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Nipomo Waterline Intertie Project

CONCEPT DESIGN REPORT

Volume 1 of 3

April 2009

Nipomo Waterline Intertie Project

Concept Design Report
Volume 1 of 3

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TABLE OF CONTENTS

VOLUME I – CONCEPT DESIGN REPORT

EXECUTIVE SUMMARY

Introduction and Background

Currently, the Nipomo Community Services District (District) relies on groundwater as the sole source of water for their customers, approximately 12,000 people (Urban Water Management Plan 2005 Update, SAIC). The groundwater is pumped from the Nipomo Mesa Management Area (NMMA) of the Santa Maria Groundwater Basin, an aquifer that has been the subject of ongoing litigation since 1997. Due to the diminishing source and competing claims, the California State Superior Court of Santa Clara County approved a Settlement Stipulation on August 3, 2005 containing a requirement that the District import a minimum of 2,500 acre-feet of supplemental water to the NMMA each year.

The District has investigated multiple sources of supplemental water and, as a result, signed a Memo of Understanding (MOU) with the City of Santa Maria (City) to pursue the Waterline Intertie Project. The 2005 MOU established a basis for purchase and delivery of water from the City to the District. Subsequently, the District commissioned a preliminary design study. After the Draft Waterline Intertie Project Preliminary Engineering Memorandum (Boyle, November 2006), the District Board of Directors requested additional study to establish a basis for comparison to other supplemental water alternatives. Boyle Engineering (now AECOM) submitted the Evaluation of Supplemental Water Alternatives in June 2007 which investigated the costs and constraints associated with several alternative water supplies. The evaluation indicated the preferred supplemental water sources are the Santa Maria Waterline Intertie and desalination, which met the criteria for reliability, quality, and availability. The revised Waterline Intertie Project Preliminary Engineering Memorandum was submitted in May 2008 (Boyle/AECOM).

The Waterline Intertie Project summarized in this Concept Design Report consists of the “Phase I” and “Phase II” projects described in the Waterline Intertie Project Environmental Impact Report (Douglas Wood and Associates, April 22, 2009). The project is designed to deliver 3,000 AFY at 2,000 gpm. Water delivery will be phased based on system demands and conditions to be established in the contract with the City. The water delivery rate is anticipated to be constant and to be adjusted by the District daily. District wells are to be used during peak demand months and for emergency water if the Project is out of service.

Project Components

The Waterline Intertie Project consists of over 27,000 linear feet (LF) of pipeline, a 0.5 million gallon (MG) storage tank, a 2,000 gallon per minute (gpm) pump station, and chloramination systems at the pump station and at four wells, as well as the related back-up power, controls, and electrical instrumentation. Volume 1 of this report presents the technical findings and design considerations for the project. Volume 2 consists of the appendices with background information, studies, and a preliminary outline of technical specifications for project construction documents. The 30% design plans are included as Volume 3 of this Report.

Figure ES-1 displays a summary of the transmission facilities. The Intertie design begins at the north end of the City of Santa Maria water distribution system at the intersection of Blosser Road and West Taylor Street with a new 18-inch waterline. The waterline runs north along Blosser Road to Atlantic Place, crossing underneath the Santa Maria River levee and transitioning to a 24-inch waterline. The 24-inch line will be jacked and bored underneath the levee and will cross under the Santa Maria River utilizing horizontal directional drilling, ending atop the Nipomo Mesa.

On the Nipomo Mesa, the 24-inch piping will connect to a 500,000-gallon, pre-stressed concrete reservoir. The reservoir will be mostly buried to assist the delivery of water via City system pressures (without pumping). Vertical turbine pumps will draw water from the reservoir and deliver it to an existing 12-inch waterline on Santa Maria Vista Way at 2,000 gallons per minute (gpm). Traveling northwest, to Joshua Street and Orchard Road, water will be pumped along Orchard Road (in the existing 12-inch waterline) and connect to the main District system at Orchard Road and Southland Street. The preliminary design for the pump station and reservoir is discussed in Chapter 5 and is included as part of Bid Package 4.

Dedicated 12-inch waterlines will be installed to deliver water to the system's back-bone transmission mains in order to protect smaller existing waterlines and users from high pressures. These dedicated mains will be in five areas: 1) along Orchard Road, from Southland Street to Grande Street; 2) along Southland Street, from Orchard Road to Frontage Road; 3) along Frontage Road from Southland Street to Grande Street; 4) from Grande Street, northeast underneath Highway 101 (via jack-and-bore) to Darby Lane, continuing on Darby Lane to South Oakglen Avenue; and 5) along South Oakglen Avenue from Darby Lane to Tefft Street. The dedicated mains will connect to the existing system at Orchard Road and Grande Street, Frontage Road and Grande Street, and South Oakglen Avenue and Tefft Street.

Pressure-reducing-valve (PRV) stations will protect downstream users from high pressures required for the supplemental water delivery. Five PRV stations will be installed throughout the District's system. One will be placed on Santa Maria Vista Way near the connection to the existing 12-inch waterline, lowering pressure for the Maria Vista Development. Three stations will be placed downstream of the connection points, which will create a separate pressure zone in the southwest region of the District's system. The fifth PRV station will be installed on Southland Street between the dedicated main and an existing waterline to assist the flow of water into the new pressure zone during an emergency (low pressure) situation. The pipeline alignment, materials, and design considerations are further discussed in Chapter 4 and the proposed pipeline alignments are included with Bid Package 2.

The project also includes conversion of four production wells from chlorination to chloramination systems (Figure ES-2). The Preliminary Engineering Memorandum (Boyle/AECOM, May 2008) contains a detailed discussion of disinfection and water quality issues. Disinfection alternatives, as discussed in the Section 4 of the Memorandum, consists of uncontrolled blending of City and District water without changes in treatment process, converting City water disinfection to free chlorine residual, and converting NCSD groundwater disinfection to chloramine residual. The Memorandum recommends converting NCSD groundwater disinfection to chloramine at the main wellheads and including a chloramine booster at the pump station. Chapter 6 describes the preliminary design of the wellhead chloramination systems and the chloramine booster station is discussed in Chapter 5.

Project Bidding Strategy

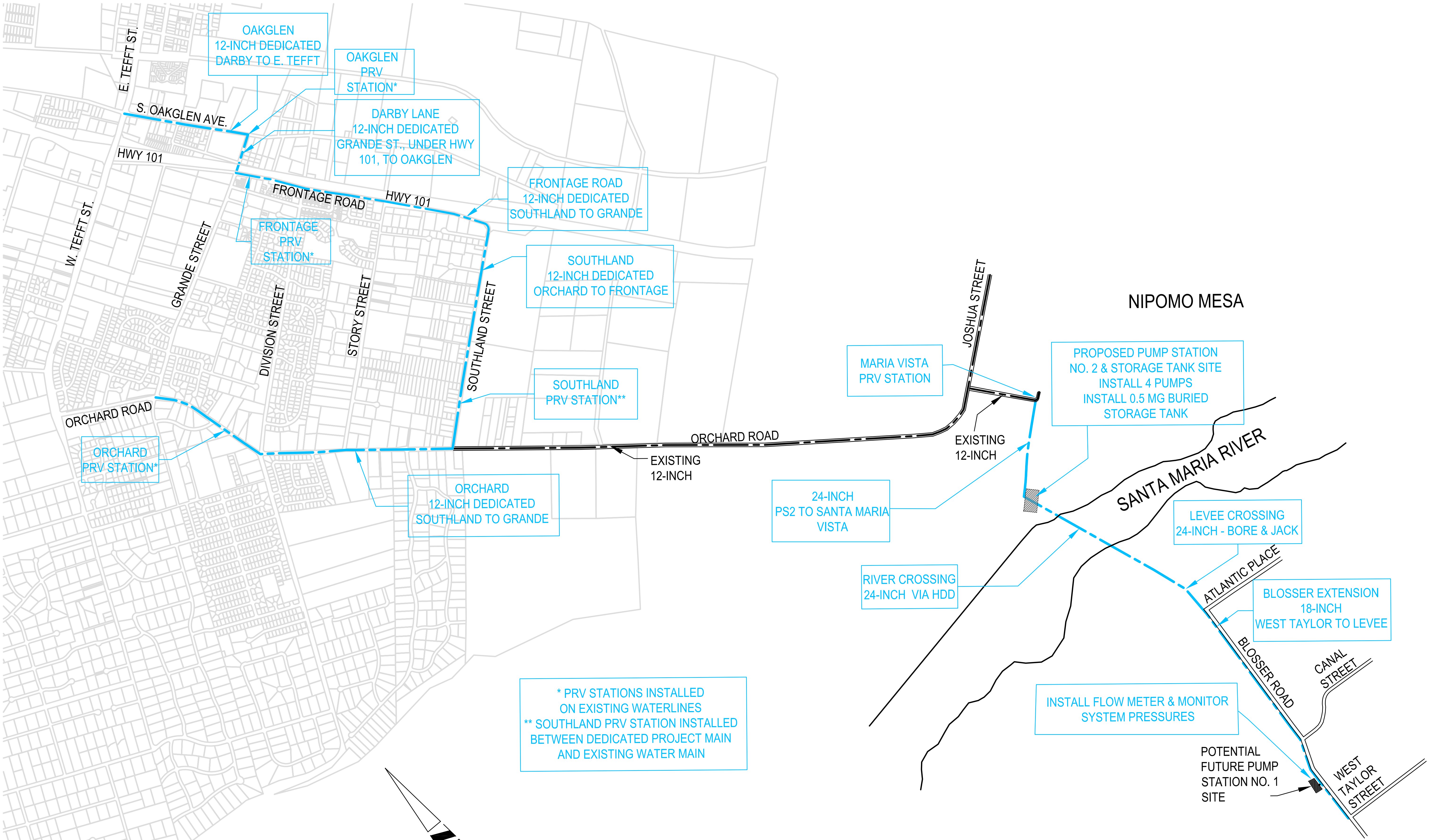
Technical Memorandum No. 2 summarizes recommendations for bidding the Waterline Intertie Project, including recommendations for multiple bid packages and optimizing the bid climate through press releases, workshops, and timing of the bid release.

Project components were grouped into bid packages based on proximity of work items to each other, unique equipment and experience required for performance of the river crossing, need to provide as few points of coordination and responsibility as possible for each project site, and desire to standardize new chloramination systems at each wellhead. Based on these criteria, AECOM recommends the project be divided into four bid packages as follows:

- Bid Package 1: Santa Maria River Water Main Crossing
- Bid Package 2: Nipomo Area Pipeline Improvements
- Bid Package 3: Blosser Road Water Main and Flow Meter

- Bid Package 4: Joshua Road Pump Station and Reservoir, and Wellhead Chloramination Improvements

AECOM recommends prequalifying only the river crossing contractors. Chapter 2 further describes the work items included in each bid package and other bidding strategies.



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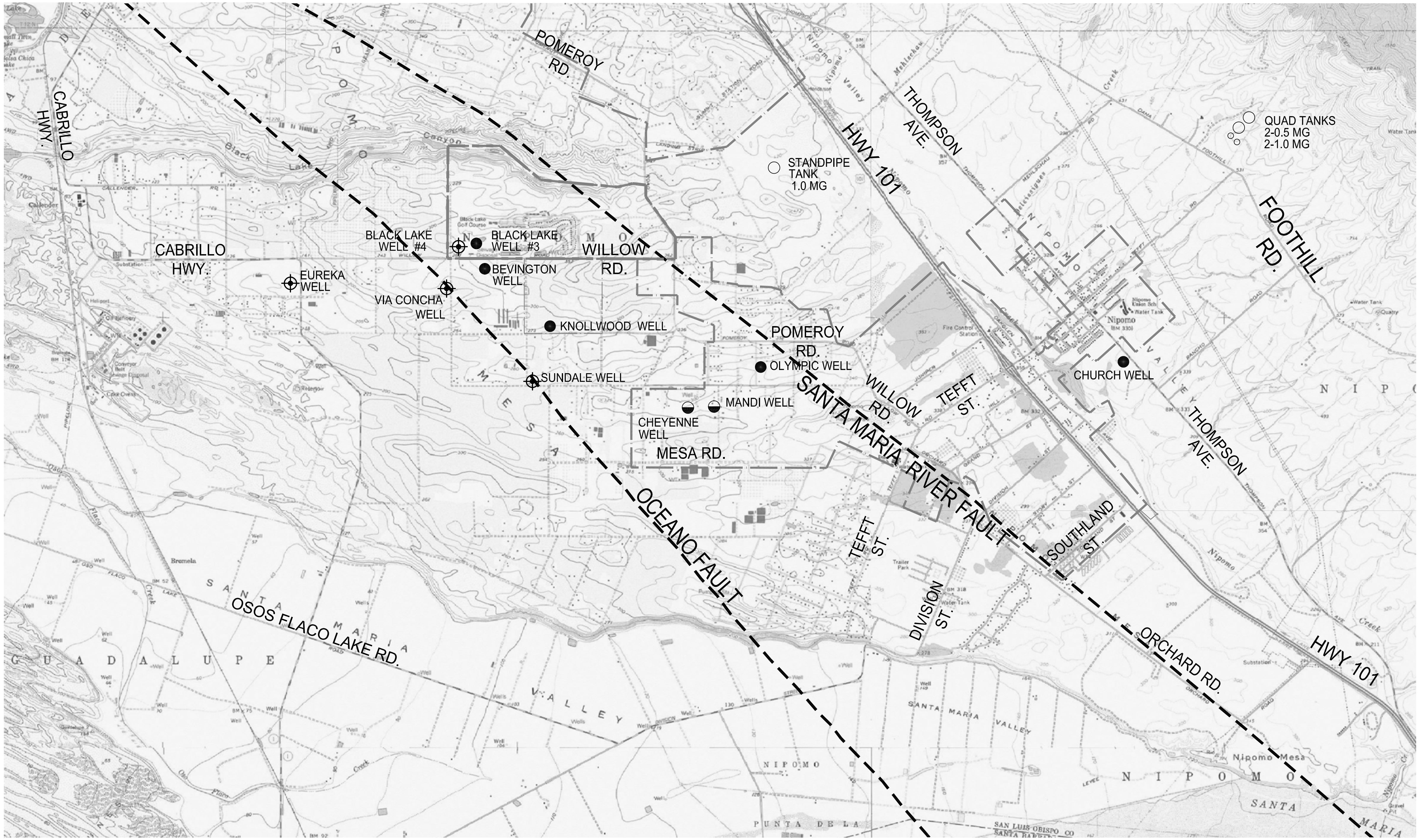
NCSD WATERLINE INTERTIE

PROJECT COMPONENTS

AECOM PROJECT NO. 19996.70

FIGURE ES-1



LEGEND

- NIPOMO CSD WELLS
- NIPOMO CSD WELLS (FUTURE)
- NIPOMO CSD TANKS
- EXISTING WATER SYSTEM SERVICE AREA BOUNDARY
- APPROXIMATE FAULT LINE
- WELLS SELECTED FOR CHLORAMINE CONVERSION

4000 0 4000 8000
SCALE IN FEET

AECOM	NCSD WATERLINE INTERTIE	BEC PROJECT NO.	FIGURE
AECOM USA, Inc. 7807 Convoy Court, Suite 200 San Diego, California 92111 T 858.268.8080 F 858.292.7432 www.aecom.com	PROJECT COMPONENTS	19996.70	ES-2

Opinion of Probable Construction Cost

The detailed opinion of probable construction costs are presented in Chapter 8. Table ES-1 is a summary.

Table ES-1. Opinion of Probable Project Costs

Item	Description	Budgeted Amount May 2008 Preliminary Engineering Memo.		Updated Amount 22-Apr-09 Concept Design Report	
	Construction Subtotal	$13,860,800		$15,577,000	
1	Contingency	$3,643,000		$3,115,400	(5)
	Construction Subtotal + Contingency	**$17,503,800**		**$18,692,400**	
2	Property Allowance	*not included*	(3)	$500,000	(3)
3	Design-Phase Engineering				
	Original Agreement (July 2008)			$744,993	
	Budget Revision 1 - Pressure Reduction			$132,798	
	Budget Revision 2 - Biological Survey for HDD			$4,050	
	Budget Revision 3 - Modeling for GSW/Woodlands Turnouts			$8,380	
	Budget Revision 4 - Additional Survey Services			$9,900	
4	Office Engineering during construction			$175,837	
5	Estimated Construction Management (2)	$2,428,000	(1)	$1,507,170	(4)
6	Permitting Fees To Date	--		$1,573	
7	Non-Final Design Funds Spent To Date	*not included*		$1,402,879	(6)
8	Estimated Other Costs (Assessment, etc)	*not included*		$415,420	(6)
	PROJECT TOTAL (Rounded to 1000)	**$19,932,000**		**$23,596,000**	

Notes: ENR CCI: March 2008 = 8109; March 2009 = 8534

(1) Engineering and Construction Management were originally presented as a "lump sum" amount

(2) Includes material testing, construction staking, and environmental monitoring

(3) Estimate only. Item not included in previous construction cost opinions, but was added to the Concept Design Report to provide a complete assessment of anticipated project costs.

(4) To be provided by CM team - Has not been revised to reflect additional work for construction management of Oakglen, Darby, and Orchard extensions.

(5) Contingency was modified to 20% which is more appropriate for 30% design phase.

(6) Provided by District staff.

not included = Item was not included in previous construction cost opinions, but was added into the Concept Design Report to provide a complete assessment of anticipated project costs.

1.0 INTRODUCTION

The Nipomo Community Services District (NCSD) serves approximately 12,000 people over an area of approximately 4,650 acres (Urban Water Management Plan 2005 Update, SAIC Engineering). The service area is currently served by groundwater from the Nipomo Mesa Management Area (NMMA) of the Santa Maria Groundwater Basin. The NMMA is at the northwestern part of the basin, and encompasses approximately 27.5 square miles.

1.1 Project Background

The 2005 Urban Water Management Plan states that "since July 1997, the Santa Maria Groundwater Basin has been the subject of ongoing litigation between nearly 800 parties with competing claims to pump groundwater, collectively called the Santa Maria Groundwater Litigation (Santa Maria Valley Water Conservation District vs. City of Santa Maria, et al. Case No. 770214)".

The California Superior Court, County of Santa Clara, approved a Settlement Stipulation on August 3, 2005, which requires that the District import a minimum of 2,500 acre-feet of Supplemental Water to the NMMA each year. The need for additional source(s) of water has been well-established; and a history of investigations has led the District to pursue imported water from the City of Santa Maria (City) through the Waterline Intertie Project. Some of these studies are summarized in the following documents:

- Preliminary Engineering Memorandum (Boyle/AECOM, May 2008)
- Supplemental Water Evaluation (Boyle, June 2007)
- Supplemental Water Alternatives Environmental and Permitting Constraints Analysis (Padre Associates, May 2007)
- Urban Water Management Plan 2005 Update (SAIC, January 2006)
- Santa Maria Inter-Tie: Project Schedule and Probable Cost (Cannon Associates, June 2005)
- Waterline Feasibility Study: Santa Maria River Crossing Alternatives (Cannon Associates, April 2005)
- Santa Maria Inter-Tie: Route and Site Alternatives (Cannon Associates, June 2005)
- Resource Capacity Study, Water Supply in the Nipomo Mesa Area (S.S. Papadopoulos & Associates, November 2004)
- Final Report, Evaluation of Water Supply Alternatives (Kennedy/Jenks Consulting, October 2001)
- Evaluation of Alternative Supplemental Water Supplies (Bookman-Edmonston Engineering, Inc, July 1994)
- Engineering Considerations of Groundwater Yields and Rights on the Nipomo Mesa Sub-Area, San Luis Obispo County, CA (Lawrence, Fisk, & McFarland, Inc, October 1993)

The 2005 Memorandum of Understanding (MOU) between the District and the City of Santa Maria became the basis for the Waterline Intertie Project. According to paragraph 2.10 of the MOU, "NCSD shall be responsible for constructing and operating an interconnection with the City's retail distribution system". Location, plans, and specifications of this connection must be approved by the City. All costs for regulatory and environmental permits, licenses, and other approvals must be paid by NCSD.

The water is intended to be the City's "municipal mix", including both City groundwater and State Water Project supplies.

The District and the City of Santa Maria are currently negotiating the terms of the water supply agreement.

In November 2006, Boyle Engineering (now AECOM) completed the Draft Waterline Intertie Preliminary Engineering Memorandum. Two general project alignments were evaluated for crossing the Santa Maria River: one near Highway 101 and one due north from the north end of Blosser Road in Santa Maria. The report also included a preliminary hydraulic analysis, water quality analysis, disinfection study, review of reservoir storage options, siting evaluation for two pump stations, environmental and permitting considerations, and a conceptual cost comparison. The hydraulic analysis was a preliminary effort and intended to be merged with the updated water model being produced as part of the District's Water Master Plan.

After the draft Memorandum was released, Carollo Engineers completed a hydraulic analysis of the City of Santa Maria's distribution system in order to evaluate impact of various delivery rates on the City's system pressures. Two connection points were compared: at Atlantic Place and Blosser Road, existing 10-inch pipelines in a residential area, and at the end of a theoretical dedicated 18-inch pipeline from Taylor Street to the south side of the Santa Maria River. Although analyses showed that connection to the existing network did not create an unacceptable pressure drop in the residential area for a continuous supply to the NCSD, the report concluded that this connection point should not be considered because other variables, such as effects of a pump station or a flow control facility, may cause the results to change. The dedicated 18-inch pipeline was recommended for water delivery to the NCSD.

After the Preliminary Engineering Memorandum for the Waterline Intertie Project was completed, the District Board of Directors commissioned an evaluation of supplemental water alternatives with which to form a basis of comparison. In order to determine the cost and constraints associated with other water supply alternatives, and provide context for evaluating cost and constraints of the Waterline Intertie Project, in June of 2007 Boyle evaluated options for supplemental water supplies. The alternative sources included:

- Santa Maria Waterline Intertie
- Santa Maria groundwater
- Desalination
- Surface water from Oso Flaco Lake
- State water from a regional interconnection
- Nacimiento pipeline
- Recycled wastewater recharge and/or reuse

The evaluation indicated the preferred supplemental water sources are the Santa Maria Waterline Intertie and desalination, which meet the criteria for availability, quality, and reliability. These sources also have an added advantage of lower salts concentrations (such as sulfates, boron, chloride, and total dissolved solids) than the Nipomo groundwater. The potential to increase salt concentrations in the groundwater through disposal of treated wastewater is a current concern for the District. This has become a more pressing issue as the Southland Wastewater Treatment Facility approaches its permitted effluent limit for maximum flow. The District plans to prepare a salts management program to reduce this potential for impact. A water supply with lower salts concentration is an important component of this program, as it will help mitigate future impact to the groundwater.

The District was interested in purchasing State Water, but significant constraints were identified in the report and were further explored in meetings with staff from the District, San Luis Obispo County Public Works Department, and City of Santa Maria.

In May 2008, the Waterline Intertie Project Preliminary Engineering Memorandum was finalized. The Waterline Intertie Project Preliminary Engineering Memorandum summarized the preliminary hydraulic analysis for the District's and the City's distribution systems and the intertie pipeline; examined water quality and disinfection alternatives; and evaluated pipeline alignment, storage, and pumping options. Three main pipeline alignments were compared based on apparent constructability, potential environmental impact, easements required, existing utilities, intertie pipeline length, cost, and geotechnical considerations. The Board of Directors selected a final alignment based primarily on environmental issues and river crossing challenges associated with the eastern alignment. Refer to Figure 1-1 for the selected alignment. The project consisted of the components summarized in Table 1-1.

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NCSD WATERLINE INTERTIE PROJECT

PROJECT ALIGNMENT -
SANTA MARIA TO NIPOMO MESA

AECOM PROJECT NO.
19996.70

FIGURE
1-1

DWG: W:\Nipomo CSD (19996)\19996.70 Waterline Intertie Project Design\Phase 0001 – Concept Design Report\Task 016 – Concept Design Report\Final Concept Design Report\Figures\Figure 1-1.dwg Layout Name: Figure 1-1 – Plotted by: Froelicher, Jim Date: 5/4/2009 – 9:37 AM
XREFS: CANNON BASE MAP IMAGES: img20600389 Aerial Photo-LITE-SMALL.jpg – img20600389 Aerial Photo-LITE.jpg –

Table 1-1. Waterline Intertie Project from Preliminary Engineering Memorandum

Component	Description
Blosser Road Water Main	– 5,000 lineal feet (lf) of 18" water main, valves, and appurtenances from West Taylor Street to Atlantic Place – Flow meter
Santa Maria River Crossing	3,700 lf of 24" water main from the north end of Blosser Road to the Horizontal Directional Drill (HDD) staging area, including: – 300 lf of bore-and-jack crossing underneath the south levee – 900 lf of open trench to the south HDD staging area – 2,500 lf of water main from the south HDD staging area across the river to the north HDD staging area
Nipomo System Pipeline Improvements	– 2,500 lf from the north side of the river (at the north HDD staging area) to the pump station site near Joshua Street – 3,200 lf of 12" main along Orchard Avenue between Southland Street and Division Street – 3,900 lf of 12" main along Southland Street between Orchard Road and Frontage Road – 6,470 lf of 12" main along Frontage Road from Southland Street to Tefft Street – 340 lf of 12" main along Division Street between Allegre Road and Meridian Road – Approximately 150 pressure regulating valves for water services (to protect homeowners from higher pressures due to the new booster station) – Pressure reducing valve station on Joshua Street between the pump station and the Maria Vista development
Booster Pump Station and Reservoir	– 1,830 to 2,000 gallon per minute (gpm) booster pumping station – Chloramination system – 500,000 gallon reservoir
Wellhead Chloramination System	Conversion of four production wells from chlorination to chloramination systems

One of the concerns with the project is an increase in distribution system pressure for existing customers near the Waterline Intertie Project connection on Joshua Road (see Figure 1-2 [Area A]). Many of these customers currently experience pressures in the 90- to 100-psi range and would experience pressures around 110 psi without pressure reducers. The Preliminary Engineering Memorandum recommended installation of 150 pressure regulating valves as shown Table 1-1. This was considered a low cost approach to protecting homeowners from higher pressure.

In Technical Memorandum 9 (included as Appendix A), AECOM developed an alternative to installation of regulating valves, as directed by the District. This alternative included additional pressure reducing valve stations, isolation valves, and dedicated pipelines (instead of pipeline replacements, as originally described in the Preliminary Engineering Memorandum) to create a pressure zone (see Figure 1-3 [Area B]) that would not be affected by the booster station.

Figure 1-2: NCSD System Improvements from May 2008 Preliminary Engineering Memorandum

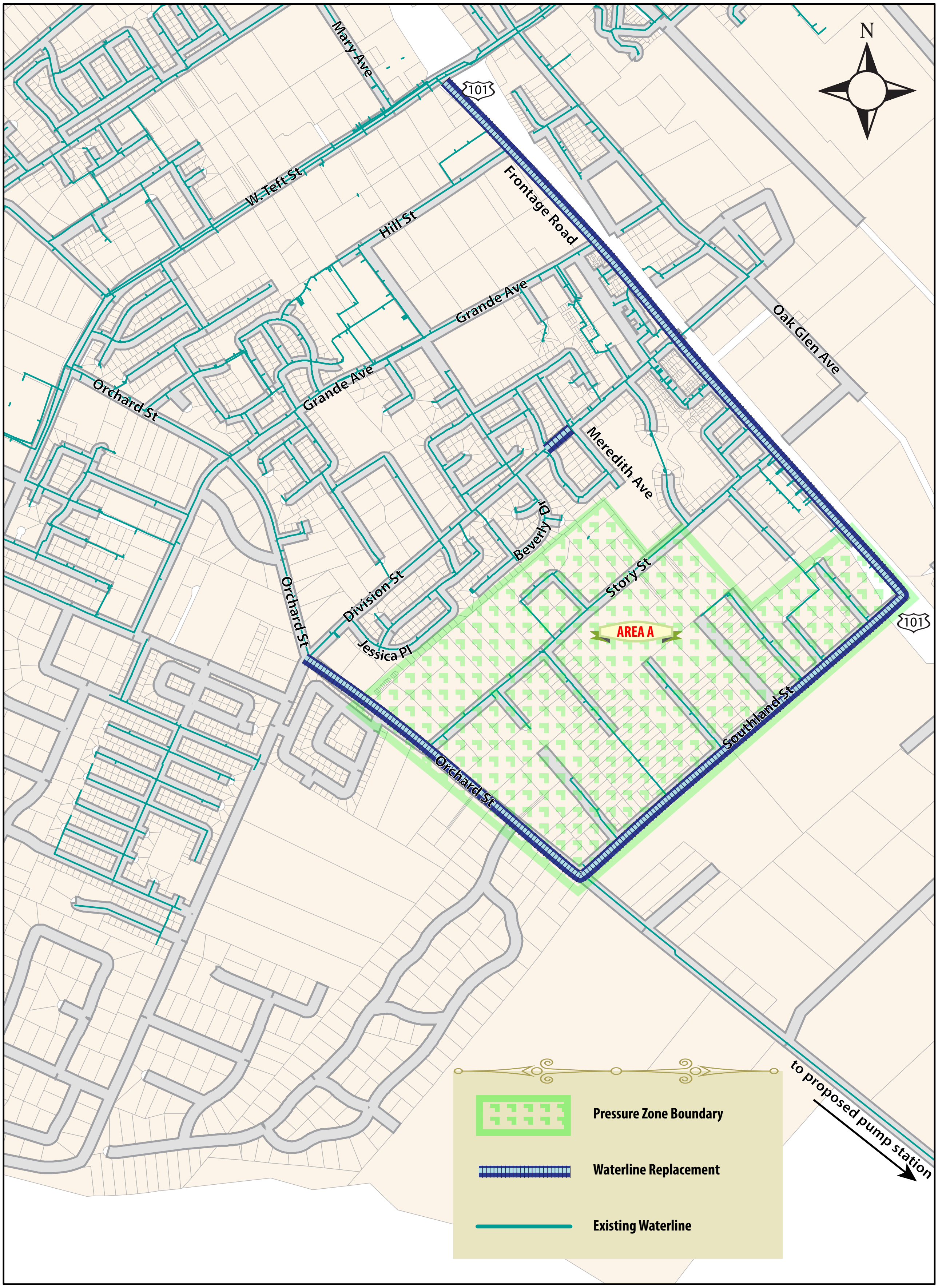
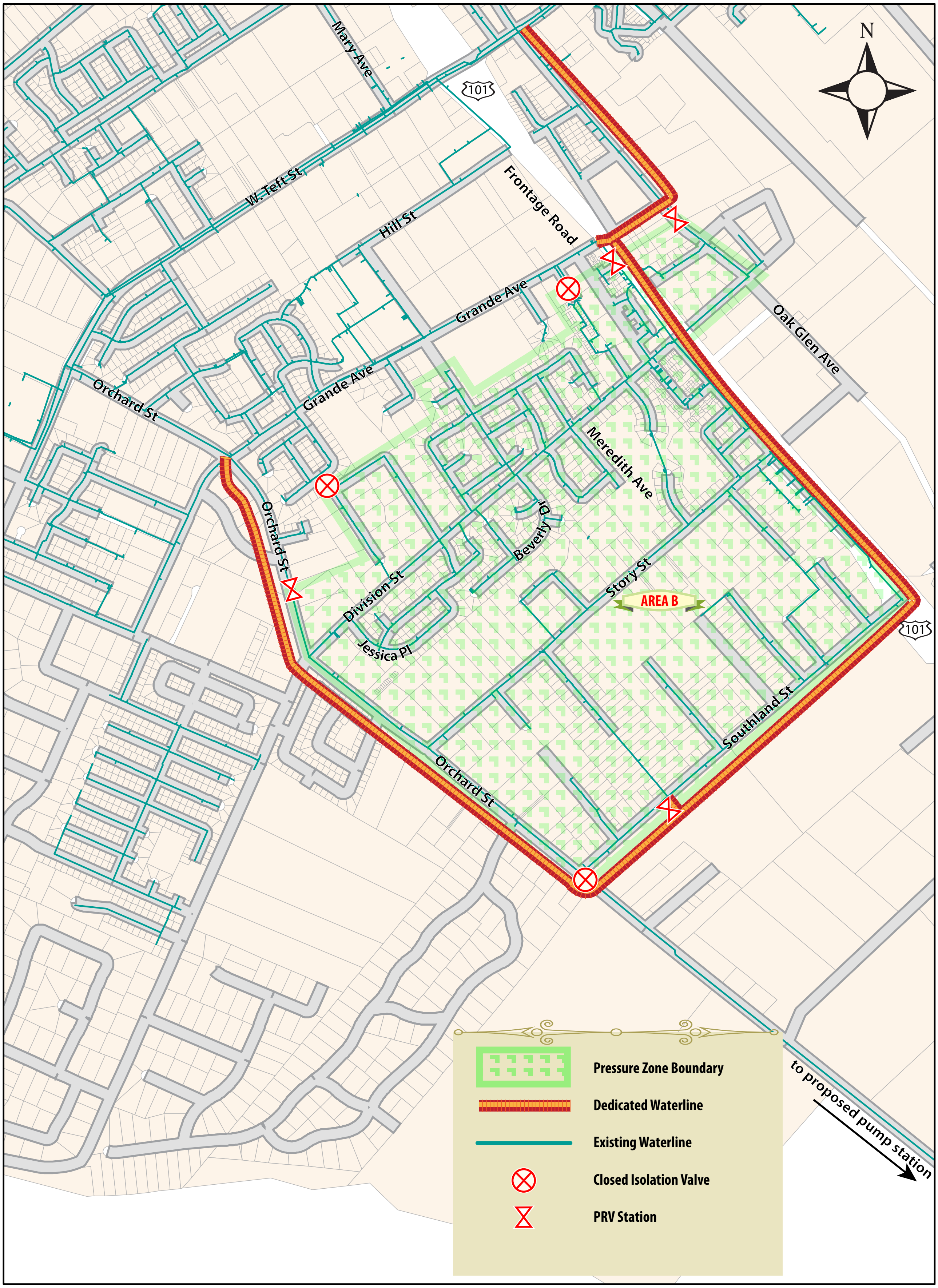


Figure 1-3: Selected NCSD System Improvements



Creating the low pressure zone and utilizing dedicated pipelines provide a project flow rate capacity of 2,000 gpm, in effect combining the "Phases I and II" as described in the Waterline Intertie Project Environmental Impact Report (EIR) (Douglas Wood and Associates, April 22, 2009). In addition, the pipeline segment along Division was not required because the current size is 10-inch, not 6-inch as shown in the District's Water and Sewer Master Plan Update. The recommendations for project components are discussed in Section 1.2.

1.2 Project Components

The Waterline Intertie Project is designed to deliver 3,000 AFY at a rate of 2,000 gpm. The project consists of a combined "Phase I" and "Phase II" components as described in the project EIR and as summarized in Table 1-2. In order to reduce the potential for future reconstruction underneath the levee and across the Santa Maria River, the River and levee crossing pipelines are designed to handle up to 6,300 AFY at a flow rate of 5,570 gpm.

Table 1-2. Revised Waterline Intertie Project

Component	Description
Blosser Road Water Main	5,000 lineal feet (lf) of 18" water main, valves, and appurtenances from West Taylor Street to Atlantic Place Flow meter
Santa Maria River Crossing	3,250 lf of 24" water main from the north end of Blosser Road to the Horizontal Directional Drill (HDD) staging area, including: – 300 lf of bore-and-jack crossing underneath the south levee – 900 lf of open trench to the south HDD staging area – 2,050 lf of water main from the south HDD staging area across the river to the north HDD staging area
Nipomo System Pipeline Improvements	– 2,500 lf from the north side of the river (at the north HDD staging area) to the pump station site near Joshua Street – 5,200 lf of dedicated main along Orchard Road between Southland Street and Grande Street – 3,900 lf of 12" dedicated main along Southland Street between Orchard Road and Frontage Road – 4,400 lf of 12" dedicated main along Frontage Road from Southland Street to Grande Street – 220 lf of 12" dedicated main with bore and jack crossing at Highway 101 from Grande Street to Darby Lane – 500 lf of 12" main along Darby Lane to South Oakglen Avenue – 2,100 lf of 12" main along South Oakglen Avenue from Darby Lane to Tefft Street – Five Pressure Reducing Valve Stations • Southland Street between Drumm Lane and Honeygrove • Orchard Road between Division Street and Apricot Lane • Frontage Road south of Grande Street • South Oakglen Avenue, south of Darby Lane • Between the pump station and the Maria Vista development
Booster Pump Station and Reservoir	2,000 gallon per minute (gpm) booster pumping station Chloramination system 500,000 gallon reservoir
Wellhead Chloramination System	Conversion of four production wells from chlorination to chloramination systems

Water delivery will remain constant, but may be adjusted daily by the District. Water delivery will be phased as demands increase per the agreement between the City and the District. District wells will be utilized for peak demand months and for emergency water delivery if the waterline intertie is out of service.

1.3 Scope of Work

In July of 2008, the District contracted with AECOM to perform the design work for the Waterline Intertie Project as summarized in Table 1-2. To present the preliminary design decisions and identify potential design challenges, the scope of work includes a Concept Design Report to be submitted with the 30% Plans. The 30% Plans are included as Volume 3 of this Report. (Volume 2 consists of the Appendices). An outline of technical specifications to be included with the Contract Documents is included as Appendix F.

This Concept Design Report is submitted in accordance with Task 113 under the project scope of work. The key components of the project design were evaluated and presented in a series of technical memoranda. The memoranda were used to compile this Draft Concept Design Report, incorporating comments received from the District staff. Descriptions and status of memoranda are summarized in Table 1-3. The current project schedule is attached as Appendix B.

Table 1-3. Technical Memoranda

Tech Memo No.	Title	Description	Submittal Status	Location in Concept Design Report
1	Geotechnical Report for HDD	A specific feasibility study of HDD for the River Crossing. Detailed geotechnical evaluation along the proposed River crossing (Fugro). Evaluation of soil requirements and design details of HDD, recommendations regarding feasibility and direction for the project (AECOM/Jacobs).	Final Submitted 3/25/09	Contained in Chapter 3
2	Project Bidding Strategy	Overview of recommendations for bidding the project, including recommendations for multiple bid packages and for optimizing the bid climate through press releases, workshops, and timing of bid release.	Final Submitted 11/19/08	Contained in Chapter 2
3	Pipeline Alignment	Preliminary pipeline design, utility research, and identification of locations and area requirements for easement acquisition.	Draft Submitted 3/3/09 Errata Letter Submitted 3/11/09	Contained in Chapter 4

Tech Memo No.	Title	Description	Submittal Status	Location in Concept Design Report
4	Pump Station and Reservoir Design	Preliminary Pump Station & Reservoir design. Building material & foundation, reservoir footprint & dimensions, site layout, pump control points, pump & motor configuration, controls/ instrumentation & power requirements, pump station layout, preliminary cost opinions.	Draft Submitted 3/20/09	Contained in Chapter 5
5	Reservoir Design	Combined with Tech Memo #4. See above.	See above	See above
6	Permitting Strategy	Strategy for obtaining required permits, recommended environmental monitoring/studies prior to and during construction, and recommended environmental mitigation measures	Draft Submitted 03/27/09	Not Included
7	Chloramination Systems	Identification of wells for chloramination, storage requirements, chemical feed system, valves and meters, controls and instrumentation, permitting and code requirements, preliminary cost opinion.	Tech Memo 7 Submitted 7/16/08 & Tech Memo 7b Submitted 11/21/08	Contained in Chapter 6
8	Back-up Power, Controls, and Instrumentation	Identify system components for back-up power, electrical controls, and instrumentation for the pipeline, tank, pump station, and chloramination systems.	Draft Submitted 3/16/09	Contained in Chapter 7
9	System Pressure Reduction Study	Review available waterline information, perform hydraulic analysis, and prepare conceptual cost opinions to compare the installation of new water mains to the proposed installation of residential pressure regulating valves.	Final Submitted 9/23/08	Attached as Appendix A
10	Frontage Road Sewer Replacement	Define proposed alignment, identify potential challenges, summarize preliminary pipeline and manhole design parameters, and prepare preliminary cost opinion.	Draft Submitted 03/19/09	Contained in Chapter 10

2.0 Project Bidding Strategy

The Project Bidding Strategy Technical Memorandum (Technical Memorandum No. 2) was submitted for Task Group 1, Task 102, as contained in this Chapter. The scope of work is to provide an overview of recommendations for bidding the project, including recommendations for multiple bid packages and optimizing the bid climate through press releases, workshops, and timing of the bid release.

2.1 Bid Packages

The following criteria were considered in establishing discrete bid packages for this project:

- Location of the work items in relation to each other
- Unique equipment and experience required for performance of the river crossing
- Need to provide as few points of coordination and responsibility as possible for each project site
- Desire to standardize new chloramination systems at each wellhead

In order to best meet these criteria, AECOM recommends dividing the project as follows:

Table 2-1. Proposed Bid Packages

Bid Package	Title	List of Work Items
1	Santa Maria River Water Main Crossing	River crossing pipeline and casing pipe (if required)
2	Nipomo Area Pipeline Improvements	– Frontage Road water main – Southland Road water main – Oakglen Avenue water main – Darby Lane water main – Orchard Avenue water main – Highway 101 Crossing at Darby & Grande – Pressure reducing valve stations (5 total) including valves, telemetry, controls, and instrumentation (system integration to be performed in Bid Package 4) – Isolation valves

Bid Package	Title	List of Work Items
3	Blosser Road Water Main and Flow Meter	– Blosser Road water main (between Taylor and Atlantic) – Flowmeter and telemetry – Water main to the Santa Barbara County Flood Control levee – Jack and bore underneath the levee – Water main to the south end of the river crossing (across existing agricultural land)
4	Joshua Road Pump Station and Reservoir; Wellhead Chloramination Improvements	– Water main from north end of river crossing to Joshua Road Reservoir – Reservoir – Pump station building and mechanical equipment – Landscaping – Onsite piping and valves, including piping between reservoir and pump station and discharge piping from pump station to Joshua Road – Chloramination equipment including chemical storage tanks, feed pumps, and instrumentation – Onsite electrical systems, instrumentation, and controls – Wellhead Improvements: – Upgrade existing chlorination storage and chemical feed pumping systems at Black Lake #4, Sundale, Via Concha, and Eureka wells – Electrical, controls, and instrumentation for new metering pumps and automated feed systems – New chemical storage buildings – System integration with pressure reducing valve and flowmeter telemetry (equipment to be installed by others)

2.2 Prequalification of Contractors

AECOM recommends prequalifying only the river crossing contractors (whether the project is HDD or open trench) and not requiring prequalification for the other bid packages since many contractors are capable of performing the other work items.

The prequalification process for the river crossing contractors will include development and issuance of a Request for Qualifications (RFQ); and a field review and mandatory prequalification conference.

The RFQ will include a 30% plan set, preliminary technical specifications, and project references. Attached as Appendix C is a similar RFQ prepared for another horizontal directional drilling (HDD) project for the District's consideration. If the format and requirements are acceptable, AECOM anticipates following a similar approach for this project.

After the prequalification process is completed, the number of contractors will be narrowed to a short-list for issuance of the final contract documents.

It is assumed the prequalification work will begin after the final Concept Report is submitted and accepted by the Board.

2.3 Press Releases and Contact

In the current construction climate, many contractors around the state will be interested in the project. In addition, the potential HDD contractors from around the country are likely to closely follow this project.

AECOM recommends sending the Project Narrative and a project schedule to the Ventura County and San Luis Obispo County Builders Exchange and the Santa Barbara County and Santa Maria Valley Contractors Association after the final Concept Design Report is completed. Interested contractors should be directed toward the District to provide contact information. This contact information will be collected and used for sending notices of preproposal meetings and bid advertisements.

All notices and announcements should be kept on the District's webpage. AECOM recommends putting this information under a "Contractors" tab that is shown prominently on the District's home page.

2.4 Bid Process

As project design proceeds, AECOM will work with District staff and the Construction Management Team to determine the appropriate time to bid the project. At this time, it appears the project should be bid as soon as design plans are completed. Bid packages should be provided at the Santa Maria Valley Contractors Association and SLO County Builders Exchange for review by contractors.

AECOM recommends scheduling one preproposal meeting for Bid Packages 2 through 4. Bid Package 1 will follow a separate prequalification procedure, as described in this Section. Many of the contractors are likely to propose on multiple packages and many vendors will be on several teams. This will give the contractors a general project understanding and comprehension of the relationship between the individual bid packages.

It is assumed the bid process for each package (other than Bid Package 1 – River Crossing) would follow this general schedule:

	Weeks											
	1	2	3	4	5	6	7	8	9	10	11	12
Bid Advertisement												
Bid Analysis and Recommendation												
Bid Award and Contract Finalization												

In the latest Project Schedule (see Appendix B), notification, bidding, and award of bids is scheduled to require approximately 3 ½ months for Bid Packages 2 through 4. Bid Package 1 (Santa Maria River Crossing) will require approximately 5 ½ months for prequalification, notification, bidding, and award of bids. In order to facilitate review by District staff and consulting team members, AECOM recommends staggering the bid opening dates by one to two weeks. The general timeline described in the latest Project Schedule is adequate.

The schedule for bid packages will be developed after the Concept Design Report is finalized. AECOM will perform the following bid-phase services to support the District:

- Provide plans and specifications for Electronic Clearinghouse and 20 copies of construction documents per bid package;
- Organize and attend the prebid meeting
- Maintain a list of bidders for distributing addenda
- Respond to inquiries from bidders
- Prepare, issue, and circulate addenda
- Assist the District in bid review and provide recommendations for award

3.0 HDD SANTA MARIA RIVER CROSSING

3.1 Background

The WIP Preliminary Engineering Memorandum (Boyle | AECOM 2008) recommended construction of a 24-inch nominal diameter Santa Maria River crossing via horizontal directional drilling (HDD) to reduce impacts to environmentally sensitive areas, avoid construction on the bluff, and streamline permitting for the project. This chapter summarizes the design basis for the HDD Santa Maria River Crossing including summaries of the site specific geotechnical evaluation performed by Fugro West, Inc., as well as the HDD evaluation performed by Jacobs Associates. The bore-and-jack and open-trench segments of the river crossing are discussed in Chapter 4.

3.2 HDD River Crossing Description

As shown on the 30% concept plans (See Volume 3 of this Report), the HDD River Crossing alignment begins within the riverbed at the HDD Entry Point located approximately 880-feet northwest of the levee crossing. From this point, the alignment extends northward towards the south facing bluff of the Nipomo Mesa, traversing approximately 2,100 feet of riverbed and gaining approximately 110 feet in elevation as it rises to the top of the bluff. At the top of the bluff, the alignment extends an additional 500 feet northward towards the HDD Exit Point near the proposed reservoir and pump station site. The carrier pipe vertical alignment (profile) includes a broadly sweeping inverted arc with a radius of 5,000-feet. There is no separate curve in the horizontal plane. Section 3.7 provides further detail regarding the HDD profile geometry.

3.3 Existing Utilities

Existing utilities were investigated as part of the preliminary design for the HDD River Crossing. Aboveground utilities, visible surface features of underground utilities, and utility easements were mapped as part of the topographic survey mapping effort. Available utility records were also reviewed and incorporated into the project base mapping where appropriate.

As shown on the 30% concept plans, there are no known utilities between the proposed HDD entry point and the base of the Nipomo bluff. As shown on Site Plan – North (Dwg C-102), there are multiple overhead electrical lines at the top of the bluff. There is also an irrigation pipeline near the HDD exit point (STA 33+75). The approximate horizontal location of an existing 10-inch petroleum pipeline was defined by mapping of a 10-feet wide oil easement including surface mounted petroleum pipeline placards. As shown on Site Plan – North, the 10-inch petroleum pipeline alignment is approximately 1,000-feet due east of the HDD alignment.

Based on this utility investigation, there appear to be no major utility conflicts. However, the existing irrigation pipelines near the HDD exit point may need to be relocated prior to construction. The need for irrigation pipeline relocation will be further assessed as the design progresses. Utility relocation logistics will be addressed during the right-of-way acquisition process.

3.4 Geotechnical Evaluation – Horizontal Directional Drilling

AECOM prepared and submitted Technical Memorandum No. 1 – Geotechnical Report for HDD to NCSD at the end of January 2009 following completion of the Draft Geotechnical Report prepared by Fugro West, Inc. This section summarizes the findings and recommendations as submitted in Technical Memorandum No. 1 (TM#1); including geotechnical considerations and recommendations for HDD project design.

The geotechnical evaluation did not identify “fatal flaws” with the proposed HDD river crossing. Furthermore, the Final Geotechnical Report indicated that the field exploration performed along the HDD alignment was successful, and provided suitable characterization of the subsurface conditions that should be anticipated along the alignment. Based on the results of the geotechnical evaluation as presented in TM#1, AECOM recommended that the District proceed with preliminary design of the HDD River Crossing.

The following conclusions regarding geologic conditions along the HDD river crossing were included in TM#1 and were based on the Geotechnical Report:

- The presence of seismic faults does not pose a significant fault rupture hazard to the pipeline project.
- There is a low potential for liquefaction to impact the pipe along the proposed river crossing.
- The potential exists for caving ground near the HDD entrance and exit locations.
- The HDD alignment transitions from Older Alluvium “OA” into Paso Robles Formation (QTp) near the vicinity of CPT C-14 and boring B-7 resulting in a potential tendency for the alignment to deflect at the QTp contact.
- HDD pipe installation at the Santa Maria River crossing will likely be relatively difficult as a result of variable subsurface conditions encountered. These conditions may include: shallow groundwater, wet soil conditions, coarse sand and gravel layers, cobbles, possible boulders, and firm to hard silt and clay layers.

The following recommendations regarding geologic conditions along the HDD river crossing were included in TM#1 and were based on the Geotechnical Report. Italicized text indicates how each measure is addressed in the 30% concept design.

- The design of the pipeline should consider the potential for the site to be subject to strong ground motion in response to nearby or regional earthquakes. *(Noted; as indicated above, the presence of seismic faults does not pose a significant “fault-rupture hazard” to the HDD river crossing. The use of relatively flexible HDPE carrier pipe material with fusion-welded joints will provide a continuous flexible buried conduit installation. Joint separation should not be an issue.)*
- Surface casings are likely needed to maintain HDD alignment, support boreholes, and to prevent ground caving near entry/exit locations. *(The use of surface conductor casings are addressed in the Jacobs Horizontal Directional Drilling Evaluation)*
- Shallow clearances and drilling pressures should be considered to prevent blowout during HDD operations. *(The minimum cover required to provide sufficient confinement against slurry pressure is addressed in the Jacobs HDD Evaluation. The 30% concept profile shows minimum cover that exceeds that required for confinement against slurry pressure)*
- Variable groundwater conditions and the potential to encounter perched groundwater when drilling through the base of dune sand deposits below the Nipomo Mesa (just beyond the bluff) should be considered. Reconditioning of drilling fluid may be needed to address changing

ground and groundwater conditions. *(Reconditioning of drilling fluid to address changing ground and groundwater will be addressed in the Technical Specifications.)*

- The HDD heading, alignment, and drilling fluids should be monitored during the HDD installation. Adjustment of HDD heading may be needed to maintain alignment. *(The HDD tracking and guidance is addressed in the Jacobs HDD Evaluation. Minimum requirements and tolerances will be addressed in the Technical Specifications)*

Following review and comment of the Draft Geotechnical Report, Fugro West, Inc. submitted the Final Geotechnical Report (dated March 02, 2009). A few additional clarifications and recommendations were made in the Final Report as follows:

- The field exploration performed along the HDD alignment was successful, and provides suitable characterization of the subsurface conditions that the contractor should anticipate along the alignment.
- The use of a "wash casing" is recommended to stabilize the borehole behind the HDD heading.

Figures 3-1 and 3-2 (on the following pages) depict the approximate Fugro soil boring locations as well as the anticipated subsurface soil profile along the HDD River Crossing as interpreted by Fugro West, Inc. Note, both the alignment stationing and pipeline profile have been revised in the Concept Design Report. Refer to Dwg No. C-201 (HDD Profile) of Bid Package No. 1 for the current pipeline profile.

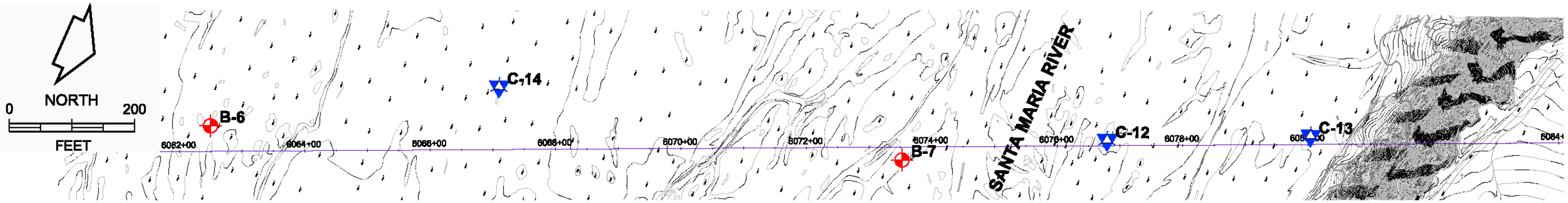
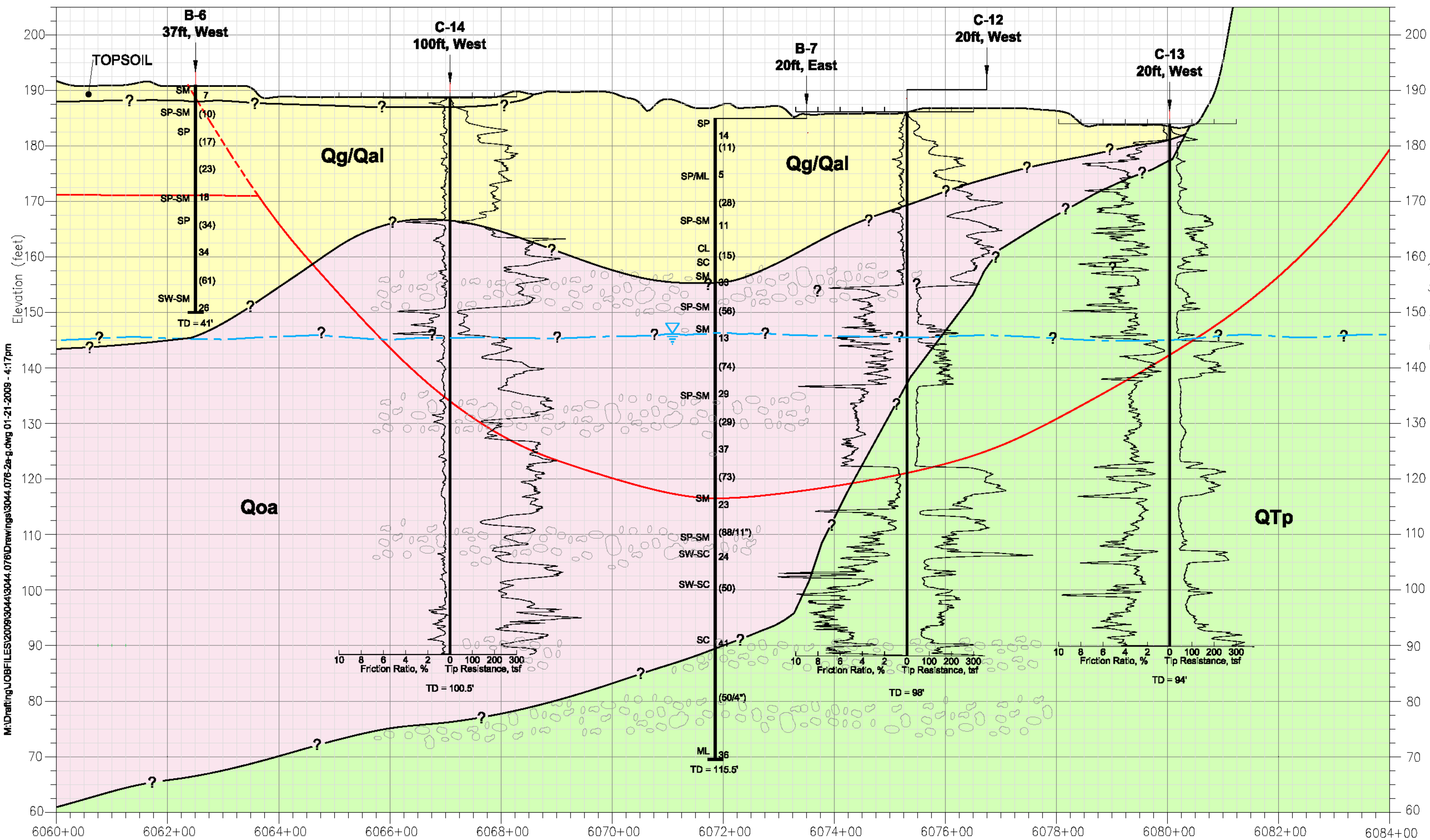
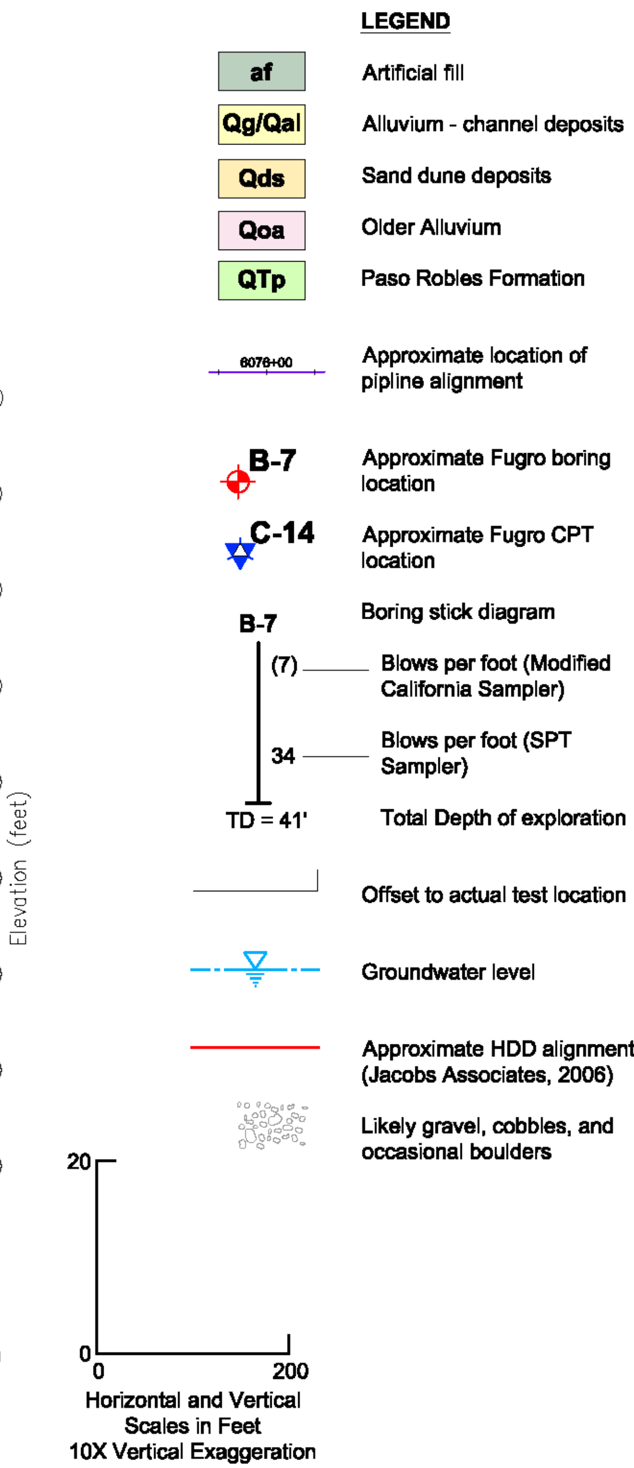


Figure Source: This figure is based on Plate 2d of project Geotechnical Report prepared by Fugro West, Inc. Project stationing and profile were revised in Concept Design Report.





BORING LOCATION PLAN AND SUBSURFACE PROFILE

Nipomo - Santa Maria Intertie

Nipomo, California

Figure 3-1

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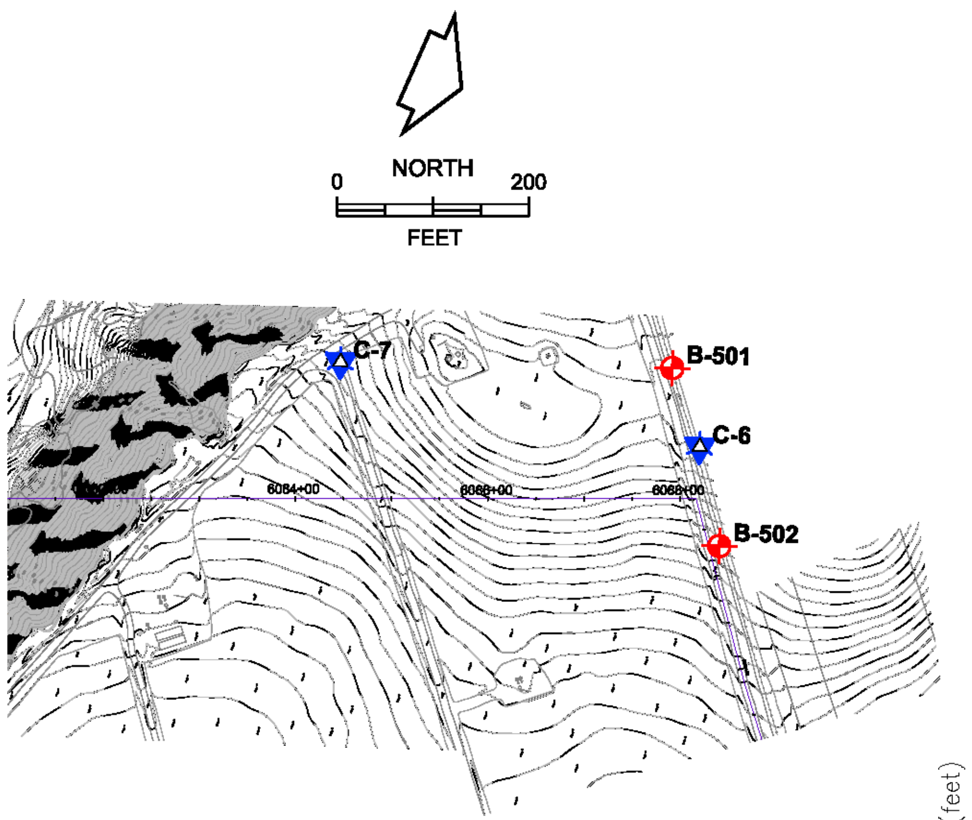
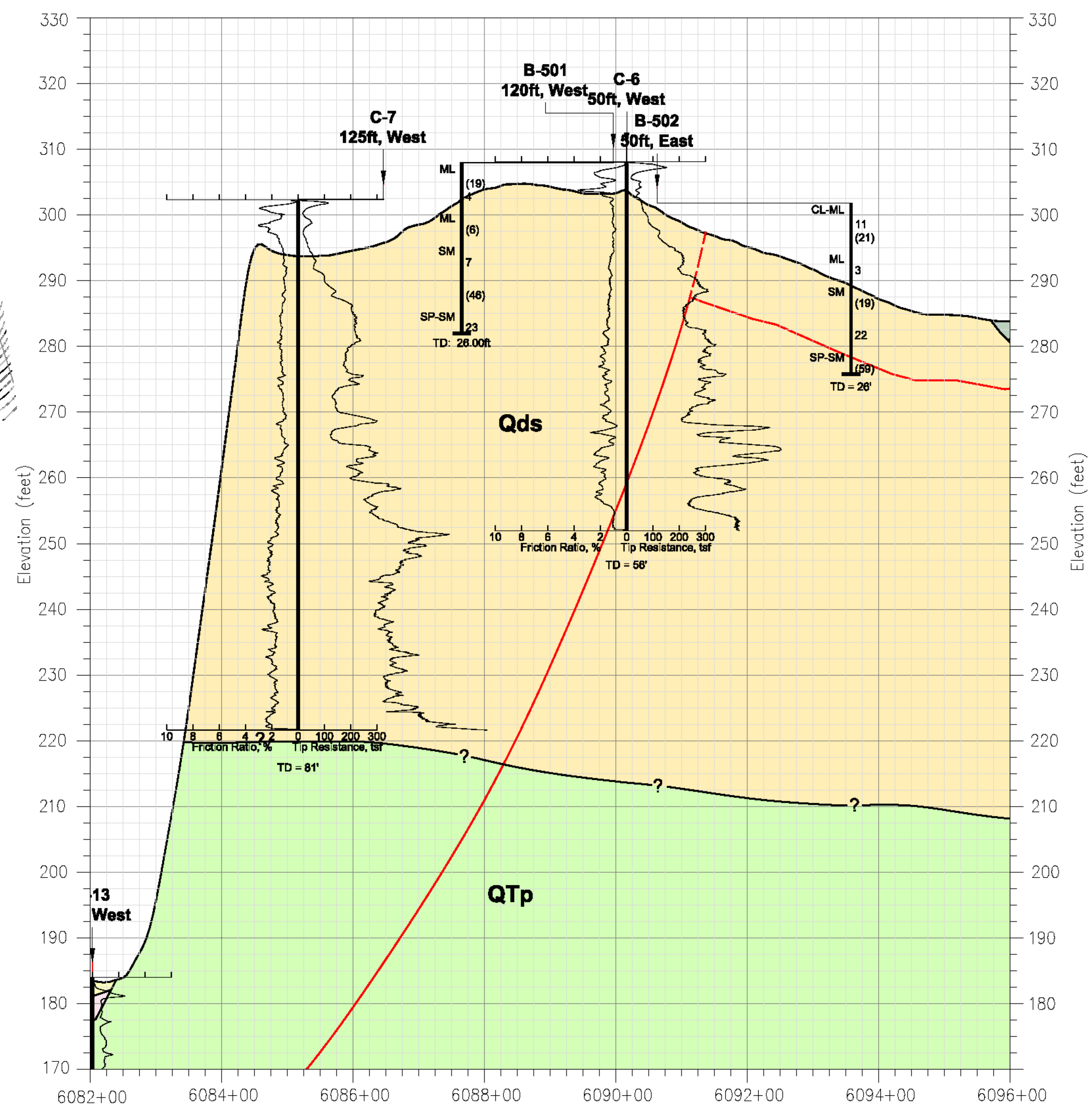


Figure Source: This figure is based on Plate 2e of project Geotechnical Report prepared by Fugro West, Inc. Project stationing and profile were changed in Concept Design Report.



LEGEND

- af — Artificial fill
- Qg/Qal — Alluvium - channel deposits
- Qds — Sand dune deposits
- Qoa — Older Alluvium
- QTp — Paso Robles Formation
- 6088+00 — Approximate location of pipline alignment
- B-502 — Approximate Fugro boring location
- C-7 — Approximate Fugro CPT location
- B-502 — Boring stick diagram
- (7) — Blows per foot (Modified California Sampler)
- 34 — Blows per foot (SPT Sampler)
- TD = 41' — Total Depth of exploration
- Offset to actual test location
- Groundwater level
- Approximate HDD alignment (Jacobs Associates, 2006)
- Likely gravel, cobbles, and occasional boulders

20 / 0 / 200

Horizontal and Vertical Scales in Feet
10X Vertical Exaggeration

BORING LOCATION PLAN AND SUBSURFACE PROFILE
Nipomo - Santa Maria Intertie
Nipomo, California

Figure 3-2

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3.5 Carrier Pipeline Design Criteria

3.5.1 Hydraulics

Chapter 4 – Pipeline Alignment summarizes the anticipated maximum working pressures for the proposed WIP pipeline improvements along Blosser Road and within the Nipomo area, including the segment from Blosser Road to the HDD entry point. Based on this evaluation, AECOM performed a preliminary hydraulic analysis of the 24-inch HDD carrier pipeline to establish both the anticipated maximum working and surge pressures along the conduit.

As indicated in Chapter 4, the carrier pipeline near the HDD entry point is anticipated to have a maximum working pressure of 100 psi. For the purposes of this analysis, it is assumed this pressure is near the ground surface. Accounting for the maximum depth of 110-feet reached at the pipe low point (See Dwg C-201, HDD Profile), the maximum anticipated working pressure at the deepest point along the HDD alignment is approximately 150 psi as shown in Table 3-1.

In order to evaluate the maximum potential surge pressure along the HDD carrier pipeline, AECOM performed a preliminary surge analysis. A hydraulic transient (also known as water hammer or surge) is a temporary flow and pressure condition that occurs in a hydraulic system following a rapid velocity change in response to the operation or shut-down of a flow-control device (for instance, a rapid valve closure). The objective of this analysis was to simulate and estimate the worst-case theoretical transient response (pressure increase over time) of the proposed carrier pipeline following rapid closure of an isolation valve along the Blosser Road transmission pipeline.

Based on the preliminary analysis, and assuming the future maximum flow of 5,570 gpm, the maximum change in pressure due to surge is estimated to be 70 psi, resulting in a total maximum pressure of 220 psi at the deepest point along the HDD alignment. The carrier pipeline material shall have sufficient internal pressure capacity to accommodate the estimated maximum pressure including surge of 220 psi as well as the design test pressure. Pipe Material is discussed below in Section 3.6.

Table 3-1. 24-inch Carrier Pipeline Hydraulics[1]

Location	Design Flow (gpm)	Design Velocity (ft/sec)	Anticipated Maximum Working Pressure (psi)	Anticipated Maximum Pressure including surge (psi)
Entry Point @ STA 7+73	2,000 gpm[2]	1.6 ft/sec	110 psi	135 psi
Centerline EL of 163.75-ft	5,570 gpm[3]	4.3 ft/sec	110 psi	180 psi
Low Point @ STA 18+75)	2,000 gpm[2]	1.6 ft/sec	150 psi	175 psi
Centerline EL of 75-ft	5,570 gpm[3]	4.3 ft/sec	150 psi	220 psi
Exit/High Point @ STA 33+75	2,000 gpm[2]	1.6 ft/sec	60 psi	85 psi
Centerline EL of 274-ft	5,570 gpm[3]	4.3 ft/sec	60 psi	130 psi

Table 3-1 Notes:

1. Assumes an average inside diameter of 22.933 inches based on selected pipe material and dimension ration (DR) discussed below in Section 3.6.
2. WIP project design flow as defined in Section 1.3.
3. Future maximum instantaneous flow rate required to deliver up to 6,300 AFY through the pipeline. The maximum flow rate of 5,570 gpm is the flow required to meet the July demand at master plan buildout as stated in Section 12.0 of the WIP Preliminary Engineering Memorandum (May 2008, Boyle|AECOM).

3.5.2 Minimum Pipeline Cover / Embedment

Multiple design criteria govern the minimum vertical ground cover required above the HDD pipeline crossing as follows:

- Slurry Confinement: Cover above the pipeline must be sufficient for confinement against drilling slurry pressure, which varies with distance along the drill hole from the entry point. As discussed below in the HDD Evaluation, ground cover for confinement is the governing condition for the proposed profile.

- Scour Protection: Pipeline cover shall be below the estimated scour elevation. This is critical within the Santa Maria Riverbed near the HDD Entry Point (STA 7+73) and near the base of the bluff (STA 26+75) where the groundcover above the carrier pipeline is at its shallowest. As discussed in Chapter 4, scour considerations within the riverbed were evaluated based on the report "Evaluation of Channel-Bed Scour at Proposed Coastal Aqueduct Crossing of Santa Maria River (Chang, 1995)". In the Chang Report the top of the CCWA State Water Pipeline was recommended to have a minimum embedment of approximately 25 feet below the active riverbed, to account for the maximum estimated general scour plus a factor of safety for potential local scour. Therefore, the pipeline should be installed below the estimated scour depth, which is anticipated to be approximately 25 ft below the existing river bottom surface.

- Surface Mining: Pipeline embedment shall consider the potential for surface mining activities (above the HDD river crossing) mentioned in the NCSD WIP EIR. Surface mining operations within the riverbed could impact scour and require deeper minimum pipeline embedment for scour protection where cover is at its shallowest. The surface mining claim mentioned in the NCSD WIP EIR was further investigated by AECOM in order to confirm its current status including site boundary limits within the Santa Maria River bed. Based on our review of the San Luis Obispo County Planning Department files, it is our understanding that there may be a potential for future surface mining to occur over the HDD river crossing. The available records indicate that the maximum mining depth (as measured from the existing river bed) is 15-feet. As a worst case, the depth of cover above the pipeline near the HDD entry point will need to be increased from 25 to 40 feet to account for the potential mining of up to 15-feet of surface material.

AECOM recommends that the District consult with legal counsel to determine the best way to proceed with this issue. It is also recommended that the District meet with SLO County planning staff for further coordination and to discuss the County's position on the potential for the full Troesh site to be mined. At NCSD Staff's request, AECOM is currently pursuing further information from SLO County Planning Department staff regarding the status of the mining claim in question.

3.5.3 Installation and Operating Loads

Pipe strength and wall thickness requirements for HDD-installed pipelines are determined by consideration of installation and operating loads. HDD pipelines are subjected to high loads and stresses during the installation process including tension, bending, and external pressure stresses. Operating loads include internal pressure (as discussed above in Section 3.5.1), bending, thermal effect, and external pressure. Jacobs Associates' HDD Evaluation (See Appendix D) discusses these loads in detail and the method of their evaluation.

3.6 Pipe Material Selection:

AECOM recommends that the 24-inch nominal carrier pipe be AWWA C906-07 High-Density Polyethylene (HDPE) pressure pipe with a standard PE code designation of PE-3408. Pipeline joints shall be butt-fusion welded. Based on the preliminary design discussed in this Chapter, AECOM recommends a standard dimension ratio of DR-9 with a corresponding pressure class/maximum working pressure rating of 200 psi. According to AWWA C906-07, DR-9 pipe has a recurring surge capacity of 100 psi (in addition to working pressure rating of 200 psi) which is more than sufficient to handle the estimated maximum pressure including surge of 220 psi. HDPE is inert to corrosive elements in water and soil and does not require corrosion protection.

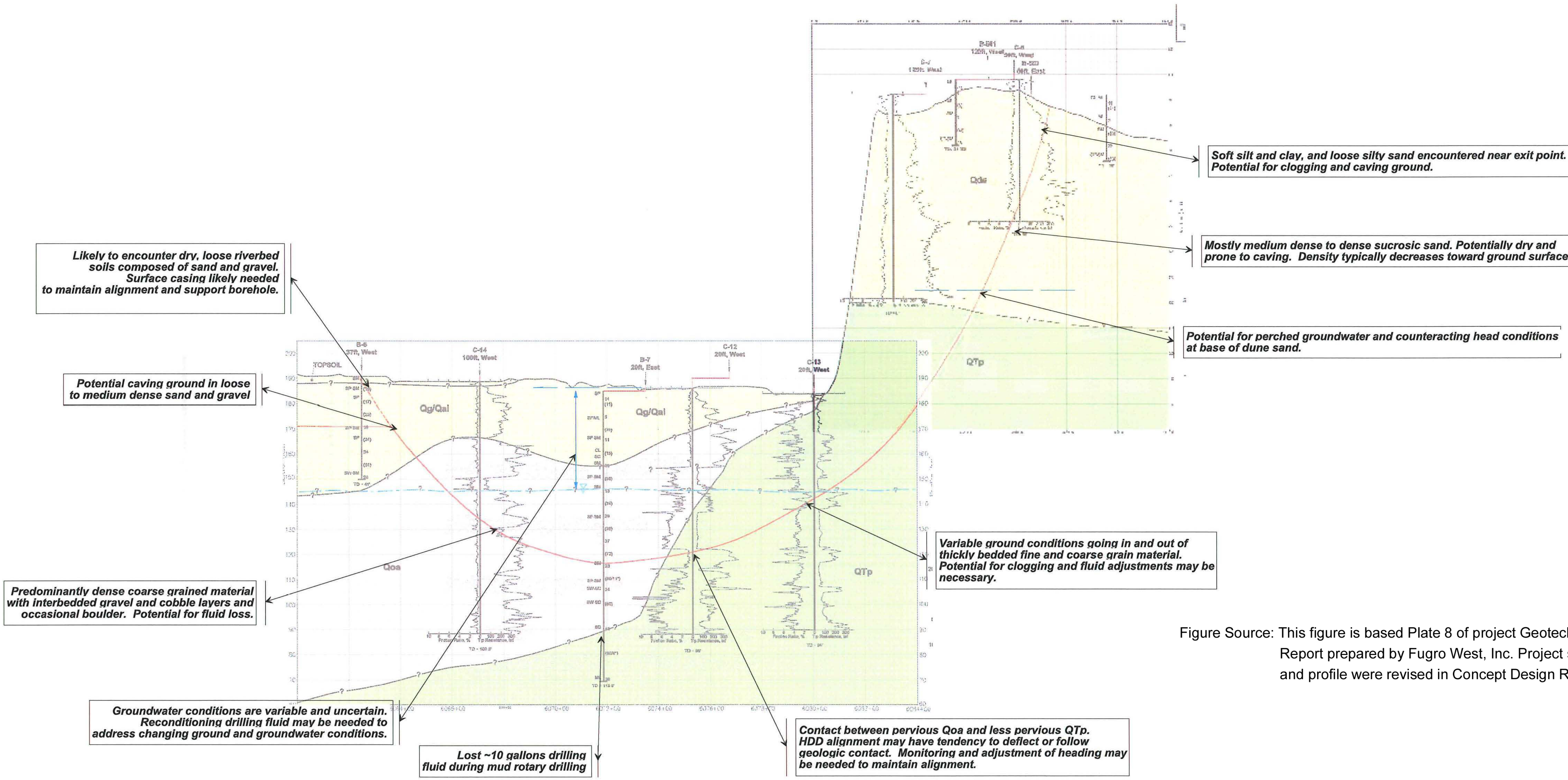
Hydrostatic field leak test pressure will not exceed 220 psi at the HDD low point. Due to profile, test pressure will vary along pipeline alignment.

3.7 Horizontal Directional Drilling Evaluation

The Jacobs Associates' Preliminary Design Report (see Appendix D) evaluates the horizontal directional drilling requirements for the Santa Maria River Crossing. The Jacobs Associates' PDR provides a favorable evaluation of the viable HDD river crossing. It identifies potential HDD risk factors and mitigation measures, sets forth key design criteria, provides minimum staging area considerations, and evaluates anticipated ground behavior during HDD as well as other construction considerations.

3.7.1 Geotechnical Considerations for HDD Design

Section 3.4 provides a summary of the geotechnical evaluation performed by Fugro West, Inc., along the proposed HDD River Crossing. Geotechnical conclusions and recommendations are discussed and figures showing the anticipated soil profile along the HDD alignment are included. Figure 3-3 (below) presents geotechnical considerations for design of the HDD River Crossing based on the project Geotechnical Report prepared by Fugro West, Inc. Note, both the alignment stationing and pipeline profile have been revised in the Concept Design Report. Refer to Dwg No. C-201 (HDD Profile) of Bid Package No. 1 for the current pipeline profile. The following evaluation further interprets geotechnical considerations shown on Figure 3-3, incorporates key geotechnical recommendations from Section 3.4, and discusses anticipated ground behavior during drilling operations based on data from the Geotechnical Report.



TYPICAL PROFILE VIEW of HDD CROSSING LOOKING WEST
Approximate Scale: 1 in. = 40 ft. vertical
1 in. = 400 ft. horizontal
(see profile and legend on Figures 3-1 and 3-2)

Figure Source: This figure is based Plate 8 of project Geotechnical Report prepared by Fugro West, Inc. Project stationing and profile were revised in Concept Design Report.

GEOTECHNICAL CONSIDERATIONS FOR HDD CROSSING
Nipomo-Santa Maria Intertie
Santa Maria, California

Figure 3-3

3.7.2 Subsurface Conditions and Anticipated Ground Behavior During HDD

The existing geotechnical information along the alignment indicates that the soils generally consist of alluvium derived from the Paso Robles Formation and dune sand. The geotechnical report also states that groundwater levels vary significantly seasonally and as such the HDD construction should take place during periods of low flow because the HDD entry point is within the active river bed.

The alluvium, including stream channel deposits within the riverbed, appears to be clean sand. The alluvial stream channel deposit is anticipated to exhibit running behavior when dry, as defined in the Tunnelman's Ground Classification (Jacobs' Appendix A) and flowing behavior when wet. The ground will require the drilling mud to help stabilize the excavation. The alluvium will tend to have high frictional forces during carrier pipe installation due to the sand content and anticipated ground behavior. A surface casing or a shored pit may be required to stabilize the soils at the entry and exit points during drilling. The use of drilling mud will also reduce frictional forces during carrier pipe installation.

The alluvium, located outside of the river channel, appears to be sand with an increased silt and clay content. This alluvium is anticipated to exhibit raveling behavior when dry and flowing behavior when wet. The ground will require the drilling mud to help stabilize the excavation. The alluvium will tend to have moderate frictional forces during carrier pipe installation due to the sand content and anticipated ground behavior. The use of drilling mud will reduce frictional forces during carrier pipe installation.

The older alluvium that underlies the alluvium is similar to the alluvium located outside of the river channel. The older alluvium includes clay and silt and is distinguished from the alluvium by an increase in gravel and cobble frequency and increased density. There is a possibility of boulders within this deposit. The older alluvium is anticipated to exhibit raveling behavior when dry and flowing behavior when wet. The ground will require drilling mud to help stabilize the excavation. The older alluvium will tend to have high frictional forces during carrier pipe installation due to the sand content and anticipated ground behavior. The use of drilling mud will reduce frictional forces during carrier pipe installation.

The Paso Robles Formation is a formational rock comprised of weakly cemented clay, silt, and sand. The formation is anticipated to exhibit firm behavior and will tend to contain the drilling mud and provide stability to the excavation. The Paso Robles Formation will tend to have low frictional forces during carrier pipe installation due to the stability of the excavation when lubricated and higher frictional forces when the drilling mud is not in the excavation. The contact line between the riverbed alluvium and the Paso Robles Formation is anticipated to project downward at the same angle as the surface topography to an elevation of 95 ft and then transition into a gentle slope to the south.

The dune sand deposits that form the Nipomo Mesa are anticipated to exhibit running behavior when dry and flowing behavior when wet. The ground will require the drilling mud to help stabilize the excavation. Since this is at a higher elevation than the entrance point, a surface casing or a shored pit may be required to prevent over-excavation and to stabilize the hole as the drilling mud may not remain in the hole. The sand dune deposits will tend to have moderate frictional forces during carrier pipe installation due to the sand

3.7.3 HDD Installation Process

A detailed discussion of the HDD method and installation process is provided in Section 5 of the Jacobs HDD PDR.

3.7.4 30% HDD Design Profile

Table 3-2, below, summarizes the preliminary geometry of the HDD profile. The 30% concept plans are attached as Volume 3 of this Report.

Table 3-2. HDD Profile Geometry

Description	Design Value
HDD Entry Angle: Santa Maria River	9 degrees (15.8%)
Entry Elevation	190 ft
Straight Length/Tangent	277 ft
Vertical Bend Radius	5,000 ft
Curve (arc) Length	2,123 ft
Minimum Elevation	75 ft
Horizontal Bend Radius	N/A
Maximum Depth at River	110 ft
HDD Exit Angle: Nipomo Mesa	14.5 degrees (25.9%)
Straight Length/Tangent	216 ft
Exit Elevation	300 ft

3.7.5 Construction Considerations

The use of a mid-path intercept may reduce risk on this project. A midpath intercept is performed by drilling with two HDD rigs, one from each end. This method may reduce risk by allowing for the entire pilot hole to be drilled with drilling mud and increasing hole stability. Other advantages of the method include:

- Fluid pressures in each drilled hole are better controlled, reducing the risk of an inadvertent return (frac-out).
- Mud return line is not required as the mud is pumped across the reach within the drill steel.
- Drill steel continuously occupies the hole until the carrier pipe is installed. Having drill steel occupy the hole reduces the risk of losing the hole due to hole collapse.

Curvature: The minimum radius of curvature during installation for a 30-in. OD HDPE pipe is approximately 30 times the pipe outside diameter, or about 100 ft (based upon dimensional information only). The actual radius of curvature is determined using critical HDPE properties, operating pressure, static pressure, surge pressure, and external loads, in addition to the induced bending stresses from the curvature. The drill steel is also a factor determining the radius of curvature. The minimum drill steel diameter is anticipated to be at least 8 in. The safe minimum radius of curvature for the drill string is approximately 1,200 times the drill steel diameter, or 800 to 1,000 ft for this project. A tighter radius or significantly thicker drill steel would require the use of shorter lengths of drill steel. The geometry dictated by the layout parameters for this project greatly exceeds the minimum radius of curvature established by constructability and pipe-material considerations.

Length: A pipe string of about 40 ft longer (20 ft on each end) than the drill path is needed to accommodate for the pipe shrinkage that occurs after the pipe is placed (the pipe is elongated approximately 10 ft due to pulling forces and temperature). The pipe shrinkage occurs as a result of cooler temperatures in the hole and the release of tension developed in the pipe (during pipe pullback) once the pullback is complete. Verification that the HDPE has shrunk will be made before the ends are cut and capped. A maximum HDPE length of about 2,655 ft is anticipated.

Pipeline Buildup: For this project, the pipeline is anticipated to be built up as one continuous string of pipe for pullback to mitigate risk of the excavation collapsing during pullback. The HDPE joints will be welded by creating a bead on the inside and outside of each joint. The inner welded bead shall be removed due to system hydraulic requirements. The outer bead may need to be removed to reduce skin friction and the probability of the pipe string becoming unable to be advanced during pullback.

3.8 Easements and Right-of-Way

Temporary Construction and Permanent Utility Easement requirements along the Santa Maria River Crossing are discussed in Table 4-5 of Chapter 4. Additional Temporary Access Easements may be required for site access to the HDD lay down and work areas shown on the 30% level HDD Plans (see Volume 3 of this Report). The need for access easements will be addressed as the design progresses.

3.9 HDD Risk Management

HDD and underground construction, in general, carry many risks which can impact the success of the project including unforeseen conditions that may arise in the field during construction. The risks typically include impacts to schedule, cost, environment, system operations, and safety.

As discussed above in the Geotechnical and HDD Evaluation Sections, HDD carrier pipe installation at the Santa Maria River crossing will likely be relatively difficult as a result of variable subsurface conditions encountered. As a risk management approach, AECOM will work with the District and the selected HDD contractor to prepare and implement an HDD Contingency Plan as part of the drilling program to systematically identify and mitigate known risks to the HDD installation.

AECOM recommends the use of the mid-path intercept method which may reduce the risk on this project. AECOM will work with Jacobs to determine the best strategy for defining this method as either a requirement or option in the Contract Documents.

In accordance with Task 303 – Geotechnical Baseline Report (GBR), a GBR will be prepared by Jacobs Associates to identify the geotechnical baseline anticipated during the HDD. The Final Geotechnical Report prepared by Fugro West, Inc., will be the basis for the GBR. The purpose of the GBR will be to establish a contractual basis for the anticipated ground conditions and ground responses during the HDD installation. This document will: 1) facilitate resolution of potential disputes regarding underground conditions and 2) provide clear indications of the risks associated with actual ground conditions and ground response. Changes from the baselines will be handled in accordance with provisions stated in the Contract Documents. The GBR will be included with the Contract Documents.

In accordance with Task 201 – Permit Applications of the authorized Scope of Work, an HDD Frac-Out Monitoring, Response, and Clean-Up Plan will also be prepared and included with the Contract Documents.

4.0 PIPELINE ALIGNMENT

4.1 Introduction

The Draft Pipeline Alignment Technical Memorandum was submitted for Task Group 1, Task 103 on March 3, 2009. Comments were received from District Staff and responded to in an errata letter dated March 11, 2009. This Chapter consists of the Draft Technical Memorandum updated per the District staff comments. AECOM's scope of work is to define the proposed alignment and identify potential challenges. Preliminary pipeline design parameters such as diameter, length, material, valve type, anticipated working pressures and pressure classes, corrosion control (if required), thrust restraint, connections to the existing system, and air/vacuum valve type and placement are presented herein. The 30% plan submittal presents many of these elements (included as Volume 3 of this Concept Design Report).

This Chapter addresses only the pipeline components of the project. AECOM is also assisting the District with development of construction plans for the Frontage Road Sewer Upgrade project on Frontage Road between the Southland Wastewater Treatment Facility and Division Street. The sewer line will need to be installed prior to installing the new waterline on Frontage Road, which (along with the potential for cost savings) is the reason for combining these projects into the same construction documents. The Frontage Road sewer design was described in the Draft Technical Memorandum #10 - Frontage Road Sewer Replacement and is contained in Chapter 9.0 of this report.

4.2 Hydraulic Design Criteria

The basis for sizing the pipes, valves, and appurtenances presented herein is the result of hydraulic modeling described in Technical Memorandum No. 9 (System Pressure Reduction Study), attached as Appendix A.

4.3 Geotechnical Design Recommendations

The Geotechnical Report by Fugro (January, 2009) presented soil parameters that are important for pipeline design. These are summarized in this Section:

The site is located within a seismically active region of Central California that is prone to moderate to large earthquakes. The design of the pipeline and associated structures should consider the potential for the site to be subject to strong ground motion in response to earthquakes. Structures should be designed to resist the forces generated by earthquake shaking in accordance with the building code and local design practice.

Based on the subsurface conditions encountered in the geotechnical investigation borings, the majority of the on-site soil should not be considered suitable for use as pipe bedding or backfill in the pipe zone. The southern portion of the Blosser Road alignment is underlain by sandy material that may be suitable for use as pipe bedding or pipe zone backfill. If the on-site soils are to be used for these purposes, the contractor will likely need to exercise care during excavation such that potentially suitable materials are not contaminated or mixed with the overlying or interbedded finer grained soils. The excavated materials can likely be used for compacted backfill above the pipe zone. Moisture conditioning of the soils and control of compaction layer thickness will be needed to achieve the recommended compaction.

The soils expected to be encountered at the site within the anticipated depth of excavation generally consist of sandy soils. The onsite soils can likely be excavated with conventional backhoe or excavator type equipment typically used for pipeline construction. Vertical cuts in sandy soils should not be considered stable unless properly shored or sloped in accordance with the requirements of OSHA. Temporary slopes and shoring will need to comply with OSHA requirements.

4.4 Blosser Road Extension

Groundwater was not encountered along the alignment. However, groundwater levels will depend on the time of year of construction and the water level in the Santa Maria River and adjacent Blosser drainage channel.

Trench depths are expected to be less than 10 feet. The bottom of the trench excavation is expected to expose loose to medium dense sand. The trench subgrade should be moisture conditioned and compacted prior to placing bedding material for the pipe.

4.5 Santa Maria River Levee Jack and Bore

Asphalt, concrete, and road base materials overlaying sand with varying amounts of silt and gravel were encountered in the borings. Groundwater was not encountered in the borings performed north and south of the levee. However, groundwater levels will depend on the time of year of construction and water level in the Santa Maria River. The rock slope protection for the levee (Fugro, 2008a) likely extends to a depth of approximately 20 feet below the top of the levee.

Jacking and boring and excavations for the jacking and receiving pits will likely encounter loose to medium dense sand with varying amounts of silt and gravel. Procedures should be followed that reduce the potential for caving of loose sands that can occur as a result of advancing the auger beyond the casing. There is a potential for the process to result in heaving or settlement of the levee. Recommendations are included in the geotechnical report for monitoring heave or settlement during construction.

4.6 Santa Maria River Crossing

Alluvium, older alluvium, and Paso Robles Formation were encountered in the soils explorations in the Santa Maria River. The alluvium and older alluvium generally consist of loose to very dense sand with varying amounts of silt, clay, and gravel. The Paso Robles Formation generally consists of dense to very dense sand with varying amounts of silt, clay, and gravel and stiff to hard silt and clay. Varying amounts of gravel, cobbles, and possibly boulders were encountered at various depths within the alluvium and Paso Robles Formation. Groundwater was encountered at a depth of approximately 38 feet below the existing ground surface in the borings in the Santa Maria River.

In the cut and cover section of the pipeline alignment, trench excavation will likely expose loose to medium dense sand with varying amounts of silt, clay, and gravel. The trench subgrade should be moisture conditioned and compacted prior to placing bedding material for the pipe. Moisture conditions at the bottom of trench excavation could change if construction is performed during the wet season or during release from Twitchell Dam. Coordination of the construction schedule to river flow conditions may reduce the need for dewatering.

4.7 Nipomo Mesa Pipelines

Artificial fill and dune sand deposits were encountered along the pipeline alignment and generally consisted of asphalt concrete, base materials, very loose to very dense sand, and local soft to stiff silt. Groundwater was encountered in borings B-102 and B-405 near the Highway 101 crossing at a depth of approximately 27.5 feet below the existing ground surface. The groundwater encountered is below the anticipated pipe depths. Various concrete, rubble, and unidentified buried objects were encountered along the alignment below the asphalt pavements. These area will be defined during potholing activities. The concrete appears to be associated with old concrete pavement in the area. We expect the bottom of the trench excavation will expose very loose to medium dense sand. The trench subgrade will likely need to be moisture conditioned and compacted prior to placing bedding material for the pipe.

4.8 Highway 101 Jack and Bore

Asphalt, concrete, road base materials, and dune sand deposits were encountered near the Highway 101 crossing and generally consist of asphalt concrete, base materials, and very loose to dense sand with varying amounts of silt. Depending on the groundwater levels during construction, groundwater may be encountered at the bottom of the jacking and receiving pits. Procedures should be followed that reduce the potential for caving of loose sands that can occur as a result of advancing the auger beyond the casing. There is a potential for the process to result in heaving or settlement of the Highway 101. Recommendations are included in the geotechnical report for monitoring heave or settlement during construction – and will be incorporated into the bid documents.

4.9 Materials and Sizes

The pipeline sizes and materials shown in the 30% design plans (Volume 3 of this Report) are summarized in Table 4-3. The Nipomo CSD requires C900 PVC for buried water mains 12” and smaller and ductile iron pipe (DIP) for buried water mains greater than 12”.

Table 4-3. Pipe Materials and Sizes

Location	Material and Size (including casing pipe if required)	Anticipated Maximum Working Pressure (psi)	Pressure Class
Blosser Road	18” DIP	100	CL 250
South Santa Maria River Levee Crossing	36” Steel casing with 24” DIP carrier pipe	N/A	Extra Strong (0.500” wall thickness)
South Riverside Alignment (levee to South HDD Staging Area Pump Station)	24” DIP	100	CL 250
Between Pump Station and Santa Maria Vista Road	24” DIP	150	CL 250
Southland Street	12” C900 PVC	110	CL 200
Frontage Road	12” C900 PVC	110	CL 200
Orchard Road	12” C900 PVC	110	CL 200
Oakglen Avenue	12” C900 PVC	100	CL 200
Darby Lane	12” C900 PVC	100	CL 200
Highway 101 Crossing	30” Steel casing with 12” C900 PVC carrier pipe	N/A	Extra Strong (0.500” wall thickness)

4.10 Fittings

Valves, pipe joints, and thrust restraints will be designed for the test pressure. The test pressure will be at least 150% of the anticipated working pressures listed in the above table.

Restrained, push-on, or mechanical joints will be specified for installation of carrier pipes in the jacked steel casings and other locations along the pipeline alignment as needed. Hydrostatic thrusts at the test pressures will be the basis for sizing thrust blocks or other means of resisting thrusts.

4.11 Valves and Appurtenances

Gate valves will be used for buried installations. Butterfly valves will be used in the PRV vaults because they require less space than gate valves, resulting in a smaller vault footprint. Valves will be flanged and equipped with 2” AWWA operating nuts for buried valves and hand wheel operated valves in vaults or above ground. Valves will be placed at all pipeline intersections (3 valves at tees, and 4 valves at crosses) and are shown on the plans at approximately every 500 feet along straight lengths of pipe for isolation purposes. Spacing should be discussed by District staff and the project team.

4.12 PRV Stations

Pressure reducing valves (PRVs) will be Cla-Val model 90-01 or approved equal. Each PRV station will be a buried vault with two valves. The smaller PRV is intended to regulate pressures during relatively low flows (average day for example). The larger PRV will regulate pressures during higher flows when the smaller valve cannot supply enough water to meet demand at the PRV pressure setting (during a fire for example) in the regulated pressure zone (see Figure 1-3). Flanged fittings will be specified for installation in the new PRV vault.

Vaults will be pre-cast structures with traffic-rated access hatches, telemetry and controls for connection to the SCADA system. Instrumentation and controls are addressed in Chapter 7.

Initial pressure valve settings are summarized in Table 4-4. Settings are based on the hydraulic modeling summarized in Technical Memorandum No. 9 (Appendix A). The PRV settings can be adjusted in the field as necessary.

Table 4-4. PRV Settings

Location	Station	High-Flow PRV Nominal Size/Downstream Pressure Setting (psi)	Low-Flow PRV Nominal Size/Downstream Pressure Setting (psi)
Southland St.	2020+00	6" / 89	2.5" / 94
Orchard Rd.	1043+00	6" / 73	2.5" / 78
Frontage Rd.	3041+00	6" / 77	2.5" / 82
S. Oakglen Ave.	4011+00	6" / 77	2.5" / 82
Santa Maria Vista Rd.	118+50	6" / 90	2.5" / 95

4.13 Air/Vacuum and Air Release Valves

Air/vacuum and air release valve (ARV) construction details will be consistent with the latest versions of the Nipomo CSD standard details for all locations. The plans show potential locations for ARVs in the profile at all local high points; actual locations will be evaluated once the pipeline profile is finalized. The physical locations for the ARV cans will be determined once the pipeline alignment plan and profile is completed to the 60% progress level.

4.14 Pigging Facilities

Pigging facilities will be considered for the transmission main between the point-of-connection in Santa Maria and the reservoir in Nipomo. Where appropriate, facilities will be incorporated and shown in the 60% design submittal. Compatibility of pigging with valves and other appurtenances will be considered.

4.15 Corrosion Control

Linings and coatings designed to protect against corrosion will be utilized to negate the need for cathodic protection. Polyethylene "baggies" and wrapping will be used for DIP and at fittings, valves, etc. per AWWA specifications.

4.15 Blosser Road Flowmeter

AECOM recommends the use of a magnetic meter which offers a high degree of accuracy and reliability, as well as requiring little maintenance. These types of meters can provide flow readings within 0.5 percent of actual flow. The meter will be installed in a precast vault with a traffic-rated access hatch, buried bypass piping, valves for shutoff, and will be connected to both the Santa Maria and NCSD SCADA systems. It will be designed so that both agencies will be able to read data from the flowmeter with no remote control capability. A pressure transducer is recommended at this location to monitor the pipeline pressure. The pressure will be SCADA monitorable for trending and troubleshooting. Instrumentation and controls are addressed in Chapter 7.

4.16 Pavement Repair

It is assumed pavement will be replaced at either the thickness specified below (from the Geotechnical Report, ibid) or at the existing thickness, whichever is greater. Pavement Repair in San Luis Obispo County will be coordinated with the County of San Luis Obispo Public Works Department, at this time one traffic lane width is anticipated to be repaved after pipeline installation is completed. Similar requirements have been assumed for the pavement in the City of Santa Maria. Final pavement repair conditions will be incorporated into the bid documents.

4.17 Easement Requirements

NCSD will be responsible for acquiring easements north of Blosser Road, across the levee and river, through the Linda Vista Farms area, and to the existing pipeline easement between Joshua Road and Maria Vista Estates. Both permanent easements and temporary construction easements will be required. Locations and widths are summarized in Table 4-5, although they are considered approximate until negotiations are finalized with the existing property owners. The width of the temporary construction easement represents the entire width during construction. The permanent utility easement width will remain.

Table 4-5. Easement Widths and Locations

Location	Assessor Parcel Numbers (APNs)	Stations	Temporary Construction Easement Width (ft)	Permanent Utility Easement Width (ft)
Blosser Road	017-030-019	Unknown (15+00 & 18+00)	30	10
South Santa Maria River Levee Crossing	090-341-019	50+91 to 53+71	100	30
South Riverside Alignment (levee to South HDD Staging Area)	090-341-019	53+71 to 56+99	100	30
Linda Vista Farms Area	090-291-042	100+00 to 105+00	100	30
	090-291-043	105+00 to 112+50	--	30
	090-291-044	112+50 to 118+40	--	30

4.18 Bore and Jack Crossings

Crossing the south Santa Maria River levee and Highway 101 at Grande/Darby will both require the use of trenchless technologies. Both the traditional bore-and-jack and guided-auger-boring methods are well suited for these installations. The traditional bore-and-jack method consists of removing the soil ahead of a steel casing pipe that is simultaneously jacked behind the cutting head of an auger. The auger is placed within the steel casing. A bore-and-jack installation will require a jacking pit (approx. 40' x 12'), a receiving pit (approx. 10'x10'), and surface access for equipment and personnel.

The guided-auger-boring method (auger boring with pilot tube guidance, including a jacked steel casing) is another trenchless process that includes elements from the conventional bore-and-jack method. This method first requires the installation of a pilot tube using a laser-guided steering head. Using this pilot tube for guidance, an auger head is then advanced behind the pilot tube to bore the required opening for the steel casing. Simultaneously, the steel casing is jacked behind the cutting head auger. Like the bore-and-jack method, this process will require a jacking pit (approx. 35' x 12'), a receiving pit (approx. 20'x10'), and surface access for equipment and personnel. This method is applicable for casing sizes up to 48 inches outside diameter.

For either method, the carrier pipe is installed inside the casing.

4.18.1 Santa Maria River Crossing

The Santa Maria River crossing will include the following sections:

1. 280 feet of bored and jacked 36 inch steel casing with 24 inch DIP;
2. 900 feet of "cut and cover" construction for 24 inch DIP;
3. 2,600 feet of HDD construction (design of HDD is covered in Chapter 3).

Both the Santa Barbara County Flood Control & Water Conservation District (SBCFC&WCD) and the United States Army Corps of Engineer (USACE) were contacted to determine design requirements for the levee crossing.

The USACE has requirements for how deep to construct a pipeline under a river levee. The Corps indicated they were in the process of preparing design documents for repairs to the levee and provided preliminary requirements. The Corps plans to extend levee improvements to 15 feet below the low flow channel elevation which is about 30 feet below the top of the existing levee. The Corps' preliminary requirement is for the top of the casing be three feet lower than the bottom of the levee. Note that these requirements may be subject to change pending the USACE's completion of design documents for the levee upgrades currently scheduled for completion by May 29, 2009. Modifications to the levee crossing depth, length, and other USACE requirements may need to be incorporated at that time.

A second criterion for determining the elevation of the top of the casing under the river is scour protection. Scour considerations were evaluated based on the report *Evaluation of Channel-Bed Scour at Proposed Coastal Aqueduct Crossing of Santa Maria River* (Chang, 1995) [1]. In the Chang Report the top of the CCWA State Water Pipeline was recommended for constructed at approximately 25 feet below the active riverbed, to account for the maximum estimated general scour plus a factor of safety for potential local scour.

The calculations shown below were used to assess the worst case for determining the elevation of the top of the casing under the levee and the uncased pipe under the river between the levee crossing and the start of the HDD section. The lower elevation of the two methods will be used to determine the depth of the casing.

Check Depth Based on Levee Repairs:

(Top of Levee Elevation) – (Depth of Levee Repairs) – (Minimum Clearance) = (Top of Casing Elevation)

202 ft – 30 ft – 3 ft = 169 ft

Check Depth Based on Scour Protection:

(Channel Elevation) – (Scour Protection Depth) = (Top of Casing)

190 ft – 25 ft = <u>165 ft</u>

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[1] Howard H. Chang Consultants, "Evaluation of Channel-Bed Scour at Proposed Coastal Aqueduct Crossing of Santa Maria River", Prepared for Fugro West, Inc., April 1995

Since the scour protection depth is lower than the required clearance for the levee repairs, use 165 feet as the elevation for the top of the casing and the top of the uncased pipe under the river. In conformance with recommendations made in the Geotechnical Report (Fugro, 2009), provisions will be included in the project technical specifications that will require the contractor to monitor the ground surface above the steel casing for settlement and/or heave prior to and during boring and jacking operations. If the heave or settlement exceeds the maximum allowable then mitigation measures such as grouting and/or repair to the levee will be required.

4.18.2 Highway 101 Crossing

The pipeline will cross the Caltrans Highway Right-Of-Way between Frontage/Grande Street and Darby Lane via a bore and jack installation aligned perpendicular to the highway. The highway crossing will include approximately 220-lf of 12-inch ductile iron carrier pipe within a 30-inch steel casing pipe. Based on Chapter 600 – Utilities Permits of the Caltrans Encroachment Permits Manual, the following design criteria are assumed in the 30% design:

- Required thickness for steel casing pipe will be ½-inch thick
- Encasement shall extend, at a minimum, to the highway right-of-way lines.
- The recommended minimum depth of cover for pipelines or casings 25-inches to 48-inches shall be 15-feet.

Based on a top of roadway elevation of 338-ft at Highway 101 (see DWG C-139), the recommended top of pipe elevation for casing placement is 338 – 15 = <u>323-feet</u>. Pending outcome of the Caltrans encroachment permit submittal and review process, modifications to the design may be required to satisfy any additional State requirements and/or conditions that may arise.

In conformance with recommendations made in the Geotechnical Report (Fugro, 2009), provisions will be included in the project technical specifications that will require the contractor to monitor the ground surface above the steel casing for settlement and/or heave prior to and during boring and jacking operations. If the heave or settlement exceeds the maximum allowable then mitigation measures such as grouting and repair to the roadway will be required.

4.19 Traffic Control

It is assumed the contractor will be responsible for preparing and submitting traffic control plans along Blosser Road, Orchard Avenue, Frontage Road, Joshua Road, Southland Road, and Darby Lane.

5.0 PUMP STATION & RESERVOIR

Draft Technical Memorandum No. 4 – Booster Pump Station No 2 – was submitted to the District on March 20, 2009. Comments have been received and incorporated, as contained in this Chapter. The pump station and reservoir site layout are included with the 30% Plans, contained in Volume 3 of this Report. This Chapter will identify and evaluate facilities required to provide capacity to deliver 2,000-gpm of supplemental water to the NCSD distribution system.

5.1 Background

The Waterline Intertie Project Preliminary Engineering Memorandum (PEM – Boyle/AECOM, May 2008), recommended a buried storage tank (approximately 0.5 MG storage capacity) and a pump station (Pump Station #2) to pump water from the WIP storage to the NCSD distribution system. In the PEM, AECOM anticipated designing a phased booster pump station with three 75-hp pumps with two capable of pumping 1,300-gpm at 300-ft of head, or four 75-hp pumps with three capable of pumping 1,860-gpm at 325-feet of head. Subsequent discussion with the City of Santa Maria and further evaluation of the District's water supply and demand needs have resulted in AECOM recommending a maximum project delivery rate of 2,000-gpm.

5.2 Water Storage Facility

The purpose of the storage facility is to provide operational and emergency storage of Santa Maria water prior to transmission to the Nipomo CSD system. The District selected a partially-buried, prestressed concrete design based on a review of lifecycle costs and aesthetic concerns for both steel and concrete designs.

5.2.1 Reservoir Size

Reservoir volume (1 tank with 500,000 gallon storage capacity) was selected in the PEM.

5.2.2 Aesthetics

The primary reason for a partially buried tank is to meet system hydraulics (discussed in Section 5.2.10). However, a positive secondary benefit to the partially buried tank design is that it will reduce visual impacts. The partially buried tank is designed with the bottom of the tank at an elevation of 278 feet (approximately 22 feet below grade). Approximately 3 to 4 feet of tank wall will be visible above grade. "Native" colors will be evaluated for the tank surface.

5.2.3 Structural

The materials of construction for the proposed reservoir will include reinforced concrete foundations, columns and roof slab. The perimeter ring wall will consist of a cast-in-place concrete wall prestressed by post-tensioned high-strength steel strand. The structural design of the reservoir will be based upon current engineering standards including the American Water Works Association (AWWA) D110-04, entitled "Wire and Strand Wound, Circular, Prestressed Concrete Water Tanks," the current Uniform Building Code, the American Concrete Institute (ACI) 318, and geotechnical criteria relative to foundation materials and site-specific seismic data.

5.2.4 Seismic

The seismic design criteria will be based upon the project site location in seismic Zone 4. The data developed by the geotechnical engineer establishes site ground accelerations based upon native subgrade formations. This project-specific seismic data will also be used for the estimate of hydrodynamic forces on the reservoir perimeter wall associated with seismic ground motions.

5.2.5 Geotechnical

Fugro performed exploratory borings in the area and prepared a soils report for the project site, including recommendations for the design of the buried reservoir. Dune sand deposits were encountered from the ground surface to depths of up to approximately 80 feet below the existing ground surface, the depth to refusal in boring C-7 (see Fugro Report). The dune sand deposits consisted of 8 to 15 feet of soft silt (ML) and loose silty sand (SM) overlying medium dense to dense silty sand (SM) and poorly-graded sand with varying amounts of silt (SP, SP-SM).

Groundwater was not encountered in the borings and CPT soundings performed at the tank and pump station site to the maximum depth explored, which was approximately 80 feet below the existing ground surface. Moist soil was encountered on the tip of the CPT after one of the soundings; however, standing water was not encountered in the sounding. Evaporate minerals and lush vegetation observed on the bluff face, near the contact between the dune sand deposits and the Paso Robles Formation, suggest that groundwater commonly perches on the finer-grained Paso Robles Formation and groundwater may be present at that depth. Variations in groundwater levels and soil moisture conditions will occur depending on changes in precipitation, runoff, irrigation schedules for agriculture, the water elevation in the Santa Maria River, and other factors.

Layers of loose sand and soft silt encountered within the upper 8 to 15 feet are prone to seismic settlement and/or potentially compressible. Grading recommendations provided by Fugro are incorporated into the typical tank cross section shown in the attached drawings. Site preparation and grading to provide uniform support and limit post construction settlements of the proposed structures will be performed according to the recommendations of the geotechnical report.

5.2.6 Site Preparation Recommendations

Fugro recommends removal of existing soil to bottom of soft to firm silt and loose sand or 2 feet below the bottom of the tank, whichever is greater, to expose medium dense to dense dune sand deposits. At least 6 inches of drainage material should be placed below the tank to help stabilize the sandy subgrade. The limits of excavation depicted on the plans incorporate clearance for the prestressing equipment.

5.2.7 Access Openings

A 4' x 6' Bilco-type roof hatch will be placed on the western side of the reservoir, with handrails in the immediate vicinity of the manway. To minimize visual impact, handrails will not extend around the entire roof.

An aluminum ladder outside the reservoir will be provided on the western side of each reservoir to access the roof hatch. A stainless-steel ladder inside the reservoir will also be provided and will incorporate "Safe-T-Climb" type fall protection devices for safety.

5.2.8 Air Vent

An air vent will be located at the center of the reservoir to prevent pressure fluctuations as the water level rises and falls. The vent will be approximately 3 feet in diameter. The vent will incorporate an insect screen to prevent insects and debris from entering the reservoir.

5.2.9 Inlet-Outlet

The tank will have a dedicated 24-inch inlet pipe, and a dedicated 24-inch outlet. The inlet and outlet will penetrate the bottom of the reservoir. A 24x12 inch reducer will increase inlet velocity to facilitate the circulation of water within the tank. The reducer will be accessible to divers should future replacement be required. At the project design flow the tank turnover will occur in approximately 4 hours. A flexible connection between the tank and inlet/outlet piping will be evaluated prior to completion of the 60% design submittal.

5.2.10 Need for Buried Tank

The system hydraulics require that the maximum operating water surface elevation of the tank be at 300 feet or less in order for the tank to fill by gravity from the Santa Maria Reservoirs. If the maximum water surface elevation is established at an elevation of higher than 300 feet Pump Station #1 in Santa Maria would be required.

The maximum operating water surface elevation in the tank was calculated based on hydraulic modeling performed by the City of Santa Maria (Nipomo Community Services District Potable Water Supply Delivery Scenarios, Carollo, 2006). The analysis estimates that an operating pressure range of 60 – 89 psi is anticipated at the connection point in Santa Maria. The pressure at the point of connection is equivalent to a hydraulic grade of 334 feet to 401 feet. Head loss through the transmission main due to friction at 2,000-gpm was estimated using the Hazen Williams equation with a roughness coefficient (C-value) of 135. Based on these calculations and a low pressure of 60 psi at the Santa Maria connection, the maximum operating water surface elevation at the tank was designed at 300 feet to allow for head losses through the transmission main, and to provide sufficient residual pressure for the operation of the flow control valve. Figure 5-1 shows the project Hydraulic Profile.

In the event of an emergency, an 18-inch overflow pipe connected to a 36-inch diameter weir inside the tank will serve as an emergency overflow. The overflow will be set at 301 feet, which is 1 foot higher than the maximum operating water surface elevation of 300 feet. And will allow for approximately 2 feet of freeboard between the roof of the tank (with an inside elevation of 302 feet) and the maximum operating water surface elevation. The overflow will drain to the onsite detention basin.

5.2.11 Reservoir Drain

The configuration of the buried reservoir and the surrounding topography prevents the use of a traditional tank drain line. However, the tank can be emptied by pumping the water level in the tank to within 1 or 2 feet of the floor (depending on the amount of sediment in the tank) and removing the remaining water by using a portable pump. Sumps will be designed into the reservoir floor to facilitate cleaning.

5.2.12 Reservoir Underdrain

A wall drain and slotted PVC underdrain will be installed to prevent water from collecting under or around the reservoir. Underdrains will be arranged such that leaks in the reservoir can be located by observing which underdrain carries water to the drainage vault. The underdrains will drain to an onsite sump that can be pumped out to the onsite detention basin.

5.2.13 Wall Drain

Wall drains convey any water that may accumulate against the outside wall of the reservoir (below grade) to the drainage vault.

5.2.14 Tank Bypass

A bypass will be constructed to allow water to flow directly from the river crossing pipeline into the pump station suction piping, allowing the tank to be taken offline for maintenance while the pump station continues to transfer City water. Operational procedures for this will need to be established between the City and Nipomo.

DWG: W:\Nipomo CSD (19996)\19996.70 Waterline Intertie Project Design\Phase 0001 – Concept Design Report\Task 016 – Concept Design ReporFig 5–1.dwg USER: FroelicherJ
DATE: Apr 27, 2009 3:27pm XREFS: IMAGES:

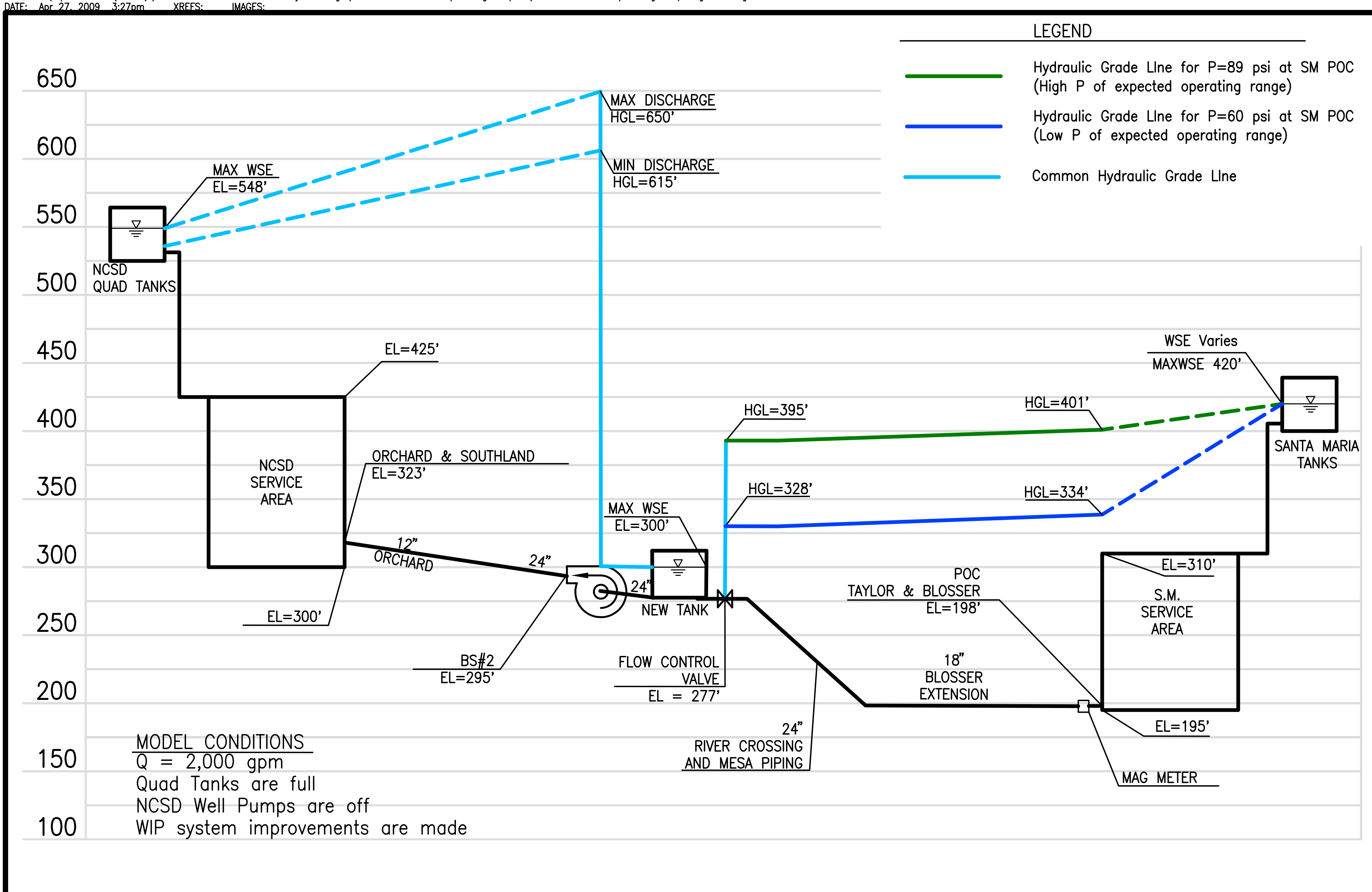

FIGURE
5-1

NCSD WATERLINE INTERTIE

HYDRAULIC PROFILE

AECOM
PROJECT NO.
19996.12

AECOM

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5.3 Site Access Considerations

The site will require routine access by a variety of vehicles including maintenance trucks, chemical supply trucks, and fuel trucks. The site will also require occasional access by a crane to pull the vertical turbine pumps from their cans (the pumps are greater than 25-ft in length). The flow control valve vault may require access by a boom truck. The site has been designed to accommodate these vehicles.

Access to the site is across active farmland in an offered road dedication. At this time the road is an unimproved dirt road. AECOM recommends the District construct a 10 foot wide all weather access road inside the existing road easement. Final alignment and design of the road will need to be discussed and negotiated with the property owner(s).

5.4 Flow Control

AECOM recommends that a detailed operations plan be prepared by NCSD and the City to outline the procedures that will govern operations of the Waterline Intertie Project, including items such as flow control, shutdown, start up, rate of delivery, changes to the delivery rate, and emergency procedures. The recommended option for flow control is an electronic flow control valve. The valve allows the operator to input a desired flow rate, and the flow control valve will throttle flow to maintain the set rate. In addition to controlling the rate of flow through the interconnect, the flow control valve will act to protect the buried tank from overflow by closing in the event that the tank reaches a preset high water level.

5.5 Variable Speed Pumping

The need to handle constant incoming flow rate against varying downstream pressure is an appropriate application for variable speed pumps. Although there are numerous methods for achieving variable speed pumping, variable frequency drives (VFDs) have been selected for the booster station. VFDs are reasonably priced, reliable, and are the most common method of achieving variable speed pumping. Alternatives to VFDs such as internal combustion engine driven pumps, slip couplings, belt drives, and pump control valves[2] are not well-suited for an application that requires high efficiency and reliability and the need to operate for an extended period.

Since the flow out of the tank through the booster station into the District's system will need to match the flow into the tank, the booster station output will be adjusted based on the buried tank level, with the pumps attempting to keep the buried tank level static. The volume of the tank allows for some variance between the flow into the tank and the flow out of the booster station, which will inherently result in the pump station adapting to downstream pressure variations in a controlled manner. The pumps need to match the inflow as closely as possible in order to maintain the tank volume for use during outages, and by keeping the tank level high the pump inlet pressure will be higher allowing the pumps to run more efficiently.

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(1) [2] Pump control valves do not change the pump speed. A pump control valve is placed downstream of the pump, and hydraulic pressure opens and closes the valve to vary flow rate. Pump control valves are effective, and generally reliable, however they are inefficient since the pump's operating point is always greater than needed, with the excess pressure (and electrical energy) being "burned off" by the valve.

5.5.1 Pump Selection

AECOM recommends vertical turbine pumps for the booster pump station. Vertical turbine pumps generally feature steep pump curves (resulting in better modulation of flow rate by VFD) and typically feature higher efficiencies. Most importantly, the pumps can be placed in pump cans which can be at or below the floor elevation of the buried tank, thereby allowing full use of the buried storage tank volume.

5.5.2 System Hydraulics

AECOM prepared Technical Memorandum No. 9: System Pressure Reduction Study in September 2008 (TM 9). TM 9 (attached as Appendix A) investigated the creation of a reduced pressure zone in Nipomo. The system pressure required to fill the Quad Tanks from Joshua & Orchard was determined during the course of that evaluation (Table 2 of TM 9). The required pressure at the intersection of Joshua & Orchard at 10% Average Day Demand (ADD) was determined to be 151 psi and 147 psi at Peak Hour Demand (PHD). Since the Booster station is not located at Joshua & Orchard, the losses were calculated in the proposed pipeline between Orchard & Joshua and the actual tank site. This information, along with the high and low Quad Tank water elevations, resulted in a required hydraulic grade line (HGL) elevation that varies between 615 feet and 650 feet. Since the pumps and pump cans are buried at an elevation of approximately 271 feet the required head from the pumps is between 344 feet (149 psi) and 379 feet (379 psi).

Figure 5-2 shows the likely range of system curves and an appropriate pump curve for the booster station based on the hydraulic grade of the system. The upper system curve, the “worst case scenario”, represents the scenario when the Quad Tanks are full, the buried reservoir is at low water elevation, and 10% ADD flow conditions are occurring in the distribution system. In the worst case scenario the pumps are producing the least amount of flow. The lower system curve represents the Quad Tanks at a low water level, the buried reservoir full and the system experiencing PHD, which is the “best case scenario”. In this scenario the pumps will produce their maximum flow.

5.5.3 Expected Range of Flow

The project design flow is 2,000 gpm but the booster station will be capable of delivering a range of flows from 600-gpm up to 2,000-gpm. Figure 5-2 shows the system curve with a suitable 100-hp vertical turbine pump curve. The pump curves are only shown at full speed in this figure. At full speed, three pumps will operate at about 2,050-gpm during the worst case scenario and a single pump is capable of about 700-gpm at the worst case scenario. However, when system conditions are more favorable the single pump will produce in excess of 900-gpm, which is more than required. The use of VFDs will allow the performance of this pump to be reduced to meet demand conditions, as shown in Figure 5-3.

By using VFDs to reduce the pump speed to 1670-rpm (an 8% reduction in speed) the pump is capable of delivering approximately 600-gpm. The best efficiency capacity (BEC) of this pump is 840-gpm; therefore it should not be operated at less than 590-gpm or about 30% less than the BEC. Individual pump manufacturers will have varying requirements for low flow limitations to prevent low flow cavitation from damaging the pump.

The 100 horsepower Fairbanks Morse pump shown on these curves meets the requirements of the pump station, and is used as the basis for laying out the pump station cans and manifolding; however, other pumps are available, and the pumps will not be sole-sourced in the plans and specifications.

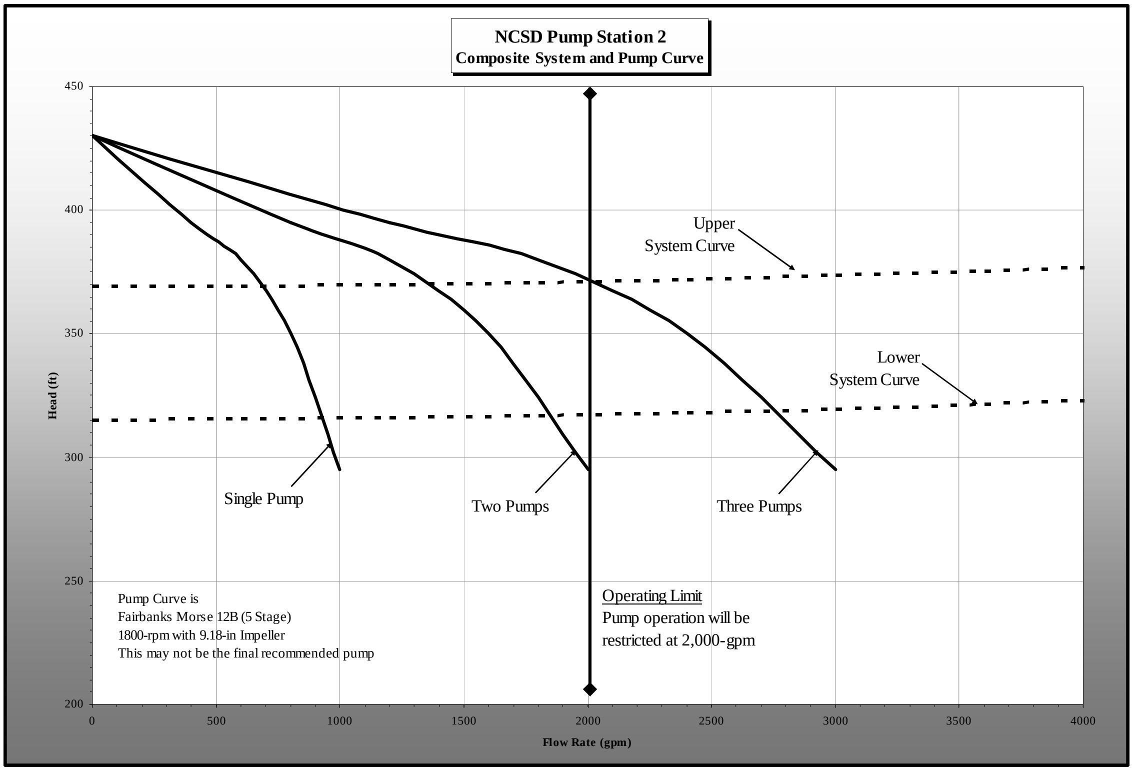


Figure 5-2. Composite System Curve and Full Speed Pump Curves

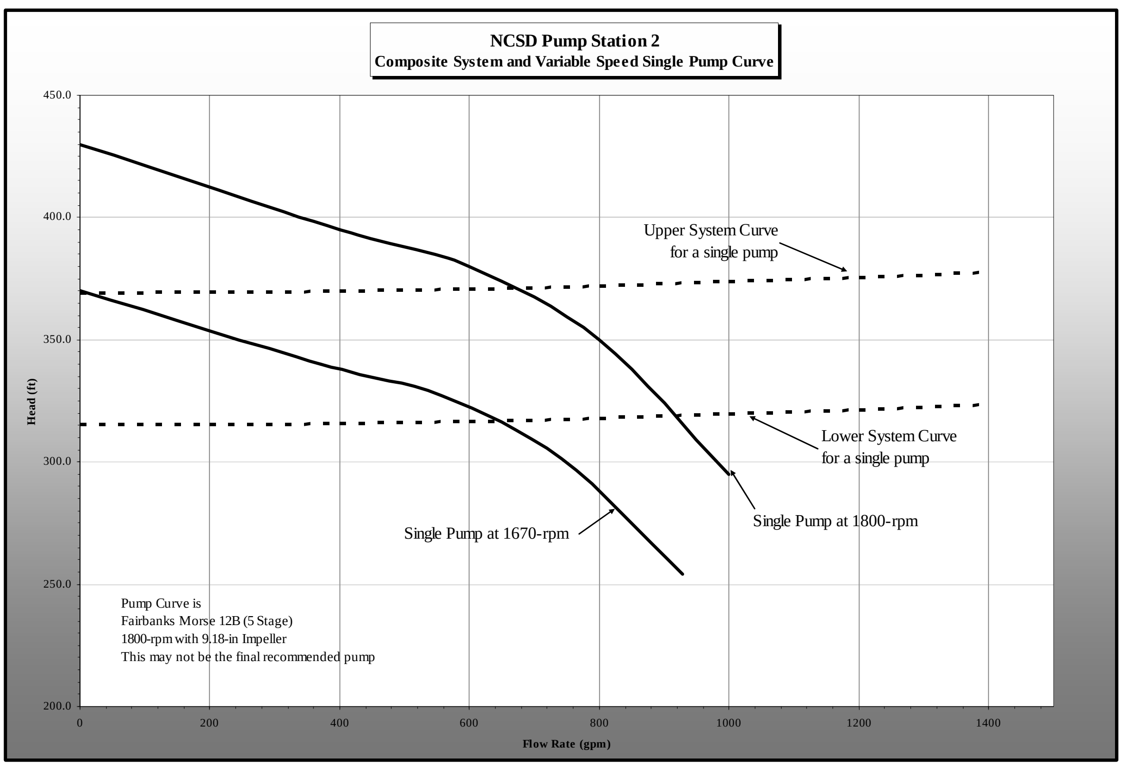


Figure 5-3. Composite System Curve and Variable Speed Pump Curve

5.5.4 Control of Pumps

The booster station will be designed to match the flow entering the buried tanks. To accomplish this, the level in the buried reservoir will be used to pace the pumps. A pressure transducer in the tank will be read by the programmable logic controller (PLC) and SCADA system, and the change of the tank level over time will be used to increase or decrease speed of the pumps and to call additional pumps as required. Since the inflow to the booster station tank will be set in advance and will remain constant, the pumps will vary flow by responding to fluctuation of the level in the buried reservoir thereby accommodating to changes in demand and fluctuating levels in the Quad Tanks. These changes in speed will be made gradually to prevent surge or other abnormal pressure and flow fluctuations.

The Quad Tanks level will be used to send an emergency “off” signal to the booster station in the even that a high water level alarm is detected at the Quad Tanks. A low water level in the buried reservoir will shut the booster pumps off to prevent cavitation in the pumps. Prior to reaching either of the “off” signals, a warning alarm will be sent to the District Operations Staff.

A magnetic flow meter will be installed downstream of the booster station as a check against the magnetic flow meter installed in Santa Maria (the official meter for determining water use). The meter at the booster station site will be used to monitor the flow rate of the booster station and as a verification of water delivery.

5.6 Pump Station Design Features

Additional design features that will be incorporated into the design are listed below:

5.6.1 Trouble shooting considerations

- Manual pressure gauges on suction and discharge piping
- Easy access to Motor Control Center (MCC) and VFDs when working on pumps and motors
- Chemical dosing pumps and residual meters located in the same room as the MCC and booster pumps
- Doors allowing easy access between the MCC, residual meters, and flow meter/ residual vault
- Access to PLC to make set point changes and/or examine trends over several days (stand alone and redundant)

5.6.2 Piping and Pump Can Layout

- Adequate clearance to access the check valves, valves, or motors
- Ability to lift a 27-ft long pump through the ceiling

- A rollup door that will allow easy access to the motors and pump cans
- MCC clearance

5.6.3 Other Items

- Chemical Storage should be isolated from the electrical and mechanical rooms
- Drainage of incidental water will be provided
- The building will be ventilated

5.7 Hydraulic Transient Analysis

In order to evaluate the potential hydraulic transients resulting from the proposed booster station, AECOM performed a preliminary surge analysis of the WIP including elements of NCSD's existing potable water distribution system. The objective of this analysis was to simulate and estimate the worst-case theoretical transient response (pressure increase / decrease over time) of the proposed booster station, new transmission pipelines, and elements of the NCSD's existing water distribution system following simultaneous stopping of pumps due to power failure.

A hydraulic transient (also known as water hammer or surge) is a temporary flow and pressure condition that occurs in a hydraulic system following a rapid velocity change in response to the operation or shut-down of a flow-control device (for instance, pump shut down following power failure). When velocity changes rapidly at the onset of a surge event, the compressibility of the liquid and the elasticity of the pipeline cause a transient pressure wave to propagate throughout the system. If the magnitude of this transient pressure wave and the resulting transient flow variation is great enough and adequate transient-control measures are not in place, a transient can cause system hydraulic components, including pipelines, to fail.

The preliminary surge analysis was performed using Bentley's HAMMER V8 XM Edition software. To serve as the starting point for the surge model, the NCSD's existing water distribution model was imported into the Hammer program from WaterCAD. Once imported, the model was updated to include the proposed WIP Improvements and skeletonized to provide a manageable representation of both the existing and proposed system elements.

The following assumptions were made to define initial conditions and the modeled surge event:

5.7.1 Initial Conditions

- Proposed Buried Reservoir and Quad Tanks at 50% water levels
- Existing wells not active
- Proposed Booster Station is active
    - All three (3) pumps operating; 2,000 gpm total flow at 330 ft, 1800 rpm.

5.7.2 Modeled Surge Event

Two (2) scenarios were run to evaluate the potential system surge response with and without surge protection. In order to characterize the surge envelope for each scenario, it was determined that each scenario needed to be run with a range of flows to estimate the potential surge behavior under different system demand conditions. To provide the required range of results, both scenarios were run under existing system Peak Hour Demand (PHD); and at 10% Average Day Demand (10% ADD). The following conditions were simulated for each surge scenario:

- Scenario 1 – System with no surge protection at both 10% ADD and PHD
    - Power failure at booster station – three (3) duty pumps simultaneously stop following power failure at time T = 0
    - Pump discharge check valves close when reverse flow is first sensed
    - Reverse pump spin not allowed
    - No surge protection
- Scenario 2 – System with surge protection at pump station discharge both 10% ADD and PHD
    - Power failure at booster station - three (3) duty pumps simultaneously stop following power failure at time T = 0
    - Pump discharge check valves close when reverse flow is first sensed
    - Reverse pump spin not allowed
    - Surge protection = 1,000-gallon bladder hydro-tank set at 50-psi initial pre-charge pressure

5.7.3 Surge Analysis Results

Surge analysis results at select locations along the transmission route are summarized below in Table 5-1. Detailed *HAMMER* Transient Reports are not attached to this memo due to size, but can be provided on DVD if requested. The range of values presented in the tables are based on combined results from runs at PHD and 10%ADD

Table 5-1. Surge Analysis Results for Scenario 1 - System with no surge protection

Location	**Elevation (ft)**	**Steady-state HGL (ft)**	**Up Surge HGL (ft)**	**Down Surge HGL (ft)**
WIP Pump Discharge Header	300	616 - 633	648 - 659	267 – 360 **(-14 psi min)**
Connect to Ex. @ Santa Maria Vista Rd (node J-5298)	302.3	614 - 631	614 - 646	288 – 364 **(-6 psi min)**
Connect to Ex. @ Orchard & Southland (node J-8539)	323	546 - 574	546 - 599	405 - 438
Connect to Ex. @ Oakglen & Tefft (node J-3411)	333.3	534 - 558	534 -575	459 - 470
Connect to Ex. @ Orchard & Grande (node J-3004)	374.4	534 - 564	534 - 580	441 - 477

Table 5-2. Surge Analysis Results for Scenario 2 - System with 1,000-gallon bladder hydro-tank

Location	**Elevation (ft)**	**Steady-state HGL (ft)**	**Up Surge HGL (ft)**	**Down Surge HGL (ft)**
WIP Pump Discharge Header	300	616 - 633	655 - 677	420 – 441 (52 psi min)
Bladder Hydro-Tank (node J-8599)	300	616 - 633	628 - 646	421 – 443 (52 psi min)
Connect to Ex. @ Santa Maria Vista Rd (node J-5298)	302.3	614 - 631	621 - 631	425 - 443
Connect to Ex. @ Orchard & Southland (node J-8539)	323	546 - 574	549 - 581	468 - 486
Connect to Ex. @ Oakglen & Tefft (node J-3411)	333.3	534 - 558	534 - 567	492 - 512
Connect to Ex. @ Orchard & Grande (node J-3004)	374.4	534 - 564	538 - 578	479 - 497

As shown in Table 5-1, potential vacuums (negative gauge pressures) may develop between the proposed booster station discharge header and part of the existing 12-inch PVC pipeline near Santa Maria Vista Rd/Orchard Rd. In comparison, pressure increases resulting from the upsurge are within 50-ft (22 psi) at the locations shown. Table 5-2 demonstrates that the inclusion of a bladder hydro-tank at the booster station discharge could be an effective means of attenuating the potential vacuum that can develop within the discharge pipeline during a down surge. Furthermore, this surge mitigation device reduces the spike in pressure resulting from upsurge as graphically represented in the figures below.

In order to graphically depict the surge analysis results, surge profiles were prepared for both scenarios showing the pressure variation along the primary transmission route (from the booster station to Southland/Frontage to Quad Tanks) and are included on the following pages. These profiles include the theoretical "steady state" HGL, Max Surge HGL, and Min Surge HGL as calculated by *HAMMER*. The Vacuum Lower Limit HGL is also included to represent the theoretical limits for "full vacuum".

Figure 5-4 is the surge analysis profile for Scenario 1 (System with no surge protection). Figure 5-5 is the surge analysis profile for Scenario 2 (System with surge protection at pump station discharge).

Figure 5-4 depicts a potential downsurge vacuum (approaching full vacuum at -14 psi) that may develop between the proposed booster station and the first 2,000-lf of existing 12" PVC along Orchard Rd. Pressure spikes resulting from upsurge are within 50-ft. The maximum pressure along the profile is 160 psi (located downstream of the new pump station).

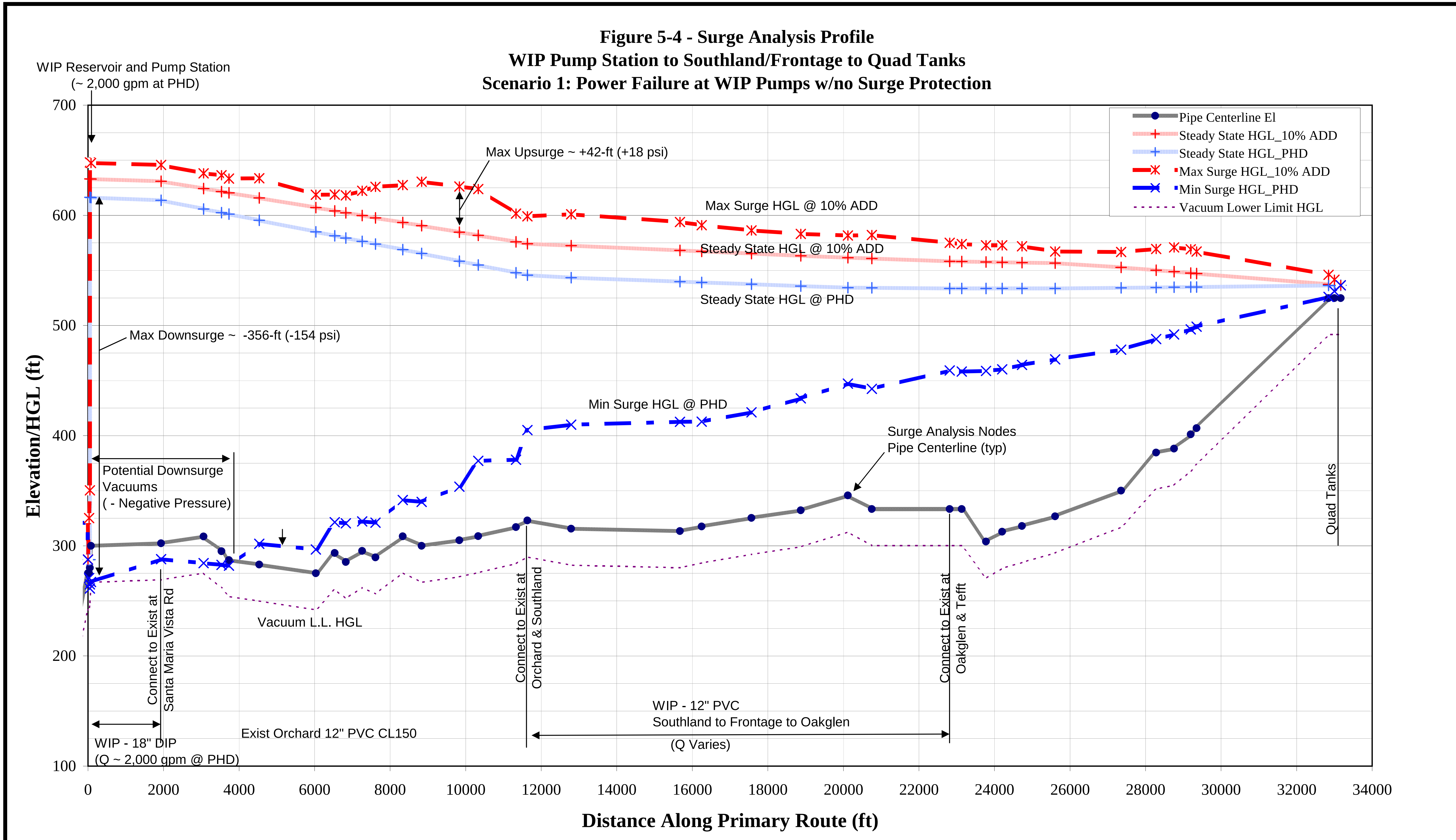
Figure 5-5 depicts a potential downsurge resulting in elevated minimum pressures in excess of 50 psi downstream of the booster station. Pressure spikes resulting from upsurge are reduced when compared to Figure 5-4 and approximate the Steady State HGL. The maximum pressure along the profile is 170 psi (located downstream of the proposed booster station).

5.7.4 Surge Control Recommendations

Based on this analysis, a surge control system including a bladder hydro-tank is recommended for the proposed booster station. Further analysis is required during the next stage of design in order to determine the optimum bladder hydro-tank size, bladder pre-charge pressure, and configuration. The drawings show a 1,000 gallon tank connected to the booster station discharge piping.

During review of the Draft Concept Design Report, a question was raised about the potential for surge in the case where the reservoir is offline and water is being delivered directly to the pump station. If hydraulic analysis of this scenario reveals the need for additional surge control measures, some options include a surge control valve or a surge tank located upstream of the pump station. AECOM will address this question in a Technical Memorandum prior to the 60% project design submittal.

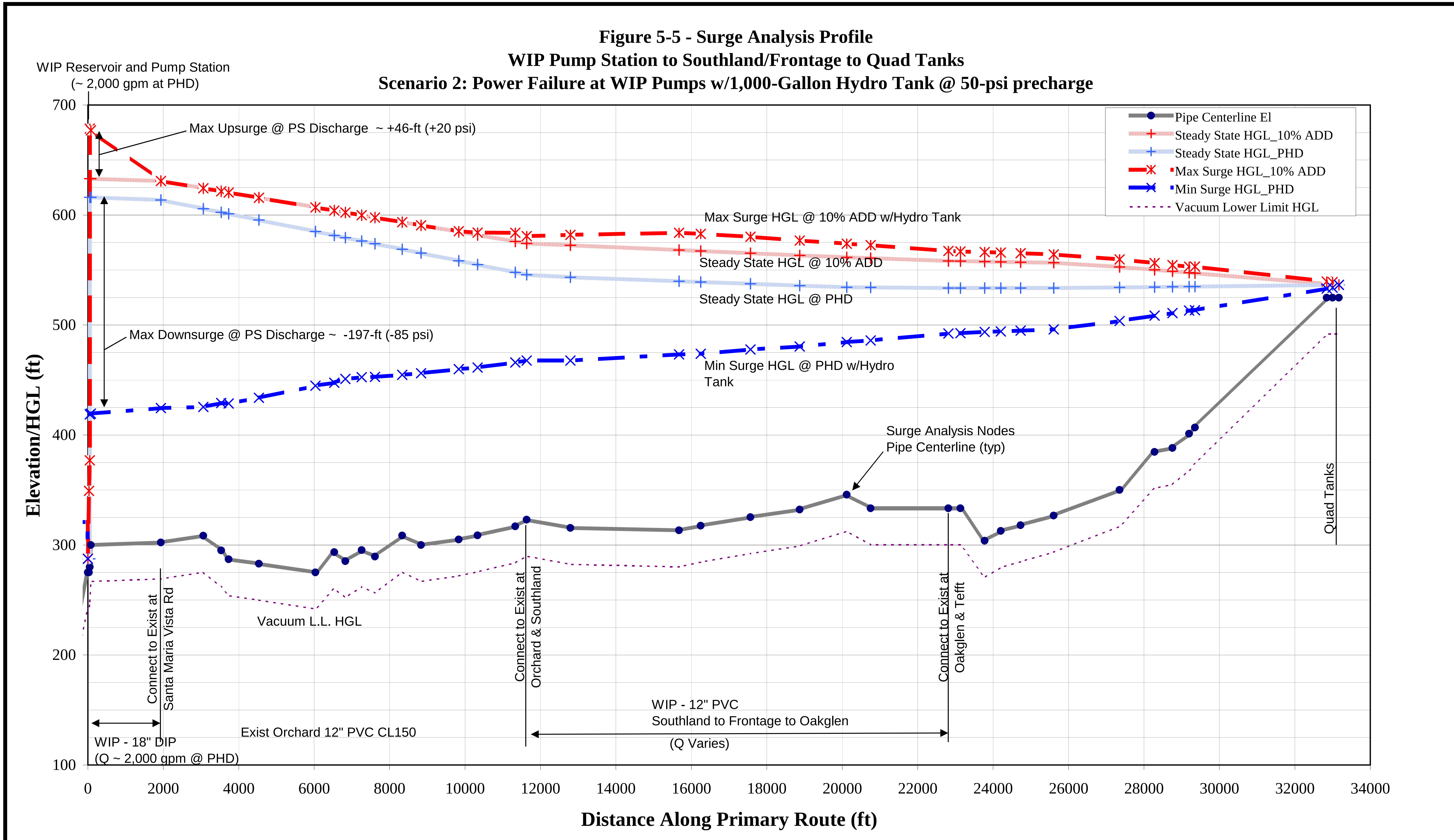
Client: NCSD
Job Title: WIP
Job #: 60061295



AECOM

Figure 4_Scenario 1_Surge Profile.xls

Client: NCSD
Job Title: WIP
Job #: 60061295



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5.8 Chloramine Boosting at Intertie

Chloramine boosting will allow addition of chloramine to Intertie Water to provide adequate chloramine residual at the farthest reaches of the distribution system. The chloramine booster facilities will be located at the booster station site and will inject sodium hypochlorite and ammonia into the water. Influent flow rate is assumed to be constant (approximately 1,000 gpm to 2,000 gpm), and may vary daily.

5.8.1 Sodium Hypochlorite Storage

AECOM recommends double-walled high density polyethylene containment vessels for sodium hypochlorite storage. Typical chloramine residuals should be above 1.0 mg/l at the end of the distribution system, and may be as high as 3.0 mg/l at the point of treatment.[3] For facility sizing, it was assumed that the chloramine residual of the water entering the booster station has been declined to 1.0 mg/L and will be boosted to 2.5 mg/L. We have therefore assumed a required chlorine dosage (as hypochlorite ion (OCl)) of 1.5 mg/l to achieve an initial chloramine residual of 2.5 mg/l (as OCL), and assuming a 1:4 ratio of NH_3 to OCl by weight to prevent formation of di- and tri-chloramines.

A standard tank size was selected to provide suitable storage capacity of 12.5% sodium hypochlorite solution. At 12.5% concentration, sodium hypochlorite is unstable and will deteriorate in strength with time. The rate at which the solution loses strength is accelerated by exposure to high temperatures and sunlight. It is therefore recommended that no more than 14 days of supply be kept on hand. Also, the tank level should be as low as possible before it is replenished. Recommended tank sizes and approximate maximum storage capacities are shown in Table 5-3.

Table 5-3. Recommended Sodium Hypochlorite Tank Size (12.5% Solution)

Design Flow Rate (gpm)	2000
Approximate NaOCL Usage (gal/day)	36
Recommended NaOCl Storage (gal)	1000
Maximum Tank Capacity (days)	27

5.8.2 Ammonia Storage

For ammonia storage, AECOM recommends the “Ultratainer” intermediate bulk container from Snyder Industries. These polyethylene containers are rugged, relatively inexpensive, and can resist the vapor pressure of ammonia, thereby eliminating the need for a scrubber.[4] The Ultratainer would be fitted with a pressure relief valve to prevent over pressurization of the tank.

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[3] “Waterline Intertie Project – Disinfection Alternatives Evaluation,” Boyle Engineering Corporation, November 2006.
[4] Ammonia volatizes easily at room temperatures and must either be refrigerated, contained in a pressure vessel, or vented to a scrubber which consists of a volume of water through which the ammonia vapor passes. A scrubber would require maintenance, connection to a fresh water supply, and connection to sanitary sewer.

Since the Ultratainer is a single-walled tank, a containment curb will be provided in the chemical storage building.

Table 5-4. Recommended Tank Size for Saturated Ammonium Sulfate Solution

Design Flow Rate (gpm)	2000
Approximate Ammonium Sulfate Solution Usage (gal/day)	8
Recommended NH_3 Tank Size (gal)	220
Maximum Tank Capacity (days)	28

AECOM has assumed that the District will mix dry ammonium sulfate with water at the booster pump station on an as-needed basis. We have also assumed that a saturated solution of ammonium sulfate (14% ammonia) will be used.[5] Assuming a 1:4 ratio of NH_3 to OCl by weight for chloramine production, ammonia storage requirements were calculated. Maximum storage capacity is shown in Table 5-4, but actual chemical volume may be reduced in practice to avoid degradation of chemical strength.

5.8.3 Chemical Delivery Equipment

Technical specifications will be provided for materials, testing, and installation of a packaged, skid-mounted chemical feed system for sodium hypochlorite and aqueous ammonia. Components will include:

- Electronic actuated diaphragm metering pumps to pump the chemical from the storage tank to the point of application. Equipment will be designed with fully redundant duplex metering pumps. Pumps will be designed to supply the maximum required chemical dose with only one of the two pumps operating.
- Pulsation damper, pressure gauges, flow switch, isolation, backpressure, pressure relief, solenoid, and control valves within the on-skid piping.
- Electrical power and control wiring and conduit between the above components.

The design will include a control panel for local control of the feed system and junction boxes and terminals for terminating alarm and control signal to or from an external control system.

Input terminals will be provided for the following:

1. Connecting to external power
2. 4– to 20-mA flow-pacing signals to metering pumps

[5] A saturated solution of ammonium sulfate contains approximately 138 mg of ammonia per liter of water at 20 degrees C (approximately 14% solution),

3. Metering pumps on/off signal

Output terminals will be provided for:

1. Sending alarm and control signals to an external PLC
2. Metering pump running (separate signal for each pump)
3. Flow switch in each metering pump discharge piping enabled/disabled
4. High pressure in metering pump discharge
5. Liquid level detected in drain pan

The design will include specifications for ammonia injection quills on process piping, and will feature a Westfall static mixer.

Pacing of the sodium hypochlorite and ammonia feed pumps will be accomplished manually. Adjustments to the metering pump stroke frequency and stroke length will be made at the metering pump local controls. The feed pumps will require regular adjustment based on measured values of the total chlorine residual, free chlorine residual, monochloramine residual, and free ammonia residual. The frequency of metering pump adjustment will depend on the consistency of intertie flow rate, the consistency of the intertie water characteristics, and the drift in the pump set point.

5.8.4 Chloramine Residual Monitoring Equipment

To monitor chloramine residual, AECOM recommends use of two Hach CL-17 colorimetric chlorine analyzers. With one meter utilizing reagent to measure Free Chlorine, and one meter utilizing reagent to measure Total Chlorine, the chloramine residual can be calculated (Total Chlorine - Free Chlorine = Combined Chlorine (chloramine)). This method requires careful monitoring of the dosage ratio of hypochlorite to ammonia, as the calculation of combined chlorine cannot distinguish between monochloramine, di and trichloramine, or free chlorine in Zone 3 of the breakpoint curve. Staff will need to ensure that the dosage ratio is kept between 4:1 and 5:1 (chlorine : free ammonia) to ensure that the measured value of Combined Chlorine represents monochloramine concentration. Each meter will have an associated waste stream. Since there are no sewer connections at the site, AECOM will include design of a small drywell for disposal.

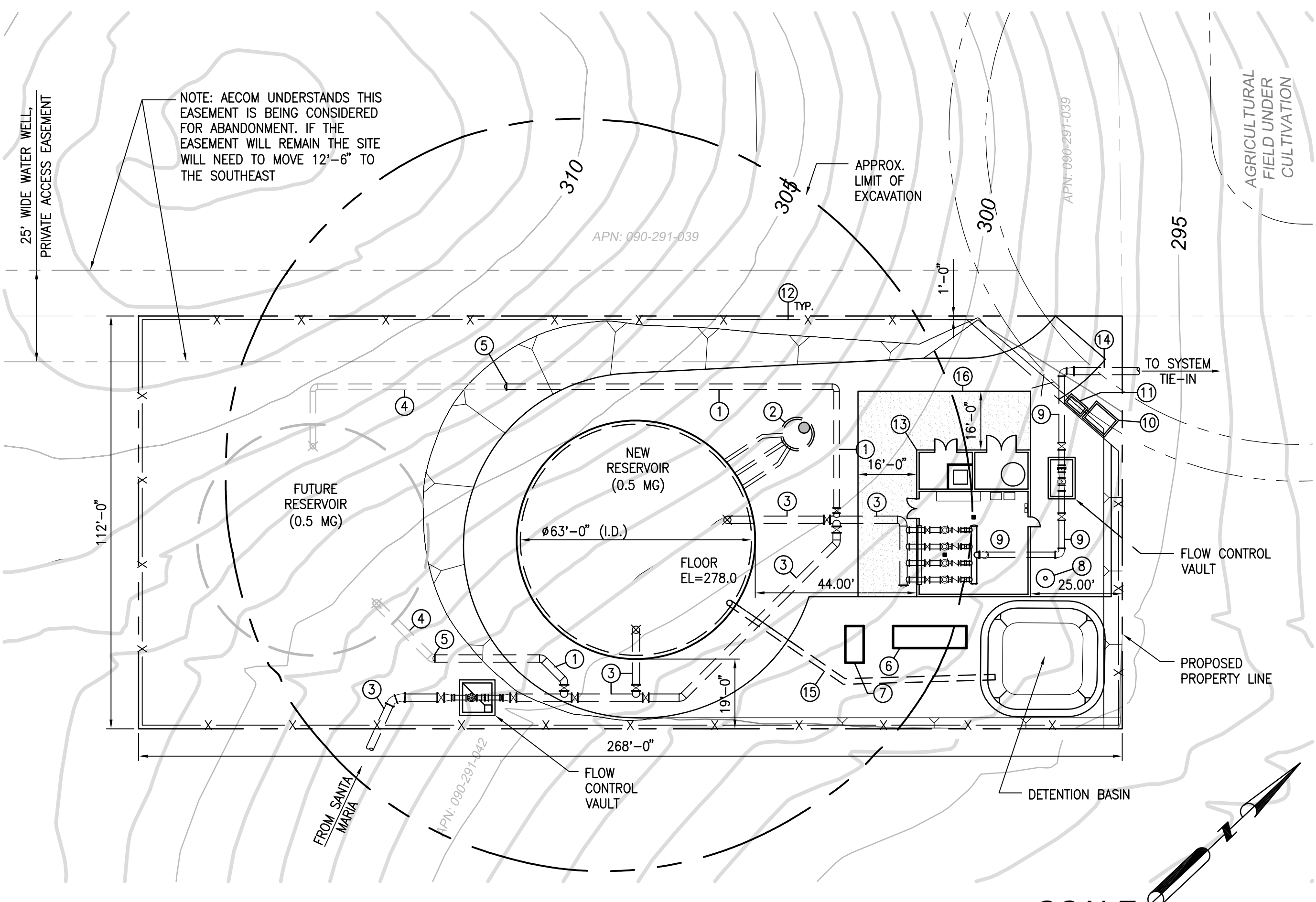
Two sets of meters will be provided, one set to monitor water received from the City, and one set to monitor water being delivered to the District's distribution system. The dual set of meters will allow a high level of control over the settings of the chemical feed pumps and control over the quality of water delivered to the system. The source water meters will also ease troubleshooting and establishment of the proper set point(s).

Meters are available that measure chloramine concentration directly. However, in our experience, these meters (such as the Hach APA 6000 ($10,000), or the ChemScan UV-2150/S ($25,000-$30,000)) are expensive and are marginally reliable. AECOM does not recommend these types of meters for NCSD.

5.9 Pump Station Building

AECOM recommends split-faced masonry block construction for the pump station building. Preliminary layouts indicate a required building size of approximately 28 feet by 45 feet to accommodate the chemical storage and pump station layout (see Figure 5-6). The contract documents will require the contractor to submit color swatches and coordinate color selection with District staff.

Storage buildings will feature two isolated rooms to provide separation between treatment chemicals. Proposed locations for the chemical storage buildings can be seen in the preliminary tank site drawings (included with the 30% Design Plans in Volume 3 of this Report).



SCALE

0 25 50 100

1 INCH = 50 FEET

NOTES:

1. FOR DETAILED INFORMATION SEE BP-4, SHEET C-114.
2. GRADING IS PRELIMINARY, TO BE REVISED IN 60% SUBMITTAL

CONSTRUCTION NOTES:

(1) 18"ø PIPE
(2) DRAINAGE MANHOLE
(3) 24"ø PIPE
(4) FUTURE 18"ø PIPE
(5) 18" BLIND FLANGE
(6) EMERGENCY GENERATOR
(7) GENERATOR FUEL TANK
(8) SURGE TANK
(9) 18"ø PIPE
(10) PG&E PAD MOUNTED TRANSFORMER
(11) METERING SWITCHBOARD
(12) 6'-0" HIGH CHAIN LINK FENCE WITH THREE STRANDS OF BARBED WIRE
(13) PUMP STATION AND CHLORAMINE BOOSTER BUILDING
(14) 24"ø PIPE
(15) 18"ø DIP TANK OVERFLOW
(16) ASPHALT APRON

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WATERLINE INTERTIE PROJECT	AECOM PROJECT NO.	FIGURE
PUMP STATION AND RESERVOIR LAYOUT	19996.70	5-6

DWG: W:\Nipomo CSD (19996)\19996.70 Waterline Intertie Project Design\Phase 0001 - Concept Design Report\Task 016 - Concept Design Report\Final Concept Design Report\Figures\Fig 5-6.dwg Layout Name: Fig 5-6 - Plotted by: Froelicher, Jim Date: 4/27/2009 - 12:42 PM
XREFS: Surface-Section2+TankV2 - WallaceBase - TANK SITE4 IMAGES: Santa Maria Waterline-1.sid - Santa Maria Waterline-2.sid -

6.0 CHLORAMINATION SYSTEMS

Technical Memorandum No. 7 – Chloramination Systems, submitted on July 16, 2008, made recommendations for the selection of wells for chloramination conversion. Subsequently, Technical Memorandum 7b, submitted on November 21, 2008, provided a summary of the preliminary design considerations and made recommendations for the design of the chloramination facilities at the selected wells. This Chapter combines information from the technical memoranda to summarize the Concept Design for the wellhead chloramination systems. The chloramination system for the booster station is discussed in Chapter 5.0 with the pump station and reservoir.

6.1 Identification of wells for chloramination

Because the supplemental water will contain chloramines, the District will convert its existing free chlorination treatment process to a chloramination system. This change in treatment will require the addition of ammonia injection at the wells, and the redesign of the chlorine feed systems because of the higher total chlorine residual typically maintained. This change will also require larger chlorine solution tanks and chemical feed pumps with greater capacities. Each well that is converted to chloramines will need online monitoring equipment to provide dosage control, as well as a building sized large enough to hold the two solution tanks and four chemical feed pumps (two primary and two backup).

In order to reduce costs, and because the introduction of supplemental water will reduce the need to pump groundwater, it may be possible to convert some of the District's wells to chloramine disinfection and reduce the use of the other wells. These other wells could be retired from service until such time as they were needed, or they could be operated periodically, using a portable chloramination system. At this time the District has budgeted to construct four chloramination facilities and one portable system. Findings from the 2007 Water Master Plan Update and from recent pumping records are summarized in Table 6.1 below.

Table 6-1. NCSD Production Well Information

Location	Rated Capacity	2007 Production	Other Features
Eureka	820 - 965 gpm	761 AF	Well Building
Via Concha	700 - 800 gpm	750 AF	Well Building
Bevington	330 - 405 gpm	358 AF	Well Building
Olympic	110 - 150 gpm	17 AF	
Church	130 - 160 gpm	12 AF	inactive
Sundale	800 - 1,200 gpm	374 AF	Well Building Natural Gas Powered
Knollwood	210 - 270 gpm	259 AF	
Blacklake #3	120 – 210 gpm	90 AF	
Blacklake #4	300 - 450 gpm	233 AF	Recently refurbished
Mandi	n/a		(construction incomplete)
Cheyenne	n/a		(construction incomplete)

6.1.1 Need to Meet Maximum Day Demand

It is recommended that wells be selected for conversion to chloramination to meet the maximum daily demand of the District. In this way, the District will be able to provide water to its customers during times that the Intertie may be inoperative due to emergency operations in the City of Santa Maria, or due to maintenance or repair of the Intertie itself.

The maximum daily demand was estimated (in the 2007 Water Master Plan update) to be 4.53 MGD (3,152 gpm) in 2007, and is projected to grow to 9.47 MGD (6,575 gpm) in 2030.

6.1.2 Well Capacity used to select Wells

The District has budgeted to install chloramination facilities at four wells, plus one portable unit, for a total of five wells. To determine which wells should receive the permanent chloramination equipment the wells were ordered from largest to smallest, based on the mid-value of their reported capacity, as shown in Table 6-2.

Table 6-2. NCSD Production Well Capacity Ranges

Location	Minimum reported Capacity (gpm)	Maximum Reported Capacity (gpm)	Average Reported Capacity (gpm)	Cumulative Minimum Capacity (gpm)	Cumulative Maximum Capacity (gpm)
Sundale	800	1200	1000	800	1200
Eureka	820	965	893	1620	2165
Via Concha	700	800	750	2320	2965
Blacklake #4	300	450	375	**2620**	**3415**
Bevington	330	405	368	2950	3820
Knollwood	210	270	240	3160	4090
Blacklake #3	120	210	165	3280	4300
Church	130	160	145	3410	4460
Olympic	110	150	130	3520	4610

Under this approach, Sundale, Eureka, Via Concha, and Blacklake #4 wells would be recommended for permanent chloramination facilities (Figure ES-2). Together these wells would produce between 2620 and 3415 gpm. With a portable unit operating at Bevington or Knollwood, between 2950 and 3090 gpm would be produced. It is very likely that this approach would produce sufficient water to meet the year 2007 maximum daily demand of 3,152 gpm.

In order to meet the existing maximum day demand (3,152 gpm), AECOM recommends that the District install chloramination equipment at Sundale, Eureka, Via Concha, and Blacklake #4 wells.

6.2 Selected Wells

As recommended the District has selected four wells for conversion to chloramination. It was recommended that the selected wells have adequate capacity to meet the maximum daily demand.[6] In this way, the District would be able to provide water to its customers during times that the Intertie may be inoperative due to emergency operations in the City of Santa Maria, or due to maintenance or repair of the Intertie itself. The selected wells and production rates are shown in Table 6-4.

[6] The maximum daily demand was estimated (in the 2007 Water Master Plan update) to be 4.53 MGD (3,152 gpm) in 2007, and is projected to grow to 9.47 MGD (6,575 gpm) in 2030.

Table 6-4. District Wells Selected for Chloramination Facilities

Location	Minimum reported Production Rate (gpm)	Maximum Reported Production Rate (gpm)	Average Reported Production Rate (gpm/MGD)
Sundale	800	1200	1000 / 1.4
Eureka	820	965	893 / 1.3
Via Concha	700	800	750 / 1.1
Blacklake #4	300	450	375 / .54

These four wells, along with the Bevington or Knollwood Well (if a chloramination system is installed in the future) would likely produce sufficient water to meet the year 2007 maximum daily demand of 3,152 gpm (4.5 MGD).

6.3 Sodium Hypochlorite Storage

AECOM recommends double-walled high density polyethylene containment vessels for sodium hypochlorite storage. Based on the 2007 Water Master Plan Update, we have assumed that chlorine demand at each of the wells is low (0.5 mg/l). Typical chloramine residuals should be above 1.0 mg/l at the end of the distribution system, and may be as high as 3.0 mg/l at the point of treatment.[7] We have therefore assumed a required chlorine dosage (as hypochlorite ion (OCl)) of 3.5 mg/l to achieve an initial chloramine residual of 3.0 mg/l (as OCL), and assuming a 1:4 ratio of NH_3 to OCl by weight to prevent formation of di- and tri-chloramines.

Standard tank sizes were selected to provide suitable storage capacity of 12.5% sodium hypochlorite solution. Sodium hypochlorite, at a 12.5% concentration, is unstable and will deteriorate in strength with time. The rate at which the solution loses strength is accelerated by exposure to high temperatures and sunlight. For this reason, we are locating storage tanks within buildings. It is also recommended that no more than 14 days of supply be kept on hand. Also, the tank level should be as low as possible before it is replenished. Recommended tank sizes and approximate maximum storage capacities are shown in Table 6-5.

[7] "Waterline Intertie Project – Disinfection Alternatives Evaluation," Boyle Engineering Corporation, November 2006.

Table 6-5. Recommended Sodium Hypochlorite Tank Sizes (12.5% Solution)

	Sundale	Via Concha	Blacklake 4	Eureka
Max Flow Rate	1200	800	450	965
Existing NaOCl Storage (gal)	500	45	10	45
Recommended NaOCl Storage (gal)	1000	500	360	500
Tank Manufacturer	Poly Processing	Snyder Industries	Snyder Industries	Snyder Industries
Approximate Storage Max (days)	19	14	18	12

6.4 Ammonia Storage

For ammonia storage, AECOM recommends the "Ultratainer" intermediate bulk container from Snyder Industries. These polyethylene containers are rugged, relatively inexpensive, and can resist the vapor pressure of ammonia, thereby eliminating the need for a scrubber.[8] The Ultratainer would be fitted with a pressure relief valve to prevent over pressurization of the tank. Since the Ultratainer is a single-walled tank, a containment curb will be provided in the chemical storage building.

We have assumed that the District will mix dry ammonium sulfate with water at the well sites on an as-needed basis. We have also assumed that a saturated solution of ammonium sulfate (14% ammonia) will be used.[9] Assuming a 1:4 ratio of NH_3 to OCl by weight for chloramine production, ammonia storage requirements were calculated for each site. The smallest Ultratainer (220 gal) was selected for each site. Maximum storage capacities are shown in Table 6-6, but actual chemical volume may be reduced in practice to avoid degradation of chemical strength.

Table 6-6. Recommended Ammonia Solution Tank Sizes

	Sundale	Via Concha	Blacklake 4	Eureka
Max Flow Rate	1200	800	450	965
Recommended NH_3 Tank Size (gal)	220	220	220	220
Maximum Storage (days)	20	30	>30 (53 days)	24

[8] Ammonia volatizes easily at room temperatures and must either be refrigerated, contained in a pressure vessel, or vented to a scrubber which consists of a volume of water through which the ammonia vapor passes. A scrubber would require maintenance, connection to a fresh water supply, and connection to sanitary sewer.

[9] A saturated solution of ammonium sulfate contains approximately 138 mg of ammonia per liter of water at 20 degrees C (approximately 14% solution),

6.4 Chemical Delivery Equipment

Technical specifications will be provided for materials, testing, and installation of packaged, skid-mounted chemical feed systems for sodium hypochlorite and aqueous ammonia. Components will include:

- Electronic actuated diaphragm metering pumps to pump the chemical from the storage tank to the point of application. All chloramination sites will be designed with fully redundant "duplex" metering pumps. Pumps will be designed to supply the maximum required chemical dose with only one of the two pumps operating.
- Pulsation damper, pressure gauges, flow switch, isolation, backpressure, pressure relief, solenoid, and control valves within the on-skid piping.
- Electrical power and control wiring and conduit between the above components.

Our design will include a control panel for local control of the feed system and junction boxes and terminals for terminating alarm and control signal to or from an external control system.

Input terminals will be provided for the following:

- Connecting to external power
- 4– to 20-mA flow-pacing signals to metering pumps
- Metering pumps on/off signal

Output terminals will be provided for:

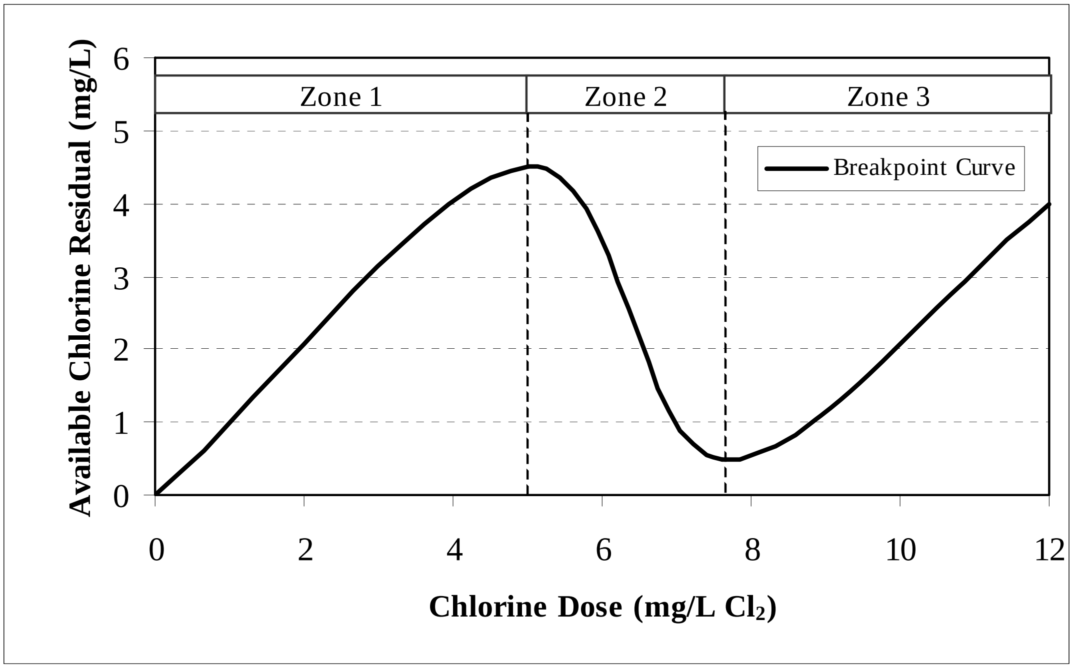
- Sending alarm and control signals to an external PLC
- Metering pump running (separate signal for each pump)
- Flow switch in each metering pump discharge piping enabled/disabled
- High pressure in metering pump discharge
- Liquid level detected in drain pan

The design will include specifications for ammonia injection quills on process piping, and will feature Westfall static mixers (or approved equal).

6.5 Chloramine Residual Monitoring Equipment

To monitor chloramine residual, AECOM recommends use of two Hach CL-17 colorimetric chlorine analyzers. With one meter utilizing reagent to measure Free Chlorine, and one meter utilizing reagent to measure Total Chlorine, the chloramine residual can be calculated (Total Chlorine - Free Chlorine = Combined Chlorine (chloramine)). This method requires careful monitoring of the dosage ratio of hypochlorite to ammonia, as the calculation of combined chlorine cannot distinguish between monochloramine, di and trichloramine, or free chlorine in Zone 3 of the breakpoint curve (See Figure 7-1). Staff will need to ensure that the dosage ratio is kept between 4:1 and 5:1 (chlorine : free ammonia) to ensure that the measured value of Combined Chlorine represents monochloramine concentration.

Figure 6-1. Theoretical Breakpoint Chlorination Curve



The primary and secondary chemical species, which are present in solution during each segment of the breakpoint curve are as described below.

Zone	Chlorine : Ammonia-N Ratio (mg Cl_2 : mg NH_3-N)	Primary Species	Secondary Species
1	<5:1	Monochloramine	Dichloramine (trace)
2	5:1 to 7.6:1	Monochloramine Nitrogen Chloride	Dichloramine Nitrate
3	>7.6:1	Free Chlorine Nitrogen Chloride	Trichloramine Nitrate

Meters are available that measure chloramine concentration directly. In our experience, these meters (such as the Hach APA 6000 ($10,000), or the ChemScan UV-2150/S ($25,000-$30,000)) are very expensive and are marginally reliable. At this time, Hach requires a service contract that includes a monthly visit from a service technician with every APA 6000 unit. Due to the expense and reliability issues, AECOM does not recommend these types of meters for NCSD.

Each meter will have an associated waste stream. Since there are no sewer connections at the District wells, AECOM will include design of a small drywell at each site for disposal.

6.6 Chemical Storage Buildings

AECOM recommends split-faced masonry block construction with a slab roof for the chloramination buildings. Storage buildings will feature two isolated rooms to provide separation between treatment chemicals. Preliminary layouts indicate a required building size of approximately 17 feet by 9 feet for Eureka, Via Concha and Blacklake 4. Eureka will require a building approximately 19 feet by 11 feet. Table 6-7 summarize chemical storage building sizes and Figures 6-2 and 6-3 provide proposed building layouts. The contract documents will require the contractor to submit color swatches and coordinate color selection with District staff.

Table 6-7. Approximate Chemical Storage Building Sizes

	Sundale	Via Concha	Blacklake 4	Eureka
Recommended NaOCl Storage (gal)	500	500	275	500
Recommended NH_3 Storage (gal)	220	220	220	220
Approximate Building Size (ft x ft)	19 x 11	17 x 9	17 x 9	17 x 9

Figure 6-2. Proposed Chloramination Building Layout (Eureka, Via Concha and Blacklake)

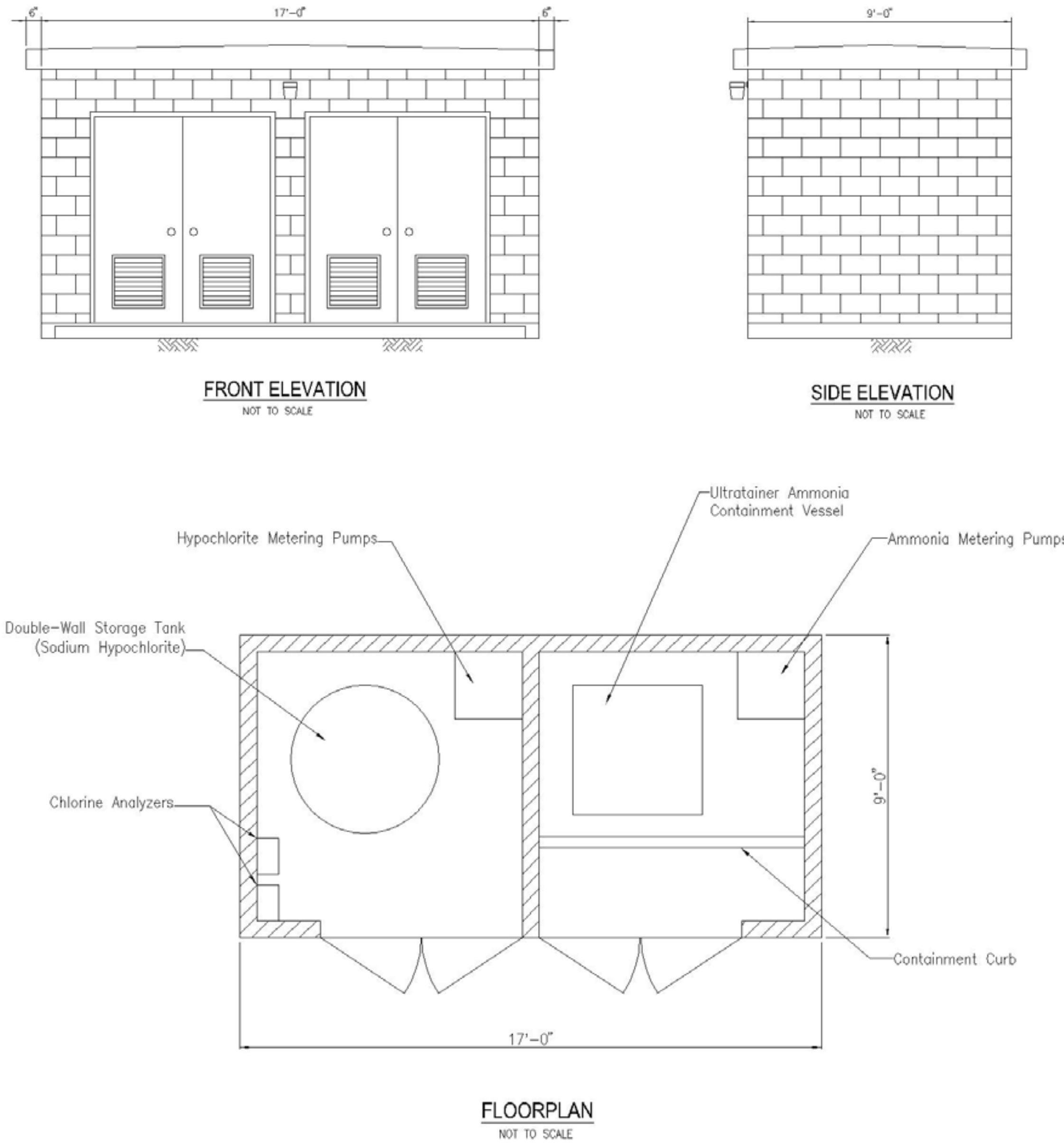
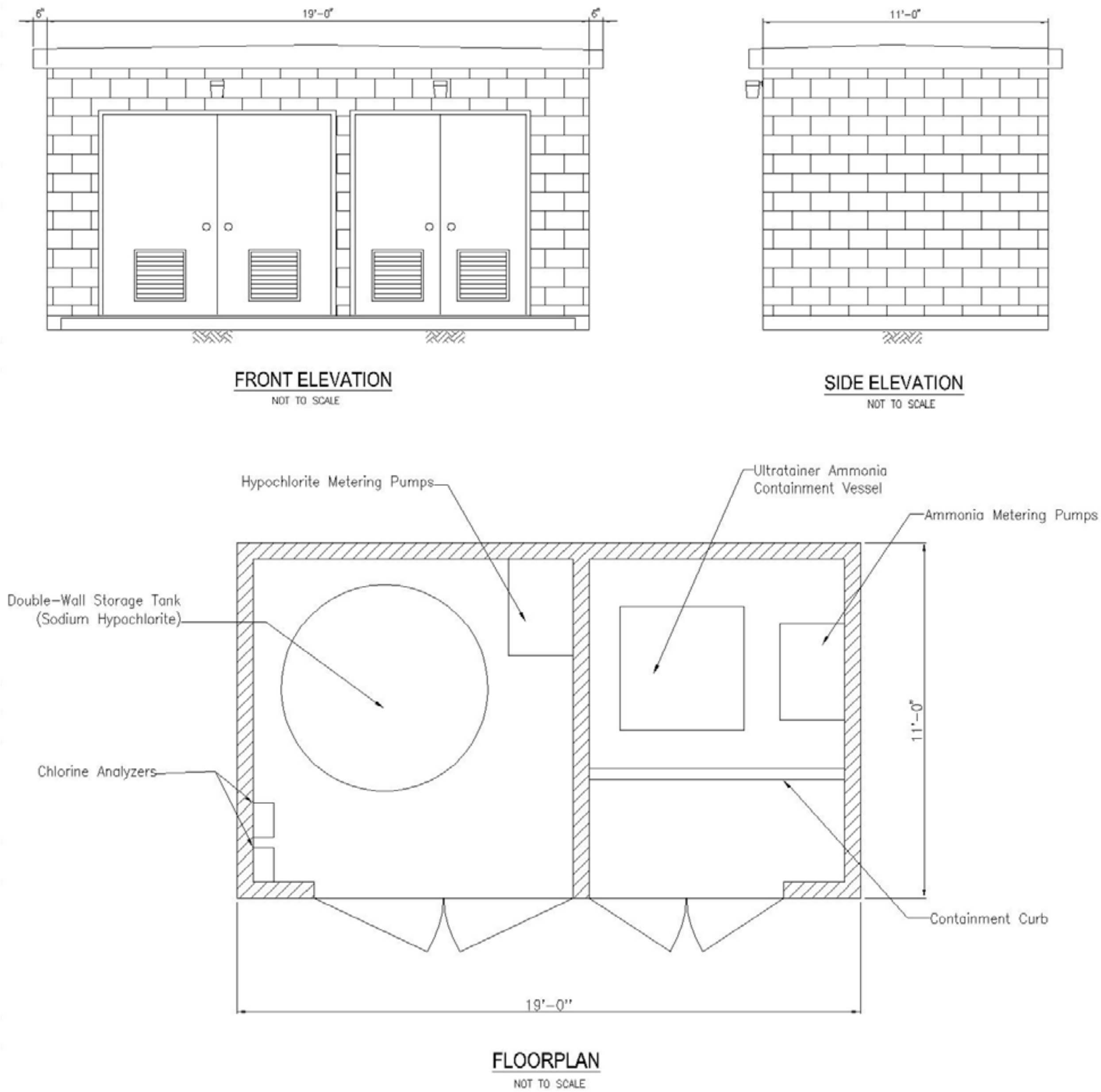


Figure 6-3. Proposed Chloramination Building Layout (Sundale)



Proposed site layouts can be seen in Figures 6-4 through 6-7.

Figure 6-4. Proposed Site Plan - Sundale

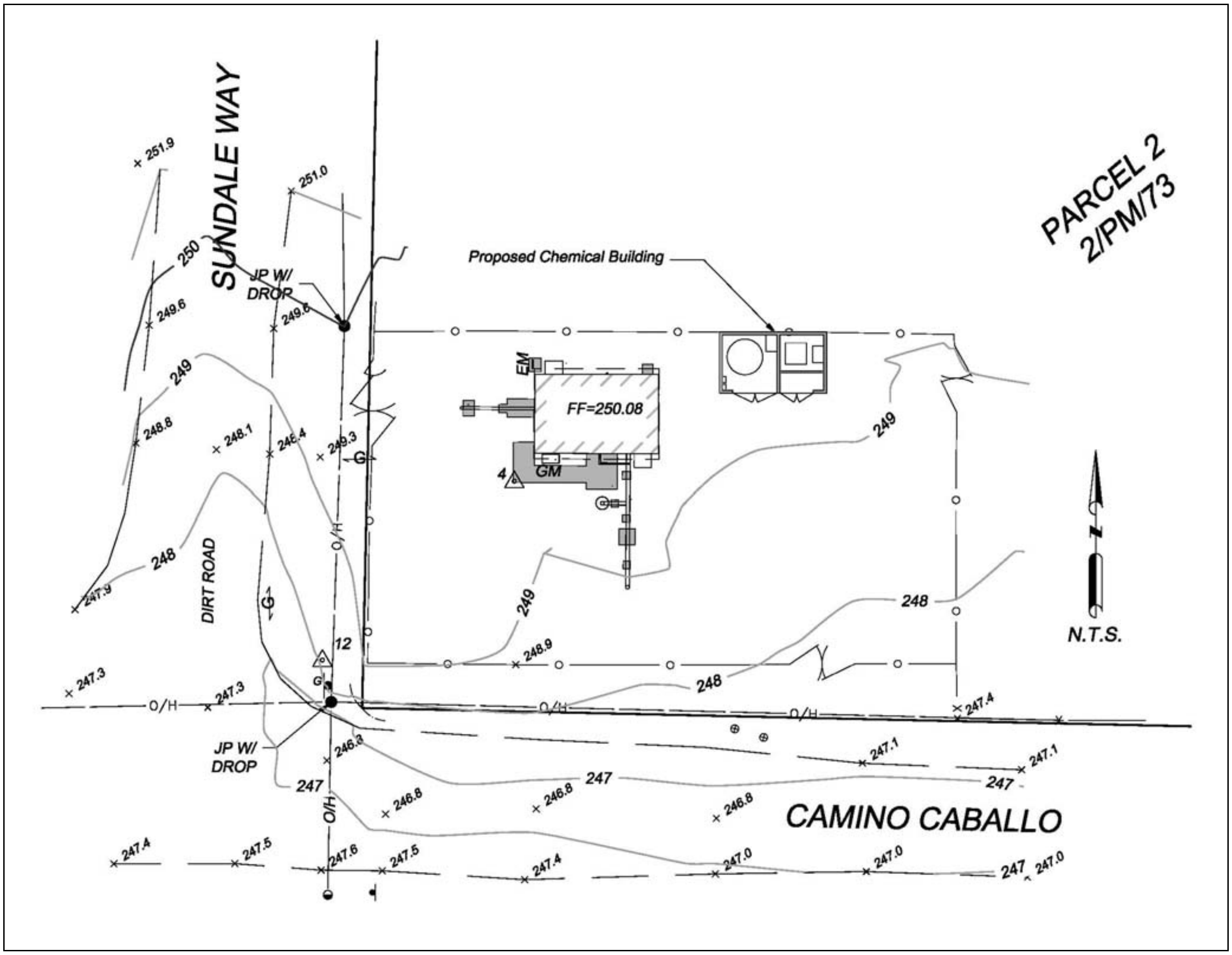


Figure 6-5. Proposed Site Plan - Eureka

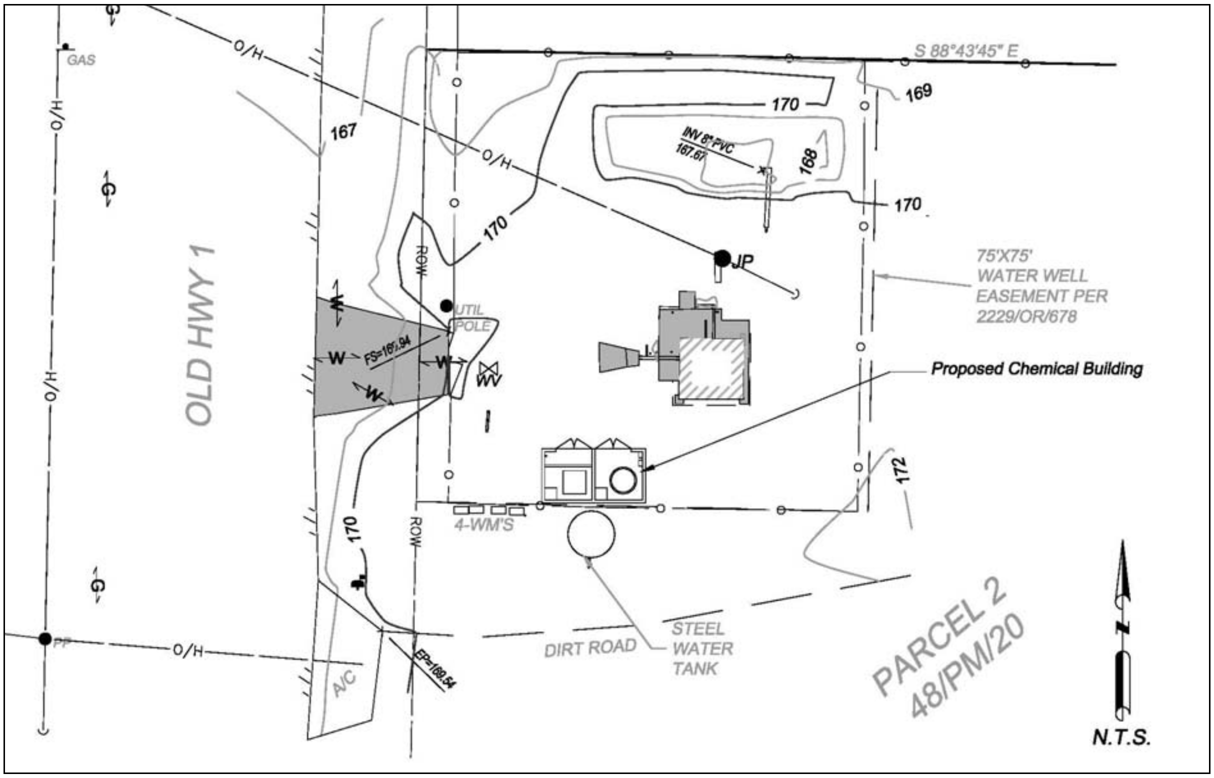


Figure 6-6. Proposed Site Plan – Via Concha

During the site visit, AECOM and District staff discussed the limited project site area, and explored the possibility of constructing a new building to enclose the existing wellhead (replacing the existing building) and the proposed treatment chemicals. Upon review of the site constraints, it is our opinion that although possible, replacing the existing building would add significant cost and complexity to the project. AECOM recommends installing a new chemical building and leaving the existing pump house and electrical panels intact, as shown on Figure 6-6. The waterline from the well to the street may have to be relocated away from the proposed chemical building, and the access gates at the front of the property will need to be reconfigured.

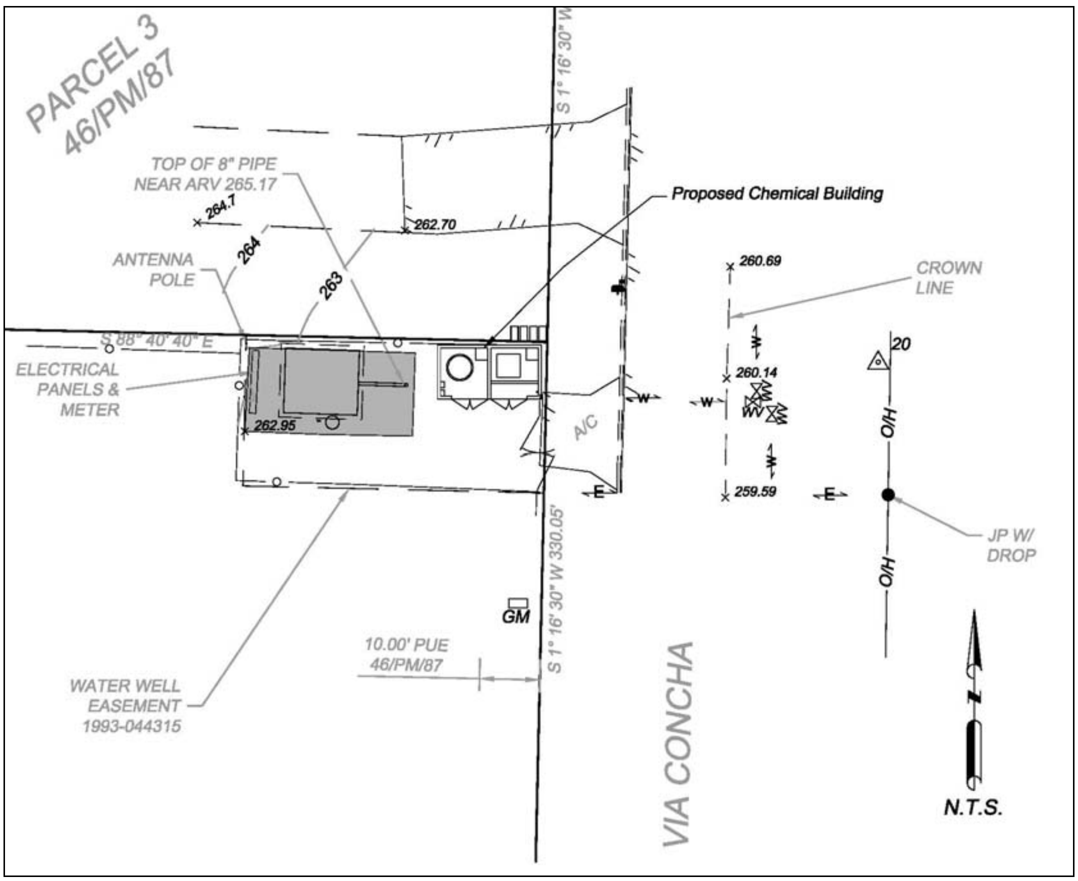
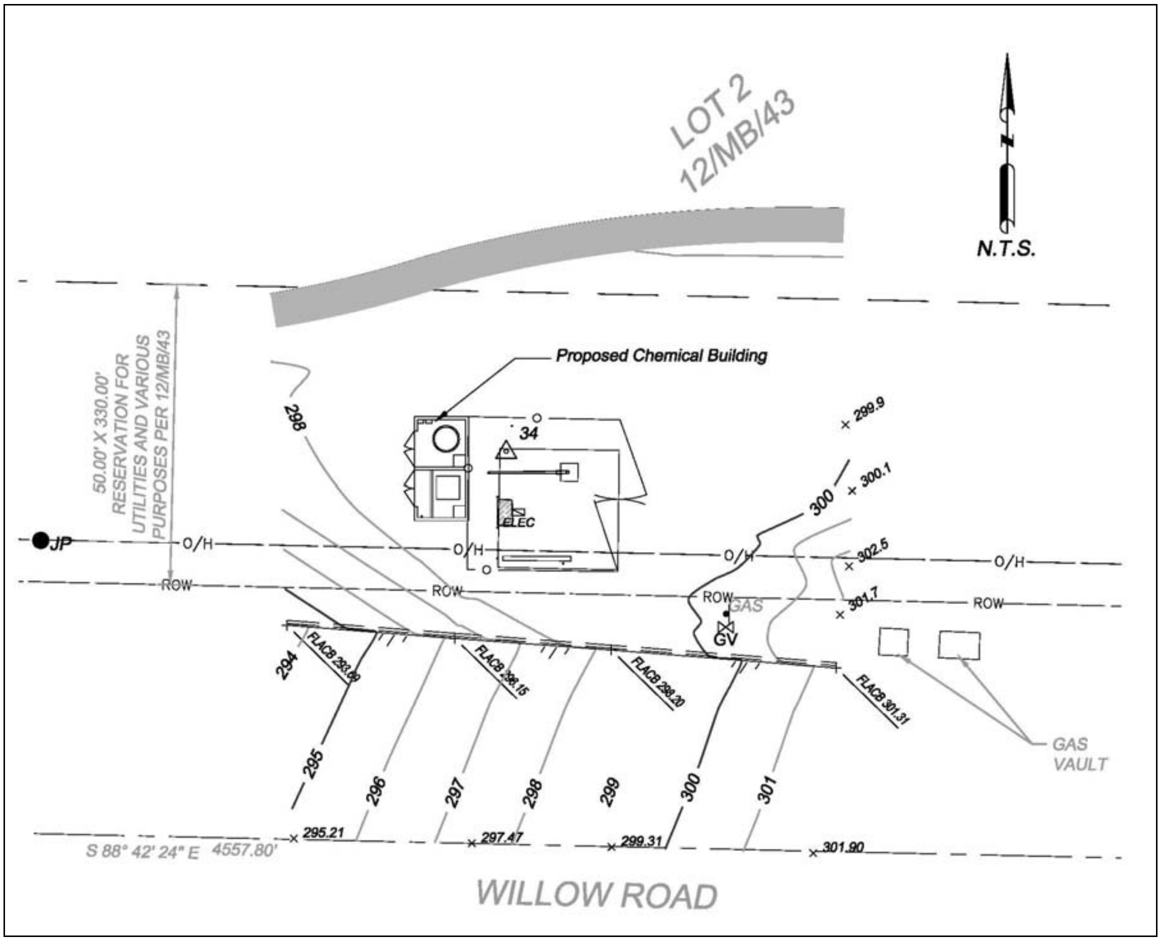


Figure 6-7. Proposed Site Plan –Blacklake 4

This site does not have adequate space to construct the proposed chemical building within the existing fence line. AECOM recommends placing the chemical building outside of the fence line as shown on Figure 6-7. This may require acquisition of additional property. District staff should determine if expansion of the project site is feasible.



6.7 Safety

The hazards at the chloramine treatment sites are primarily electrical and chemical handling. Chemicals in use at the treatment sites include sodium hypochlorite (12.5%), dry ammonium sulfate, and aqueous ammonium sulfate solution.

Caution: It is critical that ammonium sulfate and sodium hypochlorite solutions not be mixed. The undiluted mixing of these chemicals will result in the generation of a poisonous and potentially explosive gas. Special care must be taken during chemical deliveries to ensure that these chemicals are not delivered into the wrong tank.

An emergency eyewash and shower will be provided at each well site. Since some of the treatment sites are located in sparsely populated areas, it is recommended that operations staff always have a readily-available means of communicating with headquarters when working at the treatment sites.

6.8 Emergency or Temporary Chloramination Facilities

Chemical containment vessels, packaged chemical delivery systems, residual meters, and various appurtenances necessary to convert a wellhead to chloramines on an emergency basis are readily available through a wide variety of sources such as USA Bluebook. We do not recommend that this equipment to be purchased in advance and stored unused, until a need arises. It is our opinion that these emergency facilities could be purchased and assembled very rapidly if a need is identified. In our design of permanent chloramination facilities at the four wellheads, we will include technical specifications for skid-mounted, packaged chemical delivery systems that District staff can use to quickly convert a well (such as Bevington) to chloramines in an emergency.

6.9 SCADA, Telemetry and Operation

Each well facility will be equipped with remote monitoring capabilities. The District will be able to monitor the settings of the chemical feed pumps and the chlorine residual from a remote location; however, pacing of the sodium hypochlorite and ammonia feed pumps will be accomplished manually. Adjustments to the metering pump stroke frequency and stroke length will be made at the metering pump local controls. The feed pumps will require regular adjustment based on measured values of the total chlorine residual, free chlorine residual, monochloramine residual, and free ammonia residual. The frequency of metering pump adjustment will depend on the consistency of the well flow rate, the consistency of the well water characteristics, and the drift in the pump set point.

A detailed discussion of controls and instrumentation at the well sites and at Booster Pump Station #2 is included in Chapter 8 (Technical Memorandum No 8).

6.10 Preliminary Opinion of Probable O&M Cost for Water Treatment at District Wells

Table 6-8 shows the estimated monthly chemical consumption at each well. This estimate was based on a chlorine dosing rate of 3.5 mg/l (as hypochlorite ion), and a 4:1 chlorine to ammonia ratio. Average reported well capacity was used, and monthly costs were based on a 30 day month.

Table 6-8. Average Monthly Chemical Usage

Location	Average Reported Production Rate (gpm/MGD)	Monthly 12.5% Sodium Hypochlorite Usage (Gallons)	Monthly Dry Ammonium Sulfate Usage (lbs)
Sundale	1000 / 1.4	1260	1290
Eureka	893 / 1.3	1140	1152
Via Concha	750 / 1.1	960	967
Blacklake #4	375 / 0.54	480	484
	Total	3840	3893

According to the District's chemical supplier (Brenntag Pacific, Inc.), the District currently pays $1.65/gallon for 12.5% sodium hypochlorite solution. Using this rate, the average monthly cost for hypochlorite would be approximately $6400 (based on continuous operation at average production rate).

According to Brenntag Pacific, Inc., dry ammonium sulfate is supplied on pallets at $.65/lb. Using the assumptions stated above, the average monthly cost for dry ammonium sulfate would be approximately $2,530/mo (based on continuous operation at average production rate).

Table 6-9. Estimated Average Monthly Chemical Costs at District Wells

Estimated Monthly Sodium Hypochlorite Costs (Based on $1.65/gal)	$6,400
Estimated Average Monthly Dry Ammonium Sulfate Costs (Based on $.65/lb)	$2,530
Total	$8,930

It is assumed that chloramination facilities will also be installed and operated at Booster Pump Station #2. Estimated O&M costs associated with water treatment at this site is discussed separately in Chapter 5.

7.0 BACKUP POWER, CONTROLS, AND INSTRUMENTATION

7.1 General

This chapter consists of Technical Memorandum No 8, which proposes requirements for electrical power, controls, and instrumentation for the pipeline, tank, pump station, and chloramination systems. The memorandum uses the recommendations of Technical Memoranda 3, 4, and 7 as a basis for design.

7.2 Pump Station/Reservoir Power

Utility power for the site will be from a new PG&E pad-mounted transformer and will be 480 volts, 3 phase. The transformer will feed a metering switchboard that will contain the PG&E revenue meters and facility main breaker. To permit PG&E access to their transformer and meters, the transformer and metering switchboard will be located outdoors near the property line with District access from the pump station site and PG&E access from the access road. The metering switchboard will be in a weatherproof enclosure with interior lights and space heaters.

The metering switchboard will feed a motor control center (MCC) within the pump station building. The MCC will include a main breaker, transient voltage surge suppressor, automatic transfer switch, variable frequency drives for pump motors, starters for building ventilation, breakers for chloramination equipment, and station transformer and panelboard for 120-volt loads. The MCC will have a NEMA 1 gasketed enclosure and will be a standard design of a major manufacturer such as Allen Bradley, Square D, or Cutler Hammer.

The automatic transfer switch transfers the station loads from utility to the on-site generator upon loss of utility power. It will have a time-delay neutral position which will de-energize all loads prior to connection to the other power source. This prevents a transfer from one live source to another live source that is out of sync which could cause damaging voltage transients. Alternative methods of preventing these transients, such as in-phase monitors and synchronous transfer switches, are more expensive and their additional complexity is not warranted. The transfer switch will also output a contact to each variable frequency drive (VFD) to ramp down on shutoff prior to transferring, thereby eliminating pipeline pressure surges. Other features of the transfer switch will include the manufacturer's standard features and will be in accordance with National Fire Protection Association (NFPA).

An on-site diesel generator will be provided for standby power. The generator will be sized to power all non-standby pumps (three of the four), all chloramination equipment, and all auxiliary equipment at the site. Estimated generator rating is 300 kw. The generator will be in a weatherproof enclosure. A separate aboveground, double containment fuel tank sized for 24 hours of normal demand operation will be provided adjacent to the generator. Because the site is in an industrial area and not within a mile of a school, a sound-attenuated enclosure and special emission controls will not be specified outside of local Air Pollution Control District requirements. No load bank will be provided because the pump motors should provide sufficient load to exercise the generator.

7.3 Pump Station Controls

The pumps will be controlled with VFDs to enable variable speed pumping. One VFD will be provided for each pump motor. VFDs were selected over other types of variable speed equipment because of their higher efficiency and high reliability.

The VFDs will not be equipped with a bypass starter for constant speed operation in case the VFD has failed. If a VFD does fail, the standby pump can be utilized. In addition, a complete set of spare VFD parts will be specified to minimize down time.

The VFDs will be specified to comply with the harmonic requirements of IEEE 519 so that excessive harmonics are not transmitted back to the PG&E grid. Specification will allow either 18-pulse inputs or harmonic filters with isolation contactors.

Safety features for pump or motor protection will be wired to the VFD in lieu of the SCADA system so that the safety features are active when in local manual control. Safety features will include a pump discharge high/low pressure switch and a motor winding high temperature switch.

The VFD will also power the motor space heaters when the motor is not running to prevent condensation within the motor. The motor will be specified as suitable for inverter duty.

The VFD will be specified to be a standard product from a major manufacturer such as Allen Bradley, ABB, Toshiba, Cutler Hammer, or Square D.

7.4 Pump Station Security Lighting

High-pressure sodium wall-pack lights will be provided above each building door and will be controlled by a photoelectric cell.

Approximately three pole-mounted high-pressure sodium floodlights (similar to street lights) will be provided for area lighting. One floodlight at the facility entry will be controlled from the building photoelectric cell and the other floodlights will be controlled from a manual switch within the building. No motion sensor switched lights are proposed for immediate use. Conduit runs for future lighting installation flexibility are recommended.

7.5 Pump Station Security

No closed-circuit television (CCTV) or perimeter detection systems are proposed for immediate use. Conduit runs for potential future security installations are recommended. Each building door and the metering switchboard doors will be monitored with intrusion switches and connected to the SCADA system.

7.6 Pressure-Reducing Stations (PRVs)

A low-profile combination meter and SCADA panel will be provided at each PRV site. PG&E service of 120/240 volts will be provided. No provisions for standby power will be provided, with the exception of the backup battery for SCADA. Pedestal will be by Tesco or equal.

7.7 Chloramination System Monitoring and Control

7.7.1 Well Sites

The sodium hypochlorite and the ammonia feed pumps will be ratio paced on the well discharge flow. The operator will enter the desired ratio between well flow and chlorine feed in the programmable logic controller (PLC). The PLC will automatically adjust the feed of the chemical feed pumps to maintain the ratio between the well flow to chlorine feed . Hach CL-17 Chlorine residual analyzers will be utilized to measure Total chlorine and Free chlorine at each well site. The chlorine residual signals will be monitored by the PLC and sent to the SCADA computer system. The PLC will compute and record the Combined Chlorine (Chloramine) level as follows; Total Chlorine – Free Chlorine = Combined Chlorine. The SCADA system will monitor and alarm these water quality signals.

7.7.2 Booster Pump Station Site

The sodium hypochlorite feed pump will be paced on the pump station discharge flow and trimmed based on the total chlorine residual using a compound control loop system. The pump station programmable logic controller (PLC) will be programmed to provide the compound control loop and output a 4-20ma signal to control the speed of the chlorine feed pump.

A Hach CL-17 Total Chlorine residual analyzer will be utilized to measure Total chlorine and Free chlorine at the booster pump station site. The chlorine residual signals will be monitored by the PLC and sent to the SCADA computer system. The PLC will compute and record the Combined Chlorine (Chloramine) level as follows; Total Chlorine – Free Chlorine = Combined Chlorine. The SCADA system will monitor and alarm these water quality signals.

The operator will decide when ammonia feed is required based on the computed chloramine level. When it is decided that ammonia feed is required, the operator will allow the ammonia feed pump to run. The feed pump will be ratio paced on the booster discharge flow. The operator will enter the desired ratio between booster pump flow and ammonia feed in the programmable logic controller (PLC). The PLC will automatically adjust the speed of the chemical feed pump to maintain the ratio between the booster pump flow and ammonia feed.

7.8 Controls and Instrumentation

The District presently utilizes iPAAC Supervisory Control and Data Acquisition (SCADA) System for monitoring and controlling the Districts' water and wastewater systems. Our understanding is the iPAAC system is a Web based application provided by a remote access system provider.

Additionally, the District utilizes two models of Automation Direct Remote Terminal Units (RTU) at each remote site. The Direct Logic 06 model for monitoring and control requirements and the Direct Logic 05 model to provide a means to communicate with the iPAAC system and the MDS radios.

The District is in the process of finalizing the SCADA system upgrade requirements for future implementation. A radio survey should be performed for the new project sites (flow meter, PRVs, and booster station). The Waterline Intertie Project control and instrumentation system will be designed with the capability to interface to the new SCADA System Upgrade package. The instrumentation and control system subcontractor, as part of Bid Package 4 will closely review the above requirements and provide the appropriate design features to satisfy those constraints.

The Direct Logic 06 PLC will be utilized at the Booster Pump Station and the five PRV sites for monitoring and control purposes. It is anticipated that the Booster Pump Station will be controlled to maintain the level of the 0.5 million gallon storage tank. Level and pressure transmitter equipment will be provided to match District standards. Magnetic flow meters will be provided at the sites requiring flow monitoring. We will specify the Direct Logic PLC systems to be provided with a 12 hour battery back-up Uninterruptable Power Supply (UPS). The PLC will also be specified to be provided with an Operator Interface Terminal (OIT) (Panelview or equal) at Booster Pump Station site. The OIT will be specified with sufficient memory to store a minimum of 72 hours each of three analog values (pressure, flow, level) for trending displays on the OIT. The chloramination systems at the four well sites (see Chapter 6) and at the booster pump station (see Chapter 5) will incorporate a control panel for local control of the feed systems and for sending and receiving alarms and control signals to the PLC at the District's Southland office.

Communication between the remote sites and NCSD Southland office will be via radio communication utilizing the District's current unlicensed 900 MHz radio frequency and MDS INET radios. A radio path analysis is required for the booster pump station and PRV sites to verify line of site communication between the remote sites and the District's Standpipe, which then relays the signal to the Southland office. This will be provided by the SCADA Upgrade design engineering firm. It is also our understanding that the Quad Tanks are utilized as a radio repeater site and a backup access point. Additionally, radio communication is required from the Santa Maria flow meter site to the District's Southland Operations center possibly routed via the Booster Pump Station site. This will be verified by the SCADA Upgrade design engineer.

The flow from the Santa Maria connection to the ½ million gallon tank requires having flow control features. Our plan is to provide a hydraulically controlled valve (Cla-Val Model 631 or equal) at the tank site. The valve will be furnished with solenoid valves that will give the District the ability control the valve locally from the PLC or remotely from the SCADA computer. The PLC will be programmed to maintain an operator entered flow set point and will pulse the open/close solenoid valves to maintain the desired flow. The magnetic flow meter monitoring flow into the tank will provide the flow feedback to the PLC flow control loop. A separate flow orifice plate will not be required to be furnished with the Cla-Val, since the magnetic flow meter provides the required flow signal used for flow control.

8.0 PRELIMINARY OPINION OF PROBABLE CONSTRUCTION COST

This opinion of probable construction cost represents judgment as a design professional and is supplied for the general guidance of the District. Since AECOM has no control over the cost of labor and materials, over delays in project bidding or award, over competitive bidding or market conditions, AECOM does not guarantee the accuracy of such opinions as compared to contractor bids or actual cost to the District. Refinements to this preliminary opinion of cost will be provided with subsequent design package submittals. The project cost opinion for construction, design, and other applicable costs are summarized in Table 8-1, below, and compared to the cost opinion from the May 2008 Preliminary Engineering Memorandum. As recommended in Chapter 2, it is anticipated that the project will be split into four bid packages for construction. More detailed construction cost tables are included for each bid package in the following pages.

Table 8.1 – Opinion of Probable Project Costs

Item	Description	Budgeted Amount May 2008 Preliminary Engineering Memo.		Updated Amount 22-Apr-09 Concept Design Report	
1	Mobilization	$580,000		$607,000	
2	Blosser Extension (18-in)	$1,247,000		$1,129,000	-
3	Pump Station No. 1 turnout & meter (Blosser Rd)	$61,000		$158,000	
4	River Crossing (24-in HDD & levee jack & bore)	$6,135,000		$5,462,500	
5	24-in Pipeline to Joshua	$656,000		$400,000	
6	Reservoir (0.5-MG)	$1,361,000		$1,365,000	
7	Pump Station No. 2	$603,000		$1,572,500	
8	Pressure Regulators (200 homes)	$30,000		--	
9	Pressure Reducing Valve Stations	$18,000		$243,000	
10	Chloramination (Joshua & 5 wellheads)	$707,000		$739,500	
11	Upgrade Southland to 12-in	$799,500	(1)	$849,000	(7)
12	Upgrade Frontage to 12-in	$1,101,300	(1)	$957,000	(7)
13	Upgrade Orchard to 12-in	$509,000		$1,103,500	(8)
14	Upgrade Division to 10-in between Allegre and Meridian (6)	$53,000		--	
15	Oakglen Avenue 12-in main (5)	--		$457,000	
16	Darby Lane 12-in main (5)	--		$153,000	
17	HWY 101 Bore & Jack (5)	--		$241,000	
18	Isolation Valves (5)	--		$12,000	
19	Pump Station All Weather Access Road	--		$128,000	
	Construction Subtotal	**$13,860,800**		**$15,577,000**	
20	Contingency	$3,643,000		$3,115,400	(10)
	Construction Subtotal + Contingency	**$17,503,800**		**$18,692,400**	
21	Property Allowance	*not included*	(4)	$500,000	(4)
22	Design-Phase Engineering				
	Original Agreement (July 2008)			$744,993	
	Budget Revision 1 - Pressure Reduction			$132,798	
	Budget Revision 2 - Biological Survey for HDD			$4,050	
	Budget Revision 3 - Modeling for GSW/Woodlands Turnouts			$8,380	
	Budget Revision 4 - Additional Survey Services			$9,900	
23	Office Engineering during construction			$175,837	
24	Estimated Construction Management (3)	$2,428,000	(2)	$1,507,170	(9)
25	Permitting Fees To Date	--		$1,573	
26	Non-Final Design Funds Spent To Date	*not included*		$1,402,879	(11)
27	Estimated Other Costs (Assessment, etc)	*not included*		$415,420	(11)
	PROJECT TOTAL (Rounded to 1000)	**$19,932,000**		**$23,596,000**	

Table 8.1 Notes:

ENR CCI: March 2008 = 8109; March 2009 = 8534

(1) Costs are from the December 2007 Water and Sewer Master Plan (Cannon).

(2) Engineering and Construction Management were originally presented as a "lump sum" amount

(3) Includes material testing, construction staking, and environmental monitoring

(4) Estimate only. Item not included in previous construction cost opinions, but was added to the Concept Design Report to provide a complete assessment of anticipated project costs.

(5) These work items were added to relieve high pressures on Mesa as an alternative to service pressure regulating valves (See Tech Memo 9). One PRV station at Maria Vista was required initially. Four are recommended for revised project. This was design Budget Revision #1.

(6) Based on review of record drawings, this pipeline is already a 10-in main

(7) Initial estimate incorporated Master Plan project costs. Revised estimate includes higher unit costs to reflect paving 1 traffic lane, per County standards

(8) Updated unit costs include higher costs to reflect paving 1 traffic lane, per County standards

(9) To be provided by CM team - Has not been revised to reflect additional work for construction management of Oakglen, Darby, and Orchard extensions.

(10) Contingency was modified to 20% which is more appropriate for 30% design phase.

(11) Provided by District staff.

not included = Item was not included in previous construction cost opinions, but was added into the Concept Design Report to provide a complete assessment of anticipated project costs.

Table 8-2. Bid Package 1: Santa Maria River Crossing

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization	1	LS	$210,000	$210,000
2	Sheeting and Shoring	1	LS	$50,000	$50,000
3	24-inch CL 250 DIP Pipeline Open Trench (Deep)	290	LF	$650	$188,500
4	24-inch Fusion-welded HDPE DR-9 Carrier Pipeline including HDD bore	2,670	LF	$1,500	$4,005,000
5	42" Minimum Diameter Surface Steel Casing at HDD Entry Point	200	LF	$1,000	$200,000
6	Combination Air/Vacuum Valve at HDD Exit (Mesa)	1	LS	$15,000	$15,000
7	Site Restoration (Grading/Earthwork Only)	1	LS	$25,000	$25,000
8	Provide necessary permits and temporary fencing around laydown/work areas	1	LS	$50,000	$50,000
9	Traffic Control During HDPE laydown and buildup	1	LS	$25,000	$25,000
10	Pressure Test and Disinfection	1	LS	$15,000	$15,000
11	Allowance for Environmental Mitigation Measures	1	LS	$100,000	$100,000
	Sub Total				$4,884,000
	Contingency	20%			$977,000
	Total				**$5,861,000**

Table 8-3. Bid Package 2: NCSD Pipeline Improvements

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization	1	LS	$123,000	$123,000
2	Traffic Control and Regulation	1	LS	$189,000	$189,000
3	Sheeting and Shoring	1	LS	$245,000	$245,000
4	12-in CL 200 PVC Water Main & Appurtenances	16,300	LF	$131	$2,132,000
5	Hwy 101 Crossing from Frontage Rd to Darby Ln.	250	LF	$964	$241,000
6	Asphalt Pavement Removal & Restoration	81,500	SF	$9	$751,000
7	2.5-in Blow-offs	19	EA	$2,660	$51,000
8	1-in Combination Air / Vacuum Release Valves	19	EA	$3,192	$61,000
9	12-in Gate Valves	42	EA	$3,625	$153,000
10	Pressure-Reducing Valve Stations	5	EA	$50,958	$255,000
11	Sampling Stations	2	EA	$2,400	$5,000
	Sub Total				$4,206,000
	Contingency	20%			$841,000
	Total				**$5,047,000**

Table 8-4. Bid Package 3: Blosser Road Water Main and Flow Meter

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization		LS	$58,000	$58,000
2	Traffic Control and Regulation	1	LS	$37,000	$37,000
3	Sheeting and Shoring	1	LS	$78,000	$78,000
4	18-in CL 250 DIP Water Main and Appurtenances	5,200	LF	$145	$753,000
5	Concrete Asphalt Pavement Removal and Restoration	26,000	SF	$9	$234,000
6	Flow Metering Station	1	EA	$106,000	$106,000
7	1.5-in Blow-offs	6	EA	$2,000	$12,000
8	2-in Combination Air / Vacuum Release Valves	6	EA	$2,400	$15,000
9	18-in Butterfly Valves	13	EA	$4,000	$52,000
10	Jack & Bore under the levee - install 36-in steel casing & 24-in DI carrier pipe	220	LF	$1,085	$239,000
11	24-in CL 250 DIP Watermain (Deep Trench)	770	LF	$650	$501,000
	Sub Total				$2,085,000
	Contingency	20%			$417,000
	Total				**$2,502,000**

Table 8-5. Bid Package 4: Pump Station and Reservoir and Chloramination Systems

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization	1	LS	$207,000	$207,000
2	Booster Station Site Clearing, Stripping, Grubbing	25,000	SF	$3	$75,000
3	Protection and Restoration of Farmland	19,000	CY	$6	$114,000
4	Sheeting and Shoring	1	LS	$25,000	$25,000
5	24-in CL 250 DIP Watermain (Deep Trench)	75	LF	$650	$49,000
6	24-in CL 250 DIP Watermain (Normal Trench)	1,700	LF	$235	$400,000
7	24-in Gate Valve	3	EA	$6,500	$20,000
8	General Site Earthwork, Excavation and Non-structural Backfill	1	LS	$85,000	$85,000
9	Partially Buried Prestressed Concrete Tank (includes 15% Contractor Markup)	500,000	GAL	$2.0	$1,000,000
10	Reservoir Foundations and Subgrade Preparation	1	EA	$50,000	$50,000
11	Reservoir Structural Backfill	3,865	CY	$40	$155,000
12	Reservoir Appurtenances	1	EA	$46,000	$46,000
13	Site Piping, FCV Vault, and Appurtenances	1	LS	$284,000	$284,000
14	Pumps (100-HP, vertical turbine "barrel" pump, w/ can)	4	EA	$85,000	$340,000
15	100-HP Variable Frequency Drives	4	EA	$30,000	$120,000
16	Piping, Valves, Mag Meter & Appurtenances	1	LS	$95,000	$95,000
17	Site Paving	1,500	SF	$9	$14,000
18	Surge Control System	1	LS	$75,000	$75,000
19	Pump Station Building	1	LS	$378,360	$379,000
20	Electrical, Emergency Generator, Switchgear	1	LS	$175,000	$175,000
21	SCADA	1	LS	$45,000	$45,000
22	Visual Screening	1	LS	$30,000	$30,000
23	Wellhead Chloramination Systems	4	LS	$87,500	$350,000
24	Pump Station Chloramination System	1	LS	$200,000	$200,000
25	10-ft Wide All Weather Access Road	1,700	LF	$75	$128,000
	Sub Total				$4,461,000
	Contingency	20%			$892,000
	Total				**$5,353,000**

9.0 FRONTAGE ROAD SEWER REPLACEMENT

9.1 Introduction

The Draft Technical Memorandum No 10 – Frontage Road Sewer Replacement was submitted for Task 1 of the Frontage Road Sewer Replacement Project. This Chapter consists of a revised Draft Technical Memorandum based on comments received from the District staff, the Peer Reviewers, and the Construction Management Team. The Draft Technical Memorandum describes the preliminary design for the Frontage Road Sewer Replacement from the Southland WWTF to the intersection of Frontage Road and Division Street. The length of influent trunk main that continues onto the WWTF property to the influent lift station was to be upgraded as part of the Southland WWTF Upgrades Project. However, the District is considering combining the Frontage Road Sewer Upgrades into one construction set in order to expedite design and construction and reduce construction mobilization costs. Should this occur, the Frontage Road Sewer Upgrade alignment from Division Street to the Southland WWTF influent lift station will be included with the 60% design submittal.

This Chapter will define the proposed alignment, identify potential challenges, and summarize preliminary pipeline and manhole design parameters such as design flow, diameter, length, material, and depth. The 30% plans (included as Volume 3 of this Report) present many of these elements. Potential challenges pertaining to water main improvements, conflicting utilities, connections to the existing system, and maintaining sewer service during construction are considered.

9.2 Background

The Southland Wastewater Treatment Facility (WWTF) Master Plan (AECOM, 2009) recommended that the District upgrade the existing 12-inch Frontage Road sewer main between the WWTF and Division Street, consistent with the Water and Sewer Master Plan Update (Cannon, 2007). The District authorized AECOM to prepare construction documents for the Frontage Road Sewer Main Replacement Project in conjunction with design of Nipomo System Pipeline Improvements on Frontage Road (part of the NCSD Waterline Intertie Project).

9.3 Flow Projection and Hydraulic Capacity

An analysis of hydraulic capacity was performed for the existing Frontage Road sewer main from Division Street to the WWTF as part of the Southland WWTF Master Plan (AECOM, 2009). Wastewater flow in the existing Frontage Road sewer main was approximated between the WWTF and Division Street by counting the number of dwelling units served by the main and applying a per-dwelling unit flow factor. The flow was then divided between three tributary mains along the Frontage Road sewer (the tributary mains are located at Southland, Story Street, and Division Street). Current and projected flows for the resulting three segments of the Frontage Road main are summarized in Table 9-1.

Table 9-1. Southland WWTF Master Plan Update Frontage Road Estimated Flow Rates

Frontage Road Trunk Main Segment	Current Estimated Flow*		Projected Future Flow (2030)*	
	Average Annual Flow (AAF)	Peak Hour Flow (PHF)	Future AAF	Future PHF
Division to Story:	0.39 mgd	1.17 mgd	1.09 mgd	3.25 mgd
Story to Southland:	0.57 mgd	1.71 mgd	1.59 mgd	4.75 mgd
Southland to WWTF:	0.60 mgd	1.80 mgd	1.67 mgd	5.00 mgd

* Southland WWTF Master Plan (AECOM, 2009) hydraulic analysis, Figures 5-2 and 5-4.

The hydraulic analysis indicates current peak flow conditions may exceed design capacity in multiple segments of the trunk main and that projected 2030 peak flows would exceed capacity of the majority of the trunk main.

The Frontage Road trunk main was also recommended for upgrade in the 2007 Water and Wastewater Master Plan Update (WWMP, Cannon Associates). In the WWMP, neither current nor projected future wastewater loads were provided for individual tributary areas.

9.3.1 Design Flows

The proposed sewer upgrade is designed to accommodate build-out (2030) peak and average flow conditions presented in the Southland WWTF Master Plan. Assumptions from the WWTF Master Plan were applied to distribute flow from mains connecting to the Frontage Road main (Table 9-1).

9.3.2 Hydraulic Capacity and Pipe Sizing

Pipe diameter was determined based on design flows and according to District design standards for gravity sewer, as summarized in Table 9-2.

Table 9-2. Summary of NCSD Design Standards for Gravity Sewer

Design Parameter	Value
Manning roughness coefficient	n = 0.011 for plastic pipe
Peaking factor	3.0
Peak flow maximum design depth, d/ D	< 15-inch diameter, $d/D_{max} = 0.50$ ≥ 15-inch diameter, $d/D_{max} = 0.75$
Minimum slope	8-inch diameter 0.0035 ft/ft 10-inch 0.0025 ft/ft 12-inch 0.0020 ft/ft 15-inch 0.0015 ft/ft 18-inch 0.0012 ft/ft 21-inch 0.0010 ft/ft

Pipeline diameters of 18 and 21-inches were analyzed for the proposed Frontage Road sewer replacement alignment using the design flows presented in Table 9-1. The Manning roughness coefficient for plastic pipe (n = 0.011) was used in the analysis. Slopes of the replacement sewer range from 0.005 to 0.009 ft/ft. Table 10-3 summarizes the pipe flow depth to pipe diameter ratio (d/D) for 18 and 21-inch diameter sewers as calculated using Manning's Equation for open channel flow. Detailed calculations are included as Appendix E.

Table 9-3. Summary of Depth to Diameter Ratios for 18-inch and 21-inch Pipeline Diameters

		Current flow conditions		Buildout flow conditions	
		AAF	**PHF**	**Average Flow**	**Peak Flow**
18" PVC	d/D	0.2	0.37	0.36	*0.70* [a]
21" PVC	d/D	0.16	0.29	0.27	0.52

(a) Near design limit d/D of 0.75—not recommended.

AECOM recommends construction of a 21-inch PVC sewer for the Frontage Road sewer main replacement to accommodate current and projected peak flows and allow a factor of safety to account for inherent uncertainty in long range sewer flow projections[10]. The 21-inch sewer will provide adequate capacity for buildout and sufficient performance at current flow rates. Hydraulic parameters for the proposed 21-inch PVC sewer upgrade are tabulated in Appendix E. Velocities at current minimum flow conditions (0.3 x $Q_{ave.}$) were also evaluated to confirm sufficient velocity.

[10] Although calculations show that an 18-inch diameter replacement sewer would meet the peak flow design standard (d/D < 0.75), 21-inch diameter pipe is recommended because an 18-inch diameter sewer would be near design capacity at future peak flow conditions, and an increase in flow of 7% would exceed the design peak hour flow maximum depth-to-diameter criteria.

9.4 Pipeline Design

The following project elements and criteria were considered in design of the replacement sewer main on Frontage Road and are discussed in the following sections in further detail.

- Connection to the existing Southland WWTF influent main
- Connection to the existing 12" main at Division Street
- Tie-in to existing "side" gravity mains
- Geotechnical recommendations
- Standards for pipeline materials and separation requirements from underground utilities
- Construction phasing and Waterline Intertie Project improvements on Frontage Road

Open trench construction is recommended for the sewer replacement. Pipe reaming was considered but not recommended for the Frontage Road sewer replacement (see Section 9.11).

9.4.1 Connection to the Southland WWTF Influent Main

Connection to the WWTF influent main at the manhole immediately north of the WWTF (Sheet 1816 MH 2) is planned for the Frontage Road sewer replacement. Connection at this location will require temporary bypass of the 8-inch Southland Street main and a short section of the existing 12-inch Frontage Road main. The replacement sewer alignment encroaches on the drip line of the existing tree near the proposed connection manhole (#1816-2).

The Southland WWTF Master Plan (ibid) recommended increasing capacity of the Southland WWTF influent main, from Southland Street to the WWTF influent lift station. AECOM recommends incorporating improvement of the Southland WWTF influent main with the Frontage Road Sewer Replacement Project since there is potential for construction cost savings. The recommended alignment for the replacement influent main would cross the WWTF yard/ parking area and continue toward the headworks near the existing detention basin and drainage swale before converging on the existing alignment north of the headworks. This alternative influent trunk main alignment would increase separation from the 16-inch high pressure gas main located north-east of the WWTP, and would allow the existing influent main to remain in operation during construction, reducing the need for bypass pumping. For this alignment, the connection between the Frontage Road replacement sewer and the improved influent main would be made with a new manhole installed near manhole #1816-2, and would not require trenching within the drip line of the existing tree. Additional survey data for the WWTF and influent manholes, and geotechnical evaluation would be needed for design of influent main improvements.

9.4.2 Connection to existing 12-inch Frontage Road Main near Division Street

The replacement Frontage Road sewer is designed to extend to Division Street where a connection will be made at manhole #1915-3. A new precast manhole channeled for straight-through and branched flow will be installed on the new 21-inch sewer in the intersection of Division and Frontage. The new branched manhole will accept flow from existing manhole #1915-3 and will be installed with a stub upstream in anticipation of future upgrade to the sewer, along the path of the replacement alignment (see plan sheet C-138). The existing manhole #1915-3 will be cored and rechanneled to direct flow into the new manhole.

9.4.3 Connection to Mains and Laterals

Existing side gravity sewer mains and service laterals connecting to the 12-inch Frontage Road sewer main will be connected to the replacement sewer main. The following side main line connections are anticipated and will be shown in construction plans:

- Southland Street main
- Side-street main serving the business park at Story Street
- Story Street mains (approaching from east and west)
- Margie Place main
- Division Street main

These existing mains determine the locations and invert elevations of tie-in manholes along the replacement alignment. Approximate slopes for side mains were assumed based on information in District as-built drawings and were used to estimate maximum invert elevations on the new alignment. The need for additional surveying is not currently anticipated since intermediate manholes are being used; however, the need for additional surveying will be evaluated as the design progresses to the 60% level.

Reconnection of side mains is proposed via installation of new intermediate manholes. Short, temporary sewage bypasses would be necessary during each main connection. An alternative to constructing an intermediate manhole for each side main reconnection is to extend the existing side main with an inline coupling, however this would only be feasible if the slope of the existing main can be maintained to match the crown of the new 21-inch sewer. Its assumed all connections will be made with intermediate manholes.

A video inspection should be performed and reviewed to determine if there are additional lateral connections. AECOM recommends that the video inspection be performed during the design phase to reduce the potential for change orders during the construction phase. AECOM will coordinate this with the District.

9.4.4 Geotechnical Design Recommendations

The Geotechnical Report by prepared by Fugro (March, 2009) presents soil parameters and recommendations for pipeline design. Geotechnical findings pertaining to improvements planned for Frontage Road are summarized in the following paragraphs.

Based on the subsurface conditions encountered during the geotechnical evaluation, the majority of the on-site soil should not be considered suitable for use as pipe bedding or backfill to at least 12 inches above the top of the pipe but may be used for compacted backfill higher than 12 inches above the top of the pipe. The excavated materials can likely be used for compacted fill or trench backfill material. Moisture conditioning of the soils and control of compaction layer thickness will be needed to achieve the recommended compaction. The soils encountered within the anticipated depth of excavation are expected to consist of sandy soils. Onsite soils can likely be excavated with conventional backhoe or excavator type equipment typically used for pipeline construction. Vertical cuts in sandy soils should not be considered stable unless properly shored or sloped in accordance with the requirements of OSHA. Temporary slopes and shoring will need to comply with OSHA requirements.

Artificial fill and dune sand deposits were encountered along the pipeline alignment and generally consist of asphalt concrete, base materials, very loose to very dense sand, and local soft to stiff silt. Groundwater was encountered in boring B-102 (At Grande Avenue, approximately 1,200 ft north of the proposed replacement sewer, near the proposed) at a depth of approximately 27.5 feet below the existing ground surface. Groundwater encountered at B-102 is below the anticipated sewer pipeline depths. Groundwater was not encountered in boring B-103 (Frontage Road, near Story Street). Various concrete, rubble, and unidentified buried objects were encountered along the alignment below the asphalt pavement. The soils report states that the trench subgrade may need to be moisture conditioned and compacted prior to placing bedding material for the pipe. This recommendation will be incorporated into the plans as an additive bid item.

9.4.5 Pipe Material

Solid wall bell and spigot PVC is recommended for sewer main improvements on Frontage Road. The light weight of PVC sewer pipe allows easier open-trench installation compared to vitrified clay or reinforced concrete pipe, and PVC has minimal potential for hydrogen sulfide corrosion. Closed wall PVC pipe is not recommended due to potential for creep under load and greater effort required for trench design and inspection. Closed wall PVC is generally more expensive than solid wall PVC pipe of the same diameter.

AECOM anticipates solid wall PS 46 PVC (ASTM F679) pipe will adequately support trench and vehicular loads on the proposed pipeline. Pipe loads are discussed in Section 9.9.

9.4.6 Manholes

Depths of replacement manholes range from 11 to 20 ft below existing grade. Precast 60-inch I.D. manholes with 24-inch diameter (nominal) covers and grade rings are planned for the replacement sewer. Manhole base thickness will depend on site-specific conditions but will not be less than 9 inches. To account for energy losses and prevent stagnant conditions in manholes, the flow line will drop 0.1 feet across each manhole, invert to invert.

The sewer manhole interiors will be lined with either an epoxy or polyurethane coating system in conformance with District Standards. Sewer manhole frames and covers will also conform to District standards.

Six of the twelve proposed manholes on the replacement sewer alignment will serve as side sewer main tie-in points. As noted in Section 9.4.3, it is assumed that one intermediate manhole will be needed at each of the six tie-in locations, to allow for a change in slope for each of the approaching side mains.

9.4.7 Pipe Loads

The pipeline will be designed to resist loads resulting from the trench backfill and applicable surface loads (i.e. vehicular live loads). The magnitude of the load supported by the pipe depends on the depth of backfill, width of trench, unit weight of backfill soil, and frictional characteristics of backfill material. Design procedures outlined in the AWWA M23 Manual (PVC Pipe - Design) will be followed for evaluating flexible pipe trench loads and horizontal pipe deflection. Based on the Geotechnical Report prepared by Fugro (March, 2009) and preliminary calculations, solid-wall PS 46 PVC pipe is expected to adequately support loads. Pipe loads will be analyzed in detail prior to the 60% submittal.

9.4.8 Alignment/ Utilities

The proposed alignment of the Frontage Road replacement sewer main has been designed to minimize potential for conflicts with known underground utilities, provide space for future water pipeline improvements (Waterline Intertie Project 12-inch PVC water main), and maximize separation from existing water mains and valves. Minimum separation between the installed 21-inch sewer and the 16-inch high pressure gas main will be approximately 12 feet. No less than 8.5 feet of separation from the gas main should be maintained throughout construction activities (including manhole excavations). A minimum of 1 foot of vertical clearance will be provided between the sewer and utility crossings. California Department of Public Health (CDPH) pipeline separation criteria between existing water and sewer will be maintained where possible. Special construction measures will be specified where separation criteria can not be met. The Contractor will be responsible for confirming locations of existing utilities and protecting utilities in-place (in conformance with the respective utility's requirements).

The greatest potential for utilities conflicts are at locations where side main tie-ins and installation of connection manholes are planned. Connection of existing mains to the new sewer main may require progressive abandonment of the existing 12-inch sewer, allowing downstream mains to remain in operation while the upstream side main is connected to the new sewer. An existing storm drain crossing near Station 5011+17 should be investigated to confirm depth near on the Frontage Road replacement sewer alignment. AECOM recommends subsurface utilities exploration (potholing) for the storm drain crossing near Margie Place is performed during design phase to reduce the potential for change orders during construction. AECOM will coordinate this with the District.

9.4.9 Consideration of Pipe Reaming for Sewer Replacement

Pipe reaming is a trenchless method for replacing existing pipes using a modified back-reaming directional drilling method. Pipe reaming typically involves the insertion of a pilot line into the existing pipe, connecting a special reaming tool, then pulling the reaming head through the existing pipe. The reaming head grinds the existing pipe and simultaneous installs a replacement pipe behind the head. The ground pipe and other material are rejected in a drilling slurry. Metal fittings on the existing pipe or buried material near the pipe zone that cannot be ground are significant obstacles for pipe reaming.

Sufficient area is available for relocation of the Frontage Road sewer with minimal anticipated conflicts along the new alignment. Relocation of the sewer improves separation from the existing water mains and service valves along Frontage Road and allows continued operation of the existing 12-inch sewer while the replacement sewer is constructed. Finally, the significantly larger replacement sewer diameter (up-sizing) and inability of the pipe reaming method to match existing invert elevations at the proposed tie-in points may add further complexity to design and construction of the Frontage Road sewer upgrade. Therefore trenchless construction is not recommended for this project.

9.5 Construction Phasing and Waterline Intertie Improvements

The Waterline Intertie Project improvements on Frontage Road include a 12-inch water main near the alignment of the existing 12-inch sewer. The proposed Frontage Road replacement sewer main will be constructed along a new alignment approximately 16 feet east of the existing 12-inch sewer. This parallel alignment will allow the existing 12-inch sewer main to remain in operation during construction, thereby reducing the need for long duration bypass pumping, associated risk and project costs. Once the 21-inch replacement sewer is installed and tested, bypass pumping will be required to construct the tie-ins near the WWTF and at Division Street. Following connection of the replacement sewer, temporary diversion and/ or bypass pumping could be used for connecting side sewer mains and laterals, starting at Division Street and working downstream toward the WWTF. As connections are made, the 12-inch sewer will be abandoned in place. Once the existing 12-inch sewer and existing manholes are abandoned, waterline improvements could then be constructed on the proposed WIP alignment.

In summary, the following phasing is anticipated:

1. Construct the of 21-inch Frontage Road replacement sewer main while maintaining flow in the existing 12-inch sewer main

2. Connect the 21-inch sewer at the WWTF and Division Street

3. Connect side mains and lateral(s) and progressively abandon existing 12" sewer main

4. Install Waterline Intertie 12-inch potable water main

Sewer bypasses should be coordinated with project traffic control and will be the responsibility of the Contractor. A technical specification for temporary sewage bypass will be included in construction documents to specify the minimum requirements including anticipated flows and Contractor responsibilities. The technical specification will require the Contractor to submit a sewage bypass plan for District review/ approval.

It is assumed the contractor will be responsible for preparing and submitting traffic control plans along Frontage Road and on side streets, as necessary, for main connections and other construction activities.

9.6 Coordination with Southland WWTF Upgrades

Since the District is initiating design of the Southland WWTF with AECOM as the design engineer, there is an opportunity to reduce construction cost by packaging the planned WWTF influent sewer upgrade with the Frontage Road Sewer Replacement Project. As described in this memorandum, the current Frontage Road sewer replacement will end just outside the Southland WWTF. A contractor selected for the Southland WWTF upgrade project would begin construction of the influent main upgrade from that point. Due to the similar type of construction, necessary equipment and construction materials, obtaining a single contractor for the Frontage Road Sewer Replacement Project and the WWTF influent sewer main upgrade is expected to reduce net construction cost. Depending on the timeline for the Southland WWTF Upgrade Project, scheduling conflicts pertaining to construction of the influent sewer could also arise between contractors for the separate construction projects. AECOM will discuss combining the Frontage Road sewer replacement and the WWTF influent main upgrade further with the District. Although additional surveying and geotechnical evaluation will be necessary prior to initiating design of WWTF influent main improvements, incorporating design of the influent sewer man with the Frontage Road Sewer replacement is not expected to impact the WIP schedule.

9.7 Preliminary Plans and Technical Specifications

Preliminary drawings (30% planset) showing plan and profile views of the proposed 21-inch sewer replacement are included in Volume 3 of this Report. An outline of Technical Specifications is included in Appendix F.

9.8 Opinion of Probable Construction Cost

This opinion of probable construction cost represents judgment as a design professional and is supplied for the general guidance of the District. Since AECOM has no control over the cost of labor and materials, over delays in project bidding or award, over competitive bidding or market conditions, AECOM does not guarantee the accuracy of such opinions as compared to contractor bids or actual cost to the District. Refinements to this preliminary opinion of cost will be provided with subsequent design package submittals. The project construction cost opinion is summarized in Table 9-4, below.

Table 9-4. FRONTAGE ROAD REPLACEMENT

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization	1	LS	$23,750	$23,750
2	Traffic Control and Regulation	1	LS	$45,000	$45,000
3	Sheeting and Shoring	1	LS	$80,000	$80,000
4	Asphalt Pavement Removal & Restoration	17,050	SF	$9	$157,000
5	Furnish and install 21-inch PS 46 PVC sewer	3,100	LF	$192	$594,000
6	Furnish and install 60" sewer manhole (15 - 20 ft deep)	6	EA	$12,000	$72,000
7	Furnish and install 60" sewer manhole (10 - 14 ft deep)	5	EA	$10,000	$50,000
8	Abandon existing 12-inch sewer and remove MHs	1	LS	$30,000	$30,000
9	Connect 21-inch sewer to existing MH at Division	1	LS	$7,000	$7,000
10	Temporary sewage bypass	1	LS	$60,000	$60,000
11	Remake 8" SDR 35 Sewer lateral	1	EA	$3,000	$3,000
12	Sewer main replacement (8" SDR 35 PVC)	90	LF	$175	$15,750
13	Sewer main replacement (12" SDR 35 PVC)	30	LF	$200	$6,000
14	Connect side mains	6	EA	$3,500	$21,000
15	Pre-cast 48" sewer manhole (10 - 14 ft deep)	4	EA	$8,000	$32,000
Additive Bid Items					
A1	Scarify, condition, compact trench subgrade for 21-in Sewer	3,300	LF	$13	$43,000
	Sub Total				$1,239,500
	Contingency	20%			$247,900
	Total				**$1,488,000**