TOWN OF MARANA

MARANA water

# Marana Water Reclamation Facility
# MASTER PLAN

FINAL

May 2016

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# MARANA WATER RECLAMATION FACILITY

# MASTER PLAN

**FINAL**

May 2016

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**TOWN OF MARANA / MARANA WATER**

**MARANA WATER RECLAMATION FACILITY**
**MASTER PLAN**

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# LIST OF ABBREVIATIONS

| | |
|---|---|
| A.C.C. | Arizona Administrative Code |
| AACEI | American Association of Cost Engineers International |
| AADF | annual average daily flow |
| APP | Aquifer Protection Permit |
| avg | average |
| BADCT | best available demonstrated control technology |
| BFPs | belt filter presses |
| BNR-CAS | biological nutrient removal conventional activated sludge |
| BNR-OD | biological nutrient removal oxidation ditch |
| BOD | biochemical oxygen demand |
| CAS | conventional activated sludge |
| cfu: | coliform forming units |
| ENR | Engineering News-Record |
| gpd | gallons per days |
| IPS | influent pump station |
| max | maximum |
| mg/L | milligrams per liter |
| mgd | million gallons per day |
| mL | milliliter |
| MLSS | mixed liquor suspended solids |
| MMADF | monthly maximum average daily flow |
| MPN | most probable number |
| $NH_3$-N | ammonia nitrogen |
| NTU | nephelometric turbidity unit |
| PDF | peak daily flow |
| PHF | peak hourly flow |
| PWWF | peak wet weather flow |
| RAS | return activated sludge |
| TKN | total Kjeldahl nitrogen |
| TM | technical memorandum |
| Town | Town of Marana |
| TSS | total suspended solids |
| UV | ultraviolet |
| WAS | waste activated sludge |
| WRF | water reclamation facility |

# 1.0 INTRODUCTION

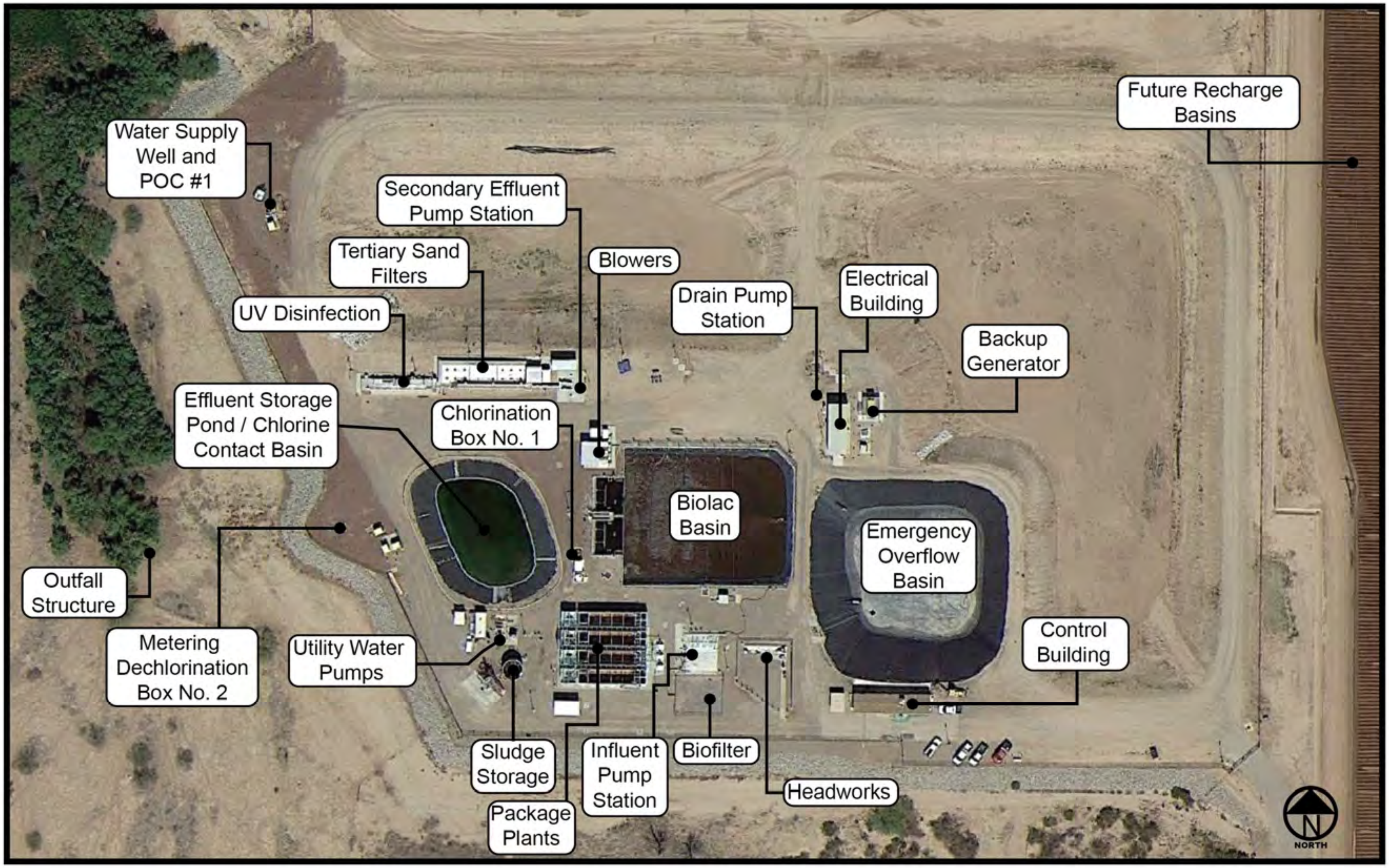
The Town of Marana (Town) owns and operates the Marana Water Reclamation Facility (WRF), which provides primary, secondary, and tertiary treatment at an operating capacity of 500,000 gallons per day (gpd).

Originally, the WRF was owned by Pima County, which approved several small expansions of the facility in 2007 and 2008, including adding the Biolac® treatment system currently in place and installing the tertiary filtration and disinfection systems. In 2012, the Town acquired the WRF, its infrastructure, and the water rights to the treated effluent from the County. Currently, the treated effluent, which is classified as Class B+ reclaimed water, is discharged to a tributary of the Santa Cruz River. However, the Town plans to construct recharge basins adjacent to the WRF property and accrue reclaimed water storage credits by recharging effluent to the below ground aquifer.

An aerial site plan of the existing treatment facility is provided in Figure 1. Process flows diagrams of the liquids and solids treatment processes are provided in Figure 2 and Figure 3, respectively.

Currently, the WRF is operating at 71 percent capacity, or an average daily flow rate of approximately 355,000 gpd, and flows are projected to increase. Initial projections suggest that the Marana WRF may require a capacity between 1.0 - 1.5 million gallons per day (mgd) within the next ten years, although economic conditions may extend or shorten this period. In its current state, the facility has limited capacity to treat wastewater with the Biolac® secondary treatment system. Thus, as part of the facility's expansion, an alternative to the Biolac® treatment system is recommended.

This Master Plan provides an evaluation of the WRF for the phased expansion of the facility to a capacity of 4.5 mgd. This evaluation includes flow projections, an assessment of the existing facilities and solids handling processes, and the alternative process selection. It also includes a final recommendation for both immediate and future phasing, proposed site plans, and other information necessary for the Town to proceed with the design of the Phase 1 plant expansion.




# MARANA WRF SITE PLAN

FIGURE 1

TOWN OF MARANA
*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

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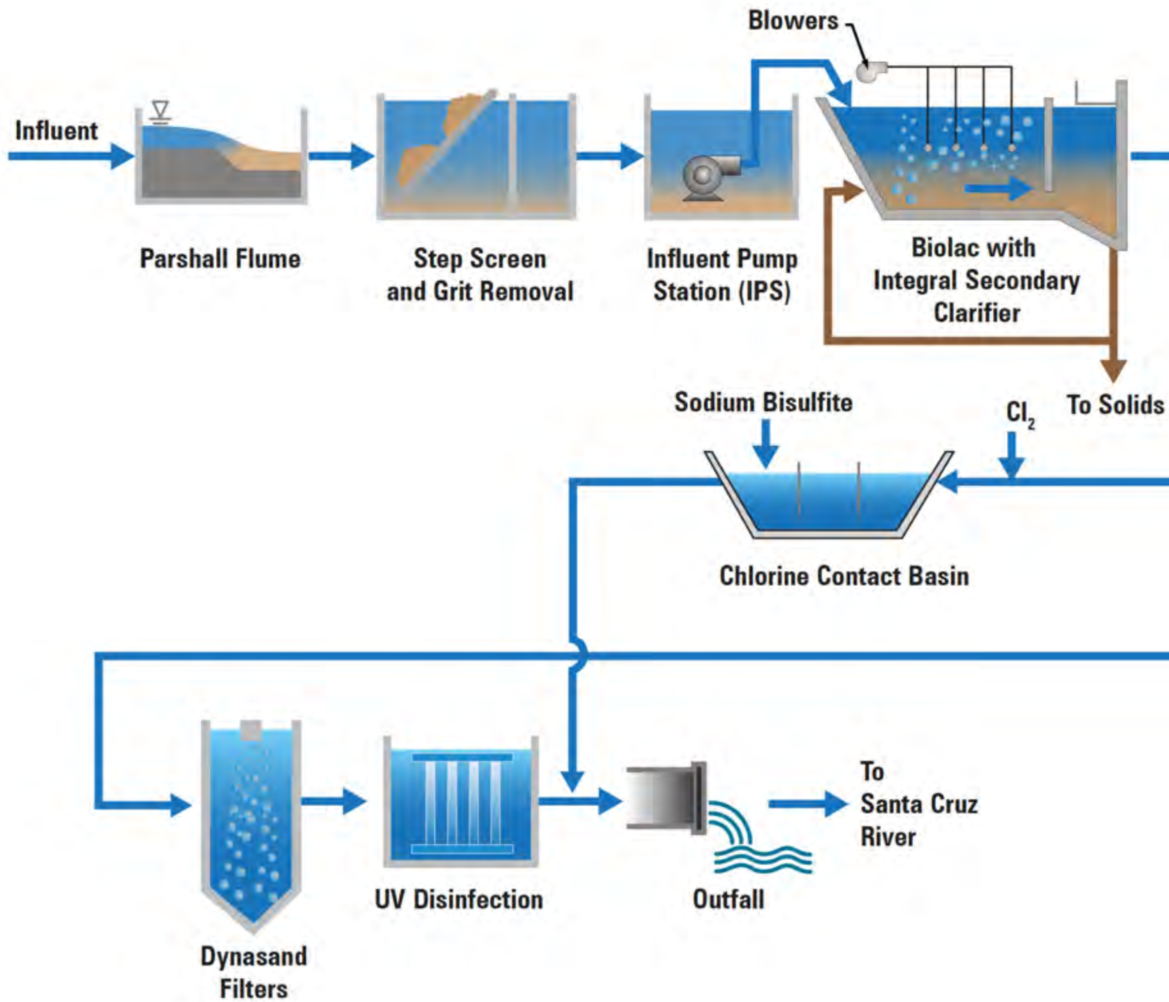


**LIQUIDS PROCESS FLOW DIAGRAM**

FIGURE 2

TOWN OF MARANA
*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

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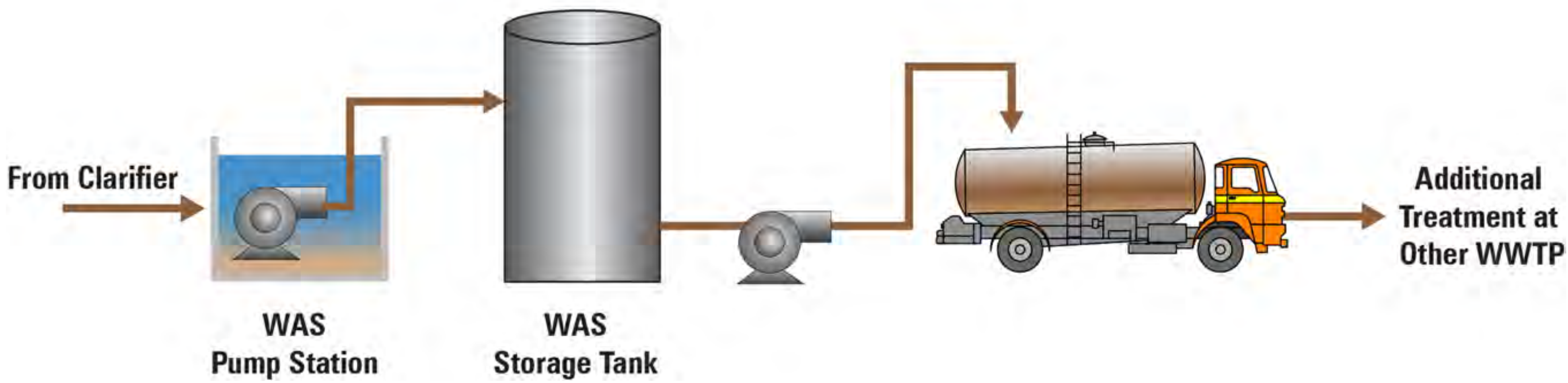


## SOLIDS PROCESS FLOW DIAGRAM

FIGURE 3

TOWN OF MARANA
*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

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# 2.0 EVALUATIONS

The Master Plan is comprised of four technical memorandums (TM) that summarize the findings and recommendations of evaluations performed on the Town's facilities. These TMs include projections of future flow, loadings, general assessments of the existing facilities, and an evaluation of various treatment alternatives. The TMs are summarized in the following sections.

## 2.1 TM-1: Flow and Loading Projections

TM-1 includes projections of flows and loadings for the Marana WRF over the next 20 years. Historical flow and loadings information was reviewed from plant operation records to determine the current conditions. Growth projections provided by the Town's Planning Department provided the basis for projections of future flows and similar loadings. These projections summarize the flow, biochemical oxygen demand (BOD), and total suspended solids (TSS) by annual average, maximum month, and peak day loads for the planning phases identified.

TM-1 can be found in Appendix A of this Master Plan.

## 2.2 TM-2: Existing Facilities Evaluation

TM-2 documents the existing condition and capacity of the WRF's processes and equipment. It also summarizes the current major regulatory and operating permits for the WRF and provides evaluations of the floodplain, hydraulic capacity, electrical equipment, and standby power.

TM-2 can be found in Appendix B of this Master Plan.

## 2.3 TM-3: Solids Handling Evaluation

TM-3 evaluates the existing solids handling operations, summarizes future solids production estimates, and makes recommendations for future solids handling operations. This includes an evaluation of the available solids dewatering technologies applicable to the WRF.

TM-3 can be found in Appendix C of this Master Plan.

## 2.4 TM-4: Alternative Process Evaluation

TM-4 evaluates the current and alternative treatment processes for their applicability and feasibility at the WRF for both short-term and long-term planning phases. Evaluated treatment processes include the Biolac® system, biological nutrient removal oxidation ditch (BNR-OD), and conventional activated sludge (CAS) treatment for their ability to consistency meet the desired reclaimed water requirements.

TM-4 can be found in Appendix D of this Master Plan.

# 3.0 PROJECTED FLOWS AND WATER QUALITY GOALS

To properly plan and budget for the timely expansion of the WRF, a planning evaluation was completed to establish plant capacity and design criteria, which are discussed in this section.

## 3.1 Projected flows and expansion phasing

As the Town grows, the WRF will need to be expanded to serve population and industrial growth and the resulting increase in wastewater flow and/or loadings. Thus, the projections in this Master Plan are based on wastewater growth trends developed from the Town's planning department's population and development growth projections.

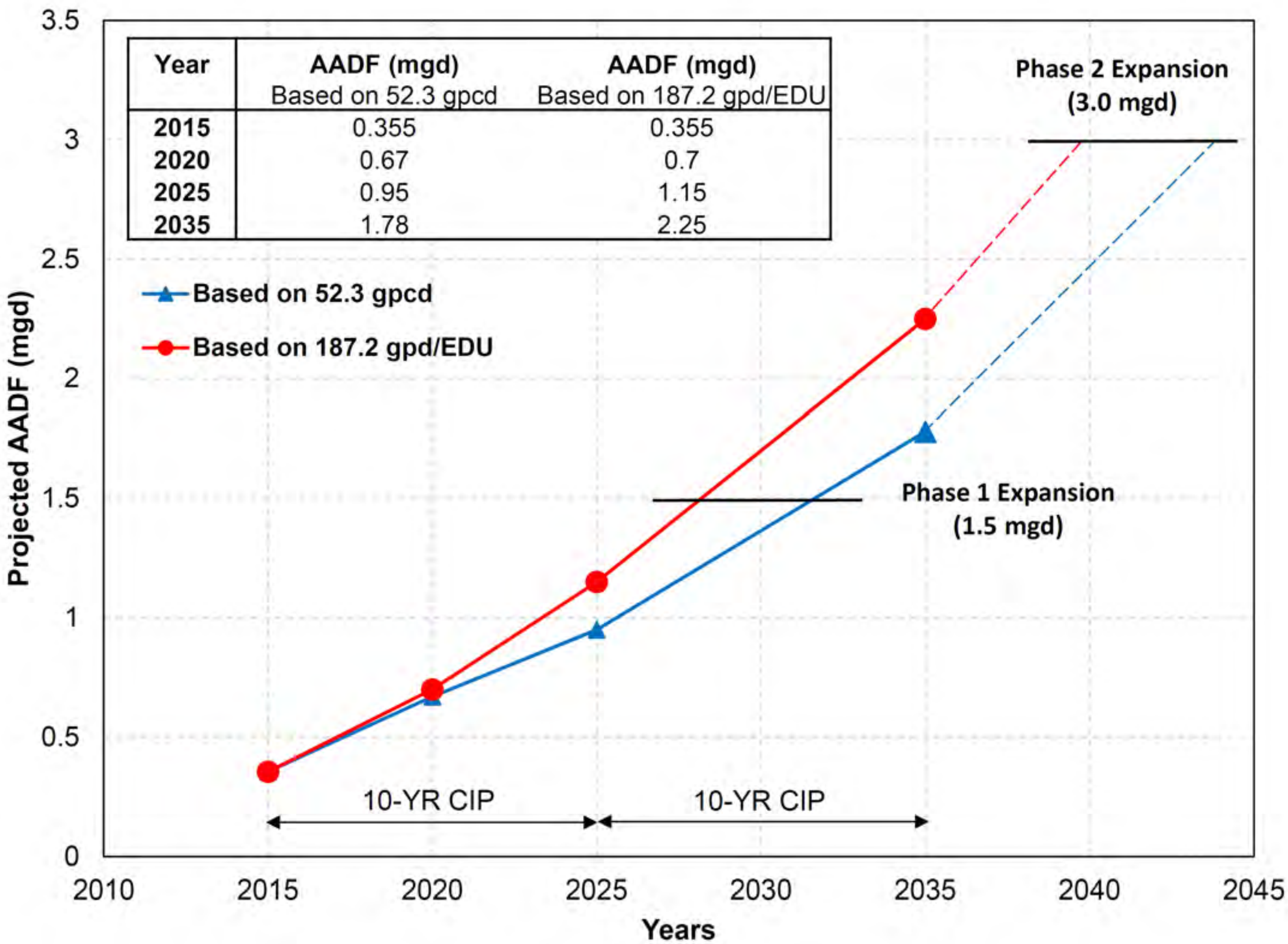
TM-1 documented the planning projections for the Marana service area and included wastewater flow projections for the next 20 years. These projections are shown in Figure 4 The flow projection graph shows a range of wastewater flow over the next 20 years based on two different criteria:

1. Future flows projected by using the current actual wastewater generation rate on a per capita basis, 52.3 gallons per day per capita (gpcd), and

2. Future flows projected using the committed, or assured, capacity rate for current and future development of residential and commercial properties, 187.2 gallons per day per Equivalent Dwelling Unit (gpd/EDU).

The initial evaluations of the wastewater growth projections lead to phasing the future plant expansions in 1.2-mgd increments. Phase 1 would be sized for an AADF flow of 1.2 mgd, Phase 2 for 2.4 mgd, and Phase 3 for 3.6 mgd. The evaluations presented in TM-3 and TM-4 are based on this sizing.

However, after the initial evaluations were completed, the Town staff reviewed its budgeting and financing projections. Additional sizing and cost estimating exercises were completed to weigh the WRF's needed capacity for the foreseeable future versus the financing abilities of the Town. During this review, it was noted that a Phase 1 AADF capacity of 1.5 mgd provided 25 percent more treatment capacity for only an additional 12 percent in funding. This is attributed to the portion of construction costs related to contractors' mobilization effort of labor and equipment, general indirect and risk management expenses often contribute significant costs, more than the increase cost of materials on a unit quantity basis.



| 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 PROJECTIONS

FIGURE 4

TOWN OF MARANA
*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

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It was decided that the WRF would be expanded to a Phase 1 AADF capacity of 1.5 mgd. Phases 2 and 3 would then follow in 1.5-mgd increments, to 3.0 and 4.5 mgd, respectively, as future growth and development required. Increasing the initial plant capacity facilitates the Town's desire to undergo only one plant expansion within the next 10-year Capital Improvement Plan budgeting period and meets its financing objectives.

A plant capacity of 1.5 mgd would provide sufficient treatment capacity to sometime between 2027 and 2032, at which time the Town must plan for an additional WRF expansion. This timeframe may be adjusted depending on the growth rate of new development and the subsequent wastewater flows generated.

Table 1 summarizes the proposed design capacity of WRF expansion phases. The Phase 3 expansion discussed in this evaluation, with a projected design capacity of 4.5 mgd, is based on assuming a modular expansion of 1.5 mgd in each phase. Phase 3 is expected to occur later than 2035 and is evaluated here for comparison and site layout purposes. Because the actual design capacity is subject to change, it is not practical to estimate wastewater flow more than 20 years into the future.

**Table 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 |
|---|---|---|---|---|
| Projected Year to Implement | -- | 2016 | 2027-2032 | Unknown |
| AADF, mgd | -- | 1.5 | 3.0 | 4.5 |
| MMADF, mgd | 1.1 | 1.65 | 3.3 | 4.95 |
| PDF, mgd | 2.0 | 3.0 | 6.0 | 9.0 |
| PHF, mgd | Varies (1) | 4.4 | 8.4 | 10.4 |

Note:
(1) Peak hour factors typically decrease as growth occurs and wastewater systems flows increase.

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

The recommendations and site plans presented in the following sections are based on the phasing and sizing presented in Table 1. The recommendations are different from the evaluations presented in TM-3 and TM-4.

## 3.2 Water Quality Goals

To design an expansion or improvement to a treatment process, a thorough understanding of the wastewater characteristics is important. Design wastewater characteristics were determined from an analysis of the plant's historical wastewater quality data, which were obtained from plant records of grab and composite samples of the plant influent.

Table 2 summarizes the key influent water quality constituents under both average annual day (AADF) and maximum month (MMADF) conditions. This water quality data was used for the process evaluation and for the sizing presented in TM-4 and subsequent sections. These wastewater characteristics are generally averaged for monitoring purposes, however, biological treatment systems should be prudently sized to meets their treatment goals under a maximum month loading condition.

**Table 2 Design Influent Wastewater Quality Characteristics**
**Marana Water Reclamation Facility Master Plan**
**Town of Marana**

| Parameter | Units | Average Concentration Under AADF | Average Concentration Under MMADF |
|---|---|---|---|
| BOD | mg/L | 228 | 269 |
| TSS | mg/L | 233 | 297 |
| TKN (1) | mg/L | 57 | 67 |
| $NH_3$-N | mg/L | 42 | 50 |

Notes:

(1) Additional subsequent sampling indicated higher events with TKN ranging between 70 - 85 mg/L. Process basin configuration can be optimized so treatment process can sufficiently treat events of higher than average TKN loading.

(2) Wastewater Quality Data from April 2012 - March 2015.

Abbreviations:

mg/L: milligrams per liter; TKN: total Kjeldahl nitrogen; $NH_3$-N: ammonia nitrogen

To determine wastewater treatment process alternatives for expanding the WRF, effluent quality goals must be enumerated. Per requirements in the Arizona Administrative Code (A.C.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, depending upon the classification and final use of the effluent produced.

The Marana WRF currently produces Class B+ effluent under authority of its Aquifer Protection Permit. However, considerations for treatment to Class A+ reclaimed water quality standards are incorporated into the evaluations. Table 3 details the treatment goals for future WRF expansion.

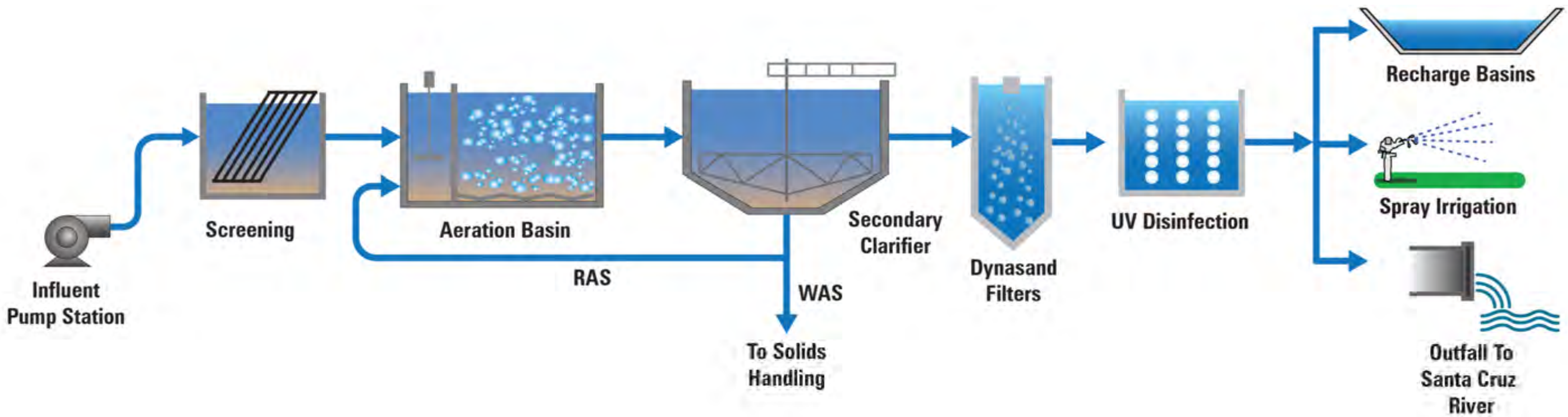
| Table 3 Effluent Water Requirements<br>Marana Water Reclamation Facility Master Plan<br>Town of Marana | |
|---|---|
| **Criteria** | **Treatment Requirement** |
| Treatment Requirement | Secondary Treatment meeting<br>$BOD_5$ < 30 mg/L (30-day avg)<br>Or<br>$CBOD_5$ < 25 mg/L<br>(30-day avg) |
| Filtration | Class A+: Required with coagulant addition.<br>Class B+: Not required. |
| Total Suspended Solids | < 30 mg/L (30-day avg) |
| Turbidity Limit | Class A+: 2 NTU (24-hour avg)/5 NTU (max)<br>Class B+: Not specified. |
| pH | Between 6.0 – 9.0 |
| Removal Efficiency | 85% of $BOD_5$, $CBOD_5$, and TSS |
| Total Nitrogen | <10 mg/L<br>5-month rolling mean |
| Fecal Coliform Limits | Class A+: Non-detectable in 4 out of 7 daily samples<br>23 MPN or cfu/100 mL max<br>Class B+: 200/100 mL in 4 out of 7 daily samples<br>800 MPN or cfu/100 mL max |

Source:
A.A.C. R18-9-part B, September 30, 2005
Abbreviations:
avg: average; NTU: nephelometric turbidity unit; max: maximum; MPN: most probable number; cfu: coliform forming units; mL: milliliter

## 4.0 RECOMMENDATIONS

The existing Marana WRF consists of the following facilities: preliminary treatment, influent pumping, secondary treatment, secondary effluent pumping, tertiary filters, ultraviolet (UV) disinfection, and a plant effluent outfall structure. The WRF also includes backup systems for chlorination/dechlorination as well as auxiliary systems for odor control, utility water, and standby power generation.

During the Phase 1 Expansion, most of these facilities need to be expanded or modified for a higher treatment capacity and to provide provisions for future expansion. Recommendations for the expansion are described in the following sections. The recommended new liquids and solids process flow diagrams are shown in Figure 5 and Figure 6, respectively. A conceptual site plan of the new facilities is shown in Figure 7.

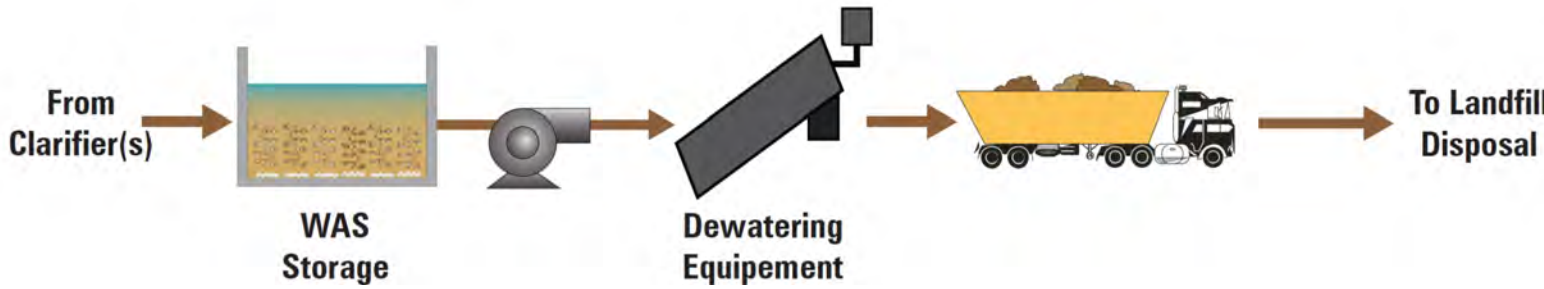



# NEW LIQUID PROCESS FLOW DIAGRAM

FIGURE 5

TOWN OF MARANA

*MARANA WATER RECLAMATION FACILITY MASTER PLAN*




**NEW SOLIDS PROCESS FLOW DIAGRAM**

FIGURE 6

TOWN OF MARANA
*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

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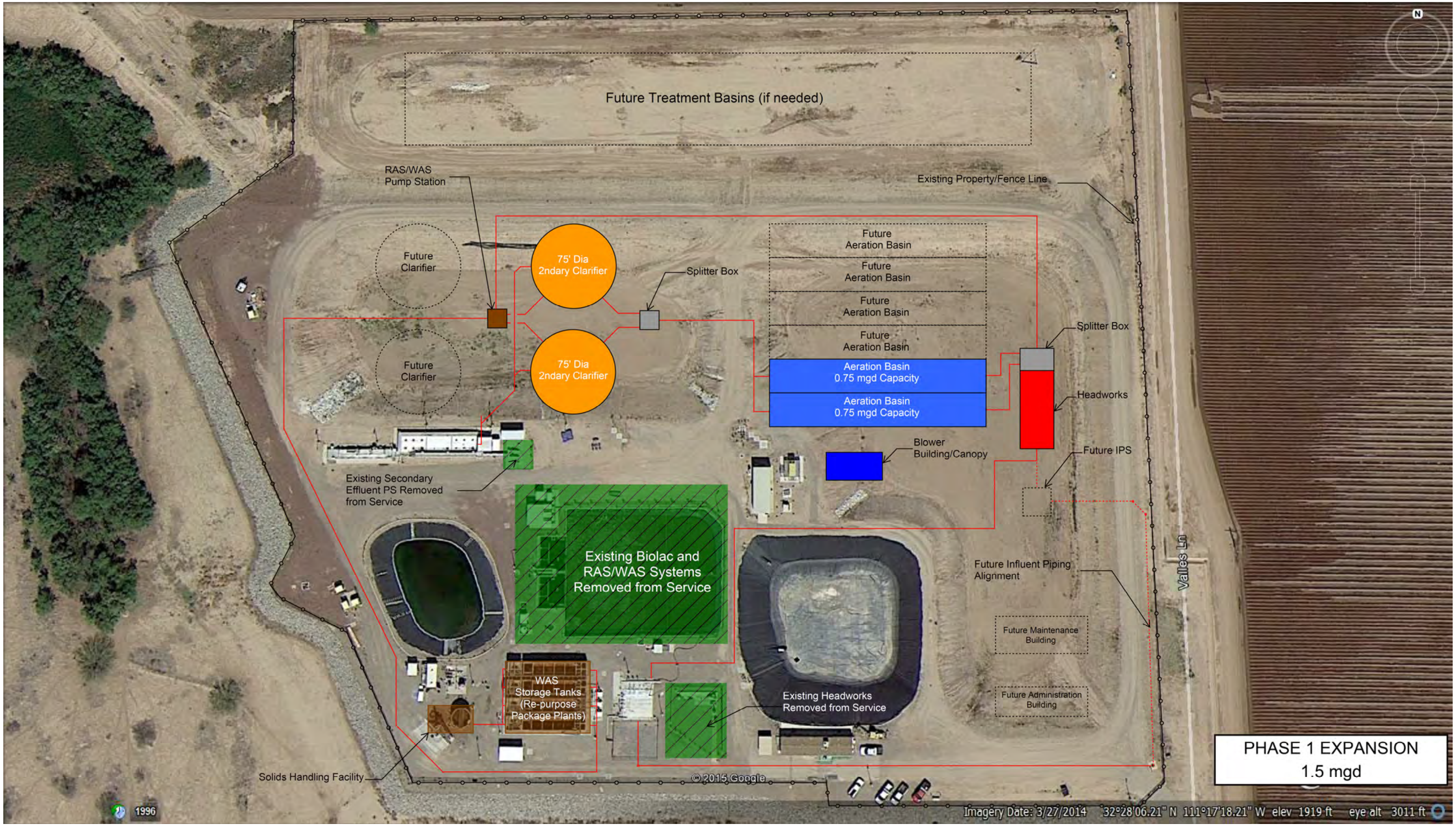


**PHASE 1 CONCEPTUAL SITE PLAN**

FIGURE 7

TOWN OF MARANA

*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

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## 4.1 Preliminary Treatment

Currently, wastewater enters the WRF and travels to the headworks through a 12-inch diameter pipe at the end of the 24-inch Gladden interceptor. The headworks consist of influent flow monitoring, mechanical bar screens, a grit removal chamber, and influent sampling. From the headworks, raw wastewater flows to the influent pump station (IPS) through an 8-inch pipe.

### 4.1.1 Necessary Improvements

Given the increased flows and loads projected for the future, the Town's current preliminary treatment facilities lack sufficient capacity. The influent pipe has an estimated capacity of 1.4 mgd, which is below the design daily average flow of 1.5 mgd required for the Phase 1 Expansion. Furthermore, the headworks has a maximum peak hydraulic capacity of 1.5 mgd and is limited by the size of the Parshall flume (designed for 0.22 to 1.2 mgd) and the 8-inch pipe between the headworks and the IPS. To mitigate these issues, several improvements are needed.

First, a new influent sewer main is required from the plant entry manhole to the IPS. The new sewer pipe size should be sufficient to convey the Phase 1 design peak hour flows.

As detailed in TM-4, a new IPS will be required at some point in the future. However, for this initial plant expansion, Phase 1, the current structure may continue to be used and the existing IPS pumps can be replaced with higher capacity pumps sized for higher flows and at a higher discharge head. Complete replacement of the IPS structure can be deferred until some point in the future, when the collection system of the entire service area has been more clearly defined and the depth of future interceptors have been established.

For the headworks, upgrading the existing headworks is not physically feasible, nor economically practical. As a result, building a new headworks at a nearby location is recommended.

A new discharge force main will be required to convey flows from the IPS to a new headworks structure. A new headworks structure would include mechanically screening of the influent to remove large solids and other inorganic and non-biodegradable solids. The new headworks structure would be elevated sufficiently to raise the hydraulic grade line of the secondary process above the existing filters, thereby eliminating the need for internal pumping. Currently, secondary effluent is lifted up by the secondary effluent pump station to the existing filtration system. It is recommended to eliminate this internal pump station since it contributes to both operating and maintenance expenses over the life of the WRF.

In addition to these facilities, the Town has the option to install a grit removal system, after the screening process. Grit removal is typically done to eliminate the build-up of grit, such as sand and dirt, in downstream basins and to eliminate the abrasive wear on downstream equipment.

Adding a new grit removal system is not recommended now because of its high capital cost, and it is not included in the Phase 1 Expansion. However, it is recommended that the Phase 1 design documents make structural and hydraulic provisions for a future grit removal system. This includes reserving the physical space for future grit systems and leaving sufficient room in the hydraulic profile for the energy losses associated with a future grit system. The Town should also weigh the costs of including a grit system with the costs of having to periodically clean the downstream basins if no grit system is in place.

## 4.2 Secondary Treatment

The existing secondary treatment process at the Marana WRF is an extended aeration activated sludge system based on a Parkson-manufactured Biolac® system. This system consists of a lined earthen basin (aeration basin) and integral rectangular secondary clarifiers. Aeration is achieved with a diffused air system and uses positive displacement blowers and tubular membrane diffusers. The system uses airlift pumps to control return activated sludge (RAS) and waste activated sludge (WAS), with an additional WAS lift station onsite to waste excess solids to the solids storage tank before disposal.

### 4.2.1 Biological Treatment

For this Master Plan, three alternatives for biological nutrient removal, including the current process, were evaluated. These alternatives are described below.

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

From the evaluation, the CAS process was selected for future expansions at the Marana WRF. This process will provide the most flexibility for capacity, basin arrangement, and treatment needs. The CAS system was selected for the following reasons.

1. Its custom-designed anoxic and aeration volumes provide maximum flexibility under different conditions. Since there are many uncertainties and the Town is experiencing rapid change, this flexibility will help the Town meet its treatment needs even with changing influent conditions. Furthermore, by compartmentalizing the treatment basins into separate zones for nitrification and denitrification, the CAS system can provide optimum conditions to nitrify and denitrify under a relatively wide range of wastewater characteristics. With this, the WRF can consistently meet is Aquifer Protection Permit (APP) discharge limits.

2. In contrast, the current Biolac® system is not sufficient to meet the WRF's treatment needs. It presents significant challenges for process operations, including poor settleability that affects filter and disinfection system performance and poor nitrogen removal that may lead to permit violations. As the Town grows and flows increase, these challenges will become more difficult to overcome.

3. The site footprint requirements of the Biolac® system do not align with the plant's future growth and expansion needs. The Biolac® system requires large basins for treatment, which limits the site's capacity to approximately 2.4 mgd. Since the Town anticipates growth beyond 2.4 mgd, the Biolac® system may limit development.

4. Although the BNR-OD is a viable technology, the cost estimates developed in TM-4 indicate that it costs more than a CAS because it has a larger footprint. In addition, an oxidation ditch process is less flexible in accommodating unknown future wastewater concentrations.

5. The CAS system utilizes more efficient diffuser technology than the Biolac® system, requiring less aeration blower horsepower, which can lower electrical operating costs. The CAS also had more efficient operations and maintenance than the BNR-OD, leading to superior process control.

6. Of the three alternatives evaluated, the CAS had the lowest footprint, allowing for the most flexibility for site planning and the largest ultimate treatment capacity at the site. Since the Town is experiencing rapid growth, this capacity is particularly important.

When compared with the other alternatives evaluated, the CAS system will perform better under future changing flow and loading conditions. The CAS system will meet the Town's needs to provide cost-effective, reliable treatment as the development occurs. Ultimately, the CAS system will allow the Town to achieve its goal of recharging high-quality reclaimed water to accrue water storage credits.

### 4.2.2 Secondary Clarification

The WRF's current configuration involves integral-type, rectangular "V-shaped" bottom clarifiers, which provide no redundancy during maintenance or process failure. Thus, operating the secondary clarifiers independently from the bioreactors is recommended to maintain the system capacity when a bioreactor or a secondary clarifier is taken out of service.

For the Marana WRF, a system with two bioreactors and two circular secondary clarifiers is recommended. The system will be designed to 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 mixed liquor suspended solids (MLSS) would be increased to make up for the lost volume, since both clarifiers would be in service to handle the increased MLSS at the design capacity.

These circular clarifiers will perform better than the current clarifiers. Because the circular clarifier mechanisms include a flocculating well, they allow operators to dose polymer to help flocculate the mixed liquor if sludge bulking occurs. The increased side water depth of the clarifiers and more efficient sludge and scum removal mechanisms also provide better overall performance. This enhanced performance helps downstream processes, such as filtration and disinfection, operate more efficiently.

### 4.2.3 RAS/WAS Pumping

For the Marana WRF, a dedicated RAS/WAS pump station is proposed. This could be accomplished with either submersible pumps in a wet well or with dry-pit style pumps. This pump station will provide a reliable, yet economical way to control RAS and WAS flow accurately.

A mechanical pumping mechanism is recommended for the WRF. The existing airlift pumping mechanisms rely on process air to control the sludge flow from the secondary clarifiers, which makes controlling the flow of liquid difficult. Mechanical pumping will provide greater control of RAS and WAS flows from the secondary clarifier, allowing operators to adjust the amount of liquid based on flow meter readings attached to the pumps.

## 4.3 Tertiary Treatment

Currently, DynaSand® tertiary filters treat the effluent from secondary treatment. For these filters, three concrete basins house six filter modules, allowing for a peak wet weather flow (PWWF) of 5.4 mgd. In addition, an in-channel Trojan UV disinfection system treats filtration effluent before discharge. For the UV system, three UV banks are in one channel, which can also accommodate a PWWF of 5.4 mgd.

### 4.3.1 Necessary Improvements

Both the current filter system and the UV disinfection system have adequate treatment capacity and will not need to be expanded during Phase 1 construction. However, some repairs are needed, which are explained below.

Although the filter basins' concrete structure is in excellent condition, one filter cell is out of service due to a collapsed module. This filter cell should be repaired.

Furthermore, backwash rates are significantly higher than the design level. Returned backwash waste flow is approximately 42 percent of the current average daily flow, which dramatically affects treatment performance. Carollo recommends retrofitting the three filters with Parkson's EcoWash, a new filter wash system designed for the DynaSand® filters. This system claims to reduce the power and volume of returned backwash flows by up to 85 percent. Depending upon available budgets, the filter repair and backwash system improvements could be completed as an operations and maintenance improvement project and not necessarily as part of the plant expansion project.

## 4.4 Solids Handling

Currently, the Town pays a third-party contractor to haul liquid waste sludge to the City of Casa Grande WRF, which is 54 miles away. This method of sludge disposal incurs high costs due to the distance of the trip and the volume of liquids being disposed. For the Marana WRF, the cost for sludge hauling and treatment represents the second highest operational cost after the process blowers.

### 4.4.1 Necessary Improvements

The Town would like to construct a sludge dewatering facility to produce dewatered sludge that can be disposed of at a nearby landfill. Based on the economic analysis presented in TM-3, this dewatering facility can significantly lower solids disposal costs. As such, installing the facility during the Phase 1 Expansion is recommended.

For the facility, the following four dewatering technologies were evaluated for their performance, operation, footprint, and cost: belt filter presses (BFPs), centrifuges, screw presses, and rotary fan presses. Recommendations for selecting the technology from the four evaluated and for installing them after they are selected are summarized below.

- **Type of technology recommended:** Of the four technologies evaluated, the screw press and rotary fan press are recommended for further consideration during preliminary design. If feasible, pilot testing could be performed during preliminary design to determine the optimal polymer usage and to familiarize plant staff with the equipment.

- **Number of units required:** Installing one duty unit and one stand-by unit is recommended for redundancy. However, if budget constraints require it, the Town could install a single dewatering unit instead of two in the Phase 1 expansion. Although a second unit is typically recommended for redundancy, it can be postponed until additional funding is available.
- **Location of the new facility:** Removing the existing sludge storage tank and installing the dewatering facility in the same location is recommended to make use of the current truck driving route.
- **Method of installation:** Installing the 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 a hauling truck.

In addition to the dewatering facility, sludge storage is recommended to provide additional operational flexibility for the main treatment and dewatering processes. With sludge storage, if something happens to the solids handling process, sludge can be stored for a designated amount of time and the secondary process can continue to operate.

Under AADF, at least four days of WAS storage is recommended. Three days of storage is recommended under MMADF.

As discussed in TM-3, the existing package treatment plants are no longer in service and have not been operated since 2006. Since these package plants are not being utilized, it could be economical to repurpose them for WAS storage instead of constructing a new WAS storage facility.

To meet the storage time requirement, installing a new coarse bubble diffuser is recommended to utilize the entire available storage volume of all four package plant trains. During preliminary design, the decanting capability of the repurposed packaged plant can be further evaluated.

## 5.0 PHASE 1 EXPANSION

The Phase 1 Expansion will increase the WRF's total treatment capacity from 0.5 mgd to 1.5 mgd and change the main treatment process employed at the facility.

## 5.1 New Facilities

Several new facilities are recommended during the Phase 1 Expansion. These facilities are as follows:

- New influent sewer to the existing IPS.
- Install new pumps in the existing IPS.
- Construct new force main from existing IPS to new headworks
- Install new headworks with mechanical screening.
- Install a new conventional activated sludge system, consisting of two concrete basins, air delivery system, and blower system.
- Install two 75-foot secondary clarifiers, with scum pump station(s).
- Install a new combination RAS/WAS pump station.
- Install a new solids handling facility, including dewatering equipment, pumps, and storage as necessary.
- Electrical and control systems improvements as required to serve the new facilities.

The existing IPS wet well will remain in service. However, new pumps will be required to convey raw influent to the new headworks.

A new secondary treatment system is recommended to provide reliable treatment and to consistently meet APP permit limitations. The conventional activated sludge system is recommended and would include two, one-pass aeration basins equipped with anoxic zones, mixers, air distribution and diffuser system, and a means for internal mixed liquor return. Following the aeration basins would be two secondary clarifiers, which will settle out the solids and send treated effluent to the existing downstream processes of final filtration and disinfection.

At the solids handling facility, the existing package plants will be repurposed for WAS storage. Adjacent to the package BNR plants, a new dewatering facility will be constructed. This facility will consist of WAS feed pumps, one dewatering equipment, a polymer system, a cake conveyor, and other appurtenances as required. Purchasing a second redundant dewatering unit can be deferred until additional funding is available.

A maintenance facility is recommended as repairing pumps and equipment in the field is not always feasible. A warehouse style metal building with large roll-up doors is appropriate for a work area as well as parts and tools storage.

## 5.2 Existing Facilities Removed from Service

Several existing facilities at the WRF will no longer be required for treatment and will be removed from service during the Phase 1 Expansion. These facilities are as follows:

- Demolish existing headworks.
- Remove existing Biolac® system from service. Repurpose if appropriate.
- Secondary effluent pump station may be taken out of service - but does not need to be physically removed.
- Demolish existing air lift RAS system and WAS wet well and pump.
- Remove existing 16,500-gallon sludge storage tank.

## 5.3 Existing Facilities Remaining in Service

Several existing facilities at the WRF will remain in service following the Phase 1 Expansion. These facilities are as follows:

- Influent pump station - wet well.
- Tertiary filters.
- UV disinfection system.
- Package plants - to be refurbished and used as WAS storage.
- Effluent storage pond.
- Chlorination and dechlorination systems.
- Reuse water system.

## 5.4 Opinion of Probable Cost

The Engineer's Opinion of Probable Cost for the Phase 1 Expansion is in Table 4.

| Table 4 | WRF Phase 1 Expansion to 1.5-mgd Opinion of Probable Cost<br>Marana WRF Master Plan<br>Town of Marana | |
|---|---|---|
| **Description** | | |
| General Requirement | 8% | $765,500 |
| Yard/Site Facilities | 12% | $1,151,300 |
| New Influent Sewer Line | | $99,200 |
| Headworks Facilities | | $726,100 |
| Influent Pump Station | | $157,500 |
| Bioreactor Splitter Box | | $229,300 |
| Secondary Treatment | | $3,818,300 |
| Existing Secondary Effluent Pump Station Demolition | | $ -- |
| Secondary Clarifier Splitter Box | | $206,400 |
| Secondary Clarifier | | $2,840,900 |
| RAS/WAS Pump Station | | $455,100 |
| Tertiary Filter Repair and Backwash Improvements(4) | | $ -- |
| WAS Storage Tank | | $243,000 |
| Dewatering Facilities' | | $917,600 |
| ***Subtotal "Baseline" Direct Costs*** | | ***$11,612,200*** |
| Escalation Contingency | 3% | $348,400 |
| Design Contingency | 25% | $2,903,100 |
| **TOTAL DIRECT COSTS** | | **$14,863,700** |
| General Conditions | 10% | $1,486,400 |
| ***Subtotal 1*** | | ***$16,350,100*** |
| Bond and Insurance | 2.5% | $371,600 |
| ***Subtotal 2*** | | ***$16,721,700*** |
| Sales Tax (at 65% of current Marana tax rate) | 8.6% | $830,900 |
| **TOTAL ESTIMATED CONSTRUCTION COST** | | **$17,552,600** |

Notes:

(1) Does not include engineering design and construction administration services, Town's legal and administrative costs, and permitting fees.
(2) Costs presented are a Budget Level, Class 4 estimate as defined by the American Association of Cost Engineers International (AACEI) and as such can be -30% to + 50% in accuracy. Carollo Engineers has 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 determining prices, competitive bidding or market conditions, practices or bidding strategies.
(3) Engineering News-Record (ENR) 20-Cities Average Construction Cost Index for February 2016 = 10,182.
(4) Filter cell repair and backwash improvements assumed to be completed as an O&M project, and not included in this budget for plant expansion.

## 6.0 PHASE 2 EXPANSION

The Phase 2 Expansion will increase the WRF's total treatment capacity from 1.5 mgd to 3.0 mgd. Figure 8 presents a conceptual site plan of the Phase 2 Expansion of the WRF.

The estimated timing of the Phase 2 Expansion would be sometime between 2027 and 2032. Since that timeframe is so far in the future, it is not reasonable to prepare a detailed construction cost estimate, as technologies and level of automation may change. However, a reasonable expectation of plant expansion costs could be between $15 and $18 million dollars, based on a current average cost of $10-$12 per gallon (in 2016 dollars). Escalated over time, at 3 percent per year to 2027, costs could be between $21 and $25 million dollars.

### 6.1 New Facilities

Several new facilities are recommended with the construction of the Phase 2 Expansion. These facilities are listed below:

- Influent sewer to new IPS.
- Deeper influent pump station.
- Secondary Treatment - conventional activated sludge basins and blower system.
- Secondary Treatment - secondary clarifiers.
- Solids Handling Facility expansion.
- Administration Building with laboratory facilities.

Expanding the Solids Handling Facility will involve adding one or more new sludge storage tanks. The Dewatering Facility will also be expanded to accommodate additional dewatering equipment and appurtenances as required.

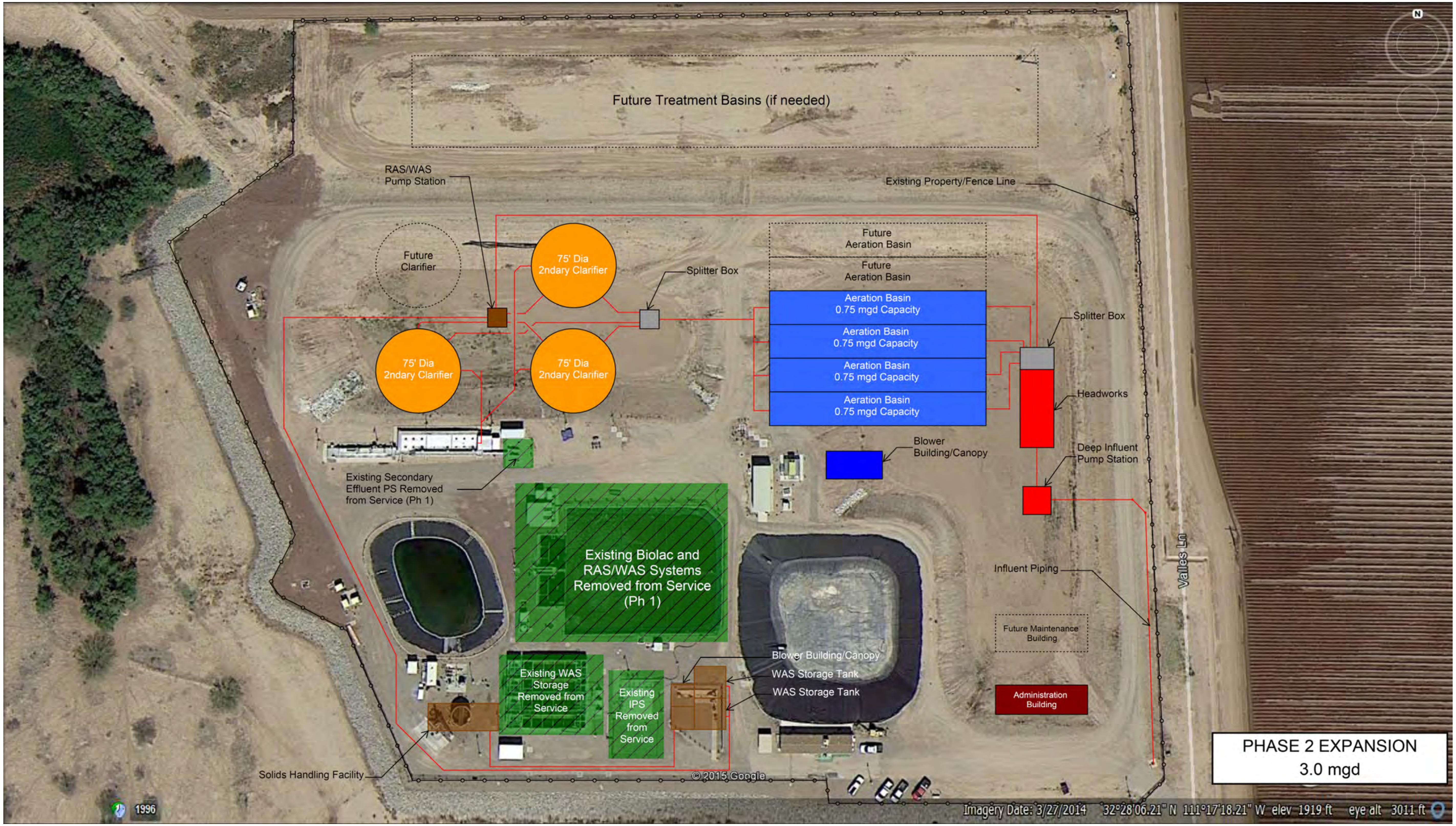
### 6.2 Existing Facilities Removed from Service

The package plants for WAS storage are the only existing facilities no longer required for treatment. These facilities will be removed from service during the Phase 2 Expansion.

### 6.3 Existing Facilities Remaining in Service

Several existing facilities at the WRF will remain in service following the Phase 2 Expansion. These facilities are as follows:

- Tertiary filters.
- UV disinfection system.
- Effluent storage pond.
- Reuse water system.




## PHASE 2 CONCEPTUAL SITE PLAN

FIGURE 8

TOWN OF MARANA
*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

carollo

pw:\\Carollo/Documents\Client/AZ/Marana/10067A00/Deliverables\Figure 8

# 7.0 PHASE 3 EXPANSION

The Phase 3 Expansion will increase the WRF's total treatment capacity from 3.0 mgd to 4.5 mgd. For a conceptual site plan of the Phase 3 Expansion, consult Figure 9.

## 7.1 New Facilities

Several new facilities are recommended with the construction of the Phase 3 Expansion. These facilities are as follows:

- Secondary Treatment - conventional activated sludge basins and blower system.
- Secondary Treatment - secondary clarifiers.
- Tertiary Treatment - filters.
- Tertiary Treatment - UV disinfection.
- Solids Handling Facility.

Expanding the Solids Handling Facility will include additional sludge storage tanks to maintain required storage time. The Dewatering Facility will also be expanded to accommodate additional dewatering equipment and appurtenances as required.

## 7.2 Existing Facilities Removed from Service

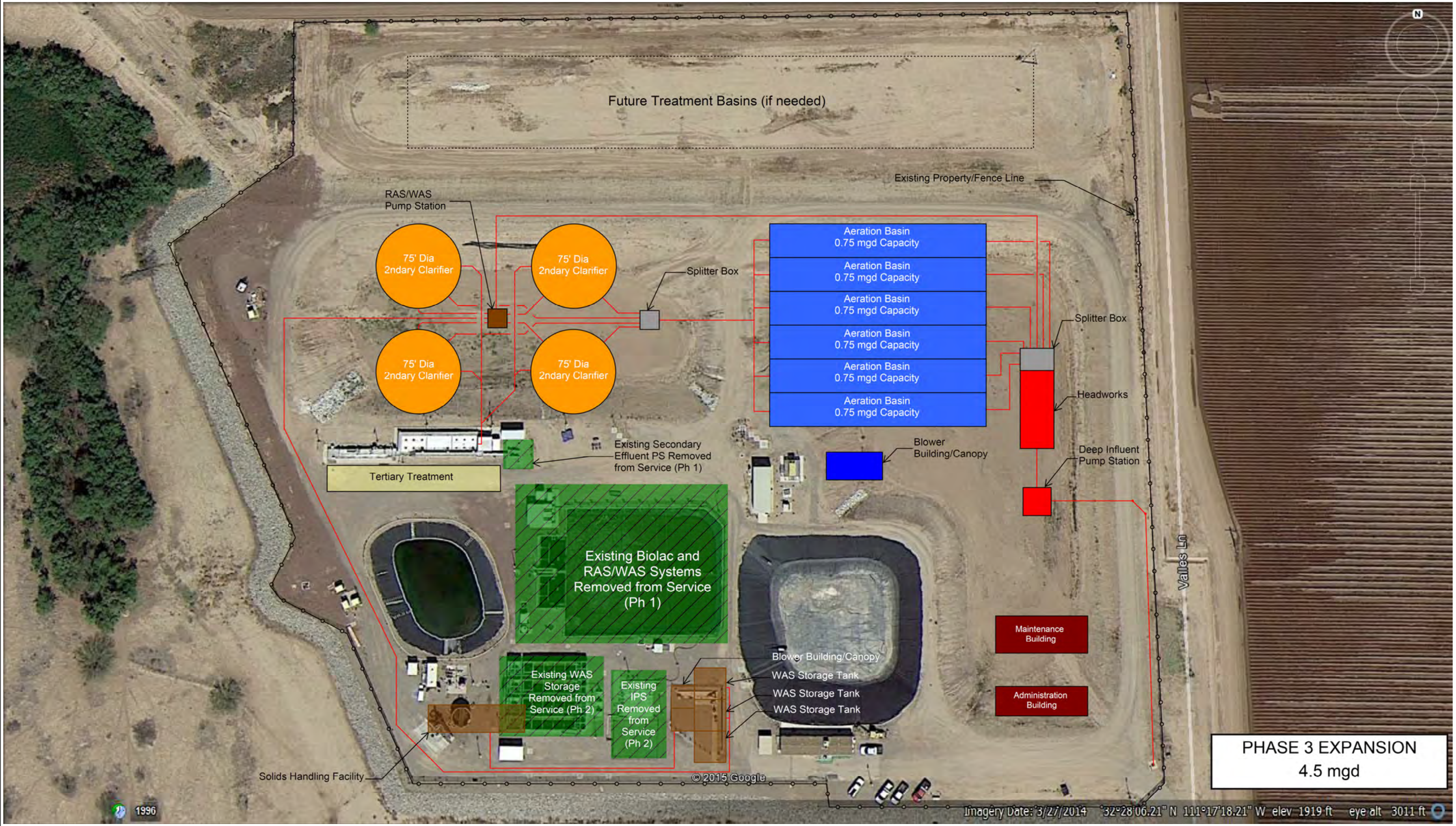
No existing facilities are expected to be removed from service during the Phase 3 Expansion. However, given the timeline for implementing the Phase 3 Expansion, aging equipment or facilities may require replacement or rehabilitation.

## 7.3 Existing Facilities Remaining in Service

Existing facilities at the WRF that will remain in service following the Phase 3 Expansion are as follows:

- Tertiary filters.
- UV disinfection system.
- Effluent storage pond.
- Reuse water system.

Given the timeline for implementing the Phase 3 Expansion, preparing a detailed opinion of probable cost is impractical.




**PHASE 3 CONCEPTUAL SITE PLAN**

FIGURE 9

TOWN OF MARANA

*MARANA WATER RECLAMATION FACILITY MASTER PLAN*

carollo

pw:\\Carollo/Documents\Client/AZ/Marana/10067A00/Deliverables\Figure 9

## 8.0 STAFFING REQUIREMENTS

An exercise to determine staffing needs was completed so that the Town may plan and budget for the increased staff that will be necessary to operate and maintain the new facilities. Our estimation is based on The New England Interstate Water Pollution Control Commission's published guide entitled *"The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants"* (November 2008). A preliminary placeholder for collection system staff was included in the total staff estimate. This number may be revised as actual O&M demands require additional personnel. A summary of the staffing needs are provided in Table 5.

**Table 5 Estimated Staffing Needs**
**Marana WRF Master Plan**
**Town of Marana**

| | **Phase 1** | **Phase 2** | **Phase 3** |
|---|---|---|---|
| AADF Flow | 1.5 mgd | 3.0 mgd | 4.5 mgd |
| WRF and Recharge Basins Staff | 5 - 6 | 8 - 9 | 11 |
| Collection System Staff | 1 | 2 | 3 |
| Supervisory Staff | 1 | 1 | 1 |
| Total Staff | 7 - 8 | 11 - 12 | 14 - 16 |

Note:

(1) Based on The New England Interstate Water Pollution Control Commission's published guide entitled *"The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants"* (November 2008).

The detailed manhour estimation for operation and maintenance activities are provided in Appendix E.

The Arizona Administrative Code (A.A.C.) establishes different grades of treatment facilities and collection systems based upon the complexity of the system and the number of people served. The regulations also require a facility owner must ensure that a treatment facility has an operator in direct responsible charge who is certified at the facility grade or higher. When the operator in direct responsible charge is absent, an operator in charge may be at no more than one grade lower than the grade of the facility. Table 6 summarizes the facility grade requirements.

| **Table 6 Facility Classification and Operator Certification<br>Marana WRF Master Plan<br>Town of Marana** | | |
|---|---|---|
| **Facility Classification and Operator Certification** | **WW Treatment Plant** | **WW Collection System** |
| Grade 1 | Stabilization pond that serves 2,000 persons or less | Serves 2,500 persons or less |
| Grade 2 | Stabilization pond that serves more than 2,000 persons, or<br>An aerated lagoon, or<br>Biological treatment based upon activated sludge or trickling filter that serves 5,000 or fewer persons. | Serves between 2,501 and 10,000 persons |
| Grade 3 | Biological treatment based upon activated sludge principle and is designed to serve 5,001 to 20,000 persons; or<br>A facility that employs trickling filtration and is designed to serve 5,001 to 25,000 persons; or<br>A variation of biological treatment based on the activated sludge principle that requires specialized knowledge, including contact stabilization, and is designed to serve 20,000 or fewer persons. | Serves between 10,001 and 25,000 persons |
| Grade 4 | Biological treatment based upon activated sludge principle and is designed to serve more than 20,000 persons; or<br>A facility that employs trickling filtration and is designed to serve more than 25,000 persons. | Serves more than 25,000 persons |

The service area for the WRF currently serves a population less than 10,000 persons. A Grade 2 operator is required for operation of the collection system. When the population of the service area exceeds 10,000 persons, a Grade 3 Collection System operator will be required. The WRF currently employs an activated sludge treatment system and, therefore, is classified as a Grade 3 facility, requiring an operator of equal certification. When the population of the service area grows larger than 20,000 persons, the facility will require a Grade 4 operator.

# APPENDIX A – TM-1 FLOW AND LOADING PROJECTIONS

# MARANA WATER RECLAMATION FACILITY FLOW AND LOADING PROJECTIONS

Registered Professional Engineer (Civil)
CERTIFICATE NO. 43796
ROBERT JOSEPH ARCHER
DATE SIGNED 2/15/16
ARIZONA U.S.A.

Expires 3/31/2018

| | |
|---|---|
| **Prepared for:** | Carollo Engineering |
| **Prepared by:** | Robert J. Archer, P.E. WestLand Resources, Inc. |
| **CC:** | Thomas A. Martinez, P.E., WestLand Resources, Inc. |
| **Date:** | February 15, 2016 |
| **Project No.:** | 527.75 |

## TABLE OF CONTENTS

## TABLES



## FIGURES

## EXHIBITS

(*follow text*)



## APPENDICES

(*follow text*)

# 1. INTRODUCTION

WestLand Resources, Inc. (WestLand) was retained by Carollo Engineers (Carollo) to prepare flow and loading projections for the Town of Marana and Carollo in support of the Marana Water Reclamation Facility (Marana WRF) Master Plan. The Marana WRF is owned by the Town of Marana, and is operated by the Town of Marana Utilities Department.

The Marana WRF treats about 0.35 million gallons per day (MGD) and has the physical capacity to treat 0.5 MGD; although parts of the plant can treat up to 3.5 MGD. The purpose of the master plan is to plan for future growth, and the purpose of this study is to provide projections of flow and loading over the next 20 years. The projections are based on historical flows and flow peaking, historical influent water quality, and growth projections from the Town of Marana Planning Department.

The study is presented in the following 4 sections:

- Section 2: Flows and Peaking Factors – analysis of historical flows and calculation of peaking factors,
- Section 3: Water Quality – analysis of historical water quality,
- Section 4: Constituent Loading – calculation of average and peak loadings for flows from 0.5 MGD to 3.5 MGD based on historical flow and water quality.
- Section 5: Projections – projections of flow for years 2020, 2025, and 2035. Also contains buildout projections by sewer basin.

# 2. FLOWS AND PEAKING FACTORS

This section contains an analysis of historical flows, historical per capita flow, and peak flows associated with projected flows. There are several flows that are useful in treatment plant design: the annual average day flow (AADF), the maximum month average day flow (MMADF), the peak day flow (PDF), and the peak-hour flow (PHF). Peaking factors are the relationship of each peak flow to the annual average flow. Per capita flow is used to calculate future flows based on growth projections within the service area.

## 2.1. HISTORICAL DAILY FLOW

***Figure 1*** shows the historical daily flows recorded at the Marana WRF influent flume from April 1, 2012 through September 30, 2015. ***Figure 1*** also shows a 30-day moving average of the daily flow for the same period. ***Appendix A*** contains tables of the daily flow data for the same period.

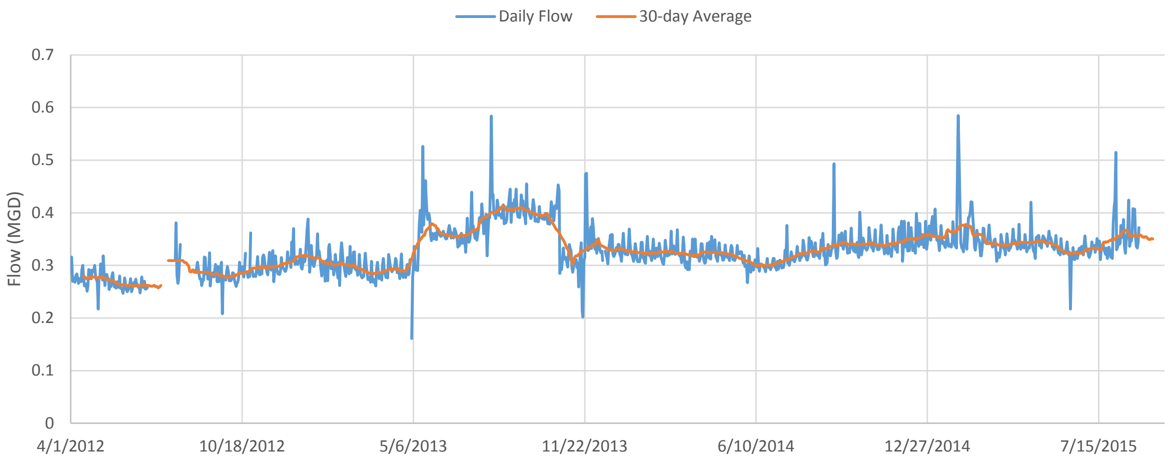



Figure 1. Marana WRF daily flow from 4/2/2012 – 9/30/2015 with 30-day average

***Figure 1*** also shows that there are periods where data was not recorded, and that there are unusually high flows from early May 2012 through late October 2013. Based on communications with the Marana WRF operator, this unusually high recorded flow could have been caused by either a problem with the flow sensor, or water backing up due to a clogged screen. The operator does not think the reported values represent actual flows, therefore, as shown in ***Figure 2***, data from May 3, 2013 through October 24, 2013 were not used in the flow analyses.

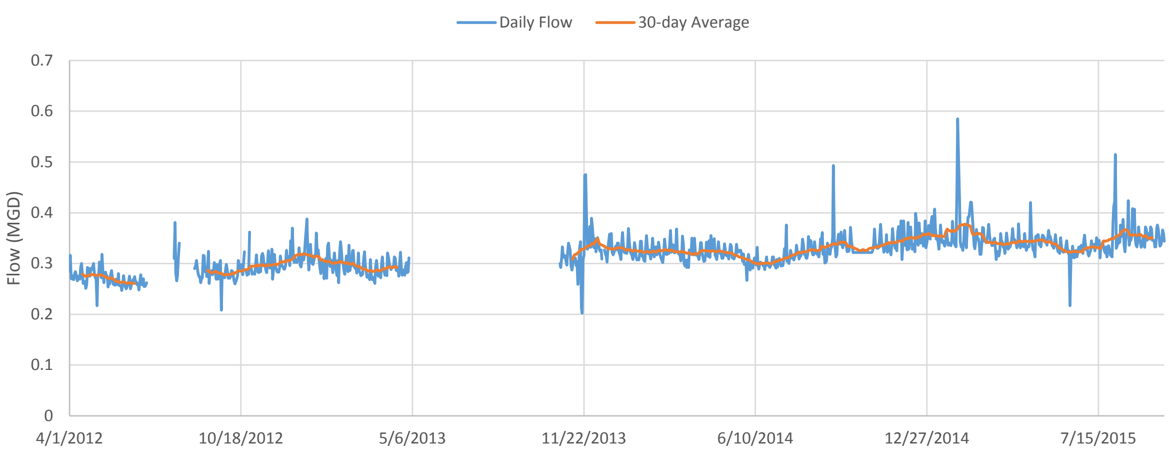



Figure 2. Daily flow data and 30-day average from 4/2/12-9/30/15, but without 5/3/13-10/24/13

To eliminate the general trend from the daily flow data in order to calculate peaking factors, the daily flow regressed on the date (***Figure 3***). The resulting line explains about 44 percent of the variation ($R^2$=0.44, p<0.01), and shows an average 61 gpd/day increase in flow.

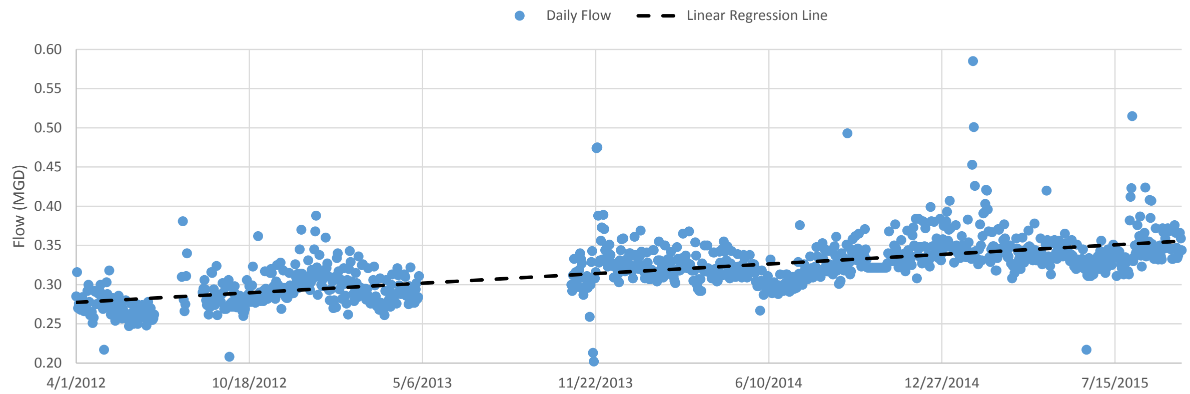



Figure 3. Linear regression of daily flow data

## 2.2. AVERAGE ANNUAL DAY FLOW

***Figure 4*** shows the daily flow for the last year (October 2014 through September 2015). ***Figure 4*** also shows the 30-day average flow for the same period. The average flow for the year preceding September 30, 2015 – shown in red – is approximately 0.345 MGD. The regression shown in ***Figure 3*** results in a predicted average flow of 0.355 MGD for September 30, 2015, and the average for September 2015 is 0.350 MGD.

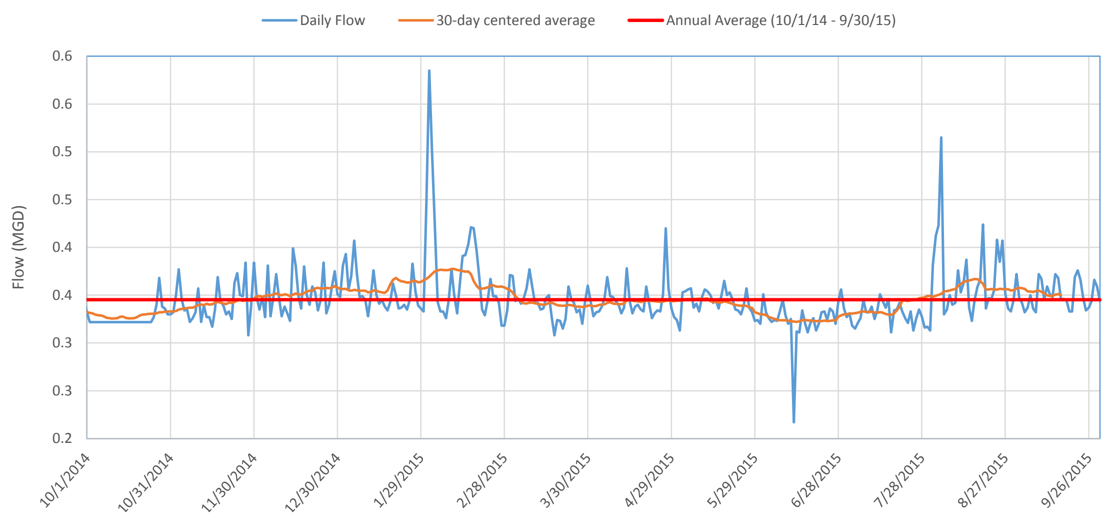



Figure 4. Daily, 30-day average, and annual average flow, 10/1/14 – 9/30/15

For purposes of this study, the existing AADF is based on the regression line, and future AADF is based on projections. Existing AADF as of September 30, 2015 is 0.355 MGD.

## 2.3. PER CAPITA FLOW

Per capita flow is the average amount of flow, usually in gallons per day, produced by a person. Based on the 2010 census, there are an average of 2.7 persons per residential dwelling (ppdu) unit in the Town of

Marana. Non-residential sources are assigned a number of equivalent dwelling units (EDU) and a virtual population of 2.7 persons per EDU.

***Figure 5*** shows the average flow and number of connections for each month between April 2012 through September 2015 (except for flows from July 2012 and May through October 2013). From April 2012 through October 2014 there were 18 non-residential connections, and from November 2014 through September 2015, there were 19 non-residential connections.

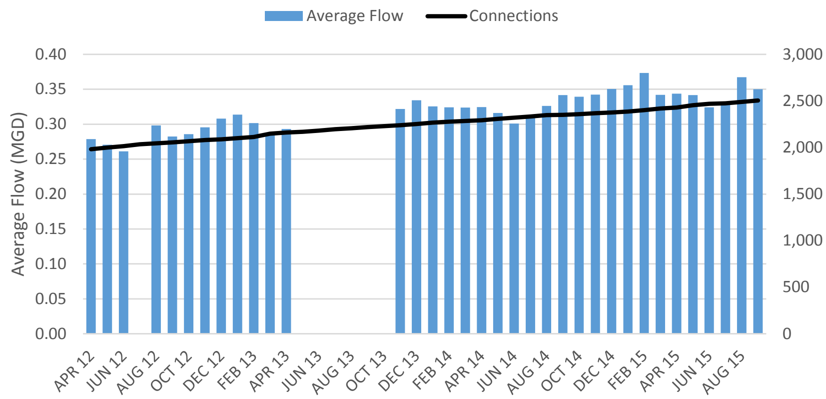


Figure 5. Average flow and number of connections by month

Because there are no data collected that differentiate between residential and non-residential flow, and because there are such a small number of non-residential connections, per capita flow is calculated assuming that all of the connections are residential. This will result in a slightly higher than actual per capita flow because most of the non-residential connections discharge more flow than a residence. ***Appendix B*** is a list of non-residential connections and the average water demand for 2014.

***Figure 6*** shows the average flow and the per capita flow for each month. Per capita flow is in gallons per capita per day (gpcd) along the right axis.

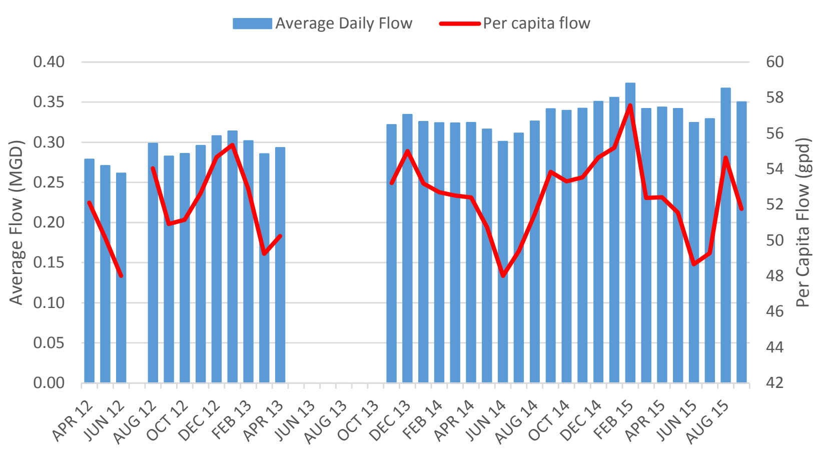


Figure 6. Average flow and per capita flow by month

The per capita flow varies from 48.0 gpcd to 57.6 gpcd with an average of 52.3 gpcd. There is no discernable trend in per capita flow (p>0.45). Assuming 2.7 ppdu, flow per EDU ranges from 129.6 gpd to 155.6 gpd with an average flow of 141.1 gpd/EDU.

The Arizona Department of Environmental Quality (ADEQ) requires an assured treatment capacity of 187.2 gpd/EDU.

Loadings and flow projections (***Sections 4*** and ***5***) are calculated for both the historical mean per capita flow of 52.3 gpcd, and the ADEQ capacity assurance requirements of 187.2 gpd/EDU or 69.3 gpcd at 2.7 persons per dwelling unit.

## 2.4. MAXIMUM MONTH AVERAGE DAY FLOW

For this study, the MMADF is represented by a 30-day moving average. The MMADF peaking factor (PF) is equal to the maximum 30-day average divided by the AADF. In ***Figure 7*** the 30-day average is shown by the brown line. The black dashed line represents the regression line of the daily data. The solid black line represents the ratio of the 30-day average to the regression line (right hand axis). The maximum ratio of the ratio of the 30-day average to the regression line is about 1.1.

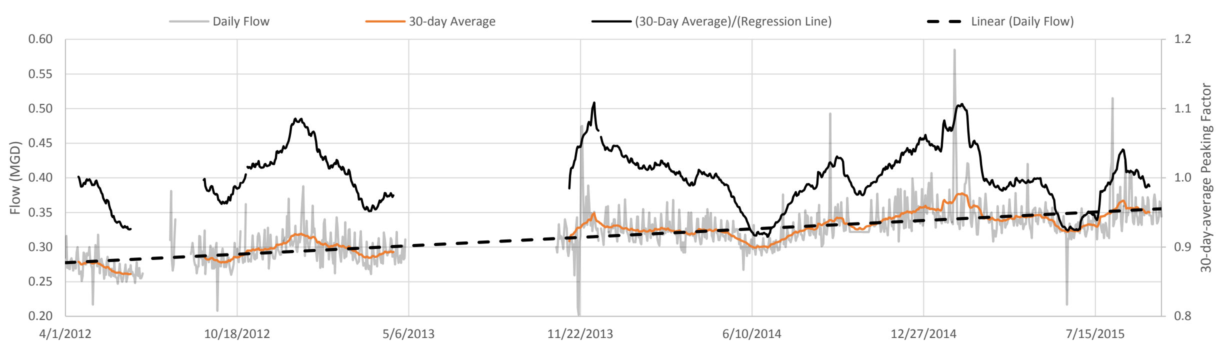



Figure 7. 30-day average peaking factor (right axis) relative to annual averages

## 2.5. PEAK DAY FLOW

The PDF is the highest one-day flow, and the PDF PF is the highest ratio of one-day flow to the AADF, where the AADF is represented by the regression line of daily flows from ***Section 2.1***.

Daily flow varies due to changes in discharge to the sewer system from customers and because of rainfall. The highest peaks are usually associated with rainfall. ***Figure 8*** shows the daily flows and rainfall from the National Oceanic and Atmospheric Administration (NOAA) database (2015). Rainfall amount is shown on the right-hand axis. The rainfall shown in ***Figure 8*** is the maximum rainfall for a given date from 1 of 5 metrological stations in the Marana area. Rain at any given station does not always equate to increased flow at the Marana WRF, as the rain may not fall on a significant portion of the sewer basin served by the Marana WRF.

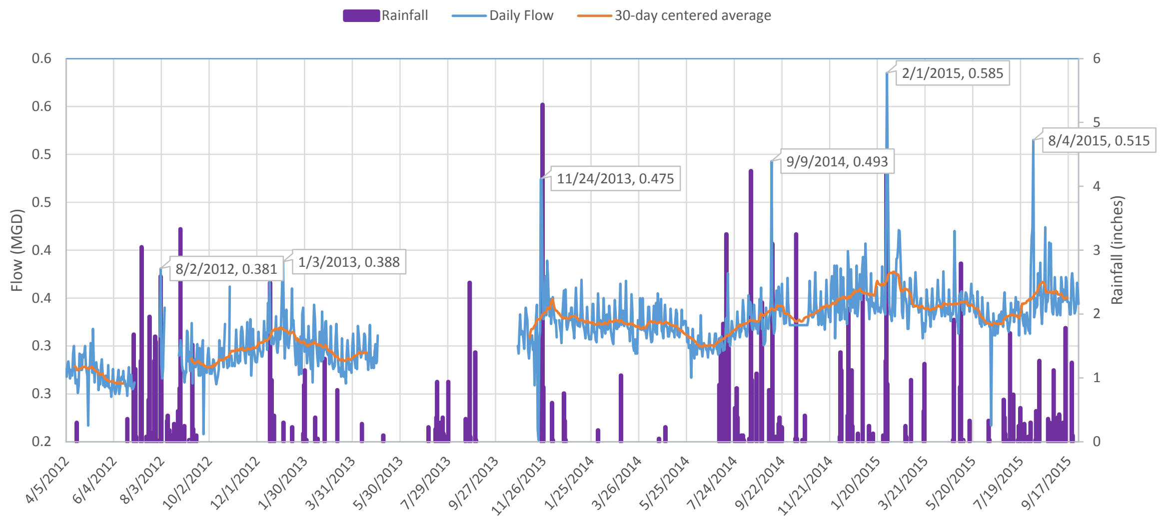



Figure 8. Daily flows and rainfall (NOAA 2015)

Some of the peak flow days in ***Figure 8*** are identified by date and amount in MGD. ***Table 1*** shows the peak days and associated rainfall, if any.

Table 1. Peak day flow events and rainfall

| Date | Flow (MGD) | Rainfall |
|---|---|---|
| 8/2/2012 | 0.381 | No rain on 8/2, but 3.9 inches in preceding days. |
| 1/3/2013 | 0.388 | No rain recorded on 1/3, but 0.3 inches on 12/31. |
| 11/24/2013 | 0.475 | 7.64 inches on preceding 3 days. |
| 9/9/2014 | 0.493 | 3.4 inches |
| 2/1/2015 | 0.585 | 6.42 inches preceding 3 days |
| 8/4/2015 | 0.515 | No rain near or on 8/4. Operators log book stated that trends showed that an object may have been stuck in flume. |

Based on the data in ***Figure 8*** and ***Table 1***, it appears that most peak-day events are associated with rainfall events, and the largest peak was associated with a particularly wet period.

***Figure 9*** shows the ratio of daily flow to the AADF, where the AADF is represented by the regression line. The maximum recorded flow is 0.585 MGD, and is associated with a PDF PF of 1.72. The maximum flow occurred on February 1, 2015, and occurred during a particularly wet period, as the records show approximately 6.42-inches rainfall over three days.

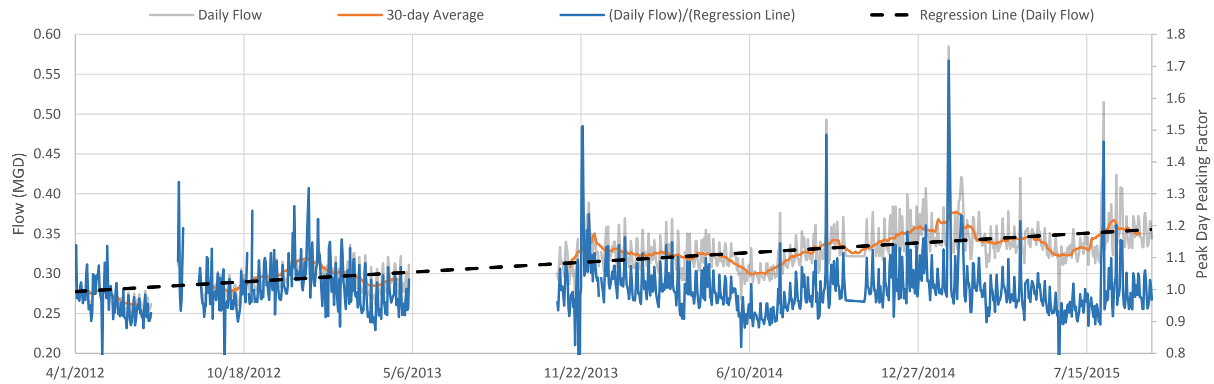



Figure 9. Peak-day flow peaking factor: ratio of daily flow to regression line through daily flow

***Figure 10*** is a frequency distribution of the ratio of daily flow to the regression line. The maximum ratio, or peaking factor, is significantly larger than the next larger peaking factors, which is a group of three days with a peaking factor value around 1.5. Over 99 percent of values are below 1.25, and over 99.9 percent of values fall below 1.52. For this study, a PDF peaking factor of 2 is used to project peak-day flows.

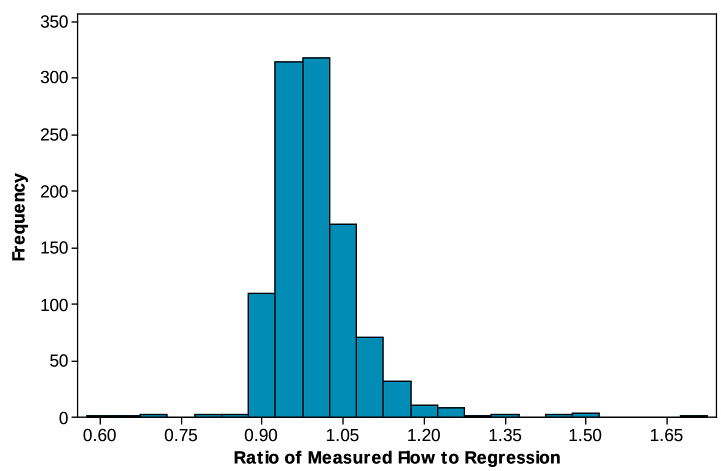



Figure 10. Frequency distribution of the ratio of daily flow to regression line flow

## 2.6. PEAK-HOUR FLOW

Peak-hour flows are the highest flows occurring during a one hour period. The PHF PF is the ratio of the PHF to the AADF. For this study, the peak-hour flow (PHF) peaking factor is based on an equation recommended by the 10-States Standards (GLUMRB 2014). The GLUMRB (2014) PHF equation is a function of population, which means that for a specific AADF, the PHF peaking factor will depend on whether you use the historical per-capita flow (52.3 gpcd) or the ADEQ Capacity Assurance flow of 187.2 gpd/EDU and 2.7 persons per dwelling unit (69.3 gpcd). ***Figure 11*** shows the GLUMRB (2014) peaking factor as a function of flow for both cases. The dashed lines in ***Figure 11*** relate flow to population for each method on the right-hand axis.

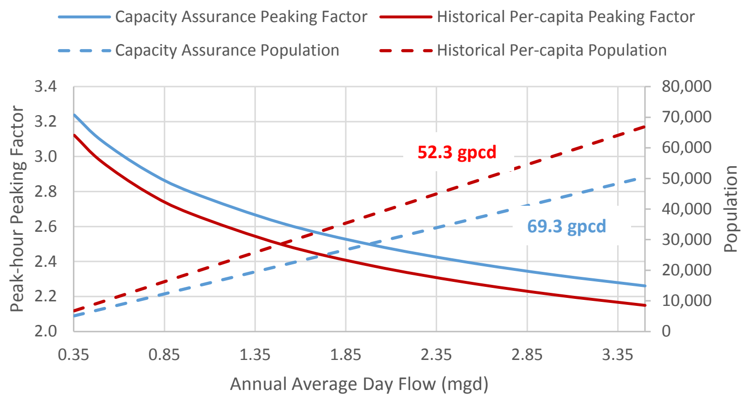



Figure 11. Peak-hour Flow peaking factor as a function of AADF

The PHF peaking factor is larger for the capacity assurance case because there is a smaller population associated with a given AADF. The GLUMRB (2014) PHF peaking factor equation is

$$PHF\ PF = \frac{18 + \sqrt{P/1{,}000}}{4 + \sqrt{P/1{,}000}} \qquad \text{Equation (1)}$$

Where $PHF\ PF$ = peak-hour peaking factor, no units
$P$ = population

***Appendix C*** contains graphs of the continuous flow measured at the headworks from October 1, 2015 through October 15, 2015, and ***Appendix D*** contains graphs of the continuous flow measured at the headworks during 9 days associated with rainfall and having high daily flows. ***Table 2*** shows the maximum flows for the 9 days associated with rainfall in ***Appendix D,*** the AADF as predicted by the regression line developed in ***Section 2.1***, and the ratio of the maximum flow to the AADF.

Table 2. Maximum flows associated with rainfall

| Date | Maximum Flow (gpm) | AADF (gpd) | AADF (gpm) | Ratio of Maximum to Average Flow |
|---|---|---|---|---|
| 8/12/2013 | 452 | 0.308 | 214 | 2.1 |
| 2/1/2015 | 668 | 0.340 | 236 | 2.8 |
| 8/8/2015 | 492 | 0.352 | 244 | 2.0 |
| 8/11/2015 | 517 | 0.352 | 245 | 2.1 |
| 8/17/2015 | 521 | 0.352 | 245 | 2.1 |
| 8/25/2015 | 656 | 0.353 | 245 | 2.7 |
| 8/26/2015 | 797 | 0.353 | 245 | 3.3 |
| 10/21/2015 | 528 | 0.356 | 248 | 2.1 |
| 10/29/2015 | 705 | 0.357 | 248 | 2.8 |

The maximum ratio of peak flow to AADF is 3.3, and occurred on August 26, 2015. The peak-hour flow is difficult to determine from the graphs in ***Appendix D***, but is slightly less than 3.3. The historical peak instantaneous flow can be compared to the PHF PF predicted by ***Equation 1***, based on the historical per capita flow (52.3 gpcd) and the per capita flow based on the Capacity Assurance requirements (69.3 gpcd).

Based on the regression line from ***Section 2.1***, the AADF for August 26, 2015, was 0.353 MGD. Assuming a per capita flow of 52.3 gpcd, the equivalent population is 6,750, and from ***Equation 1***, the PHF PF is 3.1. Assuming a per capita flow of 69.3, the equivalent population is 5,091, and from ***Equation 1***, the PHF PF is 3.2.

***Table 3*** shows the PHF PF based on ***Equation 1*** for flows from 0.5 MGD to 3.5 MGD.

Table 3. Peak-hour peaking factor versus average daily flow.

| Annual Average Day Flow (MGD) | Peak-hour Flow Peaking Factor | |
|---|---|---|
| | Assuming 52.3 gpcd (Historical) | Assuming 69.3 gpcd (Capacity Assurance) |
| 0.5 | 3.0 | 3.2 |
| 0.75 | 2.8 | 3.1 |
| 1.0 | 2.7 | 2.9 |
| 1.5 | 2.5 | 2.8 |
| 2.5 | 2.3 | 2.4 |
| 3.5 | 2.1 | 2.3 |

## 2.7. SUMMARY OF PEAKING FACTORS

***Table 4*** shows the peaking factors for the MMADF, the PDF, and the PHF for flows from 0.5 MGD to 3.5 MGD.

Table 4. Summary of peaking factors.

| AADF (MGD) | MMADF Peaking Factor | PDF Peaking Factor | PHF Peaking Factor | |
|---|---|---|---|---|
| | | | 52.3 gpcd | 69.3 gpcd |
| 0.5 | 1.1 | 2.0 | 3.0 | 3.2 |
| 0.75 | | | 2.8 | 3.1 |
| 1.0 | | | 2.7 | 2.9 |
| 1.5 | | | 2.5 | 2.8 |
| 2.5 | | | 2.3 | 2.4 |
| 3.5 | | | 2.1 | 2.3 |

***Table 5*** shows the peak flows based on the MMADF PF, the PDF PF, and the PHF PF for flows from 0.5 MGD to 3.5 MGD.

Table 5. Summary of peak flows.

| AADF (MGD) | MMADF (MGD) | PDF Peaking (MGD) | PHF (MGD) | |
|---|---|---|---|---|
| | | | 52.3 gpcd | 69.3 gpcd |
| 0.5 | 0.55 | 1.0 | 1.50 | 1.60 |
| 0.75 | 0.83 | 1.5 | 2.10 | 2.33 |
| 1.0 | 1.10 | 2.0 | 2.70 | 2.90 |
| 1.5 | 1.65 | 3.0 | 3.75 | 4.20 |
| 2.5 | 2.75 | 5.0 | 5.75 | 6.00 |
| 3.5 | 3.85 | 7.0 | 7.35 | 8.05 |

# 3. WATER QUALITY

This section contains an analysis of historical influent water quality for the Marana WRF. Unless otherwise noted, all samples were collected following screening and grit removal. The headworks screen was replaced November 26, 2013. This appears to have lowered influent Biochemical Oxygen Demand (BOD), but did not affect other water quality parameters.

The parameters analyzed include 5-day Biochemical Oxygen Demand ($BOD_5$), Total Suspended Solids (TSS), Total Kjeldahl Nitrogen (TKN), ammonia, and alkalinity. This section also includes an analysis of the ratio of ammonia to TKN and the ratio of $BOD_5$ to TKN.

Loadings, which combine flows with constituent concentrations, are presented in ***Section 5***. Loadings will be based on mean constituent concentration and on 92$^{nd}$ percentile for maximum loads.

## 3.1. 5-DAY BIOCHEMICAL OXYGEN DEMAND ($BOD_5$)

Of 109 $BOD_5$ samples available for this analysis, 101 are composite samples taken in the headworks following the screen and the grit-removal channel, and 8 composite samples taken before screening and grit removal. Of the 101 samples taken, 52 were taken before the screen was replaced on November 26, 3013, and the remaining 49 were taken after the new screen was installed.

### 3.1.1. $BOD_5$ Samples Collected Following Screening

***Table 6*** shows the results of the 101 $BOD_5$ samples collected after screening and grit removal; April 3, 2012 through November 4, 2015.

Table 6. Influent BOD5 test results from April 2012 through November 2015.

| Date | $BOD_5$ (mg/L) | Date | $BOD_5$ (mg/L) | Date | $BOD_5$ (mg/L) | Date | $BOD_5$ (mg/L) |
|---|---|---|---|---|---|---|---|
| 4/3/2012 | 177 | 12/19/2012 | 166 | 11/12/2013 | 270 | 11/11/2014 | 171 |
| 4/4/2012 | 232 | 12/20/2012 | 169 | 11/26/2013 | 252 | 12/1/2014 | 264 |
| 5/8/2012 | 246 | 1/22/2013 | 184 | 12/3/2013 | 223 | 12/16/2014 | 220 |
| 5/9/2012 | 262 | 1/29/2013 | 149 | 12/17/2013 | 280 | 12/29/2014 | 250 |
| 5/17/2012 | 180 | 2/20/2013 | 279 | 1/8/2014 | 224 | 1/13/2015 | 166 |
| 5/23/2012 | 379 | 2/21/2013 | 286 | 1/21/2014 | 234 | 1/28/2015 | 165 |
| 6/5/2012 | 234 | 3/12/2013 | 238 | 1/30/2014 | 200 | 2/11/2015 | 183 |
| 6/7/2012 | 226 | 3/13/2013 | 235 | 2/5/2014 | 201 | 2/25/2015 | 139 |
| 6/11/2012 | 186 | 4/17/2013 | 487 | 2/19/2014 | 211 | 3/11/2015 | 196 |
| 6/18/2012 | 287 | 4/18/2013 | 502 | 3/5/2014 | 193 | 3/25/2015 | 158 |
| 6/19/2012 | 124 | 4/23/2013 | 322 | 3/20/2014 | 232 | 4/8/2015 | 177 |
| 6/20/2012 | 258 | 5/23/2013 | 430 | 4/30/2014 | 213 | 4/22/2015 | 232 |
| 6/21/2012 | 274 | 6/18/2013 | 210 | 5/15/2014 | 214 | 5/6/2015 | 218 |
| 6/28/2012 | 259 | 6/25/2013 | 254 | 5/29/2014 | 241 | 5/20/2015 | 229 |
| 7/10/2012 | 326 | 7/9/2013 | 239 | 6/11/2014 | 194 | 6/3/2015 | 220 |
| 7/11/2012 | 220 | 7/17/2013 | 119 | 6/25/2014 | 205 | 6/17/2015 | 228 |
| 7/26/2012 | 266 | 7/23/2013 | 204 | 7/9/2014 | 186 | 7/15/2015 | 212 |
| 8/9/2012 | 239 | 8/7/2013 | 210 | 7/24/2014 | 186 | 7/29/2015 | 232 |

| Date | $BOD_5$ (mg/L) | Date | $BOD_5$ (mg/L) | Date | $BOD_5$ (mg/L) | Date | $BOD_5$ (mg/L) |
|---|---|---|---|---|---|---|---|
| 9/6/2012 | 486 | 8/20/2013 | 224 | 8/6/2014 | 253 | 8/14/2015 | 241 |
| 9/26/2012 | 301 | 9/5/2013 | 187 | 8/20/2014 | 209 | 8/28/2015 | 193 |
| 10/4/2012 | 141 | 9/17/2013 | 214 | 9/4/2014 | 210 | 9/9/2015 | 197 |
| 10/10/2012 | 108 | 9/25/2013 | 172 | 9/17/2014 | 156 | 9/23/2015 | 232 |
| 10/11/2012 | 148 | 10/2/2013 | 174 | 10/2/2014 | 213 | 10/7/2015 | 213 |
| 10/17/2012 | 298 | 10/15/2013 | 198 | 10/15/2014 | 175 | 10/21/2015 | 272 |
| 11/14/2012 | 244 | 10/29/2013 | 214 | 10/29/2014 | 170 | 11/4/2015 | 222 |
| 11/19/2012 | 298 | | | | | | |

***Figure 12*** shows the $BOD_5$ concentration over time. On Tuesday, November 26, 2013, a ladder screen with a 1/8-inch spacing replaced an auger screen with a 1/8-inch mesh; the washer compactor was also replaced at this time. The red vertical line represents the installation date of the new screen. The horizontal yellow and green lines represent the average $BOD_5$ concentration before and after the screen was replaced.

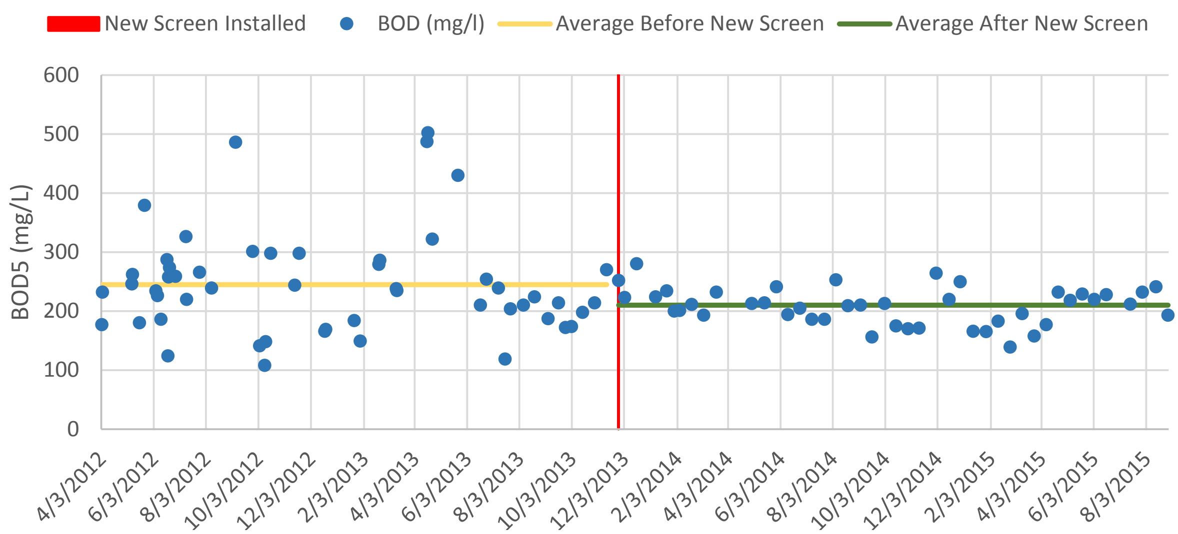



Figure 12. $BOD_5$ concentration with average before and after screen replacement

The average $BOD_5$ concentration for all samples is 228 mg/L. According to operators, $BOD_5$ concentrations have been lower since the new screen was installed. The average $BOD_5$ concentration before the new screen was installed is 245 mg/L, and the average $BOD_5$ concentration after the screen was installed is 210 mg/L; a reduction of about 35 mg/L. Tests for trends showed no significant trends in the data with the exception of the drop in averages related to the screen replacement ($p<0.04$). $BOD_5$ loading presented in ***Section 5*** is based on all 101 $BOD_5$ measurements were used to calculate loadings.

***Figure 13*** is the frequency distribution of the 101 samples collected before screen replacement. The figure shows most of the values around the 200 to 300 mg/L range, but outliers near the 400 to 500 mg/L range.

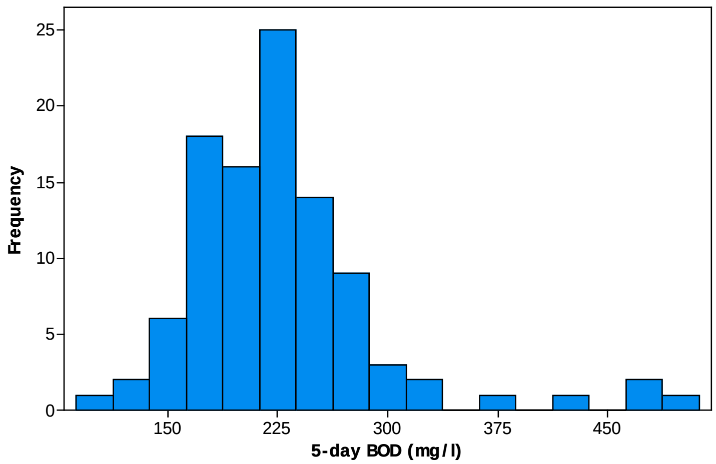



Figure 13. Frequency Distribution of all BOD5 Data (sampled following the screen and grit channel)

***Table 7*** shows statistics for the 101 $BOD_5$ samples collected between April 2012 and November 2015. These samples were collected after the screening and grit removal. The average $BOD_5$ concentration of the 101 samples is 228 mg/L, and the 92nd percentile is 375 mg/L.

Table 7. $BOD_5$ sample statistics.

| Statistic | Units | Value |
|---|---|---|
| Number of samples | | 101 |
| Mean | mg/L | 228 |
| Standard Deviation | mg/L | 68.6 |
| Minimum | mg/L | 108 |
| Median | mg/L | 220 |
| 92nd Percentile | mg/L | 298 |
| Maximum | mg/L | 502 |

### 3.1.2. $BOD_5$ Samples Collected Following Screening

***Table 8*** shows the $BOD_5$ concentrations for 8 composite samples collected before entering the headworks screen. The values ranged from 170 to 569 mg/L, with an average concentration of 331 mg/L.

Table 8. $BOD_5$ concentration for samples collected before headworks screen

| Date | $BOD_5$ (mg/L) | Date | $BOD_5$ (mg/L) |
|---|---|---|---|
| 6/16/2015 | 170 | 9/2/2015 | 310 |
| 7/1/2015 | 218 | 9/22/2015 | 460 |
| 7/14/2015 | 417 | 10/6/2015 | 240 |
| 8/5/2015 | 262 | 10/20/2015 | 569 |

## 3.2. TOTAL SUSPENDED SOLIDS

***Figure 14*** shows results of 94 samples of influent Total Suspended Solids (TSS) concentration over time. All of the samples are composite samples taken following the headworks screening and grit removal.

Statistical tests showed no significant trend ($p>0.8$), and no significant relationship with flow ($p>0.2$), or with the installation of the new screen ($p=0.18$).

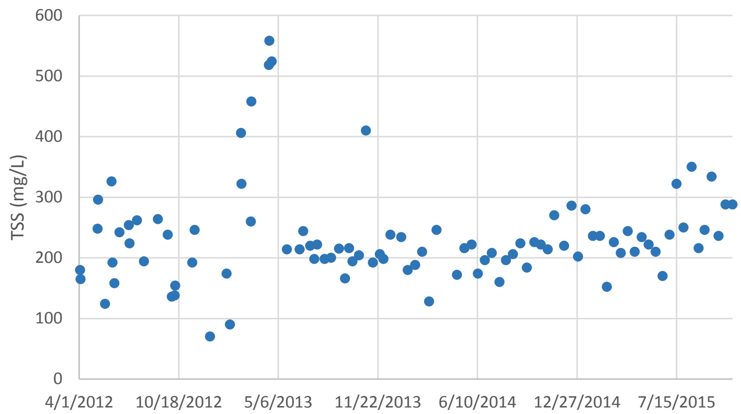



Figure 14. Total Suspended Solids Concentration

***Table 9*** shows statistics for the 94 TSS samples collected between April 2012 and November 2015. These samples were collected after the screening and grit removal. Values ranged from 70 to 558 mg/L. The average TSS concentration of the 94 samples is 233 mg/L, and the 92nd percentile is 330 mg/L.

Table 9. TSS sample statistics.

| Statistic | Units | Value |
|---|---|---|
| Number of samples | | 94 |
| Mean | mg/L | 233 |
| Standard Deviation | mg/L | 82 |
| Minimum | mg/L | 70 |
| Median | mg/L | 218 |
| $92^{nd}$ Percentile | mg/L | 330 |
| Maximum | mg/L | 558 |

## 3.3. NITROGEN

This section covers Total Kjeldahl Nitrogen (TKN), ammonia ($NH_3$), and nitrates and nitrites ($NO_x$). TKN is a combination of $NH_3$ and organic nitrogen, but does not include nitrates and nitrites. Some samples were taken before the headworks screen and some were taken following screening and grit removal.

The ratio of $BOD_5$ to TKN and the percentage of $NH_3$ in TKN are also covered, as these can affect nitrogen removal treatment design and operating costs.

### 3.3.1. Total Kjeldahl Nitrogen

***Figure 15*** shows the results of 12 composite samples taken after screening and grit removal, and 11 composite samples taken prior to screening and grit removal.

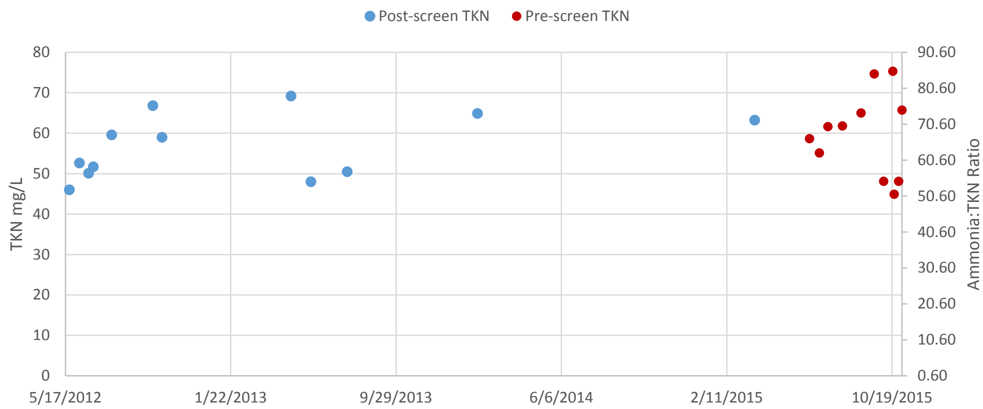



Figure 15. Total Kjeldahl Nitrogen from samples before and after screening

***Table 10*** shows the raw TKN data and whether it was taken before or after screening and grit removal.

Table 10. Total Kjeldahl Nitrogen from samples before and after screening

| Post Screening and Grit Removal | | Post Screening and Grit Removal | |
|---|---|---|---|
| **Date** | **TKN (mg/L)** | **Date** | **TKN (mg/L)** |
| 5/23/2012 | 46.0 | 6/16/2015 | 66.6 |
| 6/7/2012 | 52.6 | 7/1/2015 | 62.6 |
| 6/21/2012 | 50.1 | 7/14/2015 | 69.9 |
| 6/28/2012 | 51.7 | 8/5/2015 | 70.1 |
| 7/26/2012 | 59.6 | 9/2/2015 | 73.7 |
| 9/26/2012 | 66.8 | 9/22/2015 | 84.6 |
| 10/10/2012 | 59.0 | 10/6/2015 | 54.7 |
| 4/23/2013 | 69.2 | 10/20/2015 | 85.3 |
| 5/23/2013 | 48.0 | 10/22/2015 | 51.1 |
| 7/17/2013 | 50.5 | 10/29/2015 | 54.7 |
| 1/30/2014 | 64.9 | 11/3/2015 | 74.5 |
| 3/25/2015 | 63.2 | | |

***Table 11*** shows statistics for the 23 TKN samples collected between May 2012 and November 2015. For those samples collected after screening and grit removal, values ranged from 46 to 69 mg/L. The average TKN concentration of the 12 samples is 57 mg/L, and the 92$^{nd}$ percentile is 67 mg/L. For the samples collected prior to entering the screen, values ranged from 51 to 85 mg/L. The average concentration is 42 mg/L and the 92$^{nd}$ percentile value is 85 mg/L.

Table 11. TKN sample statistics

| **Statistic** | **Units** | **Value** | |
|---|---|---|---|
| | | **Post-screen** | **Pre-screen** |
| Number of samples | | 12 | 11 |
| Mean | mg/L | 57 | 42 |
| Standard Deviation | mg/L | 8 | 7 |
| Minimum | mg/L | 46 | 51 |
| Median | mg/L | 56 | 42 |
| 92$^{nd}$ Percentile | mg/L | 67 | 85 |
| Maximum | mg/L | 69 | 85 |

There is a statistically significant trend in the post-screening TKN data ($p<0.01$), but this may be an anomaly based on the distribution of the samples over time. For loading projections, it is assumed that the average concentration does not change over time.

***Figure 16*** shows the 20 $NH_3$ samples that were collected following screening and grit removal between May 2012 and March 2012. Fourteen of the 20 samples are in 2012. ***Figure 16*** also shows the ratio of $NH_3$-N to TKN (right-hand axis).

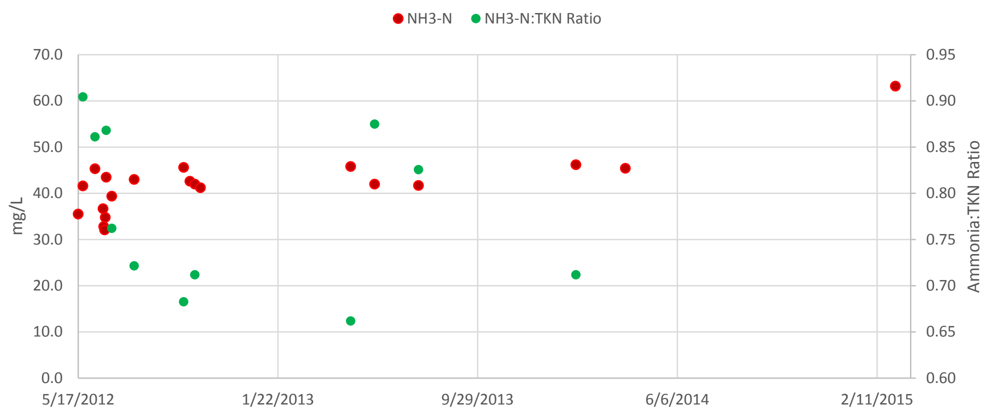



Figure 16. $NH_3$-N and $NH_3$-N:TKN for post-screening samples

***Table 12*** shows the 20 $NH_3$ samples that were taken post-screening, and the $NH_3$-N:TKN ratio for 11 samples.

Table 12. Ammonia and ammonia/TKN ratio for post-screening samples.

| Date | $NH_3$-N (mg/L) | TKN (mg/L) | Ammonia/TKN Fraction |
|---|---|---|---|
| 5/17/2012 | 35.5 | | |
| 5/23/2012 | 41.6 | 46.0 | 0.90 |
| 6/7/2012 | 45.3 | 52.6 | 0.86 |
| 6/17/2012 | 36.7 | | |
| 6/18/2012 | 32.8 | | |
| 6/19/2012 | 32.1 | | |
| 6/20/2012 | 34.8 | | |
| 6/21/2012 | 43.5 | 50.1 | 0.87 |
| 6/28/2012 | 39.4 | 51.7 | 0.76 |
| 7/26/2012 | 43.0 | 59.6 | 0.72 |
| 9/26/2012 | 45.6 | 66.8 | 0.68 |
| 10/4/2012 | 42.6 | | |
| 10/10/2012 | 42.0 | 59.0 | 0.71 |
| 10/17/2012 | 41.2 | | |
| 4/23/2013 | 45.8 | 69.2 | 0.66 |
| 5/23/2013 | 42.0 | 48.0 | 0.88 |
| 7/17/2013 | 41.7 | 50.5 | 0.83 |
| 1/30/2014 | 46.2 | 64.9 | 0.71 |
| 4/2/2014 | 45.4 | | |
| 3/6/2015 | 63.2 | 46.0 | |

***Table 13*** shows the sample statistics for the 20 ammonia samples, and the $NH_3$-N:TKN fractions for the 11 ammonia samples that were also tested for TKN. $NH_3$-N values ranged from 32 to 63 mg/L with an average of 42 mg/L. The $NH_3$-N:TKN ratio ranged from 0.66 to 0.90 with an average of 0.78.

Table 13. $NH_3$ and $NH_3$-N:TKN Ratio Statistics.

| Statistic | $NH_3$-N (mg/L) | $NH_3$-N:TKN Ratio |
|---|---|---|
| Number of samples | 20 | 11 |
| Mean | 42 | 0.78 |
| Standard Deviation | 7 | 0.09 |
| Minimum | 32 | 0.66 |
| Median | 42 | 0.76 |
| $92^{nd}$ Percentile | 46 | |
| Maximum | 63 | 0.90 |

There were 2 samples taken before the headworks screen: On October 20, 2012, the $NH_3$-N concentration was 48.6 mg/L and the $NH_3$-N:TKN ratio was 0.57; and on November 3, 2015, the $NH_3$-N concentration was 41.6 mg/L and the $NH_3$-N:TKN ratio was 0.56.

### 3.3.2. Nitrite and Nitrate

Two samples were tested for nitrite and nitrate. Both samples were taken before the headworks screen. The samples were taken on October 22 and 29 of 2015, and both samples were below the Practical Quantification Limit (PQL) of 0.1 mg/L for nitrite as N and 0.2 mg/L for nitrite and nitrate as N.

### 3.3.3. $BOD_5$:TKN Ratio

Twenty samples included both $BOD_5$ and TKN; 16 of the samples were collected following screening and grit removal, and 4 of the samples were collected prior to screening and grit removal. ***Table 14*** shows the ratio of $BOD_5$:TKN for the 20 samples.

Table 14. Pre- and post-screen $BOD_5$:TKN ratio.

| Date | Location | $BOD_5$ (mg/L) | TKN (mg/L) | BOD5:TKN Ratio |
|---|---|---|---|---|
| 5/23/2012 | post-screen | 379 | 46.0 | 8.2 |
| 6/7/2012 | post-screen | 226 | 52.6 | 4.3 |
| 6/21/2012 | post-screen | 274 | 50.1 | 5.5 |
| 6/28/2012 | post-screen | 259 | 51.7 | 5.0 |
| 7/26/2012 | post-screen | 266 | 59.6 | 4.5 |
| 9/26/2012 | post-screen | 301 | 66.8 | 4.5 |
| 10/10/2012 | post-screen | 108 | 59.0 | 1.8 |
| 4/23/2013 | post-screen | 322 | 69.2 | 4.7 |
| 5/23/2013 | post-screen | 430 | 48.0 | 9.0 |
| 7/17/2013 | post-screen | 119 | 50.5 | 2.4 |
| 1/30/2014 | post-screen | 200 | 64.9 | 3.1 |
| 3/25/2015 | post-screen | 158 | 63.2 | 2.5 |
| 6/16/2015 | post-screen | 170 | 66.6 | 2.6 |
| 7/1/2015 | post-screen | 218 | 62.6 | 3.5 |

| Date | Location | $BOD_5$ (mg/L) | TKN (mg/L) | BOD5:TKN Ratio |
|---|---|---|---|---|
| 7/14/2015 | post-screen | 417 | 69.9 | 6.0 |
| 8/5/2015 | post-screen | 262 | 70.1 | 3.7 |
| 9/2/2015 | pre-screen | 310 | 73.7 | 4.2 |
| 9/22/2015 | pre-screen | 460 | 84.6 | 5.4 |
| 10/6/2015 | pre-screen | 240 | 54.7 | 4.4 |
| 10/20/2015 | pre-screen | 569 | 85.3 | 6.7 |

***Table 15*** shows the $BOD_5$:TKN ratio statistics for pre-screen, post-screen, and all 20 samples. The average value ranged from 4.4 for the 6.2 post-screen samples to 5.2 for the 4 pre-screen samples, with an overall average of 4.6 for the 20 pre- and post-screen samples.

Table 15. Pre- and post-screen $BOD_5$:TKN ratio statistics.

| Statistic | $BOD_5$:TKN Ratio | | |
|---|---|---|---|
| | Post-screen | Pre-screen | Pre- and Post-screen |
| Number of samples | 16 | 4 | 20 |
| Mean | 4.4 | 5.2 | 4.6 |
| Standard Deviation | 2.0 | 1.1 | 1.9 |
| Minimum | 1.8 | 4.2 | 1.8 |
| Median | 4.4 | 4.9 | 4.4 |
| Maximum | 9.0 | 6.7 | 9.0 |

## 3.4. ALKALINITY

Twenty samples were tested for influent alkalinity from May 2012 through October 2014. Twelve samples were collected following screening and grit removal, and 8 samples were collected before screening and grit removal. ***Table 16*** shows sample results for all 20 samples.

Table 16. Pre- and post-screen alkalinity

| Date | Collection Location | Alkalinity (mg/L as $CaCO_3$) | Date | Collection Location | Alkalinity (mg/L as $CaCO_3$) |
|---|---|---|---|---|---|
| 5/17/2012 | Pre-screen | 312 | 4/23/2013 | Pre-screen | 364 |
| 5/23/2012 | Pre-screen | 326 | 5/23/2013 | Pre-screen | 349 |
| 6/7/2012 | Pre-screen | 338 | 6/16/2015 | Post-screen | 366 |
| 6/21/2012 | Pre-screen | 334 | 7/1/2015 | Post-screen | 354 |
| 6/28/2012 | Pre-screen | 330 | 7/14/2015 | Post-screen | 354 |
| 7/26/2012 | Pre-screen | 337 | 8/5/2015 | Post-screen | 359 |
| 9/26/2012 | Pre-screen | 336 | 9/2/2015 | Post-screen | 357 |
| 10/4/2012 | Pre-screen | 361 | 9/22/2015 | Post-screen | 531 |
| 10/10/2012 | Pre-screen | 346 | 10/6/2015 | Post-screen | 306 |
| 10/17/2012 | Pre-screen | 336 | 10/20/2015 | Post-screen | 361 |

***Table 17*** shows the statistics for the pre- and post- screen alkalinity. Post-screen samples ranged from 312 to 364 mg/L as $CaCO_3$, with a mean of 339 mg/L as $CaCO_3$. Pre-screen samples ranged from 306 to 531 mg/L as $CaCO_3$, with a mean of 374 mg/L as $CaCO_3$.

Table 17. Pre- and post-screen alkalinity statistics

| Statistic | Units | Alkalinity | |
|---|---|---|---|
| | | Post-screen | Pre-screen |
| Number of samples | | 12 | 8 |
| Mean | mg/L as $CaCO_3$ | 339 | 374 |
| Standard Deviation | mg/L as $CaCO_3$ | 14 | 66 |
| Minimum | mg/L as $CaCO_3$ | 312 | 306 |
| Median | mg/L as $CaCO_3$ | 337 | 358 |
| Maximum | mg/L as $CaCO_3$ | 364 | 531 |

### 3.5. Summary Statistics for Key Constituents

***Table 18*** is a summary of the mean and $92^{nd}$ percentile for $BOD_5$, TSS, TKN, and $NH_3$-N. The statistics in ***Table 18*** will be used to calculate loadings in the ***Section 4***. All of the statistics are based on samples taken following screening and grit removal.

Table 18. Mean and $92^{nd}$ percentile concentrations for key constituents.

| Constituent | Mean (mg/L) | $92^{nd}$ Percentile (mg/L) | Ratio of $92^{nd}$ Percentile: Mean |
|---|---|---|---|
| $BOD_5$ | 228 | 298 | 1.3 |
| TSS | 233 | 330 | 1.4 |
| TKN | 57 | 67 | 1.2 |
| $NH_3$-N | 42 | 46 | 1.1 |

## 4. CONSTITUENT LOADING

Loadings are calculated for average and maximum conditions for five pollutants: $BOD_5$, TSS, TKN, and $NH_3$-N. The average loading (AADL) is based on the average flow and the average constituent concentration. The maximum month average day loading (MMADL) is based on a peaking factor applied to the AADL, where the MMADL PF is based on the ratio of the $92^{nd}$ percentile concentration to the average concentration (see ***Table 18*** above) and adjusted based on engineering judgement.

***Table 19*** shows the ratio of the $92^{nd}$ percentile pollutant concentration to the average concentration, and the MMADL PF chosen for each constituent.

Table 19. Maximum-month average-day loading peaking factor (MMADL PF)

| Constituent | Ratio of $92^{nd}$ Percentile:Mean | MMADL |
|---|---|---|
| $BOD_5$ | 1.3 | 1.3 |
| TSS | 1.4 | 1.4 |
| TKN | 1.2 | 1.3 |
| $NH_3$-N | 1.1 | 1.3 |

***Table 20*** shows the AADL and MMADL for flows ranging from 0.5 MGD through 3.5 MGD.

Table 20. AADL and MMADL constituent loadings

| AADF (MGD) | $BOD_5$ | | TSS | | TKN | | $NH_3$-N | |
|---|---|---|---|---|---|---|---|---|
| | AADL | MMADL | AADL | MMADL | AADL | MMADL | AADL | MMADL |
| | Pounds per Day | | | | | | | |
| 0.5 | 951 | 1,236 | 972 | 1,360 | 238 | 309 | 175 | 228 |
| 0.75 | 1,426 | 1,854 | 1,457 | 2,040 | 357 | 463 | 263 | 342 |
| 1.0 | 1,902 | 2,472 | 1,943 | 2,721 | 475 | 618 | 350 | 455 |
| 1.5 | 2,852 | 3,708 | 2,915 | 4,081 | 713 | 927 | 525 | 683 |
| 2.5 | 4,754 | 6,180 | 4,858 | 6,801 | 1,188 | 1,545 | 876 | 1,138 |
| 3.5 | 6,655 | 8,652 | 6,801 | 9,522 | 1,664 | 2,163 | 1,226 | 1,594 |

# 5. PROJECTIONS

Flow projections were estimated for the years 2020 (5 years), 2025 (10 years), and 2035 (20 years). In addition, flow projections were estimated by basin for buildout.

## 5.1. 20-YEAR FLOW PROJECTIONS

Flow projections for the next 20 years, including years 2020, 2025, and 2035, were based on growth projections provided by the Town of Marana Planning Department. ***Table 21*** shows the projected number of dwelling units and non-residential area for developments that are expected to be served by the Marana WRF. The remaining columns show the percent of dwelling units and the percent of non-residential acres completed during three periods: 2016-2020, 2021-2025, and 2026-2035. Non-residential refers to facilities other than residential, such as schools and commercial development. ***Exhibits 1*** through ***3***, show the location of the developments that will contribute to increased flow at the Marana WRF.

Table 21. Growth projections by development

| Development | Buildout | | Percent of Buildout During Period | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Dwelling Units | Non-Residential Acres | Dwelling Units | | | Non-Residential Acres | | |
| | | | 2016-2020 | 2021-2025 | 2026-2035 | 2016-2020 | 2021-2025 | 2026-2035 |
| Arboles Viejo | 1,857 | | | | 25% | | | |
| Barrios de Marana | 315 | 20 | | 20% | 50% | | 20% | 50% |
| Cypress Gardens | 165 | | 75% | 25% | | | | |
| Fianchetto Farms | 114 | | 20% | 65% | 10% | | | |
| Gladden Farms | 1,850 | 36 | 25% | 10% | | | 50% | 50% |
| Gladden Farms II | 2,111 | 142 | | 10% | 60% | | | 50% |
| Mandarina | 2,500 | 215 | | 10% | 10% | | 10% | 10% |
| Marana Main Street | | 28 | | | | 15% | 35% | 25% |
| Marana Mercantile | | 39 | | | | | 20% | 80% |
| Marana Towne Center | 1,840 | 259 | | 5% | 15% | | 5% | 15% |
| Payson Farms | 367 | | | 10% | 90% | | | 90% |
| Rancho Marana Town Center | | 99 | | | | 5% | 5% | 25% |
| Saguaro Bloom | 2,250 | | 25% | 25% | 25% | | | |
| San Lucas | 784 | 24 | 34% | | | | | 25% |
| Sanders Grove | 2,250 | 19 | | 5% | 25% | | | |
| Shops At Tangerine | | 281 | | | | | 5% | 45% |
| Tangerine Business Park | | | | | | 30% | | 20% |
| Tangerine Commerce Park | 0 | 112 | | | | 20% | 20% | 20% |

| Development | Buildout | | Percent of Buildout During Period | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Dwelling Units | Non-Residential Acres | Dwelling Units | | | Non-Residential Acres | | |
| | | | 2016-2020 | 2021-2025 | 2026-2035 | 2016-2020 | 2021-2025 | 2026-2035 |
| The Villages Of Tortolita | 5,850 | 505 | | | 5% | | | |
| Uptown At Marana | 930 | 121 | | | 20% | | 5% | 5% |
| Vanderbilt Farms | 2,300 | 124 | 10% | 10% | 20% | | | |
| Whitney Farms | 12 | | 50% | 50% | | | | |

Equivalent dwelling units (EDU) are used to simplify calculations. It is assumed that the flow from each acre of non-residential development is equivalent to the flow from four dwelling units; therefore, one acre of non-residential development equals four EDUs. ***Table 22*** shows the increase in EDUs during each period for each development.

Table 22. Increase in equivalent dwelling units by development

| Development | Increase in Equivalent Dwelling Units | | |
|---|---|---|---|
| | 2016-2020 | 2021-2025 | 2026-2035 |
| Arboles Viejo | | | 464 |
| Barrios de Marana | | 79 | 198 |
| Cypress Gardens | 124 | 41 | |
| Fianchetto Farms | 23 | 74 | 11 |
| Gladden Farms | 463 | 257 | 72 |
| Gladden Farms II | | 211 | 1,551 |
| Mandarina | | 336 | 336 |
| Marana Main Street | 17 | 39 | 28 |
| Marana Mercantile | | 31 | 125 |
| Marana Towne Center | | 144 | 431 |
| Payson Farms | | 37 | 330 |
| Rancho Marana Town Center | 20 | 20 | 99 |
| Saguaro Bloom | 563 | 563 | 563 |
| San Lucas | 267 | | 24 |
| Sanders Grove | | 113 | 563 |
| Shops At Tangerine | | 56 | 506 |
| Tangerine Business Park | | | |
| Tangerine Commerce Park | 90 | 90 | 90 |
| The Villages Of Tortolita | | | 293 |
| Uptown At Marana | | 71 | 210 |
| Vanderbilt Farms | 230 | 230 | 460 |
| Whitney Farms | 6 | 6 | |
| **Total** | **1,803** | **2,398** | **6,354** |

The projected AADF is based on the following assumptions.

- Existing flow is 0.355 MGD,
- On average, there are 2.7 persons per dwelling unit (2010 U.S. Census),
- In the period 2016 through 2020, approximately 4,360 gpd of Rillito flows will be treated at the Marana WRF, and
- Flows will be projected for two cases:
    - Average per capita flow is 52.3 gpcd based on historical data, and
    - Average EDU flow is 187.2 based on capacity assurance requirements.

***Table 23*** shows the projected AADF for both per capita flow assumptions.

Table 23. Projected AADF from present to 2035

| Year | Projected AADF (MGD) | |
|---|---|---|
| | Assuming 52.3 gpcd | Assuming 187.2 gpd/EDU |
| Existing | 0.355 | 0.355 |
| 2020 | 0.61 | 0.70 |
| 2025 | 0.95 | 1.15 |
| 2035 | 1.78 | 2.25 |

## 5.2. BUILDOUT FLOW PROJECTIONS

Buildout flow projections are based on the combination of data from two sources: 1) the Town of Marana 2010 General Plan Land Use Planning Map, and 2) the buildout number of residences and non-residential acres of the development areas that are in the planning process. ***Exhibit 4*** shows the Designated Management Area (DMA), which is the Town of Marana's wastewater service area, and all of the known development areas within the DMA. Saguaro Bloom is not within the DMA boundary at this time, but will be when Saguaro Bloom is connected the Marana WRF.

***Exhibit 4*** also shows the sewer basins used for this analysis. The sewer basins are based on physical barriers such as the CAP Canal, Interstate 10 and the railroad, the Santa Cruz River, and the proposed Barnett Channel, which forms the boundary between Basin 5 and Basin 10; governmental barriers – Basin 12 is outside of the Marana Town Limits; and topography.

The buildout analysis includes all of the developments from ***Table 21*** (above) plus several others that have no predicted growth over the next 20 years or will not be connected the Marana WRF for at least 20 years. ***Table 24*** shows all of the developments that are used in the buildout flow calculation, the number of EDUs associated with buildout, the sewer basin or basins that contains each development, and the number of development EDUs associated with each sewer basin. Most of the developments are within a specific sewer basin, but 3 of the developments are split between two sewer basins (***Exhibit 4***). Rancho Marana is in Basins 7 and 11, Tucson Commerce Park is in Basins 10 and 14, and The Villages at Tortolita is in Basins 3 and 6.

Table 24. Buildout Development Areas and Sewer Basins

| Development | Dwelling Units | Non-residential Acres | EDUs | Sewer Basin | EDUs by Basin |
|---|---|---|---|---|---|
| Anway Farms | | 70 | 280 | 5 | 280 |
| Arboles Viejo | 1,857 | | 1,857 | 12 | 1,857 |
| Barrios de Marana | 315 | 20 | 395 | 5 | 395 |
| Cypress Gardens | 165 | | 165 | 5 | 165 |
| Fianchetto Farms | 114 | | 114 | 10 | 114 |
| Gladden Farms | 1,850 | 36 | 1,994 | 10 | 1,994 |
| Gladden Farms II | 2,111 | 142 | 2,679 | 10 | 2,679 |
| Honea Heights III | 150 | | 150 | 10 | 150 |
| Mandarina | 2,500 | 215 | 3,360 | 11 | 3,360 |
| Marana Gardens | 44 | | 44 | 5 | 44 |

| Development | Dwelling Units | Non-residential Acres | EDUs | Sewer Basin | EDUs by Basin |
|---|---|---|---|---|---|
| Marana Main Street | | 28 | 112 | 5 | 112 |
| Marana Mercantile | | 39 | 156 | 5 | 156 |
| Marana Towne Center | 1,840 | 259 | 2,876 | 5 | 2,876 |
| Payson Farms | 367 | | 367 | 10 | 367 |
| Rancho Marana | 507 | 78 | 819 | 11 | 217 |
| | | | | 7 | 602 |
| Rancho Marana Town Center | | 99 | 396 | 5 | 396 |
| Saguaro Bloom | 2,250 | | 2,250 | 18 | 2,250 |
| San Lucas | 784 | 24 | 880 | 5 | 880 |
| Sanders Grove | 2,250 | 19 | 2,326 | 5 | 2,326 |
| Shops At Tangerine | | 281 | 1,124 | 10 | 1,124 |
| Tangerine Business Park | | 45 | 180 | 15 | 180 |
| Tangerine Commerce Park | | 112 | 448 | 14 | 222 |
| | | | | 10 | 226 |
| The Villages of Tortolita | 5,850 | 505 | 6,004 | 3 | 3,439 |
| | | | | 6 | 2,564 |
| Uptown At Marana | 930 | 121 | 1,414 | 5 | 1,414 |
| Vanderbilt Farms | 2,300 | 124 | 2,796 | 10 | 2,796 |
| Whitney Farms | 12 | | 12 | 10 | 12 |
| | | | | **Total** | **33,197** |

***Exhibit 4*** also shows the Town of Marana's 2010 General Plan, and, in Basins 4 and 8, the area designated for no expansion of urban services. Except for the Arboles Viejo development, Basin 12 does not have land use designation. This is because the area is outside the Town of Marana's planning area. Currently, all of the area within Basin 12, but outside Arboles Viejo, is developed and served by onsite septic systems. Therefore, for this analysis, it is assumed that Arboles Viejo is the only growth in Basin 12.

***Table 25*** shows the number of EDUs assigned to each acre for each type general plan category. It was assumed that the average density in Rural Density Residential would be about 10 acres per residence, or 0.1 residences per acre (RAC), but only 10 percent of those would be able to connect to the public sewer, therefore, the resulting impact on wastewater is 0.01 EDU/acre. The *Industrial2* designation was added for this analysis to represent the sand and gravel operation which has low sewage per acre.

Table 25. EDUs per served in general plan areas

| General Plan Category | Built EDU/acre | Percent Served | Served EDU/acre | Total Area (acres) | Total EDUs |
|---|---|---|---|---|---|
| Airport | 0.25 | 100% | 0.25 | 3,335 | 834 |
| Commercial | 4 | 100% | 4 | 2,281 | 9,124 |
| Industrial | 4 | 100% | 4 | 11,637 | 46,549 |
| Industrial2 | 0.25 | 100% | 0.4 | 1,346 | 323 |
| Low Density Residential | 1.5 | 100% | 1.5 | 16,097 | 24,145 |
| Medium Density Residential | 4 | 100% | 4 | 5,293 | 21,167 |
| Mixed Rural | 0.5 | 100% | 0.5 | 207 | 104 |
| Public\Institutional | 4 | 100% | 4 | 262 | 1,046 |
| Rural Density Residential | 0.1 | 10% | 0.01 | 9,846 | 98 |

| General Plan Category | Built EDU/acre | Percent Served | Served EDU/acre | Total Area (acres) | Total EDUs |
|---|---|---|---|---|---|
| No Extension of Urban Services | 0.1 | 0% | 0 | 10,436 | 0 |
| Specific Plans | 4 | 100% | 4 | 695 | 5,556 |
| | | | **Totals** | **61,435** | **108,946** |

***Table 26*** shows the development buildout, general plan buildout, total buildout for each basin. ***Table 26*** also shows the buildout flows by basin for two conditions: 1) assuming the per capita flow equals the historical average of 52.3 gpcd, assuming the flow per EDU equals the capacity assurance requirement of 187.2 gpd/EDU.

Table 26. Buildout flow projections by sewer basin

| Sewer Basin | General Plan EDUs | Development EDU | Total EDU | Flow (MGD) | |
|---|---|---|---|---|---|
| | | | | Assuming 52.3 gpcd | Assuming 187.2 gpd/EDU |
| 1 | 9,009 | | 9,009 | 1.27 | 1.69 |
| 2 | 5,048 | | 5,048 | 0.71 | 0.94 |
| 3 | 3,124 | 3,439 | 6,563 | 0.93 | 1.23 |
| 4 | No expansion of urban services | | | | |
| 5 | 7,644 | 9,044 | 16,688 | 2.36 | 3.12 |
| 6 | 1,890 | 2,564 | 4,454 | 0.63 | 0.83 |
| 7 | 2,233 | 602 | 2,835 | 0.40 | 0.53 |
| 8 | 2,359 | | 2,359 | 0.33 | 0.44 |
| 9 | 9,575 | | 9,575 | 1.35 | 1.79 |
| 10 | 2,523 | 9,462 | 11,985 | 1.69 | 2.24 |
| 11 | 1,681 | 3,577 | 5,258 | 0.74 | 0.98 |
| 12 | | 1,857 | 1,857 | 0.26 | 0.35 |
| 13 | 4,015 | | 4,015 | 0.57 | 0.75 |
| 14 | 1,964 | 222 | 2,186 | 0.31 | 0.41 |
| 15 | 6,829 | 180 | 7,009 | 0.99 | 1.31 |
| 16 | 19,706 | | 19,706 | 2.78 | 3.69 |
| 17 | 16,376 | | 16,376 | 2.31 | 3.07 |
| 18 | 11,192 | 2,250 | 13,442 | 1.90 | 2.52 |
| 19 | 3,778 | | 3,778 | 0.53 | 0.71 |
| **Totals** | **108,946** | **33,197** | **142,143** | **20.06** | **26.61** |

## 6. REFERENCES

Great Lakes-Upper Mississippi River Board (GLUMRB). 2014. 10 States Standards. http://10statesstandards.com/waterstandards.pdf

National Oceanic and Atmospheric Administration (NOAA). 2015. NOAA Climate Data Online: Marana (5 stations). www.ncdc.noaa.gov/cdo-web/. Accessed October 22, 2015.