APPENDIX C – TM-3 SOLIDS HANDLING EVALUATION

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

TECHNICAL MEMORANDUM NO. 3
SOLIDS HANDLING EVALUATION

FINAL
April 2016

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TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

TECHNICAL MEMORANDUM
NO. 3
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TABLE OF CONTENTS

Page No.

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

AADF	average annual daily flow
BFP	belt filter press
BNR	biological nutrient removal
cy/year	cubic yards per year
ENR	Engineering News-Record
ea	each
ea/year	each per year
EPA	Environmental Protection Agency
ft	foot/feet
gal	gallon/gallons
gal/train	gallons per train
gpd	gallons per day
gpm	gallons per minute
hp	horsepower
in	inch/inches
lb	pound/pounds
lb/day	pounds per day
lb/hour	pounds per hour
lb/dry-ton	pounds per dry-ton
mgd	millions gallons per day
MMADF	maximum month average day flow
O&M	operations and maintenance
RAS	returned activated sludge
rpm	revolutions per minute
times/hr	times per hour
TM	technical memorandum
Town	Town of Marana
VFD	variable frequency drive
WAS	waste activated sludge
WMI	Waste Management, Inc.
WRF	water reclamation facility

SOLIDS HANDLING EVALUATION

1.0 INTRODUCTION

1.1 Existing Solids Handling Facilities

The Town of Marana (Town) owns and operates the Marana Water Reclamation Facility (WRF), which has a permitted capacity of 700,000 gallons per day (gpd). The WRF uses a 500,000 gpd Biolac® treatment system. A separate treatment system consisting of four biological nutrient removal (BNR) package plants with a total combined capacity of 200,000 gpd is also on site but is not in use.

The return activated sludge (RAS) system of the Biolac® system is continuously operated by use of airlift pumps. It delivers a maximum sludge return rate of 150 percent of the basin's average design flow.

The waste activated sludge (WAS) pump station is located to the west of the Biolac® basin. A 4-inch buried isolation valve is used to introduce flow to the WAS pump station. The WAS pump station includes two submersible pumps installed within a manhole. WAS is pumped in a 4-inch pipe from the pump station to a sludge storage tank in the southwest corner of the site.

The existing sludge storage tank has a capacity of approximately 16,500 gallons. Coarse bubble diffusers mounted inside the sludge tank are used to prevent the sludge from becoming septic and creating odor. The diffusers can be fed by a positive displacement blower adjacent to the storage tank.

The typical practice is to decant the waste activated sludge (WAS) by turning off the air supply and letting the sludge settle and decanted water to rise to the top. Before the sludge is hauled away, the decanting pump is operated manually to return the decant to the main treatment process. Based on historical operational data, decanting the sludge can reduce the WAS volume by 30 to 40 percent.

The sludge from the storage tank is transferred to trucks by a single dry-pit pump and hauled 54 miles to the City of Casa Grande WRF by a private contractor for treatment and disposal. This hauling and treatment cost is the second highest operational cost of the WRF.

The Town would prefer to dispose of its waste solids at the Marana Regional Landfill, operated by Waste Management, Inc. (WMI), which is only 11 miles away. However, for the solids to be acceptable for landfill disposal, they must be dewatered sufficiently to pass a paint filter test, or approximately 15-18 percent solids.

The reduced hauling distance and volume of dewatered solids would significantly lower costs for the WRF operations. Therefore, the purpose of this technical memorandum (TM) is to evaluate and compare various dewatering technology alternatives that could be implemented at the Marana WRF to reduce the volume of solids disposed of at the landfill.

1.2 Objectives

To meet the demands of expected growth in Marana's service area, the Town wants to complete a Master Plan evaluation of the WRF for the phased expansion of the facility. A major task of this Master Plan is to evaluate alternatives of future solids handling technologies. The objectives of this TM are to:

- Summarize solids production for the current conditions and potential future phased expansions.
- Discuss available solids thickening/dewatering technologies and evaluate technologies that would be applicable at the WRF, including preliminary sizing and possible site layouts.
- Provide opinion of probable costs for equipment and facility installation.
- Make a recommendation(s) of solids handling equipment and facilities that can be furthered in the preliminary or detailed design phase.

2.0 PLANT FLOWS AND SOLIDS PRODUCTION

Historical plant operational data was collected and analyzed to estimate the solids production for the WRF's current operating conditions. Biological process modeling was performed to size and define the future operating conditions of the liquids and solids treatment trains.

The process modeling output of plant flow rate and solids production is summarized in Table 3.1. These estimates are the basis for the sizing of the solids dewatering facility and the evaluation in subsequent sections of this TM.

Solids productions of current and future phases under average annual daily flow (AADF) and maximum month daily flow (MMADF) conditions are depicted in Figure 3.1.

Future required plant capacities were calculated using the projected growth and wastewater flow generation rates that are detailed in TM No. 1.

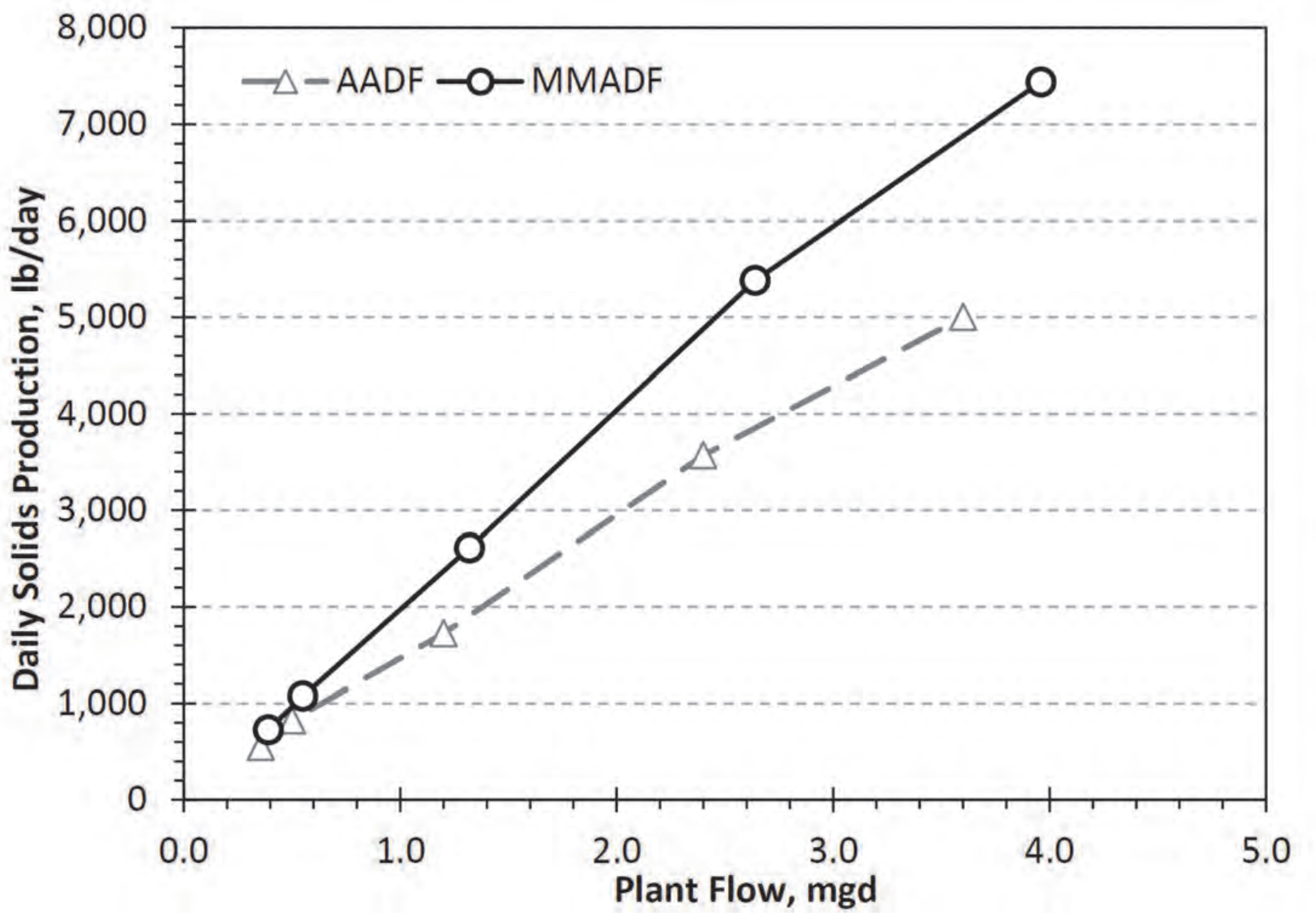
For this TM, it is assumed the WRF will expand in three phases to capacities of 1.2, 2.4, and 3.6 mgd (based on AADF).

Table 3.1 Design Phases and Solids Production
Marana Water Reclamation Facility Master Plan
Town of Marana

		Current Operation		Phase 1		Phase 2		Phase 3	
Criteria	**Units**	**AADF**	**MMADF**	**AADF**	**MMADF**	**AADF**	**MMADF**	**AADF**	**MMADF**
Plant Flow	mgd	0.35	0.39	1.2	1.3	2.4	2.6	3.6	4.0
WAS Flow	mgd	0.010	0.014	0.033	0.049	0.049	0.078	0.088	0.126
Solids Loading (1)	lb/day	551	728	1,725	2,612	3,571	5,384	5,005	7,441
Solids Content	%	0.68%	0.62%	0.62%	0.64%	0.87%	0.83%	0.68%	0.71%

Note:

(1) Solids production in current operation is based on historical operational data. Solids production in Phase 1, 2, and 3 are based on biological modeling outputs. For conservative estimating, it is assumed to continue with the Biolac® system in Phase 1, as it produces the highest volume of solids at the lowest concentration. It is assumed the treatment process will be converted to a more traditional process for Phases 2 and 3, which produce less solids at higher concentrations.



DAILY SOLIDS PRODUCTION IN CURRENT AND FUTURE PHASES

FIGURE 3.1

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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3.0 SOLIDS HANDLING SYSTEM EVALUATION

3.1 Sludge Storage

Sludge storage can be liquid WAS storage, dewatered cake storage, or a combination of both. Based on Town's inputs and the relatively small volume of cake production projected, cake storage is not necessary for the Phase 1 expansion but may be included in future phases.

WAS storage should be provided before the dewatering process. The major purposes for WAS storage are:

- General operational flexibility of the main treatment process.
- Planned shutdown of dewatering equipment for weekends, holidays, long holiday weekends, and dewatering equipment maintenance.
- Unplanned shutdowns of dewatering equipment or reduction in capacity due to a mechanical component failure.
- Planned or unplanned shutdown or reduction in cake hauling practices.
- The opportunity to decant some of the liquid, reducing the volume of solids to be dewatered.

To accommodate long holiday weekends, providing at least four days of WAS storage at the WRF under AADF and three days under MMADF is recommended before the dewatering process. In addition, sludge storage and equipment sizing were recommended based on a weekday-only operation of the dewatering equipment, assuming that dewatering operations will not be required on regular weekends

As discussed earlier in this TM, the existing package treatment plants are not in service and have not been operated since 2006. Each packaged treatment train is a steel tank that includes an aeration zone, anoxic zone, clarifier, sludge storage, and chlorine contact basin. As discussed in TM No. 2, the blowers, mixers, diffusers, piping, and pumps are not in good condition. It would require a significant rehabilitation effort to make the package plants operational again. However, the steel tanks were observed to be in fairly good condition during a site inspection. They had some surface rust in places but no significant structural deterioration.

Instead of constructing a new WAS storage facility, it could be an economical option to repurpose the package plants for WAS storage after proper surface preparation and coating.

Table 3.2 summarizes the recommended WAS storage volume, the dimensions of the existing packaged plants, and the available WAS storage time in current and future phases. This table evaluated the following two scenarios:

- Using aeration zone only of each train for WAS storage. Each aeration zone has a storage volume of 28,500 gallons. Using aeration zones of all four trains allows enough storage time for current operation flow (0.35 mgd AADF) and current design flow (0.5 mgd AADF). When capacity is expanded to 1.2 mgd in Phase 1, the storage times are 3.4 days and 2.3 days under AADF and MMADF, respectively, which does not meet storage time recommendations. Therefore, using all available storage volume of the package plants is necessary to give enough WAS storage volume for Phase 1.
- Using all available volume of each train for WAS storage. All available storage volume, including the anoxic zone, aeration zone, and sludge storage zone, are used for WAS storage. The chlorine contact basin is small and is not recommended for WAS storage.

  Total available storage volume per train is approximately 40,100 gallons. Using all four trains can provide enough storage time for Phase 1. When WRF treatment capacity is further expanded in Phase 2 and Phase 3, the WAS storage time is less than 4 days under AADF and 3 days under MMADF. Therefore, new WAS storage facilities are recommended for Phase 2 and 3.

Table 3.3 shows conceptual design criteria for future WAS storage tanks. Rectangular tanks can be constructed with common walls and typically require a smaller footprint than circular tanks. To provide operational flexibility, multiple tanks are recommended rather than a single tank.

As mentioned in Section 1.1, the current practice is to decant the WAS in the existing sludge storage tank to reduce the volume by 30 to 40 percent before the sludge is hauled off. Decanting also can help the downstream dewatering process by improving dewatering equipment performance, increasing the solids content of dewatered cake, reducing daily operation hours, and lowering polymer usage.

Detailed design of decanting will be further evaluated during the design phase. However, to develop a conservative estimate of the costs and footprint, we have assumed no decanting for sizing the dewatering equipment and related ancillary facilities.

Similarly, with the existing sludge storage tank, coarse bubble diffusers are recommended to prevent the sludge from becoming septic and creating odor in the repurposed packaged plant. The existing diffusers and small diameter piping appeared to be in poor condition. We recommend replacing all the diffusers and piping for WAS storage. It is assumed that one of existing aeration blowers can be used for aeration of the WAS storage tanks, if desired.

Table 3.2 Repurposing Package Plants for WAS Storage
Marana Water Reclamation Facility Master Plan
Town of Marana

Parameter	Current Operation	Current Design	Phase 1	Phase 2	Phase 3
Plant Flow @ AADF, mgd	0.35	0.5	1.2	2.4	3.6
WAS Flow, gpd					
AADF	9,700	15,300	33,200	49,500	87,800
MMADF	14,100	19,800	48,600	77,600	126,000
Recommended Storage Volume (1), gal	42,300	61,000	145,800	232,800	378,000
No. of Packaged Plant Trains			4		
Current Design Water Depth, ft			10.5		
Recommended Water Depth (2), ft			9.5		
Anoxic Zone Volume, gal/train			6,500		
Aeration Zone Volume, gal/train			28,500		
Sludge Storage Zone Volume, gal/train			5,100		
Total Available Storage Volume (3), gal					
per train			40,100		
all trains			160,400		
Storage Time					
Using One Aeration Zone, days					
AADF	3.0	1.9	0.9	0.6	0.3
MMADF	2.0	1.4	0.6	0.4	0.2
Using All Aeration Zone, days					
AADF	11.8	7.5	3.4	2.3	1.3
MMADF	8.1	5.8	2.3	1.5	0.9
Using One Train, days					
AADF	4.2	2.6	1.2	0.8	0.5
MMADF	2.8	2.0	0.8	0.5	0.3
Using All Train, days					
AADF	16.6	10.5	4.8	3.2	1.8
MMADF	11.4	8.1	3.3	2.1	1.3

Notes:
(1) Recommended storage volume is based on 4 days storage under AADF and 3 days under MMADF.
(2) Recommended water depth of 9.5 feet provides 2 feet freeboard for sludge storage and is used to calculate volume of each compartment in existing packaged plant trains.
(3) Total available storage volume is the combined volume of anoxic zone, aeration zone and sludge storage zone in each train.
(4) Recommended storage time is 4-day storage at AADF and 3 days at MMADF. Green cells indicate meeting the target storage time. Orange cells indicate not meeting the target storage time.

Table 3.3 New WAS Storage Facility Marana Water Reclamation Facility Master Plan Town of Marana		
Parameter	**Phase 2**	**Phase 3**
Plant Flow, AADF, mgd	2.40	3.60
WAS Flow [(1)], gpd		
AADF	49,500	87,800
MMADF	77,600	126,000
Target Storage Time		
AADF	4	4
MMADF	3	3
Required Storage Volume, gal	232,800	378,000
New WAS Storage Tanks		
No. of Tanks [(2)]	2	3
Required Volume, each, gal	116,400	126,000
Assumed Water Depth [(3)], ft	20	20
Assumed Freeboard, ft	3	3
Rectangular Tanks [(3)]		
Length, ft	30	30
Width, ft	30	30
Storage Time, days		
AADF	5.4	4.6
MMADF	3.5	3.2

Notes:

(1) Sludge concentration is 0.83%-0.87% in Phase 2, 0.68%-0.71% in Phase 3, as shown in Table 3.1.

(2) Constructing more than one tank is recommended to provide operational flexibility.

(3) Design parameters for the rectangular tank are considered a conceptual design assumption for the purpose of comparison. Actual dimensions need to be further evaluated during design phase.

The functionality of the existing sludge transfer pumps is also unknown. We recommend installing new positive displacement pumps with variable frequency drives (VFD) to feed the downstream dewatering process.

The existing sludge storage tank has a capacity of approximately 16,500 gallons. The tank has a single dry-pit pump for sludge transfer, coarse bubble diffusers, and blowers for aeration, and a manually operated pump for decanting.

In Phase 1, removing the existing sludge storage tank to provide space and adequate truck access for the new dewatering facility is recommended. However, the WAS storage and hauling must remain operational during Phase 1 construction.

The sludge storage tank should remain in service until the repurposed package plants are online for WAS storage. Hauling trucks may then fill from the newly refurbished steel tanks rather than the existing smaller tank. At that time, the existing sludge tank could be removed to allow for construction of the new dewatering facilities.

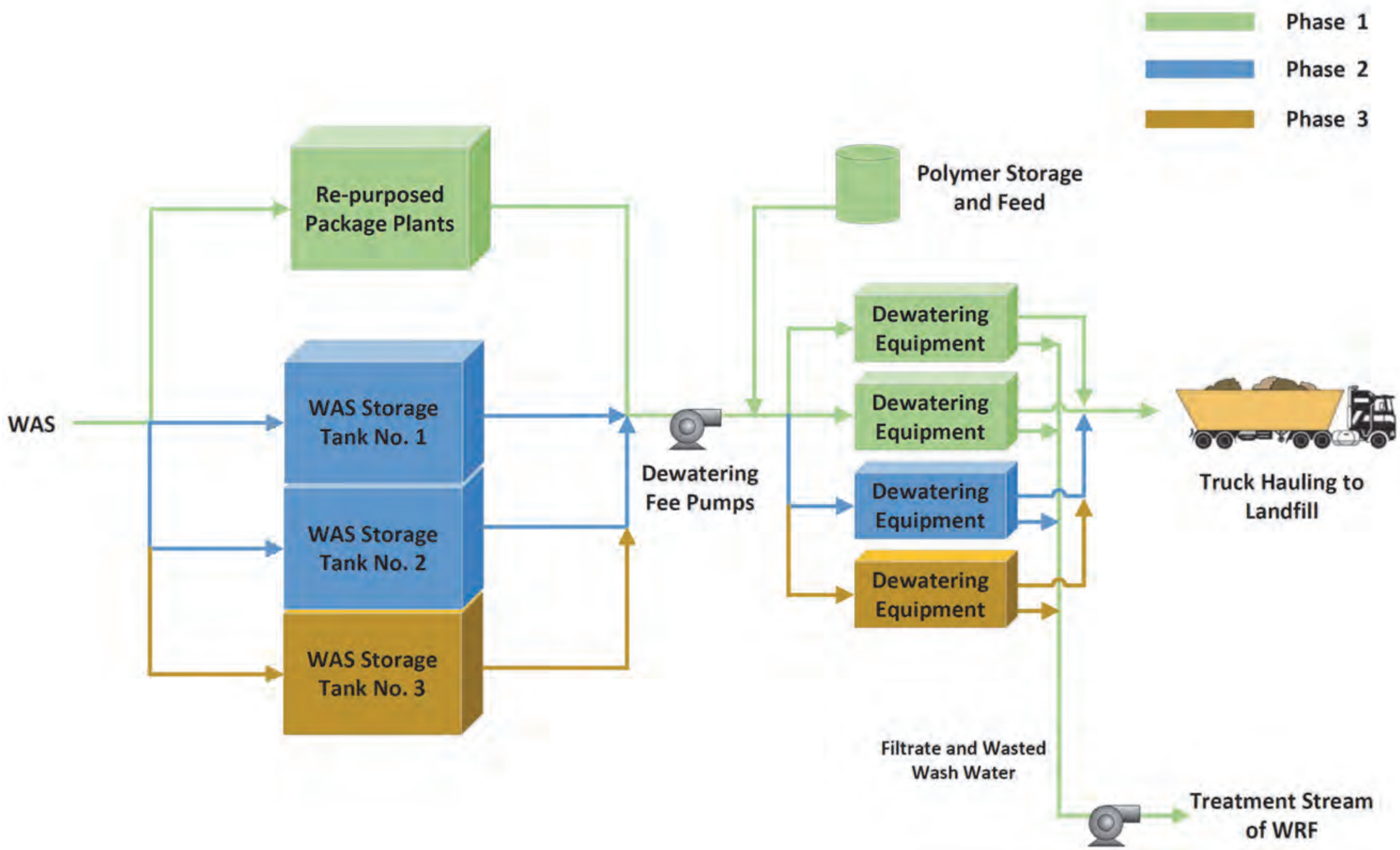
As indicated in Table 3.2, the repurposed package plants will provide adequate storage volume for the Phase 1 expansion. As discussed in TM No. 1 and No. 4, the Phase 1 expansion should provide sufficient capacity for the next 10-year planning period (2025) and the Phase 2 expansion may not be required until 2035. At that time, additional or new sludge storage facilities will be required to maintain 3 to 4 days storage volumes.

Table 3.3 details the required sizing and possible configuration of future sludge storage facilities.

3.2 Evaluation Assumptions for Dewatering Technology Alternatives

Four solids dewatering technology alternatives are evaluated and compared in subsequent sections this TM, including belt filter press, centrifuge, screw press, and rotary fan press. Figure 3.2 is a process flow diagram for the dewatering operation. Preliminary sizing and a site layout for the dewatering facility in Phase 1 are shown in Figure 3.3 and are based on the following assumptions:

- The estimated flow and loading are those presented in Table 3.1.
- A normal operation schedule is 5 days per week and a maximum of 8 hours per day, including start-up and shutdown activities.
- All dewatering facilities are installed outdoors and not inside a building. A canopy structure is constructed to cover the dewatering facility.
- Dewatering equipment installed on a concrete pad at grade level and a conveyor is used to deliver the dewatered cake into a roll-off container or truck bed. No dewatered cake storage (such as a silo) is installed.
- The existing package plants are repurposed for WAS storage in Phase 1. New WAS storage tanks will be required for future Phases 2 and 3.



DEWATERING PROCESS FLOW DIAGRAM

FIGURE 3.2

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

PROPOSED LOCATION OF DEWATERING FACILITY IN PHASE 1

FIGURE 3.3

TOWN OF MARANA
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3.3 Solids Handling Technology Alternatives

Solids thickening and dewatering are the processes that reduce the volume of the sludge by reducing the moisture content of the solids, which in turn reduces the volume of solids being hauled, applied, conveyed, and stored.

The thickening process typically produces sludge containing approximately 4 to 6 percent solids. The main purpose of a thickening process is to reduce the volumetric loading of downstream treatment processes, such as digestion.

In comparison, the dewatering process typically thickens the sludge to approximately 15 to 20 percent or higher, processing the sludge into a form that is suitable for land application or final disposal.

This TM evaluates only the dewatering technology alternatives because the Town is investigating a feasible way to send the dewatered sludge to Marana landfill.

This section evaluates the available dewatering alternatives, including the belt filter press, screw press, centrifuge, and rotary fan press.

3.3.1 Belt Filter Press

A belt filter press (BFP) is a commonly used technology for dewatering sludge or biosolids in Arizona and nationwide. This technology uses the principles of chemical conditioning, gravity drainage, and mechanically applied pressure to dewater sludge.

The solids are initially spread out across a continuously moving belt to allow free water to drain naturally by gravity. The solids then proceed, compressed between two tensioned porous belts run through a series of rollers to squeeze out additional water. A polymer solution is typically mixed with the solids upstream of the BFP to enhance the dewatering process.

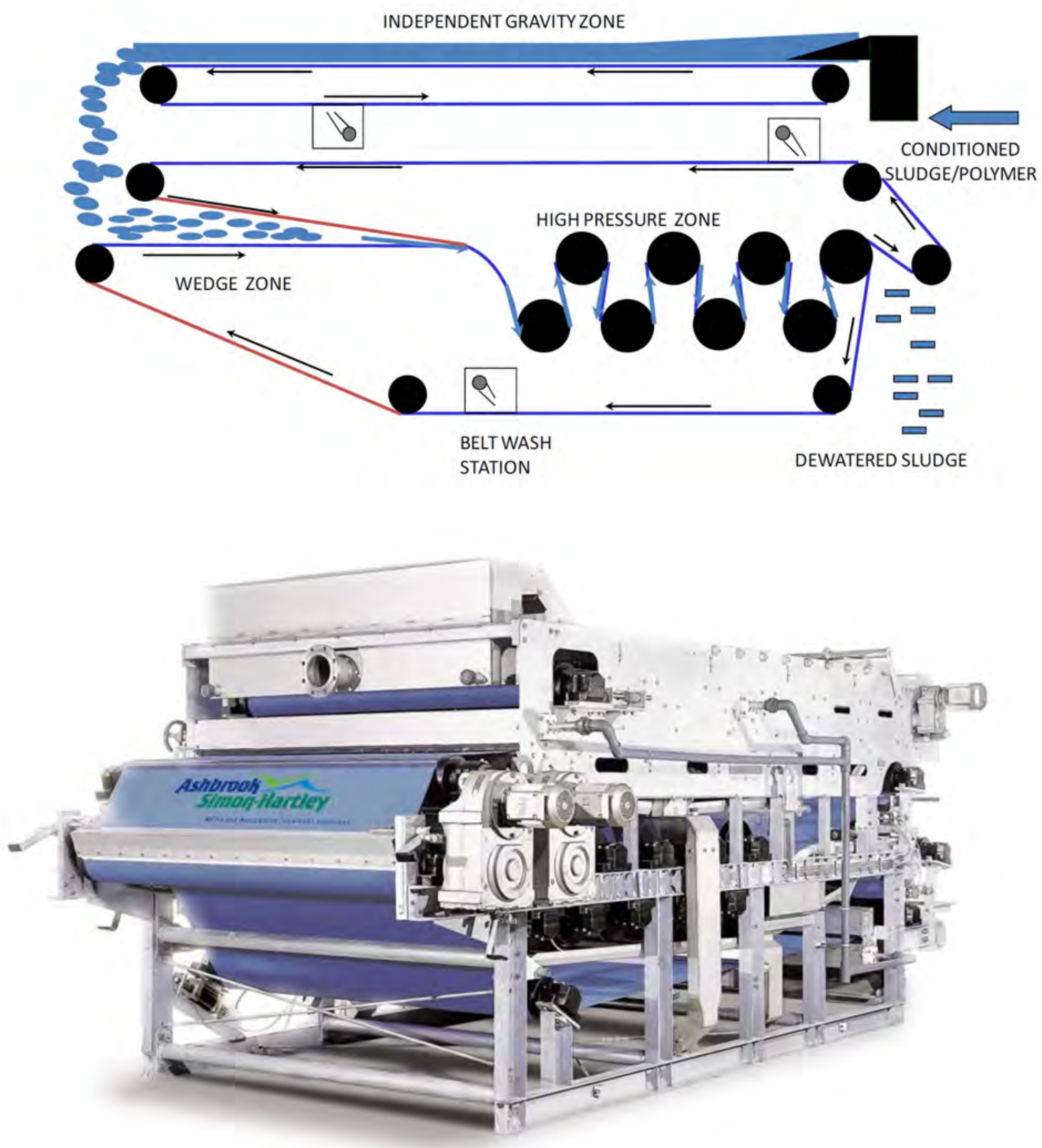
A BFP typically produces a 15 to 20 percent solids content sludge cake and a filtrate recycle stream. The filtrate can be recycled back to the main treatment process of the plant.

Many designs of BFPs are available, varying in belt width (1 to 3 meters) and the number of belts (2-belt versus 3-belt), but all incorporating the basic features of polymer conditioning zone, gravity drainage zone, low-pressure wedge zone, and high-pressure squeezing zone.

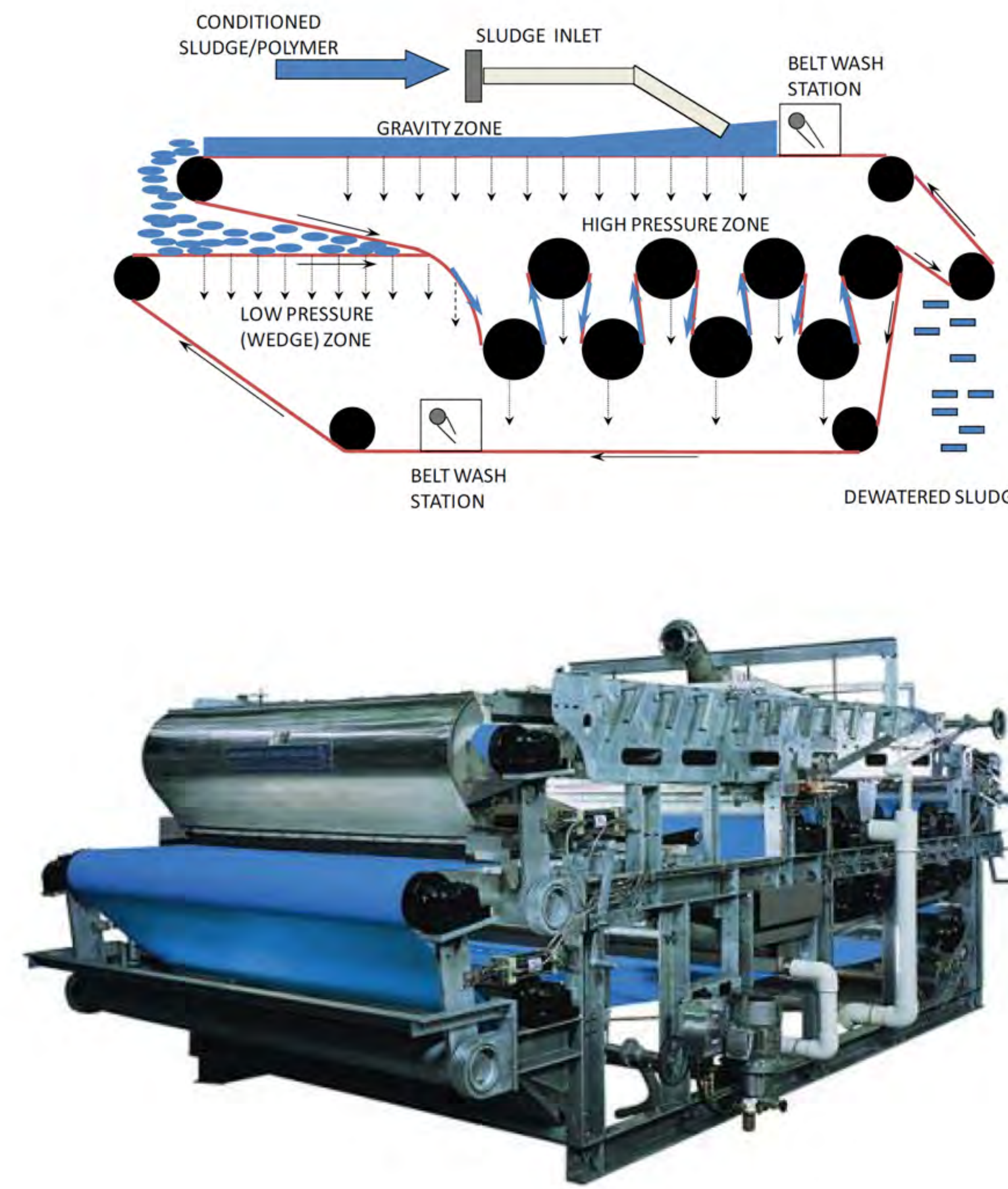
The belts are typically cleaned by a continuous spray of water, which can use as much as 60 to 150 gallons per minute (gpm). Non-potable effluent is typically used for this purpose.

Primary BFP manufacturers include Ashbrook Simon-Hartley and Andritz.

Figure 3.4 illustrates 2-belt and 3-belt BFP operation schematics.



3-Belt BFP Operation
(Courtesy of Ashbrook Simon-Hartley)



2-Belt BFP Operation
(Courtesy of Ashbrook Simon-Hartley)

BELT FILTER PRESS SCHEMATIC AND PICTURES

FIGURE 3.4

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Figure 3.5 shows the conceptual site layout for a BFP. In this site layout, a conveyor is used to deliver the dewatered cake from BFP to a roll-off container or the bed of a truck for hauling. A small, elevated platform allows access to the top of BFP for inspection and maintenance.

3.3.2 Centrifuge

Centrifuges are widely used in the industry for a variety of applications in Arizona and nationwide, including separating liquids of different density, thickening slurries, or removing solids.

The basic type is called the solid-bowl centrifuge shown in Figure 3.4, which consists of a long bowl, mounted horizontally and tapered at one end. Sludge is fed into the centrifuge at a constant flow rate, and, due to the centrifugal force acting on the varying densities of the constituents, the sludge is separated into a solid cake and centrate. The centrate is then drained and recycled, while the dewatered cake is discharged from the centrifuge by a conveyor or cake pump.

Depending on the centrifuge's settings, varying sludge solids concentrations can be put through. The dewatered cake can have 15 to 20 percent solids content or more, depending on the feed sludge type (unstabilized sludge versus stabilized biosolids) and polymer dose. The centrate is typically recycled back to the main treatment process.

The primary suppliers of centrifuges include Alfa Laval, Westfalia, Centrisys, and Andritz.

Figure 3.6 illustrates centrifuge operation schematics.

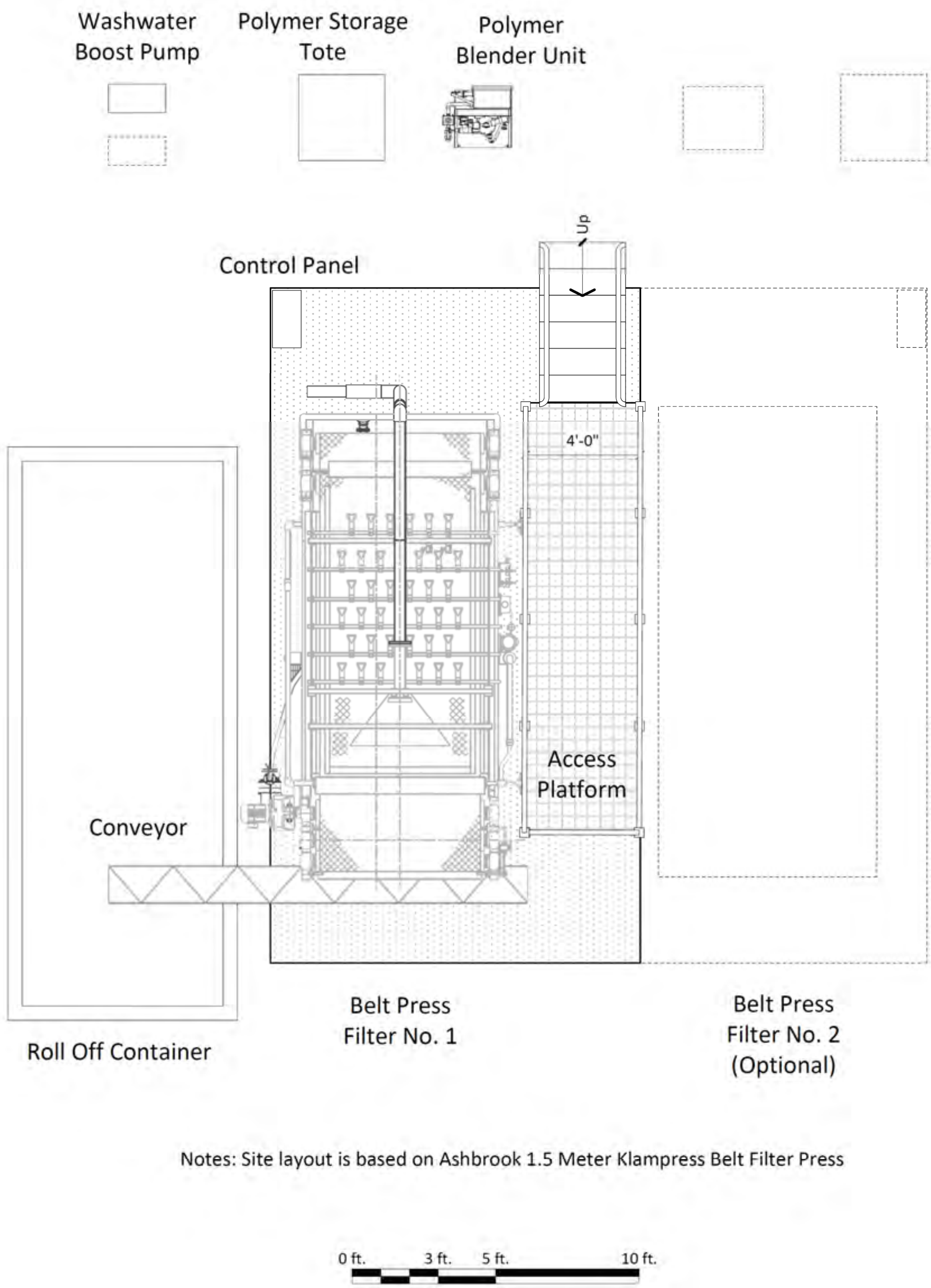
Figure 3.7 shows the conceptual site layout for centrifuge. Note that in this layout, the centrifuges are elevated by 4 to 5 feet to accommodate a chute, which discharges the dewatered cake to a conveyor. The conveyor delivers the dewatered cake into a roll-off container or the bed of a hauling truck.

A small elevated platform allows access to the centrifuge for inspection and maintenance

3.3.3 Screw Press

A screw press system is another continuously operated sludge dewatering technology.

Feed solids are dewatered by a combination of gravity drainage, at the inlet of the screw, and pressure, which is created by conveying the material along a rotating shaft toward the outlet as the interior size of the equipment decreases. Solids are loaded into the bottom of the unit, where they pass through a continually decreasing volume due to an enlarging cone screw. This increases the pressure along the length of the screw press, separating the solids from the liquids and forcing the liquid through the screen.



CONCEPTUAL SITE LAYOUT FOR BELT FILTER PRESS

FIGURE 3.5

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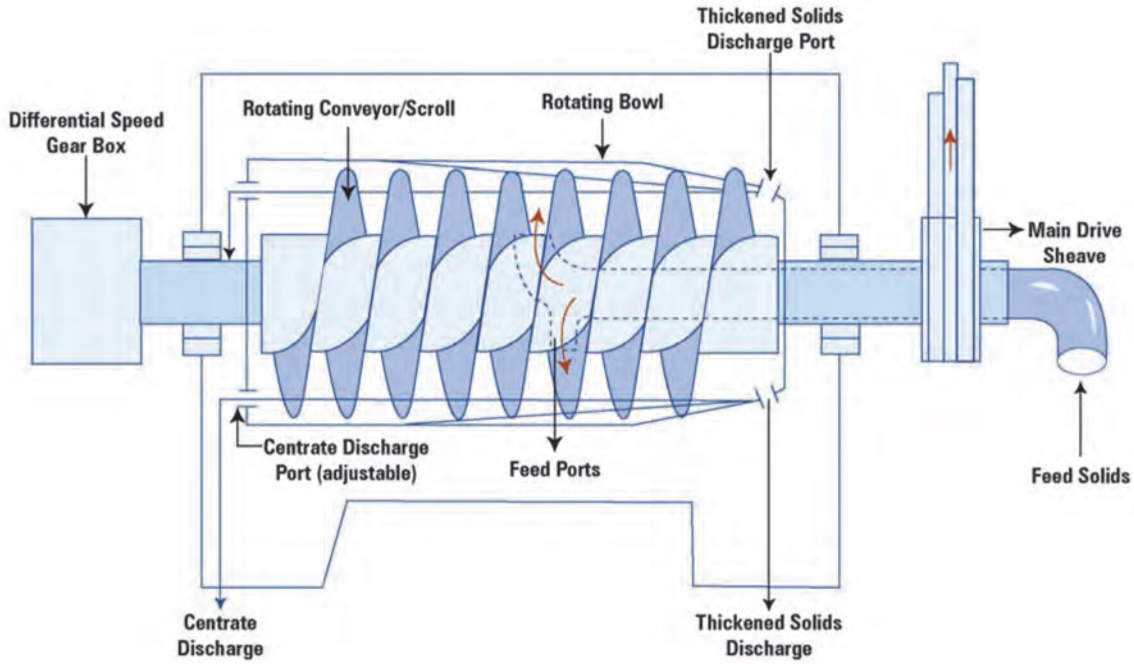

(Courtesy of Alfa Laval)

CENTRIFUGE SCHEMATIC AND PICTURE

FIGURE 3.6

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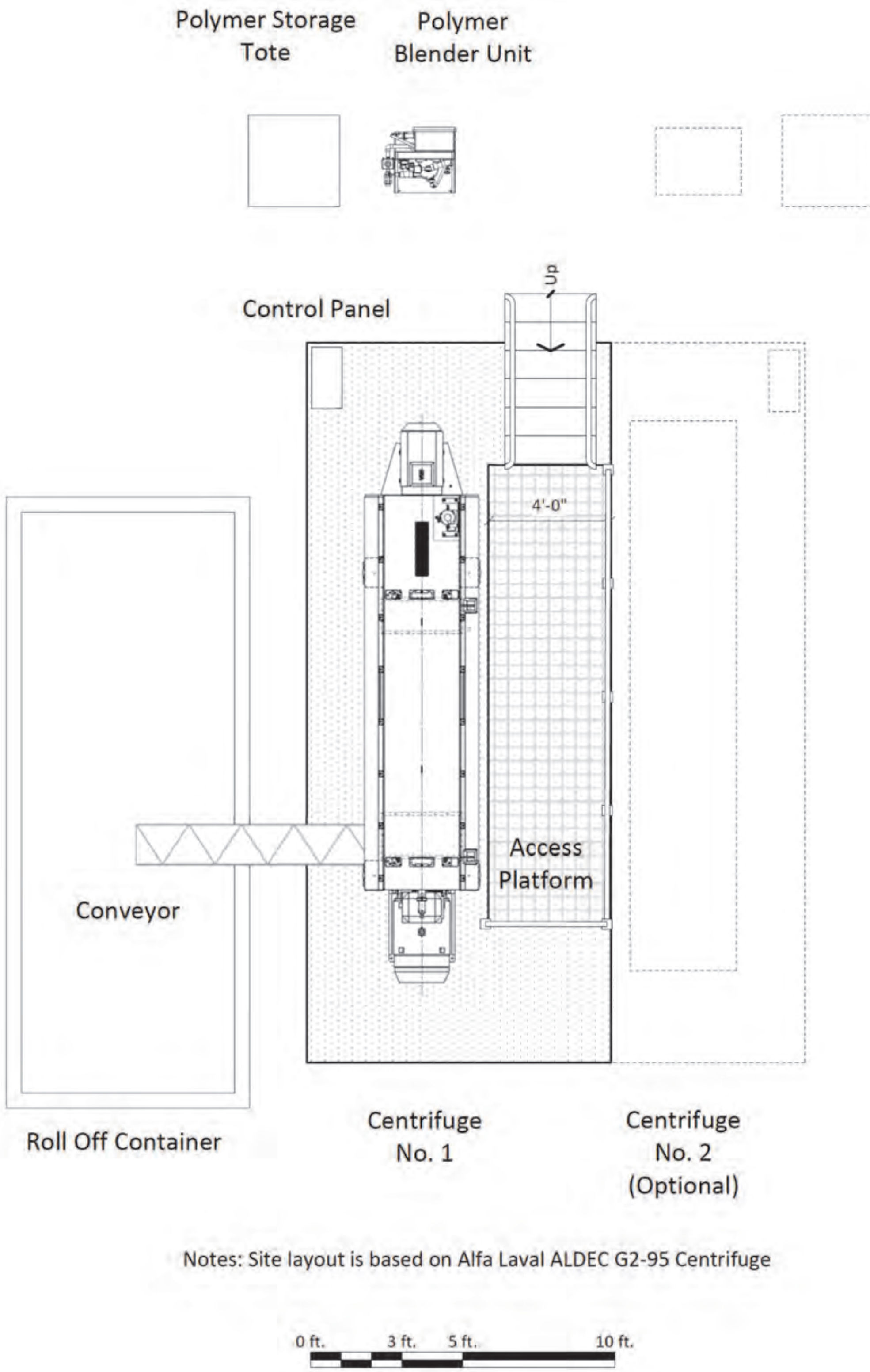


CONCEPTUAL SITE LAYOUT FOR CENTRIFUGE

FIGURE 3.7

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

A screw press typically produces 15 to 20 percent solids content dewatered solids and pressate. The separated water (pressate) is collected and discharged at the bottom of the screw press, while the dewatered cake is discharged at the end of the screw press.

Pressate is typically returned to the main treatment process. Primary screw press manufactures in the current market include FKC, Huber, PWTech, and Schwing.

Screw presses are available in two different configurations – horizontal and inclined. Figure 3.8 illustrates horizontal and inclined screw press schematics.

Figure 3.9 shows the conceptual layout for a screw press. In this site layout, the screw press is installed at grade level without an elevated access platform. A conveyor is used to collect dewatered cake from the screw press and deliver it to a roll-off container or the bed of a hauling truck.

3.3.4 Rotary Fan Press

The rotary fan press is a relatively new sludge dewatering technology with only limited installations in Arizona.

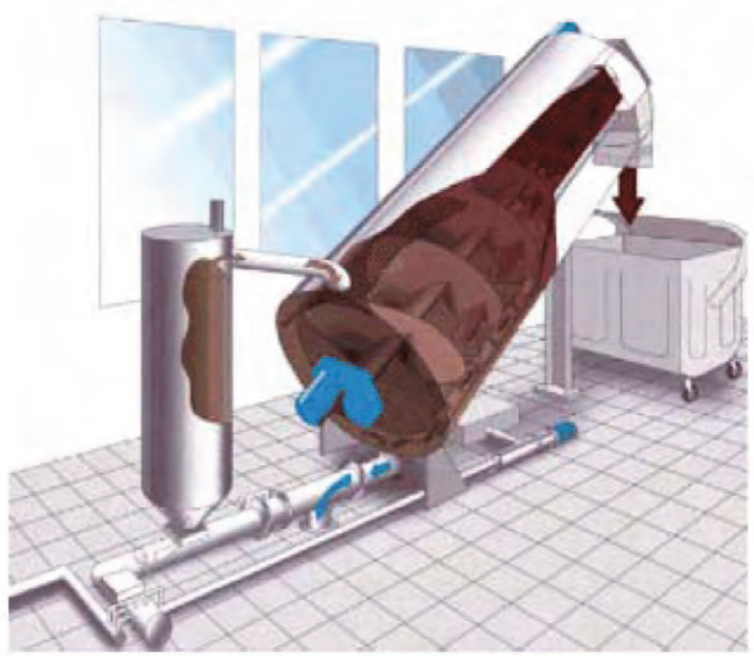
The rotary fan press operates using the low differential pressure between the incoming conditioned sludge and the outgoing sludge cake combined with the very slow (< 1 revolutions per minute [rpm]) rotational motion of the filter screens to advance the sludge through the press. (Water will seek the path of least resistance through the filter screens.) As the conditioned sludge enters the annular space between the two wedge wire filter screens, a pressure differential develops within the press and the liquid portion of the conditioned sludge seeks the path of least resistance through the filter screens.

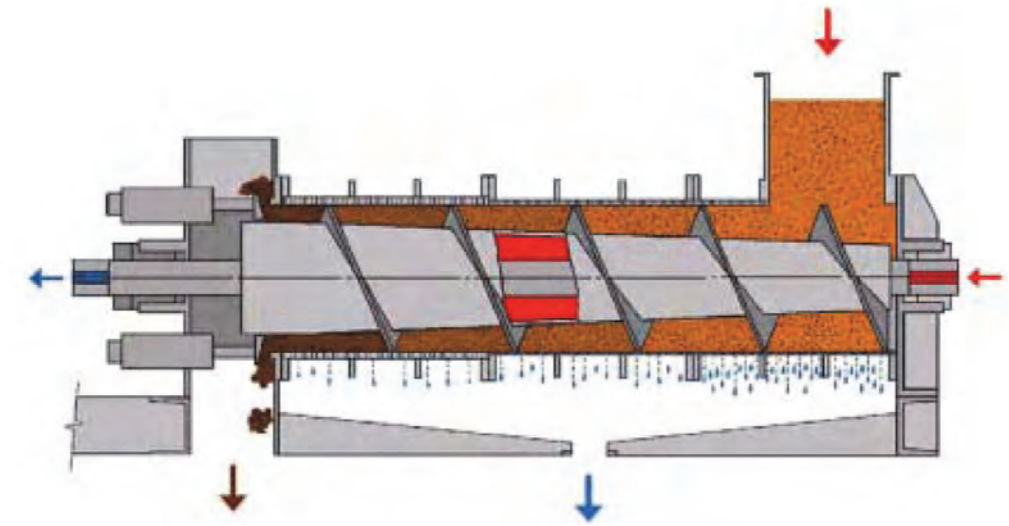
The remaining solids are collected inside the two filter screens traveling toward the solids discharge of the press. At the discharge of the press an adjustable restrictor arm slows down the solids, forming a "cake" plug. As the plug builds within the restriction discharge area, it pushes toward the inside walls of the filter screens and the slow rotation/friction of the filter screens continuously moves the cake solids past the restrictor's arm to be discharged for disposal or further processing.

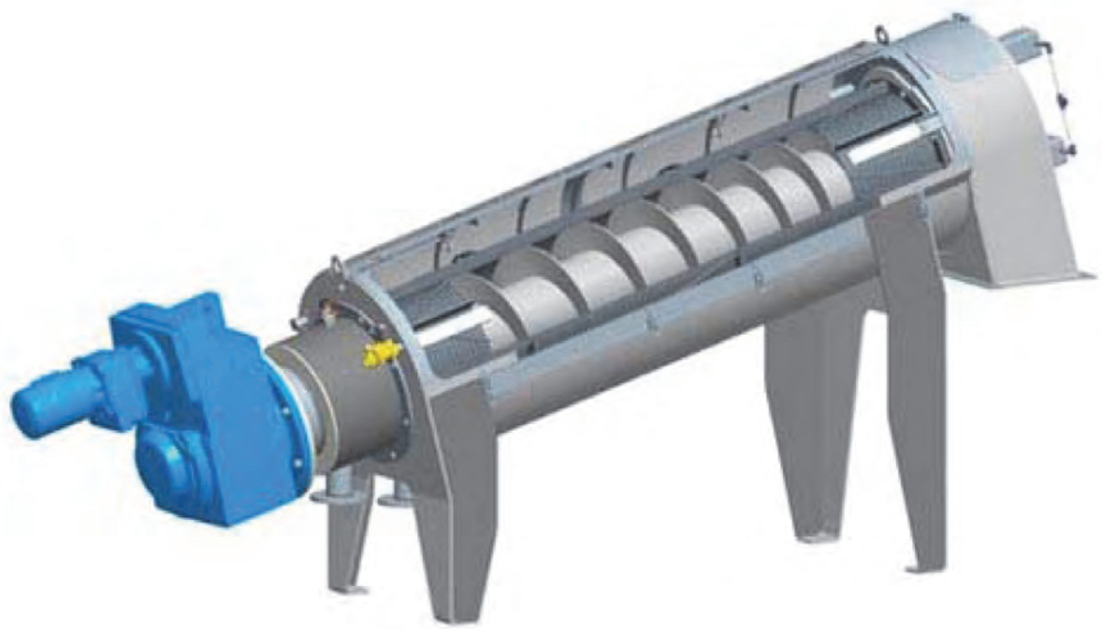
Operation of the rotary fan press can either be continuous or intermittent depending on the application. Primary rotary fan press manufactures in the current market include Prime Solution and Fournier.

Figure 3.10 illustrates rotary fan press schematics.

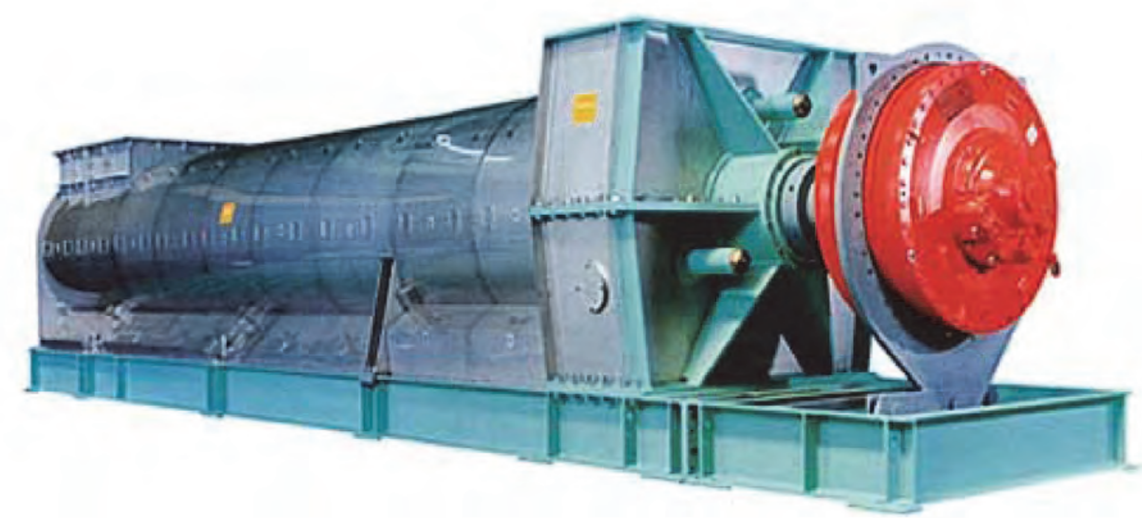
Figure 3.11 shows the conceptual layout for the rotary fan press. The press is installed at grade level. There are multiple discharges from the modular rotary fan press, and a conveyor collects all of the dewatered cake and delivers it to a roll-off container or the bed of the hauling truck.







Inclined Screw Press
(Courtesy of Huber)



Horizontal Screw Press
(Courtesy of FKC)

SCREW PRESS SCHEMATIC AND PICTURE

FIGURE 3.8

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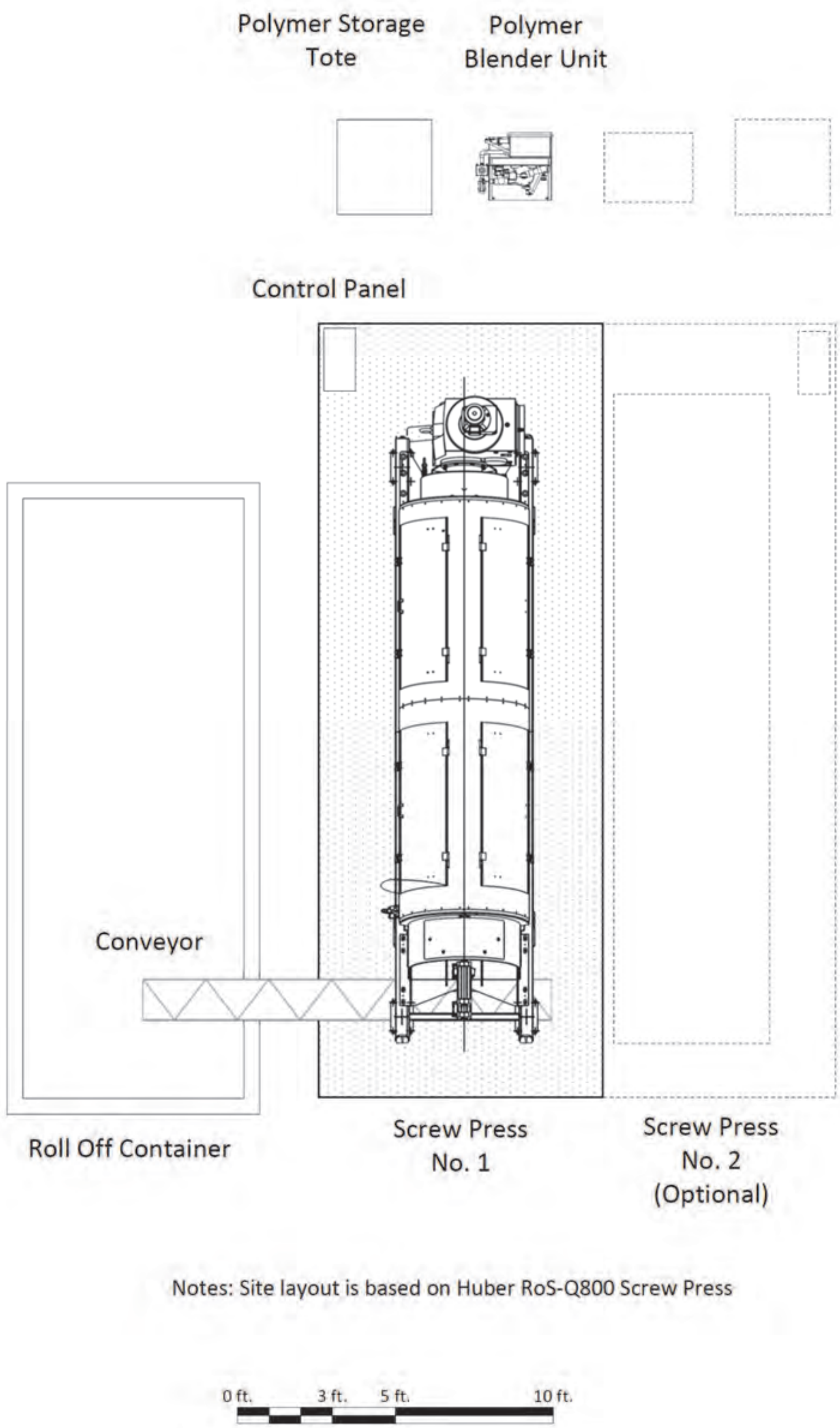
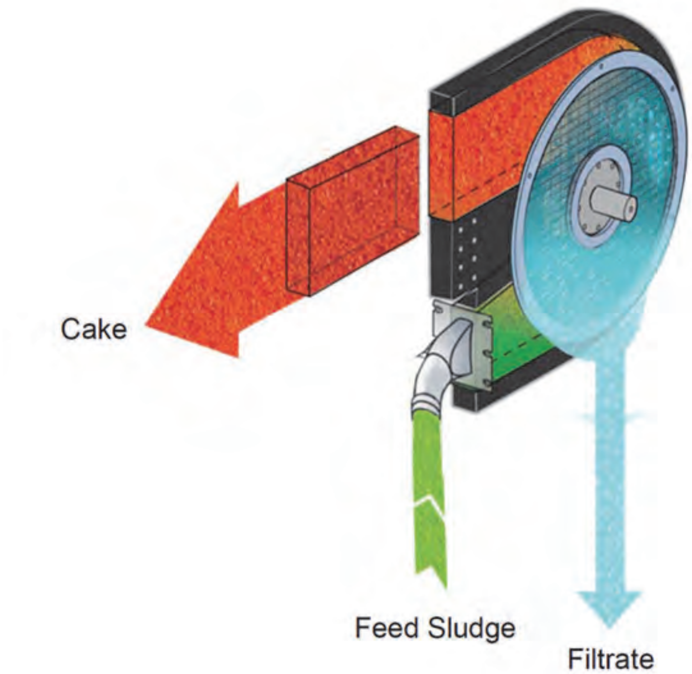
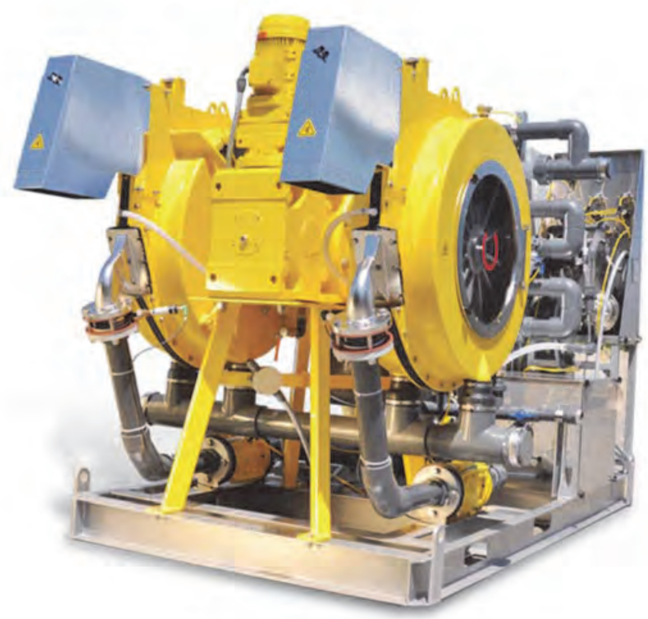


CONCEPTUAL SITE LAYOUT FOR SCREW PRESS

FIGURE 3.9

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN





(Courtesy of Prime Solution)

ROTARY FAN PRESS SCHEMATIC AND PICTURE

FIGURE 3.10

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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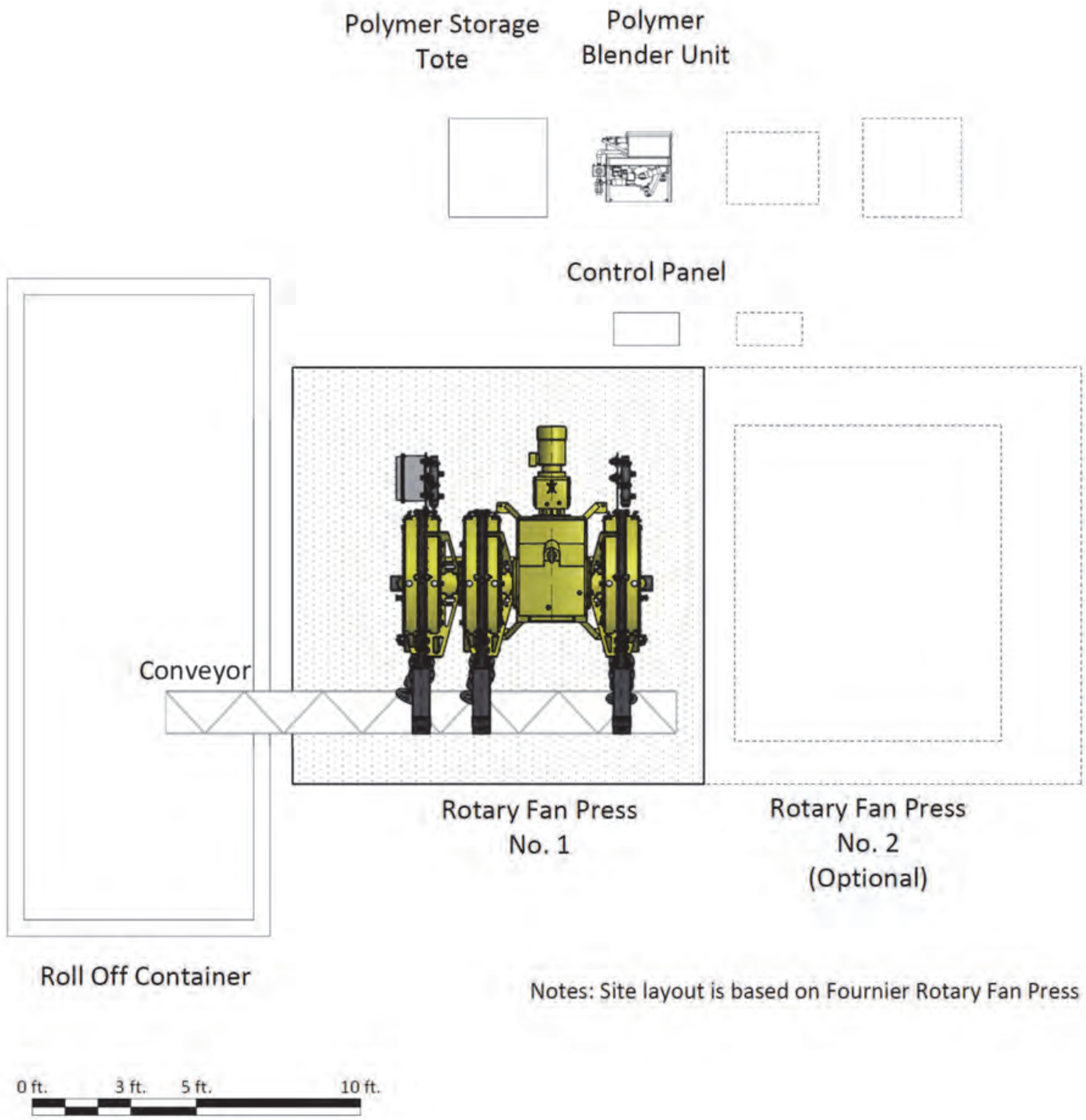


CONCEPTUAL SITE LAYOUT FOR ROTARY FAN PRESS

FIGURE 3.11

3.4 Dewatering Technology Alternatives Comparison

Table 3.4 summarizes the comparison of four dewatering technology alternatives. Design parameters of each alternative are based on the information provided by equipment manufacturers. Following are the key findings from our technology comparison.

- Multiple major manufacturers are available for each alternative. Using a competitive bid process is recommended for efficient and cost-effective purchasing, regardless of the technology selected.
- The BFP, centrifuge, and screw press have good records of installations locally and nationwide. The rotary fan press is a relatively new technology, with limited numbers of installation in Arizona. However, some current Arizona users offer positive feedback for rotary fan presses because of the small footprint, operational convenience, and relatively lower maintenance needs.
- The BFP and screw press have the advantage of lower power consumption than the centrifuge and rotary fan press.
- The BFP and centrifuge show the highest hydraulic loadings, which could potentially reduce daily operational hours compared with the screw press and rotary fan press.
- The screw press and rotary fan press require higher polymer dosages than the BFP and centrifuge. However, the dosages are based on the preliminary estimate of the equipment manufacturers without knowing detailed characteristics of the sludge. Field or lab testing is always recommended to optimize the polymer dosage, regardless of technology selection.
- The BFP requires a large amount of wash water during operation (60 gpm of continuous flushing), while the other three alternatives require significantly less water for multiple times cleanings every day.
- All alternatives show good solids capture, ranging from 93 to 98 percent, producing dewatering cake with solids content ranging from 14 to 24 percent. All alternatives should be able to produce dewatered cakes that can pass the paint filter test for land fill. However, like polymer dosage, solids capture and solids content of the cake need to be optimized along with other operation parameters during the startup and operation.

Table 3.4 Dewatering Technology Alternatives Comparison
Marana Water Reclamation Facility Master Plan
Town of Marana

Criteria	Belt Filter Press	Centrifuge	Screw Press	Rotary Fan Press
Major Manufacturers	Ashbrook Andritz	Alfa Laval Andritz Westfalia	Huber FKC	Prime Solutions Fournier
Local Installation in Arizona	Yes	Yes	Yes	Limited
Estimated Horsepower (1), hp	11.5	65	8.2	21
Estimated Hydraulic Loading (1), gpm	160	170	103.4	138
Estimated Solids Loading (1), dry-lb/hour	560	765	336	420
Operation Days per Week	5	5	5	5
Recommended Polymer Dosage, active lb/dry-ton	13	18	20 - 28	20
Required Washwater Flow, gpm	60	8~40	103	40
Cleaning Time and Frequency	Continuous	2 times/day	60 seconds 3 times/hr	5 minutes 4 times/day
Estimated Water Use (2) (gpd/each)	17,000 - 26,000	~2,000	~2,500	~800
Expected Solids Capture	~93%	~95%	~94%	~98%
Expected Cake Solids Content	16% - 17%	18% - 20%	14% - 24%	17% - 19%

Notes:

(1) Design parameters of belt press filter are based on Ashbrook Klampress 1.5 meter BFP. Design parameters of centrifuge are based on Alfa Laval ALDEC G2-95. Design parameters of screw press are based on Huber RoS-Q800. Design parameters of rotary fan press are based on Fournier Model 6-900/6000CV, six-channel unit. All hydraulic loadings and solids loadings are based on ~0.65 percent feed sludge.

(2) Estimated water used is based on 8 hours operation per day

Table 3.5 shows the preliminary design criteria for Phase 1 to 3 for all four technology alternatives. Following are the key findings from preliminary sizing.

- One piece of dewatering equipment is installed due to budget constraints in Phase 1. The repurposed package plants can provide 3 to 4 days WAS storage, allowing the dewatering equipment to be offline temporarily for maintenance or repair. Keeping flexibility in the current sludge hauling contract is also recommended to retain the ability to waste un-dewatered sludge to Casa Grande WRF (current practice) if the dewatering equipment should need to be offline for an extended period of time. The second unit can be purchased later, when additional funding is available.
- In Phase I, the BFP and centrifuge allow for operation of one unit to treat WAS for less than an 8-hour work day under both AADF and MMADF. The screw press and rotary fan press are required to operate for extended hours (>8 hours per day) under MMADF due to relatively low hydraulic loading. However, screw presses or rotary fan presses can operate unattended for extended periods of time during daily operation, according to the inputs of current users and manufacturers. Additionally, as discussed in Section 3.1, the daily operation hours of dewatering can be reduced if decanting is utilized to reduce WAS volume in the re-purposed package plants.
- In Phase 2 and 3, a fully redundant unit is recommended regardless of the technology selected. The BFP and centrifuge require to operate duty units only to meet the operation schedule (< 8 hours per day) under both AADF and MMADF. In comparison, the screw press and rotary fan press need to operate both duty and standby units under MMADF to meet the operation schedule, as shown in Table 3.5.
- When the treatment capacity of WRF is further expanded in Phase 2 and 3, the proposed area shown in Figure 3.2 is not enough to house additional dewatering facilities. Expanding the canopy and concrete pad are recommended to accommodate more dewatering equipment.

  As discussed in Section 3.1, new WAS storage tanks are recommended since the package plant can no longer provide enough WAS storage time in Phase 2 and 3. Therefore, the package plant can be removed to create space for dewatering facility expansion at that time.

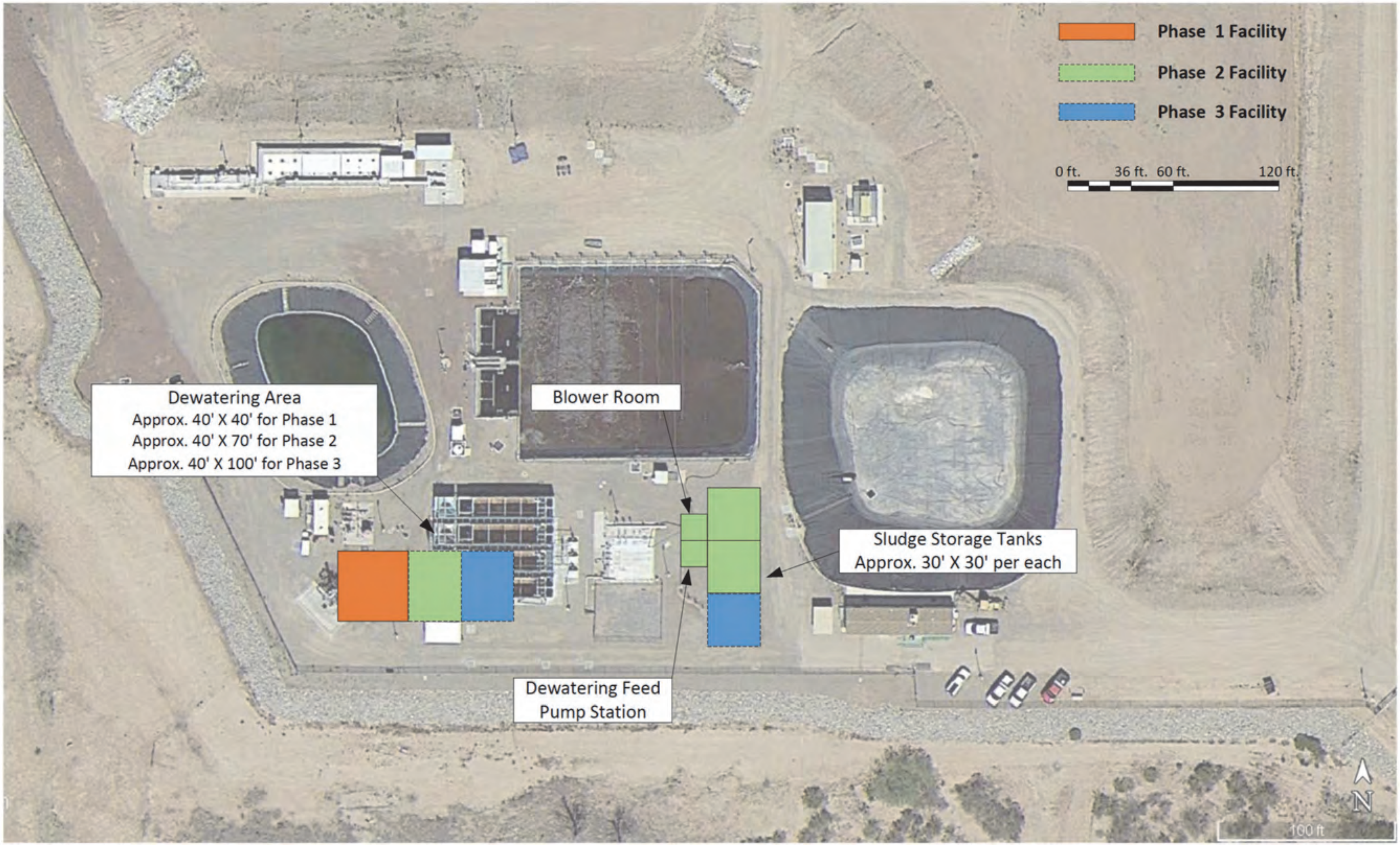
  Figure 3.12 illustrates the conceptual layout of WAS storage and dewatering facility in Phases 2 and 3.

Table 3.5 Preliminary Sizing of Dewatering Technology Alternatives Marana Water Reclamation Facility Master Plan Town of Marana			
Comparison Criteria	**Phase 1**	**Phase 2**	**Phase 3**
WAS Flow, mgd			
AADF	0.033	0.049	0.088
MMADF	0.049	0.078	0.126
Dewatering Operation Schedule		5 days/week, 8 hours/day	
Technology Alternative 1		**Belt Filter Press**	
Required No. of Equipment			
Duty	1	2	3
Standby	0	1	1
Total	2	3	4
Estimate Operation Hours per Day			
AADF	4.8	4.5	4.3
MMADF	7.1	6.7	6.2
Technology Alternative 2		**Centrifuge**	
Required No. of Equipment			
Duty	1	2	3
Standby	0	1	1
Estimate Operation Hours per Day			
AADF	4.6	3.4	4.0
MMADF	6.7	5.3	5.8
Technology Alternative 3		**Screw Press**	
Required No. of Equipment			
Duty	1	2	3
Standby	0	1	1
Estimate Operation Hours per Day			
AADF	7.5	7.2	6.9
MMADF (2)	11.0	7.2	7.5
Technology Alternative 4		**Rotary Fan Press**	
Required No. of Equipment			
Duty	1	2	3
Standby	0	1	1
Estimate Operation Hours per Day			
AADF	5.8	4.8	5.7
MMADF (2)	8.7	5.0	6.1

Notes:

(1) Estimate daily operation hours of each technology alternative are calculated using hydraulic and solids loading listed in Table 3.4

(2) Daily operation hours of screw press and rotary fan press under MMADF in Phase 2 and 3 are calculated assuming all units (duty and standby) are online including standby unit.



CONCEPTUAL SITE LAYOUT OF DEWATERING FACILITY IN PHASE 2 AND 3

FIGURE 3.12

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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pw:\\Carollo/Documents\Client/AZ/Marana/10067A00/Deliverables/TM03/Graphics\Figure 3-12

Table 3.6 summarizes the relative advantages and disadvantages of the each dewatering technology alternative evaluated.

Table 3.6 Advantages and Disadvantages of Dewatering Technology Alternatives
Marana Water Reclamation Facility Master Plan
Town of Marana

Advantages	Disadvantages
Belt Filter Press	
• Relatively low capital cost • Relatively low energy requirements • Many local installations	• Larger footprint • Unit is open to the atmosphere, increasing odor potential • Unattended operation not recommended • Requires large amount of high-pressure spray water/high filtrate volume
Screw Press	
• Relatively low energy requirements • Small unit footprint • Simple operation and maintenance • Relatively low wash water requirement • Contained process minimizes odor considerations and housekeeping	• Relatively low hydraulic loading • Relatively low solids capture rate and cake solids concentration • Higher polymer usage • Limited number of local installations
Centrifuge	
• Relatively small footprint • Higher solids capture rate and cake solids concentration • Contained process minimizes odor considerations and housekeeping	• Relatively high capital cost • Relatively high energy requirements • May require more operator attention • More sophisticated maintenance required compared to others
Rotary Fan Press	
• Relatively simple operation • Smaller unit footprint • Higher solids capture rate and cake solids concentration • Contained process minimizes odor considerations and housekeeping • Relatively low wash water requirement	• Relatively high capital cost • Relatively high energy consumption • Relatively high polymer usage • Relatively new technology. Limited number of local installations

3.5 Estimated Construction, O&M and Life Cycle Cost

A conceptual (planning level) cost estimating exercise was performed to estimate the capital costs, annual operations and maintenance (O&M) costs, and a 10-year life cycle cost for each of the dewatering technology alternatives evaluated. This cost estimate was developed on a conceptual level to compare the technology based on preliminary process sizing, partial quantity take-off, equipment quotes from vendors, reference project cost, and assumptions made for direct and indirect cost components.

Table 3.7 shows the estimated costs for all technology alternatives we evaluated. Key assumptions for the cost estimate include:

- Cost estimate is based on estimated sludge production in Phase 1 as shown in Table 3.1.
- Capital cost includes repurposing existing package plants for WAS storage, removing existing sludge storage tank, new dewatering facilities, and required site construction and yard piping.
- All dewatering equipment and facilities are located in an approximately 40-foot by 40-foot area where the existing sludge storage tank is located, as shown in Figure 3.2. Dewatering equipment is installed on an 18-inch concrete slab at grade level. A metal canopy is installed in the dewatering area. No building will be constructed.
- Dewatered cake is discharged to a roll-off container by using a conveyor. No cake storage is installed in Phase 1.
- One dewatering equipment unit is installed in Phase 1. Space and structure are planned and reserved for a second unit.
- Chemical storage totes are used for polymer solution. No permanent chemical storage tank(s) is installed in Phase 1.
- A drain pipe is connected to the existing 4-inch drain line south of package plants to return filtrate / centrate and wasted wash water back to the main treatment stream of WRF. No drain pump station is installed in Phase 1.
- WMI or a third-party contractor will provide the roll-off container. The cost for purchasing the containers is not included.
- Annual differential O&M costs include dewatering equipment power costs, polymer costs, daily operation labor, maintenance labor, and replacement parts costs. The costs for other O&M activities are considered similar among all technology alternatives and are not included in the O&M cost estimate in this TM.
- 10-year life cycle cost is based on 3 percent annual interest.

Table 3.7 Conceptual Cost Estimate for Dewatering Technology Alternatives
Marana Water Reclamation Facility Master Plan
Town of Marana

Cost Items	Belt Press Filter	Centrifuge	Screw Press	Rotary Fan Press
Capital Cost				
General Conditions	$94,000	$123,000	$101,000	$107,000
Site Construction	$102,000	$132,000	$110,000	$115,000
Structure	$78,000	$78,000	$74,000	$74,000
Equipment and Mechanicals	$654,000	$954,000	$734,000	$787,000
Electrical, Instruments and Control	$131,000	$191,000	$147,000	$158,000
Contingency (25%)	$335,000	$440,000	$362,000	$381,000
Total Project Cost	**$1,394,000**	**$1,918,000**	**$1,528,000**	**$1,622,000**
Annual Differential O&M Cost				
Power Cost	$1,700	$6,900	$900	$3,700
Chemical Cost	$29,200	$40,400	$53,900	$44,900
Operation Labor	$48,800	$48,800	$19,500	$19,500
Maintenance Labor	$24,900	$16,500	$3,900	$4,400
Replacement Parts	$1,400	$4,800	$1,800	$4,800
Annual Differential O&M Cost	**$106,000**	**$117,400**	**$80,000**	**$77,300**
10-year Life Cycle Cost	**$2,254,000**	**$2,871,000**	**$2,177,000**	**$2,249,000**

Notes:

(1) Capital cost and O&M cost are developed for Phase 1 capacity.
(2) The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.
(3) All costs are 2015 dollars. Engineering News-Record (ENR) Index (20-city average) of Nov. 2015 = 10092.

Following are the key findings from our conceptual cost estimate for dewatering alternatives.

- The BFP has the lowest installed capital cost of other technology alternatives due to its lower equipment cost. However, the BFP requires a significantly large volume and rate of wash water, which could further negatively affect the liquids treatment process which has been already affected by a large wash water return from the filters.
- The screw press and rotary fan press show lower annual differential O&M costs than the BFP and centrifuge because they require significantly less labor for daily operation and routine maintenance and have a lower power cost compared to the centrifuge. Both technologies use minimal amounts of wash waster for their process.
- The centrifuge has a significantly higher power cost than the other options due to the high horsepower it requires. The centrifuge also has the highest installed capital cost. The high O&M and life cycle cost of the centrifuge do not make it an economical option for WRF and it is not recommended.
- The screw press and rotary fan press require a slightly higher polymer usage than the BFP and the centrifuge, based on the manufacturer's recommendation. However, the recommended polymer usages are based on conservative preliminary estimates without knowing the actual characteristics of the WAS. The actual polymer usage may be reduced when operation parameters are further optimized.

3.6 Matrix Evaluation of Dewatering Technology Alternatives

For the matrix evaluations, a list of evaluation criteria and their relative scores were developed and reviewed during the Technology Alternative Workshop with Town staff to evaluate alternative technologies and unit processes. A total of 10 criteria were used for comparing the alternatives. Each of the alternatives was evaluated, with "--" being the worst, and "++" being the best.

Table 3.8 shows the results of matrix evaluation of the four dewatering technology alternatives.The results show that the centrifuge has good scores in treatment performance but requires high capital cost and lots of operator attention during daily operation. The BFP shows advantages in low capital cost and low power and chemical cost but has low scores in operation and maintenance requirements. The screw press and rotary fan press show medium capital cost but have advantages in ease of operation and less maintenance required.

Overall, the screw press and rotary fan press show higher scores in the matrix evaluation, making them the preferred options for WRF. The screw press and rotary fan press are recommended for further consideration during design phase.

Table 3.8 Dewatering Technology Alternatives Matrix Evaluation Marana Water Reclamation Facility Master Plan Town of Marana				
Criteria	**Belt Filter Press**	**Centrifuge**	**Screw Press**	**Rotary Fan Press**
Continuous Operation	+	+	+	+
Totally Enclosed	-	+	+	+
High Cake Dryness	-	++	+	+
Start-Stop Easy Procedures	-	--	+	+
Low Power Use	+	--	+	-
Small Footprint	-	+	+	+
Low Wash Water Requirement	--	+	++	++
Low Chemical Usage	++	+	-	-
Low Maintenance Requirement	-	-	+	+
Low Capital Cost	+	--	-	-

Note:
(1) "++" is the best; "--" is the worst

3.7 Cake Conveyance

In the dewatering facility site layout presented in previous sections, dewatering equipment is assumed to have been installed on an elevated platform to discharge dewatered cake to the roll-off container directly without a conveyor. However, if a truck is used during daily operation instead of a container, a conveyor is recommended to prevent dewatering equipment from being raised to a high elevation (> 15 feet above grade).

The dewatering operation shall produce dewatered cakes with an expected solids content of 15 to 20 percent or greater. The cakes within that range of solids content will not flow by gravity in a pipe or channel from the dewatering equipment to cake storage or hauling truck. Instead, it must be transported from the dewatering equipment by either mechanical means (e.g., conveyors or pumping) or gravity drop into a storage container or truck that is positioned directly under the dewatering equipment.

This section briefly evaluates dewatered sludge/biosolids conveyance alternatives, including a belt conveyor and screw conveyor. Cake pump is another cake conveyance option but is not included in this evaluation because a conveyor is preferred by the Town.

Figure 3.13 shows examples for both belt and screw conveyors. Table 3.9 summarizes the relative advantages and disadvantages of two alternatives.

Belt Conveyor

Screw Conveyor

BELT CONVEYOR AND SCREW CONVEYOR

FIGURE 3.13

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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Table 3.9 Advantages and Disadvantages of Cake Conveyance Alternatives Marana Water Reclamation Facility Master Plan Town of Marana	
Advantages	**Disadvantages**
Belt Conveyor	
• Relatively low capital cost • Relatively low energy requirements • Durable operation • Relatively simple operation and maintenance • Operations staff familiarity	• Potential problems with sludge carryover causing belt tracking issues. • Requires cover for odor control • Requires sidewalls to stop cake spillage
Screw Conveyor	
• Ability to handle varying flow rates and consistencies of dry solids contents. • Complete enclosure minimizes odor considerations and housekeeping • Can go steep or vertical, good for small footprint • Can be outfitted with multiple discharge points • Minimum ragging or blockage • Low operator intervention • No hanger bearings, less moving parts to wear (in case of shaftless screw)	• Lumpy, fibrous, or sticky materials may cause problems. • Power requirements can be high with solids that tend to pack • Conveying efficiency is considerably reduced when screws are inclined or mounted vertically • Thyrotrophic transformation of the biosolids cake

Belt Conveyor

A belt conveyor is one of the common means of transportation for bulk solids and is capable of carrying a great diversity of products at high rates and long distance. Belt conveyors use a belt to transport the material and can be fed in one or multiple inlet areas. Belt conveyors can be flat, inclined, cleated, or equipped with a trough, side wall, scraper, and/or cover, etc. The bulk material rides on top of the belt and is contained by the belt trough or side walls. Primary belt conveyor suppliers in the current market include Serpentix Conveyor Corporation and Custom Conveyor Corporation.

Screw Conveyor

A screw conveyor consist of an external housing and an inside rotating spiral. It can be shafted or shaftless, and can be employed in horizontal, inclined, or vertical installation. Shafted screw conveyors have the spiral coiled around a center shaft, driven at one end, and held at the other. The screw is supported by bearings at trough ends. Intermediate bearings (hanger bearings) can be used to limit the deflection of the spiral, if the shaft is long enough to require additional support. Shaftless screws are driven at one end and free at the other end, and usually have the spiral supported on a sacrificial wear liner. Each type has its own advantages and disadvantages, which include the following:

- Screw conveyors convey materials on a volumetric basis. Shafted screws usually have a limitation of spiral fill level, at around 30 percent maximum, to keep the product away from the hanger bearing of the screw conveyor. For the same design capacity, shafted screws require a larger housing diameter than shaftless conveyors.
- Over time of operation, shafted conveyors require bearing replacement, and shaftless conveyors require liner replacement. Liner replacement can be challenging to perform while leaving the spiral in place; whereas bearing replacement is relatively easier in terms of maintenance convenience.

Primary screw conveyor suppliers in the current market include Custom Conveyor Corporation and JDV Equipment Corporation.

3.8 Cake Hauling and Disposal

Currently, the Town hires a private contractor to haul the liquid waste sludge from the existing WRF storage tank to the City of Casa Grande WRF for further treatment and disposal. Casa Grande WRF is more than 50 miles away from Marana, resulting in a high hauling fee, which represents the second highest operational cost for the WRF.

With the use of dewatering equipment, the Town could haul dewatered solids to the nearby Marana Regional Landfill for disposal. Before disposing of the wastewater treatment residual sludge at a landfill, dewatering of the liquid waste solids is required to meet the "paint filter" liquids test (EPA Method 9095B), which determines the presence of free liquids in a representative sample of waste. Most dewatering equipment can reliably meet this limit by producing sludge with a minimum of 15 percent total solids.

The Marana Regional Landfill is operated by WMI and is located about 10 miles south of WRF.

Figure 3.14 shows the locations of Casa Grande WRF, Marana WRF, and Marana Regional Landfill.

LOCATIONS OF WATER RECLAMATION FACILITY AND LANDFILL

FIGURE 3.14

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

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Table 3.10 summarizes the estimated hauling and disposal costs for both current operation and future WRF capacities.

Total solids disposal costs also can be reduced significantly by the dewatering process due to both the reduced volume of sludge and the reduced hauling distance.

As shown in Table 3.10, dewatering facilities could reduce the current solids disposal cost by an estimated 80 percent. Total solids disposal cost at the Phase 1 plant capacity would be approximately 30 percent lower than the current operation, even when treating about three times more wastewater.

It should be noted that the costs in Table 3.10 were based on an initial cost proposal by WMI of $35.25 per ton of sludge (plus 17 percent variable fees). The Town has indicated it may have made an agreement with WMI in which disposal fees would be waived or at least significantly reduced. Significantly reducing disposal costs to the landfill would make installing dewatering facilities even more economically attractive and shorten the return-on-investment period.

**Table 3.10 Estimated Costs for Cake Hauling and Disposal
Marana Water Reclamation Facility Master Plan
Town of Marana**

Comparison Criteria	Current Operation	Current Operation w/ Dewatering	Phase 1	Phase 2	Phase 3
Plant Flow AADF, mgd	0.355	0.355	1.2	2.4	3.6
Cake Production(1), cy/year	10,114	987	3,335	6,903	9,675
Cake Solids Content, %	1.0% - 1.2%	14% - 20%	14% - 20%	14% - 20%	14% - 20%
Required No. of Truck Hauling(2), ea/year	371	-	-	-	-
Required No. of Roll Off Containers(3), ea/year	-	33	111	230	323
Solids Disposal Destination	Casa Grande WRF	Marana Landfill	Marana Landfill	Marana Landfill	Marana Landfill
Truck Driving Distance(4), one way, miles	54	10	10	10	10
Estimated Annual Hauling Cost(5)	$169,755	$3,300	$11,200	$23,100	$32,300
Estimated Annual Treatment/ Landfill Cost(6)	$20,634	$35,993	$121,668	$251,814	$352,959
Total Solids Disposal Cost	$191,000	$40,000	$133,000	$275,000	$386,000

Notes:
(1) Sludge production of current operation is based on 5,500 gallons per day, 7 days per week. Cake production of Phase 1 - 3 are based on WAS production under AADF of each phase estimated by biological modeling. Cake production for current operation with dewatering is linearly calculated based on Phase 1 sludge production.
(2) Truck capacity is 5,500 gallon based on invoice on August, 2015 from Synagro West LLC.
(3) It is assumed to use 30-cubic yard roll off container.
(4) Driving distance is estimate by using Google Maps.
(5) It uses $485 hauling fee and $28 fuel surcharge per load for current operation based on invoice on August, 2015 from Synagro West LLC. It is assumed $100 hauling fee/load for future phases.
(6) Sludge treatment cost is $10.1/1000 gallon based on invoice of September, 2014, Casa Grande WRF. Landfill fee is $35.25 per ton plus 17% variable fees, proposed by WMI.

4.0 CONCLUSION AND RECOMMENDATIONS

A summary of findings and recommendations from this dewatering technology alternative evaluation are as follows.

1. Repurposing the existing package plant tanks for WAS storage is a feasible and economical option to provide the needed WAS storage required for the WRF Phase 1 expansion. By refurbishing and repurposing the four steel tanks, more than 4 days of WAS storage is provided under AADF conditions. Under MMADF, 3 days is provided. WAS storage gives significant operational flexibility for dewatering and sludge hauling. Also, decanting the stored liquid waste solids can reduce the volume of solids to be dewatered.
2. Based on this technology alternative evaluation, further consideration of the screw press and rotary fan press dewatering equipment is recommended during the preliminary design phase. Pilot testing could be performed, if feasible, to determine the optimal polymer usage and familiarize the plant staff with equipment operations.
3. If budget constraints require it, a single dewatering equipment unit could be installed rather than two. A second unit is typically recommended for redundancy but can be postponed until additional funding is available. The facility space and structure would be planned and reserved for the second unit, which the Town could purchase and install in the future. However, provisions for the disposal of liquid waste solids would need to remain in place if the single dewatering unit is out of service for planned or unplanned maintenance.
4. Installing dewatering equipment at grade on a concrete slab is recommended. An inclined conveyor would be required to transfer dewatered cake from the dewatering unit(s) to a roll-off container or the bed of hauling truck.
5. A dewatering facility can be fit into an approximately 40- by 40-foot area, where the existing sludge storage tank is located to make use of the current truck route, which still allows access around the facility. Construction phasing and timing of work will be furthered in the preliminary and detailed design phases.
6. Sludge disposal represents the second highest operations cost in the current operation of the WRF. A dewatering process can significantly reduce the cost for sludge hauling and disposal and provide operational flexibility.

APPENDIX D – TM-4 ALTERNATIVE PROCESS EVALUATION

Registered Professional Engineer (Civil)
CERTIFICATE NO.
41462
ANDREW C.
GILMORE
DATE SIGNED 4.19.16
ARIZONA, U.S.A.
EXPIRES 09-30-2016

Registered Professional Engineer (Civil)
CERTIFICATE NO.
37702
DANA R.
TROMPKE
DATE SIGNED 4.19.16
ARIZONA, U.S.A.
EXPIRES 06-30-2017

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TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

TECHNICAL MEMORANDUM NO. 4 ALTERNATIVE PROCESS EVALUATION

FINAL
April 2016

4600 EAST WASHINGTON STREET, SUITE 500 • PHOENIX, ARIZONA 85034 • P. 602.263.9500 • F. 602.265.1422

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY
MASTER PLAN

TECHNICAL MEMORANDUM
NO. 4
ALTERNATIVE PROCESS EVALUATION

TABLE OF CONTENTS

Page No.

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

A.A.C.	Arizona Administrative Code
AADF	average annual daily flow
APP	Aquifer Protection Permit
AZPDES	Arizona Pollution Discharge Elimination System
BADCT	Best Available Demonstrated Control Technology
BNR	biological nutrient removal
BNR-CAS	biological nutrient removal conventional activated sludge
BNR-OD	biological nutrient removal oxidation ditch
BOD	biochemical oxygen demand
CAS	conventional activated sludge
CBOD	carbonaceous biochemical oxygen demand
cfu/100 mL	coliform forming units per 100 milliliters
COD	chemical oxygen demand
CSF	clarifier safety factor
DO	dissolved oxygen
ENR	Engineering News-Record
F/M	food to microorganism ratio
gpd	gallons per day
HDPE	high density polyethylene
IMLR	internal mixed liquor return
mg/L	milligrams per liter
mgd	millions gallons per day
MLE	Modified Ludzak-Ettinger
mL/g	milliliter per gram
MLSS	mixed liquor suspended solids
MMADF	maximum month average day flow
MPN	most probable number
NH_3-N	ammonia nitrogen
NO_2-N	nitrite
NO_3-N	nitrate
NTU	nephelometric turbidity units
O&M	operations and maintenance
PDF	peak daily flow
PFDs	process flow diagrams
PHF	peak hourly flow
RAS	returned activated sludge
SRT	solids retention time
SVI	sludge volume index
TKN	total Kjeldahl nitrogen
TM	technical memorandum
TN	total nitrogen
Town	Town of Marana
TSS	total suspended solids
UV	ultraviolet
WAS	waste activated sludge
WRF	water reclamation facility

ALTERNATIVE PROCESS EVALUATION

1.0 INTRODUCTION

The Town of Marana (Town) owns and operates the Marana Water Reclamation Facility (WRF), which consists of the following facilities: preliminary treatment (headworks), influent pumping, secondary treatment, secondary effluent pumping, filters, ultraviolet (UV) disinfection, and plant effluent outfall structure. The WRF also includes backup systems for chlorination/dechlorination and auxiliary systems for odor control, utility water, and standby power generation.

The Marana WRF currently operates utilizing a 500,000 gpd Biolac® treatment system. A separate treatment system consisting of four biological nutrient removal (BNR) package plants with a total combined capacity of 200,000 gpd is also located on site, but is not currently in use. The current Aquifer Protection Permit allows the plant to operate up to a maximum month average flow (MMAF) rate of 3.5 mgd, by phases. The existing WRF may operate up to 0.7 mgd, a Phase 1 expansion up to 2.0 mgd, and a Phase 2 expansion up to 3.5 mgd (MMAF).

Effluent is currently being discharged to a tributary of the Santa Cruz River; however, a recharge facility is planned to be located on the property adjacent to the Marana WRF. Construction for the recharge facility is planned to be completed in 2016. Once complete, the Town may recharge its tertiary effluent to the underlying aquifer and accrue recharge credits, which may be used as part of the Town's integrated water supply portfolio.

Waste solids are currently hauled off-site on a daily basis and represent a significant operational cost to the WRF. A detailed evaluation of the solids handing alternatives is provided in Technical Memorandum No. 3 (TM No. 3).

Currently, the Marana WRF is operating at an average daily flow rate of approximately 355,000 gpd, which is 76 percent of the secondary treatment system's capacity. Initial projections anticipated that the Marana WRF may require a capacity of 1.0 to 1.5 million gallons per day (mgd) within the next 10 years. Prior to starting a plant expansion, the Town desires to complete a Master Plan evaluation of the WRF in order to lay out a methodical plan for future phased expansions to meet the needed capacity and to evaluate the most appropriate treatment process to meet the Town's goals.

1.1 Purpose of Technical Memorandum

The purpose of this TM is to evaluate the current and alternative treatment processes for their applicability and feasibility at the Marana WRF, for both the short-term immediate project and the long-term, efficient operation of the WRF. Specifically, the following tasks were included in this evaluation:

- Evaluate applicable treatment technologies, including Biolac® systems, biological nutrient removal oxidation ditch (BNR-OD) per the prior Pima County conceptual design (Stantec, 2008-2009), conventional activated sludge treatment for discharges to the existing recharge ponds and river outfall, and/or potential reclaimed water application. Consideration should be given to effluent nitrogen limitations and future permit requirements.
- Develop process flow diagrams (PFDs) to appropriate level of detail and general site plans for the identified treatment scenarios.
- Develop alternative(s) to sufficiently verify space requirements and establish "order of magnitude costs" sufficient for CIP planning purposes.

2.0 RECLAIMED WATER USES

The WRF produces effluent classified as Class B+ Reclaimed Water as defined in the Arizona Administrative Code (A.A.C.) R18-11-305 and R18-9-206. The quality of the effluent is regulated by Aquifer Protection Permit (APP) No. P-100631.

Under legal authority of the current Arizona Pollution Discharge Elimination System (AZPDES) Permit No. AZ0024520, the WRF discharges treated effluent to an unnamed wash that flows to the Santa Cruz River. The permit was issued on April 13, 2012, and runs through April 12, 2017. At that date, the permit must be renewed. The permit was last modified to allow a discharge flow up to 3.5 mgd.

The Town plans to construct recharge basins on the property immediately east of the WRF. The APP permit has been revised to allow for treated effluent to be recharged at these new recharge facilities. Once the facilities are operating, the Town intends to primarily recharge all of the Class B+ reclaimed water at the new recharge basins and maintain the AZPDES outfall to the Santa Cruz River only as a secondary means of discharge. As part of the recharge basin project and APP permit expansion, additional monitoring wells have been constructed to monitor the groundwater below and hydraulically upstream and downstream of the recharge basins.

The Town's priority for effluent use or disposal is as follows:

1. Recharge all effluent at the recharge basins to accrue reclaimed water storage credits that will be included in the Town's water supply portfolio.
2. Irrigate the property directly south of the WRF for beneficial grass/crop uptake.
3. Discharge to Santa Cruz River.

No recharge credits can be accrued for irrigating the adjacent property or discharging to the Santa Cruz River. However, for operational flexibility, retaining the AZPDES permit, which allows for such discharge, is recommended. The permit can be revised to require sampling and reporting only for discharge events.

3.0 PHASING AND FLOWS

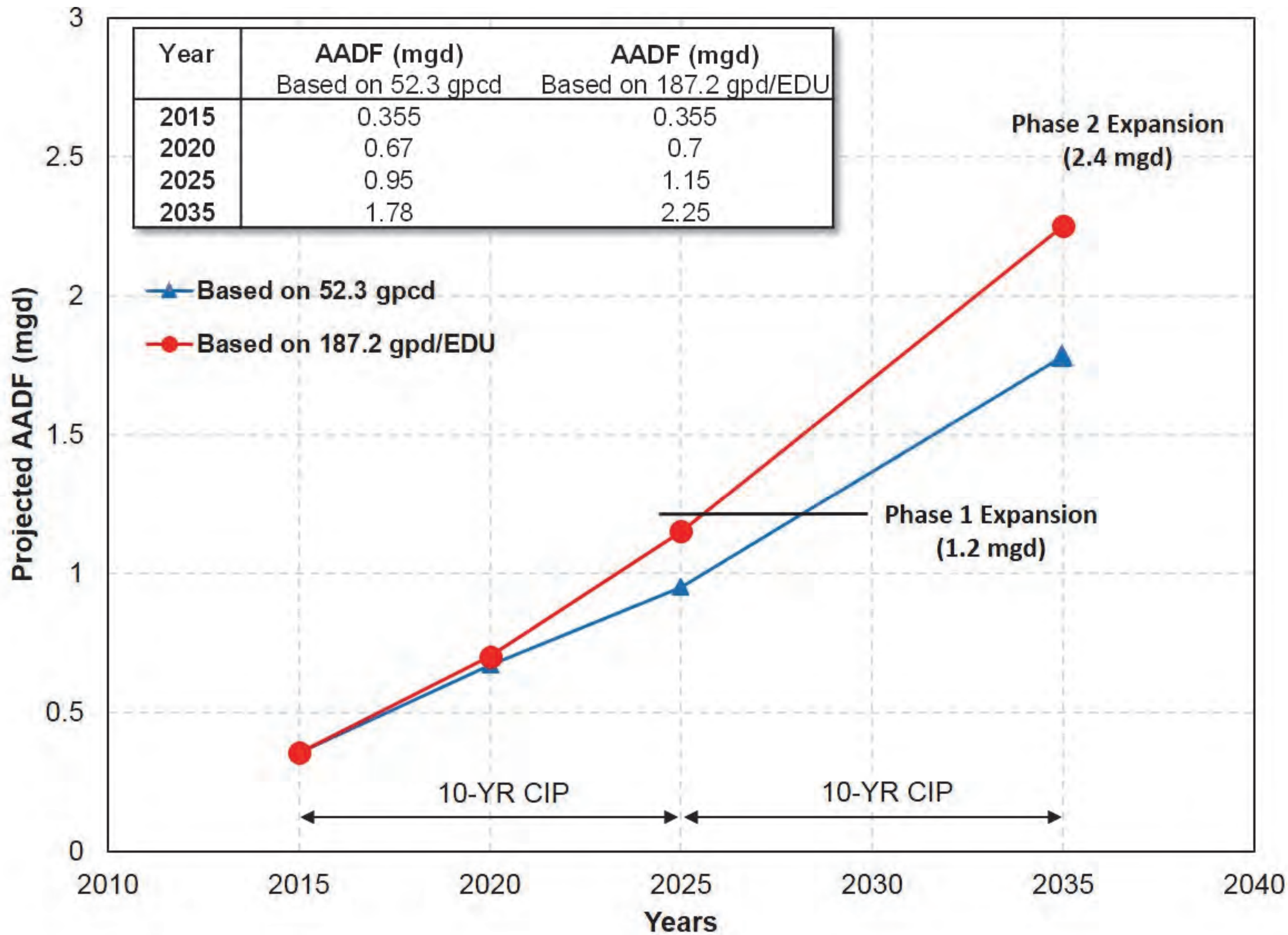
As the Town grows, the WRF expansions will be required to serve population and industrial growth and resulting wastewater flow and/or increased loadings. Therefore, to plan for future services, the projections in this Master Plan are based on wastewater growth trends TM No. 1 - Marana Water Reclamation Facility Flow and Loading Projections includes wastewater flow projections for the next 20 years, including years 2020, 2025, and 2035. These projections are based on growth projections provided by the Town of Marana Planning Department. Figure 4.1 presents the projected annual average daily flow (AADF) in the next 20 years. Table 4.1 summarizes the proposed design capacity of WRF expansion phases.

The Phase 3 expansion with a projected design capacity of 3.6 mgd is included in this evaluation is based on assuming of modular expansion of 1.2 mgd in each phase. Phase 3 is expected to occur later than 2035 and is evaluated here for comparison and site layout purpose. The actual design capacity is subject to change since it is not practical to estimate wastewater flow accurately more than 20 years into the future.

Table 4.1 Design Phases and Projected Capacity
Marana Water Reclamation Facility Master Plan
Town of Marana

Design Phases	Peaking Factors	Phase 1	Phase 2	Phase 3
Year	--	2025	2035	Unknown
AADF, mgd	--	1.2	2.4	3.6
MMADF, mgd	1.1	1.3	2.6	4.0
PDF, mgd	2.0	2.4	4.8	7.2
PHF, mgd	Varies	3.5	6.7	8.3

Abbreviations:
MMADF: monthly maximum average daily flow; PDF: peak daily flow; PHF: peak hourly flow



Year	AADF (mgd) Based on 52.3 gpcd	AADF (mgd) Based on 187.2 gpd/EDU
2015	0.355	0.355
2020	0.67	0.7
2025	0.95	1.15
2035	1.78	2.25

FLOW PROJECTION FOR WRF

FIGURE 4.1

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

carollo

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4.0 PROCESS GOALS

To design an expansion or improvement to a treatment process, a thorough understanding of the wastewater characteristics is important. For this Master Plan, wastewater characteristics were determined by analyzing the plant's historical wastewater quality data. Influent characteristics were obtained from composite samples of the plant influent collected before or after screening and grit removal. The Town provided the following wastewater quality data:

- 5-day biological oxygen demand (BOD): 101 samples taken from April 2012 through November 2015.
- Total suspended solids (TSS): 94 samples taken from April 2012 through November 2015.
- Total Kjeldahl Nitrogen (TKN): 23 samples taken from May 2012 through November 2015.
- Ammonia Nitrogen (NH_3-N): 20 samples taken from May 2012 through March 2015.
- Nitrite and Nitrate: 2 samples taken on October 2015.
- Alkalinity: 20 samples taken from May 2012 through October 2014.

Table 4.2 summarizes the key influent water quality under both AADF and MMADF, which can be used to prepare process evaluation in this TM.

Table 4.2 Wastewater Influent Water Quality
Marana Water Reclamation Facility Master Plan
Town of Marana

Parameter	Units	Under AADF	Under MMADF
BOD	mg/L	228	269
TSS	mg/L	233	297
TKN	mg/L	57	67
NH_3-N	mg/L	42	50
Wastewater Quality Data from April 2012 - March 2015			

To determine wastewater treatment process alternatives for expanding the WRF, effluent quality goals must be enumerated. Per Article 2 – Individual Aquifer Protection Permits of Chapter 9, Title 18 of the A.A.C. new treatment facilities or processes must demonstrate that Best Available Demonstrated Control Technology (BADCT) processes or operating methods are employed to reduce discharge to the greatest degree. In addition to demonstrating BADCT technologies or methods, specific treatment requirements must be met. Table 4.3 summarizes BADCT requirements.

Table 4.3 BADCT Requirements Marana Water Reclamation Facility Master Plan Town of Marana	
BADCT Criteria	**Treatment Requirement**
Setbacks	350 feet (For facilities over 1 mgd, with full noise, odor and aesthetic controls) 150 feet with an ordinance or waiver
Treatment Requirement	Secondary Treatment meeting BOD_5 < 30 mg/L (30-day avg) Or $CBOD_5$ < 25 mg/L (30-day avg)
Total Suspended Solids	< 30 mg/L (30-day avg)
pH	Between 6.0 – 9.0
Removal Efficiency	85% of BOD_5, $CBOD_5$, and TSS
Total Nitrogen	<10 mg/L 5-month rolling mean
Fecal Coliform Limits	Non-detectable in 4 out of 7 daily samples 23 MPN or cfu/100 mL max
Disinfection	Use chlorination-dechlorination, ultraviolet light and/or ozone to achieve pathogen removal and minimize trihalomethane generation

Source:
A.A.C. R18-9-part B, September 30, 2005

Table 4.4 summarizes the treatment and water quality requirements of Class A+ and B+ reclaimed water per A.A.C. R18-11-Article 3, December 31, 2008. It also identifies the treatment goal in addition to the BADCT requirements above. The Marana WRF currently produces Class B+ effluent. At the time of this writing, no changes are proposed to the requirements for treatment and effluent water quality. The treatment goal for future expansion will meet current permit requirements.

Table 4.4	**Arizona Reclaimed Water Classification and Treatment Goal Marana Water Reclamation Facility Master Plan Town of Marana**		
Classification	**A+ (Future)**	**B+ (Current)**	**Treatment Goal**
Allowable Uses	Irrigation of food crops, open-access irrigation, fire protection systems, vehicle washing, snowmaking	Surface irrigation of orchards, golf course irrigation, restricted access landscape irrigation, dust control, livestock watering (dairy), street cleaning	--
Treatment Requirement	Secondary Treatment, Filtration w/ Coagulant Addition, Nitrogen Removal, & Disinfection	Secondary Treatment, Nitrogen Removal, & Disinfection	Secondary Treatment, Filtration w/ Coagulant Addition, Nitrogen Removal, & Disinfection Provided
Turbidity Limits	2 NTU (24-hour avg) 5 NTU (max)	--	Meet Current Permit
Total Nitrogen Limits	10 mg/L (5-sample mean)	10 mg/L (5-sample mean)	< 7 mg/L (~85% of alert level)
Fecal Coliform Limits	Non-detectable in 4 out of 7 daily samples 23 MPN or cfu/100 mL max	200/100 mL in 4 out of 7 daily samples 800/100 mL max	Meet Current Permit of 200/100 mL in 4 out of 7 daily samples 800/100 mL max

Note:
(1) Reclaimed water classification is based on A.A.C. R18-11-Article 3, December 31, 2008.

Tertiary treatment was installed at the WRF in 2008. It consists of sand filtration followed by UV disinfection and was designed to treat up to 3.5 mgd. With tertiary treatment, the WRF can meet the Class A+ reclaimed water requirements in the future if better effluent water quality is desired. Since the facilities are in good condition and have more than sufficient capacity for the next 10 to 20 years, they will not be evaluated further for expansion or replacement.

5.0 SECONDARY TREATMENT PROCESS OVERVIEW

The function of the secondary treatment process is to remove BOD, chemical oxygen demand (COD), TSS, suspended and non-settleable colloidal solids, nitrogen, and sometimes phosphorous from the raw wastewater to below effluent goals and limits. The secondary treatment system consists of the biological process reactors (i.e., bioreactors), and the secondary clarifiers that separate solids and liquids between the bacteria (activated sludge) and the secondary effluent.

At the Marana WRF, the secondary treatment process is an extended aeration activated sludge system based on a Biolac® system (manufactured by Parkson). It consists of a lined earthen basin (aeration basin) and integral rectangular secondary clarifiers. The aeration system uses a diffused air system and includes positive displacement blowers and tubular membrane diffusers.

5.1 Process Alternatives

Similar to the current process used at the Marana WRF, the process alternatives evaluated for future phases of the facility are based on suspended growth and secondary clarification. While these alternatives differ in some aspects, such as design parameters, aeration system type, and approach to nutrient removal, they are all based on bioreactors with suspended bacterial growth and secondary clarifiers. One of the main differences among the alternatives evaluated is the approach to nitrogen removal and the associated flexibility in coping with varying wastewater characteristics.

The alternatives in the detailed evaluation for the Marana WRF are all followed by secondary clarification. Details about each process are supplied in the subsequent sections of this TM. The alternatives are as follows:

- **Biolac®:** This is an extended aeration activated sludge system, where the bioreactors are lined earthen basins with diffused air operated in cyclic mode.
- **Biological Nutrient Removal Oxidation Ditch (BNR-OD):** These are "race track" type oxidation ditch bioreactors, concrete basins with surface aeration, and anoxic zones for nitrogen removal.
- **Biological Nutrient Removal Conventional Activated Sludge (BNR-CAS):** These are multi-stage concrete basins with multiple internal zones that are custom-designed to meet the treatment goals.

Table 4.5 shows a qualitative comparison of the three alternatives in terms of process criteria and parameters relevant to the secondary treatment process design and operation.

Table 4.5 Process Alternatives Qualitative Comparison
Marana Water Reclamation Facility Master Plan
Town of Marana

Parameter	Biolac®	BNR-OD	BNR-CAS
F/M Ratio	Low	Mid	High
Solids retention time (SRT)	Long (25+ days)	Mid (10 to 20 days)	Short (6 to 10 days)
Bioreactor Volume	High	Mid	Low
Dissolved Oxygen (DO)	Low	Mid (stratified)	Optimum
Sludge Volume Index (SVI)	High (150 to 220 mL/g)	Low to Mid (100 to 150 mL/g)	Low to Mid (100 to 150 mL/g)
Secondary Clarifier Size	Larger	Smaller	Smaller
Aeration Power	Mid	High	Low
Volume of WAS produced	High	Mid	Low

5.2 Secondary Process Design Parameters and Approach

Below is a high-level description of the process and design parameters and approach, since they form the basis for sizing the alternatives. Process modeling was used to size the different treatment alternatives and to verify compliance with the treatment goals stated earlier in this TM.

5.2.1 Bioreactor Sizing Parameters

The alternatives considered differ from the process design parameters that dictate bioreactor sizing, mainly due to the different design solids retention time (SRT) for each system (shown in Table 4.5, above). The SRT is directly proportional to the required bioreactor volume. Therefore, alternatives based on longer SRTs (i.e., Biolac® and BNR-OD) result in a larger bioreactor volume required compared to BNR-CAS.

Another key design parameter for sizing the bioreactor process is the design mixed liquor suspended solids (MLSS). The design MLSS affects not only the bioreactor size but also the secondary clarifier size. For a given design capacity, a higher design MLSS results in a smaller bioreactor size and a larger secondary clarifier size. Conversely, a lower design MLSS results in a larger bioreactor size and a smaller secondary clarifier size.

Typical design MLSS concentrations range from 2,500 to 3,500 mg/L. As discussed in TM No. 2 - Existing Facilities Evaluation, the operating MLSS for the existing facility has generally ranged between 2,000 mg/L and 3,200 mg/L. For these evaluations, a design MLSS of 3,000 mg/L was used. The same MLSS concentration was used for every alternative considered.

5.2.2 Secondary Clarification Approach

Secondary clarification is a unit process that separates, via gravity, the MLSS from the treated secondary effluent. The secondary clarifier must offer sufficient surface area and depth to allow the activated sludge to settle, resulting in low secondary effluent total suspended solids concentrations. The settled activated sludge is recycled to the bioreactors (return activated sludge, RAS), while a smaller portion is wasted (waste activated sludge, WAS) to control the SRT of the secondary treatment process.

Multiple secondary clarifier units are required for system redundancy when one unit is taken out of service for maintenance or repairs. Operating the secondary clarifiers independently from the bioreactors is recommended so system capacity can be maintained when either a bioreactor or a secondary clarifier is taken out of service.

To illustrate this point, consider a system with two bioreactors and two secondary clarifiers. This system would operate at the design capacity with one clarifier out of service but with both aeration basins in service. When one aeration basin is taken out of service, the operating MLSS would be increased to make up for the lost volume, but both clarifiers would then need to be in service to handle the increased MLSS at the design capacity.

In contrast to the recommended approach mentioned above, the existing secondary treatment system of the Marana WRF has secondary clarifiers integral to the bioreactor basin. Although the two clarifiers are integral to one aeration basin, there are no isolation gates at the inlet of each clarifier.

This integral clarifier approach is not recommended for future expansions of the Marana WRF. This is for several reasons. One, this approach does not allow a clarifier unit to be taken out of service. Two, if a second train were added with the same approach, the system would not allow one bioreactor to be taken out of service while using all the secondary clarifiers, since each bioreactor is directly associated with its own secondary clarifiers.

Another reason to provide separate circular secondary clarifiers is that they perform better than the integral-type, rectangular "V-shaped" bottom clarifiers currently at the Marana WRF. Circular clarifiers are able to dose polymer to aid flocculation of the mixed liquor under sludge bulking events. They also offer better clarification with an increased side water depth and more efficient mechanical sludge removal and scum removal mechanisms. Improved performance of the secondary clarifiers helps downstream processes, such as filtration and disinfection, operate more efficiently.

Note that, like other treatment alternatives, bioreactors using the Biolac® system can still be designed with "external" secondary clarifiers based on suspended growth. The Biolac® alternatives presented are based on providing separate secondary clarifiers and do not use the integral clarifier approach usually proposed by Parkson.

5.2.3 Secondary Clarifier Sizing

The secondary clarifier sizing approach was based on flow, MLSS concentration, and sludge settleability (i.e., SVI). These factors are all considered when calculating a clarifier safety factor (CSF) that provides the ratio between the sludge settling velocity and the upflow velocity of the clarifier effluent.

CSF values higher than 1.0 mean that the sludge settles faster than the upflow velocity, achieving efficient clarification. The minimum target CSF under peak day flow conditions is 1.15, meaning there is a 15 percent safety factor under peak day flow at the design MLSS and SVI. As mentioned, the design MLSS for the evaluations presented in this TM is 3,000 mg/L.

The design SVI has a significant impact on the secondary clarifier size. Thus, establishing this design parameter is important for the process evaluations. Operational data for the existing Marana WRF presented in TM No. 2 show that the 90th percentile of the SVI data for most recent operations is approximately 190 mL/g, with periods in which the 30-day running average ranged from 200 to 220 mL/g. Such high SVIs suggest settleability challenges for the Biolac® process, since an SVI value of 150 mL/g is typically considered the threshold for bulking sludge.

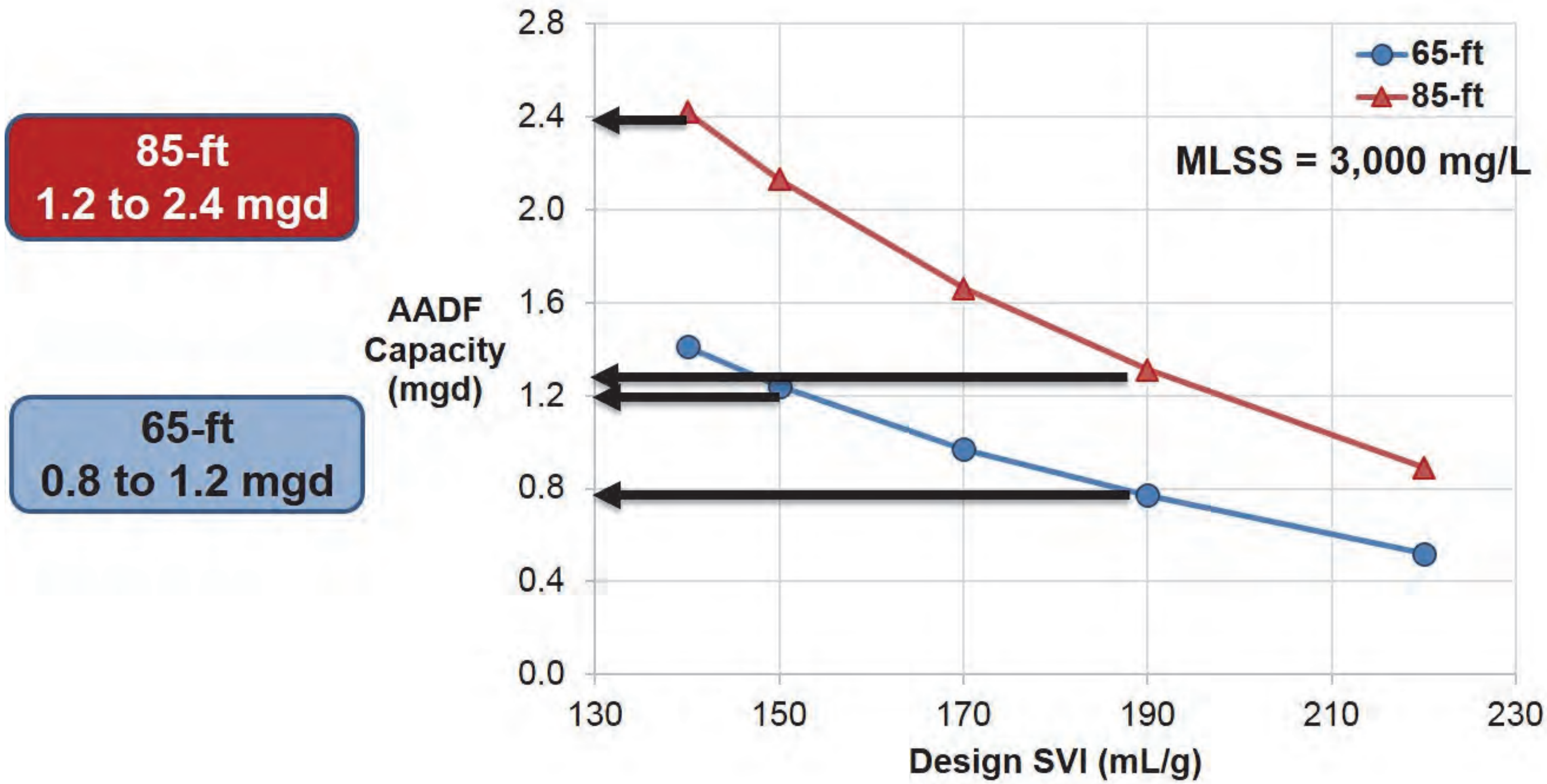
Reasons for such high SVIs were discussed in TM No. 2 - Existing Facilities Evaluation. They are associated with the nature of the Biolac® process in which process conditions (including low F/M ratios, long SRT, and low DO) are all present and promote filamentous bacteria growth in the system. A typical approach is to design for an SVI that covers sludge settleability problems. For most activated sludge processes, the normal practice is to design for an SVI of 150 mL/g and provide a redundant clarifier. Given the historical data of the Biolac® system at the Marana WRF, using a similar approach would require designing for an SVI for Biolac® alternatives at 190 mL/g.

For the initial evaluation, the recommend 65-foot circular secondary clarifiers, with a duty and standby unit, are recommended. Figure 4.2 shows the effect of SVI on the secondary clarifier capacity, at a design MLSS of 3,000 mg/L (for one duty unit).

The capacity of a 65-foot diameter circular clarifier can range between 0.8 and 1.2 mgd for SVIs of 190 and 150 mL/g, respectively. Similarly, an 85-foot clarifier can treat 1.2 to 2.4 mgd for SVIs of 190 and 140 mL/g, respectively. Providing an 85-foot clarifier would cover the high SVI of 190 mL/g needed for Biolac® alternatives, and still have a redundant unit (a standard sizing approach).

Due to budget considerations, 65-foot clarifiers are recommended over 85-foot clarifiers. Two 65-foot clarifiers adequately provide the capacity of 1.2 mgd at a normal design SVI value of 150 mL/g, allowing for one redundant unit. However, for the Biolac® alternatives, the redundant unit must be online when the SVI values exceed design values of 150 mL/g at the design flow, or, in other words, during times of bulking sludge that is not easily settled.

As noted above, the Biolac® process has challenges to sludge settleability. For further reference, and based on the information presented in Figure 4.2, with two 65-foot clarifier units in service the system would be able to handle a design flow of 1.2 mgd at an SVI of 210 mL/g.



IMPACT OF SVI ON SECONDARY CLARIFIER CAPACITY

FIGURE 4.2

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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6.0 BIOLAC®

The Biolac® process alternative is an extended aeration activated sludge process. It uses lined earthen basin bioreactors, secondary clarifiers, and a diffused air system. Figure 4.3 shows an overview of a typical Biolac® system.

The Marana WRF has a Biolac® system manufactured by Parkson, with air diffusers replaced by units from Bioworks, Inc. While Parkson's standard approach is to include secondary clarifiers integral to the earthen basin bioreactors, the alternatives presented in this TM assume the use of separate circular secondary clarifiers for reasons explained in Section 5.2.2. The aeration system is a diffused air system and includes positive displacement blowers and tubular membrane diffusers.

The treatment process is designed for biological nitrogen removal, which is achieved in the bioreactor by cycling aeration on and off in alternating groups of aeration diffusers throughout the bioreactor basin, providing alternating oxic and anoxic conditions in the bioreactor volume under a relatively long solids and hydraulic retention time.

This system is often economical because it uses lined earthen basins as bioreactors. However, several challenges are also associated with this process, as discussed in TM No. 2 - Existing Facilities Evaluation. These challenges are as follows:

- BNR performance is not optimized. Both nitrification and denitrification occur in the same basin, meaning the process conditions are not optimized for either process. The long SRT of the system (i.e., large volume) makes up for biological process inefficiencies by providing a relatively large biomass inventory.
- The long SRT and lack of selector zones results in a continuously low F/M, which promotes filament growth.
- The alternating oxic and anoxic conditions result in low DO conditions in the presence of BOD, which is another condition that promoted filament growth.
- The low DO required for denitrification can also cause incomplete nitrification (i.e., production of nitrite), which may adversely affect chlorine disinfection and potentially the overall nitrogen removal process.
- Process conditions (long SRT, low F/M, low DO) naturally favor the growth of filaments in the activated sludge, resulting in poor settleability and reduced performance of the secondary clarifiers.

Parkson (Biolac® system manufacturer) was contacted to coordinate the facility requirements for future expansions, which were also cross-checked with Carollo's process modeling results. Secondary clarifier sizing (separate circular clarifiers) and process criteria was based on the criteria and approach described in Section 5.2.

- Single, large basin of HDPE liner
- Alternating oxic/anoxic conditions for N removal
- Integral Secondary Clarifiers
- Air Lift for RAS/WAS control

Courtesy of Parkson

BIOLAC® SYSTEM

FIGURE 4.3

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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6.1 Phase 1 Expansion (1.2 mgd)

The Phase 1 Expansion to 1.2 mgd AADF is based on adding capacity with Biolac® process bioreactors, and separate secondary clarifiers. The approach is based on upgrades and improvements to the existing Biolac® system to use its capacity and to add facilities required for a total design capacity of 1.2 mgd.

In summary, the new additional facilities for the secondary process include:

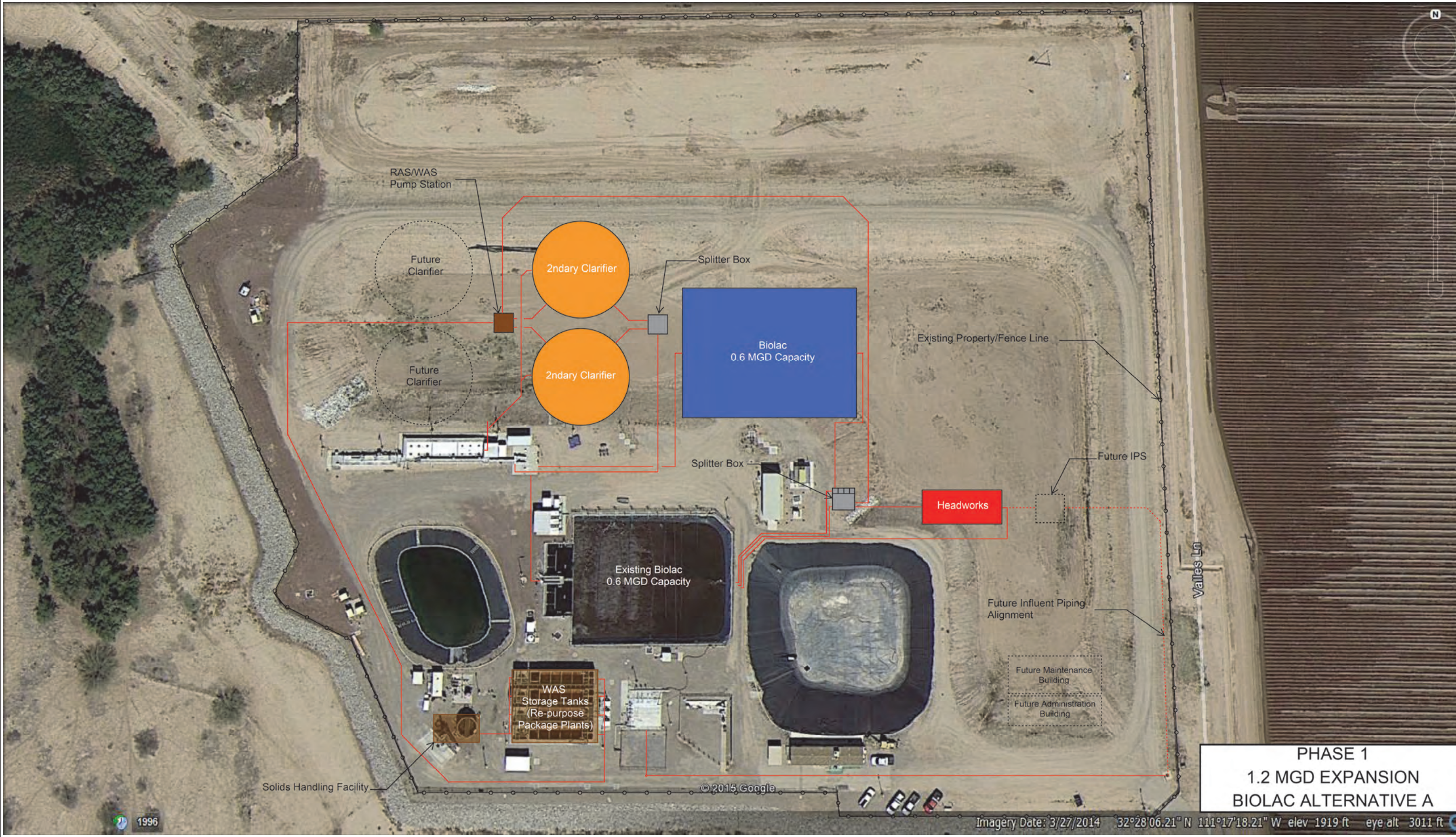
- Additional lined earthen basin bioreactor with capacity for 0.6 mgd AADF.
- Additional blowers to meet the air demand for 1.2 mgd (adding to existing system).
- Mixed liquor splitter box between bioreactors and secondary clarifiers.
- Two circular 65-foot secondary clarifiers.
- New RAS/WAS pump station. Wet well type with submersible pumps.

The improvements for the existing Biolac® system include:

- Repairs for damaged high density polyethylene (HDPE) liner or complete replacement of entire HDPE liner.
- Complete air diffuser system, including diffusers, flexible piping, and restraining cables.
- Updates to existing control system for additional blowers and aeration chains in new Biolac® basin.
- With external secondary clarifiers, the operating MLSS in the bioreactor basin can be increased to increase the SRT and provide a capacity of 0.6 mgd AADF.

Figure 4.4 shows the Phase 1 Biolac® process alternative site plan, with the new facilities located to the north of the existing facilities.

Figure 4.5 shows a site plan with a new Biolac® system, rather than refurbishing the existing one. Figure 4.6 presents the general process flow diagram of the Phase 1 Biolac® alternative.



PHASE 1
BIOLAC® SITE PLAN

FIGURE 4.4

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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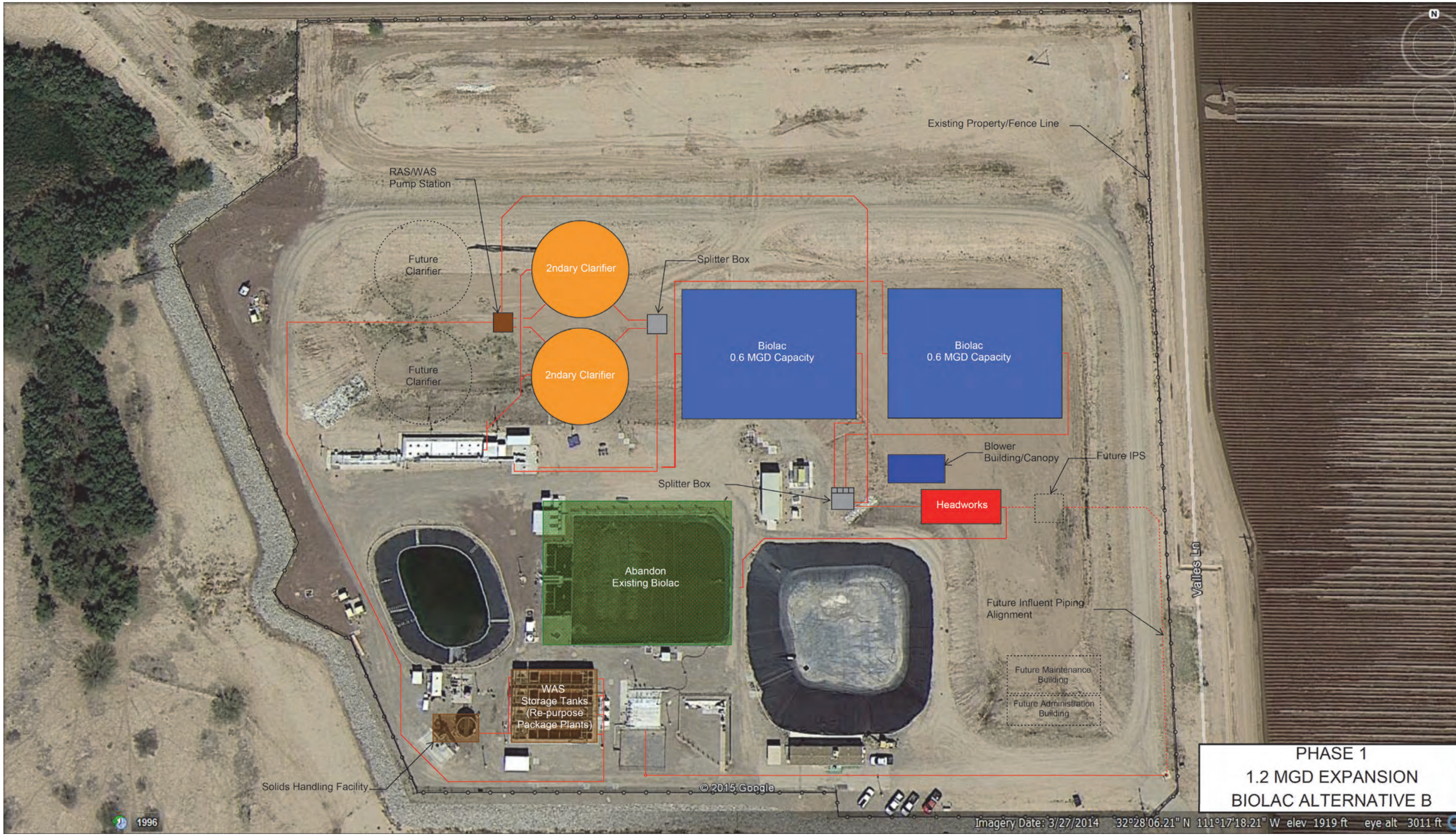

PHASE 1
BIOLAC® SITE PLAN -
ALTERNATIVE B

FIGURE 4.5

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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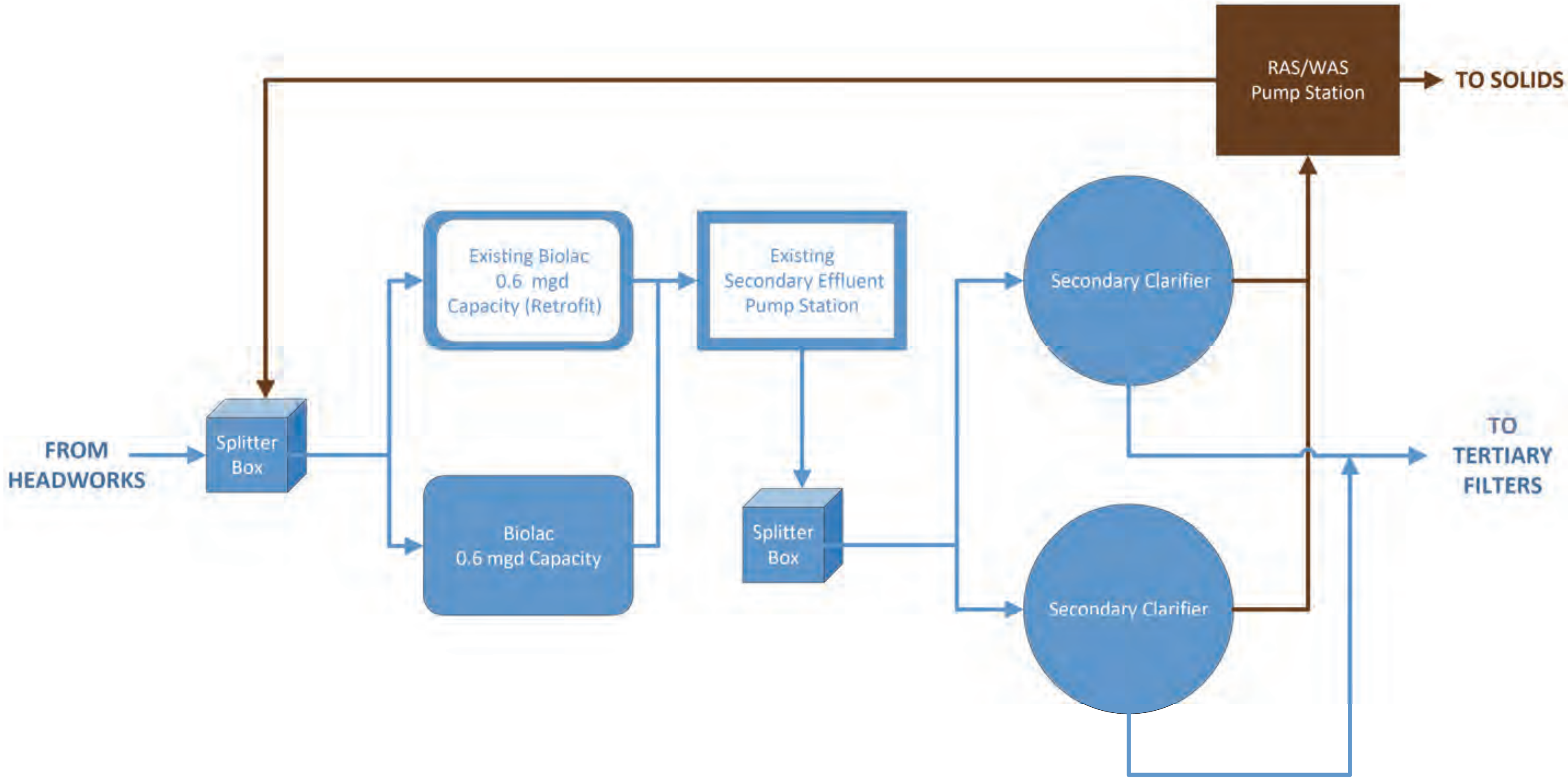


PHASE 1 BIOLAC®
PROCESS FLOW DIAGRAM

FIGURE 4.6

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

6.2 Phase 2 Expansion (2.4 mgd)

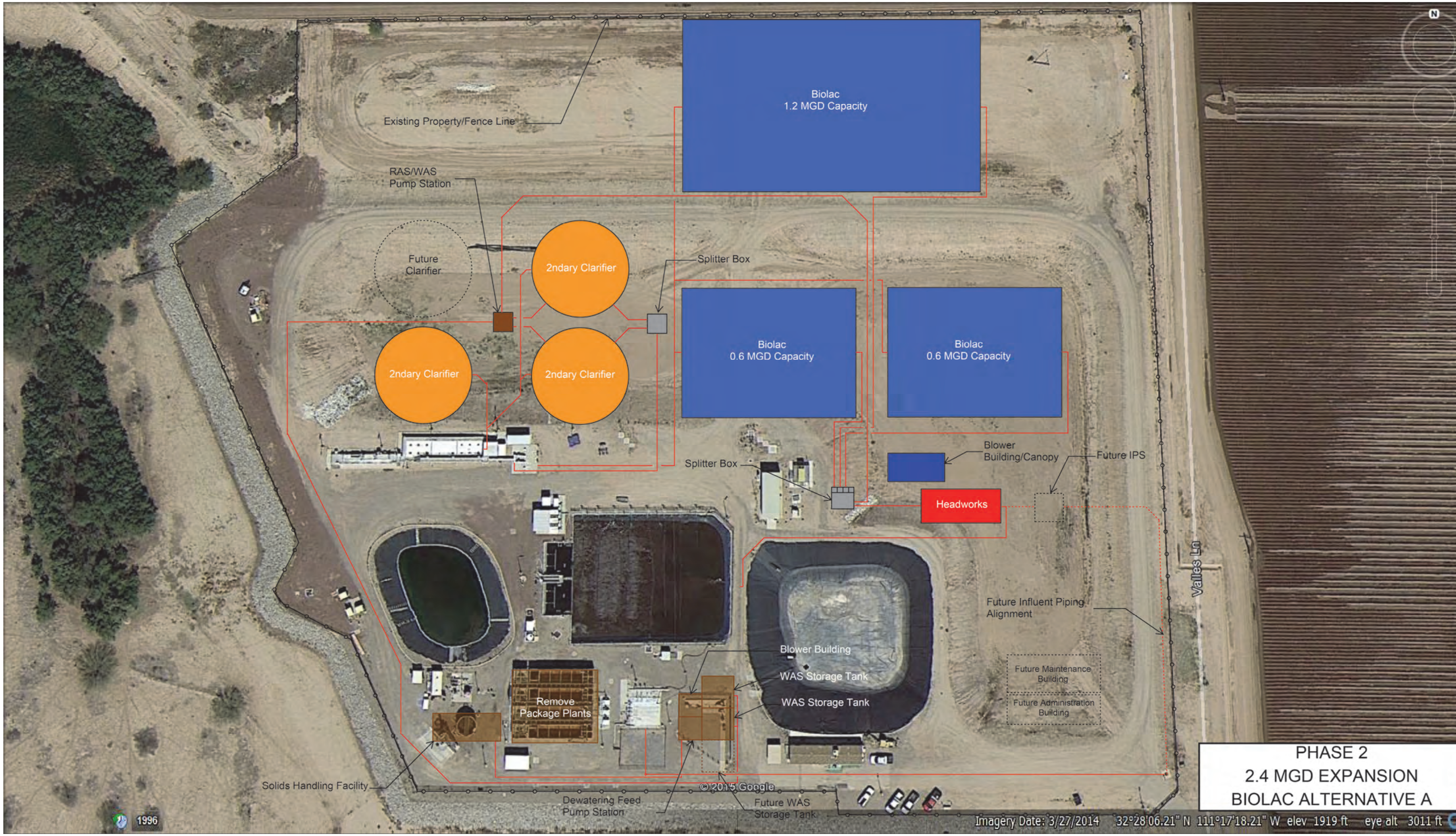
Phase 2 Expansion of the Biolac® alternative involves adding additional bioreactor and secondary clarifier capacity for a total of 2.4 mgd AADF, which includes replacing the existing Biolac® facility. The new additional facilities for the secondary process include:

- Additional lined earthen basin bioreactor with capacity for 0.6 mgd AADF to replace the current Biolac® bioreactor.
- Additional lined earthen basin bioreactor with capacity for 1.2 mgd AADF.
- Additional blowers to meet the air demand for 2.4 mgd (adding to existing system).
- One additional circular 65-foot secondary clarifier.
- Additional RAS pumping capacity in RAS/WAS pump station.

Figure 4.7 shows the Phase 2 Biolac® process alternative site plan. Figure 4.8 presents the general process flow diagram of the Phase 2 Biolac® alternative.

6.3 Phase 3 Expansion (3.6 mgd)

The Phase 3 Expansion using the Biolac® process would require one additional 1.2 mgd bioreactor and one additional secondary clarifier. Site planning evaluations revealed that, although there is space for a fourth secondary clarifier, additional bioreactor capacity using the Biolac® approach does not appear feasible beyond 2.4 mgd, as shown in Figure 4.9. While there is space at the 2.4 mgd site layout, once roads, yard piping, electrical ductbank routing, etc. are considered, adding another Biolac® bioreactor train does not appear to be feasible. This is a significant limitation of this approach since the site would be restricted to approximately 2.4 mgd due to space constraints.



PHASE 2
BIOLAC® SITE PLAN

FIGURE 4.7

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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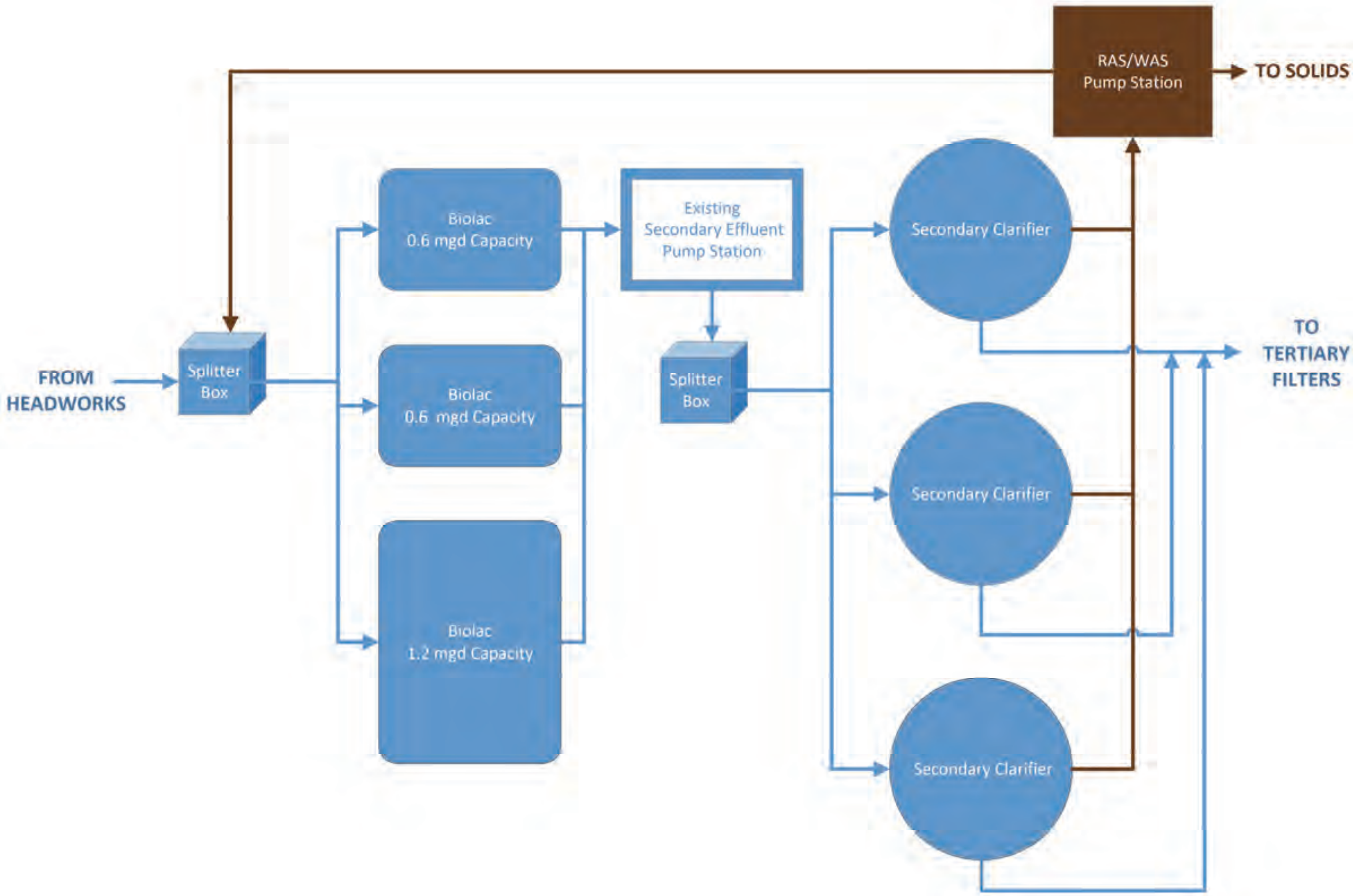

PHASE 2 BIOLAC®
PROCESS FLOW DIAGRAM

FIGURE 4.8

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

carollo

PHASE 3
BIOLAC® SITE PLAN

FIGURE 4.9

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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7.0 BIOLOGICAL NUTRIENT REMOVAL OXIDATION DITCH (BNR-OD)

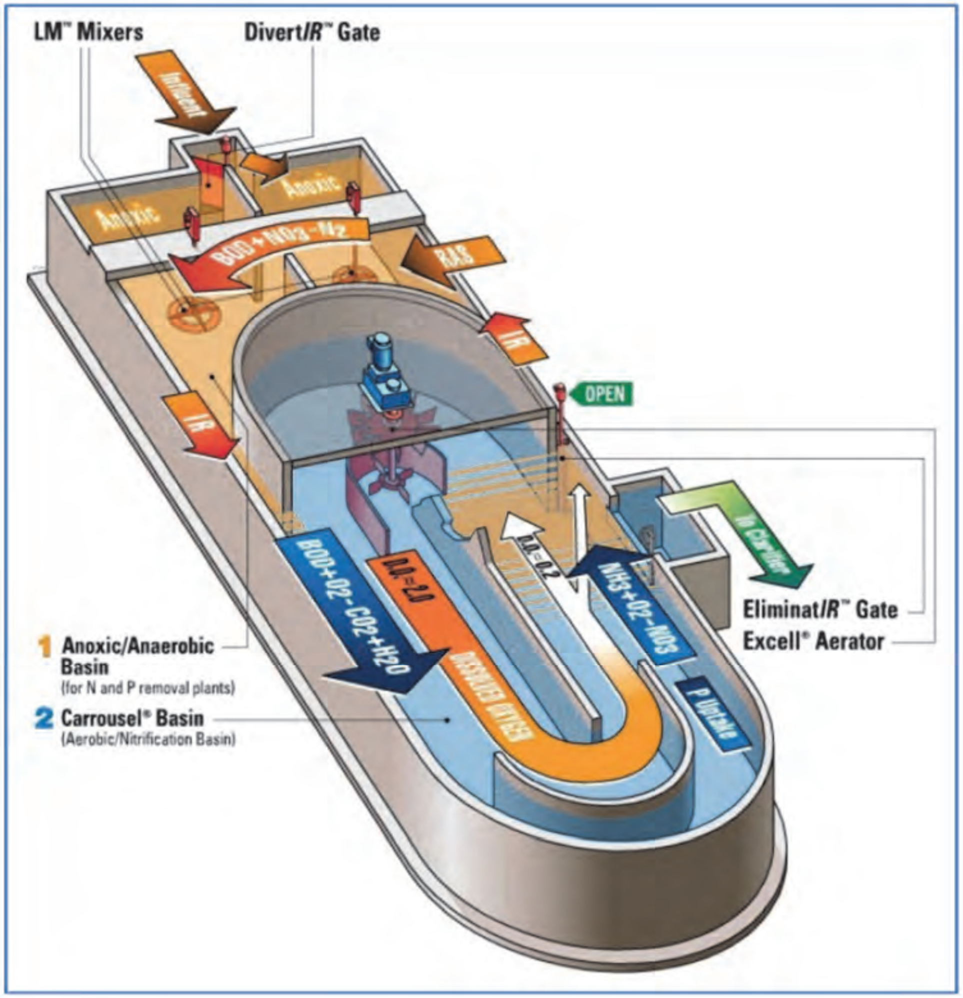
Oxidation ditches use "race track" (ring or oval shaped channel) type concrete basins equipped with mechanical aeration and mixing, which is typically accomplished by surface mechanical aerators. The tank configuration, aeration, and mixing devices promote plug flow for a system with moderate to long hydraulic detention times. The design SRT for oxidation ditches is typically 20 to 25 days when they are designed as extended aeration systems. However, they can also be designed with SRTs of 10 to 15 days, closer to the range used for conventional activated sludge processes. Figure 4.10 presents an overview of one type of oxidation ditch configured for nitrogen removal.

BNR-ODs include separate anoxic zones that allow nitrogen removal. Although the additional anoxic zones can be arranged in several configurations a more common approach is to arrange them for a two-stage Modified Ludzak-Ettinger (MLE) process. This arrangement is an initial anoxic zone where influent wastewater, RAS, and an internal mixed liquor return (IMLR) are introduced and mechanical mixing is used to avoid introducing oxygen and to maintain anoxic conditions for denitrification. In BNR-ODs, the IMLR is controlled with an internal gate that allows flow from the aerated portion of the oxidation ditch into the anoxic zone.

BNR-OD systems require little maintenance because not much mechanical equipment is required. This system can be an economical treatment alternative for small treatment plants because blower facilities aren't required. The oxidation ditch process is a proven technology, with multiple manufacturers able to supply the equipment. This process is capable of achieving biological nutrient removal and is both flexible and reliable under variable wastewater conditions.

Two oxidation ditch system manufacturers were contacted to coordinate the facility requirements for future expansions, which were also cross-checked with Carollo's process modeling results. Secondary clarifier sizing (separate circular clarifiers) and process criteria were based on the criteria and approach described in Section 5.2.

- "Race track" type basin with separate anoxic zone(s)
- Gate controls internal mixed liquor return rate and water level
- Surface aeration (mechanical)
- DO gradient – some simultaneous NdN
- Low Maintenance



BNR-OXIDATION SYSTEM

FIGURE 4.10

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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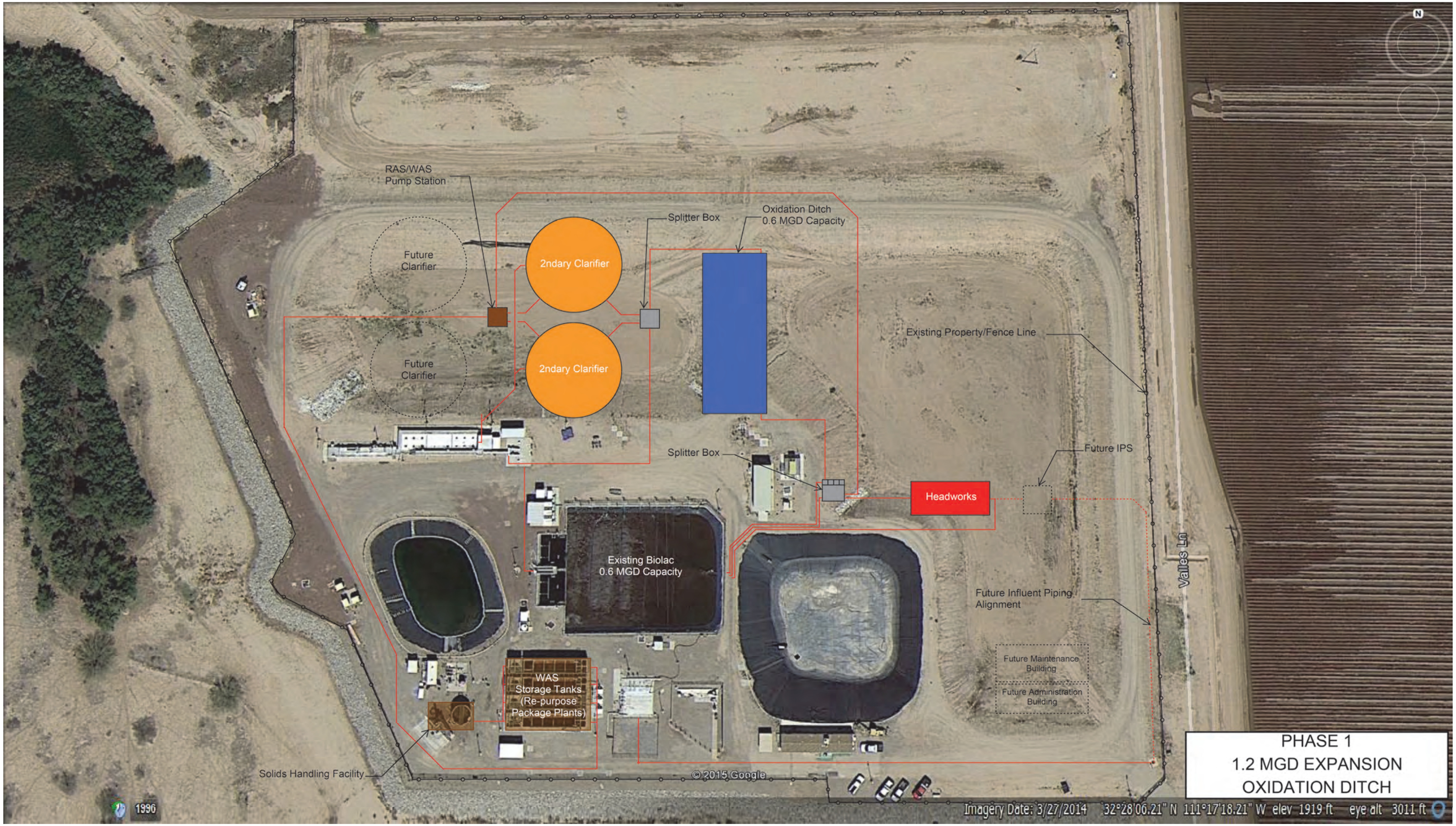
7.1 Phase 1 Expansion (1.2 mgd)

The Phase 1 Expansion to 1.2 mgd AADF adds capacity with BNR-OD bioreactors and separate secondary clarifiers. The approach is based on upgrades and improvements to the existing Biolac® system to use its capacity and adds the required facilities for a total design capacity of 1.2 mgd. The same improvements for the existing Biolac® system discussed in Section 6.1 also apply to the Phase 1 improvements.

In summary, the new facilities for the secondary process include:

- BNR-OD bioreactor with capacity for 0.6 mgd AADF.
- Surface aerators and mechanical anoxic mixers for 0.6 mgd BNR-OD basins. Other mechanical appurtenances that are part of the BNR-OD package are also included (i.e., effluent weir gate, IMLR gate).
- Mixed liquor splitter box between bioreactors and secondary clarifiers.
- Two circular 65-foot secondary clarifiers.
- New RAS/WAS pump station. Wet well type with submersible pumps.

Figure 4.11 shows the Phase 1 BNR-OD process alternative site plan, with the BNR-OD new facilities located north of the existing facilities and assuming refurbishing the existing Biolac® system. Figure 4.12 shows the site footprint of a 1.2 mgd BNR-OD. Figure 4.13 presents the general process flow diagram of the Phase 1 BNR-OD alternative.



PHASE 1 BNR-
OXIDATION DITCH SITE PLAN -
ALTERNATIVE A

FIGURE 4.11

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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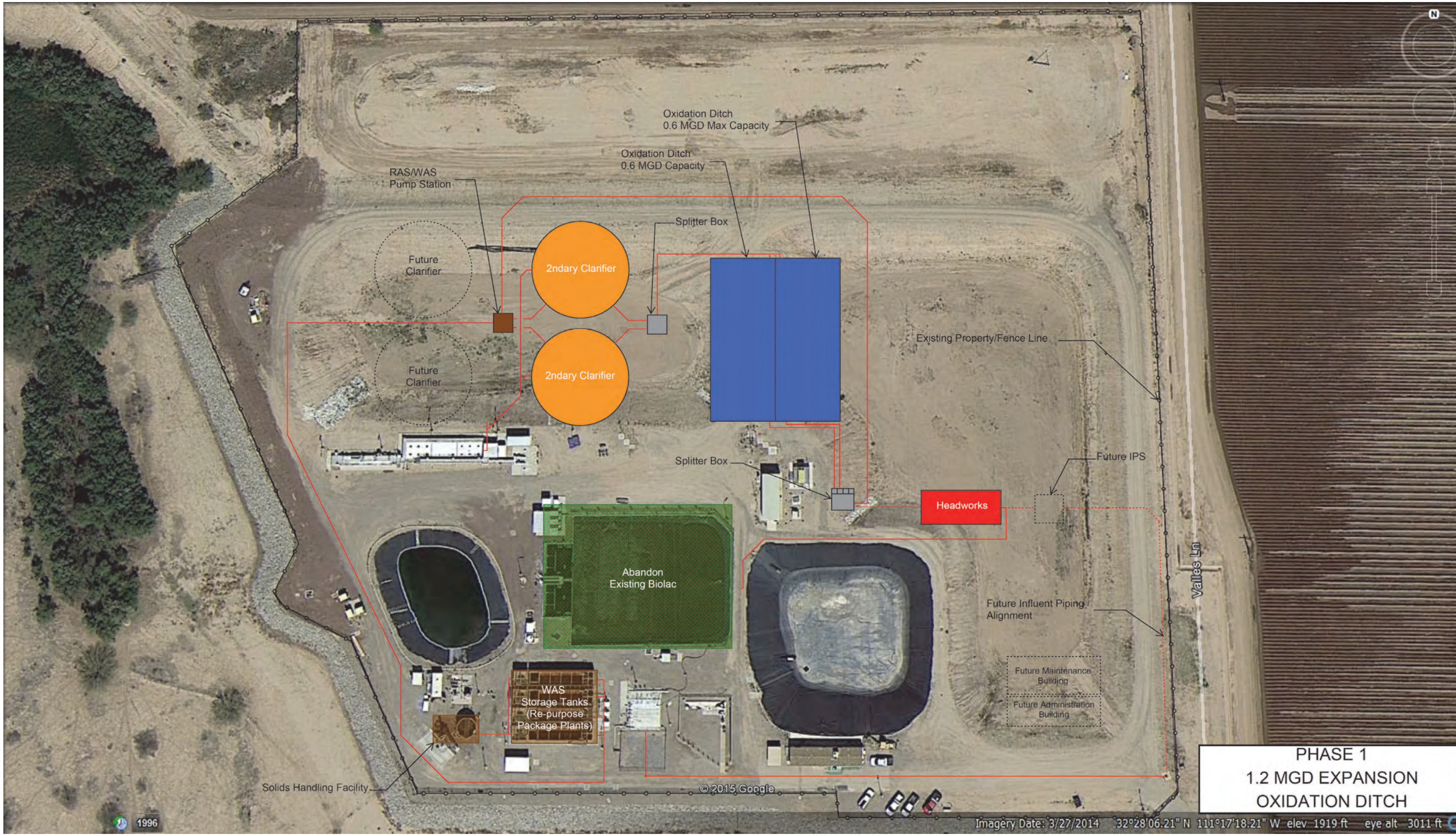

PHASE 1 BNR-
OXIDATION DITCH SITE PLAN -
ALTERNATIVE B

FIGURE 4.12

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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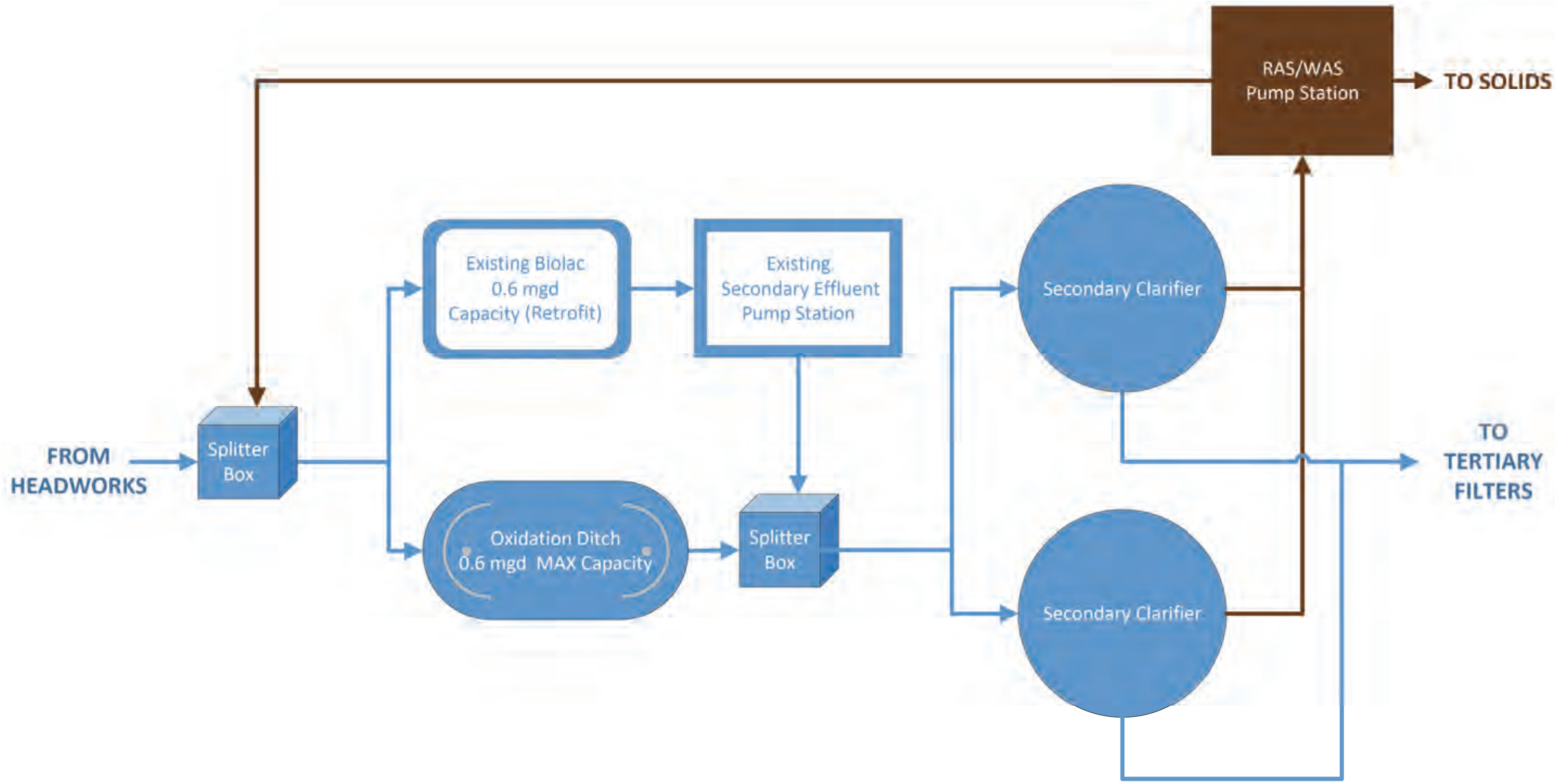

PHASE 1 BNR-OXIDATION DITCH PROCESS FLOW DIAGRAM

FIGURE 4.13

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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7.2 Phase 2 Expansion (2.4 mgd)

The Phase 2 Expansion of the BNR-OD alternative adds 2.4 mgd AADF of capacity for the bioreactor and secondary clarifier and replaces s the existing Biolac® facility. The new facilities for the secondary process include:

- Additional BNR-OD bioreactor with capacity for 0.6 mgd AADF to replace the current Biolac® bioreactor.
- Additional BNR-OD bioreactor with capacity for 1.2 mgd AADF.
- Surface aerators and mechanical anoxic mixers for 0.6 mgd BNR-OD and 1.2 mgd basins. Mechanical appurtenances part of the BNR-OD package are also included (i.e., effluent weir gate, IMLR gate).
- One additional circular 65-foot secondary clarifier.
- Additional RAS pumping capacity in RAS/WAS pump station.

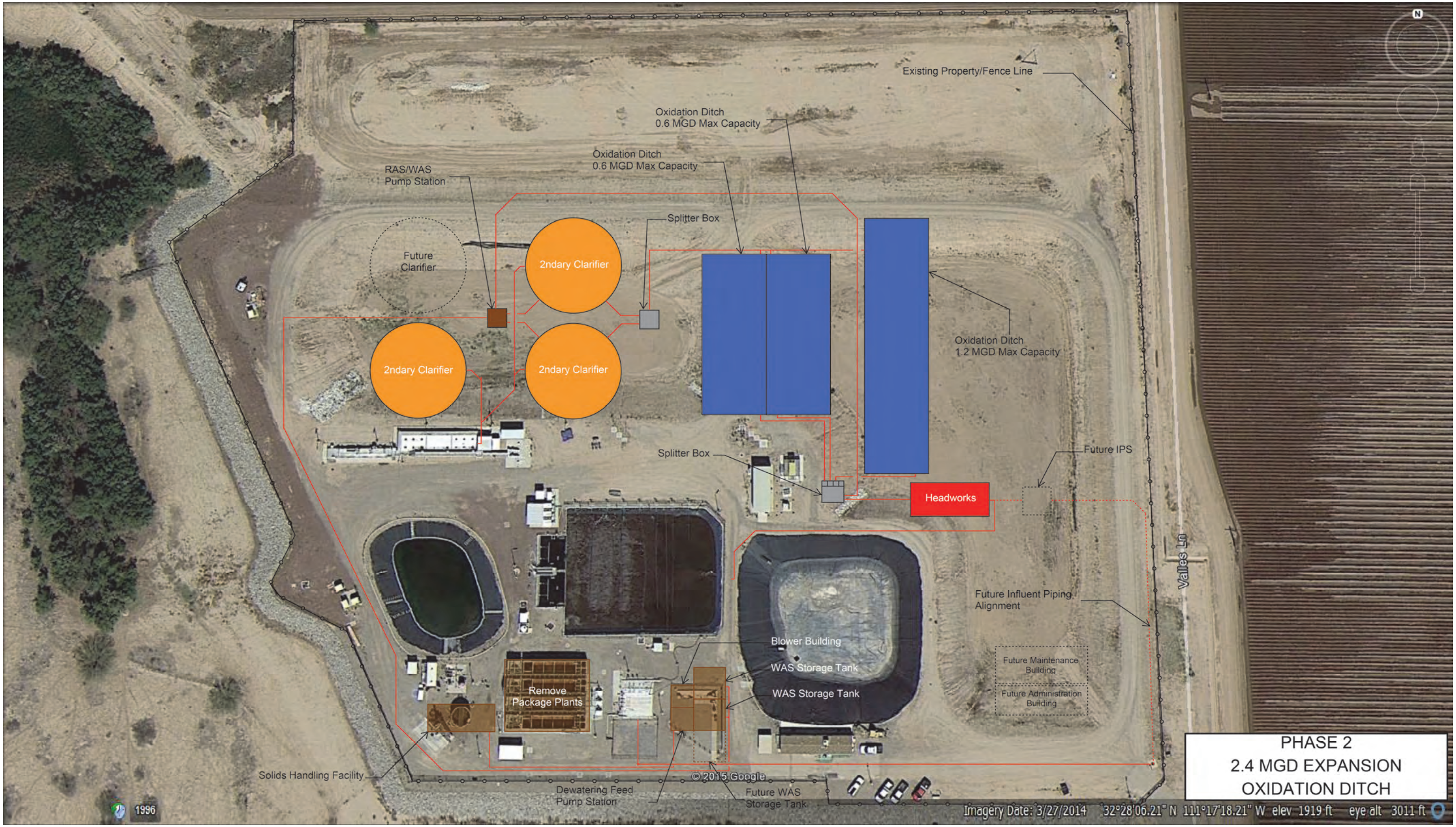
Figure 4.14 shows the Phase 2 BNR-OD process alternative site plan. Figure 4.15 presents the general process flow diagram of the Phase 2 BNR-OD alternative.

7.3 Phase 3 Expansion (3.6 mgd)

Phase 3 Expansion of the BNR-OD alternative adds bioreactor and secondary clarifier total capacity for 3.6 mgd AADF. The new facilities for the secondary process include:

- Additional BNR-OD bioreactor with capacity for 1.2 mgd AADF.
- Surface aerators and mechanical anoxic mixers for 1.2 mgd BNR-OD basins. Mechanical appurtenances that are part of the BNR-OD package are also included (i.e., effluent weir gate, IMLR gate).
- One additional circular 65-foot secondary clarifier.
- Additional RAS pumping capacity in RAS/WAS pump station.

Figure 4.16 shows the Phase 3 BNR-OD process alternative site plan. Figure 4.17 presents the general process flow diagram of the Phase 3 BNR-OD alternative.



**PHASE 2 BNR-
OXIDATION DITCH SITE PLAN**

FIGURE 4.14

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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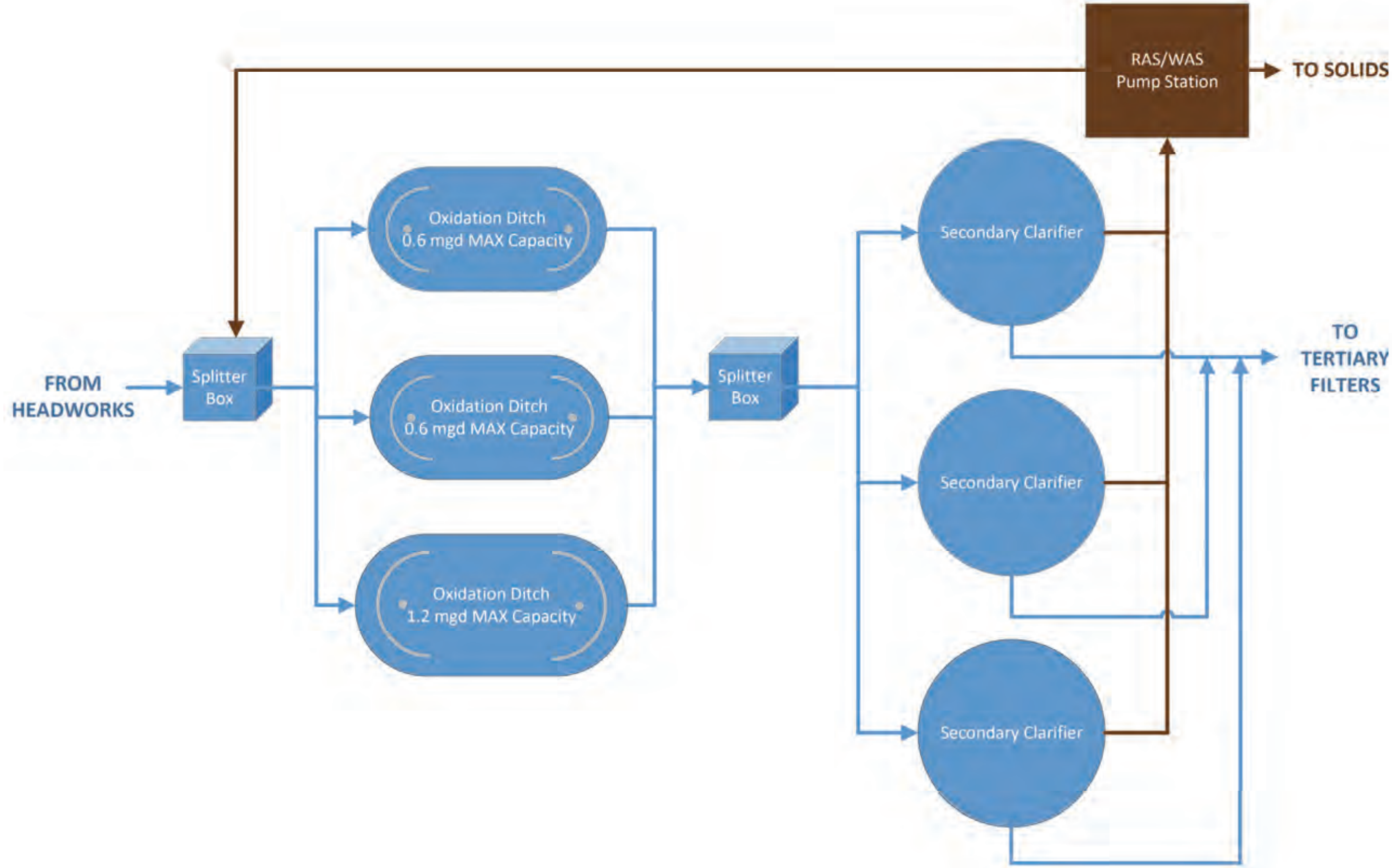

PHASE 2 BNR-OXIDATION DITCH PROCESS FLOW DIAGRAM

FIGURE 4.15

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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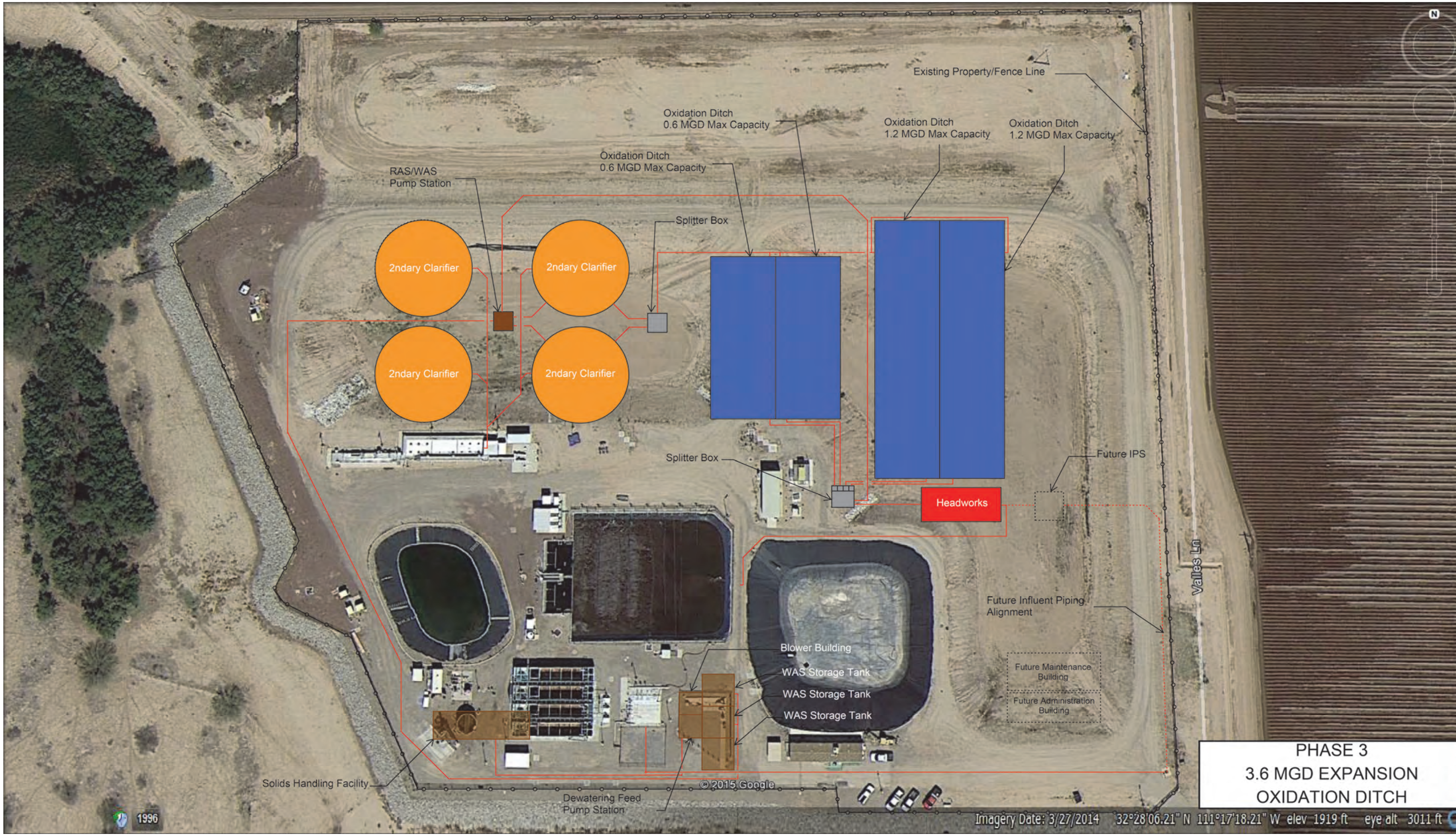

**PHASE 3 BNR-
OXIDATION DITCH SITE PLAN**

FIGURE 4.16

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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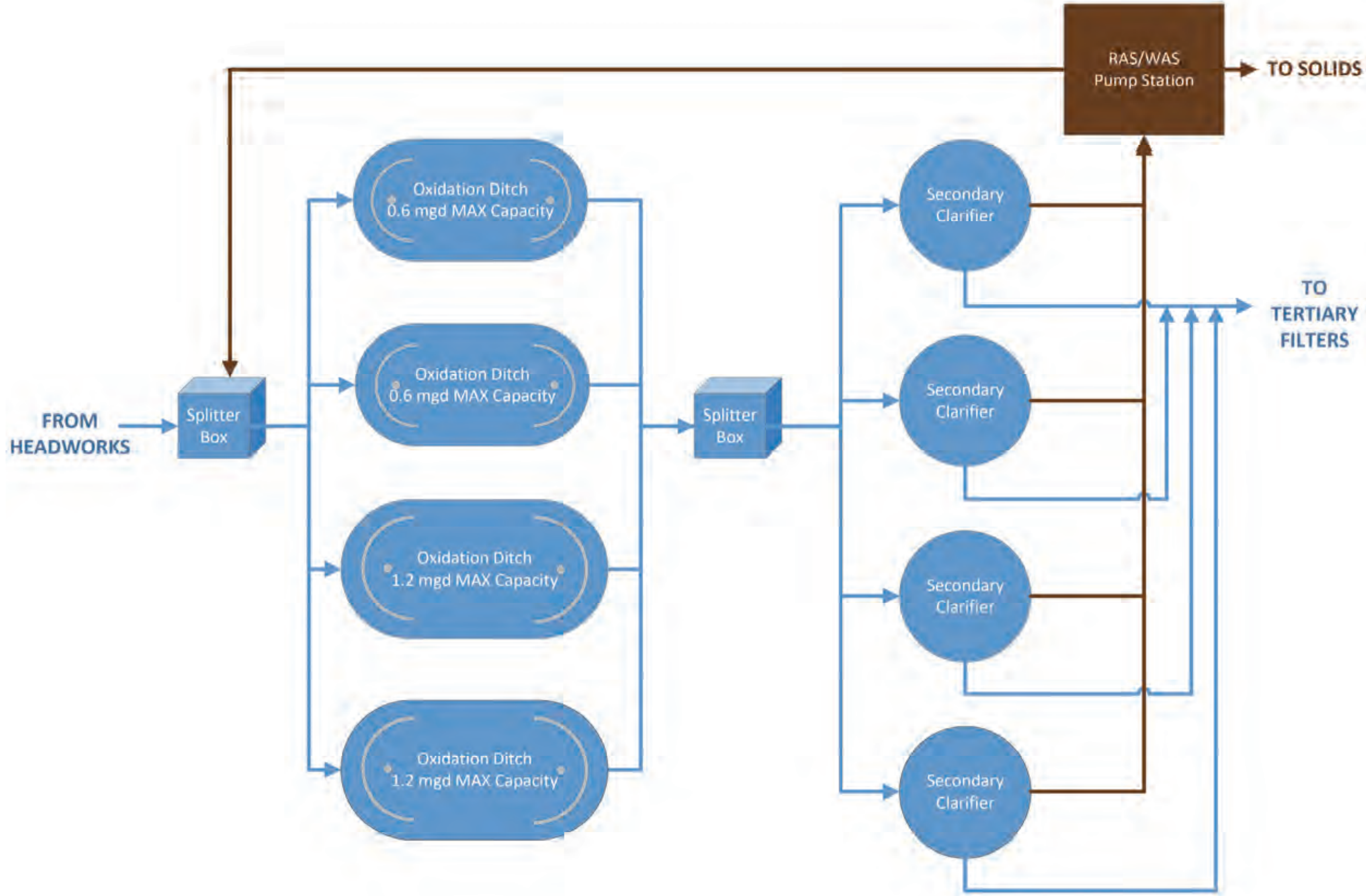

PHASE 3 BNR-OXIDATION DITCH PROCESS FLOW DIAGRAM

FIGURE 4.17

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

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8.0 CONVENTIONAL ACTIVATED SLUDGE (CAS)

The bioreactor basins in conventional activated sludge (CAS) systems include internal zones separated by baffles. These zones function either as un-aerated (anaerobic or anoxic) or aerated zones. Internal recycles and wastewater feed configurations in the aeration basins are incorporated for specific objectives (e.g., nitrate return, step feed, etc.).

The bioreactor's aerobic zones have diffusers that distribute air for biological treatment. The diffuser density is typically highest in the first aerobic zone and decreases in subsequent zones for a tapered aeration effect. The anoxic zones have mixers that keep the mixed liquor in suspension and well mixed at all times. A typical CAS system aerator basin is presented in Figure 4.18.

Depending on the arrangement of the internal aeration basin zones, variations of CAS bioreactors for nitrogen removal at the Marana WRF include:

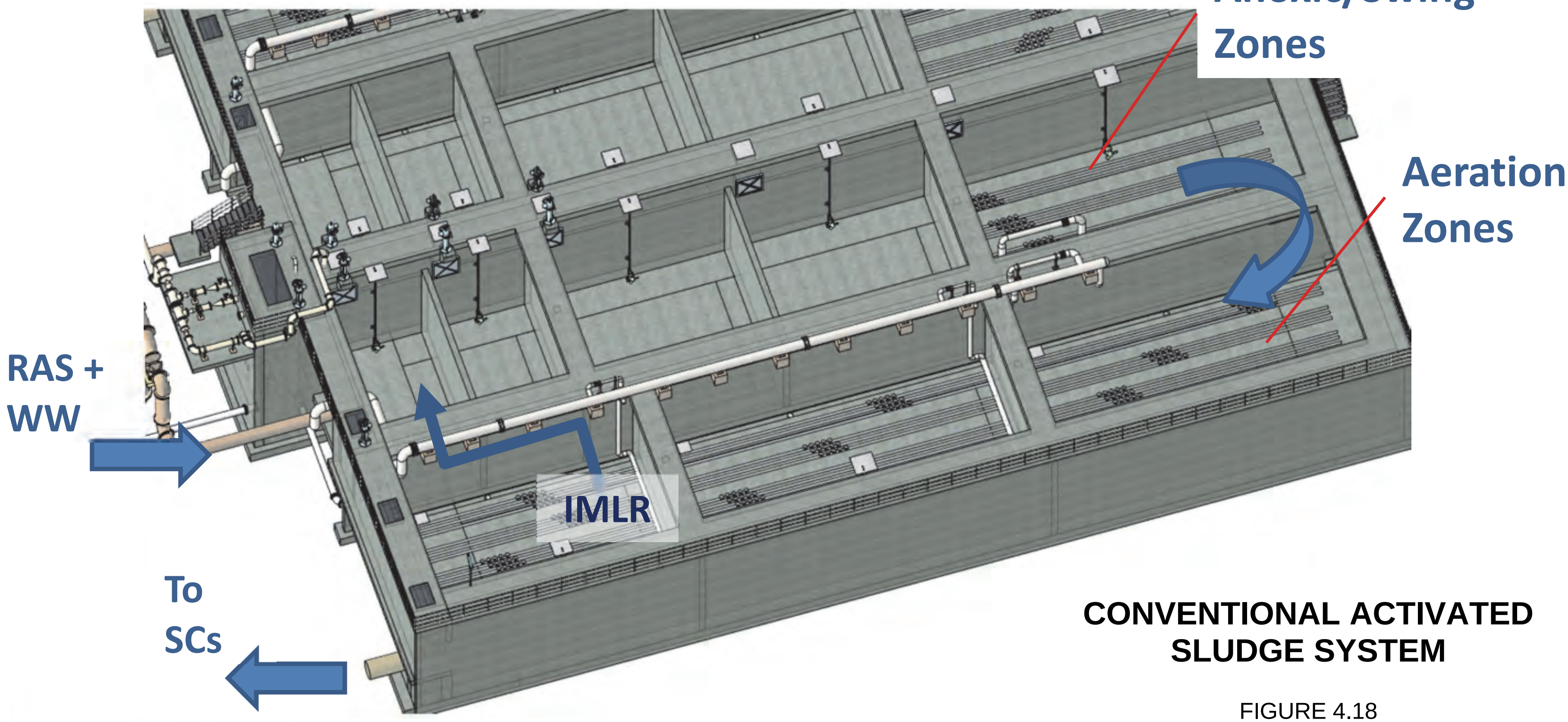
- **Modified Ludzak-Ettinger (MLE):** The MLE process is an initial anoxic zone followed by an aeration zone, and an IMLR from the final aeration zone to the first anoxic zone brings nitrates back for denitrification. Wastewater and RAS also are fed to the first anoxic zone.
- **Four-stage Bardenpho™:** This process is essentially an MLE process with an additional sequence of anoxic and aerobic stages for further denitrification. IMLR is routed from the end of the first stage aeration zone to the first anoxic zone. Wastewater and RAS are also fed to the first anoxic zone.

MLE is a common process for meeting TN limits of 10 mg/L and is widely used in facilities across Arizona. However, the MLE process' effectiveness heavily depends on the magnitude of the IMLR flow ratio to the plant influent flow, with typical values generally ranging between 3 and 4, and sometimes up to 5.

The IMLR ratio required depends on a number of factors, but the influent and effluent nitrogen concentrations are the most important. For a fixed TN effluent goal of 7 mg/L, influent nitrogen concentrations above 55 to 60 mg/L would require IMLR ratios higher than 5. In practice, when the IMLR is too high, the main challenge for nitrogen removal is returning dissolved oxygen from the aerated zones back to the anoxic zones of the bioreactors, which reduces the TN removal.

The four-stage Bardenpho™ process is very similar to MLE, with minor modifications to include a second sequence of anoxic-aerated zones. Process modeling for the Marana WRF for the four-stage Bardenpho™ showed that the second anoxic zone removes additional nitrogen while still maintaining the IMLR ratio below 5 and without requiring supplemental carbon feed. However, if the influent nitrogen concentrations increase above the design criteria shown in this TM, supplemental carbon can be dosed to the second anoxic zone, offering flexibility for variable conditions.

- Multi-stage BNR custom design, w/ internal zones
- "Swing" zones allow for flexible operation to meet changing conditions
- Mechanical mixers and internal mixed liquor return pumping req'd
- Efficient Fine Bubble Diffusers



CONVENTIONAL ACTIVATED SLUDGE SYSTEM

FIGURE 4.18

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

The CAS alternatives presented in this TM are based on a four-stage Bardenpho™ process configuration. They're designed with flexibility to also operate in MLE (two-stage) mode, which gives the Town significant flexibility in handling the high influent nitrogen concentrations that have recently been noted. (See discussions in TM No. 1 and TM No. 2).

The aeration system for the CAS alternative is based on diffused air. For the evaluations in this TM, Carollo has assumed a fine bubble diffuser system. This is standard practice for bioreactor design because it provides the most efficiency in blower power.

A blower system similar to the existing system is also assumed for this alternative. Mechanical mixing was assumed in the anoxic and swing zones, which maintains the MLSS in suspension and provides anoxic conditions without introducing air.

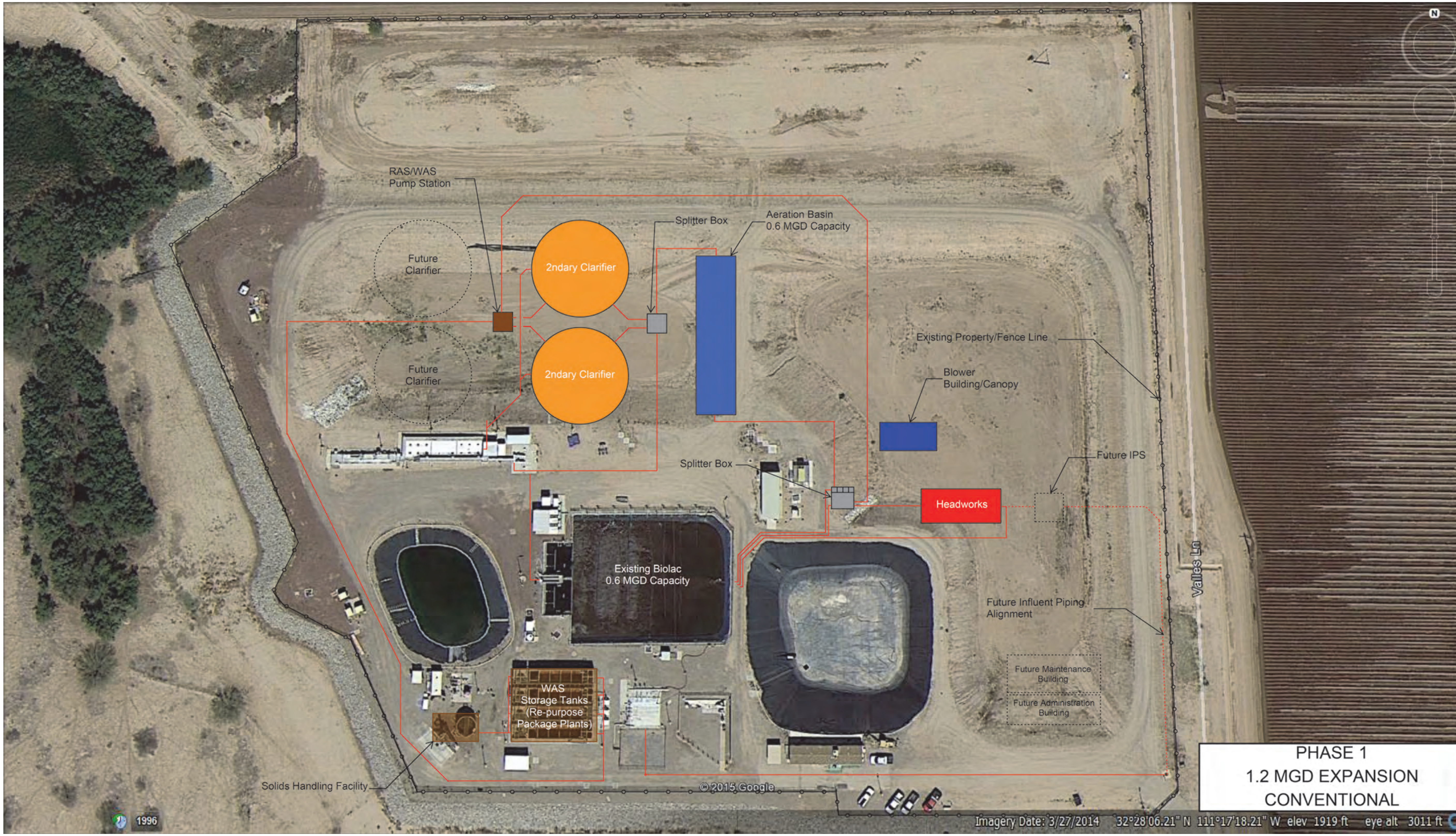
8.1 Phase 1 Expansion (1.2 mgd)

The Phase 1 Expansion to 1.2 mgd AADF is based on adding capacity with CAS bioreactors and using separate secondary clarifiers. This approach is based on upgrades and improvements to the existing Biolac® system to use its capacity and add required facilities for a total design capacity of 1.2 mgd. The same improvements for the existing Biolac® system that were discussed in Section 6.1 also apply to Phase 1 improvements.

In summary, the new facilities for the secondary process include:

- CAS bioreactor with capacity for 0.6 mgd AADF.
- Diffused air system including fine bubble diffusers, air piping, and blower system.
- Mechanical mixers for anoxic zones and swing zones. Other mechanical appurtenances are also included (i.e., gates, piping, etc.).
- Internal mixed liquor pumping system.
- Mixed liquor splitter box between bioreactors and secondary clarifiers.
- Two circular 65-foot secondary clarifiers.
- New RAS/WAS pump station, which is a wet well type with submersible pumps.

Figure 4.19 shows the Phase 1 CAS process alternative site plan, with the new facilities located north of the existing facilities, which assumes the existing Biolac® system will be refurbished and continued in service. Figure 4.20 shows the site layout with 1.2 mgd of capacity of CAS. Figure 4.21 presents the general process flow diagram of the Phase 1 CAS alternative.



PHASE 1 CONVENTIONAL ACTIVATED SLUDGE SITE PLAN ALTERNATIVE A

FIGURE 4.19

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

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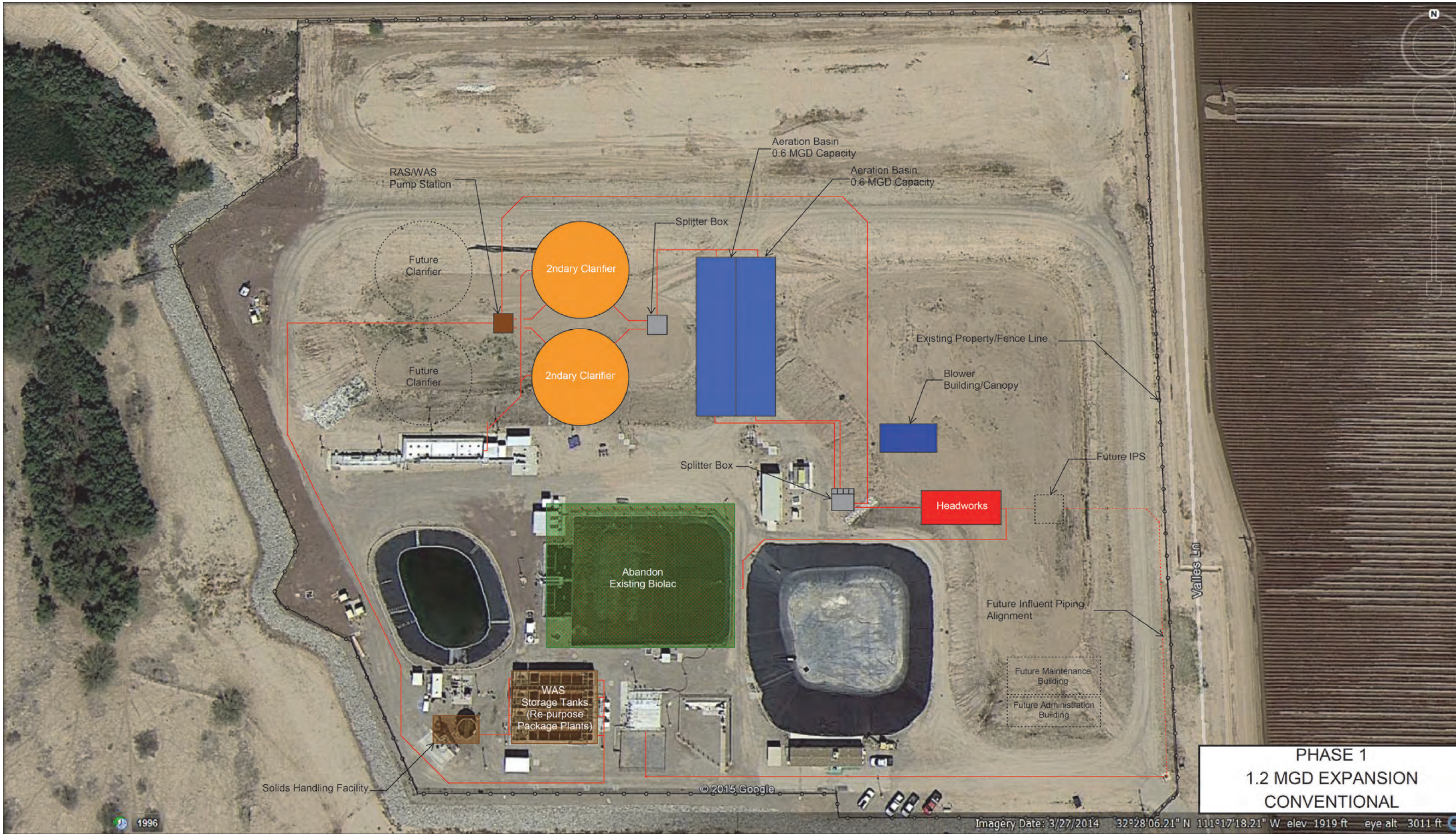

PHASE 1 CONVENTIONAL ACTIVATED SLUDGE SITE PLAN ALTERNATIVE B

FIGURE 4.20

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

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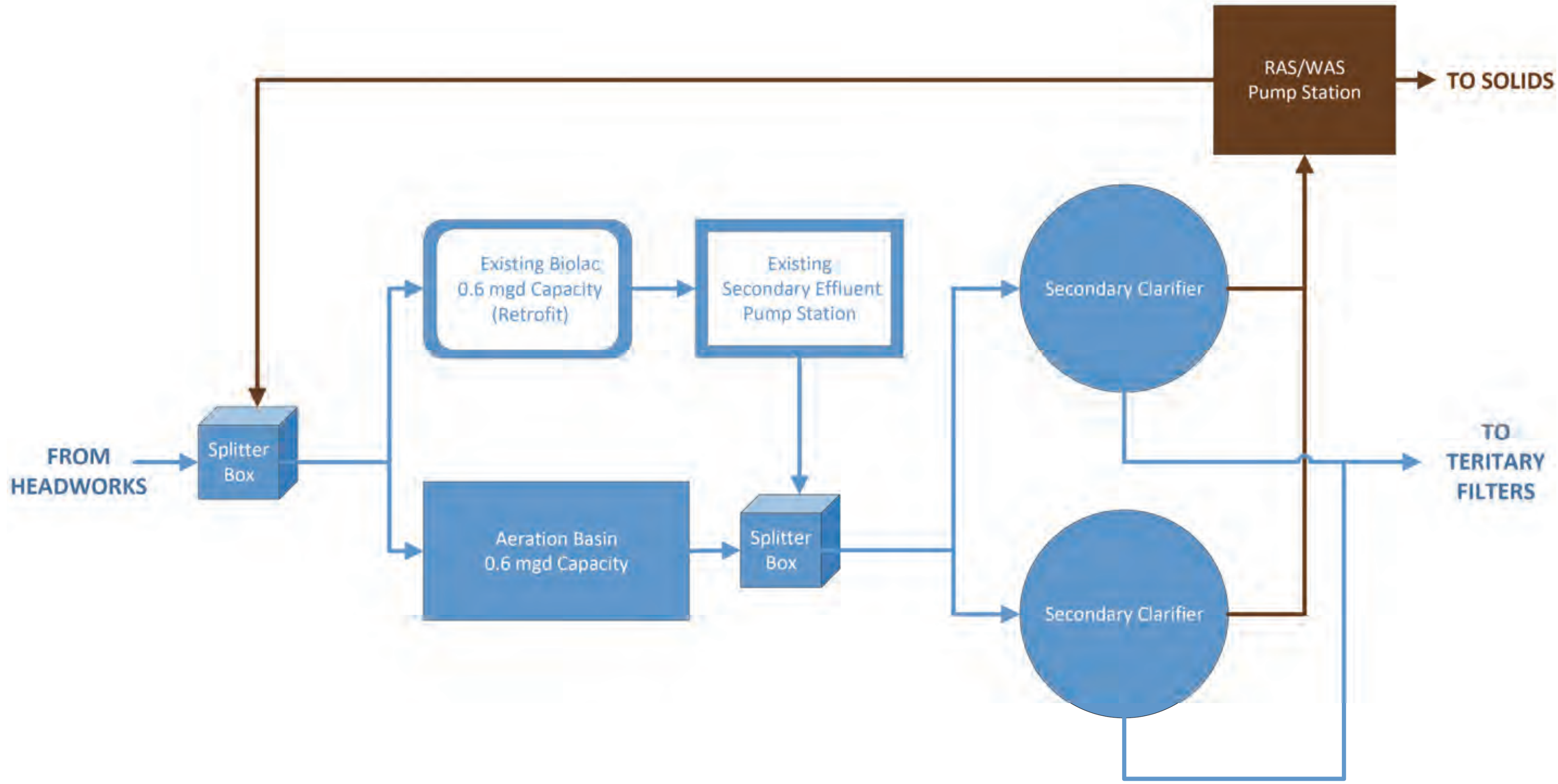

PHASE 1 CONVENTIONAL ACTIVATED SLUDGE PROCESS FLOW DIAGRAM

FIGURE 4.21

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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8.2 Phase 2 Expansion (2.4 mgd)

The Phase 2 Expansion of the CAS alternative adds bioreactor and secondary clarifier total capacity for 2.4 mgd AADF and replaces the existing Biolac® facility. The new facilities for the secondary process include:

- Additional CAS bioreactor with capacity for 0.6 mgd AADF to replace the current Biolac® bioreactor.
- Additional CAS bioreactor with capacity for 1.2 mgd AADF.
- Diffused air system including fine bubble diffusers, air piping, and expansion to blower system.
- Mechanical mixers for anoxic zones and swing zones. Other mechanical appurtenances are also included (i.e., gates, piping, etc.).
- Internal mixed liquor pumping system.
- One additional circular 65-foot secondary clarifier.
- Additional RAS pumping capacity in RAS/WAS pump station.

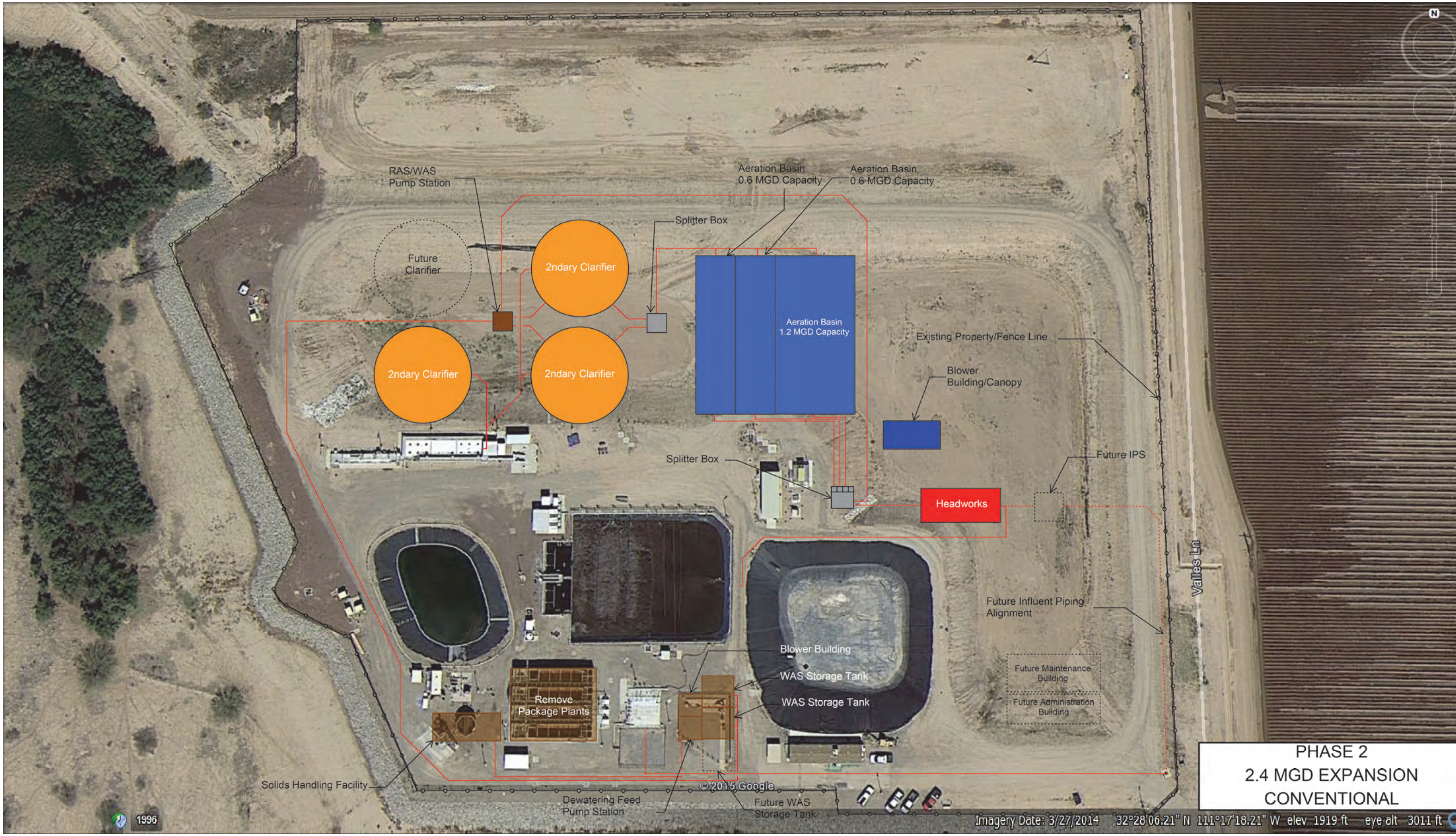
Figure 4.22 shows the Phase 2 CAS process alternative site plan. Figure 4.23 presents the general process flow diagram of the Phase 2 CAS alternative.

8.3 Phase 3 Expansion (3.6 mgd)

Phase 3 Expansion of the CAS alternative adds bioreactor and secondary clarifier total capacity for 3.6 mgd AADF. The new facilities for the secondary process include:

- Additional CAS bioreactor with capacity for 1.2 mgd AADF.
- Diffused air system including fine bubble diffusers, air piping, and expansion to blower system.
- Mechanical mixers for anoxic zones and swing zones. Other mechanical appurtenances are also included (i.e., gates, piping, etc.).
- Internal mixed liquor pumping system.
- One additional circular 65-foot secondary clarifier.
- Additional RAS pumping capacity in RAS/WAS pump station.

Figure 4.24 shows the Phase 3 CAS process alternative site plan. Figure 4.25 presents the general process flow diagram of the Phase 3 CAS alternative.



PHASE 2 CONVENTIONAL ACTIVATED SLUDGE SITE PLAN

FIGURE 4.22

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

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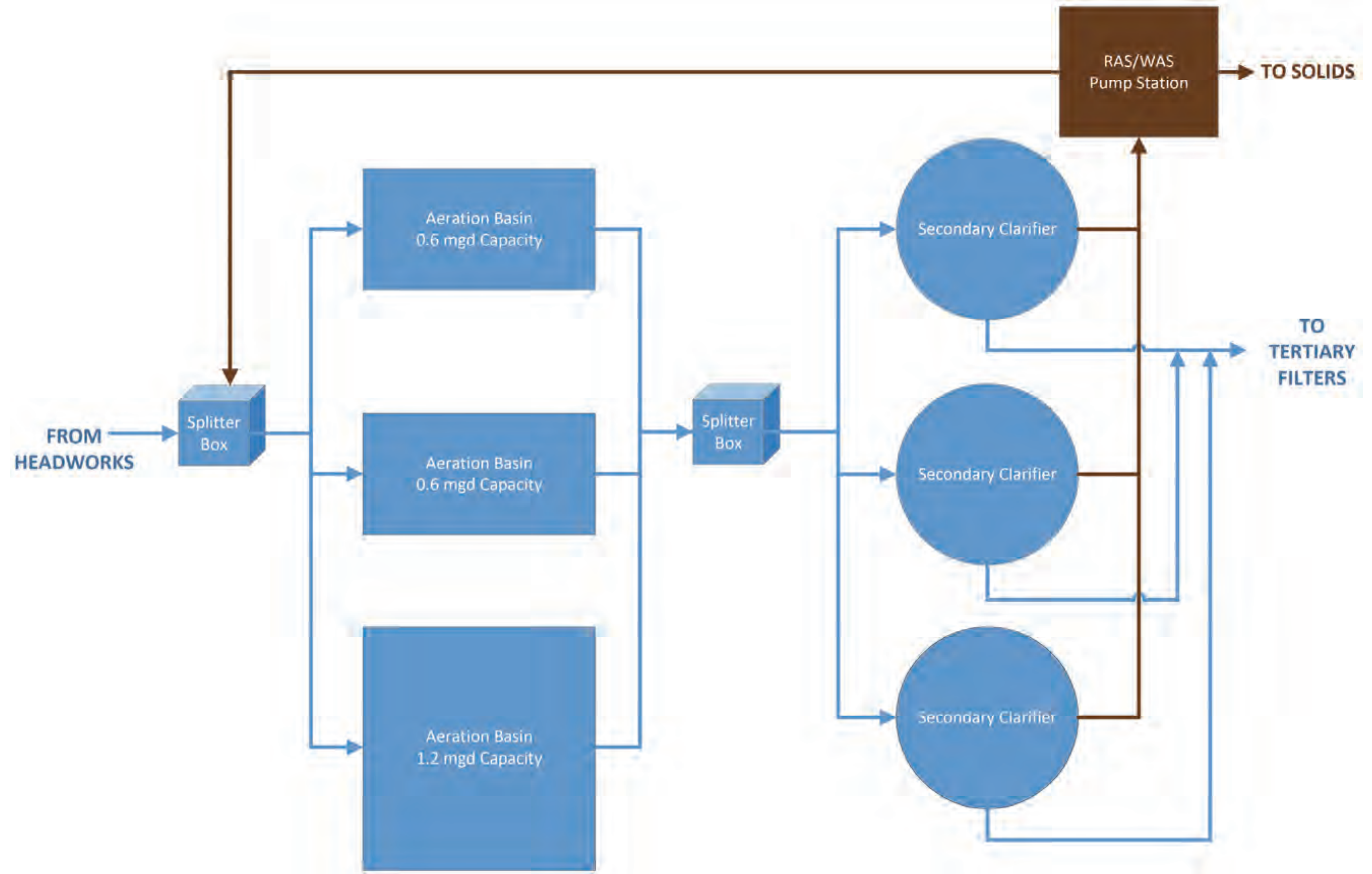

PHASE 2 CONVENTIONAL ACTIVATED SLUDGE PROCESS FLOW DIAGRAM

FIGURE 4.23

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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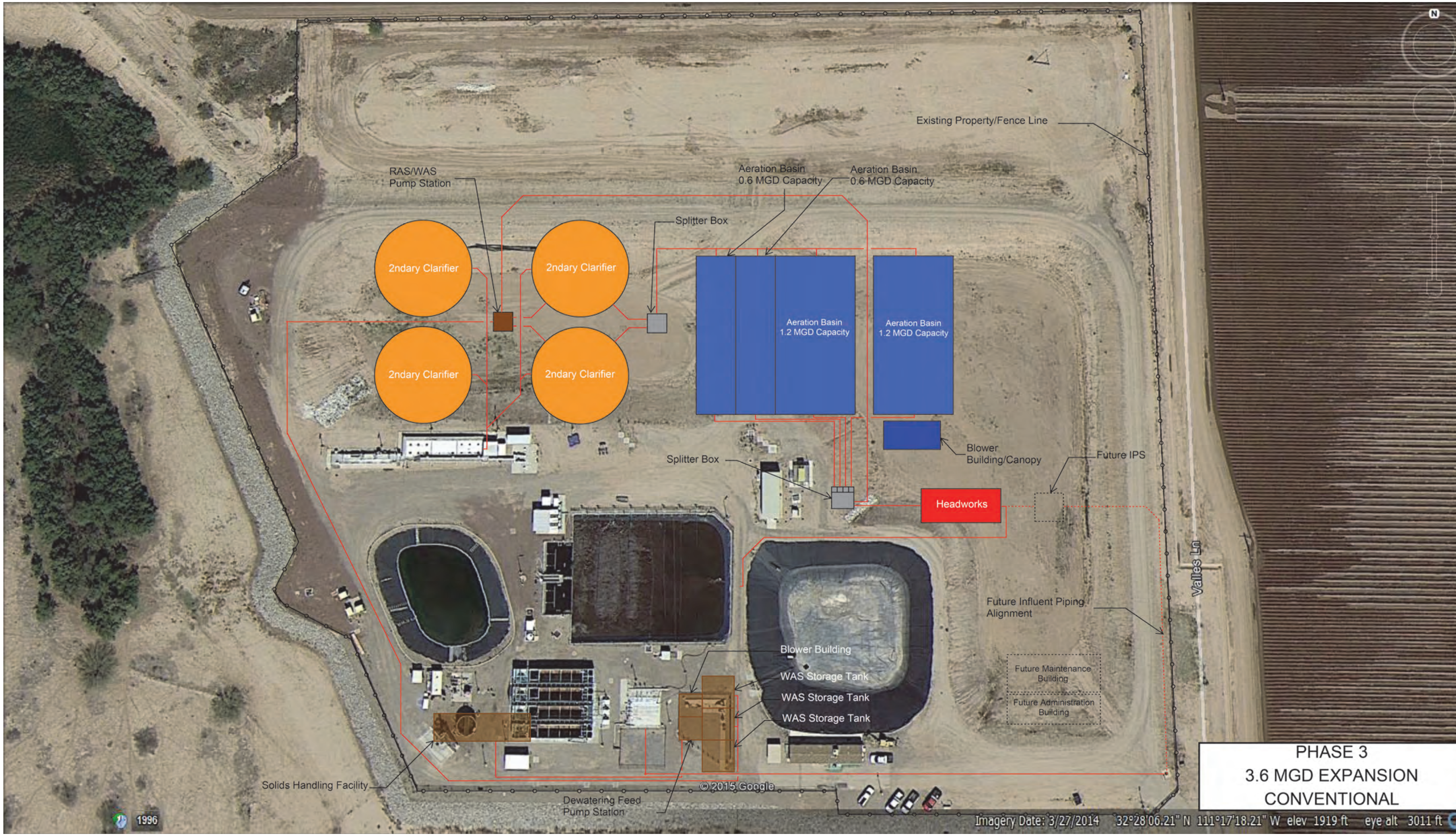

PHASE 3 CONVENTIONAL ACTIVATED SLUDGE SITE PLAN

FIGURE 4.24

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

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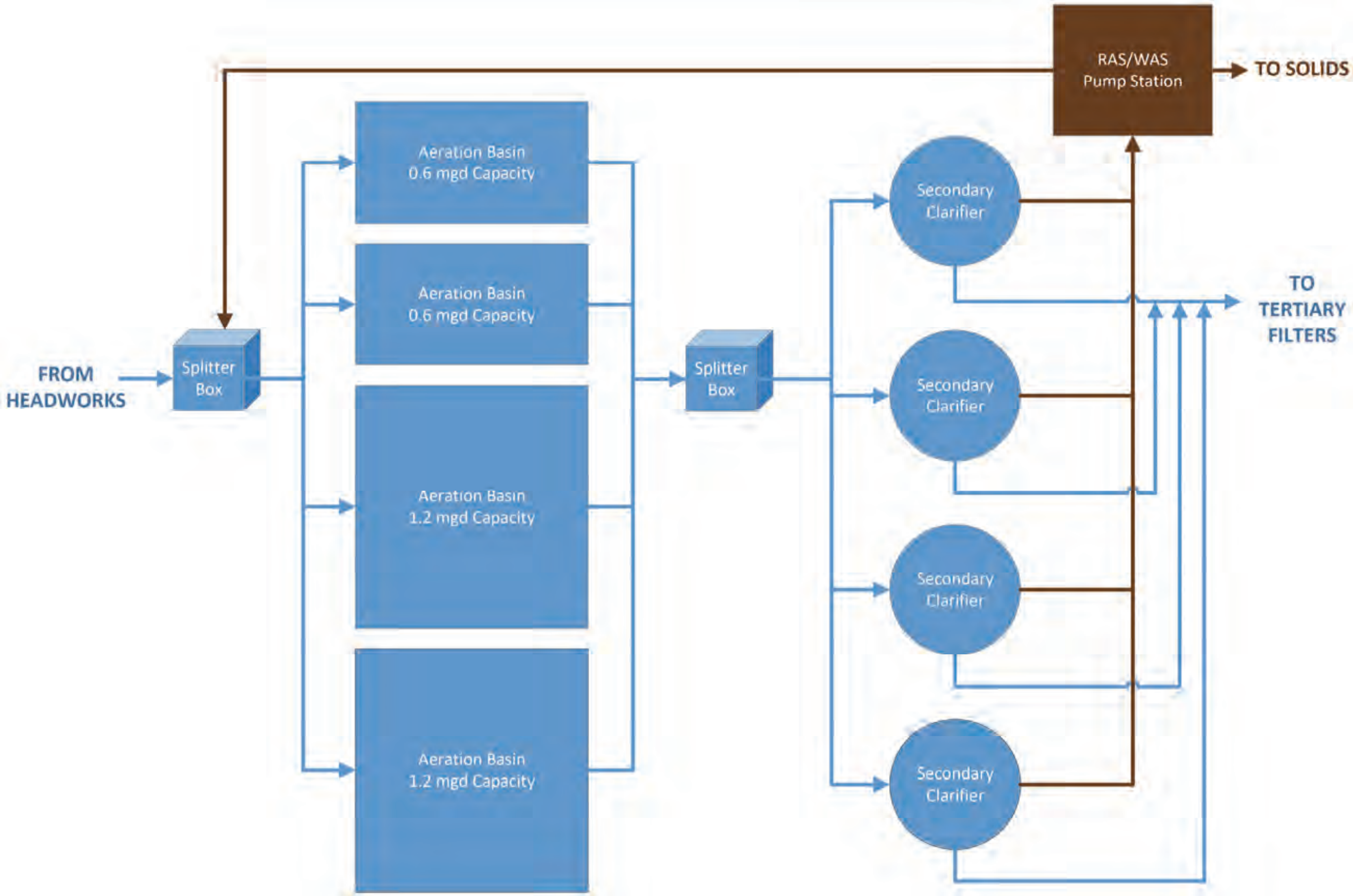

PHASE 3 CONVENTIONAL ACTIVATED SLUDGE PROCESS FLOW DIAGRAM

FIGURE 4.25

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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9.0 COST ESTIMATES

A conceptual (planning level) cost estimating exercise was performed to estimate the direct construction costs for the three secondary treatment process alternatives evaluated. These cost estimates were developed on a conceptual level to compare and screen the processes and are based on preliminary process sizing, partial quantity take-off, equipment quotes from vendors, reference project cost, and assumptions made for direct, and indirect cost components.

Table 4.6 shows the estimate of direct construction costs. These costs represent the direct construction costs only for the secondary treatment process and associated facilities.

Complete project costs, including contractor's overhead, insurance, and general requirements, engineering design fees, and Town administration costs, are detailed in the overall Master Plan Report.

Key assumptions for these cost estimate include:

- All processes were sized based on process modeling to meet the same water quality objectives detailed earlier in this report.
- Electrical costs were assumed to be 15 percent of the equipment costs.
- Instrumentation and controls were assumed to be 10 percent of the equipment costs.
- Mechanical items, such as piping, pipe supports, were assumed to be 50 percent of the equipment costs.

Table 4.6 Comparison Level Cost Estimates
Marana WRF Master Plan
Town of Marana

Component	Biolac®	BNR Oxidation Ditch		Conventional Activated Sludge	
Summary of Process Modifications/ Additions	Rurbish Existing Biolac® and Add 0.6 mgd of Biolac®	Rurbish Existing Biolac® and Add 0.6 mgd of BNR-OD	Replace Existing Biolac®, Add 1.2 mgd of BNR-OD	Rurbish Existing Biolac® and Add 0.6 mgd of BNR-CAS	Replace Existing Biolac®, Add 1.2 mgd of BNR-CAS
New Treatment Basin(s)	$983,200	$2,243,500	$3,331,400	$1,898,300	$3,156,900
Rehabilitation of Existing Biolac® Basin	$287,800	$287,800	--	$287,800	--
Bioreactor and Clarifier Splitter Boxes	$435,700	$435,700		$435,700	
External Secondary Clarifiers	$2,489,900	$2,489,900		$2,489,900	
RAS/WAS Pump Station	$401,100	$401,100		$401,100	
TOTAL ESTIMATED DIRECT COSTS	**$4,597,700**	**$5,858,000**	**$6,6658,100**	**$5,512,800**	**$6,483,600**

Notes:
(1) Capital costs are developed for Phase 1 capacity of 1.2 mgd AADF only.
(2) Costs are shown in this table are for comparisons purposes, are not inclusive total construction costs
(3) This cost estimate is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment, or services others provide, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices, or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.
(4) All costs are 2015 dollars. Engineering News-Record (ENR) Index (20-city average) of Nov. 2015 = 10092.

10.0 SUMMARY AND RECOMMENDATIONS

The following is a summary of considerations for selecting the treatment process for the Marana WRF expansion. After the process considerations is the recommendation for the treatment process that will serve the Town's short-term and long-term goals of providing reliable, cost effective treatment operations.

10.1 Process Considerations

1. The existing Biolac® process presents significant challenges for process operations, including poor settleability (which affects filter and disinfection system performance), and poor nitrogen removal (which may result in permit limit violations). As flows increase with Town growth, these challenges will become more prominent.
2. Due to the large size of the treatment basin required, the Biolac® system limits the site capacity to approximately 2.4 mgd. Since the Town anticipates growth beyond that, the Biolac® system would limit development and growth unless one or more new sites for wastewater treatment facilities are developed with an overall sewer collection and wastewater treatment facilities master plan.
3. The Biolac® treatment process is not recommended for long-term planning due to its large footprint and poor performance. The existing wastewater data shows significantly elevated total nitrogen (TN) concentrations in the influent raw sewer, most likely due to the long-term agricultural activities in the sewer basin. Should this continue, meeting the effluent TN limit in the operating permits will be difficult with Biolac®. Because the system is designed with two competing processes (nitrification and denitrification) trying to occur in the same basin, it is inherently flawed.
4. If the Town continues with the Biolac® treatment process for the next plant expansion, it will limit the WRF to a maximum capacity of 2.4 mgd, after which it must be replaced. It will cost more at future plant expansions to increase treatment capacities, as Biolac® is not a sustainable process for the long term growth of the Town. The cost to demolish the Biolac® infrastructure and replacing it with another technology becomes increasingly larger as if the Town were to continue to invest in this treatment technology. This immediate expansion phase would replace only 0.6 mgd of Biolac® capacity. However, future expansions will have to replace 1.2 or 2.4 mgd of capacity at a significantly higher cost.
5. It is recommended to repurpose the existing Biolac® earthen, lined basin for another use once a change in treatment process occurs. This will make beneficial use of the sunk investment the Town has made in that system. The Biolac® basin could be retrofitted as a reclaimed water storage basin, an emergency overflow basin, an influent equalization basin, or as an aerobic digester to further reduce sludge quantities.

6. Oxidation ditches are a viable technology, but cost estimates suggest it costs more than CAS technology because it has a larger footprint. In addition, an oxidation ditch process is less flexible in addressing unknown future wastewater concentrations.

7. The CAS process offers a custom design of both anoxic and aeration volumes, providing the greatest flexibility that will in turn optimize the treatment process under different conditions. CAS also has more efficient blowers than Biolac® systems, thereby requiring less aeration horsepower and electrical operating costs.

8. For process control, CAS is also recommended over oxidation ditch technology because of its improved operations and maintenance (O&M) efficiency and lower or equivalent initial capital cost. Lastly, the CAS process is the most compact footprint of the alternatives, providing flexibility in site planning and the largest ultimate treatment capacity at the site.

10.2 Recommendation

A conventional activated sludge treatment process provides the most flexibility, in terms of capacity, basin arrangement, and treatment needs (i.e. high influent TKN). By compartmentalizing the treatment basin into zones for separate nitrification and denitrification, the treatment process can:

1. Be optimized to current conditions.

2. Be flexible to meet changing influent conditions over time.

3. Provide the optimum environment to fully nitrify and denitrify, allowing the plant to consistently meet its APP permit limits.

Given the uncertainties and rapidly changing environment for the Town's system, a conventional activated sludge treatment process can be a robust solution with flexibility to meet these needs.

In addition, a CAS system has the smallest footprint of the alternatives evaluated, allowing for the largest ultimate capacity at this site to meet the Town's needs as it grows.

APPENDIX E – STAFFING ESTIMATE

Town of Marana Water Reclamation Facility
Summary of Staffing Estimate

FINAL ESTIMATES	Phase 1 (1.5 MGD)	Phase 2 (3.0 MGD)	Phase 3(4.5 MGD)
Chart # *(Charts follow summary page)*	**Annual Hours**	**Annual Hours**	**Annual Hours**
1 - Basic and Advanced Operations and Processes	3666	6266	8736
2 - Maintenance	2382	3750	4910
3 - Laboratory Operations	1501	1605	1709
4 - Biosolids/Sludge Handling	260	390	520
5 - Yardwork	530	530	610
Estimated Operation and Maintenance Hours	8339	12541	16485
WRF & Recharge Basins Operations Hours	5427	8261	10965
WRF & Recharge Basins Maintenance Hours	2912	4280	5520
Operations Staff	3.6	5.5	7.3
Maintenance Staff	1.9	2.9	3.7
Estimated WRF Operation and Maintenance Staff	5.6	8.4	11.0
Estimated Collection System Staff	1.0	2.0	3.0
Estimated Additional Staff from Chart 7 (Supervisory)	1	1	1
Total Staffing Estimate	7.6	11.4	15.0
Recommended Staff	**7 - 8**	**11 - 12**	**14 - 16**

*** Based on "The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants" (Nov 2008) by the New England Interstate Water Pollution Control Commision**

*** Divide the total of Annual Hours by 1500 hours per year to get the Estimated Operation and Maintenance Staff needed. This assumes 5-day work week; 29 days of vacation, sick leave, holidays; and 6.5 hours per day of productive work.**

*** Collection System Staffing is not estimated in the NEIWPCC Guide. The number of staff is a placeholder and should be revised per actual needs.**

Process	1.5 MGD Phase 1 (# of Units)	3.0 mgd Phase 2 (# of Units)	4.5 mgd Phase 3 (# of Units)	0.25-0.5	0.5-1.0	1.0-5.0	5.0-10.0	10.0-20.0	≥ 20.0	Total Hours (1.5MGD)	Total Hours (3.0 MGD)	Total Hours (4.5 MGD)
CHART 1 (One Shift) **BASIC AND ADVANCED OPERATIONS AND PROCESSES**												
Preliminary Treatment	1	2	2	130	130	260	520	780	1040	260	520	520
Primary Clarification (multiply by # of units)	-	-	-	130	130	130	260	260	260	-	-	-
Activated Sludge	-	-	-	520	1040	1560	1560-2080	2080-2600	6240	-	-	-
Activated Sludge w/ BNR	2	4	6	780	1560	2080	2340-3120	3120-6240	7280	2080	4160	6240
Rotating Biological Contactor	-	-	-	260	390-780	780-1560	1560	X	X	-	-	-
Sequencing Batch Reactor (per tank)	-	-	-	260	260	260	260	260	260	-	-	-
Extended Aeration (w/o primary)	-	-	-	650	1300	2080	X	X	X	-	-	-
Extended Aeration w/ BNR	-	-	-	910	1820	2600	X	X	X	-	-	-
Pure Oxygen Facility	-	-	-	X	X	X	2080-2600	2600	4680	-	-	-
Pure Oxygen Facility w/ BNR	-	-	-	X	X	X	2600-3900	3900	6240	-	-	-
Trickling Filter	-	-	-	260	260	520	780	1040	2080	-	-	-
Oxidation Ditch (w/o primary)	-	-	-	650	1300	2080	X	X	X	-	-	-
Oxidation Ditch w/ BNR	-	-	-	910	1820	2600	X	X	X	-	-	-
Aeration Lagoon	-	-	-	390	390	390	X	X	X	-	-	-
Stabilization Pond	-	-	-	260	260	260	X	X	X	-	-	-
Innovative Alternative Technologies	-	-	-	520	780	X	X	X	X	-	-	-
Nitrification	X	X	X	65	65	130	130	260	520	130	130	130
Denitrification	X	X	X	65	65	130	130	260	520	130	130	130
Phosphorus Removal (Biological)	-	-	-	65	65	130	130	260	520	-	-	-
Phosphorus Removal (Chemical/Physical)	-	-	-	65	130	260	520	780	1040	-	-	-
Membrane Processes	-	-	-	65	65	130	130	260	260	-	-	-
Cloth Filtration	-	-	-	65	65	130	130	130	130	-	-	-
Granular Media Filters (carbon, sand, anthracite, garnet)	3	6	9	130	260	260	390	390	780	260	520	780
Water Reuse	-	-	-	65	65	130	130	130	130	-	-	-
Plant Reuse Water	X	X	X	26	26	26	39	65	65	26	26	26
Chlorination	X	X	X	130	130	260	260	260	260	260	260	260
Dechlorination	X	X	X	130	130	260	260	260	260	260	260	260
Ultraviolet Disinfection	1	1	1.5	130	130	260	260	260	260	260	260	390
Wet Odor Control (multiply by # of systems)	-	-	-	130	130	260	260	260	260	-	-	-
Dry Odor Control (multiply by # of systems)	-	-	-	65	65	130	130	130	130	-	-	-
Septage Handling	-	-	-	130	130	260	260	260	260	-	-	-
									Total:	**3666**	**6266**	**8736**

CHART 2 (One Shift) MAINTENANCE												
	1.5 MGD	3.0 mgd	4.5 mgd	Flow (MGD)								
Activity	Phase 1 (# of Units)	Phase 2 (# of Units)	Phase 3 (# of Units)	0.25-0.5	0.5-1.0	1.0-5.0	5.0-10.0	10.0-20.0	≥ 20.0	Total Hours (1.5MGD)	Total Hours (3.0 MGD)	Total Hours (4.5 MGD)
Manually Cleaned Screens	1	1	1	65	65	65	65	130	260	65	65	65
Mechanically Cleaned Screens	-	-	-	65	65	65	260	780	1040	-	-	-
Mechanically Cleaned Screens with grinders/washer/compactors	1	2	2	65	130	260	520	1040	1300	260	520	520
Comminutors/Macerators	-	-	-	65	65	65	130	195	260	-	-	-
Aerated Grit Chambers	-	-	-	26	26	65	130	195	260	-	-	-
Vortex Grit Removal	-	-	-	26	26	65	130	195	260	-	-	-
Gravity Grit Removal	-	-	-	26	26	39	52	104	130	-	-	-
Additional Process Tanks	-	-	-	26	26	26	26	26	26	-	-	-
Chemical Addition (varying dependent upon degree of treatment)	2	2	2	26	26	26	26-78	78-156	208	52	52	52
Circular Clarifiers (Primary and Secondary)	2	3	4	65	65	130	130	195	260	260	390	520
Chain and Flight Clarifiers	-	-	-	65	65	130	130	195	260	-	-	-
Traveling Bridge Clarifiers	-	-	-	X	X	X	X	195	260	-	-	-
Squircle Clarifiers	-	-	-	65	65	130	130	195	260	-	-	-
Pumps	1	2	3	100	100	250	500	750	1500	250	500	750
Rotating Biological Contactor	-	-	-	39	39	65	65	X	X	-	-	-
Trickling Filters	-	-	-	39	39	39	65	104	130	-	-	-
Sequencing Batch Reactor	-	-	-	39	39	39	65	104	130	-	-	-
Mechanical Mixers	14	21	28	26	26	26	26	39	52	364	546	728
Aeration Blowers	3	4	5	52	52	52	52	78	104	156	208	260
Membrane Bioreactor	-	-	-	26	26	26	52	78	104	-	-	-
Subsurface Disposal System	-	-	-	26	26	26	26	78	104	-	-	-
Groundwater Discharge (Recharge Basins)	3	3	6	26	26	26	26	39	52	78	78	156
Aerobic Digestion	-	-	-	26	26	26	26	39	52	-	-	-
Anaerobic Digestion	2	3	3	X	52	52	78	156	260	104	156	156
Gravity Thickening	-	-	-	26	26	26	26	78	104	-	-	-
Gravity Belt Thickening	-	-	-	39	39	39	65	104	130	-	-	-
Belt Filter Press	-	-	-	39	39	39	65	104	130	-	-	-
Mechanical Dewatering (Plat Frame and Centrifuges)	2	3	3	39	39	39	65	104	130	78	117	117
Dissolved Air Floatation	-	-	-	X	26	26	26	78	104	-	-	-
Chlorination (gas)	-	-	-	26	26	26	52	78	104	-	-	-
Chlorination (liquid)	X	X	X	52	52	52	78	117	156	52	52	52
Dechlorination (gas)	-	-	-	26	26	26	52	78	104	-	-	-
Dechlorination (liquid)	X	X	X	52	52	52	78	117	156	52	52	52
Ultraviolet	1	1	1.5	26	26	26	39	65	78	26	26	39
Biofilter	-	-	-	130	130	130	130	130	130	-	-	-
Activated Carbon	-	-	-	130	130	130	195	195	260	-	-	-
Wet Scrubbers	1	4	5	X	X	X	39	65	78	39	156	325
Microscreens	-	-	-	26	26	26	39	65	78	-	-	-
Pure Oxygen	-	-	-	X	X	X	52	78	104	-	-	-
Final Sand Filters	3	6	9	52	52	52	52	78	156	156	312	468
Probes/Instrumentation/Calibration	15	20	25	26	26	26	26	26	26	390	520	650
									Total:	2382	3750	4910

CHART 3 (One Shift)
LABORATORY OPERATIONS

Test Required by Permit	Process at Avondale	Testing Time (hrs)	Tested Weekly X 52	Tested Monthly X 12	Tested Quarterly X4	1.5 MGD Annual Hours	3.0 mgd Annual Hours	4.5 mgd Annual Hours
Acidity		0.75	39	9	3			
L -Alkalinity, Total		0.75	39	9	3			
L -Biochemical Oxygen Demand (BOD)	Weekly	1	52	12	4	52	52	52
~~Chemical Oxygen Demand (COD)~~		2.5	130	30	10			
Chloride		0.5	26	6	2			
Chlorine, Total Residual		0.25	13	3	1			
L - Coliform, Total, Fecal, E. Coli	4x/wk	1.0	52	12	4	52	52	52
Dissolved Oxygen (DO)		0.25	13	3	1			
Hydrogen Ion (pH)	5x/wk	0.25	13	3	1	65	65	65
L -Metals	Quarterly	0.5	26	6	2	2	2	2
Toxicity (WET)	2x/yr	7.5	390	90	30	15	15	15
L - Ammonia		2.0	104	24	8			
L - Total Nitrogen	Monthly	2.0	104	24	8	24	24	24
L - Oil and Grease	1x/2wk	3.0	156	36	12	78	78	78
L - Total and Dissolved Phosphorus		2.0	104	24	8			
Solids, Total, Dissolved, and Suspended	Weekly	3.0	156	36	12	156	234	312
Specific Conductance	Monthly	0.25	13	3	1	3	3	3
L -Sulfate		1.0	52	12	4			
L - Surfactants		1.0	52	12	4			
Temperature	Daily	0.25	13	3	1	91	91	91
L - Total Organic Carbon (TOC)		0.25	13	3	1			
Turbidity	Daily	0.25	13	3	1	65	65	65
~~Bacteriological Enterococci~~		1.0	52	12	4			
Lab QA/QC Program	Quarterly	1.0	52	12	4	4	4	4
Process Control Testing	Daily	3.0	156	36	12	780	780	780
Sampling for Contracted Lab Services	Weekly	1.5	78	18	6	78	104	130
Sampling for Monitoring Groundwater Wells	monthly	3	156	36	12	36	36	36
L - Analysis provided by outside Laboratory. Plant staff collect, label and store samples.					**Total:**	**1501**	**1605**	**1709**

CHART 4 (One Shift) BIOSOLIDS/SLUDGE HANDLING										
		Flow (MGD)						1.5 MGD	3.0 mgd	4.5 mgd
Activity	Process at Marana	0.25-0.5	0.5-1.0	1.0-5.0	5.0-10.0	10.0-20.0	≥ 20.0	Annual Hours	Annual Hours	Annual Hours
Belt Filter Press	No	260	780	1560	2080	2080	2080/shift	-	-	-
Plate & Frame Press	No	260	390	780	2080	2080	2080	-	-	-
Gravity Thickening	No	65	65	130	130	260	260	-	-	-
Gravity Belt Thickening	No	65	65	130	130	260	520	-	-	-
Rotary Press	Yes	65	65	130	130	260	520	-	-	-
Dissolved Air Floatation	No	X	130	130	260	260	260	-	-	-
Alkaline Stabilization	No	65	65	65	65	65	65	-	-	-
Aerobic Digestion	No	130	130	130	260	390	520	-	-	-
Anaerobic Digestion	No	65	65	130	390	650	1040	-	-	-
Centrifuges	No	260	260	780	2080	2080	2080	-	-	-
Screw Press or Rotary Fan Press (Assumes 1 hr/day, 5 days/week.)	Yes			260				260	390	520
Composting	No	260	520-780	1040	2080	2080	2080/shift	-	-	-
Incineration	No	X	X	X	X	6240	6240	-	-	-
Air Drying - Sand Beds	No	130	130	X	X	X	X	-	-	-
Land Application	No	65	130	130	X	X	X	-	-	-
Transported Off-site for Disposal	No	65	260	1040	2080	2080	2080	-	-	-
Static Dewatering	No	260	260	X	X	X	X	-	-	-
							Total:	**260**	**390**	**520**

CHART 5 (One Shift) YARDWORK						
	Size of Plant			1.5 MGD	3.0 mgd	4.5 mgd
Work Done	**Small (≤1 MGD)**	**Average (1-10 MGD)**	**Large (≥10 MGD)**	**Annual Hours**	**Annual Hours**	**Annual Hours**
Janitorial/Custodial Staff	100	200	400	200	200	200
Snow Removal	60	120	400	NA	NA	NA
Mowing (includes Recharge Basin ripping)	100	120	400	120	120	200
Vehicle Maintenance per vehicle	25	25	25	50	50	50
Facility Painting	60	80	160	80	80	80
Rust Removal	60	80	160	80	80	80
			Total:	**530**	**530**	**610**

CHART 6 (One Shift) AUTOMATION/SCADA		
Type of Automation	**Yes**	**No**
Automated attendant or interactive voice recognition (IVR) equipment		X
Automated meter reading (AMR), touchpad meters or other automated metering technology	X	
Automatic call director (ACD)	X	
Billing system		X
Computerized facilities management (FM) system		X
Computerized preventative maintenance	X	
Computerized recordkeeping	X	
E-mail	X	
Geographical information system (GIS)		X
Integrated purchasing and inventory		X
Internet website		X
Laboratory information management system (LIMS)		X
Local area network (LAN)	X	
Supervisory control and data acquisition (SCADA)	X	
Telemetry		X
Utility customer information system (CIS) package		X

CHART 7 (One Shift) CONSIDERATIONS FOR ADDITIONAL PLANT STAFFING	
Management responsibilities (i.e. human resources, budgeting, outreach, training, town/city meetings, scheduling, etc.) and responsibility for clerical duties (i.e. billing, reports, correspondence, phones, time sheets, mailings, etc.)	YES
Plant staff responsible for collection system operation and maintenance, pump station inspections, and/or combined sewer overflows	YES
Plant operators responsible for snow plowing, road/sidewalk repair, or other municipal project	NO
Plant staff involved in generating additional energy	NO
Plant receives an extra high septage and/or grease load (higher than desired organic and grease loadings) or plant takes in sludge from other treatment plants	NO
Plant is producing a Class A Biosolid product	NO
Plant operators responsible for operating generators and emergency power	YES
Plant responsible for industrial pre-treatment program	NO
Plant staff responsible for plant upgrades and large projects done both on-site and off-site (i.e. collection systems, manholes, etc.)	YES
Plant operators responsible for machining parts on-site	YES
Age of plant and equipment (over 15 years of age)	NO