METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

FINAL

REGIONAL RECYCLED WATER PROGRAM

Backbone Conveyance System | Feasibility Level Design Report

Volume III of III

June 2020

Black & Veatch
Project No. 191628

BLACK & VEATCH

IN ASSOCIATION WITH
CDM Smith

FINAL

REGIONAL RECYCLED WATER PROGRAM

Backbone Conveyance System
Feasibility-Level Design Report
Volume 3 - Appendices L-W

BLACK & VEATCH PROJECT NO. 191628

PREPARED FOR

Metropolitan Water District of Southern California

30 JUNE 2020

In association with

Potential Regional Recycled Water Program
Black & Veatch Project 191628

Backbone Conveyance System Feasibility-Level Design Report
June 2020

Prepared by:

Andrew Stanton, P.E.
Black & Veatch

Matthew Thomas, P.E.
Black & Veatch

Lane Pagano, P.E.
Black & Veatch

Approved by:

Jay Arabshahi, P.E.
Project Manager

Bruce Chalmers, P.E.
Program Manager
Engineering Services Group

Table of Contents

LIST OF TABLES

LIST OF FIGURES

ACRONYM AND ABBREVIATIONS LIST

The following abbreviations or acronyms are used in this document.

AACE	Association for the Advancement of Cost Engineering
ARVV	air-release and vacuum valve
AWT	advanced water treatment
Black & Veatch	Black & Veatch Corporation
BEP	best-efficiency point
CalOSHA	California Occupational Safety and Health Administration
Caltrans	California Department of Transportation
CEQA	California Environmental Quality Act
cf	cubic feet
CGS	California Geologic Survey
CM	construction method
CNDDB	California Natural Diversity Database
DPR	direct potable reuse
EPBM	earth pressure balance tunnel boring machine
FEWWTP	F.E. Weymouth Water Treatment Plant
ft	feet
FLDR	Feasibility-Level Design Report
fps	feet per second
GAC	granular activated carbon
GeoPentech	GeoPentech Inc
GIS	geographic information system
gpm	gallons per minute
HDD	horizontal directional drilling
HGL	hydraulic grade line
HI	Hydraulic Institute
HP	horsepower
ID	inside diameter
in	inches
IPR	indirect potable reuse
IRRP	Indirect Reuse Replenishment Project
IPR	indirect potable reuse
JWPCP	Joint Water Pollution Control Plant
kWh	kilowatt hour
LA	Los Angeles
LACDPW	Los Angeles County Department of Public Works

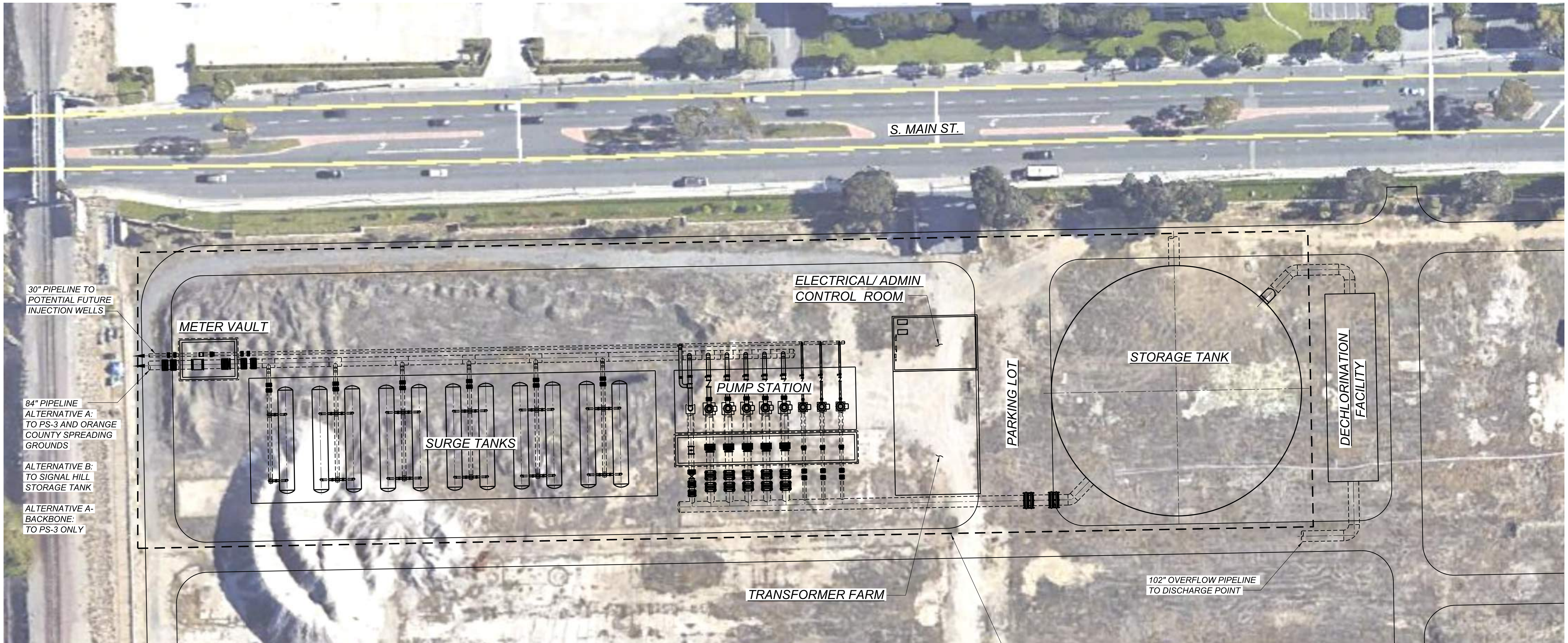
LACFCD	Los Angeles County Flood Control District
LACSD	Sanitation Districts of Los Angeles County
LADWP	Los Angeles Department of Water and Power
LUFT	leaking underground storage tank
MCAA	Mechanical Contractors Association of America
MCCs	motor control centers
Metropolitan	Metropolitan Water District of Southern California
MG	million gallons
mg/L	milligrams per liter
mgd	million gallons per day
Minagar	Minagar & Associates, Inc.
MJA	McMillan Jacobs Associates
MT	microtunneling
M_W	moment magnitude scale
NECA	National Electrical Contractors Association
OC	Orange County
OC Reach	optional branch to the Orange County Spreading Grounds
OCSD	Orange County Sanitation District
OCWD	Orange County Water District
OD	outside diameter
O&M	operations and maintenance
OPCC	opinion of probable construction cost
Project	design of the conveyance facilities of the Regional Recycled Water Program
PS	pump station
PS-1	Pump Station 1
PS-2	Pump Station 2
PS-3	Pump Station 3
RPM	revolutions per minute
RRWP	Regional Recycled Water Program
RVs	recreational vehicles
SCE	Southern California Edison
SFSG	Santa Fe Spreading Grounds
SG	San Gabriel
SWRCB	State Water Resources Control Board
TBM	tunnel boring machine
TCE	trichloroethylene
USGMWD	Upper San Gabriel Municipal Water District
VFD	variable frequency drive

WBS	work breakdown structures
WRD	Water Replenishment District of Southern California
WSE	water surface elevation

Appendix L. Concept Pump Station Site Layouts

A B C D E F G H I J K L

1 2 3 4 5 6 7 8



PUMP STATION 1, SEE

1	1
M-1	M-2

NOTES:

1. PUMP STATION LAYOUT REFLECTS ALTERNATIVE A CONFIGURATION.

PLAN | 1
SCALE: 1"=40'-0" | -

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

SCALE BARS
1" = 40'
0 40 80 120 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

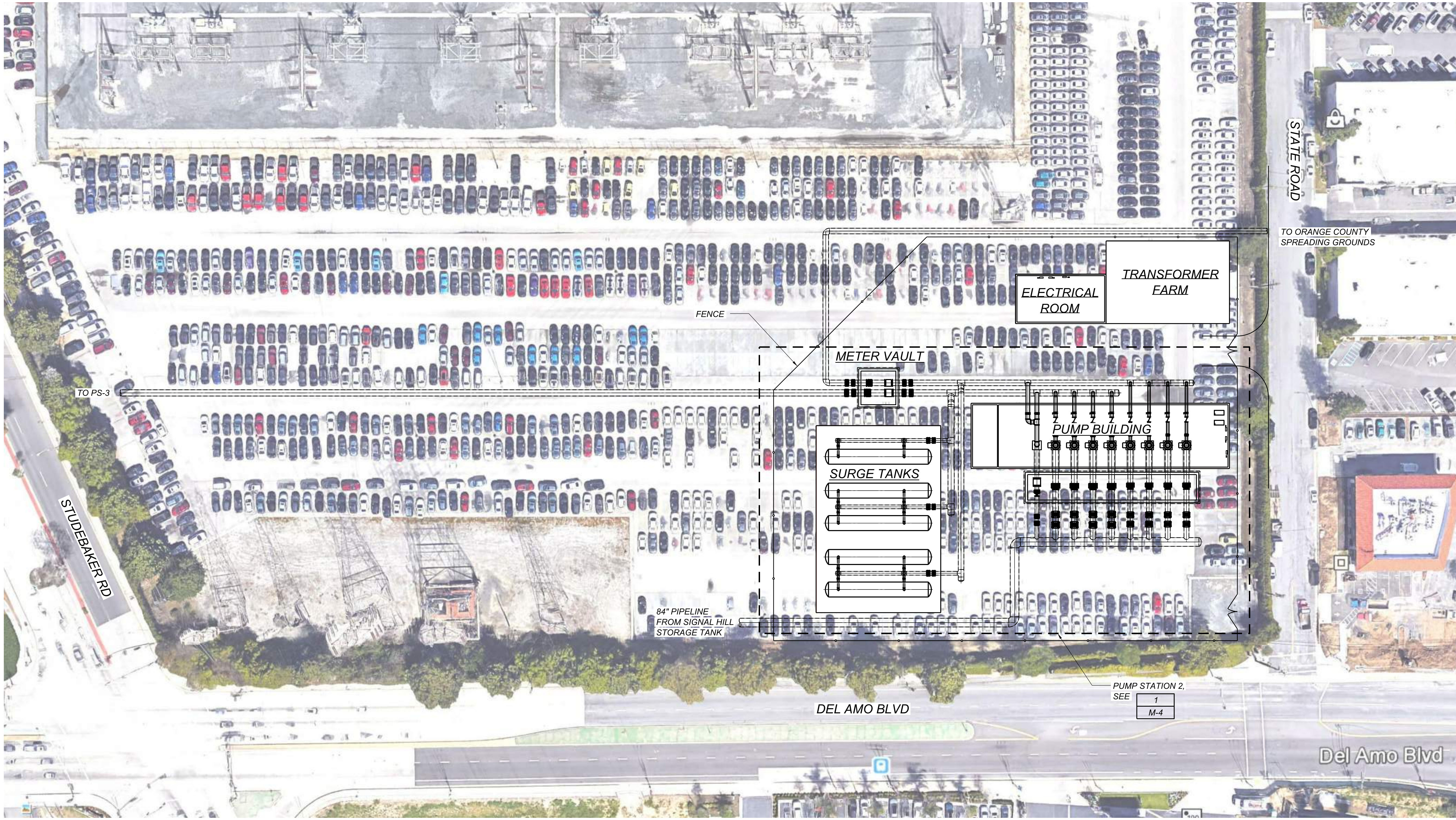
DESIGNED
DRAWN
CHECKED
FOR DRAWING APPROVALS SEE

WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

PS-1 SITE PLAN

SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET C-1
DWG
REV 0

BRDR DATE: 05/10/2016
PEN TABLE: MWD-HalfSizeBW.pen
PLOT TIME: 24-SEP-2018 12:03
USERID: svc_projectwise
FILE NAME: \\Usmwd235\pw_ics_pdf\\59095\120804_2\Study_104168_00c001.dgn



PLAN 1 -
SCALE: 1"=40'-0"

SCALE BARS
1" = 40' 0 40 80 120 FEET

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

DESIGNED
DRAWN
CHECKED
FOR DRAWING APPROVALS SEE

WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

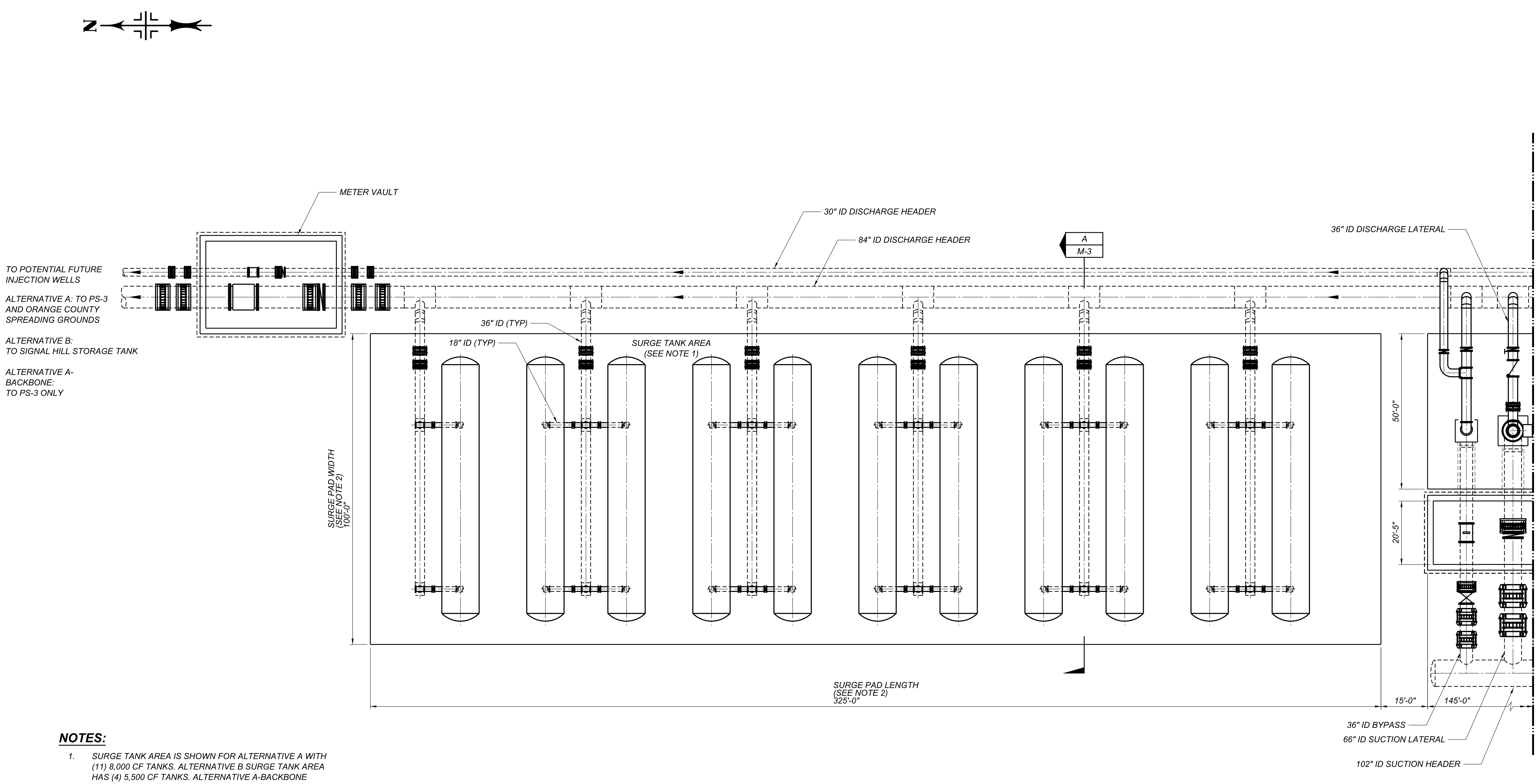
PS- 2 SITE PLAN
ALTERNATIVE B ONLY

SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET C-2
DWG
REV 0

BRDR DATE: 05/10/2016
PEN TABLE: MWD-HalfSizeBW.pen
PLOT TIME: 07-NOV-2017 09:50
USERID: svc_projectwise
FILE NAME: \\Usmwd235\pw_ics_pdf\45514\120804_4\Study_104168_00c002.dgn

A B C D E F G H I J K L

1 2 3 4 5 6 7 8



NOTES:

1. SURGE TANK AREA IS SHOWN FOR ALTERNATIVE A WITH (11) 8,000 CF TANKS. ALTERNATIVE B SURGE TANK AREA HAS (4) 5,500 CF TANKS. ALTERNATIVE A-BACKBONE SURGE TANK AREA HAS (6) 6,000 CF TANKS.
2. SURGE PAD IS SHOWN FOR ALTERNATIVE A WITH DIMENSIONS 325'x100'. ALTERNATIVE B SURGE PAD IS 141'x100'. ALTERNATIVE A-BACKBONE SURGE PAD IS 202'x100'.

PLAN 1
SCALE: 1/16" = 1'-0" -

SCALE BARS 1/16" = 1'-0" 0 16 32 48 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

DESIGNED | FOR DRAWING APPROVALS SEE
DRAWN
CHECKED

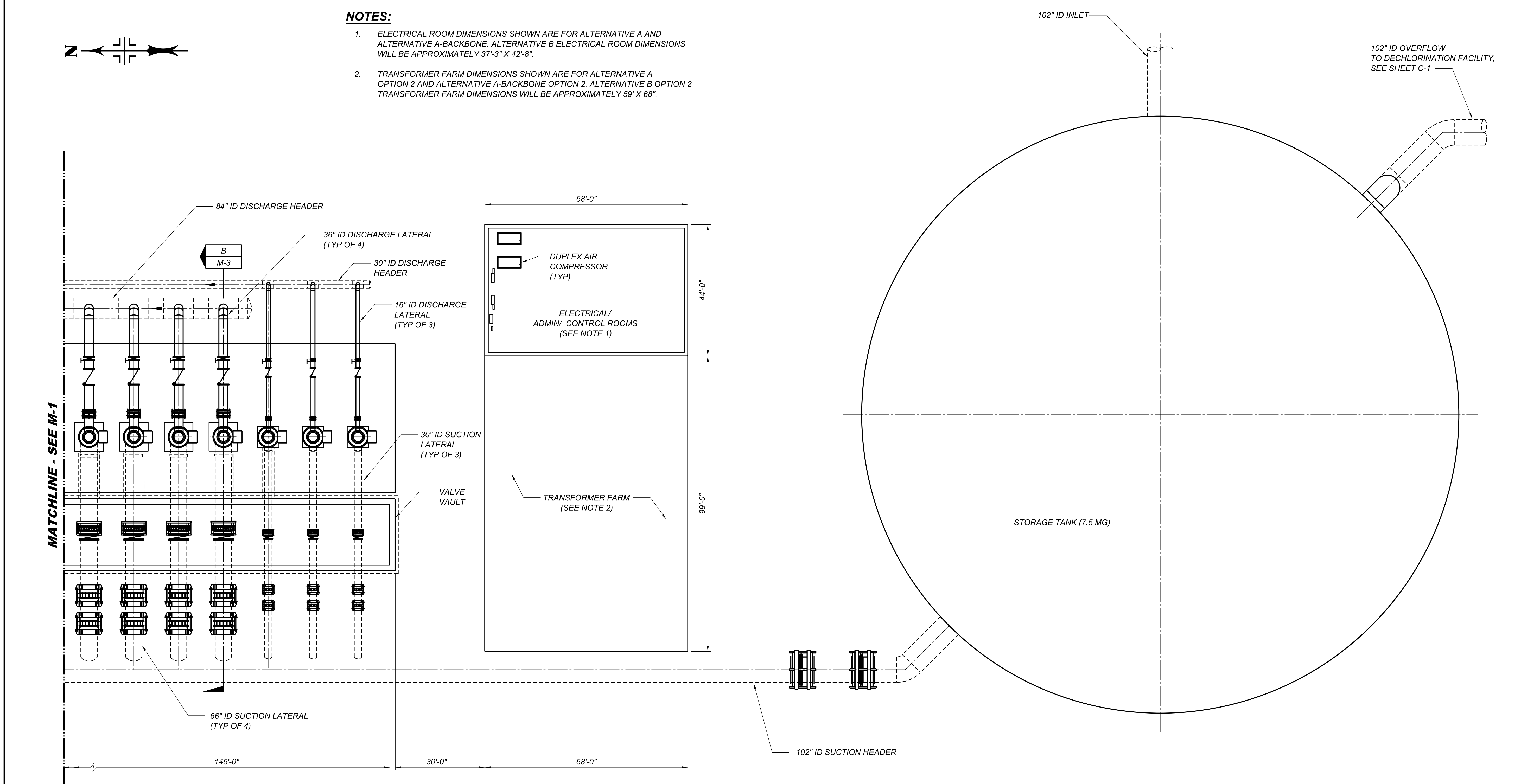
WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

PUMP STATION 1
PLAN
SHEET 1 OF 2

SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET M-1
DWG
REV 0

NOTES:

1. ELECTRICAL ROOM DIMENSIONS SHOWN ARE FOR ALTERNATIVE A AND ALTERNATIVE A-BACKBONE. ALTERNATIVE B ELECTRICAL ROOM DIMENSIONS WILL BE APPROXIMATELY 37'-3" X 42'-8".
2. TRANSFORMER FARM DIMENSIONS SHOWN ARE FOR ALTERNATIVE A OPTION 2 AND ALTERNATIVE A-BACKBONE OPTION 2. ALTERNATIVE B OPTION 2 TRANSFORMER FARM DIMENSIONS WILL BE APPROXIMATELY 59' X 68".



PLAN	1
SCALE: 1/16" = 1'-0"	-

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

SCALE BARS 1/16" = 1'-0" 0 16 32 48 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

DESIGNED | DRAWN | CHECKED

FOR DRAWING APPROVALS SEE

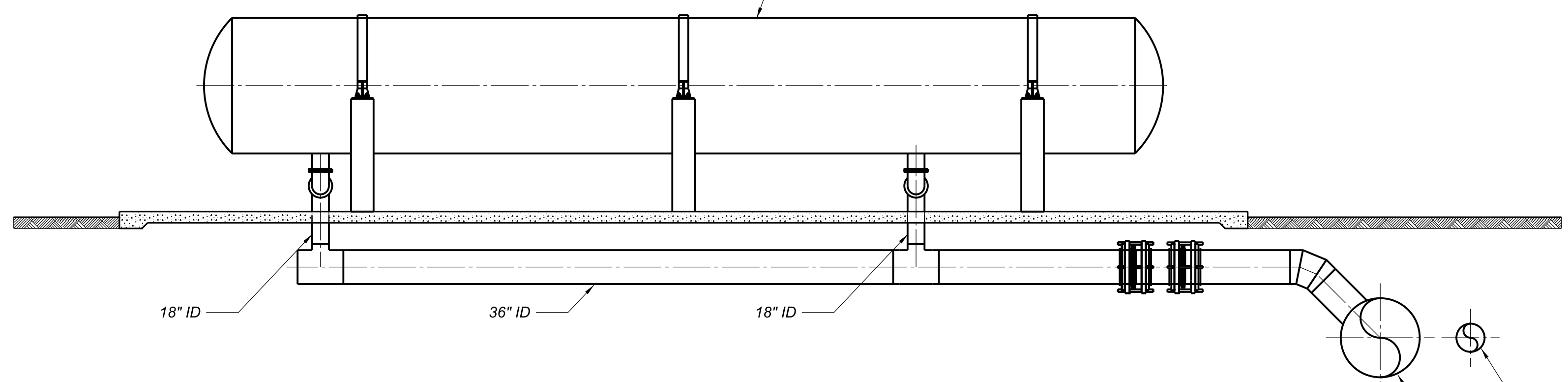
WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

PUMP STATION 1
PLAN
SHEET 2 OF 2

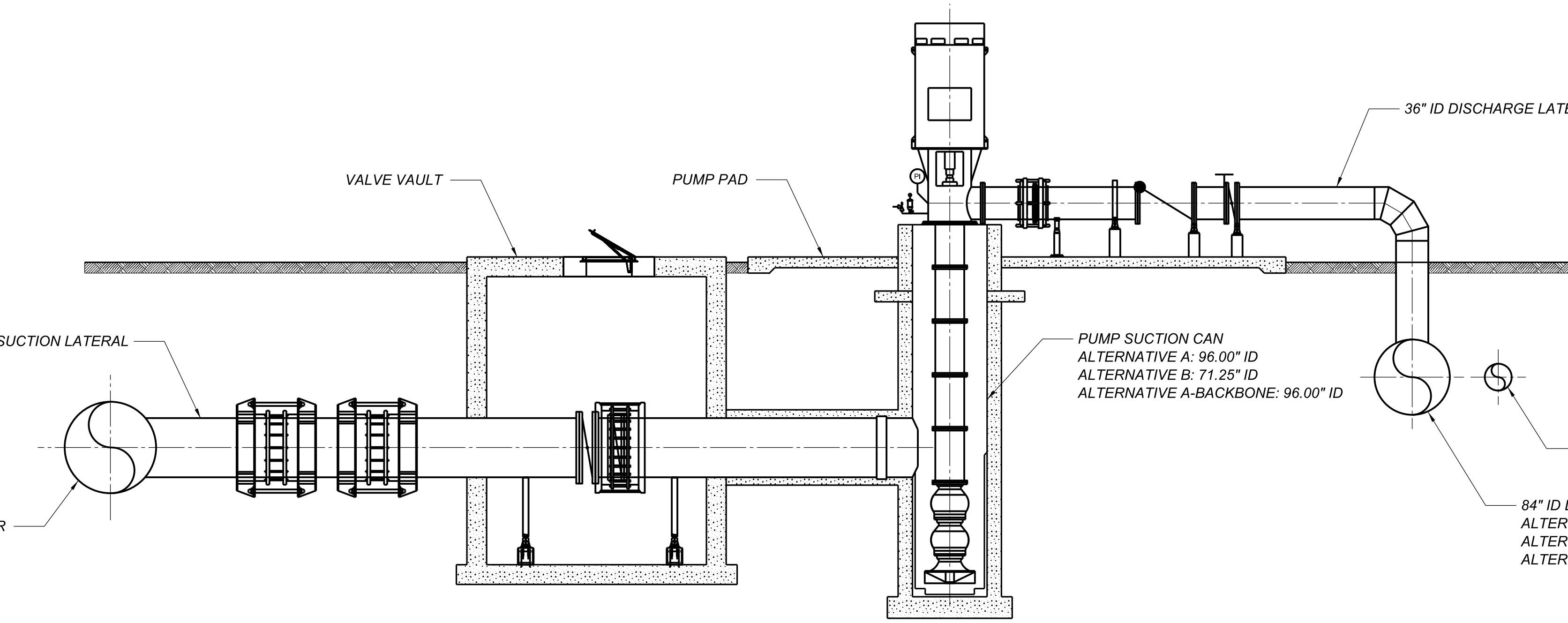
SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET M-2
DWG
REV 0

BRDR DATE: 05/10/2016 PEN TABLE: MWD-HalfSizeBW.pen PLOT TIME: 24-SEP-2018 12:50 USERID: svc_projectwise FILE NAME: \\Usmwd235\pw_ics_pdf\59102\120812_8\Study_104168_00m002.dgn

A B C D E F G H I J K L



SECTION A / M-1
SCALE: 1/8" = 1'-0"



SECTION B / M-2
SCALE: 1/8" = 1'-0"

1 2 3 4 5 6 7 8

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

SCALE BARS
1/8" = 1'-0" 0 8 16 24 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

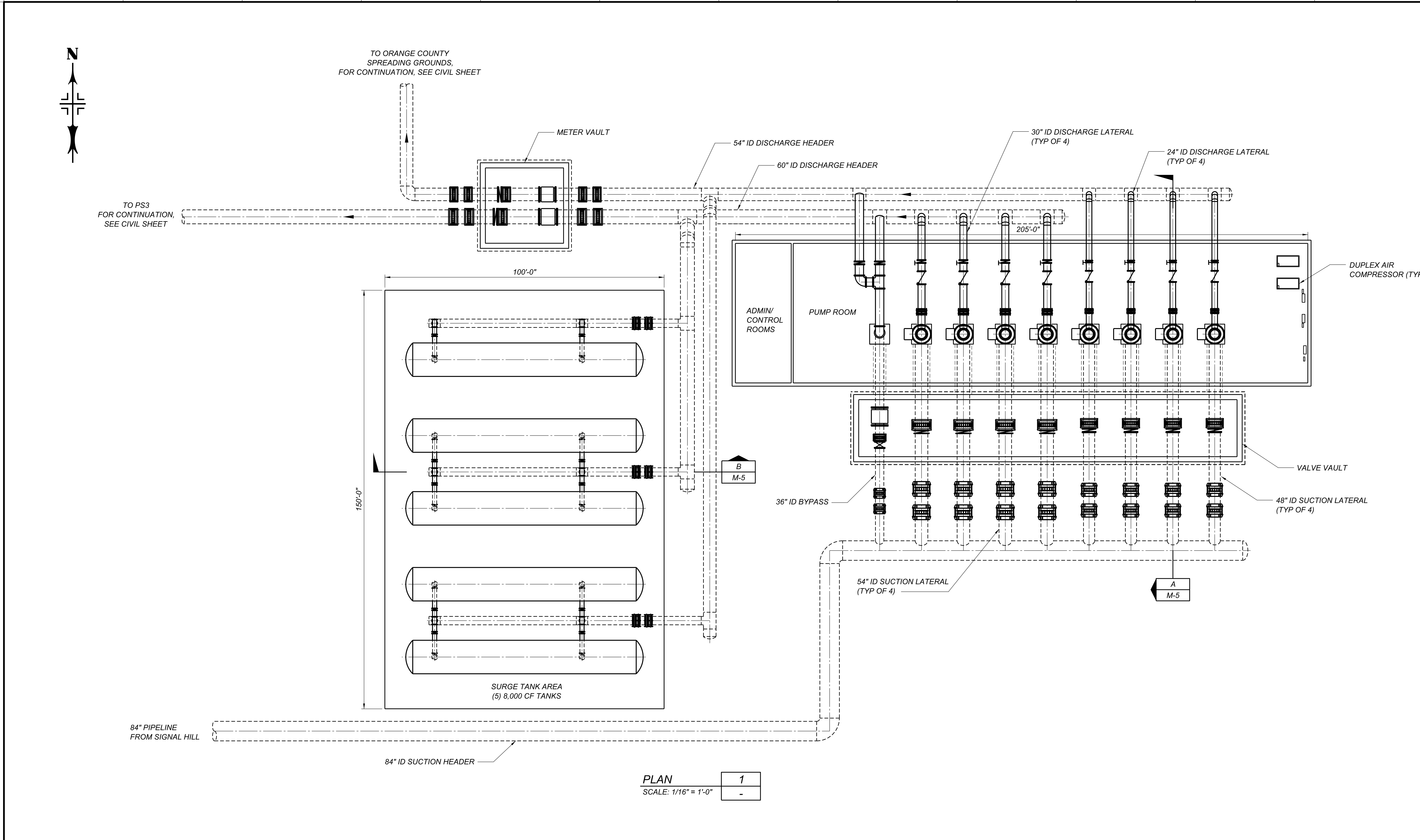
DESIGNED
DRAWN
CHECKED
FOR DRAWING APPROVALS SEE

WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

PUMP STATION 1
SECTIONS

SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET M-3
DWG
REV 0

BRDR DATE: 05/10/2016 | PEN TABLE: MWD-HalfSizeBW.pen | PLOT TIME: 20-SEP-2018 06:44 | USERID: svc_projectwise | FILE NAME: \\Usmwd235\pw_ics_pdf\59009\120812_11\Study_104168_00m003.dgn



PLAN | 1
SCALE: 1/16" = 1'-0" | -

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

SCALE BARS
1/16" = 1'-0"   0   16   32   48 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF
SOUTHERN CALIFORNIA

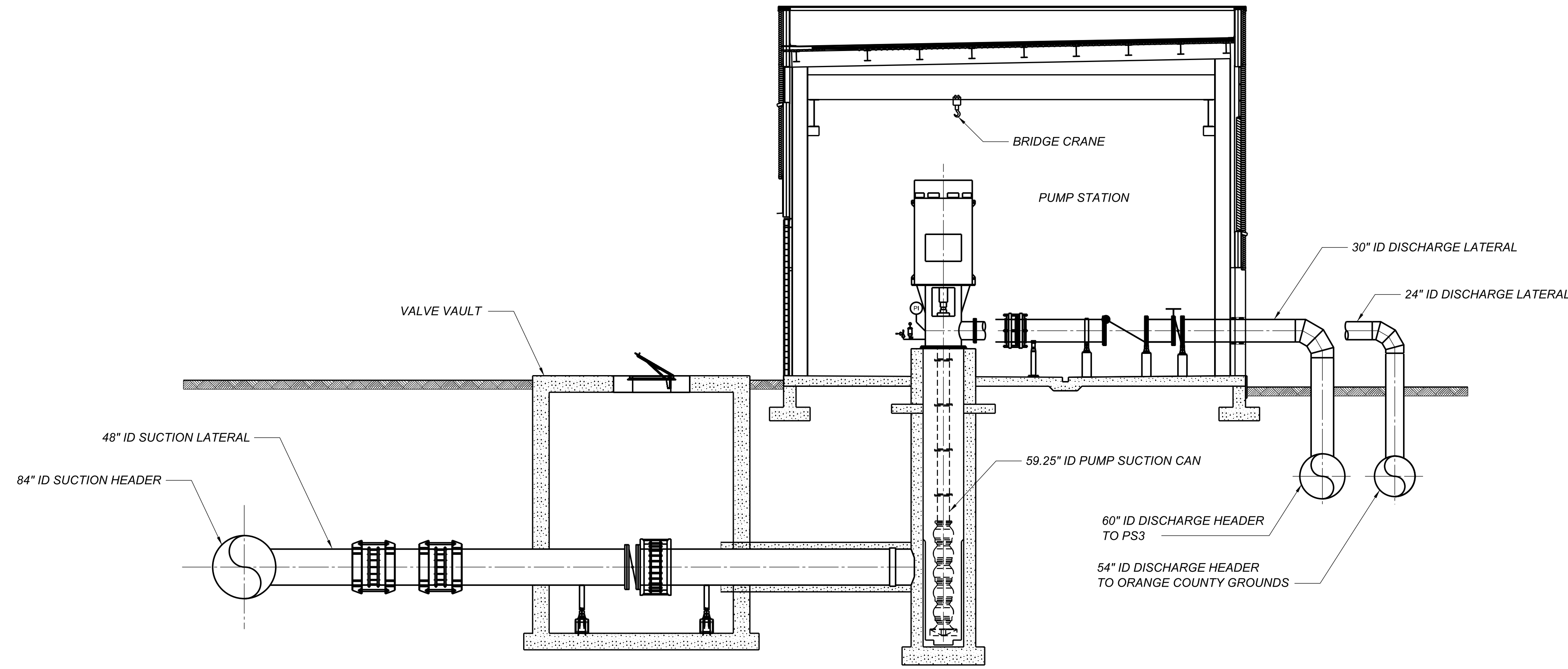
DESIGNED | FOR DRAWING APPROVALS SEE
DRAWN
CHECKED

WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

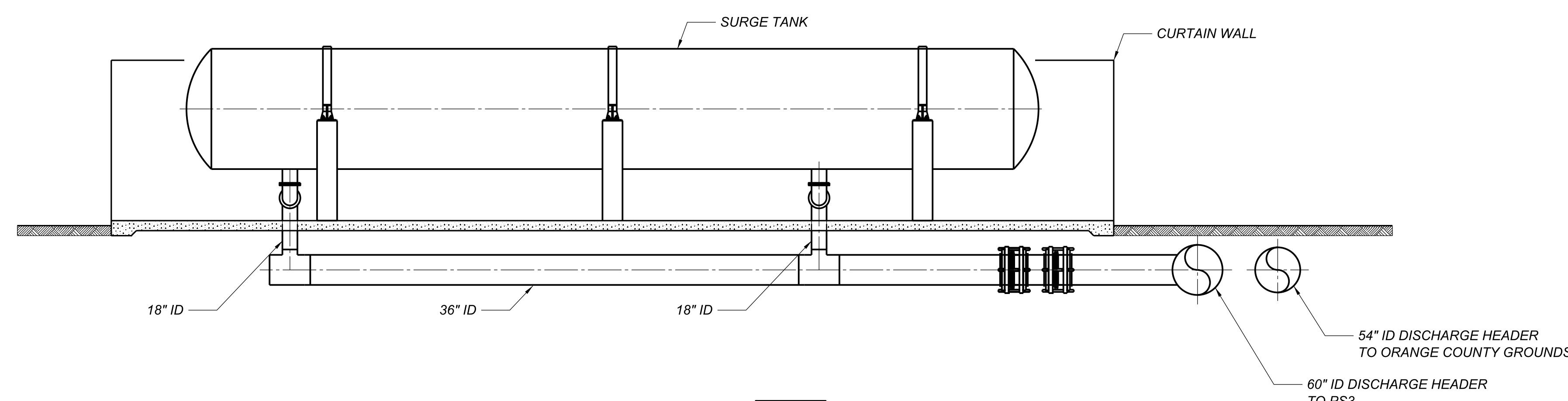
PUMP STATION 2 PLAN
ALTERNATIVE B ONLY

SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET M-4
DWG
REV 0

BRDR DATE: 05/10/2016 | PEN TABLE: MWD-HalfSizeBW.pen | PLOT TIME: 03-NOV-2017 14:36 | USERID: svc_projectwise | FILE NAME: \\Usmwd235\pw_ics_pdf\45415\120812_4\Study_104168_00m004.dgn



SECTION A
SCALE: 1/8" = 1'-0" M-4



SECTION B
SCALE: 1/8" = 1'-0" M-4

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

SCALE BARS
1/8" = 1'-0" 0 8 16 24 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF
SOUTHERN CALIFORNIA

DESIGNED
DRAWN
CHECKED
FOR DRAWING APPROVALS SEE

WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

PUMP STATION 2 SECTION
ALTERNATIVE B ONLY

SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET M-5
DWG
REV 0

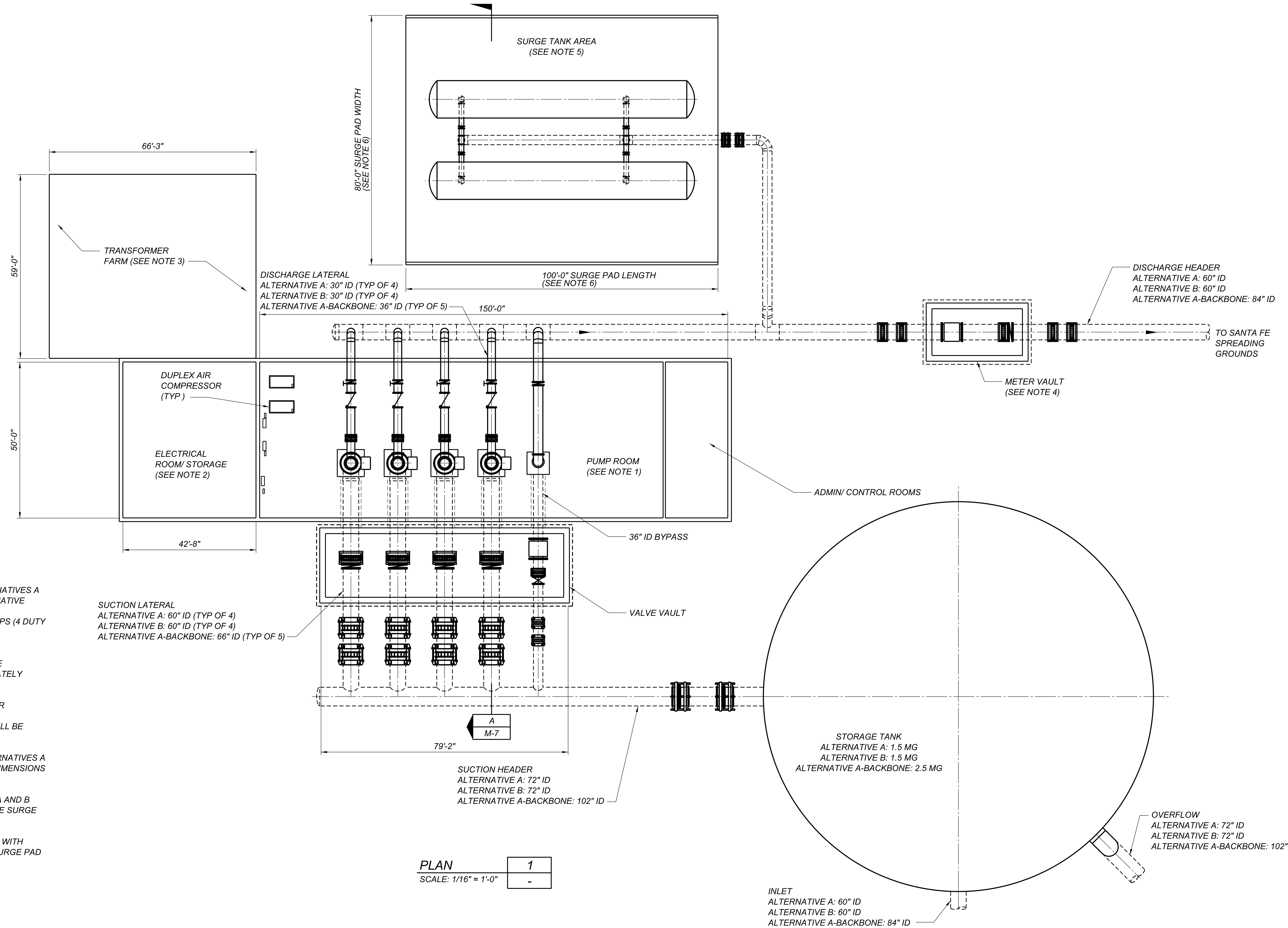
BRDR DATE: 05/10/2016
PEN TABLE: MWD-HalfSizeBW.pen
PLOT TIME: 14-NOV-2017 09:40
USERID: svc_projectwise
FILE NAME: \\Usmwd235\pw_ics_pdf\45834\120812_6\Study_104168_00m005.dgn

A B C D E F G H I J K L

1 2 3 4 5 6 7 8

K

N



NOTES:

1. PUMP ROOM DIMENSIONS SHOWN ARE FOR ALTERNATIVES A AND B FOR 4 PUMPS (3 DUTY + 1 STANDBY). ALTERNATIVE A-BACKBONE PUMP ROOM DIMENSIONS WILL BE APPROXIMATELY 165'x50' TO ACCOMMODATE 5 PUMPS (4 DUTY + 1 STANDBY).
2. ELECTRICAL ROOM DIMENSIONS SHOWN ARE FOR ALTERNATIVES A AND B. ALTERNATIVE A-BACKBONE ELECTRIAL ROOM DIMENSIONS WILL BE APPROXIMATELY 68'x44'.
3. TRANSFORMER FARM DIMENSIONS SHOWN ARE FOR ALTERNATIVES A AND B, OPTION 2. ALTERNATIVE A-BACKBONE TRANSFORMER FARM DIMENSIONS WILL BE APPROXIMATELY 99'x66'-3".
4. METER VAULT DIMENSIONS SHOWN ARE FOR ALTERNATIVES A AND B. ALTERNATIVE A-BACKBONE METER VAULT DIMENSIONS WILL BE APPROXIMATELY 17'x28'.
5. SURGE TANK AREA IS SHOWN FOR ALTERNATIVES A AND B WITH (2) 8,000 CF TANKS. ALTERNATIVE A-BACKBONE SURGE TANK AREA HAS (4) 6,000 CF TANKS.
6. SURGE PAD IS SHOWN FOR ALTERNATIVES A AND B WITH DIMENSIONS 100'x80'. ALTERNATIVE A-BACKBONE SURGE PAD IS 141'x100'.

PLAN | 1
SCALE: 1/16" = 1'-0" | -

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

SCALE BARS
1/16" = 1'-0" 0 16 32 48 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF
SOUTHERN CALIFORNIA

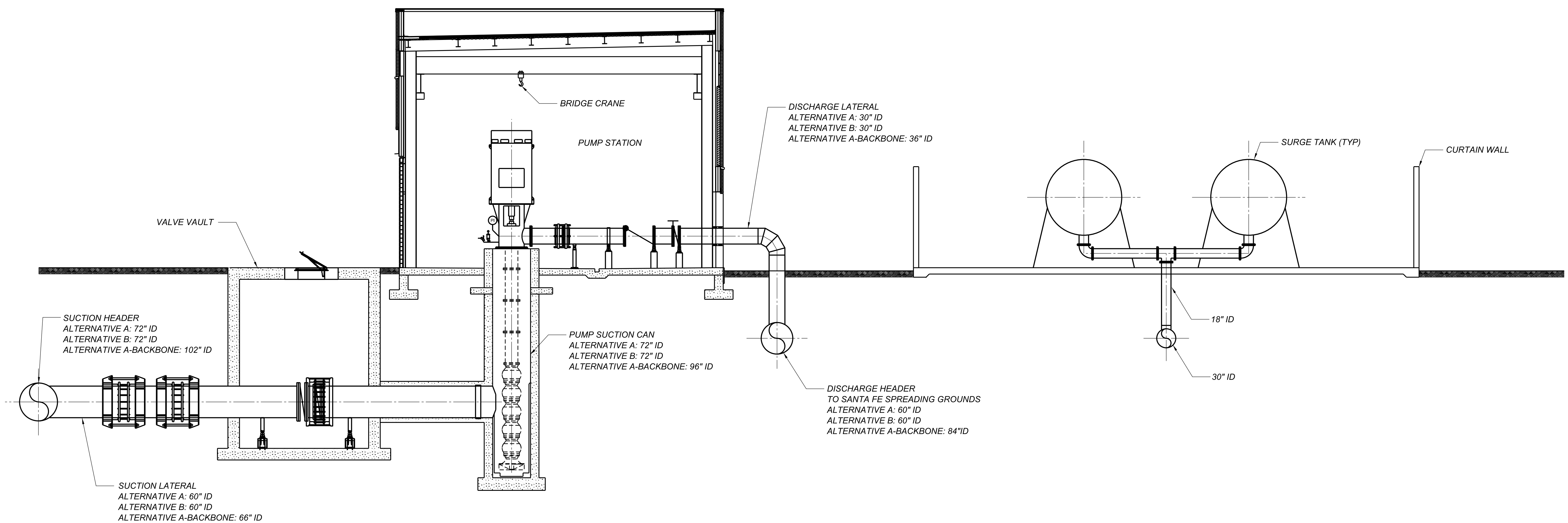
DESIGNED | FOR DRAWING APPROVALS SEE
DRAWN
CHECKED

WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

PUMP STATION 3
PLAN

SPECIFICATIONS
STUDY
PROJECT NUMBER
104168
SHEET
M-6
DWG
REV
0

BRDR DATE: 05/10/2016 PEN TABLE: MWD-HalfSizeBW.pen PLOT TIME: 24-SEP-2018 12:05 USERID: svc_projectwise FILE NAME: \\Usmwd235\pw_ics_pdf\\59097\120812_12\Study_104168_00m006.dgn



SECTION A / M-6
SCALE: 1/8" = 1'-0"

CDM Smith
600 Wilshire Boulevard, Suite 750
Los Angeles, CA 90017
Tel: (213) 457-2200

SCALE BARS
1/8" = 1'-0" 0 8 16 24 FEET

STAMPING ENGINEER IS RESPONSIBLE FOR CURRENT DRAWING REVISION

ISSUE DESCRIPTION

ISSUE DATE

MWD
METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

DESIGNED
DRAWN
CHECKED
FOR DRAWING APPROVALS SEE

WATER TREATMENT PLANTS
RECYCLED WATER DEMONSTRATION PLANT
REGIONAL RECYCLED WATER SUPPLY

PUMP STATION 3
SECTION

SPECIFICATIONS STUDY
PROJECT NUMBER 104168
SHEET M-7
DWG
REV 0

BRDR DATE: 05/10/2016
PEN TABLE: MWD-HalfSizeBW.pen
PLOT TIME: 20-SEP-2018 07:07
USERID: svc_projectwise
FILE NAME: \\Usmwd235\pw_ics_pdf\59011\120812_13\Study_104168_00m007.dgn

Appendix M. Pipeline Unit Cost Development for Construction Methods and Adders

M.1 CM1 - ROADWAYS

M.2 CM2 - SCE EASEMENTS

M.3 CM3 - LACFCD EASEMENTS

M.4 CM4 - TRENCHLESS

M.5 COST "ADDERS"

CONFIDENTIAL

Appendix N. Pipeline Quantity Take-Off

CONFIDENTIAL

Appendix O. Pipeline Opinion of Probable Construction Cost

CONFIDENTIAL

Appendix P. Pump Station Opinion of Probable Construction Cost

CONFIDENTIAL

Appendix Q. Hydraulic High Point Memo

CONFIDENTIAL

DRAFT/FINAL

HYDRAULIC HIGH POINT IN REACH 1

Potential Regional Recycled Water Program

B&V PROJECT NO. 191628

PREPARED FOR

Metropolitan Water District of Southern California

16 AUG 2017

Table of Contents

LIST OF TABLES

LIST OF FIGURES

1 Introduction and Purpose

A comparison of the Reach 1 Preferred Alignment's elevation profile and the initial hydraulic grade line (HGL) reveals a high point in the alignment between Pump Station 1 (PS1) and Pump Station 2 (PS2). When the system is operated at its full 150 mgd capacity, the HGL will be above the top of the pipeline. However, as shown in Figure 1-1, the HGL falls below the top of pipe elevation for flowrates less than approximately 140 mgd.

Six concept level alternatives were identified and evaluated for conveying flows over (or in the case of Alternative 3, around) the high point and were presented to the Metropolitan Water District of Southern California (Metropolitan) staff at a coarse screening workshop on June 14th, 2017. The six alternative concepts presented were as follows:

- Alternative 1 – Preferred Alignment: Pressurized and Gravity Flow
- Alternative 2 – Preferred Alignment: Pressurized Flow
- Alternative 3 – Reroute the Preferred Alignment to Del Amo Boulevard
- Alternative 4 – Relocate PS2's Wet Well and Use Can Pumps at PS2
- Alternative 5 – Tunnel Below HGL
- Alternative 6 – Eliminate PS2

At the coarse screening workshop, Alternatives 2, 3, and 4 were dismissed and additional analysis was requested on Alternatives 1, 5, and 6 to assist in the selection of a preferred alternative. Referred to as "Fine Screening," this Memorandum documents those additional evaluations completed on Alternatives 1, 5, and 6, which include:

- Conceptual level cost estimates
- Pipe wall thickness analysis
- Brief comparison on surge control
- Benefits of liner options

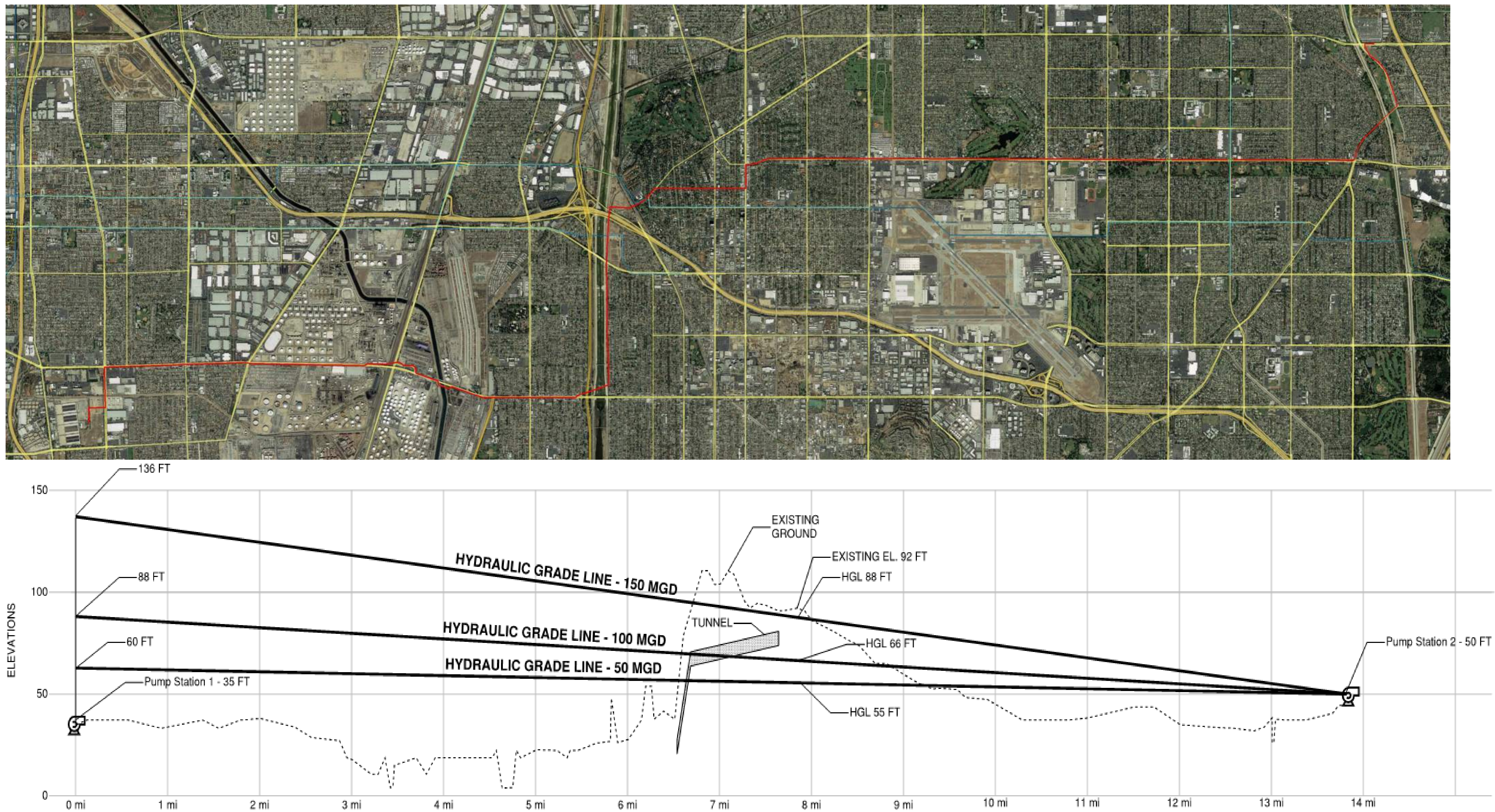


Figure 1-1 - Base Hydraulic Grade Line for Preferred Alignment from PS1 to PS2

This page is intentionally left blank.

2 Fine Screening

2.1 ALTERNATIVE 1 – PRESSURIZED AND GRAVITY FLOW

2.1.1 Description

Alternative 1 maintains the Preferred Alignment. The HGL for a range of flows is depicted in Figure 2-1. PS1 would pump at a pressure sufficient to convey flow over the high point. During lower flow conditions (less than approximately 140 mgd), the pipeline would transition from pressurized to gravity flow at the pipeline high point. This is similar in concept to how Metropolitan's Santiago Lateral is operated. Combination air and vacuum release valves would be required at the Reach 1 pipeline high point as well as at all other pipeline high points.

2.1.1.1 Complexities of a Combination Gravity and Pressurized Flow System

Due to the pipeline transitioning between fully pressurized flow and gravity flow under different flow scenarios, the functionality of the air release and intake system for the pipeline would be significantly more critical to operation. Various air release and intake systems, such as combination or individual air-vacuum valves and stand pipes, have been used successfully. Determination of the best air release and intake system for this project would be determined when the alignment and profile are finalized.

For all air release and intake systems, the system would be sized to allow large volumes of air to enter and exit the pipeline as the system transitions between pressurized and gravity flows. For vertical standpipes, the standpipe would need to extend high enough to remain above not only the peak flow HGL but also any surge pressures. The top of the standpipe would have to be protected to prevent foreign material from entering the system (i.e., a goose neck and screen).

Individual or combination air release and vacuum valves (ARVV) are mechanical systems relying on mechanical components to operate. As such, ARVVs require regular maintenance and testing to ensure reliable operation. Additionally, ARVVs will need to be carefully selected to assure they remain seated at low pressures. None of this is unusual for Metropolitan; Metropolitan has thousands of ARVVs in its system that require the same maintenance. The difference in this case is that in a fully pressurized system and under normal operation, the ARVVs are typically only releasing small pockets of air that accumulate at high points and/or relieving small vacuum issues that arise in the line. If the ARVVs malfunction or are not maintained in a timely manner, the system can continue to operate under its normal operation. The ARVVs are needed to allow large volumes of air in or out of the pipe only under controlled filling or draining operations, or to let air into the line to prevent pipe collapse if the main should break and rapidly drain.

In the case presented in this Alternative 1, the ARVVs will be relied upon to let large volumes of air in and out of the main routinely. Their operation will be more important under this Alternative. Risks associated with this alternative can be mitigated through inclusion of redundant ARVVs, and diligence in maintenance.

Standpipes are a passive system requiring little if any maintenance. A noise analysis may be required to assess system breathing sound impacts to nearby residences.

Water flowing over the high point and cascading down to the HGL on the downstream side under gravity flow could cause a large amount of air to be entrained. The entrained air would need to be accounted for in the design of the air valves downstream of the high point.

2.1.1.2 Liner Requirements

During the coarse screening workshop, concern was expressed that the wet-dry cycling of the liner material may accelerate deterioration of the liner material. Based on experience at other operating facilities, accelerated deterioration is not anticipated as long as water continues to flow through the system. The flow and conditions within the pipe will provide sufficient moisture (the water itself and humidity) to keep the liner material wet. Further hydraulic analysis may need to be conducted to ensure flow velocities within the gravity flow sections do not exceed maximum velocities recommended for the type of pipeline lining material. Based on a preliminary review of the alignment and potential pipe slopes, high pipeline flow velocities are not anticipated however.

At locations of hydraulic jumps, due to the transition of gravity flow to pressurized flow, there are additional concerns regarding longevity of the lining material. The problem generally occurs when the liner is field placed and is "thin" at the pipe crown. The solution is to ensure careful inspection and validation of the mortar lining thickness, cement content, and curing time. Additionally, a thicker liner (i.e., ¾ inch minimum with zero negation tolerance) or a welded-wire reinforcing fabric that is tack-welded to the pipe interior surface prior to field applying the mortar lining may be specified at the location of any hydraulic transitions to enhance longevity. Finally, carbon fiber or cured-in-place linings could be used. The cost and/or functionality impact at those specific locations are not significant enough to be a differentiator at this level of evaluation.

2.1.1.3 Reach 1 Pumping Inefficiencies

In Alternative 1, at flows of less than 140 mgd, PS1 would pump up to the high point in Reach 1, cascade over the high point, then flow open channel to a hydraulic jump where it would resume full pipe flow by gravity to PS2. This would result in system inefficiencies as higher pumping heads would be required for lower flow rates to reach the top of the high point than would otherwise be necessary to reach PS2 if there were no high point. At 50 mgd, approximately 26 feet of additional pumping head would be required to reach the high point, and, at 100 mgd, approximately 14 feet of additional pumping head would be required. If pumping above 140 mgd, the pipeline would be fully pressurized and no additional energy loss would be present due to the high point.

During the initial phases of the project when the system will likely be operated below its ultimately planned capacity, and at all other points of operation at lower than capacity flows, energy losses would be experienced in Alternative 1. The table below displays annual energy losses assuming consistent flows of 50 mgd or 100 mgd.

Table 2-1: Alternative 1 – Estimated Annual Energy Losses at Low Flows

VARIABLES	FLOW SCENARIO 1	FLOW SCENARIO 2
Discharge Flow (mgd)	50 mgd	100 mgd
Additional Pumping Head Required (ft)	26	14
Pump Efficiency (%)	75	75

VARIABLES	FLOW SCENARIO 1	FLOW SCENARIO 2
Horsepower (HP)	300	330
Additional Annual Power Consumption (kWhr/yr)	1,960,000	2,156,000
Pump Station Power Cost ($/kWh)	0.15	0.15
Annual Energy Losses ($/yr)	$294,000	$323,400

2.1.1.4 PS2 Wet Well Size and Control Complexities

Should PS1 stop operating for any reason, the water down stream of the high point in Reach 1 would continue to drain towards PS2, even after the PS1 pumps have stopped operating. In order to keep the PS2 wet well level from rising, the control system would need to keep PS2 operating until the gravity section of line had stopped draining. Alternatively, a motorized isolation valve could close at the PS2 wet well inlet to keep the pipe from draining, but the time for an 84-inch valve to fully close would typically be on the order of several minutes. For this reason the wet well volume at PS2 would need to be increased by up to 3 MG (over the currently identified size of 2.0 MG) or the chances of an overflow during an unexpected system shutdown would increase. Given the requirement for a buried wet well as the PS2 site and the constrained site conditions, the increase in wet well volume could be challenging to accommodate in the current sites being considered. Note that the currently identified 2.0 MG wet well was sized to accommodate the condition where PS2 stops operating while PS1 continues to pump, providing time to deactivate PS1 in a controlled manner.

2.1.1.5 Surge Control

In the event that the pumps at PS1 suddenly stop due to a loss of power (i.e., a ‘trip’ condition), the surge control in Reach 1 for Alternative 1 relies heavily upon ARVV’s and/or a vertical standpipe located in the vicinity of the high point to prevent potentially damaging negative pressure conditions. Although pressurized hydro-pneumatic surge tanks can be provided at PS1, the surge tanks themselves cannot prevent the negative pressure conditions at the higher elevations along the pipeline. Relying upon ARVV’s for primary surge control is not recommended according to Metropolitan’s standard hydraulic design approach. Using mechanical devices for surge control comes with additional risks and requires more intense transient flow analysis to ensure the design properly controls surges and maintains system integrity. In general, using ARVVs as the primary surge control device is only implemented when absolutely necessary and when no other passive means of protection are available.

2.1.2 Advantages

- Maintains PS2 for positive flow control to PS3 and Orange County via dedicated variable speed pumping equipment

2.1.3 Disadvantages

- Functionality of air release and intake system is more critical to operation. If a standpipe is used, care in siting would be required, a tall new facility at the high point would be a visual impact, and it may require land acquisition. .

- Potential travelling hydraulic jump in gravity sections requires special liner design in those areas
- Pumping inefficiencies at flow rates below 140 mgd
- Increased control complexity and need for larger wet well at PS2 due to pipeline draining from high point
- Larger wet well at PS2 results in additional congestion at potential PS2 sites
- System relies upon ARVV's and/or vertical standpipes for surge control, which is not recommended in Metropolitan's hydraulic design standards.

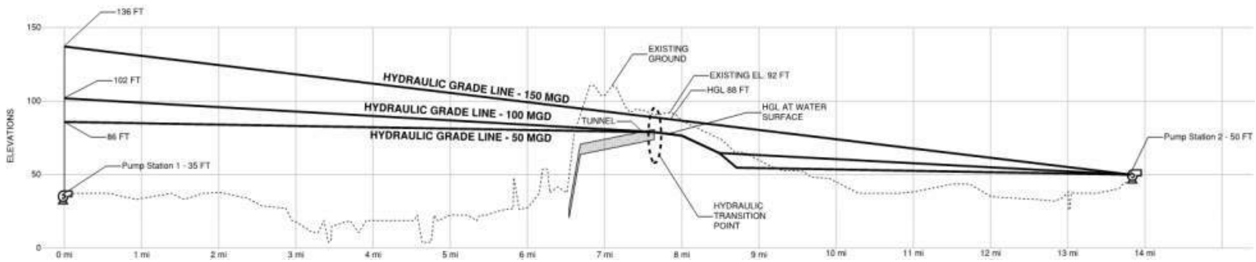


Figure 2-1 – Alternative 1 HGL

2.2 ALTERNATIVE 5 – TUNNEL BELOW HGL

2.2.1 Description

Alternative 5 includes lowering the high point of the Preferred Alignment by tunneling at a depth such that the top of pipeline is below the HGL for all operating conditions. Due to issues other than hydraulics (constructability as well as social and community issues), the Preferred Alignment already included approximately 1 mile of tunneling from the Los Angeles River to Carson Street, under the highest portion in the hill. In this alternative, the tunnel would continue at a depth below the HGL until the existing grade is low enough to accommodate open trench construction. The resulting tunnel would be approximately 3 miles in length. At the deepest point, the tunnel would have approximately 85 feet of cover.

2.2.1.1 Tunnel

Due to the depth of the proposed tunnel and to minimize the number of bends required, the 3 mile long tunnel could travel below residences and commercial buildings to find the shortest and straightest available route. This would require Metropolitan to obtain tunnel easements from a significant number of property owners. A significant risk would be placed on the project associated with these tunnel easements. Additionally, evaluations may need to be conducted documenting the existing conditions of any properties impacted to reduce project risk.

2.2.1.2 Hydraulics

Building the tunnel below the base HGL would allow the system to operate at the lowest pumping heads, allowing the system to operate at its peak efficiency (similar to Alternative 6). No pumping head would be wasted to pump over the hill in the middle of Reach 1.

2.2.1.3 Surge Control

The low static pumping head of this alternative (suction and discharge tanks at similar elevations) makes it highly unlikely to be able to provide adequate surge protection at PS1 in the form of pressurized hydropneumatic/surge tanks to prevent negative pressures along the pipeline during a PS1 trip condition. Thus, similar to Alternative 1, protection from negative pressure conditions would need to be provided by multiple ARVVs. which is not considered the preferred surge mitigation approach per Metropolitan's standard hydraulic design approach. Further complicating this situation is that such ARVVs may not be able to be installed coincident with where the negative pressure conditions occur since the pipeline will be in a tunnel. It is conceivable that hydropneumatic tanks could be installed at either or both tunnel portals to help mitigate this concern; this would require acquisition of a site or sites for these facilities. A more extreme solution would be to install the pipeline in a casing such that ARVVs could be installed coincident with areas where negative pressures are predicted. A third alternative would be to design the pipeline steel cylinder to be capable of absorbing the negative pressures. Needless to say, a more detailed analysis is required to determine a preferred approach. Note that the costs presented at the end of this memo for this Alternative 5 only account for the cost of the tunnel itself; depending on the surge mitigation solution, additional cost could occur.

2.2.1.4 Cost

Traditional tunneling allows long distances between shafts but requires an excavated diameter large enough for the man operated equipment, as well as to provide power and ventilation to the work zone. This tunnel is of significant length and diameter, allowing for conventional tunneling to be considered. Multiple methods of traditional tunneling are available, two of which are potentially applicable based on the desktop geotechnical evaluation: open shielded tunnel boring machine (TBM) and earth pressure balance tunnel boring machine (EPBM).

For the purposes of this evaluation, the traditional tunnel section identified has been assumed to be EPBM excavated with precast concrete segment initial support and steel pipe final lining. This is a conservative approach given the conceptual level of analysis and lack of geotechnical field investigations at this stage of project planning. If following a geotechnical investigation it is determined that the soils along the alignment have low permeability that could allow a shielded TBM, the tunnel cost will be lower than estimated here. Additionally if the cost of EPBM tunneling with a secondary steel lining is cost prohibitive the alignments could be excavated with microtunneling equipment with intermediate pits every 1,500 to 2,000 ft. Many of these would be extremely deep, however.

EPBM tunneling unit cost criteria is based on recent bid pricing for a similar sized EPBM tunnel project with a regional factor applied.

Assumptions:

- EPBM or Slurry would require installation of a steel liner after concrete segment installation due to internal pressure of recycled water transmission
- Costs for standard launch and retrieval pits are included in per foot price
- Dewatering for launch and retrieval pits in excess of sump pumping is not included

- Contingency not included

Table 2-2: EPBM Unit Cost Assumptions

ITEM	UNIT	ITEM	DESCRIPTION
Method		EPBM	
Length	(ft)	> 2000	
Diameter	(in)	84	
Direct Cost	($/ft)	$4,500	
General Requirement	($/ft)	$680	15% of the Direct Cost
General Contractor OH&P	($/ft)	$680	15% of the Direct Cost
Contingencies	($/ft)	N/A	
Bonds & Insurance	($/ft)	$210	3.6% (Direct Cost + General Requirements + Contractor OH&P)
Indirect Costs	($/ft)	$1,600	General Requirements + Contractor OH&P + Bonds/Insurance
Total Costs	**($/ft)**	**$6,100**	**Direct Cost + Indirect Costs**

As previously discussed, approximately 1 mile of tunneling was already included in the Preferred Alignment (and other Alternatives) due to issues other than hydraulics. Alternative 5 would propose extending the tunnel under the HGL for approximately 3 miles, or about 2 miles (10,560 feet) of additional tunneling. Therefore, the additional cost to the project is for 10,560 feet of tunneling, less the cost of in-street construction that was included for the Preferred Alignment (and other Alternatives). Per the Engineer's Opinion of Probable Construction Cost for Pipelines Associated with the RRWSP Base Case completed for Metropolitan on September 16, 2016, the total cost per foot ($/ft), including direct and indirect costs, for in-street construction is $2,315.00.

Table 2-3: Alternative 5 Cost Breakdown

ITEM	QUANTITY (FT)	UNIT COST ($/FT)	TOTAL COST ($)
Street Cost	10,560	2,315	-$24,446,000
Tunnel Cost	10,560	6,100	+$64,416,000
Difference in Cost			**+$39,970,000**

2.2.1.5 Alignment

The "Base Case" alignment between PS1 and PS2 identified by Metropolitan and Black & Veatch as part of the development of the Business Case Report presented to the Board of Directors in October of 2016 was routed through Signal Hill. The "Base Case" alignment was not selected as the Preferred Alignment during the detailed evaluation phase of the project in large part due to the length and depth of the tunnel required under Signal Hill. Since Alternative 5 also includes a long tunnel to maintain a pipe elevation below the HGL, the spreadsheet based decision model used during the detailed alternative alignment evaluation was rerun to compare the "Base Case" alignment through Signal Hill to the Preferred Alignment with an extended tunnel. The results of the new model run show that the alignment with a tunnel through Signal Hill is superior to the Preferred Alignment with the extended tunnel in Carson under one weighting scenario emphasizing construction risk (Weighting A) and inferior under the other weighting scenario emphasizing community impacts (Weighting B). Due to the lack of differentiation in the decision model results, a comparative cost has been provided later in the memo for the Signal Hill route compared to the Preferred Alignment. Table 2-4 below provides the results of the comparison using the analytical tool from the alternative alignment evaluation.

It should be noted that the spreadsheet based decision model does not factor cost into the evaluation. The tunnel required for Alternative 5 would be a mile longer than the tunnel required for the "Base Case" alignment through Signal Hill resulting in an additional cost of approximately $20 million to the project. This reinforces that an alignment through Signal Hill is preferred over the Preferred Alignment.

2.2.2 Advantages

- Lower pumping head required at PS1 compared to all other alternatives (for flows up to 140 mgd; for peak flows of 150 mgd the pumping head is the same as other alternatives)
- No additional facilities required at PS2 site (compared to current station concept).
- Maintains PS2 for positive flow control to PS3 and Orange County via dedicated pumping equipment

2.2.3 Disadvantages

- Increased construction cost due to increased tunnel depth and length
- Increased cost and risk due to tunnel easements
- Relies on ARVV's for negative pressure surge control along pipeline alignment

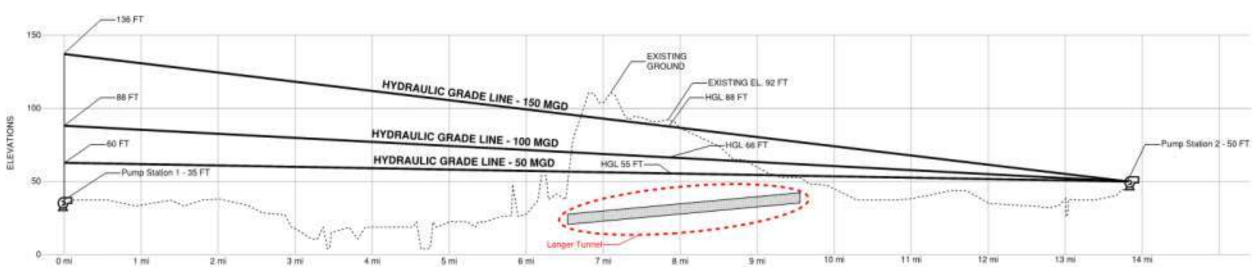


Figure 2-2 – Alternative 5 HGL

Figure 2-4: Alternative Alignment Evaluation Scoring Results

		Segments	Major Utilities	Trenchless Construction	Depth to Water	Seismic Hazard	Contaminated Soils Risk	Ease of Operations/ Accessibility	Ease of Operation Sub-Score	Length in Operational Situation	Parks	Non- SCE Parks & Rec Areas	SCE Parks & Rec Areas	Public Facilities	Length in Streets	Road Category & Traffic Impact	Center Medians	Major Intersections	Residential/ Minor Commercial	Total Alignment Length	Waters of the US and State	Critical Habitats and Listed Species	Cost	Pipe Length		Trench Length
			ea	ft	ft	Y/N	# Hits			length		length	length	ea	length		lf	ea	Length	Length	length	Y/N	$ mil	ft		ft
Alternative 5	**Start Point:**	Willow St/LA River																								
	End Point:	Carson St/Los Coyotes Diagonal																								
	Signal Hill	5	8	5489	0	Y	4	Roadway	3	5521		0	0	1	5521	5.00	3800	3	3279	11011	0	N	45.94	11011		5521
		5A	25	6950	0	Y	7	Roadway	3	19778		0	0	0	19778	3.00	8150	6	1819	26729	159	N	68.68	26729		19778
		sum	**33**	**12439**	**0**	**Y**	**11**		**3.00**	**25300**		**0**	**0**	**1**	**25300**	**2.63**	**11950**	**9**	**5098**	**37739**	**159**	**N**	**114.61**	**37739**		**25300**
		Criteria	0.87	33.0%	0%	Y	0.29		3.33		1.00			0.14		2.63	31.7%	1.26	14%	**0%**	0%	N				
		Score	1	5	1	5	3		3.33		1.00			1		2.63	3	3	3	1	1	1	1	**36**		**Raw Total**
		Weighted "A"	12.00	60.00	3.00	15.00	9.00		39.96		1.50			6.00		15.80	18.00	13.50	18.00	15.00	2.00	8.00	0.00	**236.76**	**52.65%**	**"A" Total**
		Weighted "B"	6.00	30.00	1.50	7.50	4.50		19.98		2.75			11.00		28.97	33.00	24.75	24.75	7.50	3.00	12.00	0.00	**217.19**	**55.33%**	**"B" Total**
	Pref. Align.	2(a)	18	211	0	Y	0	LAFCD	1	8065		0	0	0	0	1.00	0	0	0	8276	0	N	15.08	8276		8065
		2A	6	4774	0	Y	0	LAFCD	1	821		0	0	0	0	1.00	0	0	0	5595	0	N	30.52	5595		821
		4(c)X	5	6031	0	N	3	Roadway	3	0		0	0	0	0	3.00	0	1	1446	6031	0	N	36.79	6031		0
		8(a)X	8	4529	0	N	2	Roadway	3	2844		0	0	0	2844	5.00	3850	1	1917	7373	0	N	34.21	7373		2844
		8(b)	10	715	0	N	3	Roadway	3	9876		0	0	1	9876	5.00	8750	3	1744	10591	82	N	27.31	10591		9876
		8(c)	2	0	0	N	1	Roadway	3	2629		0	0	0	2629	5.00	2470	0	254	2629	0	N	6.09	2629		2629
		sum	**49**	**16260**	**0**	**Y**	**9**		**2.27**	**24235**		**0**	**0**	**1**	**15349**	**2.52**	**15070**	**5**	**5361**	**40495**	**82**	**N**	**150.00**	**40495**		**24235**
		Criteria	1.21	40.2%	0%	Y	0.22		2.96		1.00			0.13		2.52	37.2%	0.65	13%	**7%**	0%	N				
		Score	3	5	1	5	3		2.96		1.00			1		2.52	3	1	3	1	1	1	2	**36**		**Raw Total**
		Weighted "A"	36.00	60.00	3.00	15.00	9.00		35.55		1.50			6.00		15.10	18.00	4.50	18.00	15.00	2.00	8.00	0.00	**246.65**	**50.67%**	**"A" Total**
		Weighted "B"	18.00	30.00	1.50	7.50	4.50		17.78		2.75			11.00		27.68	33.00	8.25	24.75	7.50	3.00	12.00	0.00	**209.20**	**56.98%**	**"B" Total**
Alternative 6	**Start Point:**	Willow St/LA River																								
	End Point:	Carson St/Los Coyotes Diagonal																								
	Signal Hill	5X	8	348	0	Y	4	Roadway	3	10663		0	0	1	10663	5.00	8900	4	5690	11011	0	N	26.55	11011		10663
		5AX	25	1904	0	Y	7	Roadway	3	24825		0	0	1	24825	3.00	13535	7	2899	26729	159	N	55.05	26729		24825
		sum	**33**	**2252**	**0**	**Y**	**11**		**3.00**	**35488**		**0**	**0**	**2**	**35488**	**3.45**	**22435**	**11**	**8589**	**37740**	**159**	**N**	**81.60**	**37740**		**35488**
		Criteria	0.87	6.0%	0%	Y	0.29		3.06		1.00			0.28		3.45	59.4%	1.54	23%	**0%**	0%	N				
		Score	1	3	1	5	3		3.06		1.00			1		3.45	5	3	3	1	1	1	1	**37**		**Raw Total**
		Weighted "A"	12.00	36.00	3.00	15.00	9.00		36.72		1.50			6.00		20.67	30.00	13.50	18.00	15.00	2.00	8.00	0.00	**226.39**	**54.72%**	**"A" Total**
		Weighted "B"	6.00	18.00	1.50	7.50	4.50		18.36		2.75			11.00		37.90	55.00	24.75	24.75	7.50	3.00	12.00	0.00	**234.51**	**51.77%**	**"B" Total**
	Pref. Align.	2(a)	18	211	0	Y	0	LAFCD	1	8065		0	0	0	0	1.00	0	0	0	8276	0	N	15.08	8276		8065
		2A	6	4774	0	Y	0	LAFCD	1	821		0	0	0	0	1.00	0	0	0	5595	0	N	30.52	5595		821
		4(c)	5	258	0	N	3	Roadway	3	5773		0	0	0	5773	3.00	0	1	1446	6031	0	N	14.91	6031		5773
		8(a)	8	452	0	N	2	Roadway	3	6921		0	0	1	6921	5.00	3850	1	1917	7373	0	N	18.84	7373		6921
		8(b)	10	715	0	N	3	Roadway	3	9876		0	0	1	9876	5.00	8750	3	1744	10591	82	N	27.31	10591		9876
		8(c)	2	0	0	N	1	Roadway	3	2629		0	0	0	2629	5.00	2470	0	254	2629	0	N	6.09	2629		2629
		sum	**49**	**6410**	**0**	**Y**	**9**		**2.48**	**34085**		**0**		**2**	**25199**	**3.20**	**15070**	**5**	**5361**	**40495**	**82**	**N**	**112.74**	**40495**		**34085**
		Criteria	1.21	15.8%	0%	Y	0.22		2.72		1.00			0.26		3.20	37.2%	0.65	13%	**7%**	0%	N				
		Score	3	5	1	5	3		2.72		1.00			1		3.20	3	1	3	1	1	1	2	**37**		**Raw Total**
		Weighted "A"	36.00	60.00	3.00	15.00	9.00		32.63		1.50			6.00		19.22	18.00	4.50	18.00	15.00	2.00	8.00	0.00	**247.86**	**50.43%**	**"A" Total**
		Weighted "B"	18.00	30.00	1.50	7.50	4.50		16.32		2.75			11.00		35.24	33.00	8.25	24.75	7.50	3.00	12.00	0.00	**215.31**	**55.72%**	**"B" Total**

2.3 ALTERNATIVE 6 – ELIMINATE PS2

2.3.1 Description

Alternative 6 maintains the Preferred Alignment and uses PS1 to pump flow directly to the Orange County Spreading Basins and PS3, eliminating the need for PS2. The pumping head requirement from PS1 would significantly increase due to the additional friction loss resulting from the longer pumping distance, and because of the higher discharge elevations of the Orange County Spreading Basins and PS3, resulting in the HGL of this reach to be significantly over the high point in Carson Street.

2.3.1.1 Flow Control

To allow Metropolitan operational flexibility to adjust flow delivery to each end point, based upon the different downstream groundwater recharge needs, the project would still require one or more flow control facilities, comprising control valves and flow meters to control the splitting of flow between the two discharge locations. Flow regulation could be accomplished in one combined control facility, located at the proposed PS2 location, or it could be accomplished in a facility at any point along the alignments to at least one or both points of delivery. Since the flow control facilities could be located along the alignment to the points of delivery, there is greater flexibility in site selection.

If it was certain that Metropolitan would need to deliver flows to each end user at a consistent flow rate, it is possible to optimize such a control facility to minimize inefficiencies. However, should the flow rates vary, it would be necessary to throttle flow in one or both of the pipelines. For example, in order to reduce the water sent to Orange County while maintaining the amount of water to PS3, the control facility on the Orange County line would need to dissipate head. This throttling operation could reduce overall system efficiency depending on the extent and duration of throttling and whether any energy recovery is included.

2.3.1.2 PS1 Size

As mentioned earlier, eliminating PS2 increases the pumping head requirement at PS1. With PS2 in the project, the estimated size of the pumping equipment at PS1 is four 1,000-HP duty pumps. If PS2 were eliminated, the size of pumping equipment at PS1 would increase to an estimated four 4,500-HP duty pumps in order to pump to the terminal discharge points at PS3 and Orange County. Table 2-1 provides a comparison of pumping equipment at PS1 both with and without PS2. Essentially, the pumping power previously placed at PS2 would be relocated and incorporated into PS1. Although pumping head is increased at PS1, the overall system pumping and energy use could actually be reduced due to the associated elimination of pumping equipment at PS2 (actual overall energy use will depend on how flow control is achieved).

Table 2-5 – Changes in Pump Station Sizes Assuming the Elimination of PS2

STATION/PUMPS	WITH PS2	WITHOUT PS2
PS1/Pump Set A	15 mgd at 100 ft (2 x 350 HP duty pumps + 1 standby)	15 mgd at 100 ft (2 x 350 HP duty pumps + 1 standby)
PS1/Pump Set B	150 mgd at 100 ft (4 x 1,000 HP duty pumps + 1 standby)	150 mgd at 425 ft (4 x 4,500 HP duty pumps + 1 standby)
PS2/Pump Set A	60 mgd at 368 ft (3 x 1,750 HP duty pumps + 1 standby)	Eliminated
PS2/Pump Set B	80 mgd at 338 ft (3 x 2,500 HP duty pumps + 1 standby)	Eliminated
PS3/Pump Set A	80 mgd at 372 ft (3 x 2,500 HP duty pumps + 1 standby)	80 mgd at 372 ft (3 x 2,500 HP duty pumps + 1 standby)

2.3.1.3 Long Beach Injection Wells

The pressure in Reach 1, from PS1 to PS2, will increase by approximately 150 psi with PS2 eliminated. If injection wells are ultimately included in the project along this reach, such as those being considered in Long Beach, this additional excess pressure will need to be dissipated, reducing the overall system efficiency.

2.3.1.4 Reach 1 Operating Pressure

With PS2 in the project, the atmospheric storage tank at PS2 would limit the working pressure in the 84-inch transmission main in Reach 1 to under 50 psi. Including a 50 psi allowance for surge, the resultant required pipe wall thickness would be approximately 3/8-inch thick. It would also provide a stable and uniform hydraulic grade in Reach 1 by providing a hydraulic break. Both factors would enhance the ability to select pumps to operate efficiently over a range of desired flow rates.

If PS2 were eliminated, the pressure in Reach 1 would increase by up to an additional 150 psi, requiring an increase in pipe wall thickness to approximately ½-inch to account for the increased internal pressure. This would equate to approximately 16,000 cubic feet of steel over this reach, or about 4,060 tons. Based upon preliminary quotes received from pipe manufacturers, the 1/8" increase in pipe wall thickness would result in an additional cost of between $50 and $100 per linear foot of pipe installed.

2.3.1.5 Surge Control

Surge conditions are related to pipeline velocities, steady state operating pressures, and pipeline lengths between open reservoirs. Pipeline velocities remain unchanged regardless of the presence of PS2. With PS2 eliminated the steady state operating pressure will increase. The benefit of this increased pressure is the downsurge from a pump trip at PS1 can likely be fully mitigated with pressurized hydro-pneumatic surge tanks located at PS1 with little or no reliance on ARVV's (unlike Alternative 1 and 5). However, elimination of PS2 increases the length of pipeline between PS1 and the nearest atmospheric tank or discharge, which means the time for the surge wave to travel through the system and back is increased. The result of the increased pressure wave travel time is an associated increase in required surge tank volume. On balance, eliminating PS2 is expected to increase the surge tank volume requirements at PS1, but is also expected to improve overall system surge protection by eliminating the reliance on ARVV's, and also eliminating surge tanks at PS2.

2.3.1.6 System Control

Eliminating the storage reservoir and pumps at PS2 will require that the flow control to the PS3 storage tank and to Orange County be achieved by both the operation of the PS1 pumps as well as the flow control valves. This could be a slightly more complex control approach than having dedicated pumps to each discharge area.

Another aspect of system control that would change with the elimination of PS2 is the regular starting and stopping time for the pump stations. With longer transmission piping downstream of PS1, the optimal pump speed change rates to achieve stable operation is likely slightly longer than if PS2 were present.

2.3.1.7 PS3 Site Selection

If PS2 were eliminated, it would likely be replaced with a flow control station to provide Metropolitan the ability to control the amount of flow going to both the Orange County Spreading Basins and PS3. Although still of some size and complexity depending on the ultimate design criteria, it would likely have a much smaller footprint than PS2. Additionally and as noted above, the control facility could be located at any point along the alignments or at the points of delivery and have less stringent site criteria, allowing for greater flexibility in site selection and property acquisition. Overall, the siting challenges for a flow control station(s) are expected to be significantly reduced compared to a pump station with a large wet well or storage tank.

Additionally, with the elimination of PS2, PS3 would be located to minimize hydraulic inefficiencies between pumping from PS1 to PS3 and to the Orange County Spreading Grounds. Initial hydraulic calculations have been performed to optimize PS3's location. The optimal location is between the Whittier Narrows Dam and the San Jose Creek Water Reclamation Plant. We have identified several potentially viable sites for PS3 in this general vicinity. These sites are in the same general location as had been identified as part of the "Base Case" system. Potentially viable sites for Metropolitan consideration are depicted on Figure 2-3.

2.3.1.8 Alignment

As discussed for Alternative 5, the "Base Case" alignment was routed through Signal Hill. The "Base Case" alignment was not selected as the Preferred Alignment during the detailed evaluation phase of the project in large part due to the length and depth of the tunnel required to traverse under Signal Hill in order for the pipe to remain under the HGL. As Alternative 6 would eliminate PS2 causing the pumping head requirement from PS1 to increase and the HGL of this reach to be significantly over the high point in Signal Hill, it was warranted to revisit the spreadsheet based decision model used during the detailed alternative alignment evaluation in order to compare the "Base Case" alignment through Signal Hill to the Preferred Alignment. As with Alternative 6, the results of the new model run show that the alignment with a tunnel through Signal Hill is superior to the Preferred Alignment with the extended tunnel in Carson under one weighting scenario emphasizing construction risk (Weighting A) and inferior under the other weighting scenario emphasizing community impacts (Weighting B). Due to the lack of differentiation in the decision model results, a comparative cost has been provided later in the memo for the Signal Hill route compared to the Preferred Alignment. Table 2 4 provides the results of the comparison using the analytical tool from the alternative alignment evaluation. This affirms the initial results of the project in selecting the "Base Case" alignment.

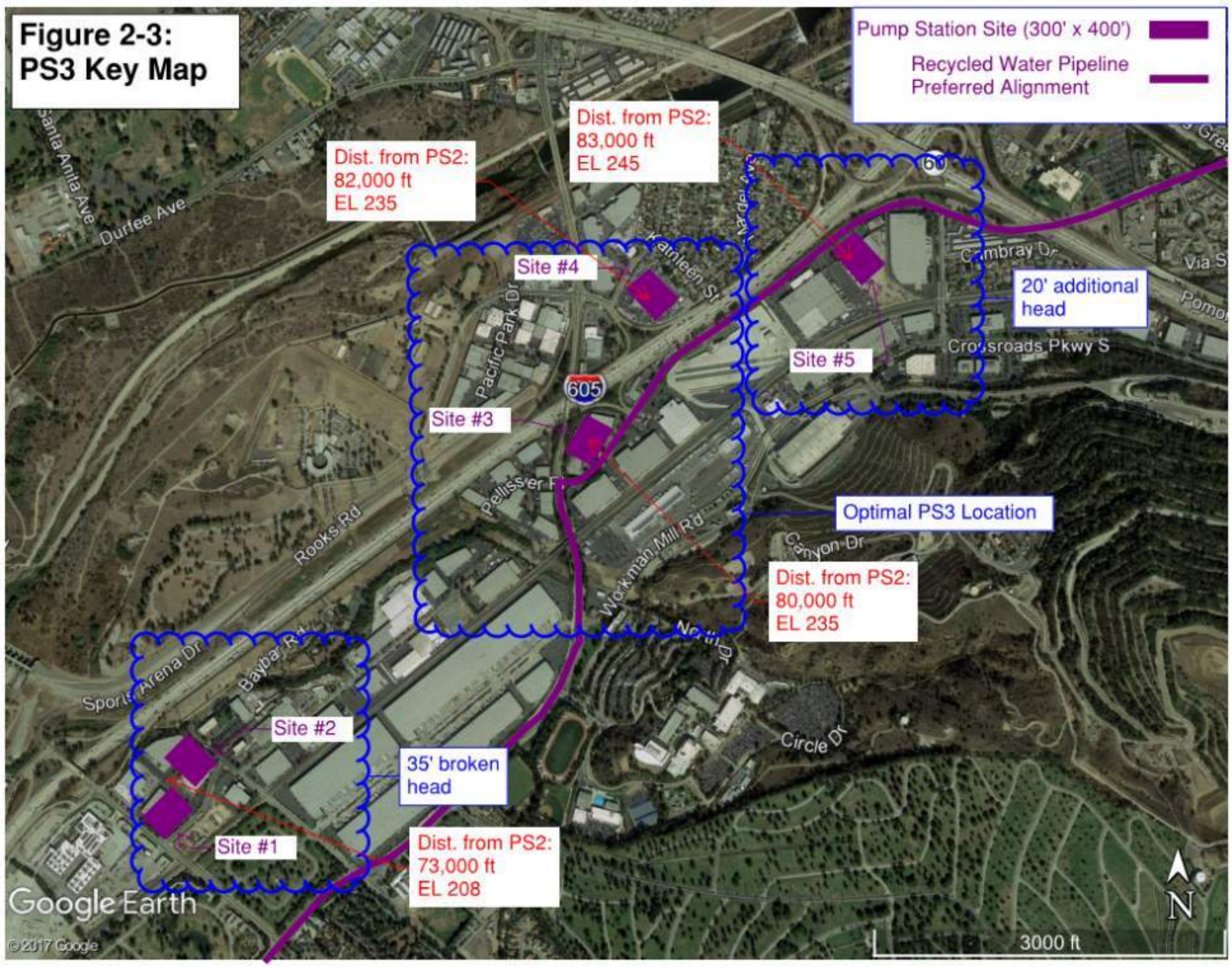


Figure 2-3 – PS3 Key Map

2.3.2 Advantages

- Eliminates PS2 capital and O&M costs
- Flexibility in locating control device structure(s)
- No pumping head is wasted
- Primary surge control can be provided at PS1 with limited reliance on ARVV's

2.3.3 Disadvantages

- Increased operational complexity
- Increased PS1 capital and O&M costs
- Potential increased piping costs to accommodate higher HGL
- Includes cost for additional control station(s)

- Possible reduction to energy efficiency if significant flow control is necessary

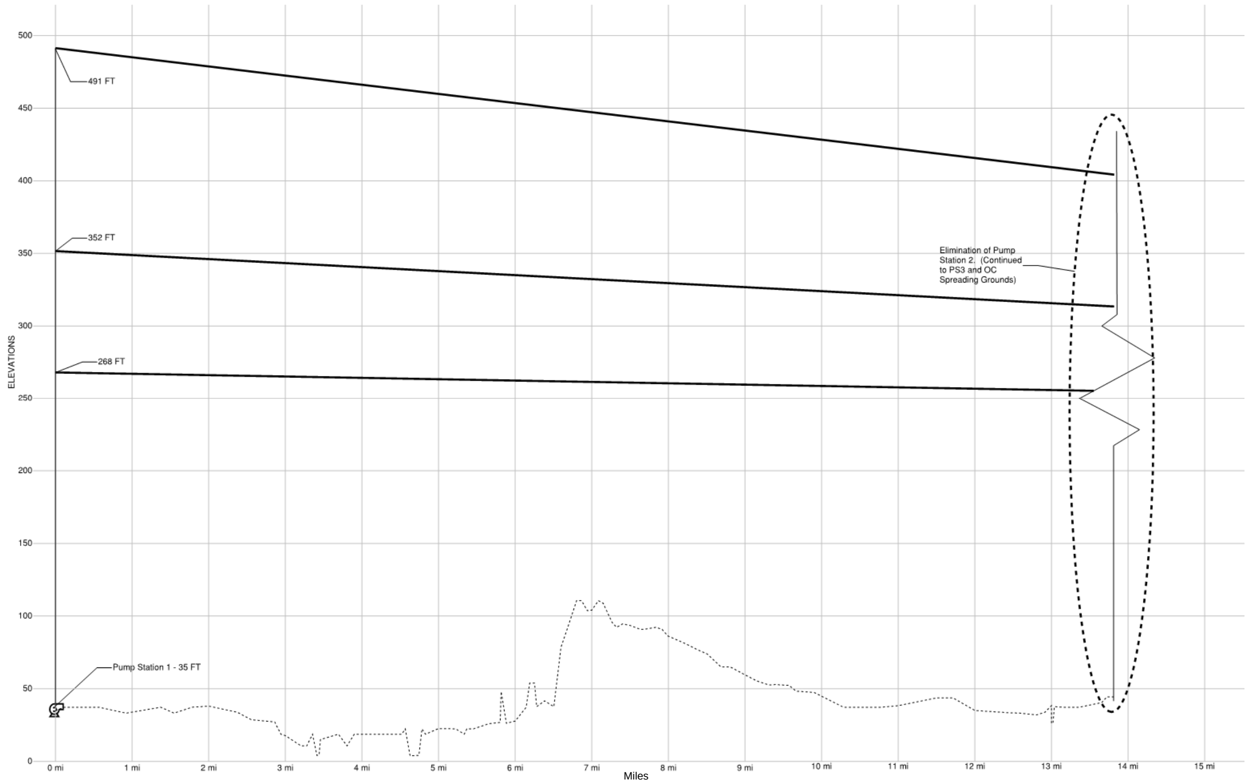


Figure 2-4 – Alternative 6 HGL

2.4 COMPARATIVE COST SUMMARY

Table 2-6 provides a comparative cost summary of the six alternatives. Only costs that change as compared to the Preferred Alignment scheme are shown. Costs are conceptual level only, but are based on engineer's opinions of probable cost provided for the Base Case project concept. Land acquisition costs are excluded from this comparison. Contingency and soft costs are also not included.

Table 2-6 – Comparative Cost Summary

	ALT 1	ALT 5 PREF. ALIGN.	ALT 5 SIGNAL HILL	ALT 6 PREF. ALIGN.	ALT 6 SIGNAL HILL
Larger Wet Well at PS2	+$6,000,000	-	-	-	-
Present Value of Additional Operational Cost[1]	+$9,000,000	-	-	-	-
Tunnel	-	+ $39,970,000	+ $22,800,000	-	- $15,700,000
Larger PS1	-	-	-	+ $5,000,000	+ $5,000,000
Eliminate PS2	-	-	-	- $34,300,000	- $34,300,000
Increased Pipe Thickness	-	-	-	+ $4,000,000	+ $4,000,000
Control Station	-	-	-	+ $10,000,000 (x2)	+ $10,000,000 (x2)
Length	-	-	- $6,400,000	-	- $6,400,000
Total	**+ $15,000,000**	**+ $39,970,000**	**+ $16,400,000**	**- $5,300,000**	**- $27,400,000**

Notes:

1) Assumes 30 year project life, energy costs for 100 mgd typical flow, 3.5% inflation/escalation rate, and a 4.0% interest/discount rate.
2) Does not include property acquisition or soft costs
3) See Appendix A for additional information on Signal Hill cost development.

2.5 RECOMMENDATION

All of the alternatives identified are viable operational strategies to address the hydraulic high point located in Reach 1.

Alternative 6 provides the most quantifiable potential benefits for the RRWP project and resolves the hydraulic high point concerns independent of alignment (Base Case or Preferred Alignment). Reverting the alignment from the Preferred Alignment back to Signal Hill provides additional cost advantages due to its shorter length and elimination of tunnels needed in other alternatives. The Signal Hill alignment also provides additional advantages that will be discussed in the next section of this memo.

Alternative 6 with the alignment through Signal Hill is therefore the recommended alternative.

3 Signal Hill Alternative – LADWP Option

3.1 BACKGROUND

The evaluation provided earlier in this memo established that Alternative 6, with the alignment through Signal Hill,provided the most quantifiable potential benefits to the RRWP project. In that concept, PS2 was eliminated.

Recently, Metropolitan has had discussions with Los Angeles Department of Water & Power (LADWP) about the potential for LADWP to become a customer of the proposed recycled water system to supply its South Bay recycled water customers. The exact quantity of supply is still being evaluated, but early discussions suggest it could be in the range of 15 to 20 MGD. Given the location of LADWP's existing recycled water pipeline infrastructure in this area, a likely point of connection would be in the vicinity of Signal Hill. This potential is another reason the Signal Hill alignment is desirable.

In the recommended Alternative 6 concept, PS1 would pump to a hydraulic grade that eliminates PS2 and delivers flows all the way to Orange County Spreading Grounds and PS3. The resulting hydraulic grade in the vicinity of the potential service connection to LADWP would be excessive, however. Several potential alternatives could be considered to address this concern:

a) Provide a service connection facility that is capable of reducing the pressure from the Metropolitan system HGL to the LADWP HGL. While a plausible approach, this is clearly energy inefficient.

b) Provide a separate, dedicated set of pumps at PS1 that only serve LADWP at its hydraulic grade, and provide a dedicated pipeline to connect from PS1 to LADWP's existing distribution system. A detailed evaluation of this approach is beyond the scope of this memo; it requires additional infrastructure to implement.

c) Provide a storage tank on Signal Hill which can serve LADWP closer to its hydraulic grade requirements as well as accommodate diurnal demands. This option would require reinstating PS2 to the recommended Alternative 6, but has several other offsetting benefits. This option is described herein.

3.2 LADWP OPTION

3.2.1 Storage Tank

In this concept, a storage tank would be located nearest to the highest elevations along the Signal Hill alignment in East Willow Street. Several potentially viable sites have been identified for this tank as shown in Figure 3-1. These sites were selected based on their proximity to the Signal Hill alignment, site access, and land use/potential availability. Property ownership was not evaluated during the identification of these sites. The site selection assumed 2.0 MG for the tank volume and 20 feet side water depth. This results in a tank diameter of approximately 135 feet. Sites #2, #5, and #6 are potentially not large enough to feature a single above grade circular tank. However, other tank configurations are possible at these locations.

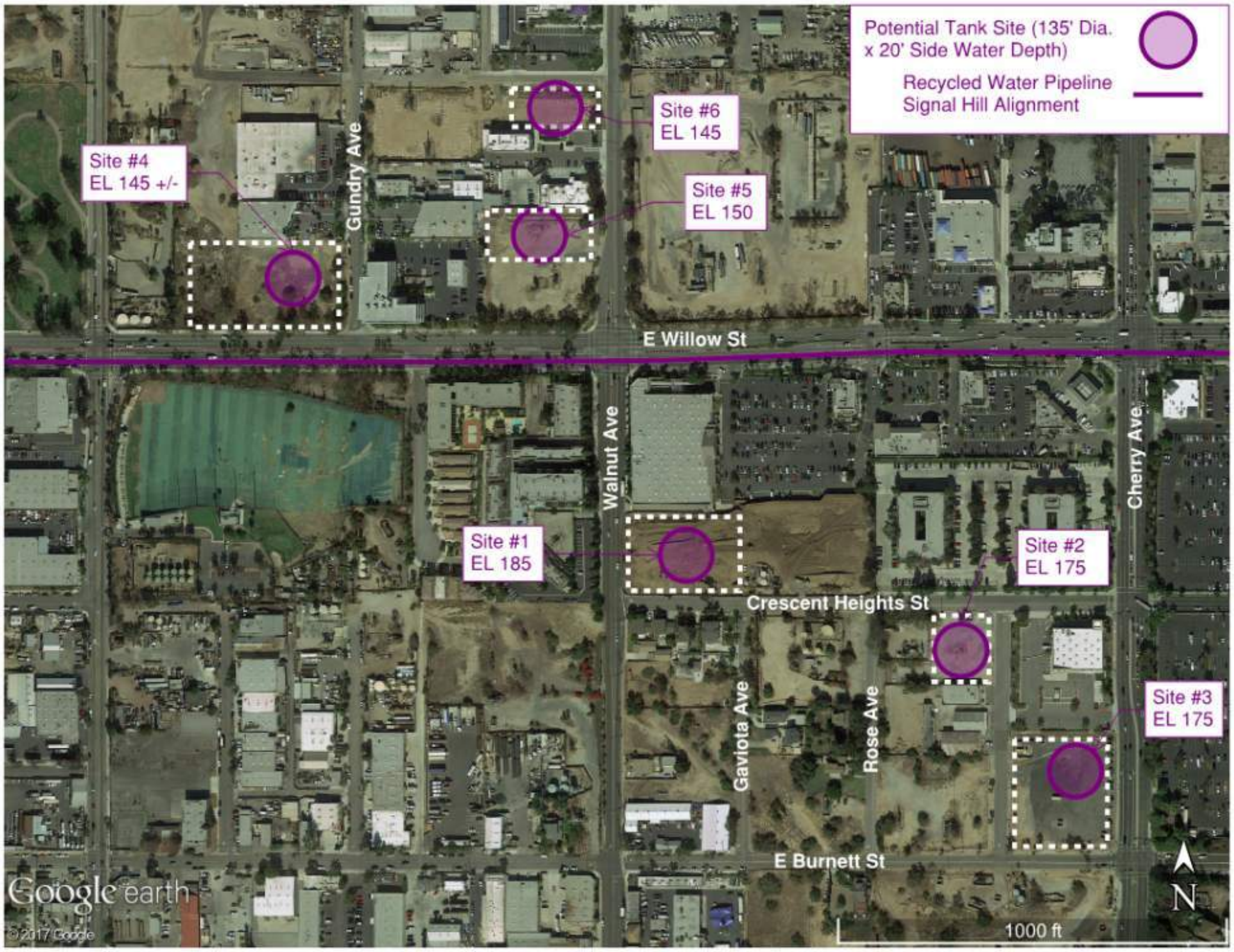


Figure 3-1: Signal Hill Tank Location Map

3.2.2 Hydraulics/Pump Station 2

In this configuration, PS1 would not pump to Orange County Spreading Grounds and PS3. Instead, PS1 would be designed to pump to the Signal Hill Storage Tank (SHST). To complete the system, it would therefore be necessary to include PS2 in this scheme. PS2 would be fed by gravity from the SHST. This provides several benefits:

- SHST will effectively serve as the wet well for the pumps at PS2. Therefore the cost (construction and O&M) and space consumption of the wet well storage volume at PS2 would be eliminated.
- Likewise, the pumps at PS2 could be installed as in-line can pumps.
- Reinstating PS2 eliminates the need for additional flow control facilities needed in Alternative 6 as PS2 will serve that function.
- Section 2 of this memo discussed hydraulic and surge control issues for all the alternatives. By introducing the SHST, hydraulic system control would remain as originally conceived,

but surge mitigation would be improved. The hydraulic break provided by SHST would in smaller surge tanks at PS1, sized for a lower maximum surge pressure than in the original Base Case and Preferred Alignment configurations. Surge control at PS2 would remain as originally conceived.

3.2.3 Storage Tank Size

As previously envisioned, the storage volume of the wet well at PS2 would be 2.0 MG. This storage volume is sized to provide operational control, allow coordinated and synchronized controls between stations to limit imbalances, and to minimize risk if a pump station operationally . Additionally, it is sized to provide limited surge control benefits. By moving the wet well at PS2 to the high point of Signal Hill, the size of the storage tank could conceivably remain the same. However, if LADWP or other project customers have diurnal flow demands, then the size of the storage tank would need to be reevaluated and could potentially get larger. Additional evaluations to determine the storage volume size should be completed once agreements with potential customers have been reached and the diurnal curves of their demands have been obtained.

As noted above, the SHST sites that have preliminarily been identified were reviewed to accommodate 2.0 MG of storage. By inspection, Sites #1, #3, and #4 have additional space to accommodate larger volumes. Different tank types of configurations could be used to accommodate larger volumes at all of the sites.

3.2.4 Cost Analysis

The table below provides a comparison of the cost of Alternative 6 Signal Hill Alternative (higher HGL PS1, PS2 eliminated) and the Alternative 6 LADWP Option.

Table 3-1 – Comparative Cost Summary

	ALT 6 SIGNAL HILL	ALT 6 LADWP OPTION
Larger PS1	+ $5,000,000	0
Eliminate PS2	- $34,300,000	0
Increased Pipe Thickness	+ $4,000,000	0
Control Station	+ $10,000,000 (x2)	0
Storage Tank	0	+4,000,000
Present Value of Additional Operational Costs (Energy)		+3,500,000
Total	**- $5,300,000**	**+7,500,000**

Notes:

1) Assumes 30 year project life, energy costs for 100 mgd typical flow, 3.5% inflation/escalation rate, and a 4.0% interest/discount rate.
2) Does not include property acquisition or soft costs

Appendix A - Signal Hill Costs

ALTERNATIVE 5 – SIGNAL HILL COSTS

As previously discussed, approximately 1 mile of tunneling was already included in the Preferred Alignment due to issues other than hydraulics. Alternative 5 - Signal Hill would propose extending the tunnel under the HGL for approximately 6,029 feet of additional tunneling as compared to the Preferred Alignment. Therefore, the additional cost to the project is for 6,029 feet of tunneling, less the cost of in-street construction that was included for the Preferred Alignment (and other Alternatives).

Table A-1: Alternative 5 Signal Hill Tunnel Cost Breakdown

ITEM	QUANTITY (FT)	UNIT COST ($/FT)	TOTAL COST ($)
Street Cost	6,029	2,315	-$14,000,000
Tunnel Cost	6,029	6,100	+$36,800,000
Difference in Cost			**+$22,800,000**

Alternative 5 - Signal Hill is also shorter than the Preferred Alignment by approximately 2,756 feet. Therefore, the benefit to the project is for 2,756 feet of in-street construction cost that was included for the Preferred Alignment (and other Alternatives).

Table A-2: Alternative 5 Signal Hill Alignment Length Cost Breakdown

ITEM	QUANTITY (FT)	UNIT COST ($/FT)	TOTAL COST ($)
Street Cost	2,756	2,315	-$6,400,000
Difference in Cost			**-$6,400,000**

ALTERNATIVE 6 – SIGNAL HILL COSTS

As previously discussed, approximately 1 mile of tunneling was already included in the Preferred Alignment due to issues other than hydraulics. Alternative 6 - Signal Hill would eliminate approximately 4,158 feet of tunneling as compared to the Preferred Alignment. Therefore, the benefit to the project is for 4,158 feet of tunneling, less the cost of in-street construction that was included for the Preferred Alignment.

Table A-3: Alternative 6 Signal Hill Tunnel Cost Breakdown

ITEM	QUANTITY (FT)	UNIT COST ($/FT)	TOTAL COST ($)
Street Cost	4,158	2,315	+$9,600,000
Tunnel Cost	4,158	6,100	-$25,400,000
Difference in Cost			**-$15,700,000**

As with Alternative 5 - Signal Hill, Alternative 6 - Signal Hill is also shorter than the Preferred Alignment by approximately 2,756 feet. Therefore, the benefit to the project is for 2,756 feet of in-street construction cost that was included for the Preferred Alignment.

Table A-4: Alternative 6 Signal Hill Alignment Length Cost Breakdown

ITEM	QUANTITY (FT)	UNIT COST ($/FT)	TOTAL COST ($)
Street Cost	2,756	2,315	-$6,400,000
Difference in Cost			**-$6,400,000**

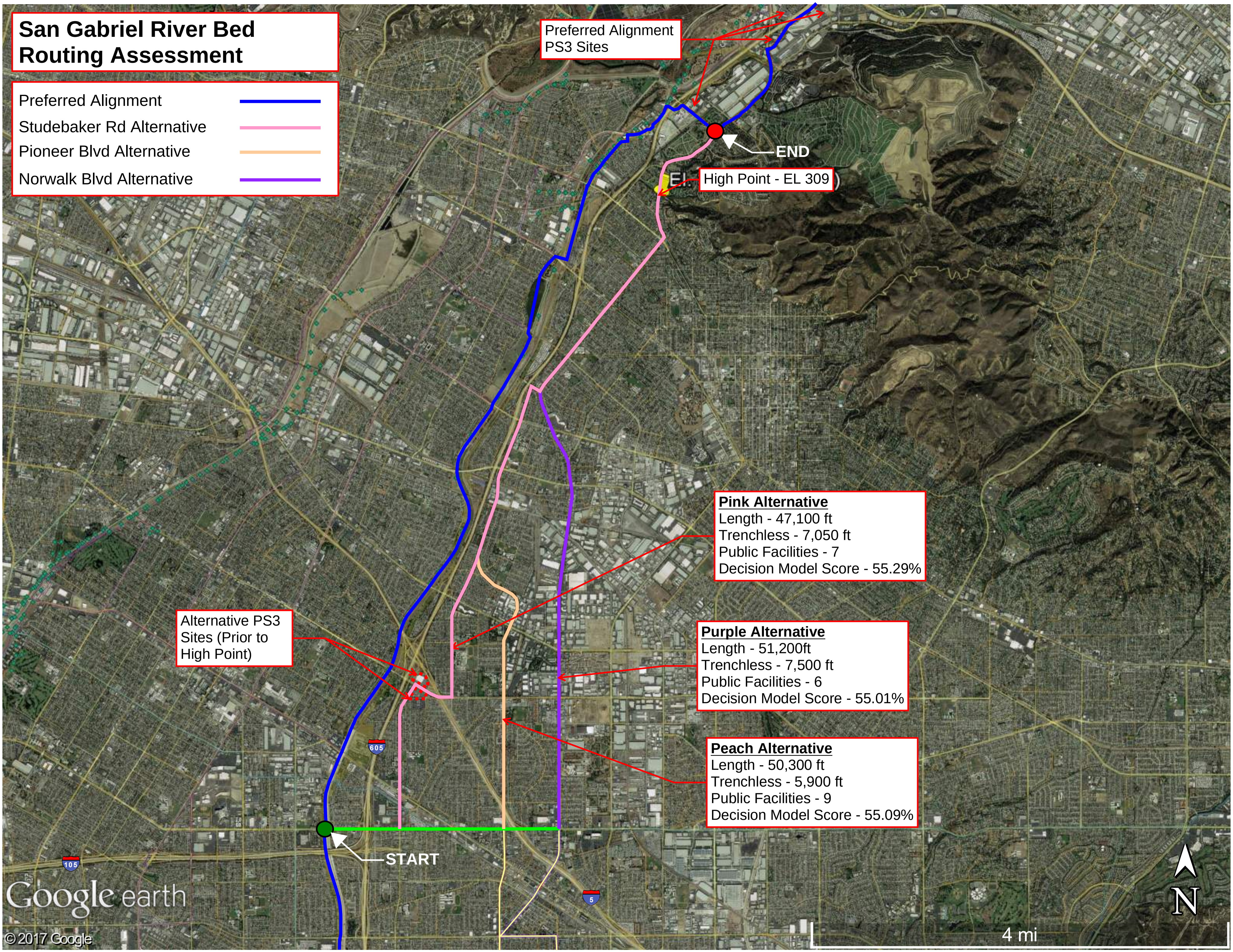
Appendix R. Alignment Verification Analysis

SAN GABRIEL RIVER ALTERNATIVES

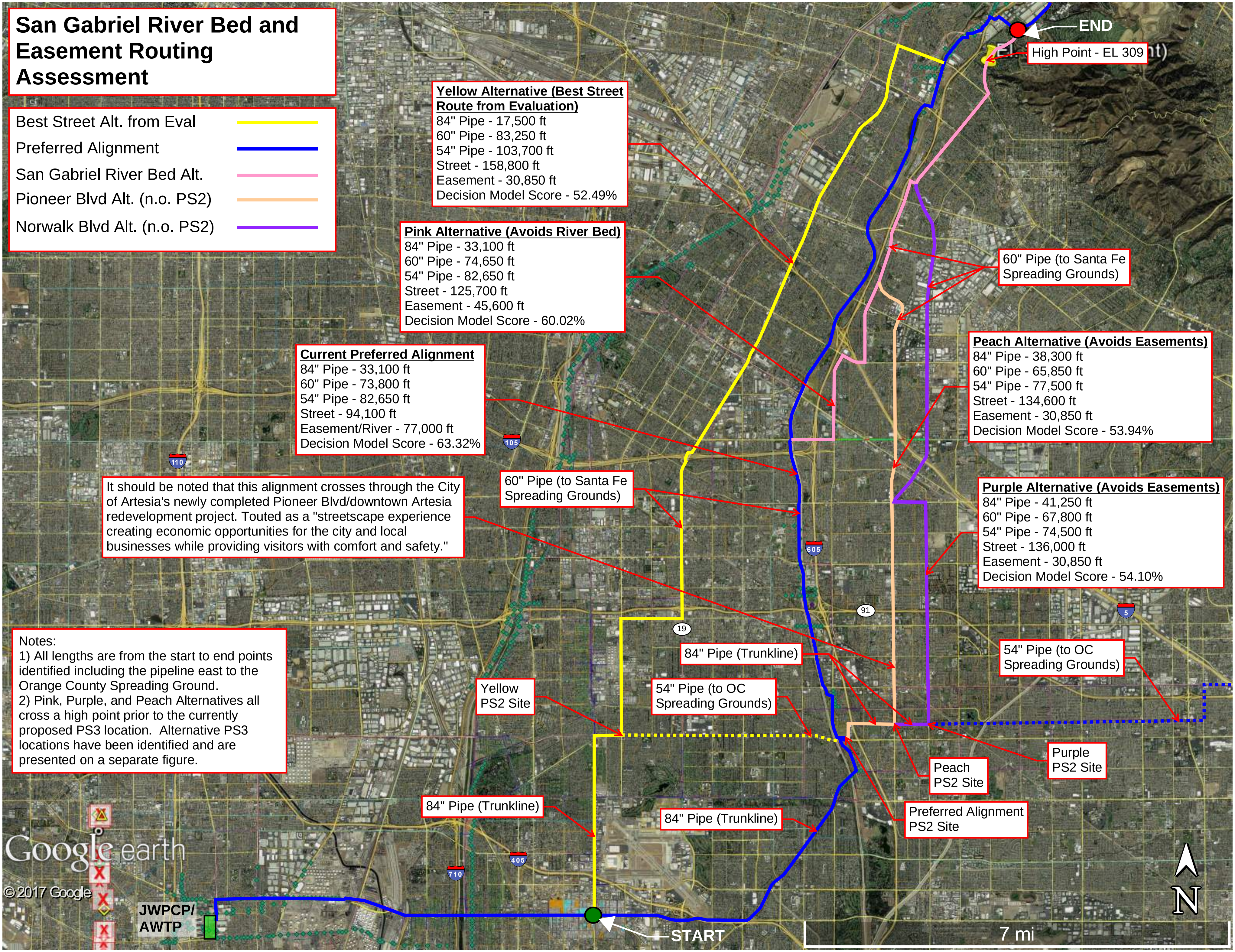
As discussed in Chapter 5, the Initial Preferred Alignment proposed constructing pipe in the San Gabriel River bed from approximately Imperial Highway to Whittier Boulevard. However, constructing pipe in the San Gabriel River bed would introduce risk to the Project schedule and budget due to potential permitting issues and the additional interagency coordination required. Metropolitan's staff asked Black & Veatch to identify alternatives to constructing in the San Gabriel River bed as a backup plan should constructing pipe in the river bed prove to be unfeasible.

Together, Black & Veatch and Metropolitan staff identified multiple routes that utilize public rights-of-way in city streets to avoid the San Gabriel River bed. The spreadsheet-based decision model used during the detailed alternative alignment evaluation was rerun to compare the different alternatives to the San Gabriel River bed. The Initial Preferred Alignment, utilizing the San Gabriel River bed, remained the favored alternative through the additional analysis. However, should an alternative route be needed, several other viable routes were identified. The results of the analysis were presented to Metropolitan staff at a workshop on August 31, 2017, and it was agreed that no changes to the Initial Preferred Alignment were required.

The following Figures present the alternative routes identified. The first figure identifies routes that exclusively avoid the portion of the Preferred Alignment within the San Gabriel River bed. The second figure presents alternatives beginning at PS-2 and extending past the portion of the Preferred Alignment within the San Gabriel River bed. The decision model results are subsequently presented.



San Gabriel River Bed
Routing Assessment
Preferred Alignment
Studebaker Rd Alternative
Pioneer Blvd Alternative
Norwalk Blvd Alternative
Preferred Alignment PS3 Sites
END
High Point - EL 309
Pink Alternative
Length - 47,100 ft
Trenchless - 7,050 ft
Public Facilities - 7
Decision Model Score - 55.29%
Alternative PS3 Sites (Prior to High Point)
Purple Alternative
Length - 51,200ft
Trenchless - 7,500 ft
Public Facilities - 6
Decision Model Score - 55.01%
Peach Alternative
Length - 50,300 ft
Trenchless - 5,900 ft
Public Facilities - 9
Decision Model Score - 55.09%
START
605
105
5
Google earth
© 2017 Google
N
4 mi



San Gabriel River Bed and Easement Routing Assessment
Best Street Alt. from Eval
Preferred Alignment
San Gabriel River Bed Alt.
Pioneer Blvd Alt. (n.o. PS2)
Norwalk Blvd Alt. (n.o. PS2)
Yellow Alternative (Best Street Route from Evaluation)
84" Pipe - 17,500 ft
60" Pipe - 83,250 ft
54" Pipe - 103,700 ft
Street - 158,800 ft
Easement - 30,850 ft
Decision Model Score - 52.49%
Pink Alternative (Avoids River Bed)
84" Pipe - 33,100 ft
60" Pipe - 74,650 ft
54" Pipe - 82,650 ft
Street - 125,700 ft
Easement - 45,600 ft
Decision Model Score - 60.02%
Current Preferred Alignment
84" Pipe - 33,100 ft
60" Pipe - 73,800 ft
54" Pipe - 82,650 ft
Street - 94,100 ft
Easement/River - 77,000 ft
Decision Model Score - 63.32%
Peach Alternative (Avoids Easements)
84" Pipe - 38,300 ft
60" Pipe - 65,850 ft
54" Pipe - 77,500 ft
Street - 134,600 ft
Easement - 30,850 ft
Decision Model Score - 53.94%
Purple Alternative (Avoids Easements)
84" Pipe - 41,250 ft
60" Pipe - 67,800 ft
54" Pipe - 74,500 ft
Street - 136,000 ft
Easement - 30,850 ft
Decision Model Score - 54.10%
END
High Point - EL 309
60" Pipe (to Santa Fe Spreading Grounds)
60" Pipe (to Santa Fe Spreading Grounds)
It should be noted that this alignment crosses through the City of Artesia's newly completed Pioneer Blvd/downtown Artesia redevelopment project. Touted as a "streetscape experience creating economic opportunities for the city and local businesses while providing visitors with comfort and safety."
54" Pipe (to OC Spreading Grounds)
84" Pipe (Trunkline)
54" Pipe (to OC Spreading Grounds)
Yellow PS2 Site
Peach PS2 Site
Purple PS2 Site
Preferred Alignment PS2 Site
84" Pipe (Trunkline)
84" Pipe (Trunkline)
Notes:
1) All lengths are from the start to end points identified including the pipeline east to the Orange County Spreading Ground.
2) Pink, Purple, and Peach Alternatives all cross a high point prior to the currently proposed PS3 location. Alternative PS3 locations have been identified and are presented on a separate figure.
JWPCP/ AWTP
START
7 mi
N
Google earth
© 2017 Google

Evaluation Criteria Scoring Table - Orange County Paths

	Segments	Major Utilities	Trenchless Construction	Depth to Water	Seismic Hazard	Contaminated Soils Risk	Ease of Operations/ Accessibility	Ease of Operation Sub-Score	Length in Operational Situation	Parks	Non- SCE Parks & Rec Areas	SCE Parks & Rec Areas	Public Facilities	Length in Streets	Road Category & Traffic Impact	Center Medians	Major Intersections	Residential/ Minor Commercial	Total Alignment Length	Waters of the US and State	Critical Habitats and Listed Species	Cost	Pipe Length		Trench Length
		ea	ft	ft	Y/N	# Hits			length		length	length	ea	length		lf	ea	Length	Length	length	Y/N	$ mil	ft		ft
Start Point:																									
End Point:																									
Pink	GreenLeft	0	1647	0	N	0	Roadway	3	2081		0	0	1	2081	5.00	2020	1	765	3728	0	N		3728		2081
	Pink1	0	2912	0	N	3	Roadway	3	14031		0	0	3	14031	3.00	8275	4	2150	16943	0	N		16943		14031
	Pink2	0	1438	0	N	1	Roadway	3	8059		0	0	1	8059	3.00	2255	2	1275	9497	0	N		9497		8059
	Pink3	0	1055	0	N	4	Roadway	3	15884		0	0	2	15884	3.00	5730	3	2300	16939	0	N		16939		15884
	sum	**0**	**7052**	**0**	**N**	**8**		**3.00**	**40055**		**0**	**0**	**7**	**40055**	**2.79**	**18280**	**10**	**6490**	**47107**	**0**	**N**		**47107**		**40055**
	Criteria	0.00	15.0%	0%	N	0.17		3.15		1.00			0.78		2.79	38.8%	1.12	14%	**0%**	0%	N				
	Score	1	3	1	1	3		3.15		1.00			5		2.79	3	3	3	1	1	1		**33**		**Raw Total**
	Weighted "A"	12.00	36.00	3.00	3.00	9.00		37.80		1.50			30.00		16.73	18.00	13.50	18.00	15.00	2.00	8.00		**223.53**	**55.29%**	**"A" Total**
	Weighted "B"	6.00	18.00	1.50	1.50	4.50		18.90		2.75			55.00		30.68	33.00	24.75	24.75	7.50	3.00	12.00		**243.83**	**49.86%**	**"B" Total**
Peach	GreenLeft	0	1647	0	N	0	Roadway	3	2081		0	0	1	2081	5.00	2020	1	765	3728	0	N		3728		2081
	GreenCenter	0	774	0	N	3	Roadway	3	4501		0	0	0	4501	5.00	1115	2	1850	5275	0	N		5275		4501
	PeachNorth	0	977	0	N	2	Roadway	3	13901		0	0	5	13901	3.00	8235	2	700	14878	0	N		14878		13901
	Pink2	0	1438	0	N	1	Roadway	3	8059		0	0	1	8059	3.00	2255	2	1275	9497	0	N		9497		8059
	Pink3	0	1055	0	N	4	Roadway	3	15884		0	0	2	15884	3.00	5730	3	2300	16939	0	N		16939		15884
	sum	**0**	**5891**	**0**	**N**	**10**		**3.00**	**44426**		**0**	**0**	**9**	**44426**	**3.03**	**19355**	**10**	**6890**	**50317**	**0**	**N**		**50317**		**44426**
	Criteria	0.00	11.7%	0%	N	0.20		3.12		1.00			0.94		3.03	38.5%	1.05	14%	**7%**	0%	N				
	Score	1	3	1	1	3		3.12		1.00			5		3.03	3	3	3	1	1	1		**33**		**Raw Total**
	Weighted "A"	12.00	36.00	3.00	3.00	9.00		37.40		1.50			30.00		18.16	18.00	13.50	18.00	15.00	2.00	8.00		**224.57**	**55.09%**	**"A" Total**
	Weighted "B"	6.00	18.00	1.50	1.50	4.50		18.70		2.75			55.00		33.30	33.00	24.75	24.75	7.50	3.00	12.00		**246.25**	**49.36%**	**"B" Total**
Purple	GreenLeft	0	1647	0	N	0	Roadway	3	2081		0	0	1	2081	5.00	2020	1	765	3728	0	N		3728		2081
	GreenCenter	0	774	0	N	3	Roadway	3	4501		0	0	0	4501	5.00	1115	2	1850	5275	0	N		5275		4501
	GreenRight	0	1049	0	N	3	Roadway	3	1749		0	0	0	1749	5.00	1651	2	980	2798	0	N		2798		1749
	PurpleNorth	0	2979	0	N	6	Roadway	3	19482		0	0	3	19482	3.00	4750	6	4150	22461	0	N		22461		19482
	Pink3	0	1055	0	N	4	Roadway	3	15884		0	0	2	15884	3.00	5730	3	2300	16939	0	N		16939		15884
	sum	**0**	**7504**	**0**	**N**	**16**		**3.00**	**43697**		**0**	**0**	**6**	**43697**	**3.03**	**15266**	**14**	**10045**	**51201**	**0**	**N**		**51201**		**43697**
	Criteria	0.00	14.7%	0%	N	0.31		3.15		1.00			0.62		3.03	29.8%	1.44	20%	**9%**	0%	N				
	Score	1	3	1	1	3		3.15		1.00			5		3.03	3	3	3	1	1	1		**33**		**Raw Total**
	Weighted "A"	12.00	36.00	3.00	3.00	9.00		37.76		1.50			30.00		18.19	18.00	13.50	18.00	15.00	2.00	8.00		**224.95**	**55.01%**	**"A" Total**
	Weighted "B"	6.00	18.00	1.50	1.50	4.50		18.88		2.75			55.00		33.36	33.00	24.75	24.75	7.50	3.00	12.00		**246.48**	**49.31%**	**"B" Total**
Start Point:																									
End Point:																									
Peach	5AX	25	1904	0	Y	7	Roadway	3	24825		0	0	1	24825	3.00	13535	7	2899	26729	158.59271	N		26729		24825
	10A.1(a)	3	674	0	N	0	Roadway	3	1855		0	0	0	1855	3.00	0	0	177	2529	0	N		2529		1855
	10A.1(b)	2	359	0	N	0	Roadway	3	2625		0	0	0	2625	3.00	0	0	0	2984	0	N		2984		2625
	10A.2	0	83	0	N	0	Private	1	384		0	0	0	0	1.00	0	1	0	468	0	N		468		384
	10A.3	1	0	0	N	0	Roadway	3	435		0	0	0	435	3.00	375	0	0	435	0	N		435		435
	10A.4	0	0	0	N	0	SCE	1	911		0	0	0	0	1.00	0	0	0	911	0	N		911		911
	10A.5	0	0	0	N	0	SCE	1	271		0	0	0	0	1.00	0	0	0	271	0	N		271		271
	11.1(a)Peacl	0	370	3954	N	0	SCE	1	3584		0	2500	1	0	1.00	0	0	0	3954	0	N		3954		3584
	PeachSouth	0	3769	0	N	16	Roadway	3	20779		0	0	5	20779	3.00	12308	8	10208	24548	0	N		24548		20779
	PeachNorth	0	977	0	N	2	Roadway	3	13901		0	0	5	13901	3.00	8235	2	700	14878	0	N		14878		13901
	Pink2	0	1438	0	N	1	Roadway	3	8059		0	0	1	8059	3.00	2255	2	1275	9497	0	N		9497		8059
	Pink3	0	1055	0	N	4	Roadway	3	15884		0	0	2	15884	3.00	5730	3	2300	16939	0	N		16939		15884
	sum	**31**	**10630**		**Y**	**30**		**2.89**	**93512**		**0**	**2500**	**15**	**88362**	**2.70**	**42438**	**23**	**17559**	**104142**	**159**	**N**		**104141.67**		**93512.17**
	Criteria	0.30	10.2%		Y	0.29		3.00		1.05			0.76		2.70	40.8%	1.17	17%	**3%**	0%	N				
	Score	1	3		5	3		3.00		1.05			5		2.70	3	3	3	1	1	1		**36**		**Raw Total**
	Weighted "A"	12.00	36.00		15.00	9.00		36.04		1.57			30.00		16.18	18.00	13.50	18.00	15.00	2.00	8.00		**230.29**	**53.94%**	**"A" Total**
	Weighted "B"	6.00	18.00		7.50	4.50		18.02		2.88			55.00		29.67	33.00	24.75	24.75	7.50	3.00	12.00		**246.57**	**49.29%**	**"B" Total**
Purple	5AX	25	1904	0	Y	7	Roadway	3	24825		0	0	1	24825	3.00	13535	7	2899	26729	158.59271	N		26729		24825
	10A.1(a)	3	674	0	N	0	Roadway	3	1855		0	0	0	1855	3.00	0	0	177	2529	0	N		2529		1855
	10A.1(b)	2	359	0	N	0	Roadway	3	2625		0	0	0	2625	3.00	0	0	0	2984	0	N		2984		2625
	10A.2	0	83	0	N	0	Private	1	384		0	0	0	0	1.00	0	1	0	468	0	N		468		384
	10A.3	1	0	0	N	0	Roadway	3	435		0	0	0	435	3.00	375	0	0	435	0	N		435		435
	10A.4	0	0	0	N	0	SCE	1	911		0	0	0	0	1.00	0	0	0	911	0	N		911		911
	10A.5	0	0	0	N	0	SCE	1	271		0	0	0	0	1.00	0	0	0	271	0	N		271		271
	11.1(a)Purpl	0	570	6943	N	0	SCE	1	6373		0	3851	1	0	1.00	0	0	0	6943	0	N		6943		6373
	PurpleSouth	0	3742	0	N	12	Roadway	3	24644		0	0	7	24644	3.00	8435	8	11420	28386	0	N		28386		24644
	PurpleNorth	0	2979	0	N	6	Roadway	3	19482		0	0	3	19482	3.00	4750	6	4150	22461	0	N		22461		19482
	Pink3	0	1055	0	N	4	Roadway	3	15884		0	0	2	15884	3.00	5730	3	2300	16939	0	N		16939		15884
	sum	**31**	**11366.5**		**Y**	**29**		**2.84**	**97688**		**0**	**3851**	**14**	**89748.9**	**2.65**	**32825**	**25**	**20946**	**109054**	**159**	**N**		**109054**		**97688**
	Criteria	0.28	10.4%		Y	0.27		2.96		1.07			0.68		2.65	30.1%	1.21	19%	**8%**	0%	N				
	Score	1	3		5	3		2.96		1.07			5		2.65	3	3	3	1	1	1		**36**		**Raw Total**
	Weighted "A"	12.00	36.00		15.00	9.00		35.50		1.61			30.00		15.88	18.00	13.50	18.00	15.00	2.00	8.00		**229.49**	**54.10%**	**"A" Total**
	Weighted "B"	6.00	18.00		7.50	4.50		17.75		2.94			55.00		29.11	33.00	24.75	24.75	7.50	3.00	12.00		**245.80**	**49.45%**	**"B" Total**

Blue	5AX	25	1904	0	Y	7	Roadway	3	24825		0	0	1	24825	3.00	13535	7	2899	26729	158.59271	N	26729		24825
	10A.1(a)	3	674	0	N	0	Roadway	3	1855		0	0	0	1855	3.00	0	0	177	2529	0	N	2529		1855
	10A.1(b)	2	359	0	N	0	Roadway	3	2625		0	0	0	2625	3.00	0	0	0	2984	0	N	2984		2625
	10A.2	0	83	0	N	0	Private	1	384		0	0	0	0	1.00	0	1	0	468	0	N	468		384
	10A.3	1	0	0	N	0	Roadway	3	435		0	0	0	435	3.00	375	0	0	435	0	N	435		435
	20.1(a)	3	0	0	N	0	Roadway	3	3572		0	0	0	3572	3.00	2500	0	0	3572	0	N	3572		3572
	20.1(b)	0	0	0	N	0	Roadway	3	1125		0	0	0	1125	3.00	1050	0	0	1125	0	N	1125		1125
	20.2	1	191	0	N	0	SCE	1	1001		395	606	0	0	1.00	0	0	0	1192	0	N	1192		1001
	20.3	1	0	0	N	0	Roadway	3	636		0	0	1	636	1.00	0	0	0	636	0	N	636		636
	20.4	0	0	0	N	0	SCE	1	1199		0	0	1	0	1.00	0	0	0	1199	0	N	1199		1199
	20.5	1	169	0	N	0	Roadway	3	766		0	0	1	766	1.00	0	0	0	934	0	N	934		766
	20.6	0	0	0	N	0	Private	1	2180		0	0	0	0	1.00	0	0	0	2180	0	N	2180		2180
	20.7	2	1133	0	N	0	SCE	1	922		0	0	0	0	1.00	0	0	0	2055	0	N	2055		922
	20.8	5	0	0	N	0	SCE	1	1781		0	1780.8	0	0	1.00	0	0	0	1781	2.648382	N	1781		1781
	20.9	7	1211	2402	N	0	SCE	1	1192		0	0	0	0	1.00	0	0	0	2402	280.705414	N	2402		1192
	20.15	2	261	5118	N	0	SCE	1	4858		0	3040	0	0	1.00	0	0	0	5118	0	N	5118		4858
	20.11	0	0	2752	N	0	Roadway	3	2752		0	0	0	2752	1.00	0	0	2687	2752	0	N	2752		2752
	20.12	1	422	1741	N	0	LAFCD	1	1318		0	0	0	0	1.00	0	0	766	1741	0	N	1741		1318
	20.13	0	0	867	N	0	Roadway	3	867		0	0	0	867	1.00	0	0	0	867	0	N	867		867
	20.14	1	164	4211	N	0	River	5	4047		0	0	0	0	1.00	0	0	0	4211	1160.096708	N	4211		4047
	22.1	0	0	1219	N	0	River	5	1219		0	0	0	0	1.00	0	0	0	1219	1219.195151	N	1219		1219
	22.2	5	1001	18750	N	0	River	5	17749		0	0	0	0	1.00	0	0	0	18750	18004.97321	N	18750		17749
	36	0	0	4265	N	0	LAFCD	1	4265		0	0	0	0	1.00	0	0	0	4265	2669.82204	N	4265		4265
	38.1	5	0	0	N	0	LAFCD	1	4032		0	0	0	0	1.00	0	0	0	4032	2712.742123	N	4032		4032
	38A	4	4545	0	N	0	Tunnel	1	0		0	0	0	0	1.00	0	0	0	4545	303.206411	N	4545		0
	60Alt	0	800	0	N	0	Roadway	3	8417		0	0	0	8417	3.00	0	1	0	9217	0	N	9217		8417
	sum	**69**	**12916.5**		**Y**	**7**		**3.00**	**94020**		**395**	**5426.8**	**4**	**47874.2**	**1.80**	**17460**	**9**	**6529**	**106936**	**26512**	**N**	**106936**		**94020**
	Criteria	0.65	12.1%		Y	0.07		3.12		1.12			0.20		1.80	16.3%	0.44	6%	6%	25%	N			
	Score	1	3		5	1		3.12		1.12			1		1.80	3	1	1	1	5	1	**29**		**Raw Total**
	Weighted "A"	12.00	36.00		15.00	3.00		37.42		1.67			6.00		10.81	18.00	4.50	6.00	15.00	10.00	8.00	**183.41**	**63.32%**	**"A" Total**
	Weighted "B"	6.00	18.00		7.50	1.50		18.71		3.07			11.00		19.82	33.00	8.25	8.25	7.50	15.00	12.00	**169.60**	**65.12%**	**"B" Total**
Pink	5AX	25	1904	0	Y	7	Roadway	3	24825		0	0	1	24825	3.00	13535	7	2899	26729	158.59271	N	26729		24825
	10A.1(a)	3	674	0	N	0	Roadway	3	1855		0	0	0	1855	3.00	0	0	177	2529	0	N	2529		1855
	10A.1(b)	2	359	0	N	0	Roadway	3	2625		0	0	0	2625	3.00	0	0	0	2984	0	N	2984		2625
	10A.2	0	83	0	N	0	Private	1	384		0	0	0	0	1.00	0	1	0	468	0	N	468		384
	10A.3	1	0	0	N	0	Roadway	3	435		0	0	0	435	3.00	375	0	0	435	0	N	435		435
	20.1(a)	3	0	0	N	0	Roadway	3	3572		0	0	0	3572	3.00	2500	0	0	3572	0	N	3572		3572
	20.1(b)	0	0	0	N	0	Roadway	3	1125		0	0	0	1125	3.00	1050	0	0	1125	0	N	1125		1125
	20.2	1	191	0	N	0	SCE	1	1001		395	606	0	0	1.00	0	0	0	1192	0	N	1192		1001
	20.3	1	0	0	N	0	Roadway	3	636		0	0	1	636	1.00	0	0	0	636	0	N	636		636
	20.4	0	0	0	N	0	SCE	1	1199		0	0	1	0	1.00	0	0	0	1199	0	N	1199		1199
	20.5	1	169	0	N	0	Roadway	3	766		0	0	1	766	1.00	0	0	0	934	0	N	934		766
	20.6	0	0	0	N	0	Private	1	2180		0	0	0	0	1.00	0	0	0	2180	0	N	2180		2180
	20.7	2	1133	0	N	0	SCE	1	922		0	0	0	0	1.00	0	0	0	2055	0	N	2055		922
	20.8	5	0	0	N	0	SCE	1	1781		0	1780.8	0	0	1.00	0	0	0	1781	2.648382	N	1781		1781
	20.9	7	1211	2402	N	0	SCE	1	1192		0	0	0	0	1.00	0	0	0	2402	280.705414	N	2402		1192
	20.15	2	261	5118	N	0	SCE	1	4858		0	3040	0	0	1.00	0	0	0	5118	0	N	5118		4858
	20.11	0	0	2752	N	0	Roadway	3	2752		0	0	0	2752	1.00	0	0	2687	2752	0	N	2752		2752
	20.12	1	422	1741	N	0	LAFCD	1	1318		0	0	0	0	1.00	0	0	766	1741	0	N	1741		1318
	20.13	0	0	867	N	0	Roadway	3	867		0	0	0	867	1.00	0	0	0	867	0	N	867		867
	GreenLeft	0	1647	0	N	0	Roadway	3	2081		0	0	1	2081	5.00	2020	1	765	3728	0	N	3728		2081
	Pink1	0	2912	0	N	3	Roadway	3	14031		0	0	3	14031	3.00	8275	4	2150	16943	0	N	16943		14031
	Pink2	0	1438	0	N	1	Roadway	3	8059		0	0	1	8059	3.00	2255	2	1275	9497	0	N	9497		8059
	Pink3	0	1055	0	N	4	Roadway	3	15884		0	0	2	15884	3.00	5730	3	2300	16939	0	N	16939		15884
	sum	**54**	**13458.8**		**Y**	**15**		**2.69**	**94346**		**395**	**5426.8**	**11**	**79512.2**	**2.42**	**35740**	**18**	**13019**	**107805**	**442**	**N**	**107805**		**94346**
	Criteria	0.50	12.5%		Y	0.14		2.85		1.12			0.54		2.42	33.2%	0.88	12%	7%	0%	N			
	Score	1	3		5	1		2.85		1.12			3		2.42	3	1	3	1	1	1	**29**		**Raw Total**
	Weighted "A"	12.00	36.00		15.00	3.00		34.20		1.67			18.00		14.52	18.00	4.50	18.00	15.00	2.00	8.00	**199.89**	**60.02%**	**"A" Total**
	Weighted "B"	6.00	18.00		7.50	1.50		17.10		3.07			33.00		26.63	33.00	8.25	24.75	7.50	3.00	12.00	**201.29**	**58.60%**	**"B" Total**
Yellow	6	20	1474	0	N	5	Roadway	3	8850		0	0	4	8850	5.00	1050	2	2854	10324	0	N	10324		8850
	7.1(a)	4	0	0	N	3	Roadway	3	5105		0	0	0	5105	5.00	5100	0	1338	5105	0	N	5105		5105
	4A(b)	6	202	0	N	3	Roadway	3	1873		0	0	0	1873	3.00	1100	1	392	2075	0	N	2075		1873
	19A(a)	20	412	0	N	6	Roadway	3	12192		0	0	0	12192	3.00	4200	3	5061	12604	0	N	12604		12192
	19A(b)	7	307	0	N	1	Roadway	3	2476		0	0	0	2476	3.00	0	2	2231	2783	0	N	2783		2476
	19.1(c)	4	0	0	N	2	Roadway	3	1689		0	0	0	1689	3.00	1175	0	588	1689	0	N	1689		1689
	19.1(d)	7	174	0	N	8	Roadway	3	8691		0	0	1	8691	3.00	4685	2	6262	8865	0	N	8865		8691
	19.1(e)	11	993	7409	N	2	Roadway	3	6416		0	0	0	6416	5.00	5382	1	2040	7409	0	N	7409		6416
	19.1(f)	4	365	6044	N	1	Roadway	3	5679		0	0	1	5679	5.00	4700	1	988	6044	0	N	6044		5679
	35	9	1828	0	N	4	Roadway	3	17359		0	0	2	17359	5.00	17300	7	7019	19187	0	N	19187		17359
	40	1	160	0	N	0	Roadway	3	3686		0	0	1	3686	3.00	3700	1	1262	3846	0	N	3846		3686
	37Alt1	0	400	0	N	4	Roadway	3	7060		0	0	4	7060	3.00	5600	1	3200	7460	0	N	7460		7060
	37Alt2	0	0	0	N	0	Roadway	3	3135		0	0	1	3135	5.00	2500	0	300	3135	0	N	3135		3135
	38.2(b)	2	346	0	N	0	Roadway	3	680		0	0	0	680	3.00	0	0	0	1027	309.716581	N	1027		680
	60Alt	0	800	0	N	0	Roadway	3	8417		0	0	0	8417	3.00	0	1	0	9217	0	N	9217		8417
	sum	**95**	**7461.8**		**N**	**39**		**3.00**	**93308**		**0**	**0**	**14**	**93307.6**	**3.78**	**56492**	**22**	**33535**	**100769**	**310**	**N**	**100769**		**93308**
	Criteria	0.94	7.4%		N	0.39		3.07		1.00			0.73		3.78	56.1%	1.15	33%	0%	0%	N			
	Score	1	3		1	3		3.07		1.00			5		3.78	5	3	3	1	1	1	**35**		**Raw Total**
	Weighted "A"	12.00	36.00		3.00	9.00		36.89		1.50			30.00		22.65	30.00	13.50	18.00	15.00	2.00	8.00	**237.54**	**52.49%**	**"A" Total**
	Weighted "B"	6.00	18.00		1.50	4.50		18.44		2.75			55.00		41.53	55.00	24.75	24.75	7.50	3.00	12.00	**274.73**	**43.50%**	**"B" Total**

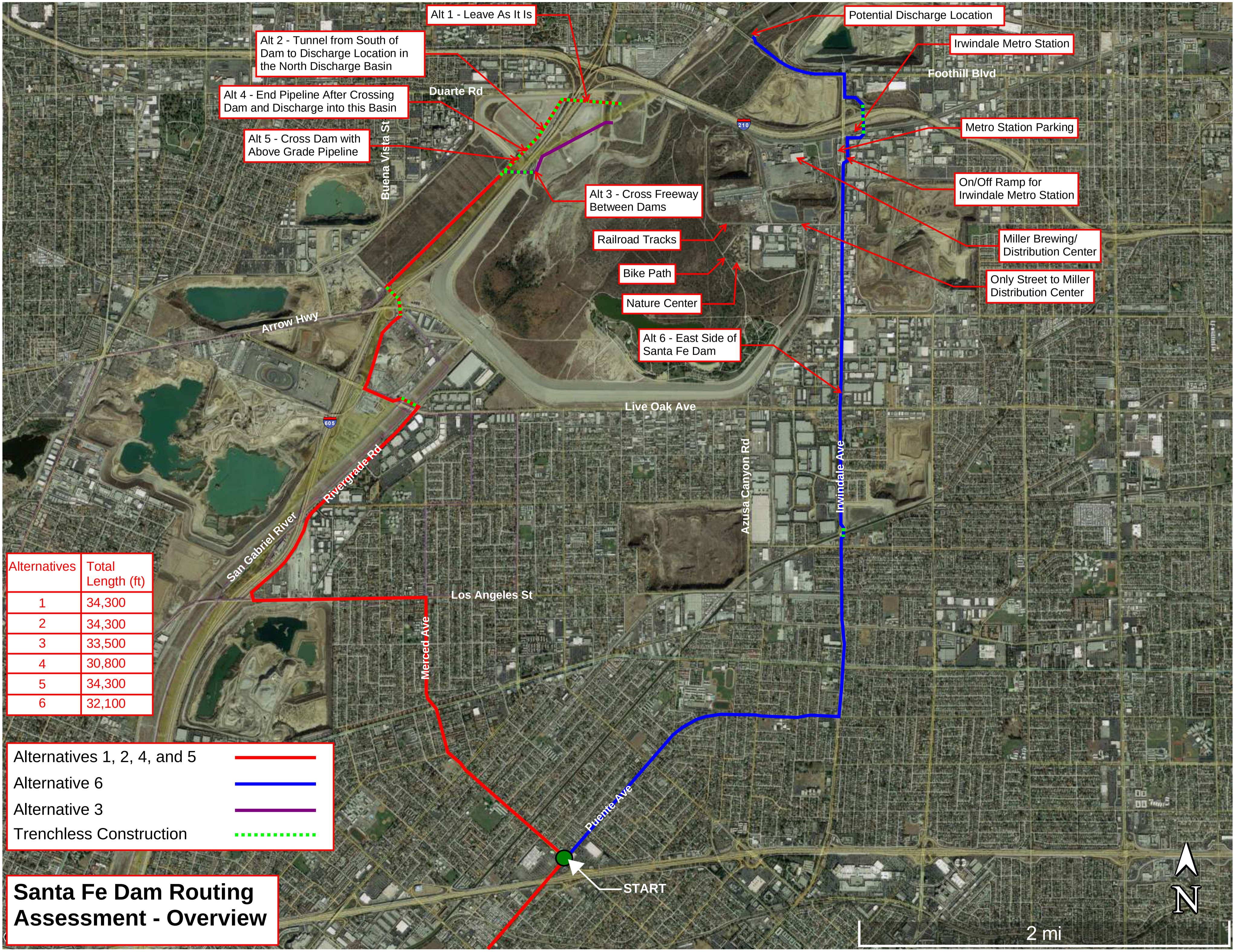
Yellow (Best Street Route From Eval				
Construction Method 1 - Roadway (Open Cut)				
84"	17,500	LF	$ 2,315.00	$ 40,512,500
60"	83,250	LF	$ 1,650.00	$ 137,362,500
54"	65,525	LF	$ 1,537.00	$ 100,712,540
Subtotal -				$ 278,587,540
Construction Method 2 - SCE Easement (Open Cut)				
84"		LF		$ -
60"		LF	$ 1,066.00	$ -
54"	38,175	LF	$ 956.00	$ 36,494,918
Subtotal -	204,450			$ 36,494,918
			Total	**$ 315,082,457**
Blue (Preferred Alignment)				
Construction Method 1 - Roadway (Open Cut)				
84"	33,100	LF	$ 2,315.00	$ 76,626,500
60"	18,135	LF	$ 1,650.00	$ 29,922,750
54"	46,225	LF	$ 1,537.00	$ 71,047,825
Subtotal -				$ 177,597,075
Construction Method 2 - SCE Easement (Open Cut)				
84"		LF	$ 1,723.00	$ -
60"	55,665	LF	$ 1,066.00	$ 59,338,890
54"	36,425	LF	$ 956.00	$ 34,822,300
Subtotal -	189,550			$ 94,161,190
			Total	**$ 271,758,265**
Pink (Avoids River Bed)				
Construction Method 1 - Roadway (Open Cut)				
84"	33,100	LF	$ 2,315.00	$ 76,626,500
60"	49,773	LF	$ 1,650.00	$ 82,125,450
54"	46,225	LF	$ 1,537.00	$ 71,047,825
Subtotal -				$ 229,799,775
Construction Method 2 - SCE Easement (Open Cut)				
84"		LF	$ 1,723.00	$ -
60"	24,877	LF	$ 1,066.00	$ 26,518,882
54"	36,425	LF	$ 956.00	$ 34,822,300
Subtotal -	190,400			$ 61,341,182
			Total	**$ 291,140,957**
Peach (Avoids San Gabriel and Easements)				
Construction Method 1 - Roadway (Open Cut)				
84"	33,100	LF	$ 2,315.00	$ 76,626,500
60"	65,850	LF	$ 1,650.00	$ 108,652,500
54"	46,225	LF	$ 1,537.00	$ 71,047,825
Subtotal -				$ 256,326,825
Construction Method 2 - SCE Easement (Open Cut)				
84"	5,200	LF	$ 1,723.00	$ 8,959,600
60"		LF	$ 1,066.00	$ -
54"	31,225	LF	$ 956.00	$ 29,851,100
Subtotal -	181,600			$ 38,810,700
			Total	**$ 295,137,525**
Purple (Avoids San Gabriel and Easements)				
Construction Method 1 - Roadway (Open Cut)				
84"	33,100	LF	$ 2,315.00	$ 76,626,500
60"	67,800	LF	$ 1,650.00	$ 111,870,000
54"	46,225	LF	$ 1,537.00	$ 71,047,825
Subtotal -				$ 259,544,325
Construction Method 2 - SCE Easement (Open Cut)				
84"	8,150	LF	$ 1,723.00	$ 14,042,450
60"		LF	$ 1,066.00	$ -
54"	28,275	LF	$ 956.00	$ 27,030,900
Subtotal -	183,550			$ 41,073,350
			Total	**$ 300,617,675**

SANTA FE DAM ALTERNATIVES

The Initial Preferred Alignment proposed a route on the west side of Interstate 605 to reach the Santa Fe Spreading Grounds. However, to reach the Santa Fe Spreading Grounds, this route would require crossing a dam. Although feasible, dam crossings would require additional permits and engineering work, in addition to coordination with various jurisdictions. Metropolitan asked Black & Veatch to investigate alternatives that would eliminate the dam crossing.

Black & Veatch identified a route on the east side of the Santa Fe Dam to reach the Santa Fe Spreading Grounds. However, the route would be significantly longer, require difficult freeway, river, and/or dam crossings, and have greater social and community impacts. Black & Veatch presented the results of the analysis, along with the recommendation to leave the Initial Preferred Alignment unaltered in this location, to Metropolitan staff at the August 31 workshop. Metropolitan's staff agreed that the Initial Preferred Alignment did not require any modifications in this area.

The following figures present the alternative route identified to reach the Santa Fe Spreading Grounds along with key details.



Alternatives	Total Length (ft)
1	34,300
2	34,300
3	33,500
4	30,800
5	34,300
6	32,100

Santa Fe Dam Routing Assessment - Overview

Alternatives	Total Length (ft)
1	34,300
2	34,300
3	33,500
4	30,800
5	34,300
6	32,100

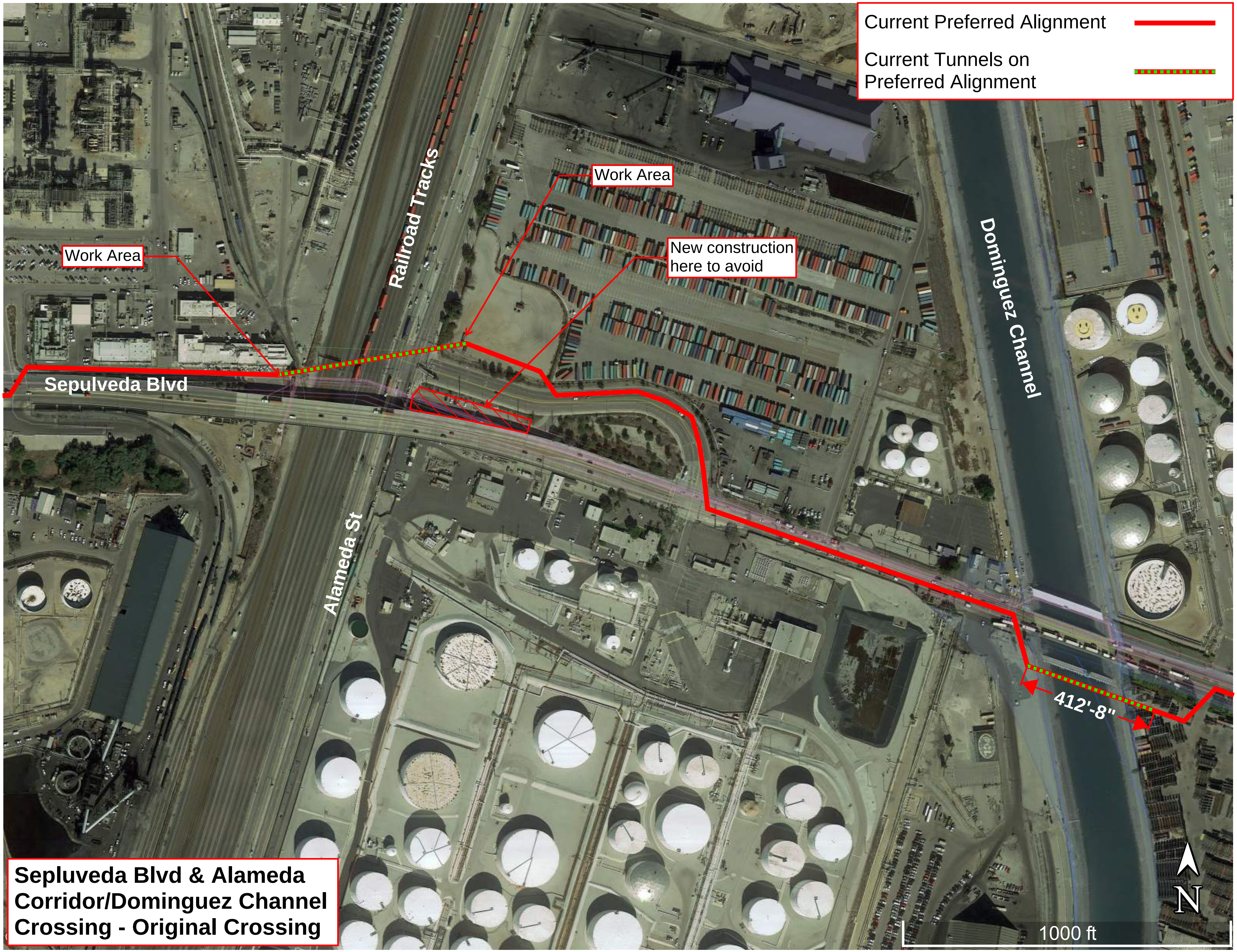
Santa Fe Dam Routing Assessment - Detail

ALAMEDA CORRIDOR/DOMINGUEZ CHANNEL CROSSING

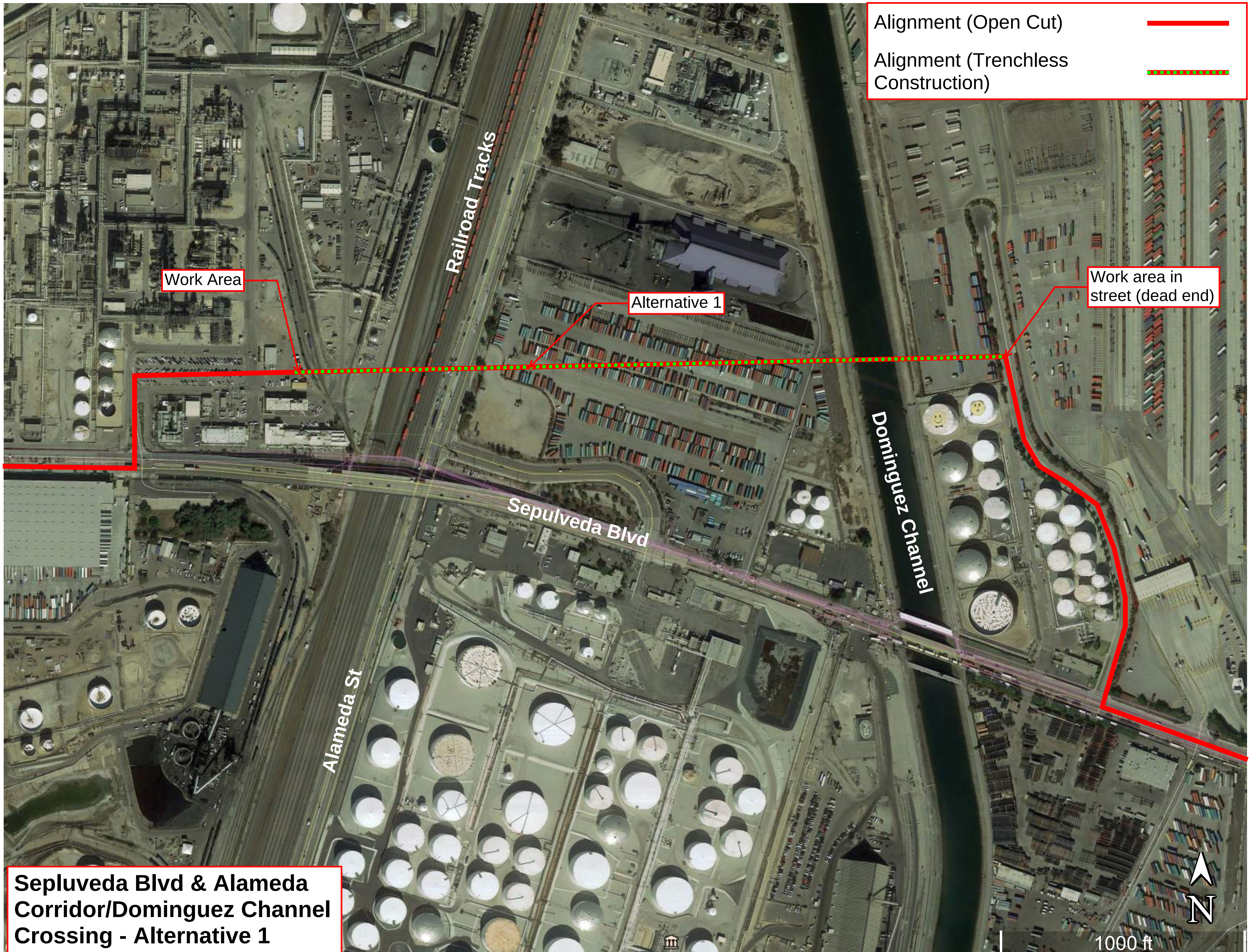
The Initial Preferred Alignment would require crossing the Alameda Corridor at Sepulveda Boulevard and then, approximately 1,700 ft later, crossing the Dominguez Channel. The Alameda Corridor includes multiple railroad tracks and a state highway (Alameda Street), and trenchless construction methods would be required to cross. Crossing the Dominguez Channel also would require trenchless construction methods. However, the land adjacent to Sepulveda Boulevard at these crossings is used as oil and gas refineries and is congested with tanks, below and above grade utilities, and other manufacturing facilities. Therefore, very limited space would be available for the launching and receiving portals required for any trenchless construction method and no clear cut route between the two crossings.

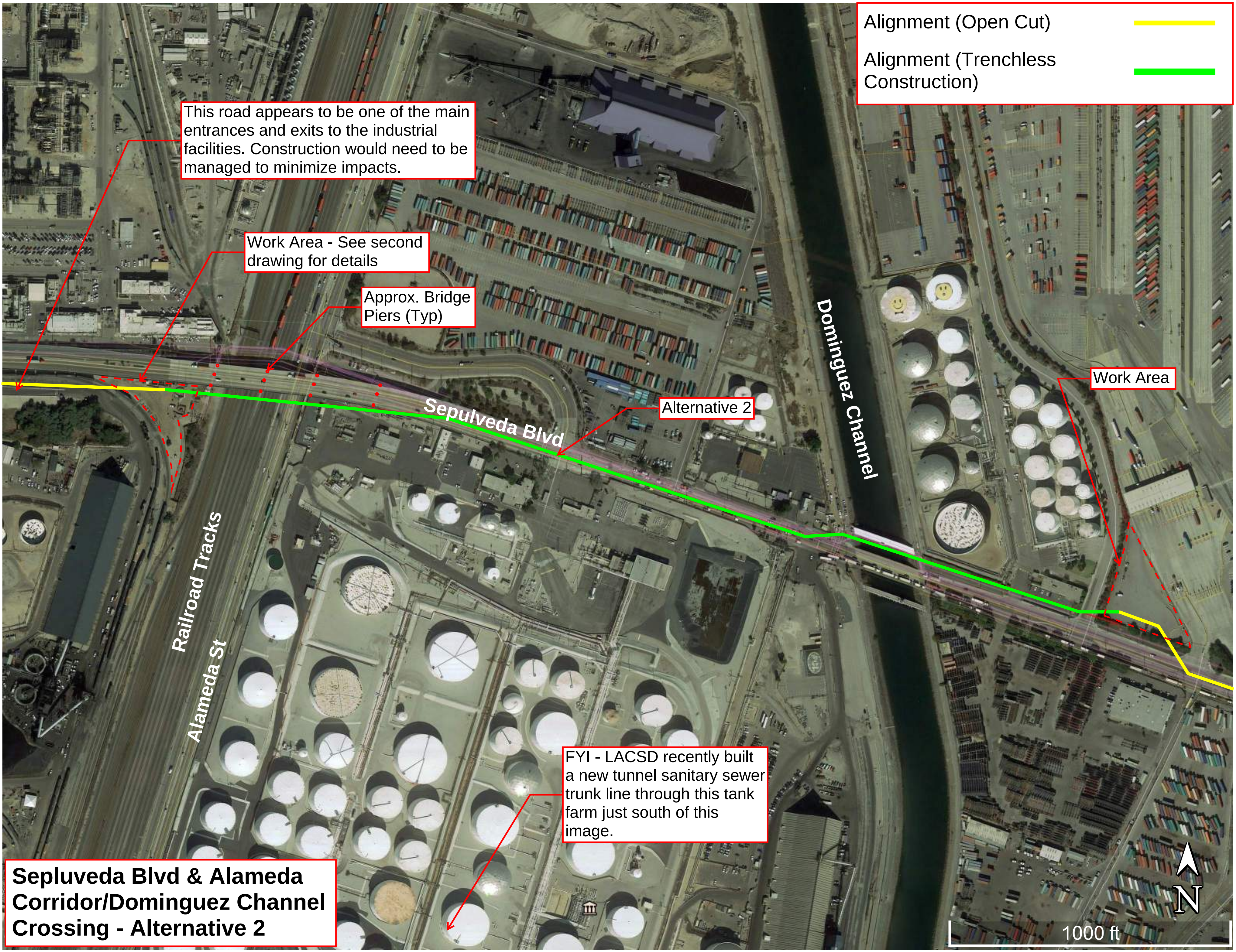
After discussions with Metropolitan staff, Black & Veatch developed three alternatives to construct these crossings and presented them during the August 31 workshop. All three alternatives were viable options for constructing the crossings. Further evaluation should be completed during the preliminary design phase of the Project to verify this crossing is preferred. Additional details of this crossing are discussed in Chapter 6.

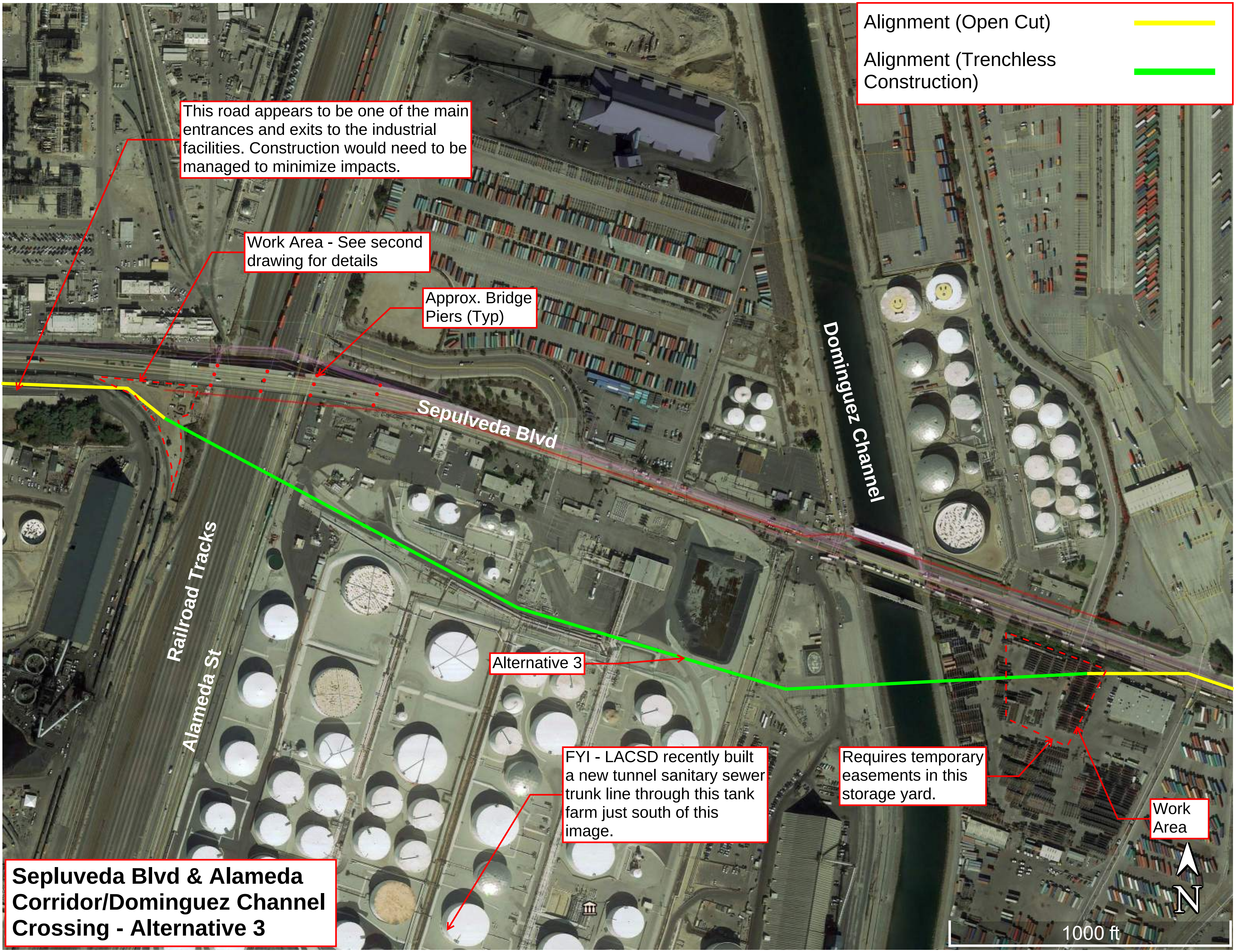
The following figures present the alternative crossings identified by Black & Veatch along with key details for each crossing.



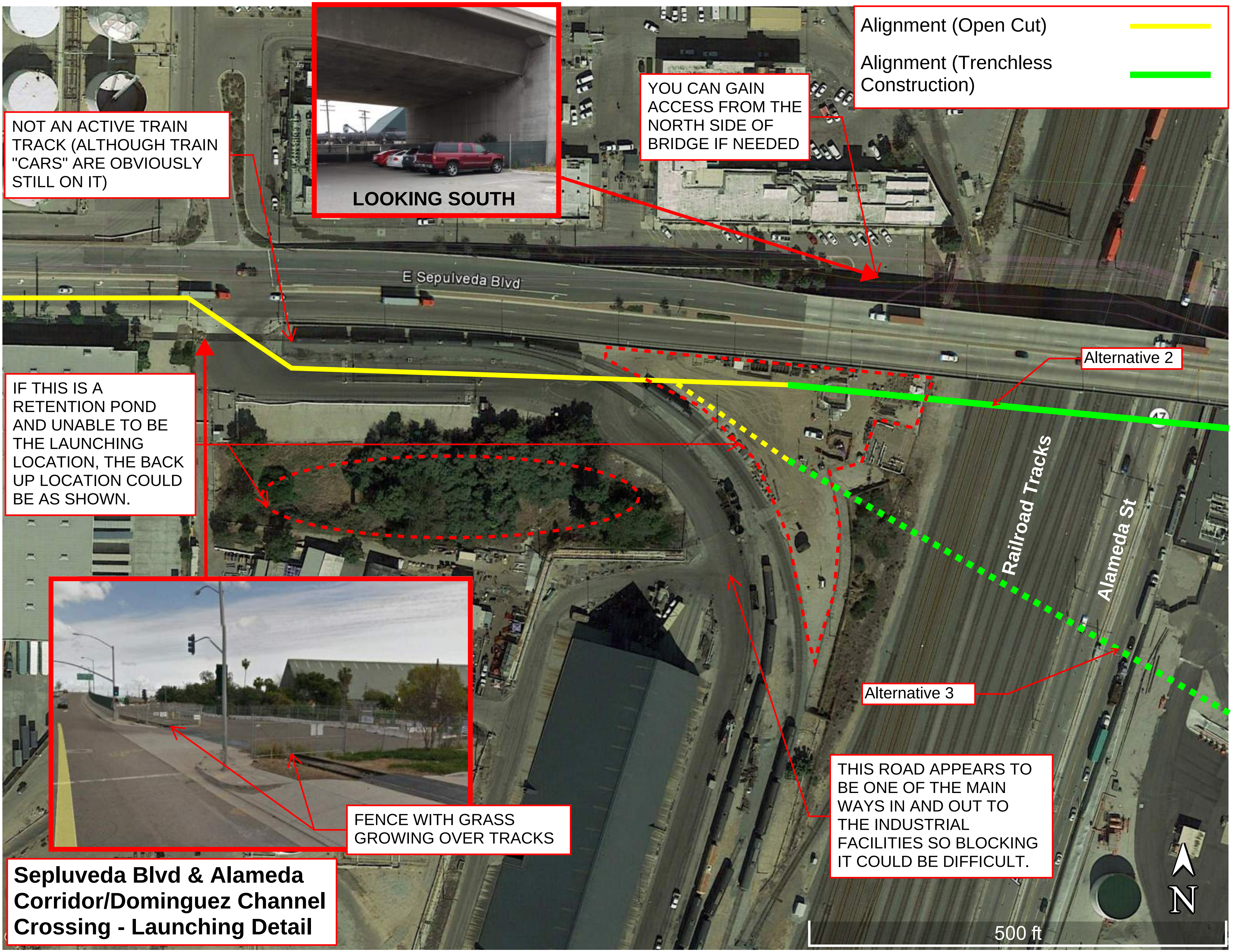
Sepluveda Blvd & Alameda Corridor/Dominguez Channel Crossing - Original Crossing


Alignment (Open Cut)
Alignment (Trenchless Construction)
Work Area
Alternative 1
Work area in street (dead end)
Railroad Tracks
Dominguez Channel
Sepulveda Blvd
Alameda St
N
1000 ft
Sepluveda Blvd & Alameda Corridor/Dominguez Channel Crossing - Alternative 1


Alignment (Open Cut)
Alignment (Trenchless Construction)
This road appears to be one of the main entrances and exits to the industrial facilities. Construction would need to be managed to minimize impacts.
Work Area - See second drawing for details
Approx. Bridge Piers (Typ)
Alternative 2
Work Area
Sepulveda Blvd
Dominguez Channel
Railroad Tracks
Alameda St
FYI - LACSD recently built a new tunnel sanitary sewer trunk line through this tank farm just south of this image.
N
1000 ft
Sepluveda Blvd & Alameda Corridor/Dominguez Channel Crossing - Alternative 2



Alignment (Open Cut)
Alignment (Trenchless Construction)
This road appears to be one of the main entrances and exits to the industrial facilities. Construction would need to be managed to minimize impacts.
Work Area - See second drawing for details
Approx. Bridge Piers (Typ)
Sepulveda Blvd
Dominguez Channel
Railroad Tracks
Alameda St
Alternative 3
FYI - LACSD recently built a new tunnel sanitary sewer trunk line through this tank farm just south of this image.
Requires temporary easements in this storage yard.
Work Area
N
1000 ft
Sepluveda Blvd & Alameda Corridor/Dominguez Channel Crossing - Alternative 3



Sepluveda Blvd & Alameda Corridor/Dominguez Channel Crossing - Launching Detail

Appendix S. Backbone Alignment Decision Model Details

Regional Recycled Water Supply System - Conveyance Feasibility Study

Evaluation Criteria and Data for Pipeline Segments and Sub-Segments

Alignment No	Alignment Sub-Segment	Pipe Length	Trenchless Construction	Trenched Construction	Major Utilities	Depth to Water	Seismic Hazard	Contaminated Soils Risk	Ease of Operations/ Accessibility	Non- SCE Parks & Rec Areas	SCE Parks & Rec Areas	Public Facilities	Length in Street	Lanes of Traffic	Road Category & Traffic Impact	Median Improvements	Major Intersections	Residential/ Minor Commercial	Property Description	Waters of the US and State	Critical Habitats and Listed Species
		ft	ft	ft	ea	ft	Y/N	# Hits		length	length	ea	length	# of lanes		lf	ea	length		length	Y/N
1	1(a)	1,800		1,800	13	11	N	0	Roadway			1	1800	4	Collector	1675	1	0	Roadway		N
1	1(b)	3,407		3,407	3	13	N	1	Roadway			1	3407	4	Collector	3340		1192	Roadway		N
1	1(c)	9,263		9,263	60	17	N	7	Roadway				9263	4	Collector	7490	2	7456	Roadway		N
1	1(d)	9,559	5,030	4,529	48	19	N	5	Roadway				4529	4	Collector	3700	2	5997	Roadway	655	N
10	10	3,372	248	3,124	-	15	N	0	SCE		3123.9		0	0	Easement	0	1		SCE		N
10A	10A.1(a)	2,529	1,005	1,524	3	19	N	0	Roadway				1524	4	Collector	0		177	Roadway		N
10A	10A.1(b)	2,984		2,984	2	14	N	0	Roadway				2984	4	Collector	0		0	Roadway		N
10A	10A.1(a)OC	2,529		2,529	3	19	N	0	Roadway				2529	4	Collector	0		177	Roadway		N
10A	10A.1(b)OC	2,984		2,984	2	14	N	0	Roadway				2984	4	Collector	0		0	Roadway		N
10A	10A.2	468		468	-	15	N	0	Private					0	Easement	0	1	0	Private		N
10A	10A.3	435		435	1	14	N	0	Roadway				435	4	Collector	375		0	Roadway		N
10A	10A.2OC	468		468	-	15	N	0	Private					0	Easement	0		0	Private		N
10A	10A.3OC	435		435	1	14	N	0	Roadway				435	4	Collector	375		0	Roadway		N
10A	10A.4	911		911	-	11	N	0	SCE					0	Easement	0		0	SCE		N
10A	10A.5	271		271	-	11	N	0	SCE					0	Easement	0		0	SCE		N
11	11.1(a)	9,733	1,402	8,332	8	10	N	0	SCE		3851	2		0	Easement	0		0	SCE	175	N
11	11.1(b)	2,039		2,039	-	10	N	0	SCE		2039.2			0	Easement	0		0	SCE		N
11	11.2	501		501	1	10	N	0	LAFCD					0	Easement	0		0	LAFCD	215	N
11	11.3(a)	2,689	149	2,541	1	10	N	0	SCE		2540.6			0	Easement	0		0	SCE		N
11	11.3(b)	2,534		2,534	-	10	N	0	SCE		2534.2			0	Easement	0		0	SCE		N
11A	11A(a)	2,544	421	2,123	-	17	N	0	Roadway				2123	2	Local	0		0	Roadway	154	N
11A	11A(a)OC	2,544	421	2,123	-	17	N	0	Roadway				2123	2	Local	0		0	Roadway	154	N
11A	11A(b)	5,243	2,004	3,239	6	10	N	0	Roadway				3239	Closure	Closure	2500		0	Roadway		N
11A	11A(c)	3,015	185	2,830	3	10	N	0	Roadway			1	2830	4	Collector	2040	1	139	Roadway		N
11A	11A(d)	2,636	162	2,474	4	10	N	0	Roadway			1	2474	4	Collector	1860	1	1190	Roadway	159	N
11B	11B(a)	3,001	169	2,832	4	10	N	1	Roadway			1	2832	4	Collector	1820	1	2338	Roadway		N
11B	11B(b)	2,601	200	2,401	3	10	N	2	Roadway			2	2401	4	Collector	1490	1	1210	Roadway		N
11B	11B(c)	7,961	1,965	5,996	3	10	N	4	Roadway			1	5996	4	Collector	5135	3	1909	Roadway		N
12	12(a)	9,211	2,662	6,549	12	10	N	2	Roadway				6549	2	Local	0	3	3341	Roadway	17	N
12	12(b)	5,287	524	4,763	3	10	N	0	Roadway				4763	2	Local	0		913	Roadway	309	N
12	12(c)	2,779	251	2,528	1	10	N	2	Roadway				2528	2	Local	0	1	842	Roadway		N
13	13	4,135		4,135	2	10	N	1	Roadway			4	4135	4	Collector	0		2225	Roadway	70	N
13A	13A	4,166	388	3,779	3	10	N	0	Roadway				3779	4	Collector	3500	1	566	Roadway	131	N
13C	13C	4,122	457	3,665	3	10	N	1	Roadway				3665	4	Collector	2330	2	440	Roadway	84	N
14	14	3,121		3,121	-	10	N	2	Roadway				3121	4	Collector	500	0	614	Roadway		N
14A	14A	1,932	235	1,697	-	10	N	1	Roadway			2	1697	4	Collector	920	2	999	Roadway		N
14B	14B	1,868	176	1,692	3	10	N	1	Roadway				1692	4	Collector	160	2	1400	Roadway	121	N
14C	14C	1,879	209	1,669	2	10	N	0	Roadway				1669	4	Collector	1670	2	987	Roadway		N
15	15	13,257	2,055	11,202	5	10	N	6	Roadway			3	11202	6	Arterial	2790	4	9299	Roadway	26	N
16	16	13,375	990	12,385	5	10	N	0	SCE		2529	4		0	Easement	0		1146	SCE		N
17	17	3,148	117	3,032	-	10	N	0	Roadway			1	3032	4	Collector	0		689	Roadway		N
18	18.1	1,629		1,629	-	11	N	0	Roadway				1629	4	Collector	520		635	Roadway		N
18	18.2	1,894	564	1,329	2	14	N	0	SCE					0	Easement	0		0	SCE		N
18	18.3	43,931	2,366	41,565	4	39	N	12	Roadway			2	41565	6	Arterial	2090	10	25869	Roadway	56	N
19	19.1(a)	5,538	261	5,277	4	21	N	0	Roadway				5277	4	Collector	4550	1	775	Roadway	16	N
19	19.1(b)	10,058	519	9,538	8	25	N	1	Roadway			2	9538	4	Collector	7725	3	2961	Roadway		N
19	19.1(c)	1,689		1,689	4	19	N	2	Roadway				1689	4	Collector	1175		588	Roadway		N
19	19.1(d)	8,865	174	8,691	7	11	N	8	Roadway			1	8691	4	Collector	4685	2	6262	Roadway		N
19	19.1(e)	7,409	993	6,416	11	8	N	2	Roadway				6416	6	Arterial	5382	1	2040	Roadway		N
19	19.1(f)	6,044	290	5,754	4	8	N	1	Roadway			1	5754	6	Arterial	4700	1	988	Roadway		N
19A	19A(a)	12,604	167	12,437	20	23	N	6	Roadway				12437	4	Collector	4200	3	5061	Roadway		N
19A	19A(a).1	10,006	167	9,839	20	23	N	4	Roadway				9839	4	Collector	4200	3	5061	Roadway		N

Regional Recycled Water Supply System - Conveyance Feasibility Study

Evaluation Criteria and Data for Pipeline Segments and Sub-Segments

Alignment No	Alignment Sub-Segment	Pipe Length	Trenchless Construction	Trenched Construction	Major Utilities	Depth to Water	Seismic Hazard	Contaminated Soils Risk	Ease of Operations/ Accessibility	Non- SCE Parks & Rec Areas	SCE Parks & Rec Areas	Public Facilities	Length in Street	Lanes of Traffic	Road Category & Traffic Impact	Median Improvements	Major Intersections	Residential/ Minor Commercial	Property Description	Waters of the US and State	Critical Habitats and Listed Species
		ft	ft	ft	ea	ft	Y/N	# Hits		length	length	ea	length	# of lanes		lf	ea	length		length	Y/N
19A	19A(b)	2,783		2,783	7	24	N	1	Roadway				2783	4	Collector	0	2	2231	Roadway		N
19B	19B.1	1,765	531	1,234	9	26	N	0	Roadway				1234	4	Collector	0	1	750	Roadway		N
19B	19B.2	2,643	113	2,530	2	22	N	1	SCE				0	0	Easement	0	1	0	SCE		N
19C	19C	9,190	1,274	7,916	10	8	N	0	Roadway			2	7916	Closure	Closure	0	3.5	8136	Roadway		N
1A	1A	9,731	462	9,269	18	14	N	0	Roadway			2	9269	4	Collector	6070	3	2803	Roadway		N
1B	1B(a)	5,964	358	5,606	11	15	N	2	Roadway				5606	4	Collector	5500	2	1364	Roadway		N
1B	1B(b)	8,572	1,170	7,402	16	15	N	5	Roadway			4	7402	6	Arterial	6850	3	3232	Roadway	146	N
1B	1B(c)	19,384	2,224	17,160	36	20	Y	6	Roadway			2	17160	6	Arterial	17000	6	3756	Roadway	418	N
1C	1C	16,200	1,286	14,914	74	14	N	7	Roadway			1	14914	4	Collector	4700	2	7067	Roadway		N
2	2(a)	12,936	6,383	6,553	18	30	Y	1	LAFCD				0	0	Easement	0		0	LAFCD		N
2	2(b)	7,405	5,393	2,013	12	30	Y	0	LAFCD				0	0	Easement	0		0	LAFCD		N
20	20.1(a)	3,572		3,572	3	14	N	0	Roadway				3572	4	Collector	2500		0	Roadway		N
20	20.1(b)	1,125		1,125	-	12	N	0	Roadway				1125	4	Collector	1050		0	Roadway		N
20	20.11	2,752		2,752	-	5	N	0	Roadway				2752	0	Easement	0		2687	Roadway		N
20	20.12	1,741	475	1,266	1	5	N	0	LAFCD				0	0	Easement	0		766	LAFCD		N
20	20.13	867		867	-	5	N	0	Roadway				867	0	Easement	0		0	Roadway		N
20	20.14	4,211	125	4,086	1	6	N	0	River				0	0	Easement	0		0	LAFCD	1,160	N
20	20.14T	4,211	4,211	-	-	6	N	0	River				0	0	Easement	0		0	LAFCD	-	N
20	20.15	5,118	205	4,913	2	5	N	0	SCE		3040		0	0	Easement	0		0	SCE		N
20	20.2	1,192	205	987	1	12	N	0	SCE	395	606		0	0	Easement	0		0	SCE		N
20	20.3	636		636	1	12	N	0	Roadway			1	636	0	Easement	0		0	Roadway		N
20	20.4	1,199		1,199	-	12	N	0	SCE			1	0	0	Easement	0		0	SCE		N
20	20.5	934	169	766	1	13	N	0	Roadway			1	766	0	Easement	0		0	Roadway		N
20	20.6	2,180		2,180	-	15	N	0	Private					0	Easement	0		0	Private		N
20	20.7	2,055	818	1,237	2	17	N	0	SCE					0	Easement	0		0	SCE		N
20	20.8	1,781		1,781	5	14	N	0	SCE		1780.8			0	Easement	0		0	SCE	3	N
20	20.9	2,402	553	1,849	7	8	N	0	SCE					0	Easement	0		0	SCE	281	N
20A	20A	8,655	1,176	7,480	13	14	N	0	River				0	0	Easement	0		0	LAFCD	8,250	N
20B	20B	12,168	1,270	10,898	18	9	N	4	Roadway			5	10898	4	Collector	10890	3	5074	Roadway	87	N
21	21.1	2,193	2,128	65	-	8	N	0	SCE		415			0	Easement	0		0	SCE		N
21	21.2	939	939	-	-	8	N	0	Roadway				0	Closure	Closure	0	1	939	Roadway		N
21	21.3(a)	2,748	2,600	148	-	8	N	0	SCE		2747.7			0	Easement	0		2477	SCE		N
21	21.3(b)	1,495	250	1,245	1	8	N	0	SCE					0	Easement	0		0	SCE		N
21	21.4	7,180	533	6,647	3	9	N	0	SCE					0	Easement	0		0	SCE	149	N
21	21.5	4,964	740	4,224	4	17	N	2	LAFCD					0	Easement	0		0	LAFCD	1,599	N
21	21.6	3,900		3,900	-	32	N	0	LAFCD					0	Easement	0		0	LAFCD		N
21A	21A	5,773	248	5,525	6	8	N	0	River					0	Easement	0		0	LAFCD	5,339	N
21B	21B	6,176	372	5,803	4	8	N	1	Roadway				5803	4	Collector	210	2	3344	Roadway		N
22	22.1	1,219		1,219	-	8	N	0	River					0	Easement	0		0	LAFCD	1,219	N
22	22.1T	1,219	1,219	-	-	8	N	0	River					0	Easement	0		0	LAFCD	-	N
22	22.2	18,750	420	18,330	5	10	N	0	River					0	Easement	0		0	LAFCD	18,005	N
22	22.2T	18,750	18,750	-	-	10	N	0	River					0	Easement	0		0	LAFCD	-	N
23	23.1	9,872	550	9,322	3	20	N	0	Roadway				2900	6	Arterial	2900	1	1000	Roadway	6,800	N
23	23.2	9,134	973	8,161	6	5	N	2	Roadway			1	8161	6	Arterial	6,529	3	2900	Roadway	86	N
23	23.3	5,364	223	5,141	1	3	N	0	LAFCD					0	Easement	0		0	LAFCD		N
24	24.1	515		515	1	12	N	0	Roadway				515	6	Arterial	500		250	Roadway		N
24	24.2	139		139	-	12	N	0	LAFCD					0	Easement	0		0	LAFCD	22	N
25	25(a)	3,447		3,447	2	8	N	0	Roadway				3447	6	Arterial	1300		1800	Roadway		N
25	25(b)	3,849	254	3,595	2	8	N	0	Roadway				3595	6	Arterial	3500	2	3471	Roadway		N
26	26	3,100	372	2,728	4	14	N	0	Roadway			1	2728	6	Arterial	2700	2	1791	Roadway		N
27	27	19,619	1,263	18,356	14	11	N	1	Roadway			3	18356	4	Collector	10600	6	8177	Roadway		N
28	28.1	4,700		4,700	2	13	N	1	Roadway			1	4700	6	Arterial	560		1547	Roadway		N

Regional Recycled Water Supply System - Conveyance Feasibility Study

Evaluation Criteria and Data for Pipeline Segments and Sub-Segments

Alignment No	Alignment Sub-Segment	Pipe Length	Trenchless Construction	Trenched Construction	Major Utilities	Depth to Water	Seismic Hazard	Contaminated Soils Risk	Ease of Operations/ Accessibility	Non- SCE Parks & Rec Areas	SCE Parks & Rec Areas	Public Facilities	Length in Street	Lanes of Traffic	Road Category & Traffic Impact	Median Improvements	Major Intersections	Residential/ Minor Commercial	Property Description	Waters of the US and State	Critical Habitats and Listed Species
		ft	ft	ft	ea	ft	Y/N	# Hits		length	length	ea	length	# of lanes		lf	ea	length		length	Y/N
28	28.2	63		63	1	10	N	0	LAFCD					0	Easement	0		0	LAFCD		N
29	29	8,719	367	8,352	9	11	N	0	Roadway			1	8352	4	Collector	5000	2	1737	Roadway		N
2A	2A	5,595	4,774	821	6	30	Y	0	LAFCD				0	0	Easement	0		0	LAFCD		N
2AOC	2AOC	5,595	4,774	821	6	30	Y	0	LAFCD				0	0	Easement	0		0	LAFCD		N
3	3.1	4,632		4,632	4	30	N	0	LAFCD				0	0	Easement	0		0	LAFCD		N
3	3.2	3,274		3,274	8	29	N	0	Roadway				3274	2	Local	0		0	Roadway	18	N
3	3.3(a)	2,195	2,126	69	-	17	N	0	LAFCD					0	Easement	0		0	LAFCD		N
3	3.3(b)	16,575	11,543	5,032	24	9	N	3	LAFCD	4130				0	Easement	0		0	LAFCD		N
3	3.4(a)	4,696	200	4,496	10	8	N	0	SCE		2150			0	Easement	0		0	SCE		N
3	3.4(b)	860		860	-	8	N	0	SCE					0	Easement	0		0	SCE		N
3	3.4(c)	2,689	371	2,318	1	8	N	0	SCE		570			0	Easement	0		0	SCE		N
30	30	125		125	1	15	N	0	LAFCD					0	Easement	0		0	LAFCD	42	N
31	31	1,834	150	1,684	1	2	N	0	Roadway				1684	4	Collector	0	1	0	Roadway		N
32	32	1,890	176	1,715	3	10	N	0	Roadway				1715	2	Local	0	1	390	Roadway		N
33	33	4,950	497	4,454	1	8	N	1	Roadway			1	4454	6	Arterial	3325	4	3498	Roadway		N
34	34.1	4,063		4,063	-	10	N	0	Roadway				4063	2	Local		1	438	Roadway		N
34	34.2	263		263	-	10	N	0	LAFCD					0	Easement			0	LAFCD	64	N
35	35	19,187	1,828	17,359	9	13	N	4	Roadway			2	17359	6	Arterial	17300	7	7019	Roadway		N
36	36	4,265		4,265	-	10	N	0	LAFCD			0		0	Easement			0	LAFCD	2,670	N
37	37	9,977	125	9,852	7	10	N	4	Roadway			4	9852	4	Collector	9000	1	4316	Roadway		N
38	38.1	4,032		4,032	5	11	N	0	LAFCD					0	Easement			0	LAFCD	2,713	N
38	38.2(a)	7,549	68	7,480	6	13	N	1	Roadway			2	7480	Closure	Closure	3735	1	5017	Roadway		N
38	38.2(b)	1,027	346	680	2	10	N	0	Roadway				680	4	Collector			0	Roadway	310	N
38	38.3	3,075		3,075	2	8	N	0	LAFCD					0	Easement			0	LAFCD		N
38	38.4(a)	11,474		11,474	6	5	N	0	Roadway			1	11474	4	Collector	3190	1	3315	Roadway	54	N
38	38.4(b)	716	666	50	-	5	N	0	Roadway				50	0	Easement			0	Roadway	515	N
38A	38A	4,646	3,749	897	4	11	N	0	Tunnel					0	Easement			0	Tunnel	303	N
38B	38B.1	3,525	141	3,384	3	8	N	0	Roadway				3384	4	Collector	1950	1	753	Roadway		N
38B	38B.2	580	513	66	-	5	N	0	LAFCD					0	Easement			0	LAFCD	341	N
39	39	576		576	-	1	N	0	Roadway				576	4	Collector	575		0	Roadway		N
3A	3A	19,580	1,686	17,894	27	10	N	3	Roadway			1	17894	Closure	Closure	4200	1	8019	Roadway		N
3B	3B	2,669	76	2,593	-	8	N	1	Roadway				2593	Closure	Closure		1	0	Roadway		N
4	4(a)	2,282		2,282	3	30	N	1	Roadway			1	2282	6	Arterial	2280		1121	Roadway		N
4	4(a)OC	2,282		2,282	3	30	N	1	Roadway			1	2282	6	Arterial	2280		1121	Roadway		N
4	4(b)OC	6,019	159	5,861	9	20	N	2	Roadway			1	5861	4	Collector	1500	1	2072	Roadway	16	N
4	4(c)OC	6,031	243	5,788	5	20	N	3	Roadway				5788	4	Collector		1	1446	Roadway	-	N
4	4(c)X	6,031	160	5,871	5	20	N	3	Roadway				5871	4	Collector		1	1446	Roadway	-	N
4	4(b)	6,019	159	5,861	9	20	N	2	Roadway			1	5861	4	Collector	1500	1	2072	Roadway	16	N
4	4(c)	6,031	243	5,788	5	20	N	3	Roadway				5788	4	Collector		1	1446	Roadway	-	N
40	40	3,846		3,846	1	10	N	0	Roadway			1	3846	4	Collector	3700	1	1262	Roadway		N
41	41.1	2,644		2,644	-		N	0	Roadway				2644	2	Local	2600		0	Roadway	64	N
41	41.2	1,106	755	351	-		N	0	LAFCD				0	0	Easement			0	LAFCD	1,106	N
41	41.3	1,100		1,100	1		N	0	LAFCD				0	0	Easement			0	LAFCD	1,054	Y
41A	41A	1,165		1,165	1		N	0	Roadway				1165	4	Collector	1100		0	Roadway	1,072	N
42	42	4,236		4,236	2	14	N	1	Roadway				4236	4	Collector	325		948	Roadway		N
43	43	9,627	188	9,439	3	1	N	2	Roadway				9439	Closure	Closure	550	1	2717	Roadway		N
43A	43A	654		654	1	1	N	1	Roadway				654	4	Collector	654		360	Roadway	-	N
44	44.1	1,768	350	1,418	-	5	N	0	LAFCD					0	Easement			0	LAFCD		N
44	44.2(a)	3,959	1,826	2,133	1	5	N	0	SCE					0	Easement			0	SCE	1,169	N
44	44.2(b)	15,794	2,085	13,709	6	8	N	0	SCE					0	Easement			0	SCE	748	N
44	44.3(a)	5,485		5,485	-	24	N	0	SCE					0	Easement			0	SCE	3,919	N
44	44.3(b)	1,885		1,885	1	34	N	0	SCE					0	Easement			0	SCE	25	N

Regional Recycled Water Supply System - Conveyance Feasibility Study

Evaluation Criteria and Data for Pipeline Segments and Sub-Segments

Alignment No	Alignment Sub-Segment	Pipe Length	Trenchless Construction	Trenched Construction	Major Utilities	Depth to Water	Seismic Hazard	Contaminated Soils Risk	Ease of Operations/ Accessibility	Non- SCE Parks & Rec Areas	SCE Parks & Rec Areas	Public Facilities	Length in Street	Lanes of Traffic	Road Category & Traffic Impact	Median Improvements	Major Intersections	Residential/ Minor Commercial	Property Description	Waters of the US and State	Critical Habitats and Listed Species
		ft	ft	ft	ea	ft	Y/N	# Hits		length	length	ea	length	# of lanes		lf	ea	length		length	Y/N
44A	44A.1	4,931	892	4,039	4	5	N	0	LAFCD					0	Easement			0	LAFCD	177	N
44A	44A.2	1,352		1,352	1	5	N	0	SCE					0	Easement			0	SCE	36	N
45	45(a)	4,244		4,244	1	5	N	0	Roadway				4244	4	Collector	1750	1.5	2501	Roadway		N
45	45(b)	7,235	175	7,060	4	8	N	0	Roadway				7060	4	Collector			6689	Roadway		N
45A	45A	8,833	455	8,378	9	7	N	2	Roadway				8378	4	Collector	1825	3	4584	Roadway		N
46	46	5,605	353	5,252	2	11	N	0	Roadway				5252	4	Collector	1120	2	4839	Roadway		N
47	47	9,118	286	8,832	4	17	N	2	Roadway			1	8832	6	Arterial	6700	2	6165	Roadway		N
47A	47A	16,619	1,214	15,405	3	58	N	5	Roadway			1	15405	6	Arterial	11900	4	10026	Roadway	96	N
48	48	3,505		3,505	-	23	N	0	Roadway				3505	4	Collector		1	3500	Roadway		N
4A	4A(a)	8,473	536	7,937	21	20	N	2	Roadway			4	7937	6	Arterial	6175	3	1931	Roadway	18	N
4A	4A(b)	2,075	203	1,872	6	20	N	3	Roadway				1872	4	Collector	1100	1	392	Roadway		N
4A	4A(c)	5,497	452	5,045	8	20	N	1	Roadway				5045	4	Collector	2800	2	171	Roadway	15	N
4A	4A(d)	10,493	607	9,885	12	28	N	4	Roadway			1	9885	6	Arterial	2450	3	1442	Roadway	79	N
4A	4A(e)	2,290	740	1,550	-	23	N	0	Roadway				1550	4	Collector	0		500	Roadway		N
4A	4A(a)OC	8,473	906	7,567	21	20	N	2	Roadway			4	7567	6	Arterial	6175	3	1931	Roadway	18	N
4A	4A(b)OC	2,075	202	1,873	6	20	N	3	Roadway				1873	4	Collector	1100	1	392	Roadway		N
4A	4A(c)OC	5,497	452	5,045	8	20	N	1	Roadway				5045	4	Collector	2800	2	171	Roadway	15	N
4A	4A(d)OC	10,493	607	9,885	12	28	N	4	Roadway			1	9885	6	Arterial	2450	3	1442	Roadway	79	N
4A	4A(e)OC	2,290	740	1,550	-	23	N	0	Roadway				1550	4	Collector			500	Roadway		N
4B	4B	3,326	497	2,829	1	14	N	0	Roadway			1	2829	4	Collector	1915	2	1332	Roadway	165	N
4B	4BOC	3,326	497	2,829	1	14	N	0	Roadway			1	2829	4	Collector	1915	2	1332	Roadway	165	N
5	5	11,011	617	10,394	8	17	Y	4	Roadway			1	10394	6	Arterial	3800	3	3279	Roadway		N
5X	5X	11,011		11,011	8	17	Y	4	Roadway			1	11011	6	Arterial	8900	4	5690	Roadway		N
51	51.1	2,871		2,871	1	27	N	1	Roadway				2871	4	Collector			662	Roadway		N
51	51.2	929		929	-	31	N	0	LAFCD				0	0	Easement			0	LAFCD	382	N
52	52.1(a)	2,605		2,605	2	43	N	0	Roadway				2605	Closure	Closure			1260	Roadway		N
52	52.1(b)	3,513		3,513	4	81	N	0	Roadway				3513	4	Collector			1884	Roadway		N
52	52.2	600		600	-	105	N	0	LAFCD					0	Easement			0	LAFCD	276	N
52A	52A.1	4,266	1,207	3,060	1	39	N	0	River					0	Easement			0	LAFCD	3,724	N
52A	52A.2	289		289	-	50	N	0	LAFCD					0	Easement			0	LAFCD	78	N
52B	52B.1	3,777	608	3,169	-	79	N	0	River					0	Easement			0	LAFCD	3,777	N
52B	52B.2	409	56	353	-	106	N	0	River					0	Easement			0	LAFCD	234	N
52C	52C.1	1,531	141	1,391	-	113	N	0	River					0	Easement			0	LAFCD	1,531	N
52C	52C.2	2,119		2,119	-	121	N	0	Roadway				2119	Closure	Closure			0	Roadway	224	N
53	53(a)	5,769	665	5,104	1	41	N	0	Roadway				5104	4	Collector			3416	Roadway		N
53	53(b)	2,674		2,674	-	76	N	0	Roadway				2674	4	Collector			2269	Roadway		N
54	54	5,215	219	4,996	1	110	N	0	Roadway				4996	4	Collector		2	3795	Roadway		N
54A	54A	6,556	124	6,433	2	91	N	0	Roadway				6433	2	Local			5015	Roadway		N
55	55(a)	1,293		1,293	-	120	N	0	Roadway				1293	4	Collector	1290	1	1292	Roadway		N
55	55(b)	1,819	256	1,563	1	110	N	0	Roadway				1563	4	Collector	1560	1	926	Roadway	42	N
56	56	1,080		1,080	-	104	N	0	Roadway				1080	4	Collector	1080		0	Roadway		N
57	57.1	5,416	717	4,699	7	8	N	1	Roadway			1	4699	6	Arterial	4900	3	5200	Roadway		N
57	57.2	92		92	-	8	N	0	LAFCD					0	Easement			0	LAFCD	11	N
58.1	58.1	1,705		1,705	-	109	N	0	SCE					0	Easement			0	SCE		N
58.2	58.2(a)	682		682	-	119	N	0	Private					0	Easement			0	Private		N
58.2	58.2(b)	963	961	2	-	124	N	0	Private					0	Easement			0	Private		N
59	59	9,247	792	8,454	-	156	N	0	LAFCD					0	Easement			0	LAFCD	2,535	N
5A	5A	26,729	558	26,171	25	23	Y	7	Roadway				26171	4	Collector	8150	6	1819	Roadway	159	N
5AX	5AX	26,729		26,729	25	23	Y	7	Roadway			1	26729	4	Collector	13535	7	2899	Roadway	159	N
6	6	10,324	1,474	8,850	20	20	N	5	Roadway			4	8850	6	Arterial	1050	2	2854	Roadway		N
60	60	4,900	530	4,370	4	81	N	0	Roadway				4370	4	Collector			1884	Roadway		N
60	60-Road	51,797	2,331	49,466	38	81	N	4	Roadway				49466	4	Collector	23545	5	9125	Roadway	-	N

Regional Recycled Water Supply System - Conveyance Feasibility Study

Evaluation Criteria and Data for Pipeline Segments and Sub-Segments

Alignment No	Alignment Sub-Segment	Pipe Length	Trenchless Construction	Trenched Construction	Major Utilities	Depth to Water	Seismic Hazard	Contaminated Soils Risk	Ease of Operations/ Accessibility	Non- SCE Parks & Rec Areas	SCE Parks & Rec Areas	Public Facilities	Length in Street	Lanes of Traffic	Road Category & Traffic Impact	Median Improvements	Major Intersections	Residential/ Minor Commercial	Property Description	Waters of the US and State	Critical Habitats and Listed Species
		ft	ft	ft	ea	ft	Y/N	# Hits		length	length	ea	length	# of lanes		lf	ea	length		length	Y/N
60	60-ALT	5,330	530	4,800	4	81	N	0	Roadway				4800	4	Collector			1884	Roadway		N
7	7.1(a)	5,105		5,105	4	20	N	3	Roadway				5105	6	Arterial	5100		1338	Roadway		N
7	7.1(a)OC	5,105		5,105	4	20	N	3	Roadway				5105	6	Arterial	5100		1338	Roadway		N
7	7.1(b)	34,046	2,215	31,831	53	13	N	12	Roadway			1	31831	4	Collector	8660	5	15355	Roadway		N
7	7.2	3,275	445	**2,830**	9	8	N	0	SCE					0	Easement		2	0	SCE		N
8	8(a)	7,373	452	6,921	8	20	N	2	Roadway			1	6921	6	Arterial	3850	1	1917	Roadway		N
8	8(a)X	7,373	4,529	2,844	8	20	N	2	Roadway				2844	6	Arterial	3850	1	1917	Roadway		N
8	8(b)	10,591	715	9,876	10	27	N	3	Roadway			1	9876	6	Arterial	8750	3	1744	Roadway	82	N
8	8(c)	2,629		2,629	2	27	N	1	Roadway				2629	6	Arterial	2470		254	Roadway		N
8	8(d)	2,170	652	1,517	3	18	N	0	Roadway			1	1517	6	Arterial	1500		281	Roadway	165	N
8	8(a)OC	7,373	452	6,921	8	20	N	2	Roadway			1	6921	6	Arterial	3850	1	1917	Roadway		N
8	8(b)OC	10,591	715	9,876	10	27	N	3	Roadway			1	9876	6	Arterial	8750	3	1744	Roadway	82	N
8	8(c)OC	2,629		2,629	2	27	N	1	Roadway			0	2629	6	Arterial	2470		254	Roadway		N
8	8(d)OC	2,170	652	1,517	3	18	N	0	Roadway			1	1517	6	Arterial	1500		281	Roadway	165	N
9	9	2,353		2,353	5	15	N	0	SCE		975			0	Easement			0	SCE	16	N
9	9OC	2,353		2,353	5	15	N	0	SCE		975			0	Easement			0	SCE	16	N
9A	9A	5,456		5,456	4	29	N	0	Roadway			1	5456	Closure	Closure		0	5400	Roadway		N
9A	9AOC	5,456		5,456	4	29	N	0	Roadway			1	5456	Closure	Closure	0	0	5400	Roadway		N
100	100.1	2,269	-	2,269	-	20	N	0	LAFCD	-	-	-	0	0	0	0	0	0	LAFCD	-	N
100	100.2	6,495	-	6,495	5	20	N	2	Roadway	-	-	2	84	4	4	0	1	3075	Roadway	-	N
100	100.3	10,772	1,316	9,456	6	20	N	1	SCE	-	2590	1	0	0	0	40	0	0	SCE	200	N
100	100.4	4,964	-	4,964	1	20	N	0	SCE	-	3015	-	0	0	0	0	0	0	SCE	-	N
101	101.1	3,062	3,062	-	-	25	Y	1	LAFCD	-	-	-	0	0	0	0	0	0	LAFCD	635	N
101	101.2	5,756	1,195	4,561	-	30	N	0	LAFCD	-	-	-	0	0	0	0	0	0	LAFCD	380	N
102	102	26,770	1,212	25,558	3	20	N	3	Roadway	-	-	9	84	4	4	8870	7	3900	Roadway	-	N
103	103	31,363	1,500	29,863	4	11	N	-	Roadway	-	-	-	4,110	4	4	4,000	2	3,721	Roadway	-	N
104	104	483		483		11	N	-	Roadway	-	-	-	483	4	4	483	-	-	Roadway	-	N
105	105-Alt	30,571	3,238	27,333	13	15	N	4	Roadway	-	-	4	27,333	4	4	10,500	6	3,425	Roadway	-	N
105	105	47,900	2,825	45,075	20	15	N	8	Roadway	-	-	6	45,075	4	4	16,260	9	5,725	Roadway	-	N

Screening Results Summary Table

Routes	San Gabriel River Alignment				LA River Alignment			
	sum	Raw Score	Weight "A"	Weight "B"	sum	Raw Score	Weight "A"	Weight "B"
Major Utilities	223	3	36	18	211	3	36	18
Trenchless Construction	21,140	3	36	18	35,853	5	60	30
Depth to Water	78,111	5	15	8	67,299	5	15	8
Seismic Hazard	Y	5	15	8	Y	5	15	8
Contaminated Soils Risk	24	3	9	5	22	3	9	5
Ease of Operation Sub-Score	3	3	23	11	2	2	17	8
Parks	1	1	2	3	1	1	2	3
Public Facilities	7	3	18	33	7	3	18	33
Road Category & Traffic Impact	2	2	12	22	2	2	11	20
Center Medians	36,350	3	18	33	29,944	3	18	33
Major Intersections	16	3	14	25	14	3	14	25
Residential/ Minor Commercial	29,832	3	18	33	31,495	3	18	33
Total Alignment Length	198,197	1	15	8	190,337	1	15	8
Waters of the US and State	36,131	5	10	15	19,487	3	6	9
Critical Habitats and Listed Species	N	1	8	12	N	1	8	12
Scour	Y	5	15	8	N	1	3	2
Weighted Score			59%	61%			59%	62%
		Raw Total	"A" Total	"B" Total			"A" Total	"B" Total

Appendix T. Santa Fe to Weymouth WTP Alignment Evaluation Memo

DRAFT

SANTA FE TO WEYMOUTH WTP ALIGNMENT EVALUATION

Regional Recycled Water Program

B&V PROJECT NO. 191628

PREPARED FOR

Metropolitan Water District of Southern California

30 APR 2020

BLACK & VEATCH

Table of Contents

LIST OF TABLES

LIST OF FIGURES

1.0 Introduction

In order to improve water supply reliability in Southern California, the Metropolitan Water District of Southern California (Metropolitan) is studying the feasibility of a Regional Recycled Water Program (RRWP). The RRWP would utilize advanced water treatment (AWT) processes to purify secondary treated effluent from the Sanitation Districts of Los Angeles County's (LACSD) Joint Water Pollution Control Plant (JWPCP) in Carson, California and then pump the advanced treated water to select locations for beneficial reuse. The full implementation of the distribution system would include construction of the AWT plant, a new regional distribution system, pump stations, and various additional appurtenant facilities to convey advance treated water for beneficial reuse. Additional smaller diameter piping would be required for laterals and connections to discharge locations.

As originally envisioned, the RRWP would convey the advanced treated water to select locations to recharge the groundwater basins within Metropolitan's service area, including the Santa Fe Spreading Grounds (SFSG). Black and & Veatch Corporation (Black & Veatch) and CDM Smith prepared a Draft Conceptual Design Report (CDR) documenting the conceptual design for the conveyance system facilities of the RRWP.

Currently, the California State Water Resources Control Board (SWRCB) is working to develop regulations that if propagated would allow for direct potable reuse (DPR) of advanced treated wastewater effluent. DPR could consist of either introduction of purified, recycled water directly into a potable water supply distribution system, or into the raw water supply upstream of a water treatment plant. The timeline for approval of DPR regulations and the details of that approval remains uncertain.

Many options exist on how to best incorporate the advanced treated water into Metropolitan's potable water distribution system depending on the exact requirements of the SWRCB's regulations (i.e., raw water augmentation or direct connection). At this time, Metropolitan has identified the most promising option would be to pump all 150 million gallons per day (mgd) of advanced treated water produced at the AWT to the SFSG and then pump some percentage of the advanced treated water on to the F.E. Weymouth Water Treatment Plant (FEWWTP). Under this scenario, the RRWP system would deliver up to 150 mgd to the SFSG until such time as the DPR regulations were implemented. At that time, a new pumping plant, or plants, and pipeline would be constructed to convey the water on to the Weymouth WTP.

This technical memorandum (TM) documents the evaluations completed comparing the alignment alternatives identified from the SFSG to the FEWWTP. An assessment of hydraulics and pumping requirements is not included within this TM.

1.1 BACKGROUND

The CDR prepared by Black & Veatch in September of 2018 focused on a conveyance system designed to deliver the advanced treated water to multiple spreading grounds and injection well locations, the farthest of which were the SFSG and the Orange County Spreading Grounds. At the time, the conveyance system was envisioned to split the flows with up to 80 mgd being conveyed to the SFSG and up to 60 mgd being conveyed to the Orange County Spreading Grounds. Figure 1-1 presents a schematic representation of the conveyance system focused on in the CDR.

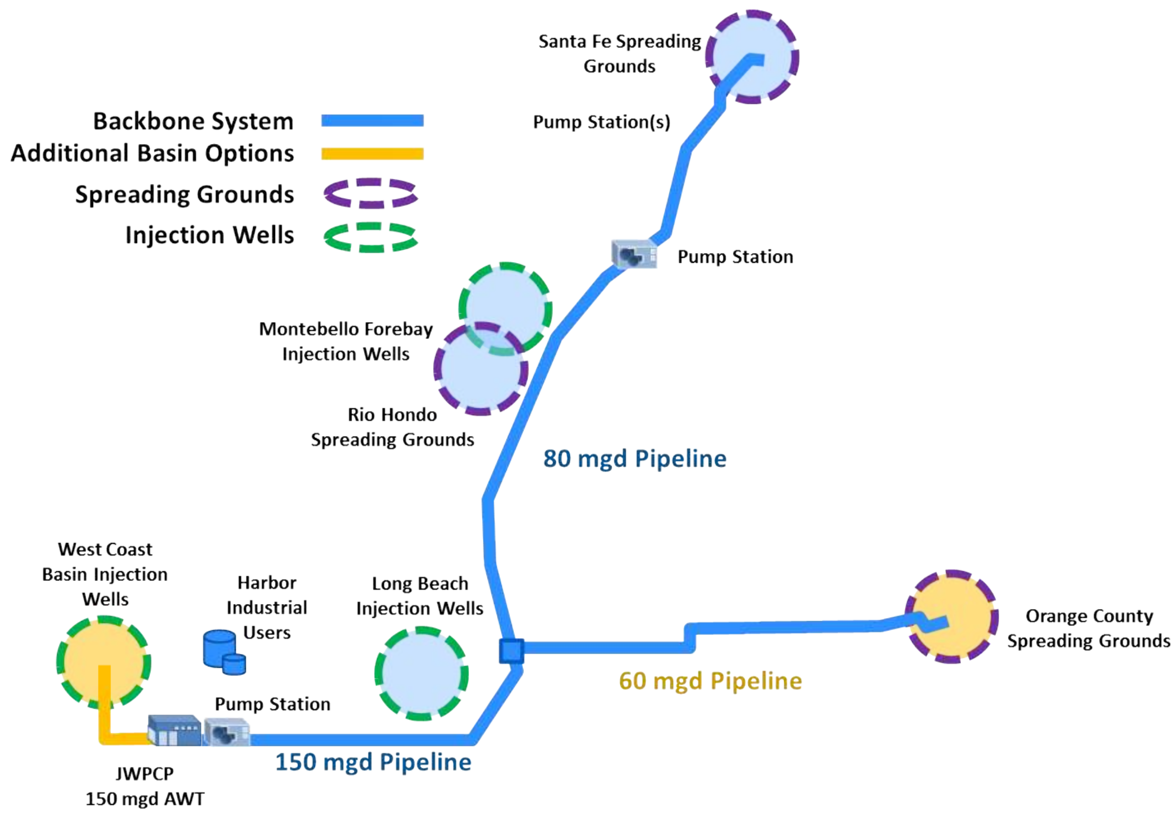


Figure 1-1: 2018 CDR Conveyance Configuration

In February of 2019, Metropolitan issued the Conceptual Planning Studies Report which presents the results of further technical studies related to the RRWP conducted by Metropolitan. The studies presented in the Conceptual Planning Studies Report evaluate, among other things, program phasing and the potential for the program to accommodate DPR. The report recommended that the organization should "proceed with the environmental review process" for the RRWP.

In July of 2019, Metropolitan issued the RRWP White Paper No. 1 – Program Implementation and Delivery. In this document, Metropolitan examines two items in detail: (1) what are the implementation options to accelerate the program to construct conveyance facilities and/or make initial deliveries of purified water and (2) how would Metropolitan proceed in developing raw water augmentation opportunities if DPR regulations get promulgated.

Through the studies mentioned above, a proposed implementation strategy emerged that would provide the flexibility to adapt the initial system for DPR, allow phasing opportunities to accelerate the program, and facilitate the addition of expanded treatment capacity at the JWPCP beyond the initial 150 mgd. The proposed approach includes an AWT plant sized to meet near-term existing and planned future demands and a "backbone conveyance system" (Backbone System) that is sized convey the full 150 mgd from the AWT plant in Carson to the SFSG through an 84-inch pipeline.

The Backbone System forgoes the pipeline branch to the Orange County Spreading Grounds (OC Reach) described as part of the "Preferred Alignment" in the CDR from the initial phases of the program. Instead, the full 150 mgd would be conveyed to SFSG. Raw water augmentation for DPR can be incorporated into the Backbone System by adding at least one additional pumping station and pipeline from the SFSG to Metropolitan's FEWWTP. Once the Backbone System is connected to the FEWWTP, Metropolitan could utilize their existing distribution system (the Yorba Linda Feeder and East Orange County Feeder Number 1) to convey water to the Orange County Spreading Grounds. In this scenario, a new pipeline to Orange County would not be required.

Another benefit of the Backbone System is that it would allow for a potential interconnection to other purified water reuse programs. The City of Los Angeles is in the early stages of a program to reuse 100% of the available secondary effluent at the Hyperion Water Reclamation Plant by 2035. By building a Backbone System that is sized to convey 150+ mgd to the SFSG with the ability to connect to the Weymouth WTP, it provides partnership opportunities to make dual use of the facilities for both Metropolitan and the City of Los Angeles.

Figure 1-2 presents a schematic of the Backbone System with future options to incorporate DPR.

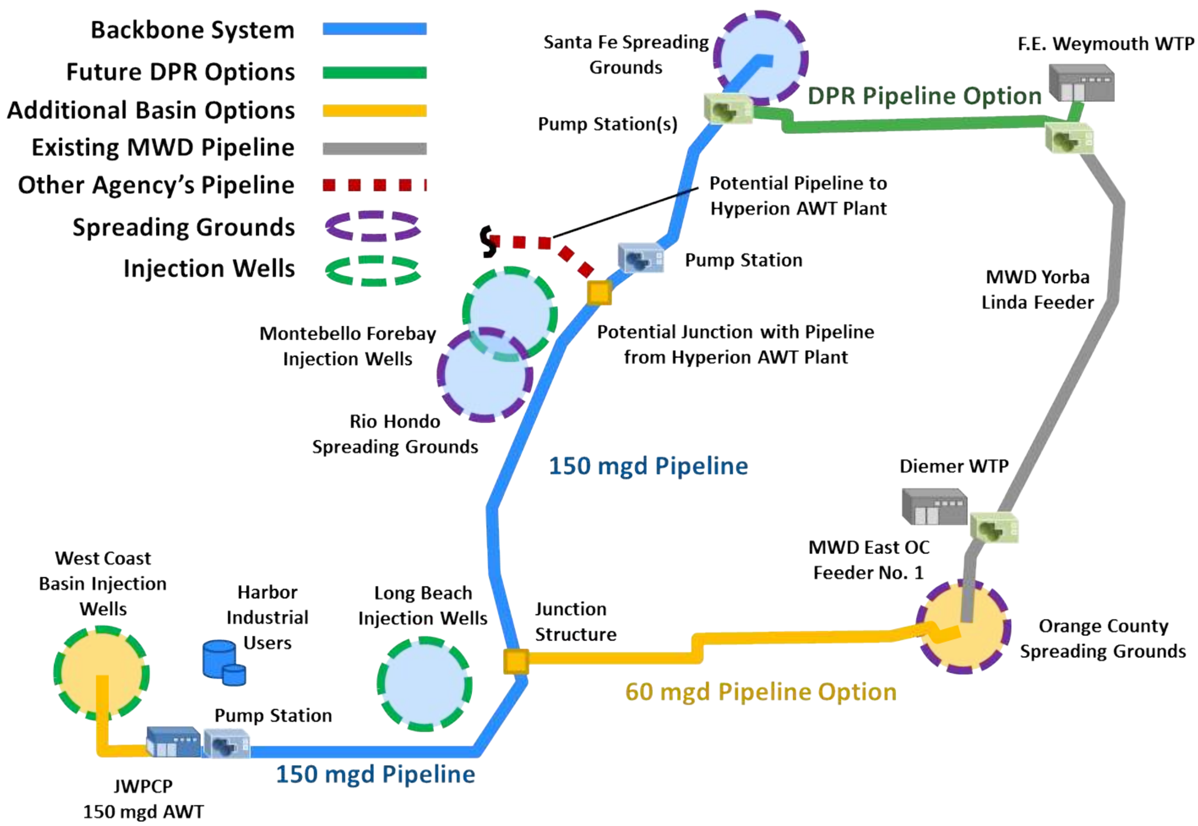


Figure 1-2: Proposed Regional Recycled Water Program

1.2 METHODOLOGY

Metropolitan retained Black & Veatch to conduct an alignment evaluation for the conveyance pipeline connecting the SFSG to the F.E. Weymouth WTP. The scope of work for this study includes utilizing the evaluation process developed as part of the CDR to identify and rank the alignment alternatives. A multi-step approach for conducting the alignment evaluation was used, as outlined herein:

- Verification of Alignment Alternatives. Metropolitan identified multiple potential alignments to construct a pipeline from the Backbone Alignment near the SFSG to the FEWWTP. These alternatives were reviewed to verify their suitability for construction of a large diameter pipeline. Data was collected on the feasible alignment alternatives to provide the basis of comparison. Data collection included:

- A desktop review of available electronic and paper documents relating to the project area.
- Field visits to confirm above grade features along the alignments.

- Scoring and Weighting System. A comprehensive system to compare and rank alignment alternatives was developed to rank potential alignment alternatives as part of the CDR. The system includes criteria to assess the various alternatives on factors relating to construction risk, community impacts, and cultural and biological impacts. The scoring and weighting system developed for the CDR was reviewed to confirm its suitability to assess the alignments from the SFSG to the FEWWTP. Additional weighting scenarios were provided by Metropolitan's internal stakeholders after the development of the CDR. These weighting scenarios were used as a sensitivity analysis to determine the impact changes in the weights have on the results of the evaluation.
- Alignment Evaluation. Using the data collected, the alignment alternatives were ranked based on their ability to satisfy the Project's objectives.

1.3 PIPELINE CORRIDORS

Metropolitan identified various alignment alternatives to convey water from the Backbone System (as identified in the CDR) near the SFSG to the FEWWTP. These alignment alternatives were provided to Black & Veatch and serve as the basis of this alignment evaluation. For the purposes of this evaluation, the alternatives were divided into a number of separate "segments." Each segment starts and ends at a junction with another segment and can be combined to form the various alignment options from the SFSG to the FEWWTP.

The alignments identified by Metropolitan generally follow four east-west corridors between the SFSG and the FEWWTP. Three of these east-west corridors are generally within existing public street rights of way. In addition to these roadways, a potential alignment utilizing Metropolitan's existing Glendora Tunnel is considered. This corridor allows for the construction of a new transmission pipeline north in roads to the westerly end of the Glendora Tunnel. The Glendora Tunnel would be re-purposed to convey water east to the FEWWTP.

These four main east-west corridors form the basis for the pipeline segments.

- Gladstone Street
- Arrow Highway
- Cypress Street
- Glendora Tunnel

Figure 1-3 presents the segments assessed in this evaluation. Descriptions of the four main east-west corridors are provided in the sections that follow.

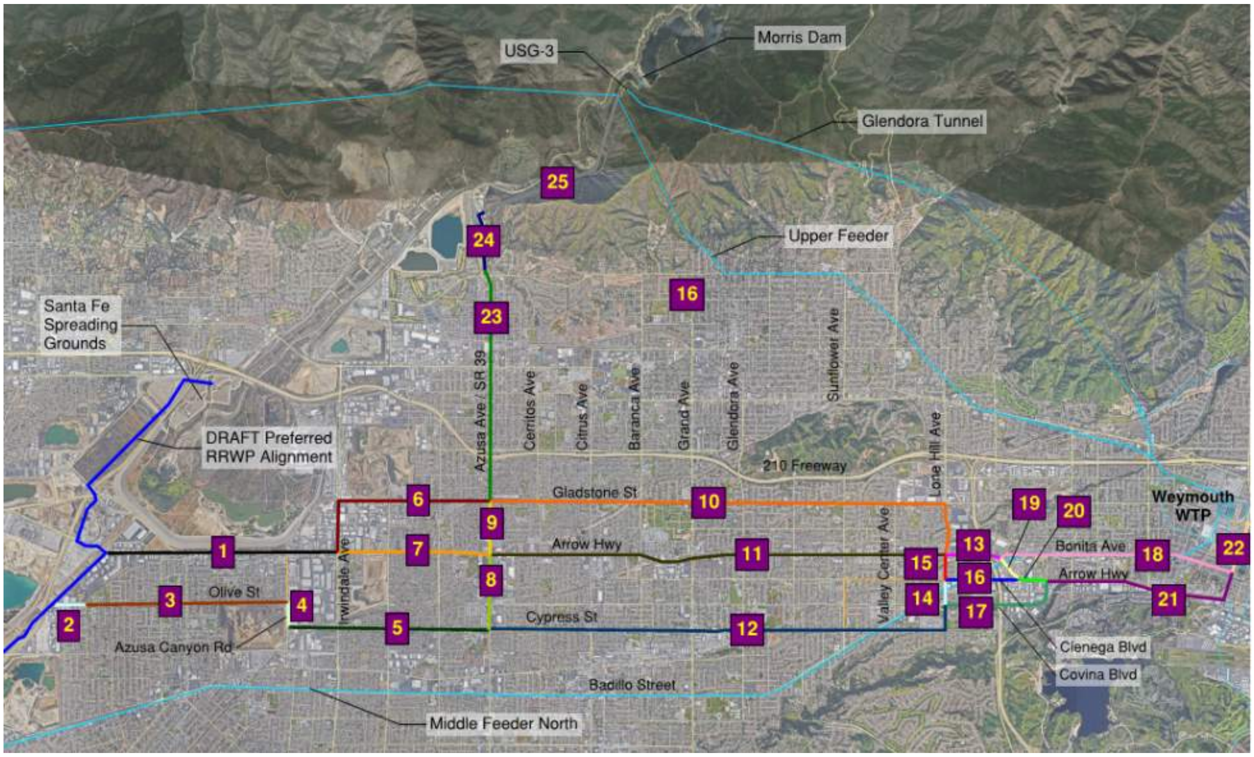


Figure 1-3: Pipeline Segment Alternatives

1.3.1 Gladstone Street

This corridor is located in the existing public rights of way of Gladstone Street and is the northern most east-west road being considered. Gladstone Street primarily has four lanes of traffic without an improved center median. The street is 60 to 65-feet wide from curb to curb. Residential access is primarily from side streets or frontage roads with only sporadic residential driveways directly on the street. The remainder of the road is primarily industrial or light commercial. Seven (7) schools are located on Gladstone Street.

1.3.2 Arrow Highway

This corridor is located in the existing public rights of way of Arrow Highway and is the middle of the east-west roads being considered. West of Valley Center Avenue, Arrow Highway primarily has four lanes of traffic with an intermittent raised but not landscape. East of Valley Center Avenue, Arrow Highway primarily has six lanes of traffic with an improved center median with mature trees. The street is 80 to 90-feet wide from curb to curb. Residential access is primarily off side streets or frontage roads. The majority of the road is industrial or commercial. One (1) school is located on Arrow Highway.

1.3.3 Cypress Street

This corridor is located in the existing public rights of way of Cypress Street and is the southernmost of the east-west roads being considered. To reach Cypress Street, the corridor follows a combination of Olive Street and Azusa Canyon Road. From there, Cypress Street primarily has four lanes of traffic, with two lanes of traffic east of Valley Center Avenue. Cypress Street has no improved center median and is 60 to 65-feet wide (curb to curb). Cypress Street is heavily residential and has many driveways directly on the street. Although there appears to be viable corridors available, many regional utilities are also found in Cypress Street. Eight (8) schools are located on Cypress Street.

1.3.4 Glendora Tunnel

This corridor consists of using Metropolitan's existing Glendora Tunnel to pump water east to the FEWWTP, reverse of its current operation. The Glendora Tunnel's primary purpose is to convey raw water from the Rialto Pipeline and / or the Upper Feeder to the USG-3 service connection for discharge to the San Gabriel Canyon and ultimately to spreading basins for groundwater recharge. With the implementation of the RRWP, the Upper San Gabriel Municipal Water District (USGMWD) would receive their replenishment water via the RRWP at the SFSG, just downstream of the USG-3 service connection, in lieu of from USG-3. Therefore, the Glendora Tunnel could be available for this new use.

To reach the Glendora Tunnel, the corridor would follow either Arrow Highway or Gladstone Street to Azusa Avenue. From there, the corridor would traverse north on Azusa Avenue and then north on Ranch Road. Metropolitan, and their consultant McMillan Jacobs and Associates, evaluated three options to construct the pipeline from Ranch Road to the terminus of the Glendora Tunnel. The first option was to open cut the pipeline within San Gabriel Canyon Road and Old San Gabriel Canyon Road and then tunnel the final 4,400-feet. The second option involved two tunnels with 2,000-feet of open cut on Old San Gabriel Canyon Road between them. The third option was a single tunnel for the entire stretch.

For the purposes of this analysis, the third option, a single tunnel, was assumed for this section due to its lower overall community impact as compared to the other options. San Gabriel Canyon Road is also a portion of State Route 39 and is the primary point of access for the Mountain Cover residential development located along this corridor. Further, Old San Gabriel Road serves as access to the Azusa River Wilderness Park, a popular hiking and pedestrian trail. By tunneling this section, it minimizes the impacts on the community.

The corridor then follows the Glendora Tunnel east to the La Verne Pipeline. The La Verne Pipeline connects the east portal of the Glendora Tunnel to the Upper Feeder Junction Structure, approximately 2 miles to the south. The Upper Feeder Junction Structure has the ability to blend the advanced treated water with Colorado River water and State Water Project water before discharging into the FEWWTP's inlet conduit.

Metropolitan conducted a preliminary hydraulic analysis and determined that the hydraulic grade line required to pump water east through the Glendora Tunnel is less than the design hydraulic grade for the tunnel. Therefore, this study assumes that no structural improvements to the tunnel are required. This assumption should be confirmed during subsequent evaluations.

2.0 Verification of Pipeline Alternatives

Black & Veatch performed an independent assessment of the pipeline alignment alternatives provided by Metropolitan. Goals associated with the assessment of alignment alternatives included:

- Verifying the alignments identified are suitable for the construction of a new large diameter pipeline. To minimize the construction zone required the following assumptions were used: 1) all trenching was assumed to be vertically shored, with a minimum of 10-foot depth to top of pipe to reduce utility conflicts, 2) excavation and pipe laying equipment would be positioned ahead of or behind the pipe being placed, 3) trenching would be positioned on one side of the construction zone such that deliveries, hauling, and staging could occur on the other side, and 4) stockpiling of excavated soils would occur at temporary off-site locations.
- To minimize community impacts, construction would be in wider collector-type streets that could accommodate the minimum work zone construction width and still maintain two-way traffic flow. Trenchless construction methods would be utilized to cross freeways, railroads, large flood control / storm drain channels, and major intersections. Alignments were reviewed for the presence of large diameter utilities from other regional entities – such as sewers, storm drains, etc. – to ensure a sufficiently wide corridor was available for the proposed pipeline.
- The Project study area was reviewed to identify biological constraints from the construction of a large diameter pipeline, including impacting wetlands, critical habitats, and cultural resources. Potential segments requiring construction through sensitive habitats or wetlands were not considered.

The alignment alternatives presented were all confirmed as feasible for construction of a large diameter pipeline. The following sections present the data collected on the alignment alternatives.

2.1 DATA GATHERING

Metropolitan collected data in both electronic and paper format from agencies, municipalities, and regional utilities in the Project study area. This data was provided to Black & Veatch for the preparation of the CDR and serves as the basis of the information used to compare and rank the alternatives from the SFSG to the FEWWTP. Black & Veatch independently gathered data to supplement the data provided by Metropolitan. Data gathering involved a desktop review of electronic and paper records and field visits to confirm the at grade characteristics of each alternative. This section documents the data gathered on each alternative.

2.1.1 Desktop Analysis

Available electronic and paper records were reviewed and logged into a GIS database to help compare and assess each alternative.

The desktop evaluations allowed for an expedited review and comparison of pipeline alignment alternatives. The desktop evaluations allowed the identification of potential obstacles and screen alignments that included high risk construction areas. Also, readily discernible were areas that presented potential community related concerns, such as schools, hospitals, and police and fire stations.

The type of information collected is shown in Table 2-1.

Table 2-1 Electronic Information Collected

INFORMATION COLLECTED	
Contaminated sites (soil) Environmental constraints (critical habitats) Historical landfills Jurisdictional boundaries Land use	Streets Regional utility records (LACSD, LACFCD, MWD) Faults Historical groundwater depths Waters of the US and State

2.1.1.1 Existing Utilities

The existing utility information collected by Metropolitan included regional utilities, such as the Sanitation Districts of Los Angeles County (LACSD), Los Angeles County Flood Control District (LACFCD), and Metropolitan's own distribution system. Regional utilities were deemed to be indicative of the feasibility of an alignment for the construction of the new distribution system.

For this conceptual-level study, utility information was not collected from the local cities and municipalities along the alignment alternatives. This information should be collected during subsequent evaluations to verify the feasibility of the preferred alignment. Telecommunications and electrical utilities were not evaluated in this study but were provided in the GIS database to be referenced in future design phases.

2.1.2 Field Investigations

Black & Veatch performed field reconnaissance to confirm the findings of the desktop evaluation. The reconnaissance was limited to visible at, or above, grade features. During the visits, actual field conditions and constructability concerns were further identified and evaluated. Attention was given to identifying high risk construction areas. Visible utilities, land use restrictions, traffic flow, and environmental concerns were noted.

2.2 SUMMARY

A summary of the data gathered on each segment is provided in Appendix A.

3.0 Alignment Alternative Evaluation

This chapter documents the technical analysis to support the ranking of alignment alternatives, including the completion of the following tasks.

- Validate Scoring and Weighting System. In this step, the scoring and weighting system developed for the CDR was reviewed to ensure its applicability to assess the alignment alternatives from the SFSG to the FEWWTP.
- Conduct a Coarse Screening. A coarse screening focusing on relatively short sections where two or more pipeline route options were available was conducted to reduce the number of alignment combinations possible.
- Develop Full Alignment Alternatives. The pipeline segments identified in Section 2.0 were combined into full alignment alternatives starting at the Backbone System near the SFSG and ending at the FEWWTP.
- Conduct Screening Analysis. Compare the alignment alternatives to achieve a ranking.

Figure 3-1 presents the evaluation methodology completed as part of this study.

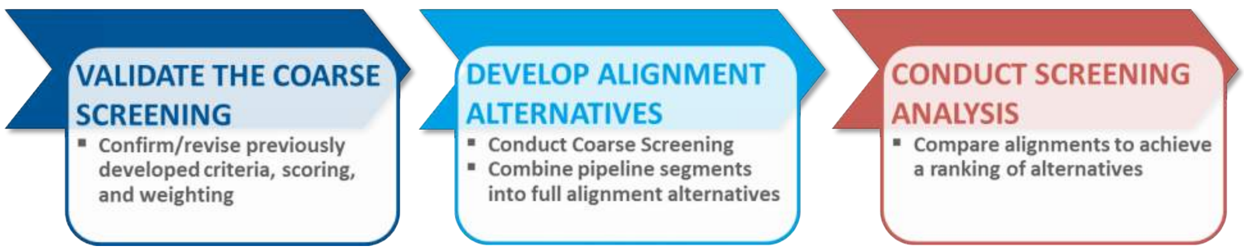


Figure 3-1: Evaluation Methodology

3.1 VALIDATE THE SCORING AND WEIGHTING SYSTEM

The CDR established a robust evaluation process consisting of a scoring and weighting system reflecting Metropolitan's goals for the RRWP and a comprehensive set of screening criteria. This study used the evaluation methodology established for the CDR as the basis to assess the alignment alternatives being considered.

The following sections present the scoring and weighting system developed for the CDR and discusses the revisions, if any, made to the systems for the evaluation of the alignments to FEWWTP.

3.1.1 Scoring System

The CDR used a scoring system to quantitatively compare the alignments based upon their ability to satisfy the project's objectives using a 1 to 5 scale. Lower scores represent more favorable comparisons, while higher scores are indicative of unfavorable comparisons. This same scoring system was used in this evaluation.

A low rating score (i.e., a score at or near "1") signaled that the impacts related to the evaluation criterion either do not exist or would occur at a rate that is generally less than the average occurrence for that alignment. Conversely, a rating score of "5" indicated the alignment alternative would not compare favorably to the screening criteria and the impacts related to the criterion would occur at a rate that is generally higher than average. In some cases, it was appropriate to calculate a weighted or proportional score between 1 and 5 for screening criteria that do not score uniformly along an entire alignment.

3.1.2 Evaluation Criteria

The CDR organized the evaluation criteria into three major categories: factors that would add construction risk, factors that would result in social and community impacts, and factors that would potentially have biological impacts. The screening criteria were generally consistent with the Project description information required for preparation of CEQA and NEPA review.

All the screening criteria established in the CDR remain applicable to the development and ranking of alignments from the SFSG to the FEWWTP. One update was made to the way the remaining criteria factors were scored. Rating scores based upon a hard count (i.e., number of major utilities crossed, number of public facilities passed, number of major intersections, etc.) were updated to reflect the statistical data of the alternatives being considered. The scoring system for the remaining criteria remained unchanged.

Table 3-1 presents the evaluation criteria and the rating scores used in this study. For a detailed description of the evaluation criteria, see Appendix B.

Table 3-1: Evaluation Criteria: Scoring Summary Matrix

FACTOR	UNIT	SCORING RANGE		
		(1)	(3)	(5)
Construction Risk				
Major Utility Crossings	#	<22	>=22 and <=36	>36
Trenchless Construction Crossings	% of length	<5%	>=5% and <=15%	>15%
Depth to Ground Water	% of length	<30%	>=30% and <=50%	>50%
Alignment Length	% of shortest alignment	0-5% of the shortest alignment	>=5% and <=20% of the shortest alignment	>20% of the shortest alignment
Seismic Hazard	Y/N	N	-	Y
Soil Contamination Risk	#	<4	>=4 and <=5	>5
Ease of Operation and Maintenance	Score	SCE/LACFCD Easements	Streets	River/Caltrans
Community Impacts				
Park and Recreation Areas	% of length	No Park	-	In a Park
Public Facilities	#	<3	>=3 and <=10	>10

FACTOR	UNIT	SCORING RANGE		
		(1)	(3)	(5)
Traffic Impacts	% of trench	No Streets	Collector / Local Road	Arterial Road / Full Road Closure Req.
Center Medians	% of length	<18%	>=18% and <=35%	>35%
Major Intersection Crossings	#	<15	>=15 and <=22	>22
Residential / Minor Commercial	% of length	<15%	>=15% and <=30%	>30%
Biological Impacts				
Waters of the U.S. / Wetland Crossings	% of length	<5%	>=5% and <=15%	>15%
Critical Habitats	Y/N	Does not cross a known critical habitat	n/a	Crosses a known critical habitat

3.1.3 Weighting Factors

To account for the difference in relative importance that each evaluation factor contributes to the overall evaluation, weighting factors reflecting Metropolitan's priorities for the RRWP were assigned at the category level and also to each screening factor. The weighting factors developed during the CDR were reviewed as part of this task to ensure their applicability to the assessment of the alternative alignments to FEWWTP.

Two weighting scenarios were considered in the CDR. Scenario A places an emphasis on the construction risk category, while Scenario B emphasizes the community and biological categories. Both scenarios were considered as part of this evaluation to illustrate how changes to the weights could impact the evaluation.

Table 3-2 summarizes the weighting factors used in this analysis.

After the development of the CDR, workshops were held with Metropolitan's internal stakeholder groups to review the evaluation process. The internal stakeholders (Environmental Planning Section, Real Property Section, and External Affairs Section) provided additional weighting scenarios to consider. These weighting scenarios were used as a sensitivity analysis to check the impact changes to the weights would have on the results of the evaluation. These additional weighting scenarios are presented in Table 3-3.

The results of the analysis considering the additional weighting scenarios were presented to the internal stakeholders at a workshop. The additional weighting scenarios did not change the results of the analysis that are presented in Chapter 4.0. This confirmed the results of the evaluation.

Table 3-2: Evaluation Criteria: Weighting Factors Matrix

Evaluation Factor	Scenario A (Emphasis on Construction Risk)		Scenario B (Emphasis on Community and Biological)	
Construction Risk	**Category Weight:**	**60%**	**Category Weight:**	**30%**
	Factor Weight	**Factor Score**	**Factor Weight**	**Factor Score**
Major Utility Crossings	20.0%	12.00	20.0%	6.00
Trenchless Construction	20.0%	12.00	20.0%	6.00
Groundwater Conditions	5.0%	3.00	5.0%	1.50
Alignment Length	25.0%	15.00	25.0%	7.50
Seismic Hazard	5.0%	3.00	5.0%	1.50
Soil Contamination Risk	5.0%	3.00	5.0%	1.50
Ease of Operations/ Accessibility	20.0%	12.00	20.0%	6.00
Social and Community	**Category Weight:**	**30%**	**Category Weight:**	**55%**
Parks/Recreation Areas	5.0%	1.50	5.0%	2.75
Public Facilities	20.0%	6.00	20.0%	11.00
Traffic Impacts	20.0%	6.00	20.0%	11.00
Street/Median Improvements	20.0%	6.00	20.0%	11.00
Major Intersections	15.0%	4.50	15.0%	8.25
Residential/Minor Commercial	20.0%	6.00	20.0%	11.00
Biological and Cultural	**Category Weight:**	**10%**	**Category Weight:**	**15%**
Waters of the US and State	20.0%	2.00	20.0%	3.00
Critical Habitats	40.0%	4.00	40.0%	6.00

Table 3-3: Additional Weighting Scenarios Provided from Metropolitan's Internal Stakeholders

Criteria	Internal Stakeholder Input					
	Environmental Group		Real Property		External Affairs	
	A	B	A	B	A	B
Construction Risk		**30%**	**60%**	**30%**	**55%**	**30%**
Major Utilities	N/A	5%	25%	25%	20%	20%
Trenchless Construction	N/A	10%	20%	20%	20%	20%
Depth to Groundwater	N/A	25%	5%	5%	5%	5%
Total Alignment Length	N/A	0%	20%	20%	25%	25%

Criteria	Internal Stakeholder Input					
	Environmental Group		Real Property		External Affairs	
	A	B	A	B	A	B
Seismic Hazard	N/A	5%	5%	5%	5%	5%
Contaminated Soils Risk	N/A	25%	5%	5%	5%	5%
Ease of O&M	N/A	15%	15%	15%	15%	15%
Scour Potential	N/A	15%	5%	5%	5%	5%
Social and Community		**20%**	**30%**	**60%**	**35%**	**55%**
Parks & Rec Areas	N/A	29%	5%	5%	5%	5%
Public Facilities	N/A	29%	20%	20%	15%	15%
Road Category & Traffic Impact	N/A	7%	20%	20%	30%	30%
Center Medians	N/A	7%	10%	10%	10%	10%
Major Intersections	N/A	6%	15%	15%	15%	15%
Residential/ Minor Commercial	N/A	22%	30%	30%	25%	25%
Environmental / Biological		**50%**	**10%**	**10%**	**10%**	**15%**
Waters of the US and State	N/A	20%	20%	20%	20%	20%
Critical Habitats and Listed Species	N/A	80%	80%	80%	80%	80%

3.1.4 Decision Model

The CDR developed a spreadsheet-based decision model that takes the raw data collected for each alternative and applies the scoring methodology and weighting factors described in the previous sections to determine a comparative scoring. The same decision model was used for this Study. The results for the alignment alternatives are summarized in the following sections.

3.2 COARSE SCREENING

The coarse screening process evaluated relatively short segments, or combinations of segments, where two or more pipeline route options were available to determine the preferred route. In many cases, these comparisons evaluated routes along parallel and adjoining streets. The following sections present the results of the screening. For details on the scoring matrix, see Appendix A.

3.2.1 Comparison 1 – Cienega Blvd vs. Arrow Highway

The first area evaluated for the coarse screening was the preferred route to get from the intersection of Arrow Highway and Lone Hill Avenue to the intersection of Cienega Boulevard and Arrow Highway. As can be seen on Figure 3-2, two options were considered. Option 1 followed Arrow Highway the entire way (Segments 13 and 19), while Option 2 followed Lone Hill Avenue and Cienega Boulevard (Segments 15 and 16).

Figure 3-2 presents the two options considered for Comparison 1.

Option 1 – Arrow Highway scored more favorably due to its shorter overall length, fewer major utility crossings, and less residential impacts.

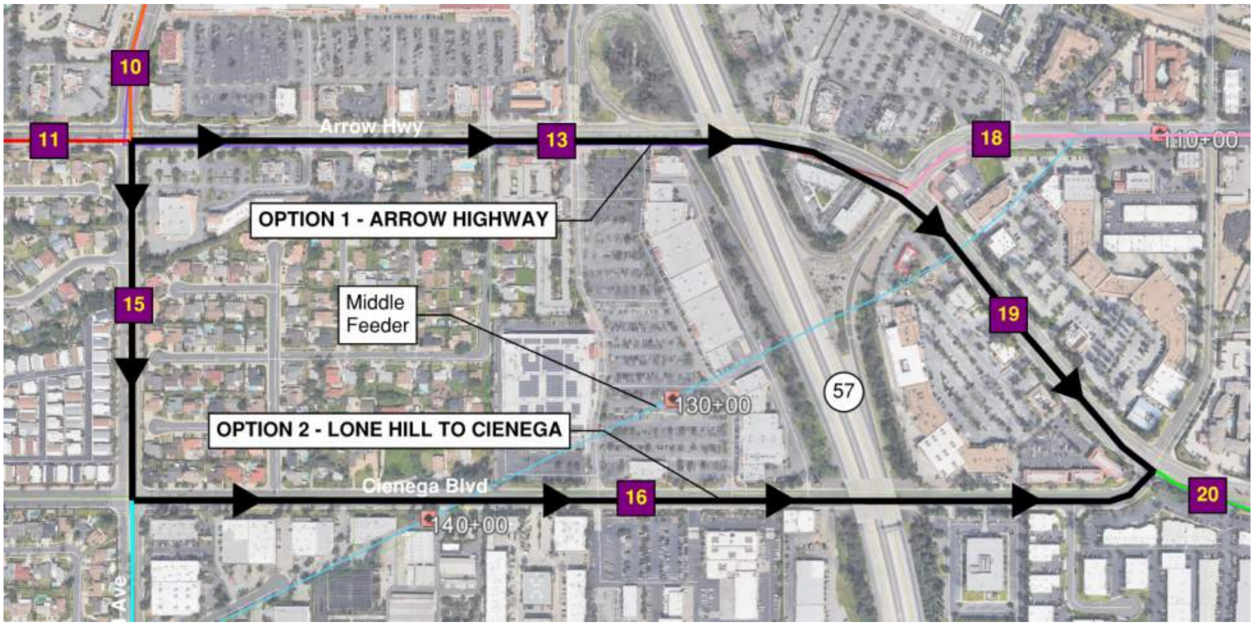


Figure 3-2 Coarse Screening Comparison 1 – Cienega Blvd vs. Arrow Highway

3.2.2 Comparison 2 – Bonita Avenue vs. Arrow Highway

The second comparison of the coarse screening was between the intersection of Arrow Highway and Bonita Ave to the intersection of Wheeler Avenue and Bonita Ave. As can be seen on Figure 3-3, two options were considered. Option 1 followed Bonita Ave the entire way (Segment 18), while Option 2 followed Arrow Highway to Wheeler Ave (Segments 19, 20, and 21).

Figure 3-3 presents the two options considered for Comparison 2.

Figure 3-3 Coarse Screening Comparison 2 – Bonita Avenue vs. Arrow Highway

Bonita Avenue is home to the improved downtown San Dimas district, which includes downtown shops, large walkable sidewalks, and a narrow two-lane street. Further, Bonita Avenue already contains Metropolitan's Middle Feeder, another large diameter pipeline. To minimize the impacts to the community to the extent possible, this study has assumed that the entire downtown section would need to be constructed in a tunnel. While Option 2 was longer, because of these reasons it scores more favorably. This is in part due to it avoiding the highly impactful downtown San Dimas located on Bonita Avenue.

3.2.3 Comparison 3 – Gladstone Street vs. Arrow Highway

The third comparison of the coarse screening was between the intersection of Arrow Highway and Irwindale Ave to the intersection of Gladstone Street and Azusa Avenue / SR 39. As can be seen on Figure 3-4, two options were considered. Option 1 followed Irwindale Avenue and Gladstone Street (Segment 6), while Option 2 followed Arrow Highway to Azusa Avenue /SR 39 (Segments 7 and 9)

Figure 3-4 presents the two options considered for Comparison 3.

Option 1 – Irwindale Avenue to Gladstone Street scored more favorably due to its lesser traffic impacts. Arrow Highway and Azusa Ave/SR 39 are both primary arterial roadways and principal trafficways through the area. Gladstone Street appears to be less traveled than Arrow Highway. Further, with the landfill on the north side of Gladstone Street, there are fewer overall driveways along this stretch.

Figure 3-4 Coarse Screening Comparison 3 – Gladstone Street vs. Arrow Highway

3.2.4 Summary of Coarse Screening Results

As mentioned earlier, the goal of the coarse screening was to screen out the worse scoring and correspondingly higher risk segments, thereby reducing the number of possible alignment iterations to a more manageable number.

Table 3-4 summarizes the results of the coarse screening, including the segments that were screened from consideration for the assessment of full alignments.

Table 3-4 Summary of Coarse Screening Results

COMPARISON	DESCRIPTION	JUSTIFICATION
Comparison 1 - Cienega Blvd vs. Arrow Highway	For the Gladstone Street and Arrow Highway Corridors, Segments 13 and 19 that remain on Arrow Highway are preferred over Segments 15 and 16, which follow Lone Hill Avenue and Cienega Boulevard	Arrow Highway (Segments 13 and 19) scored more favorably due to its shorter overall length, fewer major utility crossings, and less residential impacts
Comparison 2 – Bonita Ave vs. Arrow Highway	Segments 19, 20, and 21, which would follow Arrow Highway to Wheeler Ave, are preferred to Segment 18, which would be in Bonita Ave	Arrow Highway scored more favorable due to it avoiding the highly impactful downtown San Dimas district located on Bonita Avenue
Comparison 3 – Gladstone St vs. Arrow Highway	Segment 6, which would follow Irwindale Avenue and Gladstone Street was preferred to Segments 7 and 9, which would follow Arrow Highway to Azusa Avenue /SR 39	Arrow Highway and Azusa Ave /SR 39 are both primary arterial roadways and principal trafficways through the area. Gladstone Street offers a route with significantly less impact on traffic

3.3 FULL ALIGNMENT ALTERNATIVES

Building upon the outcomes of the coarse screening, four full pipeline alignment alternatives were identified by combining the resulting segments. The four full pipeline alignment alternatives are described in the following sections.

3.3.1 Alignment 1 – Gladstone Street

Alignment 1 would generally be located within Gladstone Street and is described as follows. Alignment 1 would start in Arrow Highway heading east. At Azusa Avenue / SR 39, Alignment 1 would turn north and then east at Gladstone Street. From there, Alignment 1 is in Gladstone Street for 4.5 miles before turning south in Lone Hill Avenue, west in Arrow Highway and finally north in Wheeler Avenue. Alignment 1 is comprised of the following segments: 1, 6, 10, 13, 19, 20, 21, and 22.

Gladstone Street is a mix of industrial and residential with most residential driveways located off frontage roads or side streets with only an occasional driveway directly on Gladstone Street. Gladstone Street is considered a collector road and is one of the primary continuous east-west roadways in the area.

Figure 3-5 presents a photo of a typical section on Gladstone Street.

Figure 3-5 Photo on Gladstone Street looking East – Typical View

3.3.2 Alignment 2 – Arrow Highway

Alignment 2 would generally be located within Arrow Highway and is described as follows. Alignment 2 would start in Arrow Highway and travel east all of the way to Wheeler Avenue. Alignment 2 is comprised of the following segments: 1, 7, 11, 13, 19, 20, 21, and 22.

Alignment 2 is the most direct route from the SFSG to the FEWWTP.

Arrow Highway is mostly comprised of minor commercial and industrial land uses. Residential areas off of Arrow Highway utilize frontage roads for driveway access. Arrow Highway is considered an arterial road and is one of the primary east-west roadways in the area.

Figure 3-6 presents a photo of a typical section on Arrow Highway.

3.3.3 Alignment 3 – Cypress Street

Alignment 3 would generally be located within Cypress Street and is described as follows. Alignment 3 would begin in a parking lot/ existing utility easement traveling east to get from the Backbone System (as identified in the CDR) on Rivergrade Road to Olive Street. The utility easement has existing LACFCD pipes and overhead SCE transmission lines within it and would likely requiring tunneling to avoid impacts to existing facilities. The alignment would then follow Olive Street to Azusa Canyon Road before turning east in Cypress Street. Alignment 3 would follow Cypress Street for 6.5 miles before turning north in Lone Hill Avenue, then East in Covina Boulevard, east again in Arrow Highway and finally north in Wheeler Avenue. Alignment 3 is comprised of the following segments: 2, 3, 4, 5, 12, 17, 21, and 22.

Figure 3-6 Photo on Arrow Highway looking East – Typical View

Cypress Street is heavily residential with driveways commonly directly on the street. Due to the residential nature of the area, overhead power lines cross the street at a higher rate than the other alternatives considered.

Figure 3-7 presents a photo of a typical section on Cypress Street.

Figure 3-7 Photo on Cypress Street looking West – Typical View

3.3.4 Alignment 4 – Azusa Avenue / SR 39 to Glendora Tunnel

Alignment 4 would follow Arrow Highway and then turn north in Irwindale Avenue. At Gladstone Street the alignment would turn east before turning north in Azusa Avenue / SR 39, which it is on for 2.75 miles. Upon reaching Ranch Road, Alignment 4 would turn right and continue heading north to the potential tunnel portal south of the City of Azusa Water Filtration Plant. From there, Alignment 4 would tunnel to the end of the Glendora Tunnel located near Morris Dam. Alignment 4 would then repurpose the Glendora Tunnel to convey water towards F.E. Weymouth WTP. Alignment 4 is comprised of the following segments: 1, 6, 23, 24, 25, and the Glendora Tunnel (known as Segment 26).

South of the 210 Freeway, Azusa Avenue is considered a primary arterial road and is one of the principal north-south trafficways with large on and off ramps to the 210 Freeway in the north and the 10 Freeway to south.

North of the 210 Freeway, Azusa Avenue transitions into heavily residential areas. Between the 210 Freeway and Fifth Street, most of the driveways in the residential areas are off of frontage roads and not directly on the street. However, north of Fifth Street Azusa Avenue travels through an improved downtown district with many driveways and commercial businesses having access directly from Azusa Avenue. Significant impacts would be anticipated for open trench pipeline construction through this area. Therefore, it was assumed that this section would need to be tunneled for the purposes of this evaluation. Alternate routes that avoid this localized issue, such as San Gabriel Avenue may warrant consideration in subsequent design phases.

Figure 3-8 presents a photo of a typical section on Azusa Avenue north of the 210 Freeway.

Figure 3-8 Photo on Azusa Avenue looking North – Typical View North of the 210 Freeway

As mentioned previously, Metropolitan currently provides replenishment water to the USGMWD via USG-3, which is located at the westerly end of the Glendora Tunnel. If this alignment moves forward, approximately 14,000 feet of the Backbone Alignment associated with discharging to the SFSG could be eliminated. Instead, the advanced treated water could be discharged to the San

Gabriel River at, or near, USG-3 (or at another location north of the SFSG) which the Los Angeles County Department of Public Works (LACDPW) has indicated is preferred to the discharge location shown in the CDR.

Figure 3-9 illustrates the eliminated section of the Backbone Alignment and the connection to USG-3 for Alignment 4 schematically. The line in red represents Alignment 4, which connects the Backbone Alignment to the Glendora Tunnel and USG-3. The blue line represents the Backbone Alignment and the dashed blue line represents the 14,000 feet of alignment that could be eliminated if a new discharge location along Alignment 4 was implemented.

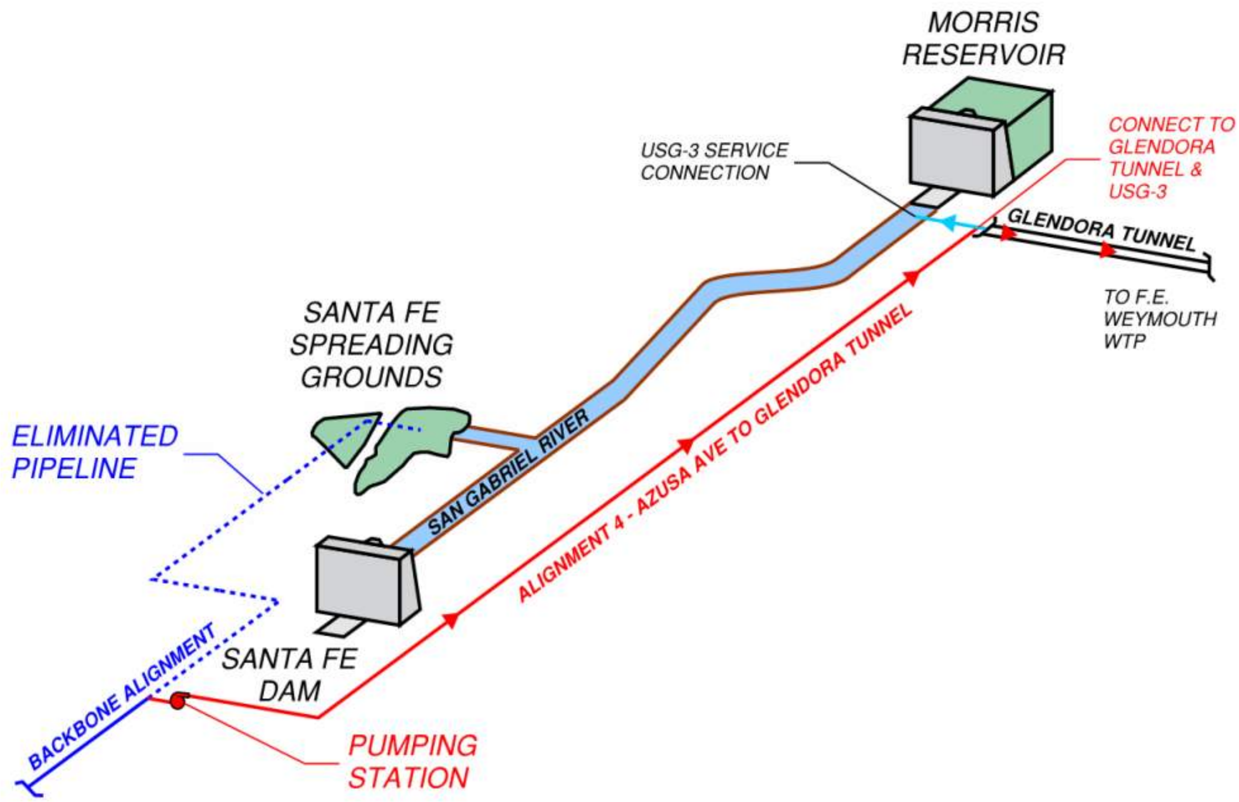


Figure 3-9 Alternative Discharge Location for Alignment 4 – Schematic View

3.3.5 Summary of Alignment Alternatives

Table 3-5 presents a summary of the raw data collected for each of the alignment alternatives. The data presented was input into the decision model to be scored and weighted to achieve a ranking of alternatives.

Table 3-5 Summary of Data Collected for Each Alternative

ITEM	ALIGNMENT 1 GLADSTONE ST	ALIGNMENT 2 ARROW HWY	ALIGNMENT 3 CYPRESS AVE	ALIGNMENT 4 GLENDORA TUNNEL
Major Utility Crossings (#)	22	36	26	20
Trenchless Construction (ft)	2,258	4,276	2,008	12,936
Depth to Water	Note 1	Note 1	Note 1	Note 1
Seismic Hazard (Y/N)	Y	Y	Y	Y
Contaminated Soil Encounters (#)	4	5	4	4
Public Facilities (#)	3	15	8	2
Length in Streets (ft)	61,266	65,027	66,639	37,275
Center Medians Impacted (ft)	28,685	10,915	25,110	18,955
Major Intersections (#)	23	19	25	13
Residential/ Minor Commercial Impacted (ft)	8,860	22,790	9,670	11,590
Total Alignment Length (ft)	63,524	69,303	68,647	50,761
Waters of the US and State (ft)	195	150	200	1,508
Critical Habitats and Listed Species (Y/N)	N	N	N	N

Notes:

1) The historical depth to groundwater for the project study area was deeper than the anticipated depth of the trench to construct the new conveyance pipeline. No subsurface exploration was completed as part of this study to verify the depth to groundwater.

4.0 Results & Conclusions

This chapter presents the results of the evaluation of alignments from the Backbone Alignment near the SFSG to the FEWWTP. This chapter also documents some of the conclusions that were made from the evaluation.

4.1 DECISION MODEL RESULTS

As described previously, outcomes from the decision model were dependent upon the evaluation criteria rating scores and category weights. To provide a more intuitive final scoring system, each total weighted score was summed for each alignment and then converted to a percentage (out of 100) so that the highest final score out of 100 percent was considered the preferred path for each comparison.

Table 4-1 summarizes the results of the alignment evaluation.

Table 4-1 Summary of Alignment Evaluation Results

ALIGNMENT	SEGMENTS	WEIGHTING A SCORE	WEIGHTING B SCORE
Alignment 1 – Gladstone Street	1, 6, 10, 13, 19, 20, 21, and 22	51%	53%
Alignment 2 – Arrow Highway	1, 7, 11, 13, 19, 20, 21, and 22	51%	53%
Alignment 3 – Cypress Street	2, 3, 4, 5, 12, 17, 21, and 22	45%	49%
Alignment 4 – Azusa Ave / SR 39 to Glendora Tunnel	1, 6, 23, 24, 25, and 26	68%	72%

As can be seen in the table above **Alignment 4 – Azusa Avenue / SR 39 to the Glendora Tunnel** was the best scoring and most favorable alignment.

Alignment 4 offers many potential benefits, including:

- Requiring the shortest length of new pipe due to repurposing the Glendora Tunnel
- Having the fewest number of major utility crossings
- Having the fewest public facility impacts
- Having the fewest major intersection crossings

Outside of the scoring system, Alignment 4 also offers other benefits to the RRWP, such as being able to eliminate 14,000 feet of pipe associated with the Backbone Alignment and providing a more preferred discharge location for the replenishment water being supplied to the USGMWD.

Table 4-2 summarizes the details of the decision model inputs, scoring, weighting, and results. Figure 4-1 presents **Alignment 4 – Azusa Avenue / SR 39 to the Glendora Tunnel**.

Table 4-2: Summary of Overall Route Results

ROUTES	ARROW HWY				ALIGNMENT 2 – ARROW HIGHWAY				ALIGNMENT 3 – CYPRESS STREET				ALIGNMENT 4 – AZUSA AVE / SR 39 TO GLENDORA TUNNEL			
	SUM	RAW SCORE	WEIGHT "A"	WEIGHT "B"	SUM	RAW SCORE	WEIGHT "A"	WEIGHT "B"	SUM	RAW SCORE	WEIGHT "A"	WEIGHT "B"	SUM	RAW SCORE	WEIGHT "A"	WEIGHT "B"
Major Utilities	26	3	45	23	22	3	45	23	36	3	45	23	20	1	15	8
Trenchless Construction	2,008	1	12	6	2,258	1	12	6	4,276	3	36	18	12,936	5	60	30
Depth to Water	0	1	3	2	0	1	3	2	0	1	3	2	0	1	3	2
Seismic Hazard	Y	5	15	8	Y	5	15	8	Y	5	15	8	Y	5	15	8
Contaminated Soils Risk	4	3	9	5	4	3	9	5	5	3	9	5	4	3	9	5
Ease of Operation Sub-Score	3	3	27	14	3	3	27	14	3	3	27	14	4	4	35	18
Parks	1	1	2	3	1	1	2	3	1	1	2	3	1	1	2	3
Public Facilities	8	3	18	33	3	3	18	33	15	5	30	55	2	1	6	11
Length in Streets	66,639				61,266				65,027				37,275			
Road Category & Traffic Impact	3	3	20	37	3	3	21	38	3	3	19	35	3	3	19	35
Center Medians	25,110	5	30	55	28,685	5	30	55	10,915	1	6	11	18,955	5	30	55
Major Intersections	25	5	23	41	23	5	23	41	19	3	14	25	13	1	5	8
Residential/ Minor Commercial	9,670	1	6	11	8,860	1	6	11	22,790	5	30	55	11,590	3	18	33
Total Alignment Length	68,647	5	75	38	63,524	5	75	38	69,303	5	75	38	50,761	1	15	8
Waters of the US and State	200	1	2	3	195	1	2	3	150	1	2	3	1508	1	2	3
Critical Habitats and Listed Species	N	1	8	12	N	1	8	12	N	1	8	12	N	1	8	12
Weighted Score			**51%**	**53%**			**51%**	**53%**			**45%**	**49%**			**65%**	**66%**

4.2 REFINEMENT O F ALIGNMENT 4

This Study recognizes that construction of a large diameter pipeline within Azusa Avenue will have significant impacts on the community. Azusa Avenue is one of the most heavily traveled surface streets in the area and is a popular through street from the 10 Freeway in the south to the 210 Freeway in the north. North of the 210 Freeway, Azusa Avenue is home to downtown Azusa, an improved, walkable downtown district with shops, wide sidewalks, and narrow streets.

Towards that end, this Study identified two alternate alignments to Azusa Avenue to get from Arrow Highway to the Glendora Tunnel. Both alternative alignments follow Alignment 4 from the Backbone Alignment to the intersection of Irwindale Avenue and Gladstone Street. When Alignment 4 turns east in Gladstone Street, both alternatives would continue north in Irwindale Avenue. Upon reaching Foothill Boulevard, Alternative 4A would turn west for approximately one-half mile and then head north in the open land adjacent to the San Gabriel River multi-purpose trail. The pipe would be constructed parallel to the trail outside of the influence of the levee. North of the San Gabriel Canyon Spreading Grounds, Alternative 4B would turn east. As of the time of this writing there is a vacant parcel north of the City of Azusa's Filtration Plant that could serve as the portal for a tunnel. Alternatively, the tunnel portal could be located west of San Gabriel Canyon Road. The alignment would then tunnel east and connect back with Alignment 4.

Alternative 4A has several "pinch points" where the distance between the San Gabriel River and the adjacent railroad tracks narrows. At the time this Study was prepared, information was not available on the levee to determine if there would be enough space to construct a large diameter pipeline. Additional evaluations are required to confirm the feasibility of this alignment.

Alternative 4B would be located entirely within existing public rights of way. From Irwindale Avenue Alternative 4B would turn east in Foothill Boulevard, north in Todd venue, and then east in Sierra Madre Avenue back to Alignment 4. While still entirely located within existing public rights of way, Alternative 4B avoids Azusa Avenue and would be located on much less impactful streets.

Figure 4-1 presents Alternatives 4A and 4B. Both alternatives carry the same benefits as the base Alignment 4 located in Azusa Avenue but were developed to try to avoid the more challenging sections of the alignment.

4.3 CONCLUSIONS

In addition to being the preferred alignment in the assessment completed, **Alignment 4 – Azusa Avenue / SR 39 to the Glendora Tunnel** offers other qualitative benefits to the RRWP outside of those strictly considered in the screening criteria. Among these benefits are the ability to eliminate 14,000 feet of the Backbone Alignment and provide replenishment water at a more preferred location.

The use of the Glendora Tunnel is the preferred alignment to get from the SFSG to the FEWWTP. Several alternatives appear feasible to get from the Backbone Alignment near the SFSG to the Glendora Tunnel. These alternatives are recommended to be carried forward for additional evaluation in subsequent design phases to confirm their feasibility and to select the preferred route.

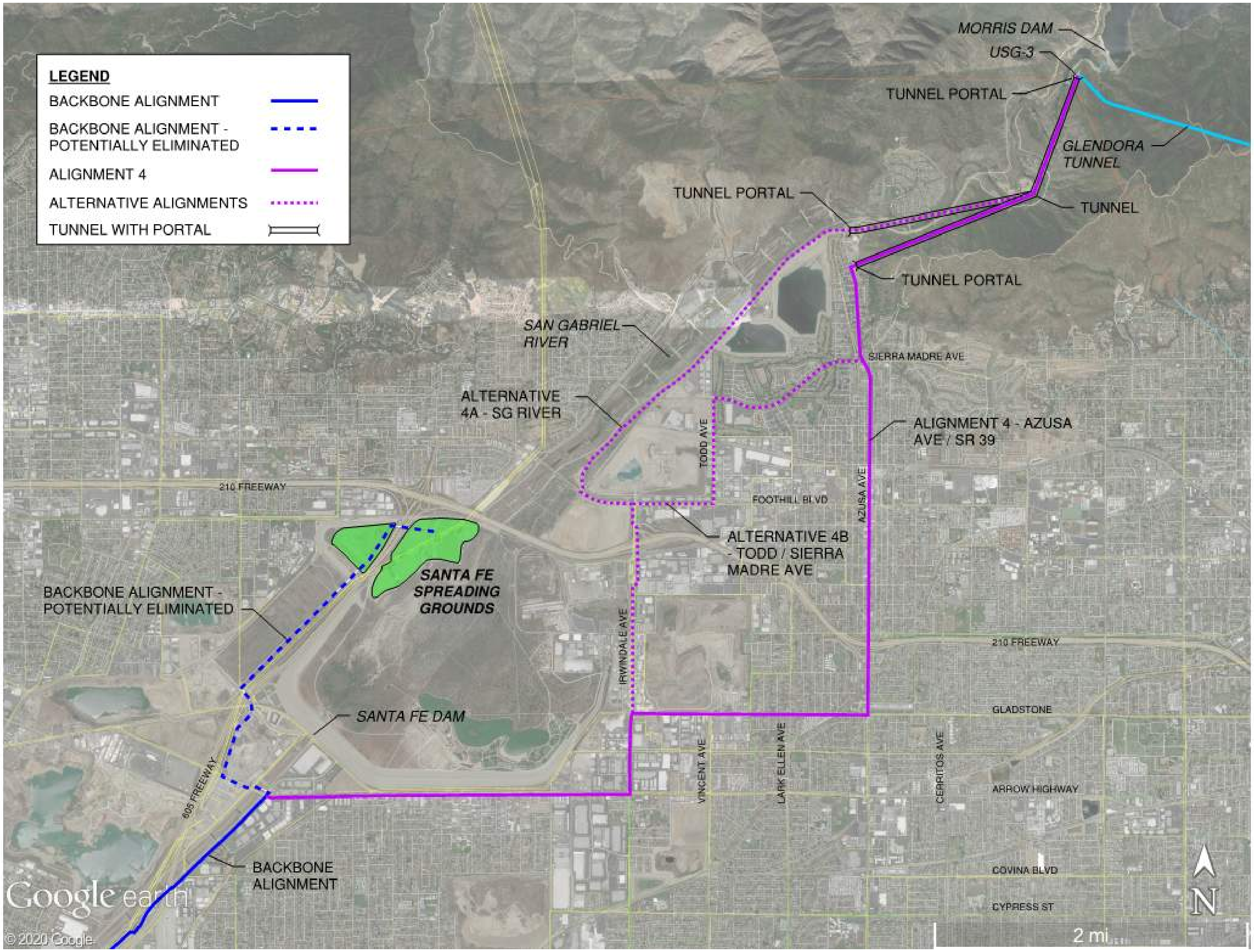


Figure 4-1 Alignment 4 – Azusa Avenue / SR 39 to Glendora Tunnel and Alternatives

Appendix U. Orange County Reach Evaluation

This page is intentionally left blank.

1.0 Orange County Reach Feasibility-Level Design

This section presents feasibility-level design information for the Orange County (OC) Reach of the Final Preferred Alignment, presented previously in the 2018 Draft Report in October 2018. This chapter is intended to provide feasibility-level design information for the OC Reach in the event that the option to deliver advanced treated water to the OC Spreading Grounds is revisited in the future.

Figure 1-1 summarizes the Project methodology as it applies to this chapter. In addition to the items listed on Figure 1-1, this chapter summarizes the OC Reach (Reach 2) of the Final Preferred Alignment and develops all the components of the feasibility-level pipeline design.

Phase	Phase 1 Metropolitan's Initial Evaluation	Phase 2 Alignment Verification and Initial Screening	Phase 3 Detailed Alternative Alignment Evaluation	Phase 4 Final Refinements	Phase 5 Conceptual Pipeline and Pump Station Design
Tasks	• Identification of potential pipeline alignments • Identification of Initial Base Case	• Data collection • Review of Metropolitan studies • Desktop analysis • Alternate alignment development • Field investigations • Initial screening • Desktop Geotechnical Report • Traffic Analysis and Impact Report • Constructability evaluations	• Development of decision model • Evaluation criteria • Weighting of evaluation criteria • Coarse screening • Secondary screening • Final screening • Ranking of alternatives	• Incorporation of stakeholder input • Conduct supplemental evaluations	• **Steel cylinder thickness** • **Conceptual pipeline plan drawings** • **Pipe optimization** • **Hydraulic analysis and profile** • **Special construction zones** • **Preliminary alignment cross-sections** • Pump station site assessment • Conceptual pump station site plans • Pump station building • Hydraulic architectural and landscaping theme • Cost development • Quantity take-off • Preliminary construction duration
Workshops		• Initial screening workshops	• Detailed evaluation workshops	• Workshops with Stakeholders • Workshops with Metropolitan's Environmental Team	• **Pipeline focus meetings/ workshops** • Pump station focus meetings/ workshops • Unit cost development workshops
Outcomes	• Initial Base Case alignment • Report entitled, "Potential Regional Recycled Water Supply Program – Conveyance System Feasibility Assessment"	• Revised Base Case alignment	• Initial Preferred Alignment	• Final Preferred Alignment	• **Conceptual pipeline and pump station design** • Engineer's cost opinion and Project schedule

Figure 1-1 Chapter Methodology

1.1 ORANGE COUNTY REACH

The Orange County Reach of the Final Preferred Alignment was the result of feasibility-level engineering development, input from internal and external stakeholders, and the ability to procure rights-of-way and easements. Details of construction activities, including but not limited to construction sequencing, contractor access and storage area, and traffic control and road closures, would be assessed during the preliminary design phase. This alignment provides a means to deliver advanced treated water to the OC Spreading Grounds.

A summary of key features of the OC Reach of the Final Preferred Alignment is presented in Table 1-1. Additionally, areas along the OC Reach requiring specific considerations during subsequent design phases are described in Table 1-2.

This page is intentionally left blank.

Table 1-1 Summary of the Orange County Reach

SEGMENT	PIPE DIAMETER (IN.)	TOTAL LENGTH (FT)	TRENCHLESS CONSTRUCTION (FT)	CITIES	DESCRIPTION	STREET	STREET WIDTH (FT)	TRAFFIC LANES (NO.)	TYPICAL CONSTRUCTION METHOD ASSUMED
11	54	17,847	964	Cerritos, La Palma	SCE	-	-	-	CM2
16	54	13,138	-	La Palma, Buena Park	SCE	-	-	-	CM2
17	54	3,146	-	Buena Park	Roadway	Stanton Ave.	60	4 + center lane	CM1
18	54	47,770	1,769	Buena Park, Fullerton, Anaheim, Placentia	Roadway	Orangethorpe	80 to 100	6 + median	CM1
						N. Kraemer Blvd.	84	4 + center lane	
						E. Miraloma Ave.	62	4 + center lane	
TOTALS		**81,901**	**2,733**						

Table 1-2 Areas Requiring Specific Consideration During Subsequent Design Phases

SEGMENT	CONSIDERATIONS FOR SUBSEQUENT DESIGN PHASES
General	Where the Final Preferred Alignment would cross a seismic hazard/ fault, a detailed seismic assessment which may include finite element analysis would be required in subsequent design phases to design for seismic resiliency.
	At this feasibility-level of planning, sufficient information is not available to determine the preferred construction method, open trench or trenchless construction, at intersections crossing the Preferred Alignment. For planning purposes, this CDR assumes that all intersections will be crossed using open trench construction unless there are known jurisdictional requirements prohibiting it (i.e., crossing rail road tracks, rivers, bridges, and Caltrans roads or highways). The CDR applies a premium to account for the higher cost of construction at all intersections that the Minagar report considered to be a Major Intersection. Further evaluation will be completed during the Preliminary Design when a comprehensive investigation and mapping of buried utilities, additional traffic control analysis, and coordination with local jurisdictions would be completed.
	The CDR assumed the crossings at freeways with adequate height and no on or off-ramp access would be constructed using open trench construction methods. However, Caltrans may require these crossings to be installed using trenchless construction. Conversations should be conducted with Caltrans during subsequent design phases to better understand their design requirements.
11	None.
16	From Reach 2, Sta. 220+15 to Reach 2, Sta. 242+30, the workspace available for construction would be limited due to congestion in the SCE corridor. The typical construction section developed for SCE easements would not be possible in this segment, and the speed of construction may be impacted.
17	The CDR assumed the crossing at Artesia Freeway (SR-91) would be constructed using trenched construction methods due to Artesia Freeway's above grade crossing at adequate height and no on or off-ramp access from Stanton Ave.
18	The CDR assumed trenchless construction would be required for the crossing of the 57 freeway and associated on and off ramps. Due to the segments above grade crossing of Orangethorpe Avenue and adequate overhead clearance, trenchless construction may not be required. Additional investigation into the rights-of-way and associated requirements regarding on and off ramps would be required during subsequent design efforts.

Notes:
1. See Section 1.2.8 for additional details.
2. See Section 1.2.9 for typical cross-sections.

1.2 FEASIBILITY-LEVEL PIPELINE DESIGN

The following section establishes the pipeline design basis, including the pipeline flow rate, hydraulic profile, diameter, material, and governing design standards. This section references two alternate pumping control strategies (Alternative A & B) that are described in further detail in Appendix V.

1.2.1 Design Flow

Pipeline diameters were sized for the full program build out of 150 mgd.

1.2.2 Optimization of Pipe Sizes and Pumping Costs

A feasibility-level analysis to optimize the pipe size of the Final Preferred Alignment to balance pumping power cost with capital construction cost was completed as part of this CDR. The analysis compared the amortized capital costs and the annual energy consumption to determine the most cost-effective pipe diameter. A more detailed evaluation should be conducted during preliminary design to validate the results. The pipe size optimization calculation is presented in Appendix H. The pipeline diameters selected for each reach are presented in Table 1-3. The stated diameter shall be the clear inside diameter after application of linings.

Table 1-3 Pipe Sizes

REACH	PIPE DIAMETER (IN.)
1	84
2	54
3	60
4	60

1.2.3 Hydraulic Profile

Preliminary hydraulic profiles were developed for the Final Preferred Alignment and are presented on Figure 1-2 through Figure 1-5.

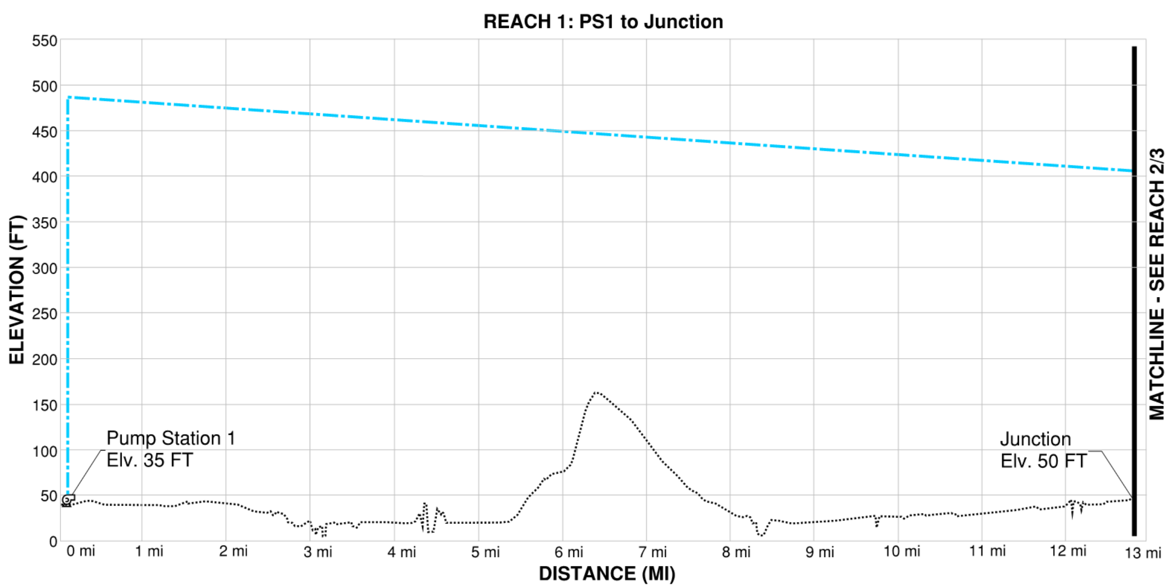


Figure 1-2 Reach 1 Hydraulic Profile (Alternative A)

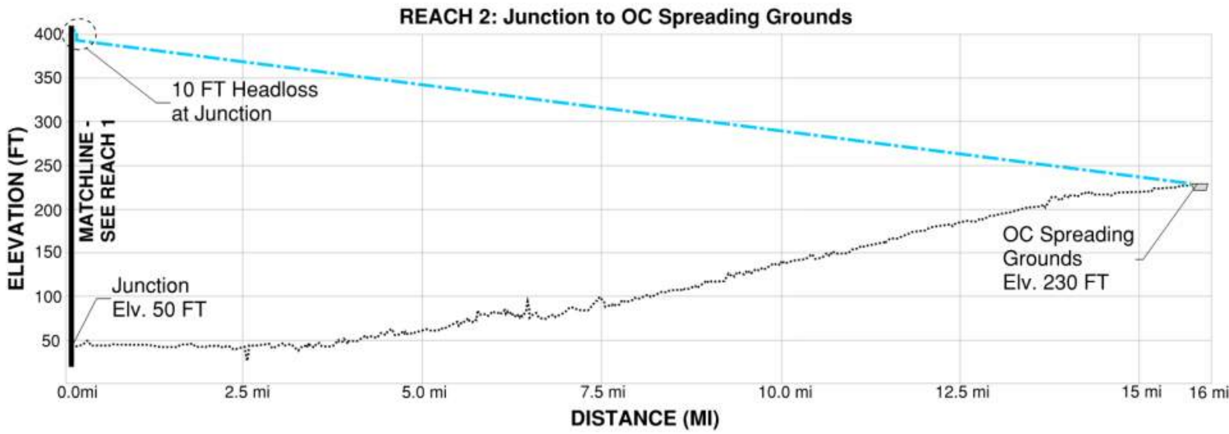


Figure 1-3 Reach 2 Hydraulic Profile (Alternative A)

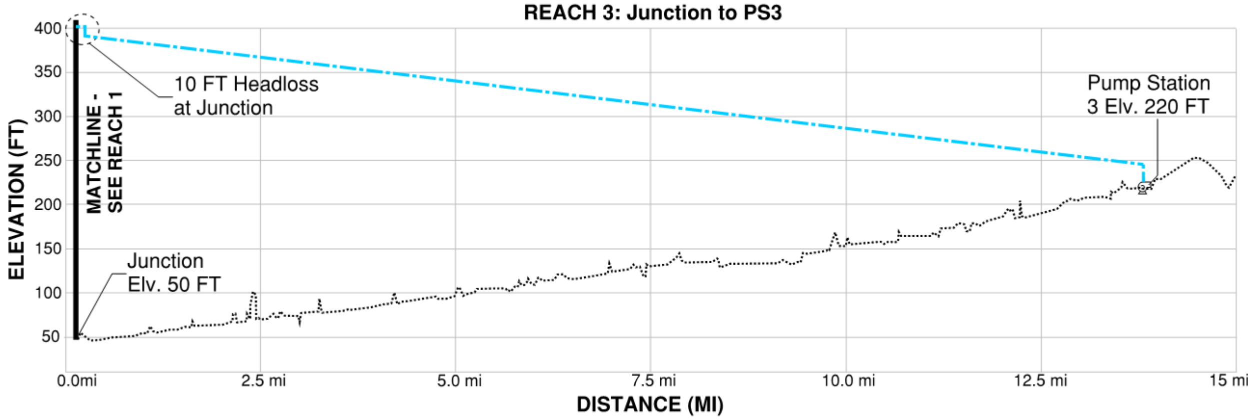


Figure 1-4 Reach 3 Hydraulic Profile (Alternative A)

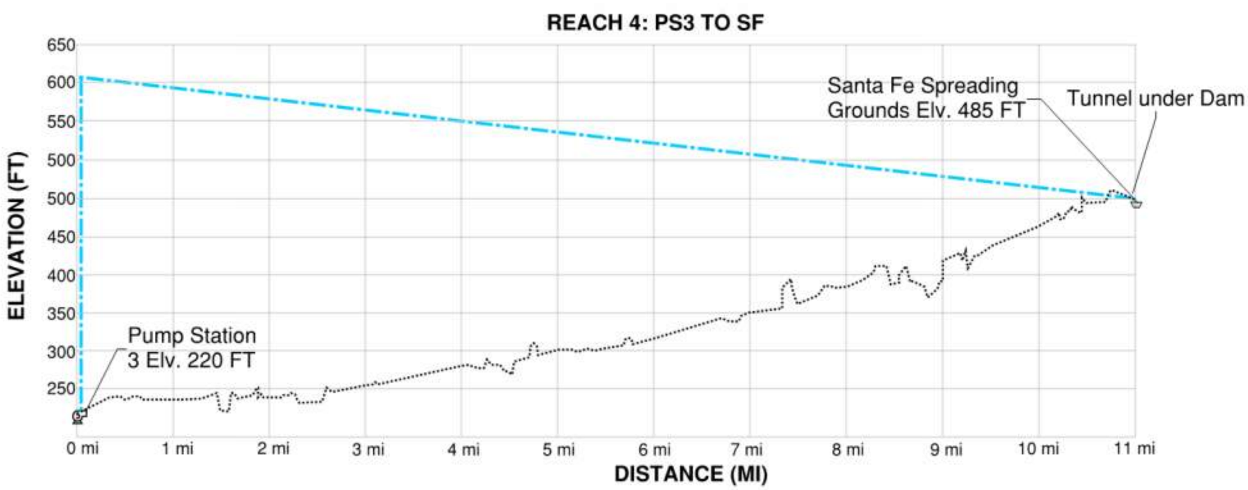


Figure 1-5 Reach 4 Hydraulic Profile (Alternative A)

1.2.4 Pipe Materials

Pipeline materials would be welded steel pipe in accordance with Metropolitan standards.

1.2.4.1 Steel Cylinder Design Calculations

Initial pipeline plate thickness calculations were completed for the OC Reach of the Final Preferred Alignment. The steel plate thickness was determined based on four loading conditions: permanent loads, semi-permanent loads, transient loads, and exceptional loads. Loads included both internal and external conditions. In addition, a minimum plate thickness due to handling and installation was considered. The evaluation was limited to a basic segment by segment analysis to support cost estimating and provide an initial basis for preliminary design development. It was assumed that more detailed, site specific calculations would be completed during preliminary design.

The recommended steel plate thicknesses for the pipe segment are summarized in Table 1-4. Details of the initial pipeline plate thickness calculations are presented in Appendix I.

Table 1-4 Steel Cylinder Thicknesses

REACH	ALTERNATIVE A PLATE THICKNESS (IN.)	ALTERNATIVE B PLATE THICKNESS (IN.)
2	0.375	0.375

Note:
1. Steel cylinder thickness calculations assume 42 kips per square inch steel and a minimum plate thickness of 0.375 inches per Metropolitan's standard specification Section 02662.

1.2.5 Pipeline Appurtenances

Pipeline appurtenances would be required for the proper operation and maintenance of the RRWP conveyance system. Appurtenances would include combination air-release and vacuum valves (ARVV), blow-offs, access manways, isolation valves, discharge connections, pumping wells, and other miscellaneous appurtenances. Metropolitan's standard drawings would be used to develop typical details for these appurtenances.

As part of the preliminary design, a study would be performed to determine potential blow-offs and ARVV locations along the alignment. Locations where blow-offs could be connected to storm drains, existing channels, or drainage courses would also be identified during preliminary design.

In general, blow-offs would be located at low points along the pipeline and ARVVs would be located at high points.

1.2.6 Intersections

A list of all the Major and Minor Intersections, as designated by the Traffic Impact Analysis, for each Segment of the OC Reach of the Final Preferred Alignment is provided in Table 1-5.

Table 1-5 Summary of Intersection Designations for the OC Reach

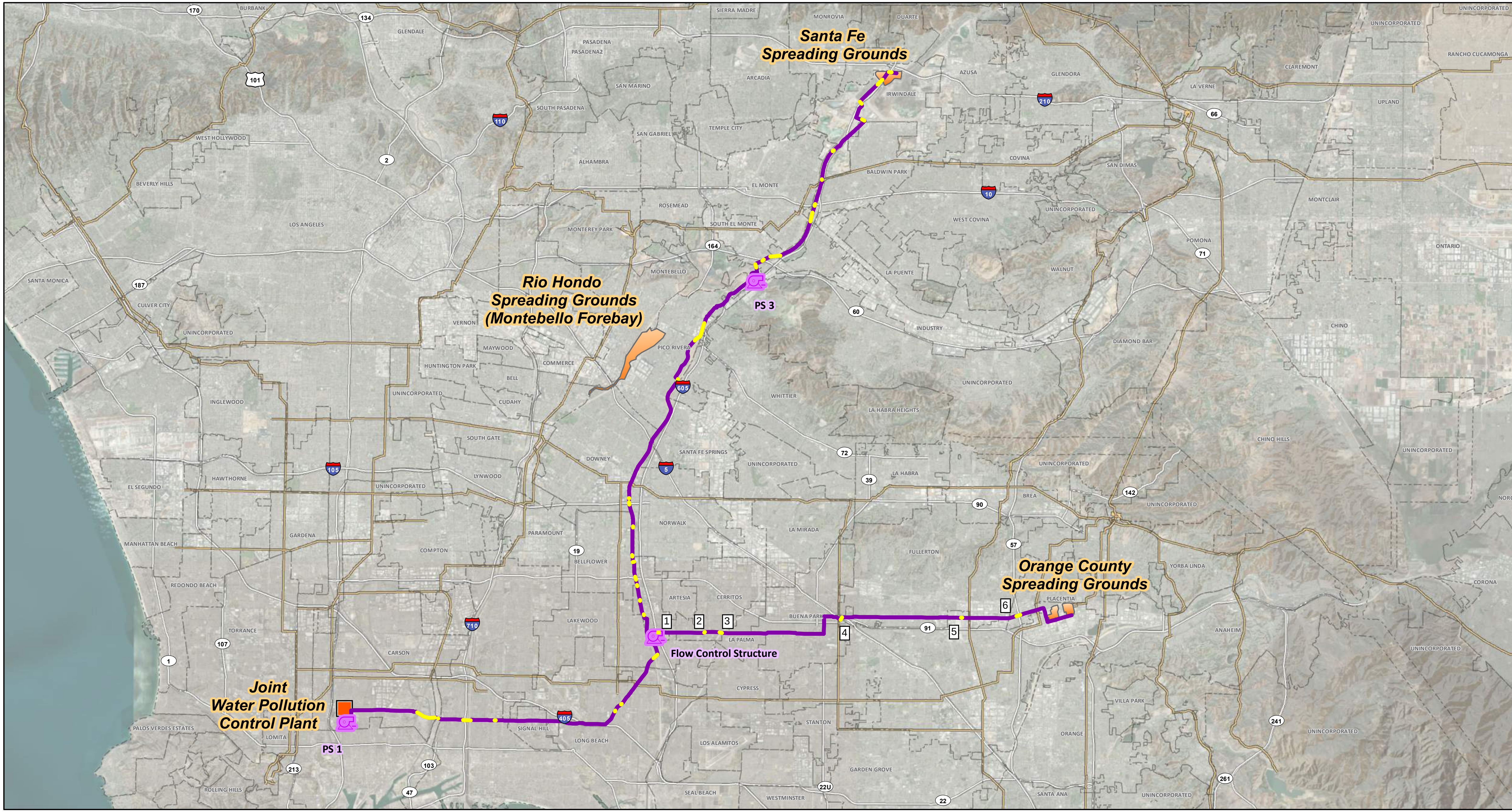
SEGMENT	INTERSECTION	CLASSIFICATION
11	None	N/A
16	None	N/A
17	Orangethorpe @ Page St.	Minor
18	Orangethorpe @ Auto Center Dr.	Minor
	Orangethorpe @ Magnolia Ave.	Major
	Orangethorpe @ Gilbert Street	Minor
	Orangethorpe @ Brookhurst St.	Major
	Orangethorpe @ Pacific Dr.	Minor
	Orangethorpe @ Basque Ave.	Minor
	Orangethorpe @ Euclid St.	Major
	Orangethorpe @ Woods Ave.	Minor
	Orangethorpe @ Richman Ave.	Minor
	Orangethorpe @ Highland Ave.	Minor
	Orangethorpe @ Harbor Blvd.	Major
	Orangethorpe @ Orangefaire Marketplace	Minor
	Orangethorpe @ Lemon St.	Major
	Orangethorpe @ Cypress Via	Minor
	Orangethorpe @ R/R Xing	Minor
	Orangethorpe @ Raymond Ave.	Major
	Orangethorpe @ Acacia Ave.	Minor
	Orangethorpe @ State College Blvd.	Major
	Orangethorpe @ Placentia Ave.	Major
	Orangethorpe @ SR-57 SB On-Off/Iowa Pl.	Minor
	Orangethorpe @ SR-57 NB On/Off-Ramps	Minor
	Orangethorpe @ Melrose St.	Major
	Orangethorpe @ Kraemer Blvd. (alignment turn)	Major
	Kraemer @ La Jolla St.	Minor
	Kraemer @ Miraloma Ave.	Minor
	Mira Loma @ Miller St.	Minor

1.2.7 Trenchless Construction Recommendations

To establish a conservative budgetary construction cost for the portions of the alignment preliminarily identified for trenchless installation, it was necessary to select a conservative trenchless construction method for each location. Within the Desktop Geotechnical Evaluation (Appendix C), a desktop level review of geotechnical and hydrogeologic conditions was conducted and applicable trenchless methods were identified for each of the trenchless sub-segments along the Final Preferred Alignment. Black & Veatch reviewed the trenchless methods that GeoPentech identified as applicable and selected a feasible method for each trenchless installation site based on its location, length, pipeline size, and the foreseeable subsurface geotechnical and hydrogeologic conditions available from the desktop studies.

The next phase of the Project is expected to include site specific subsurface geotechnical explorations and a comprehensive investigation and mapping of buried utilities. These site-specific analyses will allow for a final selection of trenchless installation methods to be used at each location and may warrant that the trenchless methods described herein for planning and budgeting purposes be revised.

The selected trenchless methods provided the basis for development of the feasibility-level Engineer's OPCC for the Project. Figure 1-6 correlates the Tunnel identification number given in Table 1-6 (seen below) with the location of each trenchless sub-segment along the OC Reach of the Final Preferred Alignment.



Existing MWD Distribution System

Preferred Alignment

Trenchless / Tunnel Undercrossing with ID #

Pump Station or Flow Control Structure

Spreading Basins

BLACK & VEATCH
Building a world of difference.

Feasibility-Level Design of Conveyance for Potential RW Supply Program

Figure 1-6: OC Reach Alignment Trenchless/Tunnel ID

N

1 in = 3 miles

Table 1-6 Trenchless Construction Method Recommendations and Key Details for the OC Reach

TUNNEL NO.	LENGTH (FT)	DESCRIPTION	PIPE INTERNAL DIAMETER (FT)	CASING OR TUNNEL OUTER DIAMETER (FT)	MINIMUM DEPTH (FT)[2]	GROUND WATER IMPACT	METHOD SELECTED	COBBLES, GRAVEL, BOULDERS	FAULT CROSSING	OIL FIELD	COMMENTS
1	351	Freeway	4.5	6.5	19.5	Yes	Microtunneling	-	-	-	Too large diameter for HDD, not suitable for jack & bore as a river crossing.
2	134	River	4.5	6.5	27.5	Yes	Microtunneling	-	-	-	Not long enough to justify HDD, MT best suited.
3	478	River	4.5	6.5	29.5	Yes	Microtunneling	-	-	-	Too short for a drive length for HDD due to large diameter unless shafts were excavated to launch HDD.
4	518	Freeway	4.5	6.5	19.5	Yes	Microtunneling	-	-	Yes	Not long enough to justify HDD, MT best suited.
5	201	Railroad	4.5	6.5	19.5	No	Microtunneling	-	-	-	MT best suited to manage risk under railroad.
6	1,050	Freeway	4.5	6.5	19.5	No	Microtunneling	-	-	-	Length is too long to reliably complete with jack & bore, MT recommended.

Notes:
1. Tunnel identification number corresponds with Figure 1-6.
2. Depth below ground surface or river channel to top of pipe or crown of tunnel; generally equal to 3 diameters of the excavated hole.

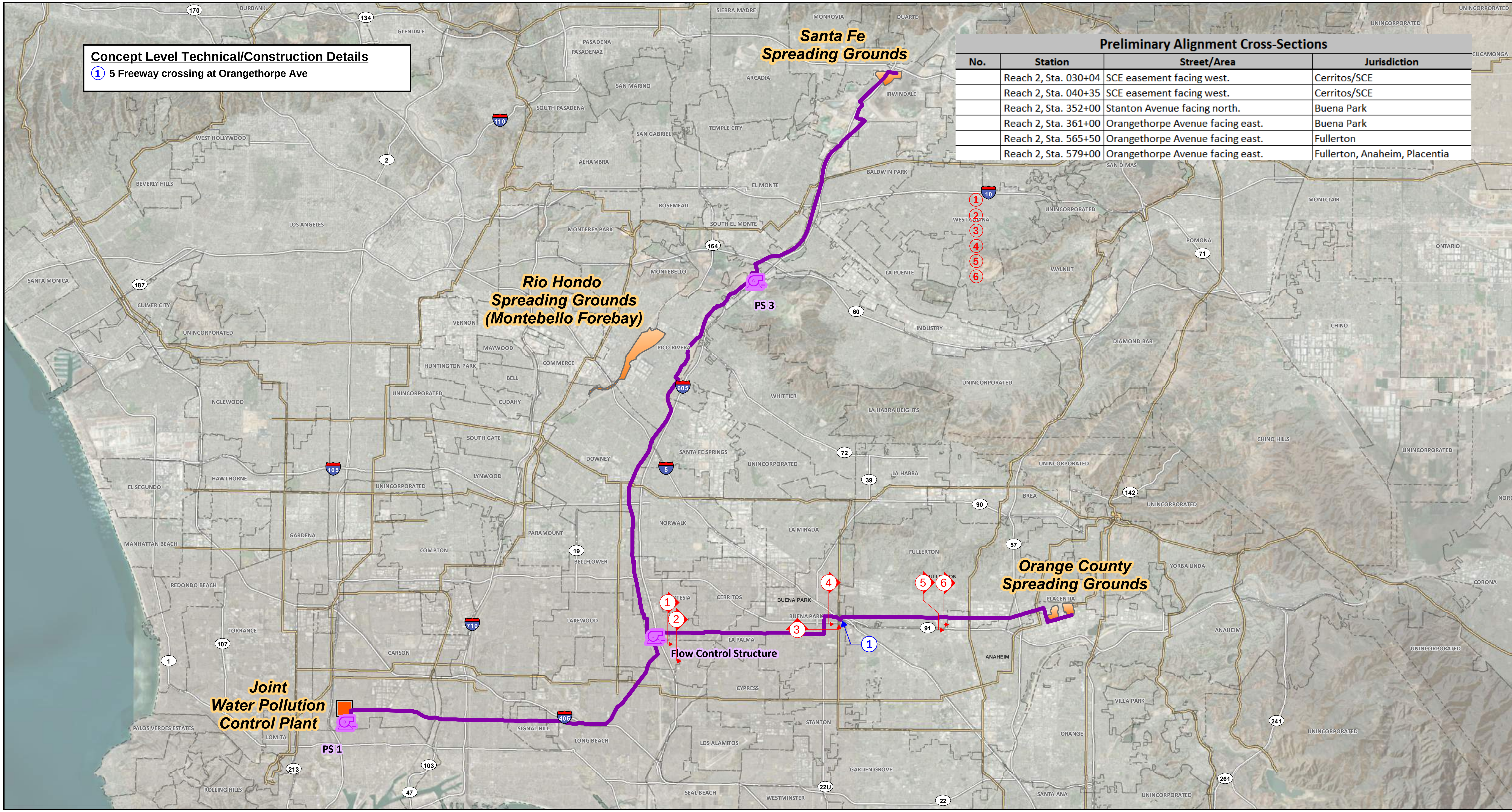
1.2.8 Feasibility-Level Technical/Construction Details

This section discusses segments of the OC Reach of the Final Preferred Alignment where the typical construction methods would not be sufficient to construct the pipeline due to terrain, such as rivers, and/or physical barriers, such as freeways or railroads, or to avoid impacts to above the above ground community. A preliminary review of the Final Preferred Alignment identified ten locations warranting feasibility-level technical / construction details. The ten feasibility-level technical/construction detail locations are identified in Table 1-7 and presented on Figure 1-7.

Descriptions of each of the ten feasibility-level technical/construction detail locations are provided in the following subsections, including details on site conditions, existing utilities, easements, and trenchless methodology. Additionally, plan and profiles have been developed for each of the ten locations. All ground elevations shown were obtained through Google Earth and are approximate. No ground surveys were completed for this CDR.

Table 1-7 Feasibility-Level Technical/Construction Detail Locations

NO.	STATION	DESCRIPTION
1	Reach 2, Sta. 385+58 – Reach 2, Sta. 390+80	Trenchless crossing of 5 Freeway along Orangethorpe Avenue.



Preliminary Alignment Cross-Sections

No.	Station	Street/Area	Jurisdiction
1	Reach 2, Sta. 030+04	SCE easement facing west.	Cerritos/SCE
2	Reach 2, Sta. 040+35	SCE easement facing west.	Cerritos/SCE
3	Reach 2, Sta. 352+00	Stanton Avenue facing north.	Buena Park
4	Reach 2, Sta. 361+00	Orangethorpe Avenue facing east.	Buena Park
5	Reach 2, Sta. 565+50	Orangethorpe Avenue facing east.	Fullerton
6	Reach 2, Sta. 579+00	Orangethorpe Avenue facing east.	Fullerton, Anaheim, Placentia

Existing MWD Distribution System
Preferred Alignment
Pump Station or Flow Control Structure
Spreading Basins
1 Preliminary Alignment Cross-Section
1 Location of Concept Construction Details

BLACK & VEATCH
Building a world of difference.

Feasibility-Level Design of Conveyance for Potential RW Supply Program

Figure 1-7: Concept Level Technical/ Construction Detail and Cross Section Locations

N
1 in = 3 miles

1.2.8.1

1.2.8.1 5 Freeway Crossing

The OC Reach Alignment proposes crossing the 5 Freeway south of Orangethorpe Avenue from Reach 2, Sta. 385+58 to Reach 2, Sta. 390+80 using trenchless construction methods. The proposed crossing is shown in plan on Figure 1-8 and in profile on Figure 1-9. Key details of the crossing are provided in Table 1-8.

Table 1-8 Trenchless Method Summary of 5 Freeway Crossing

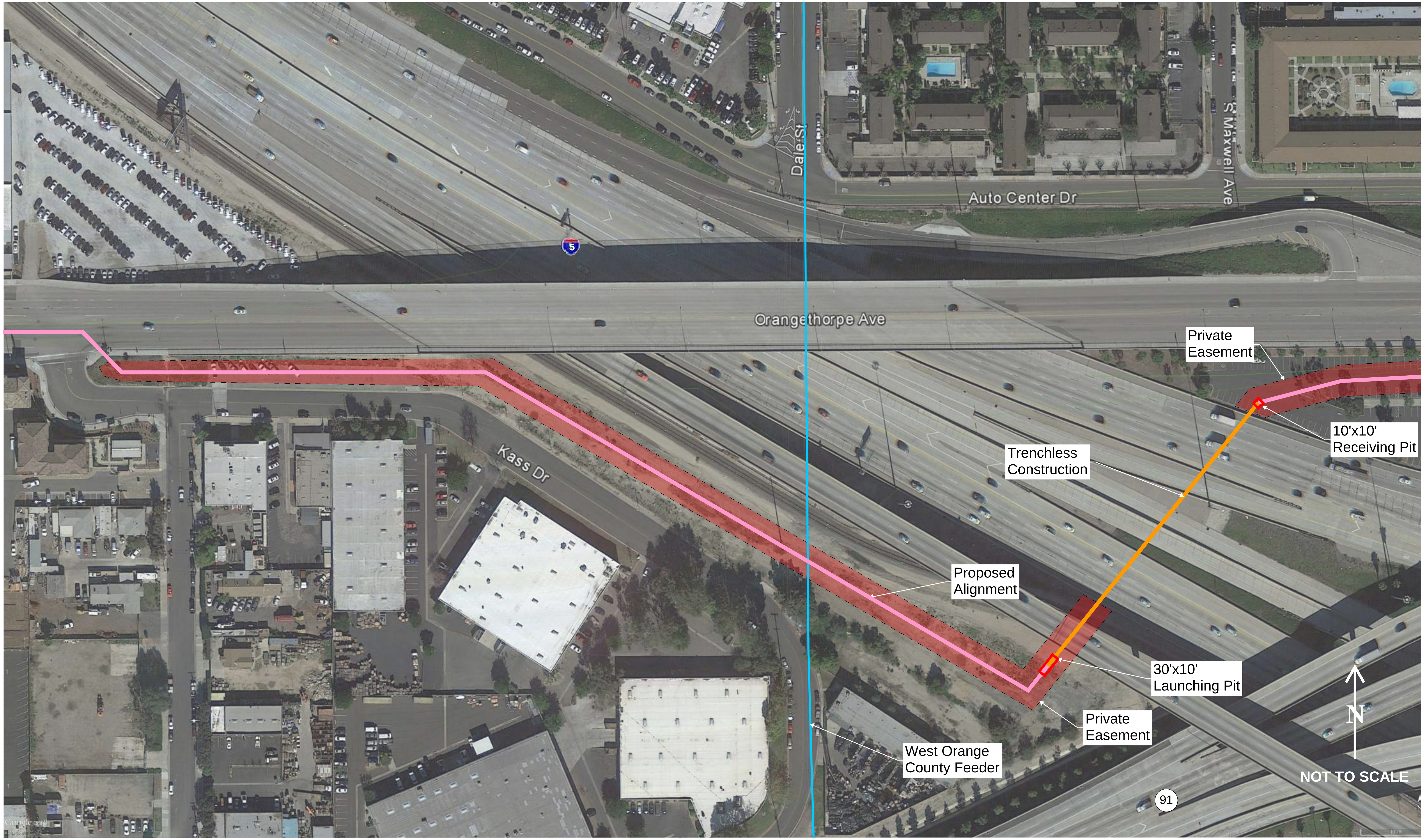
LENGTH (FT)	UNDERCROSSING DESCRIPTION	PIPE DIAMETER (FT)	MINIMUM DEPTH (FT)	GW LEVEL ABOVE TUNNEL (Y/N)	RECOMMENDED TRENCHLESS METHOD	DEWATERING FOR PITS (Y/N)	DEWATERING ALONG ALIGNMENT (Y/N)	COBBLES, GRAVEL, BOULDERS (Y/N)	FAULT CROSSING (Y/N)	OIL FIELD (Y/N)
518	Freeway	4.5	19.5	Yes	MT	Y	N	N	N	Y

To complete the crossing, launching and receiving pits would be constructed on either side of the freeway. Launching would be recommended from the southwest side of the freeway based upon potentially available space for pit excavation and contractor staging. Since the initial field investigations associated with this CDR, this land was developed from a vacant field to a parking lot. Further review of the property would be required to finalize pit location and availability. The receiving pit would be recommended on the northeast side of the freeway due to limited available space and the proximity of LACSD sewer pipes. Both properties for both the lunching and receiving pits are in commuter parking lots. Construction and easements would have a significant impact on both properties, and early real property acquisition would be recommended to confirm the alignment and acquire access.

An existing LACSD sewer line and the West Orange County Feeder both cross the 5 Freeway close to the Preferred Alignment. Potholing these utilities would be recommended to confirm the alignment. No other utilities are anticipated.

The exact location of bridge piers for the 5 Freeway and the on and off ramps will require further investigation during subsequent design phases to confirm the alignment.

Acquisition of temporary and permanent easements would be required.



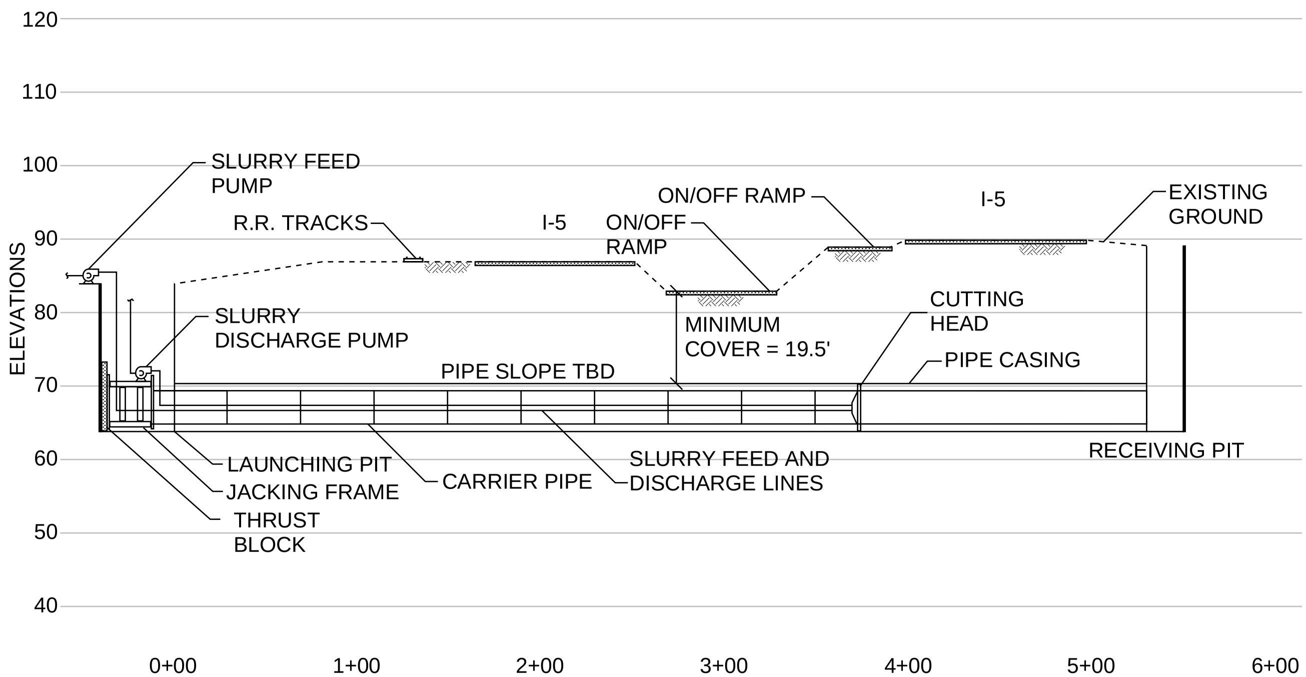
BLACK & VEATCH

- Open-Trench Construction Method - Roadways
- Trenchless Construction Method - Microtunnel
- Private Easement
- Launching/Receiving Pit
- West Orange County Feeder

POTENTIAL REGIONAL RECYCLED WATER PROGRAM

5 Freeway Crossing

FIGURE 1-8



MICROTUNNEL CROSSING OF I-5 (1)

HORZ: 1" = 50'
VERT: 1" = 12.5'

1" = 50'   0   50'   100'   150'   FEET

BLACK & VEATCH

POTENTIAL REGIONAL RECYCLED WATER PROGRAM

Profile of 5 Freeway Crossing

FIGURE 1-9

1.2.9

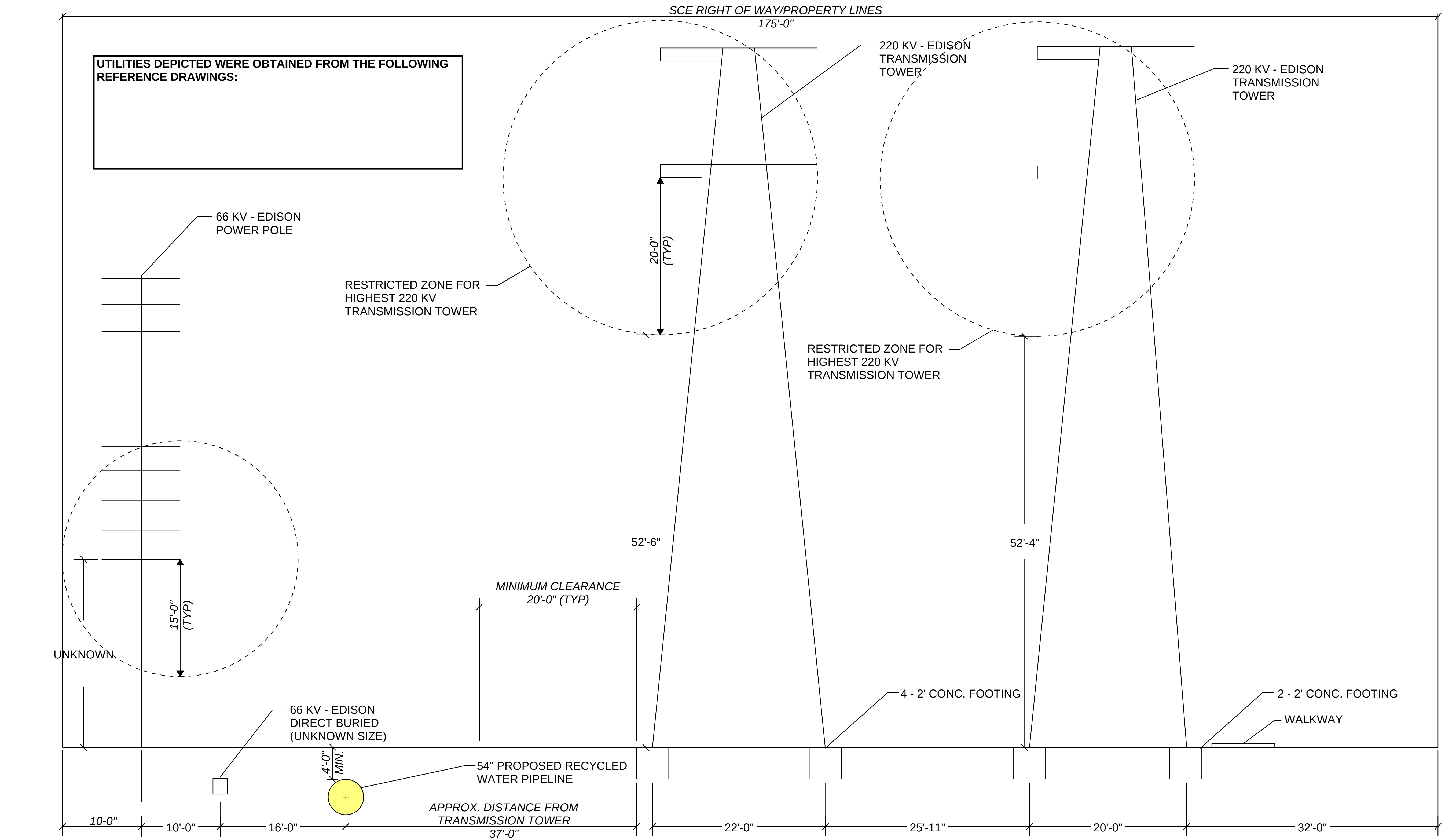
1.2.9 Preliminary Alignment Cross-Sections

Utilizing GIS mapping and right-of-way information, feasibility-level alignment cross-sections were developed to depict the approximate location of the Final Preferred Alignment relative to known major utilities and key surface features. The proposed location of the Final Preferred Alignment was developed based on extensive research of existing utilities based on above grade features and available utility maps. The cross-sections are graphical in nature and are not intended to represent design-level detail. However, the alignment does reflect a general corridor that the pipeline could be built in that avoids known major utilities, surface obstructions, and minimizes traffic impacts. Additional utility investigations, including subsurface investigations, will be completed during subsequent design phases and the alignment is anticipated to be adjusted accordingly.

Since the Final Preferred Alignment would traverse long stretches of existing streets with utilities varying in location, no "typical" section is provided to represent the location of the pipeline along the entire alignment. Instead, the alignment attempts to account for the presence of existing utilities and constructability concerns at each specific location. The representative cross-sections at key corridors of the OC Reach are identified in Table 1-9 and presented on thru Figure **1-15**. Figure 1-7 presents the location of each representative cross-section.

Table 1-9 Preliminary Alignment Cross-Section Locations

NO.	STATION	DESCRIPTION
1	Reach 2, Sta. 030+04	SCE easement facing west.
2	Reach 2, Sta. 040+35	SCE easement facing west.
3	Reach 2, Sta. 352+00	Stanton Avenue facing north.
4	Reach 2, Sta. 361+00	Orangethorpe Avenue facing east.
5	Reach 2, Sta. 565+50	Orangethorpe Avenue facing east.
6	Reach 2, Sta. 579+00	Orangethorpe Avenue facing east.



NOTE:

* WIDTH CAN REDUCE TO 6'-0" IF THERE IS 4'-0" OF TEMPORARY CONSTRUCTION EASEMENT AVAILABLE ON ADJACENT PROPERTY.

CROSS SECTION 1
NO SCALE

NOTE:
1. THE LOCATIONS OF EXISTING UTILITIES WERE IDENTIFIED USING CITY PROVIDED GEOGRAPHIC INFORMATION SYSTEM (GIS) DRAWINGS, UTILITY BASE MAPS, AND EXISTING UTILITY RECORD DRAWINGS. THE EXACT LOCATION OF THESE EXISTING UTILITIES AND CRITICAL POTENTIAL CONFLICTS SHOULD BE FIELD INVESTIGATED DURING THE DESIGN PHASE USING POT HOLES. ANY UTILITY CROSSING WITH LESS THAN 24 INCHES OF SEPARATION SHOULD BE CAREFULLY CONSIDRED TO AVOID FUTURE EXPOSURE OR CONFLICT.
2. ALL DEPTHS OF EXISTING UTILITES ARE ASSUMED.
3. HEIGHT OF POWER CABLES FOR TRANSMISSION TOWERS AND POWER POLES ARE ASSUMED.

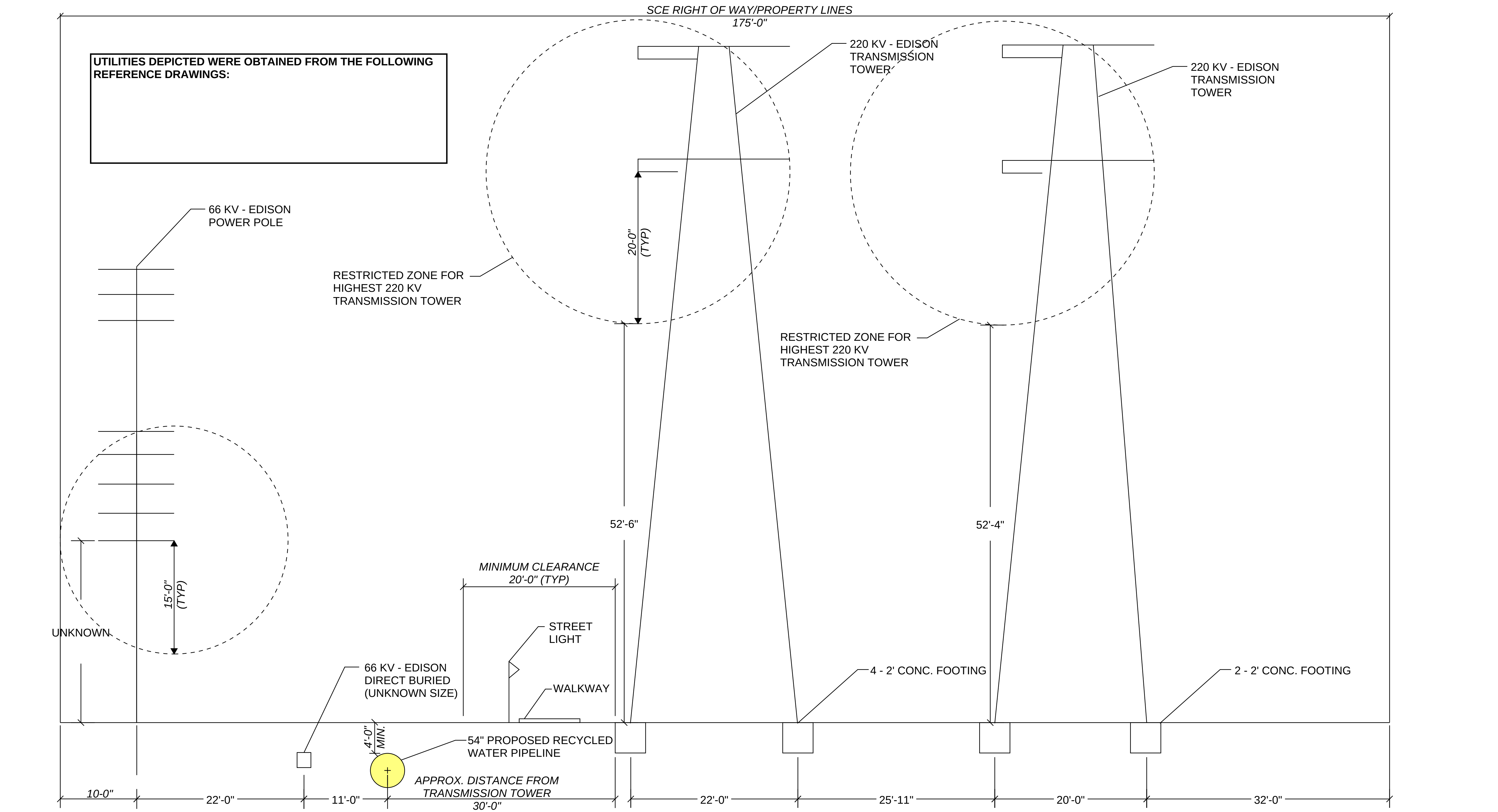
BLACK & VEATCH

0 5 10 15 FEET

POTENTIAL REGIONAL RECYCLED WATER PROGRAM

PRELIMINARY ALIGNMENT CROSS-SECTION 8 – SCE EASEMENT FACING WEST (REACH 2, STA 030+04)

FIGURE 1-10



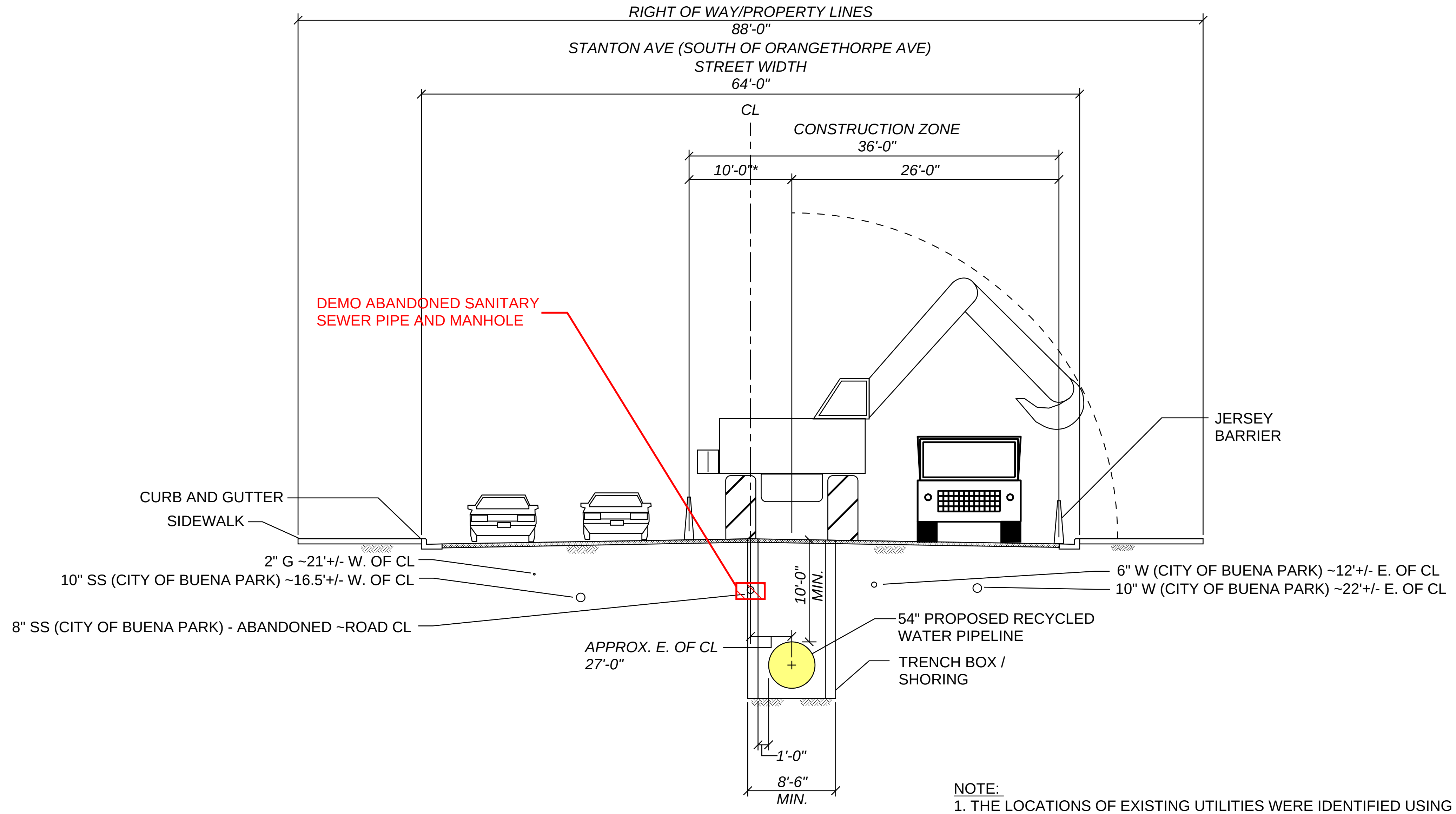
NOTE:

* *WIDTH CAN REDUCE TO 6'-0" IF THERE IS 4'-0" OF TEMPORARY CONSTRUCTION EASEMENT AVAILABLE ON ADJACENT PROPERTY.*

CROSS SECTION 2
NO SCALE

NOTE:
1. THE LOCATIONS OF EXISTING UTILITIES WERE IDENTIFIED USING CITY PROVIDED GEOGRAPHIC INFORMATION SYSTEM (GIS) DRAWINGS, UTILITY BASE MAPS, AND EXISTING UTILITY RECORD DRAWINGS. THE EXACT LOCATION OF THESE EXISTING UTILITIES AND CRITICAL POTENTIAL CONFLICTS SHOULD BE FIELD INVESTIGATED DURING THE DESIGN PHASE USING POT HOLES. ANY UTILITY CROSSING WITH LESS THAN 24 INCHES OF SEPARATION SHOULD BE CAREFULLY CONSIDRED TO AVOID FUTURE EXPOSURE OR CONFLICT.
2. ALL DEPTHS OF EXISTING UTILITES ARE ASSUMED.
3. HEIGHT OF POWER CABLES FOR TRANSMISSION TOWERS AND POWER POLES ARE ASSUMED.

BLACK & VEATCH	0 5 10 15 FEET	**POTENTIAL REGIONAL RECYCLED WATER PROGRAM** **PRELIMINARY ALIGNMENT CROSS-SECTION 9– SCE EASEMENT FACING WEST (REACH 2, STA 040+35)**	**FIGURE 1-11**



NOTE:
1. THE LOCATIONS OF EXISTING UTILITIES WERE IDENTIFIED USING CITY PROVIDED GEOGRAPHIC INFORMATION SYSTEM (GIS) DRAWINGS, UTILITY BASE MAPS, AND EXISTING UTILITY RECORD DRAWINGS. THE EXACT LOCATION OF THESE EXISTING UTILITIES AND CRITICAL POTENTIAL CONFLICTS SHOULD BE FIELD INVESTIGATED DURING THE DESIGN PHASE USING POT HOLES. ANY UTILITY CROSSING WITH LESS THAN 24 INCHES OF SEPARATION SHOULD BE CAREFULLY CONSIDRED TO AVOID FUTURE EXPOSURE OR CONFILCT.
2. ALL DEPTHS OF EXISTING UTILITES ARE ASSUMED.

CROSSSECTION 3 -
NO SCALE

UTILITIES DEPICTED WERE OBTAINED FROM THE FOLLOWING REFERENCE DRAWINGS:
- CITY OF BUENA PARK DEPT. OF PUBLIC WORKS DWG ST-796
- CITY OF BUENA PARK ENGINEERING DEPT SPECIFICATION NO. 63 W-171, PIPELINE PLAN AND PROFILE
- CITY OF BUENA PARK WATER SYSTEM ATLAS (4/21/2013)
- CITY OF BUENA PARK SEWER SYSTEM ATLAS (4/21/2013)

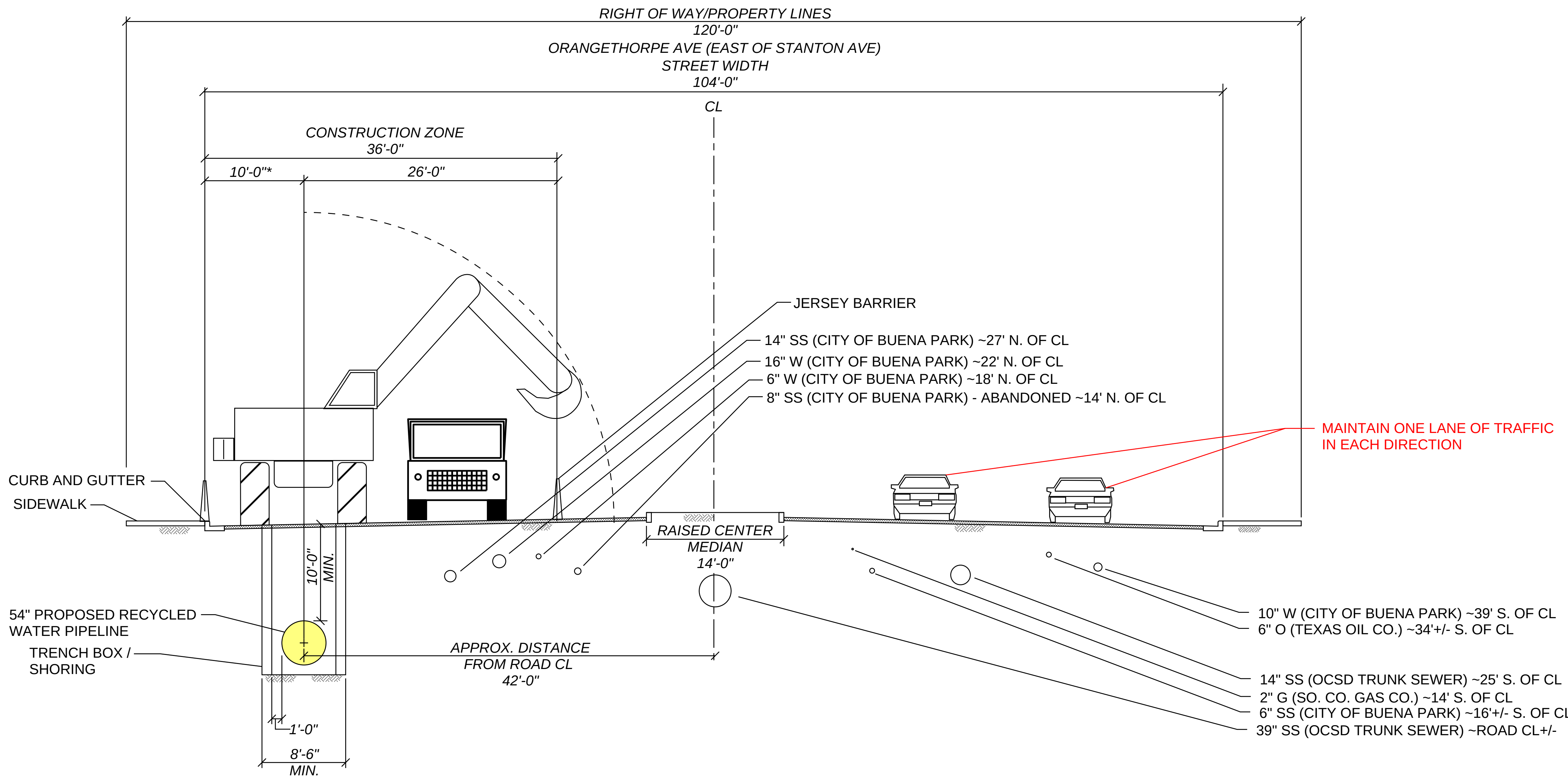
NOTE:
* WIDTH CAN REDUCE TO 6'-0" IF THERE IS 4'-0" OF TEMPORARY CONSTRUCTION EASEMENT AVAILABLE ON ADJACENT PROPERTY.

BLACK & VEATCH

0 5 10 15 FEET

POTENTIAL REGIONAL RECYCLED WATER PROGRAM
PRELIMINARY ALIGNMENT CROSS-SECTION 10 –
STANTON AVE FACING NORTH (REACH 2, STA 352+00)

FIGURE 1-12



NOTE:
1. THE LOCATIONS OF EXISTING UTILITIES WERE IDENTIFIED USING CITY PROVIDED GEOGRAPHIC INFORMATION SYSTEM (GIS) DRAWINGS, UTILITY BASE MAPS, AND EXISTING UTILITY RECORD DRAWINGS. THE EXACT LOCATION OF THESE EXISTING UTILITIES AND CRITICAL POTENTIAL CONFLICTS SHOULD BE FIELD INVESTIGATED DURING THE DESIGN PHASE USING POT HOLES. ANY UTILITY CROSSING WITH LESS THAN 24 INCHES OF SEPARATION SHOULD BE CAREFULLY CONSIDRED TO AVOID FUTURE EXPOSURE OR CONFILCT.
2. ALL DEPTHS OF EXISTING UTILITES ARE ASSUMED.

CROSS SECTION 4 -
NO SCALE

UTILITIES DEPICTED WERE OBTAINED FROM THE FOLLOWING REFERENCE DRAWINGS:
- CITY OF BUENA PARK SPECIFICATION NO. W-120, ORANGETHORPE TRANSMISSION MAIN EXTENSION PLAN AND PROFILE STA 42+00 TO STA 62+00
- CITY OF BUENA PARK DRAWING NO. ST-902, SHEET 3
- CITY OF BUENA PARK WATER SYSTEM ATLAS (4/21/2013)
- CITY OF BUENA PARK SEWER SYSTEM ATLAS (4/21/2013)

NOTE:
* WIDTH CAN REDUCE TO 6'-0" IF THERE IS 4'-0" OF TEMPORARY CONSTRUCTION EASEMENT AVAILABLE ON ADJACENT PROPERTY.

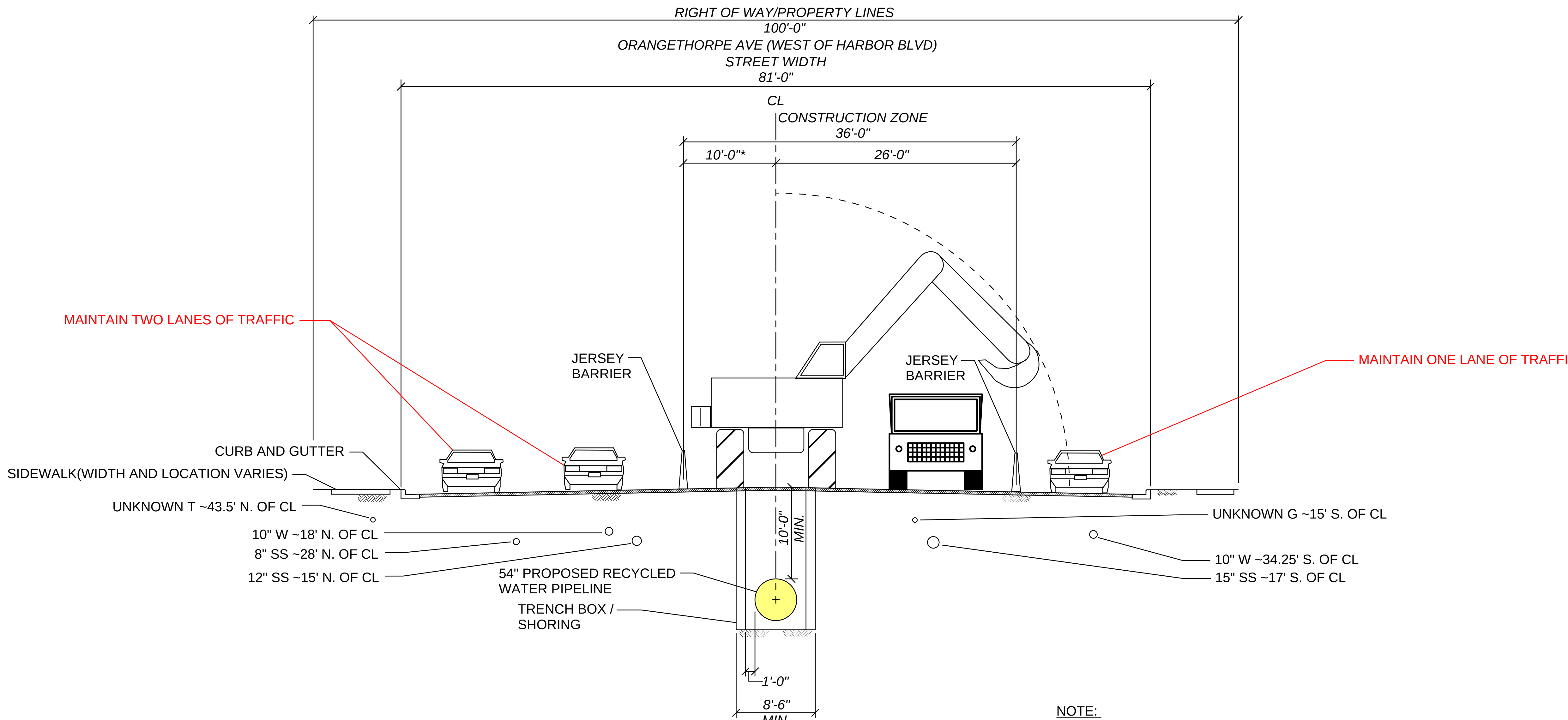
BLACK & VEATCH

0 5 10 15 FEET

POTENTIAL REGIONAL RECYCLED WATER PROGRAM

PRELIMINARY ALIGNMENT CROSS-SECTION 11 –
ORANGETHORPE AVE FACING EAST (REACH 2, STA 361+00)

FIGURE 1-13



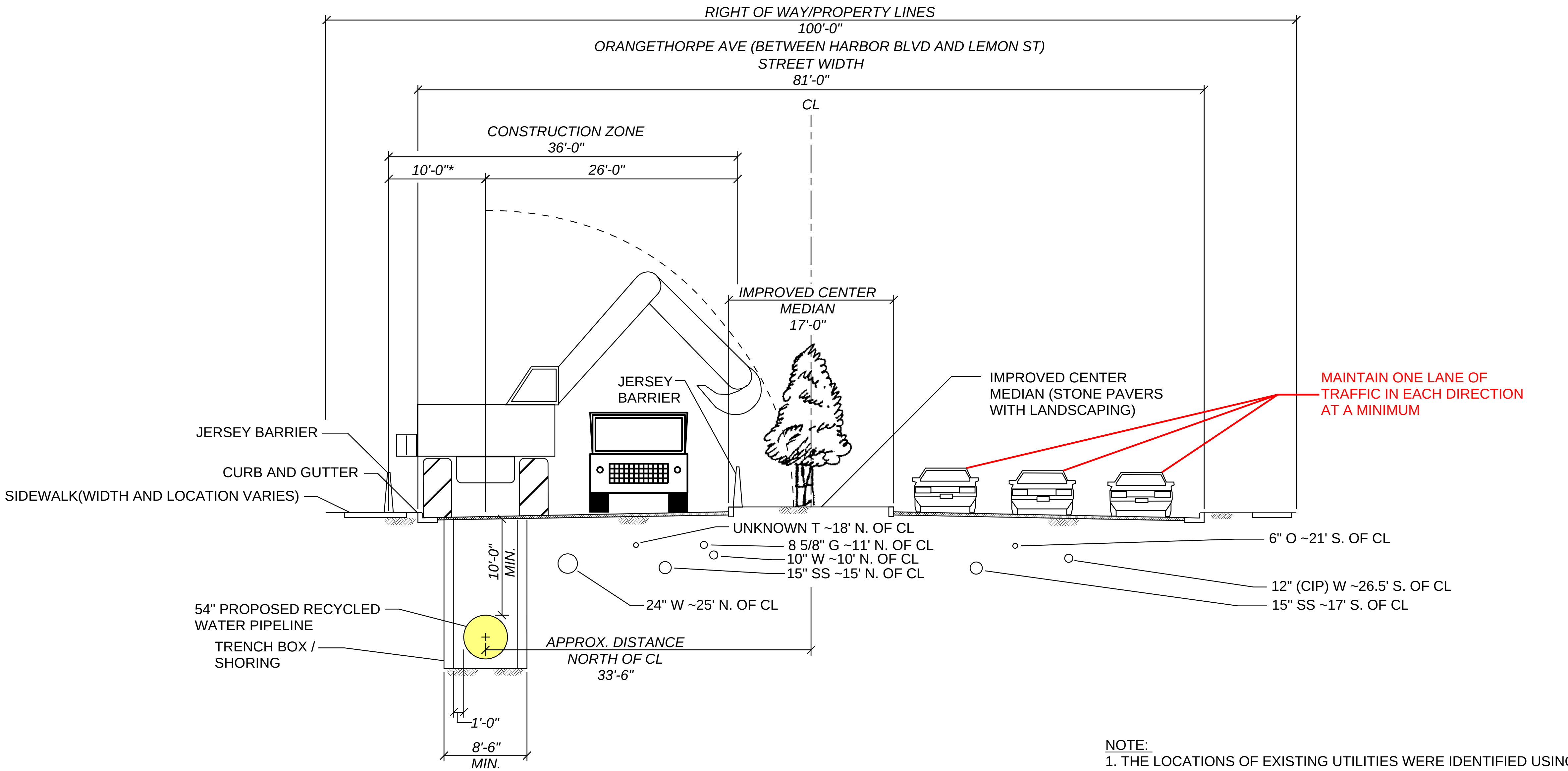
NOTE:
1. THE LOCATIONS OF EXISTING UTILITIES WERE IDENTIFIED USING CITY PROVIDED GEOGRAPHIC INFORMATION SYSTEM (GIS) DRAWINGS, UTILITY BASE MAPS, AND EXISTING UTILITY RECORD DRAWINGS. THE EXACT LOCATION OF THESE EXISTING UTILITIES AND CRITICAL POTENTIAL CONFLICTS SHOULD BE FIELD INVESTIGATED DURING THE DESIGN PHASE USING POT HOLES. ANY UTILITY CROSSING WITH LESS THAN 24 INCHES OF SEPARATION SHOULD BE CAREFULLY CONSIDRED TO AVOID FUTURE EXPOSURE OR CONFILCT.
2. ALL DEPTHS OF EXISTING UTILITES ARE ASSUMED.

CROSSECTION 5
-
NO SCALE

UTILITIES DEPICTED WERE OBTAINED FROM THE FOLLOWING REFERENCE DRAWINGS:
- CITY OF FULLERTON OFFICE OF THE DIRECTOR OF ENGINEERING DWG 879-S - ORANGETHORPE AVENUE SEWER MANHOLE

NOTE:
* WIDTH CAN REDUCE TO 6'-0" IF THERE IS 4'-0" OF TEMPORARY CONSTRUCTION EASEMENT AVAILABLE ON ADJACENT PROPERTY.

BLACK & VEATCH	0 5 10 15 FEET	**POTENTIAL REGIONAL RECYCLED WATER PROGRAM** **PRELIMINARY ALIGNMENT CROSS-SECTION 12 –** **ORANGETHORPE AVE FACING EAST (REACH 2, STA 565+50)**	**FIGURE 1-14**



NOTE:
1. THE LOCATIONS OF EXISTING UTILITIES WERE IDENTIFIED USING CITY PROVIDED GEOGRAPHIC INFORMATION SYSTEM (GIS) DRAWINGS, UTILITY BASE MAPS, AND EXISTING UTILITY RECORD DRAWINGS. THE EXACT LOCATION OF THESE EXISTING UTILITIES AND CRITICAL POTENTIAL CONFLICTS SHOULD BE FIELD INVESTIGATED DURING THE DESIGN PHASE USING POT HOLES. ANY UTILITY CROSSING WITH LESS THAN 24 INCHES OF SEPARATION SHOULD BE CAREFULLY CONSIDRED TO AVOID FUTURE EXPOSURE OR CONFILCT.
2. ALL DEPTHS OF EXISTING UTILITES ARE ASSUMED.

CROSS SECTION 6
-
NO SCALE

UTILITIES DEPICTED WERE OBTAINED FROM THE FOLLOWING REFERENCE DRAWINGS:
- CITY OF FULLERTON OFFICE OF THE DIRECTOR OF ENGINEERING DWG 879-S - PLAN AND PROFILE ORANGETHORPE AVENUE SEWER MANHOLE
- CITY OF FULLERTON OFFICE OF THE DIRECTOR OF PUBLIC WORKS DWG MISC. 5111 - TRAFFIC SIGNAL INTERCONNECT PLANS LEMON STREET

NOTE:
* WIDTH CAN REDUCE TO 6'-0" IF THERE IS 4'-0" OF TEMPORARY CONSTRUCTION EASEMENT AVAILABLE ON ADJACENT PROPERTY.

BLACK & VEATCH

0 5 10 15 FEET

POTENTIAL REGIONAL RECYCLED WATER PROGRAM
PRELIMINARY ALIGNMENT CROSS-SECTION 13 –
ORANGETHORPE AVE FACING EAST (REACH 2, STA 579+00)

FIGURE 1-15

Appendix V. 2018 Draft Report Pump Station Analysis

1.0 Pump Station Analysis

The pump station analysis presented in this section is based on the Final Preferred Alignment which provides a means to convey advanced treated water to the Santa Fe Spreading Grounds as well as the Orange County Spreading Grounds. This chapter was originally presented in the 2018 Draft Report in October 2018 as Chapter 7 "Pump Station Analysis" and is presented here for informational purposes.

This chapter provides feasibility-level design information for the pump stations that would be necessary to convey water from the Advanced Water Treatment Facility (AWTF) to the various groundwater recharge locations. The section begins with an overview of the pump station system and continues through more detailed discussions of key feasibility-level design criteria and features that would serve as a basis for subsequent design activities. Figure 1-1 summarizes the Project methodology as it applies to this chapter.

Phase	Phase 1 Metropolitan's Initial Evaluation	Phase 2 Alignment Verification and Initial Screening	Phase 3 Detailed Alternative Alignment Evaluation	Phase 4 Final Refinements	**Phase 5 Conceptual Pipeline and Pump Station Design**
Tasks	• Identification of potential pipeline alignments • Identification of Initial Base Case	• Data collection • Review of Metropolitan studies • Desktop analysis • Alternate alignment development • Field investigations • Initial screening • Desktop Geotechnical Report • Traffic Analysis and Impact Report • Constructability evaluations	• Development of decision model • Evaluation criteria • Weighting of evaluation criteria • Coarse screening • Secondary screening • Final screening • Ranking of alternatives	• Incorporation of stakeholder input • Conduct supplemental evaluations	• Steel cylinder thickness • Conceptual pipeline plan drawings • Pipe optimization • Hydraulic analysis and profile • Special construction zones • Preliminary alignment cross-sections • **Pump station site assessment** • **Conceptual pump station site plans** • **Pump station building** • **Hydraulic architectural and landscaping theme** • Cost development • Quantity take-off • Preliminary construction duration
Workshops		• Initial screening workshops	• Detailed evaluation workshops	• Workshops with Stakeholders • Workshops with Metropolitan's Environmental Team	• Pipeline focus meetings/ workshops • **Pump station focus meetings/ workshops** • Unit cost development workshops
Outcomes	• Initial Base Case alignment • Report entitled, "Potential Regional Recycled Water Supply Program – Conveyance System Feasibility Assessment"	• Revised Base Case alignment	• Initial Preferred Alignment	• Final Preferred Alignment	• **Conceptual pipeline and pump station design** • Engineer's cost opinion and Project schedule

Figure 1-1 Chapter Methodology

1.1 PUMP STATION OVERVIEW

This section describes the pump station system, the associated pump station components, and the analysis approach for developing the feasibility-level design information.

1.1.1 System Description

As described in Chapter 5, multiple pump stations would be required to convey recycled water flow from the AWTF to the anticipated discharge locations, which are located several miles away and at higher elevations than the AWTF. Table 1-1 summarizes the approximate ground elevations of these discharge points. The ground elevation at the AWTF is approximately 42 ft. All elevations are relative to mean sea level (MSL).

Table 1-1 Groundwater Recharge Location Elevations

RECHARGE LOCATION	APPROXIMATE GROUND ELEVATION (FT)
Potential Future (West Coast Basin) Injection Wells	90
Potential Future (Central Basin/Long Beach) Injection Wells	60
OC Spreading Grounds	230
Rio Hondo Spreading Grounds (Montebello Forebay)	145
Santa Fe Spreading Grounds	485-500

As discussed in Chapter 5, it is recommended that the following two concepts be carried forward to preliminary design based on the Final Preferred Alignment:

- **Alternative A – Without PS-2.** This concept comprises two pump stations where PS-1 pumps to both the Orange County Spreading Grounds and PS-3 via a transmission pipeline which splits into two branches near Carson Street and the San Gabriel River. A flow control facility would be required on one or both transmission pipe branches beyond the split. PS-3 would further pump the flow to the Santa Fe Spreading Grounds.
- **Alternative B – Construct PS-2 with Storage Tank at Signal Hill.** This concept includes three pump stations in which a smaller PS-1 (as compared to Alternative A) would convey water to a storage tank on Signal Hill, which would then feed PS-2 located near Carson Street and the San Gabriel River. PS-2 would feed the Orange County Spreading Grounds and PS-3, with PS-3 pumping the flow to the Santa Fe Spreading Grounds similar to Alternative A.

As described in Chapter 5, Metropolitan also evaluated alternatives to modify or augment the RRWP should DPR become feasible. The pumping configuration under the selected alternative would be as follows:

- **Alternative A-Backbone System – Potential for DPR.** This concept comprises two pump stations where PS-1 pumps directly to PS-3. This concept does not include PS-2 nor a junction structure at the proposed location of PS-2. Thus, pumping to the Orange County Spreading Grounds is not included.

Table 1-2 summarizes the proposed pump stations, including their general locations, capacities, and configuration. PS-1 and PS-2 both would have two sets of pumps and discharge pipelines to deliver recycled water to two separate discharge locations. PS-3 would have one set of pumps to send recycled water to the Santa Fe Spreading Grounds, with the Rio Hondo Spreading Grounds being served by gravity from the storage tank at PS-3.

Table 1-2 Summary of Pump Station Attributes

PUMP STATION	GENERAL LOCATION (WITH APPROXIMATE GROUND ELEVATION)	PRELIMINARY FIRM CAPACITY	PUMPS TO
Alternative A			
PS-1	AWTF/JWPCP, Carson (42 ft)	Set A: 15 mgd Set B: 150 mgd	Set A: West Basin Set B: PS-3 Forebay, Orange County Spreading Grounds, Long Beach
PS-3	Near Whittier Narrows, Pico Rivera (220 ft)	Set A: 80 mgd	Set A: Santa Fe Spreading Grounds
Alternative B			
PS-1	AWTF/JWPCP, Carson (42 ft)	Set A: 15 mgd Set B: 150 mgd	Set A: West Basin Set B: Signal Hill storage tank, Long Beach
PS-2	Adjacent to San Gabriel River near Carson Street, Cerritos (44 ft)	Set A: 60 mgd Set B: 80 mgd	Set A: Orange County Spreading Grounds Set B: PS-3 Forebay
PS-3	Near Whittier Narrows, Pico Rivera (220 ft)	Set A: 80 mgd	Set A: Santa Fe Spreading Grounds
Alternative A-Backbone System			
PS-1	AWTF/JWPCP, Carson (42 ft)	Set A: 15 mgd Set B: 150 mgd	Set A: West Basin Set B: PS-3 Forebay
PS-3	Near Whittier Narrows, Pico Rivera (220 ft)	Set A: 150 mgd	Set A: Santa Fe Spreading Grounds

1.1.2 Station Components

Each pump station would have similar components that would be adjusted to account for the station's specific location and capacity. The components reflected in the feasibility-level design include, but are not limited to, the following:

- Main pump area: This area would include the pumps and motors, surge tank air compressors, and administration area. At PS-1, the pumping equipment itself would be outdoors with a building sized just for administration, storage, and air compressors. At PS-2 and PS-3, all the equipment associated with this area would be located within a building.
- Surge control area: This area would include above-grade, air-over-water hydropneumatic surge tanks and associated piping. The tanks would be located outdoors, and would be shielded by a curtain wall.
- Pump station forebay/suction storage facility: At PS-1 and PS-3, this would be an above grade circular tank. PS-2 would not have a storage facility onsite as storage is provided upstream at the Signal Hill Tank.

- Dechlorination facility on storage tank overflow: This structure, mostly located below-grade, would use granular activated carbon to dechlorinate any overflow before entering offsite drainage channels. This component would only be required at PS-1 and PS-3; it is not necessary at PS-2 since that facility does not have a storage tank.
- Electrical room/building: This building would house the main electrical equipment for the station, including variable frequency drives (VFDs) and switchgear.
- Electrical transformer area: This area would house the electrical transformers that feed the electrical room/building.
- Miscellaneous facilities, including valve and meter vaults.

1.1.3 Analysis Approach

The feasibility-level design of the pump stations described herein is based on first establishing a conceptual operating strategy describing how the multiple pump stations would be controlled. This was followed by determining the preliminary size of the pumping equipment (flow, head, and power) based on the conveyance system configuration described in the previous sections. With basic control and equipment sizing established, the ancillary facilities were sized. The information provided is at the feasibility-level and will be refined and detailed in subsequent design phases. Preliminary calculations and equipment selections supporting the feasibility-level design are included in Appendix J.

1.2 CONCEPTUAL OPERATING STRATEGY

The pump stations must operate and be controlled in a carefully coordinated manner to deliver flow at the required rates to the various discharge points. The method of control will dictate design of the pump stations, including the size of storage facilities and size and speed ranges of pumping equipment. This section describes a conceptual control strategy for the system that was developed to guide the subsequent conceptual operation of the pump stations. There are alternate control strategies that may be investigated during detailed design.

1.2.1 Overall Conceptual Control Strategy

In general, the proposed primary control strategy is based on coordinated flow set points calculated for each set of pumps/flow control stations based on AWTF production and desired delivery points. These set points would be communicated to each set of pumps/flow control stations and associated flow meters so that the flow rate entering a pump station would be equal to the flow rate leaving a pump station. The Alternative A concept is shown on Figure 1-2 while Alternative B is shown on Figure 1-3. The Alternative A – Backbone System would be similar to Alternative A, except the branch to Orange County and its associated flow control station would be omitted, and the flow control station immediately upstream of PS-3 would be omitted.

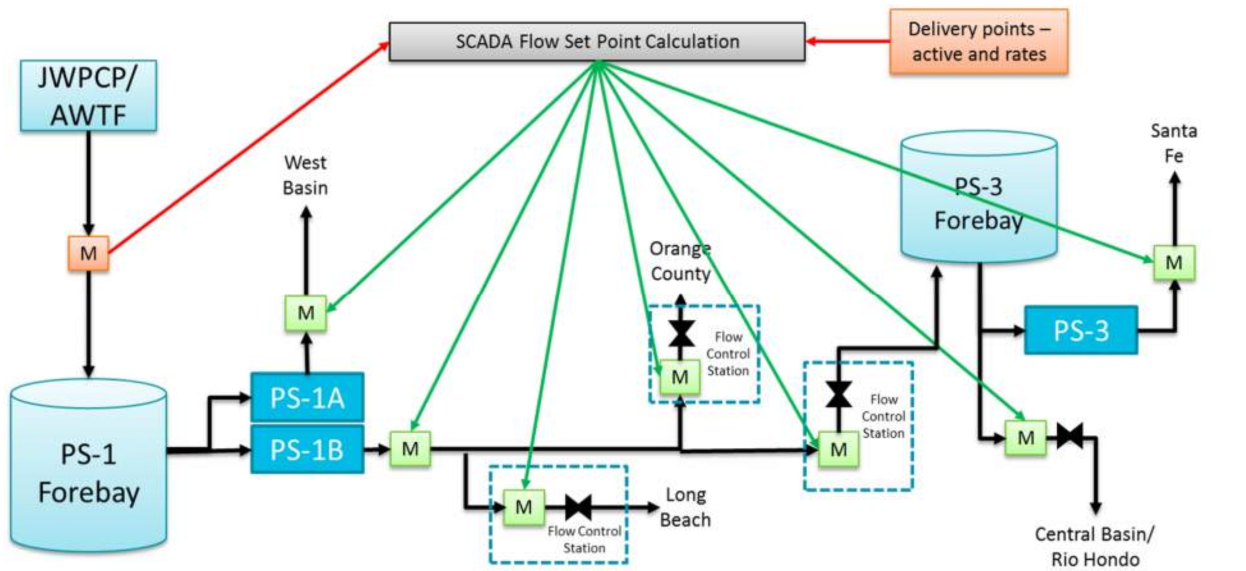


Figure 1-2 **Overall Control Strategy Concept – Alternative A**

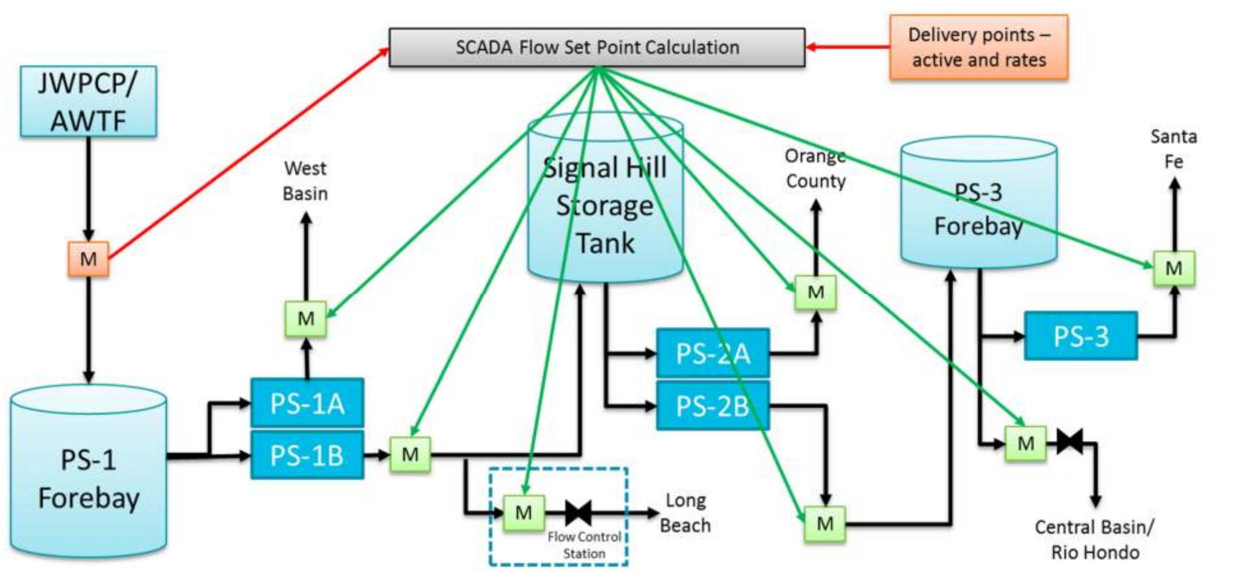


Figure 1-3 **Overall Control Strategy Concept – Alternative B**

The flow set points would be achieved by modulating the VFD-driven pumps or flow control valves to meet the flow set point. The flow set point would be modified, or trimmed, based on the level in the upstream storage tank. For example, if the level in the tank were rising above a desired level set point, the flow set point of the downstream pumps would be increased until stable tank levels are achieved. The control approach for PS-1 is illustrated on Figure 1-4. This general control framework would be supplemented by a range of control interlocks to keep the stations operating within designated parameters, which will reduce the risk of unanticipated operating scenarios. These interlocks are discussed in greater detail below.

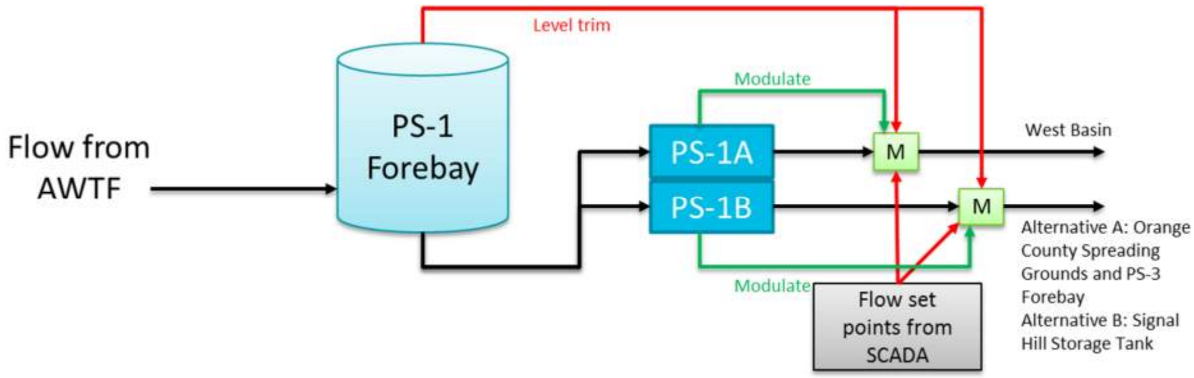


Figure 1-4 Flow Control with Level Trim and PS-1

The goal of the conceptual control strategy described above is to achieve stable tank levels, typically at around 50 percent of the forebay tank depth. When the system is stable, tank level should not change, and the need for storage would be minimal. However, there would be instances, especially during normal starting and stopping of the system, when flow imbalances would be expected to occur and the level in the forebay storage tank would either go up or down.

To estimate the volume associated with a flow imbalance during normal starting and stopping operations, a conceptual starting and stopping sequence was developed as depicted on Figure 1-5 and Figure 1-6. The ramp-up times for the system to start (time for pump to accelerate from OFF to the preset speed) were estimated at 2 minutes, which is expected to exceed the critical period for the longest length of pipe to reduce pressure surges. The "critical period" is the time required for an acoustic wave to travel from the pump station to the end of the pipe and back.

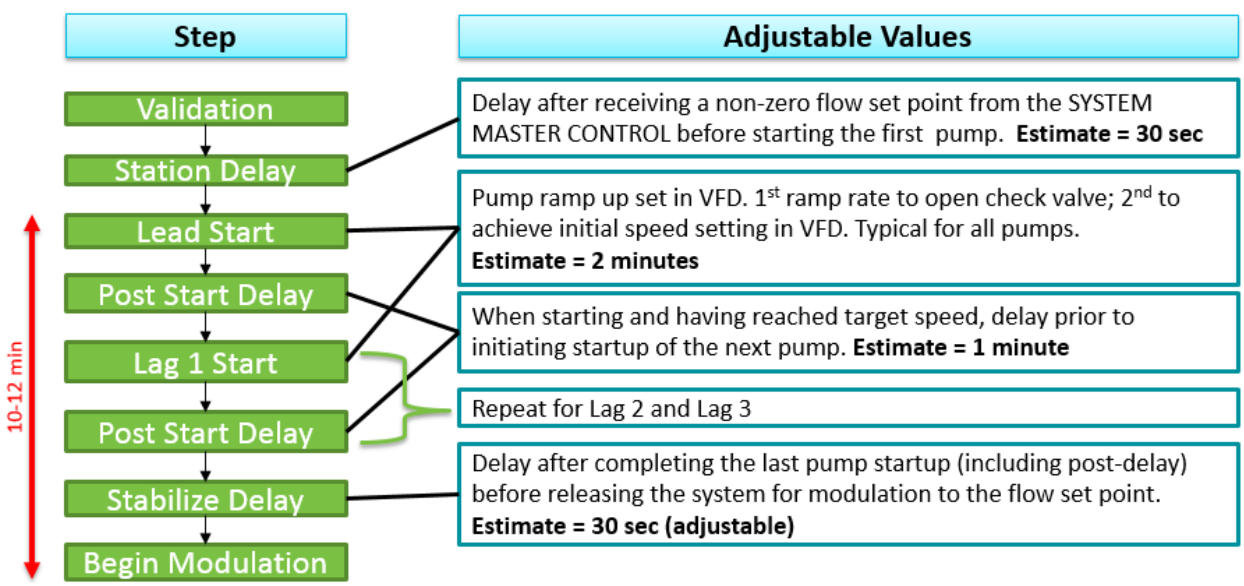


Figure 1-5 Conceptual Starting Sequence

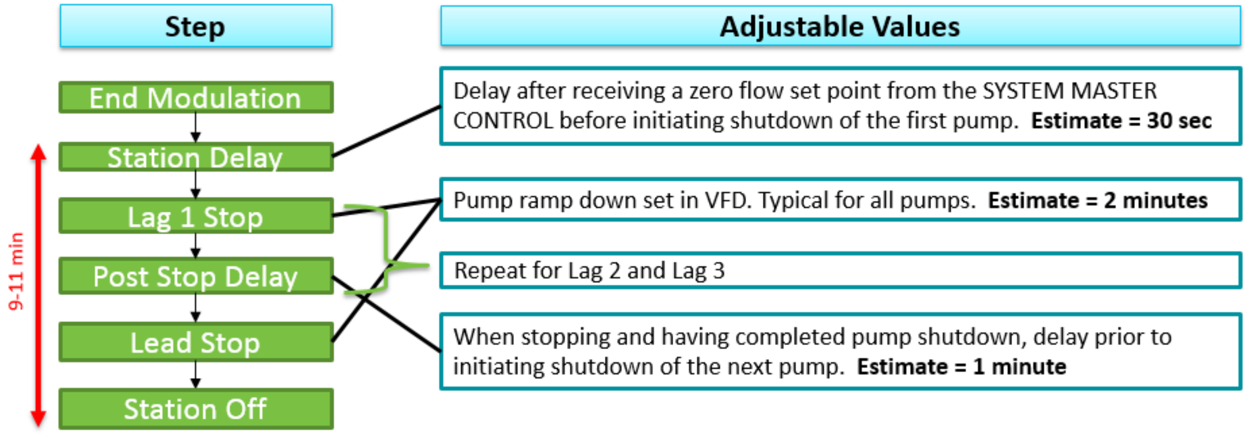


Figure 1-6 Conceptual Stopping Sequence

The estimated time for a controlled startup would range from 10-12 minutes based on the initial estimated ramping rates and control delays. The time for a controlled ramp down would range from 9-11 minutes. An emergency stop would happen essentially instantaneously as power is cut to the pumps and they decelerate (i.e., spin down) according to the system inertial characteristics. In an emergency stop scenario, the stored energy in the hydro-pneumatic surge control tanks would help to gradually reduce the flow and protect the system from damaging hydraulic surge conditions.

1.2.2 Control System Interlocks and Backup Systems

The control system for the conveyance system would be designed with various features to prevent the system from operating outside of design parameters. These features would include software and hardwired interlocks as well as backup control systems. Examples of interlocks that would be implemented include:

- Level transmitters – high or low tank level shuts down upstream/downstream of pump station.
- Redundant high and low float switches in tanks, hardwired to pumps - high or low tank level would shut down upstream/downstream of a pump station.
- Pressure transmitter/switches – out of range would shut down pump stations.
- If one station were to shut down, then all stations would shut down.
- Peer-to-peer heartbeat: if pump stations were to lose communication, all pump stations would shut down after a set delay.
- Loss of communication time-out: if a pump station would be unable to communicate, it would shut down.
- Flow coordination check routines in software to make sure flow rates at each station would match.
- Redundant operator verifications to modify automatic controls and interlocks.

Examples of backup control systems include switching to local level control if communication is lost. In this scenario, the pump station would operate to maintain the level in its associated upstream storage tank. This would prevent overflow of the local storage tank; however, it would not prevent overflow of the downstream storage tank if that facility was shut down. Thus, loss of communication is likely a scenario that would require a shutdown.

1.3 PUMP STATION HYDRAULIC ANALYSIS AND PUMP EVALUATION

This section describes the hydraulic analysis performed to determine preliminary sizing of the pumping equipment at each station. Specifically, this section describes system curve development, pumping equipment characteristics, and preliminary pump selections.

1.3.1 System Curve Development

System curves were developed for each set of pumps to document the required total dynamic head at the pump stations from the static condition to the maximum capacity. These curves were then used to select candidate pumping equipment. Detailed preliminary system curve calculations are provided in Appendix J. The following system curves were developed for each station to provide an envelope of operating points:

- High Manning's: This system curve assumes low suction tank level, high discharge tank level, and calculation of friction losses using the Manning's equation with n=0.012, as prescribed by Metropolitan's Hydraulic Design Manual. This results in the highest head condition and was the basis for the rated point on pump selections. Since this was considered to likely be a conservative condition, this point was selected left of best-efficiency point (BEP) when selecting pumps, which would provide additional runout capacity for lower head conditions when fewer pumps are operating.
- Low Manning's: This system curve assumes high suction tank level, low discharge tank level, and calculation of friction losses using the Manning's equation with n=0.012, as prescribed by Metropolitan's Hydraulic Design Manual.
- High Darcy: This system curve assumes low suction tank level, high discharge tank level, and calculation of friction losses using the Darcy-Weisbach equation with a surface roughness of 0.000225 ft, which is considered at the upper range for cement mortar lined steel pipe. The value of 0.000225 ft is 1.5 times 0.00015 ft, the surface roughness used in the Low Darcy scenario.
- Low Darcy: This system curve assumes high suction tank level, low discharge tank level, and calculation of friction losses using the Darcy-Weisbach equation with a surface roughness of 0.000015 ft, which is considered at the lower range for cement mortar lined steel pipe. This curve was the lowest estimated system curve. If possible, pumps were selected to also intercept this curve to prevent runout of a single pump at 100 percent speed. However, in some cases this would not be possible due to the relatively high friction head for some of the pump sets and would require limiting pump operating speeds for single pump operation, which is readily achievable with VFD operation and control.

1.3.1.1 PS-1 System Curves

Table 7-3 summarizes the key inputs used to develop the system curve for PS-1 and the resulting rated design point used for subsequent pump selection. The key inputs include suction tank water surface elevation (WSE) range, discharge elevation, discharge pipe length and diameter, and the rated point for pump selection.

Table 1-3 PS-1 System Curve Inputs

PARAMETER	SET A	SET B
Alternative A		
Suction Tank (PS-1) WSE Range (ft)	44 - 74[1]	44 - 74
Discharge Elevation (ft)	136	Segment 1: 50[2] Segment 2: 230[3] Segment 3: 220[4]
Discharge Pipe Length (ft)	26,400	Segment 1: 68,478 Segment 2: 83,172 Segment 3: 73,000
Discharge Pipe Diameter (in)	30	Segment 1: 84 Segment 2: 54 Segment 3: 60
Rated Point for Pump Selection	**7.5 mgd at 165 ft**	**37.5 mgd at 428 ft**
Alternative B		
Suction Tank (PS-1) WSE Range (ft)	44 - 74[1]	44 - 74
Discharge Elevation (ft)	136	180
Discharge Pipe Length (ft)	26,400	33,726
Discharge Pipe Diameter (in)	30	84
Rated Point for Pump Selection	**7.5 mgd at 165 ft**	**37.5 mgd at 174 ft**
Alterative A-Backbone System		
Suction Tank (PS-1) WSE Range (ft)	44 - 74[1]	44 - 74
Discharge Elevation (ft)	136	222
Discharge Pipe Length (ft)	26,400	141,478
Discharge Pipe Diameter (in)	30	84
Rated Point for Pump Selection	**7.5 mgd at 165 ft**	**37.5 mgd at 352 ft**

Notes:
1. Assuming ground elevation of 42 ft with a tank level range of 2 ft to 32 ft.
2. Segment 1: PS-1 to flow split junction near Carson Street and the San Gabriel River.
3. Segment 2: Junction to Orange County Spreading Grounds.
4. Segment 3: Junction to PS-3.

Figure 1-7, Figure 1-8, Figure 1-9, and Figure 1-10 present the associated system curves developed for PS-1 Set A and Set B, respectively. PS-1 Set B pumps under Alternative A and Alternative A-

Backbone System have higher head than under Alternative B and therefore will have a higher motor rating and associated costs. The curves include an overlay from one of the candidate pump selections.

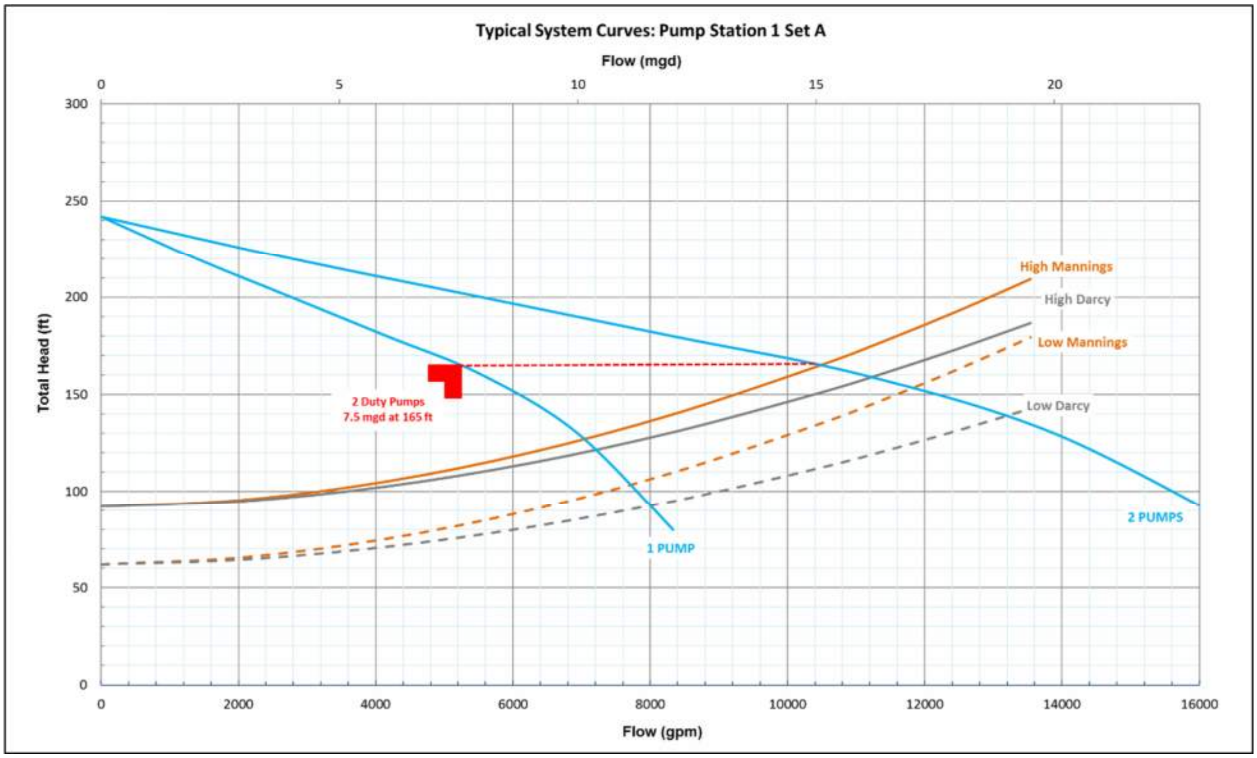


Figure 1-7 PS-1 Set A System Curves

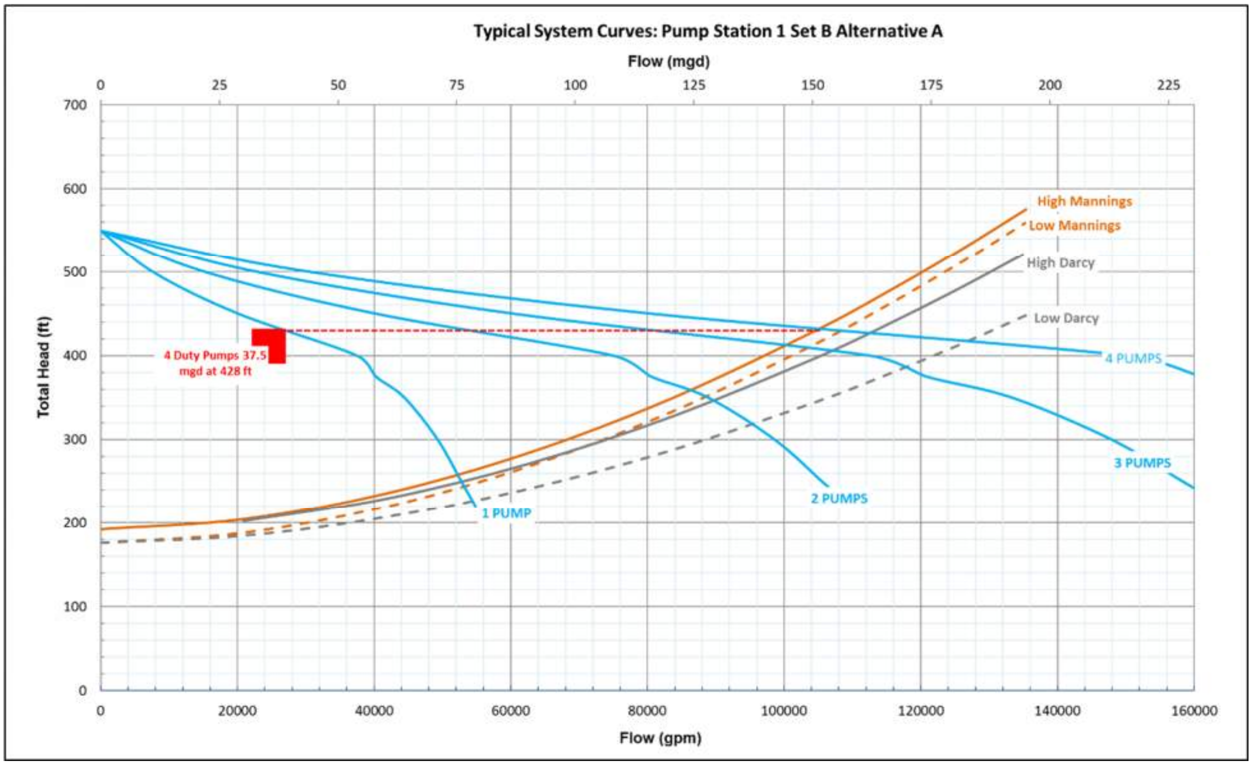


Figure 1-8 PS-1 Set B System Curves – Alternative A

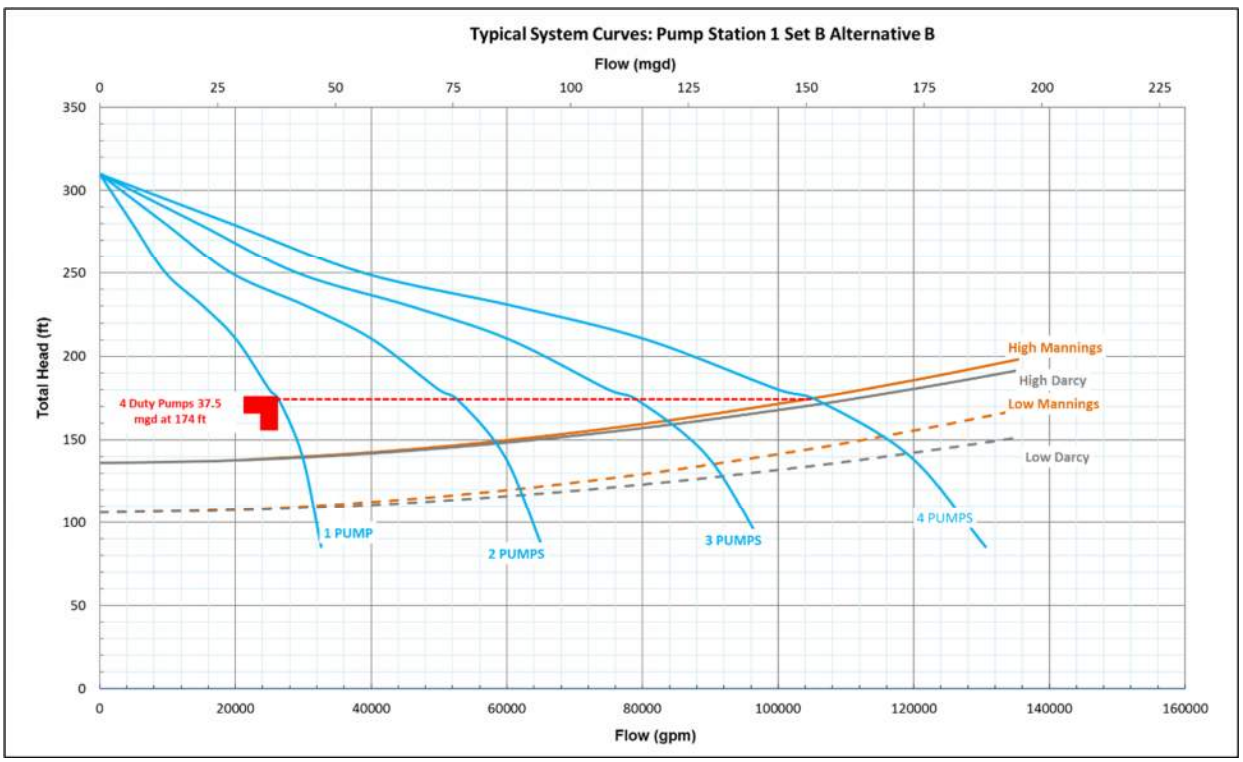


Figure 1-9 PS-1 Set B System Curves – Alternative B

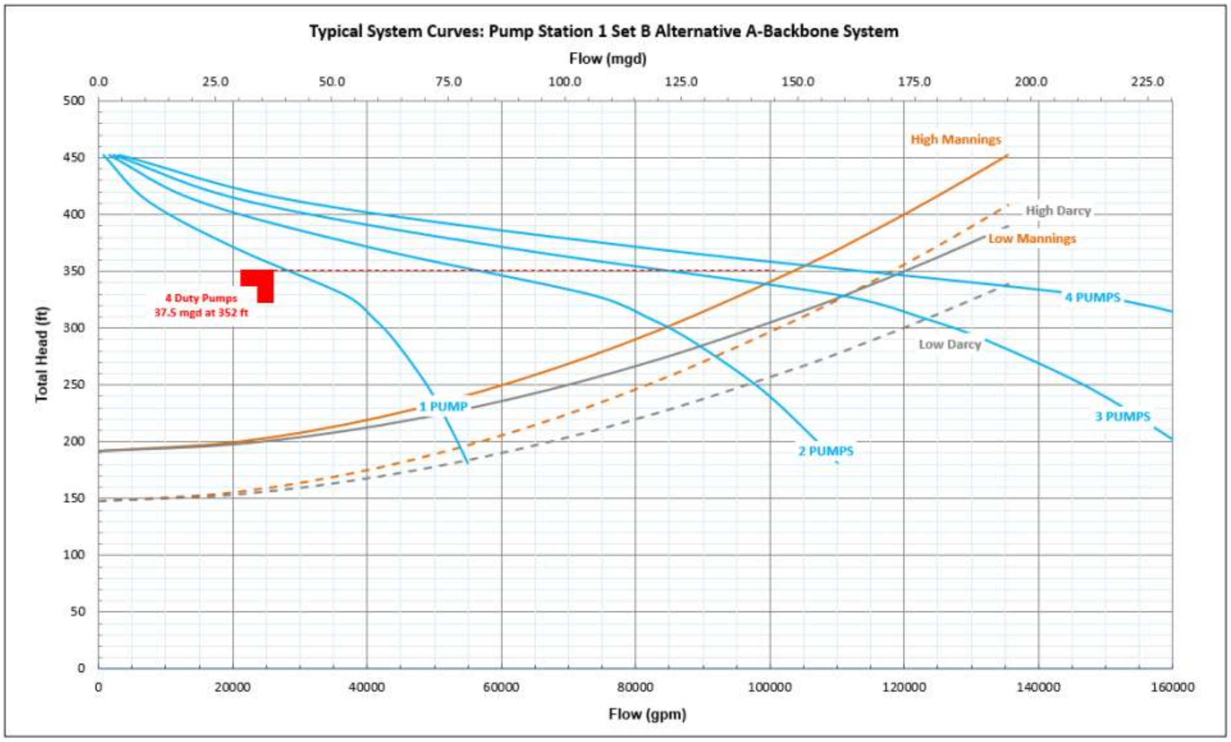


Figure 1-10 PS-1 Set B System Curves – Alternative A-Backbone System

1.3.1.2 PS-2 System Curves

Table 1-4 summarizes the key inputs used for Alternative B to develop the system curve for PS-2 and the resulting rated design point used as the basis for subsequent pump selection.

Table 1-4 PS-2 System Curve Inputs (Alternative B)

PARAMETER	SET A	SET B
Suction Tank (Signal Hill) WSE Range (ft)	182-196[1]	182-196
Suction Pipe Length (ft)	34,759	34,759
Discharge Elevation (ft)	230	220
Discharge Pipe Length (ft)	83,172	73,000
Discharge Pipe Diameter (in)	54	60
Rated Point for Pump Selection	20 mgd at 266 ft	26.7 mgd at 235 ft

Note:
1. Assuming ground elevation of 180 ft at Signal Hill with a tank level range of 2 ft to 16 ft.

Figure 1-11 and Figure 1-12 present the associated system curves developed for PS-2 Set A and Set B, respectively. The curves include an overlay from one of the candidate pump selections (see Section 7.3.3).

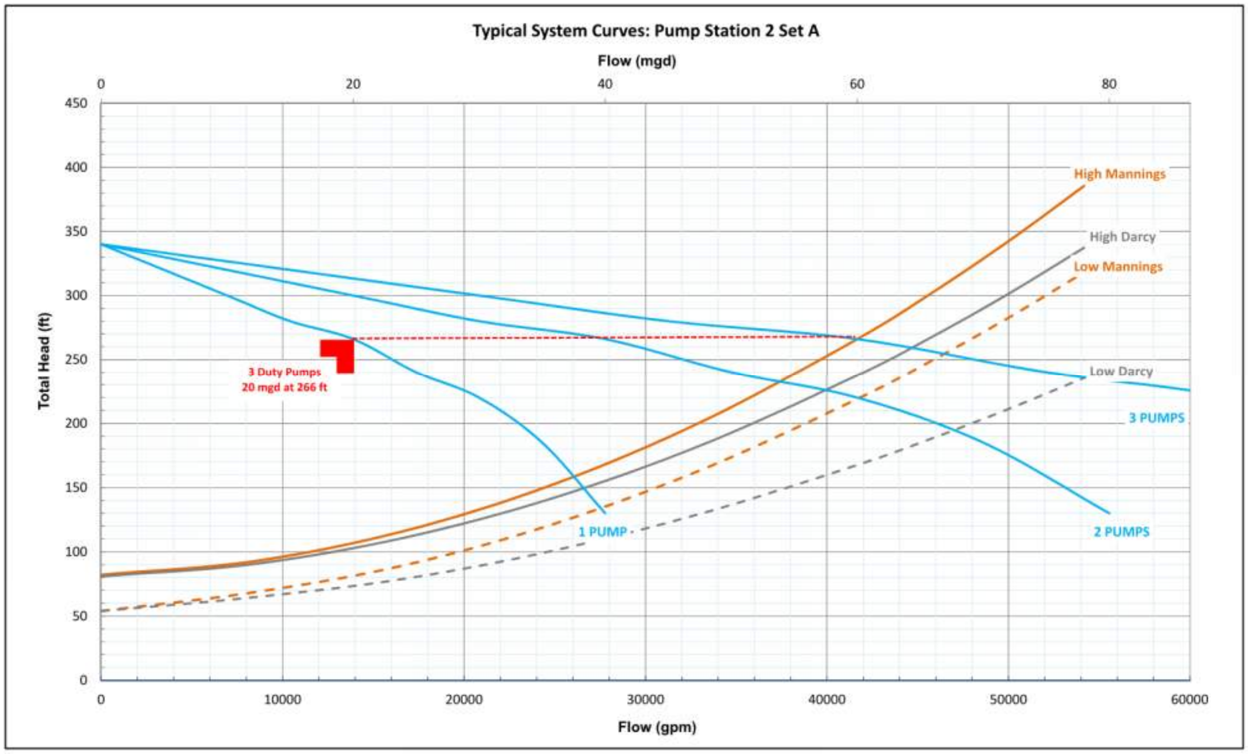


Figure 1-11 PS-2 Set A System Curves (Alternative B only)

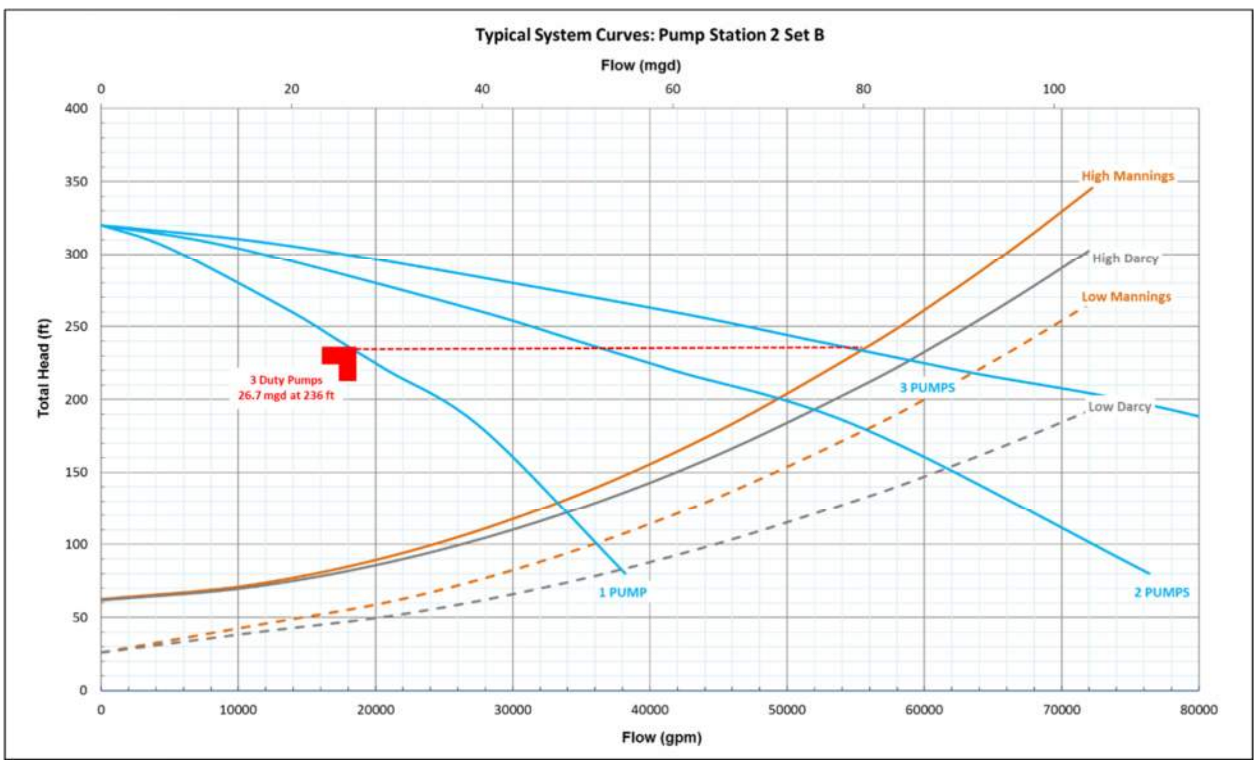


Figure 1-12 PS-2 Set B System Curves (Alternative B only)

1.3.1.3 PS-3 System Curves

Table 1-5 summarizes the key inputs used for both Alternative A and B to develop the system curve for PS-3 and the resulting rated design point used as the basis for subsequent pump selection.

Table 1-5 PS-3 System Curve Inputs

PARAMETER	SET A
Alternatives A and B	
Suction Tank (PS-3) WSE Range (ft)	222 - 236[1]
Discharge (Santa Fe Spreading Grounds) Water Surface Elevation with 20 ft Distribution Head (ft)	505
Discharge Pipe Length (ft)	58,800
Discharge Pipe Diameter (in)	60
Rated Point for Pump Selection	**26.7 mgd at 397 ft**
Alterative A-Backbone System	
Suction Tank (PS-3) WSE Range (ft)	222 - 236[1]
Discharge (Santa Fe Spreading Grounds) Water Surface Elevation with 20 ft Distribution Head (ft)	505
Discharge Pipe Length (ft)	58,800
Discharge Pipe Diameter (in)	**84**
Rated Point for Pump Selection	**37.5 mgd at 352 ft**

Note:
1. Assuming ground elevation of 220 ft with a tank level range of 2 ft to 16 ft.

Figure 1-13 and Figure 1-14 present the associated system curves developed for PS-3. The curves include an overlay from one of the candidate pump selections (see Section 7.3.3).

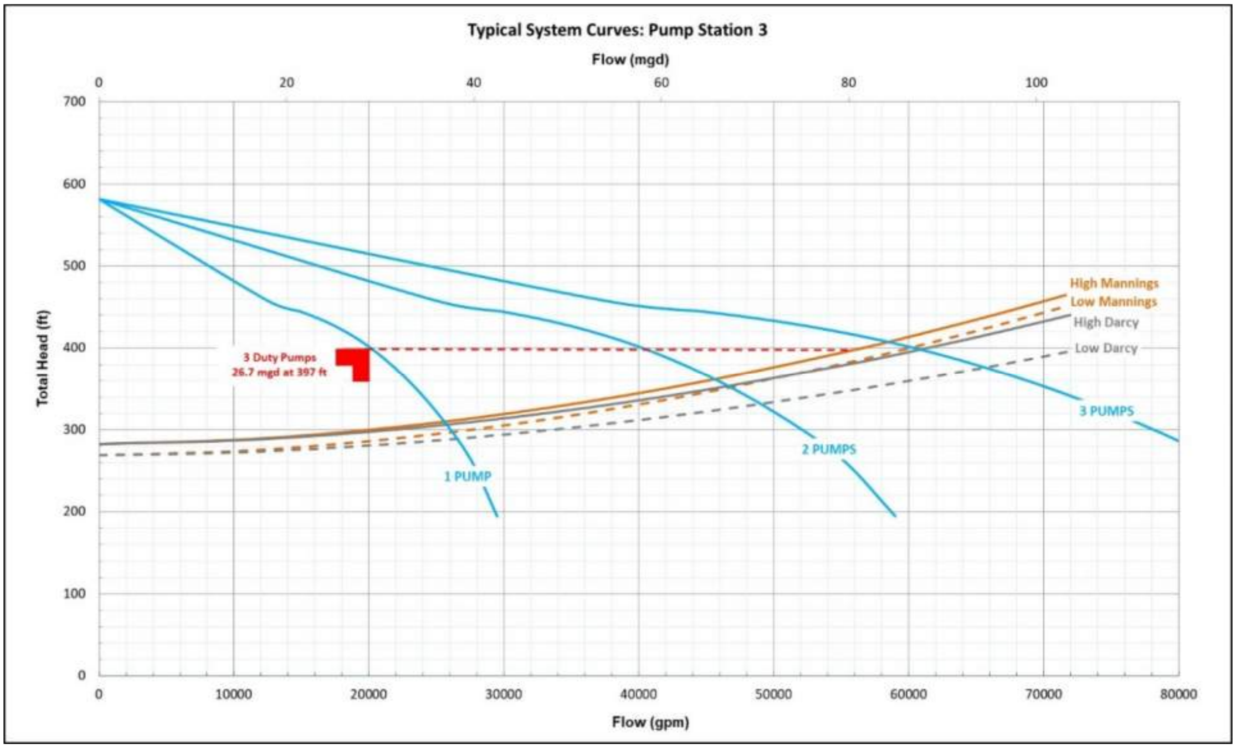


Figure 1-13 PS-3 System Curves (Alternative A and B)

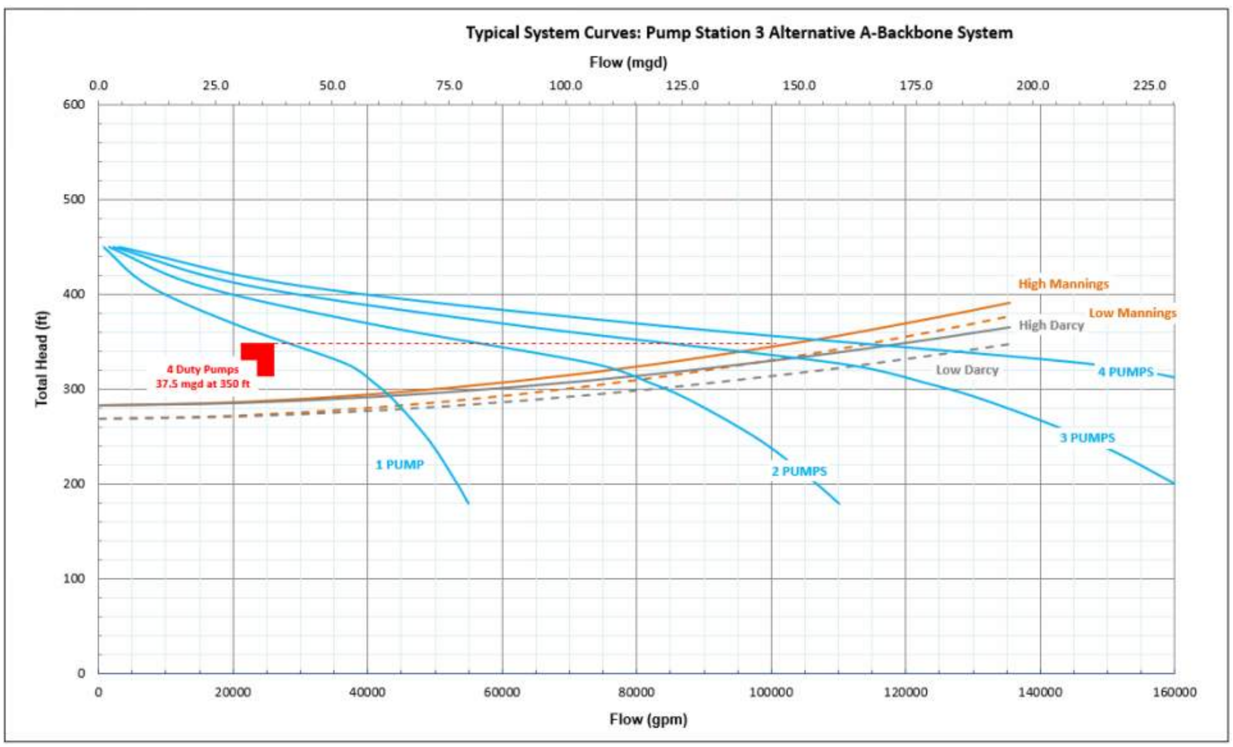


Figure 1-14 PS-3 System Curves (Alternative A-Backbone System)

1.3.2 Pumping Equipment

The recommended pumping equipment for the Project is vertical turbine pumps. These pumps have a smaller footprint than horizontal pumps, are familiar to Metropolitan staff, and offer efficient operation across the range of flows and heads that are being contemplated. It is proposed that the vertical turbine pumps would be installed in cans/barrels and separated from the water storage tank.

1.3.3 Feasibility-level Pump Selection

The hydraulic conditions described in Section 1.3.1 were used to identify candidate pumping equipment that meets the preliminary performance requirements. Initial curves were selected from three typical manufacturers: Fairbanks, Ebara, and Sulzer. These preliminary selections are summarized in Table 1-6, and the associated performance curves are included in Appendix K. The purpose of these selections was to demonstrate the availability of equipment in these sizes from multiple manufacturers and to verify motor sizes to develop the feasibility-level electrical system design (see Section 1.8.1). In subsequent design phases, the following additional analyses are recommended to optimize the pump selections:

- Refine system hydraulic calculations to include station specific losses, final pipeline alignments and hydraulic properties, and final pump station locations.
- Identify the relative frequency of various operating conditions and optimize selections to minimize power consumption.
- Investigate selections from other acceptable manufacturers to identify optimal selections and increase procurement competition.
- Develop detailed technical specifications based on Metropolitan's requirements for pumping equipment with modifications specific to the proposed service of the equipment.

Table 1-6 Summary of Feasibility-level Pump Selection

STATION	RATED DESIGN POINT	FAIRBANKS NIJHUIS	EBARA	SULZER
PS-1 Set A	7.5 mgd at 165 ft	27ML-BRZ 890 RPM, 300 horsepower (HP)	600X400VYBM 890 RPM, 350 HP	SJT-28GMC 885 RPM, 350 HP
PS-1 Set B (Alt A)	37.5 mgd at 428 ft	63HRO 7000 592 RPM, 4,500 HP	1500X1000VYB2M 710 RPM, 5,000 HP	SJT-56TMC 595 RPM, 4,000 HP
PS-1 Set B (Alt B)	37.5 mgd at 174 ft	44A-BRZ 705 RPM, 1,500 HP	1500X900VYBM 710 RPM, 1,500 HP	SJT-38KMC 705 RPM, 1,750 HP
PS-1 Set B (Alt A-Backbone)	37.5 mgd at 352 ft	63HRO 7000 592 RPM, 4,500 HP	1500X1000VYB2M 710 RPM, 5,000 HP	SJT-56TMC 595 RPM, 4,000 HP
PS-2 Set A	20 mgd at 266 ft	36G-BRZ 880 RPM, 1,500 HP	1000X800VYBM 890 RPM, 1,750 HP	SJT-BKn 680/022 880 RPM, 1,500 HP

STATION	RATED DESIGN POINT	FAIRBANKS NIJHUIS	EBARA	SULZER
PS-2 Set B	26.7 mgd at 235 ft	44B-BRZ 705 RPM, 1,750 HP	12000X900VYBM 890 RPM, 1,750 HP	SJT-BKn 840/022 705 RPM, 1,750 HP
PS-3	26.7 mgd at 397 ft	48HRO 7000 710 RPM, 2,750 HP	1200X800VYB2M 890 RPM, 2,750 HP	SJT-42CLC 705 RPM, 2,750 HP
PS-3 (Alt A-Backbone)	37.5 mgd at 352 ft	63HRO 7000 592 RPM, 4,500 HP	1500X1000VYB2M 710 RPM, 5,000 HP	SJT-56TMC 595 RPM, 4,000 HP

1.3.4 Suction and Discharge Piping Sizing

As mentioned in Section 1.3.2, the vertical turbine pumps are proposed to be installed in cans/barrels. Recycled water would be supplied from the storage tanks via a suction header pipe with suction laterals feeding each pump can.

Per Hydraulic Institute (HI) Standard 9.8 - Intake Design for Rotodynamic Pumps, the maximum flow velocity recommended for a suction lateral entering a closed-bottom can below the elevation of the discharge lateral is 4 ft per second (fps). Table 1-7 provides a summary of the flow velocities that can be anticipated in the suction laterals for the corresponding pump sets. The pipe sizes have capacity to accommodate a maximum flow rate of 150 percent of the design flow rate. The maximum flow rates were determined based on the can sizing, as discussed in Section 1.3.5, and also to provide flexibility to operate individual pumps across a wider range of flows. It was assumed that the pump VFDs would limit maximum runout conditions to maintain flow velocities below 4 fps. Detailed suction lateral sizing calculations are provided in Appendix J.

Table 1-7 Preliminary Suction Lateral Sizing

PUMPS	PIPE SIZE (IN.)	DESIGN FLOW RATE (MGD)	FLOW VELOCITIES (FPS)(1)
PS-1 Set A	30	7.5	2.4 – 3.6
PS-1 Set B	66	37.5	2.4 – 3.7
PS-1 Set B (Alt A-Backbone)	66	37.5	2.4 – 3.7
PS-2 Set A	48	20	2.5 – 3.7
PS-2 Set B	54	26.7	2.6 – 3.9
PS-3	54	26.7	2.6 – 3.9
PS-3 (Alt A-Backbone)	66	37.5	2.4 – 3.7

Note:
1. Velocity range: lower limit at design flow rate, upper limit at 150% of design flow rate.

HI Standard 9.6.6 - Rotodynamic Pumps for Pump Piping, recommends that pipe sizes for pump discharge laterals be designed to limit flow velocities to 15 fps. For the purposes of this evaluation, the maximum allowable flow velocity is assumed to be 10 fps in order to reduce both friction losses and life-cycle costs for each station. Table 1-8 provides a summary of the flow velocities that can be anticipated in the discharge laterals for the corresponding pump sets. Detailed discharge lateral sizing calculations are provided in Appendix J.

Table 1-8 Preliminary Discharge Lateral Sizing

PUMPS	PIPE SIZE (IN.)	DESIGN FLOW RATE (MGD)	FLOW VELOCITY (FPS)
PS-1 Set A	16	7.5	8.2
PS-1 Set B	36	37.5	8.2
PS-1 Set B (Alt A-Backbone)	36	37.5	8.2
PS-2 Set A	24	20.0	9.9
PS-2 Set B	30	26.7	8.4
PS-3	30	26.7	8.4
PS-3 (Alt A-Backbone)	36	37.5	8.2

1.3.5 Pump Can Sizing

As part of the initial pump sizing described in Section 1.3.3, the manufacturers provided estimated sizing for the pump cans. HI Standard 9.8 provides maximum velocities to guide the sizing of various aspects of the pump cans/barrels. The maximum velocity through the barrel at both the bowl and the bell is 5 fps. Figure 1-15 shows the standard configuration of a pump can and the acceptable dimensions and velocities per HI Standard 9.8.

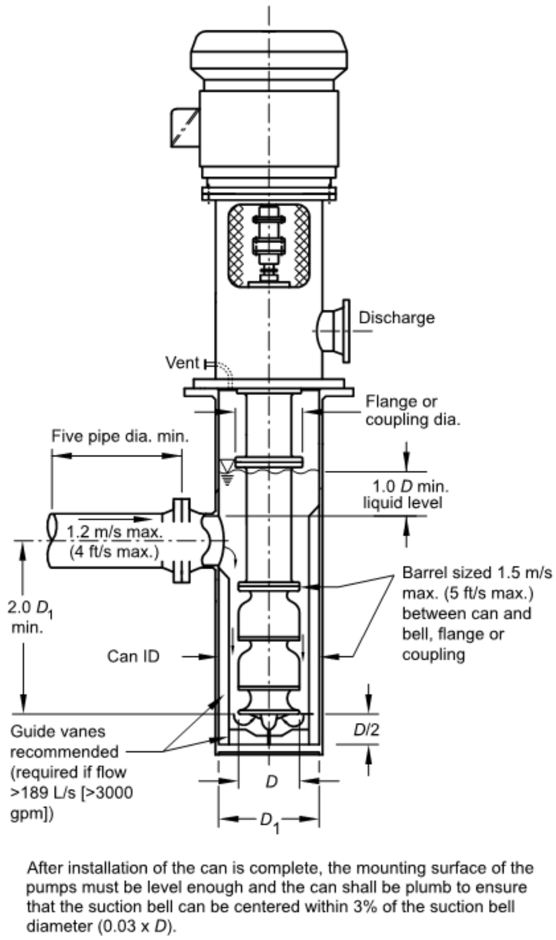


Figure 1-15 Closed Bottom Can Standard Configuration

The can sizing provided by Fairbanks Nijhuis, including the inside diameter (ID) of the barrel, outside diameter (OD) of the bowl, and OD of the bell, were used to estimate the maximum allowable flow rate through the pump can by limiting the velocity through the barrel to 5 fps. The desired maximum flow rate is 125 to 150 percent of the design flow rate. The pump can dimensions and maximum flow rates are presented in Table 1-9 and Table 1-10. Detailed can sizing calculations are provided in Appendix J.

Table 1-9 Preliminary Pump Can/Barrel Sizing – Fairbanks Nijhuis

PUMPS	ID OF BARREL (IN.)	OD OF BOWL (IN.)	OD OF BELL (IN.)
PS-1 Set A	36.75	26.60	22.50
PS-1 Set B (Alt A)	96.00	64.00	64.00
PS-1 Set B (Alt B)	71.25	43.00	40.00
PS-1 Set B (Alt A-Backbone)	96	64	64

PUMPS	ID OF BARREL (IN.)	OD OF BOWL (IN.)	OD OF BELL (IN.)
PS-2 Set A	60.00	35.75	40.00
PS-2 Set B	66.00	43.00	43.00
PS-3	72.00	48.00	48.00
PS-3 (Alt A-Backbone)	96	64	64

Table 1-10 Preliminary Pump Can/Barrel Maximum Flow Rates

PUMPS	DESIGN FLOW RATE (GALLONS PER MINUTE [GPM])	MAXIMUM FLOW RATE (GPM)(1)	MAXIMUM VELOCITY IN BARREL AT BOWL (FPS)	MAXIMUM VELOCITY IN BARREL AT BELL (FPS)
PS-1 Set A	5,208	7,813	4.98	3.63
PS-1 Set B (Alt A)	26,042	39,063	3.13	3.13
PS-1 Set B (Alt B)	26,042	39,063	4.96	4.61
PS-1 Set (Alt A-Backbone)	26,042	39,063	3.13	3.13
PS-2 Set A	13,889	20,833	3.68	4.27
PS-2 Set B	18,542	27,813	4.55	4.55
PS-3	18,542	27,813	3.96	3.96
PS-3 (Alt A-Backbone)	26,042	39,063	3.13	3.13

Note:
1. 150% of design flow rate.

1.4 PUMP STATION BUILDING

The pumping equipment, discharge piping and valves, and surge tank air compressors would be housed in a building at PS-2 and PS-3, along with areas for maintenance and administrative functions (control room, storage, etc.). Since PS-1 would be located at a treatment plant facility, the pumping equipment at that site would be outdoors, and the building would only include the air compressors and administrative facilities.

The pump buildings at PS-2 and PS-3 would be of sufficient height to allow for installation of a bridge crane for servicing the pumps and valves. Above-grade discharge laterals would include check and isolation valves for each pump before the piping extends below grade. The pumping area would also include sufficient room to assemble and disassemble a pump and perform applicable onsite maintenance. The approximate pump building/space footprint for each station is presented in Table 1-11.

Table 1-11 Preliminary Pump Building/Pad Size Estimates

PUMP STATION FACILITY	LOCATION	APPROXIMATE ROOM/ PAD SIZE
PS-1	Outdoor pad	145-ft x 50-ft
PS-2	Building[1]	205-ft x 50-ft
PS-3 (Alt A and B)	Building[1]	150-ft x 50-ft
PS-3 (Alt A-Backbone)	Building1	165-ft x 50-ft

Note:
1. Includes administration/control room.

1.5 HYDRAULIC SURGE CONTROL AND FACILITIES

Metropolitan's preferred method of surge control is to use air-over-water hydro-pneumatic tanks (also known as "air chambers"). On downsurges, as when a pump fails, the pressurized air in the tank forces fluid out into the pipeline to make up for the reduction in pipeline flow caused by the pump shutdown. As the pressure in the tank decreases from the expansion, the flow out of the tank decreases. Thus, flow changes are gradual rather than abrupt, and surge pressures are reduced. On reverse flow and upsurge, the surge chamber acts as a cushion and storage device. For a conveyance system of this size, the surge control system usually consists of several tanks, connecting pipelines with isolation valves, air compressors, liquid level sensors, and controls. The tanks themselves would be located outdoors on a pad (with appropriate curtain walls for shielding at PS-2 and PS-3), with the air compressors, add-air and vent-air solenoids, and controls panels located in the adjacent pump and/or control building.

Final sizing of the surge tanks would require detailed hydraulic transient analysis to investigate all potential surge conditions and the required system performance under each of these conditions. This level of analysis would be completed during the detailed design phase of the Project. However, for the purposes of the feasibility-level station configuration and site planning included in this report, surge tank sizes were estimated based on pipeline lengths, estimated flows, and typical surge performance requirements. The procedure used is described by Stephenson (2002) and the associated calculations are included in Appendix J. Table 1-12 summarizes the estimated surge tank sizes and associated footprints.

It should be noted that in Alternative B, there would be a significant length of suction pipe between the Signal Hill Tank and the suction side of PS-2. Depending on the final design of the facility, this length of pipe could need additional surge protection in the form of suction surge tanks or relieve valves.

Table 1-12 Preliminary Surge Tank Size Estimates

PUMP STATION FACILITY	SURGE TANK SIZE	APPROXIMATE PAD SIZE
PS-1 (Alt A)	11 tanks at 8,000 cu-ft	325-ft x 100-ft
PS-1 (Alt B)	4 tanks at 5,500 cu-ft	140-ft x 80-ft
PS-1 (Alt A-Backbone)	6 tanks at 6,000 cu-ft	202-ft x 100-ft
PS-2 (Alt B only)	5 tanks at 8,000 cu-ft	170-ft x 100-ft
PS-3 (Alt A and B)	2 tanks at 8,000 cu-ft	80-ft x 100-ft
PS-3 (Alt A-Backbone)	4 tanks at 6,000 cu-ft	141-ft x 100-ft

1.6 STORAGE FACILITIES

1.6.1 Overall Considerations

There are several features to consider when determining the optimal storage volume for a water transmission system such as the RRWP. Table 1-13 summarizes these design considerations and how they apply to this Project based on the current concept for the system.

Table 1-13 Storage Design Considerations

ITEM	STORAGE FUNCTION	APPLIES TO RRWP?	REMARKS
Diurnal Equalization	Necessary if there is a need to smooth the diurnal flow from the treatment plant so the conveyance system can pump a steady flow and not be sized for peak periods.	No	The AWTF is expected to operate at a fairly constant rate (i.e. equalization occurs upstream at the advanced treatment plant), so this storage function is not required.
Off-Peak Power Operation	Necessary if there is a desire to only operate the conveyance system during off-peak power periods.	No	The advanced treatment plant is expected to operate continuously at a near constant flow, which would require a prohibitively large storage reservoir to avoid off-peak pumping. Thus, this storage function is not being considered. If pumps at JWPCP are shut-down during off-peak periods, or for O&M, the treatment plant flows can be diverted to the existing plant outfall.

ITEM	STORAGE FUNCTION	APPLIES TO RRWP?	REMARKS
Continuous Delivery	Necessary if there is a need for the system to supply demands/customers even if the pump stations are shut down.	No	The only customers planned on the system are spreading basins and potential future injection wells, so the temporary disruption of flow will not have critical impacts. If future customers require continuous delivery they can be required to provide their own on-site storage.
Pump Cycling	If constant speed pumps are used and incoming flow does not match pumping rate enough storage must be provided to limit pump starts and stops.	No	All pumps on the RRWP will be equipped with variable frequency drives to match flow rates with adjacent stations.
Surge	Different surge control approaches require different amounts of storage to supply or accept water during a surge event.	Limited	The concept of using pressurized hydro-pneumatic tanks on the discharge side of pump stations means most of the volume is contained in pressure tanks. Currently the most volume for surge tanks is expected at PS-1, with a total volume of less than 0.7 MG; therefore, this volume would need to be available in the downstream storage facility.
Control	Storage between pump stations provides a hydraulic break and facilitates controlled ramping up and down of pumps.	Yes	The RRWP includes multiple pumps stations all with multiple pumping units as well as long transmission mains. Thus, storage facilities are necessary for improved operational control, especially during starting and stopping.
Balancing	Provides storage for short duration, low-magnitude imbalances between upstream and downstream pump stations.	Yes	Coordinated and synchronized controls between stations will limit the magnitude and duration of the imbalances.
Risk Mitigation	If a pump station fails to shut off due to upstream low reservoir level or downstream high reservoir level, pumps could be damaged or tank overflow could damage adjacent property or the environment.	Yes	The risk of such a failure can be reduced by implementation of robust control systems (as noted elsewhere in this document). If the control system fails, the facility can be located in an area that can safely convey an overflow to a drainage way.

As noted in Table 1-13, the feasibility-level storage sizing approach for the RRWP Pump Stations was based primarily on considerations of controls, balancing, and risk management. The following sections provide additional detail on the minimum volume recommended for each of these considerations.

1.6.2 Control and Balancing Volume

Storage upstream of the pump stations provide an atmospheric break between the pump stations which simplifies the controls and allows for short-duration flow imbalances between facilities, especially during starting and stopping of pumps. To determine the volume necessary for these control and balancing functions, the Project team developed a conceptual control strategy for the RRWP, which was presented in Section 1.2.

Based on the discussion in Section 1.2, the estimated duration of a flow imbalance during starting or stopping would be on the order of 12 minutes before the flow set point – level trim control algorithm engages and stabilizes tank levels. Since each station would have a slightly different size and/or number of pumps, a small flow imbalance would be likely. It is difficult to quantify the exact flow imbalance at this stage of the feasibility-level design, but it is believed it would be on the order of 5 mgd during the duration of the starting or stopping sequence. At a flow rate of 5 mgd, twelve minutes of flow imbalance would result in a total balancing storage volume of approximately 0.02 million gallons (MG), which is a relatively small volume.

1.6.3 Risk Mitigation Volume

As noted in Section 1.6.2, it is anticipated that a relatively small storage volume would be needed for pump station control. However, this assumes the station controls and interlocks are operating correctly. In the event of a control system/interlock failure, flow imbalances at a storage tank could be much higher than the controlled scenario investigated above. If a large flow imbalance occurs and is not corrected, the storage tank could either fully drain, potentially damaging the downstream pumping equipment, or it could overflow, releasing recycled water from the conveyance system. Thus, providing additional storage at each pump station would provide an increased level of risk mitigation by providing time for the control system to recover and/or for the system to shut down either automatically or via operator intervention.

1.6.4 Reaction Times

The volume of storage that should be provided for risk mitigation ultimately is a decision based on the estimated likelihood of a control failure and the potential consequences of a tank drain or overflow scenario. The probability of control failure is difficult to quantify at the feasibility level, but modern control and communication systems can be designed with high levels of reliability. The consequences of an overflow can also be managed in the design of the stations. The feasibility-level design presented in this report includes facilities to discharge to the nearest drainage way, including a system to dechlorinate the recycled water before discharge off-site.

Table 1-14 summarizes the required storage volumes in MG for a range of flow imbalances and reaction times.

Table 1-14 Required Storage Volumes in MG as a Function of Reaction Time and Flow Rate

CONDITION DESCRIPTION	FLOW RATE		REACTION TIME (MINUTES)									
	MGD	GPM	5	10	15	20	25	30	35	40	50	60
PS-1 to Flow Split Junction (Alt A) or Signal Hill (Alt B)	150	104,167	1.0	2.1	3.1	4.2	5.2	6.3	7.3	8.3	10.4	12.5
PS-2 Peak Capacity (Alt B Only)	140	97,222	1.0	1.9	2.9	3.9	4.9	5.8	6.8	7.8	9.7	11.7
Flow Split Junction (Alt A) or PS-2 (Alt B) to Orange County Capacity	60	41,667	0.4	0.8	1.3	1.7	2.1	2.5	2.9	3.3	4.2	5.0
Flow Split Junction (Alt A) or PS-2 (Alt B) to PS-3 Capacity	80	55,556	0.6	1.1	1.7	2.2	2.8	3.3	3.9	4.4	5.6	6.7
PS-1 Single Pump Capacity	37.5	26,042	0.3	0.5	0.8	1.0	1.3	1.6	1.8	2.1	2.6	3.1
PS-2 (Alt B only) Single Pump Capacity	20	13,889	0.1	0.3	0.4	0.6	0.7	0.8	1.0	1.1	1.4	1.7
Flow Split Junction (Alt A) or PS-2 (Alt B) and PS-3 Single Pump Capacity	26.7	18,542	0.2	0.4	0.6	0.7	0.9	1.1	1.3	1.5	1.9	2.2
PS-3 Peak Capacity (Alt A-Backbone)	150	104,167	1.0	2.1	3.1	4.2	5.2	6.3	7.3	8.3	10.4	12.5
PS-3 Single Pump Capacity (Alt A-Backbone)	37.5	26,042	0.3	0.5	0.8	1.0	1.3	1.6	1.8	2.1	2.6	3.1
Estimated Ramp Up/Down Imbalance	5.0	3,472	0.0	0.1	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.4

The volumes reported in Table 1-14 are total operational volumes based on the assumption that the tank would start at 50 percent full, as shown on Figure 1-16. The storage tank would also need a freeboard from the maximum level to the overflow and a minimum level to maintain pump submergence. These are estimated at 3 ft and 2 ft respectively, as shown on Figure 1-16.

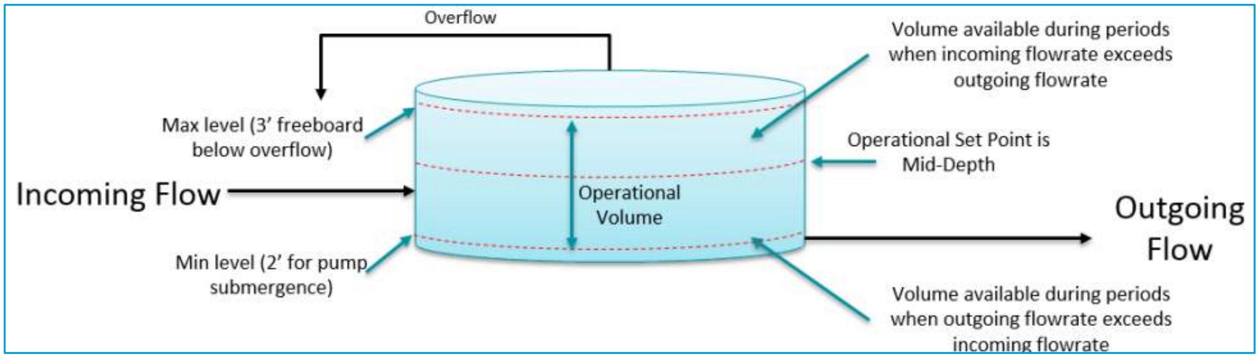


Figure 1-16 Typical Tank Level Configuration

Based on discussions with Metropolitan staff, it was determined that the AWTF would require between 35 and 40 minutes to react to an unexpected shutdown of the conveyance system. At PS-2 and PS-3, it was determined that ten minutes of reaction time would be required to trigger a shutdown of the system if communication and control were lost. Using these criteria, the following storage volumes were recommended for this feasibility-level design.

- PS-1: 7.5 MG
- PS-2: 2.0 MG (Alternative B Signal Hill storage tank)
- PS-3: 1.5 MG (Alternative A and B)
- PS-3: 2.5 MG (Alternative A – Backbone System)

Table 1-15 presents the recommended sizes and the associated storage times in minutes at the range of possible flow rates from low to high.

Table 1-15 Storage Times in Minutes at Various Flow Rates Based on Recommended Storage Volumes

CONDITION DESCRIPTION	FLOW RATE (MGD)	FLOW RATE (GPM)	STORAGE TIME (MINUTES)			
			PS-1 7.5 MG	PS-2 (SIGNAL HILL) 2.0 MG	PS-3 (ALT A AND B) 1.5 MG	PS-3 (ALT A BACKBONE) 2.5 MG
Estimated Ramp Imbalance	5	3,472	1,080	288	216	360
Flow Split Junction (Alt A) or PS-2 (Alt B) to PS-3 and PS-3 Single Pump Capacity	26.7	18,542	202	54	40	40
PS-1 Single Pump Capacity	37.5	26,042	144	38	29	29

CONDITION DESCRIPTION	FLOW RATE (MGD)	FLOW RATE (GPM)	STORAGE TIME (MINUTES)			
			PS-1 7.5 MG	PS-2 (SIGNAL HILL) 2.0 MG	PS-3 (ALT A AND B) 1.5 MG	PS-3 (ALT A BACKBONE) 2.5 MG
Flow Split Junction (Alt A) or PS-2 (Alt B) to Orange County Capacity	60	41,667	90	24	18	18
Flow Split Junction (Alt A) or PS-2 (Alt B) to PS-3 Capacity	80	55,556	68	18	14	14
PS-2 (Alt B only) Peak Capacity	140	97,222	39	10	N/A	N/A
PS-1 to Flow Split Junction (Alt A) or PS-2 (Alt B) Peak Capacity	150	104,167	36	10	N/A	N/A
PS-3 Single Pump Capacity (Alt A Backbone)	37.5	26,042	N/A	N/A	N/A	48
PS-3 Peak Capacity (Alt A Backbone)	150	104,167	36	N/A	N/A	12

Several layers of control system failure would be required for a pump station's local storage volume to reach an empty tank or overflow scenario, including:

- Failure of one or more pumps at pump station and inability of station to recover to specified flow set point.
- Failure of interlocks to trigger shut-down due to out-of-range operation.
- Failure of communication between stations to trigger shut-down if one station fails.

1.6.5 Storage Configuration

The proposed storage volume would be provided in above-ground circular tanks at PS-1 and PS-3 as well as Signal Hill (Alternative B only). Selection of the construction material for the storage tanks (i.e. steel vs. concrete) will be determined in subsequent design phases.

1.7 YARD PIPING, DECHLORINATION, AND MISCELLANEOUS FACILITIES

1.7.1 Discharge Piping and Meter Vault

Individual discharge laterals from each pump would feed a discharge header downstream of the pumps. A meter vault would be provided following the connection to the surge tanks to house and provide operator access to a flow meter and isolation vault installed in each discharge header. The approximate dimensions of the meter vaults are shown below in Table 1-16.

Table 1-16 Preliminary Meter Vault Size Estimates

PUMP STATION FACILITY	NO. OF FLOWMETERS	APPROXIMATE VAULT SIZE
PS-1	2	42-ft x 28-ft
PS-2 (Alt B only)	2	28-ft x 28-ft
PS-3 (Alt A and B)	1	15-ft x 28-ft
PS-3 (Alt A-Backbone)	1	17-ft x 28-ft

1.7.2 Dechlorination

In case of pump station failure, there may be emergency or unplanned discharges of recycled water that would ultimately reach the San Gabriel River. In order to discharge recycled water to a waterbody, it is currently anticipated that Metropolitan will need to apply for an Individual National Pollutant Discharge Elimination System Permit from the Los Angeles Recycled Water Quality Control Board, which may require additional water treatment to meet the water quality objectives for the San Gabriel River. Due to its nature as advanced treated water, it is likely that the recycled water quality would already meet basin plan requirements, with the possible exception of chlorine.

If required, dechlorination could be provided at the pump station sites to treat emergency overflows before discharging to the San Gabriel River. This is traditionally addressed in one of two ways:

- Option1: Using a liquid chemical injection system (e.g., sodium bisulfate) mixed into the overflowing volume to neutralize the chlorine during an overflow event. The benefit of this option is that its initial capital costs and overall footprint are typically less than that of a passive flow-through system. However, because the success of this approach relies on the performance of locally stored chemicals which can degrade over time, the cost of maintaining such a system and replacing these chemicals (on at least an annual basis) is viewed as excessive to most utilities- especially if an overflow event does not occur for several years.
- Option 2: Using a passive flow-through system containing media which can neutralize the chlorine during an overflow event. This approach is more likely to require a higher footprint and initial capital costs, as compared to a liquid chemical treatment system. However, because the chlorine-neutralizing capabilities of some media, such as granular activated carbon (GAC), are not exhausted with time or contact with chlorine, the need and frequency of replacement is greatly reduced. Another benefit of the passive system is that it is already 'ready' for its intended purpose; it requires no startup time, dosage metering or monitoring, and very little to no annual maintenance.

At the current feasibility-level stage of the Project, it was assumed that Metropolitan would select the flow-through system for overflow dichlorination, if required. Assuming that GAC would be utilized as the flow-through media, it is estimated that approximately 56,000 cubic ft (cf) of GAC media would be required to dechlorinate an overflow event of 150 mgd containing up to 5 milligrams per liter (mg/L) chlorine. This volume of media would correspond roughly to a facility 150-ft (long) by 40-ft (wide) by 10-ft (deep). For smaller overflow rates, the size of the facility would be reduced proportionally.

A flow-through dichlorination system is assumed for PS-1 and PS-3, both of which have on-site storage tanks. PS-2 would not have a dichlorination facility since the storage tank that it draws from is located at Signal Hill.

1.8 POWER SUPPLY AND ELECTRICAL REQUIREMENTS

1.8.1 Major Load Estimation

The major use of electricity at the pump stations will be associated with operating the pumps' motors. The pump selections discussed in Section 1.3.3 and shown in Table 1-6 were used to develop the feasibility-level electrical system design. As shown in Table 1-17, a representative manufacturer's selection for each pump station was used to estimate the amount of power that would need to be supplied to the site and to determine the required sizes of the electrical facilities.

Table 1-17 Summary of Design Motor Size

STATION	RATED DESIGN POINT	MOTOR SIZE FOR DESIGN
PS-1 Set A	7.5 mgd at 165 ft	3 pumps (2 duty + 1 standby) at 350 HP
PS-1 Set B (Alt A)	37.5 mgd at 428 ft	5 pumps (4 duty + 1 standby) at 5,000 HP
PS-1 Set B (Alt B)	37.5 mgd at 174 ft	5 pumps (4 duty + 1 standby) at 1,750 HP
PS-1 Set B (Alt A-Backbone)	37.5 mgd at 352 ft	5 pumps (4 duty + 1 standby) at 5,000 HP
PS-2 Set A (Alt B only)	20 mgd at 266 ft	4 pumps (3 duty + 1 standby) at 1,500 HP
PS-2 Set B (Alt B only)	26.7 mgd at 235 ft	4 pumps (3 duty + 1 standby) at 1,750 HP
PS-3 (Alt A and B)	26.7 mgd at 397 ft	4 pumps (3 duty + 1 standby) at 2,750 HP
PS-3 (Alt A-Backbone)	37.5 mgd at 352 ft	5 pumps (4 duty + 1 standby) at 5,000 HP

1.8.2 Electrical Facilities and Space Requirements

Each pump station would include an electrical building/room, which is anticipated to be located immediately adjacent to the pump building/pad. This building/room would house electrical equipment that cannot be located outdoors, including motor control centers (MCCs), VFD controllers, and uninterruptable power supply system. In addition to the electrical building/room, an outdoor transformer farm would be included at each pump station for medium and high voltage electrical equipment.

There are two possible electrical service options that are likely to serve the pump stations: Option 1 assumes that the medium voltage (4,160 volts) is supplied by the power utility; Option 2 assumes that higher voltage (above 4,160 volts) is supplied. The power utility would dictate which option needs to be implemented at each site. For this study, the feasibility-level layouts shown in Appendix L are based on Option 2. The power utility may require additional space either at or near the pump station sites for a switchyard, which is not currently shown on the feasibility-level layouts.

Table 1-18 summarizes the estimated footprint of the electrical facility at each pump station. Coordination with the power utility will be required in future phases of the Project.

Table 1-18 Preliminary Electrical Facility Dimensions

PUMP STATION	ELECTRICAL BUILDING/ROOM	OPTION 1 TRANSFORMER FARM	OPTION 2 TRANSFORMER FARM
PS-1 (Alt A)	68’ x 44’	36’-0” x 50’-2”	99’ x 68’
PS-1 (Alt B)	37’-3” x 42’-8”	36’-0” x 50’-2”	59’ x 68’
PS-1 Set B (Alt A-Backbone)	68’ x 44’	36’-0” x 50’-2”	99’ x 68’
PS-2 (Alt B only)	37’-3” x 70’-4”	36’-0” x 50’-2”	99’ x 66’-3”
PS-3 (Alt A and B)	50’ x 42’-8”	36’-0” x 50’-2”	59’ x 66’-3”
PS-3 (Alt A-Backbone)	68’ x 44’	36’-0” x 50’-2”	99’ x 66’-3”

1.9 PUMP STATION SITE INVESTIGATIONS

1.9.1 Methodology

The site for PS-1 was identified by Metropolitan to be located at the northeast corner of the AWTF site. It was determined that there would be enough space at the existing site for the pump station and its associated facilities.

Potential sites for PS-2 and PS-3 were evaluated based on the following criteria: (1) Current Site Uses, (2) Existing Major Utilities, (3) Site Access, (4) Overall Constructability, (5) Environmental Risks, (6) Hazardous Materials Risks, (7) Proximity to Overflow Discharge Locations, and (8) Proximity to Recycled Water Pipeline Alignment. These criteria are explained in further detail below:

- Current Site Uses: Potential sites were evaluated based on existing land use in an effort to minimize impacts to communities. Potentially sensitive sites such as religious facilities, public institutions, and community facilities were eliminated from consideration. It was assumed that Metropolitan would obtain any existing, non-Metropolitan owned properties using eminent domain.
- Existing Major Utilities: The presence of existing major utilities was investigated by performing a desktop review of the available GIS data obtained from Metropolitan and Los Angeles County, the United States Department of Transportation (DOT) National Pipeline

Mapping System and a review of aerial maps available online. Utilities analyzed included sanitary sewers, storm drains, overhead electrical lines, oil and gas transmission lines, and railroads.

- Site Access: The potential sites were evaluated for ease of construction and operational access.
- Overall Constructability: Potential sites were evaluated for ease of construction, e.g. topographic constraints of the site, demolition requirements of any existing structures, and trenchless construction requirements for the suction, discharge, and overflow pipelines.
- Environmental Risks: The presence of endangered species habitats was studied using the California Natural Resources Diversity Database.
- Hazardous Materials Risks: The presence of environmental hazard sites was analyzed using the California State Water Resources Control Board (Water Boards) Geotracker database. Sites with active environmental remediation activities were not considered viable (e,g,. environmental hazards include leaking underground storage tanks, or the presence of trichloroethylene (TCE) plumes at former dry cleaner locations).
- Proximity to Overflow Discharge Locations: Potential sites were evaluated based on their ability to gravity flow to existing storm water facilities.
- Proximity to Recycled Water Pipeline Alignment: Potential sites were evaluated based on their proximity to the Recycled Water Pipeline Preferred Alignment to minimize capital costs and pipeline construction impacts.

1.9.2 Feasibility-level Site Identification

Potential sites have been identified for PS-2 and PS-3, based on a desktop review of locations along the Recycled Water Pipeline Preferred Alignment. Further analysis will have to be conducted including onsite surveys and geotechnical studies to select the most optimal pump station location.

1.9.2.1 PS-2: Potential Siting

A potential site has been identified on the current Southern California Edison facility in the City of Cerritos. The Edison facility is located at the intersection of Del Amo Boulevard and State Road, between the 605 Freeway and the San Gabriel River. The PS-2 site is anticipated to have a footprint measuring approximate 200’ x 450’ which can fit on the southwest corner of the Edison facility parking lot.

1.9.2.2 PS-3: Potential Siting

Five potential sites for PS-3 have been identified in a commercial area near the 605 Freeway between Beverly Boulevard and Whittier Boulevard as shown in Figure 1-17. This general vicinity for PS-3 was selected so that when the system is operating at full capacity under Alternative A, minimum throttling would be required on either the downstream pipeline feeding the Orange County Spreading Grounds or the pipeline delivering flow to PS-3, thereby reflecting the most efficient operating condition. The PS-3 site, regardless of its final location, is generally anticipated to have a footprint measuring approximately 300’ x 400’, although a slightly larger footprint may be required under the PS-3 (Alt A Backbone) scenario.

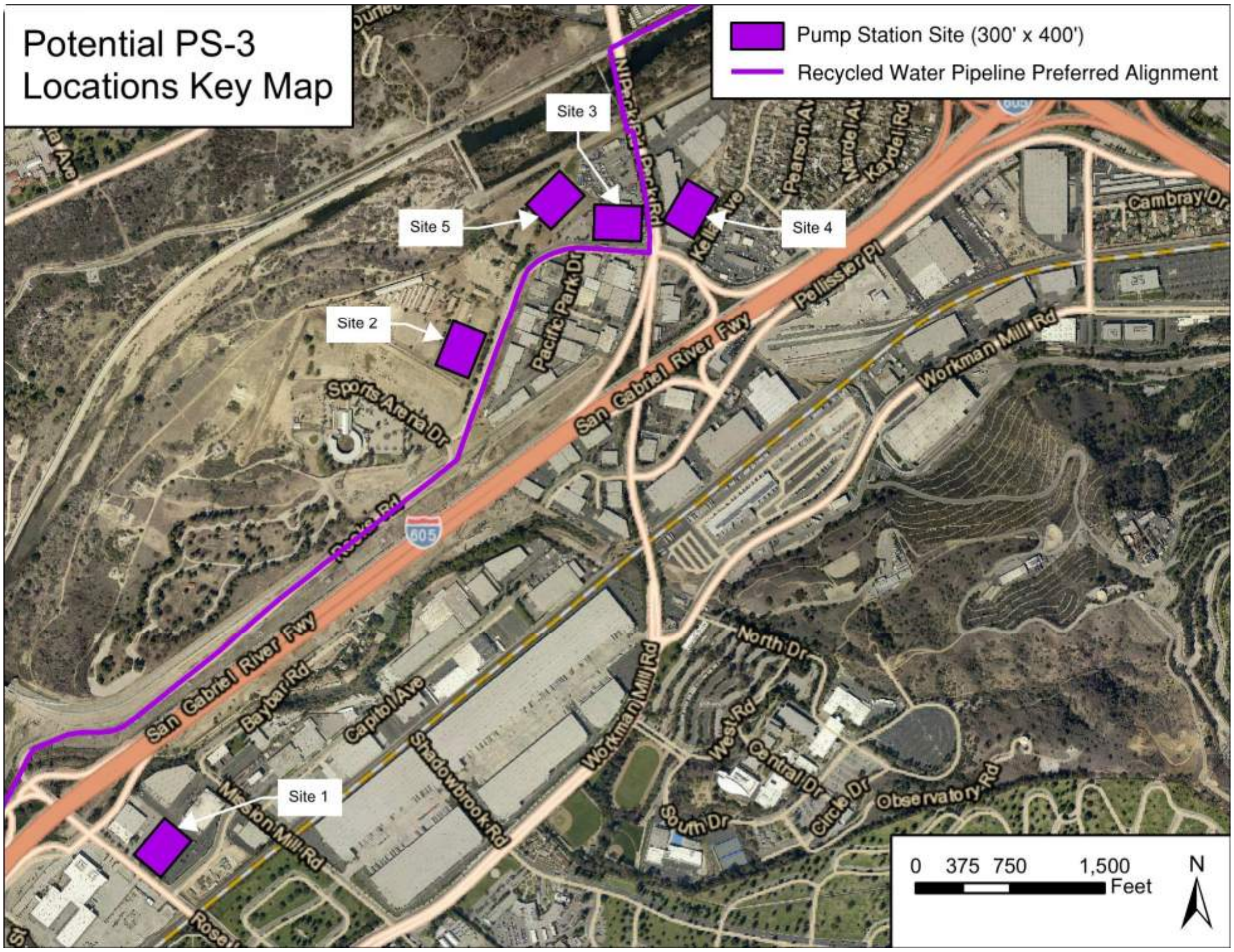


Figure 1-17 Potential PS-3 Locations Key Map

PS-3 Site No. 1 is located near the intersection of Rose Hills Drive and Capitol Avenue. Site No. 2 is located at the intersection of Rooks Road and Sports Arena Drive. Site No. 3 is located at the intersection of Rooks Road and Peck Road. Site No. 4 is located at the intersection of Rooks Road and Kella Avenue. Lastly, Site No. 5 is located west of the intersection of Rooks Road and Peck Road.

1.9.3 Site Attribute Investigation

This section describes the attributes for each potential site according to the criteria described in Section 7.9.1.

1.9.3.1 Potential PS-2 Site Attributes

The potential PS-2 site (Figure 1-18) is located approximately 600 feet to the east of the Recycled Water Pipeline Preferred Alignment (Preferred Alignment). The site is located on Edison's parking lot and would be constructed on level ground. While the site would require demolition of a portion of the existing asphalt parking lot, the removal of existing structures is not currently anticipated for construction of the new PS-2 facilities. Suction and discharge piping would be approximately 600 feet each to reach the Preferred Alignment. The overflow pipeline alignment will be oriented from east to west, and discharge into the San Gabriel River. With an anticipated drop of 6 feet over 600 feet (1% slope), the overflow pipeline could provide a design capacity of 150 MGD with a diameter of 42 inches. The overflow pipeline will cross a storm drain alignment, as well as Studebaker Road and the San Gabriel River Trail. The site contains no California Protected Areas. There is a single closed Leaking Underground Storage Tank (LUST) site on the Southern California Edison Property, and a second closed LUST site across Del Amo Boulevard near the existing Mobil station. Additional investigation may be required to ascertain whether additional remediation work is required.

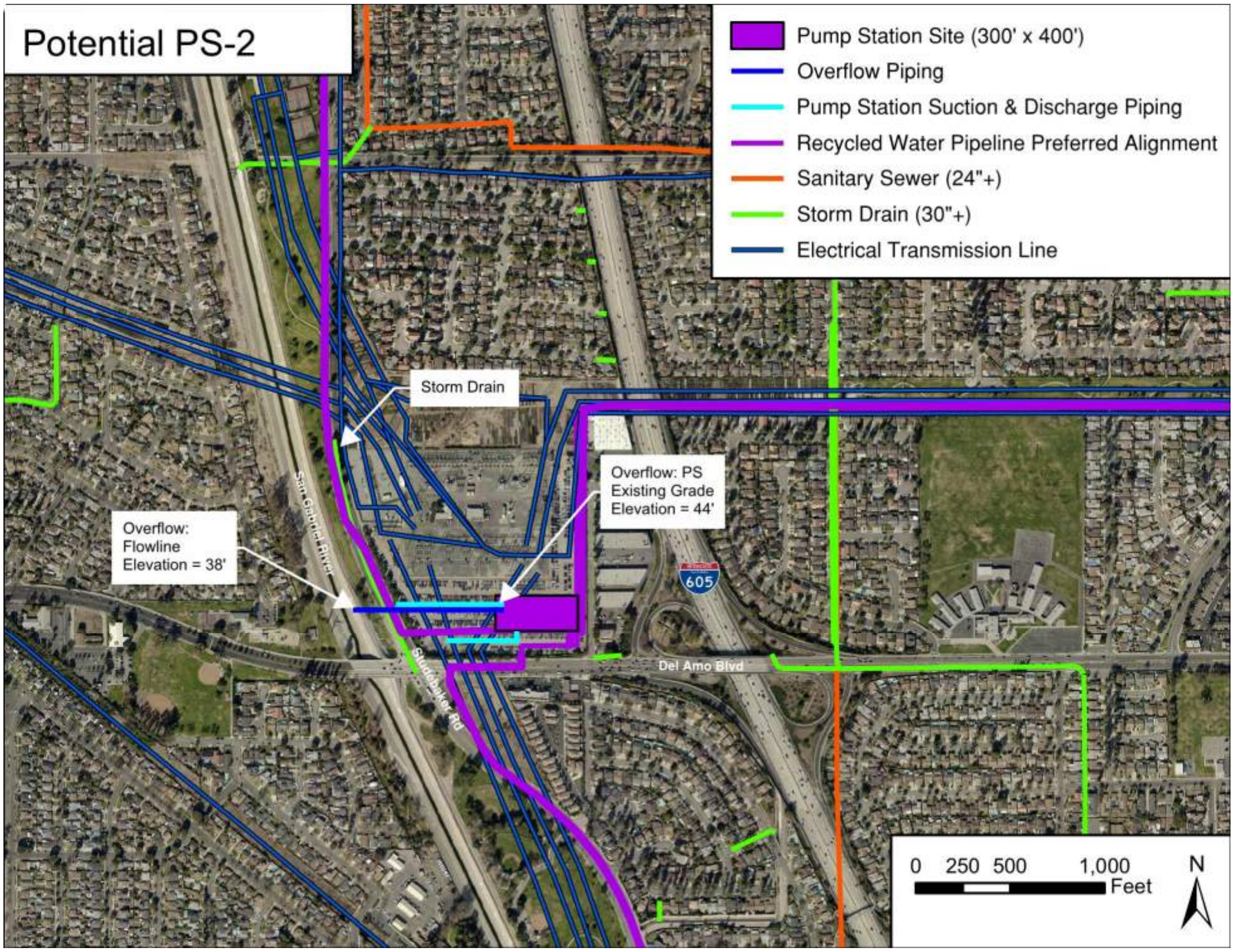


Figure 1-18 Potential PS-2 Site

1.9.3.2 Potential PS-3 Site Attributes

This section describes the site attributes for the potential PS-3 sites identified at this phase. A summary of the site attributes is presented in Table 1-19.

Table 1-19 Attributes of Potential PS-3 Sites

SITE	APPROXIMATE SITE ADDRESS	CURRENT SITE USES	EXISTING MAJOR UTILITIES	SITE ACCESS	CONSTRUCTABILITY	ENVIRONMENTAL RISKS	HAZARDOUS MATERIALS RISKS	PROXIMITY TO OVERFLOW DISCHARGE LOCATION (FEET)	PROXIMITY TO PIPELINE ALIGNMENT (FEET)	NOTES
PS-3 Site 1	10015 Rose Hills Road, City of Industry, Ca	Carpenter's Union Training Facility	An existing 54" sanitary sewer is located between the site and drainage channel that feeds the San Gabriel River. Suction and discharge pipelines would have to cross the existing 54" sanitary sewer and 605 Freeway to reach the Preferred Alignment.	The site is fronted by the four-lane Rose Hills Drive and two-lane Capitol Avenue.	The site is level and would require demolition of a commercial facility. Suction and discharge pipelines would require trenchless construction to cross the 605 Freeway.	The site does not contain any observed California Protected Areas.	No active remediation sites are observed on the property.	700	1,200	This site is close to an overflow location. However, the site is further away from the Preferred Alignment and would require trenchless pipeline crossing of the 605 Freeway. Alternative A-Backbone for this pump station would require acquisition of an additional parcel to the northeast (Industrial Bakery) to accommodate the larger site footprint.
PS-3 Site 2	11003 Sports Arena Dr, Whittier, CA	Los Angeles County Mounted Assistance Unit Training Site	An existing 25" sanitary sewer crosses the parcel. Overflow pipeline would cross the sanitary sewer and two separate vacant parcels to reach the San Gabriel River.	The site is accessible from the four-lane Rooks Road.	The site is level and is currently open space for vehicular parking. The pump station footprint may overlap with an existing training facility.	The site does not contain any observed California Protected Areas.	No active remediation sites are observed on the property.	1,300	140	The site does not require the demolition of a major building and also appears viable for the larger footprint of the Alternative A-Backbone option.
PS-3 Site 3	2429 Peck Road, Whittier, CA	Velocity Truck Centers	An existing 25" sanitary sewer and 42" storm drain are both in the vicinity of the parcel. The overflow pipeline would cross the 25" sanitary sewer in order to reach the San Gabriel River. Overhead powerlines are observed to the north of the parcel.	The site is accessible from the four-lane Rooks Road.	The site is accessible by the two-lane Rooks Road. The overflow pipeline would cross an adjacent parcel that is currently occupied by a parking lot before discharging to the San Gabriel River.	The site does not contain any observed California Protected Areas.	No active remediation sites are observed on the property.	600	150	There is little additional space near this site for the larger footprint of the Alternative A-Backbone option.
PS-3 Site 4	2450 Kella Ave, Whittier, CA	Rush Truck Center	No major utilities are present on the site. The overflow pipeline will cross an existing 25" sanitary sewer to reach the San Gabriel River.	The site can be accessed from the four-lane Rooks Road, and the 605 Freeway.	The site is level and would require demolition of a commercial facility.	The site does not contain any observed California Protected Areas.	No active remediation sites are observed on the property.	1,400	450	There is little additional space near this site for the larger footprint of the Alternative A-Backbone option.
PS-3 Site 5	10149 Rooks Road Whittier, CA 9066	Blackwill Equestrian Center (Los Angeles County Parks & Recreation)	There is an existing 25" sanitary sewer and an overhead power line at the south part of the site.	Site is accessible from the four-lane Rooks Road.	The site is level and would not require the demolition of buildings. Pump station footprint would have to avoid an existing power transmission tower on the parcel.	The site does not contain any observed California Protected Areas.	No active remediation sites are observed on the property.	150	250	The site would occupy an open space currently used for equestrian activities There is potentially enough space in the area for the larger footprint of the Alternative A-Backbone option.

1.9.3.2.1 Potential PS-3 Site 1 Attributes

Potential PS-3 Site 1 is located approximately 1,200 feet away from the Preferred Alignment and is approximately 700 feet away from a nearby drainage channel (see Figure 1-19). The existing drainage channel appears to have enough capacity to receive the overflow from the pump station. The site is currently occupied by Carpenter's Union Training Facility. The site is level, but would require demolition of the commercial facility for the construction of the pump station. Suction, discharge, and overflow piping may be constructed via open trench construction except for the 605 Freeway crossing. Suction and discharge piping may cross the 605 Freeway via trenchless technologies, which will require a Caltrans permit. There appears to be an approximate 20-foot drop in elevation between the pump station site and the drainage channel and may allow the overflow to drain by gravity. Hazardous materials requiring remediation do not appear to be present at this site. The implementation of Alternative A-Backbone for this pump station would require the acquisition of an additional parcel to the northeast (Industrial Bakery) to accommodate the larger site footprint.

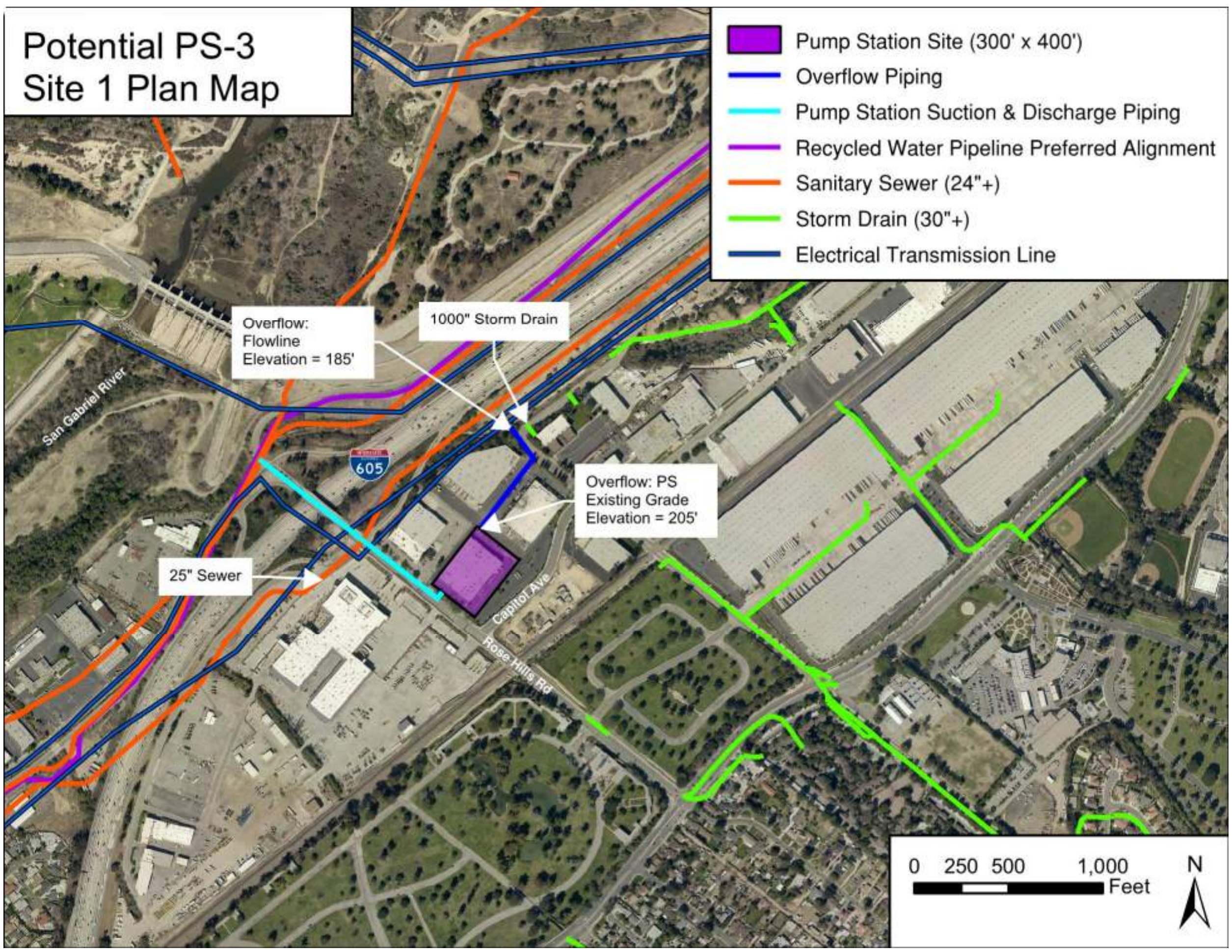


Figure 1-19 Potential PS-3 Site 1 Plan Map

1.9.3.2.2 Potential PS-3 Site 2 Attributes

Potential PS-3 Site 2 is located adjacent to the Preferred Alignment and approximately 1,300 feet away from the San Gabriel River (see Figure 1-20). The site is currently occupied by the Los

Angeles County Mounted Assistance Unit. Overflow, suction, and discharge pipelines may be constructed via open trench construction. The overflow pipeline would have to cross an existing 25" sanitary sewer pipeline and two vacant parcels to the discharge point at the San Gabriel River. There appears to be an approximate 26-foot drop in elevation between the pump station site and the river and may allow the overflow to drain by gravity. The site is level and would require minimal demolition of the existing facilities. Hazardous materials requiring remediation do not appear to be present at this site. The site appears to be viable for the larger footprint of the Alternative A-Backbone option.

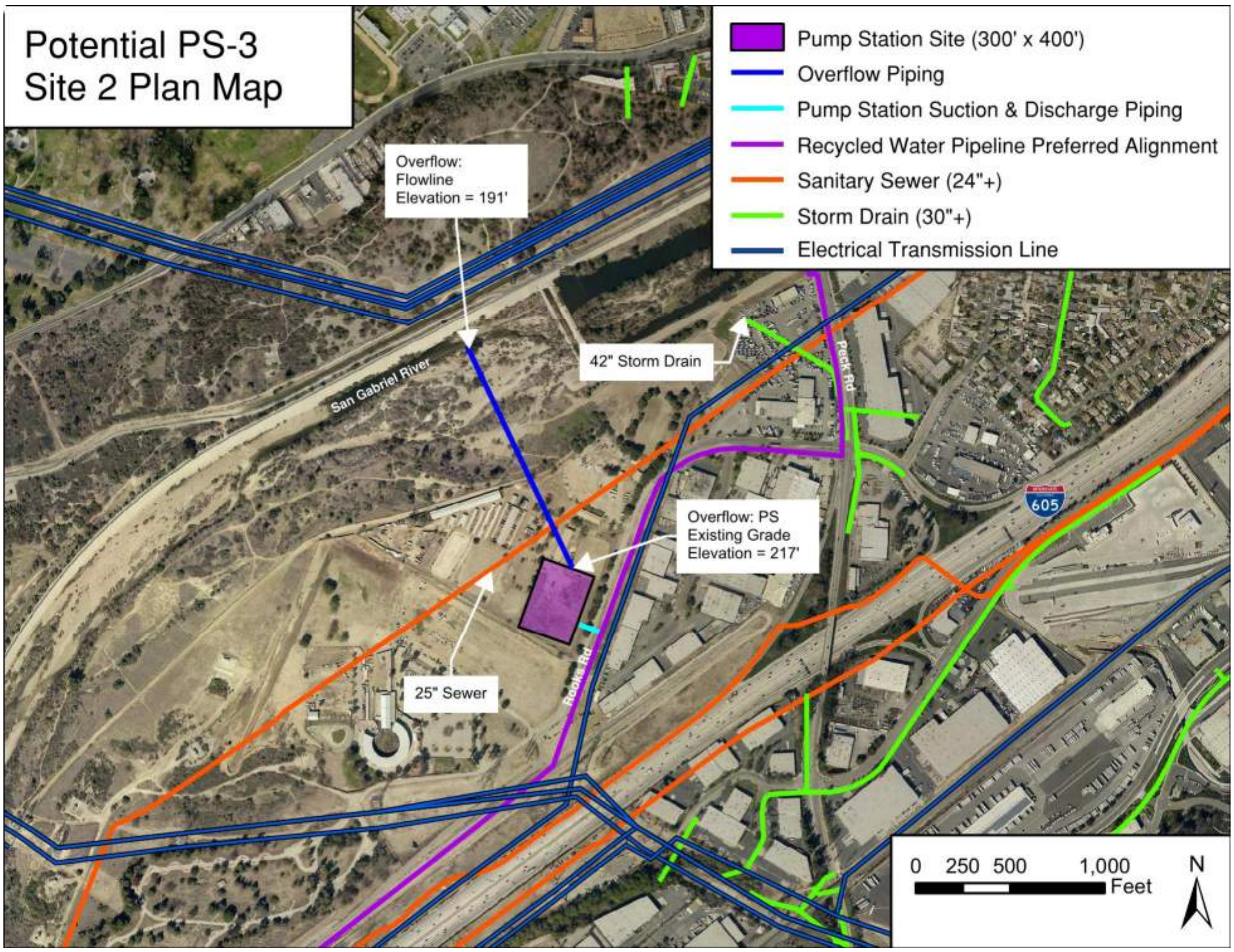


Figure 1-20 Potential PS-3 Site 2 Plan Map

1.9.3.2.3 Potential PS-3 Site 3 Attributes

Potential PS-3 Site 3 is located adjacent to the Preferred Alignment on a parcel by the intersection of Peck Road and Rooks Road (see Figure 1-21). The site is currently occupied by Velocity Truck Center. The site is level and would require the demolition of the existing building. Suction, discharge, and overflow piping may be constructed via open trench construction. The overflow pipeline would cross an existing 25" sanitary sewer and the adjacent parcel to the north that currently contains a parking lot. There appears to be an approximate 28-foot drop in elevation between the pump station site and the San Gabriel River and may allow the overflow to drain by gravity. Hazardous materials requiring remediation do not appear to be present at this site. There is little additional space near this site for the larger footprint of the Alternative A-Backbone option.

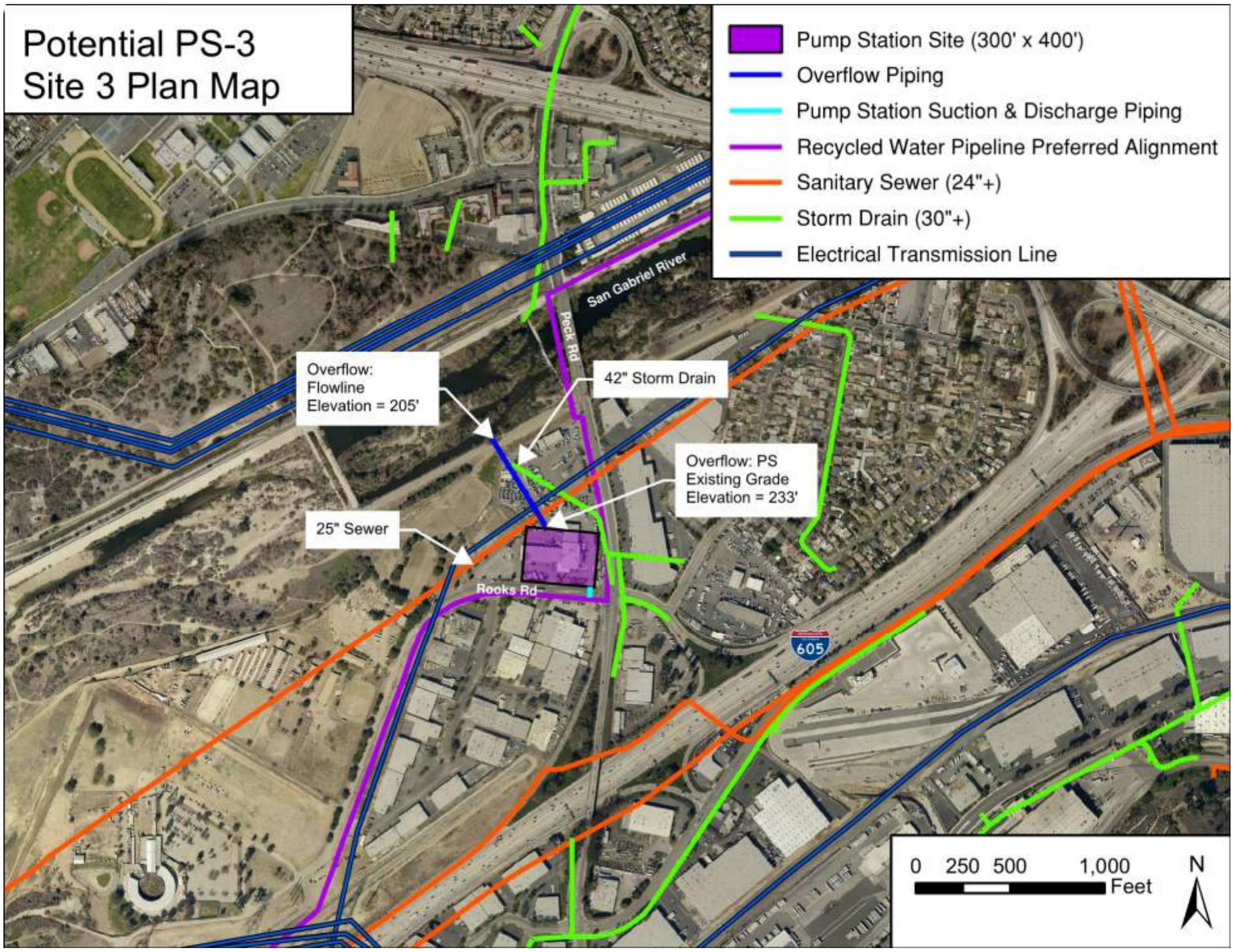


Figure 1-21 Potential PS-3 Site 3 Plan Map

1.9.3.2.4 Potential PS-3 Site 4 Attributes

Potential PS-3 Site 4 is located at a commercial facility at the intersection of Kella Avenue and Rooks Road on the west side of the 605 Freeway (see Figure 1-22). The commercial facility is occupied by Rush Truck Center. The suction and discharge piping would extend approximately 450 feet to the Preferred Alignment at the intersection of Rooks Road and Peck Road. Overflow piping may be routed north along Peck Road towards the San Gabriel River and would cross an existing 25" sanitary sewer. There appears to be an approximate 10-foot drop in elevation between the pump station site and the river which may not allow the overflow to completely drain by gravity during periods of discharge. The site is built on level ground and construction would require the demolition of the existing facilities. Hazardous materials requiring remediation do not appear to be present at this site. There is little additional space near this site for the larger footprint of the Alternative A-Backbone option.

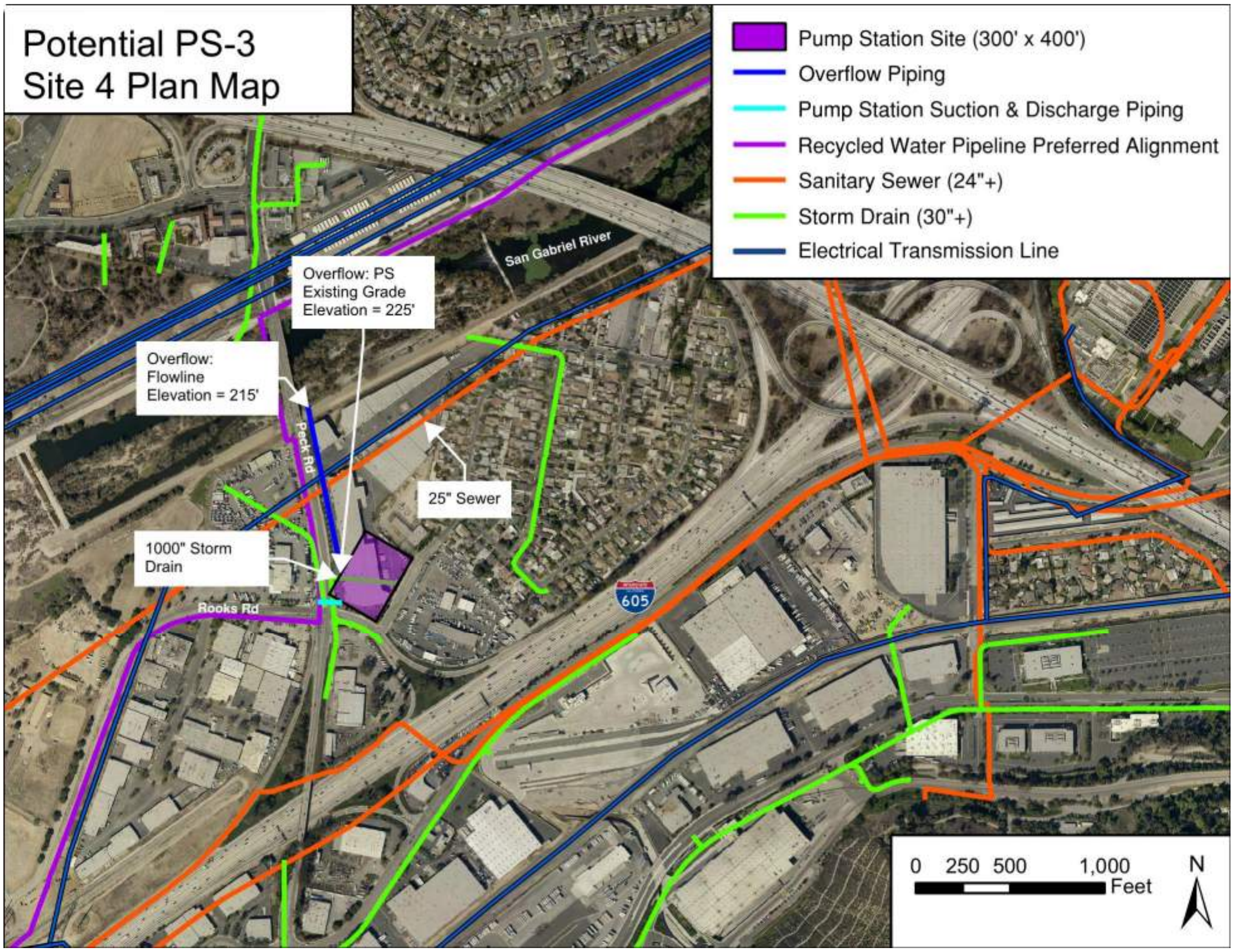


Figure 1-22 Potential PS-3 Site 4 Plan Map

1.9.3.2.5 Potential PS-3 Site 5 Attributes

Potential PS-3 Site 5 is located on an open space parcel currently occupied by the Backwill Equestrian Center (see Figure 1-23). Of the five potential sites, this site would have the shortest suction, discharge, and overflow piping. There is an existing 25" sanitary sewer and an overhead power transmission line of the site. The overflow pipeline would run north and discharge into the San Gabriel River. There appears to be an approximate 10-foot drop in elevation between the pump station site and the river which may not allow the overflow to completely drain by gravity during periods of discharge. The site is level and would not require demolition of existing buildings. Hazardous materials requiring remediation do not appear to be present at this site. The area appears to be viable for the larger footprint of the Alternative A-Backbone option.

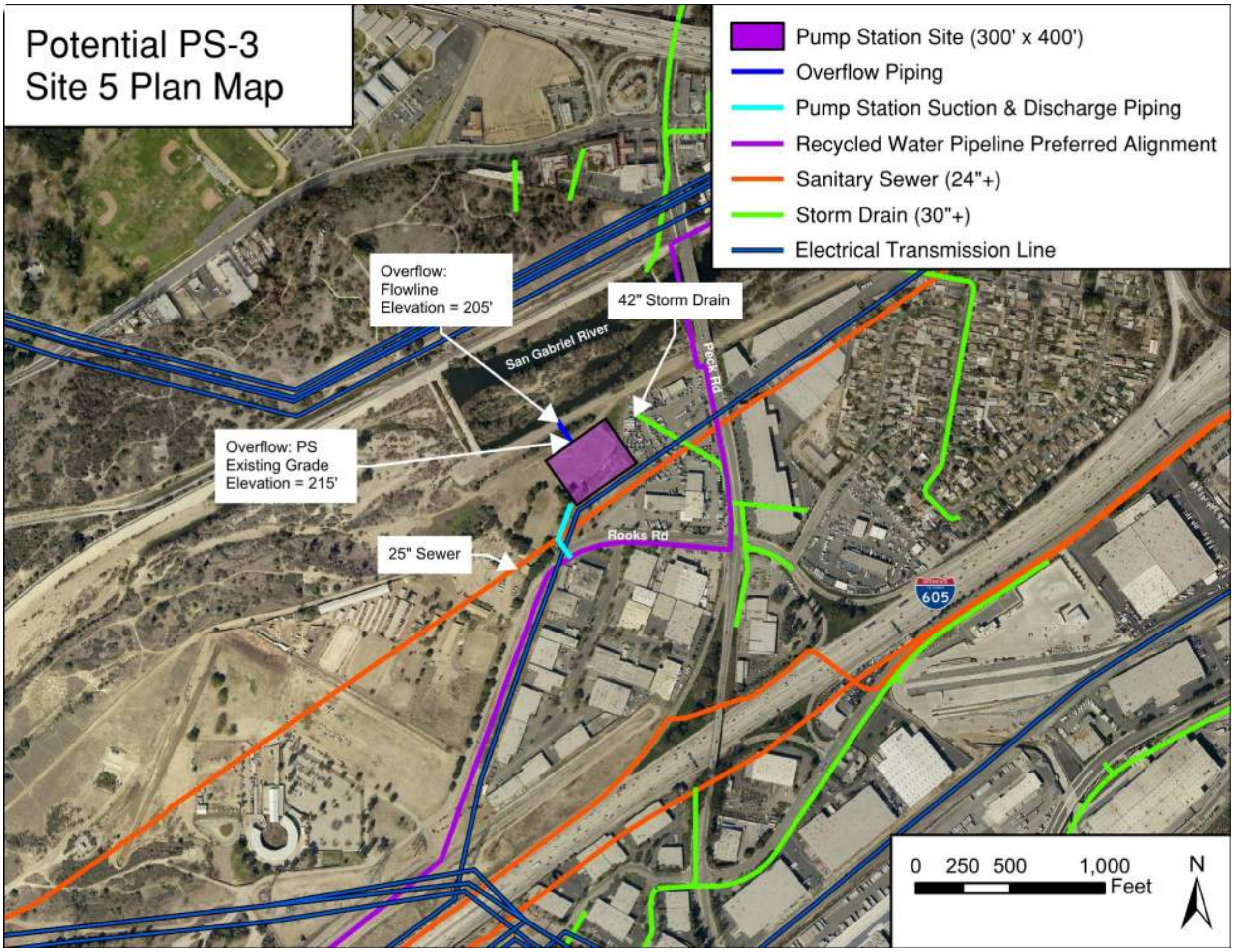


Figure 1-23 Potential PS-3 Site 5 Plan Map

1.10 SITE AND YARD PIPING DEVELOPMENT

Preliminary site plans were developed for each pump station site, as presented in Appendix L. The following sections provide details on each site.

1.10.1 PS-1 Site and Yard Piping Development

PS-1 would be located on the northeast corner of the AWTF site, as shown on Sheet C-1 in Appendix L. The circular 7.5-MG storage tank and optional dechlorination facility would be on the southern end of the pump station site. The pump pad, electrical room, transformer farm, surge tanks, and meter vault would be located on the northern portion of the site, with a parking lot between the pump facilities and the storage tank. Access to the electrical room would be provided from the east via South Main Street.

Treated recycled water would enter the storage tank from the east through a 102-inch inlet. An overflow pipeline would be provided on the southeast part of the tank and travel through the dechlorination facility, if required. From there, the overflow pipe would travel north to the drainage system. A 102-inch suction header would extend from the northwestern part of the storage tank to the pump pad. The pumps would connect to two discharge headers, which would travel north through the meter vault before existing the site. The pumps for PS-1 Set A would use a

30-inch discharge pipeline to send water to the potential future injection wells. The pumps for PS-1 Set B would use an 84-inch discharge pipeline to send water to the Orange County Spreading Grounds and PS-3 (Alternative A), or the Signal Hill storage tank (Alternative B), or PS-3 only (Alternative A-Backbone System).

Sheets M-1 and M-2 in Appendix L contain more detailed plan views for PS-1, and Sheet M-3 contains sections of a PS-1 surge tank and valve vault.

1.10.2 PS-2 Site and Yard Piping Development

For Alternative B only, PS-2 would be located on the southeast corner of the parking lot of the Edison site, as shown on Sheet C-2 in Appendix L. Preliminary section and plan drawings are presented on Sheets M-4 and M-5 in Appendix L. The 84-inch inflow pipeline would extend from the Signal Hill storage tank to the pump building. Two discharge headers would exit the site through a meter vault to the west. One header would send recycled water to the Orange County Spreading Grounds, while the other would send recycled water to PS-3.

1.10.3 PS-3 Site and Yard Piping Development

The site for PS-3 has not yet been selected, but preliminary section and plan drawings are presented on Sheets M-6 and M-7 in Appendix L. The circular 1.5-MG storage tank would be located on the southeast portion of the site. The 60-inch inflow pipeline would enter the storage tank from the south. The pump room would be located to the northwest of the storage tank, fed by a 72-inch suction header. A 60-inch discharge header would exit the site through a meter vault to the east and continue to the Santa Fe Spreading Grounds.

The measurements noted above correspond to Alternatives A and B. For the Alternative A - Backbone option, the layout is the same as for Alternatives A and B but the site is anticipated to include a 2.5-MG storage tank, 84-inch inflow pipeline, and 102-inch suction header.

Appendix W. Conceptual Review of Three New Tunnel Alignments Draft Report

Regional Recycled Water Program (RRWP)

Conceptual Review of Three New Tunnel Alignments

Draft
Revision No. 2

November 15, 2019

This page intentionally left blank.

Table of Contents

List of Tables

List of Figures

Appendices

Distribution

To:	Jay Arabshahi, PE Metropolitan Water District of Southern California
	Robb Bell, PE Metropolitan Water District of Southern California
	Tedman Tran Metropolitan Water District of Southern California
From:	Glenn M. Boyce, PhD, PE McMillen Jacobs Associates
Prepared By:	Glenn M. Boyce, PhD, PE McMillen Jacobs Associates
	Jeremy Stone, PE McMillen Jacobs Associates
	Maryam Motamed, PE McMillen Jacobs Associates
	Keith Bryant, PE McMillen Jacobs Associates
	Troy Page, PE McMillen Jacobs Associates
Reviewed By:	Rachel Martin, PE McMillen Jacobs Associates
	John Kaplin, CCM McMillen Jacobs Associates

Revision Log

Revision No.	Date	Revision Description
0	September 24, 2019	Draft
1	October 15, 2019	Draft with new appendices
2	November 15, 2019	Draft with updated cost estimates

Executive Summary

The Metropolitan Water District of California (MWD) has been assessing the feasibility of augmenting its water supplies by implementing the Regional Recycled Water Program (RRWP). The components of the program include an Advanced Water Treatment Plant and distribution system of more than 60 miles of large-diameter pipeline and pump stations to convey the highly treated reclaimed water from the Los Angeles County Sanitation Districts Joint Water Pollution Control Plant in the City of Carson to various groundwater basins in Los Angeles and Orange Counties for groundwater recharge purposes. Construction of the conveyance pipeline segments will primarily be utilizing cut-and-cover methods with trenchless methods used at special crossing locations. However, there will be numerous sections of the conveyance system where trenchless/tunneling construction methods may be preferred over cut-and-cover methods to construct the pipeline.

MWD engaged McMillen Jacobs Associates' experts for a high-level review of three alignments proposed as fully tunneled options. The conceptual review focused primarily on the "big picture" elements associated with tunnels 2.6 to 4.6 miles long in fully developed urban environments. Out of the many that could be listed, some of the elements considered in this report are ground/groundwater conditions, environmental hot zones for contamination, physical barriers that influence vertical alignment, horizontal changes in tunnel alignment to stay within existing rights-of-way (ROWs), shaft locations, tunnel diameters, and tunnel methodology. See Appendix A for a discussion of the different tunneling methods and Appendix B for a discussion of the different jacking and receiving shaft construction methods. The following three pipeline segments were evaluated in this study:

1. Carson to Long Beach: This alignment is about 4.6 miles long, starting on South Main Street and turning east following East Sepulveda Boulevard and West Willow Street to the east side of the Los Angeles River; crossing under the Dominquez Channel, Interstate 710 (I-710), the Los Angeles River, and other major roads. Two options were included in the study: Option 1A utilizing pipe jacking/microtunneling with intermediate jacking and receiving shafts along the original cut-and-cover alignment; and Option 1B utilizing an open- and closed-face tunnel boring machine (TBM) with a single jacking and receiving shaft along a partially new alignment.

2. San Gabriel River: This alignment is about 4.6 miles long, starting at Imperial Highway and following the San Gabriel River north to Pico Rivera; paralleling the river in the Southern Edison right-of-way and along I-605; and crossing Highway 42, I-5, and other major roads. Two options were included in the study: Option 2A utilizing microtunneling with intermediate jacking and receiving shafts to avoid cut-and-cover in the river bed; and Option 2B utilizing a closed-face TBM with a single jacking and receiving shaft.

3. Azusa to Glendora: This alignment is about 2.6 miles long, starting on Highway 39 adjacent to the City of Azusa Filtration Plant and trending along the San Gabriel River to the east and north to a point short of Morris Reservoir, where the new tunnel will tie into the existing Glendora Tunnel. Three options were included in the study: Option 3A utilizing an initial cut and cover section along Highway 39 and Oxbow Park with a drill-and-blast tunnel from Oxbow Park to the Glendora connection; Option 3B utilizing an initial drill-and-blast or TBM tunnel from the Azusa Filtration Plant to Oxbow Park followed by a cut-and-cover section along Oxbow Park and then

another drill-and-blast or TBM tunnel to the Glendora connection; and Option 3C utilizing a full-length drill-and-blast or TBM tunnel from the Azusa Filtration Plant to the Glendora connection.

For each pipeline segment with associated option, McMillen Jacobs Associates developed the following:

- Conceptual horizontal and vertical alignments considering property information, ROWs, and site constraints, see Appendix C
- Desktop study of the geology along the proposed alignments
- Shaft and portal locations and established pipe jack/microtunnel drive lengths or tunnel lengths
- Typical construction risks, issues, and concerns associated with trenchless/tunneling methods that factor into the trenchless/tunnel cost
- Conceptual budgetary cost estimates based on tunnel size, length, methodology, and shafts, see Appendix D

ES-1.0 Reference Documents Reviewed

This study utilized the general alignments and geotechnical data documented by others. The following reference documents were used:

1. Potential Regional Recycled Water Program Feasibility Study, Report No. 1530, November 30, 2016, Chapter 6: Conveyance System, and Appendix E: Engineer's Opinion of Probable Construction Cost for Pipelines for the Base Case, prepared by Black and Veatch and CDM Smith (Feasibility Study).
2. Potential Regional Recycled Water Program, Conveyance/Distribution System Conceptual Draft Report, November 15, 2017, Chapters 4 and 7, prepared by Black & Veatch and CDM Smith (CDR).
3. Preliminary Geotechnical/Geologic Evaluation, Proposed Regional Recycled Water Supply Program, Metropolitan Water District of Southern California, November 13, 2017, prepared by GeoPentech (Geotechnical Report).
4. Additional geotechnical reports and geologic information prepared by various government agencies and private consultants were also reviewed. References to these reports are provided in this study with a complete list included in Section 7.0 of this report.

ES-2.0 Summary of Findings

The objectives of this conceptual review are to provide MWD with an independent look at three new tunnel alignments to replace alignments proposed to be constructed using cut-and-cover methods. The most significant findings discussed in this study are as follows:

1. Class 4 cost estimates for each of the three alignments and options were developed based upon the plans and profiles provided in Appendix A. The cost and schedule for each option are summarized in Table ES-1. Additionally, Table ES-2 provides the range of budgetary total costs based on the Class 4 expected accuracy range (-30% to +50%). A 40% contingency is

recommended for budgeting purposes. The cost estimate backup materials are provided in Appendix D.

2. For the Carson to Long Beach alignment, the preferred option is Option 1B, which uses a TBM tunnel for the entire length from the Carson water treatment plant to the Los Angeles River. Option 1B estimated construction costs are $235,712,200, which includes a 40% contingency. Option 1B is estimated to take 55 months to construct. Option 1B is about $76,000,000 less than Option 1A, the pipe jacking/microtunneling option. The TBM tunnel can be constructed about 9 months faster than Option 1A.

3. For the San Gabriel River alignment, the preferred option is Option 2B, which uses a TBM tunnel for the entire length from the spreading grounds in Pico Rivera to the Imperial Highway. Option 2B estimated construction costs are $256,038,900, which includes a 40% contingency. Option 2B is estimated to take 58 months to construct. Option 2B costs about $76,000,000 less than Option 1A, the pipe jacking/microtunneling option.

4. For the Azusa to Glendora Tunnel alignment, Option 3C, the all-tunnel alternative, is the lowest cost of the three options at $63,663,700 and will take 27 months to construct. . The range of cost between the three options is about $37,000,000. Since much of the cut-and-cover work will be difficult with the large boulder field along the San Gabriel River, Option 3C, is recommended. For the option 3 tunnels, construction costs were looked at using drill-and-blast and a rock TBM. In all three options, the TBM driven tunnels were less costly and took less time to construct than tunnels excavated using drill-and-blast methods.

Table ES-1. Budgetary Cost Estimate and Schedule Summary

Alignment	Options	Tunneling Options	Tunnel or Cut-and-Cover Methods	Total Length	Segment Lengths (miles)			Duration (Months)	Budgetary Costs		
					Microtunneling or Pipe Jacking	TBM Tunneling	Cut-and-Cover		Direct and Indirect Costs	40% Contingency	Total Costs
Carson to Long Beach	Option 1A	Trenchless with 13 shafts	1/2 MTBM and 1/2 pipe jacking	4.6 mi (24,000 ft)	4.6	0	0	64	$222,736,800	$89,094,700	$311,831,500
	Option 1B	Tunnel with 2 shafts	MTBM - soft ground machine with precast concrete segmental lining	4.6 mi (24,000 ft)	0	4.6	0	55	$168,365,200	$67,347,000	$235,712,200
San Gabriel River	Option 2A	Trenchless with 14 shafts	MTBM	4.6 mi (24,100 ft)	4.6	0	0	60	$237,161,600	$94,864,600	$332,026,200
	Option 2B	Tunnel with 2 shafts	TBM - soft ground machine with precast concrete segmental lining	4.6 mi (24,100 ft)	0	4.6	0	58	$182,884,900	$73,154,000	$256,038,900
Azusa to Glendora	Option 3A	Combo Cut-and-Cover and One Rock Tunnel with 2 portals	Cut -and-C over: Sta. 1+00 to Sta. 96+00, Tunnel: St a. 96+00 to Sta. 140+00. Tunnel by rock TBM or D&B	2.6 mi (13,900 ft)	0	0.8	1.8	20	$72,398,400	$28,960,000	$101,358,400
	Option 3B	Combo Cut-and-Cover and Two Rock Tunnels with 4 portals	Tunnel: Sta. 25+00 and 76+00, Cut-and-Cover: Sta. 76+00 to Sta. 96+00, Tunnel: Sta. 96+00 to Sta. 140+00. Tunnel by rock TBM or D&B	2.2 mi (11,500 ft)	0	1.8	0.4	21	$53,804,300	$21,522,000	$75,326,300
	Option 3C	All Rock Tunnel with 2 portals	Tunnel: Sta. 24+00 to Sta. 140+00. Tunnel by rock TBM or D&B	2.2 mi (11,600 ft)	0	2.2	0	27	$45,473,700	$18,190,000	$63,663,700

Table ES-2. Budgetary Cost Estimate Ranges

Alignment	Options	Tunneling Options	Total Budgetary Costs – Class 4 Estimate		
			Low End of Range (-30%)	High End of Range (+50%)	Recommended Budgetary Total Cost 40% Contingency
Carson to Long Beach	Option 1A	Trenchless with 13 shafts	$155,915,800	$334,105,200	$311,831,500
	Option 1B	Tunnel with 2 shafts	$117,855,600	$252,547,800	$235,712,200
San Gabriel River	Option 2A	Trench less with 14 shafts	$166,013,100	$355,742,400	$332,026,200
	Option 2B	Tunnel with 2 shafts	$128,019,400	$274,327,400	$256,038,900
Azusa to Glendora	Option 3A	Combo Cut-and-Cover and One Rock Tunnel with 2 portals	$50,678,900	$108,597,600	$101,358,400
	Option 3B	Combo Cut-and-Cover and Two Rock Tunnels with 4 portals	$37,663,000	$80,706,500	$75,326,300
	Option 3C	All Rock Tunnel with 2 portals	$31,831,600	$68,210,600	$63,663,700

1.0 Overview of the Three Tunnel Alignments

The Metropolitan Water District of California (MWD) has been assessing the feasibility of augmenting its water supplies by implementing the Regional Recycled Water Program (RRWP). The components of the program include an Advanced Water Treatment Plant and distribution system of more than 60 miles of large-diameter pipeline and pump stations to convey the highly-treated reclaimed water from the Los Angeles County Sanitation Districts Joint Water Pollution Control Plant in the City of Carson to various groundwater basins in Los Angeles and Orange Counties for groundwater recharge purposes. Construction of the conveyance pipeline segments will primarily be utilizing cut-and-cover methods with trenching methods used at special crossing locations. However, there will be numerous sections of the conveyance system where trenchless/tunneling construction methods may be preferred over cut-and-cover methods to construct the pipeline.

MWD engaged McMillen Jacobs Associates' experts for a high-level review of three alignments proposed as fully tunneled options. The conceptual review focused primarily on the "big picture" elements associated with tunnels 2.6 to 4.6 miles long in a fully developed urban environment. Out of the many that could be listed, some of the elements considered in this report are ground/groundwater conditions, environmental hot zones for contamination, physical barriers that influence vertical alignment, horizontal changes in tunnel alignment to stay within existing rights-of-way (ROWs), shaft locations, tunnel diameters, and tunnel methodology. See Appendix A for a discussion of the different tunneling methods and Appendix B for a discussion of the different jacking and receiving shaft construction methods. The following three pipeline segments with associated options were evaluated in this study:

1. Carson to Long Beach:
   - Option 1A: Pipe jacking/microtunneling about 4.6 miles (Sta. 0+00 to Sta. 240+00) starting on South Main Street heading north and turning east following East Sepulveda Boulevard and West Willow Street to the east side of the Los Angeles River; crossing under the Dominquez Channel, I-710, the Los Angeles River, and other major roads.
   - Option 1B: Tunneling about 4.6 miles (Sta. 17+00 to Sta. 240+00) starting at the treatment plant on South Main Street and heading east below an existing railroad spur line. After crossing beneath Avalon Boulevard and Wilmington Avenue, the alignment crosses various industrial properties, a second railroad track, the Dominquez Channel (where it aligns on West Willow Street), and ends with the crossing of I-710 and the Los Angeles River.
2. San Gabriel River:
   - Option 2A: Microtunneling about 4.6 miles (Sta. 278+00 to Sta. 519+00) starting at Imperial Highway and following the San Gabriel River north along the Southern Edison right-of-way to Pico Rivera; paralleling the I-605; and crossing Highway 42, I-5, and other major roads.
   - Option 2B: Tunneling about 4.6 miles (Sta. 278+00 to Sta. 519+00) starting at Imperial Highway and following the San Gabriel River north along the Southern Edison right-of-way to Pico Rivera; paralleling the I-605; and crossing Highway 42, I-5, and other major roads.

3. Azusa to Glendora: Starting on Highway 39 adjacent to the City of Azusa Filtration Plant and trending along the San Gabriel River to the east and north to a point short of Morris Reservoir, where the new tunnel will tie into the existing Glendora Tunnel. Three options were included in the study:
   - Option 3A: Utilizing an initial cut and cover section (1.8 miles long) followed with a drill-and-blast tunnel (0.8 mile long) to the Glendora connection, with a total length of 2.6 miles (Sta. 1+00 to Sta. 139+00).
   - Option 3B: Utilizing an initial drill-and-blast tunnel (1.0 mile long) with a middle cut-and-cover section along Oxbow Park (0.4 mile long) and then a drill-and-blast tunnel (0.8 mile long) to the Glendora connection, with a total length of 2.2 miles (Sta. 24+00 to Sta. 139+00).
   - Option 3C: Utilizing a full-length TBM driven tunnel to the Glendora connection with a total length of 2.2 miles (Sta. 22+00 to Sta. 139+00).

For each pipeline segment and associated option, McMillen Jacobs Associates developed the following:

- Conceptual horizontal and vertical alignments considering property information, rights-of-way, and site constraints, see Appendix C
- Desk top study of the geology along the proposed alignments
- Shaft and portal locations and established pipe jacking/microtunneling drive lengths and tunnel lengths
- Typical construction risks, issues, and concerns associated with tunneling that factor into the tunnel cost
- Conceptual budgetary cost estimate based on tunnel size, length, methodology, and shafts, see Appendix D.

2.0 Estimate Development

McMillen Jacobs Associates was tasked with preparing a Class 4 construction cost estimate and schedule for each of the options. A Class 4 estimate is characterized as a concept or feasibility level estimate based on the Cost Estimate Classification Systems used by the US Department of Energy, the American Association of Cost Engineering International, and others. The primary defining characteristic of these Cost Estimate Classification Systems is the level of project definition, which for a Class 4 estimate, varies between 1 and 15 percent.

The secondary defining characteristic of the classification system is the estimating methodology used for each class of estimate. For the spectrum of low to high project definition on heavy construction projects, the corresponding spectrum of estimating methodology ranges from primarily judgement-based to deterministic as follows:

- **Class 5:** Primarily comparative, where factors and judgement, using other recent construction projects are adjusted for scale, scope, and complexity.
- **Class 3:** Primarily historical, where specific pricing information for various components of work is available from various sources, including project bid sheets. These combinations of lump sum and unit costs are applied to the gross quantifications of the major elements of work. Some comparative factors and judgement may be applied, such as adjustments for variations in geographic location.
- **Class 1:** Primarily production-based, where work is separated into discrete tasks, quantified, and production cycles are established under the specific conditions that labor crews and equipment spreads operate.

The Association for the Advancement of Cost Engineering (AACE) publishes standards for cost estimate classification. Table 2-1 provides AACE's Cost Estimate Classification Matrix.

Table 2-1. Cost Estimate Classification Matrix (AACE, 2016)

Estimate Class	**Maturity Level of Project Definition Deliverables** Expressed as % of complete definition	**End Usage** Typical purpose of estimates	**Methodology** Typical estimating method	**Expected Accuracy Range** Typical variation in low and high ranges
Class 5	0% to 2%	Concept screening	Capacity factored, parametric models, judgment, or analogy	L: -20% to -50%
				H: +30% to +100%
Class 4	1% to 15%	Study or feasibility	Equipment factored or parametric models	L: -15% to -30%
				H: +20% to +50%
Class 3	10% to 40%	Budget authorization or control	Semi- detailed unit costs with assembly level line items	L: -10% to -20%
				H: +10% to +30%

Estimate Class	**Maturity Level of Project Definition Deliverables** Expressed as % of complete definition	**End Usage** Typical purpose of estimates	**Methodology** Typical estimating method	**Expected Accuracy Range** Typical variation in low and high ranges
Class 2	30% to 75%	control or bid/ tender	Detailed unit cost with forced detailed take-off	L: -5% to -15% H: +5% to +20%
Class 1	65% to 100%	Check estimate or bid/ tender	Detailed unit cost with detailed take-off	L: -3% to -10% H: +3% to +15%

McMillen Jacobs Associates adopted a hybrid approach to developing unit costs for each of the main tunnel and shaft elements to be applied to each of the options:

- **Direct unit costs** were derived from recent Class 1 production-based estimates that were prepared for the specific means-and-methods basis envisioned to be required for the work. Since these estimates were all prepared using our proprietary in-house estimating system, the fundamental parameters of tunnel and shaft dimensions and other quantities were adjusted to the scales required for each of the project options, along with the specifics of cycle time analyses, and crew and equipment makeup. Material prices, sales tax rates, and similar material-based pricing were also adjusted as deemed necessary. These cost unit prices were then applied to the specific gross defining project quantities of pipe diameter, depth, and length.

- **Indirect costs** making up these Class 1 production-based estimates were categorized and quantified as a function of project duration or project direct cost and then applied to the direct costs.

- **Escalation** from the current 2019 4th quarter costs to the midpoint of construction, assuming a January 2021 start date.

- **Contingency** evaluated and quantified based on the following categories: (1) base cost uncertainty, such as that resulting from variability in estimating production rates or that resulting from the current or future bidding climate; (2) specific project risks such as that of encountering contaminated materials, and (3) general contingency which is a function of estimating accuracy in conjunction with project definition. Specific job risks are identified for each of the reaches and the costs estimated and carried as a direct cost add-on for each option.

The cost estimates developed for this report are for construction only. Other costs for design, construction management, internal administration, and other soft costs such as environmental and right-of-way acquisition are not included.

3.0 Carson to Long Beach

The Carson to Long Beach pipeline considered three different construction techniques and two separate alignment options. The plan and profile sheets showing the horizontal and vertical alignments with the ground conditions are included in Appendix C. Option 1A uses pipe jacking and microtunneling to construct a tunnel that follows South Main Street north and then along East Sepulveda Boulevard and West Willow Street to the east. Option 1B uses tunneling with a hybrid open/closed-face TBM that would proceed directly east along the railroad tracks and rejoin the Option 1A alignment along West Willow Street. Detailed descriptions of each of the alignments are provided below.

3.1 Option 1A – Pipe Jack and Microtunnel Alignment

The Carson to Long Beach alignment with pipe jacking and microtunneling begins on South Main Street approximately 2,000 feet north of the intersection of South Main Street and East Lomita Boulevard in the City of Carson, California. The proposed tunnel will run north approximately 2,000 feet to the intersection of South Main Street and East Sepulveda Boulevard before turning toward the east and running beneath East Sepulveda Boulevard. Along this portion of the alignment stretching from Sta. 0+00 to Sta. 80+00 both South Main Street and East Sepulveda Boulevard are surrounded by a mix of commercial and residential development.

After crossing over Wilmington Avenue at Sta. 80+00, East Sepulveda Boulevard and the proposed alignment are flanked by various heavy industrial facilities on both the north and south sides. These facilities include a large open-air surface storage lot located on the south side of Sepulveda Boulevard between Sta. 80+00 and Sta. 103+00, and the Carson location of the Los Angeles Refinery. Various other industrial facilities associated with the production and distribution of petroleum products exist along the alignment from Sta. 80+00 to approximately Sta. 185+00.

The tunnel alignment will angle slightly toward the southeast and run parallel to East Sepulveda Boulevard between Sta. 130+00 and Sta. 160+00 before rejoining and running beneath East Sepulveda Boulevard at Sta. 185+00. This segment of the tunnel will require crossing beneath various features including 14 mainline tracks and sidings of the Union Pacific Railroad (UPRR), Alameda Street, the Dominguez Channel, and additional open space used for storage and access to various tanks and other infrastructure associated with refinery operations.

After Sta. 185+00, East Sepulveda Boulevard changes its name and becomes West Willow Street. The tunnel alignment is once again surrounded by a mixture of residential and commercial properties to the north and south. The tunnel will continue to the east where it will cross below the Long Beach (I-710) freeway from Sta. 226+00 to Sta. 231+00 and the Los Angeles River from Sta. 231+00 to Sta. 237+70. In this segment the Los Angeles River is bound by levees on both the west and east banks, requiring the tunnel to cross below. The tunnel for this study will terminate at Sta. 240+00 on the east side of the Los Angeles River. Plan and profile sheets of the alignment are included in Appendix C.

The groundwater table varies along the proposed alignment. The western portion of the alignment is located above the groundwater table and will be constructed using open-face shield pipe jacking. When the pipeline extends below the groundwater table on the eastern portion, microtunneling will be used as the construction method. Details on these tunneling methods are discussed in Section 3.5.

3.2 Option 1B – Tunnel Alignment

The conventional TBM tunneling option alignment will start from the vacant land on the west side of South Main Street and head east along the BNSF railroad spur track. The tunnel alignment will follow the BNSF railroad track to the east for approximately 4,350 feet, crossing beneath Avalon Boulevard and Broad Street. The BNSF railroad alignment is approximately 95 feet wide with a single railroad spur track and occasional sidings. North and south of the alignment is a mixture of commercial development near South Main Street and residential neighborhoods near Broad Street.

The tunnel will cross below the parking lot of a commercial building between Broad Street and East Street before heading approximately 1,500 feet to the east beneath East Delores Drive to the intersection of East Delores Drive and Wilmington Avenue. This segment of the alignment consists of residential neighborhoods to both the north and south.

After crossing Wilmington Avenue, the tunnel will proceed below a large open-air surface storage lot, industrial land associated with the production and distribution of petroleum, 14 mainline tracks and sidings of the UPRR, Alameda Street, various large storage tanks associated with refinery operations, and the Dominguez Channel before intersecting West Willow Street. The tunnel will then cross below a second set of UPRR railroad tracks along West Willow Street. The alignment will then head approximately 4,400 feet to the east along West Willow Street. West Willow Street includes residential neighborhoods to the north and south.

The tunnel will continue to the east where it will cross below the Long Beach (I-710) freeway from Sta. 226+00 to Sta. 231+00 and the Los Angeles River from Sta. 231+00 to Sta. 237+70 (similar to Option 1A). In this segment the Los Angeles River is bound by levees on both the west and east banks requiring the tunnel to cross below. The tunnel for this study will terminate at Sta. 240+00 on the east side of the Los Angeles River.

The groundwater table varies along the proposed alignment. The western portion of the alignment will be above the groundwater table and will be constructed using a TBM in open face mode. When the pipeline/tunnel extends below the groundwater table on the eastern portion, the TBM will be operated in closed face mode. Details on the tunneling methods are discussed in Section 3.5.

3.3 Geotechnical Conditions

Both Carson-to-Long Beach trenchless/tunnel options will cross unconsolidated settlements of the Los Angeles Basin within the Peninsular Ranges' geomorphic province (CGS, 2002). The ground deposits along the excavation profile of either alignment can be grouped into three distinct units based on geologic origin and engineering characteristics. The following native units will be encountered along the two alignments.

3.3.1 Unit 1: Non-Marine Terrace Deposits (Qt)

Non-marine Terrace deposits are associated with Pleistocene to late Pleistocene uplift of ground deposits within the Torrance Plain (AMEC, 2011). These units are mapped separately from alluvial deposits on the Long Beach Sheet of the Geologic Map of California (Jenkins, 1962) and were characterized as older alluvium by Dibblee et al. (1998, 1999).

Non-marine terrace deposits were encountered in two separate geotechnical investigations for the Western Laboratories (north of Sta. 80+00, near Receiving Shaft R3) and for the Globus Service Building (south of Sta. 105+00 and east of Jacking Shaft J3); see plan and profile sheets in Appendix C (Western Laboratories, 1987; and Globus, 2012). These ground types consist of alternating layers of fine- and coarse-grained soils. Coarse-grained soils are predominantly medium dense to dense silty and poorly graded sand with varying fines content while fine-grained soils consist of hard to very stiff silt and clay (SM, SP, CL, and ML based on the United Soil Classification System [USCS]).

When encountered during tunneling, terrace deposits are anticipated to exhibit slow to cohesive raveling above the groundwater and running to flowing when encountered below the groundwater table.

3.3.2 Unit 2: Slough (Qs), Compressible Clay (Qem)

This segment of the tunnel will pass from the uplifted portion of the Torrance Plain and transition to the alluvial sediments of the Los Angeles River. Slough and compressible clay materials represent former low-lying areas that formed as sea-level receded. The slough materials consist of soft, highly plastic fat clay and elastic silt (CH, MH, soil types) while the compressible clays consist of low plasticity lean clay with higher sand content (CL soil type). These units are soft in consistency with Standard Penetration Test (SPT) blow counts of less than 5.

The low-lying drainage sloughs associated with the soft soil deposits were filled as the Los Angeles refinery grew and developed. The United States Geologic Survey (USGS) collection of historical topographic maps was reviewed to better understand the predevelopment location of these drainage features in relation to the tunnel. Figure 3-1 provides an overlay of the 1902 Downey Quadrangle on a recent satellite image with the proposed alignments shown.

Figure 3-1 indicates that during the time of the survey, slough features partially contained some standing water and were referred to as "Watson Lakes." The 1902 map also shows a predevelopment drainage channel of the Los Angeles river crossing the proposed alignments near the intersection of Sepulveda Boulevard and Middle Road.

AMEC and Geomatrix prepared a geotechnical investigation report for new tank development within the Kinder-Morgan Carson Terminal (tank farm). This project was located within and above one of the predevelopment drainage sloughs. As part of their work, AMEC and Geomatrix prepared several cross sections through the drainage slough and compressible slough deposits. Figure 3-2 shows two of the AMEC sections with the projected location of the proposed tunnel alignment. The soft, compressible nature of the materials suggests that it would be advantageous to put the tunnel below these soft soil deposits, which extend to a depth of approximately 30 to 40 feet along the tunnel alignment. Soft slough and compressible clay soils are anticipated to exhibit a slow raveling to squeezing type behavior when encountered during tunnel excavation.

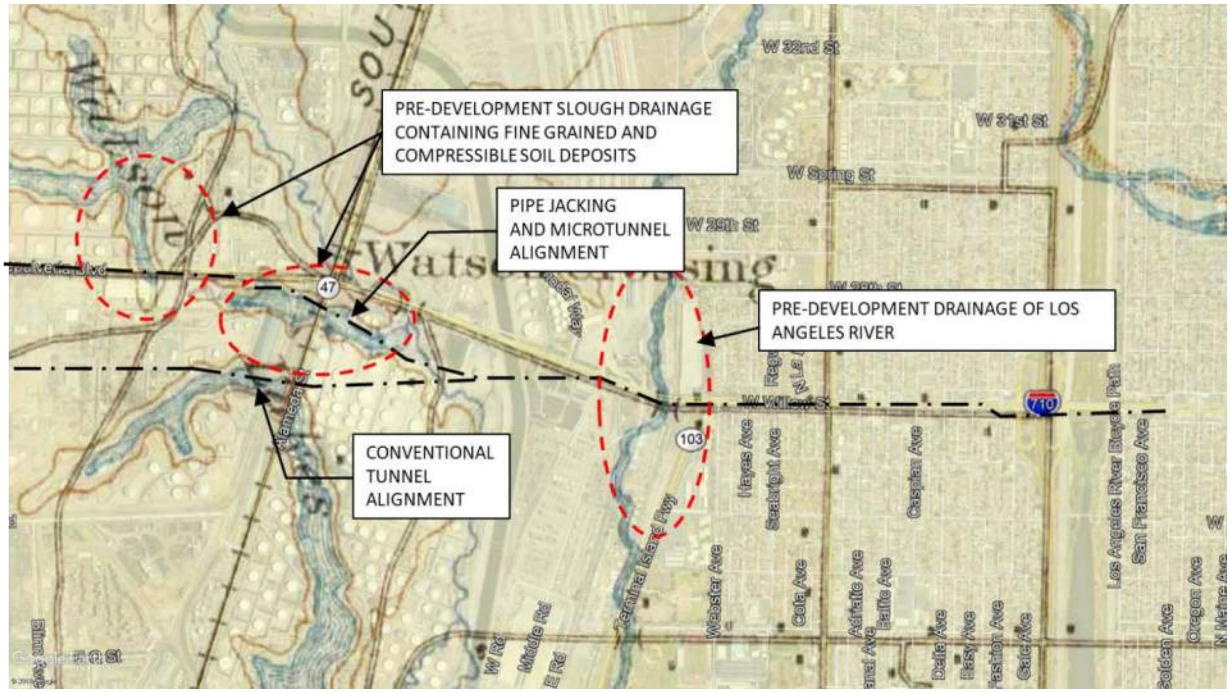


Figure 3-1. Overlay of 1902 Downey Quadrangle showing slough locations

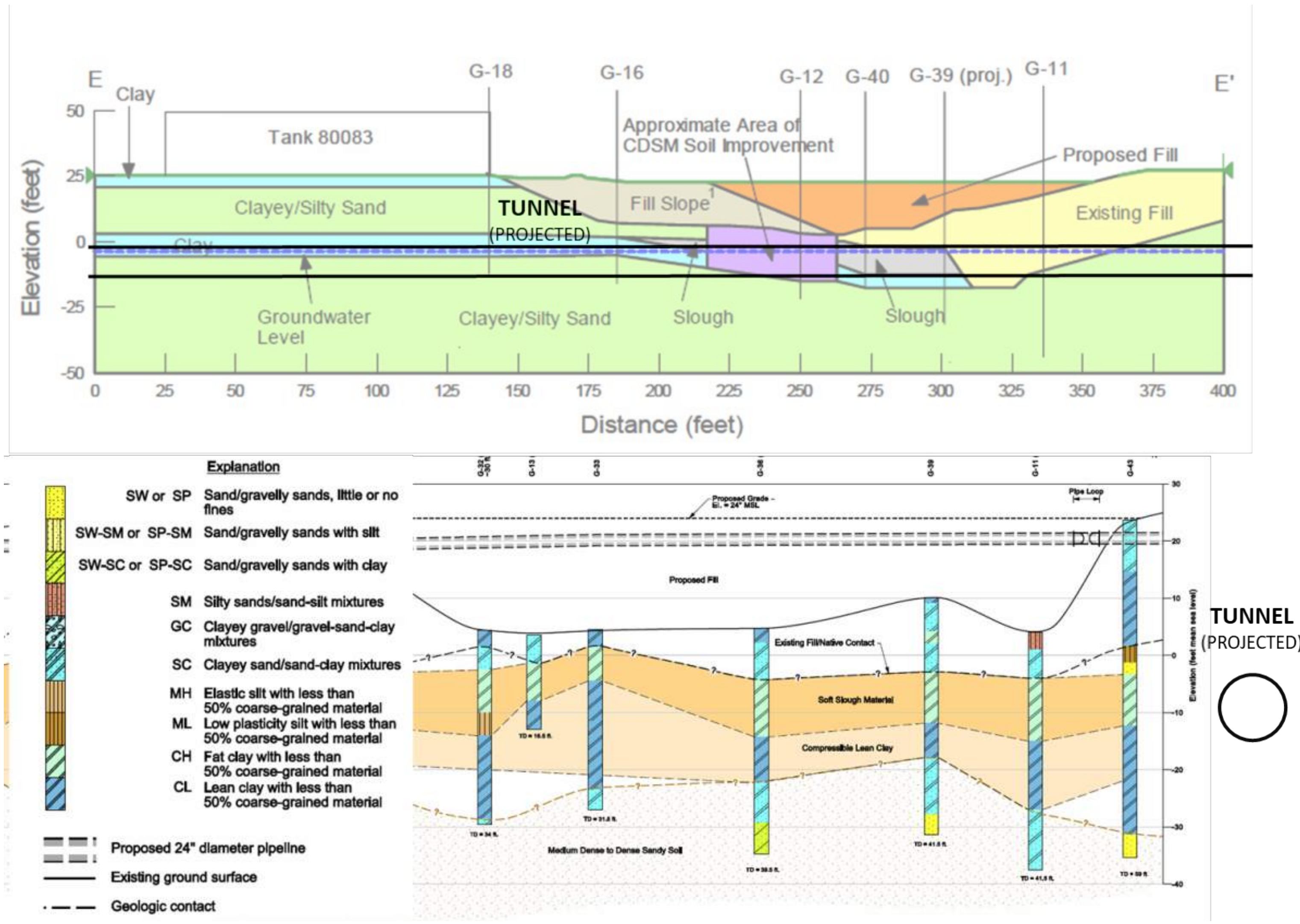


Figure 3-2. AMEC cross section through slough with proposed tunnel projected

3.3.3 Unit 3: Alluvium (Qa)

East of the Dominguez Channel and below the slough deposits described above, alluvial soils will be encountered within the excavation profile of the tunnel. These alluvial deposits are associated with the meandering path of the Los Angeles River predevelopment. Alluvial soils are expected to consist of alternating, discontinuous layers of medium dense to dense sand and gravel with varying quantities of fines (SM, SP, SW, GM, GW soil types). Occasional thin layers of stiff silt and clay (ML and CL soil types) may also be encountered. When encountered during tunneling, alluvial soils are anticipated to exhibit fast raveling to cohesive raveling when exposed above the groundwater and running to flowing when encountered below the groundwater table.

3.4 Groundwater

Groundwater information along the alignment was reviewed from several different sources. These included geotechnical investigations prepared by various consultants near the alignment, Los Angeles County Department of Public Works (LACODPW) groundwater wells database, and historical high groundwater maps prepared by the California Geologic Survey (CGS). Figure 3-3 depicts the historical high depth to groundwater map from the Torrance and Long Beach Quadrangles prepared by CGS. Plotted on Figure 3-3 are the locations of LACODPW wells that were judged to be closest to the two proposed alignments. Records of groundwater levels for each well were researched, and the historical high (lowest depth) readings are included in Table 3-1.

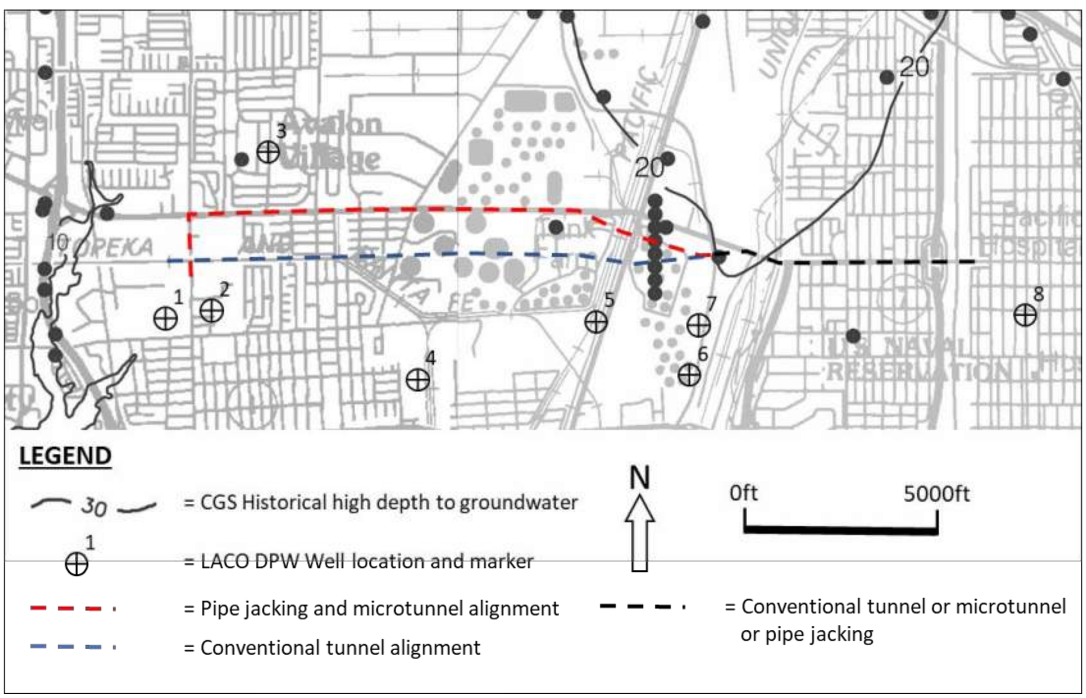


Figure 3-3. Historical high groundwater map and LACO well locations (Carson-Long Beach)

Table 3-1. LACO Groundwater Well Data (Carson-Long Beach)

Marker #	LACO Well #	Historical High Depth (ft)	Historical High Elevation (ft)	Date
1	320E	62.6	-36.5	10/10/2008
2	320L	69.0	-28.1	2/3/2009
3	838	78.0	-46.0	4/10/1995
4	340A	63.0	-17.9	1/19/1941
5	360G	73.8	-34.6	9/2/1982
6	370N	56.0	-44.9	4/6/2006
7	370T	14.9	1.6	4/10/2007
8	400	12.4	0.2	10/3/2006

Groundwater levels are highest along the eastern half of the alignment and deeper toward the west. The CGS map suggests that the historical high depth to groundwater was approximately 10 to 20 feet along Main Street. Geotechnical investigations near Sta. 80+00 and Sta. 105+00 for Option 1A (pipe jacking/microtunneling) alignment did not encounter groundwater within the maximum exploration depth of 50 feet (Western Laboratories, 1987; Globus, 2012). Recent geotechnical exploration by AMEC and URS near the Dominguez channel encountered groundwater at depths of approximately 15 feet (approximately 0 feet mean sea level). Groundwater levels have risen in the vicinity of the Dominguez Channel because of the Dominguez Gap Barrier Project (DGBP). The DGBP utilizes groundwater injection wells to recharge the Gaspur aquifer and prevent seawater intrusion.

Groundwater elevation can be considered at 3 feet mean sea level from Sta. 0+00 to Sta. 120+00 along the Option 1A alignment, with deeper depths possible from Sta. 0+00 to Sta. 120+00. After Sta. 120+00 to the end of the alignment groundwater can be assumed at approximately 15 feet below the ground surface in the vicinity of the alignment. This 15-foot depth is approximately equal to El. +3 to El. +10. Likewise, groundwater elevation should be considered at 3 feet mean sea level from Sta. 0+00 to Sta 100+00 for the Option 1B alignment and at a depth of approximately 15 feet after Sta. 100+00 (El. +3 to El. +10).

3.5 Selected Trenchless/Tunneling Methods

Two options are considered as tunneling methods for this alignment from Carson to Long Beach because of the constraints caused by the existing structures and natural features as well as the geologic conditions along the two option alignments:

- **Option 1A:** A combination of pipe jacking and microtunneling is proposed in this option along the East Sepulveda Boulevard, with a minor deviation between Alameda Street and Dominguez Channel. Seven receiving shafts and six jacking shafts are presumed along this alignment to facilitate the pipe jacking/microtunneling. As summarized in Section 3.2, this segment consists primarily of a mixture of stiff and soft material that behaves as slow to cohesive raveling above the groundwater table and running to flowing below the groundwater. Based on the historical data, the groundwater table is observed at the depths of 50 feet or deeper up to Sta. 120+00 and at the shallower depths afterwards. Consequently, pipe jacking is advised from Sta. 0+00 to Sta. 120+00, where the pipe jacking is placed above the groundwater table, and microtunneling from the Sta. 120+00 to Sta. 240+00, where the microtunnel passes through the contaminated soils and compressible slough material below the groundwater table.

- **Option 1B:** TBM excavation with a jacking and receiving shaft at either end of the alignment is proposed in this option. This option has less surface impact than Option 1A as no intermediate shafts are required. Since the groundwater table varies along the proposed alignment, the initial western portion of the alignment will be constructed using a TBM in open face mode. When the pipeline extends below the groundwater on the eastern portion, the TBM will be operated in closed face mode with slurry. The tunnel will be driven west to east and downslope. The reason is that the jacking shaft will be located on the west end, where there is more working room and easier site access. The tunnel will be driven down slope because the east end is at a lower ground elevation than the west end. Driving a tunnel downslope is done frequently, especially with a closed-face TBM, to prevent the entry of free groundwater into the tunnel. A sump pump will be required in the tunnel at the TBM to collect and remove nuisance construction water that will flow downslope to the TBM.

3.6 Traffic and Public Impacts

Option 1A using pipe jacking and microtunneling will require a total of 13 shafts along East Sepulveda Boulevard, South Main Street, and West Willow Street, which will significantly impact traffic at each shaft location. Ideally, jacking shaft sites should provide a minimum 30 foot x 200 foot area for shaft, equipment and material staging, but constrained sites can potentially be narrowed to 15 feet or 16 feet in certain cases depending on shoring methods. Receiving shaft sites require less space and can be 15 feet to 20 feet wide and 80 feet to 100 feet long.

The majority of the Option 1A alignment runs along streets with two lanes in each direction. These streets typically have a median with intermittent turn lanes in the center, and additional lanes at some of the larger intersections. Jacking and receiving shafts have been approximated along the alignment at roughly even spacing for the purposes of this conceptual evaluation. If this option were advanced to a preliminary design phase, a more detailed evaluation of each shaft site would be needed to site the shafts to minimize public impact. It is preferred that additional staging/laydown areas be obtained off of the street but adjacent to the shaft site, to minimize the impact within the traffic lanes.

Each shaft site will require the closure of one to two lanes of traffic during the construction period. Medians may need to be temporarily removed to minimize traffic impacts at certain sites. Jacking shafts will be expected to impact traffic for periods of one to four months during the construction of the shafts and tunnels. Receiving shaft sites are needed for shorter lengths of time, and can be constructed over a period of one to three weeks, then plated over to allow for traffic passage until machine removal and shaft restoration, which would take an additional one to two weeks. The contractor would likely use two machines simultaneously: one for pipe jacking and one for microtunneling. Up to four to six shafts sites along the alignment would need to be in use at any given time during the project.

For Option 1B, the conventional TBM tunnel, anticipated traffic impacts are greatly reduced by locating the jacking shaft on vacant land. The alignment requires no intermediate shafts, so traffic impact is limited to the receiving shaft at the east end of the alignment on West Willow Street. Impacts at this location will be similar to those described for the receiving shafts for Option 1A.

For both options, additional staging areas for storage of equipment and materials will be needed along the alignment at available properties. Trucks delivering pipe spools and segmental tunnel linings between the shaft and staging areas as well as off-site supply and disposal locations may also cause traffic impacts in the project area.

3.7 Identified Risks Along the Alignments

The Option 1A and Option 1B alignments poses multiple risks to the project overall. The following list describes the nature of various risks along the alignment.

- **Contaminated Ground and Groundwater:** Option 1A and Option 1B will encounter contaminants in the ground and groundwater. Contamination of these earth materials are a biproduct of industrial land use primarily for petroleum production in the vicinity of the alignment. Review of geotechnical reports (AMEC, 2011) reference the presence of contaminants but do not provide details on the nature or type of contaminants. While extensive documentation on the nature, type, and distribution of contaminated ground associated with refinery operations exists, it was not obtained as part of this study.

  Ground and groundwater removed during any shaft or tunnel construction will need to be tested on site; properly documented, drummed, and removed; and disposed of at the appropriate facility. Additional, extensive research into the documented contaminants along the alignment and a field exploration program will help to narrow down the portions of the alignment where contamination is most likely. The cost of removing contaminated ground and groundwater can more accurately estimated after identifying contaminant “hot zones.” To minimize the handling of the contaminated ground and groundwater, shafts will be constructed using watertight support systems where the pipeline is below the groundwater table. Only the ground excavated from within the shafts will need to be tested. The tunneling methods below the groundwater table will be closed systems with positive face pressure to avoid handling and drawing in the contaminated groundwater into the tunnel excavation.

- **Organic and Soft Ground Conditions:** Section 3.2 describes the ground conditions along each of the proposed alignments. Soft ground conditions consisting of organic silts, fat clays, and lean clays can be found at select locations along the alignments. The presence of these soils is associated with predevelopment sloughs.

  Soft ground conditions can result in an unstable tunnel face that extrudes or squeezes into the front of the TBM, pipe jacking shield, or MTBM. If the face is not properly supported, the inward movement of ground can result in overexcavation, leading to settlement, subsidence, and sinkholes. Option 1B carries the greatest risk of soft ground conditions, as the tunnel is deeper and passes through or in close proximity to many of the old slough deposits with existing structures located above such as tanks, piping, and other refinery infrastructure. The degree of sensitivity of this infrastructure to ground movement is not known at this time.

The development of a robust exploration program that properly characterizes the ground in terms of the material properties in the vicinity of the slough deposits shown in Figure 3-1 is essential to reducing the risk of ground movement associated with the soft deposits. Locating the lateral and vertical limits of soft deposits will allow the tunnel to potentially go deeper to avoid the soft soil units, while providing an engineering characterization of the materials will allow for meaningful, and accurate analysis of ground movement if the soft deposits cannot be avoided.

- **Cobbles and Boulders:** Cobbles and boulders are most likely to occur east of the Dominguez Channel and are associated with alluvial deposition from the Los Angeles River. They present a risk to the project because of the difficulty in removing them from the front of the shield, MTBM, or TBM. The risks associated with cobbles and boulders include slowing the rate of excavation and possible stoppage of the tunnel equipment. For this project, the diameter of the final pipeline is large relative to the likely cobble and boulder sizes, allowing the cobbles and boulders to be digested with proper cutter wheel tooling. The tunnel size is large enough that a contractor could use a machine with face access to deal with nested cobbles and boulders.
- **Buried Objects and Fill:** The presence of artificial fill occurs throughout both tunnel alignments but is primarily at or near the surface. Fill is most prevalent as roadway and railroad base and will likely be present in the top 0 to 10 feet at all shaft locations. Deep areas of fill are present where predevelopment drainage features once existed and have subsequently been filled in. Review of the 1924 Compton topographic map constructed with a 5-foot contour interval suggests that fill deposits are up to 25-feet deep in these areas.

  All artificial fill should be considered suspect in its quality and competency. In addition, debris could be present in the fill that would obstruct and require additional effort to remove while excavating a shaft. The implementation of a geotechnical exploration program will help identify the depth, lateral extent, and competency of artificial fill and help to mitigate the risk. The presence and frequency of obstructions and debris in the fill should also be part of a geotechnical baseline report that will assign the risk of encountering obstructions between MWD and contractor.

- **Pile Supported Structures:** Where the alignments cross through the tank farm, there is potential for pile supported tanks and other structures which may require re-routing or deepening alignments.
- **Gassy Ground Conditions:** Oil rigs are operating along the alignments for Option 1A and Option 1B. Gassy ground from methane and heavier petroleum products will be encountered by any underground work. Documentation on the nature, type, and distribution of gassy ground conditions will need to be identified during the geotechnical exploration phase. The mining equipment used will need to be intrinsically safe and requirements outlined by the Cal/OSHA permit followed.
- **Railroad Crossing:** The selected alignments for Option 1A and Option 1B will need to cross railroad lines at the ground surface. Excessive ground surface settlement poses a safety and operational risk to passing trains. Strict control and limitations on settlement at railroad crossings are essential. The risk of settlement can be mitigated by developing operational criteria during tunnel construction, requiring continuous mining when within the railroad zone of influence, and implementing an instrumentation and monitoring program.

- **Utilities:** Each alignment will cross below or next to many utilities owned by government entities or associated with operations in the refinery. Detailed records research, identification, and coordination with various owners will be essential. Even with due diligence, unknown utilities are likely to be encountered during construction. Risk mitigation measures for encountering utilities would include placing the alignment deeper in the profile to avoid areas of higher utility concentrations, relocating utilities prior to construction, and developing contingency measures if utilities are encountered.
- **Traffic and Public Impacts:** While tunneled pipe construction options reduce traffic and public impacts more than cut-and-cover construction, they do not eliminate these impacts completely. Traffic impacts are limited to shaft sites, but these sites may take up more roadway width than cut-and-cover construction, and a given shaft site will be affected for longer than any given cut-and-cover, which will move linearly along with the construction. As described in Section 3.6, Option 1B greatly reduces the effect to the public by eliminating all but one shaft in the public right-of-way.
- **Levees and Embankments Associated with Rivers and Channels:** The tunnel alignments will cross below the Dominguez Channel and the Los Angeles River. The Dominguez Channel flows within a riprap embankment where each alignment would cross while the Los Angeles River flows within a concrete channel that is bounded by levees. The risk of affecting these structures through vertical settlement and lateral strain is present. Risk mitigation may include placing the tunnel alignment deeper in the profile below all channels and embankments, or establishing good face control operational practices during the tunnel construction.

3.8 Budgetary Costs for the Trenchless/Tunnel Options

McMillen Jacobs Associates was tasked with preparing a Class 4 cost estimate and schedule for each of the options. McMillen Jacobs developed unit costs for each of the main tunnel and shaft elements and then applied them to each of the options. We utilized our proprietary in-house estimating software to prepare the unit costs.

The unit costs were prepared on a means-and-methods basis. The work was divided into discrete tasks, and for each component element of work making up the method, a takeoff was performed that quantified the amount of material required for that element in such terms as cubic yards of excavation, square feet of shoring, lineal feet of pipe, cubic yards of backfill, etc. A cycle time analysis was performed to determine the likely rate at which the task could be executed based on a specific crew size and equipment spread handling the relative amounts of each type of material required. In this fashion, the cost of performing each discrete task was tabulated in terms of labor, equipment, material, and subcontract costs. The construction costs are based primarily on production rates calculated for conditions specific to this contract. Historical production rates used are based on the estimator’s past records and experience, and modified as necessary for local geographic location and conditions.

The total costs are indicated in Table 3-2, with backup documentation in Appendix D. The summary sheet includes specific costs for each shaft and trenchless/tunnel drive.

For Option 1A, drives 1 through 6 were pipe jacked (work above the groundwater table), while drives 7 through 12 were microtunneled. Shafts J1, J2, J3, R1, R2, R3, and R4 were constructed using soldier beams and lagging, with internal bracing. Watertight shafts J4, J5, J6, R5, R6, and R7 were constructed using sheet piles with internal bracing. The jacking shafts were 30 feet by 20 feet, except Shaft J1, which was 30 feet by 30 feet. The receiving shafts were 15 feet by 15 feet. Option 1A is estimated to take 64 months to complete, with a pipe jacking shield and MTBM working concurrently.

Table 3-2. Summary of Tunneling Construction Costs for Options 1A and 1B

Option	Tunneling Method	Cost	Schedule
1A	Pipe Jacking and Microtunneling	Direct and Indirect $222,736,800	64 months
		40% Contingency $89,094,700	
		Total $311,831,500	
1B	Open- and Closed-Face TBM Tunneling	Direct and Indirect $168,365,200	55 months
		40% Contingency $67,347,000	
		Total $235,712,200	

The following assumptions were used in the development of the Option 1A cost estimate:

- The fixed amount for miscellaneous costs included road decking construction and maintenance at receiving shafts, traffic control, and geotechnical instrumentation.
- Used 108-inch ID casing with 1 inch wall thickness, and 84-inch-diameter steel carrier pipe with 0.5-inch wall thickness and welded connections.
- Used three levels of bracing and 5-foot-thick slab in all shafts.
- Used cellular backfill in the annular space.
- Assumed the upper 5 feet of shoring will be cut and removed, with the remaining shoring abandoned in place.
- Assumed CDF backfill of all shafts.

For Option 1B, the jacking and receiving shafts were constructed using secant piles with diameters of 40 and 24 feet, respectively. Tunnel excavation included the mobilization and use of an earth pressure balance (EPB) TBM with concrete bolted gasketed segmental lining. Option 1B is estimated to take 55 months to complete, with a single TBM used in both open and closed mode.

The following assumptions were used in the development of the Option 1B cost estimate:

- Both shafts included ground improvement for break in/out.

- Receiving shaft cost included dewatering.
- Tunnel cost assumed 10-inch-thick concrete segments.
- Used an 84-inch diameter steel carrier pipe with 0.5-inch wall thickness and welded connections.
- Carrier pipe installation included cellular backfill in the annular space, contact grouting, and lining repairs.
- Assumed the upper 5 feet of shoring will be cut and removed, with the remaining shoring abandoned in place.
- Assumed CDF backfill of all shafts.

Labor rates were established for each category of craft labor required using prevailing wage rates published for the Los Angeles area, and fully burdened to include payroll taxes and insurance. Appropriate allowances were made for shift differential pay and travel time pay, where called for.

Consumable materials (i.e., materials used in construction but not incorporated into the final product), permanent materials (materials incorporated into the final product), and subcontract items were based on a combination of published database rates for the region, and recent costs from similar projects, as the limited time allotted to prepare the costs did not allow for specific quotes for these items to be obtained from vendors.

Equipment operating rates were tabulated using algorithms established in the latest edition of the United States Army Corps of Engineers' Construction Equipment Ownership and Operating Expense Schedule for Region VII. These algorithms are based on historical records of equipment component usage and tied to specific requirements relating to the equipment model, horsepower, tire size, etc. Ownership costs for the specialized tunneling equipment will vary depending on how the project is separated out into different contract packages. Equipment depreciation is the cause of those variances.

The shaft and tunnel costs indicated are direct costs only, are calculated in 2019 dollars, and do not include mobilization/demobilization, indirect/overhead, profit, or contingency costs. Indirect/overhead and profit costs for this type of work will range between 30 and 50% of the direct costs, and are influenced by such things as: contract size and packaging, bidding climate/market conditions, and individual contractor's backlog. For this project we have assumed markups of: 5% for mobilization/demobilization; 25% for indirect costs and overhead; and 15% for profit. Contingency varies with the level of design definition, decreasing as the definition increases. At this level of design, we recommend 40% be added to the direct and indirect costs for contingency.

3.9 Recommendations if Tunnel Options Continue

The following list presents recommendations if tunneling is still considered a valid option for construction of the recycled water pipe. This list constitutes "next steps" to continue to advance the project.

- **Research and Records Requests:** Additional research is needed to supplement the documentation that was reviewed for this report. Items to research include:

  - Presence, extent, and type of environmental contamination associated with the refinery.
  - Geotechnical and construction information for the Los Angeles River levees and the Dominguez Channel embankment.
  - Groundwater recharge rates, well locations, and long-term goals for the DGBP.
  - Identification of existing utilities along the alignment.
- **Alternate Alignments:** Additional alternate alignments other than the two proposed in this report could be developed. Specifically, consideration should be given to moving the pipe jacking and microtunnel alignment slightly north of East Sepulveda Boulevard between Sta. 130+00 and Sta. 180+00. Alternate alignments in the west to east traverse of the Option 1B tunnel may also be considered. Additional shafts may be added to the alignment to facility more abrupt turns.
- **Design Criteria:** Develop design criteria for either of the alignments. The design criteria should include:
  - Crossings beneath the Los Angeles River and Dominguez Channel.
  - Railroad crossing.
  - Seismic design criteria and liquefaction susceptibility.
- **Property Inventory:** Research and collect records of all property owners and ROWs along both alignments. Collect building and foundation records for all structures above and adjacent to the alignment.
- **Geotechnical Investigation:** Develop a detailed geotechnical exploration program that adequately characterizes ground and groundwater conditions along the chosen alignment. Geotechnical data should be summarized in a Geotechnical Design Report (GDR).
- **Baseline Ground Conditions:** After completion of the GDR, the ground conditions should be baselined and included in a Geotechnical Baseline Report (GBR). This report will serve to allocate risk for subsurface conditions between MWD and the tunnel contractor.
- **Develop an Extensive Risk Registry:** An extensive risk registry should be developed that ranks risk along the selected alignment. This registry would be a working document that is continuously updated as design and exploration proceed.
- **Building and Utility Settlement Study and Protection:** After completion of a geotechnical investigation the effects of tunneling and ground movement on adjacent structures should be evaluated. Structures deemed to be at-risk of damage from tunnel excavation should be protected by appropriate mitigation measures.

4.0 San Gabriel River

The San Gabriel River segment for pipeline construction considered two different tunnel construction techniques along the same alignments. The plan and profile drawings showing ground conditions are provided in Appendix C. Option 2A considers the use of microtunneling to construct a tunnel along the east bank of the San Gabriel River. Option 2B considers the use of a closed-face TBM to construct a tunnel along the east bank of the San Gabriel River. Detailed descriptions of each option are provided below.

4.1 Option 2A – Microtunnel Alignment

The San Gabriel River alignment will run along an approximately south to north path through the communities of Downey, Santa Fe Springs, and Pico Rivera. Much of the access and right-of-way for the alignment will be obtained through the jurisdiction of the United States Army Corps of Engineers (USACE), California Edison, and UPRR.

The alignment will start on the southside of the Imperial Highway and cross beneath the highway before crossing below the USACE flood control channel for the San Gabriel River. In this section the San Gabriel River flows within a concrete channel and is contained by levees on the east and west banks. After crossing the river at Sta. 296+00, the tunnel will continue north along the river on the east bank just to the east of the existing levee. The pipeline will cross beneath various structures from Sta. 296+00 to Sta. 506+00 including three railroad crossings, the Santa Anna (I-5) freeway, and six secondary arterial roads. The alignment will terminate on the north side of Washington Boulevard.

The alignment will mainly follow the east bank of the San Gabriel River and occupy land between the USACE levees and residential or railroad property to the east. Many potential shaft locations along the alignment will need to occupy existing public parks or other municipal land.

4.2 Option 2B – Tunnel Alignment

The use of a closed-face TBM to construct the segment along the San Gabriel River has been considered. This alignment is the same as that for Option 2A. The tunnel will start from a jacking shaft constructed within the spreading basins on the north side of Washington Boulevard. Tunneling will proceed to the south and cross under Washington Boulevard and the USACE levee on the west and east banks of the river before following the Option 2A alignment on the east side of the river. The tunnel will continue to the south. The tunnel will cross back to the west side of the river. The tunnel will cross below the east and west banks of the USACE levees and Imperial highway before entering a receiving shaft located on utility right-of-way land owned by Southern California Edison.

The tunnel will cross below three railroad bridges and five highway or secondary roadway bridges. A review of California Department of Transportation (Caltrans) foundation information for the Imperial highway (I-105), the Santa Ana freeway (I-5), and the Firestone Boulevard bridges shows that all the bridges are supported on pile foundations. While not reviewed, it is likely that all the other bridges are also supported on piles. The tunnel will cross either between or below the piles. In addition, drop structures are also present along the river below which the tunnel will need to cross. The San Gabriel

River is concrete lined to approximately 1,300 feet east of Firestone Boulevard and unlined upstream of that point.

4.3 Geotechnical Conditions

The proposed alignment is located within the Los Angeles basin. The Los Angeles Basin is part of the Peninsular Ranges geomorphic province and is characterized by a series of northwest trending mountain ranges and valleys separated by faults into various structural blocks (CGS, 2002). The subject site is located within the central block of the Los Angeles Basin southwest of the Elysian Park thrust hills and northeast of the Newport-Inglewood fault (USGS, 1965; Bilodeau, 2007). The proposed alignment follows the course of the San Gabriel river south of where the river flows through the Elysian Park thrust at the Whitter Narrows.

Primary surficial deposits along the alignment consist of various sequences of alluvial deposits from the San Gabriel and Santa Ana Rivers. Recent alluvial deposits of the San Gabriel River overlie older alluvial deposits referred to as the Gaspur Aquifer and Lakewood Formation (USACE, 1963).

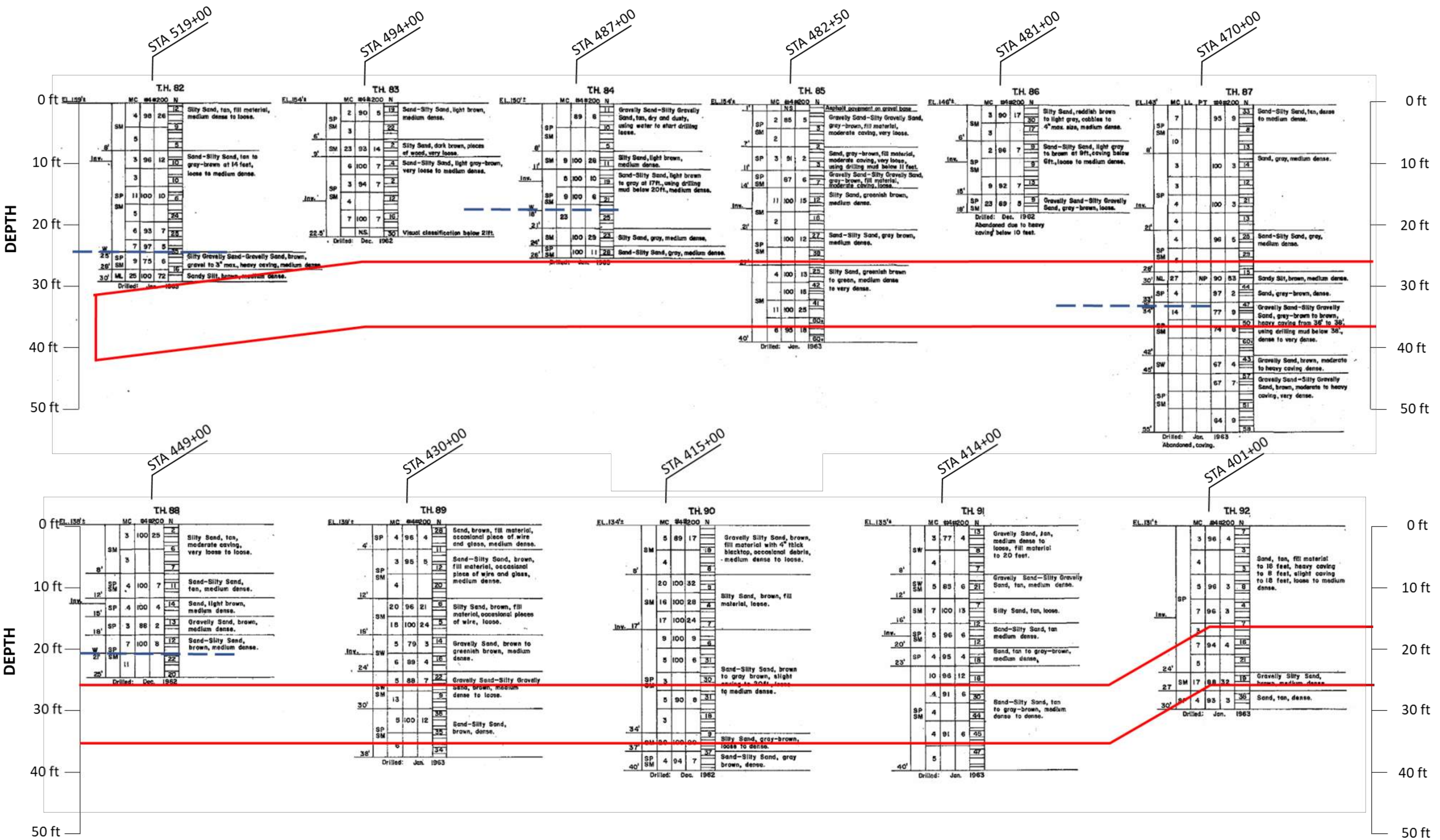
Geotechnical boring logs prepared by the USACE in 1963 for construction of the current levee system along the San Gabriel River were reviewed to assess subsurface conditions. In addition, boring logs prepared by Caltrans were also reviewed. Caltrans logs were available at various bridge crossings and at select locations along the San Gabriel River (I-605) freeway. I-605 parallels the proposed alignment at distances ranging from approximately 60 to 1,500 feet.

Subsurface conditions and soil types are illustrated in Figure 4-1 and Figure 4-2, which show a schematic cross section that presents the USACE 1963 borings and some select Caltrans borings considered during levee construction. These borings are closest to the proposed alignment and most accurately represent ground types and consistency that will be encountered. The proposed tunnel stationing is projected onto the USACE borings in Figure 4-1 and Figure 4-2, along with a schematic representation of the tunnel excavation profile. Idealized geology and to-scale plan and profile sheets of the proposed tunnel are provided in Appendix C.

Figure 4-1 and Figure 4-2 illustrate that the geotechnical profile along the alignment consists of levee fill overlying native alluvial soils. No artificial fill is assumed to underly the channel of the San Gabriel River. Native soils are primarily composed of dense to very dense silty sand, well graded sand, and poorly graded sand with varying quantities of fines and gravel (SM, SP-SM, SW-SM soil types). Occasional dense to stiff interbedded silt and clay layers are also observed in Caltrans logs. Coarse-grained soil layers rich in gravel are generally noted past a depth of 30 feet. While not explicitly mentioned in the logs reviewed, soil layers are likely to contain occasional cobbles in either discrete layers or in isolated, discontinuous deposits.

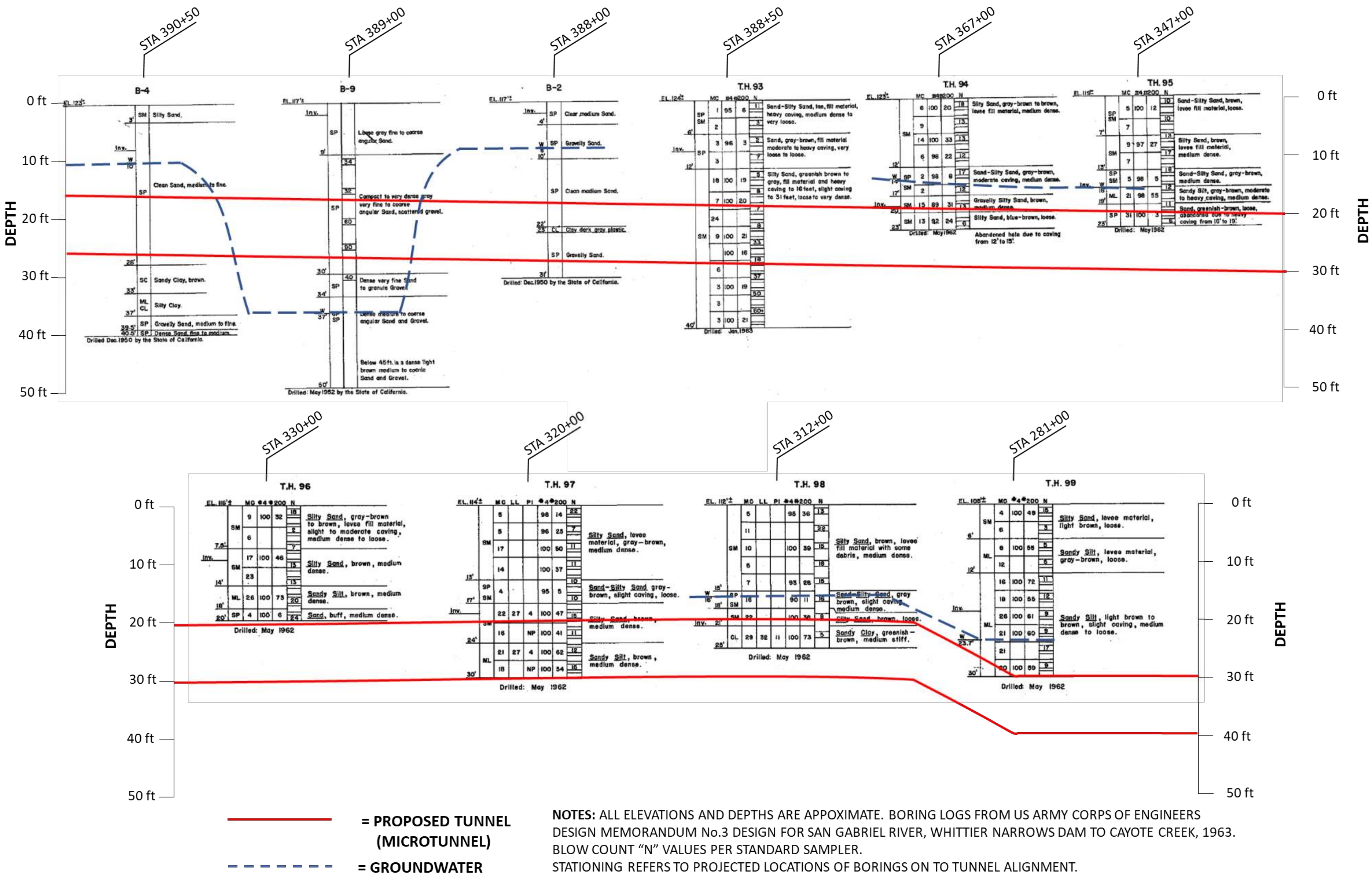
Boreholes excavated in native soils frequently require drilling mud to maintain stability and prevent caving. If encountered within an open face excavation, native soils above the groundwater table will exhibit slow raveling to cohesive running and running. Native soils below the groundwater table will exhibit fast raveling to flowing type behavior based on the Tunnelman’s ground classification system (Heuer 1974, after Terzaghi 1950).

This page intentionally left blank.



NOTES: ALL ELEVATIONS AND DEPTHS APPOXIMATE. BORING LOGS FROM US ARMY CORPS OF ENGINEERS DESIGN MEMORANDUM No.3 DESIGN FOR SAN GABRIEL RIVER, WHITTIER NARROWS DAM TO CAYOTE CREEK, 1963.
BLOW COUNT "N" VALUES PER STANDARD SAMPLER.
STATIONING REFERS TO PROJECTED LOCATIONS OF BORINGS ON TO TUNNEL ALIGNMENT.
HORIZONTAL NOT TO SCALE.

Figure 4-1. USACE logs with tunnel Sta. 519+00 to Sta. 401+00 and tunnel excavation profile



NOTES: ALL ELEVATIONS AND DEPTHS ARE APPOXIMATE. BORING LOGS FROM US ARMY CORPS OF ENGINEERS DESIGN MEMORANDUM No.3 DESIGN FOR SAN GABRIEL RIVER, WHITTIER NARROWS DAM TO CAYOTE CREEK, 1963.
BLOW COUNT "N" VALUES PER STANDARD SAMPLER.
STATIONING REFERS TO PROJECTED LOCATIONS OF BORINGS ON TO TUNNEL ALIGNMENT.
HORIZONTAL NOT TO SCALE.

Figure 4-2. USACE logs with tunnel Sta. 390+50 to Sta. 281+00 and tunnel excavation profile

4.4 Groundwater

Groundwater information along the alignment was reviewed from several different sources. These included geotechnical investigations prepared by USACE and Caltrans, Los Angeles County Department of Public Works (LACODPW) groundwater wells database, and historical high groundwater maps prepared by the California Geologic Survey (CGS). Groundwater level encountered during exploration by the USACE is shown on Figure 4-1 and Figure 4-2 above. Figure 4-3 below depicts the historical high depth to groundwater map from the Whittier Quadrangles prepared by CGS. Plotted on Figure 4-3 are the locations of LACODPW wells that were judged to be closest to the proposed alignment. Records of groundwater levels for each well were researched and the historical high (lowest depth) readings are included in Table 4-1.

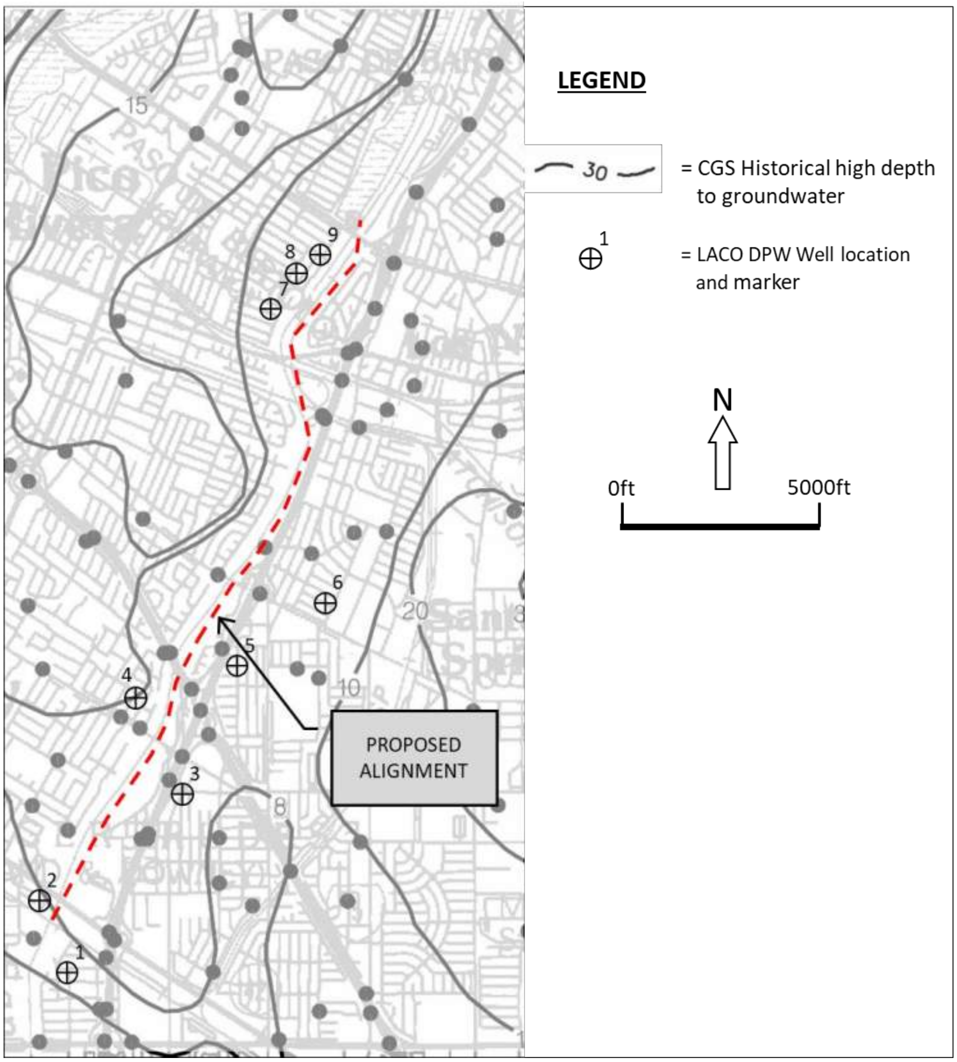


Figure 4-3. Historical high groundwater map and LACO well locations (San Gabriel River)

Table 4-1. LACO Groundwater Well Data (San Gabriel River)

Marker #	LACO Well #	Historical High Depth (ft)	Historical High Elevation (ft)	Date
1	1597Y	40.0	66.0	3/4/1949
2	1597AB	54.1	56.9	4/7/1997
3	1606U	54.0	63.5	4/1/1997
4	1596H	60.0	60.5	4/1/1997
5	1605N	21.5	105.5	11/28/1999
6	1615P	23.7	113.6	12/1/1947
7	1613V	15.3	135.7	4/9/1998
8	1612U	12.9	140.9	4/20/1995
9	1612Q	24.0	133.0	5/15/1995

The various sources of information on groundwater illustrate that groundwater levels fluctuate greatly. Groundwater levels encountered within Caltrans and USACE borings range from approximately 8 feet below the ground surface to “not encountered’ in borings up to 40 feet deep. Groundwater levels vary seasonally with the amount of water being infiltrated in upstream spreading grounds and the amount of water flowing in the San Gabriel River. The CGS historical depth shown in Figure 4-3 should be assumed for groundwater depth along the alignment. Groundwater should be assumed at a depth of 8 to 10 feet below the existing grade on either side of the levees along the San Gabriel River. This is a depth that is approximately equal to the bottom of the San Gabriel River.

4.5 Selected Trenchless/Tunneling Methods

McMillen Jacobs Associates has evaluated (1) microtunneling and pipe jacking (depending on the groundwater levels) with jacking and receiving shafts; and (2) tunneling with no shafts except at the start and end points.

Two options are considered as tunneling options for the segment of alignment along the San Gabriel river because of the constraints caused by the existing structures and natural features as well as the geologic condition along the alignment:

- **Option 2A:** Microtunneling is proposed along the San Gabriel river between Sta. 280+00 and Sta. 520+00. Seven receiving shafts and six jacking shafts are presumed for this option to facilitate the tunneling. As summarized in Sections 4.2 and 4.3, this segment is mostly consisted of soft material that behaves slow raveling to cohesive running above the groundwater table and fast raveling to flowing below the groundwater table. Because of the relatively shallow depth of water table assumed along this segment, microtunneling is recommended for this segment.
- **Option 2B:** TBM excavation with a jacking and receiving shaft, between Sta. 280+00 and Sta. 520+00, is the other proposed option for this segment. The alignment for this option is proposed along the San Gabriel cannel. This option is desirable because of the number of transportation crossings and existing features along this alignment. In addition, the use of only two shafts

represents a significant savings in terms of material costs, design costs, and impacts to the community.

4.6 Identified Risks along the Alignments

The microtunnel alignment along with the conventional TBM tunnel (Options 2A and 2B) poses multiple risks to the project overall. The following list describes the nature of various risks along the alignment.

- **Cobbles and Boulders:** Cobbles and boulders were not noted in the USACE logs, but were noted in one boring log performed for the Glen Anderson freeway (I-105) bridge crossing, south of the alignment. Cobbles and boulders are frequently encountered along many of the primary river systems that drain the Los Angeles Basin and are frequently missed during soil boring investigations.

  The implementation of a thorough geotechnical exploration will provide additional details on the extent of cobble size or larger material along either alignment. Contingency measures should be implemented to deal with the possibility of encountering cobbles and boulders along with designing a TBM or MTBM machine that can excavate this material. The amount of cobbles and boulders should be baselined in a GBR report to appropriately allocate risk between MWD and the tunnel contractor.

- **Buried Objects and Fill:** The presence of artificial fill occurs throughout both tunnel alignments. Fill is most prevalent as levee fill, embankment fill, roadway, and railroad base and will likely be present in the top 0 to 10 feet at all shaft location. All artificial fill should be considered suspect in its quality and competency unless documentation of the fill exists. In addition, debris could be present in the fill that would obstruct the advance of the tunnel and require additional effort to remove while excavating a shaft. The implementation of a geotechnical exploration program will help identify the depth, lateral extent, and competency of artificial fill and help to mitigate the risk. The presence, and frequency of obstructions and debris in the fill should also be part of a GBR that will assign the risk of encountering obstructions between the owner and contractor.
- **Utilities:** Each alignment will cross below or next to many utilities owned by government entities or associated with operations in the refinery. Detailed records research, identification and coordination with various owners will be essential. Even with due diligence, unknown utilities are likely to be encountered during construction. Risk mitigation measures for encountering utilities would include placing the alignment deeper in the profile to avoid areas of higher utility concentrations, relocating utilities prior to construction, and developing contingency measures if utilities are encountered.
- **Traffic and Public Impacts:** For Option 2A, the 14 shafts can be located off of roadways, minimizing traffic impacts. Localized portions of public recreation facilities—including Wilderness Park, Santa Fe Springs Park, and San Gabriel Mid Trail—will be impacted over a period of several weeks to several months during the construction of these portions of tunnel. Trucks delivering pipe spools and tunnel spoils between the shaft and staging areas as well as off-site supply and disposal locations may cause traffic impacts in the project area, particularly to shaft sites with limited access through narrow residential streets. Option 2B reduces the impact to

the public by reducing the number of shafts to two. Both of these shafts are located adjacent to larger thoroughfares.

- **Levees and Embankments Associated with the Rivers and Channels:** The tunnel alignments will cross below the east and west banks of the Los Angeles River, which consist of levees constructed by the USACE. Tunnel construction places the levees at risk of damage from excavation-induced ground movement, requiring strict limits on allowable settlement. In addition, the excavation of the tunnel should not undermine the hydraulic integrity of the levee by creating a preferential path for seepage and piping.
- **Railroad Crossing:** The microtunnel alignment will need to cross three different railroad lines where railroad tracks are running at the ground surface. Excessive ground surface settlement poses a safety and operational risk to passing trains. Strict control and limitations on settlement at railroad crossings are essential. The risk of settlement can be mitigated by developing operational criteria during tunnel construction, requiring continuous mining when within the railroad zone of influence, and implementing an instrumentation and monitoring program.
- **Bridge Crossings:** Option 2B, the conventional TBM tunnel, will require the machine to cross railroad bridges, highway bridges, and other bridges associated with utility and secondary road crossings. All bridges crossing the San Gabriel River are either known to be or assumed to be pile supported. Consideration will need to be given to the interaction of the tunnel with bridge foundations.

4.7 Budgetary Costs for the Trenchless/Tunnel Option

McMillen Jacobs Associates was tasked with preparing a Class 4 cost estimate and schedule for each of the options. McMillen Jacobs developed unit costs for each of the main tunnel and shaft elements and then applied them to each of the options. We utilized our proprietary in-house estimating software to prepare the unit costs.

The unit costs were prepared on a means-and-methods basis. The work was divided into discrete tasks, and for each component element of work making up the method, a takeoff was performed that quantified the amount of material required for that element in such terms as cubic yards of excavation, square feet of shoring, lineal feet of pipe, cubic yards of backfill, etc. A cycle time analysis was performed to determine the likely rate at which the task could be executed based on a specific crew size and equipment spread handling the relative amounts of each type of material required. In this fashion, the cost of performing each discrete task was tabulated in terms of labor, equipment, material, and subcontract costs. The construction costs are based primarily on production rates calculated for conditions specific to this contract. Historical production rates used are based on the estimator’s past records and experience, and modified as necessary for local geographic location and conditions.

The total costs are indicated in Table 4-2, with backup documentation in Appendix D. The summary sheet includes specific costs for each shaft and trenchless/tunnel drive.

Table 4-2. Summary of Tunneling Construction Costs for Options 2A and 2B

Option	Tunneling Method	Cost	Schedule
2A	Microtunneling	Direct and Indirect $237,161,600	60 months
		40% Contingency $94,864,600	
		Total $332,026,200	
2B	Closed Face Tunneling	Direct and Indirect $182,884,900	58 months
		40% Contingency $73,154,000	
		Total $256,038,900	

For Option 2A, drives 1 through 13 were microtunneled because all were below the groundwater table. All 14 shafts (7 jacking and 7 receiving) were watertight and constructed using sheet piles with internal bracing. The jacking shafts were 30 feet by 20 feet. The receiving shafts were 15 feet by 15 feet. Option 2A is estimated to take 60 months to complete, with two MTBMs working concurrently.

The following assumptions were used in the development of the Option 2A cost estimate:

- The fixed amount for miscellaneous costs included road decking construction and maintenance at receiving shafts, traffic control, and geotechnical instrumentation.
- Used 108-inch ID casing with 1-inch wall thickness, and 84-inch-diameter steel carrier pipe with 0.5-inch wall thickness and welded connections.
- Used three levels of bracing and 5-foot thick slab in all shafts.
- Used cellular backfill in the annular space.
- Assumed the upper 5 feet of shoring will be cut and removed, with the remaining shoring abandoned in place.
- Assumed CDF backfill of all shafts.

For Option 2B, the jacking and receiving shafts were constructed using secant piles with diameters of 40 and 24 feet, respectively. Tunnel excavation included the mobilization and use of an earth pressure balance (EPB) TBM with concrete bolted gasketed segmental lining. Option 2B is estimated to take 58 months to complete, with a single TBM used in closed mode for the entire tunnel length.

The following assumptions were used in the development of the Option 2B cost estimate:

- Both shafts included ground improvement for break in/out.
- Receiving shaft cost included dewatering.

- Tunnel cost assumed 10-inch-thick concrete segments.
- Used an 84-inch diameter steel carrier pipe with 0.5-inch wall thickness and welded connections.
- Carrier pipe installation included cellular backfill in the annular space, contact grouting, and lining repairs.
- Assumed the upper 5 feet of shoring will be cut and removed, with the remaining shoring abandoned in place.
- Assumed CDF backfill of all shafts.

Labor rates were established for each category of craft labor required using prevailing wage rates published for the Los Angeles area, and fully burdened to include payroll taxes and insurance. Appropriate allowances were made for shift differential pay and travel time pay, where called for.

Consumable materials (i.e., materials used in construction but not incorporated into the final product), permanent materials (or materials incorporated into the final product), and subcontract items were based on a combination of published data base rates for the region, and recent costs from similar projects, as the limited time allotted to prepare the costs did not allow for specific quotes for these items to be obtained from vendors.

Equipment operating rates were tabulated using algorithms established in the latest edition of the United States Army Corps of Engineers' Construction Equipment Ownership and Operating Expense Schedule for Region VII. These algorithms are based on historical records of equipment component usage and tied to specific requirements relating to the equipment model, horsepower, tire size, etc. Ownership costs for the specialized tunneling equipment will vary depending on how the project is separated out into different contract packages. Equipment depreciation is the cause of those variances.

The shaft and tunnel costs indicated are direct costs only, are calculated in 2019 dollars, and do not include mobilization/demobilization, indirect/overhead, profit, or contingency costs. Indirect/overhead and profit costs for this type of work will range between 30 and 50% of the direct costs, and are influenced by such things as: contract size and packaging, bidding climate/market conditions, and individual contractor's backlog. For this project we have assumed markups of: 5% for mobilization/demobilization; 25% for indirect costs and overhead; and 15% for profit. Contingency varies with the level of design definition, decreasing as the definition increases. At this level of design, we recommend 40% be added to the direct and indirect costs for contingency.

4.8 Recommendations if Tunnel Options Continue

The following list presents recommendations if tunneling is still considered a valid option for construction of the recycled water pipe. This list constitutes "next steps" to continue to advance the project.

- **Research and Records Requests:** Additional research is needed to supplement the documentation that was reviewed for this report. Items to research include:
  - Foundation information for railroad bridges and secondary roadway bridges.

    - USACE information on scour depth in the unlined portions of the San Gabriel River.
    - Presence of contaminated soil or groundwater along the alignment.
    - Utilities present along the chosen alignment.
- **Design Criteria:** Develop design criteria for either of the alignments. The design criteria should include:
    - Crossings beneath the Los Angeles River.
    - Railroad crossing.
    - Bridge crossing.
    - Seismic design criteria and liquefaction susceptibility.
- **Property Inventory:** Research and collect records of all property owners and ROWs along both alignments. Collect building and foundation records for all structures above and adjacent to the alignment.
- **Geotechnical Investigation:** Develop a detailed geotechnical exploration program that adequately characterizes soil and groundwater conditions along the chosen alignment. Geotechnical data should be summarized in a GDR.
- **Baseline Ground Conditions:** After completion of the GDR, the ground conditions should be baselined and included in a GBR. This report will serve to allocate risk for subsurface conditions between MWD and the tunnel subcontractor.
- **Develop an Extensive Risk Registry:** An extensive risk registry should be developed that ranks risk along the selected alignment. This registry would be a working document that is continuously updated as design and exploration proceed.
- **Building and Utility Settlement Study and Protection:** After completion of a geotechnical investigation the effects of tunneling and ground movement on adjacent structures should be evaluated. Structures deemed to be at-risk of damage from tunnel excavation should be protected by appropriate mitigation measures.

5.0 Azusa to Glendora

The Azusa to Glendora alignment will run along the upper (northern) portion of the San Gabriel River where the river exists in the San Gabriel Mountains and enters the San Gabriel Valley. Three tunnel options were considered that represent open-cut construction, a mixture of open-cut and tunnel, and an all-tunnel option. The three options are described below and are illustrated in Figure 5-1.

5.1 Option 3A – Cut-and-Cover and Tunnel Alignment

This alignment would start on land owned by the City of Pasadena and proceed up the San Gabriel River in one of two routes before entering a final segment that will terminate near Morris Reservoir and the existing Glendora Tunnel. The first route would follow Ranch Road north to the intersection of San Gabriel Canyon Road (CA Highway 39). The alignment would parallel the San Gabriel River beneath San Gabriel Canyon Road for approximately 1 mile to the intersection with Old San Gabriel Canyon Road. The alignment would turn east down the Old San Gabriel Canyon Road toward the Azusa River Wilderness Park for approximately 700 feet.

The final section of the alignment would include a second open-cut segment that would run along Old San Gabriel Canyon road approximately 1,500 feet toward the east. The alignment would then continue as a second tunnel toward the north beneath the west trending ridge spurs of Glendora Ridge. The maximum height of cover above this second tunnel is approximately 300 feet. The tunnel would exit in a portal above the existing Glendora Tunnel and below the Morris Dam.

5.2 Option 3B – Tunnel, Cut and Cover, and Tunnel Alignment

The second option would consist of a tunnel that would start at the property owned by the City of Pasadena and proceed to the northeast, continuing below the mountainous terrain of the Glendora Ridge. Maximum depth of cover above the tunnel is approximately 650 feet. The tunnel would exit the mountain in a portal located within Old San Gabriel Canyon Road. The water pipe would then follow the same path as Option 3A along Old San Gabriel Canyon Road as an open cut before entering a second tunnel and exiting in a portal above the existing Glendora Tunnel and below the Morris Dam.

5.3 Option 3C – All-Tunnel Alignment

The final option consists of constructing the alignment as one tunnel. This tunnel would start with the same alignment path as Option 3B with a portal in the City of Pasadena property. The tunnel would continue to the east and maintain cover beneath Glendora Ridge while passing to the east of the San Gabriel River. The tunnel would then join the second tunnel alignment proposed in Option 3B and daylight in a portal above the Glendora Tunnel and below Morris Dam.

5.4 Geotechnical Conditions

The proposed alignment and various options will traverse granitic rocks of the San Gabriel Mountains. The San Gabriel Mountains represent an east–west trending body of intrusive and metamorphic crystalline basement rocks contained within the Transverse Range geomorphic province (Nourse, 2002). The Transverse Ranges are characterized by east–west trending mountain ranges and sediment-filled valleys. This structure is relatively perpendicular or “transverse” to most tectonic plate movement in California, resulting in a compressional tectonic environment with rapid uplift (CGS, 2002). The San Gabriel Mountains are bounded to the south by the Sierra Madre Fault Zone and to the north by the San Andreas Fault Zone.

To understand subsurface conditions along the propose alignment, various sources of geotechnical and geological information were reviewed. These included published USGS and Dibblee foundation maps, pertinent geologic investigative work being performed by California Polytechnic University Pomona, along with engineering geology reports and information provided by MWD related to the construction of the Glendora and Monrovia Tunnels.

The principal geological unit that will be encountered during tunnel construction will be Cretaceous age quartz diorite. The diorite has been mapped and described by Dibblee et al. (1998, 1999) as being a medium grained quartz diorite composed of plagioclase feldspar, biotite, potassium feldspar, quartz, and hornblende. Occasionally, thin dikes of granite, dacite, andesite, and basalt cut the diorite. No major structural faults have been mapped along the proposed alignments by any geologic investigations. Figure 5-1 shows both the surficial and bedrock units found along the proposed tunnel alignments.

Engineering characteristics of the quartz diorite were best determined from geologic notes and sections prepared for the Glendora Tunnel and from information gained from geotechnical reports for the San Gabriel tower located above the Glendora Tunnel and south of the Morris Dam. Recent geotechnical investigation revealed an intact rock strength (through Unconfined Compressive Strength [UCS] Testing) of between 16,000 psi and 2,250 psi. Rock Quality Designation (RQD) percentages between 76% in more fresh, untethered zones and 0% in more highly fractured or weather zones were recorded (GeoPentech, 2012). Records for the Glendora Tunnel indicate that a powder factor of approximately 19 was used to drill and blast the quartz diorite. Rock conditions were characterized during tunnel excavation using the Tunnelman’s ground classification system (Heuer 1974, after Terzaghi 1950). Quartz diorite in the vicinity of the northern end of the alignment was described as “massive, moderately jointed, very firm ground” and “moderately block and seamy (firm ground). Figure 5-2 shows a segment of the Glendora Tunnel construction records in the vicinity where the proposed tunnel would cross.

In addition to quartz diorite, other surficial deposits will be encountered during tunnel construction. These include weathered diorite, artificial fill, alluvium, terrace deposits, and landslide debris. The exact thickness of each unit along the alignment, and specifically along the open-cut segment, is not known. The depth of the bedrock to soil contact is unknown along the open-cut segment. The following description provides an idealized idea of each unit.

- **Alluvium:** Alluvium will consist largely of loose to medium dense, unconsolidated deposits of sand and gravel with varying quantities of cobbles and boulders. These soils are recent deposits of the San Gabriel River and tributaries. Alluvium can be expected beneath artificial fill at an unknown depth within the open-cut portions of the alignment.
- **Terrace:** Terrace deposits are expected to consist of silt and sand deposits with varying quantities of cobbles and boulders. These deposits formed as the San Gabriel River downcut and eroded the quartz diorite and alluvial deposits, leaving alluvial terraces at higher elevations.

- **Landslide:** Talus and poorly consolidated soil with angular clasts and varying quantities of silt and sand. Localized debris flows are also possible.
- **Artificial fill:** Artificial fill will be expected along San Gabriel Canyon Road (CA-39) and Old San Gabriel Canyon Road. Fill materials will include a mixture of all the above ground types in varying consistencies and compositions.

5.5 Groundwater

The historical data available for the existing Glendora Tunnel demonstrate some seepage along the alignment of the tunnel through the rock material. However, only minor seepage is shown at the segment of the tunnel that is in the vicinity of where the proposed alignment would cross. The geology at this segment of the Glendora Tunnel, which consists of quartz diorite, verifies this observation. Other segments of the Glendora Tunnel completed within quartz diorite encountered minor seeps on the order of 2 gallons per minute.

A cut-and-cover trench along the Highway 39 and Old San Gabriel Canyon Road is likely to encounter groundwater at varying depths. No groundwater records were available for review that would provide information on the depth or elevation of the groundwater table. Groundwater elevation is likely controlled by the level of water in the San Gabriel River and likely fluctuates with the seasons and elevation of the river. Likewise, it cannot be said without a more detailed geotechnical investigation if a cut-and-cover trench would require dewatering to construct the new MWD recycled water pipeline.

5.6 Selected Tunneling/Cut and Cover Methods

Three tunneling options are considered for the segment of alignment from Azusa to Glendora, because of the constraints caused by natural features as well as the geologic condition along the alignment. The first two options are a combination of cut-and-cover method and tunneling through bedrock by TBM or drill-and-blast methods; whereas, the third option is tunneling through the bedrock all along the alignment. The following summarizes the options:

- **Option 3A:** The pipe is designed to be placed along the San Gabriel River using the cut-and-cover method from Sta. 1+00 to Sta. 96+00. The pipe is then being carried by a tunnel advanced through bedrock, using TBM or drill-and-blast from Sta. 96+00 to Sta. 140+00.
- **Option 3B:** The cut-and-cover method is used from Sta. 22+00 to Sta. 25+00 and Sta. 76+00 to Sta. 96+00 along the San Gabriel River. Tunnel advancement through the bedrock is proposed from Sta. 25+00 to 76+00 and Sta. 96+00 to Sta. 140+00.
- **Option 3C:** The tunnel will be advanced through the bedrock, using a TBM from Sta. 24+00 to Sta. 140+00.

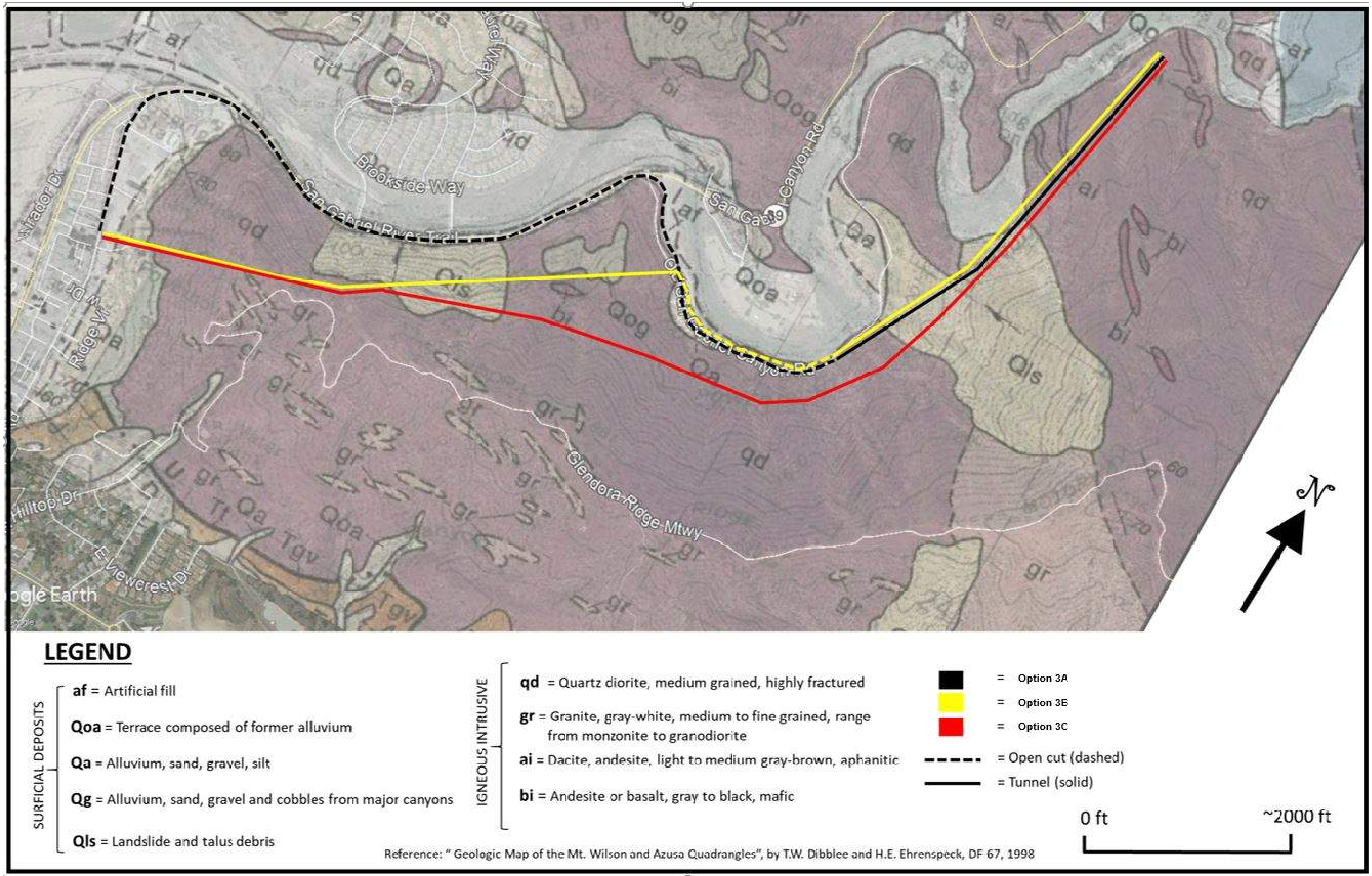


Figure 5-1. Geologic map Azusa-Glendora and proposed alignments, overlay on satellite photo

This page intentionally left blank.

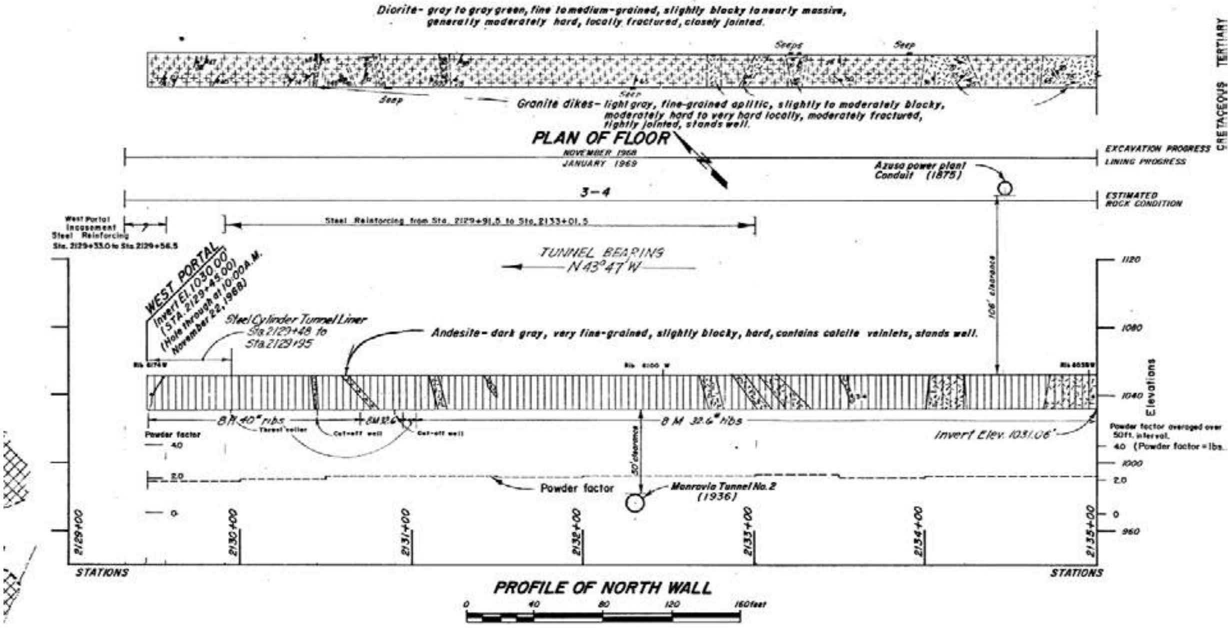


Figure 5-2. Section of Glendora Tunnel Near alignment documenting rock conditions

5.7 Traffic and Public Impacts

Traffic and public impacts vary between the three options, and the most significant traffic impacts are associated with the cut-and-cover portions of the options being evaluated.

The southern end of the alignment for all options begins at City of Pasadena land off of Ranch Road. The open land at this location would be a proposed staging area for any option and would serve as the primary tunnel staging and portal location for Options 3B and 3C. Ranch Road is a narrow, two-lane street with residential development on the west side of the road and city property on the east side. Truck deliveries to and from a tunnel portal at this location will need to be routed directly from Ranch Road to San Gabriel Canyon Road to minimize traffic through the adjacent residential area.

For Option 3A, which includes cut-and-cover pipeline installation along Ranch Road, segments of Ranch Road would need to be reduced to a single lane of traffic during construction. Likely segment lengths for closures would be 30 feet to 70 feet to allow room for equipment and trenching, with presumed material and equipment storage at the nearby staging area. The single-lane traffic control would include flaggers to actively control traffic flow past the work zone. The open trenches would be required to be covered with steel plates during nonworking hours to protect the public and allow for two-way traffic. Given that the adjacent residences have access from the west side, Ranch Road could potentially be fully closed to the public during the course of the work, which would improve public safety and likely decrease the overall length of time and cost of this section of work. City access to the City of Pasadena water treatment plant off of Ranch Road would need to be coordinated with the contractor during this period.

Public impacts along San Gabriel Canyon Road are primarily associated Option 3A, the cut-and-cover option. San Gabriel Canyon Road is also a portion of State Route 39, which runs between Huntington Beach to the west and into the San Gabriel Mountains to the west. Along the Option 3A alignment, San

Gabriel Canyon Road is primarily a two-lane road. The San Gabriel River pedestrian trail parallels the roadway along the north side between the road and the San Gabriel River. The south side of the roadway is primarily bounded by a k-rail and chain-link barrier immediately adjacent to the roadway, which would likely prevent loose rock from the adjacent hillsides from entering the roadway. Mountain Cove, a residential development, is accessed along the portion of San Gabriel Canyon Road in the project area for Option 3A. The two sole access points to this development of several dozen houses are located in this area.

For Option 3A, the pipeline could either be located under the pedestrian trail or within the roadway. Locating the pipe under the trail would require a complete trail closure but would limit traffic impacts. Truck traffic to and from the work sites would still require limited traffic control by flaggers to allow trucks to safely return to the roadway. Alternatively, if the pipe is constructed within the roadway, portions of road would need to be restricted to one-way traffic with flagger control daily. Work areas would need to take up approximately 100 feet by one lane because of the restricted area and distance from larger staging area. The open trenches would be required to be covered with steel plates during nonworking hours to protect the public and allow for two-way traffic. Trenching into the rock subgrade at this location would result in slower production rates and longer work durations than typical cut-and-cover construction.

Public impacts to San Gabriel Canyon Road associated with Options 3B and 3C are limited to increased truck traffic to deliver materials to and from work sites at Old San Gabriel Canyon Road as well as the portal and tie-in site near Morris Reservoir Dam.

Options 3A and B include cut-and-cover along portions of Old San Gabriel Canyon Road and development of one portal site along this road for Option 3A and two portal sites for Option 3B. The first 1,200 feet of the roadway consist of a narrow road that serves as access to the Azusa River Wilderness Park and abut the parking lot and ranger station. Beyond the ranger station, the road is gated off and serves as a pedestrian trail. Cut-and-cover construction along this road would require reduction to a single, narrow lane of traffic with control by flaggers. Access to the portal site at Sta. 96+00 would require temporary closure of the pedestrian trail at this location. There is the potential to limit weekend work and secure these sites to reduce impacts to weekend recreators, as well as closing the park entirely during the weekdays to maximize production and minimize public risk.

5.8 Identified Risks Along the Alignments

Options 3A, 3B, and 3C listed above present certain risks to the project. The most pronounced risks are listed and described below.

- **Groundwater Elevation and Seepage:** The groundwater elevation is not known along the open cut sections described in Options 3A and Option 3B. Dewatering may be required if groundwater is present in the bottom of the trench. In addition, groundwater elevation is not known within Glendora Ridge and the rest of the mountain ridges where rock tunneling is considered. While tunnel records for the Glendora Tunnel indicate minimal seepage and inflows of groundwater, the risk of encountering fracture zones and abundant seepage is possible. Long-term monitoring of groundwater elevation as part of a geotechnical exploration program will provide more information on the groundwater elevation. Contingency measures can be created for encountering groundwater while rock tunneling, and a dewatering program can be designed for removing groundwater if the cut and cover options are chosen.

- **Groundwater Depletion:** While tunneling through rock the risk of encountering fracture zones and high seepage is described above. Associated with this risk is the risk of temporarily lowering the static groundwater level if large inflows of groundwater are not stopped. Contingency measures should be developed to quickly arrest the inflow of groundwater to the tunnel
- **Variable Bedrock Soil Contact:** Variability of the soil rock contact presents a substantial risk to the open-cut segments presented in Options 3A and 3B. Within the steep hillsides, bedrock is either obscured by A thin layer of soil cover or exposed. Exceptions to this are in areas where terrace or landslides cover the mountain slopes. As the mountain slopes descend and reach San Gabriel Canyon, no information is available on how the bedrock continues beneath the alluvial soils of the river, and no information could be reviewed that describes the profile as the rock to soil contact passes beneath the river and reemerges on the opposing mountain side. The bedrock profile may be relatively shallow (U shaped), deep and incised (V shaped), or transition from one to the other along the river. Given the unknown elevation of this contact, it cannot be determined how much of the of the open-cut will need to excavate rock. Implementation of a geotechnical exploration program that incorporates the use of soil borings, test trench, and geophysical techniques will help to remove uncertainty around this risk.
- **Rippability of Rock:** If encountered in the cut-and-cover portions of Options 3A and 3B, the competency of the rock is expected to vary. Likewise, the amount of effort and excavation techniques required by the contractor is likely to vary. Using excavation strength described by Pettifer and Fookes (1994), rock may range from “hard digging” to “hard ripping” and in the most extreme case require blasting. The use of test trenches, geologic mapping using the Global System Integrator (GSI_ or Rock Mass Rating (RMR) systems, and geophysical methods will help to remove uncertainty around this risk.
- **Rock Quality and Competency during Tunneling:** Rock quality and competency in the quartz diorite may vary from what was encountered in the Glendora Tunnel and in other nearby geotechnical investigation.
- **Unknown Structural Features:** While not likely given the nature of the geologic mapping already performed along this segment of San Gabriel Canyon, it is possible that additional structural features have been missed. These features would include faults, shear zones, or additional rock types. Encountering unknown faults or shear zones may present a hazard to the excavation crew, and stop or slow excavation progress. In addition, unanticipated rock types that are stronger and more abrasive when excavated will cause additional costs to the contractor.
- **Traffic and Public Impacts:** As described in Section 5.7, the open-cut portions of Options 3A and 3B are likely to result in the most impact to traffic and public recreation along the alignments, including potential temporary closure of portions of the San Gabriel River Trail and Azusa River Wilderness Park. Additionally, reduction of traffic from two-way to one-way with flagger control may be needed for open-cut portions of the alignment, including San Gabriel Canyon Road/State Route 37, which provides the only access to the Mountain Cove subdivision and into portions of the Angeles National Forest. Potential impacts associated with the tunneled portions of the

alignments include increased trucking activity to and from portal sites and a potential trail closure for portal access within the Azusa River Wilderness Park.

5.9 Budgetary Costs for the Tunnel/Cut-and-Cover Option

McMillen Jacobs Associates was tasked with preparing a Class 4 cost estimate and schedule for each of the options. McMillen Jacobs developed unit costs for each of the main tunnel and shaft elements and then applied them to each of the options. We utilized our proprietary in-house estimating software to prepare the unit costs.

The unit costs were prepared on a means-and-methods basis. The work was divided into discrete tasks, and for each component element of work making up the method, a takeoff was performed that quantified the amount of material required for that element in such terms as cubic yards of excavation, square feet of shoring, lineal feet of pipe, cubic yards of backfill, etc. A cycle time analysis was performed to determine the likely rate at which the task could be executed based on a specific crew size and equipment spread handling the relative amounts of each type of material required. In this fashion, the cost of performing each discrete task was tabulated in terms of labor, equipment, material, and subcontract costs. The construction costs are based primarily on production rates calculated for conditions specific to this contract. Historical production rates used are based on the estimator’s past records and experience, and modified as necessary for local geographic location and conditions.

The total costs are indicated in Table 5-1, with backup documentation in Appendix D. The summary sheet includes specific costs for each cut-and-cover section and tunnel drive.

Option 3A includes 9,500 feet of cut-and-cover work and 4,400 feet of tunnel excavated by drill-and-blast or TBM. The tunnel will include two portals and no shafts. Option 3A is estimated to take 20 months to complete, with the cut-and-cover and tunnel working concurrently.

The following assumptions were used in the development of the Option 3A cost estimate:

- Assumed the average excavation depth of 16 feet deep and 10’ wide for the cut-and-cover work.
- Assumed support of excavation will be done using beam and lagging shoring.
- Assumed the upper 5 feet of shoring will be cut and removed, with the remaining shoring abandoned in place.
- Tunnel costs based on an excavated diameter of 9 feet.
- Tunnel excavation cost includes installation and removal of geotechnical instrumentation.
- Used 84-inch-diameter steel carrier pipe with 0.5-inch wall thickness and welded connections.
- Carrier pipe installation in tunnel includes cellular backfill in the annular space, contact grouting, and lining repair.
- Carrier pipe installation in the cut-and-cover section included CDF backfill to the top of pipe, backfill, and compaction above.

Option 3B includes 5,100 feet of tunnel excavated by drill-and-blast or TBM, 2,000 feet of cut-and-cover work, and 4,400 feet of tunnel excavated by drill-and-blast or TBM. The two tunnels will include four portals and no shafts. Option 3B is estimated to take 21 months to complete, with the cut-and-cover and two tunnels working concurrently.

Table 5-1. Summary for Tunneling Construction Costs for Options 3A, 3B, and 3C

Option	Tunneling Method	Cost	Schedule
3A	Cut-and-Cover and TBM Tunnel	Direct and Indirect $72,398,400	20 months
		40% Contingency $28,960,000	
		Total $101,358,400	
3B	TBM Tunnel, Cut-and-Cover Section, TBM Tunnel	Direct and Indirect $53,804,300	21 months
		40% Contingency $21,522,000	
		Total $75,326,300	
3C	All TBM Tunnel	Direct and Indirect $45,473,700	27 months
		40% Contingency $18,190,000	
		Total $63,663,700	

The following assumptions were used in the development of the Option 3B cost estimate:

- Assumed the average excavation depth of 16 feet deep and 10’ wide for the cut-and-cover work.
- Assumed support of excavation will be done using beam and lagging shoring.
- Assumed the upper 5 feet of shoring will be cut and removed, with the remaining shoring abandoned in place.
- Tunnel costs based on an excavated diameter of 9 feet.
- Tunnel excavation cost includes installation and removal of geotechnical instrumentation.
- Used 84-inch-diameter steel carrier pipe with 0.5-inch wall thickness and welded connections.
- Carrier pipe installation in tunnel included cellular backfill in the annular space, contact grouting, and lining repair.

- Carrier pipe installation in the cut-and-cover section included CDF backfill to the top of pipe, backfill, and compaction above.

Option 3C includes 11,600 feet of tunnel excavated by drill-and-blast or TBM. The tunnel will include two portals and no shafts. Option 3C is estimated to take 27 months to complete, with all tunnel work occurring from the Azusa portal.

The following assumptions were used in the development of the Option 3C cost estimate:

- Used 84-inch-diameter steel carrier pipe with 0.5-inch wall thickness and welded connections.
- Tunnel costs based on an excavated diameter of 9 feet.
- Tunnel excavation cost includes installation and removal of geotechnical instrumentation.
- Carrier pipe installation in tunnel included cellular backfill in the annular space, contact grouting, and lining repair.

Labor rates were established for each category of craft labor required using prevailing wage rates published for the Los Angeles area, and fully burdened to include payroll taxes and insurance. Appropriate allowances were made for shift differential pay and travel time pay, where called for.

Consumable materials (i.e., materials used in construction but not incorporated into the final product), permanent materials (or materials incorporated into the final product), and subcontract items were based on a combination of published data base rates for the region, and recent costs from similar projects, as the limited time allotted to prepare the costs did not allow for specific quotes for these items to be obtained from vendors.

Equipment operating rates were tabulated using algorithms established in the latest edition of the United States Army Corps of Engineers' Construction Equipment Ownership and Operating Expense Schedule for Region VII. These algorithms are based on historical records of equipment component usage and tied to specific requirements relating to the equipment model, horsepower, tire size, etc. Ownership costs for the specialized tunneling equipment will vary depending on how the project is separated out into different contract packages. Equipment depreciation is the cause of those variances.

The shaft and tunnel costs indicated are direct costs only, are calculated in 2019 dollars, and do not include mobilization/demobilization, indirect/overhead, profit, or contingency costs. Indirect/overhead and profit costs for this type of work will range between 30 and 50% of the direct costs, and are influenced by such things as: contract size and packaging, bidding climate/market conditions, and individual contractor's backlog. For this project we have assumed mark-ups of: 5% for mobilization/demobilization; 25% for indirect costs and overhead; and 15% for profit. Contingency varies with the level of design definition, decreasing as the definition increases. At this level of design, we recommend 40% be added to the direct and indirect costs for contingency.

5.10 Recommendations if Tunnel Options Continue

The following list presents recommendations if tunnel construction is still considered a valid option for construction of the recycled water pipe. This list constitutes “next steps” to continue to advance the project.

- **Research and Records Requests:** Additional research is needed to supplement the documentation that was reviewed for this report. Items to research include:
  - Geotechnical information for Morris Dam given its proximity to the Tunnel
  - River flood levels, and regular operational levels throughout the year
  - Presence of contaminated soil or groundwater along the alignment.
  - Utilities present along the chosen alignment.
  - United States Forest Service requirements for tunneling adjacent to federal land, specifically related to groundwater.
- **Design Criteria:** Develop design criteria for either of the alignments. The design criteria should include:
  - Rock loading
  - Groundwater loading
  - Seismic design criteria
- **Property Inventory:** Research and collect records of all property owners, and right-of-way along both alignments. Collect building and foundation records for all structures above and adjacent to the alignment.
- **Geotechnical Investigation:** Develop a detailed geotechnical exploration program that adequately characterizes soil and groundwater conditions along the chosen alignment. Geotechnical data should be summarized in a GDR. Open-cut segments should include a variety of exploration techniques including soil borings, test trenches, and geophysical techniques.
- **Baseline Ground Conditions:** After completion of the GDR, the ground conditions should be baselined and included in a GBR. This report will serve to allocate risk for subsurface conditions between the owner (MWD) and tunnel subcontractor.
- **Develop an Extensive Risk Registry:** An extensive risk registry should be developed that ranks risk along the selected alignment. This registry would be a working document that is continuously updated as design and exploration proceed.
- **Groundwater Impact and Dewatering Report:** This report should review all available exploration information contained in the GDR and consider tunnel construction impacts on the groundwater table. If open cut segments are selected this report should describe the need for dewatering and make specific recommendations to the dewatering subcontractor.

6.0 Conclusions

6.1 Summary of the Three Tunnel Alignments

MWD engaged McMillen Jacobs Associates for a high-level review of three alignments proposed as fully tunneled options for the Regional Recycled Water Program. The tunneling options are being considered in case the cut-and-cover construction is deemed not possible. The following three pipeline segments with associated options were evaluated in this study:

1. Carson to Long Beach:
   - Option 1A: Pipe jacking/microtunneling about 4.6 miles (Sta. 0+00 to Sta. 240+00) starting on South Main Street heading north and turning east following East Sepulveda Boulevard and West Willow Street to the east side of the Los Angeles River; crossing under the Dominquez Channel, I-710, the Los Angeles River, and other major roads.
   - Option 1B: Tunneling about 4.6 miles (Sta. 17+00 to Sta. 240+00) starting at the treatment plant on South Main Street and heading east below an existing railroad spur line. After crossing beneath Avalon Boulevard and Wilmington Avenue, the alignment crosses various industrial properties, a second railroad track, the Dominquez Channel (where it aligns on West Willow Street), and ends with the crossing of I-710 and the Los Angeles River.
2. San Gabriel River:
   - Option 2A: Microtunneling about 4.6 miles (Sta. 278+00 to Sta. 519+00) starting at Imperial Highway and following the San Gabriel River north along the Southern Edison right-of-way to Pico Rivera; paralleling the I-605; and crossing Highway 42, I-5, and other major roads.
   - Option 2B: Tunneling about 4.6 miles (Sta. 278+00 to Sta. 519+00) starting at Imperial Highway and following the San Gabriel River north along the Southern Edison right-of-way to Pico Rivera; paralleling the I-605; and crossing Highway 42, I-5, and other major roads.
3. Azusa to Glendora: Starting on Highway 39 adjacent to the City of Azusa Filtration Plant and trending along the San Gabriel River to the east and north to a point short of Morris Reservoir, where the new tunnel will tie into the existing Glendora Tunnel. Three options were included in the study:
   - Option 3A: Utilizing an initial cut and cover section (1.8 miles long) followed with a TBM tunnel (0.8 mile long) to the Glendora connection, with a total length of 2.6 miles (Sta. 1+00 to Sta. 139+00).
   - Option 3B: Utilizing an initial TBM tunnel (1.0 mile long) with a middle cut-and-cover section along Oxbow Park (0.4 mile long) and then a TBM tunnel (0.8 mile long) to the Glendora connection, with a total length of 2.2 miles (Sta. 24+00 to Sta. 139+00).
   - Option 3C: Utilizing a full-length TBM driven tunnel to the Glendora connection with a total length of 2.2 miles (Sta. 22+00 to Sta. 139+00).

6.2 Summary of Construction and Costs for All Three Tunnel Alignments

Class 4 cost estimates for each of the three alignments and options were developed based upon the plans and profiles provided in Appendix C. The cost and schedule for each option are summarized in Table 6-1. The cost estimate back-up materials are provided in Appendix D.

For the Carson to Long Beach alignment, the preferred option is Option 1B, which uses a TBM tunnel for the entire length from the Carson water treatment plant to the Los Angeles River. The estimated construction costs for Option 1B are $235,712,200 which includes a 40% contingency. Option 1B will take 55 months to construct. Option 1B costs about $76,000,000 less than Option 1A, the pipe jacking/microtunneling option. The TBM tunnel can be constructed 9 months faster than Option 1A.

For the San Gabriel River alignment, the preferred option is Option 2B, which uses a TBM tunnel for the entire length from the spreading grounds in Pico Rivera to the Imperial Highway. The estimated construction costs for Option 2B are $256,038,900, which includes a 40% contingency. Option 2B will take 58 months to construct. Option 2B costs about $76,000,000 less than Option 1A, the pipe jacking/microtunneling option.

For the Azusa to Glendora Tunnel alignment, Option 3C, the all-tunnel alternative, is the lowest cost of the three options at $63,663,700. The range of cost between the three options is about $37,000,000. Since much of the cut-and-cover work will be difficult with the large boulder field along the San Gabriel River, Option 3C, is recommended. For the option 3 tunnels, construction costs were looked at using drill-and-blast and a rock TBM. In all three options, the TBM driven tunnels were less cost and take less time to construct than tunnels excavated using drill-and-blast methods

For the option 3 tunnels, construction costs were looked at using drill-and-blast and a rock TBM. In all three options, the TBM driven tunnels were less than tunnels excavated using drill-and-blast methods. Estimated constructions costs for the drill-and-blast and TBM driven tunnels are included in Appendix D.

Table 6-1. Budgetary Cost Estimate Summary

Alignment	Options	Tunneling Options	Duration (Months)	Budgetary Costs		
				Direct and Indirect Costs	40% Contingency	Total Costs
Carson to Long Beach	Option 1A	Trenchless with 13 shafts	64	$222,736,800	$89,094,700	$311,831,500
	Option 1B	Tunnel with 2 shafts	55	$168,365,200	$67,347,000	$235,712,200
San Gabriel River	Option 2A	Trenchless with 14 shafts	60	$237,161,600	$94,864,600	$332,026,200
	Option 2B	Tunnel with 2 shafts	58	$182,884,900	$73,154,000	$256,038,900
Azusa to Glendora	Option 3A	Combo Cut-and-Cover and One Rock Tunnel with 2 portals	20	$72,398,400	$28,960,000	$101,358,400
	Option 3B	Combo Cut-and-Cover and Two Rock Tunnels with 4 portals	21	$$53,804,300	$21,522,000	$75,326,300
	Option 3C	All Rock Tunnel with 2 portals	27	$45,473,700	$18,190,000	$63,663,700

7.0 References

AACE International Recommended Practice No. 18R-97. 2016. Cost Estimate Classification System- As Applied in Engineering, Procurement, and Construction for the Process Industries.

ASCE Standard 36. 2015. Standard Design and Construction Guidelines for Microtunneling.

AMEC Geomatrix, Inc., 2001. Geotechnical Investigation Report, Carson 7 Tank Project – Phase II, Kinder-Morgan Carson Terminal, Carson, California, Project No. NB10160160, dated January 5.

Bilodeau, W.L, Bilodeau, S.W., Gath, E.M., Oborne, M., and Proctor, R.J. 2007. Geology of Los Angeles, California, United States of America. *Environmental & Engineering Geoscience*, XIII(2): May, 99–160.

Black & Veatch and CDM Smith. 2016. Potential Regional Recycled Water Program Feasibility Study, Report No. 1530, dated November 30.

Black & Veatch and CDM Smith. 2017. Potential Regional Recycled Water Program, Conveyance/Distribution System Conceptual Draft Report, dated November 15.

California Department of Conservation Division of Mines and Geology. 1998. Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California, Report No. 028., Revised 1/13/2006.

California Department of Conservation Division of Mines and Geology. 1998. Seismic Hazard Zone Report for the Torrance 7.5-Minute Quadrangle, Los Angeles County, California, Report No. 035., Revised 1/17/2006.

California Department of Conservation Division of Mines and Geology. 1998. Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangle, Los Angeles County, California, Report No. 037., Revised 1/17/2006.

California Geological Survey. 2002. California Geomorphic Provinces, Note 36, dated December 12.

Contech Engineered Solutions. November 2015. Contech Tunnel Liner Plate (brochure).

Dibblee, T.W. and Ehrenspeck, H.E. 1998. Geologic Map of the Mt. Wilson and Azusa Quadrangles, DF-67.

Dibblee, T.W., Ehrenspeck, H.E., Ehlig, P.L., and Bartlett, W.L., 1999. Geologic Map of the Palos Verdes Peninsula and vicinity, Redondo Beach, Torrance, and San Pedro quadrangles, Los Angeles County, California, DF-70.

GeoPentech. 2012. San Gabriel Tower Geologic and Ground Motions Assessment, dated December 21.

GeoPentech, 2017. Preliminary Geotechnical/Geologic Evaluation, Proposed Regional Recycled Water Supply Program, Metropolitan Water District of Southern California, dated November 13.

Globus Engineering Inc. 2012. Report Geotechnical Investigation, Proposed Services Building Project, Carson, California, for Conocophillips, Job No. 1087, dated April 10.

Heuer, R.E. 1974. Important ground parameters in soft ground tunneling. In *Proceedings of Specialty Conference on Subsurface Exploration for Underground Excavation and Heavy Construction*. 41–55. Henniker, NH: ASCE.

Jenkins, Olaf P., and Jennings, Charles W. 1962. Geologic Map of California, Long Beach Sheet, Published by State of California Division of Mines and Geology.

The Metropolitan Water District of Southern California. 1969. Glendora Tunnel, North Portal Area, STA.2129+45 to 2135+00, Geology and As-Built, Sheet B-20271-18.

National Utility Contractors Association (NUCA). 2004. Trenchless Construction and Rehabilitation Methods.

Nourse, J.A. 2002. Middle Miocene reconstruction of the central and eastern San Gabriel Mountains, southern California, with implications for evolution of the San Gabriel fault and Los Angeles basin, Geological Society of America, Special Paper 365.

The Pipe Jacking Association (PJA). 1995. *An Introduction to Pipe Jacking and Microtunneling Design*.

Pettifer, G.S., and Fookes, P.G. 1994. A revision of the graphical method for assessing the excavatability of rock. *Quarterly Journal of Engineering Geology and Hydrogeology*, 27: 145–164.

Proctor, R.V., and White, T.L. 1977. *Earth Tunneling with Steel Supports*. Youngstown, OH: Commercial Shearing.

Stein, D. 2005. *Trenchless Technology for Installation of Cables and Pipelines*. Bochum, Germany: Stein & Partner GmbH.

Terzaghi, K. 1950. Geologic aspects of soft ground tunneling, Chapter 11. *Applied Sedimentation*, P.D. Trask (Ed.). New York: John Wiley & Sons.

U.S. Army Engineer District, Los Angeles Corps of Engineers. 1963. Design Memorandum No.3 General Design for San Gabriel River, Whittier Narrows Dam to Coyote Creek, File No. SPLGD-H, dated June 3.

United States Geologic Survey. 1902. Topographic Map of Downey Quadrangle.

United States Geologic Survey. 1924. Topographic Map Compton Quadrangle.

United States Geologic Survey. 1965. Geology of the Los Angeles Basin California-an Introduction, Geology of the Eastern Los Angeles Basin Southern California, by R.F. Yerkes, T.H. McCulloh, J.E. Schoellhamer, and J.G. Vedder, Professional Paper 420-A.

URS. 2015. Report Geotechnical Investigation, Proposed Los Angeles Refinery Integration and Compliance (LARIC) project, Project areas Located Within the City of Carson Limits, Tesoro Los Angeles Refinery, 2101 East Pacific Coast Highway, Wilmington, California, URS Project No. 29407437, dated January 20.

Western Laboratories. 1987. Soils Investigation – Proposed Industrial Warehouse Building No. 184 – Located at the Southeasterly Corner of Wilmington Avenue and 236th Street in the City of Carson, California, Work order 9632, dated April 27.

This page intentionally left blank.

Appendix A Trenchless/Tunnel Construction Methods

This page intentionally left blank.

A.1 Introduction

Appendix A provides general background information on the trenchless/tunnel methods assumed to be used for the new pipeline construction and their potential applicability for the ground conditions. Detailed construction recommendations for the individual pipeline segments and options are provided in Sections 3.0, 4.0, and 5.0.

A number of factors must be considered in evaluating trenchless/tunnel methods for the pipeline construction. These factors include:

- Inside and outside diameter of carrier pipe and total length of installation
- Type of carrier pipe to be installed and any casing and initial support requirements
- Anticipated subsurface conditions along the alignment
  - Type of ground expected
  - Presence of cobbles, boulders, and debris
  - Groundwater presence
- Dewatering and discharge requirements
- Anticipated ground behavior
- Excavated bore stability (i.e., ability to maintain open annulus)
- Line and grade control requirements
- Alignment accessibility and provisions/contingencies for an installed drive
- Social and traffic disruption/impacts

For this study, the final recycled pipelines will have an internal diameter of 84 inches. The carrier pipe will be a steel pipe with an internal lining and external coating. The carrier pipe will be housed in either a 108-inch ID casing (for pipe jacking or microtunneling) (see Figure A-1) or a 108-inch ID segmental concrete lining (for tunneling) (see Figure A-2). The segmental lining will be the same size whether the assembled lining will be ungasketed for tunneling above the groundwater table (junk segments) or gasketed and bolted for tunneling below the groundwater table.

When these and other factors are considered, the following trenchless/tunnel construction methods are considered the most appropriate for one or more of the alignments discussed in more detail in the sections that follow.

- Pipe jacking
- Conventional shield tunneling
- Microtunneling
- Drill-and-blast tunneling
- TBM driven tunneling

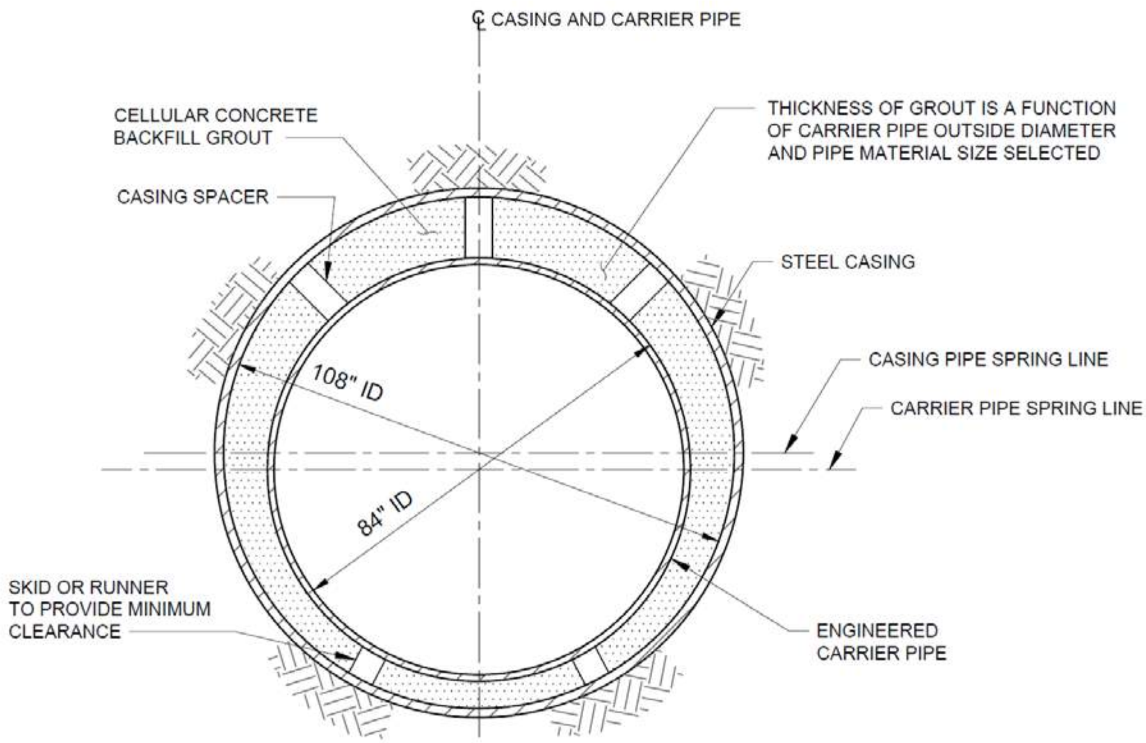


Figure A-1. Typical casing and carrier pipe configuration for pipe jacking and microtunneling

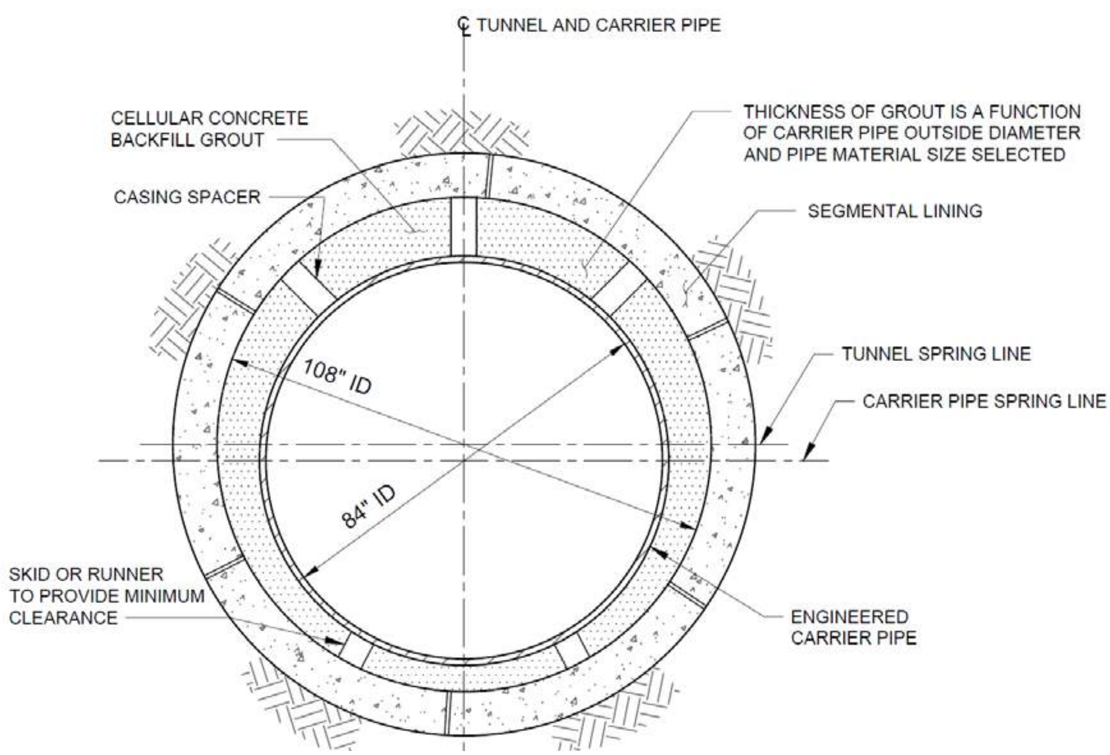


Figure A-2. Typical initial tunnel lining and carrier pipe configuration for tunneling

These trenchless methods are capable of a one- or two-pass installation to install the carrier pipe in the ground. One-pass methods install the carrier pipe directly, whereas two-pass methods first install a casing or initial lining for ground support followed by installation of the carrier pipe. Since the carrier pipe is a steel pipe with lining and coating requirements, the two-pass method will be used in all tunneling options in this study.

A.2 Pipe Jacking

Pipe jacking is a trenchless method of installing pipelines using a shield for hand mining or using mechanical means to excavate ground above the groundwater table (Figure A-3). Casing (two-pass) or specially designed pipe (one-pass) is pushed into the ground by hydraulic jacks at the back of the pipe string while excavation is taking place within the shield. The machine operator and other personnel perform the work at the tunnel heading and inside the pipe string. The operator observes ground conditions, helps determine the rate of excavation and jacking, and monitors line and grade. Spoils are transported from the face to the jacking shaft using conveyer belts, haul carts, or small locomotives and haul cars. Face access is achieved with pipe sizes of 60-inch ID or greater, given the equipment setup inside the pipe string, but face headings can be accessed through pipe diameters as small as 30 inches.

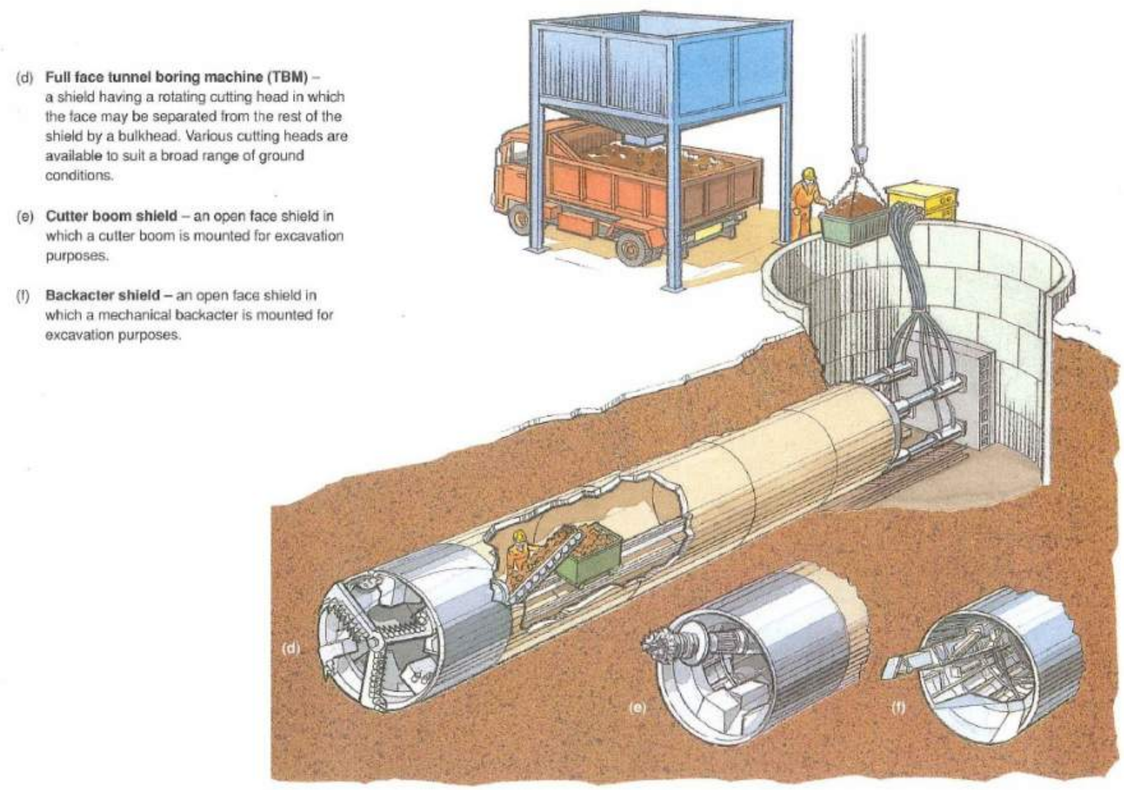


Figure A-3. Typical pipe jacking operation (PJA, 1995)

Key features that are incorporated into a pipe jacking operation include:

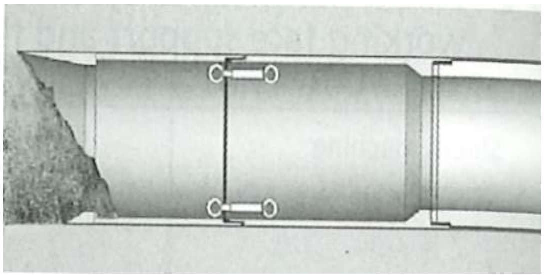
- Manned entry installation with personnel working at the face under protection of an open shield machine.
- Cyclical advancement of pipe segments installed within the jacking shaft with the aid of a main jacking station in the jacking shaft and, if required, additional intermediate jacking stations.

- Spoils removal using haul carts, conveyor belts, or small locomotives housed in casings running along the invert of the jacking pipe.
- Inclusion of different types of shields that may provide a guidance system for steering adjustments and partial or full-face mechanical support for the excavation.

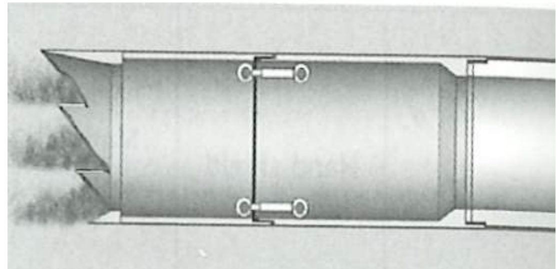
Pipe jacking allows for relatively accurate installation and control of line and grade with the use of a laser guidance system and steerable shield. Adequate space will be required around the jacking shafts for staging equipment and operations. The required size of the jacking and receiving shafts is related to the size (diameter and length) of the selected pipe or casing segments. A typical pipe length used with pipe jacking is 10 feet. The type of pipe used with pipe jacking must be capable of transmitting the required jacking forces from a thrust plate behind the hydraulic jacks to the open shield machine at the front of the pipe string. Pipe jacking is commonly used for drive lengths of 1,000 feet. Longer drives can be achieved by incorporating a lubrication program and additional intermediate jacking stations (see Figure A-4). Pipe jacking over 2,000 feet with intermediate jacking stations is routine.

Figure A-4. Intermediate jacking station

Open shields and open face machines are used in pipe jacking and conventional shield tunneling to provide ground support immediately behind the excavation face (see Figure A-5). They also incorporate excavation and spoils removal equipment and allow sufficient working space for personnel and operators. Open shields do not control groundwater pressures at the face and must be used above the groundwater table or with dewatering.



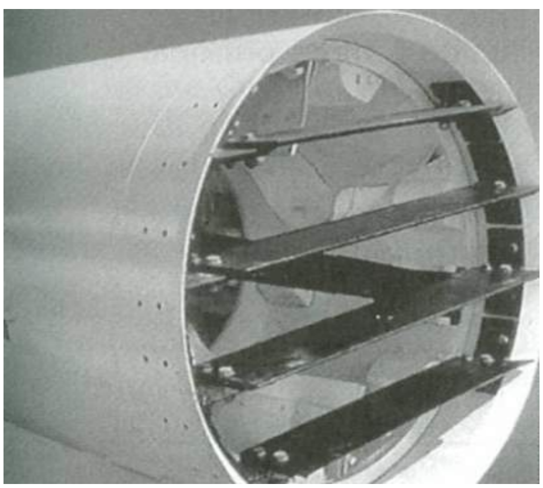
(a) Natural face support (Stein 2005)



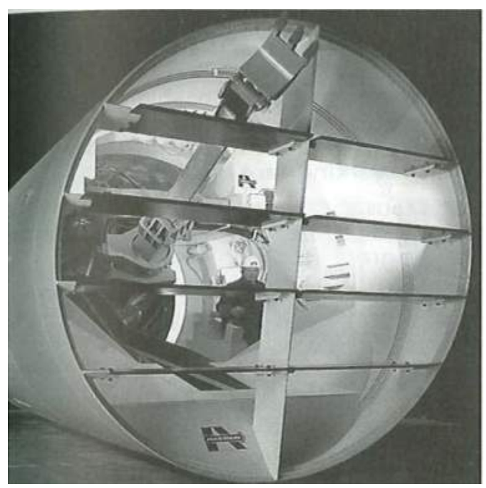
(b) Partial face support with sand shelves (Stein 2005)

(c) Rotary cutting head w/ partial face support (courtesy of Akkerman, Inc.)

(d) Rotary cutting head w/ adjustable full face support (courtesy of Horizontal Equipment Manufacturing, Inc.)



(e) Sand shelves (Stein 2005)



(f) Sand shelves w/ boom excavator and vertical web (Stein 2005)

Figure A-5. Open shield and open face machines

A variety of tunneling shields can be used with pipe jacking. A few examples are:

- Natural face support shield: This type of shield relies on natural ground support at the face. Under dry conditions, the natural angle of repose of the ground maintains face stability.
- Partial face support with sand shelves: This type of shield is suitable in loose sandy material and features horizontal plates that act as shelves to support the ground.

- Partial face rotary cutting heads: This type of shield features a partial face cutting head that is rotated using a hydraulic or electric motor incorporated within the shield. The motors provide the required torque to excavate the ground independent of the jacking station. These shields are similar to those on tunnel boring machines (TBMs).
- Full face rotary cutting shields: This type of shield is similar to the partial face shield but offers mechanical support to the ground along the entire face. This shield features hydraulically or manually adjustable doors within the cutting head that allow the operator to control the rate of excavation and remove any obstructions or over size material.

In addition, the open shields and machines are articulated to make steering adjustments to maintain the design line and grade. Line and grade are monitored using (1) a pipe laser mounted in the jacking shaft hitting a target at the back of the shield; (2) gyro guidance systems; or (3) self-leveling total stations. Curve pipe jacking is possible when using the nonlaser guidance systems.

Ground known to contain cobbles and boulders can present significant challenges for an advancing pipe jacking shield. However, with the generally open face, cobbles and boulders can be identified and broken into smaller rock pieces. The smaller pieces can then pass through the openings in the shield for removal.

A.3 Conventional Shield Tunneling

Conventional shield tunneling differs from the broader tunneling industry with respect to size and application of the tunnel. The primary use of these tunnels for pipeline projects is to house utilities and conduits. While methods of excavation for pipe jacking and conventional shield tunneling are similar, the main difference is in the type of ground support installed. In pipe jacking, the pipe or casing serves as the final lining for the excavation (see Figure A-3). With conventional shield tunneling, tunnel liner plates or steel ribs and lagging are used as temporary ground support. The lining for conventional shield tunnels is considered to be a temporary structure until the final carrier pipe is installed (see Figure A-6). The void between the carrier pipe and initial support is typically filled with cellular grout.

With conventional shield tunneling, an initial support system is installed in the tail of the shield to support the ground as the tunnel is excavated. The shield is advanced with hydraulic jacks pushing against the initial supports erected in the tail of the shield. After completing the tunnel drive, a final lining (or carrier pipe) is installed and grouted inside the tunnel to provide a finished tunnel. Considering safety, access, and mining efficiency, the minimum recommended size for a conventional shield tunnel is 72 inches. Conventional shield tunnels require a larger diameter to allow for personnel access and ease in installation of the temporary tunnel lining. These tunnels have no theoretical restriction for drive lengths as the shield is advanced by jacking against the lining immediately behind it. Conventional shield tunneling can easily be designed and constructed with curved alignments and for unlimited drive lengths.

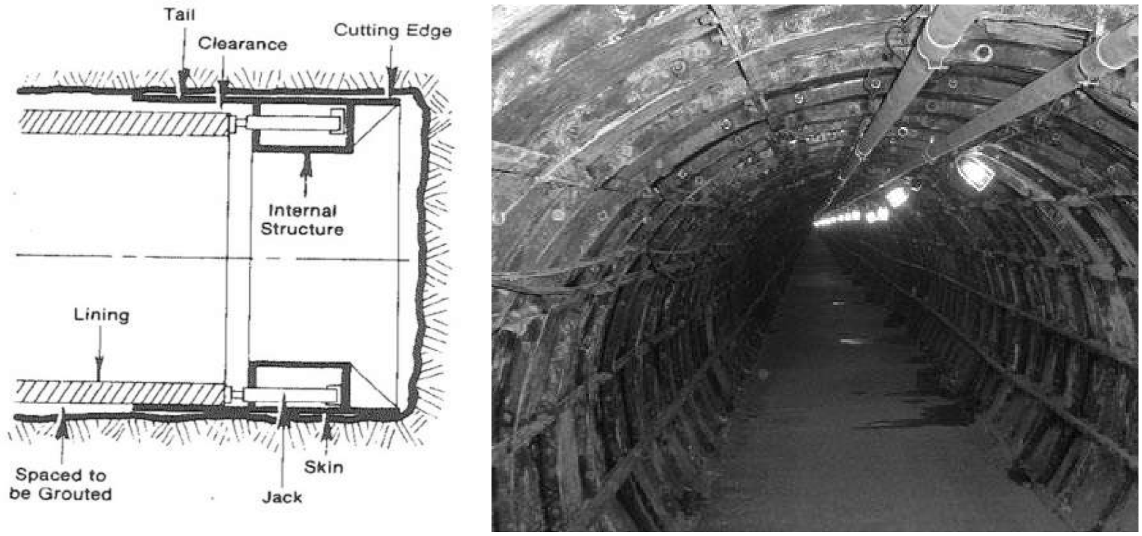


Figure A-6. Conventional shield tunneling with liner plate support (Proctor and White, 1977)

A.4 Microtunneling

Microtunneling is a pipe jacking method that simultaneously excavates the ground with a microtunneling boring machine (MTBM), counterbalances groundwater pressure with slurry, removes the excavated spoils via the slurry, and advances pipe segments to support the excavated ground. The MTBM is remotely controlled, guided, and steerable. The casing (or carrier pipe) is installed behind the machine in a pipe string to transfer jacking forces to simultaneously jack pipe and advance the machine into the ground. Excavation is carried out by the MTBM in front of the lead pipe section. The machine and transport slurry exert continuous and controllable pressure at the face of the excavation to support the ground at the same time counterbalance the groundwater pressures. Typical MTBM and pipe installation operations are shown in Figure A-7 and Figure A-8, respectively.

Excavated material and drilling fluid (slurry) are removed from a chamber behind the cutter wheel of the machine at a rate that is synchronized with the advance rate of the machine. These materials are typically transported back to the jacking shaft in slurry suspension. Besides conveying excavated ground, the slurry also counterbalances the hydrostatic pressures at the heading. The excavated materials are then separated from the slurry at the separation plant, and drilling fluid is circulated back into the closed-loop system. The spoils, together with some residual slurry, are hauled away from the site for disposal while the bulk of the slurry is recycled back into the tunneling operation. A typical microtunnel slurry plant layout is shown in Figure A-9.

Microtunneling machines are equipped with a sophisticated guidance system that utilizes a laser beam to establish a fixed reference to the design line-and-grade. The laser is independently supported in the jacking shaft with the beam set to the design line and grade. The laser beam is aimed at a target located in the rear of the MTBM. The operator is located in a surface control room and provided with a digital and/or closed-circuit display of the laser beam's position on the target. The operator uses this information to make steering corrections to maintain the beam on the target. If curved alignments are needed, gyro guidance systems or self-leveling total station survey equipment is used install of the laser.

Figure A-7. Microtunnel boring machine

Figure A-8. Placing casing for jacking operation during microtunneling

Figure A-9. Typical slurry separation plant setup

A pipeline installed by microtunneling is constructed in a series of drives from a jacking shaft to a receiving shaft. The drive length (or distance from the jacking shaft to the receiving shaft) for microtunneling methods typically ranges from a few hundred feet to over 1,500 feet. The ultimate drive length is a function of the pipe diameter and pipe materials, machine capabilities, and ground conditions. For this project's casing diameter, intermediate jacking stations (IJSs) can be installed in the casing string to extend drive lengths to over 2,000 feet. Figure A-4 shows a typical IJS.

Ground known to contain cobbles and boulders can present significant challenges for an MTBM, particularly since there is no direct face access. The MTBM must be designed with disc cutters on the cutter wheel to chip and break down the size of the cobbles or boulders to pass through the openings on the cutter wheel. Once the rock pieces are inside of the cutterhead, the rock pieces can be ground into even smaller pieces to pass through the screens at the base of the cutterhead for transport in the slurry system. Since the MTBMs for this project will be relatively large, they will have the increased horsepower and torque to chip away at any cobbles or boulders ahead of the MTBM. The only problem that could develop with cobbles and boulders is if the matrix material holding the cobbles and boulders in place is weak, allowing the cobbles and boulders to move freely within the earth. In that case, the MTBM's disc cutters on the cutter wheel are not able to effective chip away at the cobbles and boulders to make them smaller. They are MTBM therefore plowed forward by the MTBM, causing the cobbles and boulders to become nested ahead of the machine. Understanding the properties of the matrix materials will be important in assessing success of the MTBM to mine through cobbles and boulders.

For the ground conditions anticipated, we expect that rectangular or circular shafts can be used. Circular shafts utilizing liner plate or secant piles could be used to capitalize on the efficiency of circular hoop stress design. The diameter of a circular jacking shaft is generally a function of the casing or carrier pipe length being installed. For the assumed 10-foot-long casing segments, a circular jacking shaft approximately 26 feet in diameter would be required. A receiving shaft only needs to be large enough to remove the MTBM or pipe jacking shield. Their removal can generally be accomplished inside a 15-foot-diameter shaft.

A.5 Drill-and-Blast Tunneling

Some of the alignments selected will be excavated through full-face rock conditions. While an MTBM with jacked pipe could be outfitted with disc cutters for full-face rock excavation, the process is slow and inefficient. Rock tunneling is completed a number of different ways because the rock is typically self-supporting. In weak rock (less than 10,000 psi compressive strength), roadheaders are used. A roadheader is a crawling power pack with rotating arm(s) that clip the rock from the tunnel face. The tunnel is typically horseshoe shaped, with the roadheader carving out the top of the tunnel in an arch, semicircular pattern. The tunnel crown is arched to help with self-supporting the ground. The rest of the horseshoe is excavated with benches to the full tunnel height. Typically, the height of the horseshoe-shaped tunnel is equal to the width. This conventional driven tunnel is supported with rock dowels, rock bolts, shotcrete, wire mesh, steel straps, steel ribs, or a combination. As the rock becomes harder and stronger, the roadheader becomes less efficient at chipping the rock.

When the rock has a strength greater than 10,000 psi, drill-and-blast tunneling is used. A horizontal drill rig is mobilized into the tunnel at the heading/active face, where the rig drills horizontal blast holes for a set distance (round length). The pattern (spacing) of the blast hole is varied to define the tunnel opening. Closely spaced trim holes are drilled around the perimeter of the tunnel. Larger spaced load holes are drilled in the center. After the blast holes are drilled, the drill rig is moved away from the tunnel face and the holes are filled with explosives with timed detonators (or delays). Once the blast holes are charged, the explosives are detonated in a controlled pattern. First the center holes are detonated to form a hole, so the exploding rock can freely move to the newly created hole/space. As the rock explodes, the next ring of holes is detonated, until the entire rock face is blasted into small rock pieces. The time delay between the detonations is milliseconds. Once the round is completed and the air clears, the miners check to ensure all explosives have been detonated in the blast holes. The miners will check for loose rock in the tunnel crown and will scale the surface with steel rods/bars. If the tunnel needs initial ground support, it will be installed, otherwise load-haul-dumps (LHDs) are brought into the tunnel to scoop/pick up the blasted rock pieces for transport out of the tunnel. Once the tunnel is cleared of the blasted rock, the drill rig is mobilized back to the tunnel face to drill and install radial rock dowels or rock bolts. Once the tunnel is supported, the drills are turned horizontally to drill the next round of blast holes and the whole cycle is repeated. Round lengths vary based on the rock type and tunnel size. Round lengths of 8 to 10 feet are typical for a 12-foot by 12-foot horseshoe-shaped tunnel. Figure A-10 shows a drill-and-blast horseshoe-shaped tunnel with ground support.

Once the tunnel is excavated and supported, the carrier pipe will be transported into the tunnel (see Figure A-11. The pipe segments will be anchored in place to prevent flotation and welded together. The annular space between carrier pipe and the excavated tunnel will be backfilled with cement grout (see Figure A-12).

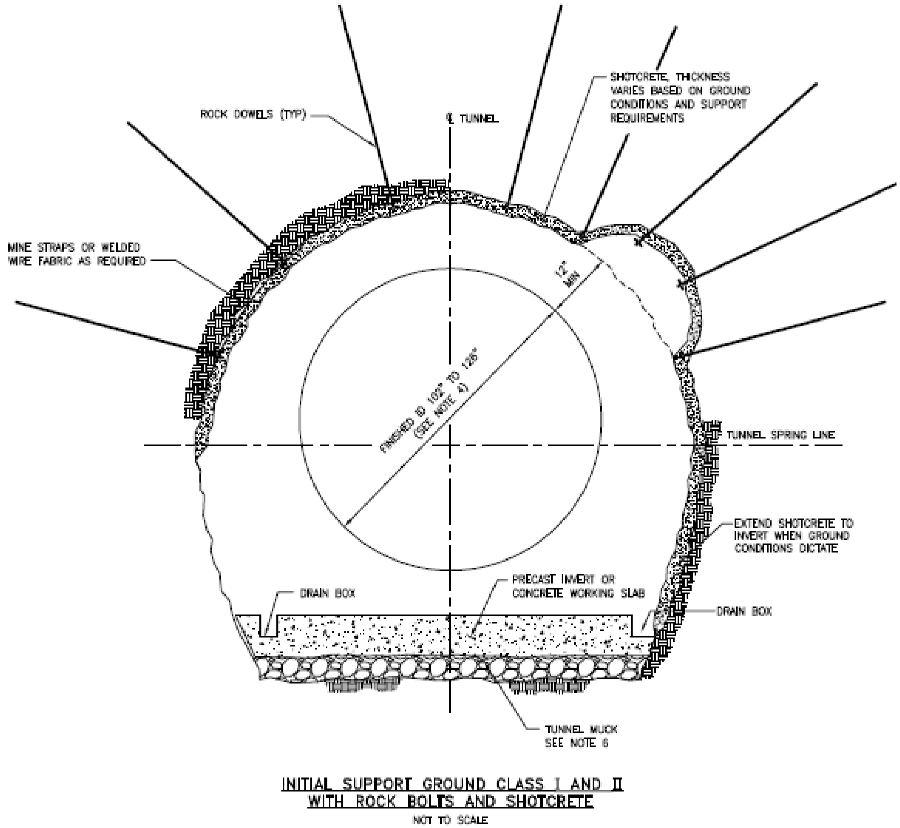


Figure A-10. Typical horseshoe-shaped drill-and-blast tunnel

Figure A-11. Carrier pipe being transported into the tunnel

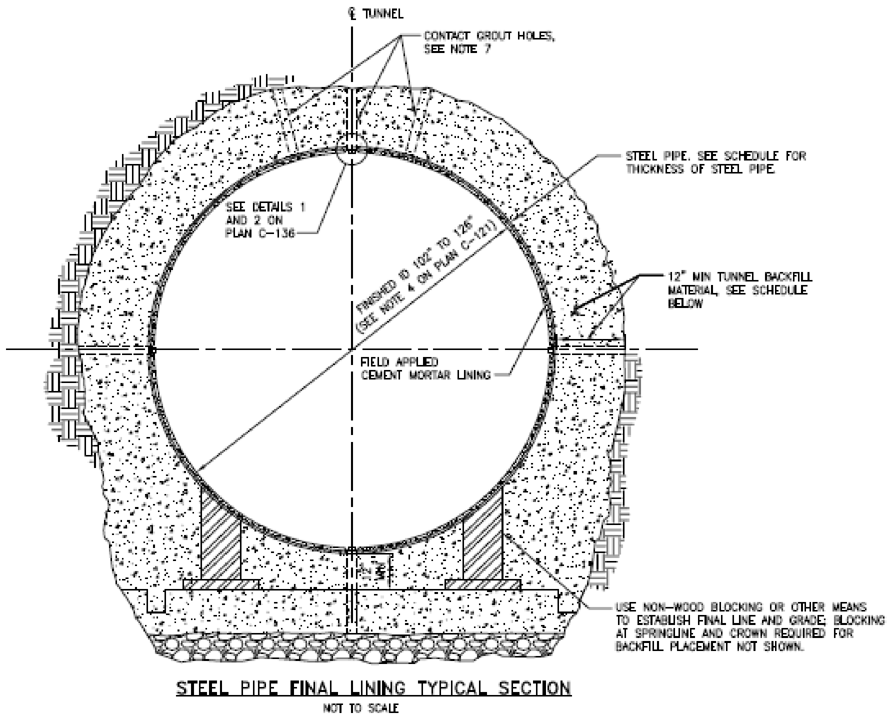


Figure A-12. Typical horseshoe-shaped tunnel with the carrier pipe inserted and grouted in place

A.6 Rock TBM Tunneling

The production rate of a conventional driven rock tunnel excavated with a roadheader or drill-and-blast methods is limited. Tunnels less than 10,000 feet with multiple headings typically use roadheaders or drill-and-blast methods. If the tunnel is longer, it become more efficient to mobilize a rock TBM. Rock TBMs are outfitted with the following:

- A cutter wheel with all rock disc cutters
- Typically more opening at the leading edge to allow the disc cutter to be replaced easily
- Grippers to engage the rock to allow forward thrust of the machine to engage the disc cutters (other TBMs use the jacking pipe or the tunnel support of the thrust reaction)
- Finger, crown, or full shield to provide rock wedges from falling on the rock TBM while it is mining

Since the forward thrust of the TBM can be developed from the grippers, the installation of any initial ground support is independent of the mining process. The same initial ground support elements mentioned above (rock dowels, rock bolts, shotcrete, wire mesh, steel straps, steel ribs, or a combination) are used in the rock TBM tunnel. Once the tunnel is excavated and supported, the carrier pipe will be installed in the tunnel and annular space between the carrier pipe and excavated tunnel will be filled with grout.

Appendix B Jacking and Receiving Shafts

This page intentionally left blank.

B.1 Introduction

Shafts are commonly required at each end of a trenchless installation or TBM operation to facilitate construction operations and allow for pipe and equipment installation and removal. Typically, the design of the temporary support of excavation systems for the shafts is made the responsibility of the contractor in the project specifications. Jacking shafts will be excavated and used to pipe jack/microtunnel in one or two directions. Receiving shafts will be excavated and used to receive the pipe jacks/MTBMs from one or two directions. The shafts will be sized based on the following considerations:

- Site constraints, including physical/cultural/man-made impedances
- Casing and carrier pipe length and diameter
- Jacking equipment to advance the pipe
- A jacking frame at the back of the shaft to advance pipe
- Space for workers to safely complete the installation
- Size of temporary shoring members to support shaft excavation

Casing and carrier pipe for each trenchless installation are assumed to be 10 feet in length. A minimum casing diameter of 108-inch ID would be required for a two-pass system. The rationale for the minimum 108-inch casing is as follows: assume 84-inch ID carrier pipe; with a wall thickness of 1 inch; assume minimum 9 inches on radius for cellular grout backfill annular space; and 4 inches on radius for line and grade adjustments of the carrier pipe.

A crane will be required outside of the jacking shaft to facilitate spoils removal and pipe and equipment transport to and from the shaft. A crane will be required outside of the receiving shaft for the retrieval of equipment. A laydown area for pipe and spoils, along with truck access for spoils transport at the ground surface, will also be required. Overall, in addition to the shaft area, a temporary construction easement of about 2,000 to 2,500 square feet would be needed to accommodate any one of the above-described trenchless/tunneling methods.

A major consideration on the selection of an underground solution for the Carson to Long Beach and San Gabriel River alignments will be the number of and location of shafts. Trenchless solutions will require up to 13 or 14 shafts for each alignment, many located in street ROWs and requiring partial lane closures for extended periods. By contrast, the conventional TBM options require only two shafts at each end at off-street work sites. Besides cost, the increased number of surface construction sites and greater traffic effects should be considered by MWD in comparing the two tunneling methodologies.

B.2 Temporary Shoring for Jacking and Receiving Shafts

Temporary shoring will be required to support shaft excavations during construction. Several shoring types are feasible for the ground conditions anticipated for the alignments. The following ground support systems are considered compatible with the anticipated ground conditions:

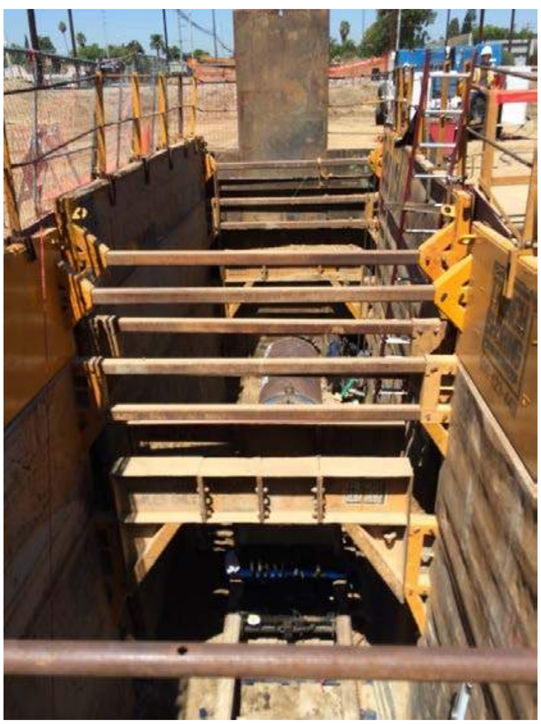
- Trench shields

- Sheet piles
- Soldier piles and lagging
- Liner plate
- Secant piles

Regardless of the support of excavation method used, some of the ground conditions will require positive support before excavation takes place. This is especially true of trench shields used in cohesionless, fine-grained soil that requires maintaining the shaft excavation level at or above the toe of the shoring system. Other ground conditions are expected to be self-supporting with favorable stand-up time that would allow top down construction of the support system (i.e., liner plates and shotcrete lining).

B.3 Trench Shields

Trench shields are often the most efficient and economical method of excavation support for relatively shallow trenchless crossings (see Figure B-1). Trench shields can typically be used for excavations up to about 10 feet wide x 30 feet long, and may be stacked to support excavations up to 25 feet deep. Trench shields used to support larger excavations are heavy and will require a large crane for installation and removal. Special provisions will be needed at the back of the trench shield to ensure there is intimate contact between the shield and the ground so that jacking forces are adequately resisted. In addition, requirements to control the gap between the trench shield and the ground will be needed to prevent raveling behavior that may lead to ground loss. Depending on the loads transferred to the ground, ground improvement may be needed to ensure the ground has enough strength so that the back wall does not deflect.



(a) Trench shields supporting shaft excavation

(b) Trench shields with auger boring

Figure B-1. Trench shields supporting launch shaft excavation

B.4 Sheet Piles

Excavation support with steel sheet piles is achieved by driving or vibrating rows of interlocking sheets to a depth sufficient to provide the required resistance to lateral earth pressures. Sheet piles would likely be installed to a depth of about 10 to 15 feet below the base of the excavation in ground conditions favorable for driving sheet piles. The interlocking sheet piles can be watertight. Struts, wales, and braces can be installed to provide additional resistance to lateral pressures. Special provisions, such as ground improvement or contact grouting, may be needed at the back wall of the shaft to ensure there is intimate contact between the sheets and the ground so that jacking forces are adequately resisted. Sheet piles will not be compatible with ground having cobbles and boulders, which is known to be present along some of the study alignments. If sheet piling is used in dense cobble and boulder ground or weak rock, slots can be pre-excavated and backfilled with sand to allow the insertion of the sheet piles in complex ground conditions.

B.5 Soldier Piles and Lagging

Excavation support with soldier piles and lagging is achieved by installing soldier piles from the ground surface and then placing lagging between the piles during excavation to retain the ground. Soldier piles are typically installed by drilling or driving steel H-piles at 4 to 8 feet centers around the perimeter of the proposed excavation. Pile depths generally range from about 8 to 10 feet below excavation base.

Excavation generally proceeds in 5-foot intervals (“lifts”) following pile installation, with lagging boards or steel plates placed to bear against the exposed beam flanges following the excavation of each lift (see Figure B-2). The ground conditions will dictate whether smaller excavation intervals will be needed to maintain stability of the ground before placement of the lagging boards. Wales and struts will likely be required to limit systemic deflection of the shoring system and provide adequate resistance to ground loads. Soldier piles placed behind the reaction wall will have to be designed to adequately resist jacking forces.

Figure B-2. Trench shields supporting launch shaft excavation

B.6 Liner Plate Shoring

Liner plate shoring systems use a system of manufactured curved steel plates (3.14 feet long) that can be interconnected to form a ring support system for a circular shaft (see Figure B-3). Liner plates are typically designed in 12-inch to 24-inch ring depths and can be installed as the shaft excavation progresses to provide support throughout the installation. Grouting is usually performed between the liner plates and the adjacent ground to ensure stability of the adjacent ground after installation. Liner plates offer the advantage of lightweight components that can be easily handled and bolted together. Liner plate systems can be removed, and the components are typically reusable. For larger and deeper excavations, liner plate systems can be used in conjunction with steel ring beams for additional support.

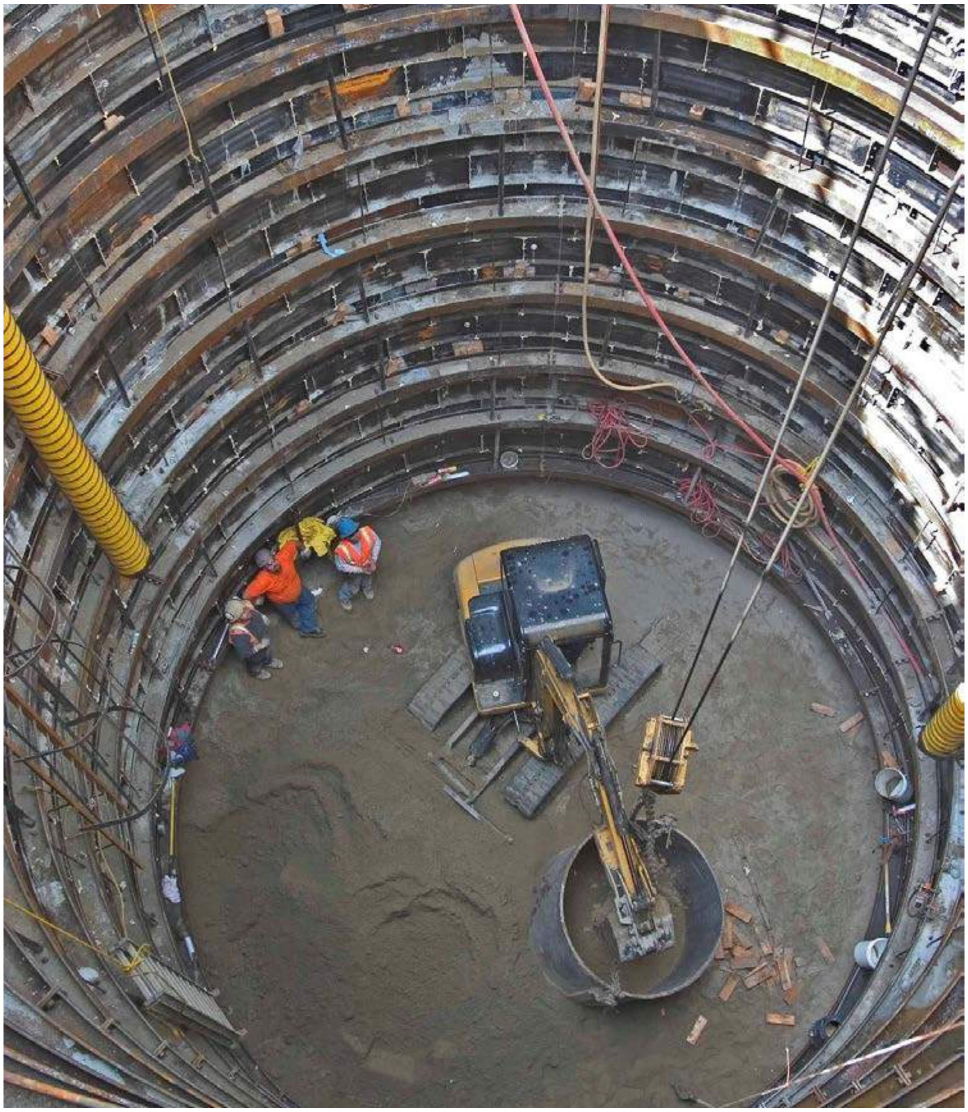


Figure B-3. Typical liner plate shaft with ring beams and tie rods

B.7 Secant Piles

When a shaft needs to be watertight, secant piles are a good solution. A drill rig will be mobilized to the site. The first pile hole is drilled/augered to the depth needed, typically up to 115 feet. The hole may be cased and/or filled with slurry during the drilling process to keep the hole stable and open. The verticality of the hole is checked for plumbness. The hole is then filled with tremie concrete from the bottom to the top of the pile, displacing the slurry and/or removing the casing. The next primary pile is drilled in line and offset from the first pile, checked for verticality, and then tremied with concrete. After a number of primary piles have been drilled and filled, the drill rig then drills between two primary piles, excavating through the ground and the two "green"/still low-strength primary piles. By drilling into the existing primary piles, an overlap is created with the adjacent (secant) piles, making a wall of overlapping concrete piles (see Figure B-4). The overlapping piles make a watertight barrier.

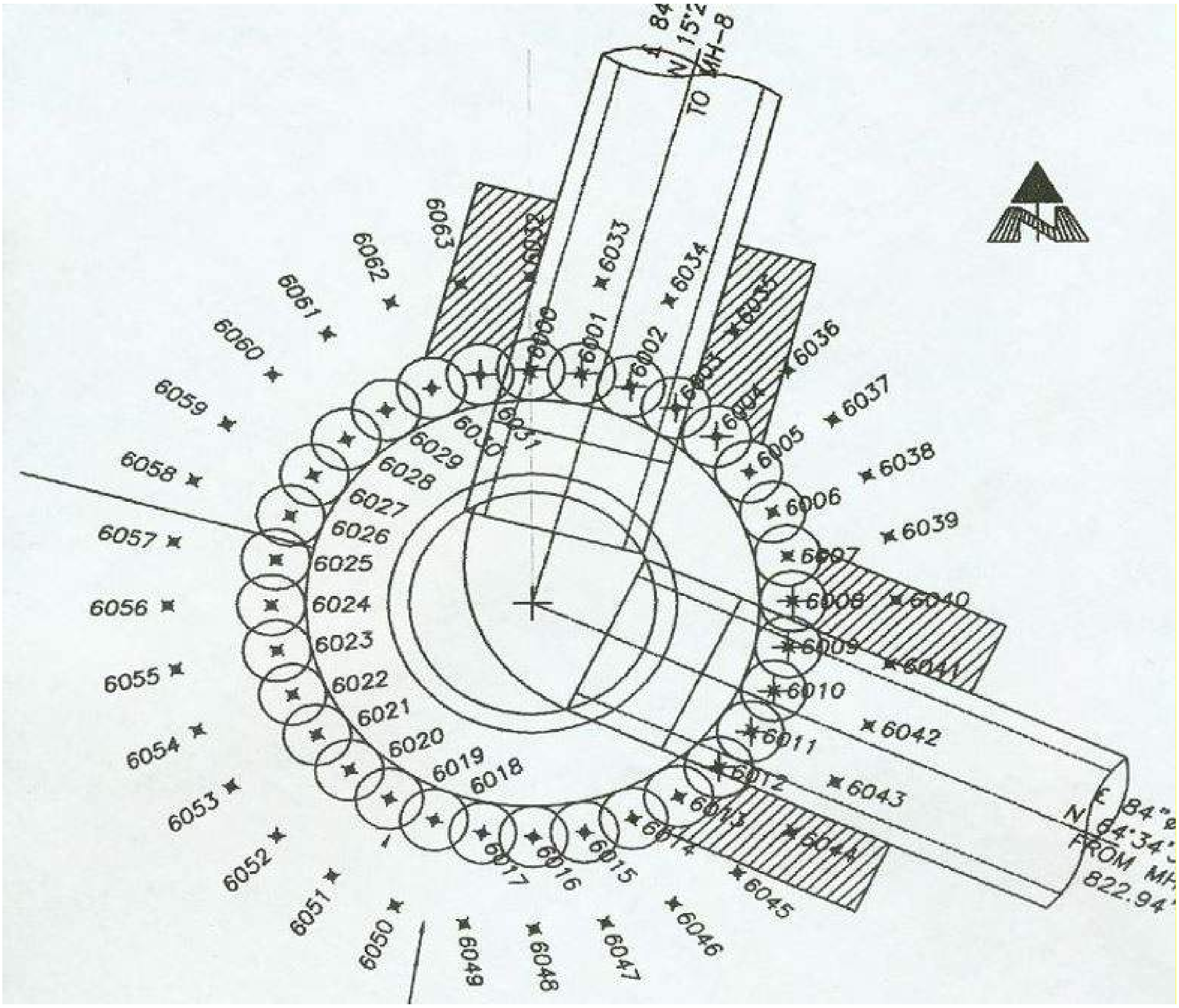


Figure B-4. Typical secant pile shaft layout with overlapping concrete secant piles

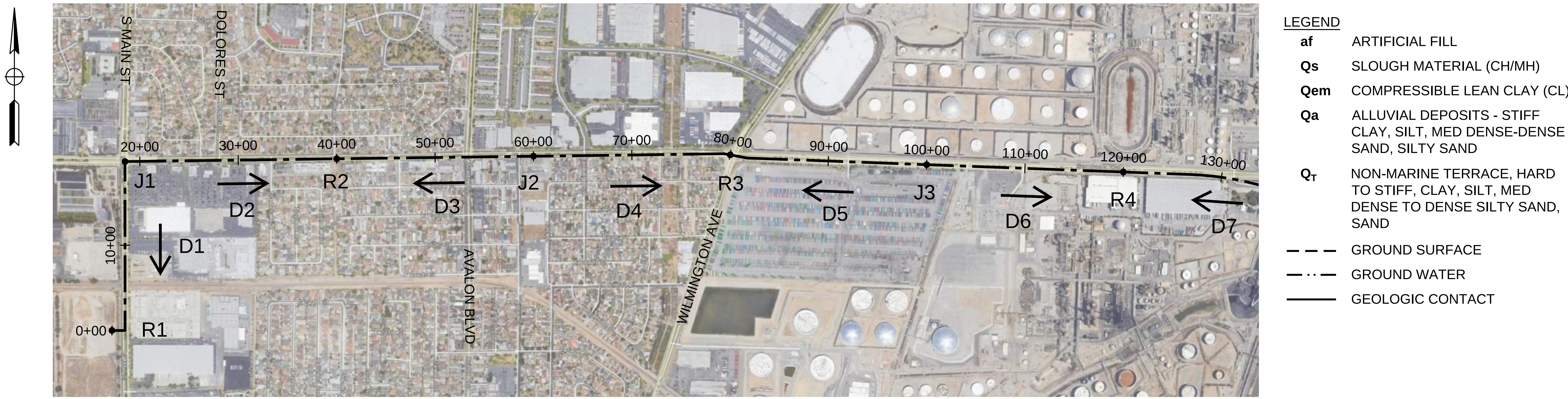
To make the shaft fully watertight, overlapping jet grouted columns can be added at the shaft invert for the needed depth and invert slab interval. The jet grouted column can seal the bottom of the shaft within the secant pile ring. Once the jet grout columns are hardened, the shaft can be excavated and used for pipe jacking, microtunneling, or tunneling (see Figure B-5.

Figure B-5. Typical secant pile shaft during the installation of a 108-inch ID casing)

This page intentionally left blank.

Appendix C Tunnel Plan and Profile Sheets

This page intentionally left blank.



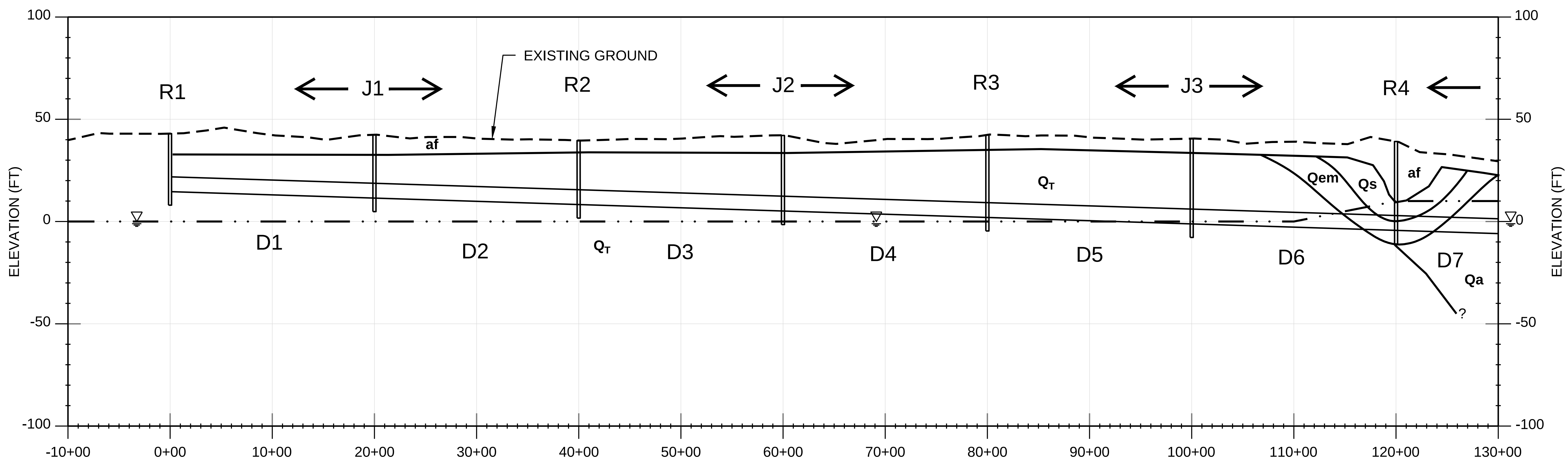


FIGURE C1
CARSON TO LONG BEACH
OPTION 1A - PIPE JACKING/MICROTUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"

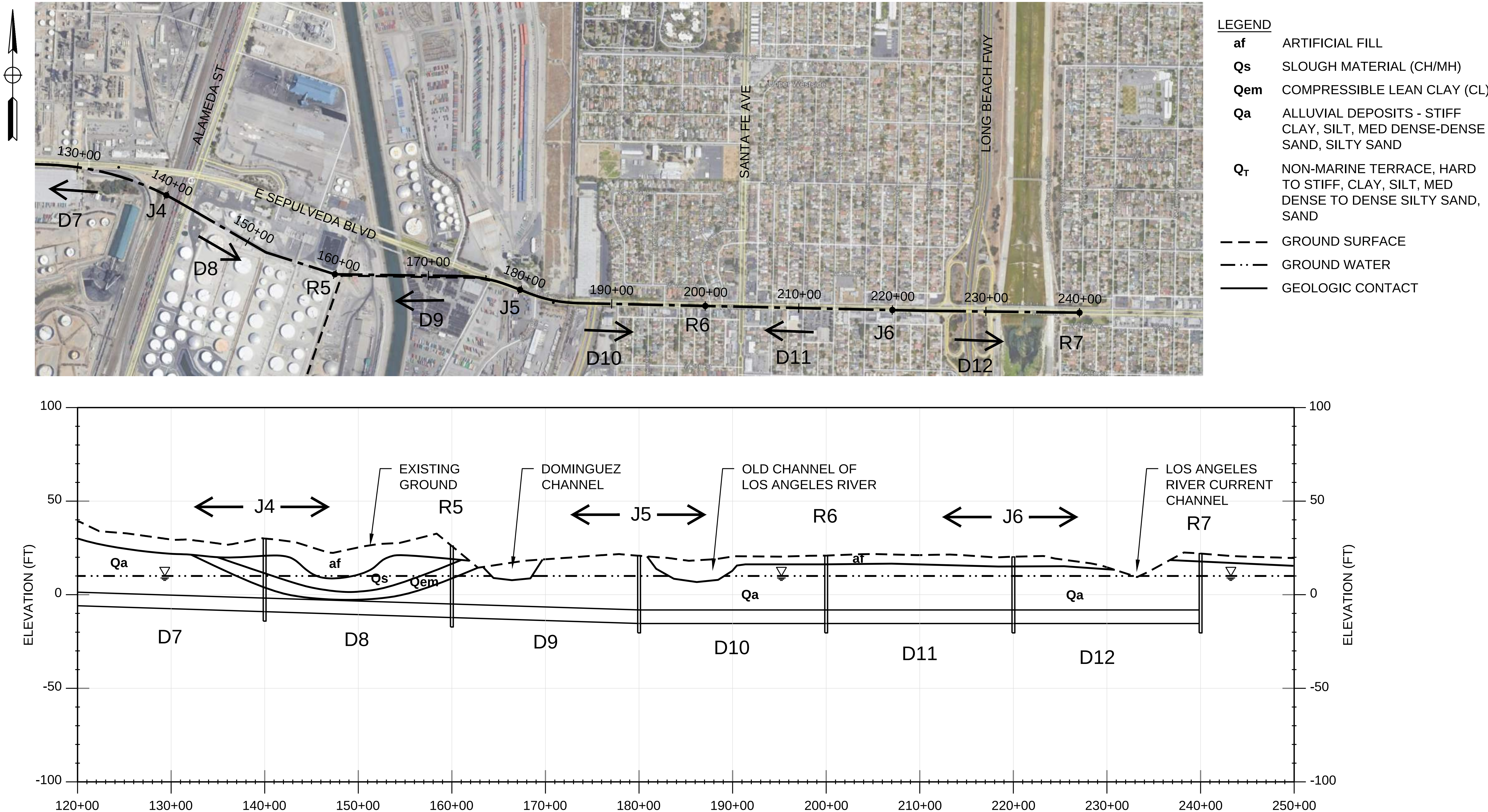


FIGURE C2
CARSON TO LONG BEACH
OPTION 1A - PIPE JACKING/MICROTUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"

LEGEND

af	ARTIFICIAL FILL
Qs	SLOUGH MATERIAL (CH/MH)
Qem	COMPRESSIBLE LEAN CLAY (CL)
Qa	ALLUVIAL DEPOSITS - STIFF CLAY, SILT, MED DENSE-DENSE SAND, SILTY SAND
Q_T	NON-MARINE TERRACE, HARD TO STIFF, CLAY, SILT, MED DENSE TO DENSE SILTY SAND, SAND
- - - -	GROUND SURFACE
—··—	GROUND WATER
———	GEOLOGIC CONTACT

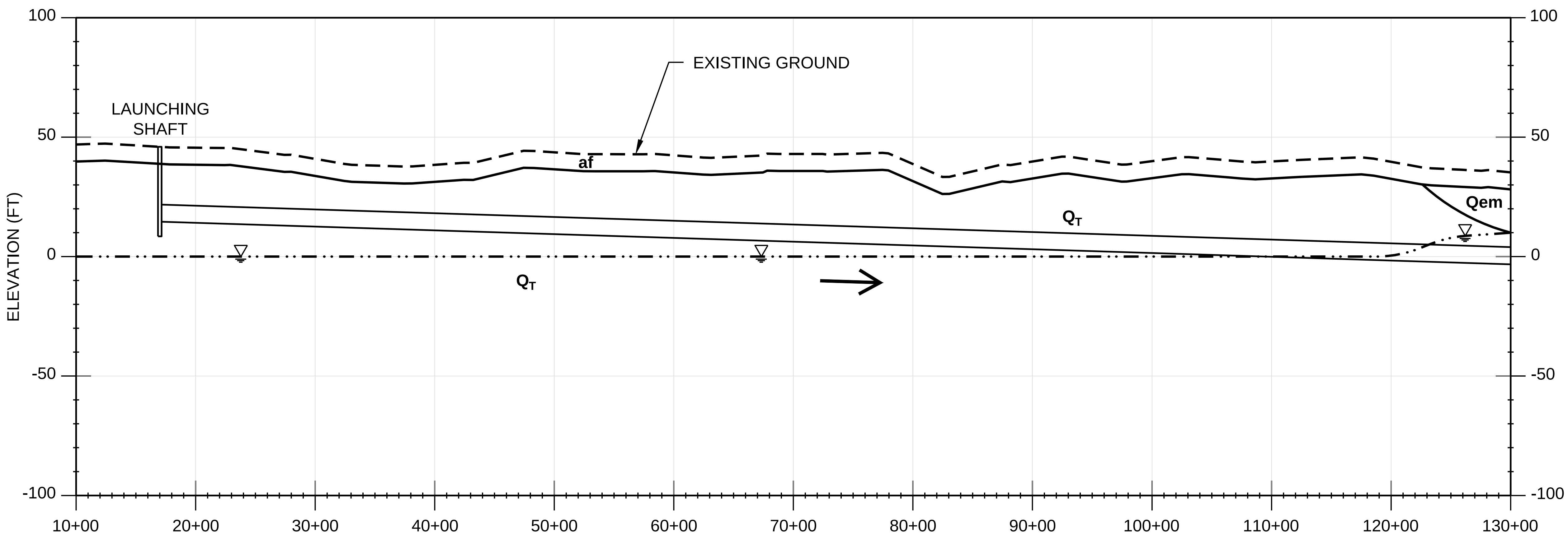


FIGURE C3
CARSON TO LONG BEACH
OPTION 1B - TUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"

LEGEND

af	ARTIFICIAL FILL
Qs	SLOUGH MATERIAL (CH/MH)
Qem	COMPRESSIBLE LEAN CLAY (CL)
Qa	ALLUVIAL DEPOSITS - STIFF CLAY, SILT, MED DENSE-DENSE SAND, SILTY SAND
Q_T	NON-MARINE TERRACE, HARD TO STIFF, CLAY, SILT, MED DENSE TO DENSE SILTY SAND, SAND
– – –	GROUND SURFACE
–·–·–	GROUND WATER
——	GEOLOGIC CONTACT

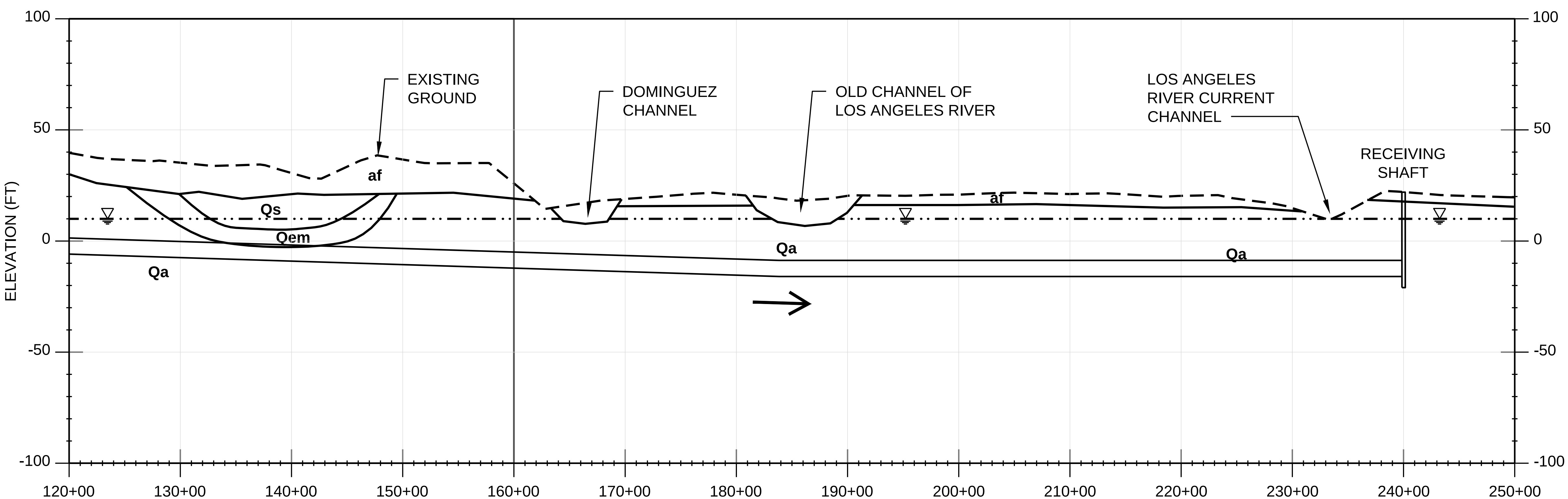
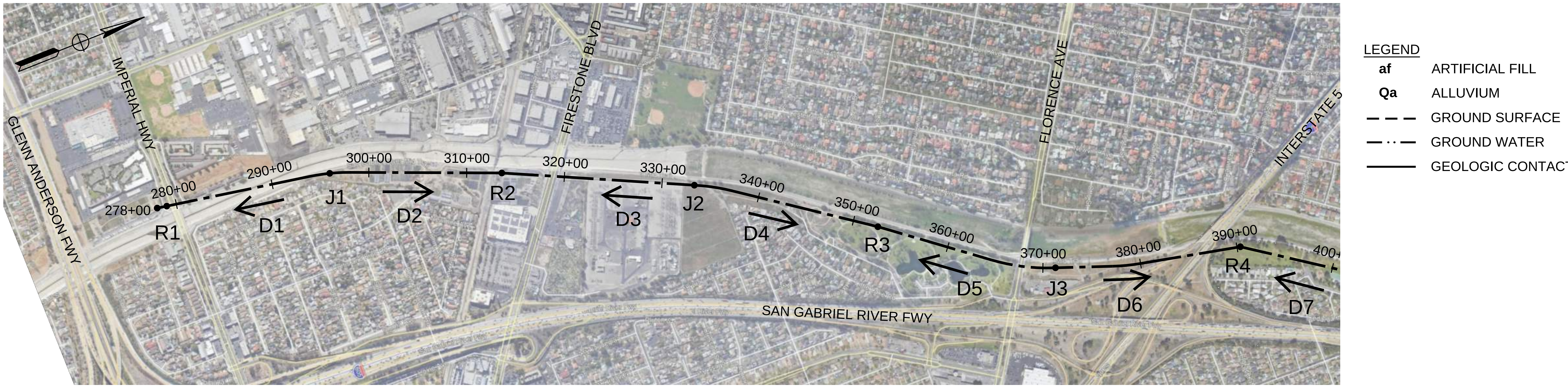


FIGURE C4
CARSON TO LONG BEACH
OPTION 1B - TUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"



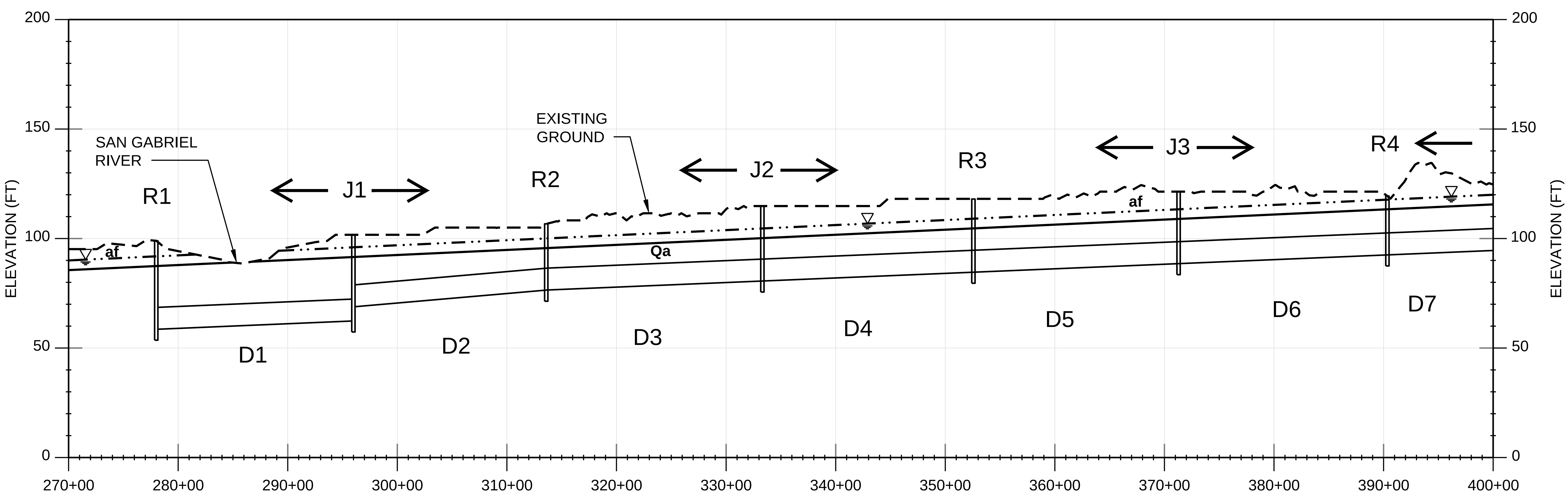
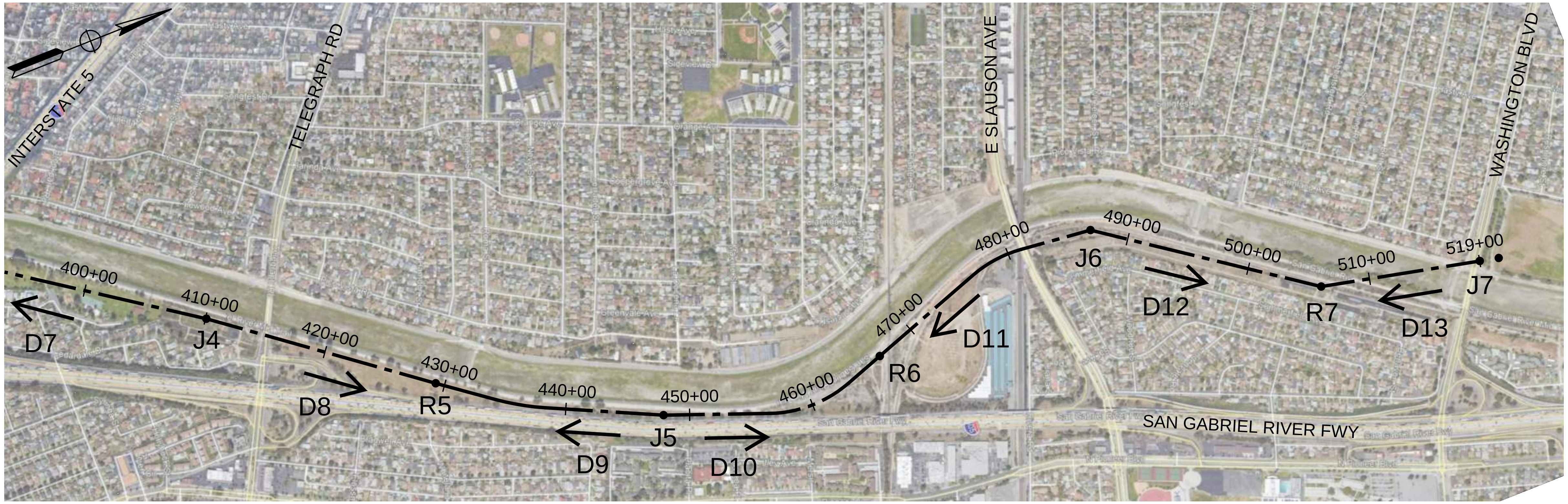


FIGURE C5
SAN GABRIEL RIVER
OPTION 2A - MICROTUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"



LEGEND

af	ARTIFICIAL FILL
Qa	ALLUVIUM
— — —	GROUND SURFACE
—·—·—	GROUND WATER
———	GEOLOGIC CONTACT

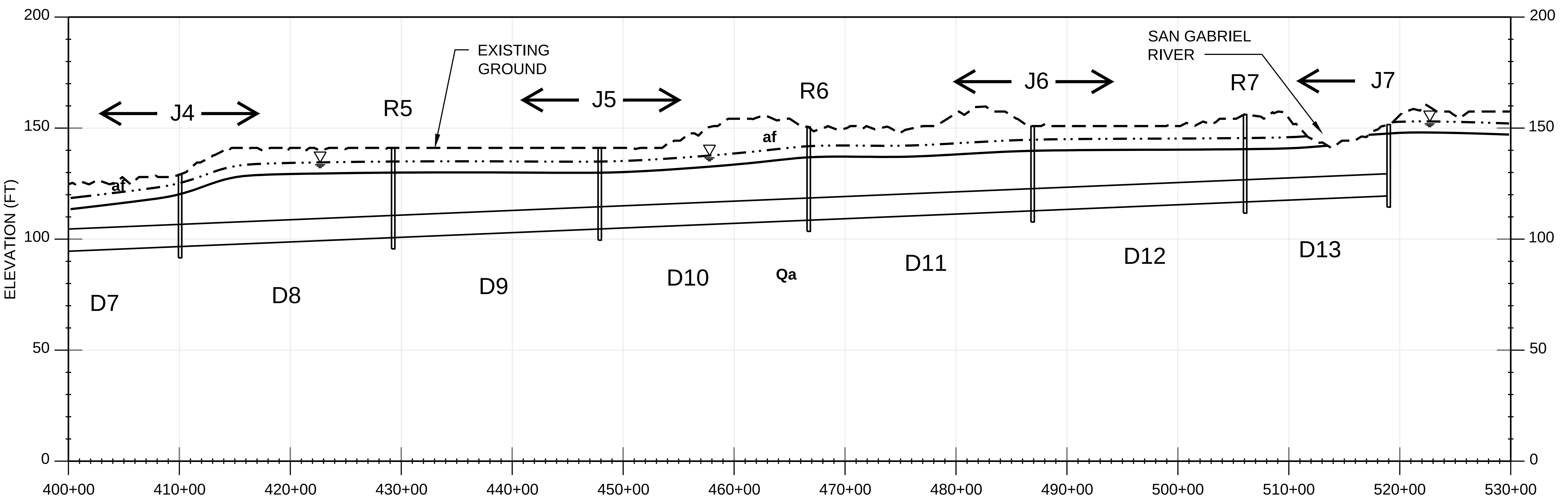
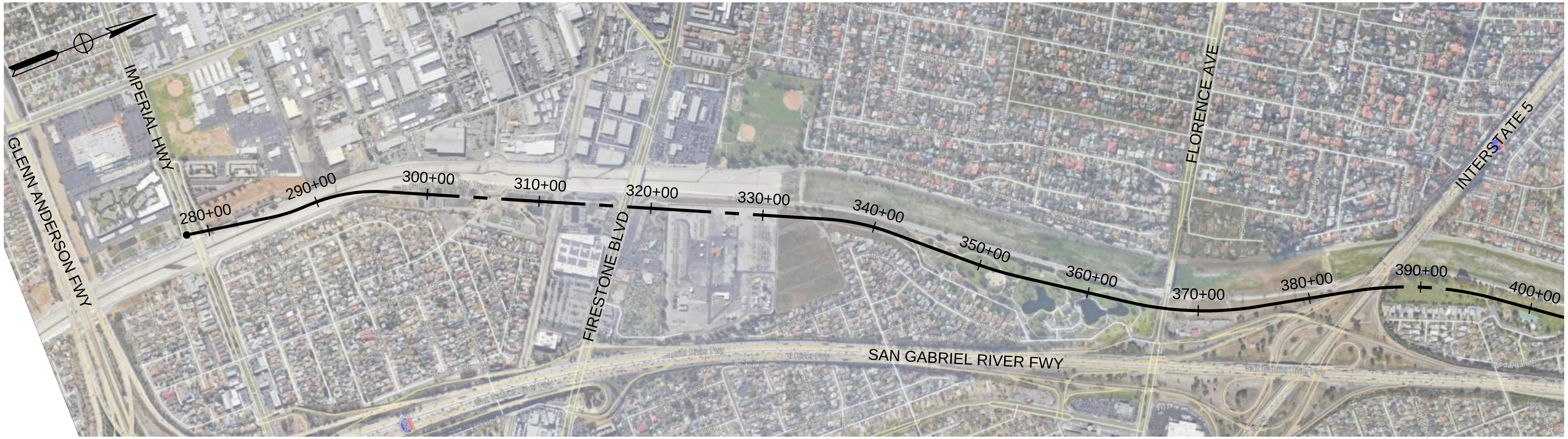


FIGURE C6
SAN GABRIEL RIVER
OPTION 2A - MICROTUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"



LEGEND
af ARTIFICIAL FILL
Qa ALLUVIUM
GROUND SURFACE
GROUND WATER
GEOLOGIC CONTACT

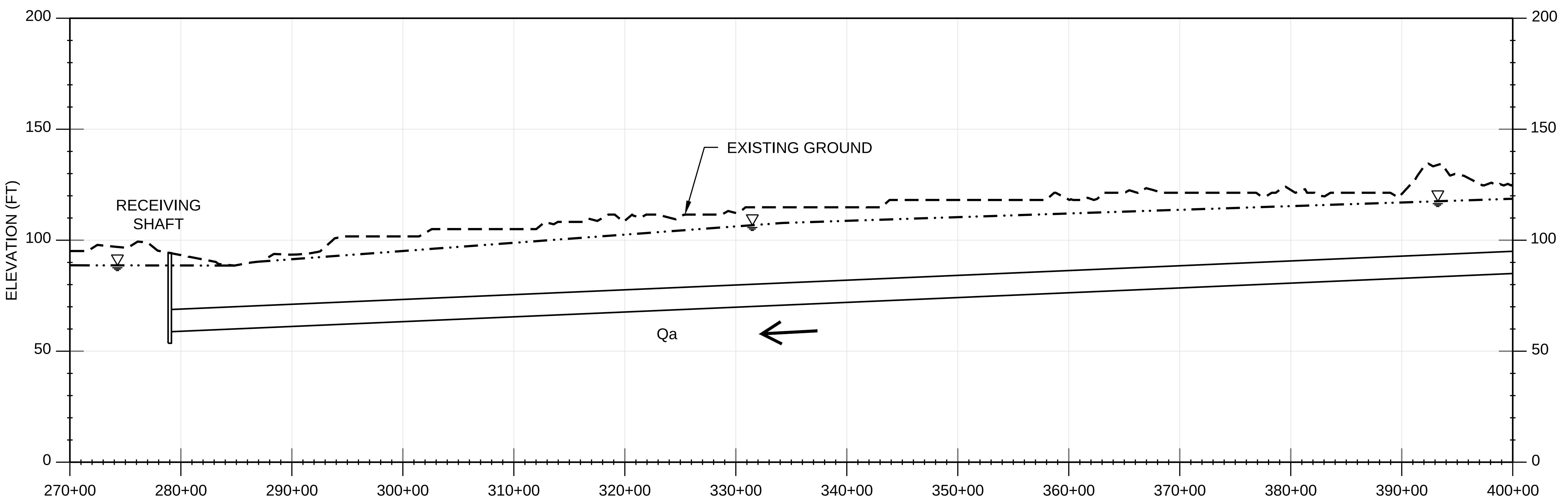


FIGURE C7
SAN GABRIEL RIVER
OPTION 2B - TUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"

LEGEND

af	ARTIFICIAL FILL
Qa	ALLUVIUM
— — —	GROUND SURFACE
—·· —	GROUND WATER
———	GEOLOGIC CONTACT

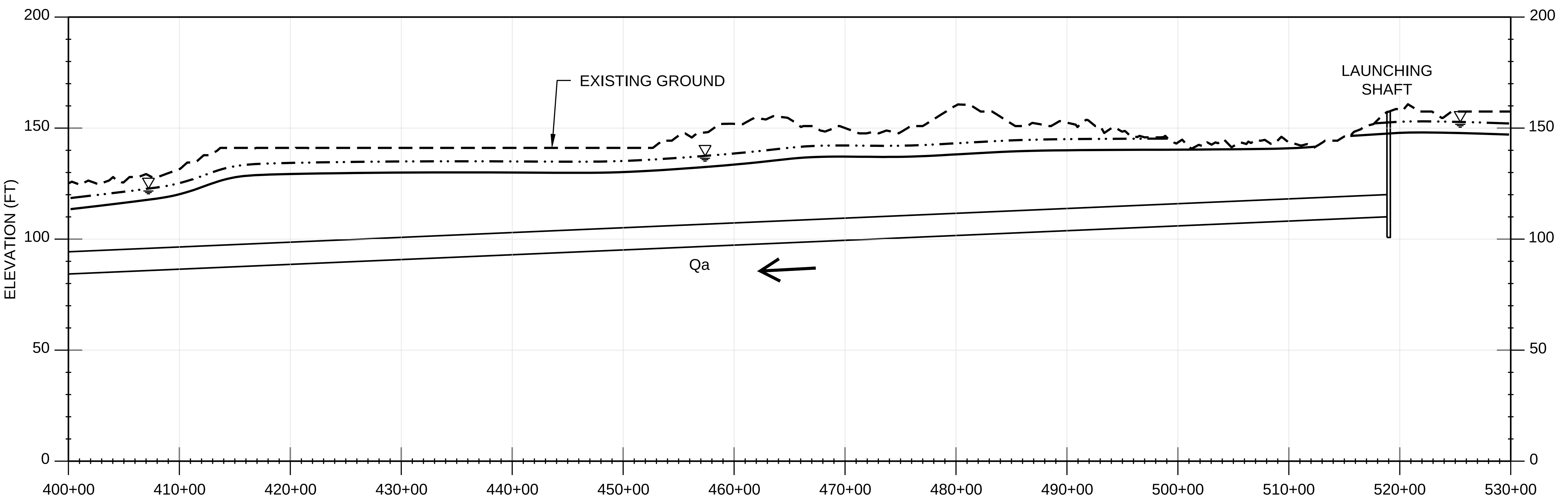
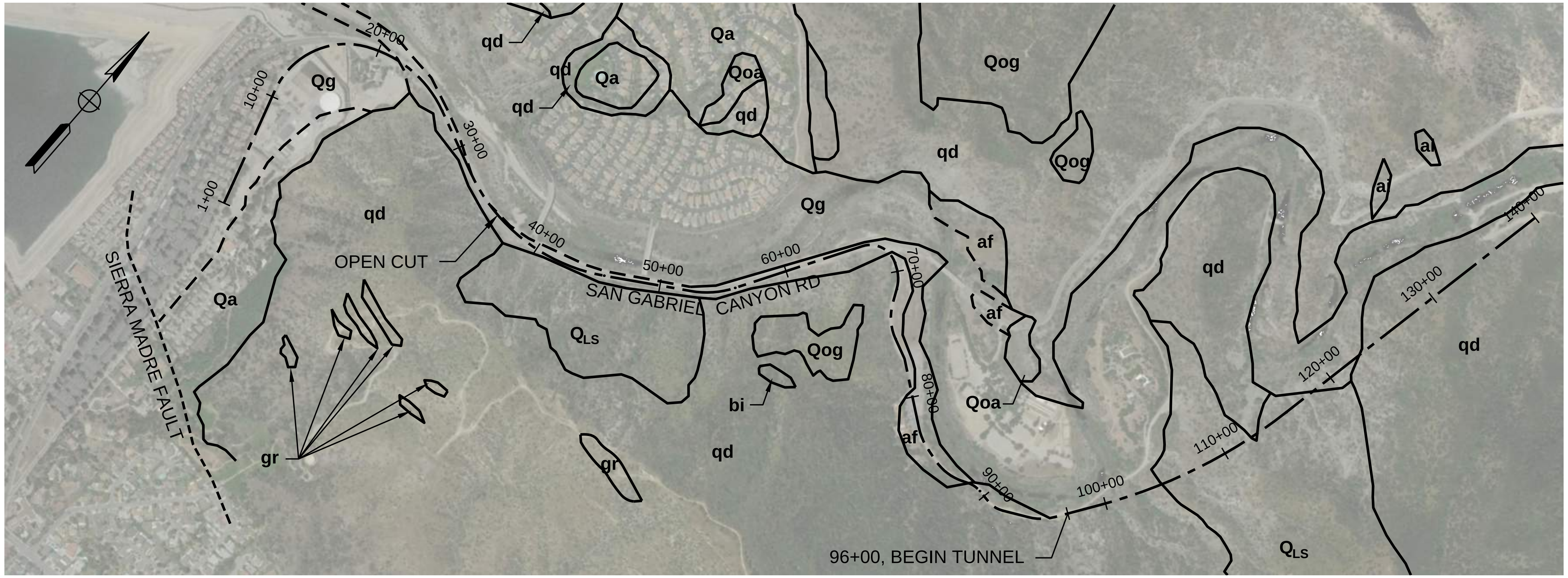


FIGURE C8
SAN GABRIEL RIVER
OPTION 2B - TUNNELING
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=50'-0"



LEGEND

af	ARTIFICIAL FILL
Qa	ALLUVIUM
Qg	STREAM DEPOSITS GRAVEL/SAND
Qoa	TERRACE
qd	QUARTIZ DIORITE
gr	GRANITIC
Q_{LS}	TALUS/LANDSLIDE
ai	DIKE, ANDESITE
bi	DIKE, BASALT
	GROUND SURFACE
	GROUND WATER
	CONTACT
	CONTACT OBSCURED

NOTES

1. ALL UNIT CONTACTS ARE APPROXIMATE. THICKNESS OF SURFICIAL UNITS IS NOT KNOWN.
2. STATIC GROUNDWATER ELEVATIONS ARE NOT KNOWN.
3. GEOLOGIC UNITS FROM DIBLEE MAP FOR AZUSA/GLENDORA QUAD DF-67.

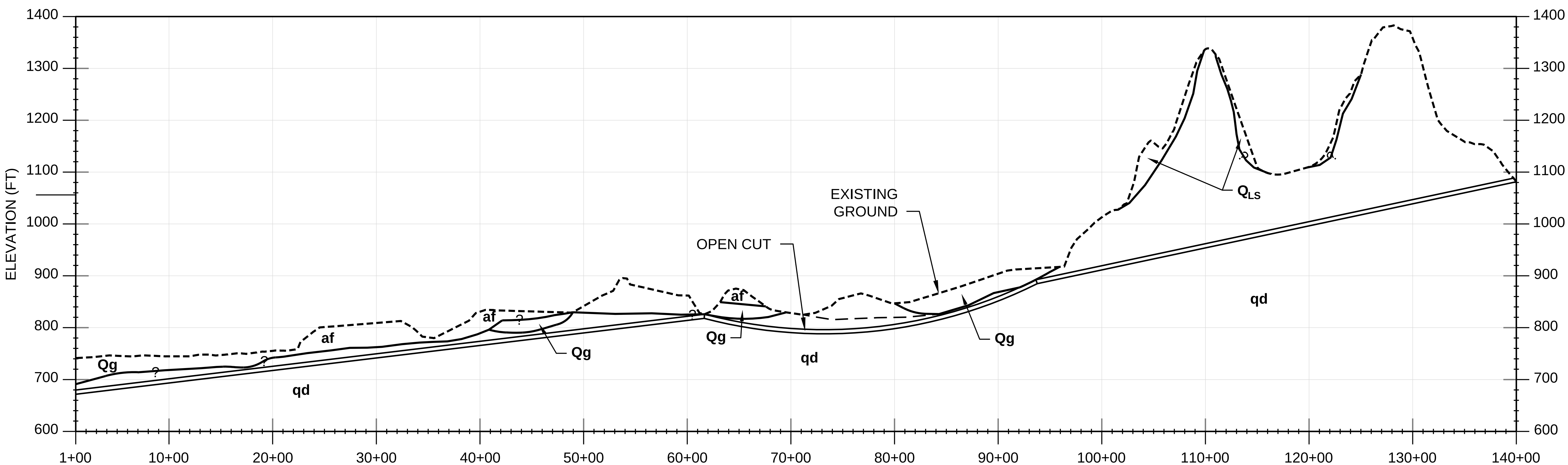
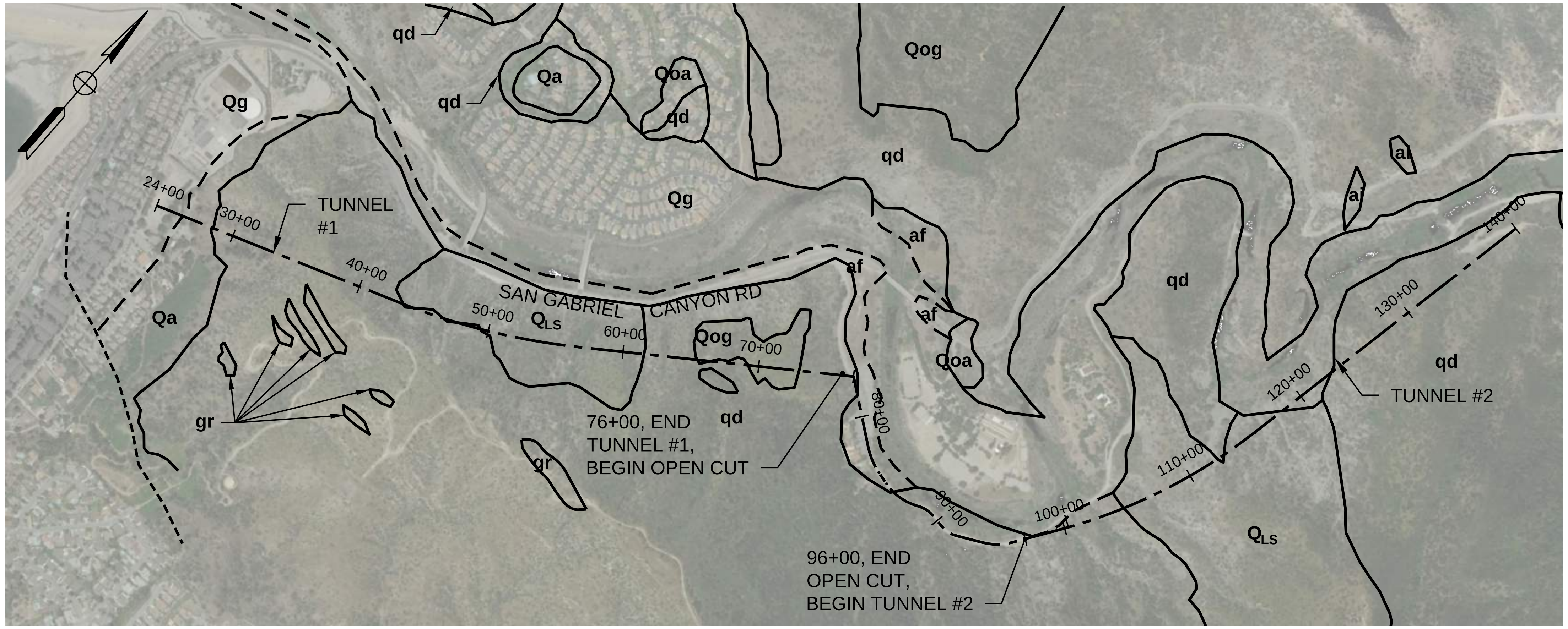


FIGURE C9 - AZUSA TO GLENDORA OPEN CUT AND TUNNEL OPTION 3A
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=200'-0"



LEGEND

af	ARTIFICIAL FILL
Qa	ALLUVIUM
Qg	STREAM DEPOSITS GRAVEL/SAND
Qoa	TERRACE
qd	QUARTIZ DIORITE
gr	GRANITIC
Q_{LS}	TALUS/LANDSLIDE
ai	DIKE, ANDESITE
bi	DIKE, BASALT
	GROUND SURFACE
	GROUND WATER
	CONTACT
	CONTACT OBSCURED

NOTES

1. ALL UNIT CONTACTS ARE APPROXIMATE. THICKNESS OF SURFICIAL UNITS IS NOT KNOWN.
2. STATIC GROUNDWATER ELEVATIONS ARE NOT KNOWN.
3. GEOLOGIC UNITS FROM DIBLEE MAP FOR AZUSA/GLENDORA QUAD DF-67.

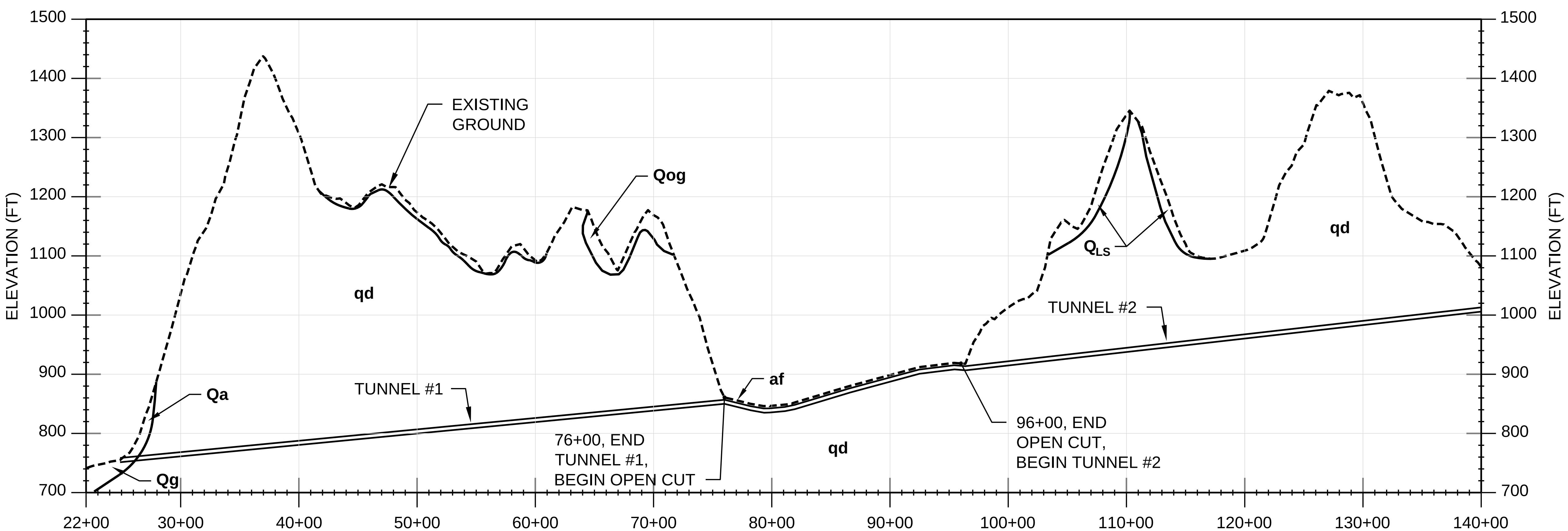
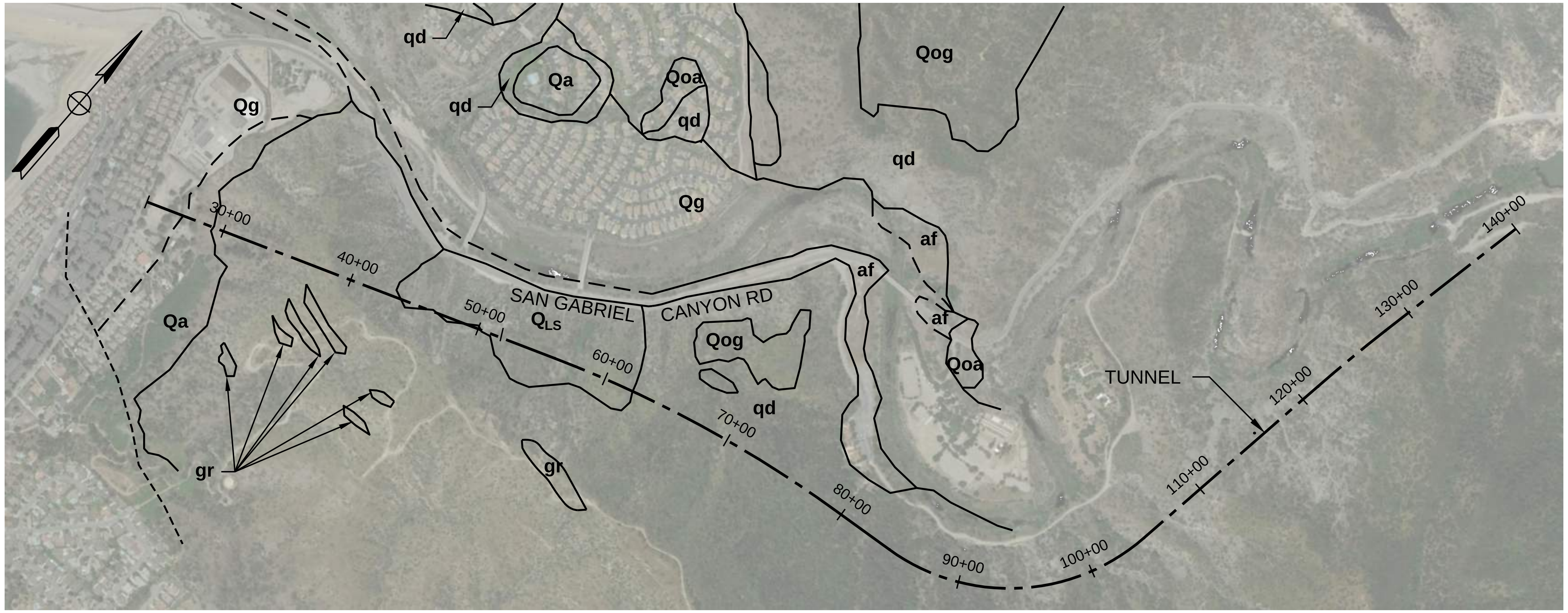


FIGURE C10 - AZUSA TO GLENDORA - OPTION 3B - TUNNEL-OPEN CUT-TUNNEL
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=200'-0"



LEGEND

af	ARTIFICIAL FILL
Qa	ALLUVIUM
Qg	STREAM DEPOSITS GRAVEL/SAND
Qoa	TERRACE
qd	QUARTIZ DIORITE
gr	GRANITIC
Q_{LS}	TALUS/LANDSLIDE
ai	DIKE, ANDESITE
bi	DIKE, BASALT
	GROUND SURFACE
	GROUND WATER
	CONTACT
	CONTACT OBSCURED

NOTES

1. ALL UNIT CONTACTS ARE APPROXIMATE. THICKNESS OF SURFICIAL UNITS IS NOT KNOWN.
2. STATIC GROUNDWATER ELEVATIONS ARE NOT KNOWN.
3. GEOLOGIC UNITS FROM DIBLEE MAP FOR AZUSA/GLENDORA QUAD DF-67.

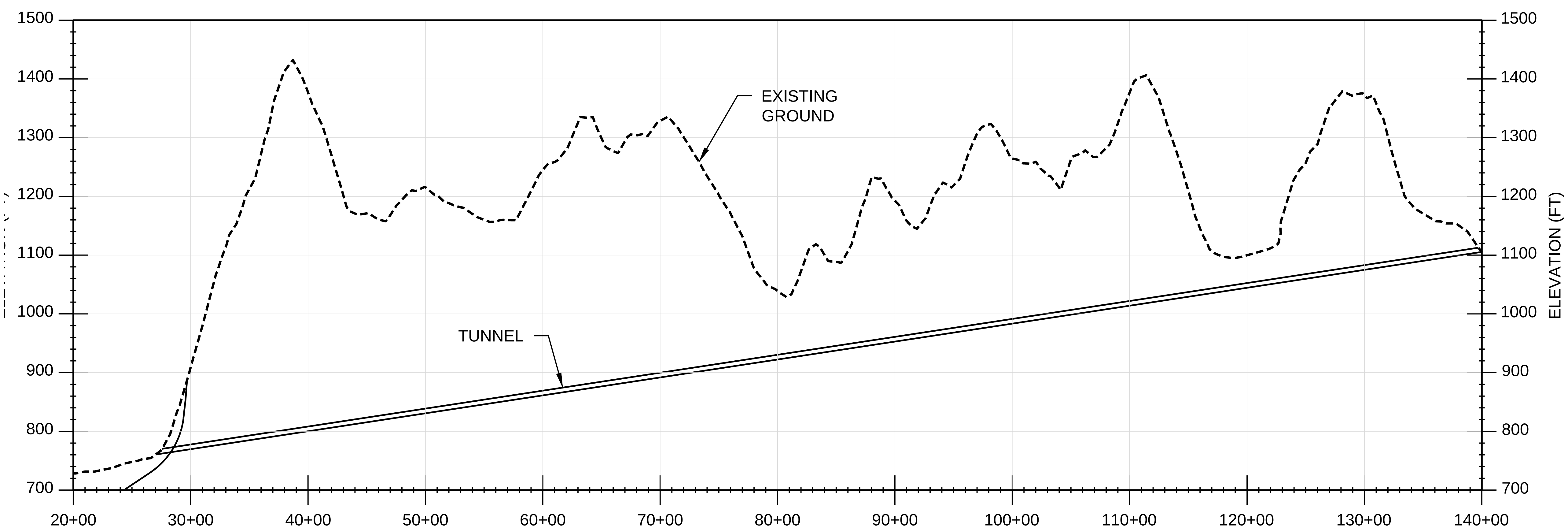


FIGURE C11 - AZUSA TO GLENDORA - OPTION 3C - TUNNEL
HORIZONTAL 1"=1000'-0"
VERTICAL 1"=200'-0"