APPENDIX 1b

West Valley Sewer Conveyance System Technical Memorandum

West Valley Water Reclamation Facility Sewer Conveyance System Technical Memorandum

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FINAL

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I.0 Introduction

I.1 Background

Mission Springs Water District's (MSWD) mission is to provide, protect, and preserve our most valuable resource, water. The District's dedication to protecting and preserving the quality of its most valuable natural resource is demonstrated through its Groundwater Quality Protection Program (GQPP). The program involves constructing municipal wastewater collection and treatment systems that will eliminate individual septic systems that overlie the Mission Creek and Desert Hot Springs aquifers. The Desert Hot Springs community demonstrated its support for protecting and preserving local groundwater supplies by approving special assessment districts to aid in funding the GQPP.

MSWD has successfully completed GQPP projects since 2006, and their continued efforts resulted in a need for additional wastewater treatment capacity. As such, MSWD has elected to pursue the completion of West Valley Water Reclamation Program (WVWRP) to meet the growing wastewater treatment capacity needs within its service area. The WVWRP has three components: construction of the West Valley Water Reclamation Facility (WVWRF), construction of a conveyance system connecting existing sewered areas to the WVWRF, and constructing a collection system for GQPP Area M2 (to be served by the WVWRF).

I.2 Scope

The objective of this Technical Memorandum (TM) is to confirm or modify the existing, interim and ultimate tributary areas and establish an appropriate design parameters (land use and wastewater flows) in order to properly size the proposed wastewater facilities such as gravity sewers, force mains and lift station. The TM will determine appropriate sewer main alignments, diameters, and depths along with the possible use of force mains and lift stations necessary, to deliver wastewater to the proposed WVWRF. In this TM, TKE Engineering, Inc. (TKE) will evaluate potential service areas, wastewater flow rates, trunk sewer and force main alignments, analyze lift station capacity, and consider other preliminary design criteria needed to identify the preferred alignment of the proposed sewer conveyance system.

I.3 Design Criteria

This TM will follow the design criteria as presented in the 2007 Wastewater System Comprehensive Master Plan (WWSCMP) and in conjunction with 2012 MSWD's Developer/Contractor Handbook and Guidelines for Design and Construction of Water and Sewer Facilities (DCH&G). Tables I-1 through I-6 identifies the various design criteria required by MWSD.

Peak Dry Weather Flow (**PDWF**) is estimated from Average Day Dry Weather Flow (**ADWF**) rates and Peak Dry Weather Flow Factors (**PF**) based on land use as presented in **Table I-1, Design Flow Criteria**. MSWD has confirmed these flow rates through flow monitoring programs.

Table I-1
Design Flow Criteria

Land Use	Average Daily Flow	Units	Peak Factor
Residential (EDU)	200	gpd/EDU	2.5
Commercial / Industrial	2,000	gpd/acre	1.33
Public Uses	1,000	gpd/acre	1.33
Schools	500	gpd/acre	2.0

Source: MSWD Developer/Contractor Handbook and Guidelines for Design and Construction of Water and Sewer Facilities, Sept. 2012.

Gravity sewer main sizing is determined on the basis of design flow rates (ADWF, and MSWD Peak Wet Weather Flow (**PWWF**)), Manning's roughness coefficient, and the hydraulic depth ratio (d/D). These criteria are provided in **Table I-2, Sewer Main Sizing Criteria.**

Table I-2
Sewer Main Sizing Criteria

Pipe Diameter	Manning's Roughness Coefficient	PWWF D/d Max
8" to 12"	0.013	0.50
15" or Greater	0.013	0.75

Source: MSWD Developer/Contractor Handbook and Guidelines for Design and Construction of Water and Sewer Facilities, Sept. 2012.

Piping material along with velocity criteria for sewer lines, force mains, and inverted siphons are provided in **Table I-3, Material and Velocity Criteria.** Additionally, head losses for force mains shall be approximately 5 feet per 1,000 feet of force main or less.

Table I-3
Material and Velocity Criteria

Type	Material	Minimum (fps)	Desired (fps)	Maximum (fps)
Sewer Pipelines	VCP	2	3	10
Force Mains	DIP	3	-	5
Inverted Siphons	VCP	3	-	5

Source: MSWD Developer/Contractor Handbook and Guidelines for Design and Construction of Water and Sewer Facilities, Sept. 2012.

Maximum allowable sewer manhole spacing is 350 feet with a drop of 0.1 feet for straight runs and bends up to 45° and 0.2 feet for 90° bends shall occur across manholes. Slope design criteria for gravity sewer lines are provided below in **Table I-4, Gravity Sewer Main Slope Criteria.**

Table I-4
Gravity Sewer Main Slope Criteria

Pipe Diameter (in.)	Extreme Min. Slope (ft/ft)	Min. Slope (ft/ft)	Preferred Min. Slope (ft/ft)	Max Slope (ft/ft)
	V = 1.5 fps	V = 2 fps	V = 3 fps	V = 10 fps
8	0.0020	0.0040	0.0076	0.086
10	0.0016	0.0028	0.0060	0.061
12	0.0012	0.0020	0.0044	0.049
15	0.0008	0.0016	0.0036	0.036
18	0.0008	0.0012	0.0024	0.029
21	0.0006	0.0010	0.0020	0.024
24	0.0004	0.0008	0.0016	0.020
27	0.0004	0.0008	0.0016	0.017
30	0.0003	0.0006	0.0012	0.015
33	0.0003	0.0006	0.0012	0.012

Source: MSWD Developer/Contractor Handbook and Guidelines for Design and Construction of Water and Sewer Facilities, Sept. 2012.

Minimum and maximum depths of cover parameters are used as general guidelines when designing sewer facilities. Special design and approval are required by MSWD for depths of cover outside of the prescribed depths as shown below in **Table I-5, Depth Criteria.**

Table I-5
Depth Criteria

Typical Cover		Typical Depth to Flowline
Min	Max	
5 ft	20 ft	7.5 ft

Source: MSWD Developer/Contractor Handbook and Guidelines for Design and Construction of Water and Sewer Facilities, Sept. 2012.

I.4 Study Area Limits

This TM will focus on the area tributary to the WVWRF, known as the "WVWRF Service Area." This area is within MSWD's service area and is generally located west and east of Little Morongo Road and north of Two Bunch Palms Trail, as shown in **Exhibit A, Study Area**.

Ultimately all flows generated in WVWRF service area are slated to be conveyed to the WVWRF. Initially, it is understood that the conveyance system will deliver existing wastewater flows from the existing MSWD GQPP Areas L and M1 at the Dos Palmas Lift Station (DPLS) along with the near term wastewater flows of the MSWD GQPP Area M2, and other MSWD identified near term developments. These existing sewered areas and near term developments will be discussed in further detail below.

Additionally, this TM will investigate the potential for a bypass system that would allow MSWD the ability to bypass the Horton Wastewater Treatment Plant (HWWTP) and divert the wastewater flows into the WVWRF Sewer Conveyance System for operational flexibility and emergency purposes.

I.5 Initial Flows

The initial flows that will be conveyed to the WVWRF will come from the identified existing and near term sewered areas. These areas are shown in **Exhibit A, Study Area**.

The existing MSWD GQPP sewered Areas L and Area M1 currently flow to the DPLS where they are conveyed to the HWWTP. This TM will allocate these flows to the WVWRF. The existing and ultimate ADF's from these existing areas were established in a previous Technical Memorandum[1].

In addition, this TM considers the following near term areas: GQPP Area M2, DHS Coalition, DHS 109, and the Palm Springs Business Park (aka Coachillin') as contributing initial flow areas which will be conveyed to the WVWRF.

Table I-5, Existing and Near Term Flows, shown below establishes design flow rates of these initial flow areas in accordance with MSWD's design criteria.

[1] MSWD, Regional Wastewater Program Flow Projections Technical Memorandum, Nov. 2017.

Table I-5
Existing and Near Term Flows

Development Name	Area (Acres)	Developed Density	Peak Factor	Average Day Dry Weather Flow (ADWF) MGD
Existing Flows				
Area L & Area M1 (*)	-	-	2.50	0.195
Subtotal				**0.195**
Near Term Flows				
Area M2	-	-	2.50	0.081
DHS Coalition	122	30%	1.33	0.073
DHS 109	122	15%	1.33	0.037
P.S. Business Park (Coachillin)	142	37%	1.33	0.105
Subtotal				**0.296**
			Total:	**0.49**

(*) 2017 and 2018 Recorded Existing Lift Station Wastewater Flows

I.6 Ultimate Wastewater Flows

As shown on the Exhibit A, MSWD consists of two Sewershed:

- Horton Sewershed (shown in blue color)
- WVWRP Sewershed (shown in green color)

The ultimate flows that will be conveyed to the WVWRF are calculated in accordance with the 2007 MSWD Wastewater System Comprehensive Master Plan (2007 WSCMP or SMP). The 2007 SMP developed sub-basin (SB) areas with corresponding ultimate average day dry weather wastewater flows for each of these areas and assigned to the existing or proposed interceptors at designated collection points. **Exhibit A, Study Area** shows these SB areas and average day dry weather wastewater flows throughout MSWD's service area. The wastewater from Horton's Sewershed (SB's shown in blue color) will be conveyed to the existing HWWTP and wastewater from WVWRP Sewershed (SB's show in green color) will be conveyed to the future WVWRF.

Note: SB-6 will transition from the HWWTP to the WVWRF at the 6-year mark after the completion of the WVWRF construction[2].

[2] MSWD, Regional Wastewater Program Flow Projections Technical Memorandum, Nov. 2017, Table 5.

These SB areas from the 2007 SMP were developed at an ultimate build out flow scenario utilizing data from the Desert Hot Springs and Riverside County land use plans. The ultimate flows were calculated using average day dry weather unit flow values, which were peaked at an average rate of 2.4[3].

Table I-6, West Valley Ultimate Flows, establishes ultimate design wastewater flow rates (ADWF) of these SB areas used herein this Table I-6.

[3] MSWD, Wastewater System Comprehensive Master Plan, April 2007, p. 9-1.

Table I-6
West Valley Ultimate Average Daily Wastewater Flows

Subbasin Area	Ultimate ADWF (MGD)
SB-1	0.47
SB-2	0.74
SB-3	0.46
SB-4	0.58
SB-5	0.46
SB-6	0.20
SB-8	0.51
SB-9	0.40
SB-10	0.44
SB-11	0.52
SB-15	0.17
SB-16	0.41
SB-17	0.40
SB-18	0.54
SB-23	0.31
SB-24	0.04
SB-25	0.32
SB-26	0.54
SB-27	0.77
SB-28	0.46
SB-29	0.53
SB-30	0.39
SB-31	0.03
SB-32	0.29
SB-33	0.02
SB-34	0.26
SB-35	0.09
SB-36	0.02
SB-37	3.30
SB-38	0.73
SB-39	0.31
SB-40	0.01
SB-41	0.01
SB-42	0.05
Total:	**14.78**

I.7 Dos Palmas Lift Station Design Capacity

The Dos Palmas Lift Station (DPLS) is located on north side of Dillon Road near the intersection with Ave Manzana. The DPLS is a submersible pump station with two (2) 60 HP ESSCO pumps rated at 700 GPM (1 MGD) and total dynamic head (TDH) of 133 feet. Therefore, the pump station design capacity is 0.35 MGD ADWF and 1.0 MGD of PWWF as presented in Table I-7.

Table I-7
Design Capacity of DPLS

	Q (PWWF) gpm	Q (PWWF) MGD	Q (ADWF) MGD
DPLS Capacity	700	1.0	0.35

I.8 Alternatives Considered

This TM will consider four (4) conveyance alternative solutions. The conveyance alternatives are selected to meet MSWD's objective of conveying wastewater to the future WVWRF. These alternatives will examine ultimate service areas, calculated design flows for existing and ultimate development from said areas, existing ground surface slopes, and required diameters for proposed sewers and/or force mains ensuring adequate cleaning velocities at minimum flows along with reliable conveyance at ultimate flows. In addition, the objective of the flow diversion alternative is to consider the potential to divert flows from the HWWTP to the DPLS by potentially converting the existing force main to a gravity sewer main.

These four (4) alternatives can be broken down in to two (2) sub-categories, as listed below, and will be discussed further in this TM.

Gravity Alternatives:
Alternative 1: Cross Country Trunk Sewer
Alternative 2: Dillon and Little Morongo Deep Trunk Sewer

Pumped Alternatives:
Alternative 3: Dillon Force Main and Little Morongo Trunk Sewer
Alternative 4: Dillon Force Main/Trunk Sewer and Little Morongo Trunk Sewer

Flow Diversion:
Horton Flow Diversion Analysis

I.9 Additional Appurtenances on the Proposed Force Main

I.9.1 Air and Vacuum Release Valves

Air and vacuum release valves are used in wastewater force mains to prevent formation of vacuums and/or to release trapped air in the force mains. Air and vacuum release valves are extremely high maintenance items, especially in the high head systems; options Alternative No. 3 and Alternative No. 4 would be considered as high head sewer systems.

Although, the proposed force main has been designed without high points, two (2) of the proposed alignments are pretty long, therefore, they will require a minimum of two (2) or three (3) air/vacuum release valves.

Preferably, these valves should be located in the street right-of-way. Costs of installation of the air/vacuum valves are included in the capital cost for both of the proposed force main alignments.

I.9.2 Check Valve and Vault

Two (2) of the proposed alignments are pretty long (over 1 mile), both of them use good size diameter pipes and each contain thousands of gallons of raw sewage.

Alternative No. 3: the approximate length of the 14-inch diameter force main is 10,192 feet and would contain approximately 81,000 gallons of raw sewage.

Alternative No. 4: the approximate length of the proposed 10-inch diameter dual force mains is 7,531 feet each. Each of the force main would contain an approximate 30,800 gallons of raw sewage (for a total of 61,600 gallons for both force main). In the event of pipe failure or the necessity to drain the proposed force main, very extensive work would be required to deal with the problem.

TKE proposes to install an additional check valve in a precast vault. The vault would be equipped with the check valve and shut off valve. Preferably, these valves should be installed around 2,000 feet away from the existing lift station and located in the street right-of-way. The benefit of using an additional check valve is specified below:

- Check valves would prevent reverse flow, therefore it would minimize the spill volume of the raw sewage.
- Check valve would also greatly mitigate water hammer of the proposed system.

1.0 Alternative 1: Cross Country Trunk Sewer

Alternative 1: Cross Country Trunk Sewer (Alt. 1), shown in **Exhibit 1.0, Alternative 1,** proposes to convey wastewater flows from the DPLS to the WVWRF via a gravity trunk sewer. The alignment of Alt. 1 generally travels south and west following the existing topography with the goal of reaching the WVWRF without relying on a lift station or having to "buck grade." This creates an alignment that is partially within the public right-of-way and partially in open areas which would require easement acquisition. To meet this criteria, portions of the Alt. 1 alignment would also require construction through an environmentally sensitive area.

1.1 Conveyance Reaches

Alt. 1 will have two reaches within its alignment as shown in **Exhibit 1.0, Alternative 1**. The first reach (R1) will run "cross country" from the DPLS to Little Morongo Road at 19th Ave. The second reach (R2) will run from the end of R1 to the headworks at the WVWRF. The types and lengths of each Alt. 1 reach are shown below in **Table 1-1, Alternative 1 Reaches**.

Table 1-1
Alternative 1 Reaches

Reach	Type	Length (ft.)
R1	Gravity Sewer	13,890
R2	Gravity Sewer	950
	Total:	**14,840**

1.2 Initial Flows

As mentioned above in Section I.5, Initial Flows, the initial wastewater flows are generated from the existing and near term developments. These initial flows are allocated to their appropriate reaches based upon existing MSWD sewer facilities and are shown below in **Table 1-2, Alternative 1 Initial Flows.** These initial flows are used to calculate gravity sewer main sizing while maintaining MSWD's minimum 2 fps cleaning velocity.

Table 1-2
Alternative 1 Initial Wastewater Flows

Reach	Area	Initial Flows (ADWF)
		MGD
R1	DHS Coalition	0.073
	DHS 109	0.037
	Area L & Area M1	0.195
	Area M2	0.081
	R1 Total:	**0.39**
R2	Palm Springs Business Park (Coachilin)	0.105
	R2 Total:	**0.11**
	Total:	**0.49**

1.3 Ultimate Flows

As mentioned above in Section I.6, Ultimate Flows, the ultimate wastewater flows are generated from the 2007 SMP SB areas. These ultimate flows are also allocated to their appropriate reaches based upon existing and future MSWD sewer facilities and are shown in **Table 1-3, Alternative 1 Ultimate Flows.** These ultimate flows are used to calculate sewer line sizing while meeting MSWD's hydraulic depth ratio criteria.

Table 1-3
Alternative 1 Ultimate Flows

Reach	Subbasin Area	Ultimate Flows (ADWF)	Peak Factor (PWF)	Ultimate Flows (PWWF)
		MGD		MGD
R1	SB-28	0.46		
	SB-29	0.53		
	SB-30	0.39		
	SB-31	0.03		
	SB-32	0.29		
	SB-33	0.02		
	SB-34	0.26		
	R1 Total:	**1.98**	**2.35**	**4.65**
R2	SB-1	0.47		
	SB-2	0.74		
	SB-3	0.46		
	SB-4	0.58		
	SB-5	0.46		
	SB-6	0.20		
	SB-8	0.51		
	SB-9	0.40		
	SB-10	0.44		
	SB-11	0.52		
	SB-15	0.17		
	SB-16	0.41		
	SB-17	0.40		
	SB-18	0.54		
	SB-23	0.31		
	SB-24	0.04		
	SB-25	0.32		
	SB-26	0.54		
	SB-27	0.77		
	SB-35	0.09		
	SB-36	0.02		
	SB-37	3.30		
	SB-38	0.73		
	SB-39	0.31		
	SB-40	0.01		
	SB-41	0.01		
	SB-42	0.05		
	R2 Total:	**12.80**	**1.90**	**24.32**
	Total:	**14.78**	**1.90**	**28.08**

1.4 Environmental Compliance Issues

Due to the existing ground topography, Alt. 1's alignment forces the sewer to run through an environmentally sensitive area as shown in **Exhibit 1.0, Alternative 1**. Alt. 1 has a number of challenges regarding compliance with the California Environmental Quality Act (CEQA).

The Coachella Valley Conservation Commission (CVCC) controls much of this environmentally sensitive area. The area contains habitat for endangered species, such as the Coachella Valley fringe-toed lizard and mesquite hummocks. All CVCC parcels have what is known as a Restrictive Covenant (RC) on them. Any modification to a RC would require an affirmative vote of CVCC's Board of Directors followed by written concurrence from United States Fish and Wildlife Service and the California Department of Fish and Wildlife. These wildlife agencies will only consider construction of public infrastructure within that area that could absolutely demonstrate necessity of the project. In TKE's communications with CVCC, it was noted that these wildlife agencies will not consider convenience or cost savings to be sufficient justification. It is estimated that even with sufficient justification, project approvals would take anywhere from 12 to 18 months to obtain.

Also, there are cultural resource challenges that would require mitigation. The proposed alignment would likely impact cultural resources as it is a known area containing tribal artifacts and remains. This would require initial archeological investigations prior to construction and rigorous tribal monitoring during construction.

The combined environmental and cultural challenges of Alt. 1's alignment would not only greatly impact the project schedule, but also greatly increase the cost of project delivery. Due to the impact this would have on the WVWRP, it is not recommended to further pursue Alt. 1.

2.0 Alternative 2: Dillon and Little Morongo Deep Trunk Sewer

Alternative 2: Dillon and Little Morongo Deep Trunk Sewer (Alt. 2), shown in **Exhibit 2.0, Alternative 2,** proposes to convey wastewater flows on Dillon Road from the DPLS to Little Morongo Road and then on Little Morongo Road from Dillon Road to the WVWRF via a gravity trunk sewer. This alignment is contained within the public right-of-way. Portions of the existing topography along the alignment create a condition where the sewer line slope is opposite of the existing ground slope.

2.1 Conveyance Reaches

Alt. 2 will have four reaches within its alignment as shown in **Exhibit 2.0, Alternative 2**. The first reach (R1) will run along Dillon Road from the MSWD Manhole No. 102 (MH102) at the DPLS to where the extension of Cactus Drive would meet Dillon Road. The second reach (R2) will run from the end of R1 to the intersection of Dillon Road and Little Morongo Drive. The third reach (R3) will run from the end of R2 along Little Morongo Road to 19th Ave. The fourth reach (R4) will run from the end R3 to the headworks at the WVWRF. The types and lengths of each Alt. 2 reach are show below in **Table 2-1, Alternative 2 Reaches**.

Table 2-1
Alternative 2 Reaches

Reach	Type	Length (ft.)
R1	Gravity Sewer	3,562
R2	Gravity Sewer	6,570
R3	Gravity Sewer	5,175
R4	Gravity Sewer	1,087
	Total:	**16,414**

2.2 Initial Flows

As mentioned above in Section I.5, Initial Flows, the initial wastewater flows are generated from the existing and near term developments. These initial flows are allocated to their appropriate reaches based upon existing MSWD sewer facilities and are shown below in **Table 2-2, Alternative 2 Initial Flows.** The table also includes Peak Wet Weather Flow Factors (PWF) and Ultimate Peak Weather Flow (PWWF).

Table 2-2
Alternative 2 Initial Flows

Reach	Area	Initial Flows (ADWF)
		MGD
R1	Area M2	0.081
	Area M1 & Area L	0.195
	R1 Total:	**0.28**
R2	DHS Coalition	0.073
	DHS 109	0.037
	R2 Total:	**0.11**
R3	-	-
	R3 Total:	**0.00**
R4	Palm Springs Business Park (Coachilin)	0.105
	R4 Total:	**0.11**
	Total:	**0.49**

2.3 Ultimate Flows

As mentioned above in Section I.6, Ultimate Flows, the ultimate wastewater flows are generated from the 2007 SMP SB areas. These ultimate flows are also allocated to their appropriate reaches based upon existing and future MSWD sewer facilities and are shown below in **Table 2-3, Alternative 2 Ultimate Flows.**

Table 2-3
Alternative 2 Ultimate Flows

Reach	Subbasin Area	Ultimate Flows (ADWF)	Peak Factor (PWF)	Ultimate Flows (PWWF)
		MGD		MGD
R1	SB-29	0.53		
	SB-30	0.39		
	SB-31	0.03		
	SB-32	0.29		
	SB-33	0.02		
	SB-34	0.26		
	R1 Total:	**1.52**	**2.40**	**3.65**
R2	SB-28	0.46		
	R2 Total:	**0.46**	**2.70**	**1.24**
R3	SB-1	0.47		
	SB-2	0.74		
	SB-3	0.46		
	SB-4	0.58		
	SB-5	0.46		
	SB-6	0.20		
	SB-8	0.51		
	SB-9	0.40		
	SB-10	0.44		
	SB-11	0.52		
	SB-15	0.17		
	SB-16	0.41		
	SB-17	0.40		
	SB-18	0.54		
	SB-23	0.31		
	SB-24	0.04		
	SB-25	0.32		
	SB-26	0.54		
	SB-27	0.77		
	SB-35	0.09		
	R3 Total:	**8.37**	**2.00**	**16.74**
R4	SB-36	0.02		
	SB-37	3.30		
	SB-38	0.73		
	SB-39	0.31		
	SB-40	0.01		
	SB-41	0.01		
	SB-42	0.05		
	R4 Total:	**4.43**	**2.20**	**9.75**
	Total:	**14.78**	**1.90**	**28.08**

2.4 Combined Flows

The flows generated from the areas tributary to each reach build upon themselves as they progress towards the WVWRF. **Table 2-4, Alternative 2 Combined Flows** shows the combined initial and ultimate **Average Dry Weather** flows in million gallons per day, gallons per minute, and cubic feet per second.

Table 2-4
Alternative 2 Combined Initial & Ultimate ADWF

	Combined Initial Flows (ADWF)			Combined Ultimate Flows (ADWF)		
Reach	**MGD**	**gpm**	**cfs**	**MGD**	**gpm**	**cfs**
R1	0.28	192	0.43	1.52	1,056	2.35
R2	0.39	268	0.60	1.98	1,375	3.06
R3	0.39	268	0.60	10.35	7,188	16.01
R4	0.49	341	0.76	14.78	10,264	22.87

Table 2-4 A, Alternative 2 shows the Wet Weather Peak Factor and combined initial and ultimate Average Dry Weather flows and Peak Wet Weather Wastewater flows in million gallons per day, gallons per minute, and cubic feet per second. These initial and ultimate values will be taking into account to properly size the gravity sewer mains. The downstream sewer depth to diameter rations will be evaluated based on the calculated peak wet weather flow plus the DPLS pump capacities.

Table 2-4 A
Alternative 2 Combined Initial and Ultimate Daily Wastewater Flows

	Initial ADWF			PWF	Combined Initial PWWF		
Reach	**MGD**	**gpm**	**cfs**		**MGD**	**gpm**	**cfs**
R1	0.276	192	0.43		1.01	700	1.56
R2	0.110	76	0.17	3.2	1.36	944	2.10
R3	0.000	-	-		1.36	944	2.10
R4	0.105	73	0.16	3.2	1.70	1,178	2.63
	Combined Ultimate ADWF			**PWF**	**Combined Ultimate PWWF**		
Reach	**MGD**	**gpm**	**cfs**		**MGD**	**gpm**	**cfs**
R1	1.52	1,056	2.35	2.4	3.65	2,533	5.64
R2	1.98	1,375	3.06	2.35	4.65	3,231	7.20
R3	10.35	7,188	16.01	2	20.70	14,375	32.03
R4	14.78	10,264	22.87	1.9	28.08	19,501	43.45

2.5 Preliminary Design

Pipe sizing and slope criteria are developed for each reach considering initial and ultimate flows. Combined ultimate peak wet weather flows are used to calculate sewer line sizing while meeting MSWD's hydraulic depth ratio criteria. Combined initial peak wet weather flows are used to calculate gravity sewer line slope while maintaining MSWD's minimum 2 fps cleaning velocity as shown below in **Table 2-5, Alternative 2 Preliminary Design.**

Table 2-5
Alternative 2 Preliminary Design

Alternative 2: Dillon and Little Morongo Deep Trunk Sewer								
Reach	**PWWF (cfs)**	**Pipe Diameter (in)**	**Min. Slope (ft/ft)**	**Manning's Roughness (n)**	**Max Depth Ratio (d/D)**	**Velocity (fps)**	**Sewer Capacity (cfs)**	**Depth of Pipe @ Inv. El.**
Alt. 2, R1, Initial Flow PWWF	1.56	21	0.0021	0.013	0.32	2.43	1.61	
Alt. 2, R1, Ultimate Flow PWWF	5.64	21	0.0021	0.013	0.68	3.36	5.85	45.71
Alt. 2, R2, Initial Flow PWWF	2.10	24	0.0017	0.013	0.34	2.45	2.27	
Alt. 2, R2, Ultimate Flow PWWF	7.20	24	0.0017	0.013	0.67	3.29	7.36	87.32
Alt. 2, R3, Initial Flow PWWF	2.10	36	0.0045	0.013	0.17	3.47	2.65	
Alt. 2, R3, Ultimate PWWF	32.03	36	0.0045	0.013	0.63	6.90	32.34	12.66
Alt. 2, R4, Initial Flow PWWF	2.63	39	0.0121	0.013	0.15	5.72	4.57	
Alt. 2, R4, Ultimate Flow PWWF	43.45	39	0.0121	0.013	0.49	10.85	43.88	8.0

Due to the existing topography where the existing ground is sloping in the opposite direction of the sewer it is desirable to run the sewer as shallow as allowable for R1 and R2. A preliminary check of sewer depths using reach lengths and slopes is critical. Not including the typical 0.1’ drop across sewer manholes spaced at every 350’ feet, the ending point for R2 at the intersection of Dillon and Little Morongo Road yields a flowline depth of approximately 87’ as shown below in **Table 2-6, Preliminary Depths.**

Table 2-6
Alternative 2 Preliminary Depths

Reach	Start Ground Elevation (ft)	Start Flowline Depth (ft)	End Ground Elevation (ft)	End Flowline Depth (ft)
R1	806.70	17.83	827.96	45.71
R2	827.96	45.71	856.43	87.32

It is not recommended to pursue Alt. 2 due to the extreme depths of R1 and R2. Construction at these depths are not only cost prohibitive, but not practical for construction of facilities within street right-of-way.

3.0 Alternative 3: Dillon Force Main and Little Morongo Trunk Sewer

Alternative 3: Dillon Force Main and Little Morongo Trunk Sewer (Alt. 3), shown in **Exhibit 3.0, Alternative 3,** proposes to convey wastewater flows within the public right-of-way through a force main on Dillon Road from the DPLS to Little Morongo Road and then through a gravity sewer line on Little Morongo Road from Dillon Road to the WVWRF. Flows would be pumped through the force main by the DPLS after modifications are made.

3.1 Conveyance Reaches

Alt. 3 will have three reaches within its alignment as shown in **Exhibit 3.0, Alternative 3**. The first reach (R1) will run along Dillon Road from the DPLS to the intersection of Dillon Road and Little Morongo Drive. The second reach (R2) will run from the end of R1 along Little Morongo Road to 19th Ave. The third reach (R3) will run from the end of R2 to the headworks at the WVWRF. The types and lengths of each Alt. 3 reach are shown below in **Table 3-1, Alternative 3 Reaches**.

Table 3-1
Alternative 3 Reaches

Reach	Type	Length (ft)
R1	Force Main	10,192
R2	Gravity Sewer	5,175
R3	Gravity Sewer	1,087
	Total:	**16,454**

3.2 Initial Flows

As mentioned above in Section I.5, Initial Flows, the initial wastewater flows are generated from the existing and near term developments. These initial flows are allocated to their appropriate reaches based upon existing MSWD sewer facilities and are shown below in **Table 3-2, Alternative 3 Initial Flows.** These initial flows will be taken into account to properly size gravity sewer main while maintaining MSWD's minimum 2 fps cleaning velocity.

Table 3-2
Alternative 3 Initial Flows

Reach	Area	Initial Flows (ADWF)
		MGD
R1	Area M2	0.081
	Area M1 & Area L	0.195
	DHS Coalition	0.073
	DHS 109	0.037
	R1 Total:	**0.39**
R2	-	-
	R3 Total:	**0.00**
R3	Palm Springs Business Park (Coachilin)	0.105
	R3 Total:	**0.11**
	Total:	**0.49**

3.3 Ultimate Flows

As mentioned above in Section I.6, Ultimate Flows, the ultimate wastewater flows are generated from the 2007 SMP SB areas. These ultimate flows are also allocated to their appropriate reaches based upon existing and future MSWD sewer facilities and are shown below in **Table 3-3, Alternative 3 Ultimate Flows.** These ultimate plans will be used to preliminary size the lift station, force main and sewer main while meeting MSWD's hydraulic depth.

Table 3-3
Alternative 3 Ultimate Flows

Reach	Subbasin Area	Ultimate Flows (ADWF)	Peak Factor (PWF)	Ultimate Flows (PWWF)	Peak Factor (PF)	Ultimate Flows (PDWF)
		MGD		MGD		MGD
R1	SB-28	0.46				
	SB-29	0.53				
	SB-30	0.39				
	SB-31	0.03				
	SB-32	0.29				
	SB-33	0.02				
	SB-34	0.26				
	R1 Total:	**1.98**	**2.35**	**4.65**	**1.69**	**3.35**
R2	SB-1	0.47				
	SB-2	0.74				
	SB-3	0.46				
	SB-4	0.58				
	SB-5	0.46				
	SB-6	0.20				
	SB-8	0.51				
	SB-9	0.40				
	SB-10	0.44				
	SB-11	0.52				
	SB-15	0.17				
	SB-16	0.41				
	SB-17	0.40				
	SB-18	0.54				
	SB-23	0.31				
	SB-24	0.04				
	SB-25	0.32				
	SB-26	0.54				
	SB-27	0.77				
	SB-35	0.09				
	R2 Total:	**8.37**	**2.00**	**16.74**	**1.51**	**12.64**
R3	SB-36	0.02				
	SB-37	3.30				
	SB-38	0.73				
	SB-39	0.31				
	SB-40	0.01				
	SB-41	0.01				
	SB-42	0.05				
	R3 Total:	**4.43**	**2.2**	**9.75**	**1.57**	**6.96**
	Total:	**14.78**	**1.9**	**28.08**	**1.4**	**20.69**

3.4 Combined Flows

The flows generated from the area tributary to each reach build upon themselves as they progress towards the WVWRF. **Table 3-4 A and Table 3-4 B, Alternative 3 Combined Flows** shows the combined flows in million gallons per day, gallons per minute, and cubic feet per second. Initial combined flows will be used to calculate gravity sewer main sizing while maintaining MSWD's minimum 2 fps cleaning velocity, and the ultimate combined peak wet weather flows will be used to calculate sewer main sizing while meeting MSWD's hydraulic depth ratio criteria. Additionally, these flows will be used to analyze the existing DPLS volumes and capacity along with sizing for force main piping. The downstream sewer depth to diameter ratio will be evaluated based on the calculated peak wet weather flow plus the DPLS pump capacities.

Table 3-4 A
Alternative 3 Combined Initial and Ultimate Average Dry Weather Wastewater Flows

	Combined Initial Flows (ADWF)			Combined Ultimate Flows (ADWF)		
Reach	**MGD**	**gpm**	**cfs**	**MGD**	**gpm**	**cfs**
R1	0.39	268	0.60	1.98	1,375	3.06
R2	0.39	268	0.60	10.35	7,188	16.01
R3	0.49	341	0.76	14.78	10,264	22.87

Table 3-4 B
Alternative 3 Combined Initial Average and Peak Wet Weather Wastewater Flows

	Combined Initial ADWF			PWF	Combined Initial PWWF		
Reach	**MGD**	**gpm**	**cfs**		**MGD**	**gpm**	**cfs**
R1	0.39	268	0.60		1.58	1,100	2.45
R2	0.00	-	-		1.58	1,100	2.45
R3	0.11	73	0.16	3.2	1.92	1,334	2.97

Table 3-4C
Alternative 3 Combined Ultimate Daily Wastewater Flows

	Combined Ultimate ADWF			PWF	Combined Ultimate PWWF			PF	Combined Ultimate PDWF		
Reach	MGD	gpm	cfs		MGD	gpm	cfs		MGD	gpm	cfs
R1	1.98	1,375	3.06	2.35	4.65	3,231	7.20	1.70	3.37	2,338	5.21
R2	10.35	7,188	16.01	2	20.70	14,375	32.03	1.50	15.53	10,781	24.02
R3	14.78	10,264	22.87	1.9	28.08	19,501	43.45	1.40	20.69	14,369	32.02

3.5 Preliminary Design

Pipe sizing and minimum slope criteria are developed for each reach considering initial and ultimate peak wet weather flows. Combined ultimate peak wet weather flows are used to calculate sewer main sizing while meeting MSWD's hydraulic depth ratio criteria. Combined initial peak wet weather flows are used to calculate gravity sewer line slope while maintaining MSWD's minimum 2 fps cleaning velocity as shown below in **Table 3-5, Alternative 3 Preliminary Design.** Force main sizing will be discussed **in Section 3.7, Lift Station Modification**.

Table 3-5
Alternative 3 Preliminary Design

Alternative 3: Dillon Force Main and Little Morongo Trunk Sewer							
Reach	**PWWF (cfs)**	**Pipe Diameter (in)**	**Min. Slope (ft/ft)**	**Manning's Roughness (n)**	**Max Depth Ratio (d/D)**	**Velocity (fps)**	**Sewer Capacity (cfs)**
Alt. 3, R1, Initial Flow	1.56	Force Main (14 inch D.I.)					
Alt. 3, R1, Ultimate Flow	7.20	Force Main (14 inch D.I.)					
Alt. 3, R2, Initial Flow	1.56	36	0.0033	0.013	0.165	2.97	2.27
Alt. 3, R2, Ultimate Flow	32.03	36	0.0033	0.013	0.75	6.14	34.94
Alt. 3, R3, Initial Flow	3.61	39	0.0038	0.013	0.153	3.21	2.58
Alt. 3, R3, Ultimate Flow	43.45	39	0.0038	0.013	0.75	6.95	46.41

R1 is a force main and will follow existing ground. R2 and R3's gravity sewer lines will be sloping in the same direction as the existing topography. The calculated minimum slopes shown above are verified against the existing ground via preliminary profiling.

3.6 Preliminary Profiles

Preliminary profiling along the R2 and R3 reaches shows that the existing ground slope is approximately 1.86%. This average ground slope is much steeper than the calculated minimum required slopes as shown in **Table 3-5**. Therefore, the pipe slopes for R2 and R3 require additional investigation. Preliminary profiles are prepared in order to determine approximate pipe slopes while meeting MSWD pipe cover criteria. These preliminary profiles can be viewed in **Exhibits 3.1, 3.2, and 3.3**.

It is proposed to begin R1's force main by connecting to the existing DPLS force main at STA 165+66.82 on Dillon Road (see Exhibit 3.3 Detail A). R1 runs along Dillon Road, with a minimum of 42" of cover, where it ends at its high point at the MH-3A manhole found at the intersection of Dillon Road and Little Morongo Road.

In order to avoid unnecessary deep construction, the depth of cover at the beginning of R2 is set at 5' (8' depth to flowline). It was determined that an average slope of approximately 1.99% for R2 with a 36" pipe and an average slope of approximately 1.37% for R3 with a 39" pipe will convey initial and ultimate flows while adhering to MSWD's design criteria. It should be noted that while R2's calculated velocity during ultimate flow is greater than 10 fps, it is still under the maximum slope allowed by MSWD for a 36" pipe (i.e. 2.0%).

Table 3-6, R2 and R3 Gravity Sewer, displays the pipe sizing and average slopes required for the gravity sewer reaches (R2 & R3) along Little Morongo Road. The **Table 3-6** also presents two options of proposed pipe diameters;

- The first option presents larger proposed sewer diameters 36" for R2 and 39" for R3.
- The Second option shows smaller sizes of the pipeline diameters which is 33" for R2 and 36" for R3.

Table 3-6: R2 and R3 Gravity Sewer

Alternative 3: Dillon Force Main and Little Morongo Trunk Sewer										
Reach	ADWF (cfs)	Estimated PDWF (cfs)	MSWDPWWF (cfs)	Pipe Dia. (in)	Average Slope (ft/ft)	Manning's Roughness (n)	Max Depth Ratio (d/D)	Velocity (fps)	Sewer Capacity (cfs)	Depth of Pipe @ Inv. El.
Alt. 3, R1, Initial Flow			2.67	Force Main (14 inch D.I.)						
Alt. 3, R1, Ultimate Flow			7.20	Force Main (14 inch D.I.)						7.8
Alt. 3, R2, Initial Flow			2.67	36	0.0199	0.013	0.11	5.67	2.40	
Alt. 3, R2, Ultimate PWWF			32.03	36	0.0199	0.013	0.40	**12.01**	31.71	
Alt. 3, R2, Ultimate PDWF		24.02		36	0.0199	0.013	0.35	**11.22**	24.74	
Alt. 3, R2, Ultimate ADWF	16.01			36	0.0199	0.013	0.28	9.94	16.11	14.47
Alt. 3, R3, Initial PWWF			2.67	39	0.0137	0.013	0.11	4.96	2.46	
Alt. 3, R3, Ultimate PWWF			43.45	39	0.0137	0.013	0.47	**11.34**	43.45	
Alt. 3, R3, Ultimate PDWF		32.02		39	0.0137	0.013	0.40	**10.51**	32.57	
Alt. 3, R3, Ultimate ADWF	22.87			39	0.0137	0.013	0.33	9.52	22.73	9.32
Alt. 3, R2, Initial Flow				33	0.0199	0.013	0.11	5.35	1.90	
Alt. 3, R2, Ultimate PWWF			32.03	33	0.0199	0.013	0.46	**12.11**	32.30	
Alt. 3, R2, Ultimate PDWF		24.02		33	0.0199	0.013	0.40	**11.33**	25.14	
Alt. 3, R2, Ultimate ADWF	16.01			33	0.0199	0.013	0.32	10.10	16.54	14.47
Alt. 3, R3, Initial PWWF				36	0.0137	0.013	0.11	4.71	1.99	
Alt. 3, R3, Ultimate PWWF			43.45	36	0.0137	0.013	0.54	**11.40**	44.38	
Alt. 3, R3, Ultimate PDWF		32.02		36	0.0137	0.013	0.45	**10.54**	32.52	
Alt. 3, R3, Ultimate ADWF	22.87			36	0.0137	0.013	0.37	9.58	22.78	9.32

Table 3-6A:
R2 and R3 Gravity Sewer Velocity Comparison

Alternative 3: Velocities Comparison for various Pipe Diameters					
Reach	Estimated PDWF (cfs)	MSWD PWWF (cfs)	Pipe Diameter (in)	Max Depth Ratio (d/D)	Velocity (fps)
Alt. 3, R2, Ultimate PWWF		32.03	36	0.40	**12.01**
Alt. 3, R2, Ultimate PDWF	24.02		36	0.35	**11.22**
Alt. 3, R3, Ultimate PWWF		43.45	39	0.47	**11.34**
Alt. 3, R3, Ultimate PDWF	32.02		39	0.40	**10.51**
Alt. 3, R2, Ultimate PWWF		32.03	33	0.46	**12.11**
Alt. 3, R2, Ultimate PDWF	24.02		33	0.40	**11.33**
Alt. 3, R3, Ultimate PWWF		43.45	36	0.54	**11.40**
Alt. 3, R3, Ultimate PDWF	32.02		36	0.45	**10.54**

As shown in Table 3-6 A, the velocities for both options are above 10 ft/s for peak wet weather flows. However, the differences in velocities are negligible; therefore, the smaller pipe diameters has been chosen as a final pipe sizes.

This allows MH-3B to have an adequate invert in elevation of 743.15’ located STA 36+70 at the intersection of Dillon Road and the extension of 19th Ave. R2 ends at this manhole and R3 begins with additional flows generated from R3’s tributary area. R3’s tributary area will flow manhole MH-3B via a 12” sewer shown on the Coachillin’ Off-Site Plans[4]. On the Coachillin plans the 12” sewer ends at MH1 with an INV out Elevation of 748.52. A stretch of sewer approximately 990’ in length will be required to connect the MH1 and MH-3B manholes with an approximate slope of 0.31%, which meets MSWD minimum slope requirements, and most likely 2 additional manholes.

Finally, R3 ends at MH-3C with 5.75’ of cover and a flowline depth of 9.3’. MH-3C will be located directly adjacent to the WVWRF influent pump station where the wastewater flows will enter the treatment process. Further refinements of the reaches, their slopes, depths, and exact locations will transpire during the design of these facilities.

3.7 Lift Station Modification

[4] MSWD, Coachilln’ Off-Site Improvement Plan Phase 2 – 19th Ave, Reference #11461, Sheet 2 of 7.

Currently, the DPLS conveys all Reach 1 (R1) tributary wastewater flows to the HWWTP. The DPLS consists of an 8' diameter wet well housing two 60 HP submersible sewage pumps each capable of 700 gpm at 133' TDH[5]. It is assumed that the DPLS has a design capacity of 1.0 MGD (700 gpm) of peak wet weather flow and 0.35 MGD of average daily flow maintaining one pump as redundant backup capacity.

Currently, the lift station serves Area L and Area M, and existing ADWF is around 0.2 MGD. The peak dry weather factor (**PF**) is estimated at 2.5 (residential area) and peak wet weather factor (**PWF**) is estimated at 3.0 per the2007 SMP. Therefore, with existing ADWF of 0.2 MGD, the current peak dry weather and peak wet weather flows are estimated at 0.5 MGD and 0.6 MGD, respectively. Consequently, there is additional capacity in the existing DPLS. The lift station can accept approximately 0.10 MGD of ADWF.

However, at the present time, the existing pumps do not meet the designed duty point. These pumps operate at 82% of design capacity and deliver approximately 575 gpm (0.83 mgd/1.28 cfs) at 140' of TDH. Therefore, the current capacity of the existing DPLS is around **0.3 mgd of ADWF** and 0.83 mgd of PWWF.

Currently, the District has purchased the Variable Frequencies Drives (VFD) to be installed at the DPLS and operate the existing pumps. The variable frequencies of the VFD's will greatly impact the operation of the existing pumps with the proposed new system.

Alt. 3 proposes to utilize the DPLS by conveying initial flows through the R1 force main as described above. This requires investigation into the capacity of the existing DPLS and the requirements of the R1 force main.

In order to adhere to MSWD's head loss per 1,000 feet requirement, the R1 force main is sized at 14-inch diameter to accommodate ultimate flows. Table 3-7 shows the velocity and head loss per 1,000 feet values for R1 at the existing and ultimate flow rates.

Table 3-7
Force Main Sizing (*)

	Q (GPM)	Pipe Diameter (in)	C Factor	Pipe Length (ft)	Head Loss (ft)	Velocity (fps)	Head Loss per 1,000 ft
Initial	1,100	14	140	10,192	14.9	2.29	1.38
Ultimate	3,231	14	120	10,192	124.74	6.23	10.15

* Note: Only Force Main head loss are included

It is necessary to create a system head curve in order to understand where the existing pumps will operate on their pump curve. A preliminary system head curve

[5] MSWD, Dos Palmas (Areas "L" and "M") Gravity Sewermain, Forcemain, and Lift Station, Sheet 12.

can be created utilizing the existing site facilities and the calculated force main properties. **Table 3-8, Alt. 3 Preliminary System Losses** shows the static losses, dynamic losses, along with the minor losses for the various sizes of valves and fittings in the system.

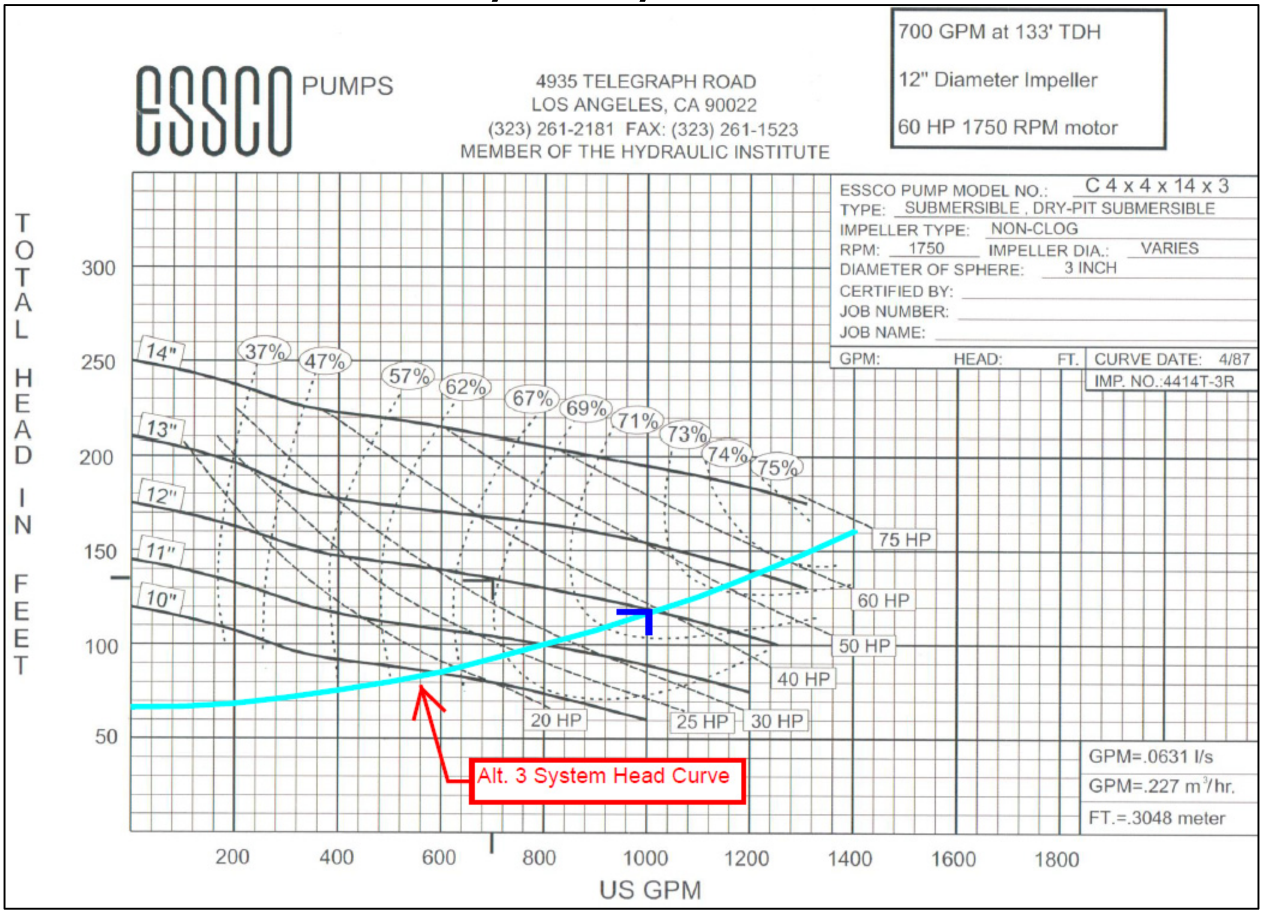
Table 3-8
Alt. 3 Preliminary System Losses

Alt. 3 Losses							
Q	H_s	$H_{f\ FM}$	$H_{f\ Site\ Piping}$	$H_{m\ 4"}$	$H_{m\ 8"}$	$H_{m\ 10"}$	TDH
0	69.00	0.00	0.00	0.00	0.00	0.00	**69**
100	69.00	0.34	0.02	0.22	0.07	0.07	**70**
200	69.00	1.21	0.06	0.78	0.24	0.27	**72**
300	69.00	2.57	0.14	1.66	0.50	0.57	**74**
400	69.00	4.37	0.23	2.82	0.85	0.97	**78**
500	69.00	6.61	0.35	4.26	1.28	1.47	**83**
600	69.00	9.26	0.49	5.97	1.80	2.06	**89**
700	69.00	12.31	0.65	7.95	2.39	2.74	**95**
800	69.00	15.77	0.83	10.17	3.06	3.51	**102**
900	69.00	19.60	1.03	12.65	3.81	4.36	**110**
1000	69.00	23.82	1.25	15.37	4.62	5.30	**119**
1100	69.00	28.42	1.50	18.34	5.52	6.32	**129**
1200	69.00	33.38	1.76	21.54	6.48	7.42	**140**
1300	69.00	38.71	2.04	24.98	7.51	8.61	**151**
1,400	69.00	44.39	2.34	28.65	8.62	9.87	**163**

A system head curve can be created using the data gathered in Table 3-8. This preliminary system head curve displays what the system curve will generally look like. This preliminary system head curve is plotted against the existing pump curve as shown on **Figure 3-1, Preliminary Alt. 3 System Head Curve**. A more detailed curve will need to be created during final design of the force main.

Figure 3-1

Preliminary Alt. 3 System Head Curve



The change in TDH will force the pumps to operate closer to the 1,000 gpm point on the curve, but may run the risk of running off the end of the curve. Throttling the discharge valves can be a viable short term solution that will shift the system head curve up and in turn moving the operating point of the pumps to a more efficient point on the curve. However, this may have an adverse effect on valve condition, pump efficiency and motor horsepower. If unavoidable, an impeller modification or pump replacement may be required. It is conceivable that under the initial flows condition the DPLS pumps may provide adequate service in conveying flows through the R1 force main.

Additionally, a check of the pump cycling times is necessary. Based on the calculations from Table 3-4, the DPLS will receive an initial design flow rate of 1,100 gpm and an ultimate design flow rate of 3,231 gpm. It should be noted that these are design flows which are peak wet weather flows values based on MSWD design criteria, which are much higher than average daily flows.. The existing 8' diameter wet well has a volume of 376 gal/ft with a total operating volume of 1,767 gallons. Traditionally, a wet well is sized where the maximum number of starts per hour occurs at an inflow rate which is half of the design pumping rate[6]. With an approximate pumping rate of 1,000 gpm, it would appear that the lift station will provide adequate service for initial design flows. Nonetheless, a check of the existing wet well capacity against the approximate pumping rate and initial flows using the storage equation shown below is necessary.

Storage Equation[7]: $T = \left(\frac{Vol}{Q_{in}} + \frac{Vol}{Q - Q_{in}}\right)$

$$Where: Vol = 1{,}767\ gal;\ Q_{in} = 471\ gpm;\ Q = 1{,}000\ gpm;\ Solve\ for\ T$$

$$T = 7.09\ min\ (pumping\ cycle\ time\ between\ starts)$$

Preliminary analysis shows that the pump cycling will be too frequent; however, this should be verified against the actual motor manufacturer's recommendations. Typically, the recommended pump cycling time should be roughly six starts per hour[8] (T = 10); however submersible motors cooled by the pumped liquid can usually withstand frequent starts, sometimes much more frequently than 20 starts per hour[9] (T=3). The use of alternators, which alternate lead and lag pump, will have the effect of reducing the required cycling time in half.

While an in depth and comprehensive analysis of the existing DPLS will be required based on the future conveyance system design, the DPLS will likely provide reliable initial service for the West Valley Water Reclamation Conveyance System. System optimization, control strategy, appropriate appurtenances, valving, and abandonments will be detailed in future design efforts.

[6] Hydraulic Institute, Pump Intake Design, 1998, Appendix B.
[7] Hydraulic Institute, Pump Intake Design, 1998, Appendix B., eq. B1.
[8] Jensen Engineered Systems, Pump Station Design Guidelines, p. 27.
[9] Pumping Station Design, G.M. Jones, p.31.21.

4.0 Alternative 4: Dillon Force Main/Sewer and Little Morongo Trunk Sewer

Alternative 4: Dillon Force Main/Sewer and Little Morongo Trunk Sewer (Alt. 4), shown on **Exhibit 4.0, Alternative 4,** proposes to convey wastewater flows to the WVWRF within the public right-of-way through a force main and gravity sewer lines similar to Alt. 3. However, the various reach lengths and corresponding tributary SB areas differ slightly. The intention is to avoid unnecessarily pumping flows generated from SB-28 by allocating the flows to the end of the proposed force main into the gravity sewer rather than allowing them to be routed to the inlet of the DPLS as proposed in Alt. 3. As with Alt. 3, flows would be pumped through a force main by the DPLS after modifications are made. Two Scenarios (No. 1 and No. 2) with variable slopes and single or dual force mains were developed for Alternative No. 4 as follows:

Scenario No. 1: Dual force main and maximum depth of sewer main 24 feet.

Scenario No. 2: Single force main and maximum depth of sewer main 35 feet.

4.1 Conveyance Reaches

Alt. 4 will have four reaches within its alignment as shown in **Exhibit 4.0, Alternative 4**. The first reach (R1) will run along Dillon Road from the DPLS to the intersection of Atlantic Ave. The second reach (R2) will run from the end of R1 to the intersection of Dillon Road and Little Morongo Drive. The third reach (R3) will run from the end of R2 along Little Morongo Road to 19th Ave. The fourth reach (R4) will run from the end of R3 to the headworks at the WVWRF. The types and lengths of each Alt. 4 reach are shown below in **Table 4-1, Alternative 4 Reaches**.

Table 4-1
Alternative 4 Reaches

Reach	Type	Length (ft)
R1	Force Main	7,531
R2	Gravity Sewer	2,661
R3	Gravity Sewer	5,175
R4	Gravity Sewer	1,087
	Total:	**16,454**

4.2 Initial Flows

As mentioned above in Section I.5 Initial Wastewater Flows, the initial wastewater flows are generated from the existing and near term developments. These initial flows are allocated to their appropriate reaches based upon existing MSWD sewer facilities and are shown below in **Table 4-2, Alternative 4 Initial Flows.** These initial flows will be taken into account to properly size gravity sewer trunk mains while maintaining MSWD's minimum 2 fps cleaning velocity.

Table 4-2
Alternative 4 Initial Flows

Reach	Area	Initial Flows (ADWF) MGD
R1	Area M2	0.081
	Area L & Area M1	0.195
	R1 Total:	**0.28**
R2	DHS Coalition	0.073
	DHS 109	0.037
	R2 Total:	**0.11**
R3	-	-
	R3 Total:	**0.00**
R4	Palm Springs Business Park (Coachilin)	0.105
	R4 Total:	**0.11**
	Total:	**0.49**

4.3 Ultimate Flows

As mentioned above in Section I.6, Ultimate Wastewater Flows, the ultimate wastewater flows are generated from the 2007 SMP SB areas. These ultimate flows are also allocated to their appropriate reaches based upon existing and future MSWD sewer facilities and are shown below in **Table 4-3, Alternative 4 Ultimate Flows.** These ultimate flows will be used to properly size the lift station, force main and sewer mains while meeting MSWD's hydraulic depth ratio criteria.

Table 4-3
Alternative 4 Ultimate Flows

Reach	Subbasin Area	Ultimate Flows (ADWF)	Peak Factor (PWF)	Ultimate Flows (PWWF)	Estimated Peak Factor (PF)	Estimated Ultimate Flows (PDWF)
		MGD		MGD		MGD
R1	SB-29	0.53				
	SB-30	0.39				
	SB-31	0.03				
	SB-32	0.29				
	SB-33	0.02				
	SB-34	0.26				
	R1 Total:	**1.52**	**2.40**	**3.65**	**1.75**	**2.66**
R2	SB-28	0.46				
	R2 Total:	**0.46**	**2.70**	**1.24**	**2.02**	**0.93**
R3	SB-1	0.47				
	SB-2	0.74				
	SB-3	0.46				
	SB-4	0.58				
	SB-5	0.46				
	SB-6	0.20				
	SB-8	0.51				
	SB-9	0.40				
	SB-10	0.44				
	SB-11	0.52				
	SB-15	0.17				
	SB-16	0.41				
	SB-17	0.40				
	SB-18	0.54				
	SB-23	0.31				
	SB-24	0.04				
	SB-25	0.32				
	SB-26	0.54				
	SB-27	0.77				
	SB-35	0.09				
	R3 Total:	**8.37**	**2.00**	**16.74**	**1.51**	**12.64**
R4	SB-36	0.02				
	SB-37	3.30				
	SB-38	0.73				
	SB-39	0.31		1.00		1.00
	SB-40	0.01				
	SB-41	0.01				
	SB-42	0.05				
	R4 Total:	**4.43**	**2.20**	**9.75**	**1.57**	**6.96**
	Total:	**14.78**	**1.90**	**28.08**	**1.40**	**20.69**

4.4 Combined Flows

The flows generated from the area tributary to each reach build upon themselves as they progress towards the WVWRF. **Table 4-4A, Table 4-4B, Table 4-4C, Alternative 4,** Combined Initial and Ultimate Average Day Dry Weather Wastewater Flows shows the combined flows in million gallons per day, gallons per minute, and cubic feet per second. Initial combined flows will be taken into account to size the gravity sewer main while maintaining MSWD's minimum 2 fps cleaning velocity and the ultimate combined peak wet weather wastewater flows will be used to calculate sewer trunk main sizing while meeting MSWD's hydraulic depth ratio criteria. Additionally, these flows will be used to analyze the existing DPLS volumes and capacity along with sizing for force main piping.

Table 4-4 A
Alternative 4 Combined Initial & Ultimate Average Day Dry Weather Wastewater Flows

	Combined Initial Flows (ADWF)			Combined Ultimate Flows (ADWF)		
Reach	**MGD**	**gpm**	**cfs**	**MGD**	**gpm**	**cfs**
R1	0.28	192	0.43	1.52	1,056	2.35
R2	0.39	268	0.60	1.98	1,375	3.06
R3	0.39	268	0.60	10.35	7,188	16.01
R4	0.49	341	0.76	14.78	10,264	22.87

Table 4-4 B
Alternative 4 Combined Initial Daily Wastewater Flows

	Initial ADWF			PWF	Combined Initial PWWF		
Reach	**MGD**	**gpm**	**cfs**		**MGD**	**gpm**	**cfs**
R1	0.28	192	0.43		1.58	1,100	2.45
R2	0.11	76	0.17	3.2	1.94	1,344	3.00
R3		-	-		1.94	1,344	3.00
R4	0.11	76	0.17	3.2	2.29	1,589	3.54

Table 4-4 C
Alternative 4 Combined Ultimate Daily Wastewater Flows

	Combined Ultimate ADWF			PWF	Combined Ultimate PWWF			PF	Combined Ultimate PDWF		
Reach	**MGD**	**gpm**	**cfs**		**MGD**	**gpm**	**cfs**		**MGD**	**gpm**	**cfs**
R1	1.52	1,056	2.35	**2.40**	3.65	**2,533**	5.64	**1.75**	2.66	1,847	4.12
R2	1.98	1,375	3.06	**2.35**	4.65	3,231	7.20	**2.35**	4.65	3,231	7.20
R3	10.35	7,188	16.01	**2.00**	20.70	14,375	32.03	**1.50**	15.53	10,781	24.02
R4	14.78	10,264	22.87	**1.90**	28.08	19,501	43.45	**1.40**	20.69	14,369	32.02

4.5 Preliminary Design

Pipe sizing and minimum slope criteria are developed for each reach considering initial and ultimate peak wet weather flows. Combined ultimate flows are used to calculate sewer line sizing while meeting MSWD's hydraulic depth ratio criteria. Per MSWD requirements, the proposed sewer mains shall be designed so that the depth at peak wet weather flow to pipe diameter ratio shall not exceed 0.75 for pipe 15-inches and greater. Combined initial flows are used to calculate gravity sewer line slope while maintaining MSWD's minimum 2 fps cleaning velocity as shown below in **Table 4-5,** Alternative 4 Preliminary Design. The force main sizing will be also discussed in **Section 4.7** of this TM. Two Scenarios with variable slope were developed for Alternative No. 4 and are presented in **Table 4-5.**

Scenario 1 was developed with the proposed slope of the sewer main matching the existing ground surface slope along Little Morongo Rd. However, as shown below in **Table 4-5A**, the calculated velocities of peak ultimate wet weather flows exceeds the maximum velocity allowed by MSWD of 10 fps. Therefore, Scenario 2 was developed with the proposed slope of the sewer main reduced. As shown in **Table 4-5B**, the calculated velocity of ultimate MSWD peak wet weather flow still which exceeds the maximum velocity allowed by MSWD. However, this velocity would only be seen in the distant future during the peak wet weather flow.

As a comparison, the EMWD standard peak dry weather factors were used in this hydraulic analysis. As shown in **Table 4-5A and Table 4-5B**, maximum velocities for ultimate peak dry weather wastewater flows were reduced slightly, and are in closer range to comply with MSWD's maximum allowable velocity.

Table 4-5A Scenario 1
Alternative 4 Preliminary Design

Alternative 4: Dillon Force Main/Sewer and Little Morongo Trunk Sewer (Scenario 1)										
Reach	ADWF (cfs)	Estimated PDWF (cfs)	PWWF (cfs)	Pipe Dia. (in)	Min. Slope (ft/ft)	Manning's Roughness (n)	Max Depth Ratio (d/D)	Velocity (fps)	Sewer Capacity (cfs)	Depth of Pipe @ Inv. El.
Alt. 4, R1, Initial Flow			2.45	Dual Force Main (10 inch D.I.)						
Alt. 4, R1, Ultimate Flow			5.64	Dual Force Main (10 inch D.I.)						7.37
Alt. 4, R2, Initial Flow		3.27	4.09	**24**	0.0017	0.013	0.410	2.71	3.29	
Alt. 4, R2, Ultimate Flow			7.20	**24**	0.0017	0.013	0.66	3.28	7.21	24.12
Alt. 4, R3, Initial Flow			5.73	**33**	0.0167	0.013	0.18	6.64	4.83	
Alt. 4, R3, Ultimate PWWF			32.03	**33**	0.0167	0.013	0.49	11.41	33.02	
Alt. 4, R3, Ultimate PDWD		24.02		**33**	0.0167	0.013	0.41	10.51	24.09	
Alt. 4, R3, Ultimate ADWF	16.01			**33**	0.0167	0.013	0.27	8.42	10.90	14.47
Alt. 4, R4, Initial Flow			7.79	**36**	0.0137	0.013	0.21	6.99	7.55	
Alt. 4, R4, Ultimate PWWF			43.45	**36**	0.0137	0.013	0.55	11.48	45.73	
Alt. 4, R4, Ultimate PDWF		32.02		**36**	0.0137	0.013	0.45	10.54	32.52	
Alt. 4, R4, Ultimate ADWF	22.87			**36**	0.0137	0.013	0.30	8.57	15.29	9.32
Alt. 4, R4, Initial Flow			7.79	**39**	0.0137	0.013	0.21	7.38	9.34	
Alt. 4, R4, Ultimate Flow			43.45	**39**	0.0137	0.013	0.47	11.34	43.45	
Alt. 4, R4, Ultimate Flow		32.02		**39**	0.0137	0.013	0.40	10.51	32.57	
Alt. 4, R4, Ultimate Flow	22.87			**39**	0.0137	0.013	0.27	8.53	15.41	9.32

Table 4-5B Scenario 2
Alternative 4 Preliminary Design

Alternative 4: Dillon Force Main/Sewer and Little Morongo Trunk Sewer (Scenario 2)										
Reach	ADWF (cfs)	Estimated PDWF (cfs)	PWWF (cfs)	Pipe Dia. (in)	Min. Slope (ft/ft)	Manning's Roughness (n)	Max Depth Ratio (d/D)	Velocity (fps)	Sewer Capacity (cfs)	Depth of Pipe @ Inv. El.
Alt. 4, R1, Initial Flow			2.45	Force Main (12 inch D.I.)						
Alt. 4, R1, Ultimate Flow			5.64	Force Main (12 inch D.I.)						
Alt. 4, R2, Initial Flow		3.27	4.09	**24**	0.0060	0.013	0.310	4.41	3.66	
Alt. 4, R2, Ultimate Flow			7.20	**24**	0.0060	0.013	0.50	5.58	8.76	34.8
Alt. 4, R3, Initial Flow			5.73	**33**	0.0150	0.013	0.18	6.30	4.58	
Alt. 4, R3, Ultimate PWWF			32.03	**33**	0.0150	0.013	0.49	10.81	31.29	
Alt. 4, R3, Ultimate PDWD		24.02		**33**	0.0150	0.013	0.41	9.96	22.83	
Alt. 4, R3, Ultimate ADWF	16.01			**33**	0.0150	0.013	0.27	7.98	10.33	16.5
Alt. 4, R4, Initial Flow			7.79	**36**	0.0106	0.013	0.22	6.32	7.29	
Alt. 4, R4, Ultimate PWWF			43.45	**36**	0.0106	0.013	0.58	10.30	43.77	
Alt. 4, R4, Ultimate PDWF		32.02		**36**	0.0106	0.013	0.48	9.55	32.02	
Alt. 4, R4, Ultimate ADWF	22.87			**36**	0.0106	0.013	0.32	7.81	15.23	8.0

Due to the existing topography where the existing ground is sloping in the opposite direction of the sewer, it is desirable to run the sewer as shallow as allowable for R2 while still remaining low enough to accept the SB-28 flows at the intersection of Atlantic Ave and Dillon Road. A preliminary check of sewer depths using reach lengths and slopes is critical. Including the typical 0.1' drop across sewer manholes spaced at every 350' feet, the ending point for R2 at the intersection of Dillon and Little Morongo Road yields a flowline depth of approximately 23' for Scenario 1 and 35' for Scenario 2 as shown below in **Table 4-6A and 4-6B, Preliminary Depths.**

Table 4-6A
Alternative 4, Scenario 1: Preliminary Maximum Depths
At the intersection of Dillon and Little Morongo Road

Reach	Start Ground Elevation (ft)	Start Flowline Depth (ft)	End Ground Elevation (ft)	End Flowline Depth (ft)
R1	Force Main		844.55	7.50
R2	844.55	7.50	856.43	24.79

Table 4-6B
Alternative 4, Scenario 2: Preliminary Maximum Depths
At the intersection of Dillon and Little Morongo Road

Reach	Start Ground Elevation (ft)	Start Flowline Depth (ft)	End Ground Elevation (ft)	End Flowline Depth (ft)
R1	Force Main		844.55	7.50
R2	844.55	7.50	856.43	34.79

At this flowline depth, the pipe cover is approximately 22' for Scenario 1 and 33' for Scenario 2. While this depth is beyond MSWD's typical pipe cover allowance of 20', the alternative should be further explored. Construction costs at this depth may be slightly higher than normal, but a reduction in pumping costs and long term O&M would be experienced.

Since, the cover depth of the proposed sewer main at the intersection of Dillon Road and Little Morongo would be 22' in Scenario 1 and 33' in Scenario 2, the Alternative 4, Scenario 2 has not been chosen as a preferred alternative of this project.

4.6 Preliminary Profiles

Preliminary profiles are prepared in order to determine approximate pipe slopes while meeting MSWD pipe cover criteria. These preliminary profiles can be viewed in **Exhibits 4.0, 4.1.S1&S2, 4.2.S1** through **4.5.S1**, and **4.6.S2** through **4.9.S2.**

For both Scenario 1 and Scenario 2, it is proposed to begin R1's dual force main by connecting to the existing DPLS force main at STA 165+66.82 on Dillon Road (see Exhibit 4.3 Detail A). R1 runs along Dillon Road, with a minimum of 42" of cover, where it ends at its high point at the MH-4A manhole found at the intersection of Atlantic Ave and Dillon Road. MH-4A will not only be the highpoint of the force main, but will also be the inlet location for SB-28 flows. In this alternative, it is anticipated that future wastewater flows from SB-28 will flow from MH No. 26 with an invert elevation of 880.00'[10] in a 12" sewer main at an average slope of approximately 1.6%, which is within MSWD's design criteria.

In order to limit potential deep construction, the depth of cover at the beginning of R2 is set at 6' (7.5' depth to flowline), this approach was applied to both Scenarios. However, the proposed slopes for R2, R3, and R4 runs varies for Scenario 1 and Scenario 2 as presented in **Table 4-5A** and **Table 4-5B**.

Scenario 1:

R2 runs at the minimum slope of 0.17% calculated from **Table 4-5A**. R2 ends at the intersection of Dillon Road and Little Morongo Road. At this point, we have an actual flowline depth of 24' (22' of cover).

Preliminary profiling along the R3 and R4 reaches shows that the existing ground slope is approximately 1.86%. This average ground slope is much steeper than the calculated minimum required slopes from Section 4.5. Therefore, the pipe slopes for R3 and R4 require additional investigation. It was determined that an average slope of approximately 1.67% for R3 with a 33" pipe, and an average slope of approximately 1.37% for R4 with a 36" pipe, will convey initial and ultimate wastewater flows while adhering to MSWD's design criteria. **Table 4-5A** displays the pipe sizing and average slopes required for the gravity sewer reaches (R3 & R4).

Scenario 2:

R2 runs at the minimum slope of 0.60% calculated from **Table 4-5B**. R2 ends at the intersection of Dillon Road and Little Morongo Road. At this point, we have an actual flowline depth of 35' (33' of cover).

Preliminary profiling along the R3 and R4 reaches shows that the existing ground slope is approximately 1.86%. This average ground slope is much steeper than the calculated minimum required slopes from Section 4.5. Therefore, the pipe slopes

[10] MSWD, DHS 109 – Offsite Sewer Improvement Plan, Reference #11509, Sheet 10 of 10

for R3 and R4 require additional investigation. It was determined that an average slope of approximately 1.50% for R3 with a 33" pipe, and an average slope of approximately 1.06% for R4 with a 36" pipe, will convey initial and ultimate flows while adhering to MSWD's design criteria. **Table 4-5B** displays the pipe sizing and average slopes required for the gravity sewer reaches (R3 & R4).

Scenario 1 and Scenario 2:

For both Scenarios (1 and 2), this approach will allow MH-4C to have an adequate invert elevation located at STA 36+70 at the intersection of Little Morongo Road and the extension of 19th Ave. An approximate invert elevation at MH-4C would be 745.10' for Scenario 1, and 743.10 for Scenario 2.

R3 ends at this manhole and R4 begins with additional flows generated from R3's tributary area. R4's tributary area will flow from manhole MH-4C via a 12" sewer shown on the Coachillin' Off-Site Plans[11]. On the Coahillin plans, the 12" sewer ends at MH1 with an INV out Elevation of 748.52. A stretch of sewer approximately 990' in length will be required to connect the MH1 and MH-3B manholes with an approximate slope of 0.31%, which meets MSWD minimum slope requirements, and most likely 2 additional manholes.

Finally, the R4 ends at MH-4D with approximately 5' of cover and an approximate flowline depth of 9'. MH-4D will be located directly adjacent to the WVWRF influent pump station where the wastewater flows will enter treatment process. Further refinements of the reaches, their slopes, depths, and exact locations will transpire during final design of these facilities.

4.7 Lift Station Modification

Currently, the DPLS conveys all Reach 1 (R1) tributary wastewater flows to the HWWTP. The DPLS consists of an 8' diameter wet well housing two 60 HP submersible sewage pumps, each capable of 700 gpm at 133' TDH[12]. Subsequently, the DSPLS has a design capacity of 1.0 mgd (700 gpm) of peak wet weather flow and 0.35 mgd of average daily flow, maintaining one pump as redundant backup capacity.

However, at the present time, the existing pumps do not meet the designed duty point. These pumps operate at 82% of design capacity and deliver approximately 575 gpm (0.83 mgd/1.28 cfs) at 140' of TDH. Therefore, the current capacity of the existing DPLS is around 0.3 mgd of ADWF and 0.83 mgd of PWWF.

Currently, the District has purchased the Variable Frequencies Drives (VFD) to be installed at the DPLS and operate the existing pumps.

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[11] MSWD, Coachilln' Off-Site Improvement Plan Phase 2 – 19th Ave, Reference #11461, Sheet 2 of 7.
[12] MSWD, Dos Palmas (Areas "L" and "M") Gravity Sewermain, Forcemain, and Lift Station, Sheet 12.

Alt. 4 also proposes to utilize the DPLS by conveying initial flows through the R1 force main as described above. This requires investigation into the capacity of the existing DPLS and the requirements of the R1 force main.

In order to adhere to MSWD's head loss per 1,000 feet requirement, the R1 dual force mains are sized to accommodate existing and ultimate flows, as shown in **Table 4-7A.**

Table 4-7B shows the velocity and head loss per 1,000 feet values for R1 at the existing and ultimate flow rates.

Table 4-7A
Estimated Wastewater Flows

Capacity of DPLS	Q (PWWF)	Q (PWWF)	Q (PWWF)	PWF	Q (ADWF)
Flow	**gpm**	**MGD**	**CFS**		**MGD**
Initial	950	1.37	2.12	2.75	0.50
Intermediate	1,265	1.82	2.82	2.75	0.66
Ultimate	2,533	3.65	5.64	2.40	1.52

Table 4-7B
Force Main Sizing

Flows	Q (GPM)	Pipe Dia. (in)	C Factor	Pipe Length (ft)	Head Loss (ft)	Velocity (fps)	Head Loss / 1,000 ft
Initial	950	10	120	7,531	49.21	3.88	6.53
Ultimate/ Force Main	1,250	10	120	7,531	81.76	5.11	10.86

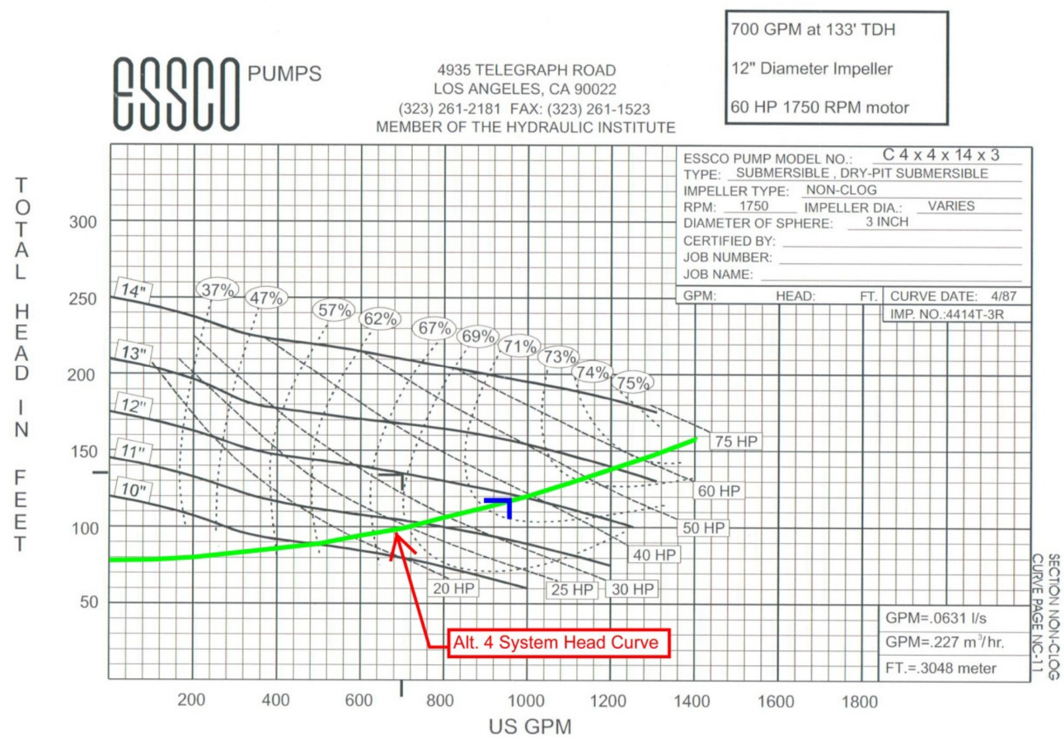
It is necessary to create a system head curve in order to understand where the existing pumps will operate on their pump curve. A preliminary system head curve can be created utilizing the existing site facilities and the calculated force main properties. **Table 4-8, Alt. 4 Preliminary System Losses** shows the static losses, dynamic losses, along with the minor losses for the various sizes of valves and fittings.

Table 4-8
Alt. 4 Preliminary System Losses for Initial Flows

Alt. 4 Losses in each Force Main							
Q	**H_s**	**$H_{f\ FM}$**	**$H_{f\ Site\ Piping}$**	**$H_{m\ 4''}$**	**$H_{m\ 8''}$**	**$H_{m\ 10''}$**	**TDH**
0	56.25	0.00	0.00	0.00	0.00	0.00	**56**
100	56.25	0.89	0.01	0.16	0.04	0.05	**57**
200	56.25	3.21	0.04	0.57	0.14	0.17	**60**
300	56.25	6.80	0.08	1.22	0.31	0.35	**65**
400	56.25	11.57	0.14	2.07	0.52	0.60	**71**
500	56.25	17.49	0.21	3.13	0.79	0.90	**79**
600	56.25	24.51	0.30	4.39	1.11	1.27	**88**
700	56.25	32.59	0.40	5.83	1.47	1.69	**98**
800	56.25	41.72	0.51	7.47	1.88	2.16	**110**
900	56.25	51.88	0.63	9.29	2.34	2.68	**123**
1000	56.25	63.05	0.77	11.29	2.85	3.26	**137**
1100	56.25	75.21	0.92	13.46	3.39	3.89	**153**
1200	56.25	88.34	1.08	15.81	3.99	4.57	**170**
1300	56.25	102.44	1.25	18.34	4.62	5.30	**188**
1400	56.25	117.49	1.44	21.03	5.30	6.08	**208**

A system head curve can be created using the data gathered in Table 4-8. This preliminary system head curve displays what the system curve will generally look like and is plotted against the existing pump curve as shown on **Figure 4-1, Preliminary Alt. 4 System Head Curve**. An adjusted system curve will be created during the final design of the force main.

Figure 4-1
Preliminary Alt. 4 System Head Curve



The change in TDH of the proposed new system will force the pumps to operate closer to the 950 gpm point on the curve. At this point there is the inherent risk of running off the end of the curve. The District is in a process to installing VFD on both existing pumps, therefore, the pumps will be operated at lower frequencies and a new duty point that will shift the system head curve up, and in turn, move the operating point of the pumps to a more efficient point on the curve. At this time, the pump replacement is not anticipated. It is conceivable that under the initial flows condition, the DPLS pumps will provide an adequate service in conveying flows through the R1 force mains without modification of the existing pump station. However, a further evaluation would be required during the final design of the force main.

Additionally, a check of the pump cycling times is necessary. Based on the calculations from **Table 4-4A**, the DPLS will receive an initial average day flow rate of 192 gpm (0.28 mgd) and an ultimate average day flow rate of 1,056 gpm (1.52 mgd). The existing 8’ diameter wet well has volume of 376 gal/ft with a total operating volume of 1,767 gallons for operating range of 4.7 ft. Traditionally, a wet well is sized where the maximum number of starts per hour occurs at an inflow rate which is half of the pumping rate[13]. With an approximate pumping rate of 950 gpm it would appear that the lift station will provide adequate service for initial flows. Nonetheless, a check of the existing wet well capacity against the approximate pumping rate and initial flows using the storage equation shown below is necessary.

Storage Equation[14]: $T = \left(\frac{Vol}{Q_{in}} + \frac{Vol}{Q - Q_{in}}\right)$

$$Where: Vol = 1{,}767\ gal;\ Q_{in} = 192\ gpm;\ Q = 950\ gpm;\ Solve\ for\ T$$

$$T = 11.53\ min\ (pumping\ cycle\ time\ between\ starts)$$

Preliminary analysis shows that the pump cycling will be with recommended frequency of 6 starts per hour for 60 HP pump; however, this will be verified against the actual motor manufacturer's recommendations. Typically the recommended pump cycling time should be roughly six starts per hour[15] (T = 10), however submersible motors cooled by the pumped liquid can usually withstand frequent starts, sometimes much more frequently than 20 starts per hour[16] (T=3). The use of alternators, which alternate pump lead and lag status, can also be employed and would have the effect of reducing the required cycling time in half.

While in depth and comprehensive analysis of the existing DPLS will be required based on the future conveyance system design, with the minor modifications (VFD's employment or pump replacement), the DPLS will be able to provide reliable initial service (existing and interim conditions) for Alt. 4 and the West Valley Water Reclamation Conveyance System. System optimization, control strategy, appropriate appurtenances, valving, and abandonments will be detailed in future design efforts.

[13] Hydraulic Institute, Pump Intake Design, 1998, Appendix B.
[14] Hydraulic Institute, Pump Intake Design, 1998, Appendix B., eq. B1.
[15] Jensen Engineered Systems, Pump Station Design Guidelines, p. 27.
[16] Pumping Station Design, G.M. Jones, p.31.21.

5.0 Horton Flow Diversion

MSWD has identified a desire to have flow diversion capabilities at the HWWTP. The objective of this flow diversion alternative is to consider the potential to divert flows from the HWWTP to the WVWRF Sewer Conveyance System by converting the existing force main to a gravity sewer line. This will afford MSWD the ability to bypass some flows headed to the HWWTP and divert them to the WVWRF via the conveyance system. The intent of this diversion is to give MSWD the capability of flexible operations in the event of scheduled maintenance or an emergency. The desired flow diversion capacity is 0.75 MGD, which is the capacity of the largest treatment train at the HWWTP. **Table 5-1, Diversion Flow** shows the desired flow diversion rate in MGD, gpm, and cfs.

Table 5-1
Diversion Flow

MGD	gpm	cfs
0.750	521	1.16

5.1 Existing Force Main

The existing force main is a 10” diameter C-900 PVC line that is approximately 6,600 feet in length. The general alignment of the existing force main follows Avenida Manzana from Dillon Road at the south end to MSWD Manhole No. 136 (MH136) where the extension of Avenida Manzana would meet Verbena Drive at the north end. MSWD desired to utilize as much of the existing facilities as possible in order to save costs. **Figure 5-1, Existing Force Main North** shows where the existing force main terminates adjacent to the HWWTP at MH136 and where the beginning of the flow diversion alignment will begin at MH135. **Figure 5-1, Existing Force Main South** shows where the existing force main originates at the DPLS and where the force main conversion will terminate near MH103.

Figure 5-1
Existing Force Main North

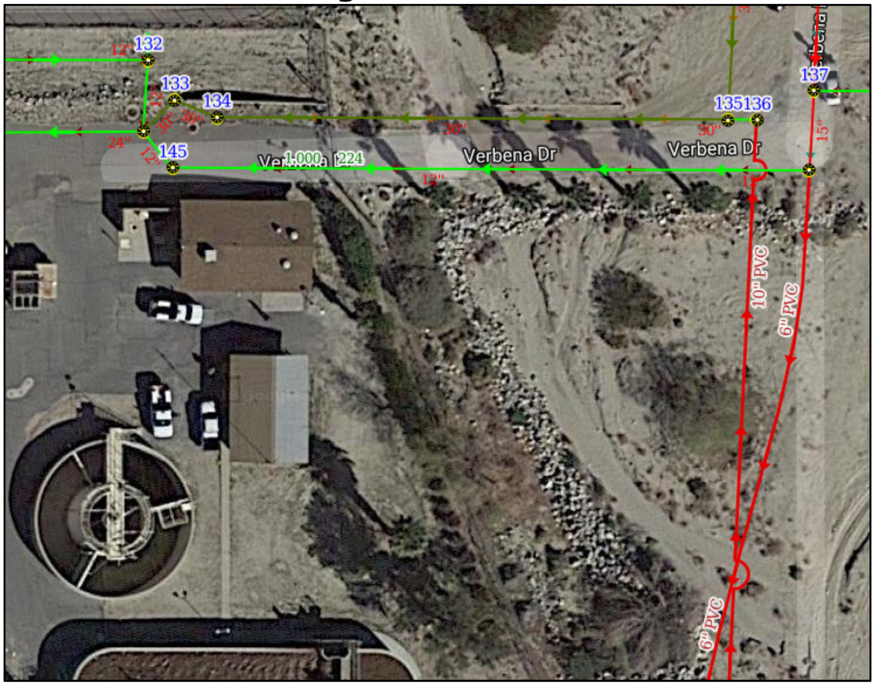
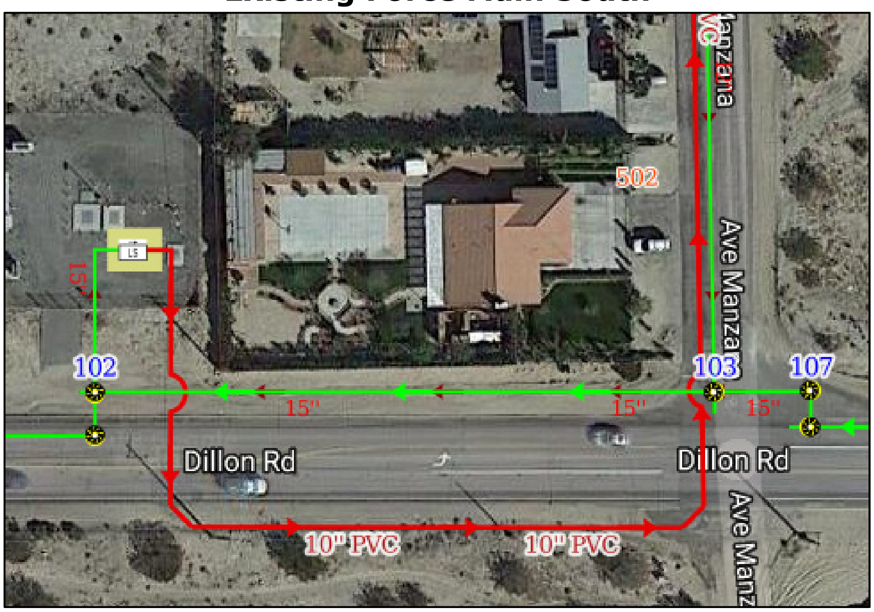


Figure 5-2
Existing Force Main South



5.2 Force Main Capacity

The existing minimum and maximum slopes of the force main are 0.4% and 3.92%[17]. These reaches need to be verified against the desired capacity of 0.75 MGD (116 cfs). **Table 5-2, Force Main Capacity** shows the velocity and depth ratio that the converted force main will experience during flow diversions. This demonstrates that there is adequate capacity in the force main to accommodate flow diversions. It should be noted that while the minimum slope reach has a higher depth ratio than typically allowed for 10" sewer (d/D=0.5), it is not of concern due to the fact that this pipe was originally designed for pressure flow and also does not have laterals along its alignment.

Table 5-2
Force Main Capacity

Alternative 5: Flow Diversion						
Reach	Pipe Diameter (in)	Pipe Slope (ft/ft)	Manning's Roughness (n)	Depth Ratio (d/D)	Velocity (fps)	Sewer Capacity (cfs)
Min Slope	10	0.0040	0.013	0.700	2.84	1.16
Max Slope	10	0.0392	0.013	0.353	6.74	1.16

5.3 North and South Connections

Construction of improvements along with some demolition and abandonments will be required in order to facilitate the conversion of the existing force main. At the north connection, construction of diversion facilities will required at MSWD's MH135 along with a portion of VCP sewer, including manholes, in order to convey flows into the existing force main. **Table 5-3, North Connection Capacity** shows the minimum required pipe slope in order to achieve the desired diversion capacity.

Table 5-3
North Connection Capacity

North Connection						
Reach	Pipe Diameter (in)	Min Pipe Slope (ft/ft)	Manning's Roughness (n)	Depth Ratio (d/D)	Velocity (fps)	Sewer Capacity (cfs)
North Connection Half Full	10	0.0113	0.013	0.500	4.27	1.16
North Connection 3/4 Full	10	0.0034	0.013	0.750	2.66	1.16

[17] MSWD, Dos Palmas (Areas "L" and "M") Gravity Sewermain, Forcemain, and Lift Station, Sheet 3 & 5.

It should be noted that at half full, the minimum required slope is quite a low and would require a great distance in order to intersect the existing force main. It is recommended that this pipe be permitted to flow at ¾ full in order to intersect the existing force main in a much shorter distance and limit the amount of construction required. **Exhibit 5.1, North Connection** shows a conceptual layout of the proposed improvements for the north connection. Curved sewers with a maximum of 2.5 degrees deflection can be utilized in order to reduce the number of manholes required.

At the south connection, construction of additional facilities, including sewer lines and manholes, will also be required in order to covey flows from the existing force main into the existing sewer that flows to the DPLS. Again, MSWD desires to limit the amount of required construction to a little as feasible. **Exhibit 5.2, South Connection** shows a conceptual layout of the proposed improvements for the south connection. Again, curved sewers may be desirable to reduce the number of manholes required.

It is calculated that an approximate slope of 6.1% will be required to connect the existing force main to the existing sewer. **Table 5-4, South Connection Capacity** shows the depth ratio and velocity that will be experienced at the desired diversion flow rate for the south connection.

Table 5-4
South Connection Capacity

South Connection						
Reach	Pipe Diameter (in)	Pipe Slope (ft/ft)	Manning's Roughness (n)	Depth Ratio (d/D)	Velocity (fps)	Sewer Capacity (cfs)
South Connection	10	0.061	0.013	0.314	7.90	1.16

While an in depth and comprehensive design of the proposed facilities is necessary, it is the opinion of this TM that the Horton flow diversion utilizing the existing force main is a viable option for MSWD. Applicable connections, details, and abandonments will be determined in future design efforts.

6.0 Cost Opinions

Alt. 3 and Alt. 4 are considered to be the two most feasible alternatives for conveyance of wastewater flows to the future WVWRF. Preliminary cost opinions for these identified alternatives, along with the Alt 5. flow diversion, have been prepared and are included in **Appendix A, Cost Opinions**. A total project cost summary of the opinions is shown in **Table 6-1, Cost Summary.**

Table 6-1
Cost Summary

Alternative	Cost ($ M)
Alt. 3	$6.3
Alt. 4, Scenario 1 (Dual FM & 24' deep Sewer Main)	$7.24
Alt. 4, Scenario 2 (Single FM & 35' deep Sewer Main)	$7.52
Alt. 5	$0.30

The cost opinions are based on a number of factors. These factors include TKE's experience with similar construction projects, vendor input, current cost trends, and data provided by MSWD. These cost estimates are planning level estimates which are based on the level of detail provided in this plan and are given in 2019 present values.

Construction costs for manholes are established on a $/each cost. Costs for force mains and sewer lines are assumed to be DIP for single FM, PVC for dual FM and VCP for sewer main, are established on a diameter and $/foot cost, and include pavement rehabilitation for the areas required. A budgetary amount has been included for improvements to the DPLS in the event that future design efforts deem it necessary.

Project soft costs are also accounted for in the cost options. A 10% contingency value is added to the project cost. Finally, a 15% value is added to account for administration, construction management, testing and inspection.

7.0 Final Recommendations

Four (4) alignments alternatives have been chosen for this project. Other routes were also evaluated but eliminated due to anticipated construction difficulties associated with the environmental hydraulic or topographic constraints.

The following conclusion and recommendations are made from the analysis and results:

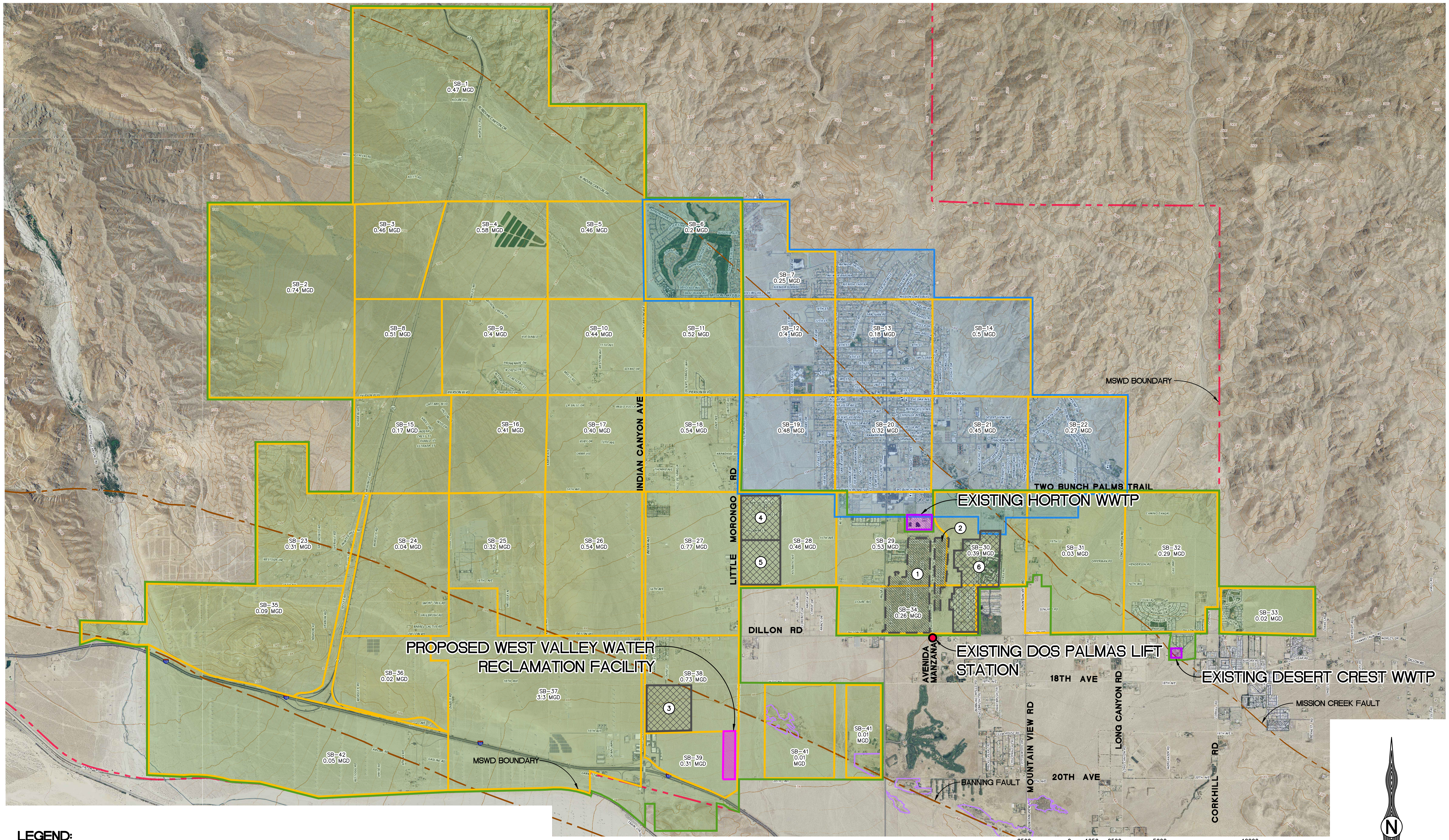
Option No. 1 is not possible due to environmental constraints and due to the existing topography. In addition, this option presents easement acquisition challenges due to its environmentally sensitive area.

Option No. 2 is not feasible due to topography constraints. Portions of the existing topography along the alignment create a condition where the sewer line slope is opposite of the existing ground slope. As a result, the depths of the sewer reaches are extreme and not practical.

Option No. 3 is feasible option. By extending the proposed force main, whether a single or dual force mains are implemented, to Little Morongo Road it avoids the deep gravity sewers encountered with Alternative No. 4. Thus, a significant construction cost savings is presented. However, this option would also have a higher total dynamic head and requires a pump upgrade at the existing lift station to meet the future flows. Additionally, the lift station would serve a larger area and as a result, long term operation and maintenance cost will be substantially higher when compared to Alternative No. 4.

Option No. 4 is also a feasible option. Two Scenarios with variable slopes and single or dual force mains were developed for Alternative No. 4. TKE recommends that the MSWD consider Option No. 4, Scenario 1 with dual force main for further consideration. Installing dual force mains will provide redundancy in the system, better flexibility to accommodate dry and wet weather flows, and greatly mitigate the water hammer effects. Further, by transitioning from force main to gravity sewer at Atlantic Avenue, MSWD can serve a much larger area by gravity and reduce the ultimate flow going to the lift station. However, this alternative also pushed the gravity sewer to a maximum depth of 24 feet, increasing construction costs. Of note, this option would have lower total dynamic head, and when paired with lower future flows, long term operation and maintenance cost will be substantially lower when compared to Alternative No. 3.

In closing, both Alternative Nos. 3 and 4 are viable option to meet MSWD’s current and future wastewater conveyance needs in delivering flows to the proposed WVWRF. The preferred alternative will ultimately be selected by MSWD as the project progresses and funding opportunities present themselves.



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LEGEND:

- WEST VALLEY WRF SERVICE AREA
- HORTON SUBBASIN
- EXISTING WWTP'S (HORTON & DESERT CREST)
- PROPOSED WEST VALLEY WATER RECLAMATION FACILITY
- EXISTING WASTEWATER COLLECTION SYSTEM
- NEAR TERM WASTEWATER COLLECTION SYSTEM
- FAULT LINES
- MESQUITE HUMMOCK AREAS
- SUBBASIN BOUNDARIES
- MISSION SPRINGS WATER DISTRICT BOUNDARY

AREA NAMES:

1. AREA L
2. AREA M1
3. PALM SPRINGS BUSINESS PARK (COACHILLIN')
4. DHS COALITION
5. DHS 109
6. AREA M2

N

2500 0 1250 2500 5000 10000

USE GRAPHIC SCALE WHEN PRINTED AT 11X17

SCALE: 1" = 2,500

TKE ENGINEERING
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RIVERSIDE, CA 92507
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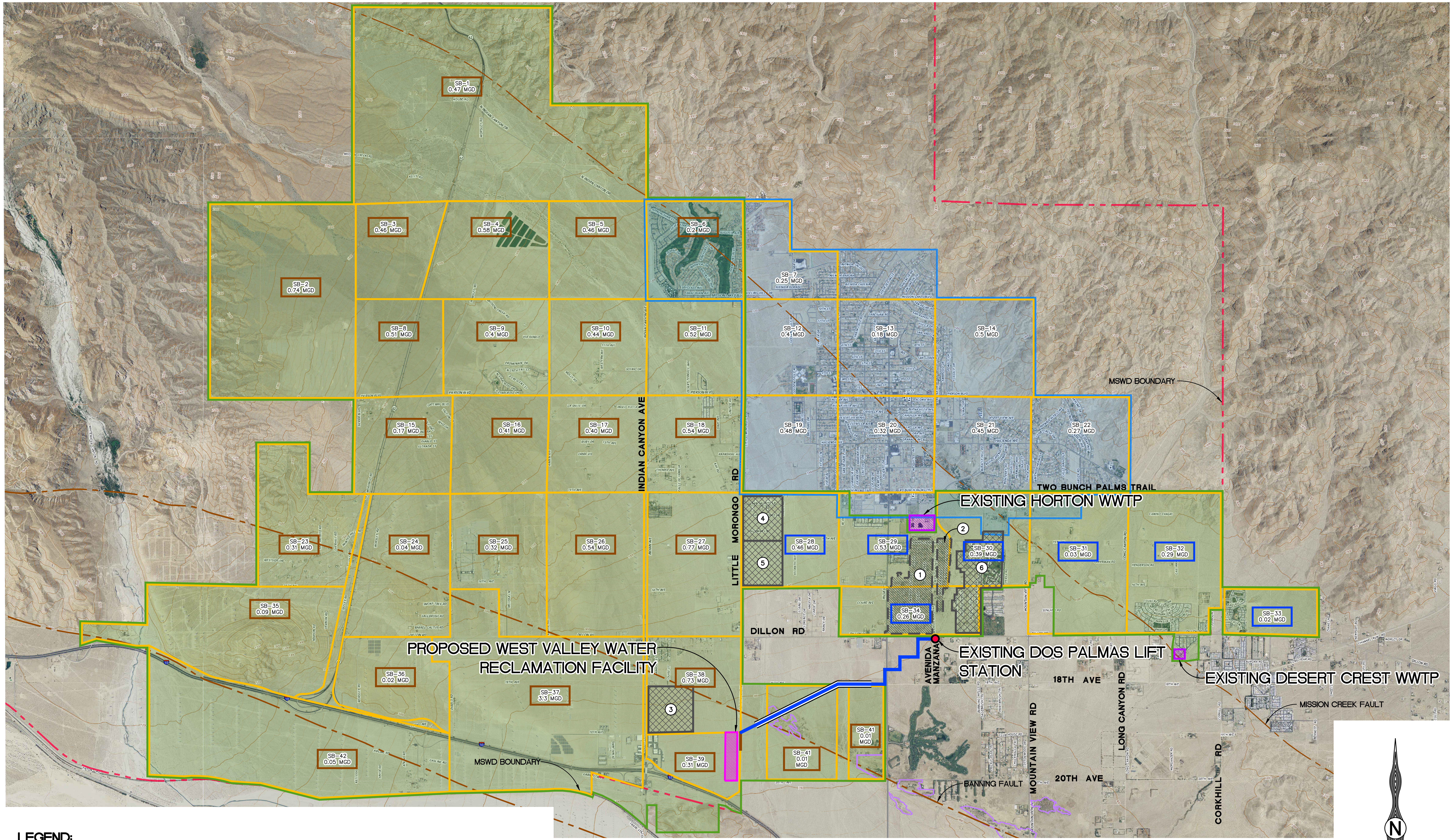
MISSION SPRINGS WATER DISTRICT

WEST VALLEY SEWER CONVEYANCE
TECHNICAL MEMORANDUM
EXHIBIT A: STUDY AREA



LEGEND:

- WEST VALLEY WRF SERVICE AREA BOUNDARY
- HORTON SERVICE AREA
- EXISTING WWTP'S (HORTON & DESERT CREST)
- PROPOSED WEST VALLEY WATER RECLAMATION FACILITY
- EXISTING WASTEWATER COLLECTION SYSTEM
- NEAR TERM WASTEWATER COLLECTION SYSTEM
- FAULT LINES
- MESQUITE HUMMOCK AREAS
- SUBBASIN BOUNDARIES
- MISSION SPRINGS WATER DISTRICT BOUNDARY
- REACH 1 GRAVITY SEWER
- REACH 2 GRAVITY SEWER
- ENVIRONMENTALLY SENSITIVE AREA

AREA NAMES:

1. AREA L
2. AREA M1
3. PALM SPRINGS BUSINESS PARK (COACHILLIN')
4. DHS COALITION
5. DHS 109
6. AREA M2

2500 0 1250 2500 5000 10000

USE GRAPHIC SCALE WHEN PRINTED AT 11X17

SCALE: 1" = 2,500

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WEST VALLEY SEWER CONVEYANCE
TECHNICAL MEMORANDUM
EXHIBIT 1.0: ALTERNATIVE 1

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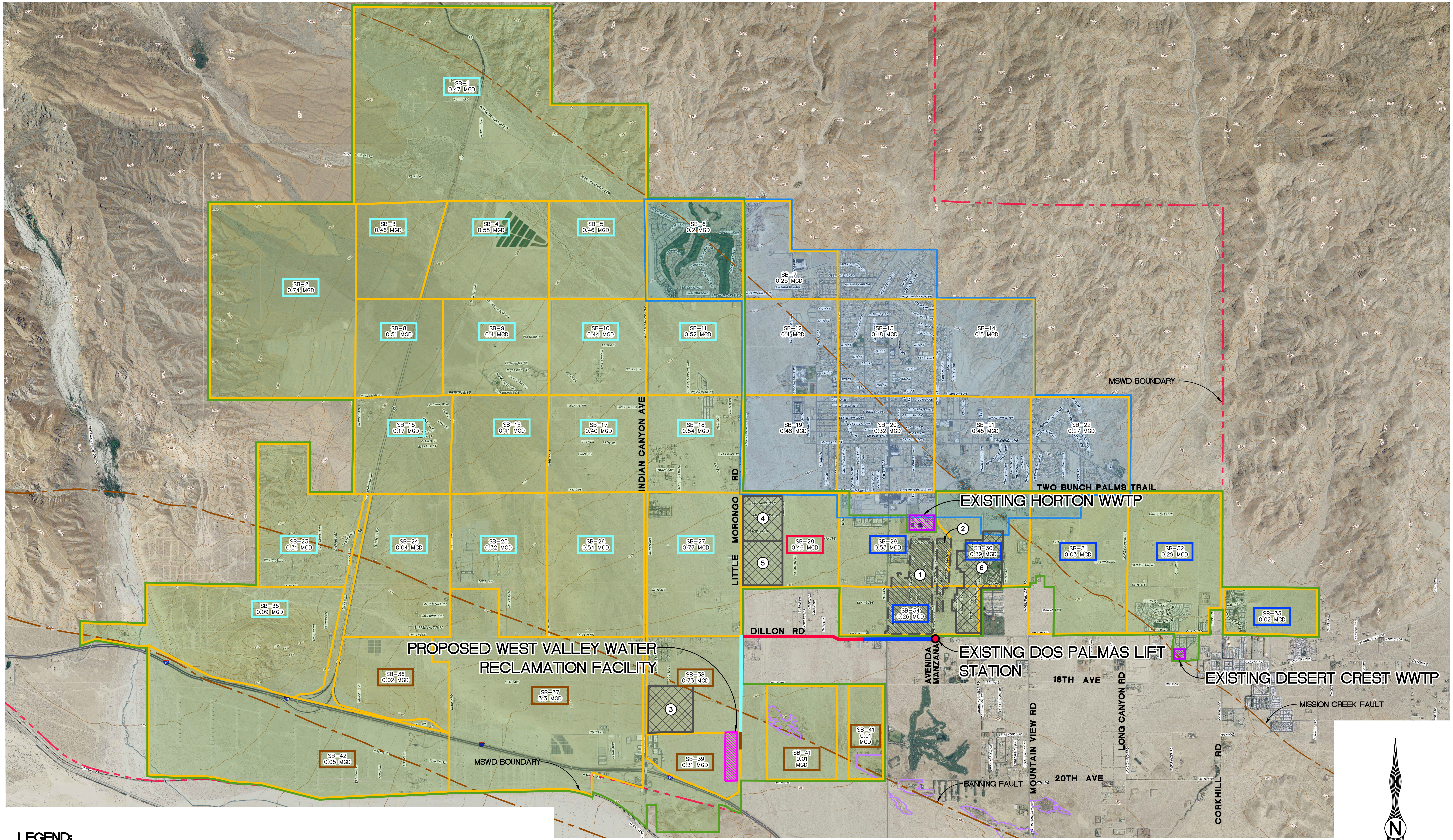

LEGEND:

- WEST VALLEY WRF SERVICE AREA BOUNDARY
- HORTON SERVICE AREA
- EXISTING WWTP'S (HORTON & DESERT CREST)
- PROPOSED WEST VALLEY WATER RECLAMATION FACILITY
- EXISTING WASTEWATER COLLECTION SYSTEM
- NEAR TERM WASTEWATER COLLECTION SYSTEM
- FAULT LINES
- MESQUITE HUMMOCK AREAS
- SUBBASIN BOUNDARIES
- MISSION SPRINGS WATER DISTRICT BOUNDARY
- REACH 1 GRAVITY SEWER
- REACH 2 GRAVITY SEWER
- REACH 3 GRAVITY SEWER
- REACH 4 GRAVITY SEWER

AREA NAMES:

1. AREA L
2. AREA M1
3. PALMS SPRINGS BUSINESS PARK (COACHILLIN')
4. DHS COALITION
5. DHS 109
6. AREA M2

2500 0 1250 2500 5000 10000

USE GRAPHIC SCALE WHEN PRINTED AT 11X17

SCALE: 1" = 2,500

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WEST VALLEY SEWER CONVEYANCE
TECHNICAL MEMORANDUM
EXHIBIT 2.0: ALTERNATIVE 2

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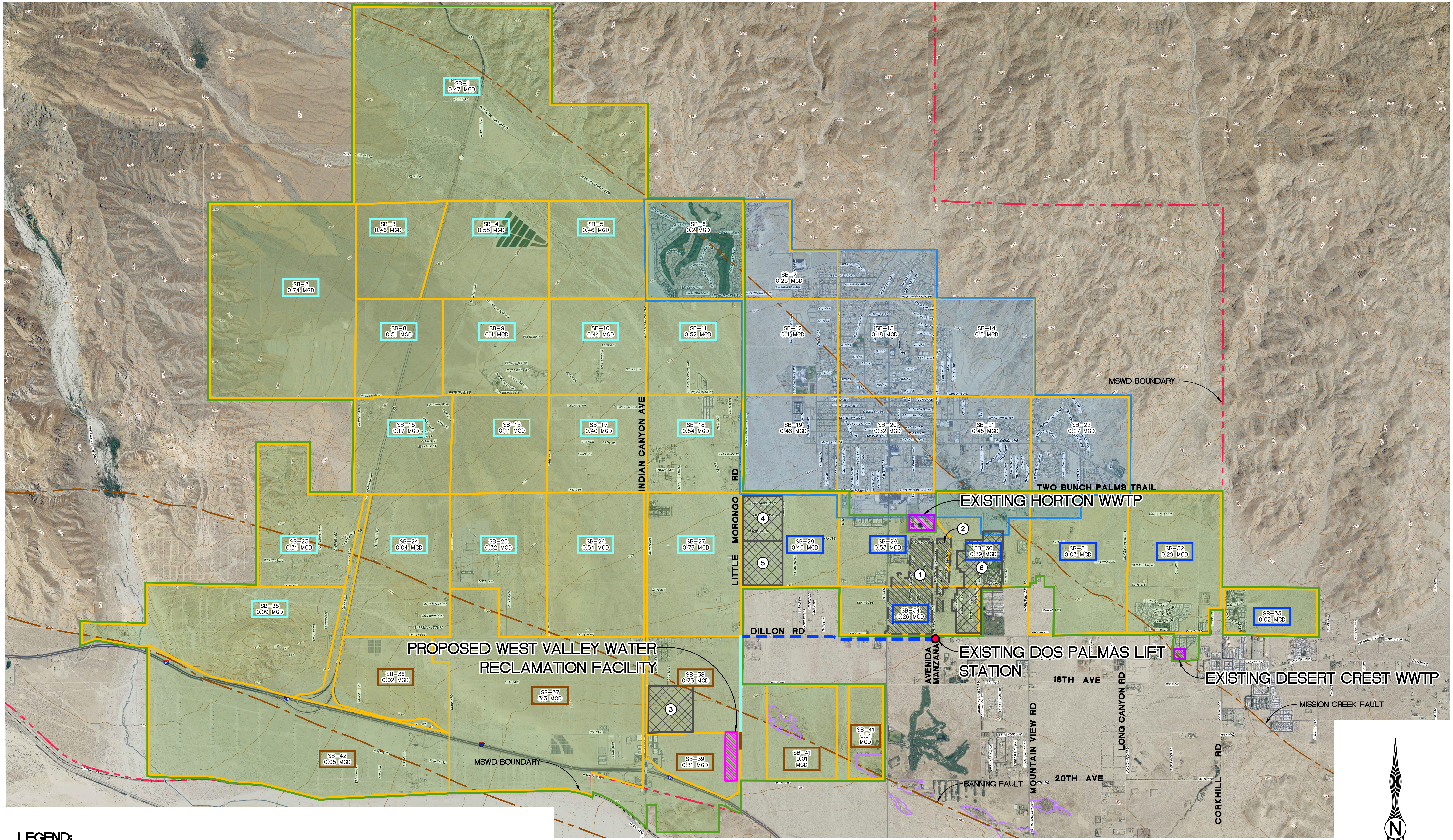

SCALE: 1" = 2,500

2500 0 1250 2500 5000 10000

USE GRAPHIC SCALE WHEN PRINTED AT 11X17

LEGEND:

- WEST VALLEY WRF SERVICE AREA BOUNDARY
- HORTON SERVICE AREA
- EXISTING WWTP'S (HORTON & DESERT CREST)
- PROPOSED WEST VALLEY WATER RECLAMATION FACILITY
- EXISTING WASTEWATER COLLECTION SYSTEM
- NEAR TERM WASTEWATER COLLECTION SYSTEM
- FAULT LINES
- MESQUITE HUMMOCK AREAS
- SUBBASIN BOUNDARIES
- MISSION SPRINGS WATER DISTRICT BOUNDARY
- REACH 1 FORCE MAIN
- REACH 2 GRAVITY SEWER
- REACH 3 GRAVITY SEWER

AREA NAMES:

1. AREA L
2. AREA M1
3. PALM SPRINGS BUSINESS PARK (COACHILLIN')
4. DHS COALITION
5. DHS 109
6. AREA M2

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WEST VALLEY SEWER CONVEYANCE TECHNICAL MEMORANDUM
EXHIBIT 3.0: ALTERNATIVE 3

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GRAPHIC SCALE

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(IN FEET)
1 INCH = 120 FEET

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MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT
EXHIBIT 3.1 – ALTERNATIVE 3 PLAN AND PROFILE

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GRAPHIC SCALE

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(IN FEET)

1 INCH = 120 FEET

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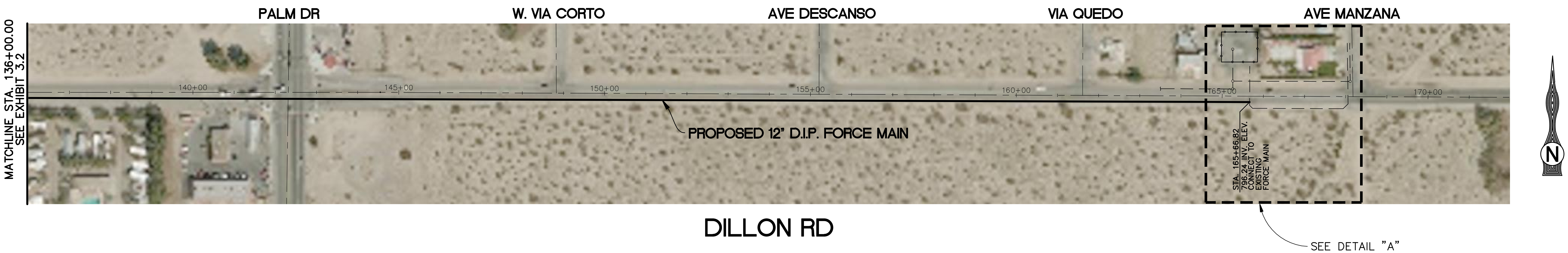
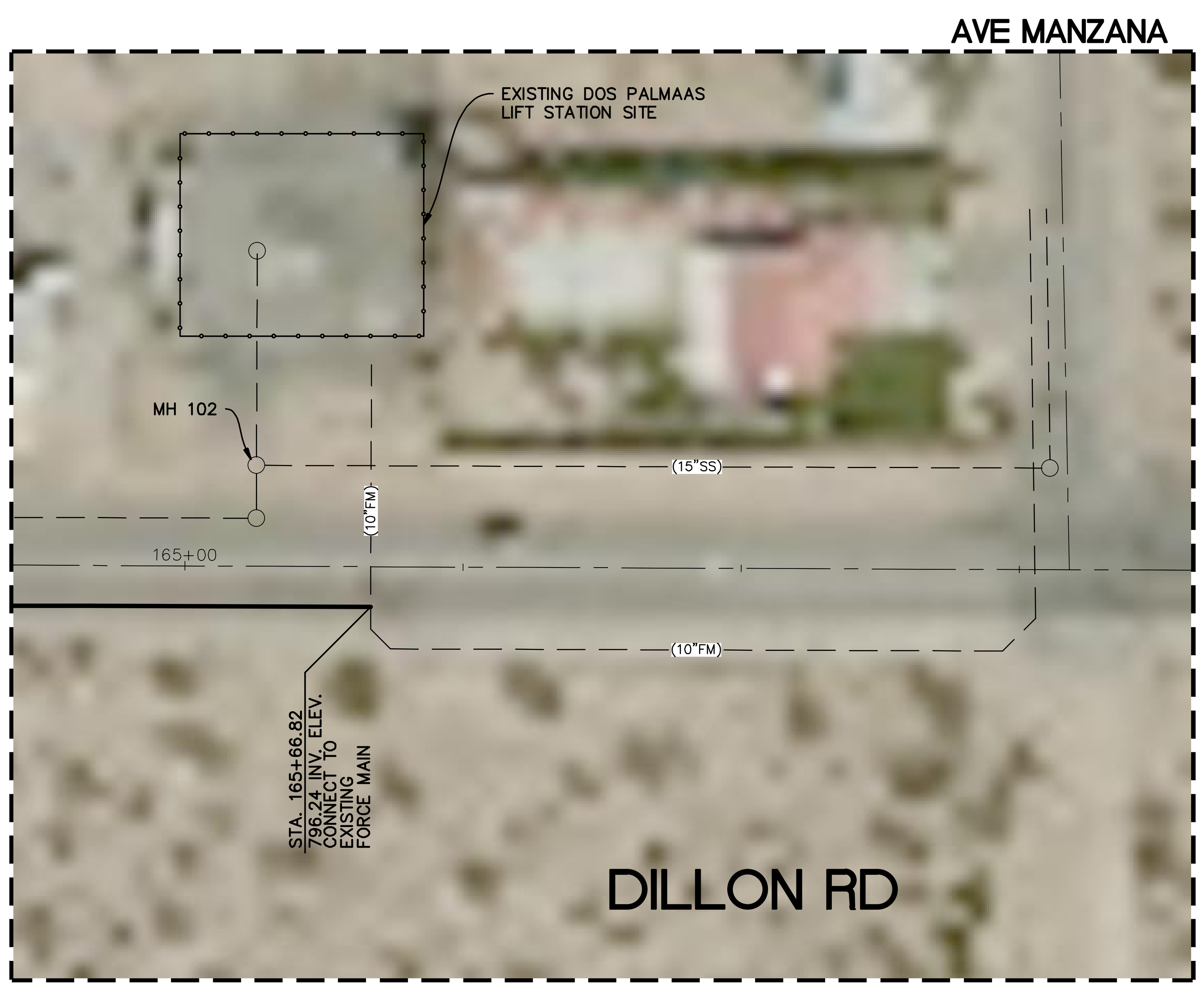
WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT
EXHIBIT 3.2 – ALTERNATIVE 3 PLAN AND PROFILE





DETAIL "A"
1 INCH = 40 FEET

GRAPHIC SCALE
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(IN FEET)
1 INCH = 120 FEET

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WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT
EXHIBIT 3.3 – ALTERNATIVE 3 PLAN AND PROFILE

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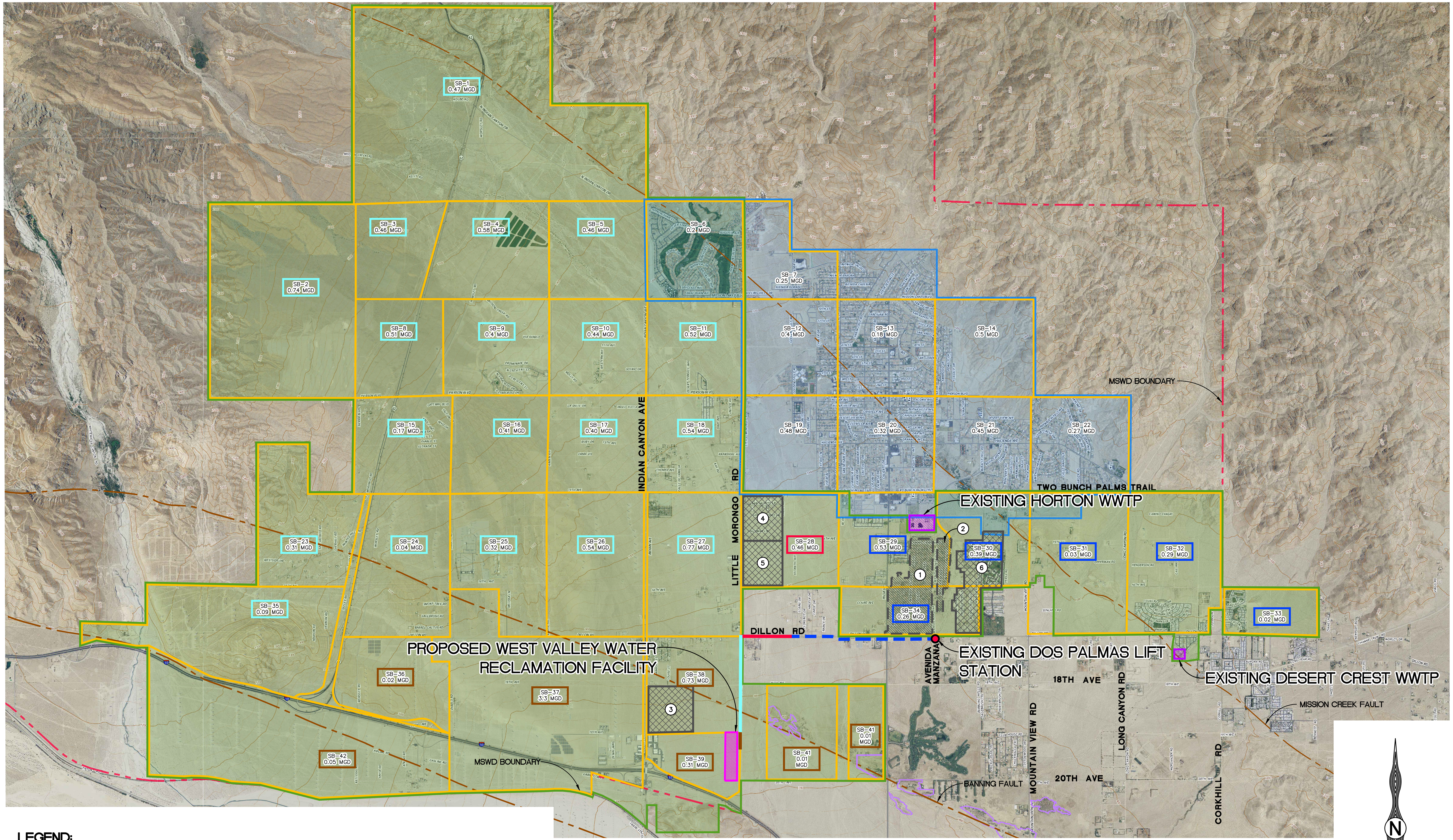

LEGEND:

- WEST VALLEY WRF SERVICE AREA BOUNDARY
- HORTON SERVICE AREA
- EXISTING WWTP'S (HORTON & DESERT CREST)
- PROPOSED WEST VALLEY WATER RECLAMATION FACILITY
- EXISTING WASTEWATER COLLECTION SYSTEM
- NEAR TERM WASTEWATER COLLECTION SYSTEM
- FAULT LINES
- MESQUITE HUMMOCK AREAS
- SUBBASIN BOUNDARIES
- MISSION SPRINGS WATER DISTRICT BOUNDARY
- REACH 1 FORCE MAIN
- REACH 2 GRAVITY SEWER
- REACH 3 GRAVITY SEWER
- REACH 4 GRAVITY SEWER

AREA NAMES:

1. AREA L
2. AREA M1
3. PALM SPRINGS BUSINESS PARK (COACHILLIN')
4. DHS COALITION
5. DHS 109
6. AREA M2

2500 0 1250 2500 5000 10000

USE GRAPHIC SCALE WHEN PRINTED AT 11X17

SCALE: 1" = 2,500

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WEST VALLEY SEWER CONVEYANCE
TECHNICAL MEMORANDUM
EXHIBIT 4.0: ALTERNATIVE 4

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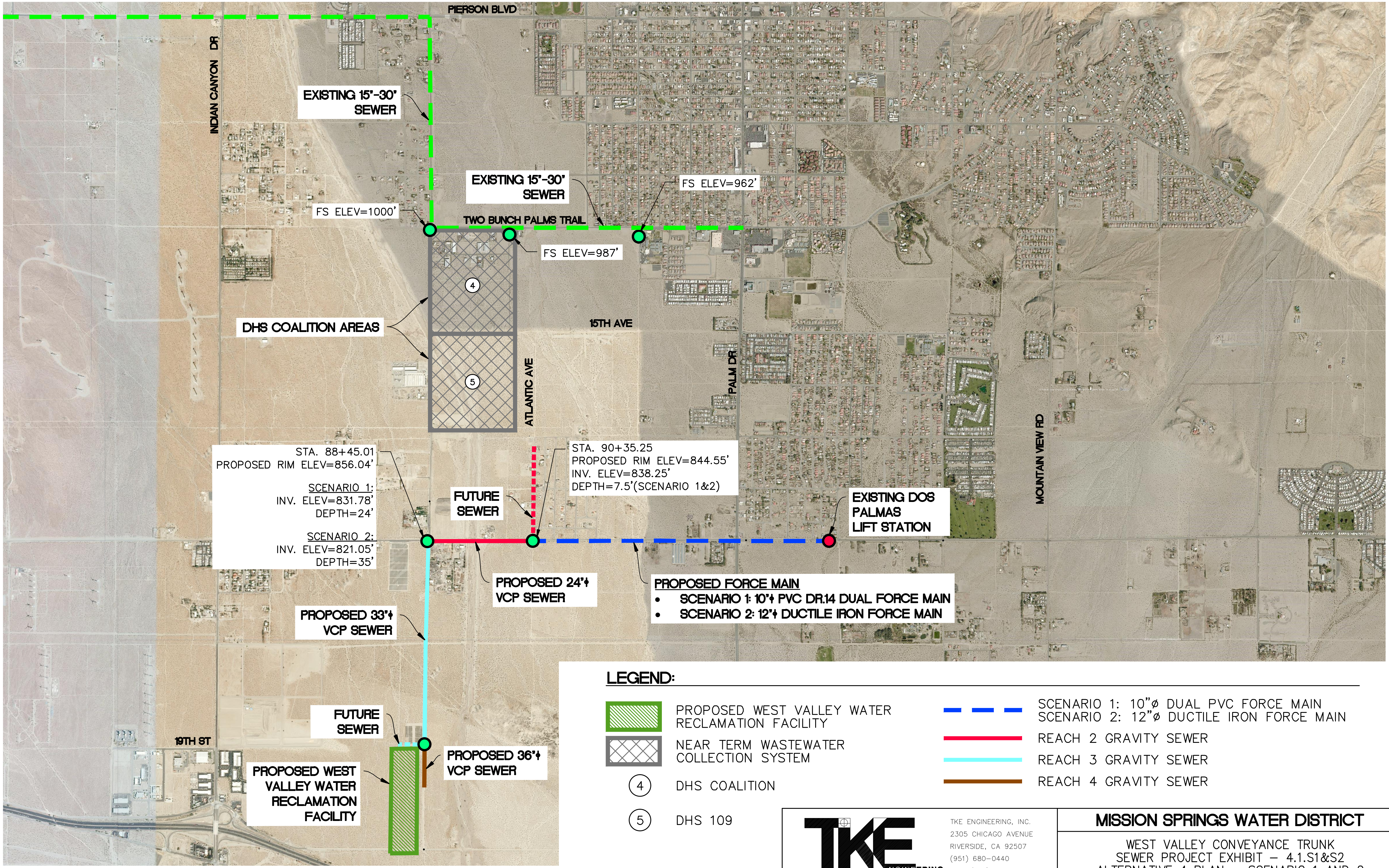

LEGEND:

(green hatched box)	PROPOSED WEST VALLEY WATER RECLAMATION FACILITY
(grey crosshatched box)	NEAR TERM WASTEWATER COLLECTION SYSTEM
4	DHS COALITION
5	DHS 109
(blue dashed line)	SCENARIO 1: 10"ϕ DUAL PVC FORCE MAIN SCENARIO 2: 12"ϕ DUCTILE IRON FORCE MAIN
(red line)	REACH 2 GRAVITY SEWER
(cyan line)	REACH 3 GRAVITY SEWER
(brown line)	REACH 4 GRAVITY SEWER

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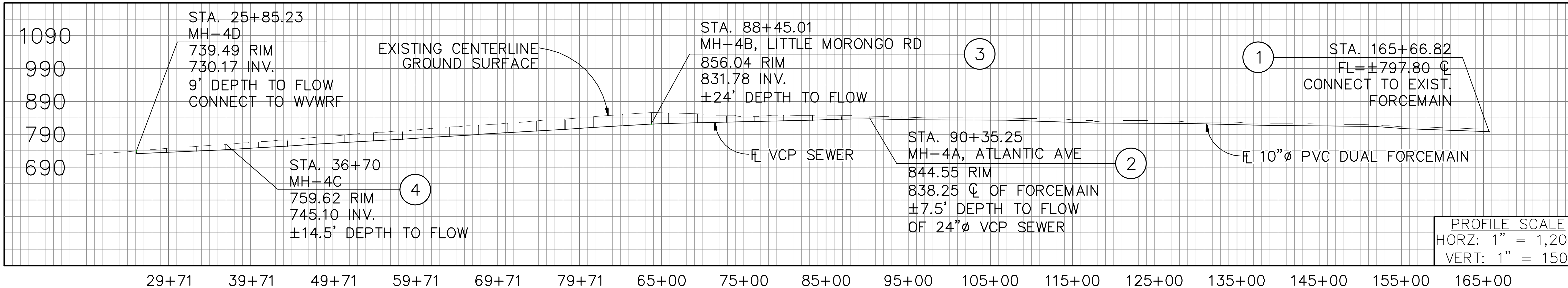
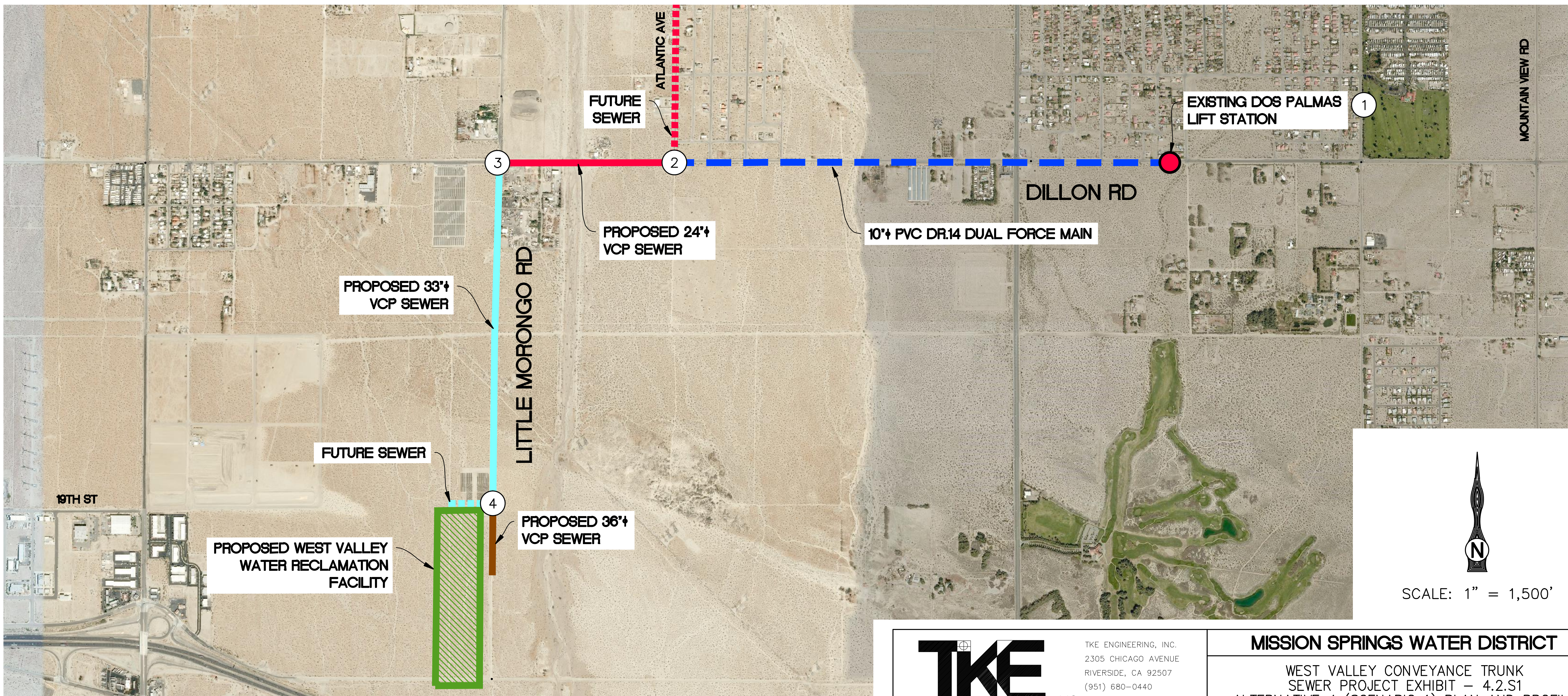
WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT – 4.1.S1&S2
ALTERNATIVE 4 PLAN – SCENARIO 1 AND 2





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MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT – 4.2.S1
ALTERNATIVE 4 (SCENARIO 1) PLAN AND PROFILE

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WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT 4.3.S1
ALTERNATIVE 4 (SCENARIO 1) PLAN AND PROFILE

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FAX: (951) 680-0490

MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK SEWER PROJECT EXHIBIT 4.4.S1
ALTERNATIVE 4 (SCENARIO 1) PLAN AND PROFILE

DETAIL "A"

1 INCH = 40 FEET

GRAPHIC SCALE

120 0 60 120 240 480

(IN FEET)

1 INCH = 120 FEET

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MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT 4.5.S1
ALTERNATIVE 4 (SCENARIO 1) PLAN AND PROFILE

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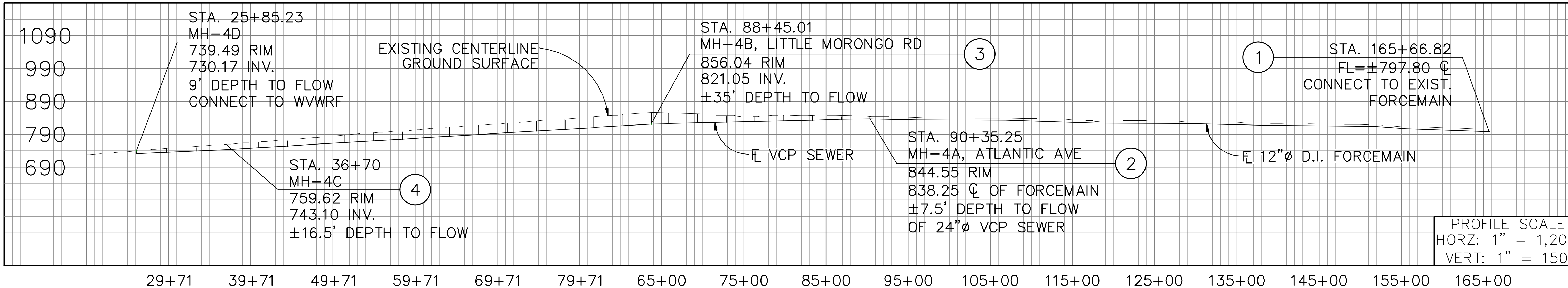

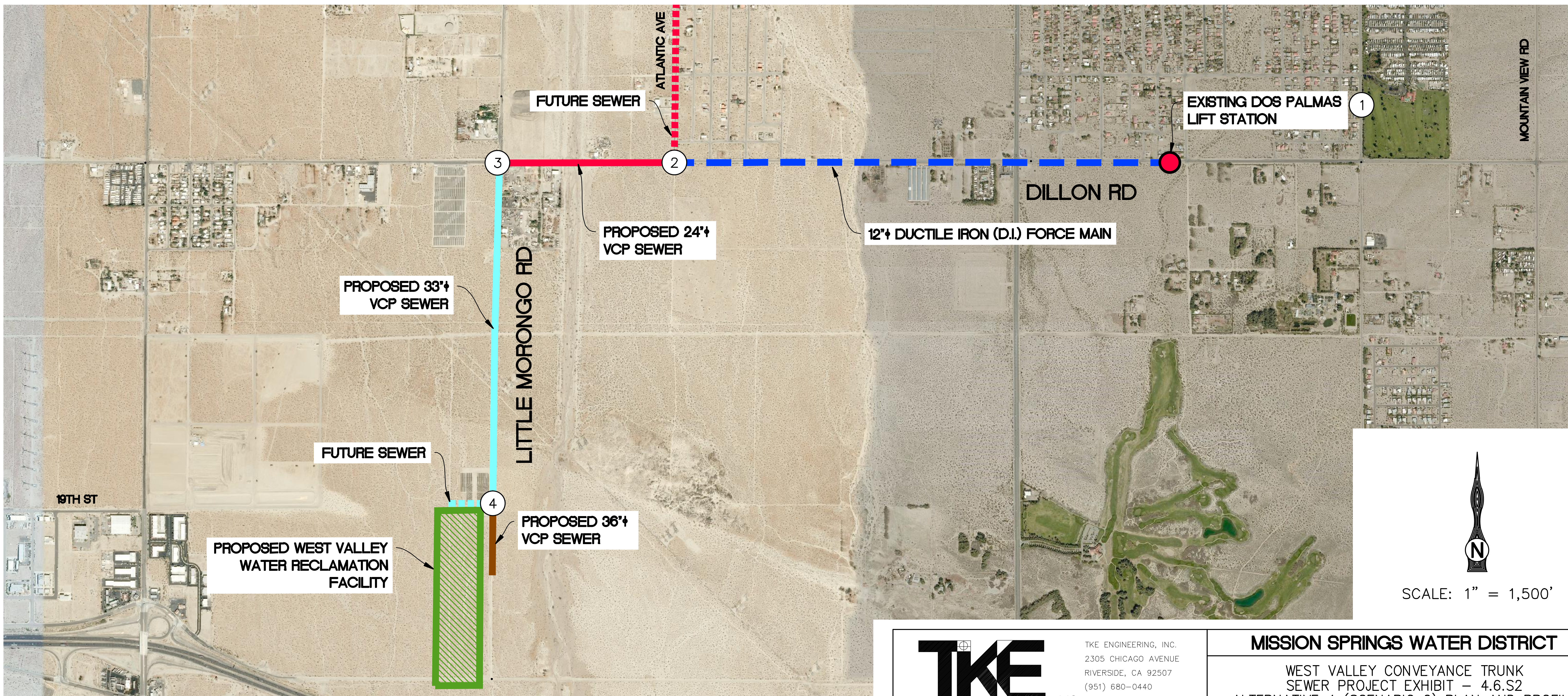

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MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT – 4.6.S2
ALTERNATIVE 4 (SCENARIO 2) PLAN AND PROFILE

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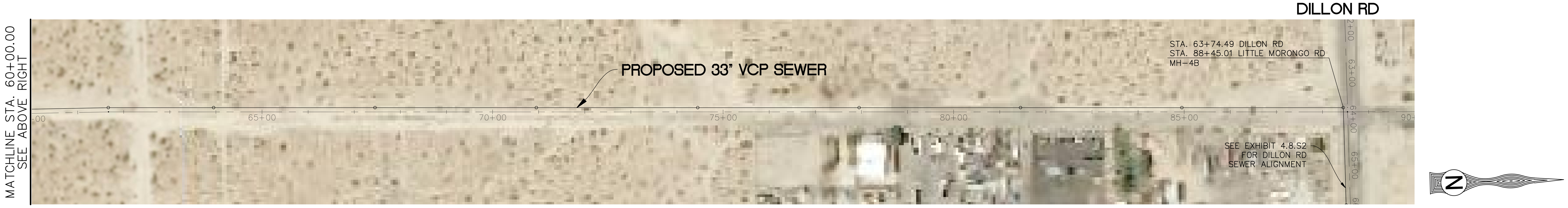
19TH STREET

LITTLE MORONGO RD

DILLON RD



LITTLE MORONGO RD

GRAPHIC SCALE

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(IN FEET)
1 INCH = 120 FEET

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RIVERSIDE, CA 92507
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FAX: (951) 680-0490

MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT 4.7.S2
ALTERNATIVE 4 (SCENARIO 2) PLAN AND PROFILE

GRAPHIC SCALE

120 0 60 120 240 480

(IN FEET)

1 INCH = 120 FEET

TKE ENGINEERING

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RIVERSIDE, CA 92507
(951) 680-0440
FAX: (951) 680-0490

MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT 4.8.S2
ALTERNATIVE 4 (SCENARIO 2) PLAN AND PROFILE

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DETAIL "A"

1 INCH = 40 FEET

GRAPHIC SCALE

120 0 60 120 240 480

(IN FEET)

1 INCH = 120 FEET

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RIVERSIDE, CA 92507
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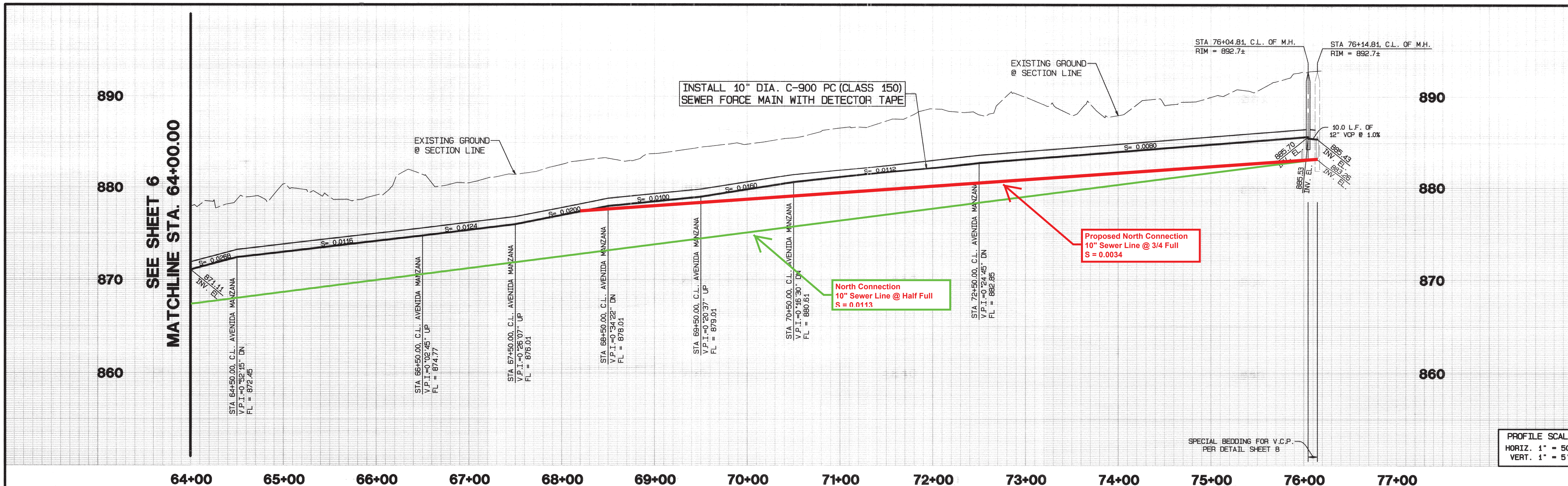
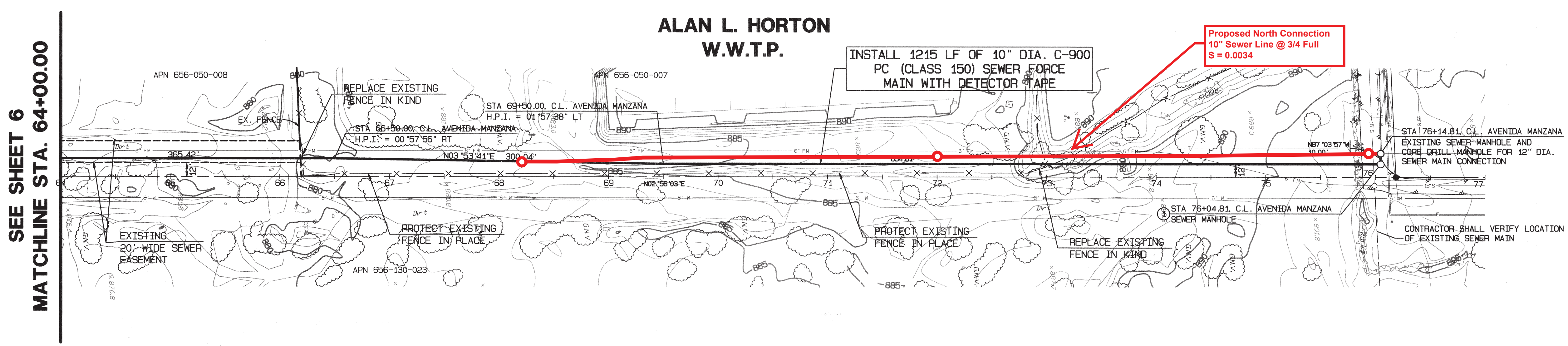
MISSION SPRINGS WATER DISTRICT

WEST VALLEY CONVEYANCE TRUNK
SEWER PROJECT EXHIBIT 4.9.S2
ALTERNATIVE 4 (SCENARIO 2) PLAN AND PROFILE



* CONTRACTOR SHALL DETERMINE EXACT LOCATION AND ELEVATION OF EXISTING FACILITIES PRIOR TO CONSTRUCTION.



CONSTRUCTION NOTES:

1. CONSTRUCT MANHOLE PER M.S.W.D. STD. DWG. NOS. S-4 & S-5.
3. CONSTRUCT 12" DIA. V.C.P. SEWER W/ SPECIAL BEDDING PER M.S.W.D. STD. DWG. NO. S-3.

FORCEMAIN NOTES:

1. THRUST PROTECTION FOR FORCEMAIN SHALL BE THRUST BLOCKS OR RESTRAINED JOINTS PER DISTRICT STANDARD.
2. FORCEMAIN SHALL CROSS UNDER WATERMAIN AT ALL WATERMAIN CROSSINGS.

GRAPHIC SCALE 1"=50'
0 50 100 150

DIGALERT — DIAL BEFORE YOU DIG — TWO WORKING DAYS BEFORE YOU DIG — TOLL FREE 1-800-227-2600 — A PUBLIC SERVICE BY UNDERGROUND SERVICE ALERT

THE PRIVATE ENGINEER SIGNING THESE PLANS IS RESPONSIBLE FOR ASSURING THE ACCURACY AND ACCEPTABILITY OF THE DESIGN HEREON. IN THE EVENT OF DISCREPANCIES ARISING AFTER M.S.W.D. APPROVAL OR DURING CONSTRUCTION, THE PRIVATE ENGINEER SHALL BE RESPONSIBLE FOR DETERMINING AN ACCEPTABLE SOLUTION AND REVISING THE PLANS FOR APPROVAL BY M.S.W.D.

DATE	BY	MARK		APPR	DATE
	ENGINEER			M.S.W.D.	

DESIGNED BY: N.J.H. DRAWN BY: N.J.H. CHECKED BY: S.Y.

RECOMMENDED FOR APPROVAL:
CHECKED:
DATE:

SEAL - DISTRICT

SEAL — REGISTERED PROFESSIONAL ENGINEER — WILLIAM T. MALONE — NO. C47569 — EXP. 12-31-05 — CIVIL — STATE OF CALIFORNIA

APPROVED BY: DATE:
GENERAL MANAGER
PREPARED UNDER THE SUPERVISION OF: DATE: 8/16/05
William T. Malone
P.E. NO. 47569 EXP. DATE 12-31-05

ALBERT A. WEBB ASSOCIATES
CIVIL ENGINEERS
3788 McCRAY ST. RIVERSIDE, CA 92506
PH. (951) 686-1070 FAX (951) 788-1256

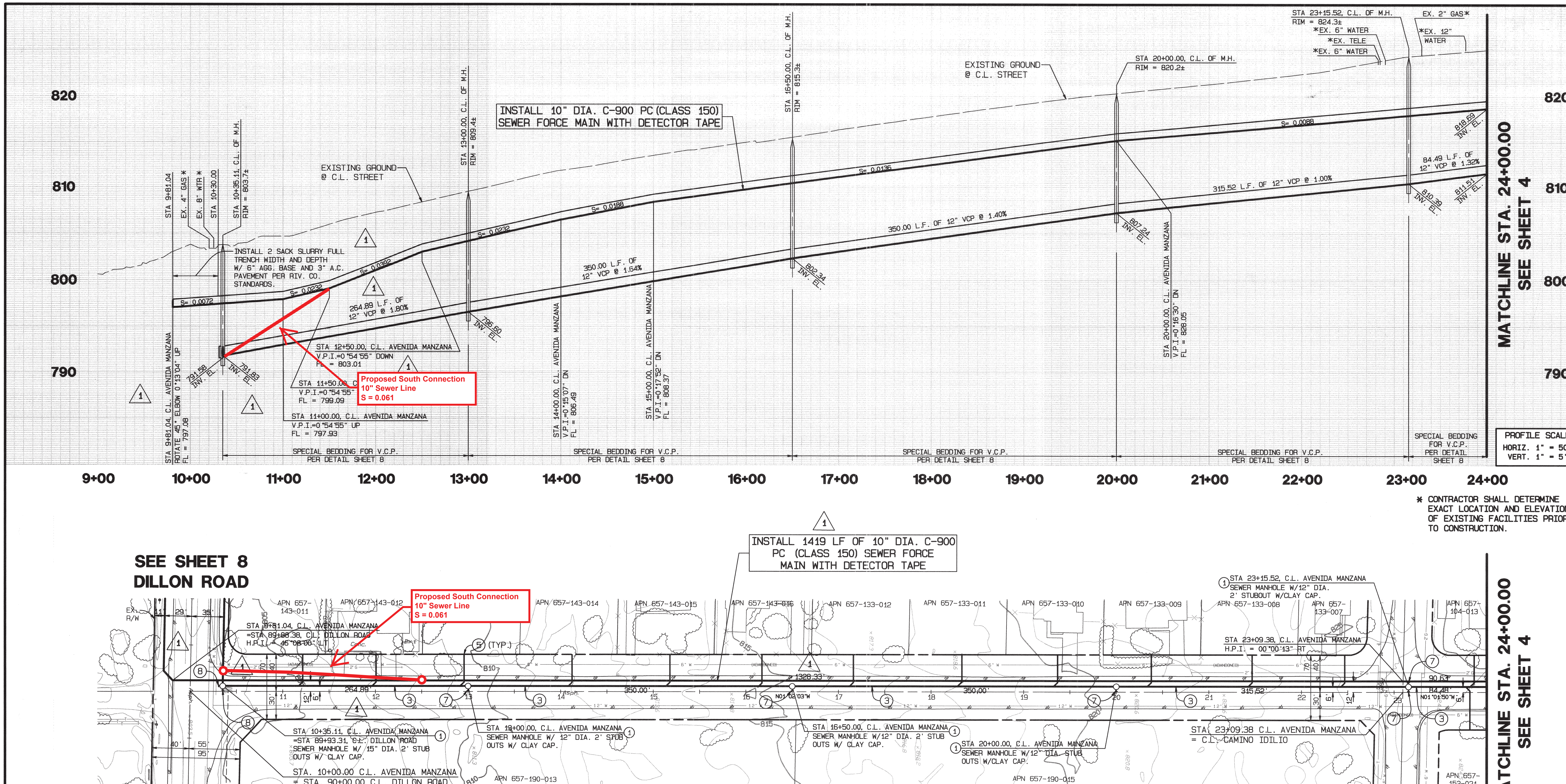
SCALE: 1" = 50'
PLAN: 040396SE.pro
BENCHMARK: SEE SHEET 2

MISSION SPRINGS WATER DISTRICT
DOS PALMAS (AREAS "L" AND "M")
GRAVITY SEWERMAIN, FORCEMAIN AND LIFT STATION
ALAN L. HORTON W.W.T.P.
STA. 64+00.00 TO STA. 76+14.81
FOR: M.S.W.D.
W.O. 04-0396

SHEET 7 OF 19 SHEETS
DWG. NO. 04-0396

\\Remote\wcoeiek\2004\04-0396\DwgPro\040396SE.pro -- Aug 16, 2005 11:31AM

Exhibit 5.1 - North Connection



CONSTRUCTION NOTES:

1. CONSTRUCT MANHOLE PER M.S.W.D. STD. DWG. NOS. S-4 & S-5.
3. CONSTRUCT 12" DIA. V.C.P. SEWER W/ SPECIAL BEDDING PER M.S.W.D. STD. DWG. NO. S-3.
5. CONSTRUCT 4" DIA. V.C.P. SEWER LATERAL PER M.S.W.D. STD. DWG. NO. S-8.
7. CONSTRUCT 12" DIA. 2' LONG V.C.P. STUBOUT WITH CLAY CAP PLUG
8. CONSTRUCT 15" DIA. 2' LONG V.C.P. STUBOUT WITH CLAY CAP PLUG

FORCEMAIN NOTES:

1. THRUST PROTECTION FOR FORCEMAIN SHALL BE THRUST BLOCKS OR RESTRAINED JOINTS PER DISTRICT STANDARD.
2. FORCEMAIN SHALL CROSS UNDER WATERMAIN AT ALL WATERMAIN CROSSINGS.

GRAPHIC SCALE 1"=50'

DIGALERT — DIAL BEFORE YOU DIG — TWO WORKING DAYS BEFORE YOU DIG — TOLL FREE 1-800-227-2600 — A PUBLIC SERVICE BY UNDERGROUND SERVICE ALERT

AVENIDA MANZANA

THE PRIVATE ENGINEER SIGNING THESE PLANS IS RESPONSIBLE FOR ASSURING THE ACCURACY AND ACCEPTABILITY OF THE DESIGN HEREON. IN THE EVENT OF DISCREPANCIES ARISING AFTER M.S.W.D. APPROVAL OR DURING CONSTRUCTION, THE PRIVATE ENGINEER SHALL BE RESPONSIBLE FOR DETERMINING AN ACCEPTABLE SOLUTION AND REVISING THE PLANS FOR APPROVAL BY M.S.W.D.

DATE	BY	MARK		APPR	DATE
9-05	N.J.H.	1	RIVERSIDE COUNTY ENCROACHMENT PERMIT.	WFM	10/05
	ENGINEER			M.S.W.D.	

DESIGNED BY: N.J.H. DRAWN BY: N.J.H. CHECKED BY: S.Y.

RECOMMENDED FOR APPROVAL: — CHECKED: — DATE:

SEAL - DISTRICT

SEAL: REGISTERED PROFESSIONAL ENGINEER WILLIAM T. MALONE NO. C47569 EXP. 12-31-05 CIVIL STATE OF CALIFORNIA

APPROVED BY: DATE:
GENERAL MANAGER
PREPARED UNDER THE SUPERVISION OF: William T. Malone DATE: 10/21/05
P.E. NO. 47569 EXP. DATE 12-31-05

ALBERT A. WEBB ASSOCIATES
CIVIL ENGINEERS
3788 McCRAY ST. RIVERSIDE, CA 92506
PH. (951) 686-1070 FAX (951) 788-1256

SCALE: 1" = 50' — PLAN: 040396SE.pro — BENCHMARK: SEE SHEET 2

MISSION SPRINGS WATER DISTRICT
DOS PALMAS (AREAS "L" AND "M") GRAVITY SEWERMAIN, FORCEMAIN AND LIFT STATION
AVENIDA MANZANA STA. 9+81.04 TO STA. 24+00.00
FOR: M.S.W.D. — W.O. 04-0396

SHEET 3 OF 19 SHEETS — DWG. NO. 04-0396

Exhibit 5.2 - South Connection

Appendix A

Cost Opinions

Mission Springs Water District

West Valley Sewer Conveyance System

Alternative No. 3

Dillon Force Main and Little Morongo Trunk Sewer
Preliminary Cost Opinion
March 2019

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization, Cleanup, & Demobilization	1	4%	$ 237,117	$ 237,117
2	Clearing and Grubbing	1	LS	$ 5,500	$ 5,500
3	Prepare SWPPP & BMP's	1	LS	$ 10,000	$ 10,000
4	Traffic Control	10,192	LF	$ 15	$ 152,880
5	Pavement Removal/Restoration up to 15'	10,192	LF	$ 60	$ 611,520
6	14" DIP Force Main Assembled	10,192	LF	$ 154	$ 1,569,568
7	33" VCP Sewer (8 to 16 feet depth)	5,175	LF	$ 300	$ 1,552,500
8	36" VCP Sewer (8 to 16 feet depth)	1,087	LF	$ 325	$ 353,275
9	5' Dia Manholes (up to 20')	18	EA	$ 10,000	$ 180,000
10	Sheeting Shoring & Bracing	1	2%	$ 73,107	$ 73,107
11	Valving, Fittings, and Abandonments	1	LS	$ 44,000	$ 44,000
12	AVAR Assembly and Vault	3	LS	$ 14,000	$ 42,000
13	Water Hammer Reducing Vault	2	LS	$ 24,000	$ 48,000
14	Lift Station Improvements / Initial Flow Only	1	LS	$ 100,000	$ 100,000

Construction Sub-Total:	**$ 4,979,467**
Construction Contingencies (10%):	$ 497,947
Construction Total:	**$ 5,477,414**
Administration, Construction Management, Testing & Inspection (15%):	**$ 821,612**
Project Total:	**$ 6,299,026**
Rounded Project Total:	**$ 6,300,000**

Mission Springs Water District

West Valley Sewer Conveyance System

Alternative No. 4: Scenario 1

Dillon Road Dual Force Main & Sewer Main and Little Morongo Sewer Main
Preliminary Cost Opinion
March 2019

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization, Cleanup, & Demobilization	1	4%	$ 272,386	$ 272,386
2	Clearing and Grubbing	1	LS	$ 5,500	$ 5,500
3	Prepare SWPPP & BMP's	1	LS	$ 10,000	$ 10,000
4	Traffic Control	10,192	LF	$ 15	$ 152,880
5	Pavement Removal/Restoration up to 15'	10,192	LF	$ 60	$ 611,520
6	10" PVC, DR 18 1st Force Main	7,531	LF	$ 85	$ 640,135
7	10" PVC, DR 18 2nd Force Main	7,531	LF	$ 85	$ 640,135
8	24" VCP Sewer (8 to 16 feet deep)	1,911	LF	$ 250	$ 477,750
9	24" VCP Sewer (16 to 24 feet deep)	750	LF	$ 350	$ 262,500
10	33" VCP Sewer (8 to 16 feet depth)	4,175	LF	$ 300	$ 1,252,500
11	33" VCP Sewer (16 to 24 feet depth)	1,000	LF	$ 400	$ 400,000
12	36" VCP Sewer (8 to 16 feet depth)	1,087	LF	$ 325	$ 353,275
13	Sheeting Shoring & Bracing	1	2%	$ 80,526	$ 80,526
14	5' Dia Manholes (up to 20')	16	EA	$ 10,000	$ 160,000
14	5' Dia Manholes (20' and deeper)	10	EA	$ 20,000	$ 200,000
15	Valving, Fittings, and Abandonments	1	LS	$ 35,000	$ 35,000
16	AVAR Assembly and Vault	4	LS	$ 11,000	$ 44,000
17	Bypass Connection, Water Hammer Reducing Vault	1	LS	$ 22,000	$ 22,000
	Lift Station Improvements / Initial Flows Only	1	LS	$ 100,000	$ 100,000

Construction Sub-Total:	**$**	**5,720,107**
Construction Contingencies (10%):	$	572,011
Construction Total:	**$**	**6,292,118**
Administration, Construction Management, Testing & Inspection (15%):	**$**	**943,818**
Project Total:	**$**	**7,235,935**
Rounded Project Total:	**$**	**7,240,000**

Mission Springs Water District

West Valley Sewer Conveyance System

Alternative No. 4: Scenario 2

Dillon Road Single Force Main & Sewer Main and Little Morongo Sewer Main
Preliminary Cost Opinion
March 2019

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization, Cleanup, & Demobilization	1	4%	$ 283,079	$ 283,079
2	Clearing and Grubbing	1	LS	$ 5,500	$ 5,500
3	Prepare SWPPP & BMP's	1	LS	$ 10,000	$ 10,000
4	Traffic Control	10,192	LF	$ 15	$ 152,880
5	Pavement Removal/Restoration up to 15'	10,192	LF	$ 60	$ 611,520
6	12" DIP Force Main Assembled	7,531	LF	$ 126	$ 948,906
7	24" VCP Sewer (8 to 16 feet deep)	1,500	LF	$ 250	$ 375,000
8	24" VCP Sewer (16 to 35 feet deep)	1,161	LF	$ 450	$ 522,450
9	33" VCP Sewer (16 to 35 feet depth)	1,700	LF	$ 550	$ 935,000
10	33" VCP Sewer (8 to 35 feet depth)	3,475	LF	$ 300	$ 1,042,500
11	36" VCP Sewer (8 to 16 feet depth)	1,087	LF	$ 325	$ 353,275
12	Sheeting Shoring & Bracing	1	2%	$ 83,543	$ 83,543
13	5' Dia Manholes (up to 20')	12	EA	$ 10,000	$ 120,000
14	5' Dia Manholes (20' to 30' deep)	8	EA	$ 20,000	$ 160,000
14	5' Dia Manholes (30' and deeper)	6	EA	$ 30,000	$ 180,000
15	Valving, Fittings, and Abandonments	1	LS	$ 35,000	$ 35,000
16	AVAR Assembly and Vault	4	LS	$ 11,000	$ 44,000
17	Water Hammer Reducing Vault	1	LS	$ 22,000	$ 22,000
	Lift Station Improvements / Initial Flow Only	1	LS	$ 60,000	$ 60,000

Construction Sub-Total:	**$**	**5,944,652**
Construction Contingencies (10%):	$	594,465
Construction Total:	**$**	**6,539,118**
Administration, Construction Management, Testing & Inspection (15%):	**$**	**980,868**
Project Total:	**$**	**7,519,985**
Rounded Project Total:	**$**	**7,520,000**

Mission Springs Water District
West Valley Sewer Conveyance System
Alternative 5
Horton Flow Diversion
Preliminary Cost Opinion
June 2018

Item	Description	Quantity	Unit	Unit Price	Amount
1	Mobilization, Cleanup, & Demobilization	1	5%	$ 10,230	$ 10,230
2	Clearing and Grubbing	1	LS	$ 1,500	$ 1,500
3	10" VCP Sewer	1,005	LF	$ 120	$ 120,600
4	5' Dia Manholes	5	EA	$ 10,000	$ 50,000
5	Diversion Manhole	1	EA	$ 22,000	$ 22,000
6	Valving, Fittings & Abandonments	1	LS	$ 10,500	$ 10,500
					$ -
					$ -

Construction Sub-Total:	**$ 214,830**
Construction Contingencies (20%):	$ 42,966
Construction Total:	**$ 257,796**
Administration, Construction Management, Testing & Inspection (15%):	**$ 38,669**
Project Total:	**$ 296,465**
Rounded Project Total:	**$ 300,000**