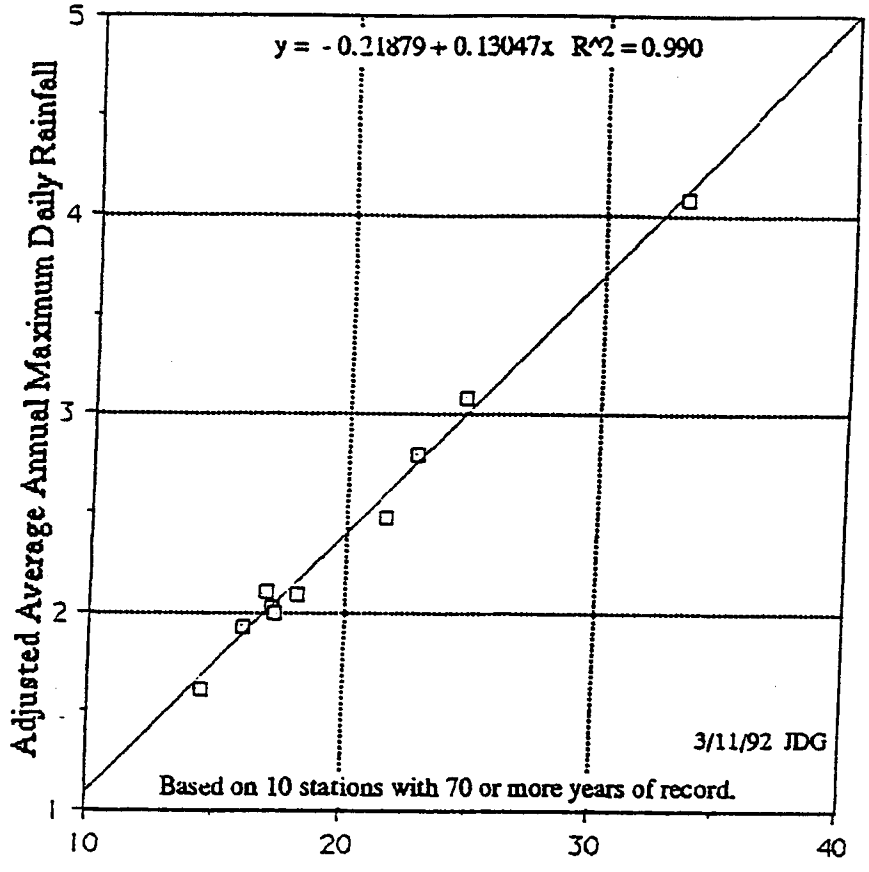


Figure 2

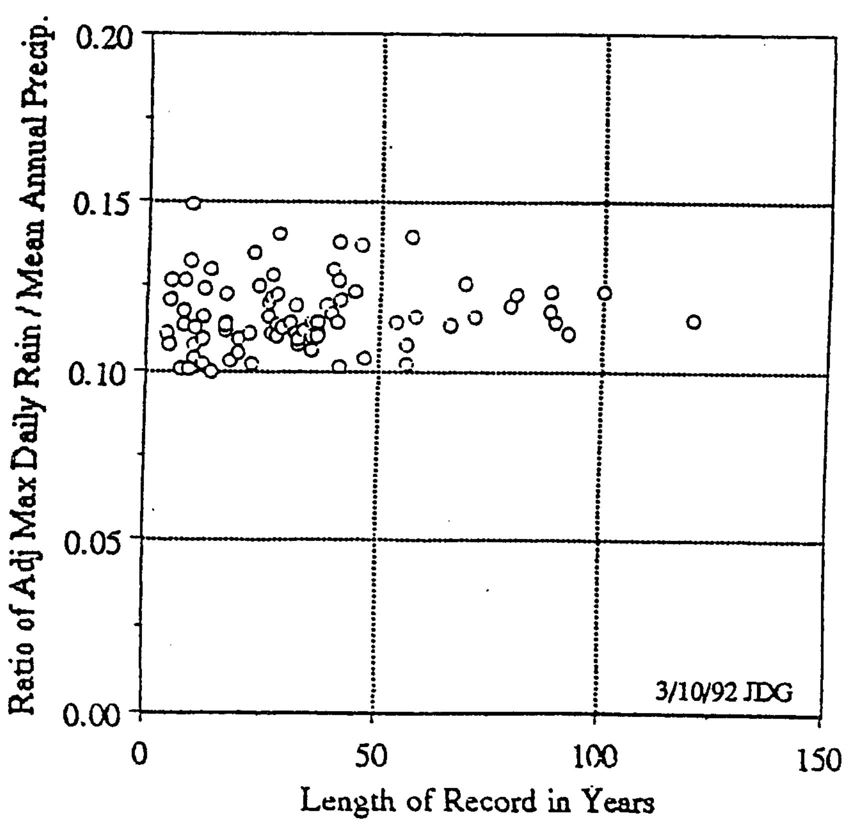


Figure 3

Solano & Yolo County Resign Rainfall

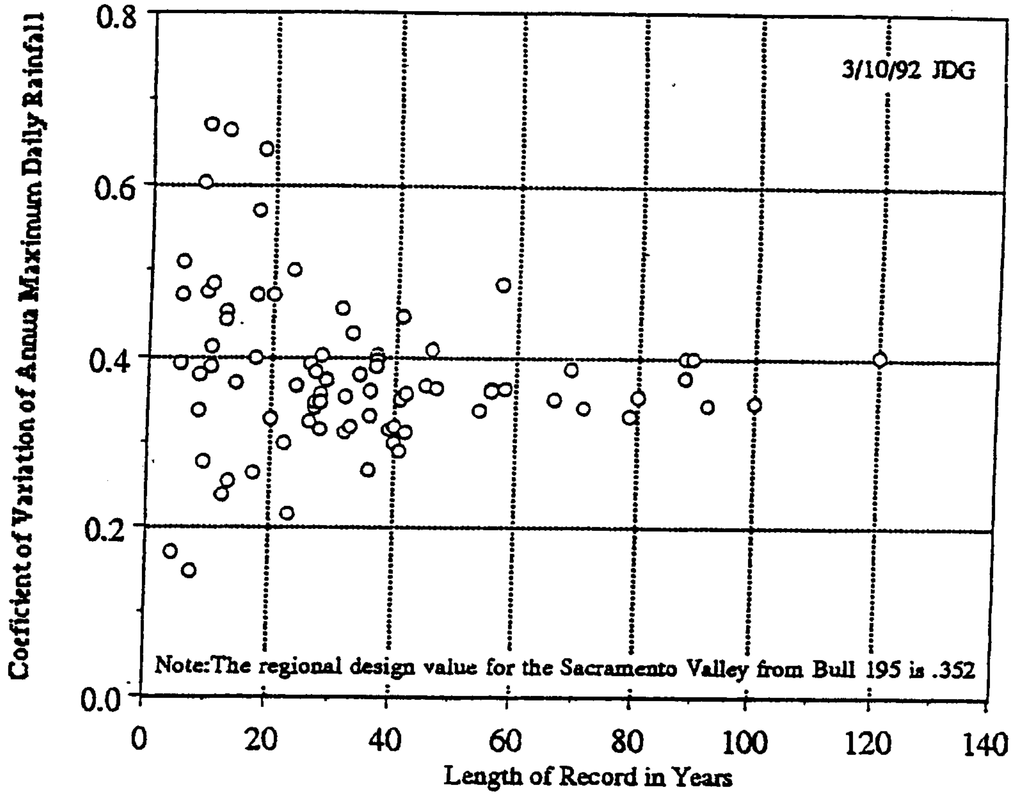


Figure 4

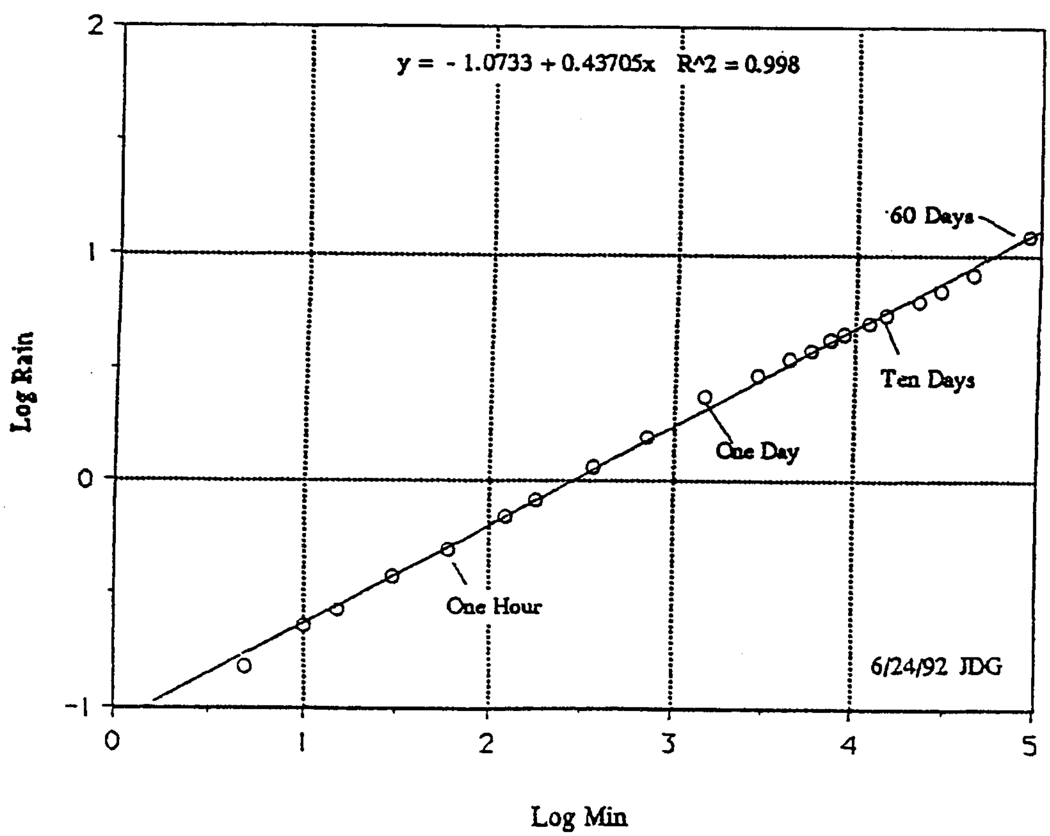


Figure 5

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 B

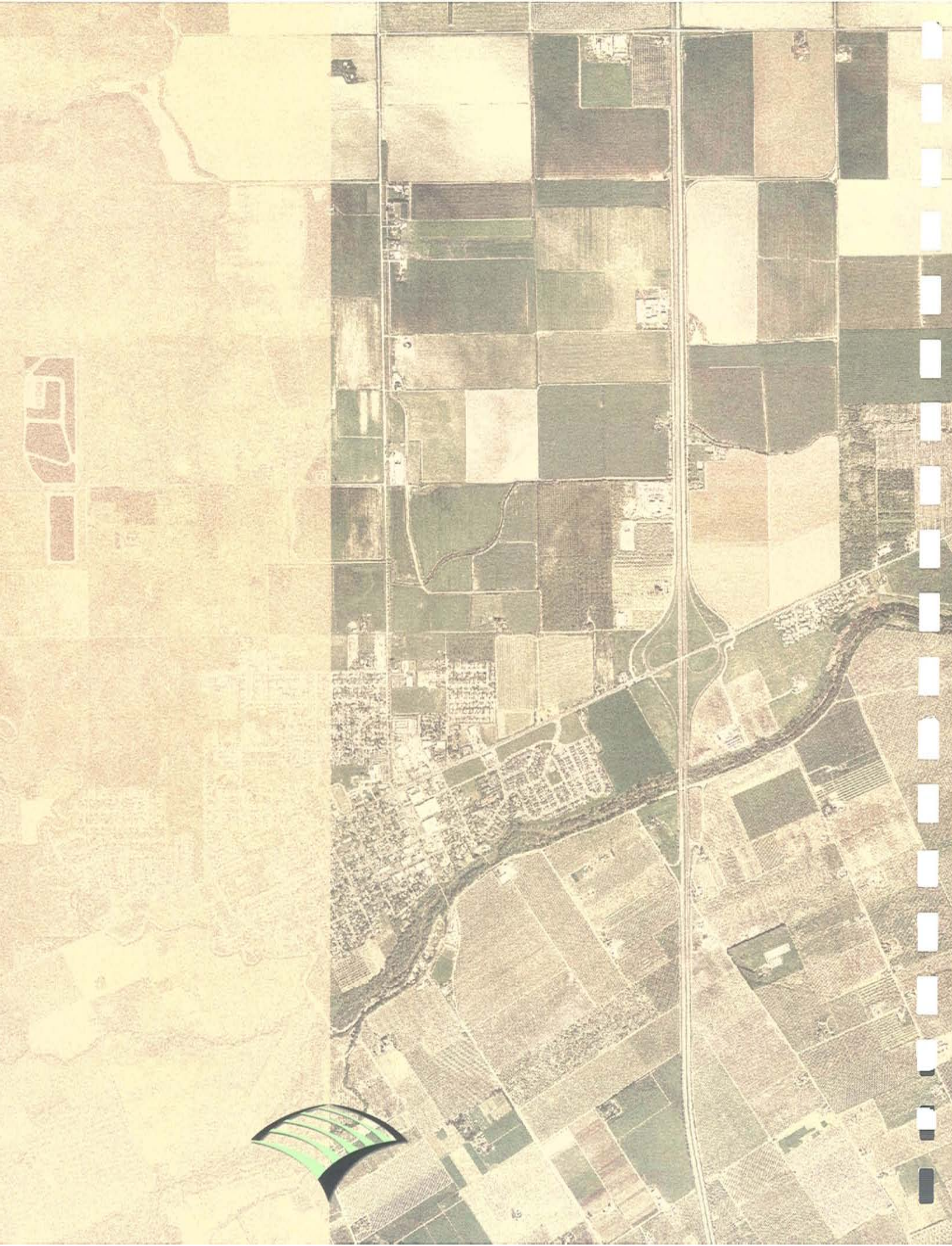
Spreadsheets and Models for Hydrologic and Hydraulic Calculations (Digital Files Available from City Upon Request)

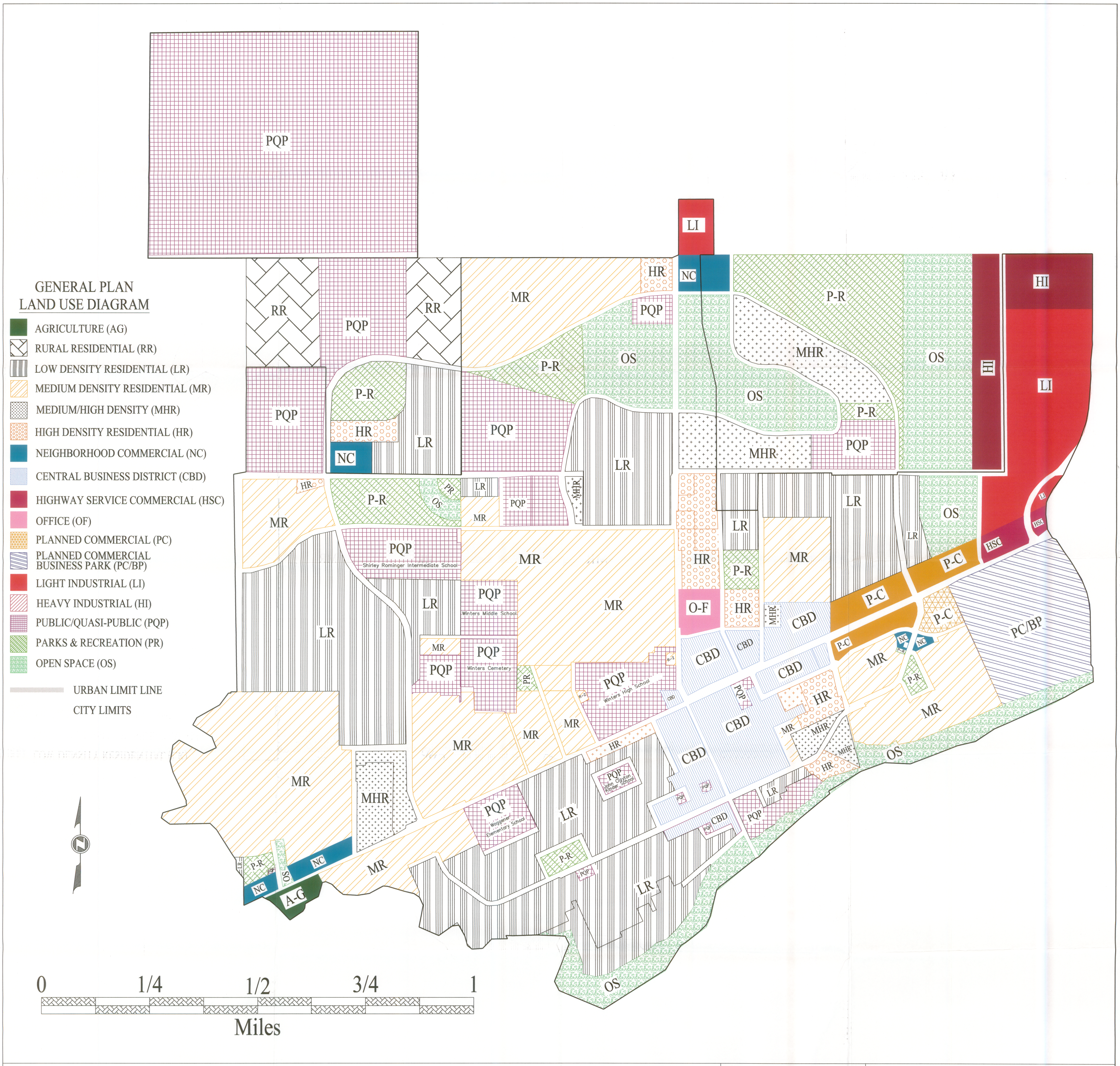
WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix C

"City of Winters General Plan Amendment," 2003 (Pocket)





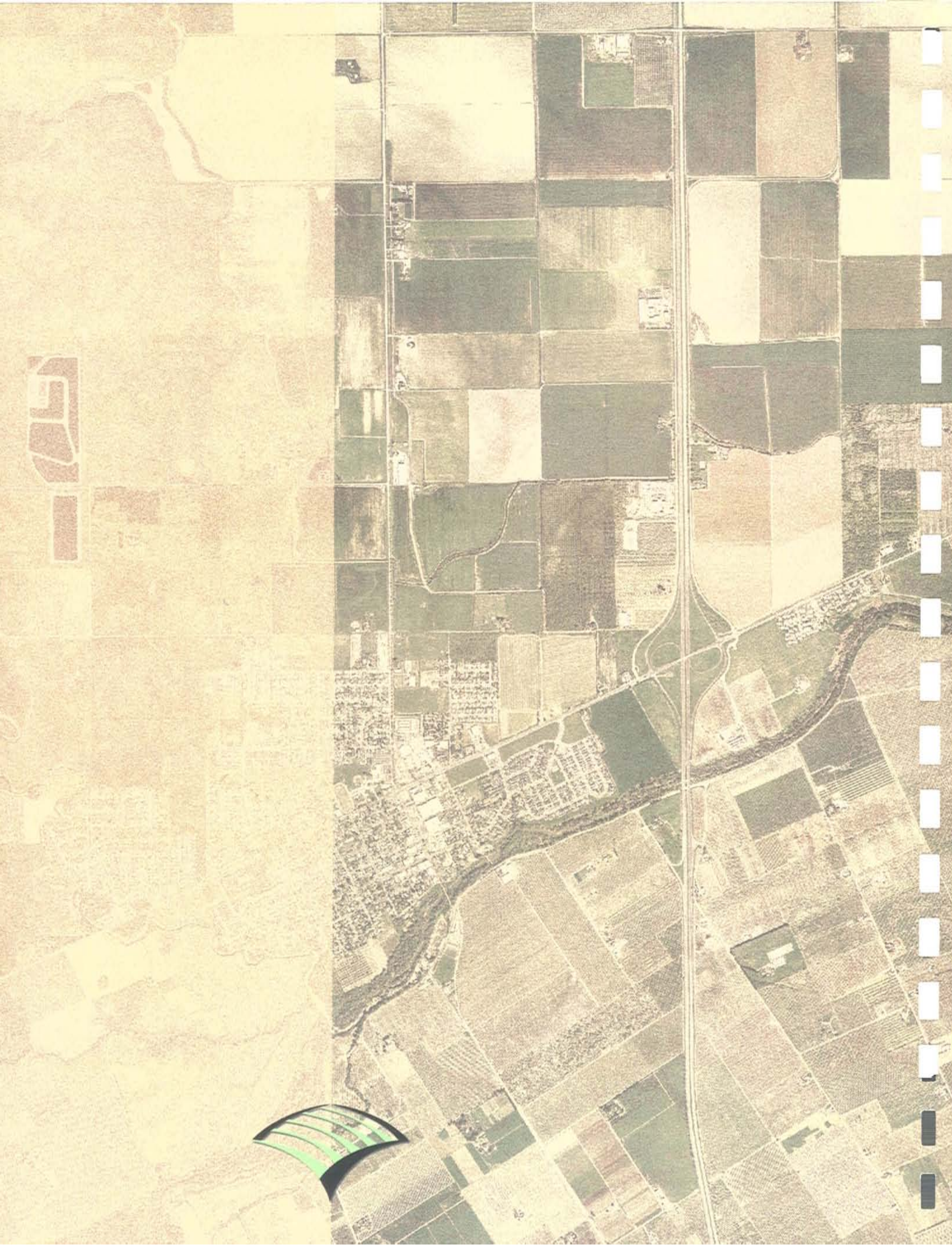
GENERAL PLAN AMENDMENT MAP
JUNE 2003
CITY OF WINTERS

MAP I

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Appendix D

Opinion of Probable Costs



CITY OF WINTERS
DRAINAGE REPORT - MOODY SLOUGH SUBBASIN

OPINION OF PROBABLE COSTS[1]
ULTIMATE CONDITIONS

			Description	Quantity	Unit	Unit Cost, $	Total Cost, $
1.	Putah Creek Diversion[2]						
	a.	Land Acquistion					
		·	Fee	10	ac	10,075.00	101,800
		·	Acquisition Allowance	1	ls	25%	25,450
	b.	Channel Construction					
		·	Excavate and Load Into Trucks	100,273	cy	1.78	178,600
		·	Haul and Dump Excess Material	100,273	cy	1.15	115,600
		·	Spread, Compact, and Shape Excess Material	100,273	cy	1.47	147,100
		·	Construct Patrol/Access Roadways	1,770	tn	15.19	26,900
		·	Construct Fencing on Both Sides of Channel	6,100	lf	16.30	99,400
	c.	Highway 128 Road Crossing (Five 5'x8' Box Culverts)					
		·	Excavate and Load Into Trucks	5,355	cy	1.78	9,500
		·	Haul and Dump Excess Material	1,190	cy	1.15	1,400
		·	Spread, Compact, and Shape Excess Material	1,190	cy	1.47	1,700
		·	Reinforced Concrete Structure	557	cy	592.01	329,700
		·	Structural Backfill	4,162	cy	10.48	43,600
		·	Pavement Replacement	833	sy	45.06	37,500
		·	Traffic Control	1	ls	52,390.00	52,400
	d.	Upstream End - Public Road Crossing (Five 5'x8' Box Culverts)					
		·	Excavate and Load Into Trucks	5,355	cy	1.78	9,500
		·	Haul and Dump Excess Material	1,190	cy	1.15	1,400
		·	Spread, Compact, and Shape Excess Material	1,190	cy	1.47	1,700
		·	Reinforced Concrete Structure	557	cy	592.01	329,700
		·	Structural Backfill	4,162	cy	10.48	43,600
		·	Pavement Replacement	833	sy	45.06	37,500
		·	Traffic Control	1	ls	52,390.00	52,400
	e.	Outfall Structure					
		·	Excavate and Stockpile/Load Into Trucks	780	cy	1.78	1,400
		·	Haul and Dump Excess Material	420	cy	1.15	500
		·	Spread, Compact, and Shape Excess Material	420	cy	1.47	600
		·	Reinforced Concrete Structure	219	cy	592.01	129,600
		·	Structural Backfill	360	cy	10.48	3,800
	Subtotal Putah Creek Diversion Improvements[2]						1,782,350
2.	Detention/Water Quality Pond #1						
	a.	Land Acquistion					
		·	Fee	29	ac	10,075.00	292,200
		·	Acquisition Allowance	1	ls	25%	73,050
	b.	Pond Construction					
		·	Excavate and Load Into Trucks	383,909	cy	1.78	683,800
		·	Haul and Dump Excess Material	383,909	cy	1.15	442,500
		·	Spread, Compact, and Shape Excess Material	383,909	cy	1.47	563,200
		·	Construct Perimeter Road	3,465	tn	15.19	52,600
	c.	Inlet Structure (Five 10'x5' Box Culverts)					
		·	Excavate and Load Into Trucks	2,585	cy	1.78	4,600
		·	Haul and Dump Excess Material	1,670	cy	1.15	1,900
		·	Spread, Compact, and Shape Excess Material	1,670	cy	1.47	2,400
		·	Reinforced Concrete Structure	605	cy	592.01	358,200
		·	Structural Backfill	915	cy	10.48	9,600
	d.	Outlet Control Structure					
		·	Obermeyer Control Gate	1	ls	249,500.00	249,500
		·	Obermeyer Control Gate Installation Cost	1	ls	15%	37,425
		·	Excavate and Load Into Trucks	1,186	cy	1.78	2,100
		·	Haul and Dump Excess Material	782	cy	1.15	900
		·	Spread, Compact, and Shape Excess Material	782	cy	1.47	1,100
		·	Reinforced Concrete Structure	263	cy	592.01	155,700
		·	Structural Backfill	404	cy	10.48	4,200
	Subtotal Detention/Water Quality Pond #1						2,934,975

CITY OF WINTERS
DRAINAGE REPORT - MOODY SLOUGH SUBBASIN

OPINION OF PROBABLE COSTS[1]
ULTIMATE CONDITIONS

			Description	Quantity	Unit	Unit Cost, $	Total Cost, $
3.	Detention/Water Quality Pond #2						
	a.	Land Acquistion					
		·	Fee	23	ac	10,075.00	231,700
		·	Acquisition Allowance	1	ls	25%	57,925
	b.	Pond Construction					
		·	Excavate and Load Into Trucks	388,503	cy	1.78	692,000
		·	Haul and Dump Excess Material	388,503	cy	1.15	447,800
		·	Spread, Compact, and Shape Excess Material	388,503	cy	1.47	569,900
		·	Construct Perimeter Road	2,228	tn	15.19	33,900
	c.	Outlet Control Weir Structure					
		·	Excavate and Load Into Trucks	200	cy	1.78	400
		·	Haul and Dump Excess Material	100	cy	1.15	100
		·	Spread, Compact, and Shape Excess Material	100	cy	1.47	100
		·	Reinforced Concrete Structure	50	cy	592.01	29,600
		·	Structural Backfill	100	cy	10.48	1,000
	d.	Road Crossing (Five 6'x10' Box Culverts)					
		·	Excavate and Load into Trucks	1,450	cy	1.78	2,600
		·	Haul and Dump Excess Material	800	cy	1.15	900
		·	Spread, Compact, and Shape Excess Material	800	cy	1.47	1,200
		·	Reinforced Concrete Structure	244	cy	592.01	144,400
		·	Structural Backfill	650	cy	10.48	6,800
		·	Pavement Replacement	500	sy	45.06	22,500
	Subtotal Detention/Water Quality Pond #2						2,242,825
4.	Detention/Water Quality Pond #3						
	a.	Land Acquistion					
		·	Fee	14	ac	10,075.00	141,100
		·	Acquisition Allowance	1	ls	25%	35,275
	b.	Pond Construction					
		·	Excavate and Load Into Trucks	234,238	cy	1.78	417,200
		·	Haul and Dump Excess Material	234,238	cy	1.15	270,000
		·	Spread, Compact, and Shape Excess Material	234,238	cy	1.47	343,600
		·	Construct Perimeter Road	1,604	tn	15.19	24,400
	c.	Road Crossing (Two 8'x10' Box Culverts)					
		·	Excavate and Load into Trucks	2,070	cy	1.78	3,700
		·	Haul and Dump Excess Material	350	cy	1.15	400
		·	Spread, Compact, and Shape Excess Material	350	cy	1.47	500
		·	Reinforced Concrete Structure	225	cy	592.01	133,200
		·	Structural Backfill	1,725	cy	10.48	18,100
		·	Pavement Replacement	500	sy	45.06	22,500
	d.	Inlet Culverts (Under Proposed Roadway)					
		·	24" Diameter (60' Length)	30	ea	4,337.89	130,100
	Open Channel Between Wetlands and Pond #3						
	a.	Land Acquistion					
		·	Fee	5	ac	10,075.00	51,500
		·	Acquisition Allowance	1	ls	25%	12,875
	b.	Channel Construction					
		·	Excavate and Load Into Trucks	47,435	cy	1.78	84,500
		·	Haul and Dump Excess Material	47,435	cy	1.15	54,700
		·	Spread, Compact, and Shape Excess Material	47,435	cy	1.47	69,600
		·	Construct Patrol/Access Roadways	1,608	tn	15.19	24,400
	Subtotal Detention/Water Quality Pond #3						1,837,650
5.	Water Quality Pond #4						
	a.	Land Acquistion					
		·	Fee	3	ac	10,075.00	26,200
		·	Acquisition Allowance	1	ls	25%	6,550

CITY OF WINTERS
DRAINAGE REPORT - MOODY SLOUGH SUBBASIN

OPINION OF PROBABLE COSTS[1]
ULTIMATE CONDITIONS

Description	Quantity	Unit	Unit Cost, $	Total Cost, $
b. Pond Construction				
· Excavate and Load Into Trucks	11,290	cy	1.78	20,100
· Haul and Dump Excess Material	11,290	cy	1.15	13,000
· Spread, Compact, and Shape Excess Material	11,290	cy	1.47	16,600
· Construct Perimeter Road	455	tn	15.19	6,900
c. Road Crossing (Two 5'x10' Box Culverts)				
· Excavate and Load into Trucks	560	cy	1.78	1,000
· Haul and Dump Excess Material	235	cy	1.15	300
· Spread, Compact, and Shape Excess Material	350	cy	1.47	500
· Reinforced Concrete Structure	115	cy	592.01	68,100
· Structural Backfill	325	cy	10.48	3,400
· Pavement Replacement	500	sy	45.06	22,500
Subtotal Water Quality Pond #4				185,150
6. Water Quality Pond #5				
a. Land Acquistion				
· Fee	2	ac	10,075.00	15,100
· Acquisition Allowance	1	ls	25%	3,775
b. Pond Construction				
· Excavate and Load Into Trucks	8,390	cy	1.78	14,900
· Haul and Dump Excess Material	8,390	cy	1.15	9,700
· Spread, Compact, and Shape Excess Material	8,390	cy	1.47	12,300
· Construct Perimeter Road	156	tn	15.19	2,400
c. 54" Diameter Siphon Pipeline				
· Excavate and Load Into Trucks	500	cy	1.78	900
· 54" Diameter Pipe	200	lf	314.34	62,900
· Spread, Compact, and Shape Excess Material	100	cy	1.47	100
· Reinforced Concrete Inlet and Outlet	30	cy	592.01	17,800
Subtotal Water Quality Pond #5				139,875
7. Open Channel Connecting Ponds 1 and 2				
a. Land Acquistion				
· Fee	2	ac	10,075.00	24,400
· Acquisition Allowance	1	ls	25%	6,100
b. Channel Construction				
· Excavate and Load Into Trucks	20,500	cy	1.78	36,500
· Haul and Dump Excess Material	20,500	cy	1.15	23,600
· Spread, Compact, and Shape Excess Material	20,500	cy	1.47	30,100
· Construct Patrol/Access Roadways	828	tn	15.19	12,600
d. Road Crossing (Five 6'x10' Box Culverts)				
· Excavate and Load Into Trucks	1,450	cy	1.78	2,600
· Haul and Dump Excess Material	800	cy	1.15	900
· Spread, Compact, and Shape Excess Material	800	cy	1.47	1,200
· Reinforced Concrete Structure	244	cy	592.01	144,400
· Structural Backfill	640	cy	10.48	6,700
· Pavement Replacement	500	sy	45.06	22,500
Subtotal Open Channel Connecting Ponds 1 and 2				311,600
8. Winters North Drain/Relocated Willow Canal				
a. Land Acquistion				
· Fee	27	ac	10,075.00	267,000
· Acquisition Allowance	1	ls	25%	66,750
b. Channel Construction				
· Excavate and Load Into Trucks	92,614	cy	1.78	165,000
· Haul and Dump Excess Material	92,614	cy	1.15	106,700
· Spread, Compact, and Shape Excess Material	45,935	cy	1.47	67,400
· Construct Patrol/Access Roadways	3,360	tn	15.19	51,000
· Fencing (Willow Canal Only)	3,500	lf	13.62	47,700
· Concrete Lining (Willow Canal Only)	2,550	lf	36.67	93,500
· Willow Canal Extension (54" Pipeline Under Proposed Roadway)	800	lf	314.34	251,500

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ULTIMATE CONDITIONS

Description	Quantity	Unit	Unit Cost, $	Total Cost, $
c. Pipeline Construction				
· Excavate and Load Into Trucks	4,282	cy	1.78	7,600
· Haul and Dump Excess Material	4,282	cy	1.15	4,900
· Spread, Compact, and Shape Excess Material	2,265	cy	1.47	3,300
· Willow Canal 54"Pipeline	2,580	lf	314.34	811,000
· Manholes - 72" Diameter	3	ea	2,923.36	8,800
d. County Road 89 Crossing (Four 8'x6' Box Culverts)				
· Excavate and Load Into Trucks	1,090	cy	1.78	1,900
· Haul and Dump Excess Material	450	cy	1.15	500
· Spread, Compact, and Shape Excess Material	450	cy	1.47	700
· Reinforced Concrete Structure	244	cy	592.01	144,400
· Structural Backfill	640	cy	10.48	6,700
· Pavement Replacement	267	sy	45.06	12,000
· Traffic Control	1	ls	20,956.00	21,000
e. Levee Improvements				
(1) Clear and Grub for Base				
· Stripping and Vegetation (6")	21,860	cy	0.84	18,300
· Subexcavation and Recompaction (Inspection Trench)	21,500	cy	2.83	60,800
(2) Fill for New Embankment				
· Haul and Dump On-Site Dry Material	0	cy	1.15	0
· Compact and Shape On-Site Fill Material	46,679	cy	6.00	280,300
f. Siphon/Spill Structure (WC Under Winters North Drain Near CR 89)				
· Excavate and Load Into Trucks	500	cy	1.78	900
· 54" Diameter Pipe	156	lf	314.34	49,000
· Spread, Compact, and Shape Excess Material	500	cy	1.47	700
· Reinforced Concrete Inlet and Outlet	50	cy	592.01	29,600
· 54" Slide Gate	1	ls	10,478.00	10,500
g. Siphon Structure (WC Pond #1 inlet box structure)				
· Excavate and Load Into Trucks	1,011	cy	1.78	1,800
· 54" Diameter Pipe	150	lf	314.34	47,200
· Spread, Compact, and Shape Excess Material	109	cy	1.47	200
· Reinforced Concrete Inlet and Outlet	50	cy	592.01	29,600
h. Siphon Structure (Under Proposed Roadway)				
· Excavate and Load Into Trucks	500	cy	1.78	900
· 54" Diameter Pipe	120	lf	314.34	37,700
· Spread, Compact, and Shape Excess Material	500	cy	1.47	700
· Reinforced Concrete Inlet and Outlet	30	cy	592.01	17,800
Subtotal Winters North Drain/Relocated Willow Canal				2,725,350
9. Winters North Drain Ultimate Levee				
a. Land Acquistion				
· Fee	2	ac	10,075.00	22,200
· Acquisition Allowance	1	ls	25%	5,550
b. Flood Barrier at Frontage Road				
· Reinforced Concrete Structure	35	cy	592.01	20,700
· Structural Backfill	16	cy	10.48	200
· Pavement Replacement	100	sy	45.06	4,500
c. Levee Improvements				
(1) Clear and Grub for Base				
· Stripping and Vegetation (6")	741	cy	0.84	600
· Subexcavation and Recompaction (Inspection Trench)	1,972	cy	2.83	5,600
(2) Fill for New Embankment				
· Haul and Dump On-Site Dry Material	6,195	cy	1.15	7,100
· Compact and Shape On-Site Fill Material	6,195	cy	6.00	37,200
Subtotal Winters North Drain Ultimate Levee				103,650

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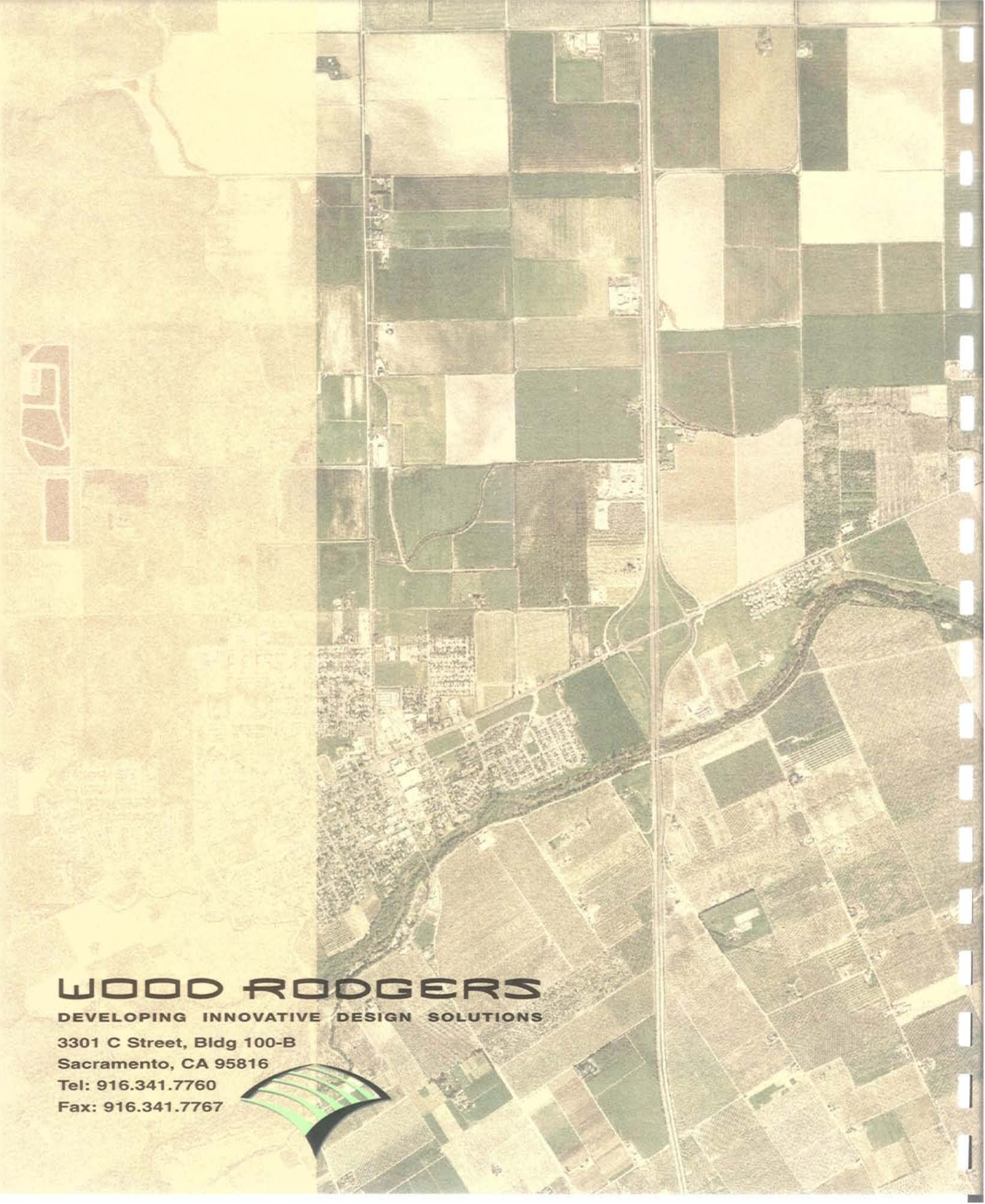
OPINION OF PROBABLE COSTS[1]
ULTIMATE CONDITIONS

Description	Quantity	Unit	Unit Cost, $	Total Cost, $
10. I-505 Floodwall				
a. Land Acquistion				
· Fee	2	ac	10,075.00	16,100
· Acquisition Allowance	1	ls	25%	4,025
b. Pond Construction				
· Excavate and Load Into Trucks	7,845	cy	1.78	14,000
· Haul and Dump Excess Material	1,162	cy	1.15	1,300
· Spread, Compact, and Shape Excess Material	1,162	cy	1.47	1,700
· Structural Backfill	6,683	cy	10.48	70,000
· Reinforced Concrete Wall	895	cy	592.01	529,800
Subtotal I-505 Floodwall				636,925
Subtotal Ultimate Drainage Improvements (Includes Land Acquisition)				12,900,350
Land Acquisition Costs[3]				1,486,625
Subtotal Ultimate Drainage Improvements (Does Not Include Land Acquisition)				11,413,725
Contingencies (25%)				2,853,431
Administration and Engineering (35%)				3,994,804
TOTAL ULTIMATE FACILITIES COST (Includes Land Acquisition Costs)				19,748,585

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

[2]Putah Creek Diversion Improvements are shared by land outside of 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.

[3]Land acquisition cost does not include runoff corridor acquisition. It is assumed either existing rights-of-way or easements are in place or that land will be dedicated.



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