REGIONAL FACILITY PLAN

Lower Rio Grande Valley

B&V PROJECT NO. 181092

PREPARED FOR

RGRWA

1 JULY 2016

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CHAPTER 1 – INTRODUCTION

Regional Facility Plan

B&V PROJECT NO. 181092

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Rio Grande Regional Water Authority

4 DECEMBER 2015

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1.0 Introduction

1.1 PURPOSE

Due to the recent drought conditions, regional concerns over local water resources have grown in the Lower Rio Grande Valley (LRGV) that encompasses Cameron, Hidalgo and Willacy County. The Rio Grande Regional Water Authority (RGRWA), with a grant from the Texas Water Development Board (TWDB) and the Border Environment Cooperation Commission (BECC), is to develop a water facility plan that identifies potential water sources that could be developed as a regional solution for the growing water reliability concerns in the LRGV. The purposes of these planning efforts are to identify and evaluate the potential water sources and develop design criteria, an implementation schedule, an organization plan and financial details for the selected alternatives. This plan takes full advantage of previous studies performed on water resources, water management strategies, populations and demands in its development and evaluation of alternatives.

1.2 LOCATION

1.2.1 Rio Grande Regional Water Authority

The RGRWA was created by the 78th Legislature to supplement the services, regulatory powers and authority of irrigation districts, water development supply corporations, counties, municipalities, and other political subdivisions within its border. The RGRWA covers six counties in the Middle and Lower Rio Grande Valley: Willacy, Cameron, Hidalgo, Starr, Zapata and Webb (Figure 1-1). The RGRWA shares an approximate boundary with the Region M Water Planning Group. The focused study area includes a large portion of the Rio Grande Regional Water Authority jurisdiction commonly referred to as the Lower Rio Grande Valley. Specifically, the area includes the three southern most counties in the state, Cameron, Hidalgo, and Willacy.

1.2.2 Watershed

The Rio Grande is the major source of water supply in LRGV region. The Rio Grande Basin extends from southern Colorado through New Mexico and Texas as shown on Figure 1-2. Between El Paso, Texas, and the Gulf of Mexico, the Rio Grande forms the International Boundary between the United States and Mexico. The Lower Rio Grande basin which lies within the Rio Grande Basin extends from Fort Quitman, Texas along the U.S./Mexico border, to the Gulf of Mexico. Located in the region are Amistad and Falcon Reservoirs which are operated as a system for flood control and water supply purposes by the International Boundary and Water Commission (IBWC). Water rights from the reservoir system are allocated from the Texas Commission on Environmental Quality (TCEQ) Watermaster's Office. Diversions have significantly depleted river-flows as the river reaches Fort Quitman, Texas, just downstream from El Paso. In Mexico, the Rio Conchos, Rio Salado, and Rio San Juan are the largest tributaries of the Lower Rio Grande Basin.

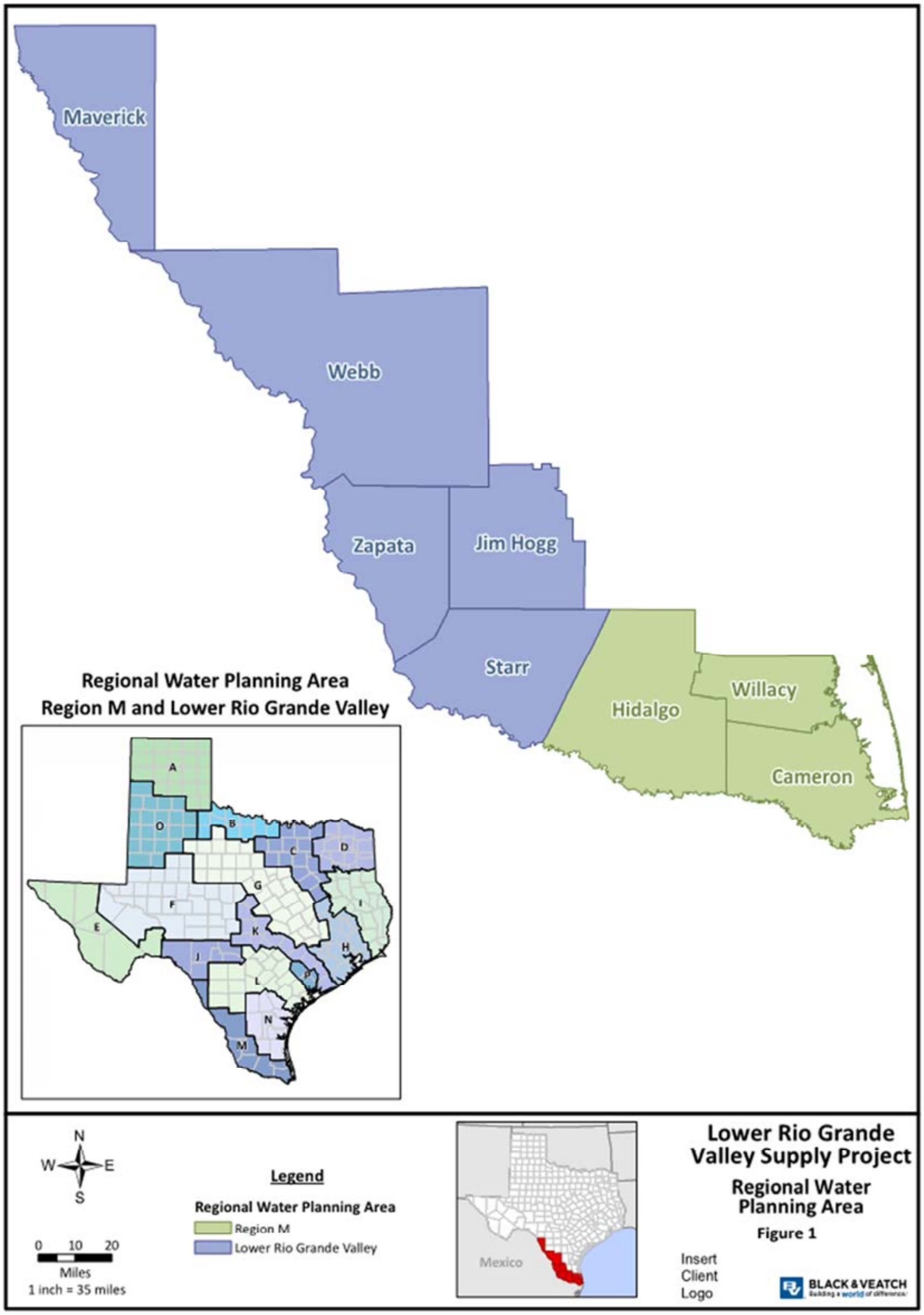

Figure 1-1 Regional Water Planning Area, Region M and Lower Rio Grande Valley

Lower Rio Grande Valley Supply Project

Rio Grande Basin

Figure 2

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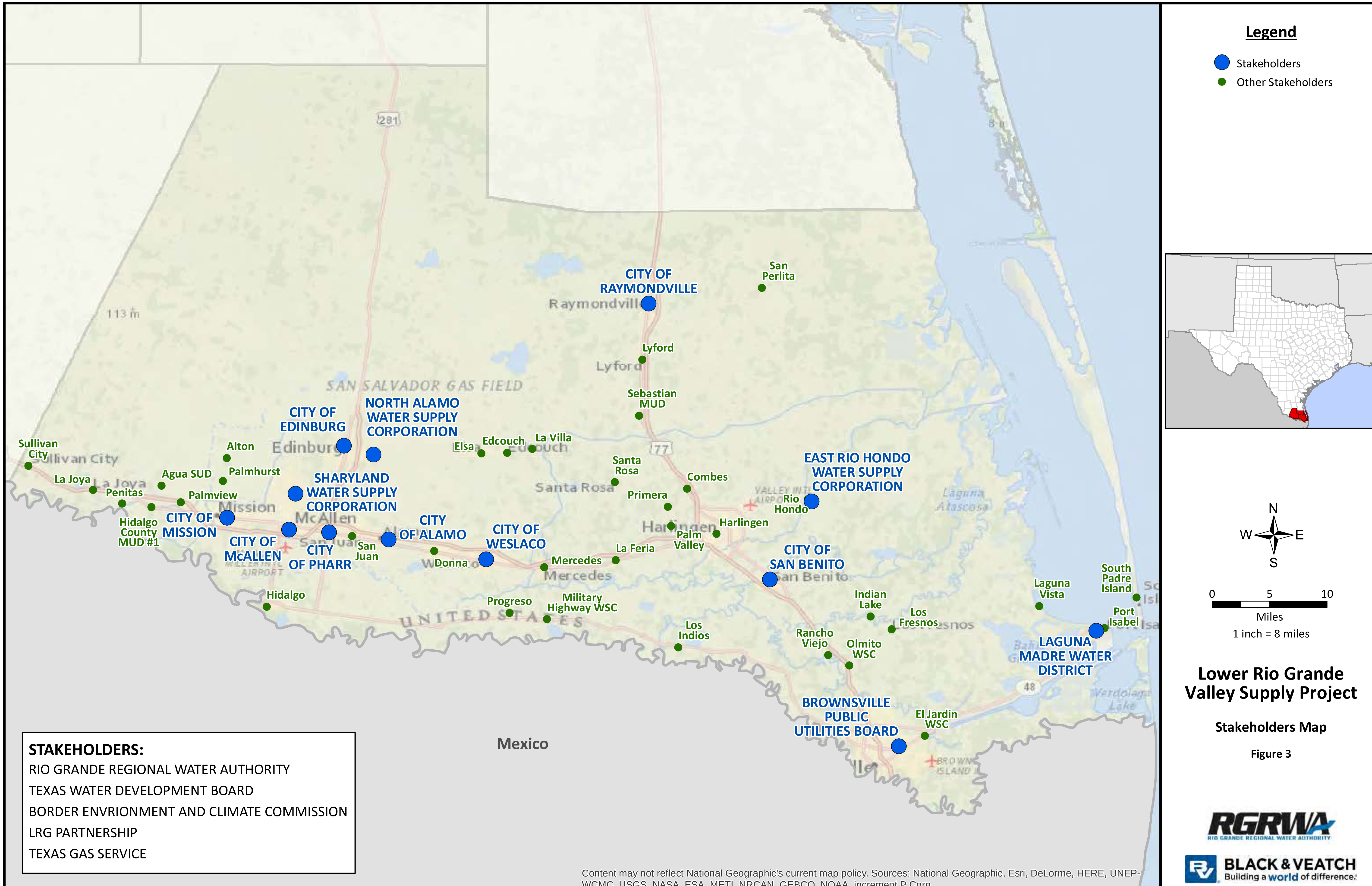
1.3 STAKEHOLDERS

Stakeholders for this plan include all potential project partners, TWDB, TCEQ, BECC, and the public. Sponsors for the plan include RGRWA, TWDB and a variety of water providers. A list of plan sponsoring and other stakeholders is included below:

- Sponsoring Stakeholders
  - Rio Grande Regional Water Authority
  - Texas Water Development Board
  - Border Environment and Climate Commission
  - Brownsville Public Utilities Board
  - City of Alamo
  - City of Edinburg
  - City of McAllen
  - City of Mission
  - City of Pharr
  - City of Raymondville
  - City of San Benito
  - City of Weslaco
  - East Rio Hondo Water Supply Corporation
  - Laguna Madre Water District
  - Lower Rio Grande (LRG) Partnership
  - North Alamo Water Supply Corporation
  - Sharyland Water Supply Corporation
  - Texas Gas Service
- Other Stakeholders
  - Other municipalities and/or water providers affected by this plan
  - Rate Payers
  - Public

Figure 1-3 illustrates the location of the study area, the counties and major stakeholders.



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1.4 PREVIOUS STUDIES

The RGRWA and other stakeholders have spent a considerable amount of resources and effort to understand the interdependencies, needs and potential resources in the area. These studies have been evaluated and integrated into this planning process to the extent that the information was useful in determining pre-disclosed purposes.

1.4.1 State Regional Planning

State and Regional Water Plans are developed on a 5 year cycle. The 2011 Regional Plan along with the 2012 State Plan has been adopted by the state. The 2016 Regional Plan is being developed and draft documents describing both demands and water management strategies have been incorporated into this plan.

1.4.2 Bureau of Reclamation Study

In an effort to address potential impacts from climate change, the US Bureau of Reclamation (BuRec) along with the RGRWA funded a resource study aimed at using brackish groundwater. The 2014 BuRec Lower Rio Grande Basin Study quantified losses in the Amistad Falcon Reservoir System due to decreases in precipitation and increases in evaporation. The total change in annual yield for an average year was an estimated reduction of 86,000 AF each year throughout the study period. The study recommended regional brackish groundwater plants be constructed around three demand centers centered around the three largest existing metropolitan areas: McAllen, Harlingen and Brownsville.

1.4.3 Brackish Groundwater Availability Studies

Groundwater availability studies have been completed in the area and are listed below. Estimates of brackish groundwater volumes and sustainable yields have varied considerably and the TWDB is commissioning an update to the GMA 16 groundwater study and hydrogeologic model based on the recent research included in the Brackish Resources Aquifer Characterization System (BRACS) Database.

- TWDB Report and Database on Brackish Groundwater in the Gulf Coast Aquifer, Lower Rio Grande Valley, Texas (BRACS). This study compiled hydrogeologic data for the brackish aquifers in the study area. It estimated aquifer thickness, salinity, depths and locations based on existing geophysical logs that were collected as part of the study.

- Southern Gulf Coast Groundwater Availability Models (GAM) by Chowdhury and Mace (2003 and 2007). This study utilizes MODFLOW to estimate approximate groundwater available in a larger area that includes both Brownsville and Corpus Christi.

- "GMA 16" model by Hutchison and others (2011). This study also utilizes MODFLOW to estimate groundwater levels and availability in the region.

1.5 DESCRIPTION OF SOURCES

1.5.1 Current Water Use

Current water use in the Region M Planning area is predominately from the Rio Grande. A small amount of fresh groundwater is being used, while brackish groundwater has become a bigger part of the regions portfolio. Reclaimed wastewater is being used to some degree for irrigation, cooling of combine cycle power plants, and other non-potable processes. The subset of the study area is very similar in its water profile. Figure 1-4 displays the various major water sources in the area as a percent of the projected 2020 use from the Region M plan. The projected demands and water resource availability will be evaluated further for comparisons and project selections.

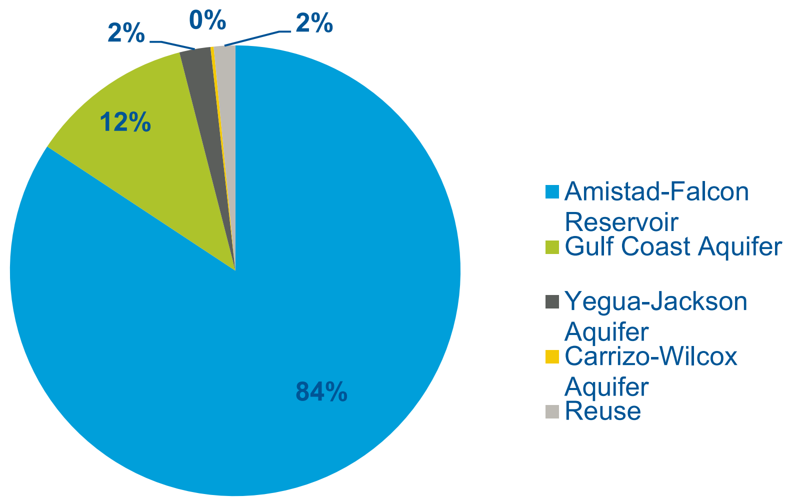

Figure 1-4 Major Water Resources, Region M (2020)

1.5.2 Amistad-Falcon Reservoir System

Practically all of the surface water used in the Rio Grande Region is from the Rio Grande, which is from the yield of the Amistad and Falcon International Reservoirs. The Falcon Reservoir releases just under 1 million AF of water in an average year. These reservoirs are operated as a system by the IBWC for flood control and water supply purposes. These impoundments provide controlled storage for over 8 million acre-feet of water owned by the United States and Mexico, of which 2.25 million acre-feet are allocated for flood control purposes and 6.05 million acre-feet are reserved for sedimentation and conservation storage (water supply). Practically all municipal, domestic, industrial, agricultural and mining water rights have been allocated from the system. Current water rights come available as irrigated land is developed. Since all water rights are adjudicated, further water right must come from non-municipal water rights that are converted to municipal water rights. Water rights are managed and allocated by the TCEQ Watermater's office. Further discussion into the management of the water rights is included in Chapter 3.

Some very limited surface water is available from sources in the Lower Rio Grande Valley in Maverick, Webb, Zapata, Willacy, Hidalgo, Cameron, and Starr Counties: from the Arroyo Colorado, which flows through southern Hidalgo County and northern Cameron County to the Laguna Madre; from the pilot channels within the floodways that convey local runoff and floodwaters from the Rio Grande throughout the Lower Rio Grande Valley to the Laguna Madre; and from isolated lakes and oxbows (locally known as resacas) in Hidalgo and Cameron Counties. Under drought of record conditions, surface water supplies from these other sources have very little flow and are of little significance.

Existing springs within the Rio Grande Basin of the Region M Planning Area (primarily Maverick, Webb, Zapata, Jim Hogg, and Starr Counties) are not numerous and are small in terms of their discharge quantities. There are no major springs that are extensively relied upon for water supply purposes. Many of the small springs do provide water for livestock and wildlife when they are flowing. Typically, the flow rate of the existing springs is less than 20 gallons per minute, with most springs in the region flowing at a rate of only a few gallons per minute. Figure 1-5 shows the Rio Grande basins major tributaries. Figure 1-6 illustrates how the local irrigation districts and water utilities receive and distribute water from the Rio Grande.

Figure 1-5 Major Tributaries of Rio Grande

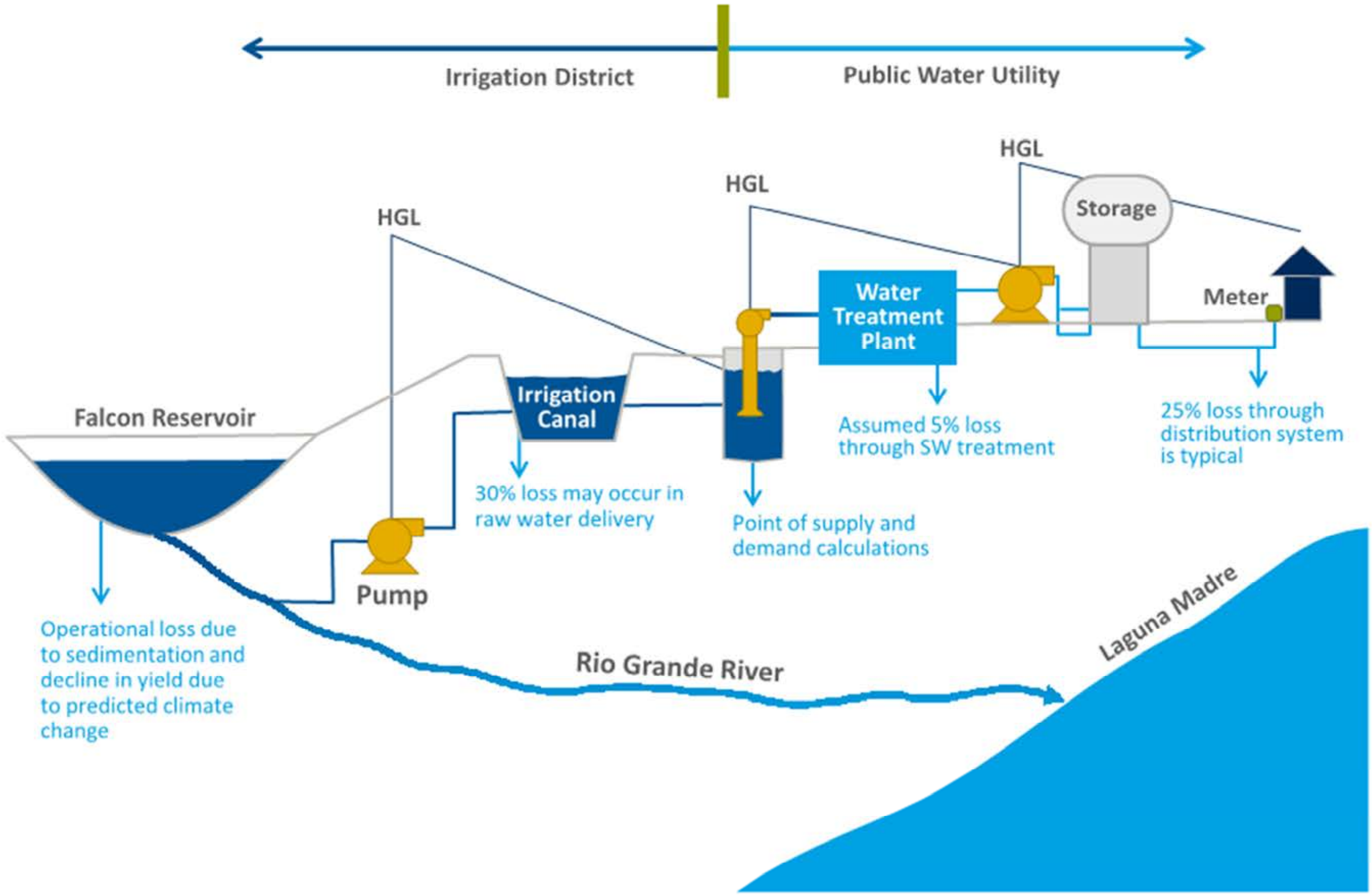

Figure 1-6 **Raw Water Distribution from the Rio Grande in the LRGV**
This graphic also shows a representative hydraulic grade line (HGL) for the system

1.5.3 Groundwater

The major aquifer within the study area is the Gulf Coast aquifer (see Figure 1-7), which underlies the entire coastal region of Texas. In general, groundwater from the aquifer in the region have total dissolved solids (TDS) concentrations exceeding 1,000 milligrams per liter (mg/L) (slightly saline) and often exceeding 3,000 mg/L (moderately saline). The salinity hazard for groundwater ranges from high to very high, resulting in restricted use for irrigation and livestock watering. Developing and desalinating groundwater in the study area are increasing in interest because of the recent droughts and competition for surface water supplies.

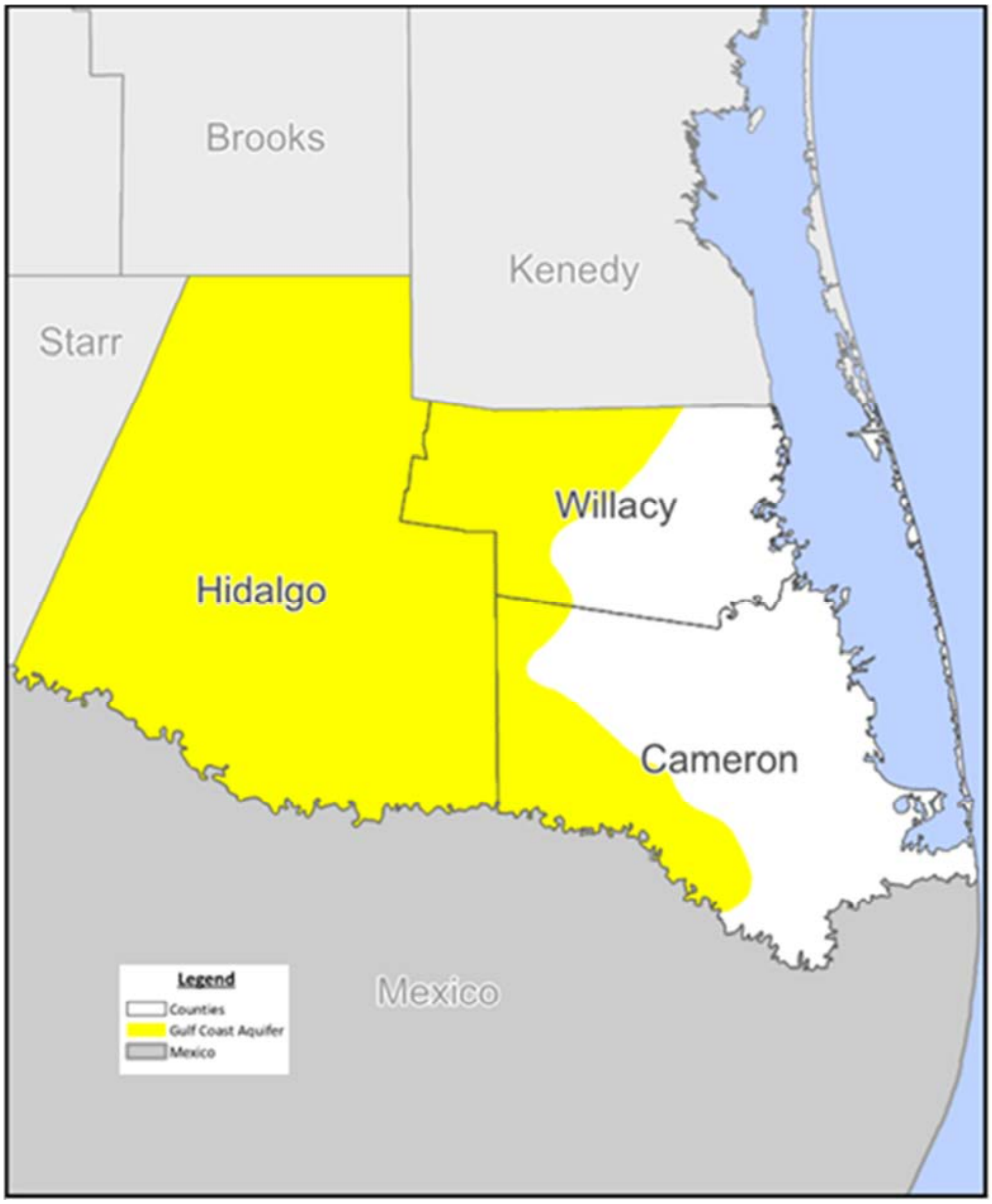

Figure 1-7 Gulf Coast Aquifer in Lower Rio Grande Valley

1.6 APPROACH AND IMPLEMENTATION

The goal of this planning effort is to provide a thorough evaluation of supplies and demands for the study area, based on previous work completed, and detail a regional solution to meet the needs of municipal water users. Figure 1-8 below presents the flowchart adopted for this effort.

The first step is to identify all the cities, water supply corporation and irrigation districts in the region and summarize the water data for them. Data from regional water plan (Draft 2016 Region M plan), which was finalized and approved by the TWDB, was used to evaluate population projections, to identify all water user groups, and to establish water demands. Water supplies were evaluated to estimate potential water availability for municipal drinking water uses. Based on the demands and available water supplies the plan recommends strategies that could be implemented to address the water needs. The plan takes into consideration stakeholder organizational structures and potential rate impacts derived from infrastructure operations and maintenance costs.

The nature of the planning process requires simplifying assumptions be made to quantify supplies, demands and their resulting needs. A technical memorandum describing these assumptions has been included in Appendix A. The individual assumptions are described in more detail in their corresponding chapters.

VALLEY WATER SUPPLY PROGRAM

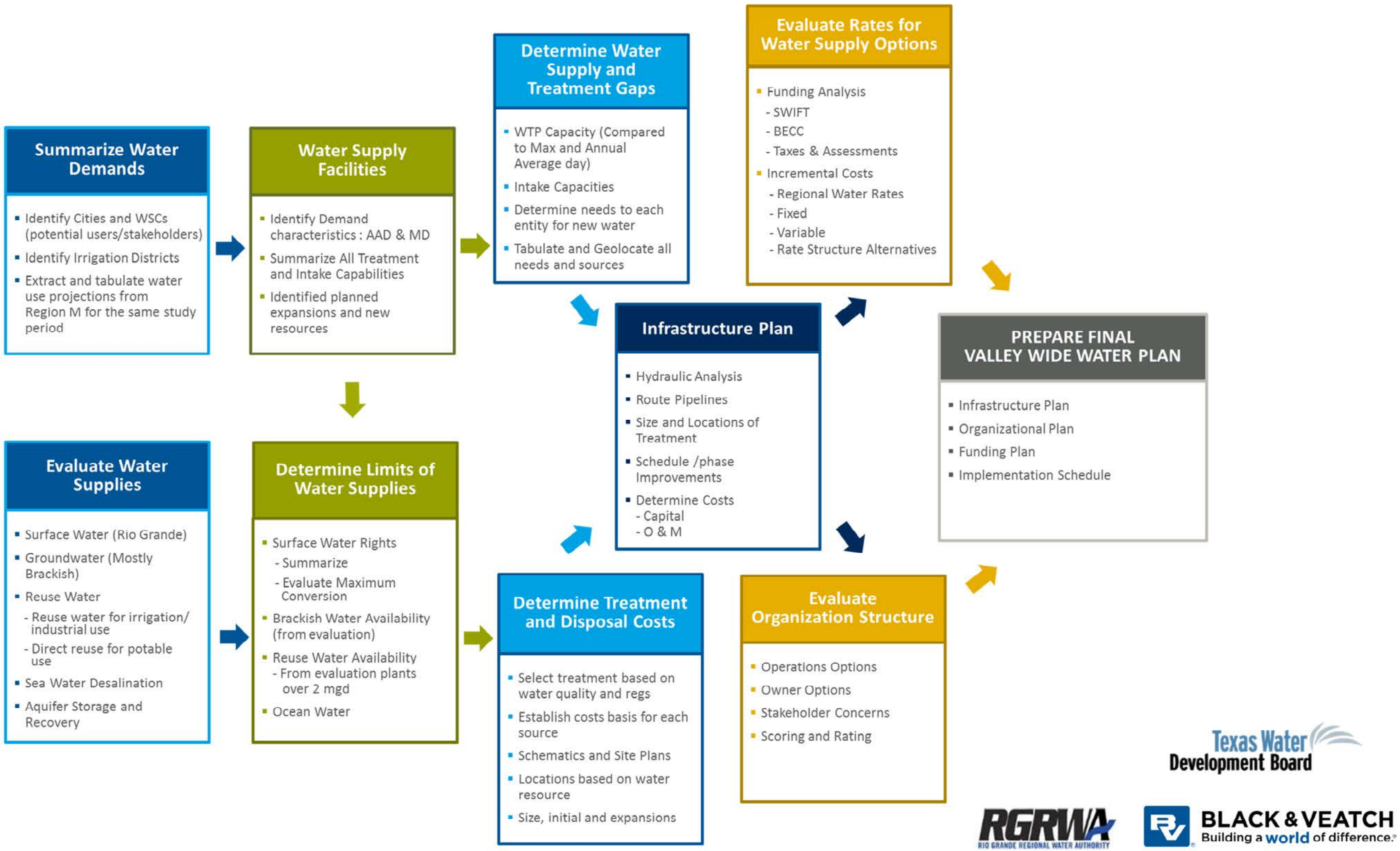

Figure 1-8 Valley Water Supply Program Process Flowchart

Appendix A. Memorandum of Understanding

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MEMORANDUM

Rio Grande Regional Water Authority
Lower Rio Grande Valley Regional Water Supply Plan — Black & Veatch PN 181092
Draft Memorandum of Understanding — March 2015

To: Rio Grande Regional Water Authority – Groundwater Committee

From: Robert Jenkins, PE

Purpose: This Memorandum of Understanding documents the assumptions and processes that will be followed during the execution of each task of the Regional Water Supply Plan.

1.0 Project Background

The Rio Grande Regional Water Authority (RGRWA) was awarded a Regional Facility Planning Grant by Texas Water Development Board (TWDB) to evaluate and determine the most feasible alternative to meet regional water supply needs for areas in the Lower Rio Grande Valley. The study area is comprised of Cameron, Willacy, Hidalgo, and the eastern portion of Starr Counties. The Lower Rio Grande Valley Regional Water Supply Plan will assess the water demand and available water resources of the planning area. Various water resource alternatives will be evaluated and a recommendation of the best solution for a regional water supply will be made. Factors for consideration will include location and capacity of potential water resources, existing treatment facilities, water provider needs and planned supply strategies, costs, organization structure, and alternative funding opportunities. A preliminary engineering report will be prepared for the recommended solution.

1.1 Project Stakeholders

All municipalities and/or water providers located in Cameron, Willacy, Hidalgo, and the eastern portion of Starr Counties

1.2 List of Deliverables

- Memorandum of Understanding (MOU) (Draft and Final)
- Water Demand Analysis TM (Draft and Final)
- Water Resources Availability TM (Draft and Final)
- Infrastructure Plan TM (Draft and Final)
- Organizational and Funding Analysis (Draft and Final)
- Preliminary Engineering Report (PER) (Draft and Final)

1.3 Administration

- Meetings with the Groundwater Committee will be held every other month after the regular RGRWA meeting.
- Presentation and workshops for Project Stakeholders will be scheduled at key times during the project to ensure adequate stakeholder input is included. It is estimated that 6 stakeholder presentations and/or workshop will be held.

- Presentations to the Public will be held at the commencement of the project, when it is 50% complete, and within 30 days after the study completion date. Refer to the Executive Schedule for tentatively scheduled dates for these meetings.
- Discussions and meetings with regulatory and funding agencies will begin during the PER stage.
- Monthly progress reports will be submitted to TWDB, BECC and RGRWA.
- Each chapter of the report will be submitted to the committee as a technical memorandum for review and comment and shall be posted to the website hosted by the RGRWA.
- Organizational and financial strategies will be identified and scored through facilitated workshops with the committee.

2.0 Project Assumptions and Design Basis

2.1 Information Sources

Information from the following sources will be used in order to reduce the amount of redundant work performed:

- 2016 Region M Regional Water Plan Draft (in progress, due May 2016)
- Bureau of Reclamation Lower Rio Grande Basin Study
- TWDB Report on Brackish Groundwater in the Gulf Coast Aquifer, Lower Rio Grande Valley, Texas (BRACS)
- Rio Grande Basin Water Availability Model (WAM) from TCEQ
- Groundwater Availability Model (GAM)
- Texas Commission on Environmental Quality (TCEQ)
    - Location and capacity of water treatment plants in the study area will be obtained from the TCEQ website
    - Annual average effluent flow data for wastewater treatment plants considered for reuse water will be obtained from the TCEQ Office of Compliance and Enforcement
- Environmental Protection Agency (EPA) website
    - Location and capacity of wastewater treatment plants in the study area will be obtained from the EPA EnviroFacts website
- Arroyo Colorado Watershed Partnership website
    - Location and water quality of the Arroyo Colorado and its tributaries will be obtained from the Arroyo Colorado Watershed Partnership website in order to determine possible discharge points from RO brine
- Rio Grande Watermaster Office
    - Information on water rights ownership
- Rio Grande Regional Water Authority
    - Reference Reports

3.0 Water Demand and Supply Analysis

3.1 Projected Potable Water Demand

The projected municipal water demand will be based on population projections and estimated water usage in gallons per capita per day (GPCD) prepared by the TWDB for the 2016 Region M Regional Water Plan Draft (Chapter 2) and as modified with additional infrastructure. More specific information will be used if provided.

3.2 Water Management Efficiency

- The projected municipal water demand does not include special regional or municipal initiatives to decrease water waste with the exception of a minimal reduction in the GPCD due to federal and state requirements for water fixture manufacturers.
- Conservative Estimates for Water Conservation and System Efficiencies will be included based on the GPCD usage as compared to the national average.
- Every municipal WUG was assigned water conservation as a possible water management strategy.

4.0 Availability Analysis

4.1 Surface Water

The regional surface water availability will be evaluated using the following:

- The amount of available water from the Rio Grande will decrease by 13% by 2070, due to sedimentation build up in the Amistad and Falcon Reservoirs.[1]
- It is assumed that there will be 86,438 AFY less available surface water in the Lower Rio Grande by 2060 due to Climate Change.[2]
- Historical urbanization rates from either irrigation districts or from municipal growth rates will be used to estimate the amount of agricultural water rights that will be converted to municipal water rights.
- It will be assumed that the urbanized agricultural land is Flat Rate acreage, which is allotted 2.5 AFY water per acre of land.
- Push water requirements will reduce surface water supplies during drought years.
- Maximum availability of surface water rights is 90% due to market limitations.
- Water Rights are portable.
- Excess surface water can be used through permit number 1838 for groundwater recharge as available.
- Typical water quality parameters for Rio Grande water will be used based on the average water quality as provided by Brownsville, Harlingen and McAllen,
- Conventional water treatment processes will be used to treat raw surface water, as is the current practice.
- New surface water capacity may be provided at existing or new facilities.

4.2 Groundwater Recharge

The use of groundwater recharge will be evaluated using the following:

- Groundwater Availability Model (GAM) Particle Tracking Simulations

[1] 2016 Region M Regional Water Plan Draft, Chapter 3
[2] Bureau of Reclamation Lower Rio Grande Basin Study, Chapter 2, Section IV.D.3

- Capacity will be calculated based on drought of record assuming multi year drought and using annual discharge from Amistad Reservoir as provided on the International Boundary & Water Commission.
- TCEQ regulations will be followed for Class V ASR injection wells.
- The water retrieved from aquifer storage will not require treatment.

4.3 Brackish Groundwater

The regional brackish groundwater availability will be evaluated using the following:

- Previous studies for existing and potential brackish groundwater desalination facilities within the study area
- Brackish Groundwater availability is to be estimated from the refined transient GAM Model.
- Transient simulations of the further defined GAM to meet desired future conditions.
- The use of particle tracking within the model will estimate location of origin of brackish groundwater withdrawals
- Assume water TDS Concentrations is below 3,000 mg/L
- Assume surface water discharge of RO Concentrate.
- An estimate TDS for the Gulf Coast Aquifer brackish water will be determined through analysis of information provided in the BRACS Report. It will be less than 3,000 mg/l.
- Recovery rate from the Reverse Osmosis (RO) process will be based on the existing water treatment plant performance.
- Pretreatment for iron, manganese and arsenic needed
- It will be assumed that concentrate will be discharged to the Arroyo Colorado as is the current practice.

4.4 Reuse Water

- Reuse water alternatives will be limited to sources that can supply an estimated 1 MGD minimum annual average flow of direct potable reuse water.
- WWTP will have an assumed peaking factor of 2.0 to calculate average daily use.
- Assume 80% of average WWTP effluent is available for reuse on a consistent basis
- Direct Reuse of water opportunities will be evaluated for potable water replacement.
- Direct Reuse water use is limited partially by the expected TDS of wastewater effluent.
- TCEQ 210 rules will be followed for infrastructure and treatment requirements
- Direct Potable Reuse will assume Advanced Treatment.
- It is assumed that the wastewater has accumulated 150 mg/l of TDS from the raw water based on recent studies in Oklahoma.
- Recovery rate from the RO process will be 85%
- Direct potable reuse treatment process will include dual membrane barrier and advanced treatment which may include advanced oxidation, Micro Filtration (MF)/RO/UV.
- Assume a 5:1 dilution with surface water prior to conventional treatment at an existing water treatment plant.

- For Direct Reuse of Wastewater Treatment will include tertiary filtration addition if it is not already provided.

4.5 Sea Water

- Sea water intake and discharge locations will not be evaluated but will be assumed to be located in an area where tidal flows will not affect the intake of raw water or dispersion of wastewater
- An unlimited supply will be assumed to be available
- A TDS of 35,000 mg/l (typical sea water salinity) will be used for the raw water quality of the Gulf of Mexico
- Recovery rate from the RO process will be 50%
- Pretreatment for boron required
- Concentrate management will be evaluated based on the location of proposed WTP(s) and quantity and quality of the brine. Options to be evaluated include:
  - o Surface water discharge
  - o Sea water discharge through an outfall

4.6 Existing and Planned Facilities

- An annual average flow for treatment facilities will be determined using a peaking factor of 1.3.
- Water Supplies will take into consideration limitations due to infrastructure capacities.

4.7 Water Quality Requirements

- Total Dissolved Solids (TDS) will be used as the chief parameter to determine water quality and treatment requirements.
- Treated water quality goals will include non-corrosive and a compatible water source

5.0 Gap Analysis

- The shortage over the specified timeframe will be developed by taking the difference of the projected demand from the amount of existing water supplies. This information will be used in assessing needs and replacing potential WMS.
- It will be assumed that all of the water management strategies (WMS) recommended in Chapter 4 of the 2016 Region M Regional Water Plan will be implemented, unless they are deemed unnecessary by the implementation of the recommendation in this plan.

6.0 Preliminary Engineering Report

Assumptions for the PER will be dependent on solutions derived from previous tasks.

7.0 Anticipated Project Schedule

Figure 1 shows the anticipated Executive Schedule for the study.

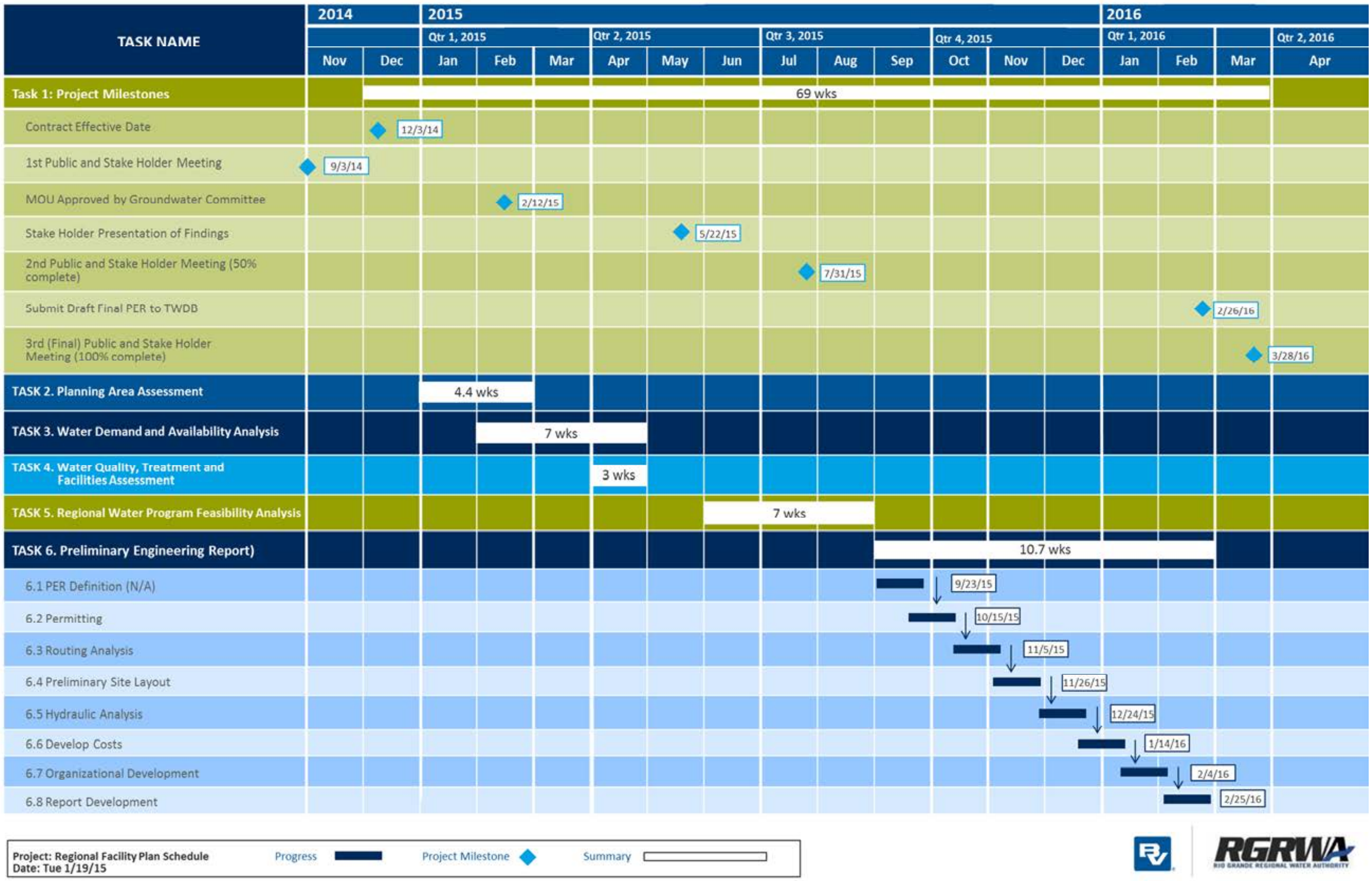

Figure 1. Lower Rio Grande Valley Regional Water Supply Study Schedule

Appendix B. Scope of Work

EXHIBIT B

SCOPE OF WORK

The ultimate intent of this program is to provide a comprehensive management plan for the study area to include all water purveyors (both potable and irrigation). Due to the extensive scope of this effort, this first phase will focus on the evaluation of the largest potable water and irrigation organizations to establish the first projects that will form the basis for additional phases in the future. Many previous studies will be utilized to form the basis of this more detailed analysis, including Region M water plans, Bureau of Reclamation studies, and academic investigations. The intent is to not duplicate any work but take high level concepts and strategies and develop entity level information. For example, the task will be to drill down in a general reuse strategy developed for water planning and explore specific details at a facility level to assess viability for an individual user or region-wide use. The following tasks will be similar for all phases of this project. None of the work will be duplicative of current regional water planning efforts.

TASK 1. Program Administration

1. Contract monitoring and administration
2. Schedule maintenance and administration
3. Program cost monitoring and management
4. Program and progress meetings
5. Public and stakeholder presentations
6. Meetings with regulatory and funding agencies
7. Develop a data base of information and interface/web site for stakeholders

TASK 2. Planning Area Assessment

The purpose of this task is to quantify all available water resources, projected population and water demands, water rights and regulatory requirements. Task will build off of work being completed and summarized for the current Region M water plan particularly where recommendations of individual strategies to satisfy needs at the planning level will limit a detailed analysis of a regionalized concept for supply. There will be an emphasis on development of GIS to help in the spatial analysis necessary to support regional project development.

1. Summarize the existing facilities types and capacities that serve the project area for wastewater treatment, water treatment and distribution of raw water for irrigation and potable water use.
2. Review the exiting water management strategies (WMS) plus identify existing plans for expansions or improvements of irrigation systems, water treatment and wastewater treatment systems not captured in the regional plan or not captured at the level of detail

needed for this effort. This will include efforts to improve efficiency and add capacity of all systems.

3. Summarize the findings and recommendations of previous reports on water facilities and management by all stakeholder participants, the State and university organizations.

Deliverable: Summarize all findings in a draft and final technical memorandum.

None of the work will be duplicative of current regional water planning efforts.

TASK 3. Water Demand and Availability Analysis

The purpose of this task is to summarize all demands and compare against the available water supplies and facilities to summarize the regional shortfall in water supply at a level of detail not included in the strategies in the planning or reclamation project documents. Planning data at county level irrigation strategy will not be the same as Irrigation District specific information intended for this task. This task will build off of work being completed and summarized in the Region M plan and recent Bureau of Reclamation projects. This effort will assist in conceptualizing a regional project in lieu of projects being developed by individual users in parallel with the normal planning process. Since the purpose of this effort is for design and construction rather than planning, the timeframe for this comparative analysis will be 15 years.

1. Compare demand and availability by entity including Irrigation Districts, specific farmland, and individual industrial users. This will be a more through analysis than provided in the Region M Plan.
2. Determine the effects of implementing efficiency/conservation measures for municipal and agricultural uses.
3. Perform gap analysis (this will be user specific) to determine the location and quantity of water shortages/overages. GIS review of this will facilitate potential interconnect information.
4. Determine physical, legal, and regulatory limits to availability of water resource alternatives.
    a. Reuse water
        i. Review existing allocations for possible redistribution or incorporation into single regional project.
        ii. Identify impact on reducing stream flows for any identified regional reuse project for this study.
    b. Brackish groundwater
        i. Identify the regions for available brackish ground water, utilizing the BRACS database and regional report.
        ii. Identify through a groundwater availability model (GAM).
        iii. Evaluate feasibility of recharging the aquifer and long term storage.
    c. Groundwater recharge
    d. Sea Water
        i. Determine locations of extraction.
        ii. Confirm limitations of availability.
5. Rules and Regulations

a. Summarize rules and regulations governing capacities and use of water alternatives.

Deliverable: Summarize all findings in a draft and final technical memorandum. None of the work will be duplicative of current regional water planning efforts.

TASK 4. Water Quality, Treatment and Facilities Assessment

The purpose of this task is to identify all treatment requirements required by water resource to be able to utilize the water for municipal and agricultural/industrial uses. Regional water quality standards are being developed for drinking water treatment requirements. This task will also assess the individual discharge treatment levels in order to identify which discharges would be available for particular use strategies, i.e., some water discharged may not be suitable for irrigation but may be suitable for industrial needs.

1. Summarize water quality requirements
    a. Municipal Water.
    b. Agricultural/Industrial water.
    c. State and Federal regulations.
2. Water Quality and Treatment Evaluation
    a. Raw Water Quality Summary by water resource.
    b. Treatment Overview by water resource.
    c. Preliminary selection of treatment processes.
    d. Budget level costs (Capital and O&M) by water resourceTWDB costing tool will be used for compatibility with Regional Water Planning costs development.

Deliverable: Summarize all findings in a draft and final technical memorandum.

TASK 5. Regional Water Program Feasibility Analysis

The purpose of this task is to evaluate the alternatives to meet the water needs through regionalized facilities including treatment facilities, conveyance and storage requirements, capacities of all facilities, organization and governance, project sequencing, and schedule for implementation.

1. Three major options for regional water systems will be investigated for the most critical areas.
    a. Single independent regional system, designed to serve a large portion of the region via individual service interfaces.
    b. Hybrid system utilizing larger regional facilities plus larger local facilities, with an emphasis on inter-connections.
    c. Sub-regional systems, potentially expansions of the capacity and capabilities of existing facilities, and identification of sub-regions with infrastructure or geographical ties.

2. Estimate the construction costs of the recommended project for each major element of the proposed improvements and new facilities. These unit costs of water will be used as the basis for evaluating all alternatives (TWDB costing tool will be used for compatibility with Regional Water Planning costs development).
   a. Well field
   b. Raw water conveyance
   c. Treatment facility
   d. Brine disposal
   e. Treated water conveyance
3. Management Agency Evaluation
   a. Summary of existing agencies
   b. Summary of financial, political and operational capabilities
   c. Evaluation of owning agency for implementation
4. Financial Analysis Results
   a. Capital Costs
   b. Operating Costs
   c. Cost per Unit of Volume
   d. Comparison and coordination with regional rates
   e. Development of preliminary rate structure
   f. Identify and Apply for Federal and/or State Fund
5. Development of feasibility report
   a. Stakeholder presentations of evaluations.
   b. Evaluation and scoring of individual alternatives
   c. Development of water resource management plan
   d. Phasing of water structures.
   e. Plan for organizational implementation
   f. Plan for funding

None of the work will be duplicative of current regional water planning efforts.

TASK 6. Preliminary Engineering Report

The purpose of this task is to further develop the first phases of the water management program.

1. Anticipated components of the program at this point in time are expected to be:
   a. Phase 1 Brackish desalination facilities
   b. Phase 1 aquifer storage and recovery facilities
   c. Phase 1 water distribution network
   d. Phase 1 water reuse system improvements
2. Permitting
   a. EPA/TCEQ
   b. COE
   c. Local

3. Routing analysis
    a. Raw water facilities

    b. Finished water delivery facilities
4. Preliminary site layouts
    a. Well fields
    b. Intake structures

    c. WTP sites
5. Hydraulic analysis
    a. System development
    b. Storage analysis
6. Develop costs
    a. Engineers opinion of probable cost
    b. O&M costs
    c. Develop rate structure
7. Organizational Development
    a. Finalize program ownership
    b. Finalize operations responsibilities
    c. Develop draft contracts for implementation
8. Report Development
    a. Develop draft report for Phase 1 improvements.
        i. Summarize all individual evaluations
        ii. Develop implementation schedule
        iii. Develop funding schedule
    b. Prepare presentation of recommendations.
    c. Finalize report.

Deliverable: Final report.

The following tasks will be completed under non-TWDB funding, and will not be included in study report.

TASK 7.Preliminary Design

This task will further define the facilities under design.

TASK 8. Final Design

Task to be defined based on the facilities selected and the project delivery methodology.

TASK 9. Construction Phase Services

Task to be defined based on the facilities to be designed and project execution plan.

Appendix C. Draft Comments Form

September 12, 2016

Addendum 1

to

Regional Facility Plan Lower Rio Grande Valley

Final Report Dated July 1, 2016

The following changes shall be made to the report:

1. Page 1-4 of the Introduction, change "Border Environment and Climate Commission" to "Border Environment Cooperation Commission"

2. On Page 1-12, of the Introduction, add the following paragraph to the end of Section 1.6:

   "As part of long term planning efforts in the Lower Rio Grande Valley (LRGV) the Rio Grande Regional Water Authority (RGRWA) has worked with and supported the improvement of agricultural efficiencies and cooperation between municipalities and irrigation districts in developing major projects. This project addresses the interface and impacts of the irrigation systems and the municipal systems as follows:

   - Agricultural efficiency strategies are included in the Region M Plan and were incorporated in Chapter 3 - Gap Analysis for the municipal water user (see appendix for detail). These were specific improvements that were coordinated between the municipal users and agricultural interests, or irrigation districts and were recognized in our plan.

   - This project was originally envisioned with a much more integrated relationship between the irrigation systems inefficiencies and the volumes of water delivered to the municipalities and resultant saved water. However; since DMI water rights have priority over agricultural water any improvements in efficiencies will not have a significant effect/increase on municipal water supplies. Agricultural water rights converted for domestic use by irrigation districts due to development are discussed in detail.

   - Originally it was thought that surface water would be delivered through an existing irrigation system and improving that system (or systems) would result in improved efficiencies (more water for municipal entities) in the delivery. But the regional surface water strategies developed in this plan only include piping Rio Grande water directly from the river to avoid the any of the water losses that are common in agricultural systems. This new intake in the Rio Grande has essentially zero losses, a huge improvement over the 40%to 60% losses common in irrigation canals. "

3. On page 13 of the Appendix A, to Chapter 13, "Financial Initiative Plan", Replace the text describing the Border Environmental Infrastructure fund (BEIF) with the following text:

2.3.1 Border Environment Infrastructure Fund (BEIF)

The U.S.-Mexico Border Water Infrastructure Program, funded by Congress through EPA, has awarded grants to water and wastewater systems in the border region through the Project Development Assistance Program (PDAP) for project development and design. The Border Environment Infrastructure Fund (BEIF) provides funding for construction, programs administered by NADB with BECC approval.

Applications are for a maximum of $30M and project sponsors are encouraged to complete final design for analysis of eligibility. The analysis shall include a comprehensive financial review of the project and eligible project costs. The agency will work with RGRWA to determine a maximum debt capacity and work from that point to a final determination of grant eligibility. The BEIF program shall not exceed $8M on any one project in grant funding. The remainder of the eligible project will be funded by a loan.

Water Division
Quality Control Review Comments

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Project Number:	181092	**Project Name:**	Regional Facility Plan
Project Manager:	Rob Jenkins	**Client:**	RGRWA
Engineering Mgr:		**Stage of Design:**	Final Report

Comment Codes:
A. Incorporate/Add
B. Confirm
C. Consider
D. Change
E. Note

Response Codes:
1. Incorporated
2. Confirmed
3. Noted
4. No Change, Designer Preference
5. Need Additional Info/Direction
6. Requires Further Investigation, Next Submittal
7. Not in Scope

Verifier						Designer Response			
Comment No.	**Date**	**Reference Dwg, Spec, Page**	**Comment Code**	**Review Comments / Questions**	**Verifier's Name**	**Response Code**	**Resolution/ Response Comment**	**Responder Name**	**Response Date**
1	7-Apr-16	1-1	A. Incorporate/Add	Add "and the Border Environment Cooperation Comission" to grant resources	BECC	1. Incorporated	Added to resources	DD	20-Jun-16
2	7-Apr-16	1-1	C. Consider	Which might these political tends be? First Sentence	BECC	3. Noted	Sentence changed to address drought instead of political trends.	DD	20-Jun-16
3	7-Apr-16	1-1	B. Confirm	Is it known if these diversions for irrigation are illegal?	BECC	2. Confirmed	These diversions are not illegal.	RJ	20-Jun-16
4	7-Apr-16	1-1	B. Confirm	Have contributions from these sources diminished over time? (tributaries to lower rio grande basin)	BECC	3. Noted	These contributions have dimished over time, but it has been from unknown reasons.	RJ	20-Jun-16
5	7-Apr-16	1-4	A. Incorporate/Add	Add BECC to project partners	BECC	1. Incorporated	Add BECC to list	DD	20-Jun-16
6	7-Apr-16	1-9	B. Confirm	Are the to water loss precentages based on project assumptions or general literature assumptions	BECC	1. Incorporated	Figure updated to convey precentages are based on previous project assumptions.	DD	20-Jun-16
7	7-Apr-16	Chapter 1 Appendix	A. Incorporate/Add	Add BECC to page two for progress reports bullet	BECC	1. Incorporated	Added in the Draft MOU under draft study documents	DD	20-Jun-16
8	7-Apr-16	Chapter 1 Appendix	D. Change	Change "brackish groundwater" to surface water on page 3	BECC	1. Incorporated	Changed in the Draft MOU under draft study documents	DD	20-Jun-16
9	7-Apr-16	Chapter 1 Appendix	B. Confirm	Is there another source of infromation on losses in distribution system? Page 3	BECC	1. Incorporated	Overall losses was not taken into account in the final report.	RJ	20-Jun-16
10	7-Apr-16	2-12	B. Confirm	Units for table 2-4 are acre-feet?	BECC	1. Incorporated	Added (AF/YR) to table title	DD	20-Jun-16
11	7-Apr-16	4-3	A. Incorporate/Add	Can the six Mexican tributaries be listed?	BECC	1. Incorporated	List of rivers added to text.	DD	20-Jun-16
12	7-Apr-16	4-9	C. Consider	Sentence describing steps 2 through 4 is repeated in first two paragraphs	BECC	1. Incorporated	Removed repeated sentence	DD	20-Jun-16
13	7-Apr-16	4-13	C. Consider	Are the conveyance losses described here different from thoseon 1-9 and in appendix A?	BECC	3. Noted	The conveyance lossed described here support those described on 1-9.	RJ	20-Jun-16
14	7-Apr-16	4-14	B. Confirm	Were there any assumptions or estimates related to efficiencies of the conveyance system if the remaining "unlined canal" were to be lined or piped? or it wouldn't make a difference in the big picture?	BECC	7. Not in Scope	No irrigation districs were included or evaluated in the report. Losses are elimated by putting intake directly on the river.	RJ	20-Jun-16
15	7-Apr-16	4-15	B. Confirm	There is no way to convert any savings in irrigation for the agricultural WUG to pumping into an Aquifer Storage and Recovery system without holding the water rights to those specific volumes?	BECC	7. Not in Scope	Not in Scope	RJ	20-Jun-16
16	7-Apr-16	4-15	C. Consider	If the information is readily available, can a table be added comparing the historical water availability for diversion from the river vs the water rights owned by each water utility vs the water actual water rights used for a reasonable past period of time? Something similar to Figure 4-5 but with historical information instead of projected information	BECC	3. Noted	This was considered, but the information was not readily avaiable.	RJ	20-Jun-16
17	7-Apr-16	4-15	A. Incorporate/Add	Can the concept of "charge their network of canals" be explained?	BECC	1. Incorporated	Explained in paragraph.	RJ	20-Jun-16
18	7-Apr-16	4-16	B. Confirm	Are water rights transferable from one property to another? do any Irrigation District allows this? For example, if a farmer owns three pieces of land, and builds a residential development in one of them, can he transfer the original water rights to any of the other two pieces of land that belong to him? Or is it that once the land is developed then water rights go to the utility always? as explained in the paragraph below?	BECC	2. Confirmed	Yes, they can be transferred, but once land is developed and platted, it must follow state laws found in Water Code, Title 4. Subchapter O.	RJ	20-Jun-16
19	7-Apr-16	4-23	B. Confirm	Have these numbers stayed constant throughout the years? the amount of annual water rights hasn't increase over the years? Table 4-12 MUNI WR	BECC	2. Confirmed	This is the amount avaiable at this time.	RJ	20-Jun-16
20	7-Apr-16	5-30	B. Confirm	Are the files of the scenarios presented in this report available? 5.1.7.3	BECC	2. Confirmed	Avaiable on request.	RJ	20-Jun-16
21	7-Apr-16	6-1	D. Change	This is the same exact text as the first paragraph as the introduction. Was this intended? or was some information left out?	BECC	1. Incorporated	Paragraph Changed	RJ	20-Jun-16
22	7-Apr-16	6-1	A. Incorporate/Add	BECC also provided FA for the study	BECC	1. Incorporated	Add to resources	DD	20-Jun-16
23	7-Apr-16	6-6	B. Confirm	Is the gpcd for South Padre Island just for it only or this data includes all communities served by LMWD?	BECC	2. Confirmed	The data only includes that for South Padre Island	RJ	20-Jun-16
24	7-Apr-16	7-25	C. Consider	Was there any water modeling done with software that we can get a copy of the files?	BECC	3. Noted	Pipeline sizing was not done with any water modeling software. All sizes were determined by using Excel.	RJ	20-Jun-16
25	7-Apr-16	9-5	B. Confirm	Which of the two desalination studies, the one conducted for BPUB or for the LMWD?	BECC	2. Confirmed	BPUB study. Clarification added to paragraph	RJ	20-Jun-16
26	13-Apr-16	7	D. Change	Change page numbers to match others in chapter 7	DD	1. Incorporated	Changed page numbers and TOC numbers	DD	20-Jun-16
27	7-Apr-16	13-3	C. Consider	Read attached document regarding BEIF and NADB	BECC	1. Incorporated	Descriptions updated/removed.	RJ	20-Jun-16
28	7-Apr-16	13-4	B. Confirm	TWDB has more than the SRF and the SWIFT funding opportunity. Are these the only ones that apply? Some of TWDB's funding I understand are for agricultural purposes (maybe lining of canals?) or maybe it's the purpose of this report to show just some of the funding opportunities?	BECC	7. Not in Scope	The funding opportunites used in the report were the ones applicable to our study and within the scope.	RJ	20-Jun-16
29	7-Apr-16	13-5	C. Consider	Read attached document regarding BECC	BECC	1. Incorporated	Table updated.	RJ	20-Jun-16
30	7-Apr-16	Chapter 13 Appendix	B. Confirm	Is this date still good? Doesn't this facility plan needs to be adopted first by the Regional Planning Group, and then it would be available for funding from the SWIFT Program? Spring 2015 Page 5	BECC	4. No Change, Designer Preference	The draft FIP was never finialized. Changes are noted and will be considered if the final FIP is completed. Yes, the facility plan would need to be adopted into the Region M plan to be avaialbe for SWIFT funding.	RJ	20-Jun-16

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Quality Control Review Comments

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Engineering Mgr:		**Stage of Design:**	Final Report

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31	7-Apr-16	Chapter 13 Appendix	C. Consider	There are some Technical Assistance funding available in order to pay for studies, PER, Environmental, etc. These funding opportunities are on the website. Page 5	BECC	4. No Change, Designer Preference	The draft FIP was never finialized. Changes are noted and will be considered if the final FIP is completed. Funding opportunities have been addressed in chapter 13.	RJ	20-Jun-16
32	7-Apr-16	Chapter 13 Appendix	D. Change	Adjust alignment beside graphic on page 9	BECC	4. No Change, Designer Preference	The draft FIP was never finialized. Changes are noted and will be considered if the final FIP is completed.	RJ	20-Jun-16
33	7-Apr-16	Chapter 13 Appendix	C. Consider	Up to 8 million dollars is allocated per project (WTP, WWTP, with significant environmental benefits) through the BEIF fund. This financial support is not annually.Please read attached documents. BECC section page 13	BECC	4. No Change, Designer Preference	The draft FIP was never finialized. Changes are noted and will be considered if the final FIP is completed. Funding opportunities have been addressed in chapter 13.	RJ	20-Jun-16
34		Routing	C. Consider	I have no comments aside from the interconnection between Brownsville PUB/SRWA and Laguna Madre Water District being located on Old Port Isabel Road (From FM 511 to SH 100, then continue along SH100 to Buena Vista Rd. At Buena Vista, transmission main should continue north toward FM 510 within Cameron County RMA right of way and continue across 2nd causeway to tie into future seawater desalination facility) (TXDOT/MPO planning should also follow this route for roadway via extension of FM 3248 to SH 100.) I strongly recommend interconnection at the intersection of FM 510 and Buena Vista Rd between East Rio Hondo WSC and LMWD. East Rio Hondo WSC's existing distribution system can eventually feed treated seawater further up the valley to minimize new distribution main costs. From our Weslaco visit, it sounds like south Pharr has low pressure issues on Military Hwy @ US 281 similar to low pressure issues on South Padre Island. A regional approach would be a good method to resolve problems and meet future water demands.	LMWD	3. Noted	Routing the pipe in the suggested location seems resonable. Further routing studies will take these comments into account and seek to utulize the future causeway. Using Buena Vista road to reach the causeway will be considered as well. Alternaitve addressed in chapter 7.	RJ	20-Jun-16
35	2-May-16	13, FIP, Page 5 table and section 2.1.3 (pg 10-11	B. Confirm	Please clarify that funding for EDAP is determined on a per biennium basis at the discretion of the Legislature. (it is not known until the lege takes it up each session whether or not funding will be allocated for this program)	TWDB	4. No Change, Designer Preference	The draft FIP was never finialized. Changes are noted and will be considered if the final FIP is completed. Funding opportunities have been addressed in chapter 13.	RJ	20-Jun-16
36	20-May-16	Appendix	A. Incorporate/Add	Please include a copy of the contract Scope of Work in the final Report, for example as an appendix.	TWDB	1. Incorporated	Added as Chapter 1 Appendix B	DD	20-Jun-16
37	20-May-16	Various (1-6)	D. Change	Please update throughout report, the current status of the final 2016 Region M Regional Water Plan that was adopted by the planning group November 2015 and subsequently approved by the TWDB Board December 2015; and the public hearing on the Draft 2017 State Water Plan was held April 18,2016. (example; page 1-6, Section 1.4.1)	TWDB	3. Noted	Status updated where necessary.	DD	20-Jun-16
38	20-May-16	Various (1-6)	D. Change	Please update throughout report the current status of the TWDB-funded study to include water quality delineations in the Gulf Coast Aquifer GAM. (example: page 1-6, Section 1.4.3)	TWDB	4. No Change, Designer Preference	The report conveys the current status of this study.	RJ	20-Jun-16
39	20-May-16	1.2.1 &1.5.2	A. Incorporate/Add	Please include a reference to the role of the TCEQ Watermaster in Sections 1.2.1 & 1.5.2 regarding operation of the Amistad-Falcon Reservoir System.	TWDB	1. Incorporated	Role added to sections	RJ	20-Jun-16
40	20-May-16	1.5.2	A. Incorporate/Add	Please clarify in Section 1.5.2 that all water rights in the Rio Grande have been adjudicated, and, that regional water planning requires drought-of-record firm-yield conversions of non-municipal water rights to municipal water rights in order to utilize for municipal water supplies.	TWDB	1. Incorporated	Sentence added to paragraph to clarify.	RJ	20-Jun-16
41	20-May-16	1-7 & 1-8	B. Confirm	Please clarify that the Arroyo Colorado is located in the Nueces-Rio Grande Basin and is not a tributary to the Rio Grande, pages 1-7 and 1-8.	TWDB	1. Incorporated	Paragraph Changed, no change to graphic.	RJ	20-Jun-16
42	20-May-16	1-11	A. Incorporate/Add	Please clarify that the Region M population and water demand projections presented in the Draft 2016 Plan and utilized in this study were the final projections approved by the TWDB Board for the 2016 Region M Plan, page 1-11. The list of Water User Groups (WUGs) and Wholesale Water Providers (WWPs) were also final versions.	TWDB	1. Incorporated	Language added to convey the data was approved by TWDB.	RJ	20-Jun-16
43	20-May-16	Task 1.7	A. Incorporate/Add	Please include missing documentation in the report of the deliverable for Task 1.7: discussion of the database of information and an interface/web site for stakeholders created for this project.	TWDB	1. Incorporated	Language added to Appendix A of chapter one to convey that all chapters are uploaded to the RGRWA supported website.	RJ	20-Jun-16
44	20-May-16	Appendix A (Ch.1)	A. Incorporate/Add	Please include missing documentation of all stakeholder, public, regulatory, and project committee meetings held for this project (meetings referenced in the Ch.I, Appendix A, Section 1.3).	TWDB	7. Not in Scope	Meeting minutes are avaiable upon request, not added to report	RJ	20-Jun-16

Water Division
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45	20-May-16	Appendix A (Ch.1)	B. Confirm	In Chapter 1, Appendix A, Section 1, the memo of understanding indicates that eastern Starr County is in the study area. Chapter 1 does not list eastern Starr Co as being included in the study area, please address this difference.	TWDB	2. Confirmed	As the study was developed, it was apparent the Starr county demands did not merit a new water supply, but can be added in the future if necessary.	RJ	20-Jun-16
46	20-May-16	Appendix A (Ch.1)	A. Incorporate/Add	Please clarify in Chapter 1, Appendix A, Section 3.2 that Advanced Water Conservation was assigned as a recommended water management strategy for every municipal WUG with a projected need.	TWDB	1. Incorporated	Section 3.2 adjusted accordingly.	RJ	20-Jun-16
47	20-May-16	2.1.2	A. Incorporate/Add	Please clarify in Section 2,1.2 that drought-year demands are actually "drought-of-record" demands; and that all planning groups hire a technical consultant to assist them with their regional water plan development. A statement should be added clarifying all non-municipal Water User Groups are defined by county or county/basin boundaries.	TWDB	1. Incorporated	Section now shows drought-of-record. All other comments already addressed in text.	RJ	20-Jun-16
48	20-May-16	2.2 and Appendix A (Ch.2)	B. Confirm	Please clarify in Section 2.2 and Chapter 2, Appendix A that population projections were based on the most recent 2010 U.S. Census.	TWDB	2. Confirmed	The population projections are based on Texas State Data Centers (TSDC)/Office of the State Demographer county-level population projections which uses 2010 census data.	RJ	20-Jun-16
49	20-May-16	2.1.2	A. Incorporate/Add	Please clarify in Section 2.1.2, paragraph 2; and in Table 2-1 that the "County-Other" municipal WUG is the compilation of all towns in a county with populations less than 500 and all remaining diffuse county populations (the criteria tor this category is not based on "unincorporated" status).	TWDB	1. Incorporated	Section adjusted to define "county-other".	RJ	20-Jun-16
50	20-May-16	2-3 & 2-4	D. Change	Please correct the names of three municipal county-other water user groups listed in Table 2-1, pages 2-3 and 2-4, as "unincorporated" misrepresents that unincorporated areas of less than 500 are included.	TWDB	1. Incorporated	Changed to "County-Other"	DD	20-Jun-16
51	20-May-16	2-10	A. Incorporate/Add	Please include missing decadal totals for municipal demand projections in Table 2-3, bottom of page 2-10.	TWDB	1. Incorporated	Added totals	DD	20-Jun-16
52	20-May-16	2-2	D. Change	Please correct the second equation in Figure 2-1, page 2-2: (Base Year GPCD) (Projected Decadal PC Savings) = (Projected Decadal GPCD),	TWDB	5. Need Additional Info/Direction	Figure Updated	DD	20-Jun-16
53	20-May-16	Table 2-2	A. Incorporate/Add	Please consider adding a footnote to Table 2-2 to clarify that the projected decadal GPCD is the Base Dry Year GPCD with anticipated per capita savings from implementation of the federal plumbing codes included	TWDB	1. Incorporated	Footnote added	DD	20-Jun-16
54	20-May-16	1-9	B. Confirm	Figure 1-6, page 1-9, indicates that transmission of municipal raw water supplies via irrigation district canal systems has an estimated 30% water loss; however, it appears that consideration of regional irrigation districts conveyance system water conservation projects were not included in Chapter 6, as part of the deliverables for Scope of Work (SOW) Task 3(2); please provide in the final report or clarify why this task was not performed. Please explain how the demand numbers were adjusted to reflect an "additional level of detail" (beyond the level used in the 2016 Regional Water Plan) for this study.	TWDB	3. Noted	Task 3.2 of the SOW refers to demand and availability and does not require agricultural focused strategies. In the gap analysis - demands were analyzed to reflect and additional level of deatil by their location and a centroid of demand was calculated for each decade.	RJ	20-Jun-16
55	20-May-16	Task 3.3	C. Consider	There are calculations showing the effects of implementing efficiency conservation measures for municipal demands. The effects of implementing efficiency conservation measures for Agricultural uses are also needed as required in Scope of Work Task 3, 3.	TWDB	3. Noted	Task 3.3 refers to gap analysis that was preformed in chapter 3 of the report. Agricultrural efficiency savings was not the focus or intent of this study.	RJ	20-Jun-16
56	20-May-16	4-25	A. Incorporate/Add	Please add some clarification to the statement in the Conclusion on page 4-25 which states there is sufficient water in the system to meet the municipal demand and the statement on the following page that indicates that municipal demands cannot be met with estimated municipal supply.	TWDB	1. Incorporated	Paragraph reorganized to better explain the term "municipal supplies".	RJ	20-Jun-16
57	20-May-16	Task 5	C. Consider	It is not clear that the three major options for regional water systems were investigated as required in the SOW Task 5. Description of the various strategies throughout the report don't necessarily identify whether the strategy is related to an independent system, a hybrid system or a sub-regional system. Please consider providing additional information/clarification statements in each strategy's summary.	TWDB	3. Noted	Each strategy would work in a hybrid, regional, or sub-regional stragety. Are discussion and recommendation of a regional system was developed in chapter 14 where the RGRWA is as the manager of the system for the LRGV. Options such as pipe routing and wellfield and sea water RO were looked at.	RJ	20-Jun-16
58	20-May-16	Chapter 1 TOC	A. Incorporate/Add	Please include the missing Appendix A in the Chapter 1 Table of Contents; and please consider revising the naming convention for report appendices to include the chapter number, as several chapters have an "Appendix A".	TWDB	1. Incorporated	Appendix A added to TOC.	DD	20-Jun-16
59	20-May-16	Various (1-6,1-7)	D. Change	Please consider correcting the many typographic errors to correct in report (examples: page 1-6, Section 1.4.2, line 7; page 1-7, paragraph!, line 2).	TWDB	1. Incorporated	Completed	DD	20-Jun-16

Water Division
Quality Control Review Comments

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60	20-May-16	Various (1-4)	A. Incorporate/Add	Please consider defining ail acronyms In report (example: 1-4, bullet 15).	TWDB	1. Incorporated	Completed	DD	20-Jun-16
61	23-May-16	Pressure Filters	C. Consider	Pressure Filters will likely end up being cartridge filters with no clarification. Should be a cost adjustment.	BM	4. No Change, Designer Preference	Design shows conservative system. Construction would be minimal to owner cost of project.	RJ	20-Jun-16
62	23-May-16	Table 8-13/14	C. Consider	Table 8-13 and 8-14 notes PE pipe for the wellfield collector lines. I'm not a fan, but there may not be a more viable option with the design parameters we have to live with.	BM	4. No Change, Designer Preference	Pipe material selection can be adjusted with future investagation. The cost of most pipeline materials in this size are fairly competitive with other materials.	RJ	20-Jun-16
63	23-May-16	Table 8-17/18	C. Consider	Table 8-17 & 8-18 also note the use of PE pipe. In this size range, I believe PVC is a much more viable option.	BM	4. No Change, Designer Preference	Pipe material selection can be adjusted with future investagation. The cost of most pipeline materials in this size are fairly competitive with other materials.	RJ	20-Jun-16
64	23-May-16	Ocean Desal	C. Consider	Ocean desal schematic shows the addition of lime. It is a mess everywhere you put it. We are currently using NaOH and CaCl for pH, alkalinity, and hardness adjustment. Lime definitely won't work in a static mixer.	BM	4. No Change, Designer Preference	Alternative processes can be evaluated in the future.	RJ	20-Jun-16
65	23-May-16	Cost	C. Consider	COST... When you look at the cost of the ocean desal versus the cost of the brackish desal and surface water plant with ASR, the ocean desal facility is not financially viable. The water quality of the brackish desal will have little to no DOC so the chloramine residual will maintain itself in the distribution system as it makes its way east and south to Brownsville. Just my opinion, make the cheapest water first and lay the pipeline. The ocean desal will come when the grant money shows up.	BM	4. No Change, Designer Preference	The initial ocean desalination plan at the ship channel is located there since BPUB intends to construct a plant and water flow to the east end of the pipeline is desirable.	RJ	20-Jun-16
66	23-May-16	Table 10-13	C. Consider	Table 10-13 shows costs per gal. Water rights alone for a firm yield are 1gal/dayX365day/yearX1Ft3/7.48galX1Acre/43,560Ft2X1.1(loss factor)X$2500/AF=$3.08. Looking down the road at the table cost values, I don't see how this includes the cost of water rights. If you ever make a large purchase of water rights, the market spikes and tightens. You can't use the 68% reduction value as we don't have a specific subdivision the Authority can pay for. I think we just need to state that entities wanting in on the Surface water Plant need to provide their own water rights or pay cash for the water rights up front or have them financed as part of their water take or pay. Not cheap though.	BM	4. No Change, Designer Preference	Water rights costs confirmed to be correct in table. These costs were included in the infastructure costs for the water treatment plant and are amoritzed over twenty years based on their inclusion with the plant costs.	RJ	20-Jun-16

CHAPTER 2 – DEMAND PROJECTIONS

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

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2.0 Demand Projections

2.1 INTRODUCTION

2.1.1 Purpose

The RGRWA is pursuing the Lower Rio Grande Regional Facility Plan (Regional Facility Plan) in order to provide preliminary engineering for a regional potable water system. In order to gauge the future need for potable water and size potential facilities, it is necessary to determine the predicted future demands for the region. The Regional Facility Plan uses data from the 2016 Rio Grande Regional Water Plan (Region M Plan) to provide initial estimates of future demands. This chapter provides an overview of Lower Rio Grande Valley's projected municipal, irrigation and other non-municipal water demands.

2.1.2 Demand Project Process

The Region M Water Plan is funded by the TWDB to meet state requirements for regional plans, updated on a 5 year cycle, and is aggregated with other regions to form the basis for the State Water Plan (SWP). The projections in the Regional Water Plans (RWPs) are intended to show drought-of-record demands, averaged over 10 year increments and projected over a 50-year planning horizon (2020-2070 in this cycle). The RWPs are developed by the regional planning groups, with technical assistance and guidance from both the TWDB staff and, in most cases, a consultant. Black & Veatch served as the consulting engineer for Region M in the fourth cycle of regional water planning, which culminates in the 2016 Region M Plan and the 2017 SWP.

The TWDB collaborated with the Region M Planning Group to develop demand projections for the region's users. Population and municipal demand were estimated for each county, city, and unincorporated areas for municipal water user group (WUG) projections. Other users, like Irrigation and Steam Electric Power Generation, were aggregated into geographical areas defined by county and river basin boundaries to form the demand projections for all other WUGs. The municipal WUG given the name "County-Other" is used to combine all the towns in a county with less than 500 people living there. TWDB estimated demands based on historical data and recent studies for each category, establishing a *base year* for each WUG. Subsequently, a rate of change was calculated for each WUG based on historic trends. Decadal estimates were projected using these criteria over the 50-year planning horizon.

The TWDB draft demand projections were distributed to the regional water planning groups for review and were revised where necessary, based on local knowledge. The Region M Planning Group agreed with the TWDB estimates for population and municipal, manufacturing, steam-electric, and livestock demands. Revisions were requested and adopted for irrigation and mining demands based on recent studies, and an alternative approach to estimating changes in irrigation demands were used. For the purposes of the Regional Facility Plan, information pertaining to the counties of Cameron, Hidalgo, and Willacy was included (the Lower Rio Grande Valley).

2.2 MUNICIPAL DEMANDS

As described previously, the TWDB generated draft projections for population and municipal demand for the Regional Water Planning Process. The population projections are based on Texas State Data Centers (TSDC)/Office of the State Demographer county-level population projections. Municipal water demands were calculated by applying the projected gallons per capita per day (GPCD) usages and the population projections for the planning period. The projected GPCD values include reductions in demands associated with replacement of existing fixtures and appliances with water-efficient ones and compliance with plumbing codes. A detailed description of the methodology can be found in Appendix A. Figure 2-1 presents the projection methodology.

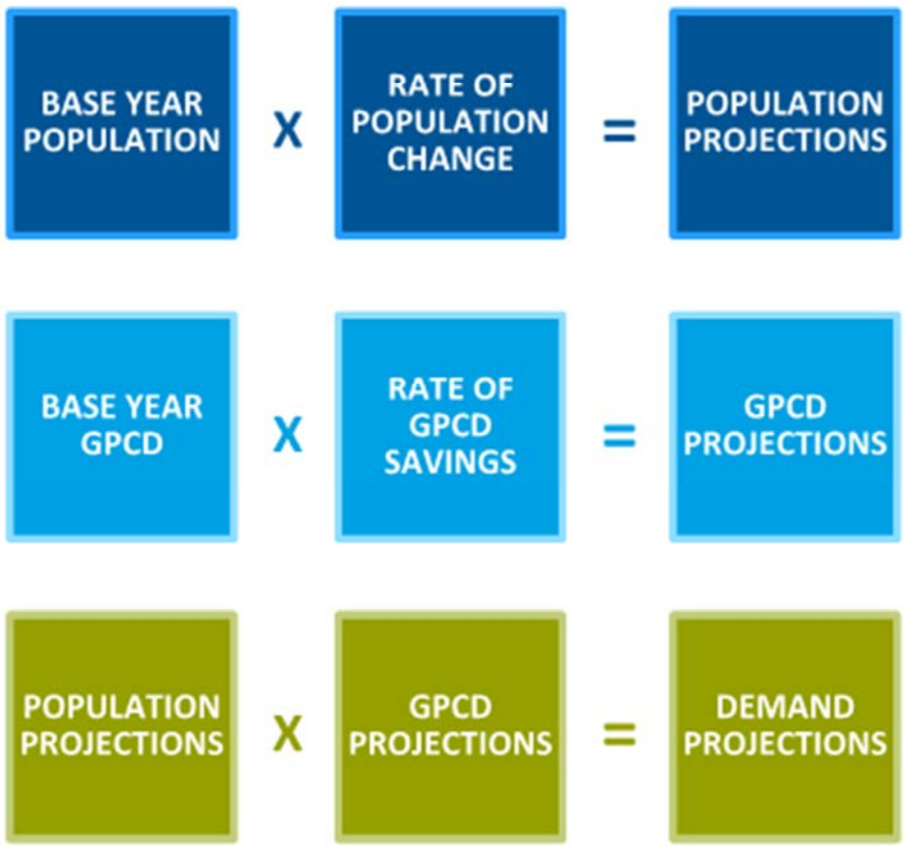


Figure 2-1 Population and Demand Projection Methodology

Table 2-1 provides population projections for the study area and Figure 2-2 provides the population projection by county in the Study area. The corresponding GPCD values are provided in Table 2-2 in 10 year increments as well as a magnitude comparison illustrated in Figure 2-3. The GPCD values for the region illustrate the tourist economy existent at the gulf coast and from seasonal residence throughout the area.

Table 2-1 Population Projections for Lower Rio Grande Valley (Rio Grande Regional Water Plan, 2016 Draft)

COUNTY	NAME	2020	2030	2040	2050	2060	2070
Cameron	Brownsville	211,200	251,288	291,955	335,755	380,809	426,990
Cameron	Combes	3,414	3,989	4,571	5,199	5,845	6,507
Cameron	County-Other	47,407	50,849	54,339	58,099	61,967	65,934
Cameron	East Rio Hondo WSC	27,435	32,052	36,736	41,782	46,971	52,291
Cameron	El Jardin WSC	15,099	17,640	20,218	22,995	25,851	28,779
Cameron	Harlingen	76,464	89,334	102,390	116,452	130,916	145,742
Cameron	Indian Lake	755	882	1,011	1,150	1,293	1,439
Cameron	La Feria	8,610	10,059	11,530	13,113	14,742	16,411
Cameron	Laguna Vista	3,676	4,294	4,922	5,598	6,293	7,006
Cameron	Los Fresnos	6,535	7,635	8,751	9,952	11,189	12,456
Cameron	Los Indios	1,277	1,492	1,710	1,945	2,187	2,434
Cameron	Military Highway WSC	19,462	22,737	26,060	29,639	33,320	37,094
Cameron	North Alamo WSC	482	563	645	733	824	917
Cameron	Olmito WSC	3,963	4,630	5,307	6,036	6,786	7,554
Cameron	Palm Valley	1,538	1,797	2,059	2,342	2,633	2,931
Cameron	Port Isabel	5,903	6,897	7,904	8,990	10,107	11,251
Cameron	Primera	4,799	5,607	6,427	7,309	8,217	9,147
Cameron	Rancho Viejo	2,874	3,358	3,848	4,377	4,920	5,477
Cameron	Rio Hondo	2,778	3,246	3,720	4,231	4,757	5,295
Cameron	San Benito	28,594	33,406	38,289	43,547	48,956	54,500
Cameron	Santa Rosa	3,388	3,958	4,537	5,160	5,800	6,457
Cameron	South Padre Island	3,321	3,880	4,447	5,057	5,685	6,329
Hidalgo	Agua SUD	52,129	64,729	77,379	90,055	102,731	115,054
Hidalgo	Alamo	23,259	28,881	34,525	40,181	45,837	51,335
Hidalgo	Alton	15,640	19,420	23,215	27,019	30,822	34,519
Hidalgo	County-Other	40,847	50,722	60,632	70,564	80,490	90,146
Hidalgo	Donna	20,021	24,860	29,719	34,587	39,456	44,189

COUNTY	NAME	2020	2030	2040	2050	2060	2070
Hidalgo	Edcouch	4,006	4,974	5,946	6,920	7,894	8,841
Hidalgo	Edinburg	97,711	121,329	145,041	168,800	192,560	215,659
Hidalgo	Elsa	7,173	8,906	10,647	12,391	14,136	15,831
Hidalgo	Hidalgo	14,191	17,621	21,065	24,516	27,967	31,322
Hidalgo	Hidalgo County MUD #1	6,858	8,516	10,181	11,848	13,516	15,138
Hidalgo	La Joya	5,050	6,271	7,496	8,724	9,952	11,146
Hidalgo	La Villa	2,480	3,079	3,681	4,284	4,887	5,474
Hidalgo	McAllen	164,597	204,382	244,325	284,348	324,372	363,284
Hidalgo	Mercedes	19,732	24,501	29,290	34,088	38,886	43,551
Hidalgo	Military Highway WSC	12,142	15,077	18,023	20,976	23,928	26,799
Hidalgo	Mission	97,658	121,263	144,962	168,708	192,455	215,541
Hidalgo	North Alamo WSC	148,138	183,945	219,894	255,915	291,937	326,957
Hidalgo	Palmhurst	3,303	4,102	4,904	5,707	6,511	7,292
Hidalgo	Palmview	6,919	8,592	10,271	11,953	13,636	15,272
Hidalgo	Penitas	5,580	6,928	8,282	9,639	10,996	12,315
Hidalgo	Pharr	89,220	110,785	132,437	154,131	175,826	196,918
Hidalgo	Progreso	6,979	8,666	10,359	12,056	13,753	15,403
Hidalgo	San Juan	42,906	53,277	63,690	74,123	84,556	94,699
Hidalgo	Sharyland WSC	45,075	55,970	66,908	77,869	88,829	99,485
Hidalgo	Sullivan City	5,071	6,297	7,528	8,761	9,995	11,194
Hidalgo	Weslaco	45,205	56,132	67,102	78,094	89,087	99,773
Willacy	County-Other	530	600	666	735	800	867
Willacy	East Rio Hondo WSC	36	40	45	49	54	58
Willacy	Lyford	2,981	3,360	3,723	4,110	4,485	4,851
Willacy	North Alamo WSC	6,088	6,862	7,604	8,395	9,159	9,908
Willacy	Raymondville	12,880	14,519	16,089	17,762	19,379	20,964
Willacy	San Perlita	655	738	817	902	985	1,065
Willacy	Sebastian MUD	2,094	2,360	2,615	2,887	3,150	3,408
	Total	**1,486,128**	**1,807,297**	**2,130,437**	**2,460,558**	**2,793,095**	**3,121,199**

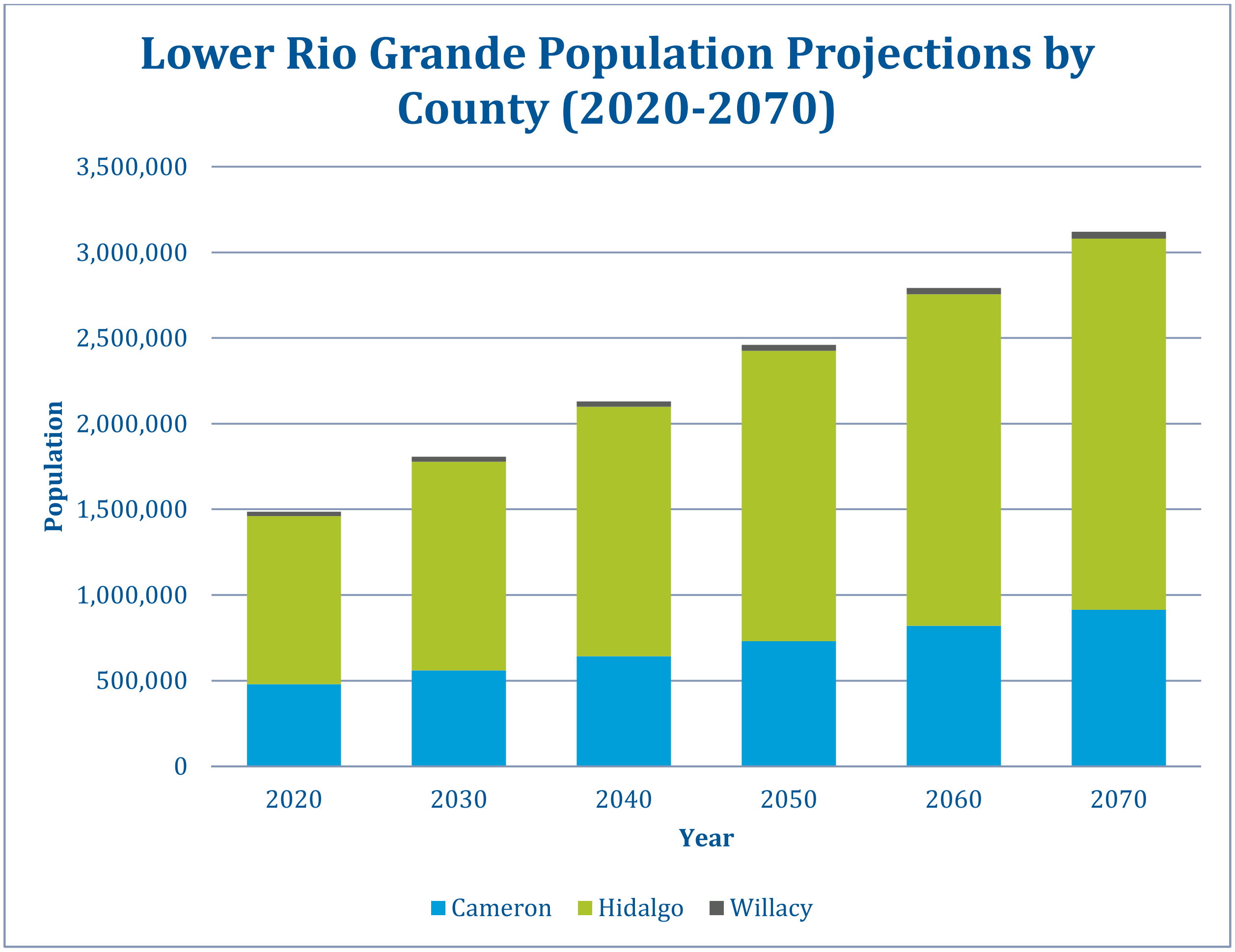


Figure 2-2 Lower Rio Grande Population Projections by County (2020-2070)

Table 2-2 Gallons Per Capita Per Day for Lower Rio Grande Valley (Rio Grande Regional Water Plan, 2016 Draft)*

COUNTY	NAME	BASE GPCD	2020	2030	2040	2050	2060	2070
Cameron	Los Fresnos	60	60	60	60	60	60	60
Cameron	Indian Lake	67	60	60	60	60	60	60
Willacy	Sebastian MUD	73	63	60	60	60	60	60
Cameron	Rio Hondo	75	65	62	60	60	60	60
Hidalgo	Hidalgo County MUD #1	82	74	71	70	70	69	69
Cameron	Primera	87	78	75	73	72	72	72
Cameron	Santa Rosa	88	78	73	70	69	69	69
Hidalgo	Edcouch	91	80	75	73	71	71	71
Cameron	Combes	94	84	80	77	76	76	76
Willacy	Lyford	96	87	83	81	80	79	79
Hidalgo	Progreso	101	92	89	88	87	87	87
Hidalgo	Penitas	103	96	94	93	93	92	92
Hidalgo	Agua SUD	104	96	93	91	91	90	90
Hidalgo	Palmview	104	96	93	92	91	91	91
Hidalgo	Sullivan City	106	96	92	90	89	88	88
Hidalgo	La Villa	108	99	95	93	92	92	92
Hidalgo	Pharr	108	99	96	95	94	94	93
Cameron	El Jardin WSC	109	101	98	96	95	95	95
Cameron	Los Indios	111	100	96	93	92	92	92
Hidalgo	Mercedes	111	101	96	94	93	93	93
Hidalgo	Elsa	112	101	96	94	93	93	92
Willacy	Raymondville	115	105	102	99	98	97	97
Willacy	County-Other	118	112	111	110	110	109	109
Hidalgo	County-Other	121	108	107	106	106	106	106
Cameron	San Benito	123	113	108	106	104	104	104
Hidalgo	La Joya	125	115	111	109	108	108	108
Hidalgo	Hidalgo	125	117	114	113	112	112	112
Hidalgo	Alton	125	118	116	115	114	114	114
Cameron	La Feria	126	117	113	111	110	110	109

COUNTY	NAME	BASE GPCD	2020	2030	2040	2050	2060	2070
Hidalgo	Donna	127	116	112	110	109	109	109
Hidalgo	Edinburg	128	120	117	116	115	115	115
Willacy	East Rio Hondo WSC	132	124	122	120	119	119	119
Cameron	East Rio Hondo WSC	132	124	122	120	119	119	119
Hidalgo	Alamo	133	124	121	119	118	118	118
Hidalgo	San Juan	137	128	125	123	122	122	122
Cameron	Military Highway WSC	144	135	132	130	129	129	129
Hidalgo	Military Highway WSC	144	135	132	130	129	129	129
Willacy	North Alamo WSC	153	145	142	140	140	140	139
Cameron	North Alamo WSC	153	145	142	140	140	140	139
Hidalgo	North Alamo WSC	153	145	142	140	140	140	139
Cameron	County-Other	155	146	142	140	138	138	138
Cameron	Brownsville	162	153	149	147	146	145	145
Hidalgo	Weslaco	165	155	152	150	149	149	149
Cameron	Harlingen	168	158	154	152	151	150	150
Hidalgo	Sharyland WSC	169	159	155	153	152	152	152
Cameron	Olmito WSC	175	165	161	158	157	157	157
Cameron	Palm Valley	176	165	161	158	157	156	156
Hidalgo	Mission	193	185	182	180	180	179	179
Cameron	Port Isabel	211	201	196	194	192	192	192
Hidalgo	McAllen	220	210	206	204	203	203	203
Hidalgo	Palmhurst	259	252	250	249	249	249	248
Cameron	Rancho Viejo	267	259	256	255	254	254	254
Willacy	San Perlita	330	319	314	312	311	311	311
Cameron	Laguna Vista	599	591	588	587	586	586	586
Cameron	South Padre Island	877	868	864	862	860	860	860

*Projected decadal GPCD is the Base Dry Year GPCD with anticipated per captai savings from implementation of federal plumbing codes included.

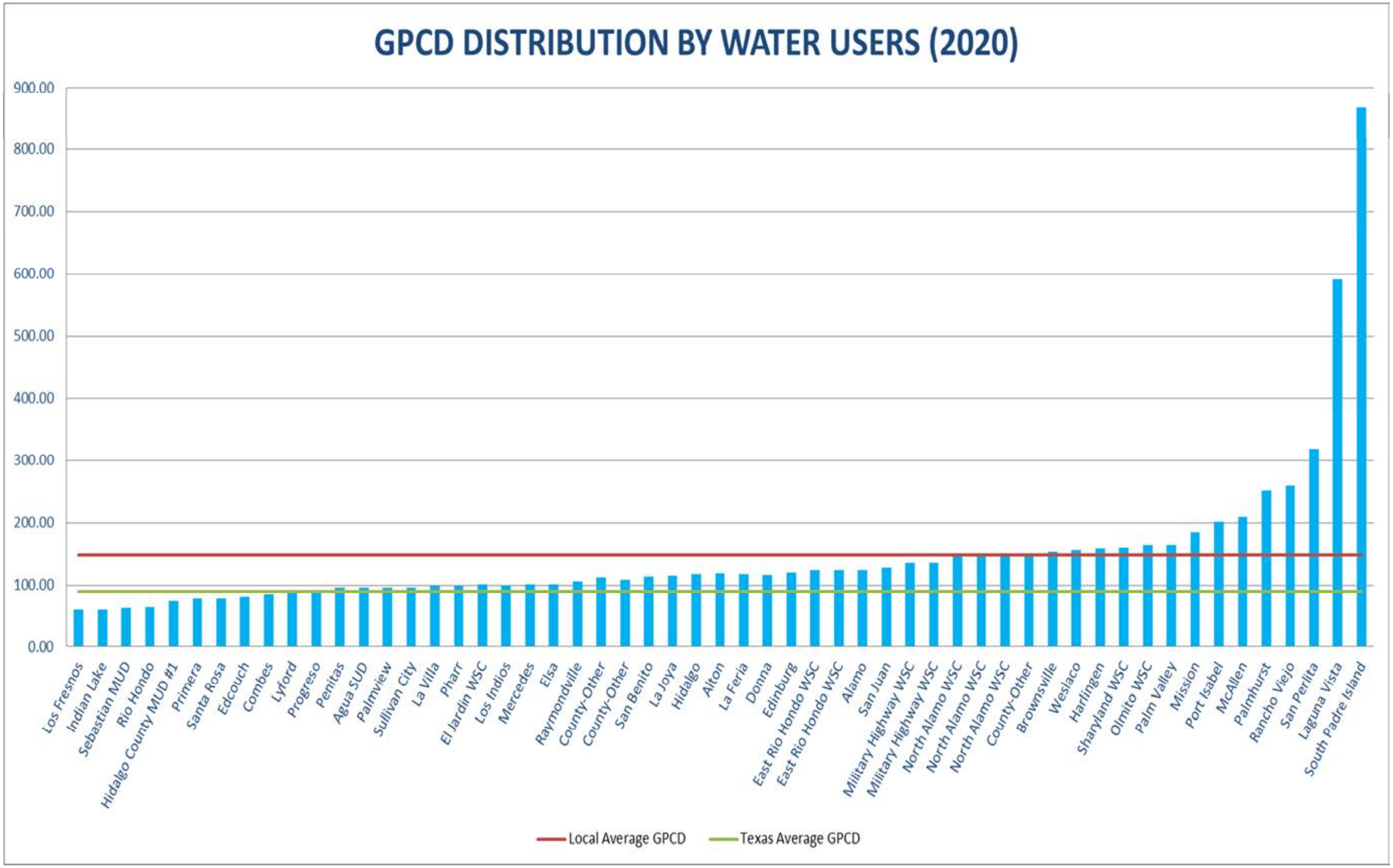


Figure 2-3 GPCD Distribution by Water Users (2020)
National Average: 88 GPCD; Local Average: 148 GPCD; Texas Average: 90 GPCD

Municipal water demands are calculated by multiplying the per person water use with the forecasted population. These demands are calculated in ten year increments for the 50 year planning horizon. Table 2-3 below presents the demand projections and the associated increase from 2020 until 2070. Figure 2-4 illustrates the demand trends in the study area by county.

Table 2-3 Municipal Demand Projections for Lower Rio Grande Valley (Rio Grande Regional Water Plan, 2016 Draft) (AF/YR)

COUNTY	NAME	2020	2030	2040	2050	2060	2070	DEMAND INCREASE
Willacy	East Rio Hondo WSC	6	6	7	7	8	8	2
Cameron	Indian Lake	51	60	68	78	87	97	46
Willacy	County-Other	67	75	83	91	99	107	40
Cameron	North Alamo WSC	79	90	102	115	129	144	65
Willacy	Sebastian Mud	149	159	176	195	212	230	81
Cameron	Los Indios	144	161	179	201	226	251	107
Cameron	Rio Hondo	204	224	251	285	320	356	152
Willacy	San Perlita	235	260	286	315	344	371	136
Willacy	Lyford	291	314	338	368	400	432	141
Cameron	Santa Rosa	295	325	358	400	448	498	203
Cameron	Palm Valley	285	324	365	411	462	514	229
Cameron	Combes	322	358	397	445	498	554	232
Hidalgo	La Villa	275	328	385	443	504	564	289
Hidalgo	Edcouch	358	419	484	554	630	705	347
Cameron	Primera	422	472	526	590	661	735	313
Cameron	Los Fresnos	440	514	589	669	752	838	398
Hidalgo	Sullivan City	544	647	755	869	989	1,107	563
Hidalgo	Hidalgo County Mud #1	570	682	801	923	1,049	1,174	604
Hidalgo	Penitas	603	732	865	1,001	1,139	1,275	672
Cameron	Olmito WSC	732	835	941	1,063	1,192	1,327	595
Hidalgo	La Joya	652	783	919	1,060	1,207	1,351	699
Hidalgo	Progreso	722	868	1,020	1,177	1,339	1,498	776
Willacy	North Alamo WSC	987	1,091	1,197	1,315	1,432	1,548	561
Hidalgo	Palmview	743	897	1,056	1,220	1,388	1,554	811
Cameron	Rancho Viejo	835	965	1,099	1,246	1,399	1,557	722
Hidalgo	Elsa	811	963	1,121	1,289	1,466	1,641	830

COUNTY	NAME	2020	2030	2040	2050	2060	2070	DEMAND INCREASE
Cameron	La Feria	1,126	1,274	1,432	1,613	1,809	2,012	886
Hidalgo	Palmhurst	932	1,149	1,369	1,591	1,813	2,030	1,098
Willacy	Raymondville	1,522	1,652	1,784	1,944	2,115	2,286	764
Cameron	Port Isabel	1,327	1,517	1,714	1,936	2,174	2,419	1,092
Cameron	El Jardin WSC	1,704	1,931	2,172	2,447	2,744	3,052	1,348
Hidalgo	Military Highway WSC	1,841	2,231	2,629	3,039	3,460	3,873	2,032
Hidalgo	Hidalgo	1,859	2,254	2,662	3,079	3,505	3,923	2,064
Hidalgo	Alton	2,071	2,524	2,990	3,464	3,943	4,413	2,342
Hidalgo	Mercedes	2,223	2,648	3,091	3,558	4,049	4,531	2,308
Cameron	Laguna Vista	2,435	2,831	3,236	3,676	4,130	4,597	2,162
Cameron	Military Highway WSC	2,950	3,364	3,802	4,294	4,818	5,360	2,410
Hidalgo	Donna	2,610	3,126	3,660	4,219	4,802	5,375	2,765
Cameron	South Padre Island	3,228	3,755	4,292	4,875	5,478	6,098	2,870
Cameron	San Benito	3,607	4,053	4,529	5,088	5,705	6,346	2,739
Hidalgo	Alamo	3,231	3,909	4,607	5,326	6,064	6,787	3,556
Cameron	East Rio Hondo WSC	3,820	4,366	4,941	5,582	6,261	6,965	3,145
Cameron	County-Other	7,749	8,100	8,494	8,992	9,569	10,176	2,427
Hidalgo	County-Other	4,952	6,075	7,232	8,393	9,553	10,691	5,739
Hidalgo	Agua SUD	5,590	6,736	7,925	9,152	10,414	11,652	6,062
Hidalgo	San Juan	6,152	7,448	8,782	10,154	11,561	12,940	6,788
Hidalgo	Weslaco	7,873	9,551	11,271	13,040	14,852	16,625	8,752
Hidalgo	Sharyland WSC	8,026	9,722	11,460	13,252	15,094	16,896	8,870
Hidalgo	Pharr	9,923	11,933	14,021	16,183	18,415	20,607	10,684
Cameron	Harlingen	13,546	15,429	17,400	19,636	22,035	24,516	10,970
Hidalgo	Edinburg	13,113	15,899	18,772	21,714	24,721	27,667	14,554
Hidalgo	Mission	20,212	24,704	29,290	33,954	38,684	43,305	23,093
Hidalgo	North Alamo WSC	24,015	29,240	34,598	40,064	45,625	51,069	27,054
Cameron	Brownsville	36,092	41,913	47,986	54,797	62,040	69,520	33,428
Hidalgo	McAllen	38,728	47,219	55,875	64,722	73,748	82,563	43,835
	Total	**243,279**	**289,105**	**336,144**	**386,114**	**437,561**	**488,730**	**245,451**

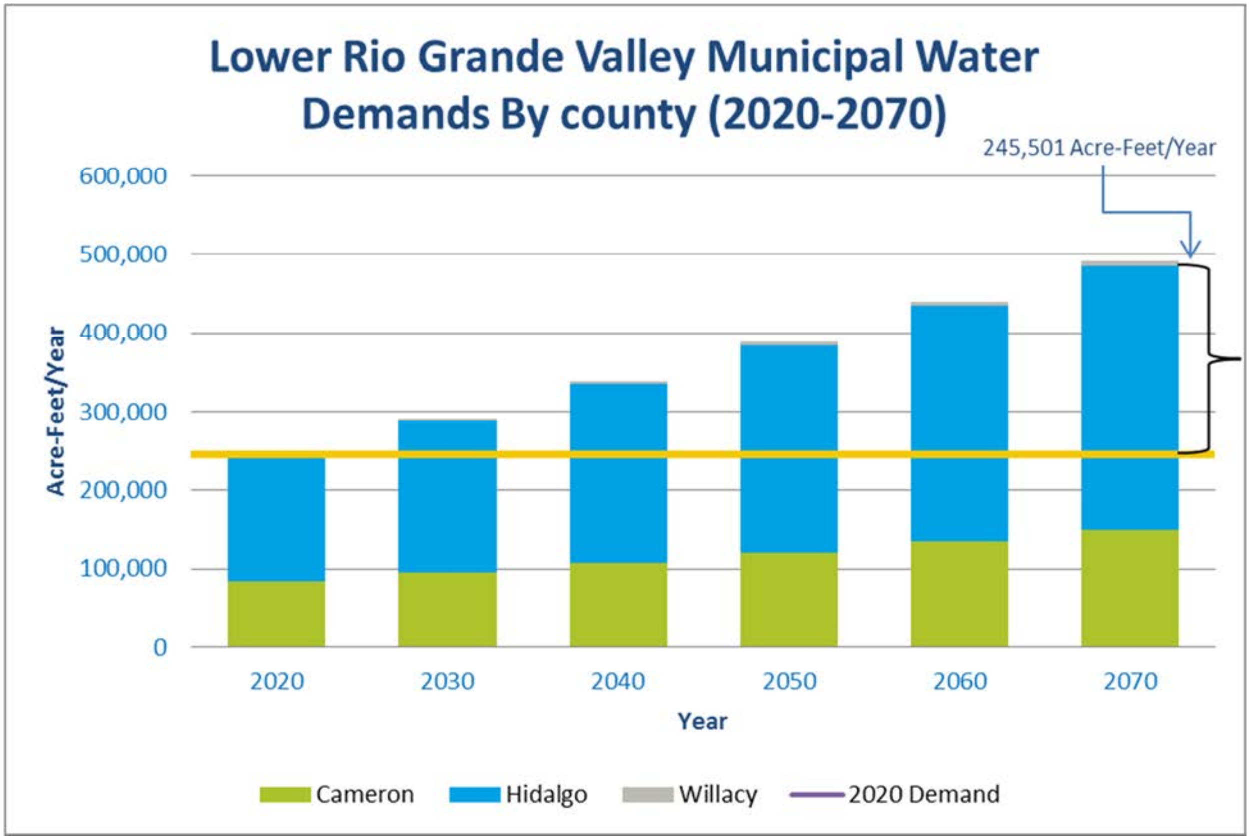


Figure 2-4 Lower Rio Grande Valley Municipal Water Demands by County (2020-2070)

2.3 IRRIGATION DEMANDS

Irrigation use within Region M and the study area is largely dependent on available supply from the Amistad-Falcon reservoir system and weather. Irrigation water rights on the Rio Grande are not guaranteed in their full amount in a drought, but are curtailed based on an allocation system when the Amistad-Falcon reservoir system falls to a certain storage level. It is important for regional planning that irrigation estimates make a distinction between irrigation water use, irrigation rights, and irrigation water demand. In most actual drought years, farmers may respond to limited water supplies by selecting crops which require less water or no 'applied' water (dry land farming). Similarly, citrus and pecan trees can tolerate minimal water for a limited time period, but their true demand for a productive crop is greater than the minimum water required to survive. Since the RWP process permits only a single demand scenario and is intended to represent a drought year, irrigation demand is best developed assuming a dry year in which irrigators do not implement water management strategies because of limited surface water availability. These assumptions produce the worst-case demand scenario for the planning process.

The base year is established by aggregating the maximum irrigation water use year for each county in TWDB water use estimates from 2005 to 2009, thus assembling a new representative demand year. A summary of the TWDB base year estimates, the average use, and the 5-year maximum use are shown in Table 2-4.

Table 2-4 Summary of TWDB Irrigation Base-Year Demand Estimates (AF/YR)

COUNTY	2005	2006	2007	2008	2009	5-YEAR AVERAGE	5-YEAR MAXIMUM
Cameron	298,503	308,571	322,976	314,353	314,597	311,800	322,976
Hidalgo	513,348	530,395	519,770	610,576	616,600	558,138	616,600
Willacy	57,532	57,000	57,457	59,300	59,700	58,198	59,700
Total	**869,383**	**895,966**	**900,203**	**984,229**	**990,897**	**928,136**	**999,276**

In addition to revising the methods for estimating the base year demand, the RWP stakeholders had concerns about previous methods used for estimating the rate of change. Specifically, the approach used to estimate irrigation demands had been based on the 2001 Regional Water Plan, and does not reflect the data and trends of the last 15 years.

Table 2-5 Irrigation Demand Projections by County (AF/YR)

COUNTY	2020	2030	2040	2050	2060	2070
Cameron	355,962	339,470	322,622	305,522	288,601	288,601
Hidalgo	639,676	609,754	577,457	540,797	502,563	502,563
Willacy	69,253	69,074	68,936	68,814	68,741	68,741
Total	**1,064,891**	**1,018,298**	**969,015**	**915,133**	**859,905**	**859,905**

Irrigation demands for the Region M plan were calculated using rigid and broad criteria that will not be re-evaluated for the specific irrigation water users in the Lower Rio Grande Valley (Table 2-4). Irrigation demands are not addressed further because they are not a significant focus of this study.

2.4 MISCELLANEOUS DEMANDS

The regional water planning groups work with the TWDB to evaluate current demands and project future water demands for each category of water user group (WUG); municipal, irrigation, livestock, steam-electric power generation, manufacturing, and mining. For this study the water demands for manufacturing, mining, steam-electric power generation and livestock are grouped into a miscellaneous category. Similarly to irrigation demands, the miscellaneous demands were calculated using broad criteria that will not be re-evaluated in this study. Since the focus of this study is to provide municipal drinking water demand, projections for miscellaneous use are not provided.

Estimates and projections for other non-municipal categories were developed and provided by TWDB with inputs from representatives of regional planning groups. In general, the methodology uses an initial base year estimate developed by gathering available data, assessing their quality, adjusting them as necessary, and reviewing their comparability among counties. A rate of change is then applied to the base year estimate for the planning period, resulting in the projections.

Figure 2-5 Miscellaneous Demand Projection Methodology

A detailed description regarding the methodology for each of the miscellaneous categories (manufacturing, mining, steam-electric power generation and livestock demands) is provided in the 2016 Draft Region M plan.

Table 2-6 Miscellaneous Demand Projections by County (AF/YR)

COUNTY	CATEGORY	2020	2030	2040	2050	2060	2070
Cameron	Manufacturing	4,708	5,111	5,510	5,856	6,324	6,829
	Mining	264	277	191	126	61	28
	Steam Electric Power Generation	1,523	1,780	2,094	2,477	2,944	3,428
	Livestock	334	334	334	334	334	334
Hidalgo	Manufacturing	5,461	5,909	6,357	6,756	7,276	7,836
	Mining	2,844	3,620	4,198	4,819	5,532	6,434
	Steam Electric Power Generation	14,151	16,545	19,462	23,018	27,354	32,507
	Livestock	830	830	830	830	830	830
Willacy	Manufacturing	136	136	136	136	136	136
	Mining	49	51	38	28	18	12
	Steam Electric Power Generation	0	0	0	0	0	0
	Livestock	261	261	261	261	261	261
	Total	**30,561**	**34,854**	**39,411**	**44,641**	**51,070**	**58,635**

2.5 SUMMARY

This section summarizes the water demand projections; regional demand projections for 2020 are shown in Figure 2-6.

- Though municipal demands in 2020 are a fifth of anticipated demands accordingly to the data compiled and calculated by the Region M Planning Group and the TSDC, these demands will double by 2070, to just over 500,000 AF/YR. To meet this demand 50,000 AF/YR of supply needs to be added each decade over the planning horizon.
- Municipal demands are dispersed throughout the valley as can be seen in Figure 2-7. However, the largest increases in demand are located in the metropolitan areas of McAllen, Harlingen and Brownsville. Regional supply projects in these areas may be an economic alternative, and will be evaluated further in subsequent chapters. Smaller demands will also be considered for alternative supply strategies and may be included in regional solutions based on their proximity to the projects.
- Irrigation changes in the study area are caused by many factors including urbanization of farmlands, farm subsidies, available work force, extreme weather, pricing and market conditions. A separate study on irrigation districts and supplies is ongoing and will further address changes in irrigation demands. Municipal demand and irrigation demand completely dominate the other water user groups in the Lower Rio Grande Valley. During the study period it is expected that municipal demands will increase, and irrigation demands will decrease both as a result of increasing cost pressure on water and because of urbanization of irrigable land. Also irrigated areas are expected to decline with expansion of urbanization into agricultural farmlands.

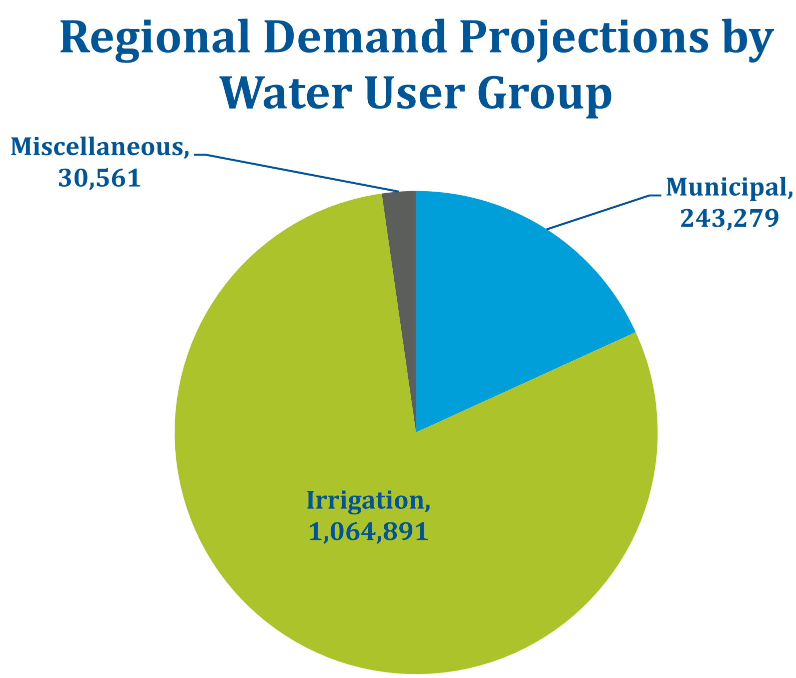
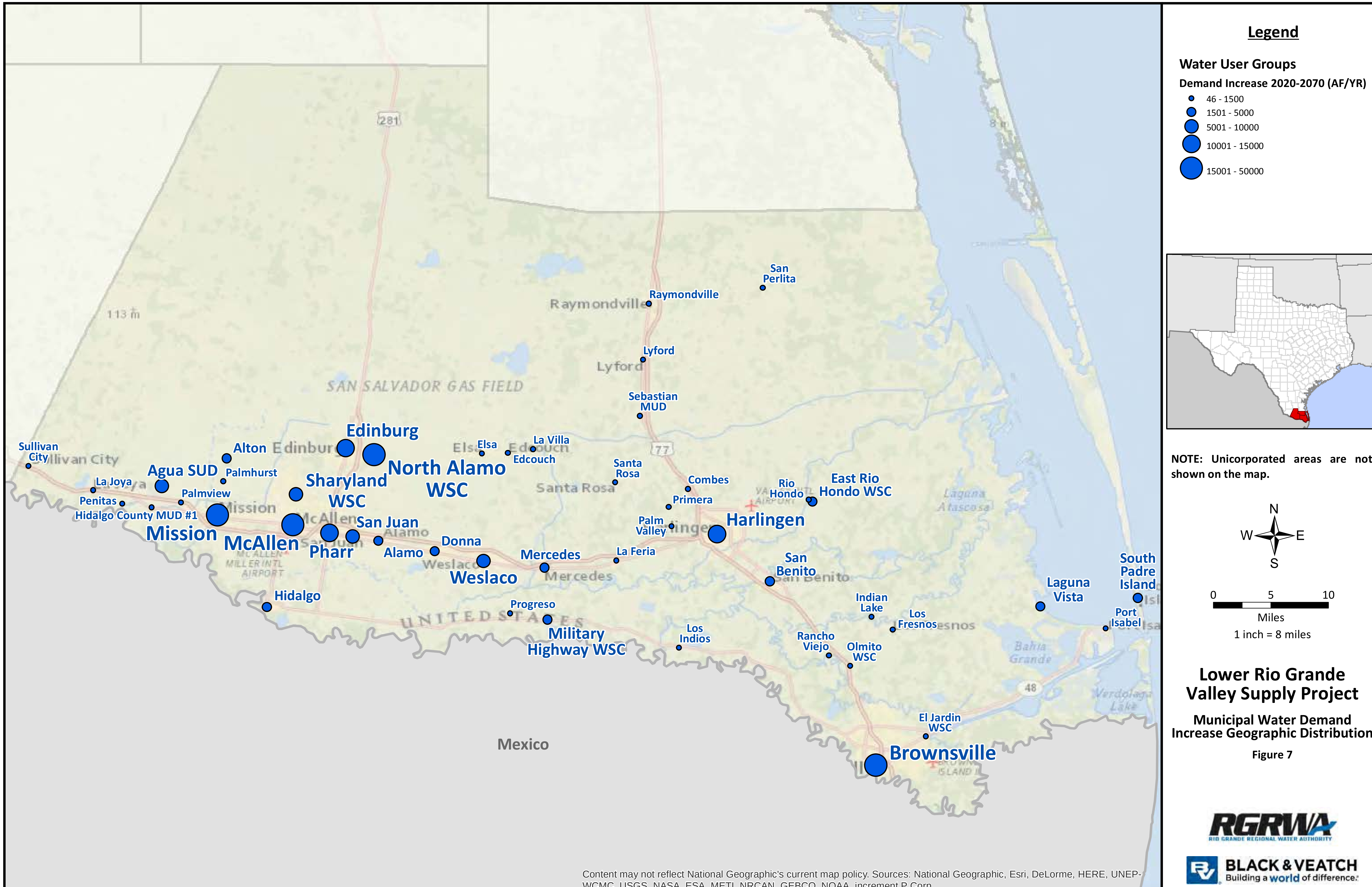


Figure 2-6 Regional Demand Projections by Water User Group (Year 2020)



C:\Projects\181092.2050_LowerRioGrandeValley\02GISData\01MapDocuments\Figure7_DemandIncrease.mxd A. Black June 09, 2015

Appendix A. Demand Methodology

Projection Methodology – Draft Population and Municipal Water Demands

1 Table of Contents

2 Population

The population projection methodology takes place in two steps: first, projections at the county level and then projections at the city/utility level.

2.1.1 County Population Projections

Draft county population projections are based on Texas State Data Center (TSDC)/ Office of the State Demographer county-level population projections. Such projections are based on recent and projected demographic trends, including the birth rates, survival rates, and net migration rates of population groups defined by age, gender and race/ethnicity.

The TSDC develops county-level population projections from 2011 to 2050 under three migration scenarios:

1) no net migration (natural growth only),
2) net migration rates of 2000-2010 ("full-migration scenario"), and
3) 2000-2010 migration rates halved ("half-migration scenario").

The State Data Center strongly recommends use of the half-migration scenario for long-term-planning. For each county, the draft projection is based on the half-migration scenario as the default, but alternatives (full-migration scenario or a composite of the scenarios) were chosen in select instances where a different scenario was more reflective of anticipated growth patterns.

While the TSDC's projections extend to 2050, the 2017 State Water Plan will require projections to 2070. TWDB staff has extended the projections to 2060 and 2070 by using the trend of average annual growth rates of the 2011-2050 TSDC projections. In 60 counties, the TSDC-projected population show a decline sometime between 2011 and 2050. For these counties, staff held the county population at its highest point prior to the decline for the following reasons:

1) Small Impact - the difference between holding the populations of these 60 counties constant or projecting continued decline in 2050 is 21,987, or 0.05 percent of the state-wide population of over 41 million. The largest county-specific difference between constant population and declining population is 2,030, the smallest is 17, and the average county difference is 366;
2) Constant System Requirements - projected population decline is often a decline in the number of people per household rather than a reduction in the number of connections that a water system must serve. The water systems must continue to have the capability to serve the customer connections regardless of population.

2.1.2 Water User Group Population Projections

The regional and state water plans require population projections for individual Municipal Water User Groups.

Water User Group Criteria

Municipal water user groups in the regional planning process include:

- Cities with a 2010 population greater than 500;
- Select Census Designated Places, such as military bases and in counties with no incorporated cities;

- Utilities (areas outside the places listed above) providing more than 280 acre-feet of municipal water per year;
- Collections of utilities with a common water supplier or water supplies (Collective Reporting Units); and
- Remaining rural, unincorporated population summarized as "County-Other"

The criterion for including only cities with populations greater than 500 has been used throughout the regional planning process, beginning with the 2001 regional water plans and the 2002 state water plan. Smaller cities are included in the aggregated "County-Other" water use, but are not separately delineated because many such small cities may not have a public water system or may not be the owner of the system. Regional planning groups do have the option of combining smaller water systems/cities into a collective water user group when the systems share a similar source or provider and are anticipated to coordinate in meeting their future water needs. In addition, regions may request the inclusion of cities or systems below the threshold criteria as distinct water user groups. This can be accomodataed in the online planning database.

2.1.2.1 Overlapping Boundaries

The previous section noted various criteria for water user groups. In some cases, the boundaries of qualifying water user groups may overlap. Examples and the method of population and water use allocation include:

•**<u>City utility serving beyond city limits</u>** - The service area boundary of a city-owned water utility may extend beyond the city boundaries; in such cases, the population and associated water use outside of the city limits are allocated not to the city but to the County-Other water user group.

•**<u>Non-city utility serving city residents</u>** – A non-city water utility may provide water directly to residents of a city that qualifies as a water user group; in such cases, the population and associated water use in the shared area are attributed to the city rather than the non-city utility in the regional water plan. Additional information regarding these shared populations and demands can be provided to the RWPGs and their technical consultants.

2.1.3 Projection Methodology

Projections for these individual water user groups are developed by allocating growth from the county projections down to the cities, utilities, and rural areas. The methods of allocating future populations from the county to the sub-county areas include:

1) Share of Growth - applying the water use group's historical (2000-2010) share of the county's growth to future growth;
2) Share of Population - applying the water user group's historical (2000-2010) share of the county population to projected county population; and
3) Constant Population - applied to military bases, and other water user groups that had population decline between 2000 and 2010 in a county with overall population growth.

The sum of all water user group populations within a county is reconciled to the total county projection prior to the finalization of draft projections.

3 Municipal Water Demands:

Draft municipal water demand projections utilize the population projections and a per-person water use volume for each city, water utility and rural area (County-Other). The draft projections will include 2011 per-person water use values (Gallons Per Capita Daily or GPCD) as the initial 'dry-year' water use estimate. Staff then applies future anticipated reductions in water use due to natural replacement rates for adoption of water-efficient fixtures and appliances required by law.

For each municipal water user group, the 2011 GPCD, minus the incremental anticipated savings for each future decade due to water-efficient fixtures/appliances, is multiplied by the projected population to develop the municipal water demand projections.

3.1.1 2011 Gallons Per Capita Daily (GPCD)

The 2011 GPCD for each water user group is calculated by:

- Calculating the net water use of each water system surveyed annually by the TWDB (total intake volume minus sales to large industrial facilities and to other public water suppliers),
- Allocating all or portions of the system net use and applicable estimates of non-system municipal water use (private groundwater) to the planning water user groups (city boundaries or water utility service areas), and
- Dividing the total water use allocated to a water user group by 365 and by the 2011 population estimate.

For city water user groups, the 2011 population estimates from the U.S. Census Bureau were used. Historically, the July 1st population estimates from the Texas State Data Center (TSDC) have been used in GPCD calculation, however because the TSDC had not released their 2011 population estimates by January 2013, staff used the available Census Bureau estimates. For non-city utility water user groups (Districts, Water Supply Corporations, and Investor Owned Utilities), the population reported in the annual water use survey was utilized, with an alternative calculation based on the reported number of connections if necessary.

3.1.2 Minimum GPCD Values

When calculating the base (2011) or projected GPCD values, TWDB staff applied a minimum of 60 GPCD. The minimum value of 60 GPCD is based upon several recent studies: *Analysis of Water Use in New Single-Family Homes*[1] and an internal TWDB report, *The Grass Is Always Greener...Outdoor Residential Water Use In Texas*, analyzing the percentage of Texas residential water used outside of the home.[2] The single-family home study studied the average per-person water use for:

1) Pre-1995 Homes (62.18 GPCD),
2) Standard New Homes built after 2001 (44.15 GPCD),
3) Standard new homes retrofitted with high-water-efficient fixtures and appliances (39.0 GPCD), and
4) New WaterSense Homes built with the best available technology for water conservation (35.6 GPCD).

[1] *Analysis of Water Use in New Single Family Homes*, Prepared by William B. DeOreo of Aquacraft Water Engineering & Management for The Salt Lake City Corporation and the U.S. Environmental Protection Agency, 2011

[2] *The Grass Is Always Greener...Outdoor Residential Water Use In Texas*, Sam Marie Hermitte and Robert Mace, Technical Note 12-01, 2012

With the assumed replacement of fixtures and appliances over the next 50 years, the indoor per-person water use of the Standard New Home Retrofitted (39.0 GPCD) can be expected under existing standards. However, this is only indoor use and the single-family home study found that there was no statistical difference in outdoor water use between types of housing.

The TWDB study of outdoor water use in Texas estimated that on average 31 percent of total residential water use is outdoor water use. Utilizing this average outdoor water use percentage (31 percent) and the indoor water use (69 percent) of 39 GPCD for retrofitted new homes produces a total residential GPCD of 56.5 GPCD. While some municipal water user groups may remain primarily residential, any water use by the local government or commercial water users will contribute some to the water user groups average GPCD. For this reason, staff rounded the minimum GPCD to 60.

3.1.3 Water Efficiency Savings

Federal standards on plumbing fixtures, dish washers, and clothes washers sold in the U.S. have recently been upgraded with potential savings due to installation of more water efficient units comprising a small, although significant, portion of total water use. Table 1 summarizes the expected savings from adoption of the standards, which apply by Federal Law to the fixtures and appliances sold in the U.S. for each of the effective date years shown. Years shown in Table 1 for each type of fixture/washer are the legislated beginning of sales of those items, with the associated water savings levels mandated by law.

Details concerning each of the pertinent pieces of legislation may be found at the websites noted in Table 2.

Anticipated savings due to water-efficient fixtures/appliances include:

1) Toilets and Showerheads – savings of 16 GPCD;
2) High-Efficiency Toilets – savings of 1.63 GPCD;
3) Dishwashers – savings of 1.61 to 1.90 GPCD; and
4) Clothes Washers – 6.45 GPCD

Table 1. Summary of Water Efficiency Savings and Implementation Years

	1995	2007	2010	2013	2015	2018
Item						
Plumbing Fixtures, 1991 (toilets, showerheads)	***Combined savings:*** **16 GPCD**					
High-Efficiency Toilet, 2009			***Savings:*** **0.32 gal/flush or 1.63 GPCD**			
Dishwashers			***Standard:*** **6.5 gal/cycle** ***Savings*:*** **7.5 gal/cycle or 1.83 GPCD**	***Standard:*** **5 gal/cycle** ***Savings:*** **9 gal/cycle or 1.93 GPCD**		
Front Load Clothes Washers		***Standard:*** **9.5 gal/cycle** ***Savings:*** **17.5 gal/cycle or 5.23 GPCD**			***Standard:*** **4.7 gal/cycle** ***Savings:*** **22.3 gal/cycle or 6.67. GPCD**	
Top Load Clothes Washers		***Standard:*** **9.5 gal/cycle** ***Savings:*** **17.5 gal/cycle or 5.23 GPCD**			***Standard:*** **8.4 gal/cycle** ***Savings:*** **18.6 gal/cycle or 5.56 GPCD**	***Standard:*** **6.5 gal/cycle** ***Savings:*** **20.5 gal/cycle or 6.13 GPCD**

*Savings for dishwashers and clothes washers are calculated versus historical average usage noted below: Dishwashers: 14 gal/cycle, Clothes Washers: 27 gal/cycle (minor use of front load clothes washer previous to 2007). GPCD savings based on assumed 2.75 people per household, 215 dishwasher loads/yr, and 300 clothes washer loads/yr.

Table 2. Background Information on Federal Standards on Water/Energy Efficiency

Item	Effective Year	Website
Plumbing Fixtures	1995	http://www.gao.gov/new.items/rc00232.pdf
High-Efficiency Toilets	2010-2014	www.capitol.state.tx.us (search House Bill 2667, 81st Legislature (Regular) 2009)
Dishwashers	2010	http://www1.eere.energy.gov/buildings/appliance_standards/residential/pdfs/74fr16040.pdf
Dishwashers	2013	http://www1.eere.energy.gov/buildings/appliance_standards/residential/dishwashers.html (see section on Energy Conservation Standards)
Clothes Washers	2007	http://www1.eere.energy.gov/buildings/appliance_standards/residential/pdfs/rcw_dfr_tsd_ch3.pdf (see section 3.7.2)
Clothes Washers	2015, 2018	http://www1.eere.energy.gov/buildings/appliance_standards/residential/clothes_washers.html (see section on Energy Conservation Standards)

3.1.4 Plumbing Fixtures Efficiency Savings, 1991 ("Plumbing Code Savings")

The suggested water savings that accompanied the water demand projections represent an estimation of the amount of water (average per-person) that will be saved by the conversion to more water-efficient fixtures as described in the State Water-Efficient Plumbing Act passed in 1991. Those housing units built before the law came into effect will, over time, replace their old fixtures with the new water-efficient fixtures. TWDB is providing a suggested schedule at which the fixture replacements will take place, and the effect that the replacement will have on the city or utility's average Gallons Per Capita Daily (GPCD).

3.1.4.1 Water Savings

From the a recent study of water conservation, it is estimated that the average savings of replacing higher water-use fixtures with more efficient fixtures mandated by state and federal laws would be 16 gallons per person, per day (10.5 gallons for toilets and 5.5 gallons for showerheads).

3.1.4.2 Replacement Schedule

The TWDB compiles population data rather than housing data, so in calculating the number of houses and the less-efficient fixtures, the Board staff used population as a proxy for the number of houses at the time the law took effect and the projection of future houses. The July 1995 population estimate is used as a benchmark to determine the potential average per-capita water savings of a city or utility. The 1995 population (as a proxy for housing and fixtures) is assumed to have less-efficient fixtures, which can be replaced, lowering their GPCD and the city's or utility's average GPCD. Any population growth after 1995 is expected to inhabit new housing that was built with the more efficient water fixtures. No additional water savings can be expected on the basis of fixture replacement for the post-1995 population. Fixture standards have not changes since the initial law was implemented.

The July 1995 population estimate was chosen as a starting point for adoption of the more efficient fixtures for several reasons. First, in both the state and federal laws affecting plumbing codes, retailers were allowed to continue selling the less-efficient fixtures that they had in stock. Second, in any areas, whether a city or a subdivision served by a utility, there are vacant housing units which will eventually be occupied. Although there was no population in the house, there were less-efficient fixtures that will be used, and replaced, by residents eventually. Third, because we are using a proxy for the number of fixtures and the proxy (population estimate) can have varying degrees of accuracy, the July 1995 estimate was felt to be a good, conservative number.

The annual rate of fixture replacement was estimated to be 2 percent of the 1995 population, implying a 50 year adoption period for the 1995 population of housing. By the year 2045, 100 percent of the 1995 population would have the new water-efficient plumbing fixtures.

STEPS IN CALCULATING THE WATER SAVINGS DUE TO FIXTURE REPLACEMENT

A) <u>Establish the Base 'Dry-Year' and Associated GPCD.</u> Due to the extreme drought experience in 2011, it was decided that the year 2011 GPCD would act as the default 'dry-year' water use figure for all municipal water user groups. However, the base year for the population projections was 2010, so the dry-year GPCD (2011) will be applied to the 2010 base year. All potential water saving calculations are therefore subtracted from this reference GPCD (year 2011, assigned as the year 2010 value) to calculate the expected GPCD for each water user group over time as adoption of the various water saving technologies (fixtures, clothes and dish washers) proceed.

B) <u>Calculate the estimated savings due to replacement between 1995 and 2010</u>. Some fixture replacement took place between the passage of the law and the year 2010. The savings that result decrease the potential water savings available after the year 2010. Using the estimate that 2% of the 1995 population will replace the fixtures each year, 30% of the 1995 replaced their fixtures by the year 2010.

$$\textit{\textbf{EQ. 1: PCS2010}} = \underbrace{((\textit{\textbf{POP1995}} * 30\%) + \textit{\textbf{G1995-10}}) / \textit{\textbf{POP2010}})}_{\text{Calculates the percentage of the 2010 population that has water-efficient fixtures.}} * \underbrace{16 \text{ GPCD}}_{\text{The per-person amount saved per replaced toilet and showerhead.}}$$

GPCD2010	Per-person, per-day water use in 2010 (GPCD)
G1995-10	Population growth between 1995 and 2010
PCS2010	The city/utility's average GPCD savings due to plumbing code changes (fixture replacement) between 1995 and 2010.
PCS2020	The city/utility's average GPCD savings due to plumbing code changes (fixture replacement) between 2010 and 2020
POP1995	July 1995 population estimate
POP2010	Census 2010 population (cities) or Year 2010 population estimate (utilities

Note: The per-person savings for each toilet and showerhead replaced is 16 gallons, however this change in GPCD applies for the portion of the 1995 population that replaced fixtures up to the point in time under consideration plus the new housing units in the water use group service area. The average GPCD savings for the entire city or utility will be considerably less than the maximum possible 16 GPCD due to non-replacement of plumbing fixtures by the majority of 1995 housing units. As noted in the calculation above (EQ 1.), the estimated water savings are a combination of the accrued savings due to 30 percent of the 1995 level housing units, plus all of the growth from 1995 to the year 2010.

C) Calculate the remaining savings that will become available in each decade.

EQ. 2: PCS2020 =

$$\left[\underbrace{((\boldsymbol{POP1995} * 50\%) + (\boldsymbol{POP2020} - \boldsymbol{P1995})) / \boldsymbol{POP2020})}_{\substack{\text{Calculates the percentage of the 2010 population} \\ \text{that has water-efficient fixtures (30\% of the 1995} \\ \text{pop plus the growth between 2010 and 1995,} \\ \text{divided by the 2010 total population).}}} * 16\ \text{GPCD} \right] \text{minus } \underbrace{\boldsymbol{PCS2010}}_{\substack{\text{These water-use savings took place} \\ \text{before the water-use base year (2000)} \\ \text{and cannot be subtracted from the base}}}$$

Similar water savings calculations (a point estimate for the year 2020 (EQ 2)) combine water savings from 50 percent of the 1995 housing population plus all of the population growth since 1995. Water savings estimated to be in place by 2010 (PCS2010), already implicit in the year 2010 estimated GPCD, are then subtracted from the potential savings to avoid double counting the potential savings.

Estimated GPCD for the year 2020 is then the baseline Dry Year GPCD (***GPCD2010***) less the water savings accumulated up to that point in time.

EQ 3: 2020 Per-Person Water Use (GPCD) =

2010 Per-Person Water Use (*GPCD2000)* MINUS Fixture Efficiency Savings (*PCS2020)*

Note: A formula similar to EQ. 3 would apply for each decade through 2070. By 2060 and 2070 all of the fixture replacements would have taken place and no additional water savings (and GPCD reductions) will occur.

3.1.5 High-Efficiency Toilet Savings, 2009

House Bill 2667 of the 81st Texas Legislature (2009) mandated that all toilets installed in residential and commercial buildings, with limited exemptions be High-Efficiency Toilet, using no more than 1.28 gallons per flush. The act also addressed water efficiency standards for showerheads, urinals, and faucet flow.

3.1.5.1 Water Savings

The 2009 law required that by January 2014, all toilets use no more than 1.28 gallons per flush. This is a 20% savings from the 1.6 gallons per flush standard set in the 1991 Texas law. Based upon an average frequency of per-person toilet use in households of 5.1 and a per-use savings of 0.32 gallons per use the estimated saving of adopting high-efficiency toilets is 1.63 GPCD. The act also required changes to standards for showerheads, from 2.75 gallons per minute to 2.5 gallons per minute, and standards for urinals and faucets, however at the regional water planning level such savings become too detailed and cumbersome to incorporate.

3.1.5.2 Replacement Schedule

To provide toilet manufacturers time to shift production to high-efficiency toilets, the 2009 law allowed a phasing in period by the percent of models offered for sale meeting the 1.28 gallons per flush standard:

- January 1, 2010 – 50% of the models offered for sale
- January 1, 2011 – 67% of the models offered for sale
- January 1, 2012 – 75% of the models offered for sale
- January 1, 2013 – 85% of the models offered for sale
- January 1, 2014 – 100% of the models offered for sale

Similar to the replacement of water-efficient fixtures required by the 1991 law, the replacement of pre-high-efficiency toilet was assumed to be 2 percent per year, with adjustments for the 2010-2014 time period as the high-efficiency toilets are being phased in.

3.1.6 Dishwasher Savings Efficiency Savings

3.1.6.1 Water Savings

The baseline water use per load of dishwashers prior to mandatory efficiency standards was 14 gallons per load. Beginning in 2010, dishwashers were required to use no more than 6.5 gallons per cycle. By 2013 the maximum water use is set at 5 gallons per cycle for all dishwashers produced or sold in the country. Thus, the savings per load for the 2010 machine standards is 7.5 gallons per load (14 gallons – 6.5 gallons) and 9 gallons for the 2013 standards (14 gallons – 5 gallons).

The water efficiency saving for the 2010 – 2020 period is a weighted average of the 2010 and 2013 standards (3 years at 7.5 gal/load plus 7 years at 9 gal/load): 8.55 gallons per load. Water savings after 2020 is the full implementation of the 2013 standards of 5 gallons per load, or a savings of 9 gallons per load.

Table 3. Use and installation assumptions

Metric	Value	Source
People/ household	2.75	Texas State Data Center
Loads/household/yr	215	DOE/EPA estimate
Percentage of new construction installing a new Dishwasher	96.7%	DOE documentation on year 2012 dishwasher standards

Per-person, per day water use saving of the installation of new dishwashers:

<u>Water Savings (2010 to 2020)</u>

= (8.55 gal/load* 215 loads/yr)/(365 days/year * 2.75 people per household)

= 1.83 GPCD max savings for each new dishwasher installed.

<u>Water Savings (2020 to 2070)</u>

= (9 gal/load*215 loads/yr)/(365 days/yr*2.75 people/household)

= 1.93 GPCD max savings for each new dishwasher installed

3.1.6.2 Replacement Schedule and Baseline Adoption Values

A ten year useful life was assumed for dishwashers, with the baseline for dishwashers statewide estimated at 78 percent of existing households for 2010. The latter value is based on metropolitan statistics from the American Housing Survey (http://www.census.gov/housing/ahs/data/metro.html). Therefore, 78 percent of the 2010 population for each water use group was assumed to be the starting point for new, more water efficient dishwasher installation. The ten year useful life implied that ten percent of the 2010 population would install the more water efficient dishwashers each year. It is assumed that all pre-2010 dishwashers have the 14 gal/load water use level, so all benefits of the new standard(s) accrue beginning in 2010, and the updated WUG-specific GPCD values do not have to be adjusted for previous new technology adoption.

3.1.7 Clothes Washer Efficiency Savings

3.1.7.1 Water Savings

The first nationwide standards for residential clothes washers took effect in 2007, requiring both top and front-loading machines to use a maximum of 9.5 gallons per load, compared to a possible use of 27 gallons in pre-efficiency-standard machines. Future efficiency standards will require a maximum usage of 8.4 gallons per load in top-loading machines and 4.7 gallons in front-loading machines in the year 2015. In 2018, the maximum usage for top-loading machines will be reduced further to 6.5 gallons.

Table 4. Parameters for Clothes Washer Savings Calculations

Metric	Value	Source
People Per Household	2.75	Texas State Data Center, 2010 Census
Loads/household/yr	300	DOE/EPA estimate
Proportion of TX households with clothes washers in 2010	75%	American Housing Survey, Metro Stats for 4 major cities in Tx
Percentage of new construction installing a new Clothes Washer	91%	DOE documentation on year 2012 Clothes washer standards
Proportion Top-Loads vs Front-Loads	40% vs 60%	DOE documentation on year 2012 Clothes washer standards
Lifespan of Clothes Washing Machines	Top Load – 14 years, Front Load – 11 years, "Composite" – 12 years	www.bankrate.com/brm/news/pf/20050810c1.asp

Potential Max savings for

- •Both Top Loading and Front Loading Machines (27 gallon -9.5 gallon) = 17.5 gallon for year 2007 standard
- •Top Loading Machines (27 gallon -8.4 gallon) = 18.6 gallon /cycle for year 2015 standard
- •Top Loading Machines (27 gallon -6.5 gallon) = 20.5 gallon /cycle for year 2018 standard
- •Front Loading Machines (27 gallon -4.7 gallon) = 22.3 gallon /cycle for year 2015 standard

3.1.7.2 Replacement Schedule

A twelve year replacement schedule is assumed for the clothes washers. New clothes washer purchases/replacements assume that forty percent of the replacements are top-loading machines and 60 percent are frontloading. A composite machine (i.e., part top-loader and part front-loader) is assumed to ease the water savings calculation process, and a weighted average savings calculation, based upon the respective potential savings of the two types of machines, is performed. The American Housing Survey of 2010 for four major cities in Texas estimated that 75 percent of households have clothes washers. This percentage was applied as a statewide average. In addition, 2012 U.S. Department of Energy studies estimate that 96.7 percent of new residential construction will have clothes washers. These two parameters are used to determine the number of clothes washers eligible for replacement, or will be installed in new constructions as the estimates of potential GPCD savings are calculated for each decade.

CHAPTER 3 – GAP ANALYSIS

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

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Building a world of difference.

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3.0 Gap Analysis

3.1 PURPOSE

The purpose of this chapter is to determine the drinking water quantities to be supplied to each water utility from the RGRWA Regional Facility. This chapter will evaluate the existing drinking water infrastructure and planned expansions throughout the study area, the current needs of each utility to meet their demands, and the water management strategies recommended in the 2016 Region M Regional Water Plan to provide proposed water quantities be fulfilled by the RGRWA Regional Facility Plan.

3.2 EXISTING DRINKING WATER SUPPLIES

The LRGV is currently supplied by numerous surface water and brackish groundwater treatment plants. These plants range in their maximum day capacity from 0.3 to 47.3 MGD. Table 3-1 indicates the names and capacities of the various water treatment plants (WTPs) in the study area as well as any additional groundwater supplies that each utility has. These supplies are what an entity treats and/or pumps and does not include transfers from other utilities. In order to estimate the annual water usage and rights for each utility, a 1.6 maximum day to annual average day peaking factor was assumed. Dividing the maximum plant by the peaking factor (1.6) and multiplying by 365 days and converting to acre-ft results in an equivalent maximum amount of annual water rights that can be utilized. This amount is shown under the "SWTP Annual Production Capacity" number in the table.

The "Surface Water Rights" indicated is based on information provide in the Region M water planning process. This value represent the firm water rights held by the utility or WUG. The lesser value of the Production capacity and the Surface Water Rights is used to determine the Total amount of the water supply. If the Production Capacity is less than the Surface Water Rights, then it means that they do not have enough treatment plant capacity to use their existing rights. If the Production Capacity is more than the surface water rights, then they have the ability to treat more water than they have firm rights for.

The "Groundwater Supply" represents the amount of brackish groundwater water (or in a few cases fresh water) that can be produced based on the size of the WTP, similar to the surface water plant calculations above. The Total water supply is therefore the lesser of the Surface Water Rights, or surface water treatment plant (SWTP) production capacity, plus the Groundwater annual production capacity. This value represents that maximum water supply that is available by entity to meet their current and projected water demands. All demands above this value are assumed to be met by the Regional Water Facilities.

Table 3-1 Water Treatment Plants and Water Supply for Lower Rio Grande Valley

LOWER RIO GRANDE VALLEY SUPPLIES						
ENTITY	WTP	PLANT CAPACITY (MGD)	SWTP ANNUAL PRODUCTION CAPACITY (AFY)	SURFACE WATER RIGHTS* (AFY)	GROUNDWATER SUPPLY (AFY)	TOTAL (AFY)
Agua SUD	492 Water Plant	4	2,800			
	Abrams Water Plant	6	4,250			
	Agua SUD Havana WTP	3.5	2,460			
	TOTAL	13.5	9,510	6,725	0	6,725
ERHWSC	Arroyo City WTP	0.6	420			
	Martha M Simpson WTP	8	5,600			
	Nelson Road WTP	3.2	2,240			
	North Cameron Regional WTP				403	
	TOTAL	11.8	8,260	3,490	403	3,893
BPUB	BPUB WTP #1	20	14,000			
	BPUB WTP #2	20	14,000			
	TOTAL	40	28,000	32,153	0	28,000
Alamo	WTP	5	3,500	1,603	624	2,227
Donna	WTP	6.5	4,550	2,975	0	2,975
Edcouch	WTP	1.5	1,050	330	0	330
Edinburg	Downtown WTP	10	7,000			
	West WTP	8	5,600			
	TOTAL	18	12,600	8,822	0	8,822
Elsa	WTP	2.5	1,750	910	0	910
La Villa	WTP	1.4	980	246	0	246
Los Fresnos	WTP	2.4	1,680	715	0	715
Mercedes	WTP	3.78	2,646	1,288	655	1,943

LOWER RIO GRANDE VALLEY SUPPLIES						
ENTITY	WTP	PLANT CAPACITY (MGD)	SWTP ANNUAL PRODUCTION CAPACITY (AFY)	SURFACE WATER RIGHTS* (AFY)	GROUNDWATER SUPPLY (AFY)	TOTAL (AFY)
Mission	North WTP	11.5	8,050			
	South WTP	8	5,600			
	TOTAL	19.5	13,650	12,078	0	12,078
LMWD	WTP 1	8.4	5,880			
	WTP 2	11.2	7,840			
	TOTAL	19.6	13,720	3,413	0	3,413
McAllen	North WTP	11.3	7,910			
	South WTP	47.3	33,110			
	TOTAL	58.5	41,020	28,196	306	28,502
NAWSC	WTP 5	2.5	1,750			
	North Cameron Regional WTP	2.3	1,610		710	
	WTP 1	3.5	2,450			
	La Sara WTP	1.3	910		1,120	
	WTP 4	3.5	2,450			
	WTP 2	2.5	1,750			
	WTP 6	1.3	910			
	Donna WTP				2,240	
	Doolittle WTP				3,360	
	Owassa WTP				1,680	
	TOTAL	16.9	11,830	14,624	9,110	20,940
Olmito WSC	WTP	2	1,400	526	0	526
Pharr	Water Plant	19	13,300	6,741	0	6,741
Lyford	WTP	0.7	490	588	0	490
Raymondville	WTP	6	4,200	3,402	2,240	5,642
San Juan	WTP 1	7	4,900			
	WTP 2	7	4,900			
	TOTAL	14	9,800	2,141	404	2,545

LOWER RIO GRANDE VALLEY SUPPLIES						
ENTITY	WTP	PLANT CAPACITY (MGD)	SWTP ANNUAL PRODUCTION CAPACITY (AFY)	SURFACE WATER RIGHTS* (AFY)	GROUNDWATER SUPPLY (AFY)	TOTAL (AFY)
Harlingen	Downtown WTP	18.7	13,090			
	MF Runnion WTP	20.4	14,280			
	TOTAL	39.1	27,370	15,231	0	15,231
Hidalgo County MUD	WTP	1.4	980	273	0	273
La Feria	WTP	4	2,600	1,020	0	1,020
La Joya	WTP	0.3	210	388	595	805
MHWSC	Las Rusias WTP	2.1	1,470			
	Progresso RO WTP	1	700			
	TOTAL	3.1	2,170	556	6,170	6,726
San Benito	WTP 1	6	4,200			
	WTP 2	6	4,200			
	TOTAL	12	8,400	4,782	0	4,782
Santa Rosa	WTP	1	700	238	0	238
Sebastian MUD	WTP	0.7	490	204	0	204
Sharyland WSC	WTP 1	6	4,200			
	WTP 2	8	5,600			
	WTP 3	8	5,600			
	TOTAL	20	15,400	7,160	0	7,160
Southmost Regional Water Authority	Desal Facility	11	0	0	7,700	7,700
Valley MUD #2	Desal Facility	0.3			280	
	SWTP	2.3	1,610			
	TOTAL	2.6	1,610	798	280	1,078
Weslaco	WTP	8.1	5,670	3,928	0	3,928
TOTAL		**366**	**249,536**	**165,544**	**29,602**	**186,808**

*Full amount of water rights had been adjusted to account for efficiency losses through the Irrigation Districts, as shown in Appendix A

In addition to the individual water plants that a utility currently operates to meet their demands, many utilities are also interconnected with other water entities. The larger water utility serves as a wholesale water provider to shore up the water supply to another utility in the case of a drought, or potentially under a push water concern that may control water delivery. These interconnections provide a sub-regional system to meet water demands collectively. The current interconnection infrastructure is detailed in Appendix B. These interconnections are regarded as emergency connections and are not considered as part of a long term water supply strategy.

3.3 PLANNED EXPANSIONS

As part of the Region M planning process, several planned expansions have been identified and included for many of the entities. These projects include expansions for SWTPs, BGD Plants, and new wells for raw water blending upstream of their SWTPs. Table 3-2 describes the planned expansions for each entity that have been submitted as part of the 2016 Regional Water Plan (Region M). Since they have been specifically identified, and they are an expansion to an existing facility, we recognize them here and not include this capacity provided as part of the development of the regional water system. Average annual production from each facility is calculated for each decade assuming a 1.6 peaking factor is needed for the plants to utilize the annual volume of water. It is assumed that the entities will buy existing water rights from other entities in order to use the full production capacity of the expanded plant. Generally, if a water utility is planning to expand their existing plant, which already is being staffed, and has the supporting infrastructure to distribute the water this capacity was NOT displaced with the Regional Water System.

Table 3-2 Planned Expansions to Water Treatment Plants in the Lower Rio Grande Valley

ENTITY	PROJECT DESCRIPTION	ADDITIONAL CAPACITY (MGD)	CURRENT SUPPLY (AFY)	EXPANDED SUPPLY (AFY)
Donna	Upgrade and expand WTP from 6.5 MGD to 10.5 MGD by 2020.	4	2,975	5,775
Elsa	Upgrade and expand WTP from 2.5 MGD to 4.5 MGD by 2020.	2	910	2,310
San Juan	Proposed a water plant upgrade to replace antiquated structures and equipment. Provide facilities to manufacture liquid chlorine due to neighborhood hazard. Install ground water wells and provide membrane treatment of the ground water. WTP will be expanded from 7 MGD to 10 MGD by 2020.	3	2,545	4,645
Sharyland WSC	Add groundwater well to WTP No. 2 and expand WTP from 8 MGD to 9 MGD by 2020.	1	7,160	8,560
	Add groundwater well to WTP No. 3 and expand WTP from 8 MGD to 9 MGD by 2020.	1		

ENTITY	PROJECT DESCRIPTION	ADDITIONAL CAPACITY (MGD)	CURRENT SUPPLY (AFY)	EXPANDED SUPPLY (AFY)
North Alamo WSC	Add well La Sara reverse osmosis plant to provide an additional source of raw water to the plant, increasing the capacity from 1.3 MGD to 2.3 MGD by 2020.	1	20,940	24,440
	Expansion to WTP 5 to provide an additional 4 MGD of potable water to area residents. WTP will be expanded from 2.5 MGD to 6.5 MGD by 2020.	4		
Weslaco	Add groundwater well to WTP and expand WTP from 8.1 MGD to 9.6 MGD by 2020.	1.5	3,928	4,978
Total of other municipalities without expansions				148,350
TOTAL Existing and Planned Treatment Capability				**199,058**

3.4 OVERALL INFRASTRUCTURE GAP

The capacity of the regional water system to be analyzed is the difference between the existing available water sources and the total water needs projected from today (The Infrastructure Gap). In previous chapters we have outlined the projected population projections and the resulting water demands by decade out to 2070.

Table 3-3 presents the gap between the available supply and the demand in 2070 by water utility. It is assumed that the infrastructure required to produce the future supply is sized 30% larger than the annual average usage (a 1.3 peaking factor is assumed). This small peaking factor is required to manage the water resources and allow some operational and seasonable flexibility.

Table 3-3 Rio Grande Valley Infrastructure Production Gap

ENTITY	CURRENT TOTAL SUPPLY (AFY)	EXPANDED SUPPLY (AFY)	2070 DEMANDS (AFY)	2070 INFRASTRUCTURE GAP (AFY)	2070 INFRASTRUCTURE NEED (MGD)
Agua SUD	6,725	6,725	11,652	-4,927	-5.7
East Rio Hondo WSC	3,893	3,893	6,973	-3,080	-3.6
Brownsville Public Utility Board	28,000	28,000	69,520	-41,520	-48.2
Alamo	2,227	2,227	6,787	-4,560	-5.3

ENTITY	CURRENT TOTAL SUPPLY (AFY)	EXPANDED SUPPLY (AFY)	2070 DEMANDS (AFY)	2070 INFRASTRUCTURE GAP (AFY)	2070 INFRASTRUCTURE NEED (MGD)
Donna	2,975	5,775	5,375	400	0.5
Edcouch	330	330	705	-375	-0.4
Edinburg	8,822	8,822	27,667	-18,845	-21.9
Elsa	910	2,310	1,641	669	0.8
La Villa	246	246	564	-318	-0.4
Los Fresnos	715	715	838	-123	-0.1
Mercedes	1,943	1,943	4,531	-2,588	-3.0
Mission	12,078	12,078	43,305	-31,227	-36.2
McAllen	28,502	28,502	82,563	-54,061	-62.7
NAWSC	20,940	24,440	52,761	-28,321	-32.9
Olmito	526	526	1,327	-801	-0.9
Pharr	6,741	6,741	20,607	-13,866	-16.1
Lyford	490	490	432	58	0.1
Laguna Madre	3,413	3,413	13,114	-9,701	-11.3
Raymondville	5,642	5,642	2,286	3,356	3.9
San Juan	2,545	4,645	12,940	-8,295	-9.6
Harlingen	15,231	15,231	24,516	-9,285	-10.8
Hidalgo County MUD	273	273	1,174	-901	-1.0
La Feria	1,020	1,020	2,012	-992	-1.2
La Joya	805	805	1,351	-546	-0.6
MHWSC	6,726	6,726	9,233	-2,507	-2.9
San Benito	4,782	4,782	6,346	-1,564	-1.8
Santa Rosa	238	238	498	-260	-0.3
Sebastian	204	204	230	-26	0.0
Sharyland WSC	7,160	8,560	16,896	-8,336	-9.7
SRWA	7,700	7,700	7,700	0	0.0

ENTITY	CURRENT TOTAL SUPPLY (AFY)	EXPANDED SUPPLY (AFY)	2070 DEMANDS (AFY)	2070 INFRASTRUCTURE GAP (AFY)	2070 INFRASTRUCTURE NEED (MGD)
Valley MUD #2 (Rancho Viejo)	1,078	1,078	1,557	-479	-0.6
Weslaco	3,928	4,978	16,625	-11,647	-13.5
TOTAL	**186,808**	**199,058**	**453,726**	**-254,668**	**-295.6**

3.5 DEMANDS MET BY RGRWA REGIONAL FACILITY PLAN

Calculated demand projections previously determined include the effects of passive conservation. Though required in the regional planning process, advanced conservation was not considered as a potential alternative in this study when the measures were developed and recommended by the planning group WITHOUT a direct request or input from the water supplier. The benefits from active conservation require considerable effort to measure, analyze, and educate the water system and the water customers. Without a noted commitment to these advanced conservation measure, we do not believe that the savings will be realized.

Specific strategies that were recommended by the water supplier were included and anticipated water savings were calculated based on the regional planning process, and their demands subsequently reduced.

Another type of conservation included in the 2016 Regional Water Plan is Irrigation District Conservation. This strategy calculated the quantity of water that would be saved if each of the Irrigation Districts made improvements to raise their system efficiency to 90% in 2070. This general reduction in water use by the increase in delivery efficiency was NOT included in this plan since it requires many major capital improvement projects to be implemented to be effective.

Some of the Irrigation Districts submitted specific projects that they intend to implement in order to reduce water loss. These were accounted for in the general Irrigation District Conservation strategy for those entities. Each customer served by an Irrigation District was assigned a portion of the water savings that came from the increased efficiencies.

The water demand projections and subsequent gap analysis for the RGRWA Regional Facility Plan was based on data in the 2016 Regional Water Plan, with the stated deviations in the effects of conservation, summarized in Table 3-4.

Table 3-4 Conservation Strategies in Region M Plan and Regional Facility Plan

CONSERVATION STRATEGY	INCLUDED IN REGION M	INCLUDED IN REGIONAL FACILITY PLAN
General Passive Conservation	Yes	Yes
General Municipal Conservation Developed by Region M	Yes	No
Specific Municipal Conservation Projects Submitted by Entity	Yes	Yes

CONSERVATION STRATEGY	INCLUDED IN REGION M	INCLUDED IN REGIONAL FACILITY PLAN
General Irrigation District Conservation Developed by Region M	Yes	No
Specific Irrigation District Conservation Projects Submitted by Entity	Yes	Yes

The final determination of the capacity to be met by the regional water system was impacted by the following constraints and considerations:

- Because conservation was included differently between the Regional Water Plan and the Regional Facility Plan, in some cases the Regional Water Plan recommended projects did not provide enough water to meet the revised need after conservation, therefore they were assumed to be met by the regional water system. The benefits of acquiring water through regional facilities include cost savings due to economies of scale on shared facilities and centralized O&M costs. TWDB funding also encourages regional projects.
- In general, the projects that were developing a brand new water resource, (in lieu of expanding an existing one) were proposed to be replaced by the RGRWA Regional Facility Plan. For example, if a municipality had a recommended strategy in Region M to build a new brackish groundwater desalination plant, that groundwater availability could instead be used by the Regional Facility Plan and that municipality could receive the same amount of water through the regional project.
- Small water right acquisitions or contract water purchases that were less than 1,000 acre-feet remained as recommendations where the regional system could not reasonably supply the municipality because of its distance from the regional system.

Once it was determined which 2016 Regional Water Plan projects would be proposed to remain or be replaced, the quantity of each entity's needs that are logical to be met by the Regional Facility Plan was calculated. Appendix A has detailed sheets indicating the demand, supply and needs for each of the municipal entities. Each data sheet indicates the proposed recommended projects and the water to be supplied by the regional system. The amount of water provided to each entity from the RGRWA Regional Facility Projects by decade is summarized in Table 3-5.

Table 3-5 Needs To Be Met by RGRWA Regional Facility Plan (AF/YR)

ENTITY	2020	2030	2040	2050	2060	2070
Agua SUD	0	700	700	2,900	4,600	6,350
Alamo	850	1,500	2,200	2,950	3,650	4,400
Brownsville	0	0	500	7,600	15,150	22,950
Donna	0	150	650	1,250	1,800	2,400
East Rio Hondo WSC	0	50	650	1,300	2,000	2,700
Edinburg	3,550	6,350	9,200	12,150	15,150	18,100
Harlingen	0	0	1,100	3,500	6,050	8,700
Hidalgo	400	800	1,200	1,600	2,050	2,450

ENTITY	2020	2030	2040	2050	2060	2070
Hidalgo County MUD1	300	450	550	650	800	950
La Feria	0	50	200	400	600	800
Laguna Vista	850	1,250	1,650	2,100	2,550	3,000
McAllen	4,350	12,800	21,500	30,350	39,350	48,150
Mercedes	250	700	1,150	1,600	2,100	2,550
Military Highway WSC	1,100	2,050	3,050	4,150	5,250	6,400
Mission	6,650	11,150	15,700	20,350	25,100	29,700
North Alamo WSC	0	1,750	3,100	8,750	12,350	16,950
Olmito WSC	0	0	0	100	250	400
Pharr	50	2,050	4,150	6,300	8,600	10,750
Port Isabel	450	650	850	1,100	1,300	1,550
Rancho Viejo	0	0	0	0	100	250
San Benito	0	0	0	0	600	1,250
San Juan	1,750	2,850	3,900	5,250	6,550	7,850
Sharyland WSC	1,050	4,300	7,700	11,200	15,700	17,850
South Padre Island	1,100	1,650	2,200	2,750	3,350	4,000
Weslaco	2,800	4,500	6,200	7,950	9,800	11,550
TOTAL	**25,500**	**55,750**	**88,100**	**136,250**	**184,800**	**232,000**

Appendix A. Individual City Decision Documents

WATER SUPPLY AND DEMAND ANALYSIS								
BROWNSVILLE								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			211,200	251,288	291,955	335,755	380,809	426,990
Water Demand (ac-ft)			36,092	41,913	47,986	54,797	62,040	69,520
Current Water Supply	**Type**							
Direct Source	MUNI AWR		31,740	31,740	31,740	31,740	31,740	31,740
Cameron County ID #6	MUNI AWR		263	263	263	263	263	263
Southmost Regional Water Authority	GW		11,448	11,448	11,448	11,448	11,448	11,448
Valley MUD #2	MUNI AWR		150	150	150	150	150	150
Total Supply (AF/yr)			**43,601**	**43,601**	**43,601**	**43,601**	**43,601**	**43,601**
Projected Supply Surplus/Deficit			**0**	**0**	**-4,385**	**-11,196**	**-18,439**	**-25,919**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**0**	**0**	**-4,385**	**-11,196**	**-18,439**	**-25,919**
Region M Recommended WMS	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brownsville Resaca Restoration	$ 12,396,000	$ 1,182	827	827	827	827	827	827
Brownsville Banco Morales Reservoir	$ 8,853,000	$ 168	3,564	3,564	3,564	3,564	3,564	3,564
Non-Potable Water Reuse Pipeline	$ 32,271,000	$ 1,094	0	0	0	0	0	0
Brownsville Southside WWTP Potable Reuse -Phase I	$ 36,282,000	$ 1,651	0	3,412	3,412	3,412	0	0
Brownsville Southside WWTP Potable Reuse -Phase II	$ 9,822,000	$ 1,153	0	0	0	0	4,715	4,715
Brownsville Seawater Desalination Demonstration (Phase I)	$ 56,002,000	$ 5,522	2,603	2,603	2,603	2,603	2,603	2,603
Brownsville Seawater Desalination Demonstration (Phase II)	$ 309,531,000	$ 3,646	0	0	0	0	26,022	26,022
Surplus/Deficit after WMS's			**6,994**	**10,406**	**6,021**	**-790**	**19,292**	**11,812**
Proposed Recommended WMS	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brownsville Resaca Restoration	$ 12,396,000	$ 1,182	827	827	827	827	827	827
Brownsville Banco Morales Reservoir	$ 8,853,000	$ 168	3,564	3,564	3,564	3,564	3,564	3,564
Non-Potable Water Reuse Pipeline	$ 32,271,000	$ 1,094	0	0	0	0	0	0
RGRWA			0	0	500	7,600	15,150	22,950
Transfer to El Jardin			-31	-257	-498	-772	-1,069	-1,376
Surplus/Deficit after WMS's			**4,360**	**4,134**	**8**	**23**	**33**	**46**
Region M Alternative WMS	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brownsville/Matamoros Weir and Reservoir	$ 20,508,000	$ 77	17,821	17,887	17,953	18,020	18,086	18,152
Valley MUD #2 New BGD Plant	$ 3,760,000	$ 6,430	0	0	0	0	10	10

Legend	
Deleted WMS	
Changed WMS	
RGRWA	

WATER SUPPLY AND DEMAND ANALYSIS								
COMBES								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			3,414	3,989	4,571	5,199	5,845	6,507
Water Demand (ac-ft)			322	358	397	445	498	554
Current Water Supply	**Type**							
Harlingen	MUNI AWR		322	322	322	322	322	322
Total Supply (AF/yr)			**322**	**322**	**322**	**322**	**322**	**322**
Projected Supply Surplus/Deficit			**0**	**-36**	**-75**	**-123**	**-176**	**-232**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**0**	**-36**	**-75**	**-123**	**-176**	**-232**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Harlingen Wastewater Treatment Plant 2 Potable Reuse	$ 19,164,000	$ 1,957	0	0	39	39	39	43
Surplus/Deficit after WMS's			**0**	**-36**	**-36**	**-84**	**-137**	**-189**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfer from Harlingen			0	36	75	123	176	232
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**0**	**0**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brackish Groundwater Desalination Plant	$ 3,891,000	$ 5,320	0	0	0	125	125	125
Harlingen New BGD Plant	$ 12,327,000	$ 2,180	0	0	21	21	21	21

Legend	
	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS									
EAST RIO HONDO WSC									
Year				**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population				27,471	32,092	36,781	41,831	47,025	52,349
Water Demand (ac-ft)				**3,826**	**4,372**	**4,948**	**5,589**	**6,269**	**6,973**
East Rio Hondo WSC				3,826	4,372	4,948	5,589	6,269	6,973
Indian Lake									
Military Highway WSC (from water use survey)									
Current Water Supply	**Type**								
Groundwater	GW			403	403	403	403	403	403
Cameron County ID #2	MUNI AWR			3,260	3,260	3,260	3,260	3,260	3,260
Harlingen	MUNI AWR			216	216	216	216	216	216
Harlingen ID	MUNI AWR			230	230	230	230	230	230
Olmito WSC	MUNI AWR			49	49	49	49	49	49
Total Supply (AF/yr)				**4,158**	**4,158**	**4,158**	**4,158**	**4,158**	**4,158**
Projected Supply Surplus/Deficit				**332**	**-214**	**-790**	**-1,431**	**-2,111**	**-2,815**
Evaluation of Selected Water Management Strategies				**Additional Supply by Decade**					
Conservation				**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: CCID #2				136	136	136	136	136	136
Need after Conservation				**468**	**-78**	**-654**	**-1,295**	**-1,975**	**-2,679**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
FM 2925 Water Transmission Line	$ 5,089,000	$ 15,967		30	30	30	30	30	30
UV Disinfection - FM 510 WTP	$ 687,000	$ 24,282		11	11	11	11	11	11
North Cameron Regional WTP Wellfield Expansion	$ 1,881,000	$ 843		240	240	240	240	240	240
Harlingen WW Interconnect	$ 3,268,000	$ 1,766		112	112	112	0	0	0
Surface Water Treatment Plant and WR Purchase	$ 34,794,000	$ 736		320	320	320	320	320	320
Harlingen WWTP 2 Potable Reuse*	$ 19,164,000	$ 1,957		0	0	26	26	26	26
Total Surplus/Deficit				**1,181**	**635**	**85**	**-668**	**-1,348**	**-2,052**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
FM 2925 Water Transmission Line	$ 5,089,000	$ 15,967		30	30	30	30	30	30
UV Disinfection - FM 510 WTP	$ 687,000	$ 24,282		11	11	11	11	11	11
RGRWA				0	50	650	1,300	2,000	2,700
Surplus/Deficit after WMS's				509	13	37	46	66	62
Transfer to Indian Lake				-12	-12	-12	-17	-26	-36
Total Surplus/Deficit				**497**	**1**	**25**	**29**	**40**	**26**
Region m Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Surface Water TP (Phase II)	$ 28,386,000	$ 414		0	0	0	2,500	2,500	2,500
Harlingen New BGD Plant	$ 12,327,000	$ 2,180		0	0	14	14	14	14

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
EL JARDIN								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			15,099	17,640	20,218	22,995	25,851	28,779
Water Demand (ac-ft)			1,704	1,931	2,172	2,447	2,744	3,052
Current Water Supply	**Type**							
Brownsville	MUNI AWR		1,480	1,480	1,480	1,480	1,480	1,480
Total Supply (AF/yr)			**1,480**	**1,480**	**1,480**	**1,480**	**1,480**	**1,480**
Projected Supply Surplus/Deficit			**-224**	**-451**	**-692**	**-967**	**-1,264**	**-1,572**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade (AF/yr)**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-224**	**-451**	**-692**	**-967**	**-1,264**	**-1,572**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
El Jardin Brackish Desalination Plant	$ 8,272,000	$ 2,557	560	560	560	560	560	560
El Jardin Distribution Pipeline Replacement	$ 23,421,000	$ 192,909	11	11	11	11	11	11
Brownsville Resaca Restoration	$ 12,396,000	$ 1,182	34	34	34	34	34	34
Brownsville Banco Morales Reservoir	$ 8,853,000	$ 168	148	149	149	150	150	151
Brownsville Southside WWTP Potable Reuse -Phase I	$ 36,282,000	$ 1,651	0	517	517	517	0	0
Brownsville Southside WWTP Potable Reuse -Phase II	$ 9,822,000	$ 1,153	0	0	0	0	196	196
Brownsville Seawater Desalination Demonstration (Phase I)	$ 56,002,000	$ 5,522	108	108	108	108	0	0
Brownsville Seawater Desalination Demonstration (Phase II)	$ 309,531,000	$ 3,646	0	0	0	0	1081	1081
Surplus/Deficit after WMS's			**637**	**928**	**687**	**413**	**768**	**461**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
El Jardin Distribution Pipeline Replacement	$ 23,421,000	$ 192,909	11	11	11	11	11	11
Brownsville Resaca Restoration	$ 12,396,000	$ 1,182	34	34	34	34	34	34
Brownsville Banco Morales Reservoir	$ 8,853,000	$ 168	148	149	149	150	150	151
Transfer from Brownsville	$ 1,407,500		31	257	498	772	1069	1376
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**0**	**0**
Region M Alternative WMS	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brownsville/Matamoros Weir and Reservoir	$ 20,508,000	$ 77	17,821	17,887	17,953	18,020	18,086	18,152

Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
HARLINGEN								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			76,464	89,334	102,390	116,452	130,916	145,742
Water Demand (ac-ft)			13,546	15,429	17,400	19,636	22,035	24,516
Current Water Supply	**Type**							
Reuse	REUSE		1,120	1,120	1,120	1,120	1,120	1,120
Harlingen ID, Cameron County #1	MUNI AWR		15,231	15,231	15,231	15,231	15,231	15,231
Total Supply (AF/yr)			**16,351**	**16,351**	**16,351**	**16,351**	**16,351**	**16,351**
Projected Supply Surplus/Deficit			**0**	**0**	**-1,049**	**-3,285**	**-5,684**	**-8,165**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: Harlingen ID			225	225	225	225	225	225
Need after Conservation			**225**	**225**	**-824**	**-3,060**	**-5,459**	**-7,940**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Harlingen Wastewater Treatment Plant 2 Potable Reuse - Harlingen Supply	$ 19,164,000	$ 1,957	0	0	1,620	1,620	1,620	1,620
Surplus/Deficit after WMS's			225	225	796	-1,440	-3,839	-6,320
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA	$ -		0	0	1,100	3,500	6,050	8,700
Surplus/Deficit after WMS's			225	225	276	440	591	760
Transfer to Combes			0	-36	-75	-123	-176	-232
Transfer to Palm Valley			0	-39	-80	-126	-177	-229
Transfer to Primera			0	-50	-78	-142	-213	-287
Total Surplus/Deficit			**225**	**100**	**43**	**49**	**25**	**12**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Harlingen New BGD Plant	$ 12,327,000	$ 2,180	0	0	888	888	888	888
Non-potable Reuse Project	$ 6,898,000	$ 1,678	677	677	677	677	677	677

Legend
Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
INDIAN LAKE								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			755	882	1,011	1,150	1,293	1,439
Water Demand (ac-ft)			51	60	68	78	87	97
Current Water Supply	**Type**							
East Rio Hondo WSC	GW/MUNI AWR		39	39	39	39	39	39
Southmost Regional Water Authority	GW/MUNI AWR		0	22	22	22	22	22
Total Supply (AF/yr)			**39**	**61**	**61**	**61**	**61**	**61**
Projected Supply Surplus/Deficit			**-12**	**0**	**-7**	**-17**	**-26**	**-36**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-12**	**0**	**-7**	**-17**	**-26**	**-36**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ERHWSC North Cameron Regional WTP Supply	$ 1,881,000	$ 843.00	40	40	40	40	40	40
ERHWSC Surface Water Treatment Plant and WR Purchase	$ 34,794,000	$ 736.00	80	80	80	80	80	80
Surplus/Deficit after WMS's			**108**	**120**	**113**	**103**	**94**	**84**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfer from East Rio Hondo WSC			12	12	12	17	26	36
Surplus/Deficit after WMS's			**0**	**12**	**5**	**0**	**0**	**0**

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
LA FERIA								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			8,610	10,059	11,530	13,113	14,742	16,411
Water Demand (ac-ft)			1,126	1,274	1,432	1,613	1,809	2,012
Current Water Supply	**Type**							
La Feria ID	MUNI AWR		1,020	1,020	1,020	1,020	1,020	1,020
Direct Source	MUNI AWR		50	50	50	50	50	50
Total Supply (AF/yr)			**1,070**	**1,070**	**1,070**	**1,070**	**1,070**	**1,070**
Projected Supply Surplus/Deficit			**-56**	**-204**	**-362**	**-543**	**-739**	**-942**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: La Feria, CCID No. 3			142	142	142	142	142	142
Need after Conservation			**86**	**-62**	**-220**	**-401**	**-597**	**-800**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Rainwater Harvesting	$ 204,000	$ 831	24	24	24	24	24	24
Water Well with R.O. Unit Providing a Backup Drinking Water Supply	$ 6,260,000	$ 1,163	1120	1120	1120	1120	1120	1120
Surplus/Deficit after WMS's			**1,230**	**1,082**	**924**	**743**	**547**	**344**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Rainwater Harvesting	$ 204,000	$ 831	24	24	24	24	24	24
RGRWA			0	50	200	400	600	800
Surplus/Deficit after WMS's			**110**	**12**	**4**	**23**	**27**	**24**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Non-Potable Wastewater Reuse	$ 2,830,000	$ 2,834	174	174	174	174	174	174

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
LAGUNA VISTA								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			3,676	4,294	4,922	5,598	6,293	7,006
Water Demand (ac-ft)			2,435	2,831	3,236	3,676	4,130	4,597
Current Water Supply	**Type**							
Laguna Madre WD	MUNI AWR		1,329	1,329	1,329	1,329	1,329	1,329
Total Supply (AF/yr)			**1,329**	**1,329**	**1,329**	**1,329**	**1,329**	**1,329**
Projected Supply Surplus/Deficit			**-1,106**	**-1,502**	**-1,907**	**-2,347**	**-2,801**	**-3,268**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-1,106**	**-1,502**	**-1,907**	**-2,347**	**-2,801**	**-3,268**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Brackish Desalination Plant	$ 22,443,000	$ 1,773	780	780	780	780	780	780
LMWD Potable Reuse	$ 13,613,000	$ 2,865	286	286	286	286	286	286
Surplus/Deficit after WMS's			**-40**	**-436**	**-841**	**-1,281**	**-1,735**	**-2,202**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Potable Reuse	$ 13,613,000	$ 2,865	286	286	286	286	286	286
RGRWA via LMWD			850	1250	1650	2100	2550	3000
Surplus/Deficit after WMS's			**30**	**34**	**29**	**39**	**35**	**18**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Non-potable Reuse Project	$ 3,931,000	$ 1,929	122	122	122	122	122	122
LMWD Seawater Desalination Plant	$ 29,609,000	$ 7,175	390	390	390	390	390	390

Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
LOS FRESNOS								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			6,535	7,635	8,751	9,952	11,189	12,456
Water Demand (ac-ft)			440	514	589	669	752	838
Current Water Supply	**Type**							
Cameron County ID #6	MUNI AWR		715	513	513	513	513	513
Southmost Regional Water Authority	GW		302	280	280	280	280	280
Total Supply (AF/yr)			**1,017**	**793**	**793**	**793**	**793**	**793**
Projected Supply Surplus/Deficit			**0**	**0**	**0**	**0**	**0**	**-45**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: CCID No. 6			114	138	161	185	208	231
Need after Conservation			**114**	**138**	**161**	**185**	**208**	**186**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Surplus/Deficit after WMS's			**114**	**138**	**161**	**185**	**208**	**186**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Surplus/Deficit after WMS's			**114**	**138**	**161**	**185**	**208**	**186**

Legend	
	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
LOS INDIOS								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			1,277	1,492	1,710	1,945	2,187	2,434
Water Demand (ac-ft)			144	161	179	201	226	251
Current Water Supply	**Type**							
Military Highway WSC	GW/MUNI AWR		123	123	123	123	123	123
Total Supply (AF/yr)			**123**	**123**	**123**	**123**	**123**	**123**
Projected Supply Surplus/Deficit			**-21**	**-38**	**-56**	**-78**	**-103**	**-128**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-21**	**-38**	**-56**	**-78**	**-103**	**-128**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
MHWSC Expand Existing Groundwater Wells (Cameron Co.)	$ 5,373,000	$ 1,254	50	50	50	50	50	50
MHWSC Acquisition of Water Rights through Urbanization	$ 510,000	$ 143	8	28	45	64	92	114
ERHWSC New Surface WTP via MHWSC	$ 34,794,000	$ 736	10	10	10	10	10	10
Surplus/Deficit after WMS's			**47**	**50**	**49**	**46**	**49**	**46**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfer from Military Highway WSC			21	38	56	78	103	128
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**0**	**0**
Region M Alternative Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase I)	$ 668,000	$ 316	6	6	6	0	0	0
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase II)	$ 810,000	$ 195	0	0	0	16	16	16

Legend	
	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
MILITARY HIGHWAY WSC								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			31,604	37,814	44,083	50,615	57,248	63,893
Water Demand (ac-ft)			4,791	5,595	6,431	7,333	8,278	9,233
Current Water Supply	**Type**							
Groundwater	GW		2,902	2,902	2,902	2,902	2,902	2,902
East Rio Hondo WSC	GW		22	22	22	22	22	22
Harlingen	MUNI AWR		120	120	120	120	120	120
Harlingen ID #1	MUNI AWR		556	556	556	556	556	556
North Alamo WSC	GW/MUNI AWR		15	15	15	15	15	15
Weslaco	MUNI AWR		146	146	146	146	146	146
Total Supply (AF/yr)			**3,761**	**3,761**	**3,761**	**3,761**	**3,761**	**3,761**
Projected SupplySurplus/Deficit			**-1,030**	**-1,834**	**-2,670**	**-3,572**	**-4,517**	**-5,472**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: Harlingen ID			9	9	9	9	9	9
Need after Conservation			**-1,021**	**-1,825**	**-2,661**	**-3,563**	**-4,508**	**-5,463**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
MHWSC Expand Existing Groundwater Wells (Cameron Co.)	$ 5,373,000	$ 1,254	401	401	401	401	401	401
MHWSC Acquisition of Water Rights through Urbanization	$ 7,735,000	$ 143	255	919	1,505	2131	2730	3489
North Cameron Regional WTP Wellfield Expansion	$ 1,881,000	$ 843	121	121	121	121	121	121
ERHWSC New Surface WTP	$ 34,794,000	$ 736	280	280	280	280	280	280
Harlingen WWTP 2 Potable Reuse	$ 19,164,000	$ 1,957	0	0	17	17	17	17
NAWSC Delta Area RO Plant 2 MGD	$ 22,709,000	$ 1,781	0	0	0	0	1	1
NAWSC La Sara RO Plant, expand well field	$ 13,260,000	$ 2,104	0	0	0	0	0	1
NAWSC Expansion of Water Treatment Plant No. 5	$ 23,794,000	$ 505	0	2	2	2	2	2
NAWSC Expansion of Delta WTP	$ 28,802,000	$ 748	0	0	2	3	3	3
Total Surplus/Deficit			36	-102	-333	-607	-953	-1148
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
NAWSC Delta Area RO Plant 2 MGD	$ 22,709,000	$ 1,781	1	3	3	3	4	5
NAWSC La Sara RO Plant, expand well field	$ 13,260,000	$ 2,104	0	0	3	3	4	4
RGRWA		$ -	1,100	2,050	3,050	4,150	5,250	6,400
Surplus/Deficit after WMS's			80	228	395	593	750	946
Transfer to Los Indios			-21	-38	-56	-78	-103	-128
Transfer to Progreso			-26	-172	-324	-481	-643	-802
Total Surplus/Deficit			**33**	**18**	**15**	**34**	**4**	**16**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase I)	$ 668,000	$ 316	209	209	209	0	0	0
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase II)	$ 810,000	$ 195	0	0	0	522	522	522
Harlingen New BGD Plant	$ 12,327,000	$ 2,180	0	0	9	9	9	9

Legend	
Deleted WMS	
Changed WMS	
RGRWA	

WATER SUPPLY AND DEMAND ANALYSIS								
North Alamo WSC								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			154,708	191,370	228,143	265,043	301,920	337,782
Water Demand (ac-ft)			25,081	30,421	35,897	41,494	47,186	52,761
Current Water Supply	**Type**							
Groundwater	GW		9,349	9,349	9,349	9,349	9,349	9,349
Delta Lake ID	MUNI AWR		4,504	4,504	4,504	4,504	4,504	4,504
Donna ID	MUNI AWR		1,759	1,759	1,759	1,759	1,759	1,759
Hidalgo County ID #1	MUNI AWR		1,137	1,137	1,137	1,137	1,137	1,137
Hidalgo County ID #2	MUNI AWR		2,453	2,453	2,453	2,453	2,453	2,453
Hidalgo & Cameron County ID #9	MUNI AWR		3,610	3,610	3,610	3,610	3,610	3,610
Santa Cruz ID #15	MUNI AWR		1,161	1,161	1,161	1,161	1,161	1,161
Total Supply (AF/yr)			**23,973**	**23,973**	**23,973**	**23,973**	**23,973**	**23,973**
Projected SupplySurplus/Deficit			**-1,108**	**-6,448**	**-11,924**	**-17,521**	**-23,213**	**-28,788**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: Donna ID			26	26	26	26	26	26
ID Conservation: HCID No. 1			124	124	124	124	124	124
ID Conservation: HCID No. 1 via Santa Cruz ID			192	192	192	192	192	192
ID Conservation: HCID No. 2			5	5	5	5	5	5
ID Conservation: H&CCID No. 9			116	116	116	116	116	116
ID Conservation: Santa Cruz ID No. 15			113	113	113	113	113	113
Need after Conservation			**-532**	**-5,872**	**-11,348**	**-16,945**	**-22,637**	**-28,212**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
NAWSC Delta Area RO WTP Expansion	$ 22,709,000	$ 1,781	0	0	0	0	1,410	1,410
NAWSC La Sara RO Plant Expansion	$ 13,260,000	$ 2,104	0	0	0	0	0	997
NAWSC Converted WR and Water Treatment Plant No. 5 Expansion**	$ 23,794,000	$ 654	381	3,533	3,533	3,533	3,533	3,533
NAWSC Converted WR and Delta WTP Expansion	$ 23,794,000	$ 505	0	0	3,753	4,900	4,900	4,900
North Cameron Regional WTP Wellfield Expansion	$ 1,881,000	$ 843	492	492	492	492	492	492
Surplus/Deficit after WMS's			**341**	**-1,847**	**-3,570**	**-8,020**	**-12,302**	**-16,880**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
NAWSC Delta Area RO Plant 2 MGD (Willacy County)	$ 22,709,000	$ 1,781	1,032	4,127	4,127	4,127	5,159	6,191
NAWSC La Sara RO Plant, expand well field (Willacy County)	$ 13,260,000	$ 2,104	0	0	4,127	4,127	5,159	5,159
RGRWA			0	1,750	3,100	8,750	12,350	16,950
Surplus/Deficit after WMS's			**500**	**5**	**7**	**60**	**31**	**88**
Transfer to San Perlita			0	0	0	-13	-22	-40
Total Surplus/Deficit			**500**	**5**	**7**	**47**	**9**	**48**

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
OLMITO WSC								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			3,963	4,630	5,307	6,036	6,786	7,554
Water Demand (ac-ft)			732	835	941	1,063	1,192	1,327
Current Water Supply	**Type**							
Cameron County ID #6	MUNI AWR		526	526	526	526	526	526
Total Supply (AF/yr)			**526**	**526**	**526**	**526**	**526**	**526**
Projected Supply Surplus/Deficit			**-206**	**-309**	**-415**	**-537**	**-666**	**-801**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-206**	**-309**	**-415**	**-537**	**-666**	**-801**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 510,000	$ 143	200	200	200	300	300	300
Surplus/Deficit after WMS's			-6	-109	-215	-237	-366	-501
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 765,000	$ 143.00	250	350	450	450	450	450
RGRWA			0	0	0	100	250	400
Surplus/Deficit after WMS's			44	41	35	13	34	49
Region M Alternative Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brackish Desalination Plant	$ 8,400,000	$ 2,582	560	560	560	560	560	560

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
PALM VALLEY								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			1,538	1,797	2,059	2,342	2,633	2,931
Water Demand (ac-ft)			285	324	365	411	462	514
Current Water Supply	**Type**							
Harlingen	MUNI AWR		285	285	285	285	285	285
Total Supply (AF/yr)			**285**	**285**	**285**	**285**	**285**	**285**
Projected Supply Surplus/Deficit			**0**	**-39**	**-80**	**-126**	**-177**	**-229**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**0**	**-39**	**-80**	**-126**	**-177**	**-229**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Harlingen Wastewater Treatment Plant 2 Potable Reuse	$ 19,164,000	$ 1,957	0	0	34	34	34	34
Surplus/Deficit after WMS's			**0**	**-39**	**-46**	**-92**	**-143**	**-195**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfers from Harlingen			0	39	80	126	177	229
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**0**	**0**
Region M Alternative Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Harlingen New BGD Plant	$ 12,327,000	$ 2,180	0	0	19	19	19	19

Deleted WMS
Changed WMS
RGRWA

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS

PORT ISABEL

Year			2020	2030	2040	2050	2060	2070
Total Population			5,903	6,897	7,904	8,990	10,107	11,251
Water Demand (ac-ft)			1,327	1,517	1,714	1,936	2,174	2,419
Current Water Supply	**Type**							
Laguna Madre WD	MUNI AWR/REUSE		724	724	724	724	724	724
Total Supply (AF/yr)			**724**	**724**	**724**	**724**	**724**	**724**
Projected Supply Surplus/Deficit			**-603**	**-793**	**-990**	**-1,212**	**-1,450**	**-1,695**

Evaluation of Selected Water Management Strategies — **Additional Supply by Decade**

Conservation			2020	2030	2040	2050	2060	2070
Need after Conservation			**-603**	**-793**	**-990**	**-1,212**	**-1,450**	**-1,695**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Brackish Desalination Plant	$ 22,443,000	$ 1,773	425	425	425	425	425	425
LMWD Indirect Potable Reuse	$ 13,613,000	$ 2,865	156	156	156	156	156	156
Surplus/Deficit after WMS's			**-22**	**-212**	**-409**	**-631**	**-869**	**-1,114**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Potable Reuse	$ 13,613,000	$ 2,865	156	156	156	156	156	156
RGRWA via LMWD			450	650	850	1,100	1,300	1,550
Surplus/Deficit after WMS's			**3**	**13**	**16**	**44**	**6**	**11**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Non-potable Reuse Project	$ 3,931,000	$ 1,929	0	0	0	0	0	0
LMWD Seawater Desalination Plant	$ 29,609,000	$ 7,175	213	213	213	213	213	213

WATER SUPPLY AND DEMAND ANALYSIS								
PRIMERA								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			4,799	5,607	6,427	7,309	8,217	9,147
Water Demand (ac-ft)			422	472	526	590	661	735
Current Water Supply	**Type**							
Harlingen	MUNI AWR		400	400	400	400	400	400
North Alamo WSC	GW/MUNI AWR		48	48	48	48	48	48
Total Supply (AF/yr)			**448**	**448**	**448**	**448**	**448**	**448**
Projected Supply Surplus/Deficit			**0**	**-24**	**-78**	**-142**	**-213**	**-287**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**0**	**-24**	**-78**	**-142**	**-213**	**-287**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Reverse Osmosis Water Treatment Plant with Ground Storage and Ground Water Well	$ 14,318,000	$ 2,190	1,120	1,120	1,120	1,120	1,120	1,120
Harlingen Wastewater Treatment Plant 2 Potable Reuse	$ 19,164,000	$ 1,957	0	0	48	48	48	48
NAWSC Delta Area RO Plant 2 MGD	$ 22,709,000	$ 1,781	0	0	0	0	4	4
NAWSC La Sara RO Plant, expand well field	$ 13,260,000	$ 2,104	0	0	0	0	0	2
NAWSC Expansion of Water Treatment Plant No. 5	$ 23,794,000	$ 505	0	6	6	6	6	6
NAWSC Expansion of Delta WTP	$ 28,802,000	$ 748	0	0	2	3	3	3
Surplus/Deficit after WMS's			**1,120**	**1,096**	**1,090**	**1,026**	**955**	**881**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfer from Harlingen			0	50	78	142	213	287
Surplus/Deficit after WMS's			**0**	**26**	**0**	**0**	**0**	**0**
Region M Alternative Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Harlingen New BGD Plant	$ 12,327,000	$ 2,180	0	0	26	26	26	26

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
RANCHO VIEJO								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			2,874	3,358	3,848	4,377	4,920	5,477
Water Demand (ac-ft)			835	965	1,099	1,246	1,399	1,557
Current Water Supply	**Type**							
Valley MUD #2	GW/MUNI AWR/REUSE		1,307	1,307	1,307	1,307	1,307	1,307
Total Supply (AF/yr)			**1,307**	**1,307**	**1,307**	**1,307**	**1,307**	**1,307**
Projected Supply Surplus/Deficit			**0**	**0**	**0**	**0**	**-92**	**-250**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**0**	**0**	**0**	**0**	**-92**	**-250**
Region M Recommended Strategy	**Total Annual Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**-92**	**-250**
Proposed Recommended Strategy	**Total Annual Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			0	0	0	0	100	250
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**8**	**0**
Region M Alternate Strategy	**Total Annual Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Valley MUD #2 New BGD Plant	$ 3,760,000	$ 6,430	0	0	0	0	87	87

Legend	
	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
RIO HONDO								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			2,778	3,246	3,720	4,231	4,757	5,295
Water Demand (ac-ft)			204	224	251	285	320	356
Current Water Supply	**Type**							
Cameron County ID #2	MUNI AWR		605	605	605	605	605	605
Total Supply (AF/yr)			**605**	**605**	**605**	**605**	**605**	**605**
Projected Supply Surplus/Deficit			**0**	**0**	**0**	**0**	**0**	**0**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: CCID #2			44	44	44	44	44	44
Need after Conservation			**44**	**44**	**44**	**44**	**44**	**44**

	Deleted WMS
	Changed WMS
	RGRWA

No strategies reccomended in Region M plan, because none are needed

WATER SUPPLY AND DEMAND ANALYSIS								
SAN BENITO								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			28,594	33,406	38,289	43,547	48,956	54,500
Water Demand (ac-ft)			3,607	4,053	4,529	5,088	5,705	6,346
Current Water Supply	**Type**							
Cameron County ID #2	MUNI AWR		4,782	4,782	4,782	4,782	4,782	4,782
Total Supply (AF/yr)			**4,782**	**4,782**	**4,782**	**4,782**	**4,782**	**4,782**
Projected Supply Surplus/Deficit			**0**	**0**	**0**	**-306**	**-923**	**-1,564**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservatin: CCID No. 2			348	348	348	348	348	348
Need after Conservation			**348**	**348**	**348**	**42**	**-575**	**-1,216**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brackish Groundwater Supply	$ 2,033,000	$ 181	1,120	1,120	1,120	1,120	1,120	1,120
Surplus/Deficit after WMS's			**1,468**	**1,468**	**1,468**	**1,162**	**545**	**-96**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			0	0	0	0	600	1,250
Surplus/Deficit after WMS's			**348**	**348**	**348**	**42**	**25**	**34**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Non-Potable Reuse	$ 1,921,000	$ 192	1,120	1,120	1,120	1,120	1,120	1,120
Potable Reuse (Phase I)	$ 11,303,000	$ 1,349	1,120	1,120	1,120	1,120	1,120	0
Potable Reuse (Phase II)	$ 18,148,000	$ 733	0	0	0	0	0	3,360

Legend	
(light green)	Deleted WMS
(peach)	Changed WMS
(green)	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
SANTA ROSA								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			3,388	3,958	4,537	5,160	5,800	6,457
Water Demand (ac-ft)			295	325	358	400	448	498
Current Water Supply	**Type**							
La Feria ID, CCID #3	MUNI AWR		238	238	238	238	238	238
Total Supply (AF/yr)			**238**	**238**	**238**	**238**	**238**	**238**
Projected Supply Surplus/Deficit			**-57**	**-87**	**-120**	**-162**	**-210**	**-260**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: La Feria, CCID No. 3			33	33	33	33	33	33
Need after Conservation			**-24**	**-54**	**-87**	**-129**	**-177**	**-227**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 297,500	$ 143	0	25	50	100	150	175
Surplus/Deficit after WMS's			**-24**	**-29**	**-37**	**-29**	**-27**	**-52**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 952,000	$ 143	25	55	90	130	180	230
Surplus/Deficit after WMS's			**1**	**1**	**3**	**1**	**3**	**3**
Region M Alternative Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brackish Desalination Plant	$ 8,272,000	$ 2,559	0	560	560	560	560	560

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
SOUTH PADRE ISLAND								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			3,321	3,880	4,447	5,057	5,685	6,329
Water Demand (ac-ft)			3,228	3,755	4,292	4,875	5,478	6,098
Current Water Supply	**Type**							
Laguna Madre WD	MUNI AWR/REUSE		1,762	1,762	1,762	1,762	1,762	1,762
Total Supply (AF/yr)			**1,762**	**1,762**	**1,762**	**1,762**	**1,762**	**1,762**
Projected Supply Surplus/Deficit			**-1,469**	**-1,996**	**-2,533**	**-3,116**	**-3,719**	**-4,339**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-1,469**	**-1,996**	**-2,533**	**-3,116**	**-3,719**	**-4,339**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Brackish Desalination Plant	$ 22,443,000	$ 1,773	1,034	1,034	1,034	1,034	1,034	1,034
LMWD Potable Reuse	$ 13,613,000	$ 2,865	379	379	379	379	379	379
Surplus/Deficit after WMS's			**-56**	**-583**	**-1,120**	**-1,703**	**-2,306**	**-2,926**
Proposed Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Potable Reuse	$ 13,613,000	$ 2,865	379	379	379	379	379	379
RGRWA via LMWD			1,100	1,650	2,200	2,750	3,350	4,000
Surplus/Deficit after WMS's			**10**	**33**	**46**	**13**	**10**	**40**
Region M Alternative Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
LMWD Non-potable Reuse Project	$ 3,931,000	$ 7,175	162	162	162	162	162	162
LMWD Seawater Desalination Plant	$ 29,609,000	$ 1,929	517	517	517	517	517	517

Legend
Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS

Agua SUD

		2020	2030	2040	2050	2060	2070
Year		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population		52,424	65,063	77,749	90,460	103,168	115,519
Water Demand		5622	6771	7963	9194	10459	11700
Current Water Supply/Supplier	**Type**	2020	2030	2040	2050	2060	2070
Hidlago County ID No. 16	MUNI AWR	2596	2596	2596	2596	2596	2596
Hidlago County ID No. 6	MUNI AWR	4129	4129	4129	4129	4129	4129
Total Supply (AF/yr)		**6,725**	**6,725**	**6,725**	**6,725**	**6,725**	**6,725**
Projected Supply Surplus/Deficit		**-769**	**-1,918**	**-1,037**	**-4,341**	**-5,606**	**-6,847**

Evaluation of Selected Water Management Strategies — **Additional Supply by Decade**

Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: HCID #16			138	138	138	138	138	138
Need after Conservation			**-631**	**-1,780**	**-899**	**-4,203**	**-5,468**	**-6,709**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West Agua SUD Potable Reuse Phase I	$ 14,455,000	$ 2,974	565	565	0	0	0	0
West Agua SUD Potable Reuse Phase II	$ 8,796,000	$ 2,145	0	0	780	780	780	780
East Agua SUD Potable Reuse Phase I	$ 13,019,000	$ 2,358	756	756	756	0	0	0
East Agua SUD Potable Reuse Phase II	$ 3,561,000	$ 3,881	0	0	0	840	840	840
Acquisition of Water Rights through Urbanization	$ 4,420,000	$ 143	180	360	900	1,620	2,340	2,340
Supply Surplus/Deficit after WMS			870	-99	1,537	-963	-1,508	-2,749
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West Agua SUD Potable Reuse Phase I	$ 14,455,000	$ 2,974	565	565	0	0	0	0
West Agua SUD Potable Reuse Phase II	$ 8,796,000	$ 2,145	0	0	780	780	780	780
East Agua SUD Potable Reuse Phase I	$ 13,019,000	$ 2,358	756	756	756	0	0	0
East Agua SUD Potable Reuse Phase II	$ 3,561,000	$ 3,881	0	0	0	840	840	840
RGRWA			0	700	700	2,900	4,600	6,350
Supply Surplus/Deficit after WMS			690	241	1,337	317	752	1,261
Transfer to Palmview			0	-81	-16	-134	-302	-468
Transfer to Penitas			0	-70	-24	-118	-256	-392
Transfer to Sullivan City			0	-50	0	-33	-153	-271
Transfer to La Joya			0	0	0	0	0	-110
Total Surplus/Deficit			**690**	**40**	**1,297**	**32**	**41**	**20**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Agua SUD New BGD Plant	$ 18,432,000	$ 5.86	0	0	0	1,212	1,212	1,212
Agua SUD Non-Potable Reuse			1,129	1,129	1,573	1,573	1,573	1,573

Legend	
Deleted WMS	
Changed WMS	
RGRWA	

WATER SUPPLY AND DEMAND ANALYSIS								
ALAMO								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			23,259	28,881	34,525	40,181	45,837	51,335
Water Demand			3231	3909	4607	5326	6064	6787
Current Water Supply/Supplier	**Type**		2020	2030	2040	2050	2060	2070
Hidlago County ID No. 2	MUNI AWR		1603	1603	1603	1603	1603	1603
Groundwater	GW		624	624	624	624	624	624
Total Supply (AF/yr)			**2,227**	**2,227**	**2,227**	**2,227**	**2,227**	**2,227**
Projected Supply Surplus/Deficit			**-1,004**	**-1,682**	**-2,380**	**-3,099**	**-3,837**	**-4,560**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: HCID #2			189	189	189	189	189	189
Need after Conservation			**-815**	**-1,493**	**-2,191**	**-2,910**	**-3,648**	**-4,371**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Alamo Groundwater Well	$ 1,181,000	$ 113	1,120	1,120	1,120	1,120	1,120	1,120
Alamo BGD Plant	$ 13,532,000	$ 2,655	1,000	1,000	1,000	1,000	1,000	1,000
Acquisition of Water Rights through Urbanization	$ 1,700,000	$ 143	0	0	0	1,000	1,000	1,000
NAWSC Converted WR and Water Treatment Plant No. 5 Expansion	$ 23,794,000	$ 505	50	50	50	50	50	50
Net Supply Surplus/Deficit			**1,305**	**627**	**-71**	**210**	**-528**	**-1,251**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			850	1,500	2,200	2,950	3,650	4,400
Net Supply Surplus/Deficit			**35**	**7**	**9**	**40**	**2**	**29**

Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
ALTON								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			15,640	19,420	23,215	27,019	30,822	34,519
Water Demand			2071	2524	2990	3464	3943	4413
Current Water Supply/Supplier	**Type**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Sharlyand WSC	MUNI AWR		1286	1286	1286	1286	1286	1286
Total Supply (AF/yr)			**1,286**	**1,286**	**1,286**	**1,286**	**1,286**	**1,286**
Projected Supply Surplus/Deficit			**-785**	**-1,238**	**-1,704**	**-2,178**	**-2,657**	**-3,127**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-785**	**-1,238**	**-1,704**	**-2,178**	**-2,657**	**-3,127**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Sharyland WSC Water Well and R.O. Unit at WTP #2	$ 13,253,000	$ 2,630	201	201	201	201	201	201
Sharyland WSC Water Well and R.O. Unit at WTP #3	$ 13,253,000	$ 2,630	201	201	201	201	201	201
Sharyland Water Rights through Urbanization	$ 12,750,000	$ 143	690	2,050	3,450	4,950	7,400	7,500
Surplus/Deficit after WMS's			**307**	**1,214**	**2,148**	**3,174**	**5,145**	**4,775**
Proposed Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfer from Sharyland WSC			785	1238	1704	2178	2657	3127
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**0**	**0**

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
DONNA								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			20,021	24,860	29,719	34,587	39,456	44,189
Water Demand (ac-ft)			2,610	3,126	3,660	4,219	4,802	5,375
Current Water Supply	**Type**							
Donna ID	MUNI AWR		2,975	2,975	2,975	2,975	2,975	2,975
Total Supply (AF/yr)			**2,975**	**2,975**	**2,975**	**2,975**	**2,975**	**2,975**
Projected Supply Surplus/Deficit			**365**	**-151**	**-685**	**-1,244**	**-1,827**	**-2,400**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: Donna ID			40	40	40	40	40	40
Need after Conservation			**405**	**-111**	**-645**	**-1,204**	**-1,787**	**-2,360**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Donna WTP Expansion	$ 24,107,000	$ 2,512	995	2,240	2,240	2,240	2,240	2,240
NAWSC Converted WR and Water Treatment Plant No. 5 Expansion	$ 23,794,000	$ 505	0	50	50	50	50	50
Net Supply Surplus/Deficit			**1,400**	**2,179**	**1,645**	**1,086**	**503**	**-70**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			0	150	650	1,250	1,800	2,400
Net Supply Surplus/Deficit			**405**	**39**	**5**	**46**	**13**	**40**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Donna Brackish Groundwater Desalination Plant with Wells (phase I)	$ 9,440,000	$ 2,349	700	700	700	0	0	0
Donna Brackish Groundwater Desalination Plant with Wells (phase II)	$ 5,849,000	$ 3,357	0	0	0	1,000	1,000	1,000

Legend	
Deleted WMS	
Changed WMS	
RGRWA	

WATER SUPPLY AND DEMAND ANALYSIS								
EDCOUCH								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			4,006	4,974	5,946	6,920	7,894	8,841
Water Demand (ac-ft)			358	419	484	554	630	705
Current Water Supply	**Type**							
Hidalgo & Cameron County ID #9	MUNI AWR		330	330	330	330	330	330
Total Supply (AF/yr)			**330**	**330**	**330**	**330**	**330**	**330**
Projected Supply Surplus/Deficit			**-28**	**-89**	**-154**	**-224**	**-300**	**-375**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: H&CC ID #9			8	8	8	8	8	8
Need after Conservation			**-20**	**-81**	**-146**	**-216**	**-292**	**-367**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights - HCCID9	$ 170,000.00	$ 143	40	40	40	100	100	100
Groundwater Supply	$ 1,106,000	$ 218	725	725	725	725	725	725
NAWSC Converted WR and Delta WTP Expansion	$ 42,504,000	$ 748	0	0	50	50	50	50
Net Supply Surplus/Deficit			**745**	**684**	**669**	**659**	**583**	**508**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Groundwater Supply	$ 1,106,000	$ 218.00	725	725	725	725	725	725
Net Supply Surplus/Deficit			**705**	**644**	**579**	**509**	**433**	**358**

Legend	
	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
EDINBURG								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			97,711	121,329	145,041	168,800	192,560	215,659
Water Demand (ac-ft)			13,113	15,899	18,772	21,714	24,721	27,667
Current Water Supply	**Type**							
Hidalgo County ID #1	MUNI AWR		6,766	6,766	6,766	6,766	6,766	6,766
Hidalgo County ID #2	MUNI AWR		2,056	2,056	2,056	2,056	2,056	2,056
McAllen	GW		32	32	32	32	32	32
North Alamo WSC	MUNI AWR		34	34	34	34	34	34
Reuse	REUSE		1	1	1	1	1	1
Total Supply (AF/yr)			**8,889**	**8,889**	**8,889**	**8,889**	**8,889**	**8,889**
Projected Supply Surplus/Deficit			**-4,224**	**-7,010**	**-9,883**	**-12,825**	**-15,832**	**-18,778**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: HCID #1			677	677	677	677	677	677
ID Conservation: HCID #2			14	14	14	14	14	14
Need after Conservation			**-3,533**	**-6,319**	**-9,192**	**-12,134**	**-15,141**	**-18,087**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Reuse Water for Cooling Tower and Landscaping	$ 9,971,000	$ 400	2,622	3,180	3,754	3,920	3,920	3,920
NAWSC Delta Area RO Plant 2 MGD	$ 13,260,000	$ 2,104	0	0	0	0	4	4
NAWSC La Sara RO Plant, expand well field	$ 13,153,000	$ 2,156	0	0	0	0	0	2
NAWSC Converted WR and Water Treatment Plant No. 5 Expansion	$ 42,504,000	$ 748	205	205	205	205	205	205
NAWSC Converted WR and Delta WTP Expansion	$ 42,504,000	$ 748	0	0	12	20	20	20
North Cameron Regional WTP Wellfield Expansion	$ 1,462,000	$ 843	0	0	0	0	0	0
Acquisition of Water Rights through Urbanization - HCID#1	$ 6,800,000	$ 143	100	1,000	1,500	2,500	4,000	4,000
Acquisition of Water Rights through Urbanization - HCID#2	$ 6,800,000	$ 143	100	1,100	2,000	3,000	4,000	4,000
Net Supply Surplus/Deficit			**-606**	**-1,934**	**-3,721**	**-5,489**	**-6,992**	**-9,936**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
NAWSC Delta Area RO Plant 2 MGD	$ 13,260,000	$ 2,104	2	8	8	8	10	12
NAWSC La Sara RO Plant, expand well field	$ 13,153,000	$ 2,156	0	0	8	8	10	10
RGRWA			3,550	6,350	9,200	12,150	15,150	18,100
Net Supply Surplus/Deficit			**19**	**39**	**24**	**32**	**30**	**36**

Legend	
Deleted WMS	
Changed WMS	
RGRWA	

WATER SUPPLY AND DEMAND ANALYSIS							
ELSA							
Year		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population		7,173	8,906	10,647	12,391	14,136	15,831
Water Demand (ac-ft)		811	963	1,121	1,289	1,466	1,641
Current Water Supply	**Type**						
Hidalgo & Cameron County #9	MUNI AWR	910	909	909	909	908	908
Total Supply (AF/yr)		**910**	**909**	**909**	**909**	**908**	**908**
Projected Supply Surplus/Deficit		**99**	**-54**	**-212**	**-380**	**-558**	**-733**

Evaluation of Selected Water Management Strategies — **Additional Supply by Decade**

Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: H&CC ID #9			21	21	21	21	21	21
Need after Conservation			**120**	**-33**	**-191**	**-359**	**-537**	**-712**
Region M Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 952,000	$ 143	0	0	70	200	260	310
NAWSC Converted WR and Delta WTP Expansion	$ 42,504,000	$ 748	0	0	200	200	200	200
Net Supply Surplus/Deficit			**120**	**-33**	**79**	**41**	**-77**	**-202**
Proposed Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 952,000		0	35	195	360	540	715
Net Supply Surplus/Deficit			**120**	**2**	**4**	**1**	**3**	**3**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
New Brackish Water Treatment Plant	$ 8,276,000	$ 2,564.00	560	560	560	560	560	560
WTP Expansion and Interconnect to Engleman ID	$ 9,836,000	$ 671.00	2,240	2,240	2,240	2,240	2,240	2,240

Legend	
(light green)	Deleted WMS
(peach)	Changed WMS
(green)	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
HIDALGO								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			14,191	17,621	21,065	24,516	27,967	31,322
Water Demand			1859	2254	2662	3079	3505	3923
Current Water Supply/Supplier	**Type**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Direct Source	GW/MUNI AWR		1499	1499	1499	1499	1499	1499
Total Supply (AF/yr)			**1,499**	**1,499**	**1,499**	**1,499**	**1,499**	**1,499**
Projected Supply Surplus/Deficit			**-360**	**-755**	**-1,163**	**-1,580**	**-2,006**	**-2,424**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-360**	**-755**	**-1,163**	**-1,580**	**-2,006**	**-2,424**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Expand Existing Groundwater Wells	$ 656,000	$ 260	300	300	300	300	300	300
Acquisition of Water Rights through Urbanization	$ 3,660,000	$ 211	400	500	1,050	1,050	1,500	1,500
Surplus/Deficit after WMS's			**340**	**45**	**187**	**-230**	**-206**	**-624**
Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			400	800	1,200	1,600	2,050	2,450
Surplus/Deficit after WMS's			**40**	**45**	**37**	**20**	**44**	**26**

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
HIDALGO COUNTY MUNICIPAL UTILITY DISTRICT NO. 1								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			6,858	8,516	10,181	11,848	13,516	15,138
Water Demand			570	682	801	923	1,049	1,174
Current Water Supply/Supplier	**Type**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Hidalgo County Irrigation District No. 1	MUNI AWR		273	273	273	273	273	273
Total Supply (AF/yr)			**273**	**273**	**273**	**273**	**273**	**273**
Projected Supply Surplus/Deficit			**-297**	**-409**	**-528**	**-650**	**-776**	**-901**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: HCID#1			27	27	27	27	27	27
Need after Conservation			**-270**	**-382**	**-501**	**-623**	**-749**	**-874**
Region M RecommendedStrategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 2,550,000	$ 143	500	500	500	1,500	1,500	1,500
Surplus/Deficit after WMS's			**230**	**118**	**-1**	**877**	**751**	**626**
Proposed Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			300	450	550	650	800	950
Surplus/Deficit after WMS's			**3**	**41**	**22**	**0**	**24**	**49**

	Deleted WMS
	Changed WMS
	RGRWA

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS

LA JOYA

		2020	2030	2040	2050	2060	2070
Year		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population		5,050	6,271	7,496	8,724	9,952	11,146
Water Demand		652	783	919	1060	1207	1351
Current Water Supply/Supplier	**Type**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Groundwater	GW	595	595	595	595	595	595
Agua SUD	MUNI AWR	159	159	159	159	159	159
Hidalgo County ID #16	MUNI AWR	388	388	388	388	388	388
Total Supply (AF/yr)		**1,142**	**1,142**	**1,142**	**1,142**	**1,142**	**1,142**
Projected Supply Surplus/Deficit		**0**	**0**	**0**	**0**	**-65**	**-209**

Evaluation of Selected Water Management Strategies — **Additional Supply by Decade**

Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: HCID No. 16			20	20	20	20	20	20
Need after Conservation			**20**	**20**	**20**	**20**	**-45**	**-189**
Region M Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974	18	18	18	18	18	18
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145	0	0	27	27	27	27
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358	25	25	25	25	25	25
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881	0	0	0	8	8	8
Surplus/Deficit after WMS's			**63**	**63**	**90**	**99**	**34**	**-110**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974	18	18	18	18	18	18
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145	0	0	27	27	27	27
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358	25	25	25	25	25	25
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881	0	0	0	8	8	8
Transfer from Agua SUD			0	0	0	0	0	110
Surplus/Deficit after WMS's			**63**	**63**	**90**	**99**	**34**	**0**
Region M Alternate Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Agua SUD New BGD Plant	$ 181,136,000	$ 2,649	0	0	0	40	40	40
Agua SUD Non-Potable Reuse	$ 4,026,000.00	$ 2,946	37	37	51	51	51	51

WATER SUPPLY AND DEMAND ANALYSIS								
LA VILLA								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			2,480	3,079	3,681	4,284	4,887	5,474
Water Demand (ac-ft)			275	328	385	443	504	564
Current Water Supply	**Type**							
Hidalgo & Cameron County ID #9	MUNI AWR		246	246	246	246	246	246
Total Supply (AF/yr)			**246**	**246**	**246**	**246**	**246**	**246**
Projected Supply Surplus/Deficit			**-29**	**-82**	**-139**	**-197**	**-258**	**-318**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: H&CC ID #9			7	7	7	7	7	7
Need after Conservation			**-22**	**-75**	**-132**	**-190**	**-251**	**-311**
Region M Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 340,000	$ 143	100	100	100	200	200	200
NAWSC Converted WR and Delta WTP Expansion	$ 42,504,000	$ 748	0	0	100	100	100	100
Net Supply Surplus/Deficit			**78**	**25**	**68**	**110**	**49**	**-11**
Proposed Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Acquisition of Water Rights through Urbanization	$ 952,000		25	75	135	190	255	315
Net Supply Surplus/Deficit			**3**	**0**	**3**	**0**	**4**	**4**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
New Brackish Water Treatment Plant	$8,276,000	$ 2,558	560	560	560	560	560	560

Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
MCALLEN								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			164,597	204,382	244,325	284,348	324,372	363,284
Water Demand (ac-ft)			38,728	47,219	55,875	64,722	73,748	82,563
Current Water Supply	**Type**							
Reuse	REUSE		2,251	2,251	2,251	2,251	2,251	2,251
Groundwater	GW		306	306	306	306	306	306
Hidalgo County ID #1	MUNI AWR		2,840	2,840	2,840	2,840	2,840	2,840
Hidalgo County ID #2	MUNI AWR		5,759	5,759	5,759	5,759	5,759	5,759
Hidalgo County WID #3	MUNI AWR		11,609	11,609	11,609	11,609	11,609	11,609
United ID	MUNI AWR		7,988	7,988	7,988	7,988	7,988	7,988
Total Supply (AF/yr)			**30,753**	**30,753**	**30,753**	**30,753**	**30,753**	**30,753**
Projected Supply Surplus/Deficit			**-7,975**	**-16,466**	**-25,122**	**-33,969**	**-42,995**	**-51,810**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: HCID #1			284	284	284	284	284	284
ID Conservation: HCID #2			221	221	221	221	221	221
ID Conservation: HC WID #3			1,452	1,452	1,452	1,452	1,452	1,452
ID Conservation: United ID			911	911	911	911	911	911
Need after Conservation			**-5,107**	**-13,598**	**-22,254**	**-31,101**	**-40,127**	**-48,942**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Raw Water Line Project	$ 1,662,000	$ 225	800	800	800	800	800	800
South WWTP Potable Reuse -Phase I	$ 20,143,000	$ 1,958	0	2,000	0	0	0	0
South WWTP Potable Reuse -Phase II	$ 6,232,000	$ 2,702	0	0	2,500	0	0	0
South WWTP Potable Reuse -Phase III	$ 9,732,000	$ 2,101	0	0	0	3,500	3,500	3,500
North WWTP Potable Reuse -Phase I	$ 14,145,000	$ 2,353	0	0	1,120	0	0	0
North WWTP Potable Reuse -Phase II	$ 8,888,000	$ 989	0	0	0	2,000	2,000	2,000
Brackish Groundwater Desalination Plant	$ 21,946,000	$ 2,043	2,688	2,688	2,688	2,688	2,688	2,688
Acquisition of Water Rights through Urbanization	$ 7,990,000	$ 143.00	0	0	800	800	2,200	4,700
Net Supply Surplus/Deficit			**-1,619**	**-8,110**	**-14,346**	**-21,313**	**-28,939**	**-35,254**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Raw Water Line Project	$ 1,662,000	$ 225	800	800	800	800	800	800
RGRWA			4,350	12,800	21,500	30,350	39,350	48,150
Net Supply Surplus/Deficit			**43**	**2**	**46**	**49**	**23**	**8**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Expand Existing Groundwater Wells - Phase I	$ 940,000	$ 235	0	500	500	500	0	0
Expand Existing Groundwater Wells - Phase II	$ 1,004,000	$ 124	0	0	0	0	1,500	1,500
McAllen Non-Potable Reuse	$ 12,123,000	$ 1,064	1,950	1,950	1,950	1,950	1,950	1,950

Legend	
Deleted WMS	
Changed WMS	
RGRWA	

WATER SUPPLY AND DEMAND ANALYSIS								
MERCEDES								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			19,732	24,501	29,290	34,088	38,886	43,551
Water Demand			2223	2648	3091	3558	4049	4531
Current Water Supply/Supplier	**Type**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Groundwater	GW		655	655	655	655	655	655
Hidalgo & Cameron County ID #9	MUNI AWR		1288	1288	1288	1288	1288	1288
Total Supply (AF/yr)			**1,943**	**1,943**	**1,943**	**1,943**	**1,943**	**1,943**
Projected Supply Surplus/Deficit			**-280**	**-705**	**-1,148**	**-1,615**	**-2,106**	**-2,588**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: H&CC ID No. 9			38	38	38	38	38	38
Need after Conservation			**-242**	**-667**	**-1,110**	**-1,577**	**-2,068**	**-2,550**
Region M Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Potable Reuse	$ 11,722,000	$ 1,958	1,670	1,670	1,670	1,670	1,670	1,670
Surplus/Deficit after WMS's			**1,428**	**1,003**	**560**	**93**	**-398**	**-880**
Proposed Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			250	700	1,150	1,600	2,100	2,550
Surplus/Deficit after WMS's			**8**	**33**	**40**	**23**	**32**	**0**
Region M Alternate Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Expand Existing Groundwater Wells	$ 1,001,000	$ 222	560	560	560	560	560	560
Brackish Groundwater Desalination Plant	$ 12,062,000	$ 4,920	0	0	435	435	435	435

Legend
Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
MISSION								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			97,658	121,263	144,962	168,708	192,455	215,541
Water Demand (ac-ft)			20,212	24,704	29,290	33,954	38,684	43,305
Current Water Supply	**Type**							
Agua SUD	MUNI AWR		28	28	28	28	28	28
McAllen	GW		84	84	84	84	84	84
United ID	MUNI AWR		12,078	12,078	12,078	12,078	12,078	12,078
Total Supply (AF/yr)			**12,190**	**12,190**	**12,190**	**12,190**	**12,190**	**12,190**
Projected Supply Surplus/Deficit			**-8,022**	**-12,514**	**-17,100**	**-21,764**	**-26,494**	**-31,115**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: United ID			1,399	1,399	1,399	1,399	1,399	1,399
Need after Conservation			**-6,623**	**-11,115**	**-15,701**	**-20,365**	**-25,095**	**-29,716**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Brackish Groundwater Desalination Plant	$ 31,914,000	$ 2,069	2,688	2,688	2,688	2,688	2,688	2,688
Potable Reuse -Phase I	$ 32,565,000	$ 1,572	3,920	3,920	3,920	3,920	0	0
Potable Reuse -Phase II	$ 27,630,000	$ 734	0	0	0	0	7,840	7,840
West Agua SUD Potable Reuse Phase I	$ 14,455,000	$ 2,974	3	3	3	3	3	3
West Agua SUD Potable Reuse Phase II	$ 8,796,000	$ 2,145	0	0	9	9	9	9
East Agua SUD Potable Reuse Phase I	$ 13,019,000	$ 2,358	4	4	4	4	4	4
East Agua SUD Potable Reuse Phase II	$ 3,561,000	$ 3,881	0	0	0	6	6	6
Acquisition of Water Rights through Urbanization	$ 5,950,000	$ 143	0	600	2,100	3,500	3,500	3,500
Net Supply Surplus/Deficit			**-7**	**-3,899**	**-6,976**	**-10,234**	**-11,044**	**-15,665**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West Agua SUD Potable Reuse Phase I	$ 14,455,000	$ 2,974	3	3	3	3	3	3
West Agua SUD Potable Reuse Phase II	$ 8,796,000	$ 2,145	0	0	9	9	9	9
East Agua SUD Potable Reuse Phase I	$ 13,019,000	$ 2,358	4	4	4	4	4	4
East Agua SUD Potable Reuse Phase II	$ 3,561,000	$ 3,881	0	0	0	6	6	6
RGRWA			6,650	11,150	15,700	20,350	25,100	29,700
Net Supply Surplus/Deficit			**35**	**43**	**16**	**8**	**28**	**7**
Region M Alternative Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Agua SUD New BGD Plant	$ 18,136,000	$ 2,649	0	0	0	7	7	7
Agua SUD Non-Potable Reuse	$ 4,026,000.00	$ 2,946	7	7	9	9	9	9

Legend
Deleted WMS
Changed WMS
RGRWA

Deleted WMS	Changed WMS	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS

PALMHURST

Year			2020	2030	2040	2050	2060	2070
Total Population			3,303	4,102	4,904	5,707	6,511	7,292
Water Demand			932	1149	1369	1591	1813	2030
Current Water Supply/Supplier	**Type**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Sharyland WSC	MUNI AWR		579	579	579	579	579	579
Total Supply (AF/yr)			**579**	**579**	**579**	**579**	**579**	**579**
Projected Supply Surplus/Deficit			**-353**	**-570**	**-790**	**-1,012**	**-1,234**	**-1,451**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-353**	**-570**	**-790**	**-1,012**	**-1,234**	**-1,451**
Region M Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Sharyland WSC Water Well and R.O. Unit at WTP #2	$ 13,253,000	$ 2,630	90	90	90	90	90	90
Sharyland WSC Water Well and R.O. Unit at WTP #3	$ 13,253,000	$ 2,630	72	72	72	72	72	72
Sharyland WSC Acquisition of Water Rights through Urbanization - United ID	$ 2,040,000	$ 143	8	15	15	15	84	90
Sharyland WSC Acquisition of Water Rights through Urbanization - HCID #1	$ 8,160,000	$ 143	10	25	60	60	60	60
Sharyland WSC Acquisition of Water Rights through Urbanization - Santa Cruz ID	$ 2,550,000	$ 143	21	78	120	210	288	288
Surplus/Deficit after WMS's			**-183**	**-393**	**-613**	**-835**	**-988**	**-1,199**
Proposed Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfer from Sharyland WSC			353	570	790	1,012	1,234	1,451
Surplus/Deficit after WMS's			**0**	**0**	**0**	**0**	**0**	**0**

WATER SUPPLY AND DEMAND ANALYSIS								
PALMVIEW								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			6,919	8,592	10,271	11,953	13,636	15,272
Water Demand (ac-ft)			743	897	1,056	1,220	1,388	1,554
Current Water Supply	**Type**							
Agua SUD	MUNI AWR		641	641	641	641	641	641
Total Supply (AF/yr)			**641**	**641**	**641**	**641**	**641**	**641**
Projected Supply Surplus/Deficit			**-102**	**-256**	**-415**	**-579**	**-747**	**-913**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-102**	**-256**	**-415**	**-579**	**-747**	**-913**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974	75	75	75	75	75	75
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145	0	0	224	224	224	224
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358	100	100	100	100	100	100
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881	0	0	0	46	46	46
Acquisition of Water Rights through Urbanization		$ 143	8	16	40	72	104	104
Net Supply Surplus/Deficit			**81**	**-65**	**24**	**-62**	**-198**	**-364**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974	75	75	75	75	75	75
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145	0	0	224	224	224	224
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358	100	100	100	100	100	100
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881	0	0	0	46	46	46
Transfer from Agua SUD			0	81	16	134	302	468
Net Supply Surplus/Deficit			**73**	**0**	**0**	**0**	**0**	**0**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Agua SUD New BGD Plant	$ 18,136,000	$ 2,649	0	0	0	160	160	160
Agua SUD Non-Potable Reuse	$ 4,026,000	$ 2,946	149	149	208	208	208	208

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
PENITAS								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			5,580	6,928	8,282	9,639	10,996	12,315
Water Demand (ac-ft)			603	732	865	1,001	1,139	1,275
Current Water Supply	**Type**							
Agua SUD	MUNI AWR		520	520	520	520	520	520
Total Supply (AF/yr)			**520**	**520**	**520**	**520**	**520**	**520**
Projected Supply Surplus/Deficit			**-83**	**-212**	**-345**	**-481**	**-619**	**-755**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-83**	**-212**	**-345**	**-481**	**-619**	**-755**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974	61	61	61	61	61	61
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145	0	0	179	179	179	179
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358	81	81	81	81	81	81
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881	0	0	0	42	42	42
Acquisition of Water Rights through Urbanization		$ 143	4	8	20	36	52	52
Net Supply Surplus/Deficit			**63**	**-62**	**-4**	**-82**	**-204**	**-340**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974.00	61	61	61	61	61	61
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145.00	0	0	179	179	179	179
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358.00	81	81	81	81	81	81
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881.00	0	0	0	42	42	42
Transfer from Agua SUD			0	70	24	118	256	392
Net Supply Surplus/Deficit			**59**	**0**	**0**	**0**	**0**	**0**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/1000 gal)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Agua SUD New BGD Plant	$ 18,136,000	$ 2,649	0	0	0	130	130	130
Agua SUD Non-Potable Reuse	$ 4,026,000	$ 2,946	121	121	169	169	169	169

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
PHARR								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			89,220	110,785	132,437	154,131	175,826	196,918
Water Demand (ac-ft)			9,923	11,933	14,021	16,183	18,415	20,607
Current Water Supply	**Type**							
Hidalgo County ID #2	MUNI AWR		6,741	6,741	6,741	6,741	6,741	6,741
Reuse	REUSE		3,076	3,076	3,076	3,076	3,076	3,076
Total Supply (AF/yr)			**9,817**	**9,817**	**9,817**	**9,817**	**9,817**	**9,817**
Projected Supply Surplus/Deficit			**-106**	**-2,116**	**-4,204**	**-6,366**	**-8,598**	**-10,790**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: HCID #2			70	70	70	70	70	70
Need after Conservation			**-36**	**-2,046**	**-4,134**	**-6,296**	**-8,528**	**-10,720**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Potable Reuse	$ 38,422,000	$ 808	6,721	6,721	6,721	6,721	6,721	6,721
Net Supply Surplus/Deficit			**6,685**	**4,675**	**2,587**	**425**	**-1,807**	**-3,999**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			50	2,050	4,150	6,300	8,600	10,750
Net Supply Surplus/Deficit			**14**	**4**	**16**	**4**	**72**	**30**
Region M Alternative Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**

Deleted WMS
Changed WMS
RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
PROGRESO								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			6,979	8,666	10,359	12,056	13,753	15,403
Water Demand (ac-ft)			722	868	1,020	1,177	1,339	1,498
Current Water Supply	**Type**							
Military Highway WSC	MUNI AWR		696	696	696	696	696	696
Total Supply (AF/yr)			**696**	**696**	**696**	**696**	**696**	**696**
Projected Supply Surplus/Deficit			**-26**	**-172**	**-324**	**-481**	**-643**	**-802**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-26**	**-172**	**-324**	**-481**	**-643**	**-802**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
MHWSC Expand Existing Groundwater Wells (Cameron Co.)	$ 5,373,000	$ 1,254	150	150	150	150	150	150
MHWSC Acquisition of Water Rights through Urbanization	$ 974,297	$ 143	34	139	227	321	460	573
ERHWSC New Surface WTP via MHWSC	$ 34,794,000	$ 736	100	100	100	100	100	100
Net Supply Surplus/Deficit			**258**	**217**	**153**	**90**	**67**	**21**
Proposed Recommended Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Transfer from Military Highway WSC			26	172	324	481	643	802
Net Supply Surplus/Deficit			**0**	**0**	**0**	**0**	**0**	**0**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase I)	$ 668,000	$ 316	31	31	31	0	0	0
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase II)	$ 810,000	$ 195	0	0	0	79	79	79

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS

SAN JUAN

Year			2020	2030	2040	2050	2060	2070
Total Population			42,906	53,277	63,690	74,123	84,556	94,699
Water Demand (ac-ft)			6,152	7,448	8,782	10,154	11,561	12,940
Current Water Supply	**Type**							
Groundwater	GW		404	404	404	404	404	404
Hidalgo County ID #2	MUNI AWR		2,141	2,141	2,141	2,141	2,141	2,141
Military Highway WSC	GW/MUNI AWR		43	43	43	43	43	43
North Alamo WSC	GW/MUNI AWR		1,672	1,672	1,672	1,672	1,672	1,672
Total Supply (AF/yr)			**4,260**	**4,260**	**4,260**	**4,260**	**4,260**	**4,260**
Projected Supply Surplus/Deficit			**-1,892**	**-3,188**	**-4,522**	**-5,894**	**-7,301**	**-8,680**

Evaluation of Selected Water Management Strategies — **Additional Supply by Decade**

Conservation			2020	2030	2040	2050	2060	2070
ID Conservation: HCID #2			74	74	74	74	74	74
Need after Conservation			**-1,818**	**-3,114**	**-4,448**	**-5,820**	**-7,227**	**-8,606**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
WTP No. 1 Upgrade and Expansion	$ 9,561,000	$ 1,058	1,792	1,792	1,792	1,792	1,792	1,792
Acquisition of Water Rights through Urbanization - HCID #2	$ 2,720,000	$ 143	200	800	1,600	1,600	1,600	1,600
MHWSC Expand Existing Groundwater Wells (Cameron Co.)	$ 5,373,000	$ 1,254	5	5	5	5	5	5
MHWSC Acquisition of Water Rights through Urbanization	$ 7,735,000	$ 143	2	9	14	20	350	350
ERHWSC New Surface WTP via MHWSC	$ 34,794,000	$ 736	5	5	5	5	5	5
NAWSC Delta Area RO WTP Expansion	$ 22,709,000	$ 1,781	0	0	0	0	800	800
NAWSC La Sara RO Plant Expansion	$ 13,260,000	$ 2,104	0	0	0	0	0	70
NAWSC Converted WR and Water Treatment Plant No. 5 Expansion	$ 23,794,000	$ 505	12	230	230	230	230	230
NAWSC Converted WR and Delta WTP Expansion	$ 28,802,000	$ 748	0	0	227	735	735	735
North Cameron Regional WTP Wellfield Expansion	$ 1,881,000	$ 843	52	52	52	52	52	52
Net Supply Surplus/Deficit			**250**	**-221**	**-523**	**-1,381**	**-1,658**	**-2,967**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
NAWSC Delta Area RO Plant 2 MGD	$ 22,709,000	$ 1,781	72	289	289	289	362	434
NAWSC La Sara RO Plant, expand well field	$ 13,260,000	$ 2,104	0	0	289	289	362	362
RGRWA			1,750	2850	3900	5250	6550	7850
Net Supply Surplus/Deficit			**4**	**25**	**31**	**9**	**47**	**40**
Region M Alternative Strategy	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase I)	$ 668,000	$ 316	2	2	2	0	0	0
MHWSC Expand Existing Groundwater Wells (Hidalgo Co. Phase II)	$ 810,000	$ 195	0	0	0	5	5	5

Deleted WMS

Changed WMS

RGRWA

Deleted WMS	Changed WMS	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS

SHARYLAND WSC

Year		2020	2030	2040	2050	2060	2070
Total Population		45,075	55,970	66,908	77,869	88,829	99,485
Water Demand		8,026	9,722	11,460	13,252	15,094	16,896
Current Water Supply/Supplier	**Type**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Hidalgo County ID #1	MUNI AWR	2,460	2,460	2,460	2,460	2,460	2,460
Santa Cruz ID #15	MUNI AWR	1,008	1,008	1,008	1,008	1,008	1,008
United ID	MUNI AWR	3,692	3,692	3,692	3,692	3,692	3,692
Total Supply (AF/yr)		**7,160**	**7,160**	**7,160**	**7,160**	**7,160**	**7,160**
Projected Supply Surplus/Deficit		**-866**	**-2,562**	**-4,300**	**-6,092**	**-7,934**	**-9,736**

Evaluation of Selected Water Management Strategies — **Additional Supply by Decade**

Conservation			2020	2030	2040	2050	2060	2070
ID Conservation: HCID1			246	246	246	246	246	246
through			128	128	128	128	128	128
ID Conservation: United ID			421	421	421	421	421	421
ID Conservation: Santa Cruz ID No. 15			90	90	90	90	90	90
Need after Conservation			**19**	**-1,677**	**-3,415**	**-5,207**	**-7,049**	**-8,851**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Sharyland WSC Water Well and R.O. Unit at WTP #2	$ 13,253,000	$ 2,630	900	900	900	900	900	900
Sharyland WSC Water Well and R.O. Unit at WTP #3	$ 13,253,000	$ 2,630	900	900	900	900	900	900
Sharyland WSC Acquisition of Water Rights through Urbanization - HCID #1	$ 2,040,000	$ 143	200	500	1,200	1,200	1,200	1,200
Sharyland WSC Acquisition of Water Rights through Urbanization - Santa Cruz ID	$ 8,160,000	$ 143	350	1,300	2,000	3,500	4,800	4,800
Sharyland WSC Acquisition of Water Rights through Urbanization - United ID	$ 2,550,000	$ 143	140	250	250	250	1,400	1,500
Surplus/Deficit after WMS's			**2,509**	**2,173**	**1,835**	**1,543**	**2,151**	**449**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			1,050	4,300	7,700	11,200	15,700	17,850
Surplus/Deficit after WMS's			1,069	2,623	4,285	5,993	8,651	8,999
Transfer to Alton			-690	-2,050	-3,450	-4,950	-7,400	-7,500
Transfer to Palmhurst			-353	-570	-790	-1,012	-1,234	-1,451
Surplus/Deficit after WMS's			**26**	**3**	**45**	**31**	**17**	**48**

Deleted WMS	Changed WMS	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
SULLIVAN CITY								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			5,071	6,297	7,528	8,761	9,995	11,194
Water Demand (ac-ft)			544	647	755	869	989	1,107
Current Water Supply	**Type**							
Agua SUD	MUNI WR		469	469	469	469	469	469
Total Supply (AF/yr)			**469**	**469**	**469**	**469**	**469**	**469**
Projected Supply Surplus/Deficit			**-75**	**-178**	**-286**	**-400**	**-520**	**-638**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-75**	**-178**	**-286**	**-400**	**-520**	**-638**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974	55	55	55	55	55	55
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145	0	0	224	224	224	224
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358	73	73	73	73	73	73
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881	0	0	0	15	15	15
Acquisition of Water Rights through Urbanization		$ 143	8	16	40	72	104	104
Net Supply Surplus/Deficit			**61**	**-34**	**106**	**39**	**-49**	**-167**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
West WWTP Potable Reuse - Phase I	$ 14,455,000	$ 2,974	55	55	55	55	55	55
West WWTP Potable Reuse - Phase II	$ 8,796,000	$ 2,145	0	0	224	224	224	224
East WWTP Potable Reuse - Phase I	$ 13,019,000	$ 2,358	73	73	73	73	73	73
East WWTP Potable Reuse - Phase II	$ 3,561,000	$ 3,881	0	0	0	15	15	15
Transfer from Agua SUD			0	50	0	33	153	271
Net Supply Surplus/Deficit			**53**	**0**	**66**	**0**	**0**	**0**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Agua SUD New BGD Plant	$ 18,136,000	$ 2,649	0	0	0	117	117	117
Agua SUD Non-Potable Reuse	$ 4,026,000.00	$ 2,946	109	109	152	152	152	152

WATER SUPPLY AND DEMAND ANALYSIS								
WESLACO								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			45,205	56,132	67,102	78,094	89,087	99,773
Water Demand			7,873	9,551	11,271	13,040	14,852	16,625
Current Water Supply/Supplier	**Type**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Reuse	REUSE		1,052	1,052	1,052	1,052	1,052	1,052
Hidalgo & Cameron County ID #9	MUNI AWR		3,928	3,928	3,928	3,928	3,928	3,928
Total Supply (AF/yr)			**4,980**	**4,980**	**4,980**	**4,980**	**4,980**	**4,980**
Projected Supply Surplus/Deficit			**-2,893**	**-4,571**	**-6,291**	**-8,060**	**-9,872**	**-11,645**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: H&CC ID No. 9			116	116	116	116	116	116
Need after Conservation			**-2,777**	**-4,455**	**-6,175**	**-7,944**	**-9,756**	**-11,529**
Region M Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
North WWTP Potable Reuse -Phase I	$ 14,444,000	$ 2,378	1,120	1,120	1,120	1,120	0	0
North WWTP Potable Reuse -Phase II	$ 19,548,000	$ 1,738	0	0	0	0	3,360	3,360
Acquisition of Water Rights through Urbanization	$ 5,950,000	$ 143	679	1375	3000	3500	3500	3500
Brackish Groundwater Mixing	$ 980,000	$ 160	560	560	560	560	560	560
NAWSC Converted WR and Water Treatment Plant No. 5 Expansion	$ 23,794,000	$ 505	370	370	370	370	370	370
Surplus/Deficit after WMS's			**-48**	**-1,030**	**-1,125**	**-2,394**	**-1,966**	**-3,739**
Proposed Recommended Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
RGRWA			2800	4500	6200	7950	9800	11550
Surplus/Deficit after WMS's			**23**	**45**	**25**	**6**	**44**	**21**
Region M Alternative Strategies	**Capital Cost ($)**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Scalping Plants	$ 1,346,000	$ 1,455,000	0.5	0.5	0.5	0.5	0.5	0.5
Brackish Groundwater Desalination Plant	$ 17,694,000	$ 1,906	0.0	1630.0	1630.0	1630.0	1630.0	1630.0

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
LYFORD								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			2,981	3,360	3,723	4,110	4,485	4,851
Water Demand (ac-ft)			291	314	338	368	400	432
Current Water Supply	**Type**							
Delta Lake ID	MUNI AWR		588	588	588	588	588	588
Total Supply (AF/yr)			**588**	**588**	**588**	**588**	**588**	**588**
Projected Supply Surplus/Deficit			**297**	**274**	**250**	**220**	**188**	**156**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**297**	**274**	**250**	**220**	**188**	**156**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Lyford Brackish Groundwater Well and Desalination	$ 6,690,000	$ 1,217	1,120	1120	1120	1120	1120	1120
Surplus/Deficit after WMS's			1,417	1,394	1,370	1,340	1,308	1,276

	Deleted WMS
	Changed WMS
	RGRWA

No strategy proposed because Lyford does not have a need

WATER SUPPLY AND DEMAND ANALYSIS								
RAYMONDVILLE								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			12,880	14,519	16,089	17,762	19,379	20,964
Water Demand (ac-ft)			1,522	1,652	1,784	1,944	2,115	2,286
Current Water Supply	**Type**							
Groundwater	GW		2,240	2,240	2,240	2,240	2,240	2,240
Delta Lake ID	MUNI AWR		3,402	3,402	3,402	3,402	3,402	3,402
Total Supply (AF/yr)			**3,402**	**3,402**	**3,402**	**3,402**	**3,402**	**3,402**
Projected Supply Surplus/Deficit			**1,880**	**1,750**	**1,618**	**1,458**	**1,287**	**1,116**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation	**Capital Cost**		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**1,880**	**1,750**	**1,618**	**1,458**	**1,287**	**1,116**
Region M Recommended Strategy	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Surplus/Deficit after WMS's			**1,880**	**1,750**	**1,618**	**1,458**	**1,287**	**1,116**

No strategy proposed because Raymondville does not have a need

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS								
SAN PERLITA								
Year			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population			655	738	817	902	985	1,065
Water Demand (ac-ft)			235	260	286	315	344	371
Current Water Supply	**Type**							
North Alamo WSC	GW/MUNI AWR		225	225	225	225	225	225
Total Supply (AF/yr)			**225**	**225**	**225**	**225**	**225**	**225**
Projected Supply Surplus/Deficit			**-10**	**-35**	**-61**	**-90**	**-119**	**-146**
Evaluation of Selected Water Management Strategies			**Additional Supply by Decade**					
Conservation			**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Need after Conservation			**-10**	**-35**	**-61**	**-90**	**-119**	**-146**
Region M Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
NAWSC Delta Area RO Plant 2 MGD	$ 22,709,000	$ 1,781	0	0	0	0	19	19
NAWSC La Sara RO Plant, expand well field	$ 13,260,000	$ 2,104	0	0	0	0	0	9
NAWSC Converted WR and Water Treatment Plant No. 5 Expansion	$ 23,794,000	$ 654	2	30	30	30	30	30
NAWSC Converted WR and Delta WTP Expansion	$ 23,794,000	$ 505	0	0	30	44	44	44
North Cameron Regional WTP Wellfield Expansion	$ 1,881,000	$ 843	7	7	7	7	7	7
Surplus/Deficit after WMS's			**-1**	**2**	**6**	**-9**	**-19**	**-37**
Proposed Recommended Strategies	**Capital Cost**	**Max Unit Cost ($/AFY)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
NAWSC Delta Area RO Plant 2 MGD	$ 22,709,000	$ 1,781	10	39	39	39	48	58
NAWSC La Sara RO Plant, expand well field	$ 13,260,000	$ 2,104	0	0	39	39	48	48
Transfer from NAWSC			0	0	0	13	22	40
Surplus/Deficit after WMS's			**0**	**4**	**16**	**0**	**0**	**0**

	Deleted WMS
	Changed WMS
	RGRWA

WATER SUPPLY AND DEMAND ANALYSIS							
SEBASTIAN MUD							
Year		**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Total Population		2,094	2,360	2,615	2,887	3,150	3,408
Water Demand (ac-ft)		149	159	176	195	212	230
Current Water Supply	**Type**						
La Feria ID, CCID #3	MUNI AWR	204	204	204	204	204	204
Total Supply (AF/yr)		**204**	**204**	**204**	**204**	**204**	**204**
Projected Supply Surplus/Deficit		**55**	**45**	**28**	**9**	**-8**	**-26**

Evaluation of Selected Water Management Strategies		**Additional Supply by Decade**					
Conservation	**Yield (AF/yr)**	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
ID Conservation: CCID No. 3, La Feria	28	28	28	28	28	28	28
Need after Conservation		**83**	**73**	**56**	**37**	**20**	**2**

No strategies reccomended in Region M plan, because none are needed

	Deleted WMS
	Changed WMS
	RGRWA

CHAPTER 4 - SURFACE WATER AVAILABILITY

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

BLACK & VEATCH
Building a world of difference.

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4.0 Surface Water Availability

4.1 PURPOSE

The Lower Rio Grande Valley (LRGV) relies primarily on surface water from the Rio Grande to meet the drinking water needs for all water user groups. A basic understanding of the operations of the Rio Grande and the rules governing water rights are critical to understanding how water is used in the LRGV. This chapter serves to describe and quantify current and future surface water availability, water rights, and water use in the region, as well conceptualize surface water solutions for the study area.

4.2 INTRODUCTION

The LRGV (Cameron, Hidalgo, and Willacy Counties) depends primarily on surface water from the Rio Grande. This segment of the Rio Grande is controlled and operated through the Amistad-Falcon reservoir system. These two reservoirs are managed by the International Boundary and Water Commission (IBWC) in order to deliver water to users in the border areas of both Texas and Mexico. The Texas Commission on Environmental Quality (TCEQ) – Rio Grande Watermaster's Office serves to operate the accounting, storage, and delivery of the United States' share of water to users in Texas.

Figure 4-1 Rio Grande Basin

The majority of Rio Grande water is used for agriculture in the LRGV. The region's farmers grow vegetables, citrus, cotton, sugar cane, and grain sorghum. In recent years, demand for municipal water has increased as the population of the region has grown. While there is expanded use of groundwater resources to meet municipal demands in the study area, only about 24,000 Acre-feet (AF) are reported used in 2013, as compared with 251,954 AF annual municipal demand.

Much of the surface water data discussed here is from the Region M planning process, and is included in the 2016 Regional Water Plan (RWP).The Rio Grande Regional Water Planning Group (Region M) is tasked with reviewing projected demands provided by Texas Water Development Board (TWDB), evaluating and summarizing existing supplies, determining the needs of the region, and evaluating new water supplies to meet those needs for an eight-county region that includes the LRGV, as well as Maverick, Jim Hogg, Zapata and Webb Counties. The Region M Plan, along with 15 other regional water plans from across the state, forms the basis of the State Water Plan for Texas. The Region M Water Plan is updated every five years, and the next update will be finalized in December of 2016. All data from Region M is based on a representative drought year, which is based on the worst recorded drought, called the Drought of Record.

4.3 AVAILABILITY

Surface water availability, using TWDB's regional water planning terminology, is intended to estimate how much water is legally available and can be reliably accessed if the drought of record was repeated. The Firm Yield is the basis of surface water availability in Regional Water Planning, which is developed using TCEQ's Water Availability Models. For planning purposes, the Firm Yield is shown in Table 4-1 in decadal annual yield over a 50-year planning horizon.

The Firm Yield that is currently being used by the Region M Plan does not take climate change into account. A separate study, the Bureau of Reclamation Lower Rio Grande Basin Study, developed an estimate of the impacts of climate change and is discussed in this chapter.

While drought scenarios and climate change - impacted drought scenarios are valuable for planning to the lowest availability year, it is also valuable to compare these estimates with annual averages and to discuss availability in an average year.

4.3.1 Rio Grande Water Availability Model

The Amistad-Falcon Reservoir System serves users from Amistad Reservoir, on the border between Val Verde County and Mexico, down to the Gulf of Mexico. The US Firm Yield for the Amistad-Falcon Reservoir system is estimated using the Rio Grande Water Availability Model (WAM). The Rio Grande WAM uses historical data from 1943 – 2000 in order to simulate the watershed. The WAM takes into account historical drought and sedimentation rates to predict Firm Yield. Figure 4-2 shows all of the control point locations in the Rio Grande Basin. The Primary Control Points are the sites where the naturalized flow data is developed for the WAM.

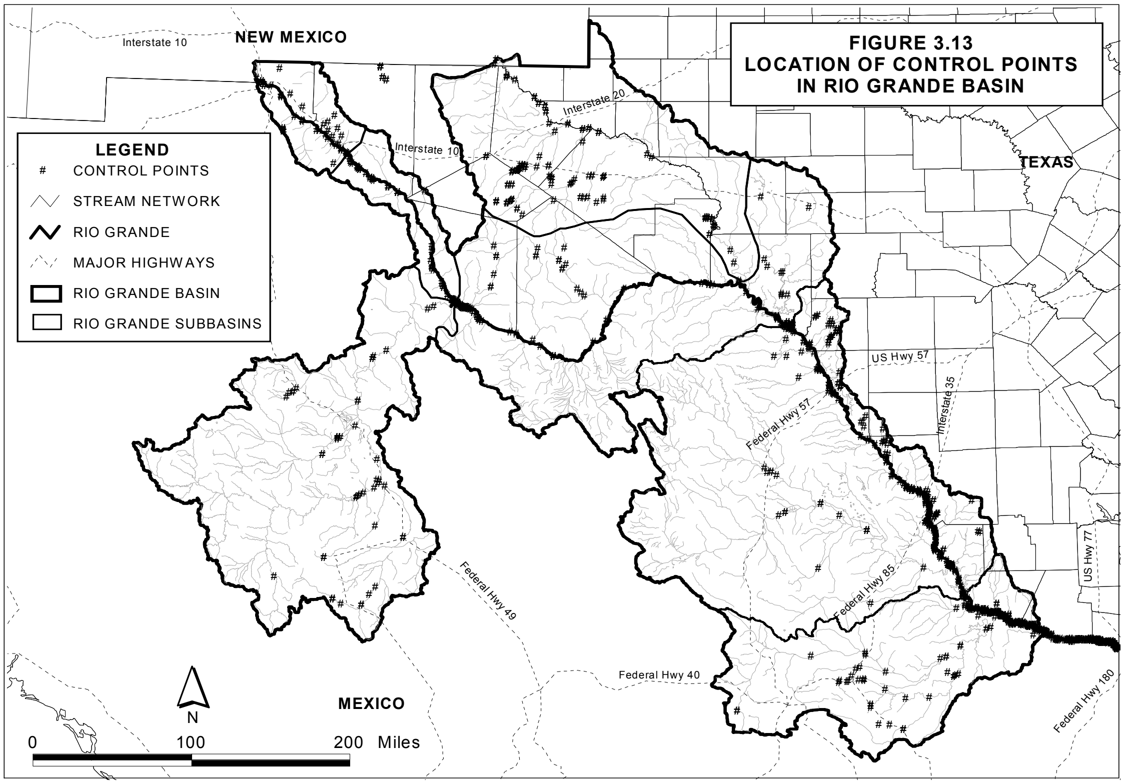


Figure 4-2 Rio Grande Basin and Control Point Locations (TCEQ Rio Grande Water Availability Model)

All of the Rio Grande Basin below the New Mexico state line, including the Mexican portion of the basin, is included in the Rio Grande WAM. However there a provision of the Treaty of February 3, 1944 for "Utilization of Waters of the Colorado and Tijuana Rivers and of the Rio Grande" requiring a minimum of 350,000 acre-ft. /year to be delivered to the U.S. from the six named Mexican tributaries, the Conchos, San Diego, San Rodrigo, Escondido, and Salado Rivers and the Las Vacas Arroyo, which has not been incorporated into the WAM, because it is not enforced on an annual basis and future compliance is uncertain. The transfer of Mexican water from the six named Mexican tributaries of the Rio Grande to the U.S. is modeled after Mexico's demands and reservoirs on these tributaries have been simulated. The U.S. is allotted one-third of the remaining flow at the mouths of each of the six named Mexican tributaries. Demands for water along the Rio Grande by both U.S. and Mexican water users downstream of these Mexican tributaries then are simulated in the model.

The Rio Grande Basin and the Amistad-Falcon Reservoir System refer to the drought spanning from February 1993 to October of 2000 as the Drought of Record. This 7.75 year period is the most severe hydrologic drought according to the Rio Grande Water Availability Model (WAM), and is used to predict firm yield over the planning horizon. The span of the current drought is limited by the extent of naturalized flow data in the WAM. The actual drought extended through approximately 2003, and if the WAM were updated to include those years, may impact the drought of record. Extending the span of the drought of record or reviewing recent droughts could change the drought of record, and therefore the firm yield projections.

The US annual firm yield, shown in Table 4-1, represents an estimate of annual availability in a protracted drought that resembles the historical drought of record. The Firm Yield is expected to gradually decline over the planning horizon as a result of reservoir sedimentation.

Table 4-1 Annual Firm Yield from the Amistad Falcon Reservoir System (AF/YR)

SOURCE	2020	2030	2040	2050	2060	2070
Amistad-Falcon Reservoir System Firm Yield	1,060,616	1,059,260	1,057,903	1,056,547	1,055,191	1,053,834

4.3.2 US Bureau of Reclamation Lower Rio Grande Basin Study

In 2013, the Bureau of Reclamation and the Rio Grande Regional Water Authority evaluated the impacts of climate change on the LRGV in a Basin Study. The study, funded by a grant through the WaterSMART program, considered 112 climate change –affected outcomes based on three different future global emission scenarios. The 112 climate scenarios simulated runoff and other water/land/atmosphere interactions in the study area.

The outputs from all 112 climate scenarios were summarized into evaporation and flow data representing the range of likely precipitation and evaporation for the study area. The WAM was used to model potential combinations of evaporation and flow. Based on results of that study, the reduction due to climate change, as determined by the difference between average delivery under the median climate scenario and the baseline, is 86,438 acre-feet. This study incorporates this estimated reduction of availability due to climate change.

4.3.3 Historical Inflows to Amistad and Falcon Reservoirs

Amistad Reservoir captures the majority of the surface water used from the Amistad-Falcon System. Falcon reservoir is used to capture and control some additional inflows, and to stage water that is eventually delivered to users in the Lower Rio Grande Valley. Recorded inflows and storage can be used to evaluate the magnitude and frequency of drought in the system. The total conservation storage capacity of Amistad Reservoir is approximate 3.15 million acre-feet and for Falcon Reservoir it is about 2.66 million acre-feet.

Total annual inflow data for the United States portion of the Amistad-Falcon reservoir system from 1945 – 2003 is shown in Figure 4-3, broken out by inflow to each reservoir. A majority of the US water flows into Amistad Reservoir, with additional water contributed by the intervening watershed below Amistad Reservoir, especially in peak years. The vertical line over 1953 represents the year Falcon Reservoir was constructed and the one on 1968 is when Amistad Reservoir was constructed. The drought of the 1950's shows years of very low inflow with high to medium inflow in years immediately preceding and following. The low inflows shown at the end of the data period are not as dramatic, but more sustained.

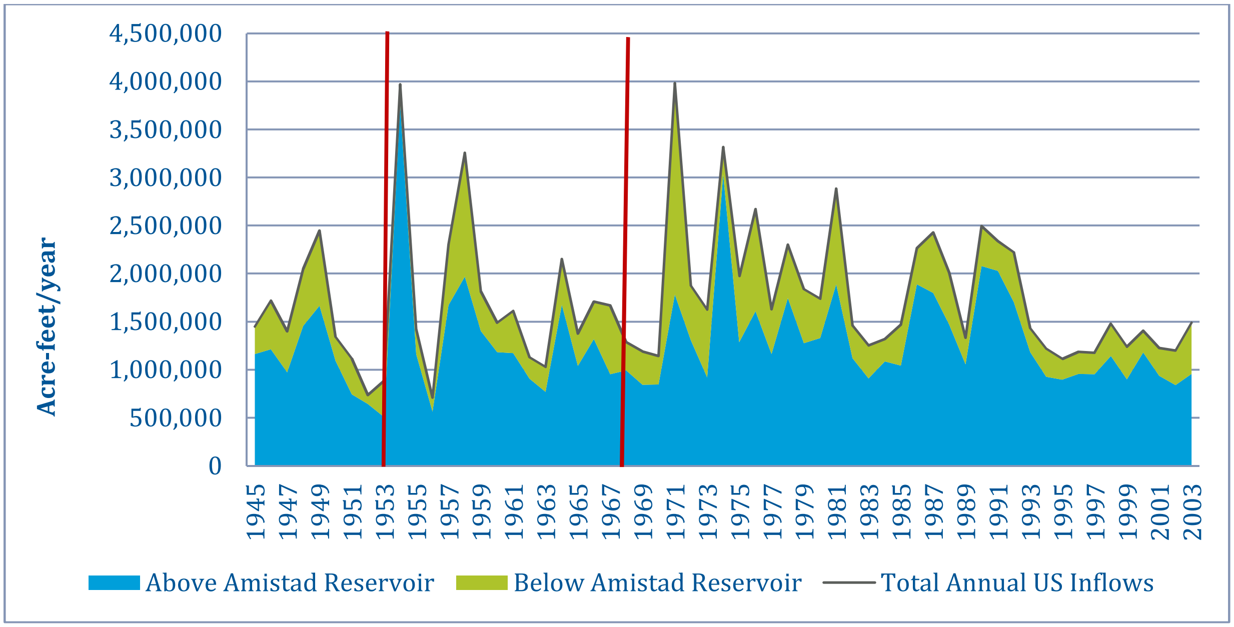


Figure 4-3 Historical Annual United States and Mexican Inflows to the Rio Grande above Amistad Reservoir and between Amistad and Falcon Reservoirs (Source: IWBC)

Table 4-2 shows both the U.S. and Mexican inflows during the period IWBC data is available (1945-2003). The intention of Firm Yield determination and WAM modeling is to estimate the reliable volume in drought years. This amount is predicted to be higher than the inflows shown in 25 of the 49 years modeled because the water in storage at the beginning of the year is also taken into account.

The lowest storage level to which Amistad Reservoir has ever fallen was approximately 770,000 acre-feet in July 1998. Since the initial filling of Falcon Reservoir, the lowest level that it has dropped to was 160,000 acre-feet in January 1957; however, its storage did fall to near 200,000 acre-feet on several occasions during the 2000-2002 period. The severity of the drought of record from 1993 to 2000 on the lower and middle Rio Grande is evident from the low storage levels experienced in Amistad and Falcon Reservoirs.

Table 4-2 Historical Annual United States and Mexican Inflows to the Rio Grande above Amistad Reservoir and between Amistad and Falcon Reservoirs (Source: IWBC)

	UNITED STATES INFLOWS, AC-FT			MEXICAN INFLOWS, AC-FT			TOTAL, AC-FT
YEAR	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	TOTAL ANNUAL INFLOWS
1945	1,163,203	285,000	1,448,203	883,389	278,000	1,161,389	2,609,592
1946	1,212,854	506,000	1,718,854	909,841	521,000	1,430,841	3,149,695
1947	973,130	426,000	1,399,130	669,063	371,000	1,040,063	2,439,193
1948	1,454,024	595,000	2,049,024	507,768	702,000	1,209,768	3,258,792
1949	1,666,097	783,000	2,449,097	1,042,898	442,000	1,484,898	3,933,995

	UNITED STATES INFLOWS, AC-FT			MEXICAN INFLOWS, AC-FT			TOTAL, AC-FT
YEAR	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	TOTAL ANNUAL INFLOWS
1950	1,093,569	248,000	1,341,569	786,227	128,000	914,227	2,255,796
1951	743,512	371,000	1,114,512	404,486	326,000	730,486	1,844,998
1952	644,293	92,000	736,293	428,901	64,000	492,901	1,229,194
1953	505,469	380,000	885,469	222,231	1,003,000	1,225,231	2,110,700
1954	3,764,424	206,368	3,970,792	788,961	325,559	1,114,520	5,085,312
1955	1,161,083	262,728	1,423,811	677,209	344,411	1,021,620	2,445,431
1956	562,134	146,131	708,265	296,764	153,390	450,154	1,158,419
1957	1,670,650	633,550	2,304,200	564,144	727,886	1,292,030	3,596,230
1958	1,969,349	1,287,790	3,257,139	1,567,841	1,933,882	3,501,723	6,758,862
1959	1,400,966	413,263	1,814,229	667,730	489,555	1,157,285	2,971,514
1960	1,183,084	304,220	1,487,304	848,707	307,596	1,156,303	2,643,607
1961	1,173,210	438,643	1,611,853	624,584	583,960	1,208,544	2,820,397
1962	906,681	222,588	1,129,269	511,070	240,095	751,165	1,880,434
1963	770,142	259,995	1,030,137	481,290	307,161	788,451	1,818,588
1964	1,673,626	478,465	2,152,091	672,900	548,188	1,221,088	3,373,179
1965	1,039,969	334,430	1,374,399	489,720	350,059	839,779	2,214,178
1966	1,318,285	391,422	1,709,707	1,003,086	417,219	1,420,305	3,130,012
1967	954,207	713,220	1,667,427	523,436	943,825	1,467,261	3,134,688
1968	991,330	294,637	1,285,967	841,232	382,091	1,223,323	2,509,290
1969	843,864	346,676	1,190,540	705,083	382,759	1,087,842	2,278,382
1970	844,695	297,120	1,141,815	620,385	283,218	903,603	2,045,418
1971	1,783,089	2,201,017	3,984,106	692,998	3,101,272	3,794,270	7,778,376
1972	1,307,088	569,612	1,876,700	802,803	670,492	1,473,295	3,349,995
1973	918,028	707,828	1,625,856	679,907	740,920	1,420,827	3,046,683
1974	3,029,423	287,805	3,317,228	1,211,470	305,682	1,517,152	4,834,380
1975	1,284,972	689,676	1,974,648	748,604	913,544	1,662,148	3,636,796

	UNITED STATES INFLOWS, AC-FT			MEXICAN INFLOWS, AC-FT			TOTAL, AC-FT
YEAR	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	TOTAL ANNUAL INFLOWS
1976	1,607,050	1,062,184	2,669,234	773,967	1,693,211	2,467,178	5,136,412
1977	1,163,283	464,282	1,627,565	550,896	554,875	1,105,771	2,733,336
1978	1,743,638	556,024	2,299,662	1,517,216	801,281	2,318,497	4,618,159
1979	1,275,063	564,636	1,839,699	878,202	688,648	1,566,850	3,406,549
1980	1,329,313	409,238	1,738,551	817,103	544,535	1,361,638	3,100,189
1981	1,888,274	994,629	2,882,903	1,238,430	1,430,420	2,668,850	5,551,753
1982	1,118,780	340,150	1,458,930	664,349	338,840	1,003,189	2,462,119
1983	910,765	342,907	1,253,672	497,472	291,291	788,763	2,042,435
1984	1,086,407	234,142	1,320,549	775,321	243,487	1,018,808	2,339,357
1985	1,043,484	424,262	1,467,746	682,379	463,802	1,146,181	2,613,927
1986	1,887,478	377,249	2,264,727	1,208,462	540,129	1,748,591	4,013,318
1987	1,797,750	630,894	2,428,644	1,203,973	748,490	1,952,463	4,381,107
1988	1,469,121	539,973	2,009,094	929,864	831,771	1,761,635	3,770,729
1989	1,055,062	278,254	1,333,316	589,071	285,024	874,095	2,207,411
1990	2,076,817	418,569	2,495,386	1,728,668	498,141	2,226,809	4,722,195
1991	2,027,658	308,733	2,336,391	1,892,590	322,749	2,215,339	4,551,730
1992	1,702,861	517,404	2,220,265	1,283,085	623,610	1,906,695	4,126,960
1993	1,181,767	250,123	1,431,890	788,586	230,123	1,018,709	2,450,599
1994	924,654	295,200	1,219,854	488,813	255,581	744,394	1,964,248
1995	895,126	218,838	1,113,964	387,891	240,841	628,732	1,742,696
1996	956,466	227,673	1,184,139	441,577	259,854	701,431	1,885,570
1997	951,291	226,163	1,177,454	398,567	242,833	641,400	1,818,854
1998	1,141,780	336,462	1,478,242	314,958	313,171	628,128	2,106,370
1999	899,246	340,210	1,239,456	379,527	410,671	790,198	2,029,654
2000	1,178,741	228,448	1,407,189	206,208	91,279	297,488	1,704,677
2001	935,554	291,632	1,227,186	183,849	133,833	317,682	1,544,868

	UNITED STATES INFLOWS, AC-FT			MEXICAN INFLOWS, AC-FT			TOTAL, AC-FT
YEAR	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	ABOVE AMISTAD RESERVOIR	BELOW AMISTAD RESERVOIR	TOTAL ANNUAL INFLOWS	TOTAL ANNUAL INFLOWS
2002	840,966	357,906	1,198,871	304,054	401,696	705,751	1,904,622
2003	954,473	533,034	1,487,507	360,704	669,445	1,030,149	2,517,656
AVG	1,288,971	456,651	1,745,622	734,924	549,786	1,284,710	3,030,332
1956 (Min)	562,134	146,131	708,265	296,764	153,390	450,154	1,158,419
1971 (Max)	1,783,089	2,201,017	3,984,106	692,998	3,101,272	3,794,270	7,778,376

4.4 AMISTAD-FALCON RESERVOIR SYSTEM OPERATIONS

The waters of the Rio Grande, treated as a stock resource, are accumulated in the Amistad-Falcon Reservoir System and released on demand in accordance with water rights set by law. The TCEQ administers the United States' share of water stored in Amistad and Falcon Reservoirs in compliance with the decision of the Thirteenth Court of Civil Appeals in the landmark case, "State of Texas, et al. vs. Hidalgo County Water Control and Improvement District No. 18, et al." commonly referred to as the Rio Grande Valley Water Case. The TCEQ Rio Grande Watermaster's Office is responsible for allocating, monitoring, and controlling the use of surface water in the Rio Grande Basin from Ft. Quitman to the Gulf of Mexico.

Since the 1960s, the U.S. portion of the Rio Grande below Amistad has been fully adjudicated, such that there are no 'unclaimed' (or unappropriated) water rights available in the system. Water rights on the river are divided into two major types: Domestic, Municipal, and Industrial (DMI) rights and irrigation and mining rights (which are sub-divided into Class A and B). These rights represent the annual allowable maximum to be diverted. Because the existing demands exceed the current supply in a drought year, only the highest priority water rights receive the full amount of their allocations. The first priority goes to DMI, the second goes to a minimum volume required for reservoir operations, and the third priority goes to the irrigation and mining accounts. In drought years, irrigation and mining water right holders may not have access to the stored water. [1]

To determine the amount of water from the mainstream Rio Grande to be allocated to various accounts, the Watermaster makes the following computations, which are given in highest to lowest priority, at the beginning of each month:

1. From the amount of water in usable storage, 225,000 acre-feet are deducted to re-establish the DMI storage pool. These uses are given the highest priority;
2. From the remaining storage, the operating reserve is deducted to account for evaporation, seepage, conveyance losses, and emergencies; and
3. From the remaining storage, the total end-of-month account balances for all lower and middle Rio Grande irrigation and mining water right holders are deducted; and,
4. Any remaining storage is allocated to the irrigation and mining accounts.

[1] Texas Administrative Code, Rule §303.21 - Amistad/Falcon Reservoirs Accounts.

Steps 2 through 4 listed above are iterative, and are all based on the reservoir volume. When there is insufficient water to fulfill the account balances for Irrigation and Mining, the requirement for operating reserve can be reduced. In years of limited availability, Class A and Class B mining or irrigation water rights are only fulfilled as water is available. Sometimes only 30% or 40% of the face value of their water right can be diverted over the course of a year.

Water that has been designated for municipal use must be used for municipal purposes, and similarly irrigation water rights for irrigation, etc. unless a water right is converted permanently through TCEQ. Class A water rights, when converted to municipal water rights, are reduced by 50%, and Class B water rights are reduced by 60%. The main mechanism for this conversion is urbanization of land with water rights associated to it. Ownership and conversion of water rights are addressed in Section 5.0, Urbanization.

4.4.1 Water Rights Accounting

When low to average flows occur in the Rio Grande, requests are made to the IBWC by water users in both the United States and Mexico for releases of water from the conservation storage pools in Amistad and Falcon Reservoirs. The Rio Grande Watermaster makes daily requests to the IBWC for releases from the reservoirs to meet municipal, industrial, and agricultural demands in the Lower Rio Grande Valley below Falcon Dam, and in the Middle Rio Grande Valley between Falcon and Amistad Reservoirs. For some users at the extreme lower end of the river, the requests are made five to seven days in advance of need to allow for the travel time required for the released water from Falcon Reservoir to flow downstream along the more than 200 miles of river channel to the various points of diversion. The WAM takes into account transmission losses as the water is conveyed downstream.

Generally, under the current rules and regulations of the TCEQ, all United States water that is diverted from the lower and middle Rio Grande by authorized diverters is accounted for by the Rio Grande Watermaster with appropriate charges against annual authorized diversion accounts in accordance with existing individual water rights and against individual storage accounts in Falcon and Amistad Reservoirs.[2]

There are some circumstances, however, when the water use and storage accounts of water rights holders along the lower and middle Rio Grande are not charged for water diverted from the river. These are referred to as “no charge pumping” periods, and diversions during such periods are authorized by an order issued by the Texas Water Commission. Generally the Rio Grande Watermaster allows no charge pumping when there are substantial flows in the river due to high runoff conditions or when there are releases from Amistad and/or Falcon Reservoirs. The intention is to operate the system to minimize the amount of water that flows to the Gulf of Mexico and maximize the amount diverted by users. When no-charge pumping is declared by the Rio Grande Watermaster, water from the Rio Grande can be diverted by authorized water rights holders in unlimited quantities; without their respective annual water use and storage accounts being charged. No-charge pumping periods may represent an opportunity for Aquifer Storage and Recovery (ASR).

Separate from no-charge pumping, there is a diversion permit, 1838, which is held by the City of Brownsville, which allows “excess” flows in the Rio Grande to be utilized. This permit allows Brownsville, as the most downstream diverter on the river, to intermittently divert and store water

[2] Texas Administrative Code, Rule §303.22 - Allocations to Accounts.

when flow is above 25 cfs. The maximum authorized diversion is 40,000 acre-feet, which has historically been used as a supplemental water source and to fill resacas, oxbow lakes and ponds.

The allotment for irrigation and mining uses is divided into the Class A and Class B water rights. The accounting for these water rights is based on their cumulative managed volume stored in the reservoir system (storage pool) and useable balance, and Class A rights accumulate water in storage at a rate 1.7 times that of Class B rights.

DMI water right accounts are not allowed to roll over any water each year, and the individual water right's maximum diversion quantity may not be exceeded. Class A and B water right accounts can accumulate up to 1.41 times the annual authorized diversion right in storage. For all water rights, if an allottee does not use any water for two consecutive years, the account is reduced to zero. Though the allotee retains ownership of the water right, no subsequent allocations can be made until the allottee advises the watermaster that water is expected to be used.

4.4.2 2013 Water Rights

TCEQ records from 2013 show the Annual Water Rights (AWR) that are held for Rio Grande water, separated into user designations:

- Domestic – Guaranteed, similar to municipal but more commonly used for lawn watering or small accounts outside of city accounts.
- Municipal – Most commonly raw water for municipal treatment plants.
- Industrial – Water used in industrial applications
- Irrigation Class A and B – Water used to irrigate crops.
- Multi-Use – (Multi) which refers to a water right that can be used for either mining or irrigation and is assigned as Class A or B), and
- Mining – Water used in Mining or Oil and Gas applications

For Regional Planning, each municipal entity or utility serving 500 people or more is considered a municipal Water User Group (WUG), while other types of users, (irrigation, mining, manufacturing, livestock, and steam-electric power generation) are aggregated into county-wide WUGs, e.g., "Cameron County Manufacturing." For comparison to the Amistad-Falcon system yield, all of the water rights that are served by the Amistad-Falcon Reservoir System, not just those in the LRGV, are discussed here. In later sections the portion of these supplies that serves the LRGV will be discussed.

A portion of reservoir storage is reserved to fulfill DMI water rights which are replenished monthly, and remaining water is used to fulfill operational uses and Class A and B water rights. The portion of Class A and B water rights that can be expected to be delivered in the representative drought year is predicted by the WAM (Volume Reliability) shown in Table 4-3.

Table 4-3 2013 Annual Water Rights, 2014 Rio Grande Water Availability Model update[3] (AF/YR)

[3] Region M Water Plan, Surface Water Modeling from Kennedy Resources Co. Performed 2013.

AWR TYPE	WAM AWR	CLIMATE-LIMITED VOLUME RELIABILITY	CLIMATE-LIMITED AVERAGE DELIVERY VOLUME
DMI	301,920	100.0%	301,920
Class A	1,624,004	61.7%	1,002,011
Class B	187,078	40.8%	76,328
TOTAL	**2,113,002**	**65.3%**	**1,380,259**

Although the percentage of a Class A or B Mining or Agricultural water right that is expected to be delivered in a drought year is low, the relationship between supply and demand in agriculture is different than other water users. While municipal or industrial users are limited in their ability to reduce usage in response to a water shortage, farmers have significant latitude. In anticipation of a shortage, crops can be planted that can survive without irrigation water and in some Irrigation Districts farmers can consolidate available water on high-value crops and leave other fields dry. It is therefore difficult to assess which demand scenario is the most appropriate for agricultural demands, which are very responsive to available supplies.

While Agriculture relies almost exclusively on surface water, mining is split between surface and groundwater. In this drought scenario, it is expected that mining operations, especially oil and gas, will pursue groundwater after all water rights that are available to be used for mining (Mining and Multi) are exhausted.

For the purposes of this study, it is estimated that only 90% of the municipal maximum authorized diversions will be utilized. According to TCEQ Watermaster, in 2013 only 85% of the maximum authorized diversions associated with municipal water rights were used in the study area. Many water rights holders may reserve a small portion of their water right for emergencies or are otherwise managed such that they are not completely used. This reduction in the amount of water used can be seen in Table 4-4.

Table 4-4 LRGV Estimated Irrigation District Conveyance Efficiencies

IRRIGATION DISTRICT	2016 REGIONAL WATER PLAN EFFICIENCY	2020 TOTAL DIVERSION (AF)	WATER LOST (AF)	MUNICIPAL WATER RIGHTS (AF)	EFFECTIVE WR USE (10% UNUSED)	MUNICIPAL WATER AFTER WATER LOSSES
Adams Garden Irrigation District No. 19	68%	8,944	2,862	0	0	0
Bayview Irrigation District	68%	10,935	3,499	7,383	6,645	4,518
Brownsville Irrigation District	68%	15,874	5,080	334	301	204
Cameron County Irrigation District No. 2, San Benito	68%	80,782	25,850	13,361	12,025	8,177
Cameron County Irrigation District No. 6, Los Fresnos	68%	44,830	14,346	2,597	2,337	1,589

IRRIGATION DISTRICT	2016 REGIONAL WATER PLAN EFFICIENCY	2020 TOTAL DIVERSION (AF)	WATER LOST (AF)	MUNICIPAL WATER RIGHTS (AF)	EFFECTIVE WR USE (10% UNUSED)	MUNICIPAL WATER AFTER WATER LOSSES
Cameron Co W.I.D No. 10, Rutherford Harding	68%	2,386	763	0	0	0
Cameron County Irrigation District No. 16	68%	1,631	522	0	0	0
Hidalgo and Cameron Counties Irrigation District No. 9, Mercedes	70%	71,931	21,579	23,380	21,042	14,729
Delta Lake Irrigation District	60%	119,305	47,722	15,651	14,086	8,452
Donna Irrigation District Hidalgo Co. No. 1	58%	91,617	38,479	6,893	6,204	3,598
Engleman Irrigation District	71%	4,930	1,430	0	0	0
Harlingen Irrigation District No. 1	68%	70,076	22,424	26,776	24,098	16,387
Hidalgo County Irrigation District No. 1, Edinburg	71%	97,172	28,180	40,114	36,103	25,633
Hidalgo County Irrigation District No. 2, San Juan	71%	97,356	28,233	27,760	24,984	17,739
Hidalgo County Irrigation District No. 5, Progresso	71%	6,488	1,882	0	0	0
Hidalgo County Irrigation District No. 6, Mission 6	71%	22,716	6,588	6,184	5,566	3,952
Hidalgo County Irrigation District No. 13	71%	1,359	394	0	0	0
Hidalgo County Irrigation District No. 16, Mission	71%	18,829	5,460	4,216	3,794	2,694
Hidalgo County Water Control and Improvement District No. 18	71%	834	242	0	0	0
Hidalgo M.U.D. No. 1	71%	1,268	368	1,120	1,008	716
Hidalgo County Water Irrigation District No. 3	71%	21,697	6,292	16,350	14,715	10,448
La Feria Irrigation District, Cameron County No. 3	68%	43,285	13,851	3,050	2,745	1,867
Santa Cruz Irrigation District No. 15	71%	26,694	7,741	4,500	4,050	2,876
Sharyland, Hidalgo County Improvement District No. 19	71%	3,885	1,127	0	0	0

IRRIGATION DISTRICT	2016 REGIONAL WATER PLAN EFFICIENCY	2020 TOTAL DIVERSION (AF)	WATER LOST (AF)	MUNICIPAL WATER RIGHTS (AF)	EFFECTIVE WR USE (10% UNUSED)	MUNICIPAL WATER AFTER WATER LOSSES
United Irrigation District	71%	59,838	17,353	36,995	33,296	23,640
Valley Acres Irrigation District	71%	3,509	1,032	0	0	0
Valley MUD	68%	2,275	728	798	718	488
TOTAL		930,446	304,027	237,462	213,717	147,707

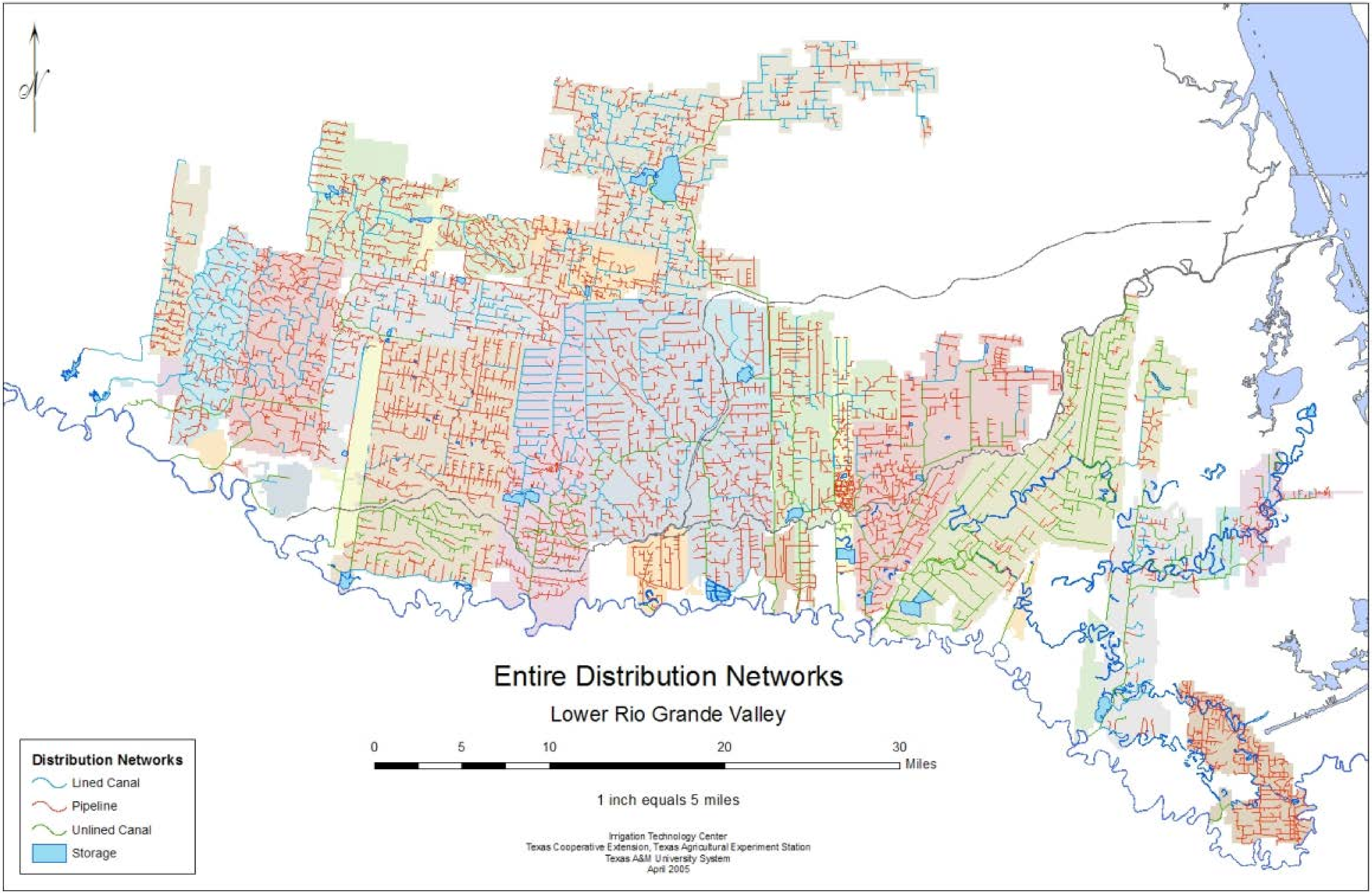
4.4.3 Role of Irrigation Districts

Water users in Rio Grande Region that are dependent upon the Amistad-Falcon Reservoir System for their water supplies operate under rules and regulations that originate from the 1969 Valley Water Case. Among other things, the judgment allocated specific amounts of water in the LRGV to individual DMI water users (typically cities) that were in existence at the time and had documented historical water usage, and it assigned these DMI water rights to specific Irrigation Districts, which had pumping facilities on the river, for the subsequent diversion and delivery of river water to the DMI users. In effect, the Irrigation Districts were assigned municipal water rights that were specifically designated for certain individual domestic, municipal, and industrial water users. Figure 4-4 shows the Irrigation Districts' conveyance network.

Today, most of the DMI water users in the LRGV continue to obtain their water supplies from the Irrigation Districts under the original water rights that are owned by the Irrigation Districts but assigned to the DMI users in their district. The Irrigation Districts request releases from Falcon Reservoir, pump this water from the Rio Grande into their own distribution systems, and deliver the water, less losses, to the DMI users.

The diversions from the Rio Grande are metered, and the intake to municipal utilities are generally metered, but many Irrigation Districts estimate the quantity of water being delivered to farmers with field measurements done by canal riders, who are employees of the Irrigation District tasked with overseeing field operations. It is difficult to get an accurate measure of conveyance efficiency without extensive metering, but most districts have efficiency estimated somewhere between 58% and 71%. Most DMI contracts include efficiency losses in their accounting, so that a water user's account is charged for the conveyance losses, although the estimates of losses for accounting purposes tend to be between 10 and 15%. Texas AgriLife Research has continued to work with Irrigation Districts to assist with ongoing mapping and improvement efforts for the conveyance networks.

Table 4-4 shows estimated Irrigation District efficiencies, diversions, and deliveries used in the 2016 Region M Plan. The Total Diversion includes water rights used for non-municipal uses, based on the amount of each water right is estimated to be available for diversion in 2020 (i.e. Class A & B water rights). The Municipal AWR are shown in full, and the Municipal Delivered shows the 90% of the total, for the 10% reduction due to management, and the reduction due to Irrigation District conveyance efficiency (2016 Regional Water Plan Efficiency).


N
Entire Distribution Networks
Lower Rio Grande Valley
Distribution Networks
Lined Canal
Pipeline
Unlined Canal
Storage
0
5
10
20
30
Miles
1 inch equals 5 miles
Irrigation Technology Center
Texas Cooperative Extension, Texas Agricultural Experiment Station
Texas A&M University System
April 2005

Conveyance losses directly impact the quantity of water available for an end user. As shown in Table 4-4, all of the individual water rights were associated with the Irrigation District(s) that divert them. For 100 acre-feet of DMI water delivered by a 71% efficient district, only 71 acre-feet were assumed to be delivered to that end user. In some cases a WUG may have multiple Districts that can deliver the same water right, which was accounted for by estimating what portion of water has historically been delivered by each District. In many cases, an Irrigation District diverts water for another district, and water passes through multiple systems before arriving at the end user. The efficiency for these deliveries was estimated by applying the appropriate loss factor for each district that the water passes through. Brownsville PUB is the only major municipal user that diverts their own water, and is therefore not included in the Irrigation District evaluation.

As most of the DMI water users continue to obtain their water supplies from the Irrigation Districts, delivery contracts are maintained between entities with agreed-upon pumping costs and estimated conveyance losses. When these delivery contracts or agreements expire, they normally are simply extended with revised rates.

There are some municipal water users that own their own water rights, and some that have specific contracts for DMI water from the Irrigation Districts under the districts' water rights exclusive of the original allotments from the Rio Grande Valley Water Case.

At times, the Irrigation District may continue to supply DMI water to the DMI user under the district's own water right when the annual allotment for DMI water is exceeded by an individual DMI water user. The DMI user is charged by the Irrigation District for this additional water. If the District does not have available municipal water rights, the City or the District can acquire municipal use water from third parties to deliver to the City. This one-time delivery of water is referred to as "contract water," which means that water is being delivered to a DMI user on a short-term contractual basis, governed by the Watermaster rules, and the original owner retains the water right for future use.

Sales of both water and water rights allow for a more adaptable, but constantly changing system. Some Irrigation Districts are in a constant state of flux with increased development of farmland or changes among their municipal customers, and have had to adapt to changing volumes and delivery locations.

4.4.4 Conservation, Drought, and Push-Water

One of the results of the water rights allocation system is that conservation in any one year by municipalities does not make a significant amount of water available for other user groups (like agriculture) and municipal availability is not impacted by agricultural conservation. Agricultural water rights absorb reduced availability in drought, and municipalities only experience a shortage if their water right is insufficient for their demands, regardless of the conditions in the reservoirs. Low precipitation or high temperatures can increase municipal demands, but if the municipality has sufficient water rights, even an extreme drought may not trigger drought restrictions.

The exception to this is the impact of reduced agricultural availabilities on the operations of Irrigation Districts, or the impact of "push water." Many of the water districts primarily deliver water for irrigation and use this water to charge their networks of canals. When there is irrigation water being delivered, the municipal water is delivered along with the irrigation water at a higher efficiency. This allows less of the municipal water to be lost to evaporation and infiltration. In years of severe drought, Irrigation Districts go on allocation and there may not be irrigation water being delivered for weeks or months at a time. In this case, municipalities may need to purchase additional water, or "push water", in order to provide a minimum operational amount of water in the system because of the anticipated losses due to the inefficiency of the canal system at lower flow rates. Cities further from the Rio Grande or in districts that deliver primarily irrigation water, are more vulnerable to conveyance issues when agricultural water is restricted. (This is in addition to the regular water losses experienced by districts as a result of seepage, evaporation, and operational losses, which are also more severe for those cities farther from the river.) Cities are less likely to experience the "push water" issue if they are close to the river, are served by efficient conveyance systems, or if their districts deliver enough municipal water to maintain their operational minimum.

To date, a few cities have leased water in anticipation of the need for push water, but none have had to use it. When an Irrigation District goes on allocation, agricultural usage slows dramatically. This reduction of usage has historically allowed for the reservoirs and irrigators' useable account balances to re-charge, and for the system to go back to normal operations with irrigation deliveries to charge the canals and make municipal water available. Additionally, cities have drought response plans that can decrease water demand through voluntary and mandatory measures.

The current authority of the Watermaster includes the ability to "take action appropriate to prevent waste or alleviate emergencies", which can apply to a pushwater crisis. In recent years, the Watermaster has recommended the use of specific criteria for allowing entities to purchase water beyond the maximum authorization of their account in the case of a pushwater emergency. This and other infrastructure recommendations, like interconnects between cities, may help to alleviate a pushwater crisis.

4.5 URBANIZATION

Land that was previously undeveloped or farmland is constantly being developed for residential or commercial use. This urbanization has direct implications on where and how water is used. In the LRGV, industrial, commercial, and population growth requires new development inside and outside of cities. Some of this development, but not all, is displacing farmland.

Some of the farmland in the LRGV is "flat rate acreage" which is served by an Irrigation District, and has a certain portion of the Irrigation District's irrigation water rights associated with it. The farmer doesn't pay for the water itself, but pays a small fee for the delivery costs. Some lands do not have water rights assigned to them but are irrigated either through water rights held by an individual or an alternate source like groundwater wells. Some lands are flat rate but not irrigated, for instance as a result of a farmer planting some portion of his land with a low value crop which does not require irrigation water and consolidating his share of irrigation water onto a higher value crop planted on only a portion of his land. Some Irrigation Districts rules may allow this practice.

As land is developed, water rights may change ownership and/or use type. If there are water rights associated with land being developed, the utility that will serve that development has the first right of refusal to purchase and legally separate those rights from the land (called exclusion). The utility has two years from when the development is platted in which to petition the water district for the water rights associated with that land, which the utility may then purchase at a rate of 68% of the current market value. [4] If the utility fails to file for the water rights, the Irrigation District can retain the water rights, but if they select to sell the water rights or contract the water, they must make the rights available to other municipal utilities within Cameron, Hidalgo, and Willacy Counties under the same terms as were offered to the developing utility for 90 days. If no municipal water utilities

[4] Water Code, Title 4. General Law Districts, Chapter 49. Provisions Applicable To All Districts, Subchapter O.

in this area elect to purchase the water rights, the Irrigation District can sell the water right or contract the water freely. The rules governing this process are described in the Texas Water Code, Title 4, Subchapter O.

When irrigation water rights are converted into DMI water rights, the maximum authorized diversion is reduced to 50% for Class A, 40% for Class B.

There are a few approaches to evaluating urbanization rates within the LRGV that are presented here for comparison. The intent of this evaluation is to estimate how much water has historically shifted from agricultural use to municipal use, and how much can be expected to be converted in the future. The studies that are summarized here are:

- Texas AgriLife Extension Irrigation District Engineering and Assistance (IDEA) Program: measures the change in flat rate acreage in selected Irrigation Districts; and
- Texas Water Resource Institute (TWRI) TR-419: discusses the rates at which urban areas are growing.
- Texas Water Resource Institute (TWRI) Technical Report (TR) 387: evaluates the rate at which land use is changing;

While the change in flat rate acreage calculated in the IDEA report is the closest correlation to the rate at which water rights may be separated from the land, there are not data for all of the Irrigation Districts in the LRGV and not all excluded water rights are converted to municipal water rights. The IDEA study was found to have too broad a range of urbanization rates to be useful for estimating regional rates, but localized information may be useful later in the study.

TR-419 focused on the rate with which urban areas are growing, but the correlation between expansion of urban areas and the reduction of irrigated acreage is shown to be a complex one. This increase in urban development may provide insight into the details of growth in local areas, but is not useful here to predict the rate at which water rights are converted.

The TR-387 estimates rates of change for acreage of irrigated land in each county and in some Irrigation Districts. These county-wide estimates for rate-of-change are applied to currently held water rights for the preferred methodology for estimating converted water rights.

4.5.1 Change in Irrigated Acreage - IDEA

Alternately, the Irrigation District Engineering Assistance (IDEA) initiative under the Irrigation Technology Program at Texas A&M AgriLife Extension did an evaluation of the change in flat rate and irrigated acreage for 10 of the 27 Irrigation Districts in the LRGV from 2000 to 2007. In Table 4-5, the rates of urbanization are separated into lands that are irrigated acreage and flat rate acreage. Flat Rate acreage is typically defined as the acreage within an Irrigation District's boundary that pays a yearly flat rate for the opportunity to utilize irrigation water and has water rights tied to the property. This can vary from the acreage of farmland in an Irrigation District because some areas do not have the right to irrigation water, and not all flat-rate land is irrigated on a regular basis. When acreage is removed from flat-rate status, the process is called exclusion.

Table 4-5 Rates of Reduction in Irrigated and Flat Rate Lands in Selected Counties, 2000 - 2007[5]

	IRRIGATED ACREAGE			FLAT RATE ACREAGE		
IRRIGATION DISTRICT	TOTAL	7-YEAR REDUCTION	YEARLY AVERAGE REDUCTION	TOTAL	7-YEAR REDUCTION	DECADAL AVERAGE REDUCTION
Cameron County ID No. 2, San Benito	18,676	1,228	0.94%	60,807	4,760	11.2%
Hidalgo and Cameron Counties Irrigation District No. 9, Mercedes	60,000	6,000	1.43%	57,737	5,157	12.8%
Adams Gardens Irrigation District No. 19	4,633	-93	-0.29%	7,242	161	3.2%
Brownsville Irrigation District	9,325	1,920	2.94%	20,350	2,993	21.0%
Delta Lake Irrigation District	70,439	333	0.07%	70,439	333	0.7%
Hidalgo County Irrigation District No. 6, Goodwin/Mission 6	11,087	1,091	1.41%	16,827	2,288	19.4%
Hidalgo County Irrigation District No. 13, Baptist Seminary	1,000	340	4.86%	0	0	0.0%
Sharyland, Hidalgo County Improvement District No. 19	12,107	4,005	4.73%	4,046	1,894	66.9%
Harlingen Irrigation District No. 1	34,500	1,000	0.41%	35,251	1,144	4.6%
Cameron County Irrigation District No. 6, Los Fresnos	13,186	7,483	8.06%	0	0	0.0%
AVERAGE	**--**	**--**	**2.2%**	**--**	**--**	**12.8%**

The average decadal rate of conversion among the listed Irrigation Districts is 12.8% of Flat Rate acreage, and the mean is 4.6%. The rates range from 0 to 66%, which is such a broad range that it is difficult to say that this can be meaningfully applied to the remaining 18 Irrigation Districts in the study area. As more specific regions are identified for evaluation in this study, the Irrigation District-based data may become useful, but at this time it is not a valuable estimate of region-wide urbanization.

[5] Special Study #2 in the 2011 Rio Grande Regional Water Plan, digital appendices.

4.5.2 Increase in Urban Area: TR-419

The Texas Water Resource Institute (TWRI) evaluated GIS data and aerial photography to estimate the rates of urbanization in the LRGV in TR-419. Acreage presented for each Irrigation District is from 1996 and 2006. The results of TR-419 for rates of urbanization by county and by district are presented in Table 4-6 and Table 4-7.

Table 4-6 Urban area within Counties in 1996 and 2006 (TWRI, TR-419)

COUNTY	TOTAL AREA (ACRES)	URBAN AREA (1996) (ACRES)	URBAN AREA (1996) (% OF TOTAL)	URBAN AREA (2006) (ACRES)	URBAN AREA (2006) (% OF TOTAL)	INCREASE (%)
Cameron	613,036	66,189	11	81,635	13	23
Hidalgo	1,012,982	118,466	12	160,095	16	35
Willacy	393,819	3,084	1	3,509	1	14
Total/Average	**2,019,837**	**187,739**	**9**	**245,239**	**12**	**31**

Table 4-7 Urban area within Irrigation Districts in 1996 and 2006 (TWRI, TR-419)

DISTRICT	TOTAL AREA (ACRES)	URBAN AREA (1996) (ACRES)	URBAN AREA (1996) (% OF TOTAL)	URBAN AREA (2006) (ACRES)	URBAN AREA (2006) (% OF TOTAL)	INCREASE (%)
Adams Garden	9,600	532	5.5	1,380	14.4	159
Bayview	10,700	24	0.2	120	1.1	400
BID	22,000	8,724	39.7	9,915	45.1	14
CCWID16	2,200	260	11.8	415	18.9	60
CCID2	79,000	8,384	10.6	10,925	13.8	30
CCID6	33,000	4,439	13.5	7,948	24.1	79
CCWID10	4,700	135	2.9	224	4.8	66
Delta Lake	85,600	1,127	1.3	1,841	2.2	63
Donna	47,000	4,357	9.3	7,310	15.6	68
Engelman	11,200	144	1.3	331	3	130
Harlingen	56,500	14,662	26	16,955	30	16
HCCID9	87,900	16,721	19	22,716	25.8	36
HCID1	38,600	22,633	58.6	25,327	65.6	12
HCID13	2,200	117	5.3	469	21.3	301
HCID16	13,600	83	0.6	1,005	7.4	1,111

DISTRICT	TOTAL AREA (ACRES)	URBAN AREA (1996)		URBAN AREA (2006)		INCREASE (%)
		(ACRES)	(% OF TOTAL)	(ACRES)	(% OF TOTAL)	
HCID19	4,800	0	0	1,908	39.8	--
HCWCID18	2,400	15	0.6	300	12.5	1,900
HCID2	72,600	33,006	45.5	39,107	53.9	18
HCWID5	8,100	1,142	14.1	1,424	17.6	25
HCID6	22,900	5,677	24.8	9,595	41.9	69
HCMUD1	2,000	1,016	50.8	1,811	90.6	78
HCWID3	9,100	6,618	72.7	6,936	76.2	5
La Feria	36,200	2,626	7.3	3,809	10.5	45
Santa Cruz	39,500	2,889	7.3	3,715	9.4	29
Santa Maria	4,000	242	6.1	365	9.1	51
United	37,800	15,336	40.6	17,794	47.1	16
Valley Acres	11,200	162	1.4	162	1.4	0
VMUD2	4,800	1,142	23.8	1,142	23.8	0
Total/Average	**759,200**	**152,213**	**17.9**	**194,949**	**26**	**45.2**

This TWRI study focused on the impact of urbanization on the operations of the Irrigation District networks. However, not all of the lands that are developed are converted from irrigated land, therefore these data describe what portion of land is urbanized, but not necessarily how much of this land was irrigated previously and could be converted for municipal use.

4.5.3 Land Cover Change: TR-378

A separate TWRI report, TR-378[6], attempted to classify the land areas that were urbanized using Landsat Satellite Multi-Spectral Classification with data from 1993 and 2003. These years had comparable average temperatures and precipitation, as well as similar combined storage levels in the Amistad-Falcon Reservoir system. Images covering three counties (Cameron, Hidalgo, and Willacy) were evaluated, and the land cover was evaluated based on five classes were adapted from the USGS' Anderson classification system.

1. Water land cover is assigned for areas that persistently are water covered, provided that, if linear, they are at least 1/8 mile wide and, if extended, cover at least 40 acres.

[6] Landsat Satellite Multi-Spectral Classification of Land Cover Change for GIS-Based Urbanization Analysis in Irrigation Districts: Evaluation in the Lower Rio Grande Valley. Yanbo Huang, Extension Associate and Guy Fipps, Professor and Extension Agricultural Engineer, Department of Biological and Agricultural Engineering, Texas A&M University, College Station, TX. 2011

2. Barren Land is defines as land in which less than one-third of the area has vegetation or other cover. In general, it is an area of thin soil, sand, or rocks. Vegetation, if present, is more widely spaced and consists mostly of scrub and brush.

3. Irrigated land includes all acreage that is irrigated for the purpose of farming.

4. Vegetated land includes lands with plant cover beyond the limited scrub and brush of barren land, but not such that the area is irrigated.

5. Urban areas are heavily used, and much of the land covered by structures. Included in this category are cities, towns, villages, strip developments along highways, transportation, power, and communications facilities, and areas such as those occupied by mills, shopping centers, industrial and commercial complexes, and institutions that may, in some instances, be isolated from urban areas.

Urbanization rates in the study area were derived using this method. Acreage for each of five types of land cover and county-wide population estimates are shown in Tables 4-5 – 4-7 for each of the three counties studied.

Table 4-8 Land Cover Change Estimation in Hidalgo County

LAND COVER CATEGORY	1993 AREA (ACRES)	2003 AREA (ACRES)	NET CHANGE (%)
Water	17,697.66	14,169.00	-19.89
Barren Land	283,980.48	309,936.39	9.14
Irrigated Land	532,531.04	477,960.38	-10.25
Vegetated Land	136,140.03	141,326.76	3.8
Urban	45,158.44	72,112.65	59.69
Population	447,508	627,164	40
Population Density (people/acre)	0.44	0.62	

For Hidalgo County, there were significant increases in both urban area and population. Hidalgo County has the highest population density, and has the closest correlation between increased population and increased urban area. The reduction in irrigated acreage may be related to the increase in vegetated land.

Table 4-9 Land Cover Change Estimation in Cameron County

LAND COVER CATEGORY	1993 AREA (ACRES)	2003 AREA (ACRES)	NET CHANGE (%)
Water	135,181.26	139,041.04	2.85
Barren Land	221,139.21	217,721.74	-1.55

LAND COVER CATEGORY	1993 AREA (ACRES)	2003 AREA (ACRES)	NET CHANGE (%)
Irrigated Land	276,330.11	257,757.70	-6.72
Vegetated Land	83,410.29	84,806.44	1.67
Urban	32,123.65	49,082.47	52.8
Population	288,297	357,097	24
Population Density (people/acre)	0.35	0.44	

Cameron County has a high rate of increase in urban land area but a lower rate of population increase, although there is a correlation. Cameron County has a number of industries that depend on tourism, shipping, and trade with Mexico, which could contribute to commercial development that may not correlate with population growth. The acreage decrease in irrigated land is the same magnitude of acreage increase in urbanized acreage, showing a direct correlation between the two.

Table 4-10 Land Cover Change Estimation in Willacy County

LAND COVER CATEGORY	1993 AREA (ACRES)	2003 AREA (ACRES)	NET CHANGE (%)
Water	51,442.32	76,567.96	48.84
Barren Land	222,700.91	189,598.72	-14.86
Irrigated Land	106,818.55	100,455.60	-5.96
Vegetated Land	87,171.23	100,665.63	15.48
Urban	3,370.51	4,237.85	25.68
Population	18,880	19,857	5
Population Density (people/acre)	0.04	0.04	--

Willacy County has a very small population in comparison with the other counties, but has seen over 25% increases in urban land cover. These data do not clearly link the small decrease in irrigated land with urbanization. Willacy County does not directly access the Rio Grande, although parts of the county are served by Irrigation Districts. Other portions of the county have access to Arroyo Colorado water or groundwater, where Irrigation Districts do not exist.

4.5.3.1 Estimated Rate of Water Right Conversion

County-wide urbanization estimates can be used to approximate rates of water right conversion for Cameron, Hidalgo, and Willacy Counties. The decadal percent reduction for irrigated acreage was used to estimate how much water would be converted from agricultural use to municipal use. Because the increase in urban area is significantly larger than the decrease in irrigated acreage, it is the decrease in irrigated acreage that is used as a predictor of the rate of water right conversion.

It is assumed that only a portion of the water rights are excluded, or separated, from associated farmland. Based on the mechanisms for conversion defined by Subchapter O, only 50% of the lost irrigated acreage is assumed to have water rights associated as flat rate acreage and subdivided in such a way to meet the Subchapter O rules. It is also assumed that only 50% of the irrigated acreage that is subdivided pursuant to the Subchapter O rules is then purchased by a utility and converted for municipal use. Therefore, the rate that water rights are converted is estimated using 25% of the rate at which irrigated land has historically been lost in each county of the study area (see Table 4-8).

Table 4-11 Decadal Rate of Change in Irrigated Acreage and Water Rights Conversion (Percent per Decade)

	CHANGE IN IRRIGATED ACREAGE PER DECADE	RATE OF AGRICULTURAL WATER RIGHT CONVERSION PER DECADE
Cameron	-6.72%	-1.7%
Hidalgo	-10.25%	-2.6%
Willacy	-5.96%	-1.5%

Other factors that are in the 25% assumption include:

- Some irrigated acreage may be irrigated with alternate water sources such as local groundwater or run-of-river surface water from drainage canals or contract water.
- Sometimes water that is associated with flat rate acreage can be consolidated on some portion of the landholders acreage for a high-value crop, in which case the irrigated acreage would be less than the flat rate acreage for that particular farm although the same water rights are associated with the land.

The agricultural and municipal water rights held in each county as of 2013 are shown in Table 4-12 (TCEQ).

Table 4-12 Annual Water Rights (AWR) for Irrigation and Municipal Use in the LRGV (AF/YR)

2013 AWR	MUNI WR	CLASS A IRRIGATION WATER RIGHTS	CLASS B IRRIGATION WATER RIGHTS
Cameron	83,317	417,158	62,270
Hidalgo	143,850	1,001,305	49,385
Willacy	273	474	415
LRGV Total	**236,975**	**1,422,330**	**138,141**

The rate of irrigated land cover lost per decade, as shown for each county in Table 4-11, was applied to the currently held agricultural water rights to calculate the additional municipal water rights per county shown in Table 4-13.

Table 4-13 Converted and Total Municipal Water Rights based on TR-387 Land Cover Urbanization Estimates (AF/YR)

MUNI WATER RIGHTS	EXISTING	2020		2030		2040		2050		2060		2070	
		Conv.	Total	Conv.	Total	Conv.	Total	Conv.	Total	Conv.	Total	Conv.	Total
Cameron County	83,317	2,746	86,063	3,876	89,939	3,811	93,751	3,747	97,498	3,684	101,182	3,622	104,805
Hidalgo County	143,850	9,335	153,185	13,096	166,281	12,761	179,042	12,434	191,475	12,115	203,590	11,805	215,395
Willacy County	273	4	277	6	283	6	289	6	295	6	300	6	306
Total	**227,440**	**12,085**	**239,525**	**16,979**	**256,503**	**16,578**	**273,081**	**16,187**	**289,268**	**15,805**	**305,073**	**15,433**	**320,506**

4.5.4 Municipal Water Availability

The estimate used on the R-378 report includes the water that is required to cover conveyance losses. Table 4-14 shows county-wide estimates of conveyance losses in the delivery infrastructure, and the impact on predicted municipal supplies after urbanization and an estimated 90% utilization of municipal water rights, as discussed in the Section 4.2 2013 Water Rights .

Table 4-14 Projected Municipal Surface Water Supplies per Decade Including Conversion of Water Rights due to Urbanization, Irrigation District Conveyance Losses, and 90% Utilization (AF/YR)

MUNICIPAL WATER AVAILABILITY PER COUNTY	CONVEYANCE EFFICIENCY	2020	2030	2040	2050	2060	2070
Cameron County	68%	52,670	55,043	57,375	59,669	61,924	64,140
Hidalgo County	71%	97,885	106,254	114,408	122,353	130,094	137,637
Willacy County	70%	175	178	182	186	189	193
Lower Rio Grande Valley Total		**150,730**	**161,475**	**171,965**	**182,207**	**192,207**	**201,971**

4.6 CONCLUSION

4.6.1 Surface Water available compared to demand

Overall findings show that there is sufficient water in the Amistad Falcon Reservoir System to meet municipal demands, as depicted in Figure 4-5. This figure compares the Amistad-Falcon Reservoir System Firm Yield (with its current projection and how it is estimated to be impacted due to climate change) to the amount of current DMI water rights and municipal demands through 2070. Even with the 86,000 acre-feet/year reduction in Firm Yield due to climate change, the total 2070 municipal demand can be met by surface water.

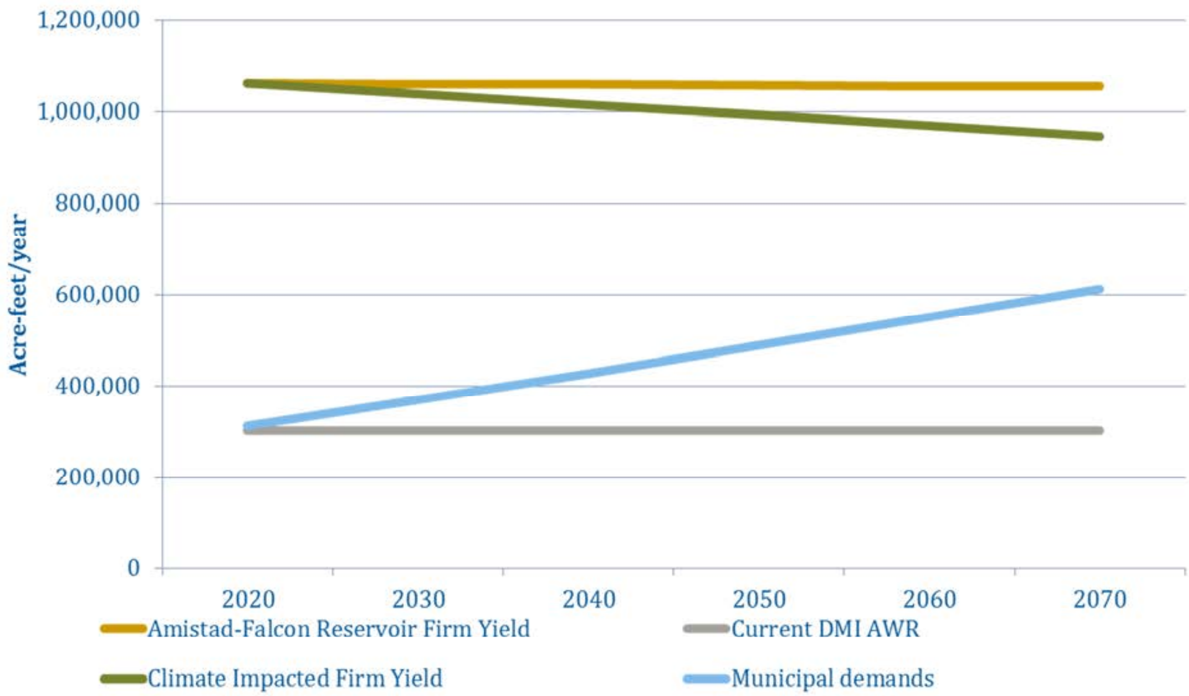


Figure 4-5 Amistad-Falcon Reservoir System Firm Yield Compared to DMI Water Rights and Municipal Demands (AF/YR)

However, the municipal demand cannot be met with the estimated municipal supplies, which are the sum of the estimated amount of water rights converted from agricultural to municipal use and the 2013 municipal water rights(taking in account distribution system efficiency and 10% not used due to operational decisions). Figure 4-6 shows the estimated municipal supplies compared with the projections for municipal demands. There are currently 301,920 acre-feet/year of DMI water rights and conversions only yields approximately 50,000 acre-feet/year in 2070. It should be noted that if converted water rights were treated through a new regional water treatment plant, more would be available due to increased conveyance efficiencies. Although water right conversion will not meet the total water needs for the region, it should be part of the solution.

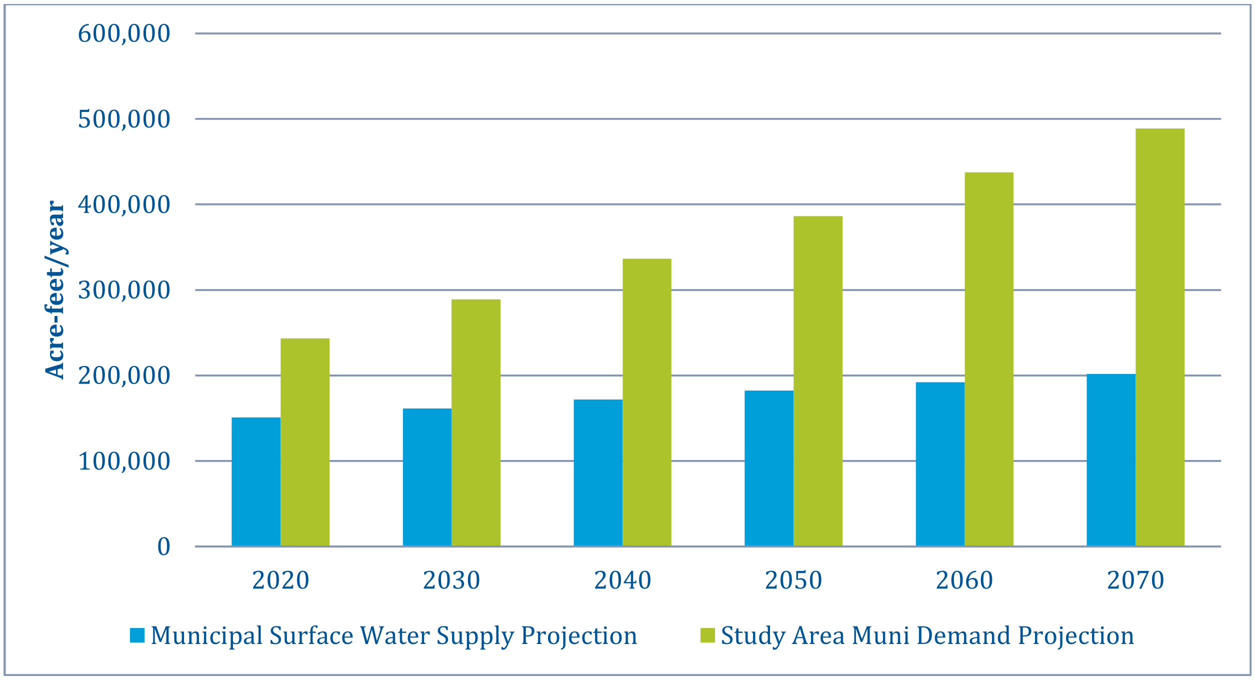


Figure 4-6 Comparison of Municipal Demands and Surface Water Supplies with the Water Right Conversion Rate Estimated using TR387with Reductions for Supply Management, and Conveyance Losses.

4.6.2 Surface Water Utilization Issues

There are changes to utilizing surface water to meet current and future demands in the Lower Rio Grande Valley. As stated, there are not enough agricultural water rights that can be converted to meet all of the future demands. Use of the available water is expected to continue using existing plants, modification through institutional changes, and possibly be treated at a regional level.

4.6.2.1 Continued Expansion and Improvements of Existing Surface Water Infrastructure

As demands increase in the region, existing surface water treatment plants would expand to meet the demands within their service areas. As discussed previously, a certain amount of agricultural water is expected to be converted to meet those growing needs, however, its use is contingent on the expansion of surface water treatment plants, storage and conveyance infrastructure. It is anticipated that the lowest cost option for water suppliers with existing excess capacity available at their surface water plants will be to purchase water rights as available and as treatment and conveyance capacity allows.

Several Factors contribute to the inefficiency of this approach. However, the most significant issue is the existing Irrigation District infrastructure contributes an average of 30% losses in surface water between diversion from the Rio Grande and delivery to a utility. Also, as agricultural water rights are converted to municipal water rights, some Irrigation Districts will need to make significant operational and infrastructure changes in order to serve the users in their districts. Concerns about push water and the ability of the Districts to respond to severe drought will persist until significant changes and improvements have been made.

Moreover, some issues that are currently faced by the study area are not addressed in this scenario. Reliance on a sole source of water, as most utilities in the region do, comes with inherent risks.

Climate variability and irregular deliveries from Mexico will continue to put pressure on the region as a whole, particularly agriculture. More specific to municipal utilities is the concern about the ability of the region to respond in the case of a spill or contamination of the Rio Grande and its reservoirs.

4.6.2.2 Legal, Administrative, and Institutional Changes

There have been incremental changes to the way that the water market operates, and to some of the rules for how the Amistad-Falcon Reservoir system is operated, and the rules governing Irrigation Districts. Further changes to the legal, administrative and institutional procedures regarding water rights would be implemented to provide more efficient use of contract water, quicker access to WRs available for transfer and increased WR use from existing DMI WR owners. These small changes have the ability to increase the ease and functionality of the water systems in the region. Although most of these rules require legislation to change, it is valuable to continue to consider ways in which the system could be improved.

4.6.2.3 Regional Surface Water Plant

Another potential solution is a regional surface water treatment plant. Large scale and centralized treatment at a large-capacity plant could improve efficiency of the treatment processes and relieve some smaller communities from operating and maintaining their own surface water treatment facilities. Treated water would be delivered to local utilities via transmission pipeline for distribution. Although the cost of water would likely increase from current rates due to pumping costs, there would be efficiencies gained through the improved method of raw water conveyance and centralized treatment. Another benefit of this alternative would be eliminating the push-water issue by creating a treated water distribution network which is no longer reliant on irrigation water for operations. The surface water plant also could utilize run of the river, excess flows during no charge events, or highly treated wastewater effluent. Any of these source waters could be stored in Aquifer Storage and Recovery facilities after treatment as well.

CHAPTER 5 - GROUNDWATER HYDROLOGY

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

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5.0 Groundwater Hydrology

5.1 BRACKISH GROUNDWATER AVAILABILITY

The Gulf Coast Aquifer in the Lower Rio Grande Valley (LRGV) is an important water resource for the region. Although the water quality in the aquifer is predominately slightly-to moderately-saline, it provides significant quantities of water for agricultural, municipal, and domestic uses. There are currently seven groundwater desalination plants in operation in the LRGV, and over 20 additional plants are recommended as part of water planning efforts. With the anticipated growth in the LRGV over the next 50 years, the Gulf Coast Aquifer will continue to be an essential component of the region's water supply.

5.1.1 Purpose

This Chapter of the report provides an overview of the Gulf Coast Aquifer, a summary of rules and regulations that govern the use of groundwater, an explanation of how two locations for regional brackish groundwater well fields were identified, and discussion of the expected impacts to groundwater and surface water that operation of each well field could cause. It also discusses aquifer storage and recovery potential in the area and its potential impacts to the hydrology. If any of the well field options are pursued, local hydrogeological analysis and testing of aquifer conditions would be required.

5.1.2 Overview of the Gulf Coast Aquifer System

The Gulf Coast Aquifer is a major aquifer within the state of Texas that parallels the Gulf Coast from the Rio Grande in the south to the Louisiana border in the north, extending roughly 100 miles inland beneath the Gulf coastal plain. The entire study area (Figure 5-1) is underlain by the Gulf Coast Aquifer, which consists of several geologic formations that comprise three designated aquifer units-the Chicot Aquifer, the Evangeline Aquifer, and the Jasper Aquifer. The geology and hydrogeology of these individual units are described below.

5.1.2.1 Geology

The Gulf Coast Aquifer consists of a discontinuous sequence of interbedded sands, silts, and clays of several Pleistocene- to Oligocene-age formations. Galloway and others (1991) refer to these Cenozoic sediments of the coastal area as a "monotonous sequence of interbedded sandstones and shales that lack distinctive lithostratigraphic units of regional extent." These discontinuous interbedded sands, silts, and clays have generally been subdivided into several formations, although the complex depositional environment makes identification of specific formations difficult (McCoy, 1990). In addition, the lateral continuity of the lithology changes, sometimes over short distances, making the mapping of units and determining the stratigraphic framework difficult (McCoy, 1990). The lithologic formations present in the study area include, from youngest to oldest, the Beaumont Clay, the Lissie Formation (sometimes divided into the Montgomery and Bentley Formations), the Willis Formation, the Goliad Formation, the Fleming Formation (sometimes called the Lagarto Clay), the Oakville Sandstone, and the Catahoula Formation.

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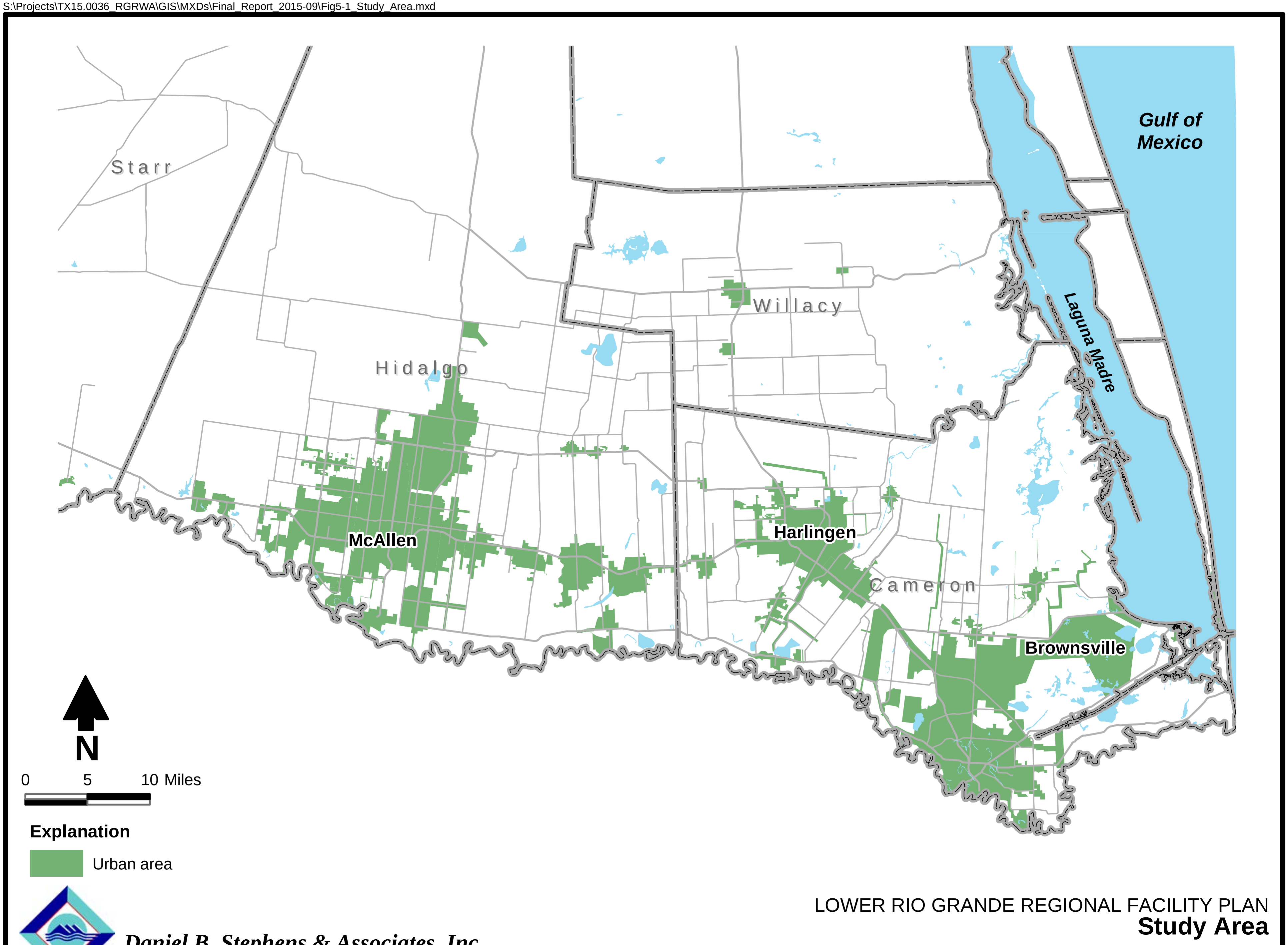

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Study Area

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Figure 5-1

Figure 5-2 shows the surface geology of the study area (Brown and others, 1976). The formations tend to crop out from the coast inland as the formations go from younger to older, although alluvium and windblown deposits can overlie and mask these, as shown in Figure 5-2. In places where alluvium overlies the sediments of the Gulf Coast Aquifer, and in particular where the alluvium has been deposited by the Rio Grande, it can be difficult to distinguish the younger alluvium from the underlying sediments of the Gulf Coast Aquifer. A brief description of the geologic formations that comprise the Gulf Coast Aquifer System is provided below from shallowest (geologically younger) to deepest (geologically oldest).

Beaumont Formation - The Beaumont Formation consists primarily of clay-rich sediments with some sandy intervals. This formation outcrops throughout much of the southern two-thirds of Willacy County, the northern third of Cameron County, and portions of Hidalgo County (Figure 5-2). The Beaumont Formation occurs as a thin veneer of sediments in the updip (outcrop) areas and thickens towards the coast to total thicknesses of over 500 feet (Young and others, 2010). Individual sand layers within the Beaumont Formation range from 20 to 50 feet thick, some of which can stack locally to attain greater effective thicknesses. The thickness of the individual sand layers decreases in the downdip direction (Young and others, 2010; Chowdhury and Mace, 2003).

Lissie Formation - The Lissie Formation lies below the Beaumont Formation and is composed of clay and some fine-grained sand and sandy clay. This formation is stratigraphically defined as the interval between the overlying Beaumont Formation and the underlying Willis Formation (Baker and Dale, 1961; Young and others, 2010). The Lissie Formation ranges in thickness from about 100 feet in its outcrop areas to more than 700 feet at the coast, where it is found at depths of 500 to 1,000 feet below ground surface (Young and others, 2010; Chowdhury and Mace, 2003).

Willis Formation - The Willis Formation lies beneath the Lissie Formation and is composed of predominantly of sand and gravelly sand. Individual sand units in the Willis Formation across the state range from 20 to 200 feet thick, and are separated by mud units of similar thickness (Young and others, 2010). The sand units tend to thin and become more isolated in the southern part of the state, including within the study area (Young and others, 2010; Chowdhury and Mace, 2003). Chowdhury and Mace (2003) report that the Willis Sand has not been identified in the Rio Grande region, while Young and others (2010) indicate that the Willis Formation does not outcrop in the Rio Grande Embayment, but that Pliocene-age sediments are present in the subsurface.

Goliad Formation - The Goliad Formation occurs beneath the Willis Formation and is composed mostly of clay and sand (Baker and Dale, 1961). This formation is between 200 and 1,400 feet thick across the state. The Goliad Sand is a coarse fluvial deposit with many sand lenses, and includes a coarse-grained basal unit containing cobbles and gravel. Net sand thicknesses can be between 100 and 800 feet, but the net sand content decreases to the south where the study area is located (Young and others, 2010; Chowdhury and Mace, 2003). The Goliad sand is the principal source of groundwater in Starr County (Dale, 1952).

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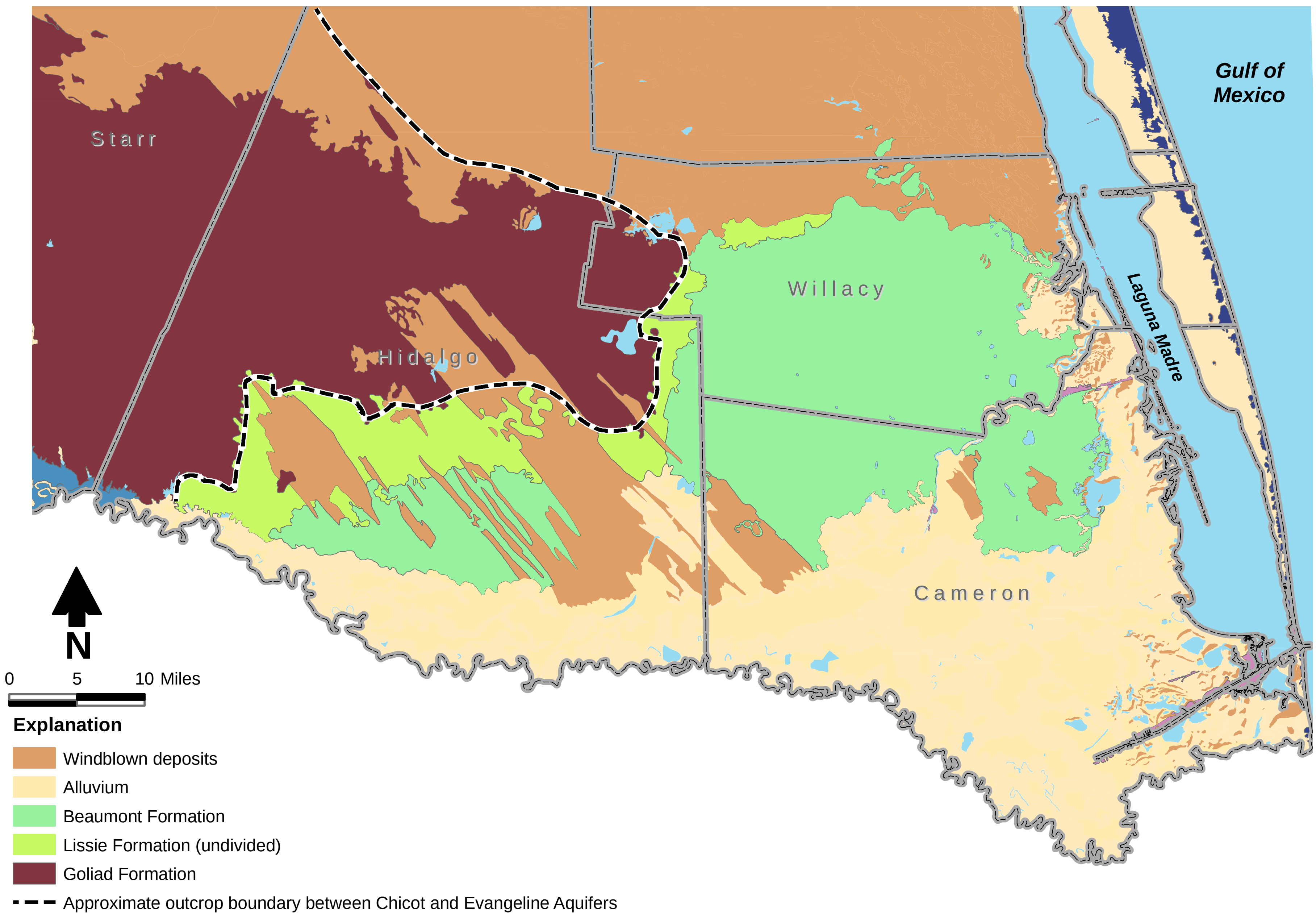

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Surface Geology

Figure 5-2

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Fleming Formation/Lagarto Clay - The Fleming Formation is separated from the underlying Oakville Sandstone by its higher clay content and less massive sand beds (Chowdhury and Mace, 2003). This formation often contains clays and chalky limestone. In South Texas the Lagarto sandstones tend to thin downdip (Young and others, 2010; Chowdhury and Mace, 2003).

Oakville Sandstone - Located stratigraphically between the Catahoula Formation and the Lagarto Clay/Fleming Formation, the Oakville Sandstone is composed of thick sand beds that thicken downdip and contain some clay. The Oakville Sandstone is distinctly sandier than the Lagarto in South Texas, with sand content ranging from about 20 to 50%. This unit pinches out in the subsurface in western Starr and Jim Hogg counties (Young and others, 2010; Chowdhury and Mace, 2003).

5.1.2.2 Hydrogeology

The hydrostratigraphy of the thick sequence of interbedded sands, silts, and clays that make up the formations described in Section 1.2.1 have been interpreted differently by multiple authors. This report follows that convention provided by Baker (1979), who subdivided the geologic units into five general hydrostratigraphic units. The convention provided in Baker (1979) was used by Chowdhury and Mace (2003) and Hutchison and others (2011) for the regional groundwater availability models (GAMs), and by Meyer and others (2014) for the Brackish Resources Aquifer Characterization System (BRACS) study.

From shallowest to deepest the major aquifer units are the Chicot Aquifer, the Evangeline Aquifer, and the Jasper Aquifer (Figure 5-3). These three aquifer units, along with the Burkeville confining unit, comprise the Gulf Coast Aquifer System, often referred to simply as the Gulf Coast Aquifer. A summary description of each of these aquifer units is provided below. These aquifer units thicken from their outcrop area in the down-dip direction toward the coast, where the sedimentary sequence is over 8,000 feet thick (Young and others, 2010). Although the Burkeville confining unit is the only regionally extensive confining unit typically identified in studies of Gulf Coast Aquifer hydrostratigraphy, net sand analysis presented in Meyer and others (2014) indicates that only about 30 to 50% of the geologic formation designated as aquifer units consist of sand, with the remainder being fine grained, low-permeability materials such as silt and clay that lack continuity over significant distances.

The Chicot Aquifer is the shallowest of the three aquifer units. Within the study area it outcrops in Cameron and Willacy County and, like the other units, deepens and thickens towards the coast. This aquifer consists of the Beaumont Clay, the Lissie Formation, and the Willis Sand or equivalent aged Pleistocene sediments. The Chicot Aquifer generally consists of approximately equal amounts of sand and clay. This aquifer can yield moderate to large quantities of water to wells (Chowdhury and Mace, 2003; Baker and Dale, 1961).

The Evangeline Aquifer lies below the Chicot Aquifer and is bounded below by the Burkeville Confining Unit. Within the study area, this aquifer outcrops in Hidalgo County and like the other units deepens and thickens towards the coast. This aquifer consists of the Upper and Lower Goliad Formation, and the Upper Lagarto Formation. This aquifer contains mostly sand, with individual sand beds that are tens of feet thick. This aquifer can yield moderate to large quantities of water to wells (Chowdhury and Mace, 2003; Baker and Dale, 1961).

Epoch and age (millions of years before present)	Geologic formation	Hydrogeologic unit	
Pleistocene (1.8-present)	Beaumont	Chicot Aquifer	Gulf Coast Aquifer
	Lissie		
Pliocene (5.6-1.8)	Willis		
	Upper Goliad	Evangeline Aquifer	
Miocene (23.8-5.6)	Lower Goliad		
	Upper Lagarto		
	Middle Lagarto	Burkeville Confining Unit	
	Lower Lagarto	Jasper Aquifer	
	Oakville		
Oligocene	(upper) Catahoula		

Source: Meyer et al., 2014

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Hydrostratigraphy

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Figure 5-3

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The Jasper aquifer is the deepest of the three aquifers that comprise the Gulf Coast Aquifer System. It occurs below the Burkeville Confining Unit and is bounded below by the Catahoula confining system. Within the study area, this aquifer outcrops in Starr County, and then deepens and thickens towards the Gulf Coast. This aquifer consists of the Lower Lagarto Formation and the Oakville Sandstone. This aquifer can yield moderate amounts of water to wells (Chowdhury and Mace, 2003; Baker and Dale, 1961).

5.1.2.3 Water Level Trends

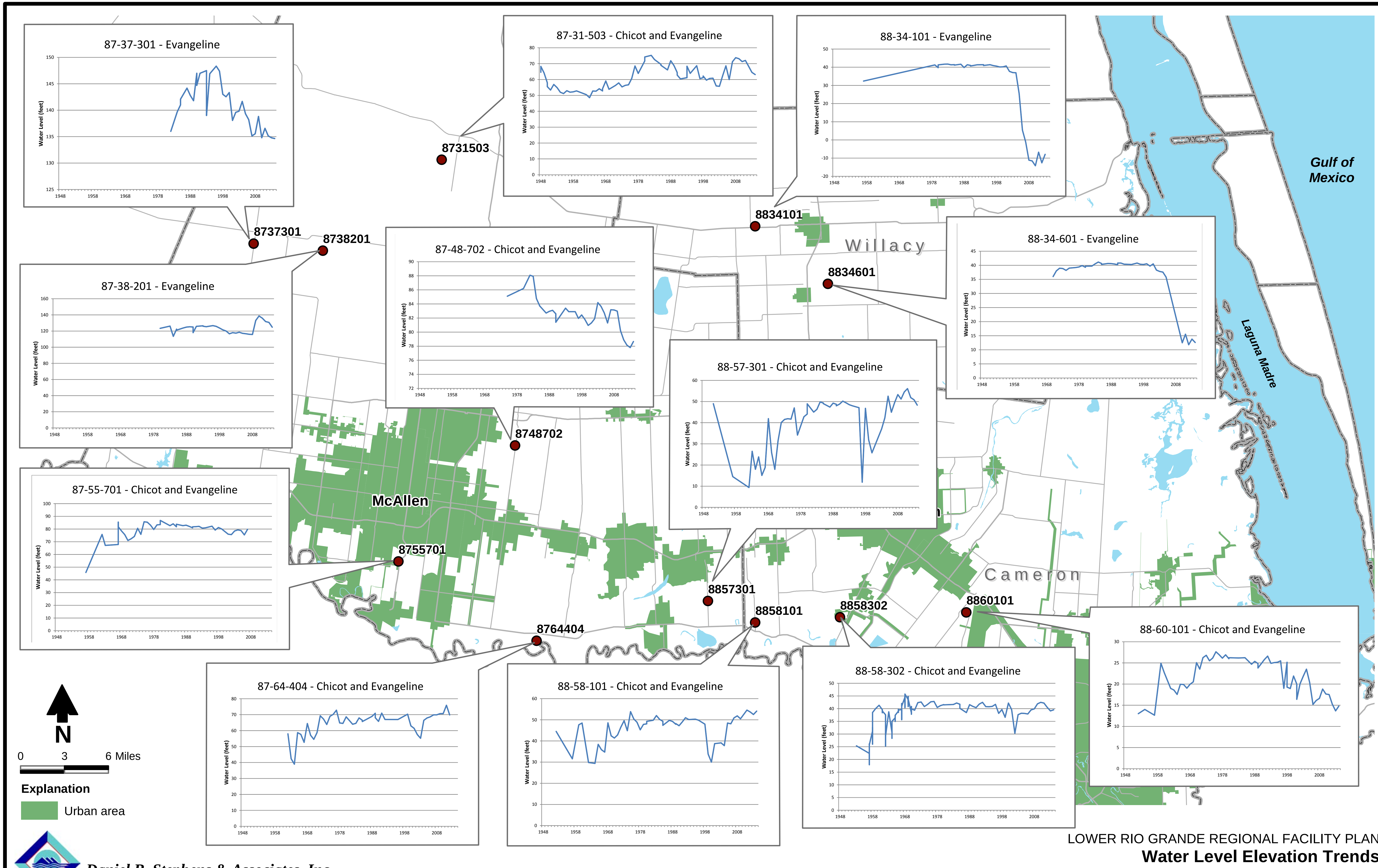
Because no groundwater conservation districts (GCDs) are present in most of the study area, and the GCDs that are present have no data collection programs, the Texas Water Development Board (TWDB) groundwater database was queried for historic groundwater level data to determine water level trends throughout the study area. Many of the wells included in the TWDB database are reportedly screened across both the Chicot and Evangeline Aquifers, or geologic formations that comprise the Chicot and Evangeline Aquifers. Therefore, water levels and trends observed for these wells are representative of combined Chicot and Evangeline Aquifer conditions. Water level elevation trends in the study area are illustrated in Figure 5-4 and are discussed below.

Hydrographs for wells completed in the Evangeline Aquifer in the west-central portion of Willacy County all show water level declines. In wells 88-34-101 and 88-34-601, water levels remained fairly constant until between 2005 and 2010, at which time water levels declined significantly. This trend is likely attributable to additional pumping from, or close to, these wells. Observed water levels in north-central Hidalgo County, in the vicinity of the Red Sands GCD and the City of Edinburg indicate that water levels have remained relatively stable over the long term in the Evangeline and Chicot Aquifers. The water levels in wells completed closer to the Rio Grande in Hidalgo and Cameron Counties in formations that comprise both the Chicot and Evangeline Aquifers have generally remained relatively stable through time. Wells 88-60-101 in Cameron County and well 87-48-702 in Hidalgo County indicate consistent overall declines over the last 20 to 30 years, probably caused by groundwater pumping in the vicinity of these wells.

5.1.3 Historic and Current Groundwater Use

The total reported groundwater produced from the Gulf Coast Aquifer in the four counties in the study area is generally between 10,000 and 40,000 acre-feet per year, as illustrated in Figure 5-5. The groundwater pumping estimates are those reported to the TWDB as part of their water use surveys, and some of the variations and outliers may be due to data entry or reporting errors.

For much of the period between 1980 and 2000, most of the groundwater production (greater than 80%) occurred in Hidalgo County, with additional significant pumpage in Cameron and Starr counties. Since 2000, groundwater production in Cameron County has increased significantly and as of 2012 accounts for approximately 40% of the total production in the region. Pumpage in Starr County has remained fairly constant from 1980 to 2012 at about 700 to 1,500 acre-feet/year which accounts for approximately 5 to 10% of the total pumpage in the region. Pumpage in Willacy County has been negligible for the period between 1980 and 2010, with some moderate increase in the past few years.



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Water Level Elevation Trends

Figure 5-4

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Throughout the study area for most of the period between 1980 and 2000, about 50 to 75% of groundwater was produced for irrigation, about 25 to 50% was produced for municipal use, and lesser amounts were produced for livestock, mining and manufacturing. However, since 2000 the amount of reported irrigation pumpage declined significantly and in the past few years over 90% of the total reported groundwater production in the study area has been for municipal purposes, with lesser amounts for mining and livestock. Very little recent reported production has been for manufacturing, steam-electric power, or irrigation.

In Cameron County, historic groundwater use from 1980 to the present has generally been between 1,000 and 3,000 acre-feet/year. A few years stand out as significantly higher groundwater use, including 2009 through 2012. In addition, the period 2003 through 2008 appears abnormally low. The accuracy of these reported pumping estimates is unknown without additional data or information. If the reported values are correct, groundwater use in Cameron County has increased significantly over the past several years, driven primarily by municipal demand.

In Hidalgo County, historic groundwater use from 1980 to the present has generally been between 10,000 and 30,000 acre-feet/year based on the TWDB water use surveys (Figure 5-5). A few years stand out as significantly lower groundwater use, including 1986-1988, due to an apparent lack of reporting of irrigation production. Irrigation was the primary use of groundwater in Hidalgo County from 1980-1999, with reported use of 8,000 to 15,000 acre-feet/year. Municipal use from1980 to 1999 was generally 3,000 to 8,000 acre-feet/year, with lesser amounts of groundwater used for manufacturing, mining, steam-electric power, and livestock. Since the year 2000, irrigation pumpage has further declined and municipal pumpage has increased steadily within Hidalgo County, so that currently virtually all of the recent reported groundwater production is for municipal purposes.

In Starr County, historic groundwater use from 1980 to the present has generally been between 500 and 1,000 acre-feet/year. Most of the groundwater production in the county has been for municipal, mining, irrigation, and livestock purposes.

In Willacy County, reported historic groundwater use has been very low. Until 2004, the total reported use of groundwater was less than 100 acre-feet/year. Beginning in 2005 the use has increased, and during the period 2011 through 2012 the reported use increased significantly. Small amounts of groundwater have been used in the county for livestock purposes. Before 2005 this was less than 25 acre-feet/year. Since 2005, this reported use has increased to 90-130 acre-feet/year. The only other use within Willacy County has been for municipal purposes. Prior to 2009, municipal use within Willacy County was low, mostly less than 60 acre-feet/year. However in the past several years the municipal use within the county increased significantly to between 1,300 and 1,500 acre-feet/year.

As noted above, groundwater pumping in the study area for municipal purposes has increased significantly during the past 15 years. Much of this increase has come through the production and desalination of brackish groundwater. There are currently seven brackish water desalination plants operating in the study area (Figure 5-6). A summary of these facilities taken from the TWDB brackish water database (http://www2.twdb.texas.gov/apps/desal/DesalPlants.aspx) is provided in Table 5-1. The approximate total average plant production of about 16 million gallons per day (MGD) is equivalent to nearly 18,000 acre-feet/year of groundwater extraction.

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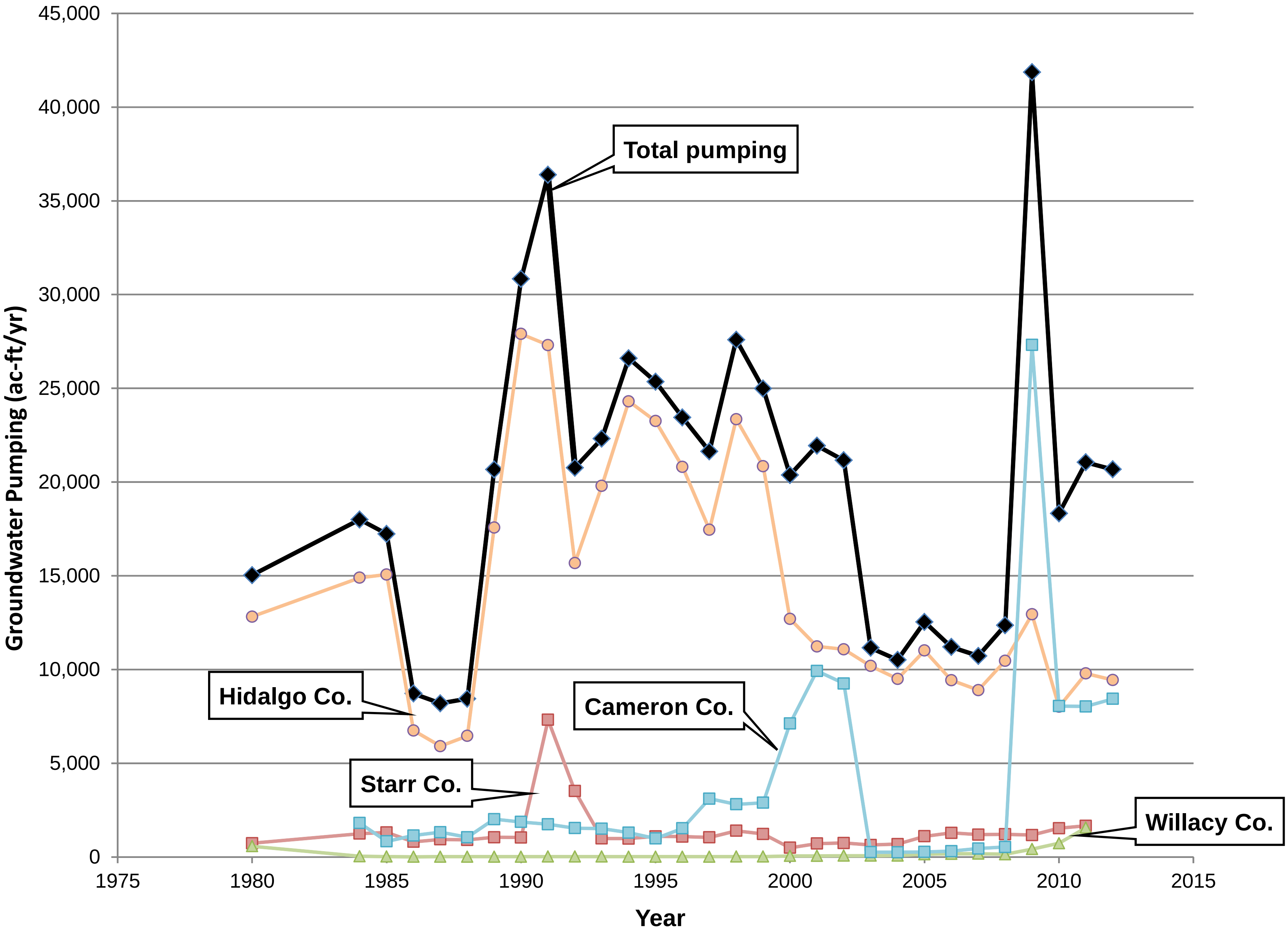

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Reported Groundwater Pumping in the Study Area

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Figure 5-5

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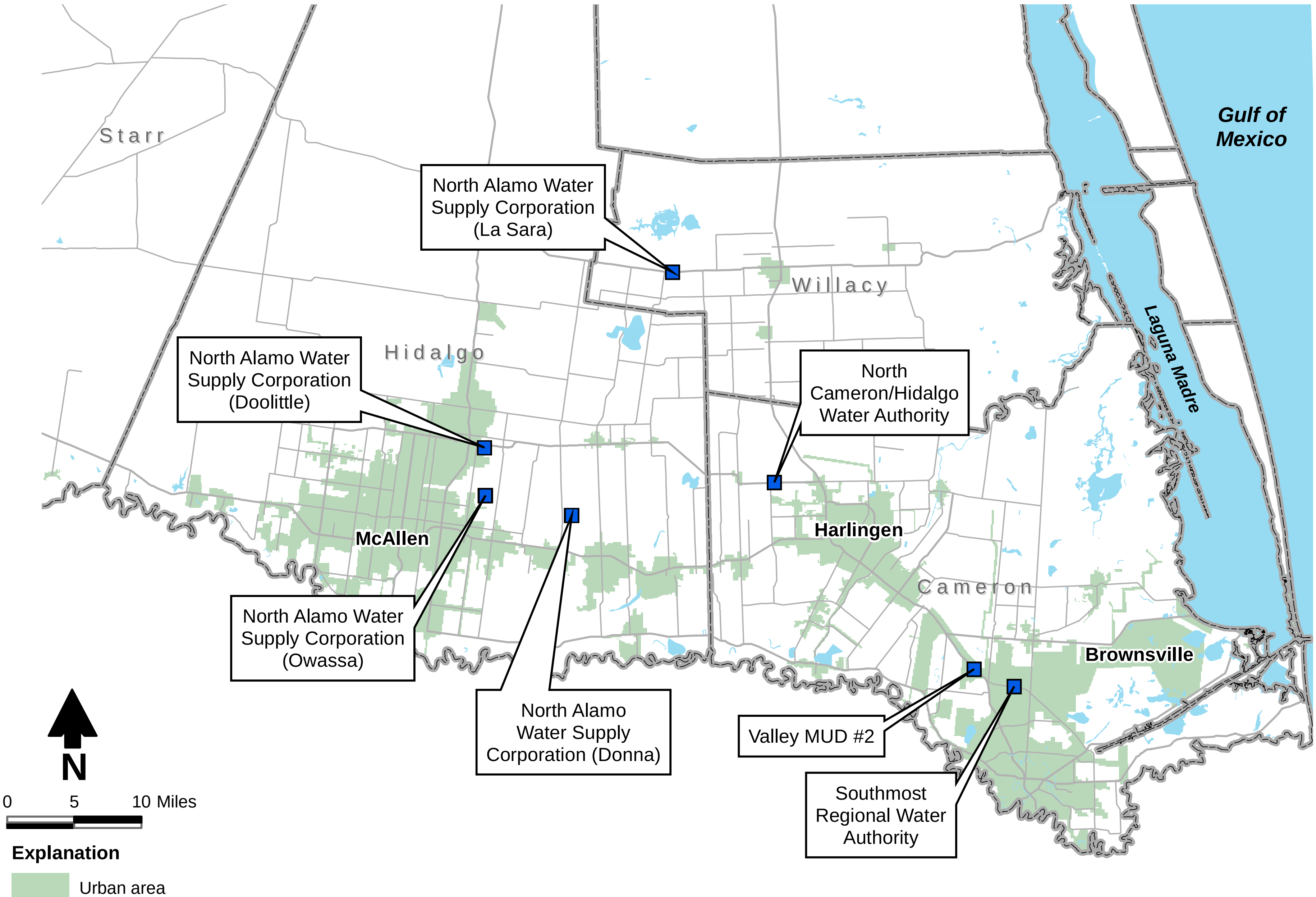

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Existing Desalination Facilities

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Figure 5-6

Table 5-1 Summary of Existing Desalination Facilities within the Study Area

PLANT NAME	AVERAGE PLANT PRODUCTION	YEAR BUILT
Victoria Rd. RO Plant	2.25	2012
North Alamo Water Supply Corporation (Doolittle)	3.50	2008
North Alamo Water Supply Corporation (La Sara)	1.20	2005
North Alamo Water Supply Corporation (Owassa)	2.00	2008
North Cameron/Hidalgo Water Authority	1.15	2006
Southmost Regional Water Authority	5.3	2004
Valley MUD #2	0.75	2000
Total	**16.15**	

5.1.4 Groundwater Management Rules and Regulations

As stated in Chapter 36 of the State Water Code, GCDs are the state's preferred method of groundwater management (TWC §36.0015). Chapter 36 establishes that GCDs will manage groundwater resources through rules developed and implemented in accordance with Chapter 36. Chapter 36 gives GCDs the tools and statutory authority to protect and manage the groundwater resources within their jurisdictional boundaries. Groundwater management does not occur in areas without a GCD. Instead the "rule of capture" applies, which generally means that a landowner or water right holder can produce as much water as needed, as long as it is not wasted, without limit and with no protection for adjacent landowners or water rights holders.

In 2005, the state of Texas initiated joint groundwater planning through HB 1763. This bill divided the state into sixteen groundwater management areas (GMAs), each of which are made up of the GCDs within the boundaries of the GMA. Each GCD within the study area and the GMA process are described in the following sections.

5.1.4.1 Groundwater Conservation Districts

Four GCDs exist in the four county study areas, including the Brush Country GCD; the Kenedy County GCD; the Red Sands GCD; and the Starr County GCD (Figure 5-7). The Brush Country GCD is only present in the far northwestern corner of Hidalgo County, and the Kenedy County GCD is only present in the northern portion of Hidalgo County and the northwestern edge of Willacy County. The Red Sands GCD is the only sub-county GCD in Texas (meaning it does not cover an entire county); it covers several non-contiguous blocks of land in north-central Hidalgo County that overlie some of the most promising areas of potential brackish groundwater resources in the study area. The Starr County GCD covers all of Starr County, which is in the far western extent of the project study area. Large portions of Hidalgo and Willacy Counties, and all of Cameron County, have no GCD.

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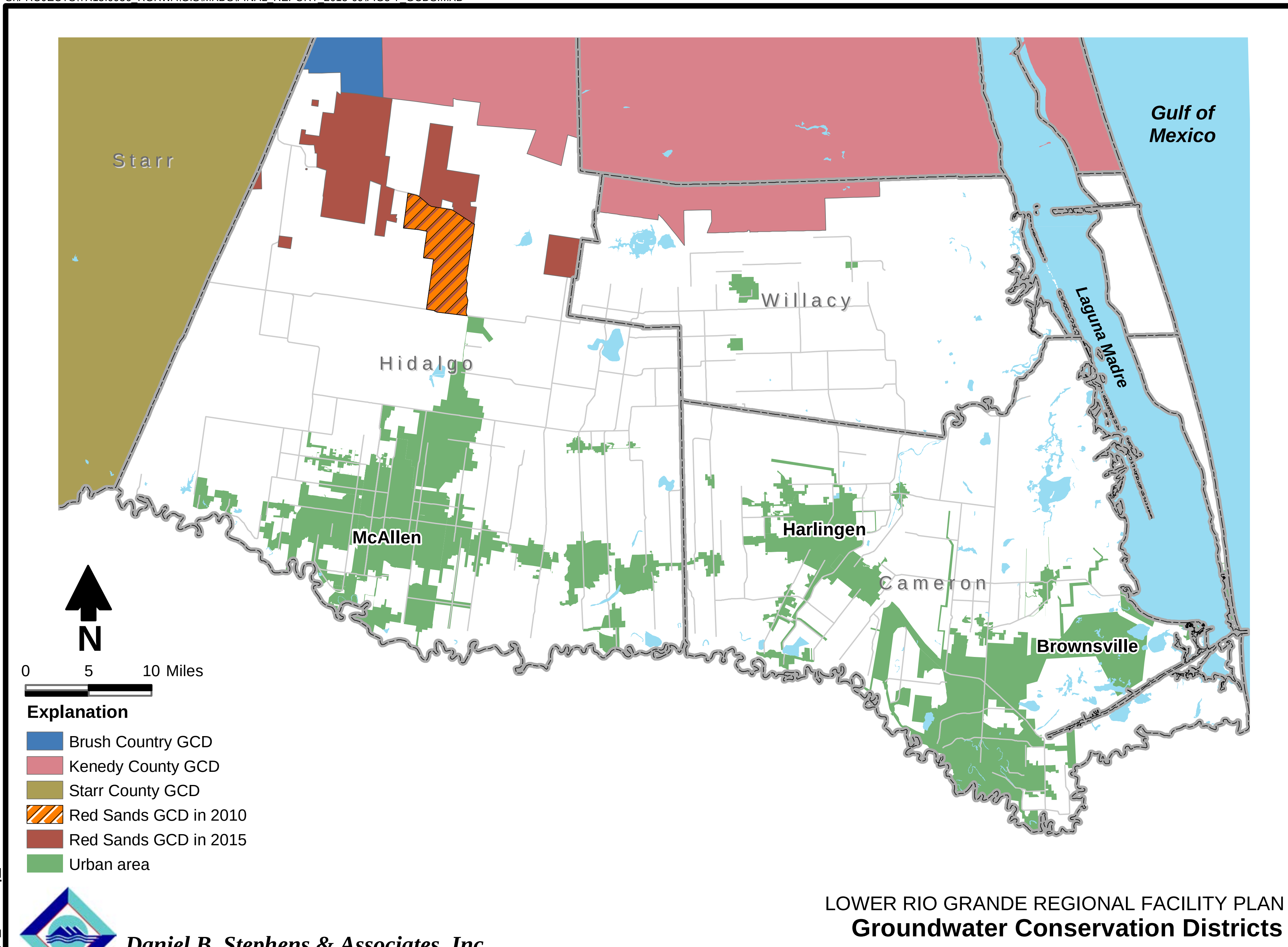

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LOWER RIO GRANDE REGIONAL FACILITY PLAN
Groundwater Conservation Districts

Figure 5-7

Each GCD has its own management plan and rules that must be reviewed and followed when groundwater resources are utilized within the GCD boundaries. The following subsections provide a summary of the significance of each GCD on groundwater production in the study area. These summaries are provided in order of the potential importance of the GCD relative to the development of brackish groundwater resources in the study area.

5.1.4.1.1 Red Sands GCD

The Red Sands GCD is a small, sub-county district covering several non-contiguous tracts in northern Hidalgo County. This district was created in 1999 by the Texas Legislature under SB 1911, and was confirmed in November 2002. The initial extent of the district was smaller than its current size; the district has been adding jurisdiction through annexation and has tripled in size during the past few years. The current size of the Red Sands GCD is slightly less than 20,000 acres.

The Red Sands GCD does not maintain a web site nor does it have regular board meetings, but rather meets once or twice per quarter on an as-needed basis. Some basic information about the district was obtained through a phone conversation with the GCD general manager, Mr. Armando Vela. The Red Sands GCD is active in the joint groundwater planning process and issues permits, although the total number of permits issued to date and the total permitted production were not obtained during the phone conversation. Mr. Vela did indicate that the largest permit issued was for a 1,000 gallon per minute (gpm) well on 260 acres of land that was approved for the irrigation of watermelons.

The rules of the Red Sands GCD include fairly typical well spacing requirements. Wells have to be 50 feet from property lines; well spacing is one foot per one gpm of well capacity up to 1000 gpm. The minimum tract size required for a piece of property to have a well is five acres. Importantly, the Red Sands GCD has maximum allowable permit amounts based on contiguous acreage owned and a maximum production limit of 2 acre-feet per acre per year.

5.1.4.1.2 Starr County GCD

The Starr County GCD is a single-county GCD that includes the entire county within its jurisdictional boundary. This district was formed in 2007, but the district does not maintain a website, does not participate in the joint groundwater planning process, and was unreachable by phone. We are unsure when or if they hold regular board meetings. It appears that the Starr County GCD is not a functional district.

The Starr County GCD rules are undated, and we are unsure if they have been formally adopted and/or approved by their board. The rules have several important aspects for entities interested in developing brackish groundwater resources within the district, including:

- A production limit of one-half an acre-foot per year per acre of land
- Well size and minimum well depth requirements that vary from 400 to 800 gpm depending on what "zone" the well is located in within the county. Although the rules refer to a zone map, the map is not included within the rules.
- Maximum well capacity of 10 gpm per contiguous acre owned.
- There are well spacing rules, but the language in the rules is unclear and difficult to interpret.

5.1.4.1.3 Kenedy County and Brush County GCDs

The Kenedy County GCD is a large district that covers all or parts of seven counties, including the northern portions of Hidalgo and Willacy Counties. The Brush Country GCD is a large district that covers parts of three counties, including most of Brooks and Jim Wells counties and a small portion of northern Hidalgo County. Neither of these districts was contacted as part of this study because of their limited extent within the study area and because potential well field sites are not near either of these districts.

5.1.4.1.4 Addition of GCDs

Much of the study area is not included within the boundaries of a GCD, and to our knowledge there are no efforts underway to create a new district. The presence of a GCD can have a significant impact on the use of groundwater in an area, and some key considerations regarding the potential formation of a GCD are provided below.

Advantages - The alternative to having a GCD is the "rule of capture", which was established as the basis for Texas groundwater law by the Texas Supreme Court in 1904 in the *Houston & Texas Central Railroad Co. v. East* case. This ruling basically states that landowners have the right to produce as much groundwater as they wish from beneath their property, as long as the water is not wasted or produced with malice. Under this doctrine, landowners are not liable to neighboring landowners for the effects of their groundwater use, even if they cause such actions as drying up a neighbor's well or depleting a neighbor's spring flow. For this reason the Rule of Capture is sometimes referred to as the "law of the biggest straw".

GCDs are the primary alternative to the Rule of Capture in Texas. GCDs are granted certain limited authority to regulate and manage groundwater use within the area encompassed by the GCD. The primary advantages of having a GCD is the protection of groundwater availability afforded through the implementation of GCD rules, which typically include well spacing requirements and in some cases limitations on allowable pumping amounts based on well size, contiguous acreage owned or possibly other mechanisms. A GCD is managed by a local board of directors that will be aware of local interests, and in many cases GCDs will collect groundwater data to help develop management plans and district rules. Finally, a GCD provides direct representation in joint groundwater planning, described in Section 5.1.4.2 below.

Disadvantages - A GCD adds another layer of government and potentially restricts the amount of groundwater that a property owner can produce. While many people are willing to accept the formation of a GCD to regulate the use of groundwater because they recognize that the use of their neighbors is also regulated, thereby protecting the groundwater resource as a whole, there are also a significant number of people opposed to the added restrictions on groundwater use, the added regulatory burden, and the introduction of taxes and fees that formation of a GCD would pose.

5.1.4.2 Joint Groundwater Planning

The Texas legislature passed HB 1763 in 2005 which created a new joint groundwater planning process which divided the state into 16 GMAs; each GMA is made up of the GCDs within its boundaries. The joint groundwater planning process requires that the GCDs collectively make policy decisions on how each aquifer within the GMA will be managed over a 50-year planning horizon. Essentially, the GCDs decide how the aquifers within the GMA boundary should "look" in the future, described as the desired future conditions (DFCs) of the aquifer. Basically, the DFCs are quantifiable goals that reflect how the each GMA wants to manage the aquifer(s) under their authority. Target conditions are established for the aquifers such as water levels, water quality, spring flows, or volumes at a specified time or times in the future (Mace and others, 2008). Based on the DFCs approved for each GMA, the TWDB will determine the availability of groundwater (how much can be used while meeting the DFC) throughout the GMA.

The LRGV study area evaluated in report is part of GMA 16, which is made up of all or parts of 16 counties extending from the Rio Grande to Bee and San Patricio counties north of Corpus Christi (Figure 5-8). Because GCDs include only a portion of the four county area, the majority of the study area is not directly represented in the GMA process, and decisions regarding joint groundwater planning are made by the other member districts of GMA 16.

5.1.4.2.1 Desired Future Conditions of GMA 16

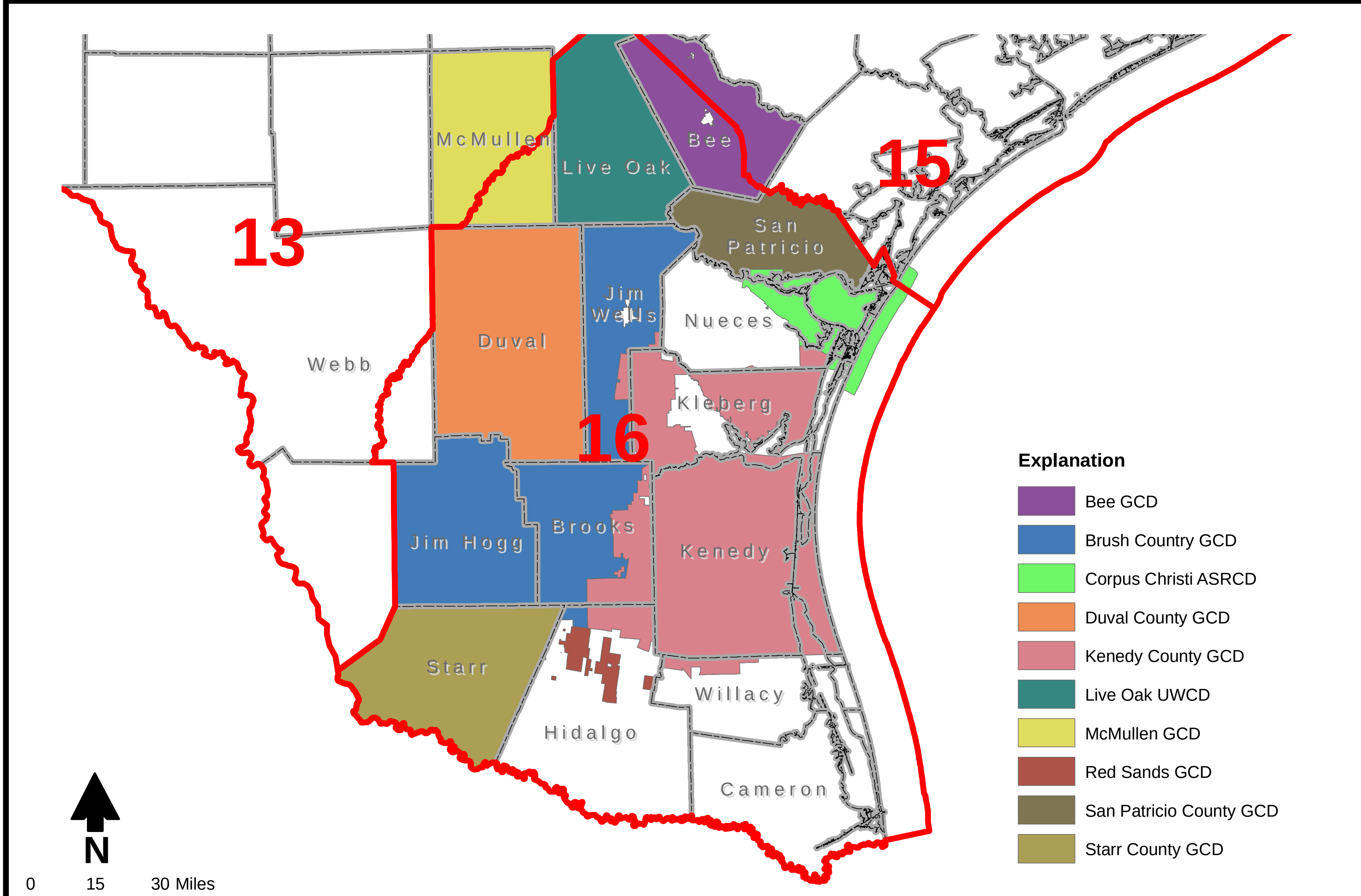
The GMA 16 DFCs include maximum groundwater level drawdown for the Gulf Coast Aquifer as a whole and for the individual aquifer units in each county, as summarized in Table 5-2.

Table 5-2 GMA 16 DFCs for the Gulf Coast Aquifer in the LRGV Study Area

COUNTY/GCD	DESIRED FUTURE CONDITION IN 2060 (FEET OF DRAWDOWN)				
	Layer 1 Chicot	Layer 2 Evangeline	Layer 3 Burkeville	Layer 4 Jasper	Gulf Coast Average
Hidalgo County	55	91	57	56	66
Cameron County	46	63	27	27	41
Willacy County	37	178	39	39	73
Starr County	--	150	137	102	127
Red Sands GCD	--	40	40	40	40

5.1.4.2.2 Modeled Available Groundwater

Based on the DFCs approved by each GMA and Groundwater Availability Models other tools, the TWDB calculates the modeled available groundwater (MAG), which is an estimate of the amount of groundwater that can be pumped over the planning time period that will result in achieving the DFC in each aquifer. The MAG is essentially the official “availability” of groundwater within the GMA.



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Groundwater Management Area 16 and Participating GCDs

Figure 5-8

The TWDB estimated the MAG for each county and for each GCD within GMA 16 based on the DFCs. These MAGs are summarized in Table 5-3. The MAGs for the Gulf Coast Aquifer for Hidalgo, Cameron, and Willacy Counties, which cover most of the study area, are between 20,000 and 50,000 acre-feet per year. The MAG for the Red Sands GCD portion of Hidalgo County is also provided in Table 5-3 to show that it is 584 acre-feet/year, which is only about 1% of the county MAG. In addition, with the Red Sands GCD covering nearly 20,000 acres, and considering their established production limit of 2 acre-feet/year per acre, the total allowable production for the GCD would be nearly 40,000 acre-feet/year. The MAG of 584 acre-feet/year is a small fraction of this potential amount of pumping and may serve to limit groundwater development within the district boundaries. The TWDB did not provide MAGs for those portions of the Kenedy County GCD or the Brush Country GCD that lie within Willacy and Hidalgo counties, respectively.

Table 5-3 Modeled Available Groundwater Values for the Gulf Coast Aquifer in the LRGV Study Area

COUNTY/GCD	MODELED AVAILABLE GROUNDWATER (ACRE-FEET/YEAR)					
	2010	2020	2030	2040	2050	2060
Hidalgo County	41,926	41,926	41,926	41,926	41,926	41,926
Cameron County	50,560	50,560	50,560	50,560	50,560	50,560
Willacy County	20,013	20,013	20,013	20,013	20,013	20,013
Starr County	7,526	7,526	7,526	7,526	7,526	7,526
Red Sands GCD	584	584	584	584	584	584

5.1.4.3 Implications for Brackish Groundwater Development

The MAG values for much of the study area (Cameron, Hidalgo, and Willacy counties) are currently much larger than the current groundwater production in these counties. The MAGs have a variety of implications depending on whether or not a GCD exists.

In areas with a GCD, the MAG serves as a permitting goal for the district. While the MAG is not technically a maximum permitting limit, and many GCDs have issued permits in total greater than the MAG, many GCDs may be reluctant to issue permits that total more than the MAG. This makes the MAG more of a permitting “goal”, but groundwater producers should not expect to be able to easily obtain permits from a GCD that are significantly above the MAG. This is important in the study area because although most of the region is not currently covered by a GCD, one could be formed in the future. If this is the case, the GCD will be bound by the DFCs and MAGs that have been established by GMA 16, and the newly created GCD will not have a chance to update these DFCs until the next round of joint groundwater planning, which must be held at least every five years.

In areas without a GCD, which currently includes most of the study area, MAGs do not serve as a permitting limit because there is no agency to issue, manage, and enforce permits. Groundwater producers are therefore free to produce as much water as they wish in accordance with the Rule of Capture. However, for a groundwater project to receive state funding in areas without a GCD, the MAG would still serve as a production limit. If a project proposes to produce more water than is “available”, it may not be included in the regional water plan and cannot receive funding from the state.

5.1.5 Summary of Previous Studies

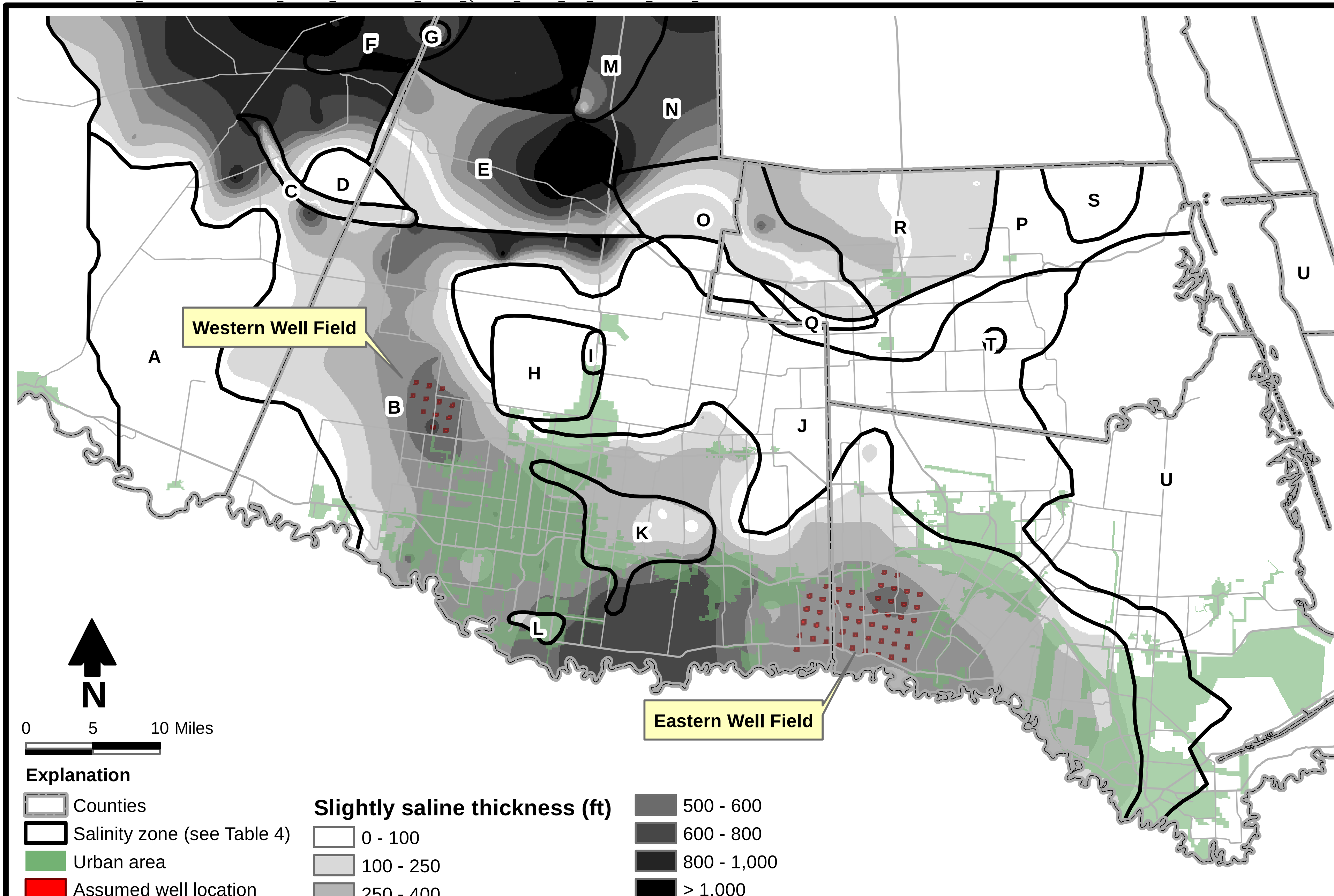
Important groundwater and hydrostratigraphic evaluations in the four county study area conducted from the 1950s to the 1990s include Dale, 1952; Dale and George, 1954; Baker and Dale, 1961; Myers and Dale, 1967; Shafer and Baker, 1973; Baker, 1979; Preston, 1983 and McCoy, 1990. More recently, the TWDB completed several groundwater flow modeling studies and a brackish groundwater evaluation for the study area. The Brackish Resources Aquifer Characterization System (BRACS) study (Meyer and others, 2014) was performed to provide information on the nature and extent of brackish groundwater in the Gulf Coast Aquifer. The findings of the BRACS study will be critical for making decisions about future groundwater development for the Gulf Coast Aquifer in Cameron, Hidalgo, Willacy, and Starr Counties. The BRACS study used the Gulf Coast Aquifer hydrostratigraphy as determined in Young and others (2014), who conducted a comprehensive evaluation of the hydrostratigraphy and hydrochemistry of the Gulf Coast Aquifer across its entire extent within Texas.

A GAM was developed by the TWDB for the southern portion of the Gulf Coast Aquifer in 2003 (Chowdhury and Mace, 2003 and 2007). Because of the limited extent of the GAM, and the need to make simulations for joint groundwater planning for GMA 16 as a whole, the TWDB developed a subsequent model in 2011 to support joint groundwater planning in GMA-16 (Hutchison and others, 2011). The purpose of the GMA-16 model was to assist with development of DFCs for GMA-16, and it has some features, such as extension of the model into Mexico south of the Rio Grande, that provide useful information and guidance for the current project.

Other groundwater flow models that have been completed in the area include Carr and others (1985), Groschen (1985), Hay (1999), and Harden and Associates (2002). While some insight into modeling the Gulf Coast Aquifer in the study area may be gained from these studies, none of these other non-TWDB modeling studies cover the study area in its entirety.

5.1.6 Determination of Potential Well Field Locations

Both groundwater quality and aquifer characteristics should be considered when identifying potential well field sites. As a first screening step for this conceptual study, the most suitable well field locations were identified based on existing information provided in the recent BRACS study (Meyer and others, 2014). The BRACS study identified 21 geographic areas (denoted A through U) that have a unique salinity zone profile from ground surface to the base of the Gulf Coast Aquifer (Figure 5-9). The salinity zones were grouped into zones of slightly saline groundwater (1,000 to 3,000 milligrams per liter [mg/L] total dissolved solids [TDS]), moderately saline groundwater (3,000 to 10,000 mg/L TDS), very saline groundwater (10,000 to 35,000 mg/L TDS), and brine (greater than 35,000 mg/L TDS). For comparison purposes, the salinity of typical seawater is approximately 35,000 mg/L TDS. Detailed descriptions of each salinity zone are provided in Meyer and others (2014).



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Water Quality Zones and Thickness of Slightly Saline Groundwater

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Figure 5-9

In order to identify the most suitable sites for the development of well fields that could extract large quantities of water, the following constraints and considerations were evaluated:

- Regions with significant thickness of slightly saline (less than 3000 mg/L TDS) groundwater would be prioritized in order to reduce long-term treatment costs, increase the likelihood that concentrate can be released to surface water for disposal, and to increase the likelihood of reasonable well field productivity. Other engineering constraints, such as the distance from population centers, were not a primary consideration.
- Some water quality zones identified in the BRACS report have slightly saline groundwater at depth overlain by poorer quality water. These zones were avoided because greater expense would be incurred to drill to the slightly saline groundwater zone at depth, and the poorer quality water that occurs above the slightly saline water will migrate downward with pumping and potentially degrade the quality of the extracted water over time.
- The well field should not be adjacent to the boundaries of the water quality zones provided in the BRACS report due to the likelihood that the well field may draw water from an adjacent area with uncertain water quality.
- Regions within the jurisdiction of a GCD should be avoided due to potential permitting constraints on the ability to develop large quantities of groundwater. In the study area this constraint applies primarily to the Red Sands GCD in northern Hidalgo County.
- Highly urbanized areas should be avoided due to the complexity of building infrastructure and the heightened potential for groundwater contamination from, for example, gas stations, dry cleaners and light industry.

Two general zones were identified for well field development based on the above constraints and some general, assumed well locations were identified (Figure 5-9). Table 5-4 provides a brief discussion for each of the 21 water quality zones identified in the BRACS study and documents why each of the zones were either considered for well field development or excluded from further consideration. As illustrated in Figure 5-9, there is an eastern well field location that straddles the Hidalgo-Cameron County line, and a western well field location west of McAllen in Hidalgo County. The assumed number of wells was estimated based on the projected well field capacity as explained in the next section.

Table 5-4 Salinity Zones Evaluation

ZONE	NOTES
A	**Exclude.** Does contain slightly saline water (moderately saline at the ground surface followed by very saline and brine zones).
B	Good zone. Spatially extensive, starts with slightly saline water at the ground surface. Thickness of the slightly saline zone ranges between 200 to 1,000 feet. Next zone down is moderately saline. The best zone to target.
C	**Exclude.** Spatially, it is a very small zone. It is hard to target given the uncertainty in zone boundaries.
D	**Exclude.** It does not have a slightly saline zone (moderately saline at the ground surface followed by very saline and brine zones).

ZONE	NOTES
E	Slightly saline zone of approximately 200 feet followed by a moderately saline zone that varies in thickness between 300 to 600 feet which is followed by another slightly saline zone of thickness of 200 to 1,200 feet.
F	**Exclude.** Mostly in Starr County, and small.
G	**Exclude.** Spatially, it is a very small zone. Hard to target given the uncertainty in zone boundaries.
H	**Exclude.** Moderately saline zone at the ground surface with a thickness of approximately 800 feet. It is followed by a slightly saline zone with a thickness of only 400 feet which is followed by another moderately saline zone of thickness of 600 feet. Deep wells would be needed, and sandwiched between two zones of higher salinity.
I	**Exclude.** Spatially, it is a very small zone. Hard to target given the uncertainty in zone boundaries.
J	**Exclude.** Does not contain a slightly saline zone along its profile.
K	**Exclude**. Moderately saline zone at the ground surface with a thickness of only 100 to 200 feet. It is followed by a slightly saline zone with a thickness of approximately 400 feet which is followed by another moderately saline zone of thickness of 800 feet. Sandwiched between two zones of higher salinity. Also, target zone of slightly saline water not that thick.
L	**Exclude.** Spatially, it is a very small zone. Hard to target given the uncertainty in zone boundaries.
M	**Exclude**. Very Saline at the ground surface with a thickness of 200 to 300 feet followed by a moderately saline zone of 400 feet thick followed by a slightly saline zone of 800 feet followed by a moderately saline zone of thickness of 1,200 feet. Deep wells would be needed, and sandwiched between two zones of higher salinity.
N	**Exclude**. Moderately Saline at the ground surface with a thickness of 600 feet followed by a slightly saline zone of another 600 feet thick followed by a moderately saline zone of thickness of 1,200 feet. Deep wells would be needed, and sandwiched between two zones of higher salinity.
O	**Exclude**. Very Saline at the ground surface with a thickness of 400 to 500 feet followed by a moderately saline zone of 200 to 300 feet thick followed by a slightly saline zone of 200 to 600 feet followed by a moderately saline zone of thickness of 1,000 to 1,400 feet. Deep wells would be needed, and sandwiched between two zones of higher salinity.
P	**Exclude.** No slightly saline zone along its profile.
Q	**Exclude.** Spatially, it is a very small zone. Hard to target given the uncertainty in zone boundaries.
R	**Exclude**. Very Saline at the ground surface with a thickness of 200 to 300 feet followed by a moderately saline zone of 300 to 400 feet thick followed by a slightly saline zone of 200 to 600 feet followed by a moderately saline zone of thickness of 600 to 1,000 feet. Limited slightly saline zone, limited zone size, deep wells would be needed, and sandwiched between two zones of higher salinity.
S	**Exclude.** Does not contain a slightly saline zone along its profile.
T	**Exclude.** Spatially, it is a very small zone. Hard to target given the uncertainty in zone boundaries. Has no slightly saline water.
U	**Exclude.** Very saline at the ground surface followed by brine.

Although potential brackish aquifer well field locations are identified in this study based on existing information, final suitable locations would have to be verified through field investigation. Even the BRACS study, although the most detailed study of water quality that encompasses the study area to date, is fairly regional in nature, and likely has not identified local hydrogeologic conditions and water quality zones that may be of importance at the scale of a municipal well field.

5.1.7 Groundwater Availability Modeling

Two groundwater availability models, the original southern Gulf Coast GAM (Chowdhury and Mace, 2003 and 2007) and the GMA-16 model (Hutchison et al., 2011), cover the study area. Both models were examined to determine which one would be most appropriate to use to simulate the hydrologic effects of the proposed well fields. Development of a new model or implementation of significant modifications to an existing model were not part of this project.

The GAM model (developed in 2003 and 2007) has four layers. The northern GAM boundary lies in the middle of Kenedy and Brooks Counties and is simulated as a no-flow boundary in all model layers. The southern model boundary is coincident with the international boundary along the Rio Grande. At that southern boundary, model layer 1 was simulated using MODFLOW (Harbaugh, 1988) River Package to represent the Rio Grande. Other model layers along the southern model boundary are simulated as no-flow, thereby eliminating the potential for groundwater flow at depth between the US and Mexico. Because the southern boundary is close to the identified well field locations, the no-flow boundary conditions are serious limitations for accurate predictive simulations, and would lead to substantially greater simulated drawdown than would likely occur. Other model considerations, such as an analysis of aquifer hydraulic conductivity, were inconclusive regarding which model may be more appropriate for conducting predictive simulations.

The GMA-16 model (developed in 2011) extends well into Mexico to the south, significantly reducing concerns about boundary conditions for the identified well field locations. The GMA-16 model also extends farther to the north than the GAM to cover several counties north of the proposed well fields. As such, the GMA-16 model was used to conduct the predictive simulations to evaluate the effects of brackish groundwater development at the identified well field sites. Although some limitations of note were also found in the GMA-16 model, they were judged to be of less importance than those of the GAM. GMA-16 model limitations are discussed at the end of this section.

The GMA-16 model consists of 6 layers. Layers 1 through 4 represent the Gulf Coast Aquifer System, comprised of the Chicot Aquifer, Evangeline Aquifer, Burkeville Confining System, and Jasper Aquifer. Layer 5 is an aggregate representation of the Yegua-Jackson Aquifer System including parts of the Catahoula Formation, and layer 6 is an aggregate representation of the Queen-City, Sparta, and Carrizo-Wilcox Aquifer System. Layer 5 is only active in a small portion southwest of the study area and layer 6 is not active in the study area. In the horizontal dimensions, the model grid consists of 284 rows and 201 columns with each grid cell being one mile by one mile. The model starts with a steady-state stress period, followed by 37 annual stress periods representing historical conditions from 1963 through 1999.

In order to better simulate the effects of the groundwater pumping at the two proposed well fields, the GMA-16 model grid was refined at the two proposed well field locations to 0.25 mile by 0.25 mile. Just outside the well fields, the grid was refined to 0.5 mile by 0.5 mile, and away from the well fields, the grid remained at its original size of one mile by one mile. The new model grid has 439 rows and 386 columns. The modified model was run for the same historical period as the original model (i.e., from 1963 through 1999) and the results were compared to those obtained from the unmodified model. Only very minor differences were observed between the two runs, indicating that the hydraulic information and other model inputs were correctly translated between the unmodified and updated models.

5.1.7.1 Predictive Simulations

The GMA-16 model with the refined grid was used to simulate predictive scenarios using 66 annual stress periods that simulate conditions from 2000 through 2065. The final simulated water levels as of the end of 1999 from the historical period simulation were used as initial hydraulic heads for each predictive model run. For the predictive simulation period, groundwater recharge was assumed to equal the steady-state recharge simulated in the historical run, thereby assuming long-term average recharge conditions.

For pumpage, the 1999 pumping was assumed to continue through 2065 with adding only the pumping from the existing desalination facilities. Within the study area, there are seven desalination plants as outlined in Section 5.1.3. Pumping from these facilities was assumed to start in the year the facility was built and continue through the end of the predictive simulation in 2065. The groundwater pumping amount and starting year for each facility is listed in Table 5-1.

The historical simulation assumed no temporal changes to model boundary conditions other than groundwater pumping and recharge, and these boundaries were also unchanged in the predictive simulations. These other model boundaries include river boundaries that simulate the effects of groundwater flow to or from surface water features, drain boundaries which simulate the discharge of groundwater at wetlands and springs, and general head boundaries that simulate groundwater discharge into or from the Gulf of Mexico, and also simulates lateral flow along the edges of the model.

A predictive base-case simulation was run using the approach described above without any additional pumping from the proposed brackish groundwater well fields. This simulation illustrates the effect of continuing estimated current pumping into the future. Two additional predictive scenarios were simulated with additional pumping in each scenario to what is described above to represent three possible well field operations. In each of these scenarios, pumping was initiated in 2016 and continued through the end of the simulation in 2065, for a predictive simulation period of 50 years. These two scenarios are as follows:

Scenario 1: Pumping from the eastern well field (Figure 5-9) of 50,000 acre-feet/year from the Chicot Aquifer (model layer 1). In this scenario 12,000 acre-feet/year of pumping is from Hidalgo County, and 38,000 acre-feet/year of pumping is from Cameron County.

Scenario 2: Pumping of 12,000 acre-feet/year from the western well field (Figure 5-9) Evangeline Aquifer (model layer 2). The entire pumping amount is from Hidalgo County.

The pumping volumes for each scenario were selected to maximize the amount of water produced based on the MAG for each county (Section 5.1.4.2.2) and the estimated current volume of pumping in the model. Since the current volume of pumping in the model (including the added existing desalination facilities) is approximately 12,000 acre-feet/year for Cameron County, and the MAG is 50,560 acre-feet/year, that leaves about 38,000 acre-feet/year for future groundwater development. Likewise in Hidalgo County since the current volume of pumping in the model including the added existing desalination facilities is approximately 30,000 acre-feet/year, the MAG of about 42,000 acre-feet/year leaves about 12,000 acre-feet/tear for future groundwater development. The predictive simulation results are presented in the following section.

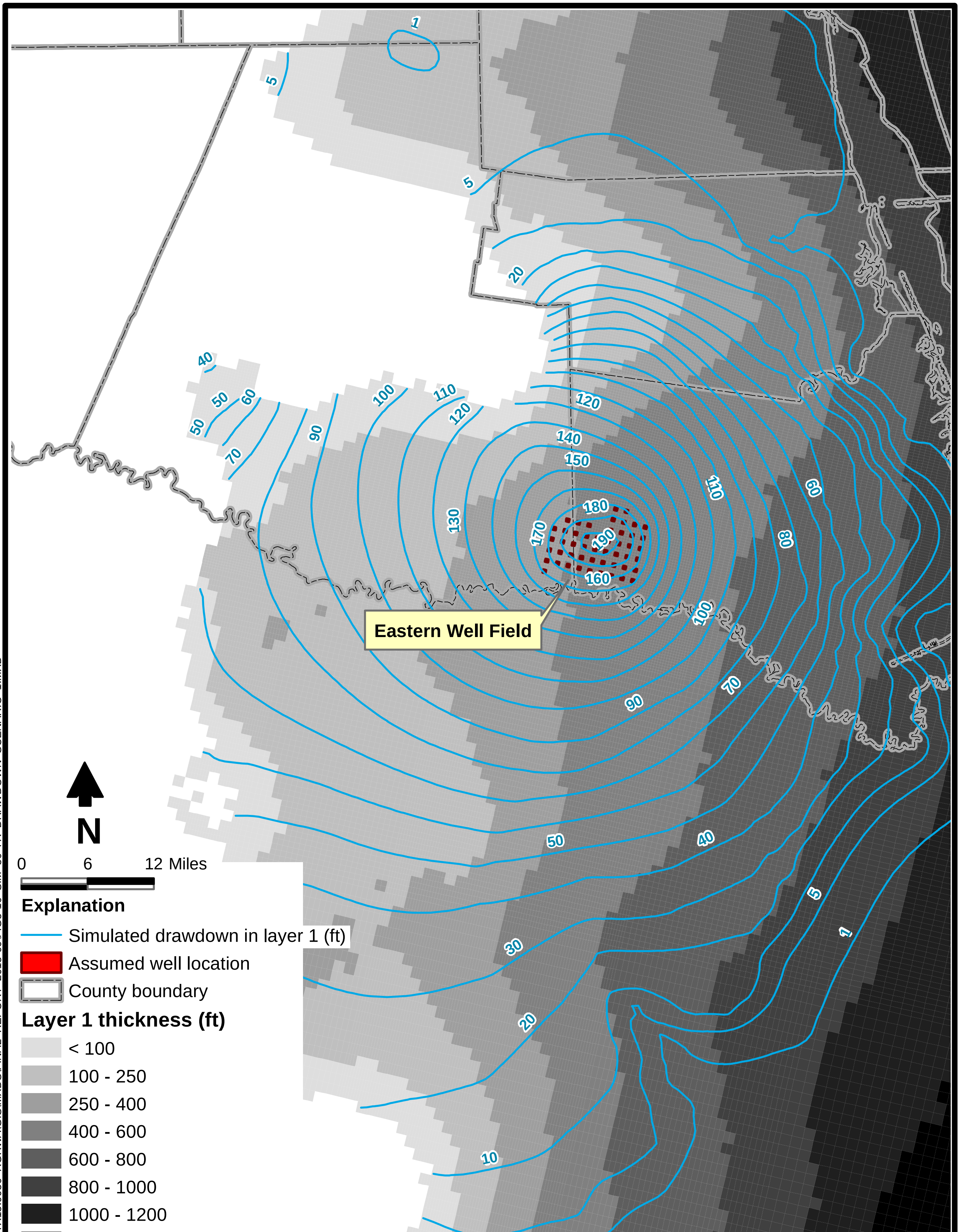
5.1.7.2 Predictive Simulation Results

For Scenario 1, the maximum simulated drawdown in the Chicot Aquifer at the center of the well field after 50 years of operation at 50,000 acre-feet/year is approximately 195 feet (Figure 5-10). At the location of the maximum simulated drawdown, this represents about 50% of the total saturated thickness in the aquifer. As indicated in the figure, the aquifer thickness is about 350 to 500 feet in the vicinity of this well field. The aquifer hydraulic conductivity in the model is 32 feet per day (ft/day) for most of the well field area, but a small portion of the well field area has a hydraulic conductivity of 68 ft/day. The storage coefficient is about 0.004 on average.

Reverse particle tracking was used to estimate the contributing zone to the well field over the same 50-year period (Figure 5-11). As indicated by the particle tracks in the figure, water pumped 50 years in the future will have travelled approximately 2 to 3 miles from areas adjacent to each production well. In addition, some wells will draw water from south of the Rio Grande. The particle tracking was completed using an effective porosity of 10%, which is a reasonable value to expect for the Chicot Aquifer. Comparison of Figure 5-11 with Figure 5-9 illustrates that the 50-year particle tracks do not cross into adjacent water quality zones, indicating that long-term water quality extracted from the well field may be relatively consistent. In addition, the upward flow of groundwater from the adjacent (deeper) model layer in the vicinity of the well field is less than 1% of the well field extraction rate, indicating that significant upward migration of poor quality water is not predicted in the simulation.

For Scenario 2, the maximum simulated drawdown in the Evangeline Aquifer at the center of the well field after 50 years of operation at 12,000 acre-feet/year is approximately 260 feet (Figure 5-12). At the location of the maximum simulated drawdown, this represents about 25% of the total saturated thickness in the aquifer. As indicated in the figure, the aquifer thickness is about 1,000 to 1,200 feet in the vicinity of this well field. The aquifer hydraulic conductivity in the model is 0.65 ft/day for most of the well field area, but a small portion of the well field area has a hydraulic conductivity of about 4 ft/day. The storage coefficient ranges from about 0.0002 to 0.04.

Reverse particle tracking was used to estimate the contributing zone to the well field over the same 50-year period (Figure 5-13). As indicated by the particle tracks in the figure, water pumped 50 years in the future will have travelled approximately 1 mile or less from areas adjacent to each production well.



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Simulated 50-Year Drawdown for Scenario 1

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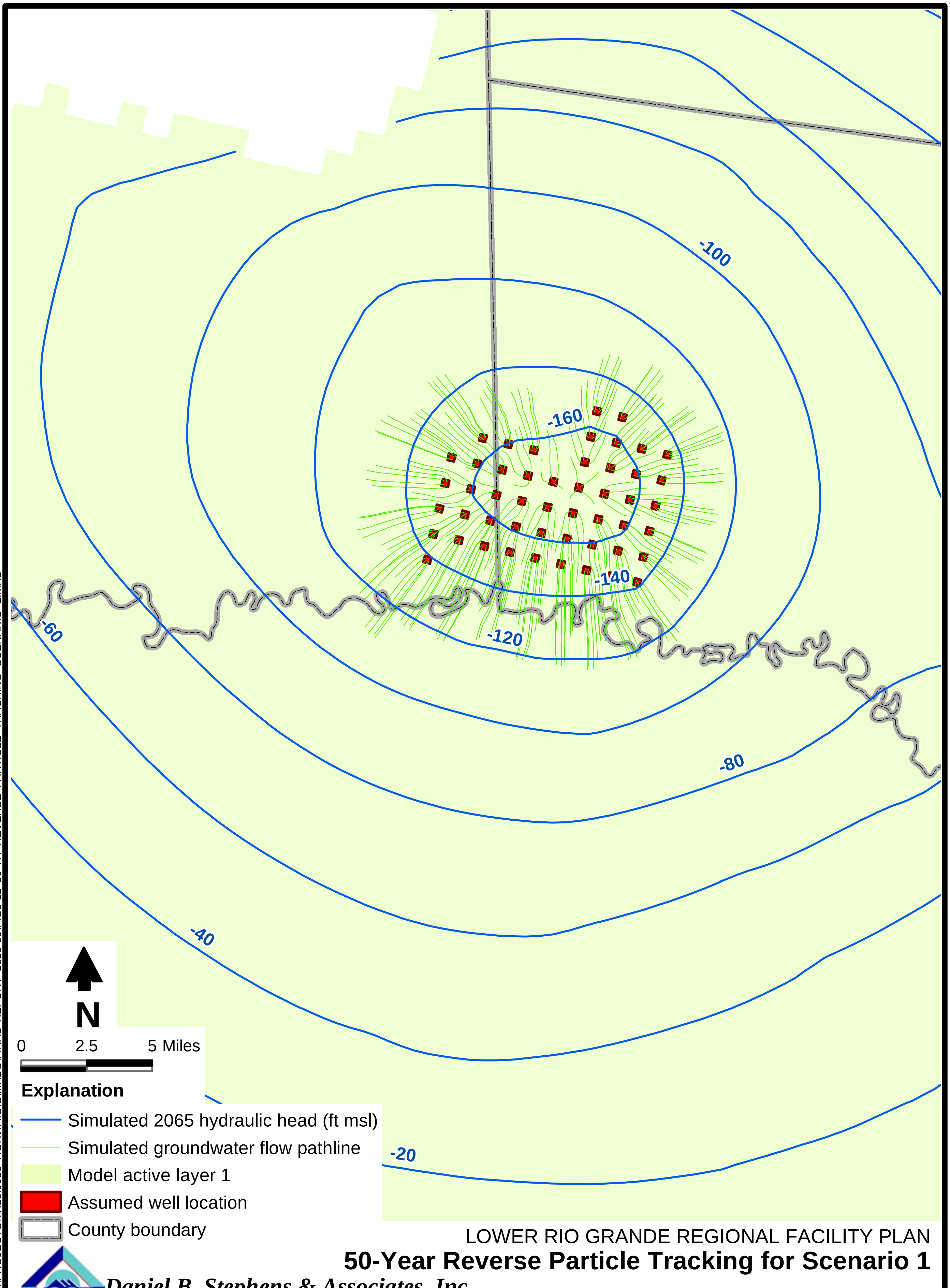
Figure 5-10



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50-Year Reverse Particle Tracking for Scenario 1

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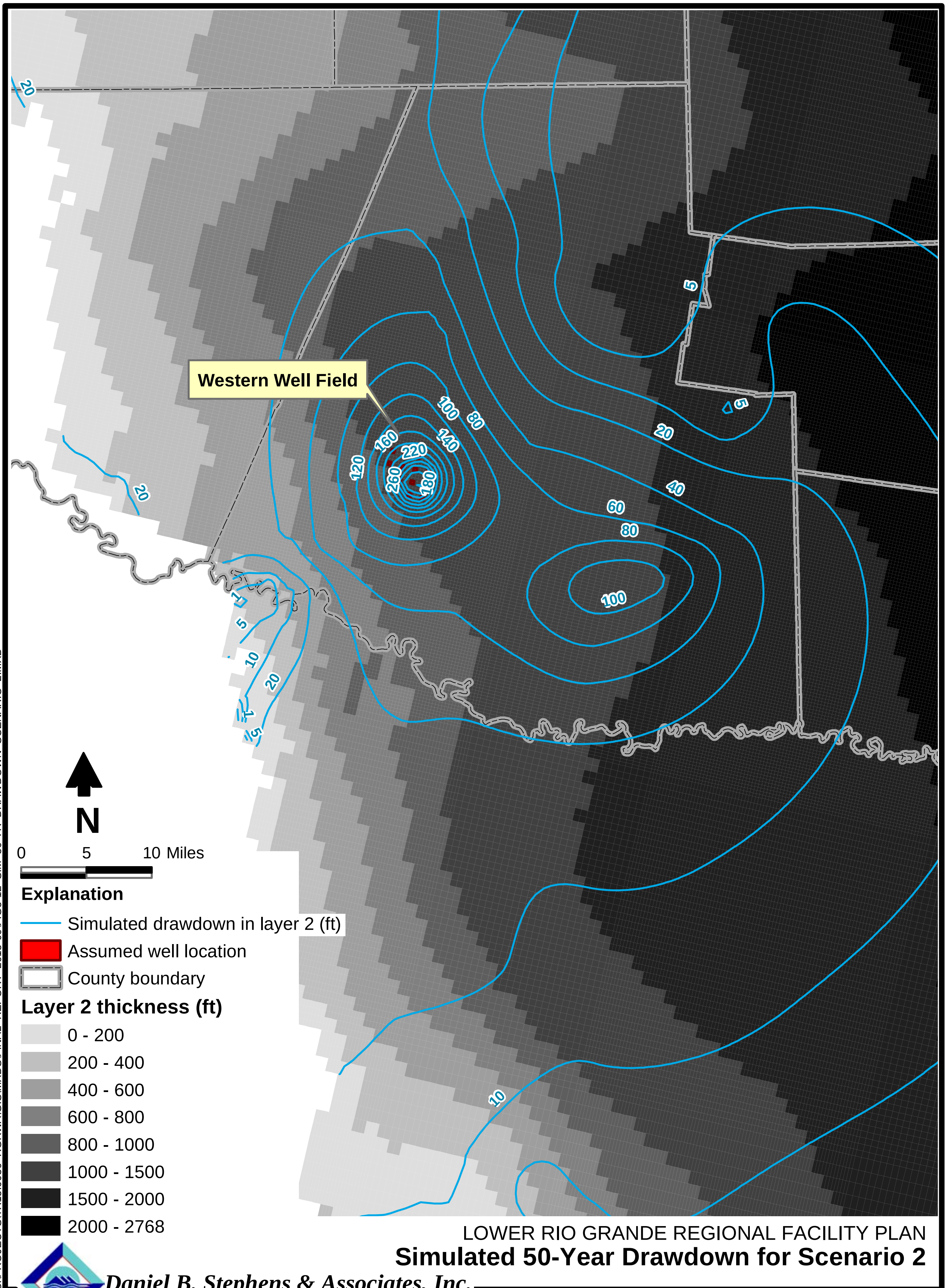
Figure 5-11



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LOWER RIO GRANDE REGIONAL FACILITY PLAN

Simulated 50-Year Drawdown for Scenario 2

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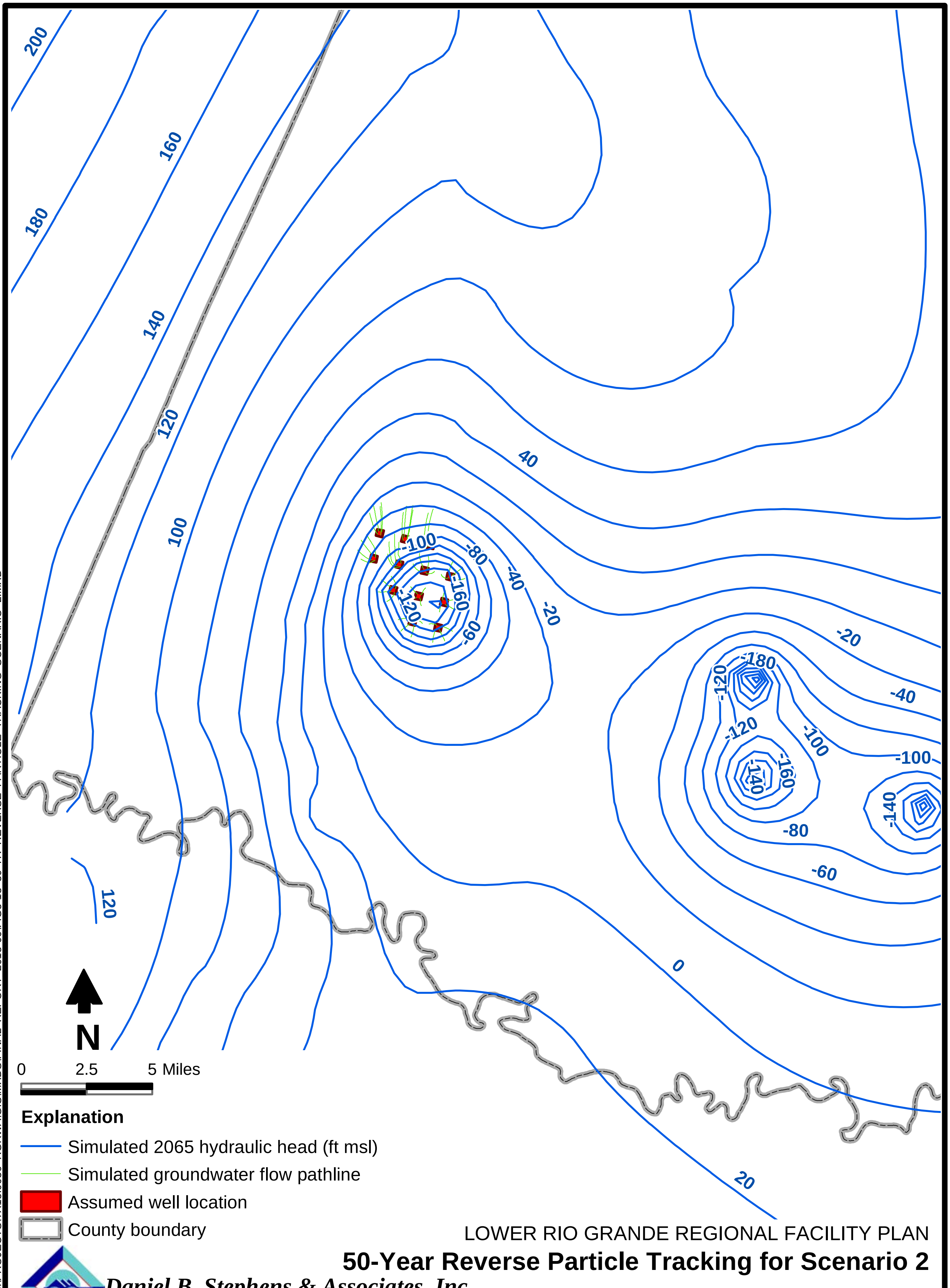
JN TX16.0036.15

Figure 5-12



LOWER RIO GRANDE REGIONAL FACILITY PLAN

50-Year Reverse Particle Tracking for Scenario 2

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Figure 5-13

This distance is less than that simulated for Scenario 1 because the western well field is located in a zone of lower aquifer hydraulic conductivity in the model. The particle tracking for Scenario 2 was also completed using an effective porosity of 10%, which is a reasonable value to expect for the Evangeline Aquifer. Comparison of Figure 5-12 with Figure 5-9 illustrates that the 50-year particle tracks do not cross into adjacent water quality zones, indicating that long-term water quality extracted from the well field may be relatively consistent. The upward flow of groundwater from the adjacent (deeper) model layer in the vicinity of the well field is small at about 3% of the well field extraction rate, indicating that significant upward migration of poor quality water is not predicted in the simulation. However, the western well field is implemented in model layer 2, which is about 1,000 feet thick and includes two water quality zones (a zone of higher salinity at depth below a shallower zone of lower salinity) identified by Meyer and others (2014). Because this model layer includes two water quality zones, the vertical migration of poor quality water from depth is more likely to occur at this well field location than it is at the eastern well field location, even if not simulated as such in the model.

The simulated mass balance of the aquifer system as of the end of the 50-year simulation period is provided for each scenario in Table 5-5. As indicated in the table, most of the groundwater extracted at each well field (75 to 82%) is from groundwater storage (water level decline in the aquifer), with only 15 to 20% from surface water features (i.e., Rio Grande and other streams simulated in the model).

Table 5-5 Simulated Source of Pumped Water for Each Predictive Scenario at 50 Years

SOURCE OF PUMPED WATER	SCENARIO 1 (AC-FT/Y)	SCENARIO 2 (AC-FT/Y)
Depletion from storage (decline in water levels)	41,250	9,020
Depletion from rivers and streams (primarily the Rio Grande)	7,540	2,255
Gulf of Mexico (seawater intrusion)	910	15
Underflow from Mexican boundary into the deep Yegua-Jackson Aquifer System	160	705
Underflow from Southern model Boundary (located south of the Rio Grande in Mexico)	135	3
Reduction in spring flow	0	3
Total	**49,995**	**12,001**

5.1.7.3 Model Limitations

Although the GMA-16 model is a better tool than the GAM to use for this study, it too has some notable limitations that will affect simulated drawdown. One of these limitations is that the model uses confined conditions for all model layers. With the confined layering configuration, the transmissivity (hydraulic conductivity of the aquifer times the aquifer thickness) of each model cell is calculated once at the beginning of the simulation, and is not subsequently updated to account for changing aquifer thickness as water levels decline. This approach can be appropriate for unconfined layers if water level does not change significantly during the simulation period, as was the case in the original GMA-16 model. However, for the predictive simulations that consider significant new pumping centers, the saturated thickness in some model cells that represent unconfined portions of the aquifer may be reduced by up to 50% or so of the initial value, yet the model assumes that the transmissivity of these cells remains unchanged. The overestimation of the calculated transmissivity in the model (i.e. neglecting that the aquifer thickness will change through time in some areas) can lead to the underestimation of future drawdown.

Although the model assumes a confined aquifer configurations in all layers, model cells located in outcrop areas, where aquifer conditions are unconfined were assigned specific yield values for the storage coefficient. However, specific yield values used in the model for the zone 1 (where well field of scenario 1 is simulated) is between 0.0039 and 0.0053, whereas a more typical specific yield for the sediments that comprise the Chicot Aquifer would be about 0.05 to 0.2. This limitation is more pronounced for model hydraulic property Zone 2 because the majority of this zone is assigned a confined storage coefficient of 0.0002. That portion of the Evangeline Aquifer should be unconfined as the Chicot Aquifer above it is dry or would be dewatered soon after wellfield pumping commenced. A higher storage coefficient would likely be more appropriate. The application of the low storage coefficients in the GMA-16 model likely leads to greater simulated drawdown than would occur in reality.

To some extent these two model limitations offset each other, and the net effect of the overestimation (or underestimation) of the simulated drawdown is unknown.

5.1.8 Conclusion

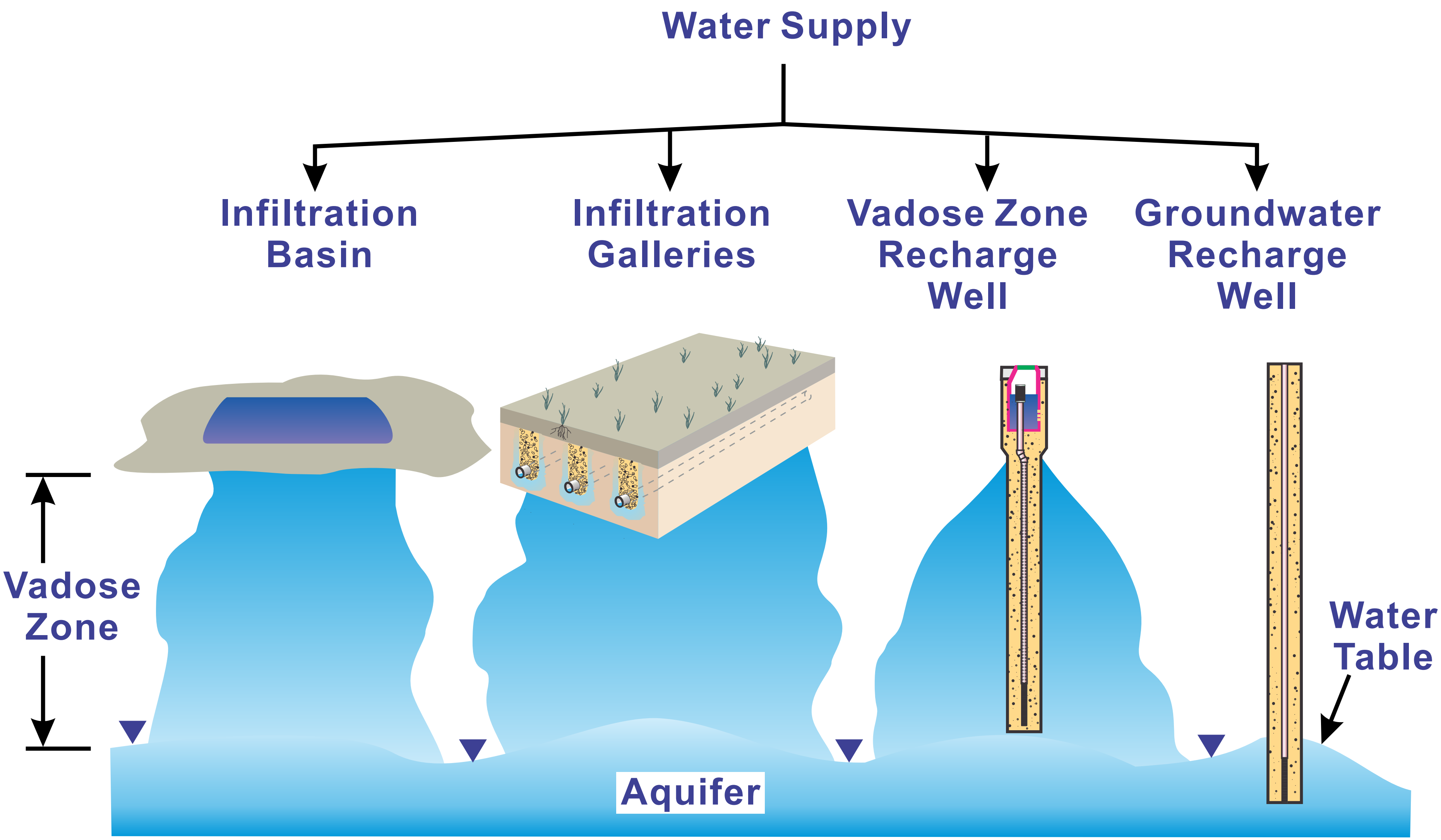
Based on the hydrologic simulations both proposed wellfields can reliably supply the anticipated brackish groundwater over a 50-year period. The required well spacing is expected to be a 1-mile radius. Well pump capacities of 900-1,000 gpm are anticipated. Well depths in the western well field are anticipated to be 350-500 feet below ground surface. Eastern well field well depths are expected to be approximately 500-600 feet.

5.2 GROUNDWATER RECHARGE

5.2.1 Introduction to Aquifer Recharge Methods

Potential aquifer recharge methods range from the use of surface infiltration basins to direct injection into the aquifer (Figure 5-14). The level of treatment and operational complexity tends to increase with proximity of the point of recharge water delivery; that is, methods that introduce water to the subsurface above the aquifer require a lesser quality source than those that inject water directly into the aquifer.

S:\PROJECTS\TX15.0036_RGRWA\VR_DRAWINGS\FIG4.3.1_AQUIFER_RECHARGE_METHODS.CDR



Not to Scale

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RIO GRANDE REGIONAL WATER AUTHORITY
Aquifer Recharge Methods

Figure 5-14

5.2.2 Recharge and ASR Wells

Recharge wells are wells constructed for the purpose of recharging water into the aquifer system. If the well is also designed for groundwater extraction, it is called a recharge and recovery well or more commonly an aquifer storage and recovery (ASR) well. Recharge and ASR wells are designed and constructed in a similar fashion to a conventional water well, with some important differences. For example, special flow-control valves are used to regulate the rate of recharge to avoid clogging of the aquifer through entrained air in the recharge water and other factors that can limit system efficiency. A common rule of thumb is that the expected rate of recharge through a well is about half of the potential pumping rate from a well, although this factor can vary widely. Periodic well maintenance is required to clean the well screen of chemical or biological precipitates that can reduce the well efficiency.

5.2.2.1 Vadose Zone Wells and Infiltration Galleries

Vadose zone wells are completed in the unsaturated zone above the water table and equipped for recharge operations only (water cannot be recovered from these wells). Because they are in the vadose zone they cannot be easily redeveloped to restore recharge rates subsequent to their inevitable clogging. Consequently, this type of recharge system tends to lose capacity over time, and wells would need to be periodically replaced. Due to the required replacement schedule and because the vadose zone is of limited thickness (several tens of feet at most) at the areas considered for recharge, vadose zone wells are not considered a useful option for large scale groundwater recharge in the study area. Infiltration galleries are similar to vadose zone wells in that they infiltrate water in the subsurface above the water table, but are shallower and cover a larger area. Infiltration galleries are also not considered due to the limited thickness of the vadose zone.

5.2.2.2 Infiltration Basins

Surface infiltration basins operate by spreading water on the basin floor for infiltration into the underlying soils and downward movement of the recharged water to the underlying aquifer. Local subsurface conditions are generally suitable for such systems in that the aquifer is unconfined and sufficiently transmissive to accommodate lateral movement of infiltrated water away from the recharge area without forming high groundwater mounds that can interfere with long-term infiltration. The feasibility of surface recharge methods is dependent on the extent and occurrence of low-permeability soils and caliche horizons above the water table that may limit the rate of infiltration beneath a basin. If low-permeability soils occur in the shallow subsurface, they can be removed during basin construction.

Periodic basin drying and scarifying is typically performed to maintain recharge capacity (AWAA RF, 2008). The primary advantage of infiltration basins are ease of construction and operation, and potentially the beneficial effect on water quality through soil-aquifer treatment. Disadvantages include land area requirements and to a lesser extent evaporative losses. The water source must be of adequate quality to limit clogging of the infiltrating surface that can occur through (1) deposition and accumulation of suspended solids (e.g., sediment, algae, and sludge) (2) formation of biofilms and biomass on and in the soil, (3) precipitation of calcium carbonate or other salts on and in the soil, and (4) formation of gases that stay entrapped in the soil (Bouwer, 2002). Clogging reduces the rate of infiltration and thus the amount of water that can be recharged to the underlying aquifer.

5.2.3 Rules and Regulations

In general, groundwater recharge projects are, or have the potential to be, regulated by the Texas Commission on Environmental Quality (TCEQ). Depending on the design of an infiltration basin, an Underground Injection Control (UIC) permit may or may not be required. In general, aquifer water quality must be protected in that the water that is infiltrated must be of equal or better quality than that in the receiving aquifer.

Recent legislation (House Bill 655) passed in June of 2015 amended the Water Code to set out provisions relating to the storage and recovery of water in aquifers using recharge wells. A previous requirement for pilot projects has been removed, and now a water right holder or an applicable water user can proceed with an aquifer storage and recovery project so long as they comply with the terms of the applicable water right and several other required authorizations. The bill also granted the TCEQ jurisdiction over the regulation and permitting of ASR wells, requires reporting of injection and recovery volumes and water quality data to TCEQ by the project operator, and allows the TCEQ to authorize a Class V injection well as an ASR well. There are also other requirements specific to ASR projects within GCD boundaries.

5.2.4 Water Recharge Analysis

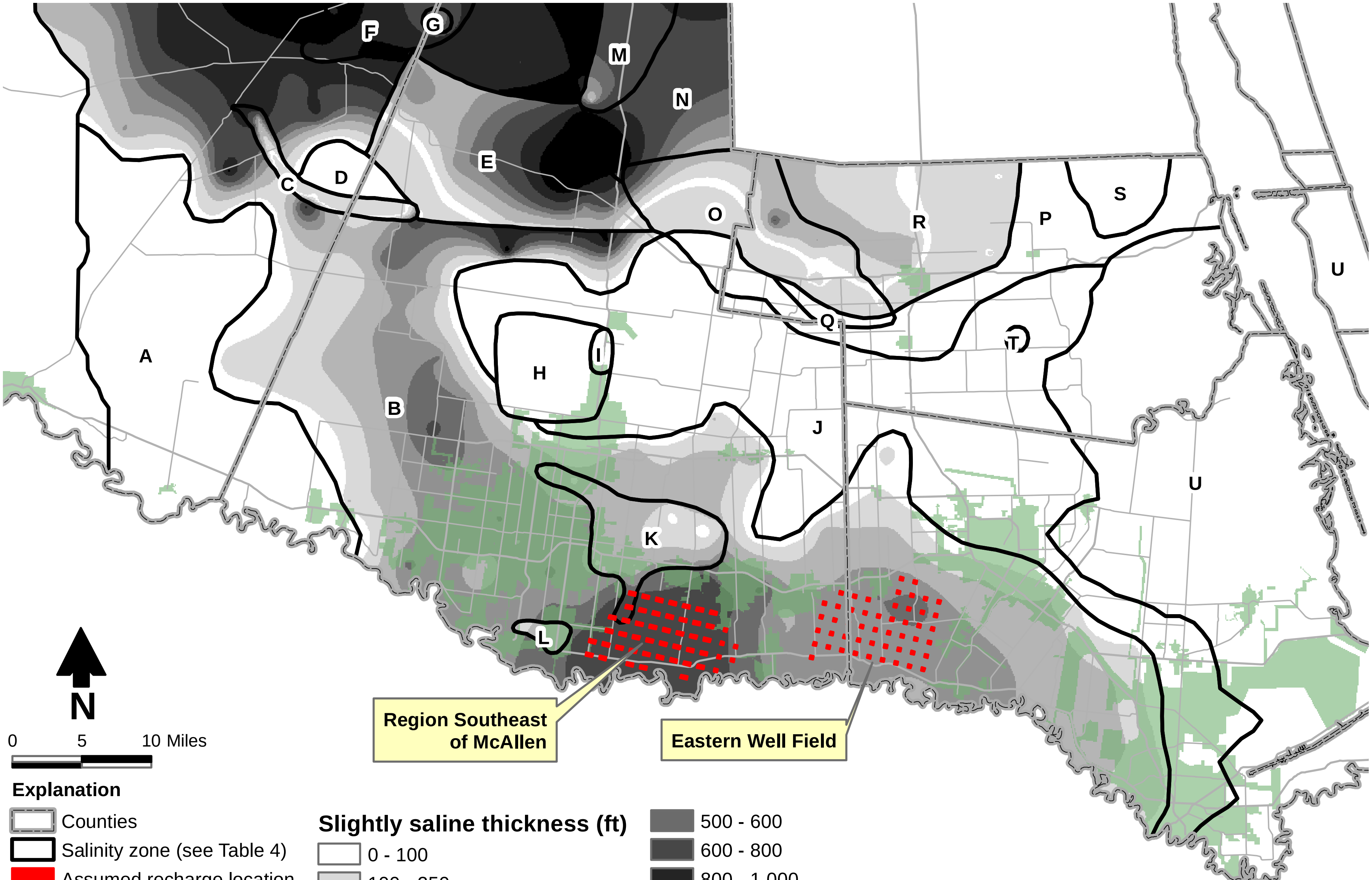
Aquifer recharge using assumed recharge well locations was evaluated for two areas illustrated in Figure 5-15. One location is the same as the eastern brackish aquifer well-field location, and the second location, southeast of McAllen, is a region where no brackish aquifer well field was proposed. For the first location, aquifer recharge could be conducted conjunctively with brackish aquifer well-field operations. At the second location southeast of McAllen, aquifer recharge and subsequent recovery could be conducted independent of a brackish water well field.

Both scenarios simulated recharge of 30 MGD (about 33,000 ac-ft/yr) of water for a five year period; the recharge rate was the same for each well location. Conceptually, the source of water would be excess surface water, collected from the Rio Grande.

Figures 5-16 and 5-17 illustrate the simulated water level rise in the Chicot Aquifer after 5 years of aquifer recharge (blue contour lines), as well as the simulated depth to water below ground surface (color shading). As indicated in the figures, the maximum simulated water level rise after 5 years of aquifer recharge is over 30 feet near the center of the eastern well field, and about 50 feet for the recharge site southeast of McAllen. Significant water level rises attributable to the assumed aquifer recharge extend approximately 10 to 15 miles from the center of each well field.

The simulated depth to water in each area ranges from zero (flooded cells colored purple in Figures 5-16 and 5-17) to greater than 30 feet (colored green in Figures 5-16 and 5-17). Where the depth to groundwater is zero, water is simulated to exit the model through the irrigation infrastructure, and is therefore lost to the aquifer system. Although it is difficult to determine the exact amount of recharge water simulated to exit the aquifer at surface water boundary conditions, the amount of simulated recharge water loss is approximately 20% of the recharged water. A portion of this simulated loss is due to how the irrigation infrastructure (canals, drains) is simulated in the model. The location and extent of these features, nor site-specific attributes such as elevation of the bottom of the canals, was not adjusted when the model grid-refinement was completed for the brackish well field development scenarios. As a consequence, these features are significantly wider in the simulation than they are in reality (see the surface water feature in model marked in Figures 5-16 and 5-17). Although it would be difficult to avoid all loss of recharged water to surface water if the assumed scenarios were implemented, more detailed simulations, combined with careful siting of recharge wells or other facilities so they are not close to surface water features, could lead to expected losses on the order of 5 to 10% of the recharge water rather than 20%. Furthermore, in the east well field no groundwater pumping was assumed. If the well field is operated and water levels decline, the loss of recharged water to surface water could be eliminated or reduced substantially.

S:\PROJECTS\TX15.0036_RGRWA\GIS\MXDS\TXMK_15_NEIL_B\FIG5-15_ASSUMED_RECHARGE_LOCS.MXD



Explanation

- Counties
- Salinity zone (see Table 4)
- Assumed recharge location
- Urban area

Slightly saline thickness (ft)

- 0 - 100
- 100 - 250
- 250 - 400
- 400 - 500
- 500 - 600
- 600 - 800
- 800 - 1,000
- > 1,000

Source: Meyer et al., 2014

LOWER RIO GRANDE REGIONAL FACILITY PLAN

Assumed Recharge Locations

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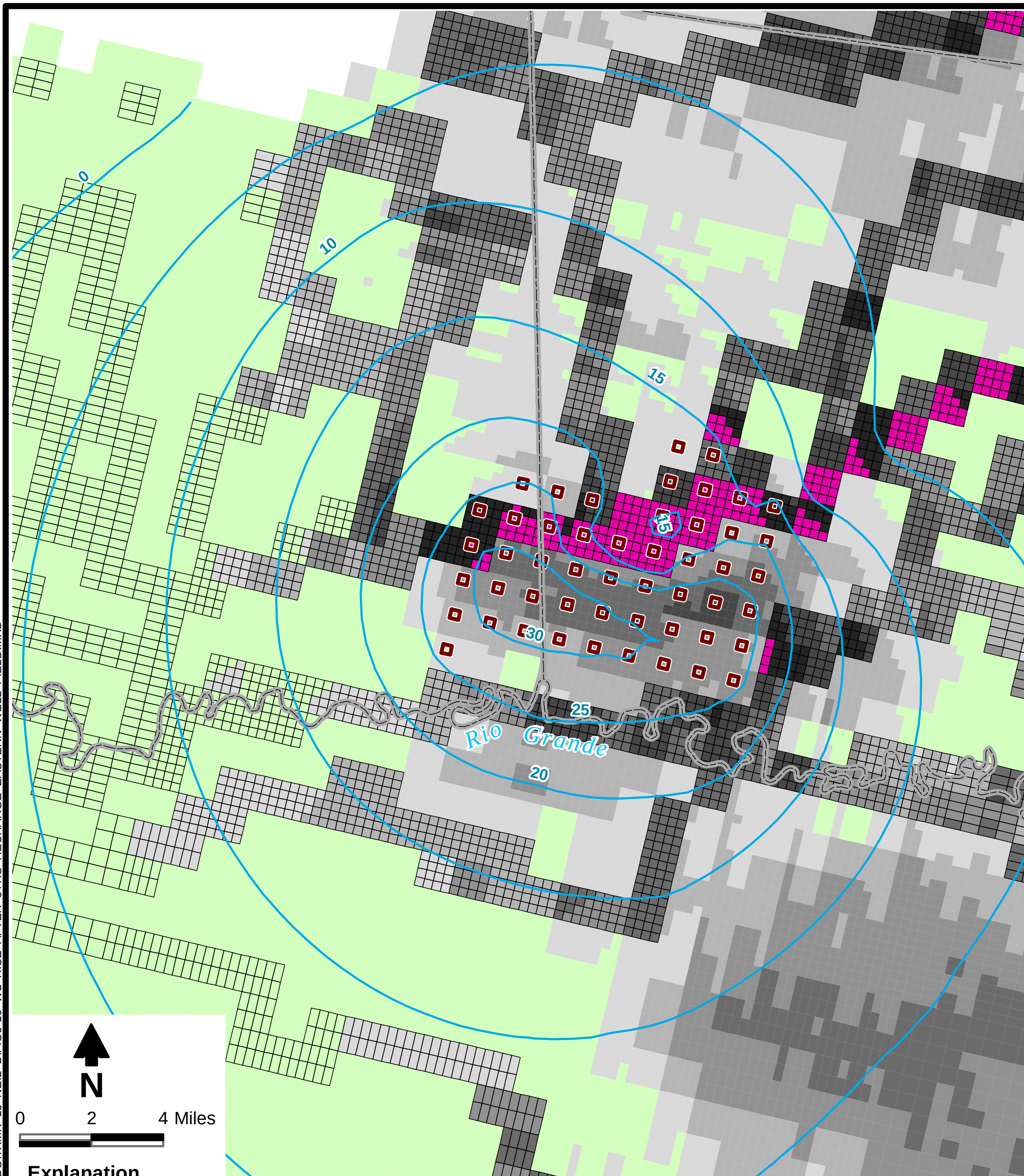
Figure 5-15



Explanation

	Water level rise at the end of Injection (ft)	**Depth to Water at the End of Injection (ft)**			15 - 20
	Counties		< 0 (Flooded cell)		20 - 25
	Assumed recharge location		0 - 5		25 - 30
	Surface water feature in model		5 - 10		> 30
			10 - 15		

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Water Level Rise After 5 Years of Recharge in Eastern Well Field

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Figure 5-16

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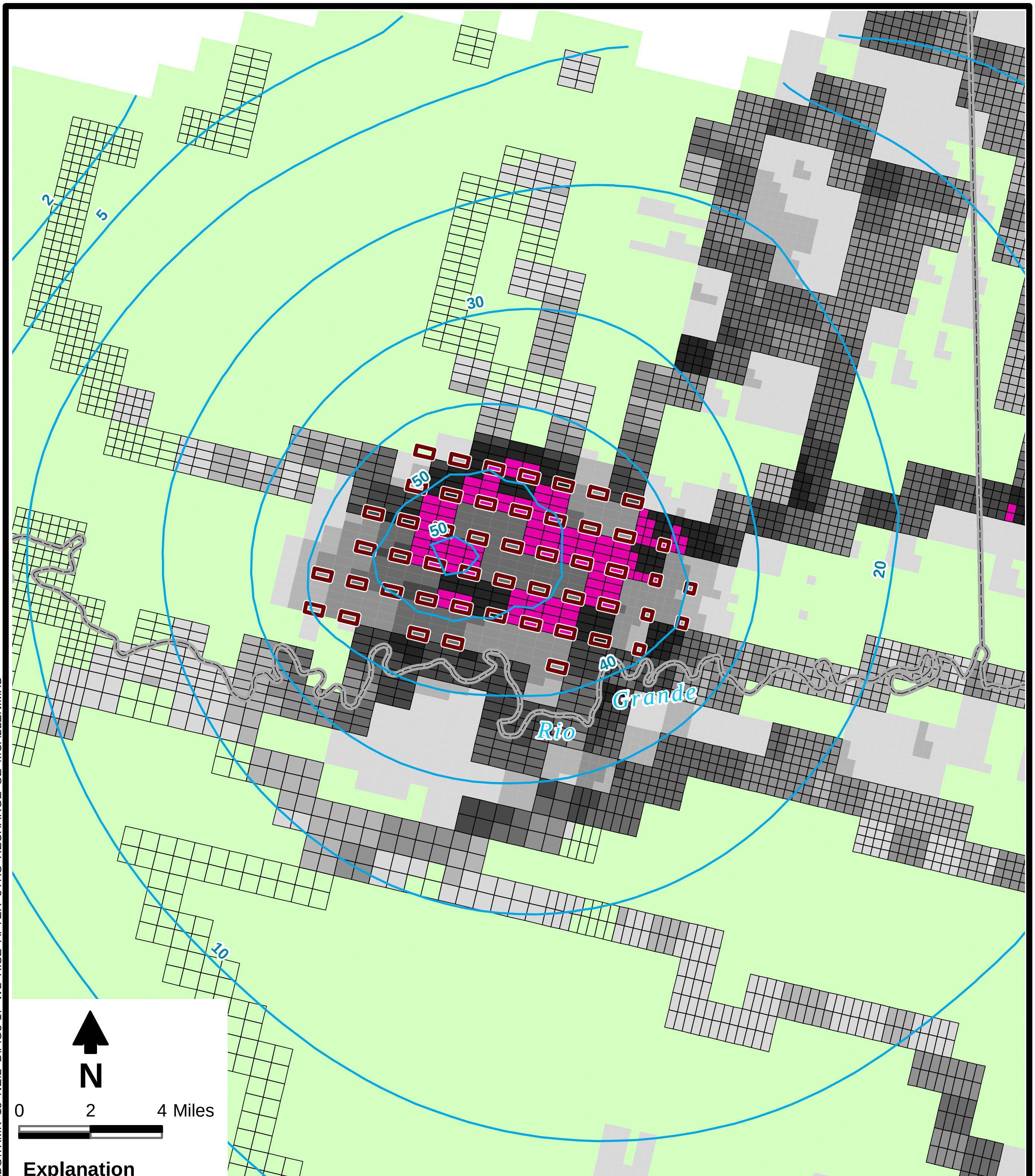

Explanation

Water level rise at the end of Injection (ft)
Counties
Assumed recharge location
Surface water feature in model

Depth to Water at the End of Injection (ft)
< 0 (Flooded cell)
0 - 5
5 - 10
10 - 15
15 - 20
20 - 25
25 - 30
> 30

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Water Level Rise After 5 Years of Recharge Southeast of McAllen

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Figure 5-17

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5.2.5 Potential Drift

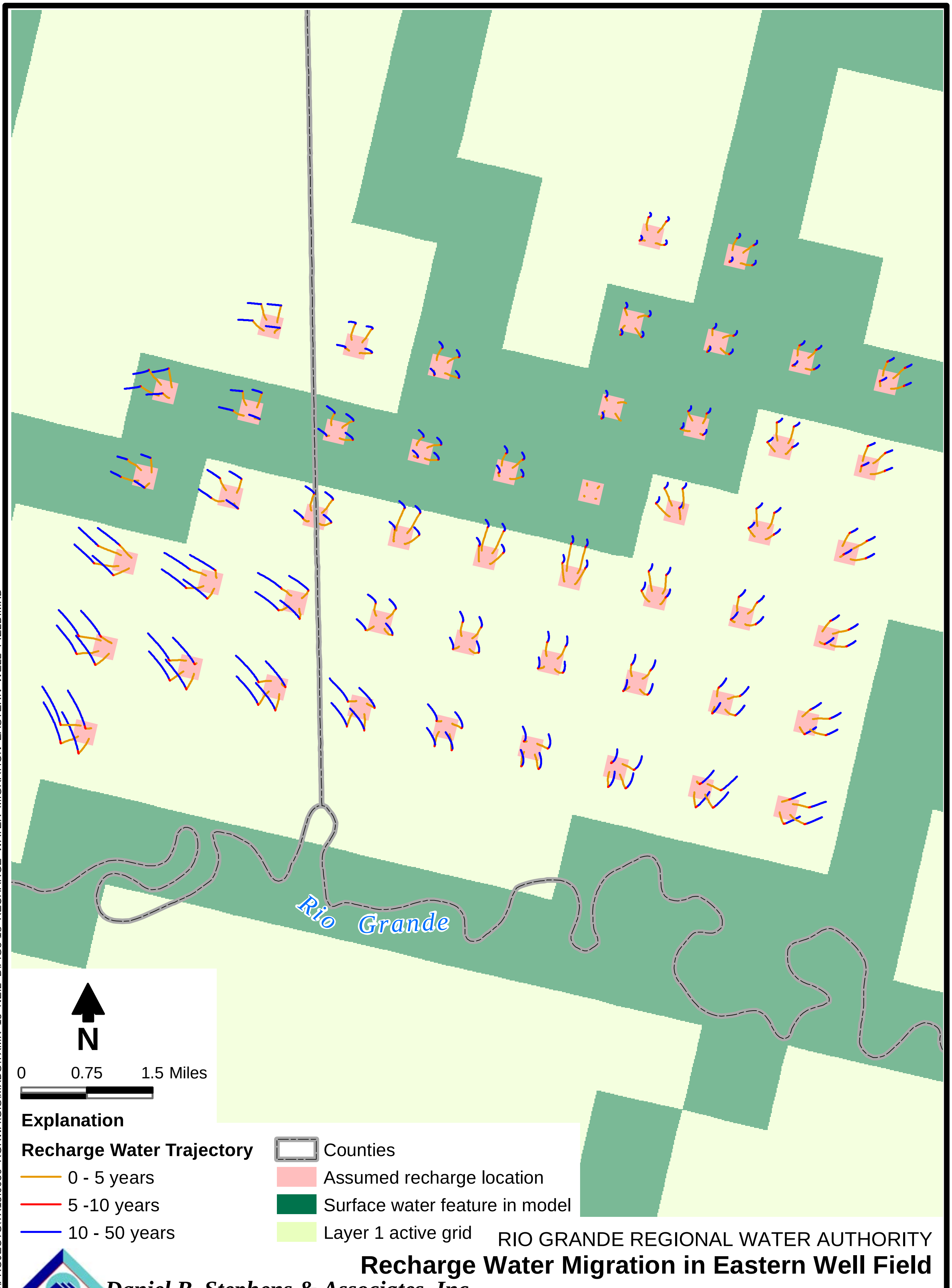
The term "drift" refers to the migration of recharge water once it reaches the aquifer storage zone, either through recharge wells, recharge basins, or other means. Consideration of drift is an important component of planning a recharge project because project operators do not wish to lose control of the water and thereby diminish recovery percentages. The amount of drift is primarily dependent on the volume of water recharged, aquifer hydraulic properties, and the time lag between aquifer recharge and recovery of the recharged water. Other factors, such as groundwater pumping from adjacent water users, can also be important.

The potential drift was simulated for each of the above scenarios using a forward particle tracking method. For each simulation, particles were released from the recharge well location at the beginning of the simulation, and the advective movement of each particle was simulated for a 50-year period, even though aquifer recharge was stopped in the simulation at 5 years. The same effective porosity of 10% (0.10) as was used for the analysis of brackish well field source water was also used in this analysis.

The simulation results for recharge in the eastern well field and recharge in the new location southeast of McAllen are provided in Figures 5-18 and 5-19, respectively. The groundwater flow pathlines (recharge water trajectories) are color-coded in these figures according to the travel time. Particles at some of the recharge locations have very small movement because they are intercepted by an overlying canal in the model for that particular cell, as described above. At most locations, the simulated movement of particles away from the recharge wells is relatively slow, indicating that recovery of recharged water should not be difficult so long as the recharged water is not lost to surface water.

5.2.6 Conclusion

Based on the hydrologic simulations the subsurface storage and recovery of 30 MGD over 5 years is a viable option to supply drinking water to the valley. Surface seepage of recharged water at surface drains or canals may occur, although through careful siting of recharge facilities this loss can be limited to perhaps 5-10% of the recharged water, or potentially eliminated entirely if the recharge facilities are operated conjunctively within a well field. The required well spacing is expected to be a 1-mile radius. Well recharge capacities of approximately 600 gpm are anticipated. Well depths in western recharge zone are 600-800 feet below ground surface. The eastern recharge well depths are 400-500 feet as simulated. Recharge basins, in lieu of or in addition to recharge wells, may be another viable recharge option.



RIO GRANDE REGIONAL WATER AUTHORITY

Recharge Water Migration in Eastern Well Field

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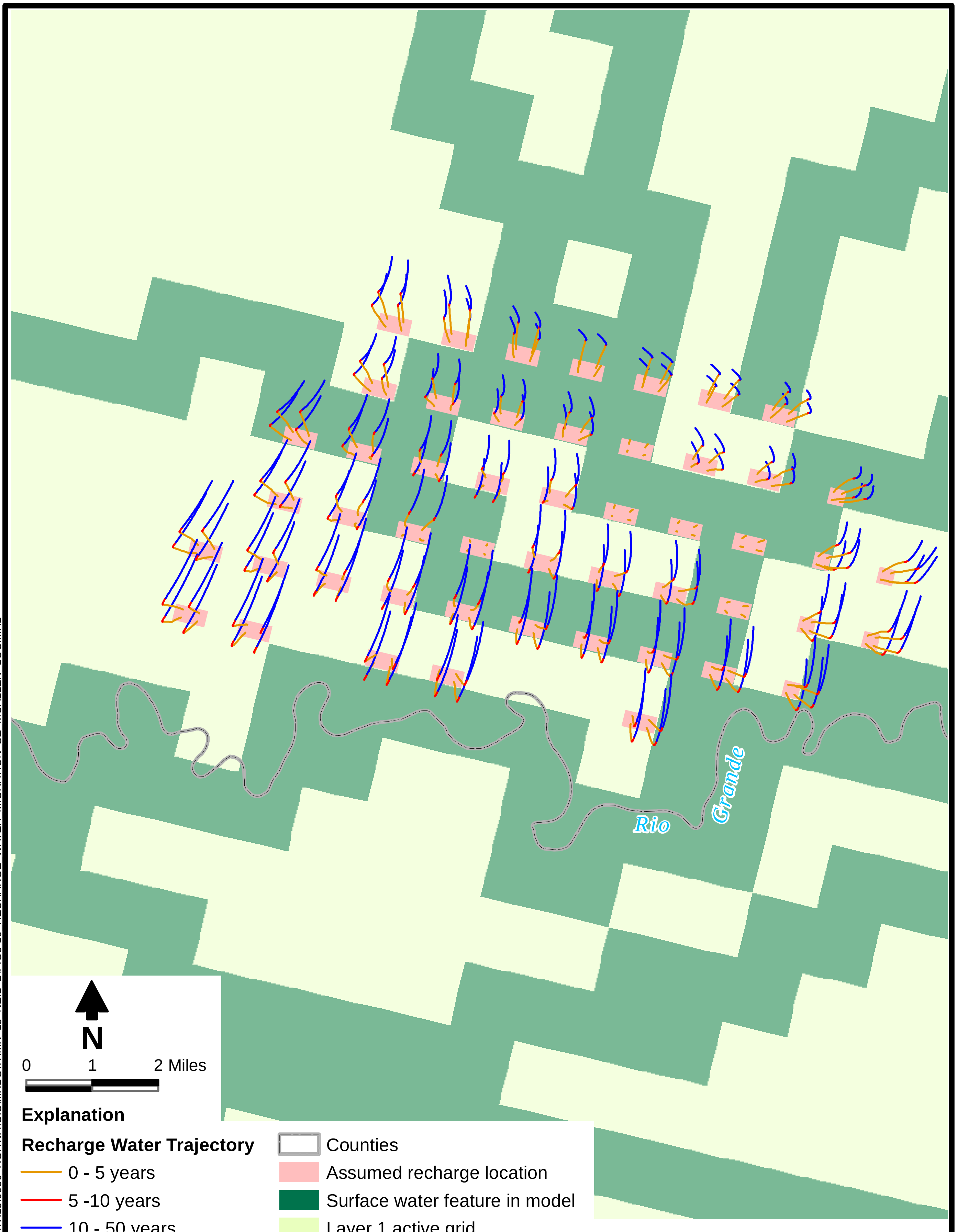
Figure 5-18



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Recharge Water Migration for Southeast of McAllen Location

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Figure 5-19

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CHAPTER 6 – WATER CONSERVATION

Regional Facility Plan

BLACK & VEATCH PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

BLACK & VEATCH
Building a world of difference.

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6.0 Water Conservation

6.1 WATER CONSERVATION ANALYSES AND PLANNING

The purpose of this chapter is to build on the strong foundation of water conservation planning in Texas and identify opportunities for additional conservation in Cameron, Hidalgo and Willacy counties. Within the broader plan, there is an important role for water conservation especially in light of the projected trend of a doubling in population in the region over the next 50 years.

6.1.1 Planning Background

All public water suppliers are required by the Texas Administrative Code Rule §288.2 to submit a Drought Contingency and Water Conservation Plan to the TCEQ for approval. These plans must include a utility profile including population and water use data (total gallons per capita per day (GPCD) and residential per capita), specific water savings goals and conservation strategies to meet those goals.

In 2001, the Texas Legislature amended the Texas Water Code to require Regional Water Planning Groups to consider water conservation and drought management strategies for every entity with a projected water shortage (need). The Water Conservation Implementation Task Force was created by Senate Bill 1094 to identify Water Conservation Best Management Practices (BMPs) and develop a BMP Guide[1] for use by Regional Water Planning Groups and utilities. Best Management Practices contained in the BMP Guide are voluntary efficiency measures that save a quantifiable amount of water, either directly or indirectly, and can be implemented within a specific timeframe.

The Texas legislature created the Water Conservation Advisory Council (WCAC), in 2007, consisting of 23 experts representing various agencies, political subdivisions, water users, and interest groups. One of the Council's roles is to improve and promote BMPs and has a website dedicated to water conservation BMPs (http://www.savetexaswater.org/bmp/), which is a foundational resource to the development of advanced water conservation plans. The WCAC continues to evaluate and update the BMPs.

Within the Lower Rio Grande area there have been a number of conservation programs which have been initiated by various individual entities. However, this has been relatively localized and have followed more passive conservation practices (such as enforcing the plumbing code) rather than detailed and advanced conservation measures. This analysis evaluates the current water conservation and provides recommendations for future advanced conservation measures.

Figure 6-1 outlines the projected population increases in the area. This increase, coupled with the associated rising demands means that conservation will be important to slow the demand for new water resources.

[1] *Best Management Practices for Municipal Water Users*. Water Conservation Best Management Practices, TWDB November, 2013.

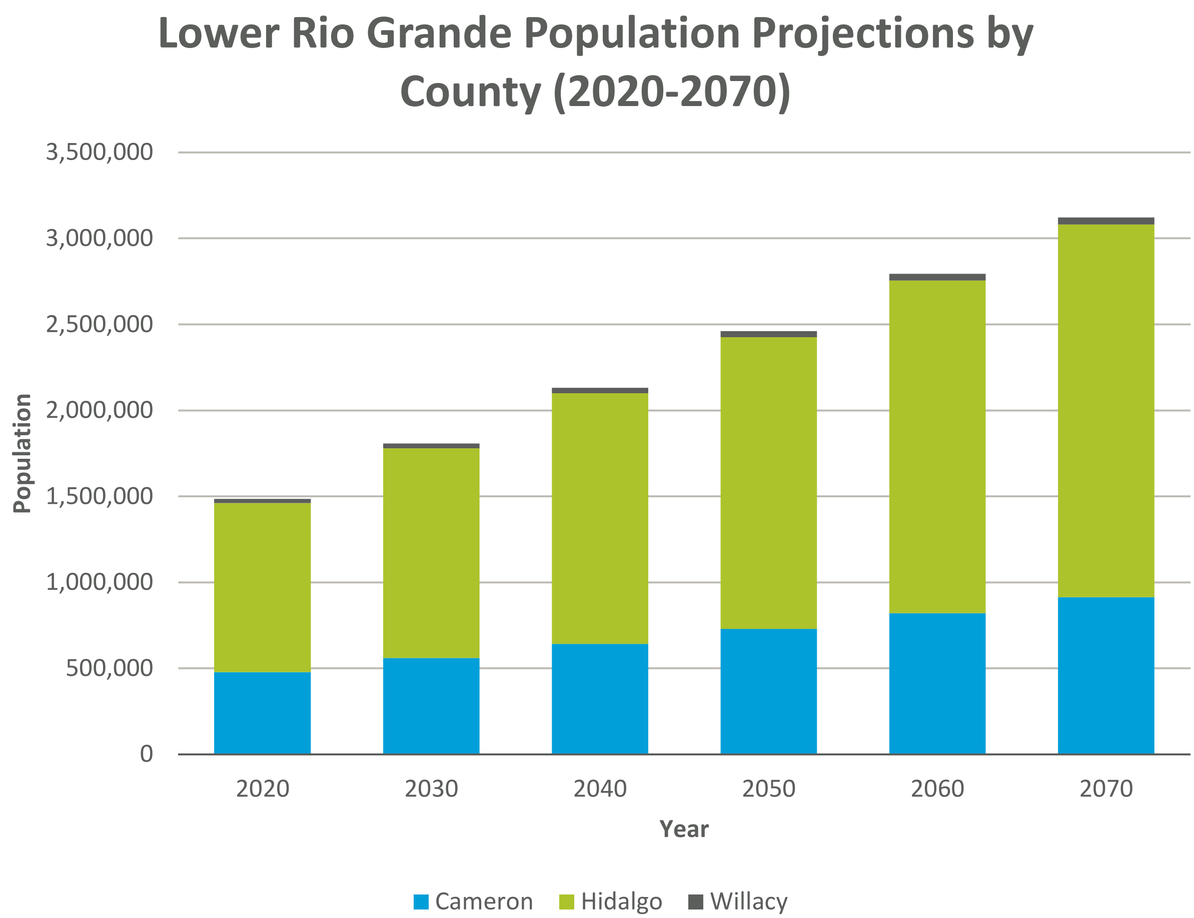

Figure 6-1 Lower Rio Grande Population Projections by County (2020-2070)

6.1.2 Water Use Overview

Current water use in the Region M Planning area is predominately from the Rio Grande. A small amount of fresh groundwater is being used, while brackish groundwater has become a bigger part of the regions portfolio. Reclaimed wastewater is being used to some degree for irrigation, cooling of power plants, and other non-potable processes. The subset of the study area is very similar in its water profile. Figure 6-2 displays the various major water sources in the area as a percent of the projected 2020 use from the Region M plan.

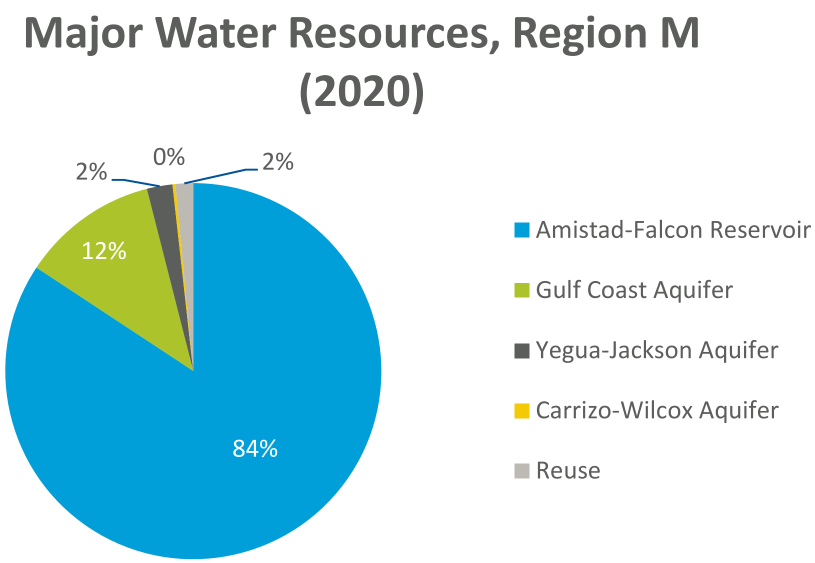

Figure 6-2 Major Water Resources, Region M (2020)

Practically all of the surface water used in the Rio Grande Region is from the Rio Grande, which is supplied from the yield of the Amistad and Falcon International Reservoirs. The Falcon Reservoir releases just less than 1 million acre-feet (AF) of water in an average year.

6.1.3 Rio Grande Regional Water Authority

The RGRWA was created by the 78th Legislature to supplement the services, regulatory powers and authority of irrigation districts, water development supply corporations, counties, municipalities, and other political subdivisions within its border. The RGRWA shares an approximate boundary with the Region M Water Planning Group. The focused study area includes a large portion of the Rio Grande Regional Water Authority jurisdiction commonly referred to as the Lower Rio Grande Valley. Specifically, the area includes 55 municipal water user groups (WUGs) in the three southernmost counties in the state, Cameron, Hidalgo, and Willacy.

6.1.3.1 Per Capita Water Use

Figure 6-6 shows total system per capita water use (in gallons per capita per day) for 2020 (the first projected year). The data are ranked and range from a minimum value of 60 gpcd to a maximum of 868 gpcd. The average value is 148 gpcd and the median value is 117 gpcd.

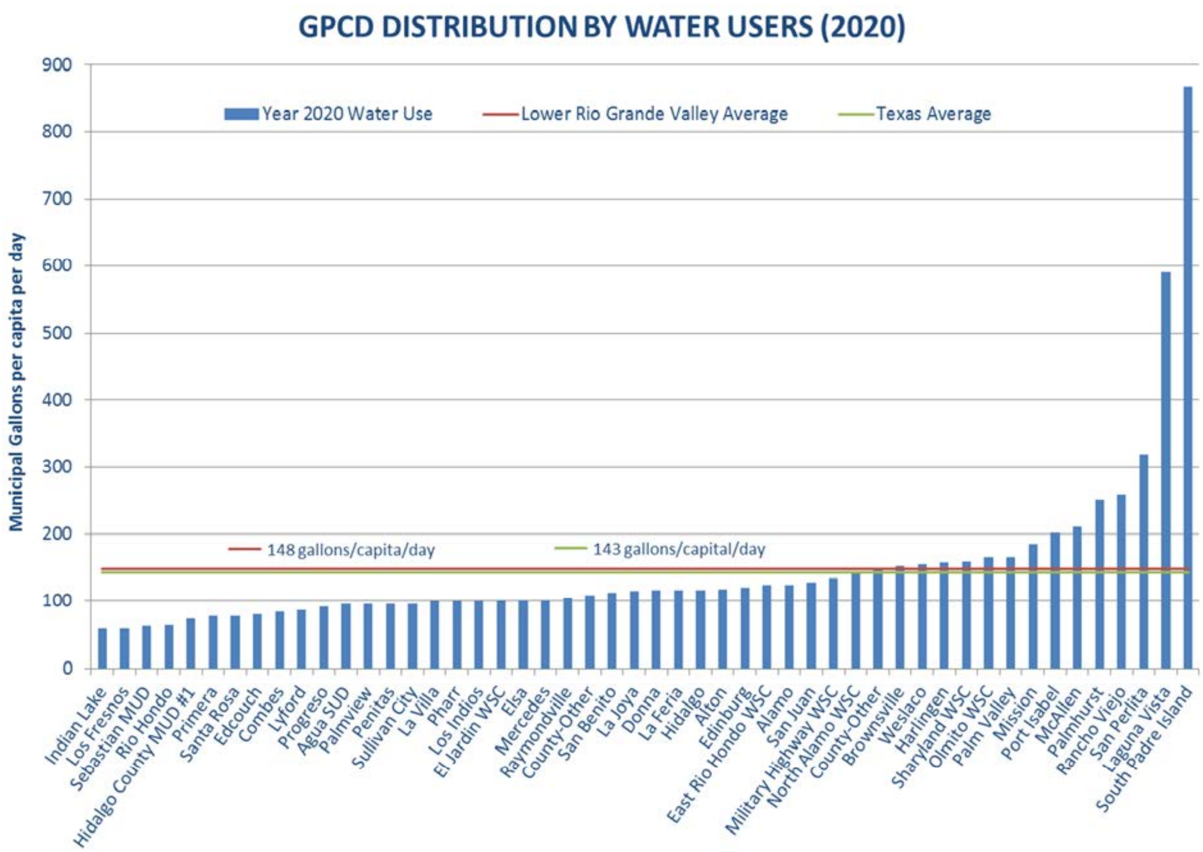

Figure 6-3 Municipal Water Demands and Population Projections (2020 – 2070)

The advanced water conservation plan strategies outlined in this chapter are focused on residential and typical municipal customers. As addressed in section 1.7 below, it is important for the planning process to first develop a sound understanding of the components of overall demand and the customer base for a specific WUG, in order to identify and prioritize effective conservation strategies. Some of the water users shown in Figure 6-6 with higher per capita use may be as a result of non-residential water uses being included in the per capita use calculation. For example, in the case of the larger communities and cities in the study area, water use will be driven up by commercial use and commuters driving to these areas which are typically employment hubs. Some of the areas are also vacation destinations and this can significantly skew some of the results on the high end of the spectrum, such as on South Padre Island, as discussed in section 1.3.3.

6.1.3.2 Projected Trends in Water Use

Municipal water demands have been projected by multiplying the per person forecasted water use by the forecasted population. These demands are calculated in ten year increments for the 50 year planning horizon (see Chapter 2 for additional details).

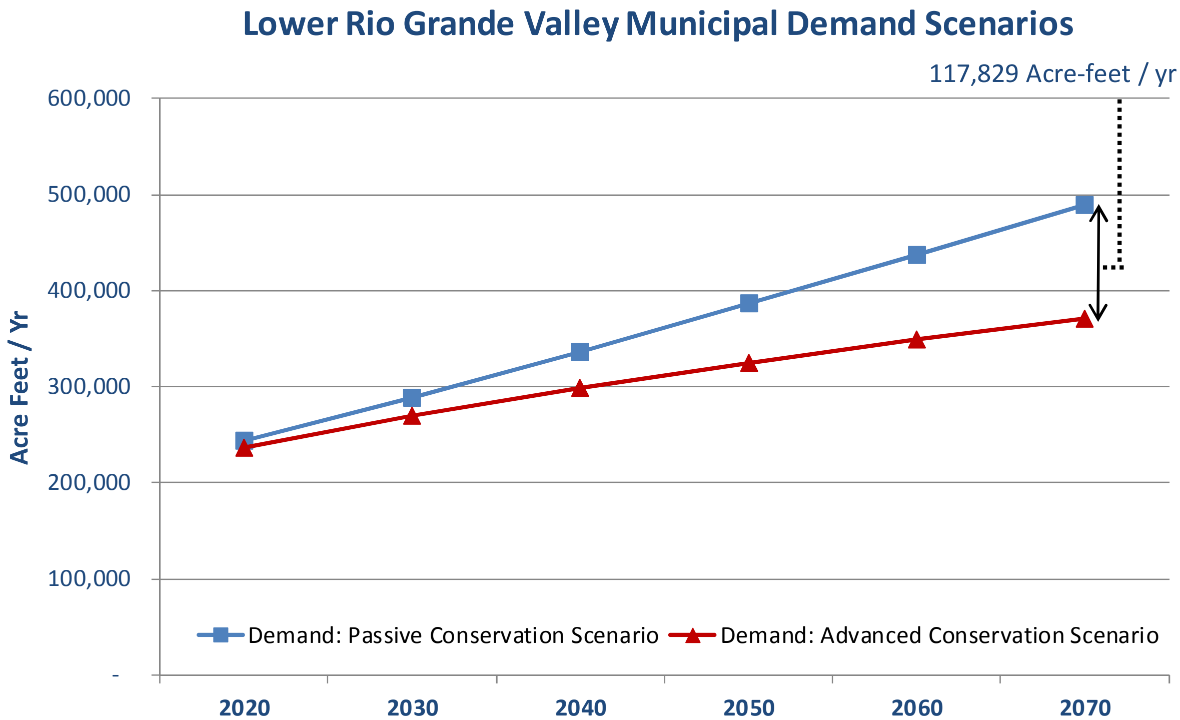

Figure 6-4 Municipal Water Demands (2020 – 2070)

Figure 6-1 indicates significant projected population growth for the study area with the population anticipated to double over the next 50 years. Municipal water demand is shown in Figure 6-4 and has been calculated under two scenarios. Municipal demand under passive conservation is shown in blue and reflects a slightly decreasing per capita consumption over the 50 year planning horizon, as documented in Chapter 2. The advanced conservation scenario shows the impact and benefits of a more aggressive set of water conservation measures, based on projected decreases in per capita use of between 0.5% and 1.0% per year (discussed in more detail in section 1.4.2). If advanced water conservation is implemented and per capita consumption decreases in line with expectations described in section 1.4.2, then these measures can be expected to reduce the increase in demand (versus the passive conservation scenario) by nearly 118,000 acre-feet (AF) per year for the year 2070. Another way of expressing the reduction in demand is that it is the equivalent of a reduction of 33.6 gallons per capita per day attributable to Advanced Water Conservation (Figure 6-5). This reduction in demand represents a significant saving in water use and will only be possible if aggressive steps are taken by water users to develop and implement an Advanced Water Conservation Plan.

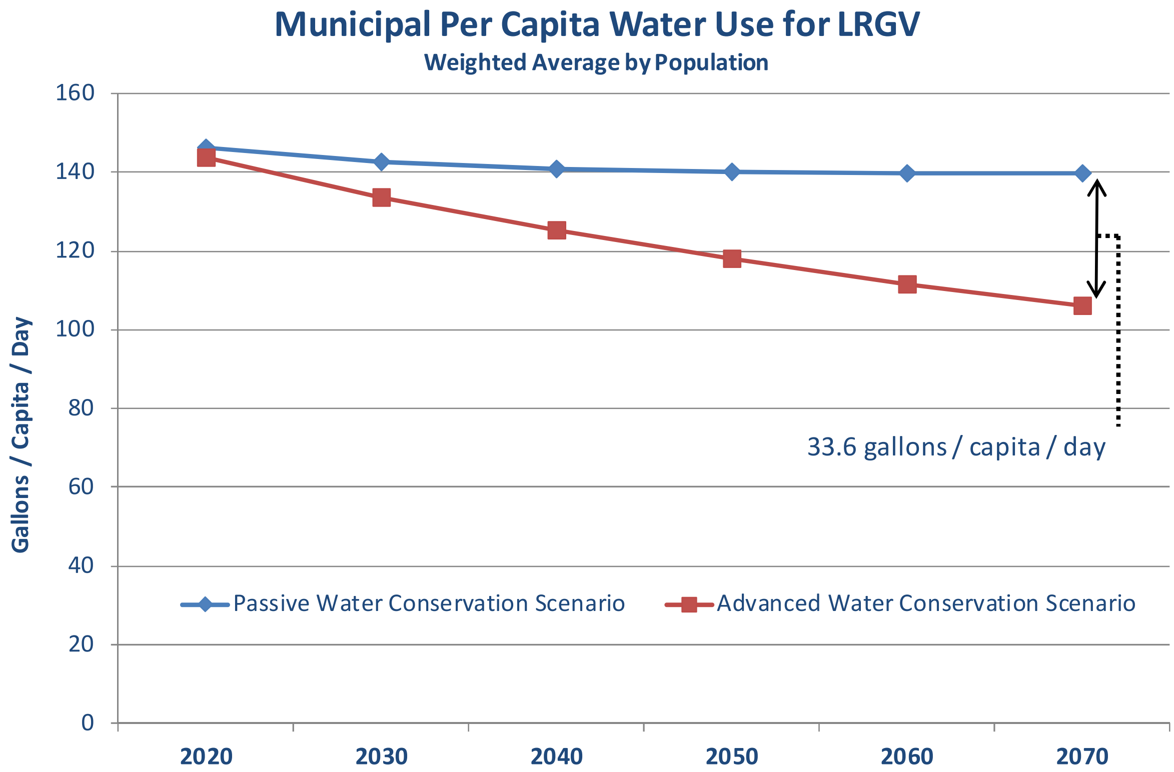

Figure 6-5 Per Capita Water Use Projections (2020 – 2070)

6.1.3.3 Top Five Users

As defined by per capita water use, the top five WUGs in the LRGV are as follows (see Figure 6-2):

1. South Padre Island (877gpcd)
2. Laguna Vista (599gpcd)
3. San Perlita (330gpcd)
4. Rancho Viejo (267gpcd)
5. Palmhurst (259gpcd)

It should be noted that the two water users with the highest per capita water use show significantly different socio-economic and customer characteristics than the typical WUGs in this area. South Padre Island (Cameron County) has the highest per capita use at 868 gpcd. South Padre Island is a popular resort town located on the barrier island. According to the 2010 US Census, its population is 2,816; however as a popular vacation destination it attracts significant visitors, especially during the summer months, which drives per capita water use higher as per capita values are typically derived from the residential (i.e., static) population. Laguna Vista (Cameron County) is a small residential community with an associated golf course and a number of irrigation customers which will increase the GPCD. These unique characteristics need to be taken into account when evaluating conservation potential. In many cases it will be necessary to separate the use data to allow reasoned comparisons. For example; golf course irrigation water use should be calculated separately from residential usage to determine a GPCD value for the residential community. This can then be used to construct dedicated water use reduction programs (if necessary) for the specific sectors.

As defined by total volume of water use, the top five WUGs in the LRGV area as follows:

1. McAllen (38,728 AF/yr)
2. Brownsville (36,092 AF/yr)
3. North Alamo WSC, Hidalgo Co (24,015 AF/yr)
4. Mission (20,212 AF/yr)
5. Harlingen (13,546 AF/yr)

Collectively, the top five WUGs by volume account for over 55% of total municipal water use in the LRGV, indicating that conservation efforts focused in these locations have the potential to significantly influence overall water demand.

6.1.4 Conservation Assumptions

Water conservation is defined as those methods and practices that either reduce demand for water supply or increase the efficiency of supply, or use facilities so that available supply is conserved and made available for future use. Water conservation is typically a non-capital intensive alternative (compared to supply-side development) that any water supply entity can and should pursue. Through conservation, the life of existing supplies can be extended which will minimize the environmental impacts associated with new supplies and delay the cost of developing additional water supplies.

6.1.4.1 Passive Conservation

The current TWDB municipal water demand projections account for expected water savings due to implementation of the 1991 State Water Efficient Plumbing Act. Any additional projected water savings from conservation programs must be listed as a separate water management strategy. The savings projected by the TWDB include complete replacement of existing plumbing fixtures to water-efficient fixtures by the year 2045. The projections also assume that all new construction includes water-efficient plumbing fixtures. It is important when including a retrofit program as a water management strategy to not double-count water savings, as savings due to retrofits are already included in the base water demand projections.

6.1.4.2 Advanced Water Conservation

Advanced Water Conservation is recommended for every WUG in Region M. A variety of conservation measures are recommended as described in the TWDB Best Management Practices, any combination of which can be used to meet the specific goals for a municipality or utility.[2] It should be noted that some of the WUG's are reportedly already at the floor or minimum water conservation level expected. These utilities will be expected to make sure the data is valid and keep the GPCD at these levels or lower.

[2] Water Conservation Implementation Task Force, "Water Conservation Best Management Practices Guide," November 2004.

For every municipal WUG with a projected need or a per capita water use rate greater than 140 gallons per capita per day (GPCD), municipal conservation yields were estimated and included in the future projected demand. The amount of water that can be conserved by implementing Advanced Municipal Conservation measures was estimated with the assistance of the Unified Costing Model (UCM) tool. The methodology is based on the "Quantifying the Effectiveness of Various Water Conservation Techniques in Texas"[3] study conducted for the TWDB.

For entities that have projected needs as defined through the regional planning process, the usage reduction rate was based on the current GPCD. Entities with needs and a per capita usage greater than 140 GPCD were assigned a 1% usage reduction per year. The usage reduction rate after the 140 GPCD goal was achieved, or for entities with a need and a GPCD below 140, was set to 0.5%. A minimum value of 60 GPCD was fixed based on the "Projection Methodology – Draft Population and Municipal Water Demands" memo from the TWDB referencing the *Analysis of Water Use in New Single-Family Homes*[4] study and internal report, *The Grass Is Always Greener…Outdoor Residential Water Use In Texas*[5]. Once the minimum value was reached, entities were projected to stop reducing their GPCD. For municipal entities that have needs starting later than 2020 and base year GPCD below 140, the Advanced Water Conservation strategy is projected to begin in the first decade with needs.

Entities that are not projected to have a need, but have per capita usage above 140 GPCD in 2011 are recommended to implement Advanced Conservation at a rate of 1% reduction per year beginning in 2020. Once these entities reach a GPCD of 140, it was assumed that Advanced Conservation would continue to yield a steady volume without an additional cost, but that additional reductions in use are not anticipated.

The 2016 Region M report recommends that entities without needs that have a 2011 per capita water use rate above the minimum of 60 GPCD implement Advanced Water Conservation.

The calculations use the GPCD estimated for each municipality, based on projected population and water demands (see Chapter 2). For every decade, the Base GPCD was calculated from the projected water demands before reductions due to Advanced Water Conservation strategies are implemented. A Base Per Capita Goal was determined by reducing the Per Capita Water Use in the decade of implementation annually by the reduction rates discussed above. The yield of Advanced Water Conservation, or the amount of water conserved in each decade, is the difference between the Per Capita Water Use and the Base Per Capita Goal, converted to acre-ft. per year.

The initial GPCD projections include reductions due to passive conservation, and in some instances the Per Capita Water Use may be lower than the Base per Capita Day. In this case, the Advanced Water Conservation is shown as zero. This may occur if the base GPCD rates projected by the TWDB decreases at a greater rate than the rates assumed for Advanced Municipal Conservation. One

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[3] GDS Associates, "Quantifying the Effectiveness of Various Water Conservation Techniques in Texas; Appendix VI, Region L," Texas Water Development Board, Austin, Texas, July 2003.

[4] Analysis of Water Use in New Single Family Homes, Prepared by William B. DeOreo of Aquacraft Water Engineering & Management for The Salt Lake City Corporation and the U.S. Environmental Protection Agency, 2011.

[5] The Grass Is Always Greener…Outdoor Residential Water Use In Texas, Sam Marie Hermitte and Robert Mace, Technical Note 12-01, 2012.

possible reason may be that if a municipality is projected to have high growth rates, then the GPCD would lower due to an increase in more efficient appliances that come with new construction.

The impact of the Base GPCD and Advanced Municipal Conservation GPCD scenarios on total water demands over a 50 year planning horizon can be seen in Figure 6-4. Based on the projection methodology described above, Advanced Municipal Conservation can be expected to reduce the projected increase in demand (versus the passive conservation scenario) by nearly 118,000 AF per year, for the year 2070.

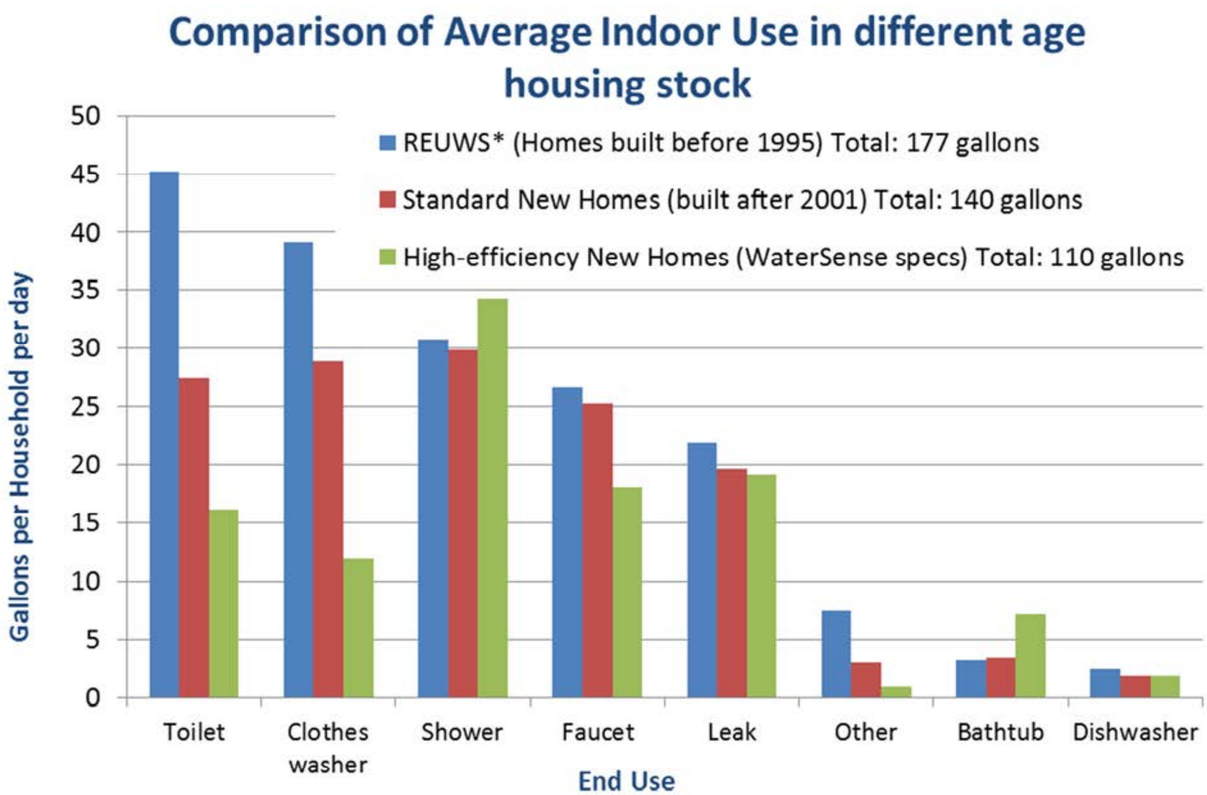
6.1.5 Comparison to National and State Statistics

Comparisons of water use efficiency typically use a per capita approach to normalize the data. Although this makes logical sense, as the number of residents in a house is the most important variable and its value varies from home to home (DeOreo, 2011), it does not always result in a meaningful comparison. Care should be taken when comparing per capita numbers as the number may be generated from a broad, top-down approach (i.e., dividing total water use by population), or by studies that specifically look at water use by the end-user (e.g., DeOreo, 2011).

A review of the per capita consumption data for the study area (presented in Chapter 2) indicates that the different WUGs have a wide range of per capita consumption values, from a minimum of 60 gpcd to a maximum of 868 gpcd. Such a wide disparity indicates that the customer base for the WUGs with higher rates likely contains a high proportion of non-residential use. The residential component of water use should be defined and isolated in order to realistically identify advanced water conservation strategies applicable to municipal uses and to allow water savings to be tracked.

6.1.5.1 National Perspective

A study of residential water use (DeOreo, 2011) quantified the savings in water use that can be expected from modern homes.



*** Mayer, Peter et al.. Residential End Uses of Water Study. AWWA Research Foundation. 1998.**

Chart adapted from: Analysis of Water Use in New Single Family Homes, Prepared by William B. DeOreo of Aquacraft Water Engineering & Management for The Salt Lake City Corporation and the U.S. Environmental Protection Agency, 2011.

Figure 6-6 Comparison of Average Indoor Use in Different Age Housing Stock

The study concluded that *"there are no technical reasons for not moving single family demands lower. The technologies for the key indoor fixtures and appliances are now available in the form of high-efficiency toilets, showers and clothes washers."* It should be noted that the water use volumes shown in Figure 6-6 are household numbers and not per capita numbers, however the important message conveyed in the figure is that significant water savings can be expected through the introduction and retro-fitting of more water efficient fixtures and fittings with the average total indoor use declining from 177 gallons *per household* to 110 gallons *per household* as reported by the study data.

6.1.5.2 State perspective

The TWDB collects and publishes annual statewide per capita water use numbers and also requires retail water suppliers submit a water loss audit according to the following schedule[6]:

- Any retail water supplier with an active financial obligation with the Texas Water Development Board is required to submit a water loss audit annually

[6] http://www.twdb.texas.gov/conservation/municipal/waterloss/

- Any retail water supplier with more than 3,300 connections is required to submit an audit annually.
- Any retail public water supplier is required to submit a water loss audit once every five years.

6.1.5.2.1 Per Capita Water Use

At the time of writing, the most recent data available covered the year 2013[7] and reported an average municipal value of 143 gpcd and an average residential value of 84 gpcd. The difference in these values is partially explained by the different types of water use that occur in larger communities (the municipal value) where a significant number of non-residential customers and uses are present, combined with the fact that the denominator in the gpcd calculation is the resident population. Additional discussion of these influences can be found in section 1.3.3. Based on the data from TWDB, per capita water use in the LRGV does not appear to be significantly different from Texas as a whole (see Figure 6-7).

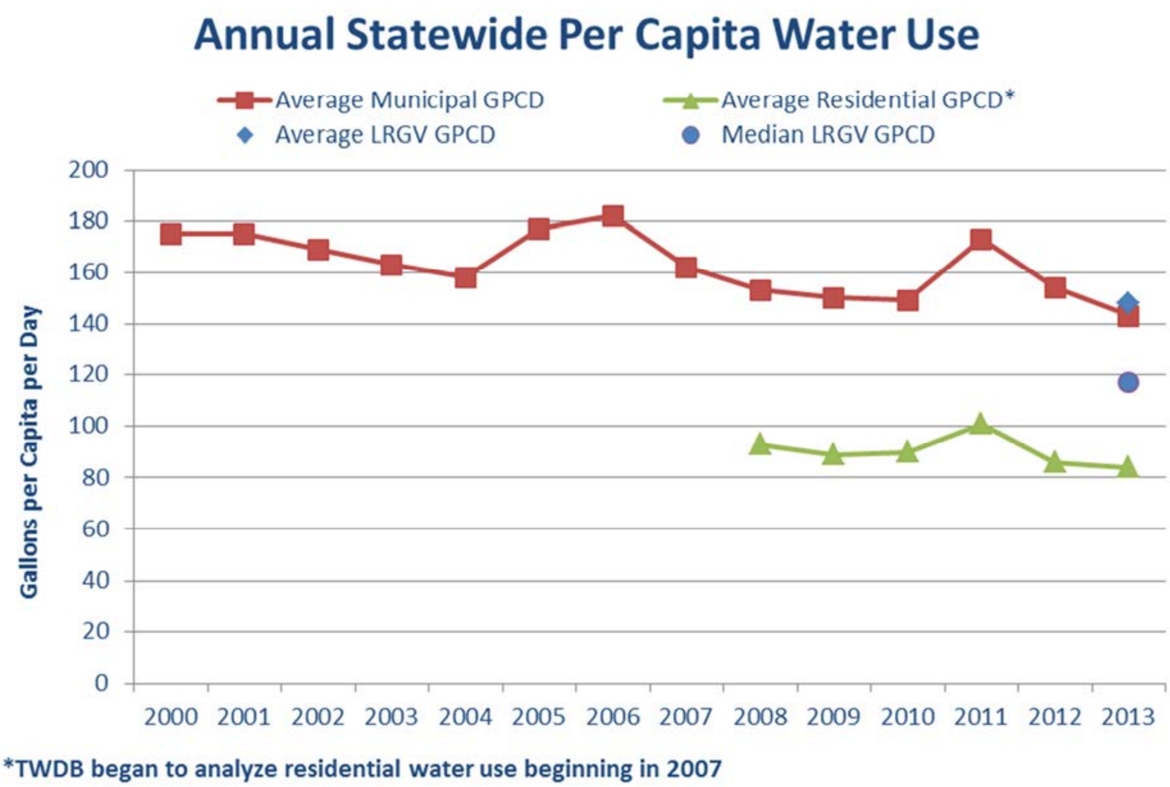


Figure 6-7 Annual State Wide Per Capita Water Use

6.1.5.3 Indoor Water Use

Indoor water use in Texas typically accounts for approximately 69% of total residential water use[8]. An analysis of water use in new single family home (DeOreo, 2011) found that the three end uses collectively accounting for the majority of indoor household use were toilets (20%), clothes

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[7] http://www.twdb.texas.gov/waterplanning/waterusesurvey/estimates/data/TexasStatewideReport_6_12_15_Revision.pdf
[8] The Grass Is Always Greener…Outdoor Residential Water Use In Texas, Sam Marie Hermitte and Robert Mace, Technical Note 12-01, 2012.

washers (21%) and showers (21%). These figures represent average values for standard new homes built after 2001 (see Figure 6-8).

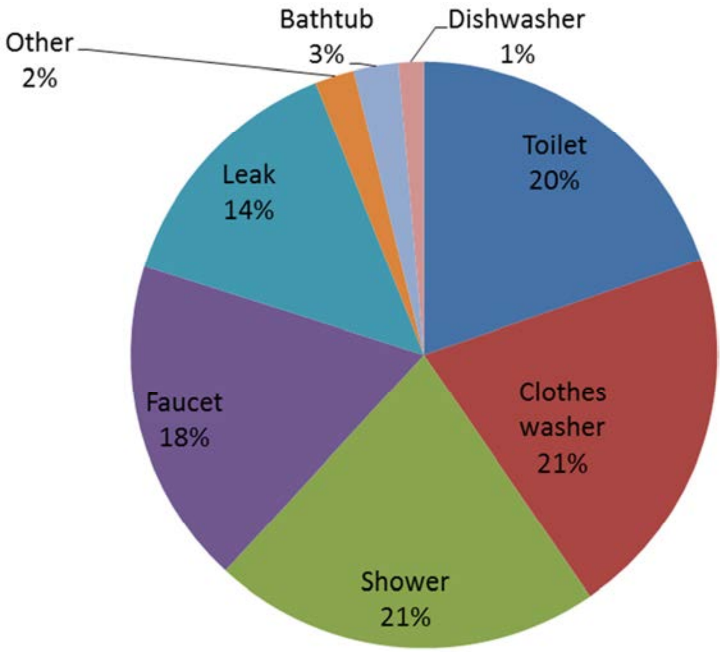


Figure 6-8 Typical indoor household water use by end use type

The information conveyed in Figure 6-8 is helpful in prioritizing areas of focus for retrofitting more water efficient fixtures and appliances. The study also showed the water conservation impact of moving to a higher level of water efficiency through retro-fitting or new construction, using more water efficient fixtures and fittings. As noted in section 1.3.3, a direct comparison of per capita use values in the LRGV study area and the DeOreo study is likely not conclusive because of the different components of demand (LRGV numbers reflect general municipal demand and the DeOreo study is focused specifically on residential end use). However, the findings of the DeOreo report suggest that retrofitting the most efficient fixtures and fittings will significantly contribute to achieving the overall water use reductions projected in the Advanced Water Conservation scenario.

6.1.5.4 Water Loss Audits

The TWDB utilizes a methodology derived from the American Water Works Association (AWWA) and the International Water Association (IWA). This new standard uses terminology such as authorized consumption, real loss, apparent loss, and non-revenue water. Traditionally, the water utility industry has used percentages to determine water loss, but the AWWA methodology uses more robust metrics that will help utilities track water loss and identify issues that may need addressing.

One of the new performance indicators is the Infrastructure Leakage Index (ILI) which is a measure of current real losses against the theoretical lowest level of real losses that could be expected given the specific water system characteristics. An ILI of 1.0 therefore represents optimal performance when it is considered with optimal management of pressure (as low a pressure as is possible within each system). Based on the data collected by TWDB in 2010, an analysis was performed to benchmark performance against national trends and within the Texas planning regions. The results are shown in Figure 6-9 and indicate that Region M shows the highest ILI score (indicating highest real water losses) of the 16 planning regions. A national ILI benchmark is also plotted that is based on published and validated water audit data from 26 water utilities in North America[9]. This finding supports the focus on water audits as an important tool in reducing overall water demand (by reducing real losses). More information on water audits and developing a water loss plan is provided in section 1.8.4.

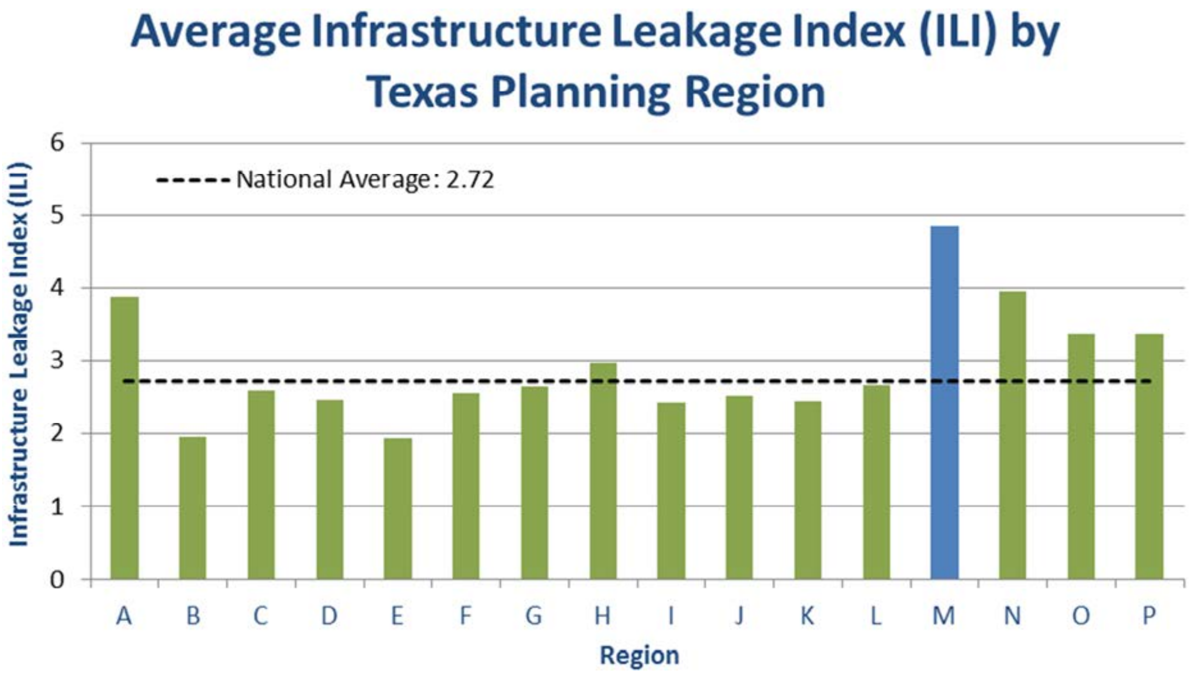


Figure 6-9 Infrastructure Leakage Index (ILI) by Planning Region

[9] AWWA 2014 Validated Water Audit Data http://www.awwa.org/resources-tools/water-knowledge/water-loss-control.aspx

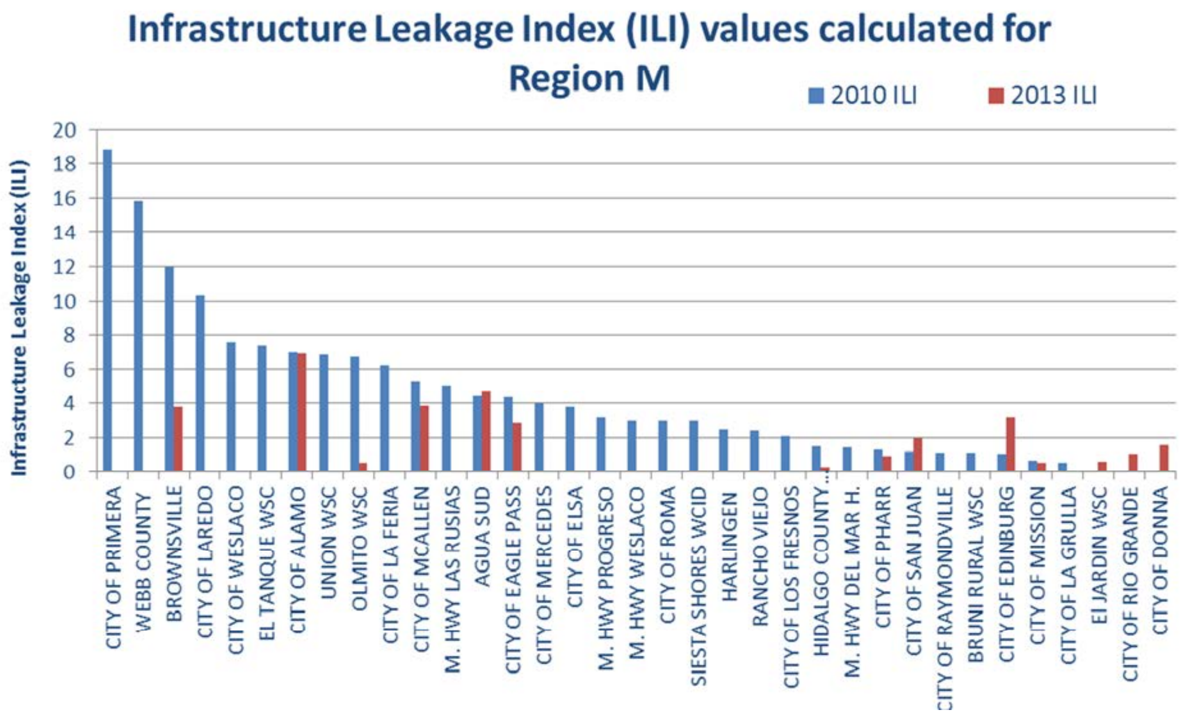


Figure 6-10 Infrastructure Leakage Index (ILI) of WUGs in Region M

The data shown in Figure 6-10 with respect to ILI outlines the level of real loss, or leakage within the systems in question. Therefore, the higher the value, the more potential there will be for reduction of leakage. It should be noted that some of the highest and lowest values may be driven by data that needs to be validated and verified, but there will likely be potential for leakage reductions in a number of the communities identified with high ILI values. Percentage values are not used as they are not good indicators between utilities of varying size and demographics.

6.1.6 Components of an Advanced Water Conservation Plan

Advanced water conservation can be achieved using a variety of strategies. Selecting the appropriate strategies will depend on a thorough understating of the baseline conditions for the individual utility and the available financial resources. Therefore the identification of utility-specific strategies is beyond the scope of this study but the options presented below should be considered by all utilities and evaluated on an individual basis. Although water conservation is typically a cheaper alternative to new supply development, implementation cost will be a factor and the financial resources available to individual utilities will influence decision making. It should be noted that different strategies will have different pay back periods. A full cost-benefit analysis is beyond the scope of this study but is recommended. The TWDB website lists the following BMPs which could be implemented to achieve advanced municipal water conservation:

Table 6-1 Texas Water Development Board Best Management Practices

BMP TYPE	BMP DESCRIPTION
Conservation Analysis and Planning	Conservation Coordinator
	Cost Effective Analysis
	Water Survey for Single-Family and Multi-Family Customers
Financial	Water Conservation Pricing
	Wholesale Agency Assistance Programs
System Operations	Metering of All New Connections and Retrofit of Existing Connections
	System Water Audit and Water Loss Control
Landscaping	Athletic Field Conservation
	Golf Course Conservation
	Landscape Irrigation Conservation and Incentives
	Park Conservation
	Residential Landscape Irrigation Evaluation
Education & Public Awareness	Public Information
	School Education
	Small Utility Outreach and Education
	Partnerships with Nonprofit Organizations
Rebate, Retrofit, and Incentive Programs	Conservation Programs for ICI Accounts
	Residential Clothes Washer Incentive Program
	Residential Toilet Replacement Programs
	Showerhead, Aerator, and Toilet Flapper Retrofit
	Water Wise Landscape Design and Conversion Programs
Conservation Technology	New Construction Graywater
	Rainwater Harvesting and Condensate Reuse
	Water Reuse
Regulatory & Enforcement	Prohibition on Wasting Water
	Conservation Ordinance Planning and Development

6.1.6.1 Data Driven Planning

Water use depends on various factors such as population, climate, land use, condition of the water distribution infrastructure and socioeconomic characteristics (e.g., cost of water relative to income level of residents). In order to design an effective water conservation strategy, it is important to gather adequate and accurate information on the factors listed above. It is also important to accurately track water use so that the impact of water conservation can be monitored and evaluated, including the assessment of progress against any targets or goal. To support data driven planning it is important to have an accurate assessment of the following aspects of water use:

- **Source metering.** It is important to accurately measure water withdrawals in order to provide accurate information to state and other agencies that have the responsibility of assessing water resource impacts and planning for future growth.
- **Production metering.** Water gains economic value when it is purified and pressurized and sent into the distribution system. To understand the efficiency of water distribution systems and track losses through a water audit, it is vital to have an accurate measurement of production metering.
- **Customer metering.** Customer meters are the 'cash registers' for the water utility operations and a metered system is the best way to equitably spread the cost of water service. Therefore it is important to ensure that the meters are functioning accurately to not only recover revenues owed to the utility but also to ensure customer equity and the effectiveness of pricing signals to encourage water conservation.
- **Customer end use.** Beyond the customer meter water, water use patterns will be influenced by regional, local and customer-specific characteristics of use. Effective, advanced water conservation planning will need to understand these characteristics and employ strategies that target specific end uses.

With appropriate tracking of water use – which integrates the impact of conservation strategies - future decision making can be improved and plans adjusted as required.

6.1.7 Developing an Advanced Water Conservation Plan

The following sections outline an example Advanced Water Conservation Plan that could be implemented by municipal water users in the LRGV.

6.1.7.1 Utility Profile

All public water suppliers are required by the Texas Administrative Code Rule §288.2 to develop a utility profile in accordance with the Texas Water Use Methodology including information on population, per capita water use and water supply and wastewater system data. In order to implement Advanced Water Conservation, it is recommended that the utility adopt additional, proactive data collection methods that will provide greater insight into water use patterns and help to target water conservation strategies.

Development of a utility profile is a good place to begin a water conservation plan. The utility profile should summarize supply and demand aspects such as sources of available water and population and major demographics. These aspects have been developed for the LRGV as a whole as noted in Chapter 2 where population and future demand has been considered. This type of planning activity should be conducted for each individual WUG, including assessment of:

- Available resources
- Current demand
- Future Demand

Once completed, the utility profile will focus attention on why conservation is important as it will reference water resources (such as reservoirs and rivers) that are familiar to the local community and is the first step towards engaging customers in advanced water conservation.

6.1.7.2 Residential Water Surveys

Beyond the utility profile, an example of data collection to inform advanced water conservation outcomes could be for a utility to develop robust data on its customer base. This could go beyond the basic understanding of customer types (i.e., residential versus commercial etc.) to include an assessment of indoor versus outdoor water use in order to target water conservation initiatives. This type of information could be developed by looking at water use profiles from actual customer data, or it could be estimated from individual parcel level data including attributes such as lot sizes.

Another example of this strategy is if the utility is unaware of the number, or percentage, of customers using automated irrigation systems, a drive-by survey can be conducted on a sample of customers to develop an estimate of how many have automatic systems (TWDB, 2013).

An important driver for many water conservation strategies is the incentive for the end user to reduce costs by saving water. This price signal relies on the appropriate rate structure but more fundamentally it relies on all customers being metered and billed accordingly. Therefore, it is important to ensure that all public water suppliers implement a policy of 100% metering of all customers.

Desk-based research can also be helpful and potentially more cost effective than an on-the-ground survey. For example, the United States Census Bureau publishes the American FactFinder website (www.factfinder.census.gov) which allows detailed information to be queried for individual towns and cities, within the study area. Information can be retrieved on household and demographic information, including the following (an example is included in Appendix x):

- Household size
- Age of housing construction
- Occupancy / vacancy rates
- Ownership / rental rates
- Household value
- Household / disposable income

This information can be useful to prioritize individual conservation strategies, or even to identify towns and cities with the greatest potential for conservation savings from a more strategic planning level. The TWDB, in partnership with the Texas Commission on Environmental Quality (TECQ) published a GPCD calculator, that incorporates this type of information, to help quantify and track water uses associated with water distribution systems.

Specific knowledge of the customer base will help determine the focus of water conservation strategies. To enhance information on customers' water use habits, a water use survey for single-family and multi-family customers can be conducted. A Water Use Survey Program can be an effective method of reducing both indoor and outdoor water usage. Surveys should be offered based on water use starting with the highest single-family and multi-family accounts, respectively. Using this approach, the utility conducts a survey of single-family and multi-family customers and uses the information gathered to provide information to them about methods to reduce indoor water use through replacement of inefficient showerheads, toilets, aerators, clothes washers, and dishwashers (TWDB, 2013). There are typically three options for conducting the survey:

- Train utility staff to conduct an onsite survey;
- Hire an outside contractor to conduct the onsite surveys; or
- Provide a printed or online survey for customers to complete on their own.

6.1.7.3 Financial Incentives

Water rates that encourage conservation can be powerful tools to reduce per capita use. Three effective conservation rate structures include volumetric pricing with uniform or increasing block rates, seasonal pricing, and allocation-based rates. Increasing block rates charge a higher amount per gallon as usage increases, which provide an incentive to keep use low. Seasonal rates charge a higher amount per gallon during the irrigation season when the water supplier's demands are highest, because the peak demands are generally most expensive for the supplier to meet. Allocation-based rates include higher per-gallon costs for usage exceeding base usage established for each customer according to customer characteristics, such as number of occupants or size of irrigated landscape. Flat rates (generally used by suppliers that do not yet meter water use) and rate structures that reduce the per-gallon price for increased usage (declining block rates) are not considered to be conservation pricing structures and are not recommended.

For any of these rate structures, retail water bills typically include two parts: fixed charges and variable charges that are based on the amount of water used by the customer. Water billing that includes a relatively small fixed portion and a significant volumetric component that increases with volume of water use provides a financial incentive to the consumer to reduce water use. The installation of water meters and billing by volume of use can reduce water use by ten percent. While increasing block rates are generally the most effective, there may be little additional cost incentive to the customer compared to uniform rates if the increase in per-gallon cost is small.

State agencies recognize the complexity and sensitivity of rate-setting. Increasing block rates should be encouraged; however local suppliers must continue to have authority for rate setting, because they have responsibility to ensure balanced budgets and fiscal solvency. Good communication can complement a conservation rate structure and help ensure that customers respond to an effective pricing signal. Billings need to communicate to the customer the amount of water used in commonly understood units such as gallons rather than units that are more commonly used by water suppliers such as hundreds of cubic feet. Water suppliers should further reinforce the conservation message by providing customers with comparisons of current and past usage, comparisons to usage by similar customers, and information on how billings are affected by increased use. More frequent billing, that is, monthly, also can be more effective (DWR, 2010).

A review of available water rates for WUGs in the LRGV was performed. A number of rates structures are in place including:

- Inclining block rates per volume unit
- Flat rate per volume unit
- Different rates for users inside/outside city limits
- Water Budget rates based on winter use
- Rate schedules for potable, irrigation and reuse

To compare rates, a standardized use of 12,000 gallons per month, per household was assumed (this is based on median per capita water use values multiplied by average household size for the LRGV). Figure 6-11 shows a wide range of water rates from a low of approximately $20 per month to a high of over $80 per month and can be used to benchmark a municipal system against its peers.

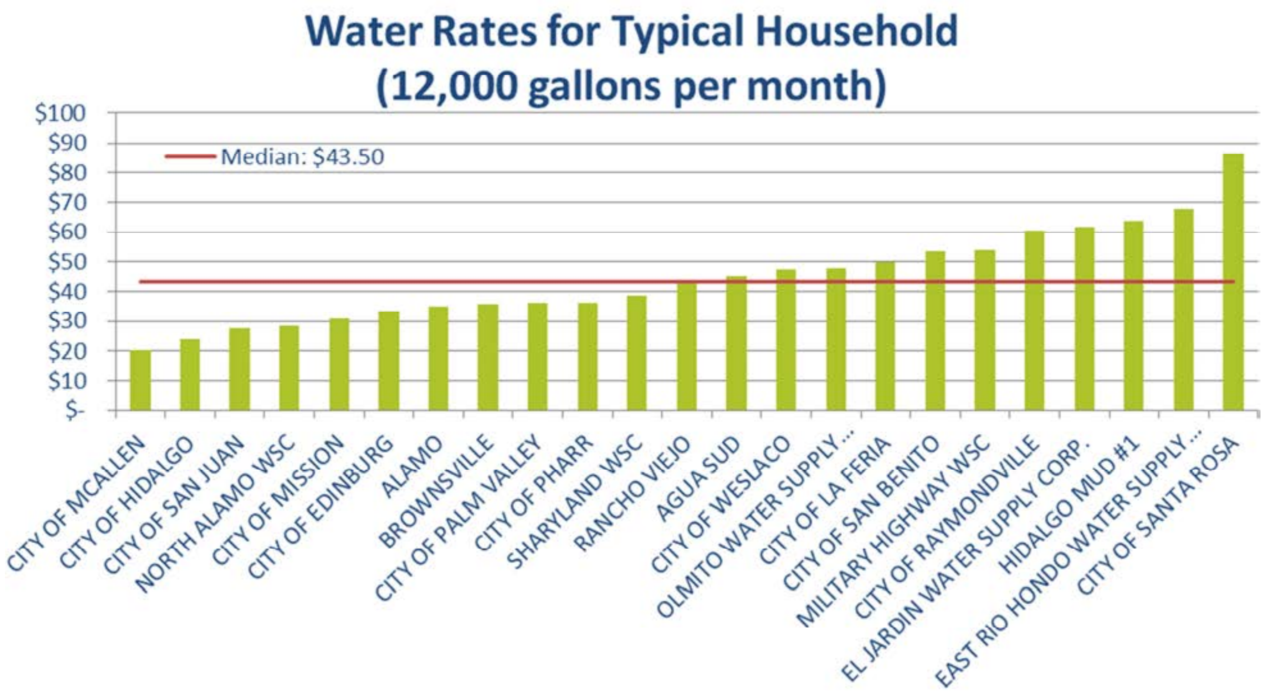


Figure 6-11 Monthly Water Bill for Typical Use in the LRGV

To illustrate the potential effectiveness of price signals influencing water use, Figure 6-12 plots typical monthly water rates against per capita water use for municipal water users in the LRGV. Although it has been noted that there are many influences that drive per capita water demand, this high-level analysis indicates that as the cost of water increases per capita water use tends to decrease. Those systems that have high per capita use should review their water rates and rate structures to determine if there is potential to incentivize water conservation through rate restructuring.

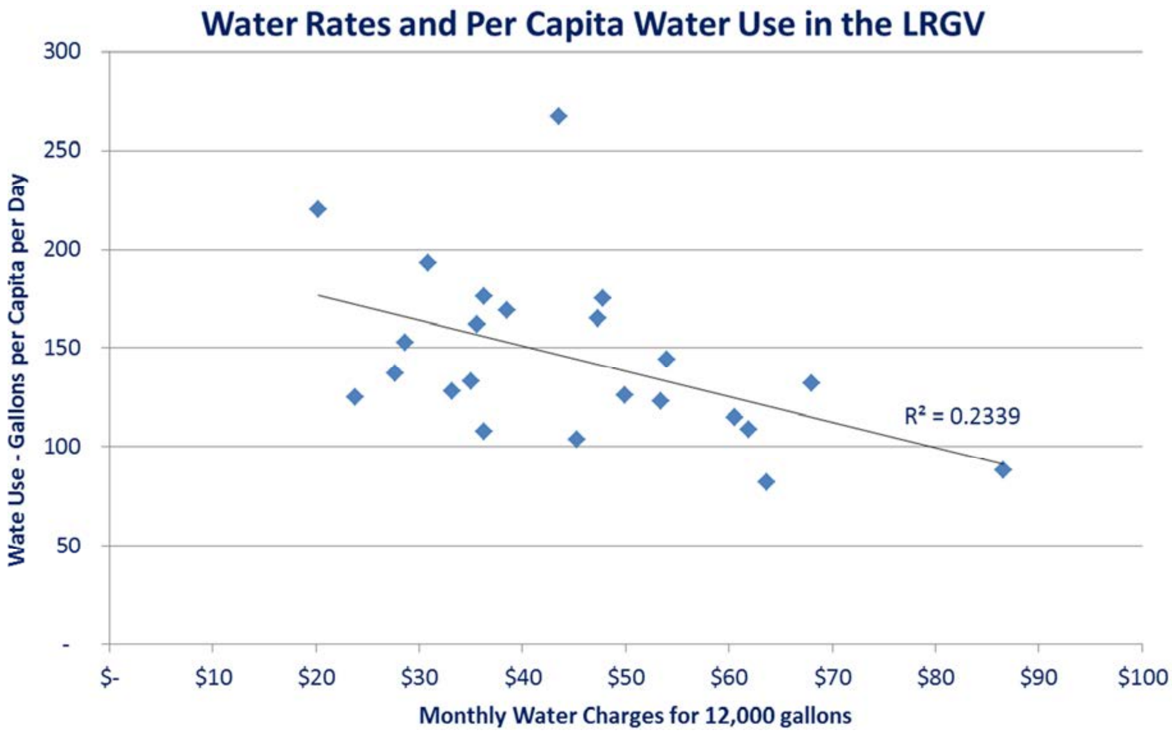


Figure 6-12 Relationship between Monthly Water Bills and Per Capita Water Use in the LRGV

6.1.7.4 Water System Audits

Traditionally, water conservation has focused on the end-user or customer and while this is still an important area of focus, it has become increasingly recognized that efficiencies, and utility cost savings, can be gained by focusing on the distribution and delivery of water to the customer. Additionally, the conservation message will be better received by the utilities' customers if the utility itself is engaging in and showing leadership by improving water supply efficiency.

The TWDB has requirements and guidance[10] for retail public utilities to perform a water loss audit that is based on the American Water Works Association (AWWA) M36 Methodology. The water loss audit provides a framework for gathering data, calculating performance measures, and reporting requirements under Texas Water Code Section 16.0121(b). Utilities implementing this Best Management Practice should use the methodology from the Texas Water Development Board manual. The American Water Works Association also offers products that can assist performing a water audit. They have published the M36 Manual, which can provide additional guidance on implementing this Best Management Practice, and offer free water loss audit software that allows utilities to quickly compile a preliminary water loss audit. As noted in Figure 6-9, water utilities in Region M showed the highest average ILI of all the planning regions. Although validation of the water audits to support those findings is required, it suggests there is significant potential for water savings in this area. Improving audit validation and real loss reduction strategies are included in the following sections.

[10] http://www.twdb.texas.gov/publications/brochures/conservation/doc/WaterLossManual_2008.pdf

6.1.7.4.1 Standardized Water Audit Approach

Water loss reduction strategies are best built upon calibrated and standardized models built on a foundation of accurate data. There are two kinds of audits that can be performed: a top-down water audit, and a bottom-up water audit, which primarily reflect the way data are gathered and derived.

The first step of the Top-Down Water Audit is to identify a group of stakeholders within the utility to aid with gathering the required data for a first look at the utility performance. Data is gathered and entered initially into a simple water balance model. The water balance model provides the level of detail for which data is currently available at this desktop analysis (top-down) level. Figure 6-13 shows the major components of the most current AWWA/IWA standard water balance model. As shown in Figure 6-13, the AWWA methodology improves on traditional approaches of measuring water loss by separating water losses into *Real Losses* and *Apparent Losses*.

Real Losses are the annual volumes lost through all types of leaks and breaks in water mains and service connections, up to the point of customer metering. Real losses also include overflows from treated water storage tanks or reservoirs.

Apparent Losses occur due to errors generated while collecting and storing customer usage data. The three categories of apparent losses include: Unauthorized Consumption, Customer Metering Inaccuracies, and Systematic Data Handling Errors.

This is an important distinction as these two categories of losses have different revenue implications for the water utility, with real losses having a more direct impact on water resources.

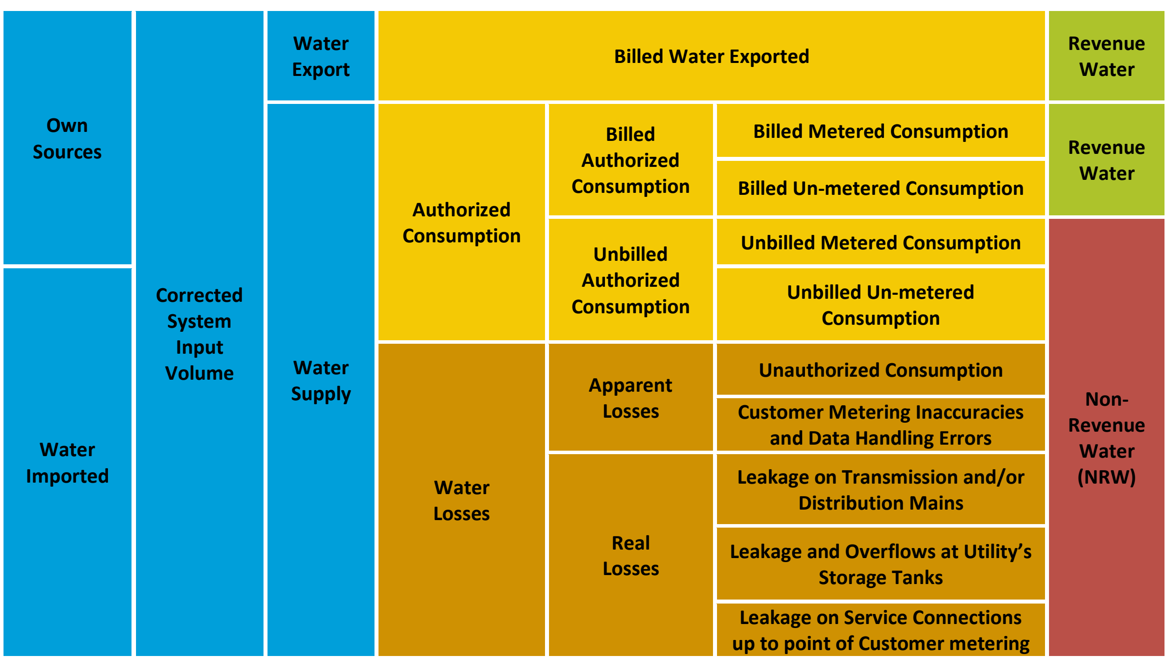


Figure 6-13 The Standard IWA Water Balance

Both the TWDB and AWWA water audits utilize a grading scheme to rate the confidence of each input audit components. Once the audit has been completed, a water audit data validity score will be generated. It is important to recognize the significance of the water audit data validity score and evaluate both the output metrics and the audit score together. For the initial audit generated by the utility, it is likely that some components of the required data are either not available or were originally derived from estimates or engineering judgments. During the top-down auditing process, these components are appropriately assigned a relatively low data grading score by reviewing a standardized *Grading Matrix* (incorporated within the AWWA software). Once an aggregate confidence level is obtained, the utility can identify the components that will have the largest impact on improving the aggregated confidence of either the apparent loss volume or the real loss volume. These input components are then typically prioritized for further verification.

It should be noted that it will likely require several years of conducting water audits to generate a high level of confidence in audit inputs. Once this level of confidence is reached, it is more realistic to base data-driven investment decisions on the water audit data and performance metrics. To generate this level of confidence in the data will require bottom-up activities and field studies that supplement the desk-top data used as entries into the audit spreadsheet.

One typical place to begin field validation is usually with the assessment of the accuracy of the supply meters. After investigation of the supply meters, the next step is an assessment of the accuracy of various categories of consumer meters. Consumer meter accuracy validation is usually done on statistically representative batches of meters; both these items are discussed in more detail below.

6.1.7.4.2 Production Meter Testing

One of the most critical measurements in the audit is the accurate measurement of water leaving the water treatment plant recorded through the production meters. Production master meters should be flow verified and calibrated annually at a minimum. It should be noted that there is an important distinction between 'flow verification' and 'calibration'. *Flow verification* is the act of confirming the accuracy of the primary metering device – the measuring element. Flow verification requires an independent measurement, typically by a second meter in series with the first, to provide comparative readings from which to quantify any discrepancy or error.

Calibration is the act of making modifications to the secondary electronic device – the output device where the flowmeter's measured values are converted and communicated. Typically this can be a differential pressure transducer or cell that converts the flowmeter measurement into a common electronic signal (i.e., 4-20 mA) used in the telemetry or SCADA system.

Both flow verification and calibration are vital in providing the highest degree of confidence in the water supplied volume within the water balance as this is perhaps the most important input value to the audit calculation[11].

6.1.7.4.3 Customer / Retail Meter Testing

Customer meters can be thought of as the "cash registers" for a utility. This means that it is critical for customer meters to be as accurate as possible to ensure that utilities capture (and then charge for) the water that a customer receives. Similarly, for the purposes of developing an accurate water balance and understanding of supply efficiency, customer meter accuracy is an important factor. Furthermore, getting an accurate picture of water use (and measuring the impact of water

[11] Georgia Water System Audits and Water Loss Control Manual, Georgia Department of Natural Resources, Jan. 2015

conservation) will depend on accurate customer metering. Due to these drivers, customer meters should be considered one of the most vital assets within the utilities' overall infrastructure and a robust program to monitor meter accuracy and repair and replace where necessary should be established. That said, attending to the accuracy of customer meters will not, *per se*, impact water conservation, but it will support accurate assessment and data driven decision making.

In order to assure water is being accounted for accurately, meters need to be selected, installed, operated and maintained using generally accepted industry standards. Meters should be regularly calibrated and tested in accordance with the manufacturer's recommendations or the guidelines issued by the American Water Works Association (AWWA), Manual for Water Meters-Selection, Installation, Testing, and Maintenance (AWWA M6).

Customer meters will range in size and it is important for a utility to keep accurate records of the number and age of meters in service and also the cumulative volume that has passed through the meter. This can help in prioritizing meter testing and selecting a representative sample of meters for testing. This information can also be used as a cross-check against actual consumption data to begin to look for data anomalies and outliers (e.g., meters likely approaching the end of their expected life, or incorrectly-sized meters for the type of account). As general guidance, The AWWA Manual M6 recommends that meters be tested in accordance with the following schedule:

- Retail meters of 6-inch and larger – Test every year
- Retail meters of 3-inch and 4-inch – Test every three years
- Retail meters of 2-inch and under - Test every ten years
- Fire Service/Detector Check meters, inspect check valve functioning, conduct testing on low flow meter only if above warranty volume

This is the protocol recommended by AWWA. However, current best practice for the small, residential meters would include testing a representative sample of the meters focusing on meters that have had the highest cumulative volume through them. This will enable the utility to develop a meter degradation curve that will allow them to improve estimates of apparent losses and make informed decisions about meter replacement. Additionally, it is suggested that each utility evaluates its billing data to determine the highest users within each category and tests these meters on a more frequent time step.

6.1.7.5 Tracking and Benchmarking Performance

Another reason that utilities should adopt the AWWA water audit methodology is that it generates more meaningful performance indicators than traditional water loss approaches and helps to identify areas where reductions in water use can be made. Real and Apparent losses are typically expressed in terms of gallons / connection / day (for rural systems real losses are expressed in gallons / mile of main /day). These are more reliable indicators than simplistic percentage approaches. An additional important indicator derived is the Infrastructure Leakage Index (ILI). The index is a ratio of actual real losses (as reported through the audit) compared to the theoretical lowest level of leakage (Unavoidable Annual Real Loss, or UARL). A calculated ILI value of 1.0 would indicate that a utility has reached a real loss level that reflects the successful application of today's best real loss control technology. As such, ILI values of 1.0 are rare within the industry and this level is often not economically achievable, unless water is very scarce, very expensive, or both. A significant advantage of the ILI approach is that it considers utility specific factors such as the number of connections, the average system pressure and the length of the customer service line, in the calculation of the system's UARL. As long as it is based on reliable data, the ILI can be a useful planning tool for benchmarking system performance. Validated water audit data has been published by AWWA on an annual basis for several years. The most recent published dataset is the2014 Water Audit Data Initiative[12] which contains audit information from 26 North American water utilities. Figure 6-14 shows performance of these utilities expressed as real losses per connection per day. Water utilities performing a water audit can benchmark their performance against this dataset.

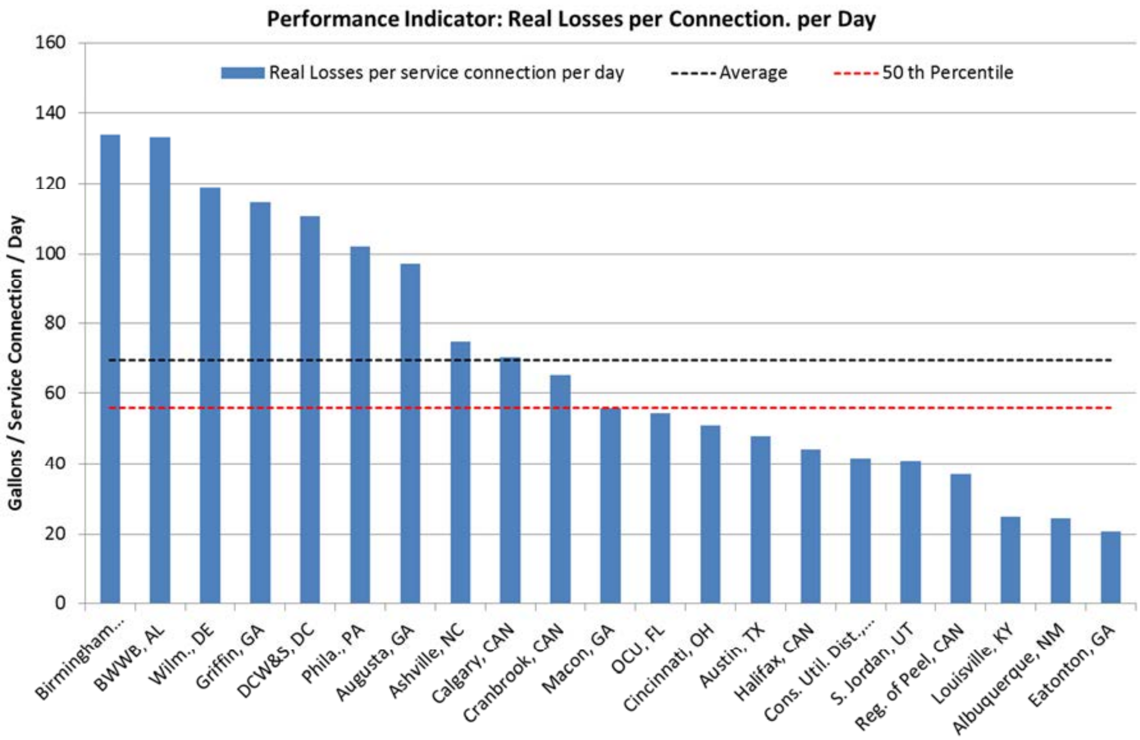


Figure 6-14 Published Water Audit Data (AWWA). Real Losses per Connection per Day

It is important to recognize that trying to achieve a water loss of zero isn't a practical or a realistic expectation. Understanding that water losses are broken down into two categories, real losses and apparent losses, is important and central to the water audit framework. Once confidence in the underlying data has reached a satisfactory level it is appropriate for the utility to develop strategies to control water losses as these are likely to be built on reliable data and will empower decision making. A review of water audit data published by AWWA, and evolving guidelines in Texas, provide three benchmark levels that a utility should consider to determine the priority for action on reducing real losses. These benchmark levels are show in Table 2

[12] AWWA 2014 Validated Water Audit Data http://www.awwa.org/resources-tools/water-knowledge/water-loss-control.aspx

Table 6-2 Benchmark Levels for Prioritizing Action on Real Losses

INDICATOR	SUGGESTED BENCHMARK LEVEL	SUPPORTING INFORMATION
Real Losses (gallons per connection per day)[1]	>50 gallons/conn. /day	55 gal/connection/day: median value from AWWA 2014 WADI dataset 50 gal/connection/day: proposed TWDB threshold for small systems
Real Losses (gallons per mile of main per day)[2]	>2,500 gallons/mile of main/day	2,634 gal/mile/day: median value from AWWA 2014 WADI dataset
Infrastructure Leakage Index (ILI)	>3.0	2.7: median ILI value from AWWA 2014 WADI dataset 3.0: proposed TWDB threshold

[1] Applicable to systems with a service connection density of greater than 32 connections /mile
[2] Applicable to systems with a service connection density of less than 32connections /mile

6.1.7.6 Developing a Water Loss Management Plan

A water loss management plan should recognize the different drivers behind real losses and apparent losses and also their financial and water resources implications. Once this is understood, the appropriate management strategies can be selected and implemented. For the purposes of developing an advanced water conservation plan, the focus here is on real loss management as reducing real losses directly benefits water resources.

A real loss management plan will encompass both the need for additional standardization and record keeping and an increased implementation of leakage detection surveying. It is recommended that an annual leak detection survey be completed by utilities that show real losses in excess of the benchmark values noted in 1.7.5. There are several types of leakage detection survey options that a utility should consider. Regardless of the type or scope of the water leak survey, it is important that the utility carefully record the leak report data in electronic format and begin tracking the water lines surveyed along with all leak data through the repair process. It is to be expected that that there are areas within the distribution network that are more susceptible to unreported leakage and as the program progresses, these suspect areas will be better defined and can be surveyed more frequently, thus making the leak detection survey more targeted, efficient and cost effective.

For larger utilities, the setting up of smaller zones to analyze demand and water loss variations more actively such as pressure zones, or District Metered Areas (DMAs) should be considered. DMA sizes will vary but typically may cover 1,000 – 3,000 connections. This will allow the distribution system to be discretized so that problem areas can be more easily identified and leak detection technologies applied with greater confidence.

Although apparent loss management is not a focus area for water conservation, the importance of the issue to a utility should not be overlooked. As retail water meters tend to deteriorate with age and use, resulting in under-registration of actual flow, this has two immediate negative impacts to the utility that may indirectly impact water conservation:

1. Utilities will lose revenue as not all the water delivered to the customer is registered through the meter unit; revenue that could be used to fund water conservation programs
2. If utilities mistakenly trust the data generated by under-registering customer meters they may erroneously conclude that end-users use less water than they actually get through their meter. Additionally, these losses may be assumed to be real loss (physical leakage from the distribution system) and a utility may mistakenly prioritize leak detection efforts when they should first focus on meter calibration and maintenance efforts.

Unauthorized consumption and systematic data handling errors are other areas within the water balance that may be addressed through a water loss plan. Although these two items are very different in their underlying causes, a review of billing data to identify trends and outliers may indicate potential accounts where these items are generating errors and impacting revenue. Although detailed analyses of billing data may require advanced data management and application of statistical techniques, it may be possible to do identify some issues by starting with a more simplified analysis.

6.1.7.7 Landscape Irrigation and Lawn Watering

Single and multi-family residential landscape irrigation and lawn watering are priority areas for focus, as reported by the TWDB Water Conservation Implementation Task Force. As noted in 1.8.2, customer surveys or sampling will help inform the utility on the extent of irrigation by utility customers. The TWDB has the following guidance specific to targeting resources towards high water users:

> If customers have automatic irrigation systems, a more detailed survey should include an evaluation of the schedule currently used and recommend any equipment repairs or changes to increase the efficiency of the irrigation system. The irrigation component of the single-family survey should target single-family customers using more than a certain amount of water per billing period that could be considered excessive for the particular geographic area and other characteristics of the service area. Typically, this is around 20,000 gallons per month in summer since that could represent an outdoor use of more than 12,000 gallons per month. Surveying outdoor water use in homes with water use below 20,000 gallons per month does not usually provide as significant an opportunity for water reductions. When conducting an onsite survey for a customer with an automatic irrigation system that is managed by an irrigation or maintenance contractor, it is beneficial to have the contractor present for the irrigation system survey (TWDB, 2013).

Studies by the California Department of Water Resources (DWR) have shown that landscape irrigation is frequently inefficient and, in some cases, a high percentage of residential landscape irrigation is wasted as a result of overwatering, poor design and poor maintenance[13] . Therefore, the survey of automatic irrigation systems should include a check of the entire system for broken, misdirected or misting heads and pipe or valve leaks. The customer's service line and meter box should also be checked for leaks. The system should be run to determine precipitation rates for typical zones. Each zone should be checked to be sure that rotors and spray heads are not on the

[13] California Department of Water Resources (DWR). 20x2020 Water Conservation Plan, February 2010

same zone since they have greatly different precipitation rates. Head spacing should be checked to determine if proper heads are installed. The schedule on the irrigation controller should be checked and the customer queried about how the schedule is adjusted during the year. A schedule should be provided based on evapotranspiration ("ETo")-based water-use budgets equal to no more than 80 percent of reference ETo per square foot of irrigated landscape. The statewide Texas Evapotranspiration Network (http://texaset.tamu.edu/) should be consulted for historical evapotranspiration data and methodology for calculating reference evapotranspiration and allowable stress. More aggressive landscape conservation programs can utilize stress coefficients lower than 80 percent. The customer should be provided a written report on the system repairs and equipment changes needed and the appropriate efficient irrigation schedule by month. The controller should be reset with the efficient schedule. If the system does not have a rain sensor, it should be installed as part of the survey if feasible or provided to the customer to be installed by a contractor. Information should be provided on the installation of dedicated landscape meters for multi-family customers if offered by the utility (TWDB, 2013).

There are many actions that may be taken to improve landscape water use efficiency. Professional landscape and irrigation design, proper installation, careful maintenance and management of the site and the selection of high quality irrigation equipment are some of the factors that can influence the efficient use of water in the landscape. Dedicated landscape meters, establishment of landscape water budgets and associated budget-based rate structures, the performance of irrigation audits, public information programs, technical training for landscape professionals, the use of alternative sources of water in the landscape, and a multitude of rebate programs to support conversion from lawns to water-smart plants and irrigation equipment are examples of actions that can be taken along with or in place of irrigation restrictions.

Irrigation restrictions can be useful in reducing water use, especially in the high demand summer months. In many areas, water use increases dramatically when customers start to irrigate their landscapes. Many utilities use irrigation restrictions during a prolonged drought or when available resources run low and are typically implemented through municipal ordinances. To increase the effectiveness of these programs, a set of enforcement actions may need to be developed, communicated to the public and implemented. An outreach program will be required to carefully communicate the necessity of water use restrictions and what end users should expect.

Voluntary elements of a comprehensive program should include (DWR, 2010):

- Widespread training programs for professional landscape maintenance contractors on water use efficiency, system maintenance and improvements
- Educational websites for consumers on landscape design, plant selection, irrigation system installation and repair
- Widespread installation of separate landscape meters for better information and water management
- More irrigation auditor training programs, and more irrigation audit programs provided by local water suppliers
- Better communication and coordination between water suppliers and local governments to ensure consistent policies and programs related to water use efficiency

- Support for rebate programs that fund improved landscape plantings, reduction of turf areas, upgrades to irrigation systems and controllers
- Use of public building landscapes as local examples of good design, installation, and maintenance
- Strong local and regional programs to encourage efficient new landscapes, replacement of older inefficient landscapes, and better management of high-water-using plantings such as turf

A suggested minimum specification list for qualifying as a "high-efficiency" home was developed for the US EPA study (DeOreo, 2011) and irrigation aspects are shown in Table 6-6.

Table 6-3 Suggested Minimum Specifications for High-Efficiency Homes (Irrigation)

FEATURE	PERFORMANCE REQUIREMENT	PERFORMANCE SPECIFICATION AND/OR REFERENCE
Water-wise landscape design and installation	Landscaped designed to require < 60% ETo overall (note, TWDB BMPs suggested 80% goal with more aggressive programs setting a lower target)	See landscape budget worksheet on www.aquacraft.com or use the GreenCo water budget calculator at www.greenco.org. Use budget tools to develop water budget for design landscape and compare this to budget for a reference landscape of cool season grass.
Smart irrigation controllers. Controller utilizes local data to adjust irrigation schedule automatically.	Devices with published SWAT testing results presumed acceptable; others on a case by case basis.	Based on SWAT performance criteria. https://www.irrigation.org/swat/control_climate/ This site lists testing criteria for both controllers and sensor based systems and provides performance reports for controllers that have passed the tests. Individuals may sign up for notices as new controller/sensor results are released.
Inspection of landscape and irrigation system by certified professional.	3rd-party field inspection/testing of landscape & irrigation system performance.	Independent party must verify that landscape was installed as designed, and that the irrigation system meets minimum performance standards. Inspector should be a certified professional.

Table adapted from: Analysis of Water Use in New Single Family Homes, Prepared by William B. DeOreo of Aquacraft Water Engineering & Management for The Salt Lake City Corporation and the U.S. Environmental Protection Agency, 2011.

6.1.7.8 Education and Public Awareness

Education and public awareness activities are covered by four separate BMPs under the TWDB BMPs for Municipal Water Providers. In addition to utility efforts to educate the public with water bill inserts and educational events promoting water conservation, it is suggested that a Customer Advisory Committee is developed, which includes representation from some of the City departments, and the spectrum of customer classes: residential, commercial, industrial, institutional, and irrigation. Allowing members of the community input into in the development of conservation planning goals and their implementation is likely to improve public acceptance and uptake of the proposed measures. Certain utilities provide educational programs for specific grades in schools within the three county area. These programs should be organized and developed with specific Rio Grande Valley geography, water resources and climate in mind.

In addition to standard education activities such as bill inserts, customer specific data will help target the education programs. For example, if summer peak usage is identified as an important planning issue then education programs should be repeated numerous times during the late spring and early summer, rather than being spaced even throughout the year[14]. The use of billboards to communicate a water conservation message will likely be an effective medium for raising public awareness.

6.1.7.9 Rebate, Retrofit and Incentive Programs

Passive water conservation savings are anticipated due to the replacement of older, less efficient fixtures, fittings and appliances due to changes in national and state plumbing codes. However, the exact schedule of replacement can only be estimated, therefore it is helpful to track the implementation of higher-efficiency products through customer surveys. This information can then be used to target rebate, retrofit and incentive programs for each water system in the LRGV.

As noted in 1.8.2, customer surveys are useful tools to establish a baseline of water efficiency products for each water system. The TWDB has the following guidance related to customer surveys:

> If the customer surveys are being performed by utility staff or an outside contractor there would be an opportunity to replace an inefficient showerhead and faucet aerators during the survey, if funding for such a program was available. A leak check should also be conducted to determine if there are any toilet leaks occurring and any dripping faucets. If 1.6 gallons per flush toilets have already been installed, the flush volume should be checked and, if needed, the water level in the tank should be adjusted to restore the flush volume to 1.6 gpf. If, after the water level in the tank is adjusted, the flush volume is still well above 1.6 gpf, it is likely that the toilet originally had an early closure flapper. Using the model number on the inside of the tank, the correct flapper to restore the 1.6 gpf can usually be determined. If the flapper is one of several early models of closure flappers, the flapper should be replaced during the survey and the information on the correct replacement flapper should be provided to the customer (TWDB, 2013).

Due to ongoing water conservation efforts, including the 1991 State Water Efficient Plumbing Act, it is likely that a high proportion of installed showerheads, faucet aerators, toilets and clothes washers will be of a water efficient design, and any remaining older inefficient products will have been projected to be replaced under the passive conservation scenario. Water efficient technologies continue to advance and it should be noted that the new benchmark for high-efficiency toilets (HETs) is 1.28gpf which is 20% more efficient than the previous benchmark Ultra-Low Flush Toilets (ULFTs), typically using 1.6gpf. In order to achieve more aggressive water conservation savings, or increase the pace of adoption of water efficient products it may be necessary for utilities to consider incentive programs to encourage the installation of water efficient fixtures, fittings and products. Surveys of individual utility customers will likely be needed to help determine what savings are still available.

[14] http://www.twdb.texas.gov/conservation/BMPs/Mun/doc/6.1.pdf

Clothes washing is a significant water use in the average home. Before new standards were adopted in 2010, traditional clothes washers used approximately 30 to 45 gallons per load. High-Efficiency Washers (HEW) significantly reduce this water use by more than 6,000 gallons per year for a typical family of four[15], with associated energy saving benefits.

A suggested minimum specification list for qualifying as a "high-efficiency" home was developed for the US EPA study (DeOreo, 2013). The specifications are very similar to the EPA WaterSense certification program (http://www.epa.gov/watersense/) and indoor features and end uses are shown in Table 6-6. WUGs can use the information provided in Table 6-4 to compare these benchmark high-efficiency standards to the currently installed fittings and appliances in the homes within their service areas. Based on the rates of adoption of the high-efficiency products, the WUGs can provide incentives to customers (subject to available funding) that will speed the adoption of these products and help achieve the advanced water conservation goals.

Table 6-4 Suggested Minimum Specifications for High-Efficiency Homes (Indoor Use)

FEATURE	PERFORMANCE REQUIREMENT	PERFORMANCE SPECIFICATION AND/OR REFERENCE
High-efficiency Toilet (HET)	1.28 gallons per flush (average)	EPA WaterSense HET spec http://www3.epa.gov/watersense/pubs/toilets.html
Faucet aerators	Bath: 1.5 gpm @ 60 psi Kitchen: 2.2 gpm @ 60 psi	Builder option
Low-flow showerheads	Single head using 1.6 gpm or less with "satisfactory" wetting performance	Builder option (e.g. Delta H2O Kinetics, Bricor, Niagara)
Horizontal-axis clothes washers	7.5 gallons, or less, per cubic foot of laundry capacity	Consortium for Energy Efficiency rating Tier 3A http://www.cee1.org/resid/seha/rwsh/reswash_specs.pdf
Energy Star dishwashers	6.5 gal/cycle or less	Energy Star rating: https://www.energystar.gov/products/appliances/dishwashers

Table adapted from: Analysis of Water Use in New Single Family Homes, Prepared by William B. DeOreo of Aquacraft Water Engineering & Management for The Salt Lake City Corporation and the U.S. Environmental Protection Agency, 2011

6.1.8 Prioritizing Advanced Water Conservation Options

The current status of water use and water conservation varies greatly between the 55 municipal water users in the LRGV. This reflects different socio-economic characteristics, water rates and end-user habits and means that a single set of conservation measures will not be applicable for all

[15] http://www.allianceforwaterefficiency.org/Residential_Clothes_Washer_Introduction.aspx

WUGs. However, the following flowchart (Figure 6-15) provides a framework that is applicable to all water users in the LRGV and can be a starting point for developing an advanced water conservation plan.

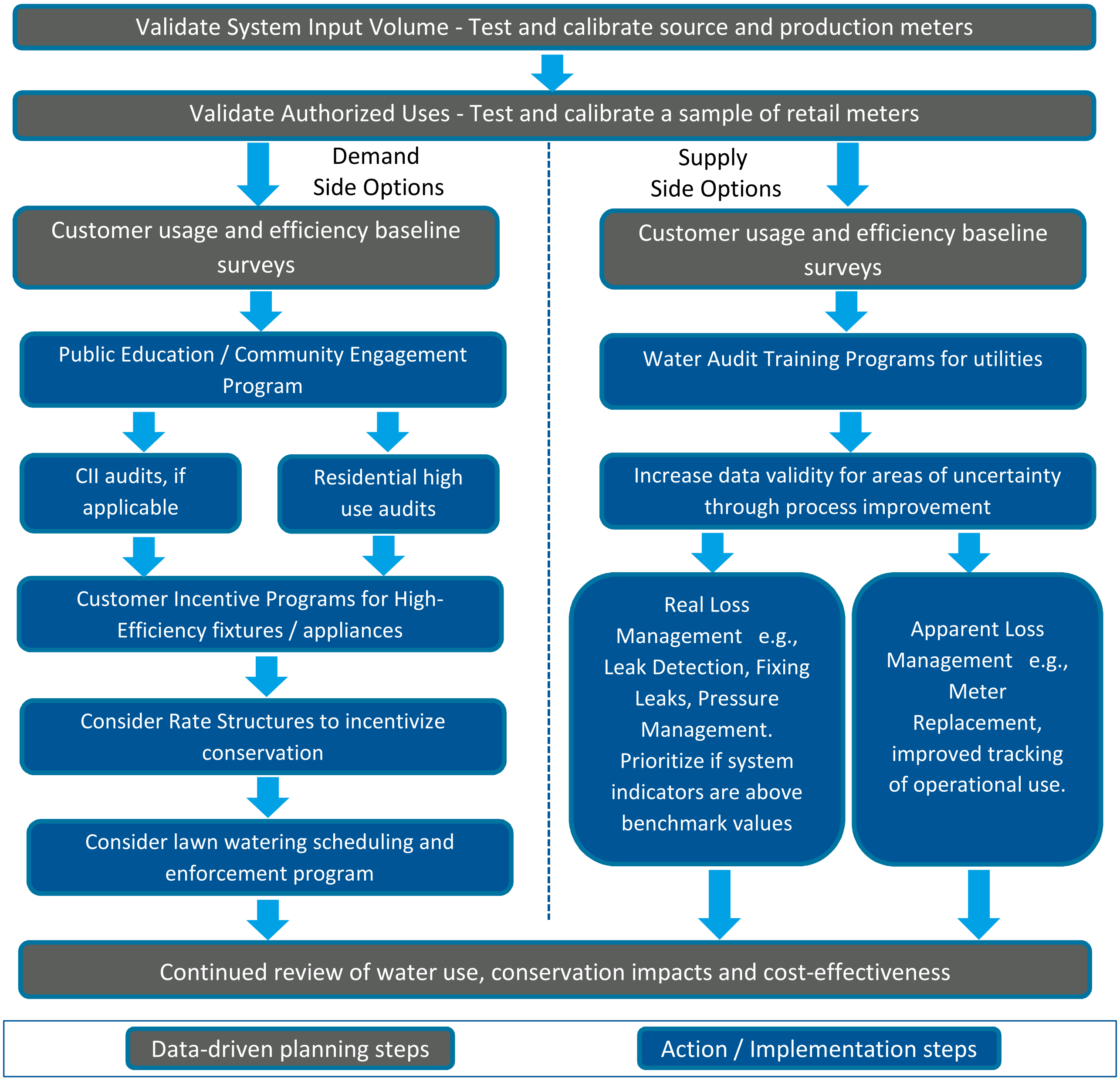


Figure 6-15 Steps to Develop an Advanced Water Conservation Plan.

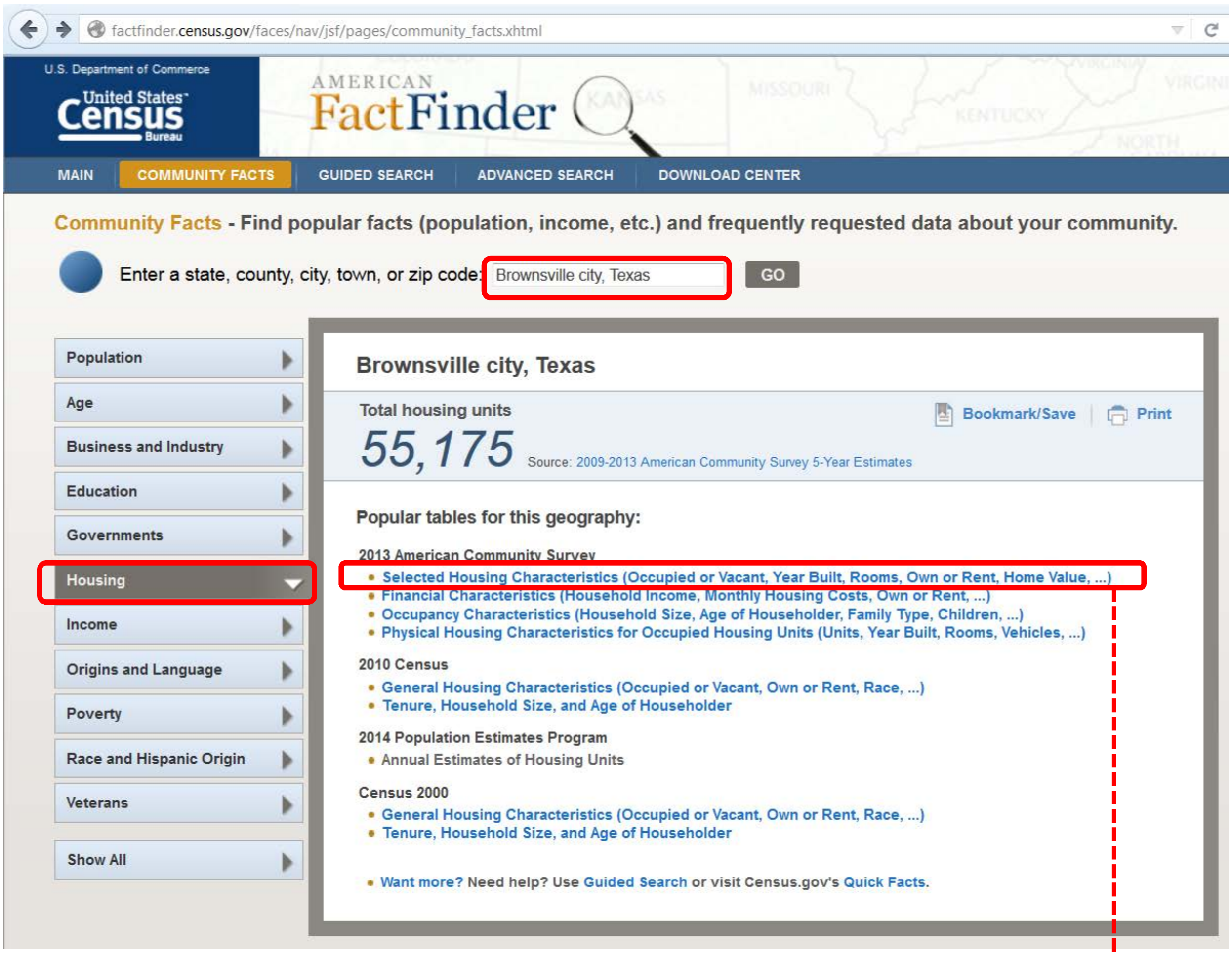
6.1.9 Conclusion

Based on the formula developed by the TWDB that estimates the potential impact of Advanced Water Conservation measures, the projected savings by 2070 (compared to passive conservation) are approximately 118,000 AF/yr. It should be recognized that these are estimates 50-years into the future and are therefore dependent on a number of different assumptions, including population projections, baseline data accuracy and the availability and willingness to supply funding for water conservation efforts over the timeframe. Future socio-economic developments will also influence the projected trends as many of the actions needed to support more efficient water use will require behavioral changes, which tend to be voluntary, as well as technological changes which can be mandated or incentivized.

There is no doubt that this level of water saving is ambitious and will require aggressive action from water systems and their customers. The scope of water savings will vary between the different systems and end users, but the areas of water conservation and efficiency identified in this chapter outline a roadmap for achieving the reduction. Based on the analysis of existing use in the LRGV, it appears that from a technical perspective a combination of demand-side efficiency measures and system efficiency or supply-side conservation (i.e. water loss control) improvements should allow the anticipated reductions to be achieved.

Appendix A – American Fact Finder

Figure 6-16 shows example data available from the American FactFinder website (www.factfinder.census.gov) that may provide useful information to a WUG to help in building the utility-specific profile and selection of relevant water conservation strategies.



YEAR STRUCTURE BUILT				
Total housing units	55,175	+/-664	55,175	(X)
Built 2010 or later	460	+/-160	0.8%	+/-0.3
Built 2000 to 2009	15,470	+/-572	28.0%	+/-1.0
Built 1990 to 1999	9,633	+/-539	17.5%	+/-0.9
Built 1980 to 1989	9,500	+/-579	17.2%	+/-1.0
Built 1970 to 1979	9,319	+/-581	16.9%	+/-1.0
Built 1960 to 1969	5,001	+/-402	9.1%	+/-0.7
Built 1950 to 1959	3,323	+/-341	6.0%	+/-0.6
Built 1940 to 1949	1,272	+/-228	2.3%	+/-0.4
Built 1939 or earlier	1,197	+/-180	2.2%	+/-0.3

Figure 6-16 Builidng a Utility Profile from the American FactFinder website

CHAPTER 7 – REGIONAL SUPPLY PHASING AND TRANSMISSION

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

23 DECEMBER 2015

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7.0 Regional Supply Phasing and Transmission

7.1 PURPOSE

The purpose of this chapter of the report is to describe the regional phasing of supplies along with the infrastructure required to transport the finished water from the water treatment facilities to the identified municipal water suppliers.

7.1.1 Summary

The proposed project includes pipelines and pump stations required to circulate water through the regional system and provides a mechanism for RGRWA to meet the future demands of user groups in the area.

Sections in this chapter include:

- Regional Supply Phasing
- Pipe Material Evaluation
- Pipeline Routing
- Pipeline Sizing
- Pipeline Hydraulics
- Cost Opinion

7.2 REGIONAL SUPPLY PHASING

In order to meet the regional demands as determined in Chapter 3, regional water supplies are phased based on their limitations, location and assumed costs for treatment. Table 7-1 provides the demands to be met by each supply per phase. It is assumed that the continual use of raw surface water from the Rio Grande will continue to be the lowest cost to implement because of the low cost of treatment and its proximity to the major population centers. However, due to the surface water right conversion limitation the Regional SWTP would not be online until 2030. Likewise, Direct Potable Reuse supplies will become available as WWTPs in the region increase in their capacities. Though seawater desalination is estimated to be the highest cost to produce, its proximity to the demands near the coast, along with potential funding incentives make it cost competitive with other sources. For discussion on limitations on each source please refer to the corresponding resource supply chapter as appropriate. Supply limits are listed below:

- Seawater supply is unlimited.
- Surface Water is limited by annual water rights. A conversion of agricultural to municipal water rights is projected for each decade.
- Direct Potable Reuse water is limited by wastewater flows collected. A projection of wastewater flows and DPR water produced is estimated for each decade starting in 2040.
- Brackish Groundwater is limited by the desired future conditions of the aquifers in each county. Additional Cameron County pumping is limited to 38,000 afy and Hidalgo pumping is limited to

12,000 afy of raw water. Estimated recoveries from the proposed plants estimates 28,500 afy and 9,000 afy of produced municipal supply for the two counties.

Table 7-1: Demands to be Met by Water Resource in acre feet per year (afy)

SUPPLY		2020	2030	2040	2050	2060	2070
Target Demand		**25,500**	**55,750**	**88,100**	**136,250**	**184,800**	**232,000**
Seawater Desalination		5,470	5,470	5,470	21,950	51,500	79,500
Surface Water Treatment Plant	Water Rights	0	12,780	28,003	38,431	48,132	57,607
	Direct Potable Reuse	0	0	17,127	38,369	47,668	57,393
Hidalgo Brackish Groundwater Desalination		0	9,000	9,000	9,000	9,000	9,000
Cameron Brackish Groundwater Desalination		20,000	28,500	28,500	28,500	28,500	28,500

System design capacities are provided with a 1.3 peaking factor to provide operational flexibility to meet seasonal variations in pumping requirements. Also, treatment production capacities are adjusted based on infrastructure sizing. For example, the Seawater Desalination Plant provides additional peaking capacity in the decade 2020 to allow for three 9 MGD treatment trains at the Cameron BGD Plant. Table 7-2 provides the system capacities for each water treatment plant.

Table 7-2: Design System Capacities (MGD)

SUPPLY		2020	2030	2040	2050	2060	2070
Target Demand (AFY)		**25,500**	**55,750**	**88,100**	**136,250**	**184,800**	**232,000**
Minimum System Capacity		**22.8**	**49.8**	**78.7**	**121.7**	**165.0**	**207.1**
Required System Capacity with 1.3 Peaking Factor		**29.6**	**64.7**	**102.3**	**158.1**	**214.5**	**269.3**
Seawater Desalination		10.0	10.0	10.0	20.0	40.0	80.0
Surface Water Treatment Plant	Water Rights	2.0*	20.0	40.0	60.0	70.0	80.0
	Direct Potable Reuse	0.0	0.0	20.0	50.0	70.0	80.0
	Total	2.0	20.0	60.0	110.0	140.0	160.0
Hidalgo Brackish Groundwater Desalination		0.0	8.0	8.0	8.0	8.0	8.0

SUPPLY	2020	2030	2040	2050	2060	2070
Cameron Brackish Groundwater Desalination	18.0	27.0	27.0	27.0	27.0	27.0
Total System Capacity	**30.0**	**65.0**	**105.0**	**165.0**	**215.0**	**275.0**

*Assume first 2 MGD of WR and SWTP Capacity is supplied through contract water into the system from existing WTPs.

7.3 REGIONAL TRANSMISSION MAIN

Regional transmission main and pumping stations will be required to transport the treated water from their sources to the recipients. This will require more than 100 miles of pipe of varying diameter pumping water from elevations as low as sea level up to a ground surface elevation of approximately 130 feet. The line will have a minimum design pressure of 85 psi to meet individual system needs and will include turn offs with valving and metering to provide water service on-demand to project partners.

7.4 PIPE MATERIAL EVALUATION

7.4.1 Soils

The Lower Rio Grande Valley has a diverse soil composition as is visible in The Hidalgo and Cameron county soils maps in Figure 7-1 and Figure 7-2. The Figures show the areas of highly corrosive soils in orange and moderately corrosive soils in yellow. The entire length of the pipeline will run through soil that is at least moderately corrosive.

Due to the variability of the soil along the pipeline route, it is preferred to use a pipe material that is naturally corrosion resistant (PVC, or HDPE) or make provisions for cathodic protection while installing the pipe.

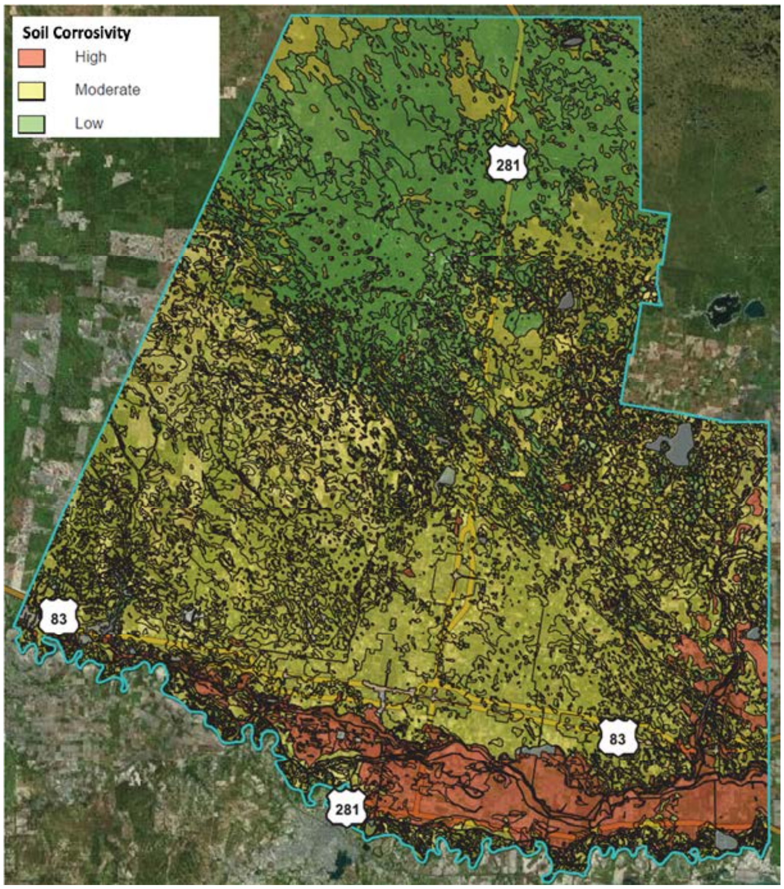


Figure 7-1: Hidalgo County Soil Corrosivity Map (NRCS)

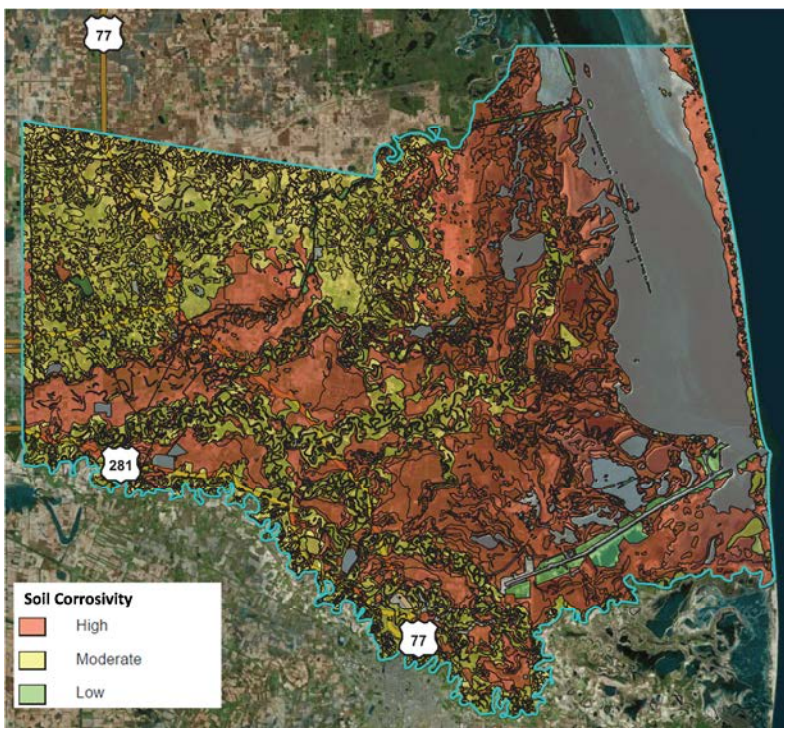


Figure 7-2: Cameron County Soil Corrosivity Map (NRCS)

7.4.2 Pipe Material Options

In order to determine the best material to recommend for the RGRWA transmission main, the following pipe materials were evaluated:

- Bar-Wrapped Steel Cylinder Concrete Pressure Pipe (BWCCP) (ANSI/AWWA C303)
- Pre-Stressed Concrete Cylinder Pipe (PCCP) (ANSI/AWWA C301)
- Ductile Iron Pipe (DIP) (ANSI/AWWA C151)
- Steel Pipe (SP)(ANSI/AWWA C200)
- High Density Polyethylene (HDPE) (ANSI/AWWA C906)
- Polyvinyl Chloride (PVC) (ANSI/AWWA C905)

Each pipe material is evaluated considering the following design criteria: history and availability, linings and coatings, joint types, fittings and appurtenances, corrosion protection, opinion of probable pipe cost, and additional considerations.

7.4.3 Assumptions

The water transmission main diameter varies throughout the project. The pipeline hydraulics between the water treatment plants and the transmission main's termination point will be evaluated during design to determine the working and test pressures along the pipeline and the required pumping head at the water treatment plant. For this evaluation, it is assumed that:

(a) the maximum pressures required for this transmission main are 250 psi working pressure, and 300 psi test pressure,
(b) the cover is 5 to 15 feet,
(c) the soil is assumed to be moderately corrosive, and
(d) the intention is to use the same pipe material for the entire length of the transmission main project.

All pipe materials require compacted granular embedment to the centerline of the pipe to provide sidewall support for vertical loads, particularly for pipe greater than 42-inch diameter.

7.4.4 Bar Wrapped Steel Cylinder Concrete Pressure Pipe (BWCCP) (ANSI/AWWA C303)

History and Availability

Concrete pressure pipe, conforming to ANSI/AWWA C303, is a semi-rigid pipe consisting of a steel cylinder that is helically wrapped with a mild steel bar reinforcement. The ANSI/AWWA C303 standard covers pipe to 72-inch diameter and working pressures to 400 psi, which is greater than the pressure requirements anticipated for this project. Concrete pressure pipe has been installed since the 1950's.

Forterra (formerly Hanson) Pipe has historically been the only pipe manufacturer to furnish concrete pressure pipe in this area. Forterra Pipe has three pipe manufacturing plants in Texas: Victoria, Grand Prairie, and Lubbock.

Linings and Coatings

Concrete pipe is provided with a shop applied mortar coating at least 3/4-inches thick over the bar reinforcement. The bar wrapping reinforces the mortar coating and locks it tightly against the steel cylinder so that cylinder, bar, and coating act as a composite structure. The bar wrapping also reduces the potential of the pipe swelling under high pressures, which reduces the potential for the cement mortar coating to spall or crack. Concrete pipe is also provided with protective shop applied cement slurry lining that is centrifugally cast on the inside of the steel cylinder.

Joint Types

The types and number of joints in the pipeline affect the construction costs and schedule. The standard length of a concrete pipe section varies between 24 and 40 feet which is dependant upon pipe size.

For non-restrained sections of the pipeline, rubber-gasketed bell and spigot (stab) type joints may be used. Carnegie joints are a type of stab joint; the Carnegie joint has a weld-on bell and spigot, with a notch in the spigot that houses the gasket. This type of joint allows for some deflection and flexibility in the joint. Forterra Pipe normally uses Carnegie joints as their standard joint for concrete pipe.

For restrained sections of the pipeline, welded joints, harnessed coupled joints, or concrete thrust blocks are normally used. Welded joints would be less costly but would provide more secure thrust restraint at higher working pressures. Harnessed coupled joints are normally used only at locations to install pipe assemblies, closures, and where future disassembly of the pipe is required. Concrete thrust blocks would be very large for the pressures anticipated and may not be practical. Therefore, welded joints are recommended for the normal thrust restraining method. Welded joints can be fillet welded either internally, externally, or both. For 60-inch pipe, the welds can easily be performed internally due to the large pipe diameter. Both internal and external welded joints may be required at the highest thrust joints. The weld types and locations will be determined during design.

After jointing of pipes, the interior joint recess is filled with cement mortar and the exterior joint space is grouted by use of a wrapper (diaper) strapped around the pipe and over the joint. For harnessed coupled joints, the space between each end of the mortar coated pipe is concrete encased or covered with a heat shrinkable coating (shrink sleeves).

Fittings and Appurtenances

Fittings such as tees and elbows are fabricated from welded steel sheets or plates and lined and coated with cement mortar similar to the pipe, with steel bell and spigot joint rings welded to the ends of the fitting. At high pressures and thrust restraints, the fittings tend to be reinforced with full saddle wrappers or crotch plates during fabrication. Specific pressure limits for welded collars are determined during the pipeline design. The fittings are manufactured in accordance with ANSI/AWWA C303.

Small outlets for appurtenances such as blowoffs and air release valves are normally shop fabricated and welded to the pipe's steel cylinder or fitting's steel sheet. Outlets can be a shop fabricated welded pipe section with a flange, mechanical joint, or plain end. Outlets smaller than 4-inch are usually made with factory welded fittings with threaded or welded connections.

Pipe connections to in-line flanged valves or flanged appurtenances are typically made with ANSI/AWWA C207 flanges. Class E flanges, for sizes up to 144 inch diameter, are rated for total pressures up to 275 psi. Class F flanges, for sizes up to 48 inch diameter, are rated at 300 psi, the highest pressure rating meeting ANSI/AWWA C207.

Corrosion Protection

Properly manufactured and installed concrete pressure pipe is corrosion resistant in most soils, except soils with high levels of sulfates or chlorides. These high sulfate and chloride soils can cause degradation of the mortar coating and eventually corrosion of steel components of the pipeline. If the mortar coating is not sound, corrosive soils can directly attack the steel components. More extensive soil information will be obtained during the geotechnical investigations. If more than the normal mortar coating is required to control corrosion for concrete pressure pipe other methods include the following:

- Use special Type II cement in the mortar pipe coating.
- Electrical isolation from dissimilar piping.
- Polyethylene encasement in areas subject to stray current pickup.
- Electrical bonding of all joints and field test stations with stationary reference electrodes for corrosion monitoring.
- Cathodic protection system consisting of either galvanic anodes or an impressed current system.

The corrosion protection system will be evaluated during final design to determine the best method of corrosion control.

Additional Considerations

Concrete pipe is a semi-rigid pipe designed specifically for the internal pressure and external load. The pipe design allows for adjustments of the pipe's structural strength, through varying the wall thickness, concrete strength, and the amount and shape of the reinforcing steel. Therefore, accommodating the assumed pressure ranges and cover depths is not anticipated to be a problem.

Concrete pipe can be field repaired; however, since concrete pipe is a composite structure and designed specifically for a project, it is more difficult to readily repair particularly for replacing a pipe section with the same pipe design. Depending on type, size, and location of damage, pipe barrel repairs are made using a weld repair plate, gasket clamp, weld on saddle, or pipe replacement with a steel pipe section and sleeve or butt strap closure.

7.4.5 Pre-Stressed Concrete Cylinder Pipe (PCCP) (ANSI/AWWA C301)

History and Availability

Pre-Stressed Concrete Cylinder Pipe (PCCP), conforming to ANSI/AWWA C301, is a semi-rigid pipe consisting of a thin steel cylinder that is helically wrapped with pre-tensioned steel wire reinforcement. The ANSI/AWWA C301 standard covers pipe to 144-inch diameter and working pressures over 400 psi.

Forterra (formerly Hanson) pipe has historically been the only pipe manufacturer to furnish concrete pressure pipe in this area. Forterra has three pipe manufacturing plants in Texas: Victoria, Grand Prairie, and Lubbock.

Linings and Coatings

Concrete pipe is provided with a shop applied mortar coating at least 3/4-inches thick over the pre-tensioned steel wire reinforcement. The wire wrapping compresses the concrete and reduces the internal force experienced by the pipe at the design working pressure. The wire wrapping also reduces the potential of the pipe swelling under high pressures, which reduces the potential for the cement mortar coating to spall or crack. Concrete pipe is also provided with protective shop applied cement slurry lining that is centrifugally cast on the inside of the steel cylinder.

Unlike BWCP, maximum depth of cover does not vary with pipe diameter. External loads (live and dead), internal pressures (working and surge), diameter, and other parameters are all components of the pipe design.

Joint Types

The types and number of joints in the pipeline affect the construction costs and schedule. The standard length of a concrete pipe section varies between 16 and 24 feet which is dependant upon pipe diameter.

For non-restrained sections of the pipeline, rubber-gasketed bell and spigot (stab) type joints may be used. Carnegie joints are a type of stab joint; the Carnegie joint has a weld-on bell and spigot, with a notch in the spigot that houses the gasket. This type of joint allows for some deflection and flexibility in the joint. Forterra Pipe normally uses Carnegie joints as their standard joint for concrete pipe.

For restrained sections of the pipeline, welded joints, harnessed coupled joints, or concrete thrust blocks are normally used. Welded joints would be less costly but would provide more secure thrust restraint at higher working pressures. Harnessed coupled joints are normally used only at locations to install pipe assemblies, closures, and where future disassembly of the pipe is required. Concrete thrust blocks would be very large for the pressures anticipated and may not be practical. Therefore, welded joints are recommended for the normal thrust restraining method. Welded joints can be fillet welded either internally, externally, or both. For 60-inch pipe and larger, the welds can be performed internally due to the large pipe diameter. Both internal and external welded joints may be required at the highest thrust joints. The weld types and locations will be determined during design.

After joining pipes, the interior joint recess is filled with cement mortar and the exterior joint space is grouted by use of a wrapper (diaper) strapped around the pipe and over the joint. For harnessed coupled joints, the space between each end of the mortar coated pipe is concrete encased or covered with a heat shrinkable coating (shrink sleeves).

Fittings and Appurtenances

Fittings such as tees and elbows are fabricated from welded steel sheets or plates and lined and coated with cement mortar similar to the pipe, with steel bell and spigot joint rings welded to the ends of the fitting. Specific pressure limits for welded collars are determined during the pipeline design. The fittings are manufactured in accordance with ANSI/AWWA C301.

Small outlets for appurtenances such as blowoffs and air release valves are normally shop fabricated and welded to the pipe's steel cylinder or fitting's steel sheet. Outlets can be a shop fabricated weld pipe section with a flange, mechanical joint, or plain end. Outlets smaller than 4-inch are usually made with threaded or welded connections.

Pipe connections to in-line flanged valves or flanged appurtenances are typically made with ANSI/AWWA C207 flanges. Class E flanges, for sizes up to 144 inch diameter, are rated for total pressures up to 275 psi. Class F flanges, for sizes up to 48 inch diameter, are rated at 300 psi, the highest pressure rating meeting ANSI/AWWA C207.

Corrosion Protection

Properly manufactured and installed concrete pressure pipe is corrosion resistant in most soils, except soils with high levels of sulfates or chlorides. These high sulfate and chloride soils can cause degradation of the mortar coating and eventually corrosion of steel components of the pipeline. If the mortar coating is not sound, corrosive soils can directly attack the steel components. More extensive soil information will be obtained during the geotechnical investigations. If more than the normal mortar coating is required to control corrosion for concrete pressure pipe other methods include the following:

- Use special Type II cement in the mortar pipe coating.
- Electrical isolation from dissimilar piping.
- Polyethylene encasement in areas subject to stray current pickup.
- Electrical bonding of all joints and field test stations with stationary reference electrodes for corrosion monitoring.
- Cathodic protection system consisting of either galvanic anodes or an impressed current system.

The corrosion protection system will be evaluated during design to determine the best method of corrosion control.

Additional Considerations

Concrete pipe is a semi-rigid pipe designed specifically for the internal pressure and external load. The pipe design allows for adjustments of the pipe's structural strength, through varying the wall thickness, concrete strength, and the amount and shape of the reinforcing wire. Therefore, accommodating the assumed pressure ranges and cover depths is not anticipated to be a problem.

PCCP cannot be easily field repaired and if the pre-tensioned wire is exposed, a specialty contractor is needed to make the pipe repair.

7.4.6 Ductile Iron Pipe (DIP) (ANSI/AWWA C151)

History and Availability

Ductile iron pipe, conforming to ASNI/AWWA C151, is a produced from molten iron centrifugally cast in a steel mold. The ANSI/AWWA standard covers pipe to 64-inch diameter and working and test pressures equal to the pressure requirements anticipated for this project. Ductile iron pipe has been utilized for the past 50 years; however its predecessor, cast iron pipe, has been utilized for many more years. Ductile iron pipe has proven durability, strength, and reliability.

American Ductile Iron Pipe Company and U.S. Pipe are two pipe manufacturers that could furnish ductile iron pipe in this area. American Ductile Iron Pipe Company has a sales office in Dallas, Texas and a pipe manufacturing plant in Birmingham, Alabama. U. S. Pipe has pipe manufacturing plant in Birmingham. Alabama.

Linings and Coatings

Ductile iron pipe is provided with a standard shop-applied asphaltic coating and a cement mortar lining for water pipelines.

Joint Types

The types and number of joints in the pipeline affect the construction costs and schedule. The standard length of a ductile iron pipe section is 18 to 20 feet.

For non-restrained sections of the pipeline, rubber-gasketed bell and spigot push-on type joints are used and easy to install. For restrained sections of the pipeline to resist hydraulic thrust, pipe manufacturer's standard fabricated restrained push-on joints or concrete thrust blocks can be used. The use of other type of restrained joints such as Megalugs is not recommended for 60-inch pipe. For larger diameter pipes and higher pressures, concrete thrust blocks usually become too large for practical use. Therefore, restrained joints are recommended for the thrust restraining method.

After jointing of pipes, the interior and exterior joints do not need grouting or additional protection. However, it is recommended that a loose polyethylene encasement be field installed around the pipe, which would include an exterior joint.

Fittings and Appurtenances

Fittings such as tees and elbows are fabricated from the manufacturer's standard fitting molds with non-restrained and restrained joints similar to the pipe. The fittings are cement mortar lined and asphaltic coated similar to the pipe. The fittings are manufactured in accordance with ANSI/AWWA C110.

Small outlets for appurtenances such as blow-offs and air release valves are normally accommodated through fittings, shop fabricated welded-on pipe outlets, or fully wrapped tapping saddles. Ends for the outlets can be flange, mechanical joint, or plain end. Outlets smaller than 4-inch can be made with factory welded fittings with threaded or welded connections.

According to AWWA M41, the rated working pressure of standard fittings for ductile iron pipe is dependent on the pipe wall thickness, fitting size, material, and configuration, while the rated working pressure on flange fittings depends on the flange. The dimensions in ANSI/AWWA C110/A21.10 show the necessary requirements for flanges used in services of 250 or 350 psi. The rating for flanged fittings can sometimes differ as well depending on the material or wall thickness. In addition, special gaskets may be used for certain ductile-iron fittings that are 24 in. or smaller so they can be rated for 350 psi.

Rubber-gasket joints, both mechanical and push-on type, are covered in ANSI/AWWA C111/A21.11. The standard specifies that the joints shall have the same pressure rating as the pipe or fittings of which they are a part.

Flanged joints for ductile iron pipe are covered in ANSI/AWWA C115/A21.15. The flanged joints included in this standard are threaded flanges and are rated 250 psi working pressure for all sizes. The minimum class thickness for pipe barrels with threaded flanges is Class 53 for all pipe sizes.

Fittings for ductile iron pipe are covered in ANSI/AWWA C110 and ANSI/AWWA C153/A21.53. Generally, mechanical joint and push-on joint fittings 12 inch and smaller are rated at 250 psi or 350 psi; 14 inch through 24 inch are rated 150 psi, 250 psi, or 350 psi; 30 inch through 48 inch are rated 150 psi or 250 psi working pressure; and 54 inch and larger are rated 150 psi working pressure. Fittings with greater pressure ratings are available; if needed, the manufacturer should be consulted. Flanged fittings 12 inch and smaller are rated 250 psi and 14 inch and larger fittings are rated 150 psi or 250 psi working pressure.

Corrosion Protection

The minimum thickness for ductile iron pipe is designed with a corrosion allowance. In addition, the asphaltic coating provides external corrosion protection. It is recommended that a loose polyethylene encasement be field installed around the pipe in accordance with ANSI/AWWA C105 to provide additional corrosion protection. Cathodic protection can be provided where additional corrosion control is needed. More extensive soil information will be obtained during the geotechnical investigations and used to determine potential corrosion issues.

The corrosion protection system will be evaluated during final design to determine if polyethylene encasement is sufficient or additional cathodic protection will be required.

Additional Considerations

Ductile iron pipe is manufactured in set pressure classes requiring the next pressure class above the design conditions for the internal pressures be selected. Ductile iron pipe can accommodate the assumed pressure ranges and cover depths.

Ductile iron pipe can be field repaired with standard ductile iron pipe and fittings that are readily available from the pipe manufacturer. Depending on type, size, and location of damage, pipe barrel repairs are made using repair clamp or replacing a pipe section with sleeves.

7.4.7 Steel Pipe (SP) (ANSI/AWWA C200)

History and Availability

Steel pipe, conforming to ANSI/AWWA C200, is a flexible pipe consisting of a steel cylinder. The ANSI/AWWA Standard covers pipe to 144-inch diameter and working pressures to 500 psi which is greater than the pressure requirements anticipated for this project. Steel pipe has an expected life of more than 100 years when properly lined and coated. Steel pipe is known for its desirable qualities such as durability, strength, economy and reliability.

Northwest Pipe, American SpiralWeld Pipe, and Forterra Pipe are three pipe manufacturers that could furnish steel pipe in this area. Northwest Pipe has a pipe manufacturing plant in Saginaw, Texas, American SpiralWeld has a pipe manufacturing plant in Columbia, South Carolina, and Forterra Pipe has a pipe manufacturing plant in Grand Prairie, Texas.

Linings and Coatings

Typical coatings for steel pipe have been cement mortar, tape wrapping, or polyurethane.

Cement mortar coating is applied in the shop by pneumatic or mechanical placement methods. The cement mortar coating is applied to a thickness of at least 3/4-inches in conformance with ANSI/AWWA C205.

Tape wrapped coating is also applied in the shop, under tension to maintain a wrinkle-free coating as described in ANSI/AWWA C214. Cold-applied tapes are usually applied as a four-layer system with a total thickness of at least 80 mils. A primer layer is covered with a filler tape which is made up of a butyl rubber compound compatible with the primer and tape which is then covered by weld stripping tape (25 mils), if required. The inner layer (20 mils), middle layer (30 mils) and outer layer (30 mils) provide corrosion protection, mechanical protection, and mechanical protection with ultraviolet light stabilizers, respectively. Tape wrapped coating requires a clean blasted surface for proper curing and adhesion.

Polyurethane coating is applied by spraying within a controlled temperature range. Minimum thickness of polyurethane is 25 mils or as recommended by the manufacturer for compliance with ANSI/AWWA C222. Polyurethane coating requires a clean blasted surface for proper curing and adhesion.

Cement mortar lining is normally applied in the shop by a spinning machine to centrifugally apply the cement-mortar. For 60-inch pipe, the cement mortar lining is applied to a thickness of 1/2-inch in conformance with ANSI/AWWA C205. Epoxy and polyurethane linings are typically more expensive than cement mortar lining.

Joint Types

The types and number of joints in the pipeline affect the construction costs and schedule. The length of a standard section of steel pipe varies between 40 and 50 feet, subject to availability and any transportation restrictions.

For non-restrained sections of the pipeline, welded joints, bolted sleeve-type couplings, flanges, grooved or shoulder couplings, and bell and spigot with rubber gaskets can all be used according to AWWA M11. The bell and spigot with a rubber gasket and the welded joints are the most common types used. Since flanges are not typically buried, they are mostly used for mating to inline valves in a vault.

For restrained sections of the pipeline, welded joints, harnessed coupled joints, or concrete thrust blocks are normally used. Welded joints would be less costly and would provide more secure thrust restraint at higher working pressures. Harnessed coupled joints are normally used only at locations to install pipe assemblies, closures, and where future disassembly of the pipe is required. Concrete thrust blocks would be very large for the pressures anticipated and may not be practical. Therefore, welded joints are recommended for the normal thrust restraining method. Several types of welded steel joints are available. Lap welds are generally considered the most economical and are recommended. The lap welded joints can be fillet welded either internally, externally, or both. For 60-inch pipe, the welds can easily be performed internally due to the large pipe diameter. Both internal and external welded joints may be required at the highest thrust joints. Butt strap welded joints provide additional restraint and are often used to install pipe assemblies, closures, or at locations where the pipe thickness changes. Butt strap joints should be considered for pipe closure assemblies and at changes of the pipe thickness in high-pressure regions. Locations for internal, external, or both lap welds will be determined during design.

After jointing of pipes, the interior joint recess is filled with cement mortar and the exterior joint space between the tape coated hold backs is covered with a heat shrinkable coating (shrink sleeve).

Fittings and Appurtenances

Fittings such as tees and elbows are fabricated from welded steel sheets or plates and lined and coated similar to the pipe, and with steel lap joints welded to the ends of the fitting. At high pressures and thrust restraints, the fittings may tend to be reinforced with full saddle wrappers or crotch plates during fabrication. Specific pressure limits for welded collars are determined during the pipeline design. The fittings are manufactured in accordance with ANSI/AWWA C200.

Small outlets for appurtenances such as blow-offs and air release valves are normally shop-fabricated and welded to the pipe's steel cylinder or fitting's steel sheet. Outlets can be a shop fabricated weld pipe section with a flange, mechanical joint, or plain end. Outlets smaller than 4-inch are made with with threaded or welded connections.

Pipe connections to in-line flanged valves or flanged appurtenances are typically made with ANSI/AWWA C207 flanges. Class D steel flanges, for sizes up to 12 inch diameter, are rated for working up to 175 psi and a test pressure up to 262.5 psi. For sizes greater than 12 inch to 144 inch diameter, flanges are rated for working pressures up to 150 psi and a test pressure up to 225 psi. Class E flanges, for sizes to 144 inch diameter, are rated for working pressure up to 275 psi and a test pressure up to 412.5 psi. Class F flanges, for sizes to 48 inch diameter, are rated at 300 psi with a test pressure up to 450 psi.

Corrosion Protection

Tape wrapping coated steel pipe can be described as a bonded dielectric coating which provides a physical barrier between the pipe and the surrounding environment. Although the tape provides a physical barrier, it might not alone provide sufficient corrosion protection for steel pipe.

Steel pipe that is dielectrically coated is more resistant to stray current than mortar coated pipe. However, with dielectrically coated pipe, any discharge of current could be concentrated at holidays in the dielectric coatings if alternate safe paths such as galvanic anodes are not provided. Dielectrically coated steel pipe requires a low initial current that remains stable over time since polarization is nearly instantaneous if isolated from rebar and grounds.

Cathodic protection is very compatible with dielectrically coated steel pipe because the cathodic protection current requirements are low and the coating system does not shield the pipe from the cathodic protection current. Accordingly, cathodic protection will be effective at protecting the pipe from corrosion at any holiday in the coating and as long as the cathodic protection system is maintained, the steel pipe should remain corrosion-free indefinitely. More extensive soil information will be obtained during the geotechnical investigation and soil corrosivity will be evaluated.

Typical methods to control corrosion for dielectrically coated steel pipe include the following:

- Electrical isolation from dissimilar piping.
- Electrical bonding of all joints and field testing stations with stationary reference electrodes for corrosion monitoring
- Cathodic protection system consisting of either galvanic anodes or an impressed current system.

Mortar coated steel pipe is corrosion resistant in most soils, except soils with high levels of sulfates or chlorides. These high sulfate and chloride soils can cause degradation of the mortar coating and eventually corrosion of steel components of the pipeline. If the mortar coating is not sound, corrosive soils can directly attack the steel components. Soil information will be obtained during the geotechnical investigations. If more than the normal mortar coating is required to control corrosion for mortar coated steel pipe other methods include the following:

- Use special Type II cement in the mortar pipe coating.
- Electrical isolation from dissimilar piping.
- Polyethylene encasement in areas subject to stray current pickup.
- Electrical bonding of all joints and field test stations with stationary reference electrodes for corrosion monitoring.
- Cathodic protection system consisting of either galvanic anodes or an impressed current system.

The corrosion protection system will be evaluated during design to determine the best method of corrosion control.

Additional Considerations

Steel pipe is a flexible pipe designed specifically for the internal pressure and external load. The pipe design allows for adjustments of the pipes structural strength by increasing the wall thickness. Since steel pipe is a flexible pipe, buckling and deflection need to be considered in the design. Steel pipe can accommodate the assumed pressure ranges and cover depth.

Steel pipe can be easily field repaired. Depending on type, size, and location of the damage, small localized damage can be repaired with tapping saddles, larger localized damage with steel plate repair, and extensive damage replacing with new pipe section and butt strap closure.

7.4.8 High Density Polyethylene Pipe (HDPE) (ANSI/AWWA C906)

History & Availability

Polyethylene pipe is mainly used in the U.S. within the natural gas piping industry and, more commonly, European water industry markets. However, polyethylene pipe has become more popular in the U.S. water industry market in recent times. The first polyethylene water piping systems in the U.S. were installed in the early 1960's and use has grown since. Published studies have shown consistently high satisfaction ratings from water utilities as early as the early 1980's up to recent years. Studies and the Plastics Pipe Institute, Inc. (PPI) have confirmed a 100-year plus life expectancy for this pipe material.

PolyPipe Inc., Performance Pipe and WL Plastics are some of the manufacturers that furnish HDPE pipe in this area. All three manufacturers have manufacturing facilities located in Texas. PolyPipe, Inc. has a pipe manufacturing plant in Gainesville, Texas. Performance Pipe has pipe manufacturing plant in Brownwood, Texas. WL Plastics has a pipe manufacturing plant in Bowie, TX.

Linings & Coatings

No linings or coatings are necessary with HDPE pipe; the pipe material by itself provides a smooth wearing surface and structural strength.

Joint Types

HDPE pipe is easily and dependably joined using the standardized butt-fusion procedure. In this process matching ends of the pipes to be joined are aligned and heated with standard tools until the surfaces have become molten. When engaged under moderate pressure, the melt faces flow together forming a monolithic, homogeneous joint that, as the material cools, yields joints that are as strong as or stronger than the pipe itself.

There are several other well established heat fusion procedures used to join HDPE pipe such as electrofusion, which is well suited for the assembly of pipe to fittings, or for making tie-ins or repairs, and saddle fusion which is used for the attachment of service fittings to HDPE mains.

More details of some of these heat fusion procedures can be found in ASTM standards such as F2620, Standard Practice for Heat Fusion Joining of Polyethylene Pipe and Fittings, AWWA Manual M55, "PE Pipe – Design and Installation, and in Chapter 9 of PPI's Handbook of Polyethylene Pipe, 2nd ed.

Fittings & Appurtenances

HDPE transition fittings, HDPE mechanical-joint adapters, gasket joint adaptors, HDPE flanges, and standard metal couplings with internal stiffeners are recommended to connect HDPE pipe to most standard valves and appurtenances. The most common method is to use an HDPE MJ (mechanical joint) adapter to connect the HDPE pipe end in a Ductile Iron MJ bell using the bolt and gland kit supplied by the HDPE MJ manufacturer. DIPS sized HDPE pipe may be inserted directly into an MJ bell with a restraint ring and insert stiffener for the HDPE pipe. HDPE and stainless steel ring flanges are also available. When joining HDPE pipe to a DI pipeline either the DI joints must be restrained or the transition connection must be anchored. More details of these mechanical joining systems can be found in Chapter 9 of PPI's Handbook of Polyethylene Pipe.

Corrosion Protection

HDPE pipe does not undergo galvanic corrosion and therefore it may be safely installed in corrosive soils that would attack metal pipes and there is no need for cathodic protection.

Additional Considerations

There are three primary additional considerations for HDPE pipe, temperature effects, pressure rating, and hydrocarbons in the soil. High water temperatures can result in pressure de-rating of plastic pipes. HDPE pipe can be constructed using different resins. The most common, 4710, will have a de-rating factor of approximately 77% at 105 degrees Fahrenheit – thus at this temperature, HDPE retains only 77% of its design pressure. For below-grade installations, the temperature vulnerability of HDPE is not as big of an issue as the ground and surrounding backfill serve as insulation for the pipe.

HDPE manufacturers are currently working on expanding their product offerings over 36" in diameter, however at this time, it is difficult to find large diameter HDPE used as pressure pipe.

Hydrocarbons in contact with HDPE pipe wall may, over time, permeate the pipe wall and contaminate the water. HDPE pipe should not be used for water service where there is likelihood that the pipe will be exposed to significant concentrations of pollutants consisting of low molecular weight petroleum products, organic vapors, or their solvents.

7.4.9 Polyvinyl Chloride Pipe (PVC) (ANSI/AWWA C905)

History & Availability

Polyvinyl chloride pipe is available up to 48-inch diameter as covered by ANSI/AWWA C905. The rated pressure decreases as the pipe diameter increases. The maximum pressure class for 24, 30, 36, 42, and 48-inch diameter pipe is 235, 235, 200, 165, and 165, respectively. Pressure class rating is temperature dependent and a reduction factor is applied for temperatures over 73.4° F. Pressure class rating is also impacted by the frequency of water system surge pressures which needs to be investigated for each project. Maximum depth of cover varies depending on pipe class and bedding/backfill conditions. Factors that impact ease of installation include light weight (significantly lighter than concrete or metal pipe), standard pipe length (20-feet), ability to lay in curves, and "flexible" pipe design (special trench and bedding/backfill requirements may apply). PVC pipe can be field cut and tapped, however, other field modifications are generally not practical or possible. PVC can be susceptible to failure due to sharp or rapid impacts from traffic loading at insufficient cover or compaction equipment.

There are several manufacturers of PVC water pipe, including JM Eagle, Diamond Plastics, North American Pipe, and Northern Pipe. JM Eagle indicated the majority of the water pipe they have supplied in Texas has been 24-inch diameter and smaller. Diamond Plastics stated they have supplied pipe to projects in Michigan and are capable of supplying up to 48-inch diameter and are planning to expand to 60-inch diameter in the near future.

Linings & Coatings

No linings or coatings are necessary with PVC pipe, the pipe material by itself provides a smooth wearing surface and structural strength.

Joint Types

For buried applications, bell and spigot joints with rubber gaskets (non-restrained) are common. Thrust blocks or mechanical restraining devices are typically used to provide thrust restraint where needed. A "fusible" PVC option is also available from one manufacturer for diameters up to 48 inches. Ductile iron fittings are typically used where needed, as C905 PVC pipe is manufactured with outside diameters equivalent to ductile iron pipe sizes.

Fittings & Appurtenances

PVC transition fittings, PVC mechanical-joint adapters, gasket joint adaptors, and standard metal couplings with internal stiffeners are recommended to connect PVC pipe to most standard valves and appurtenances. When joining PVC pipe to a DI pipeline either the DI joints must be restrained or the transition connection must be anchored.

Corrosion Protection

PVC pipe is highly corrosion resistant and does not need any additional corrosion protection for typical water transmission applications.

Additional Considerations

There are three primary additional considerations for PVC pipe, temperature effects, pressure rating, and hydrocarbons in the soil. High water temperatures can result in pressure de-rating of plastic pipes. The de-rating factor for PVC pipe is approximately 77% at 105 degrees Fahrenheit – thus at this temperature, PVC retains only 77% of its design pressure. For below-grade installations, the temperature vulnerability of PVC may not be an issue as the ground and surrounding backfill serve as insulation for the pipe.

PVC manufacturers are currently working on expanding their product offerings over 48” in diameter, however at this time, it is difficult to find large diameter PVC used as pressure pipe.

Hydrocarbons in contact with PVC pipe wall may, over time, permeate the pipe wall and contaminate the water. PVC pipe should not be used for water service where there is likelihood that the pipe will be exposed to significant concentrations of pollutants consisting of low molecular weight petroleum products, organic vapors, or their solvents.

7.4.10 Pipe Material Comparison Table

Table 7-3: Pipeline Material Comparison

PIPELINE MATERIAL	LARGEST DIAMETER[1]	MAX WORKING PRESSURE[2,3]	ADVANTAGES	DISADVANTAGES
Bar Wrapped Concrete Cylinder Pipe (BWCCP)	72-inch	400 psi	• Semi-rigid pipe with high strength • Full range of working pressures[1] • Design may be optimized to meet project specific pressures and load requirements • Additional corrosion protection typically not required	• Heavier than SP, DIP, PVC, HDPE and FRP • Limitations on depth of bury • Limited local installations
Prestressed Concrete Cylinder Pipe (PCCP)	144-inch	400 psi	• Rigid pipe with high strength • Full range of working pressures[1] • Full range of diameters[2] • Design may be optimized to meet project specific	• Heavier than SP, DIP, PVC, HDPE and FRP • Although rare, catastrophic failures have occurred with little or no warning

PIPELINE MATERIAL	LARGEST DIAMETER[1]	MAX WORKING PRESSURE[2,3]	ADVANTAGES	DISADVANTAGES
			pressures and load requirements • Additional corrosion protection typically not required • Extensive local installations	
Ductile Iron Pipe (DIP)	64-inch	250 psi	• Semi-flexible pipe with high strength • Full range of working pressures[1]	• Need for corrosion protection
Steel Pipe (SP)	144-inch	500 psi	• Flexible pipe with high strength • Full range of working pressures[1] • Full range of diameters[2] • Design may be optimized to meet project specific pressures and load requirements • Easy field modifications	• Need for corrosion protection
High Density Polyethylene Pipe (HDPE)	36-inch	250 psi	• No need for corrosion protection • Standard installation assembly is restrained • Full range of working pressures[1]	• Limited diameters • Limited installation history in larger diameters • Lower loading limits than concrete and metallic pipe • Requires significant layout area for installation
Poly Vinyl Chloride Pipe (PVC)	48-inch	Varies by diameter 235 psi (24-inch) 165 psi (48-inch)	• No need for corrosion protection	• Limited diameters and pressures • Limited installation history in larger diameter water transmission applications • Lower loading limits than concrete and metallic pipe

[1]Largest diameter only considers range included in evaluation (24 through 120-inches)
[2]Max working pressure only considers range included in evaluation (up to 250 psi)
[3]A pressure derating factor is applied to PVC, HDPE, and FRP if temperature is >73.4°F

In summary, due to the size, pressure and soil considerations it is recommended that Steel Pipe is used for entire length of Pipeline. Corrosion protection will be required. During the design process, value engineering could analyze if using different materials for different segments of the pipe would be beneficial, especially in smaller diameter sections where HDPE or PVC could be used.

7.5 PIPELINE ROUTING

Several pipeline routes were examined to transport water through the RGRWA pipeline system. There are two primary routes, a northern route using the State Route 83, Expressway 83, or the BPUB Pipeline Right of Way (ROW), and a southern route along Military Highway.

7.5.1 Northern Route

The northern route options are shown in Figure 7-3. Within the Northern Route, there are several sub-route options that have been evaluated including the use of the BPUB Pipeline ROW, Hwy 510, SR 77, Business 83 and Express 83. In general, the route runs from South Padre Island down Hwy 48 to Brownsville. Once the pipeline gets to Hwy 511 it turns north and runs along the Hwy 511 right of way to the Southmost Treatment Plant. Once at the plant, the first of the route options is available. The pipeline can use the BPUB Pipeline ROW heading north through farmland, or continue along Hwy 511. If the pipeline continues along Hwy 511, it then turns north at Hwy 77 and heads towards Harlingen. The second route option is available where Hwy 77 splits to Hwy 83 and Hwy 510. The BPUB Pipeline ROW, Hwy 83 and Hwy 510 routes all meet up again in Harlingen. Once in Harlingen, the northern route can either follow the frontage road right of way of Expressway 83, or follow Business Hwy 83 to the western end of the pipeline. Another alternative route to be considered is to travel northeast from FM 511 up to SH100, then continue north to the location of the future causeway and across to South Padre Island.

The northern route has several sub-options including the use of a new bridge access to South Padre Island, the BPUB Pipeline ROW, SR 77, Hwy 510, Business 83, and Expressway 83. Table 7-4 shows the advantages and disadvantages of each route option. The new bridge access to SPI was excluded from the analysis at this time due to uncertainty of location and timing of bridge construction.

Table 7-4: Northern Route Advantages/Disadvantages

Route	Advantages	Disadvantages
BPUB-Tenaska Pipeline ROW	• ROW acquired • Minimal Road Crossings • Less Surface Improvements	• Longer route, more pipe • Doesn't run through populated areas, lengthy connections to system
510	• Centrally located, runs through population centers	• Disruption to business access
SR 77	• Centrally located, runs through population centers	• Use of TXDOT and RR ROW • Tight work area on Frontage Road
Business 83	• Centrally located, runs through population centers • Additional ROW available where TXDOT and RR are adjacent	• Road Crossings • Use of TXDOT and RR ROW
Express 83	• Centrally located, runs through population centers	• Road Crossings • Use of TXDOT and RR ROW • Tight work area on Frontage Road

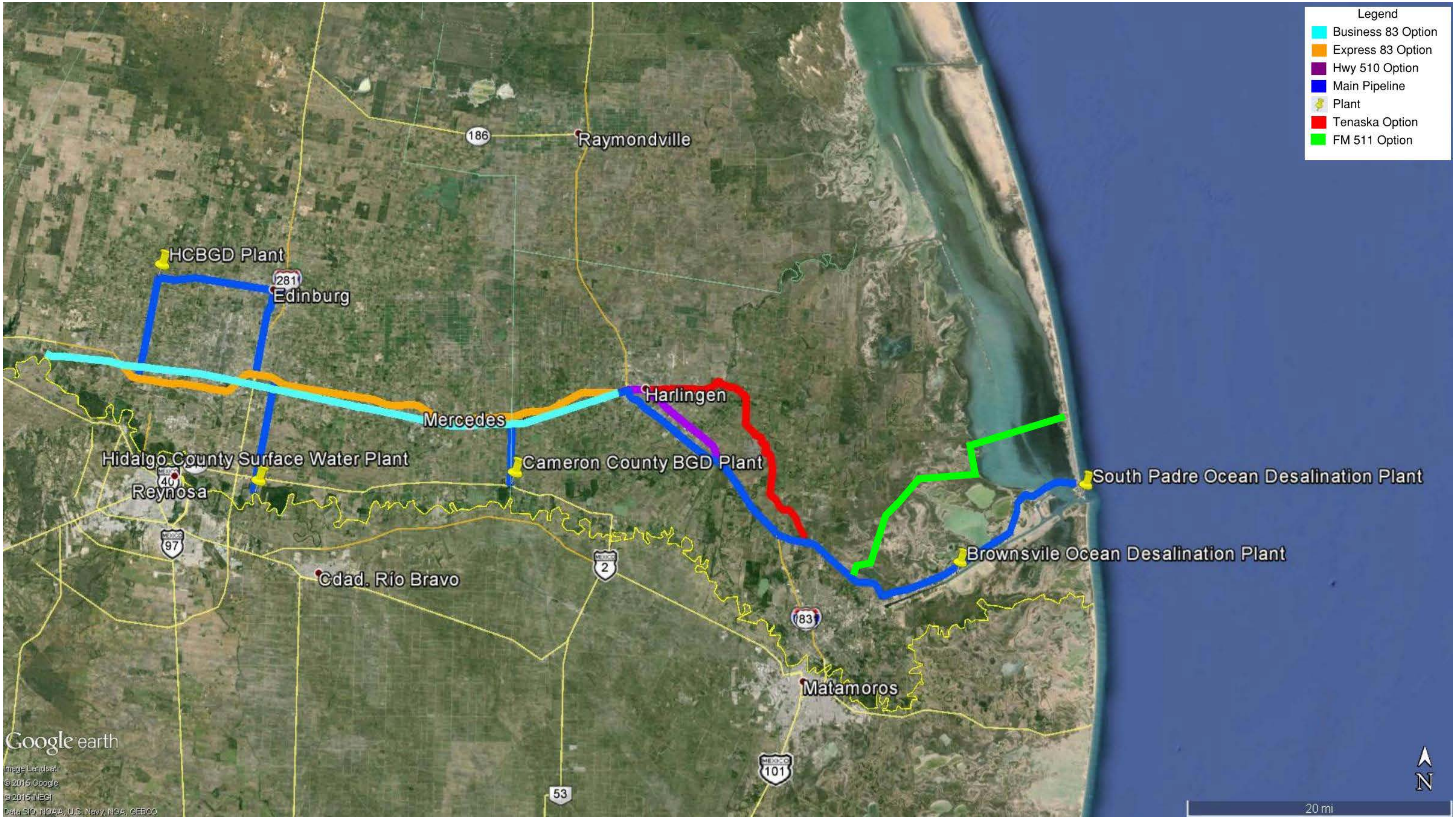


Figure 7-3: Northern Pipeline Route Map

7.5.2 Southern Route

The southern route is shown in Figure 7-4, it runs from South Padre Island down Hwy 48 to Brownsville. Once the pipeline gets to Hwy 511, it continues on Hwy 48 through Brownsville to Military Hwy. The pipeline then continues along the Military Hwy right of way all the way to the western end of the system. Because the pipeline runs to the south of the major population centers in the area, extensive pipelines running to the north will be necessary to connect the population with the water supply.

Table 7-5 summarizes the advantages and disadvantages of the southern route.

Table 7-5: Southern Route Advantages/Disadvantages

Route	Advantages	Disadvantages
Southern	• Extensive right of way available • Minimal Road Crossings • Less Extensive Surface Improvements	• Longer route, more pipe • Soils along Military Highway are known for instability and higher corrosion potential. • Doesn't run through populated areas, lengthy connections to system • Use of TXDOT ROW

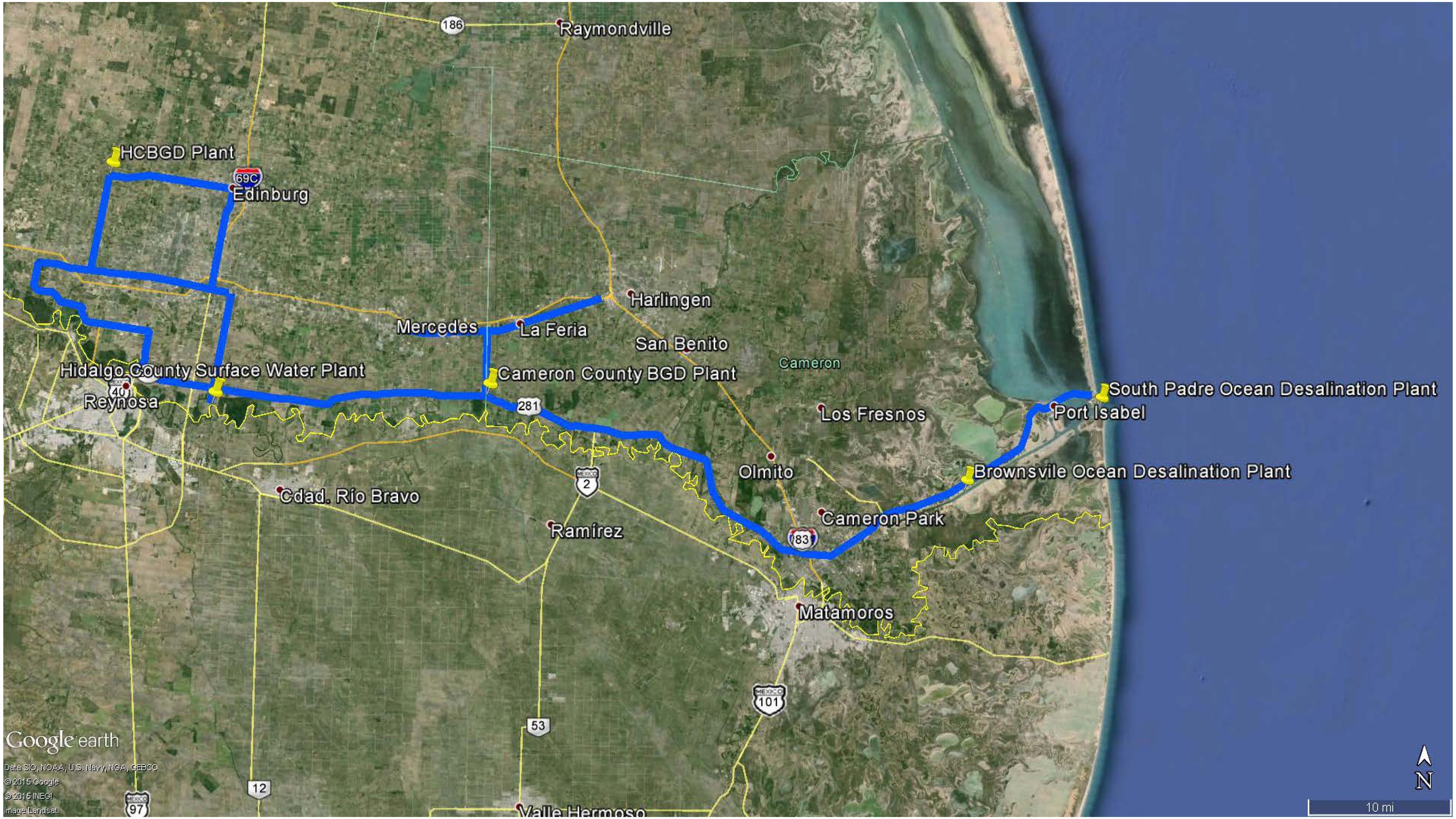


Figure 7-4: Southern Route Map

7.5.3 Route Selection

The route of the pipeline selected is the use of the BPUB right of way and then running the pipe parallel to Business 83 to the west end of the project. This route was selected because it is centered on the population to be served by the line while minimizing the length of the route and disruption from construction. Figure 7-5 shows a map of the preferred route.

It is anticipated that in final design further analysis will determine several key locations of the pipeline routing i.e. to attach the pipeline to the proposed new bridge access to South Padre Island, or alternatives due to unknown utility conflicts.

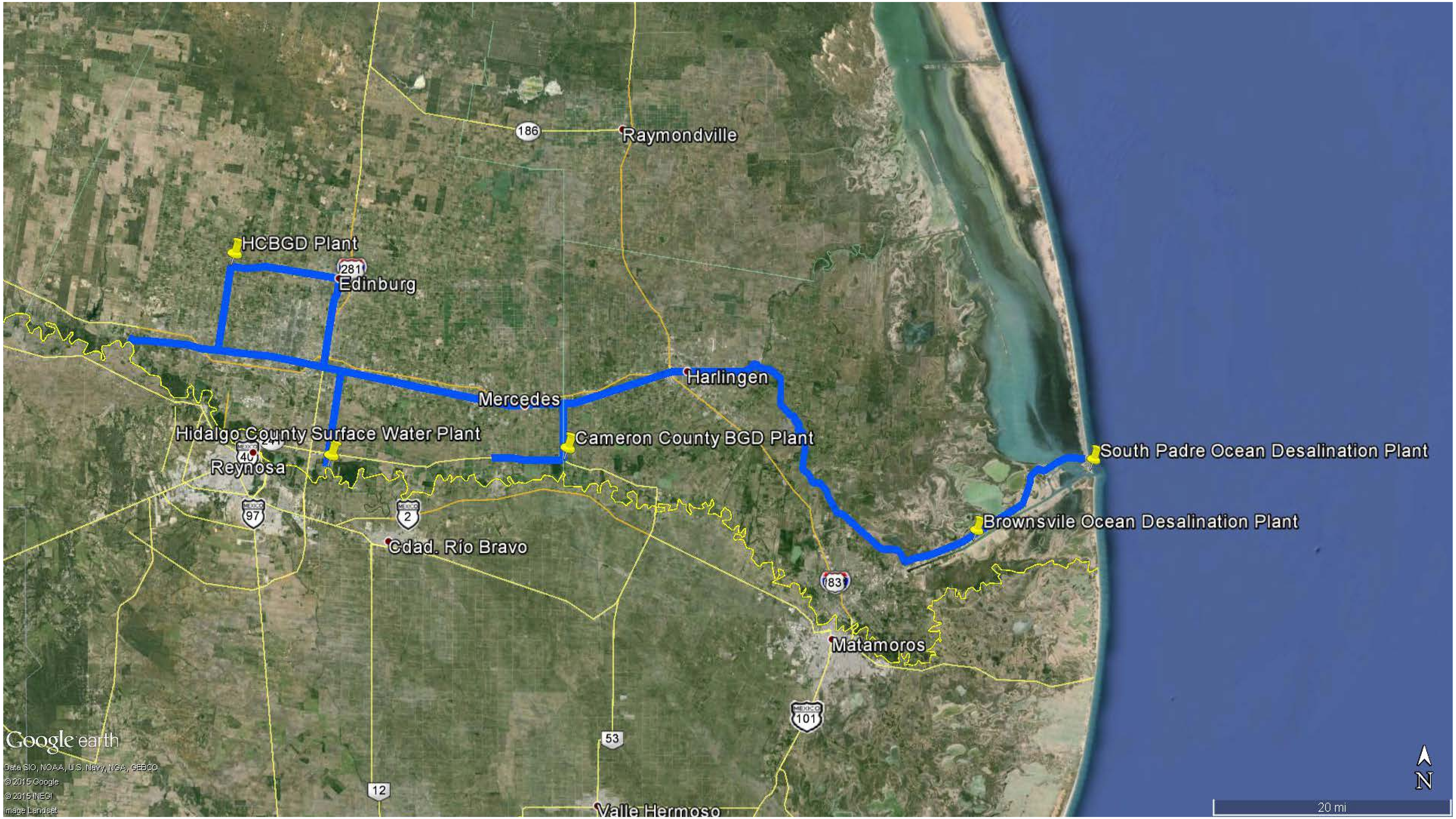


Figure 7-5: Preferred Route Alternative

7.6 PIPELINE SIZING

Pipeline sizes are calculated based on the system pressure requirements, water supply locations, velocities and corresponding pipeline headloss between supplies and demands. The pipeline headloss is calculated using the Hazen-Williams formula:

$$h_f = \frac{10.44 \cdot L \cdot Q^{1.85}}{C^{1.85} \cdot d^{4.8655}}$$

Where: h_f = head loss due to friction (ft)
L = length of pipe (ft)
Q = flow rate of water (gpm)
C = Hazen-Williams constant
d = diameter of the pipe (in.)

Pipeline diameter can be iteratively calculated based on minimum and maximum velocity requirements for the pipeline. The pipeline system is broken down in to nodes to simplify the calculation and additional pipeline sizing will take place during design.

7.6.1 Assumptions

A simplified pipeline analysis has been completed by reducing supply and demand values from individual cities into a series of nodes or clusters representing cities that are located close by. Table 7-6 identifies the demand nodes as well as the flow quantities for each decade evaluated.

Table 7-6: RGRWA Demands

	Total needed from RGRWA					
Entity	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
S. Padre Node (Ac-Ft.)						
South Padre Island	1,100	1,650	2,200	2,750	3,350	4,000
Port Isabel	450	650	850	1,100	1,300	1,550
Laguna Vista	850	1,250	1,650	2,100	2,550	3,000
Total:	2,400	3,550	4,700	5,950	7,200	8,550
MGD:	2.1	3.2	4.2	5.3	6.4	7.6
Peaking Factor (1.3x):	2.8	4.1	5.5	6.9	8.4	9.9
Brownsville Node (Ac-Ft.)						
Brownsville	0	0	500	7,600	15,150	22,950
Olmito WSC	0	0	0	100	250	400
Rancho Viejo	0	0	0	0	100	250
Total:	0	0	500	7,700	15,500	23,600
MGD:	0.0	0.0	0.4	6.9	13.8	21.1
Peaking Factor (1.3x):	0.0	0.0	0.6	8.9	18.0	27.4

Entity	2020	2030	2040	2050	2060	2070
Harlingen Node (Ac-Ft.)						
San Benito	0	0	0	0	600	1,250
East Rio Hondo WSC	0	50	650	1,300	2,000	2,700
Harlingen	0	0	1,100	3,500	6,050	8,700
La Feria	0	50	200	400	600	800
Total:	0	100	1,950	5,200	9,250	13,450
MGD:	0.0	0.1	1.7	4.6	8.3	12.0
Peaking Factor (1.3x):	0.0	0.1	2.3	6.0	10.7	15.6
Mercedes Node (Ac-Ft.)						
Mercedes	250	700	1,150	1,600	2,100	2,550
Weslaco	2,800	4,500	6,200	7,950	9,800	11,550
Donna	0	150	650	1,250	1,800	2,400
Alamo	850	1,500	2,200	2,950	3,650	4,400
Total:	3,900	6,850	10,200	13,750	17,350	20,900
MGD:	3.5	6.1	9.1	12.3	15.5	18.7
Peaking Factor (1.3x):	4.5	7.9	11.8	16.0	20.1	24.3
McAllen Node (Ac-Ft.)						
San Juan	1,750	2,850	3,900	5,250	6,550	7,850
Pharr	20	2,050	4,150	6,300	8,600	10,750
McAllen	4,350	12,800	21,500	30,350	39,350	48,150
Sharyland WSC	1,050	4,300	7,700	11,200	15,700	17,850
Total:	7,170	22,000	37,250	53,100	70,200	84,600
MGD:	6.4	19.6	33.3	47.4	62.7	75.5
Peaking Factor (1.3x):	8.3	25.5	43.2	61.6	81.5	98.2
Mission Node (Ac-Ft.)						
Agua SUD	0	700	700	2,900	4,600	6,350
Hidalgo County MUD1	300	450	550	650	800	950
Mission	6,650	11,150	15,700	20,350	25,100	29,700
Total:	6,950	12,300	16,950	23,900	30,500	37,000
MGD:	6.2	11.0	15.1	21.3	27.2	33.0
Peaking Factor (1.3x):	8.1	14.3	19.7	27.7	35.4	42.9
Edinburg Node (Ac-Ft.)						
Edinburg	3,550	6,350	9,200	12,150	15,150	18,100
North Alamo WSC	0	1,750	3,100	8,750	12,350	16,950
Total:	3,550	8,100	12,300	20,900	27,500	35,050
MGD:	3.2	7.2	11.0	18.7	24.6	31.3
Peaking Factor (1.3x):	4.1	9.4	14.3	24.3	31.9	40.7

Entity	2020	2030	2040	2050	2060	2070
Military HWY Node (Ac-Ft.)						
Military Highway WSC	1,100	2,050	3,050	4,150	5,250	6,400
Total:	1,100	2,050	3,050	4,150	5,250	6,400
MGD:	1.0	1.8	2.7	3.7	4.7	5.7
Peaking Factor (1.3x):	1.3	2.4	3.5	4.8	6.1	7.4
Hidalgo Node (Ac-Ft.)						
Hidalgo	400	800	1,200	1,600	2,050	2,450
Total:	400	800	1,200	1,600	2,050	2,450
MGD:	0.4	0.7	1.1	1.4	1.8	2.2
Peaking Factor (1.3x):	0.5	0.9	1.4	1.9	2.4	2.8

Table 7-7 identifies the supply nodes as well as the flow quantities for each decade evaluated. It should be noted that an anticipated 2 MGD of flow may be leased to the regional system in 2020 to meet the demands. It is unknown where this will enter the system at this time but will have a minimal effect on the sizing of the pipeline.

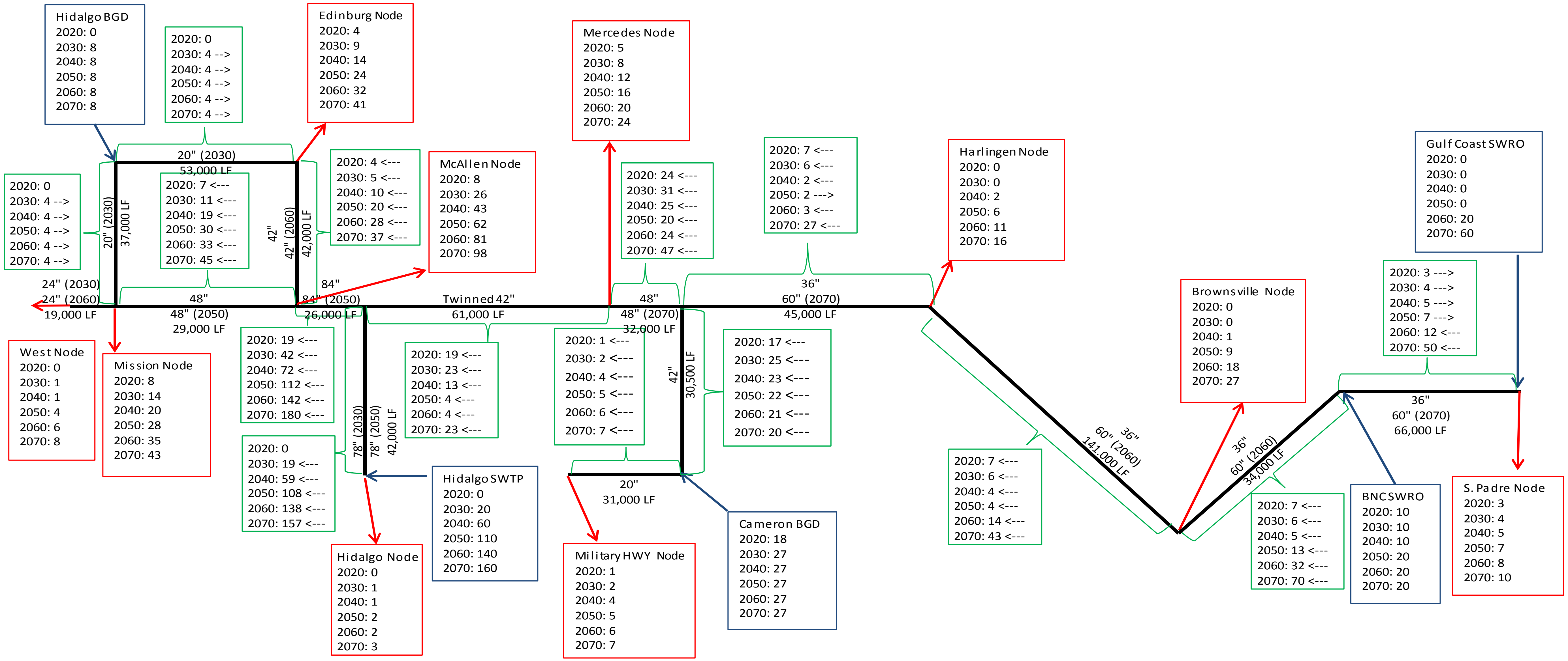
Table 7-7: RGRWA Supplies

	YEAR					
Total Supplies (MGD)	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Gulf Coast SWRO	0	0	0	0	20	60
BNC SWRO	10	10	10	20	20	20
Surface Water	0	20	60	110	140	160
Hidalgo BGD	0	8	8	8	8	8
Cameron BGD	18	27	27	27	27	27
Total:	28	65	105	165	215	275

Other pipeline design assumptions include:

- Hazen Williams C coefficient for all pipe is 120 to correspond with a smooth walled pipe that has been in service for some period of time.
- Pipeline velocities will be optimal between 0.5 feet per second (ft/s) and 4.0 ft/s to reduce headloss in the pipe and minimize pumping requirements. 4 ft/s is set as a maximum velocity and is not exceeded in any pipeline segment.

Figure 7-6 shows the system node map with supplies, demands, pipeline diameter, and pipeline length.



	YEAR					
Total Supplies (MGD)	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
Gulf Coast SWRO	0	0	0	0	20	60
BNC SWRO	10	10	10	20	20	20
Surface Water	0	20	60	110	140	160
Hidalgo BGD	0	8	8	8	8	8
Cameron BGD	18	27	27	27	27	27
Total:	28	65	105	165	215	275

	YEAR					
Total Demands (MGD)	**2020**	**2030**	**2040**	**2050**	**2060**	**2070**
S. Padre Node	3	4	5	7	8	10
Brownsville Node	0	0	1	9	18	27
Harlingen Node	0	0	2	6	11	16
Mercedes Node	5	8	12	16	20	24
McAllen Node	8	26	43	62	81	98
Mission Node	8	14	20	28	35	43
Edinburg Node	4	9	14	24	32	41
Military HWY Node	1	2	4	5	6	7
Hidalgo Node	0	1	1	2	2	3
Total:	30	65	102	158	214	269

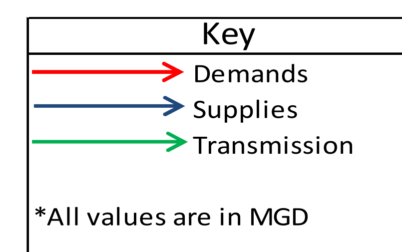


Figure 7-6: RGRWA Node Diagram

7.6.2 South Padre Island to Brownsville Navigation Channel (BNC) SWRO Plant

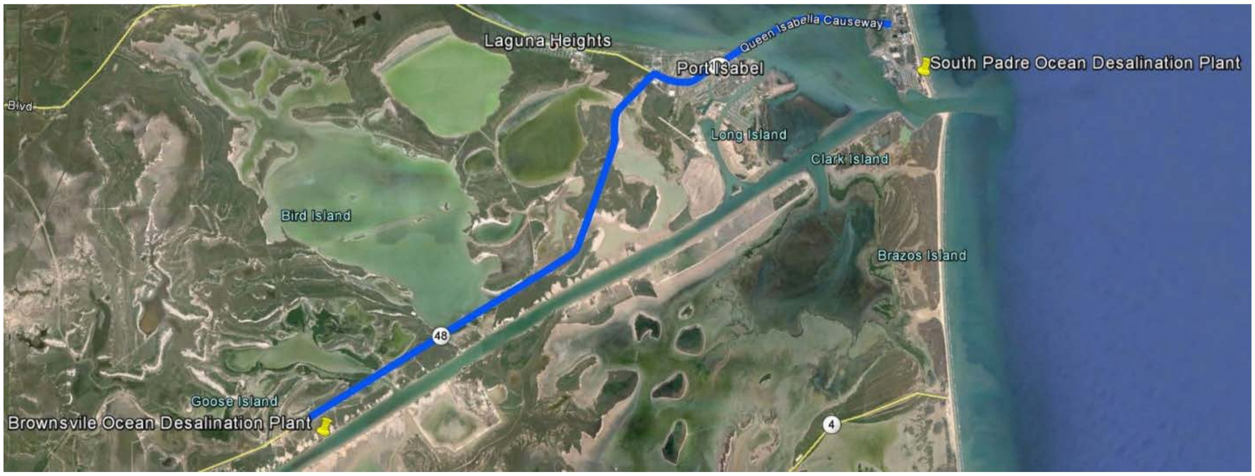


Figure 7-7: S. Padre Island to BNC SWRO Plant

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2020, the Brownsville Desalination Plant produces 10 MGD, the South Padre Island Desalination Plant produces zero MGD. The South Padre Island Node has a demand of 3 MGD, and the Brownsville Node has zero demand. Therefore in 2020, 3 MGD is flowing from the Brownsville plant to the South Padre Node.

The flow quantity and direction is summarized in Table 7-8 below.

Table 7-8: S. Padre Island to BNC SWRO Plant Pipeline Flow Summary

	2020	2030	2040	2050	2060	2070
Flow (MGD)	3 East	4 East	5 East	7 East	12 West	50 West

Route Length

The total length of the route from S. Padre Island to the BNC SWRO Plant is 66,000 LF.

Pipe Size

Table 7-9 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-9: S. Padre to BNC SWRO Velocity and Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
24	3.00	4.00	5.00	7.00	12.00	50.00	1.48	1.97	2.46	3.45	5.91	24.62	0.39	0.67	1.01	1.89	5.11	71.63
30	3.00	4.00	5.00	7.00	12.00	50.00	0.95	1.26	1.58	2.21	3.78	15.76	0.13	0.23	0.34	0.64	1.72	24.16
36	3.00	4.00	5.00	7.00	12.00	50.00	0.66	0.88	1.09	1.53	2.63	10.94	0.05	0.09	0.14	0.26	0.71	9.94
42	3.00	4.00	5.00	7.00	12.00	50.00	0.48	0.64	0.80	1.13	1.93	8.04	0.03	0.04	0.07	0.12	0.33	4.69

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	24	30	36	42	24	30	36	42
Q_{2020}	0.39	0.13	0.05	0.03	25	8	3	2
Q_{2030}	0.67	0.23	0.09	0.04	42	14	6	3
Q_{2040}	1.01	0.34	0.14	0.07	64	22	9	4
Q_{2050}	1.89	0.64	0.26	0.12	119	40	17	8
Q_{2060}	5.11	1.72	0.71	0.33	324	109	45	21
Q_{2070}	71.63	24.16	9.94	4.69	4,538	1,531	630	297

Given the flow criteria, the initial pipeline size is 36" diameter. A 42" diameter pipeline is too large and results in flow velocities that are too slow in 2020. A 30" diameter pipeline may be acceptable; however, the 36" pipeline will accommodate the flows in this pipeline from 2020 through 2050. The pipeline will need to be paralleled in 2060 when the larger seawater desalination plant comes online and provides flows to Brownsville and points west.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 36" pipe. Table 7-10 below summarizes the headloss calculations.

Table 7-10: S. Padre Island to BNC SWRO Second Pipeline

Second Pipeline											
Q_{2070} =	50	MGD	HL1 =HL2								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	36		D	48		D	54		D	60	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
9	1.97	26.4	41	5.05	107.6	41	3.99	60.7	41	3.23	36.3
9.50	2.08	29.2	40.5	4.99	105.2	40.5	3.94	59.3	40.5	3.19	35.5
10.00	2.19	32.1	40	4.92	102.8	40	3.89	57.9	40	3.15	34.7
10.50	2.30	35.2	39.5	4.86	100.5	39.5	3.84	56.6	39.5	3.11	33.9
11.00	2.41	38.3	39	4.80	98.1	39	3.79	55.3	39	3.07	33.1
11.50	2.52	41.6	38.5	4.74	95.8	38.5	3.75	54.0	38.5	3.03	32.3
12.00	2.63	45.0	38	4.68	93.5	38	3.70	52.7	38	2.99	31.5
12.50	2.74	48.5	37.5	4.62	91.3	37.5	3.65	51.4	37.5	2.95	30.8

The table illustrates that a 60" diameter pipe is an appropriate choice for the expansion. The velocities are in the acceptable range between 0.5 ft/s and 4.0 ft/s and the pipeline headloss is around 34 ft. Though a 54" would also work and would be less expensive to build, utilizing a larger line will reduce the headloss in the line and delay the construction of an intermediate booster pump station until 2070.

In summary, the pipeline from South Padre Island to the BNC SWRO Plant is 66,000 LF of 36" pipe installed in 2020 and 66,000 LF of 60" pipe installed in 2070 to accommodate increased supply from the Gulf Coast SWRO Plant that comes online in 2070.

7.6.3 BNC SWRO Plant to HWY 511

Figure 7-8: BNC SWRO Plant to Hwy 511 Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2020, the Brownsville Desalination Plant produces 10 MGD, the South Padre Island Desalination Plant produces zero MGD. The South Padre Node has a demand of 3 MGD, and the Brownsville Node has zero demand. Therefore in 2020, 7 MGD is flowing from the Brownsville plant to the Brownsville Node. The flow quantity and direction is summarized in Table 7-11 below.

Table 7-11: Brownsville Desal to Hwy 511 Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	7 West	6 West	5 West	13 West	32 West	70 West

Route Length

The total length of the route from the BNC SWRO Plant to the Brownsville Node is 34,000 LF.

Pipe Size

Table 7-12 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-12: BNC SWRO to Hwy 511 Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
24	7.0	6.0	5.0	13.0	32.0	70.0	3.4	3.0	2.5	6.4	15.8	34.5	1.9	1.4	1.0	5.9	31.4	133.5
30	7.0	6.0	5.0	13.0	32.0	70.0	2.2	1.9	1.6	4.1	10.1	22.1	0.6	0.5	0.3	2.0	10.6	45.0
36	7.0	6.0	5.0	13.0	32.0	70.0	1.5	1.3	1.1	2.8	7.0	15.3	0.3	0.2	0.1	0.8	4.4	18.5
42	7.0	6.0	5.0	13.0	32.0	70.0	1.1	1.0	0.8	2.1	5.1	11.3	0.1	0.1	0.1	0.4	2.1	8.7

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	24	30	36	42	24	30	36	42
Q_{2020}	1.89	0.64	0.26	0.12	65	22	9	4
Q_{2030}	1.42	0.48	0.20	0.09	49	16	7	3
Q_{2040}	1.01	0.34	0.14	0.07	35	12	5	2
Q_{2050}	5.93	2.00	0.82	0.39	203	69	28	13
Q_{2060}	31.37	10.58	4.35	2.06	1,077	363	149	71
Q_{2070}	133.48	45.03	18.53	8.75	4,581	1,545	636	300

Given the flow criteria, the initial pipeline size is 36" diameter. The 36" diameter pipeline satisfies the flow control rules and provides a constant pipe diameter from the previous reach. The pipeline will need to be parallel in 2060 when the South Padre Seawater Desalination Plant increases in size and provides flows to Brownsville and points west.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 36" pipe. Table 7-13 below summarizes the headloss calculations.

Table 7-13: BNC SWRO to Hwy 511 Second Pipeline

Second Pipeline											
Q_{2070} =	43	MGD	**HL1 =HL2**								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	36		D	60		D	66		D	72	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
7	1.53	9.0	36	2.84	15.5	36	2.34	9.7	36	1.97	6.4
7.50	1.64	10.2	35.5	2.80	15.1	35.5	2.31	9.5	35.5	1.94	6.2
8.00	1.75	11.5	35	2.76	14.7	35	2.28	9.2	35	1.92	6.0
8.50	1.86	12.9	34.5	2.72	14.3	34.5	2.25	9.0	34.5	1.89	5.9
9.00	1.97	14.3	34	2.68	13.9	34	2.21	8.7	34	1.86	5.7
9.50	2.08	15.8	33.5	2.64	13.5	33.5	2.18	8.5	33.5	1.83	5.6
10.00	2.19	17.4	33	2.60	13.2	33	2.15	8.3	33	1.81	5.4

The table illustrates that a 60" diameter pipe is the appropriate choice for the second pipeline. The velocities are in the acceptable range between 0.5 ft/s and 4.0 ft/s and the pipeline headloss is around 14 ft. Larger pipe diameters would also meet the criteria; however, using the 60" diameter pipe will be less expensive and is the same size used in the previous section.

The final step in sizing the pipe is verifying that the 60" pipe is acceptable for the previous flow years. Table 7-14 summarizes the headloss and velocity information for 2060.

Table 7-14: BNC SWRO to Hwy 511 Verification

Verify Second Pipeline Works for Previous Flow Years					
Q_{2060} =	32	MGD		**HL1 =HL2**	
Original Pipeline			Parallel Pipeline		
C	120		C	120	
D	36		D	60	
Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L
5	1.09	5	27	2.13	9
5.50	1.20	6	26.5	2.09	9
6.00	1.31	7	26	2.05	8
6.50	1.42	8	25.5	2.01	8
7.00	1.53	9	25	1.97	8
7.50	1.64	10	24.5	1.93	8
8.00	1.75	12	24	1.89	7

The table indicates that the velocity is still within the acceptable range of 0.5 ft/s to 4.0 ft/s and the headloss in the pipeline is at 8 ft.

In summary, the pipeline from BNC SWRO Plant to the Brownsville Node is 34,000 LF of 36” pipe installed in 2020 and 34,000 LF of 60” pipe installed in 2060 to accommodate increased supply from the Ocean Desalination plant that comes online in 2060.

7.6.4 HWY 511/BPUB-Tenaska to Harlingen

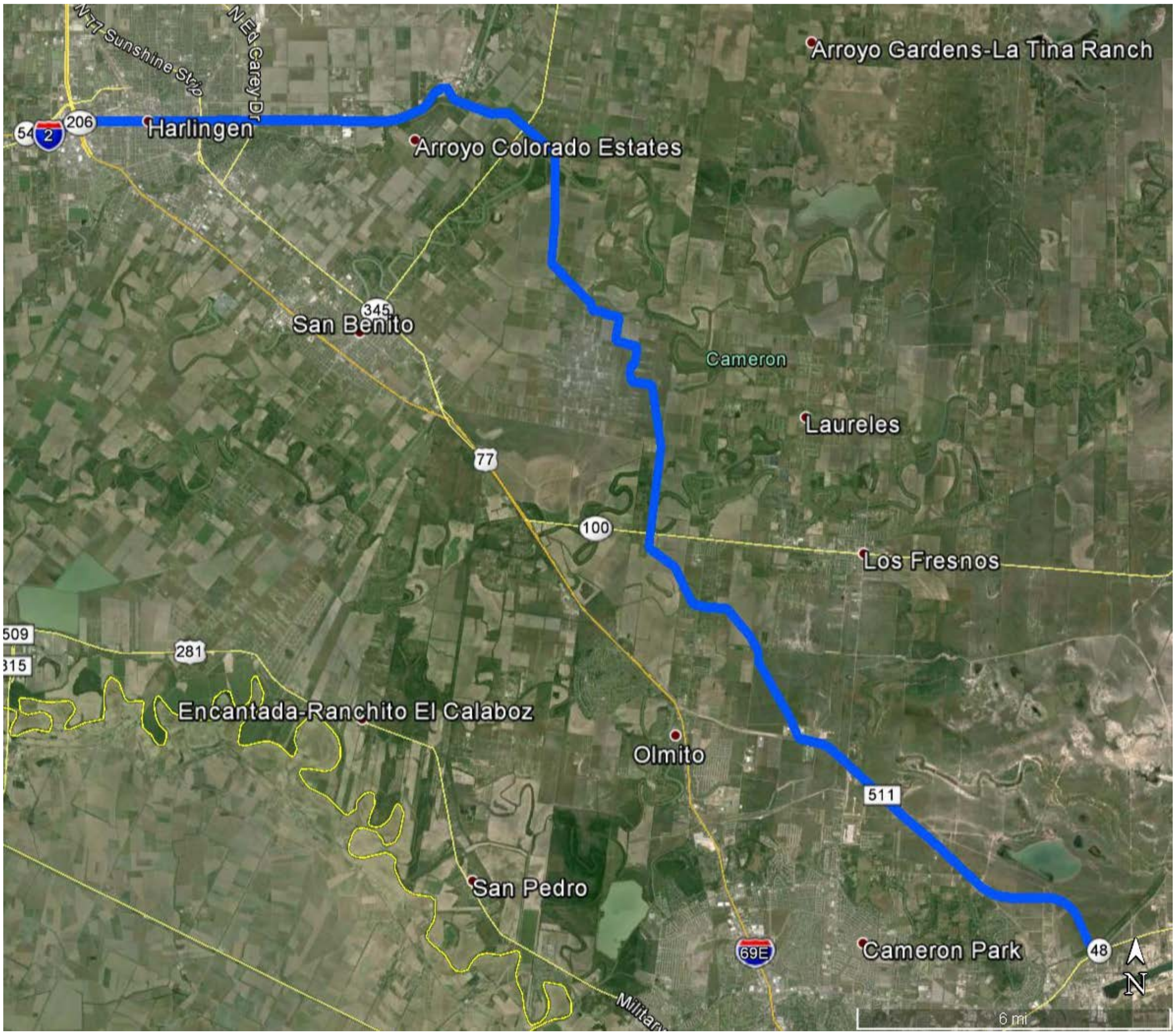


Figure 7-9: Hwy 511/BPUB-Tenaska to Harlingen

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2020, the Brownsville Desalination Plant produces 10 MGD, the South Padre Island Desalination Plant produces zero MGD. The South Padre Node has a demand of 3 MGD, and the Brownsville Node has zero demand. Therefore in 2020, 7 MGD is flowing from the Brownsville Node to the Harlingen Node. The flow quantity and direction is summarized in Table 7-15.

Table 7-15: Hwy 511 to Harlingen Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	7 West	6 West	4 West	4 West	14 West	43 West

Route Length

The total length of the route from the Brownsville Node to the Harlingen Node is 141,000 LF.

Pipe Size

Table 7-16 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-16: Hwy 511 to Harlingen Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
36	7.00	6.00	4.00	4.00	14.00	43.00	1.53	1.31	0.88	0.88	3.06	9.41	0.26	0.20	0.09	0.09	0.94	7.52
42	7.00	6.00	4.00	4.00	14.00	43.00	1.13	0.96	0.64	0.64	2.25	6.92	0.12	0.09	0.04	0.04	0.45	3.55
48	7.00	6.00	4.00	4.00	14.00	43.00	0.86	0.74	0.49	0.49	1.72	5.29	0.06	0.05	0.02	0.02	0.23	1.85
54	7.00	6.00	4.00	4.00	14.00	43.00	0.68	0.58	0.39	0.39	1.36	4.18	0.04	0.03	0.01	0.01	0.13	1.04

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	36	42	48	54	36	42	48	54
Q_{2020}	0.26	0.12	0.06	0.04	37	17	9	5
Q_{2030}	0.20	0.09	0.05	0.03	28	13	7	4
Q_{2040}	0.09	0.04	0.02	0.01	13	6	3	2
Q_{2050}	0.09	0.04	0.02	0.01	13	6	3	2
Q_{2060}	0.94	0.45	0.23	0.13	133	63	33	18
Q_{2070}	7.52	3.55	1.85	1.04	1,062	501	262	147

Given the flow criteria, the initial pipeline size is 36” diameter. The 36” diameter pipeline satisfies the flow control rules and provides continuity with the pipe diameters to the east.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 36” pipe. Table 7-17 below summarizes the headloss calculations.

Table 7-17: Hwy 511 to Harlingen Second Pipeline

Second Pipeline											
Q_{2070} =	43	MGD	**HL1 =HL2**								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	36		D	42		D	48		D	60	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
8	1.75	47.4	35	5.63	343.0	35	4.31	179.0	35	2.76	60.4
8.50	1.86	53.0	34.5	5.55	334.0	34.5	4.25	174.3	34.5	2.72	58.8
9.00	1.97	58.9	34	5.47	325.1	34	4.19	169.7	34	2.68	57.2
9.50	2.08	65.1	33.5	5.39	316.3	33.5	4.12	165.1	33.5	2.64	55.7
10.00	2.19	71.6	33	5.31	307.6	33	4.06	160.5	33	2.60	54.2

The table illustrates that a 60” diameter pipe is the appropriate choice for the second pipeline. The velocities are in the acceptable range between 0.5 ft/s and 4.0 ft/s and the pipeline headloss is around 57 ft. The 54” diameter results in velocities that are at the upper end of the acceptable range, and the 60” pipeline continues the same size of pipe from the previous sections.

The final step in sizing the pipe is verifying that the 60” pipe is acceptable for the previous flow years. Table 7-18: Hwy 511 to Harlingen Verification summarizes the headloss and velocity information for 2060.

Table 7-18: Hwy 511 to Harlingen Verification

Verify Second Pipeline Works for Previous Flow Years					
Q_{2060} =	14	MGD	**HL1 =HL2**		
Original Pipeline			Parallel Pipeline		
C	120		C	120	
D	36		D	60	
Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L
3.5	0.77	2	11	0.83	7
4.00	0.88	3	10	0.79	6
4.50	0.98	4	9.5	0.75	5
5.00	1.09	5	9	0.71	5
5.50	1.20	6	8.5	0.67	4
6.00	1.31	7	8	0.63	4
6.50	1.42	8	7.5	0.59	3

In summary, the pipeline from BNC SWRO Plant to the Harlingen Node is 141,000 LF of 36" pipe installed in 2020 and 141,000 LF of 60" pipe installed in 2060 to accommodate increased supply from the Gulf Coast SWRO Plant that comes online in 2060.

7.6.5 Harlingen to Cameron County Line

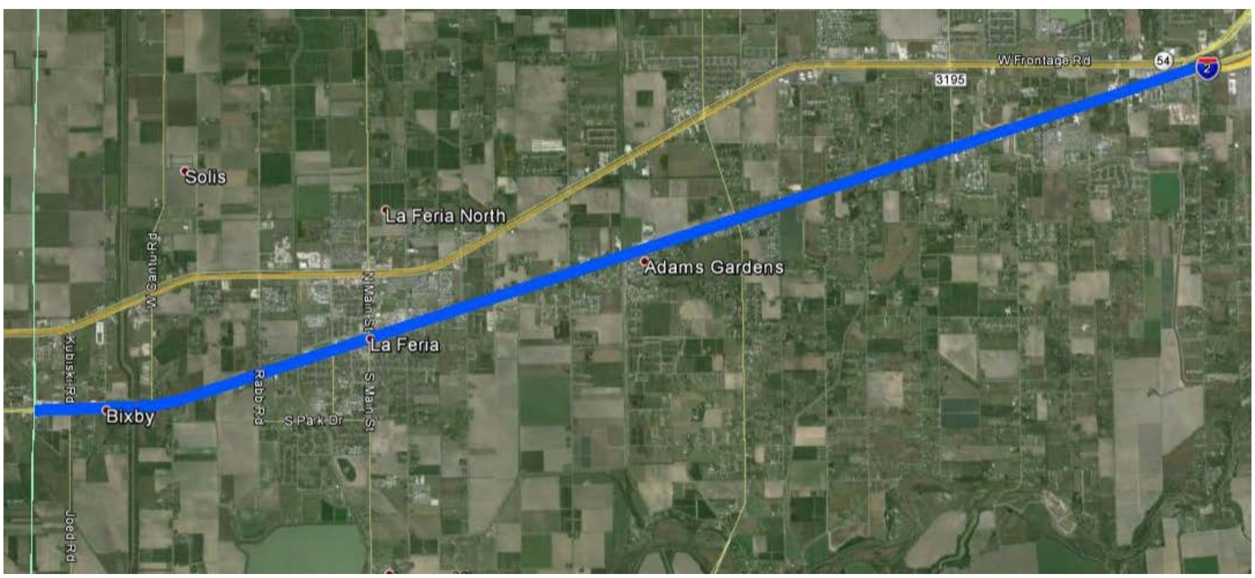


Figure 7-10: Harlingen to County Line Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. The flow quantity and direction is summarized in Table 7-19.

Table 7-19: Harlingen to County Line Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	7 West	6 West	2 West	2 East	3 West	27 West

Route Length

The total length of the route from the Harlingen Node to the Cameron County Line Node is 45,000 LF.

Pipe Size

Table 7-20 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-20: Harlingen to Cameron County Line Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
24	7.00	6.00	2.00	2.00	3.00	27.00	3.45	2.95	0.98	0.98	1.48	13.30	1.89	1.42	0.19	0.19	0.39	22.91
30	7.00	6.00	2.00	2.00	3.00	27.00	2.21	1.89	0.63	0.63	0.95	8.51	0.64	0.48	0.06	0.06	0.13	7.73
36	7.00	6.00	2.00	2.00	3.00	27.00	1.53	1.31	0.44	0.44	0.66	5.91	0.26	0.20	0.03	0.03	0.05	3.18
42	7.00	6.00	2.00	2.00	3.00	27.00	1.13	0.96	0.32	0.32	0.48	4.34	0.12	0.09	0.01	0.01	0.03	1.50

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	24	30	36	42	24	30	36	42
Q_{2020}	1.89	0.64	0.26	0.12	85	29	12	6
Q_{2030}	1.42	0.48	0.20	0.09	64	21	9	4
Q_{2040}	0.19	0.06	0.03	0.01	8	3	1	1
Q_{2050}	0.19	0.06	0.03	0.01	8	3	1	1
Q_{2060}	0.39	0.13	0.05	0.03	18	6	2	1
Q_{2070}	22.91	7.73	3.18	1.50	1,028	347	143	67

Given the flow criteria, the initial pipeline size is 36" diameter. The 36" diameter pipeline is close to the minimum flow velocity suggested but will allow more operational flexibility to transfer water into Cameron County from the Hidalgo County sources. The pipeline will need to be paralleled in 2070 when the Gulf Coast SWRO Plant increases in size and provides flows to the western end of the system.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 36" pipe. Table 7-21 below summarizes the headloss calculations.

Table 7-21: Harlingen to Cameron County Line Second Pipeline

Second Pipeline											
Q_{2070} =	27	MGD	**HL1 =HL2**								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	36		D	54		D	60		D	66	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
4	0.88	4	23	2.24	15	23	1.81	9	23	1.50	6
4.50	0.98	5	22.5	2.19	14	22.5	1.77	8	22.5	1.47	5
5.00	1.09	6	22	2.14	14	22	1.73	8	22	1.43	5
5.50	1.20	8	21.5	2.09	13	21.5	1.69	8	21.5	1.40	5
6.00	1.31	9	21	2.04	12	21	1.65	7	21	1.37	5
6.50	1.42	10	20.5	1.99	12	20.5	1.62	7	20.5	1.34	4
7.00	1.53	12	20	1.95	11	20	1.58	7	20	1.30	4

The table illustrates that a 60" diameter pipe is an appropriate choice for the second pipeline. The velocities are in the acceptable range between 0.5 ft/s and 4.0 ft/s and the pipeline headloss is around 8 ft. Utilizing a 60" diameter continues the same pipe size from the previous reaches and provides additional capacity to points east.

Because this reach is paralleled in the 2070 flow year, there is no need to verify that it is acceptable in previous flow years.

In summary, the pipeline from Harlingen to the Cameron County Line 45,000 LF of 36" pipe installed in 2020 and 45,000 LF of 60" pipe installed in 2070 to accommodate increased supply from the GCSWRO plant that comes online in 2070.

7.6.6 Cameron County Line to Mercedes Node

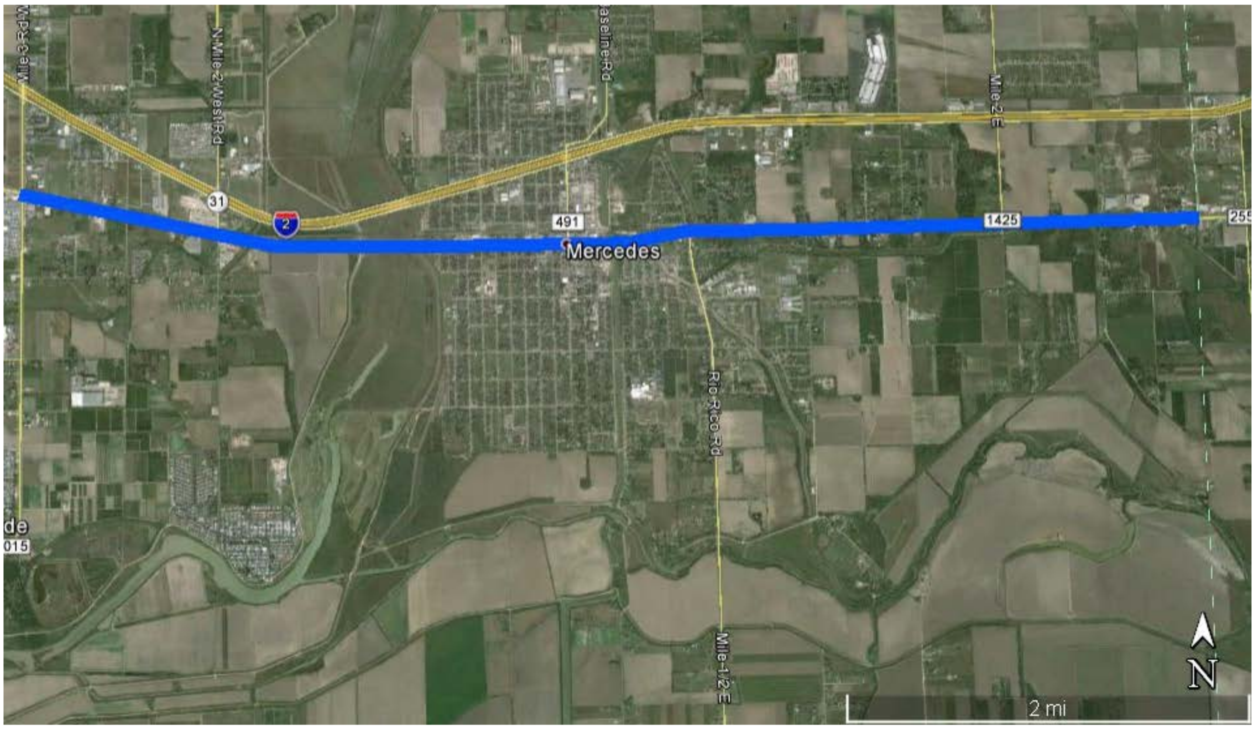


Figure 7-11: Cameron County Line to Mercedes Node Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. The flow quantity and direction is summarized in Table 7-22 below.

Table 7-22: Cameron County Line to Mercedes Node Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	24 West	31 West	25 West	20 East	24 West	47 West

Route Length

The total length of the route from the Cameron County Line Node to the Mercedes Node is 32,000 LF.

Pipe Size

Table 7-23 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-23: Cameron County Line to Mercedes Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
42	24.00	31.00	25.00	20.00	24.00	47.00	3.86	4.99	4.02	3.22	3.86	7.56	1.21	1.94	1.30	0.86	1.21	4.19
48	24.00	31.00	25.00	20.00	24.00	47.00	2.95	3.82	3.08	2.46	2.95	5.79	0.63	1.01	0.68	0.45	0.63	2.18
54	24.00	31.00	25.00	20.00	24.00	47.00	2.33	3.02	2.43	1.95	2.33	4.57	0.36	0.57	0.38	0.25	0.36	1.23
60	24.00	31.00	25.00	20.00	24.00	47.00	1.89	2.44	1.97	1.58	1.89	3.70	0.21	0.34	0.23	0.15	0.21	0.74
66	24.00	31.00	25.00	20.00	24.00	47.00	1.56	2.02	1.63	1.30	1.56	3.06	0.13	0.21	0.14	0.10	0.13	0.46

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	42	48	54	60	42	48	54	60
Q_{2020}	1.21	0.63	0.36	0.21	38	20	11	7
Q_{2030}	1.94	1.01	0.57	0.34	61	32	18	11
Q_{2040}	1.30	0.68	0.38	0.23	41	22	12	7
Q_{2050}	0.45	0.45	0.25	0.15	14	14	8	5
Q_{2060}	1.21	0.36	0.36	0.21	38	11	11	7
Q_{2070}	4.19	2.18	1.23	0.74	133	69	39	23

Given the flow criteria, the initial pipeline size is 48" diameter. The 48" diameter pipeline most closely satisfies the flow control rules. The pipeline will need to be paralleled in 2070 when the Gulf Coast SWRO Plant increases in size and provides flows to the western end of the system.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 48" pipe. Table 7-24 below summarizes the headloss calculations.

Table 7-24: Cameron County Line to Mercedes Second Pipeline

Second Pipeline											
Q_{2070} =	47	MGD	**HL1 =HL2**								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	48		D	36		D	42		D	48	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
22	2.71	17	25	5.47	87	25	3.08	25	25	3.08	22
22.50	2.77	18	24.5	3.02	29	24.5	3.02	24	24.5	3.02	21
23.50	2.89	19	23.5	2.89	27	23.5	2.89	22	23.5	2.89	19
24.50	3.02	21	22.5	2.77	25	22.5	2.77	21	22.5	2.77	18
25.50	3.14	22	21.5	2.65	23	21.5	2.65	19	21.5	2.65	16
26.50	3.26	24	20.5	2.52	21	20.5	2.52	17	20.5	2.52	15
27.50	3.39	26	19.5	2.40	19	19.5	2.40	16	19.5	2.40	14
28.50	3.51	27	18.5	2.28	17	18.5	2.28	14	18.5	2.28	12
29.50	3.63	29	17.5	2.15	16	17.5	2.15	13	17.5	2.15	11
30.50	3.76	31	16.5	2.03	14	16.5	2.03	12	16.5	2.03	10
31.50	3.88	33	15.5	1.91	12	15.5	1.91	10	15.5	1.91	9
32.50	4.00	35	14.5	1.79	11	14.5	1.79	9	14.5	1.79	8

The table illustrates that a 48" diameter pipe is an appropriate choice for the second pipeline. The velocities are in the acceptable range between 0.5 ft/s and 4.0 ft/s and the pipeline headloss is around 19 ft. Utilizing a 48” diameter provides extra capacity in the line, potentially lowers pumping costs and matches the original pipe diameter which simplifies maintenance and operation.

Because this reach is paralleled in the 2070 flow year, there is no need to verify that it is acceptable in previous flow years.

In summary, the pipeline from the Cameron County Line to the Mercedes Node is 32,000 LF of 48” pipe installed in 2020 and an additional 32,000 LF of 48” pipe installed in 2070.

7.6.7 Mercedes Node to Hidalgo County SWTP Node

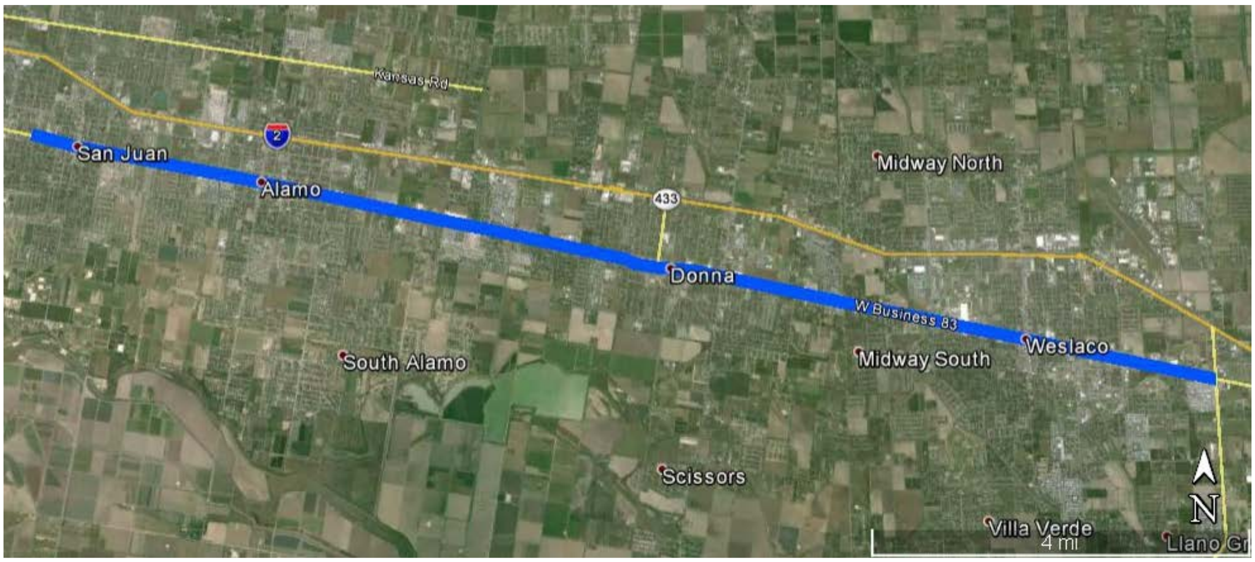


Figure 7-12: Mercedes Node to Hidalgo County Surface Water Plant Node Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. The flow quantity and direction is summarized in Table 7-25 below.

Table 7-25: Mercedes Node to HCSWTP Node Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	19 West	23 West	13 West	4 East	4 West	23 West

Route Length

The total length of the route from the Mercedes Node to the HCSWTP Node is 61,000 LF.

Pipe Size

Table 7-26 summarizes the flow, velocity, and headloss values for this section of the pipeline.

Table 7-26: Mercedes Node to HCSWTP Node Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow per Pipe, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
24	9.50	11.50	6.50	2.00	2.00	11.50	4.68	5.66	3.20	0.98	0.98	5.66	3.32	4.72	1.64	0.19	0.19	4.72
30	9.50	11.50	6.50	2.00	2.00	11.50	2.99	3.62	2.05	0.63	0.63	3.62	1.12	1.59	0.55	0.06	0.06	1.59
36	9.50	11.50	6.50	2.00	2.00	11.50	2.08	2.52	1.42	0.44	0.44	2.52	0.46	0.66	0.23	0.03	0.03	0.66
42	9.50	11.50	6.50	2.00	2.00	11.50	1.53	1.85	1.05	0.32	0.32	1.85	0.22	0.31	0.11	0.01	0.01	0.31

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	24	30	36	42	24	30	36	42
Q_{2020}	3.32	1.12	0.46	0.22	228	77	32	15
Q_{2030}	4.72	1.59	0.66	0.31	324	109	45	21
Q_{2040}	1.64	0.55	0.23	0.11	113	38	16	7
Q_{2050}	0.19	0.06	0.03	0.01	13	4	2	1
Q_{2060}	0.19	0.06	0.03	0.01	13	4	2	1
Q_{2070}	4.72	1.59	0.66	0.31	324	109	45	21

To provide redundancy throughout the length of the entire system all lines are paralleled during the program horizon; however, given the flow variation for this section the initial pipeline will consist of two identically sized pipes. The size of the lines is 42” in diameter. Though 36” diameter pipelines would have met the velocity criteria, larger lines were chosen to reduce headloss in the line, provide additional capacity to points east and to avoid adding a booster station.

Because this reach is paralleled in the 2070 flow year, there is no need to verify that it is acceptable in previous flow years.

In summary, the pipeline from the Mercedes Node to the HCSWTP Node is 61,000 LF of twinned 42” pipe installed in 2020.

7.6.8 Hidalgo County SWTP Node to McAllen Node

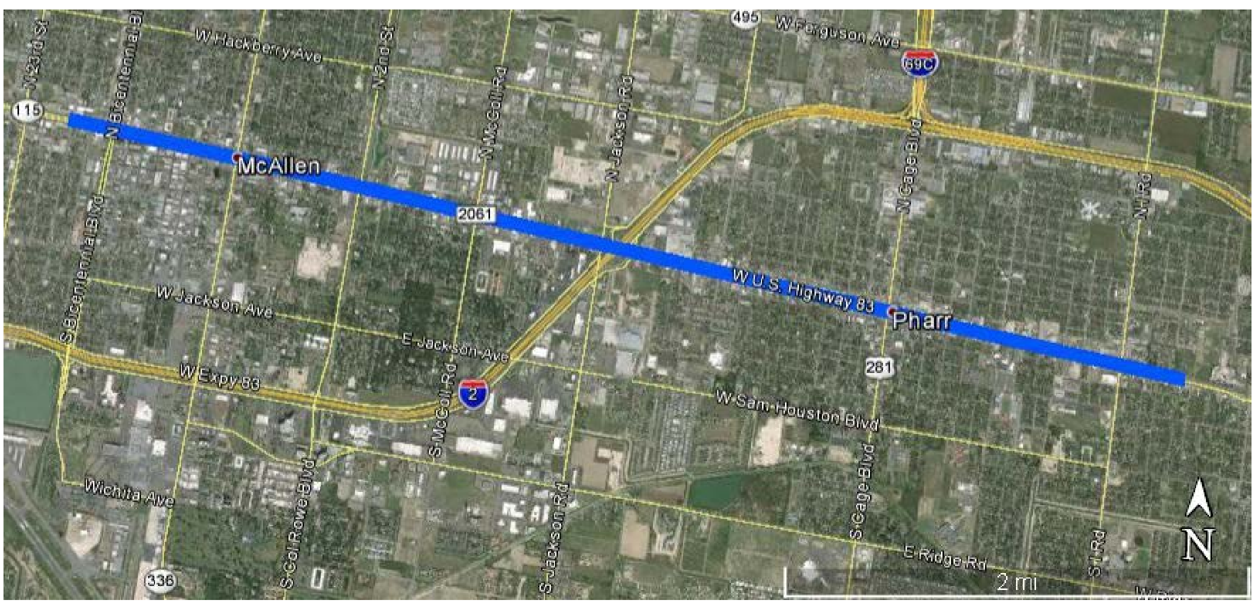


Figure 7-13: Hidalgo County SWTP to McAllen Node Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. The flow quantity and direction is summarized in Table 7-27 below.

Table 7-27: HCSWTP Node to McAllen Node Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	19 West	42 West	72 West	112 East	142 West	180 West

Route Length

The total length of the route from the HCSWTP Node to the McAllen Node is 26,000 LF.

Pipe Size

Table 7-28 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-28: HCSWTP to McAllen Node Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
66	19	42	72	112	142	180	1.24	2.74	4.69	7.29	9.25	11.72	0.09	0.38	1.02	2.31	3.58	5.56
72	19	42	72	112	142	180	1.04	2.30	3.94	6.13	7.77	9.85	0.06	0.25	0.67	1.51	2.35	3.64
78	19	42	72	112	142	180	0.89	1.96	3.36	5.22	6.62	8.39	0.04	0.17	0.45	1.02	1.59	2.46
84	19	42	72	112	142	180	0.76	1.69	2.89	4.50	5.71	7.24	0.03	0.12	0.32	0.71	1.11	1.72

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	66	72	78	84	66	72	78	84
Q_{2020}	0.09	0.06	0.04	0.03	2	1	1	1
Q_{2030}	0.38	0.25	0.17	0.12	10	7	4	3
Q_{2040}	1.02	0.67	0.45	0.32	27	18	12	8
Q_{2050}	2.31	1.51	1.02	0.71	61	40	27	19
Q_{2060}	3.58	2.35	1.59	1.11	95	62	42	29
Q_{2070}	5.56	3.64	2.46	1.72	147	96	65	45

Given the flow criteria, the initial pipeline size is 84" diameter. The 84" diameter pipeline satisfies the flow control rules. The pipeline will need to be paralleled in 2050 as the capacity of the Hidalgo County Surface Water Treatment Plant increases and provides flows to the western end of the system.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 84" pipe. Table 7-29 below summarizes the headloss calculations.

Table 7-29: HCSWTP to McAllen Node Second Pipeline

Second Pipeline											
Q_{2070} =	180	MGD	HL1 =HL2								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	84		D	78		D	84		D	90	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
75	3.02	9	105	4.90	24	105	4.22	17	105	3.68	13
80.00	3.22	10	100	4.66	22	100	4.02	15	100	3.50	12
85.00	3.42	11	95	4.43	20	95	3.82	14	95	3.33	11
90.00	3.62	13	90	4.20	18	90	3.62	13	90	3.15	10
95.00	3.82	14	85	3.96	16	85	3.42	11	85	2.98	9
100.00	4.02	15	80	3.73	15	80	3.22	10	80	2.80	8
105.00	4.22	17	75	3.50	13	75	3.02	9	75	2.63	7
110.00	4.42	18	70	3.26	11	70	2.81	8	70	2.45	6
115.00	4.62	20	65	3.03	10	65	2.61	7	65	2.28	5
120.00	4.82	21	60	2.80	9	60	2.41	6	60	2.10	5
125.00	5.03	23	55	2.56	7	55	2.21	5	55	1.93	4
130.00	5.23	25	50	2.33	6	50	2.01	4	50	1.75	3

The table illustrates that an 84" diameter pipe is the appropriate choice for the second pipeline. The velocities are in the acceptable range between 1.0 ft/s and 4.0 ft/s and the pipeline headloss is around 13 ft. Because this reach is paralleled in the 2050 flow year, it is necessary to verify that the flow in the pipeline system meets the flow requirements in both 2050 and 2060. Table 7-30 shows the headloss calculations used to verify the velocities are within the acceptable range.

Table 7-31: HCSWTP to McAllen Node Verification

Verify Second Pipe Works for Previous Flow Years											
Q_{2060} =	142	MGD	HL1 =HL2			Q_{2050} =	112	MGD	HL1 =HL2		
Original Pipeline						Original Pipeline					
C	120		C	120		C	120		C	120	
D	84		D	84		D	84		D	84	
Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L
70	2.81	7.9	72	2.89	8.3	55	2.21	5.1	57	2.29	5.4
70.50	2.83	8.0	71.5	2.87	8.2	55.20	2.22	5.1	56.8	2.28	5.4
71.00	2.85	8.1	71	2.85	8.1	55.40	2.23	5.1	56.6	2.28	5.3
71.50	2.87	8.2	70.5	2.83	8.0	55.60	2.24	5.2	56.4	2.27	5.3
72.00	2.89	8.3	70	2.81	7.9	55.80	2.24	5.2	56.2	2.26	5.3
72.50	2.91	8.4	69.5	2.79	7.8	56.00	2.25	5.2	56	2.25	5.2
73.00	2.93	8.5	69	2.77	7.7	56.20	2.26	5.3	55.8	2.24	5.2
73.50	2.95	8.7	68.5	2.75	7.6	56.40	2.27	5.3	55.6	2.24	5.2
74.00	2.98	8.8	68	2.73	7.5	56.60	2.28	5.3	55.4	2.23	5.1
74.50	3.00	8.9	67.5	2.71	7.4	56.80	2.28	5.4	55.2	2.22	5.1
75.00	3.02	9.0	67	2.69	7.3	57.00	2.29	5.4	55	2.21	5.1
75.50	3.04	9.1	66.5	2.67	7.2	57.20	2.30	5.4	54.8	2.20	5.0

In summary, the pipeline from the Hidalgo County Surface Water Treatment Plant Node to the McAllen Node is 26,000 LF of 84" pipe installed in 2020 and 26,000 LF of 84" pipe installed in 2050 to accommodate increased supply from the Hidalgo County Surface Water Treatment Plant.

7.6.9 McAllen Node to Mission Node

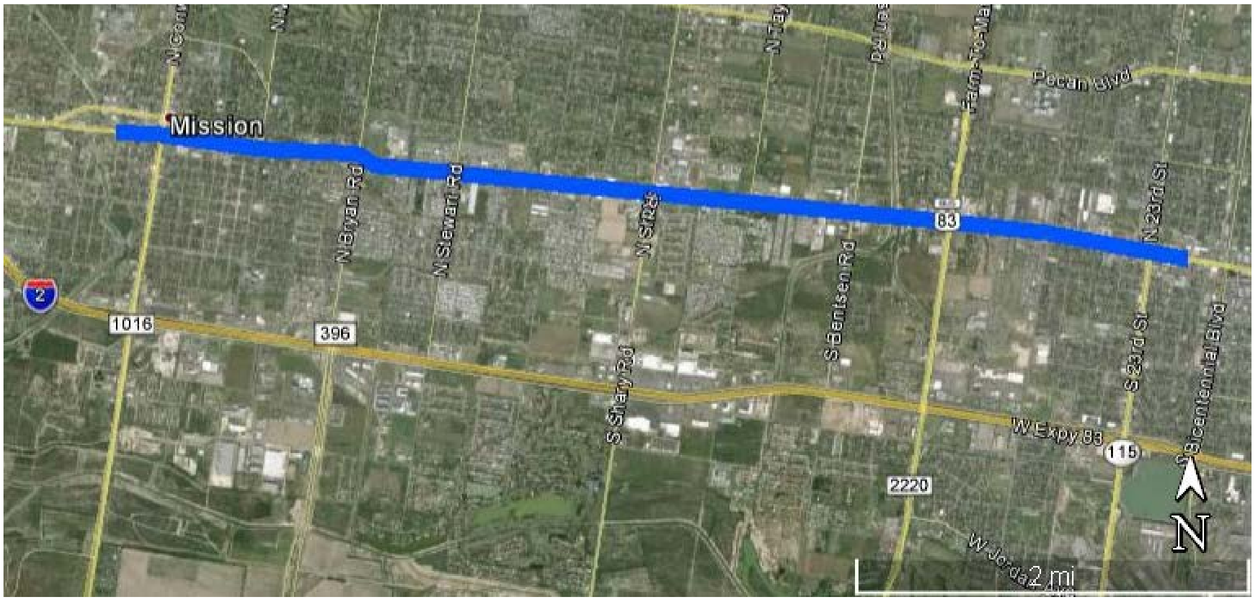


Figure 7-14: McAllen Node to Mission Node Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. The flow quantity and direction is summarized in Table 7-32.

Table 7-32: McAllen Node to Mission Node Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	7 West	11 West	19 West	30 East	33 West	45 West

Route Length

The total length of the route from the McAllen Node to the Mission Node is 29,000 LF.

Pipe Size

Table 7-33 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-33: McAllen Node to Mission Node Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
42	7	11	19	30	33	45	1.13	1.77	3.06	4.82	5.31	7.24	0.12	0.29	0.78	1.82	2.18	3.86
48	7	11	19	30	33	45	0.86	1.35	2.34	3.69	4.06	5.54	0.06	0.15	0.41	0.95	1.14	2.02
54	7	11	19	30	33	45	0.68	1.07	1.85	2.92	3.21	4.38	0.04	0.08	0.23	0.54	0.64	1.14
60	7	11	19	30	33	45	0.55	0.87	1.50	2.36	2.60	3.55	0.02	0.05	0.14	0.32	0.38	0.68

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	42	48	54	60	42	48	54	60
Q_{2020}	0.12	0.06	0.04	0.02	4	2	1	1
Q_{2030}	0.29	0.15	0.08	0.05	8	4	2	1
Q_{2040}	0.78	0.41	0.23	0.14	23	12	7	4
Q_{2050}	1.82	0.95	0.54	0.32	53	28	16	9
Q_{2060}	2.18	1.14	0.64	0.38	63	33	19	11
Q_{2070}	3.86	2.02	1.14	0.68	112	59	33	20

Given the flow criteria, the initial pipeline size is 48" diameter. The 48" diameter pipeline satisfies the flow control rules. Though a 42" line would accommodate the velocity restrictions a 48" was selected to add future capacity and reduce headloss in the pipe. The pipeline will need to be paralleled in 2050 as the capacity of the Hidalgo County Surface Water Treatment Plant increases and provides flows to the western end of the system.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 48" pipe. Table 7-34 below summarizes the headloss calculations.

Table 7-34: McAllen to Mission Second Pipeline

Second Pipeline											
Q_{2070} =	45	MGD	HL1 =HL2								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D Q (MGD)	48 V (ft/s)	H_L	D Q	42 V	H_L	D Q	48 V	H_L	D Q	54 V	H_L
21	2.59	14	24	3.86	35	24	2.95	18	24	2.33	10
21.50	2.65	15	23.5	3.78	34	23.5	2.89	18	23.5	2.29	10
22.00	2.71	16	23	3.70	32	23	2.83	17	23	2.24	10
22.50	2.77	16	22.5	3.62	31	22.5	2.77	16	22.5	2.19	9
23.00	2.83	17	22	3.54	30	22	2.71	16	22	2.14	9
23.50	2.89	18	21.5	3.46	29	21.5	2.65	15	21.5	2.09	8
24.00	2.95	18	21	3.38	27	21	2.59	14	21	2.04	8

The table illustrates that a 48" diameter pipe is the appropriate choice for the second pipeline. The velocities are in the acceptable range between 1.0 ft/s and 5.0 ft/s and the pipeline headloss is around 16 ft. and the 48" pipe is better from an operational standpoint because it is the same diameter as the original pipe and the added capacity will reduce the need for a booster pump in the 2070 decade. Since this reach is paralleled in 2050, it is necessary to verify that flows and headloss in the parallel 48" pipe also work for the previous flow years. Table 7-35 shows the headloss calculations used to verify the velocities are within the acceptable range.

Table 7-36: McAllen to Mission Verification

Verify Second Pipeline Works for Previous Flow Years												
Q_{2060} =	33	MGD	HL1 =HL2				Q_{2050} =	30	MGD	HL1 =HL2		
Original Pipeline			Parallel Pipeline				Original Pipeline			Parallel Pipeline		
C	120		C	120			C	120		C	120	
D	48		D	48			D	48		D	48	
Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L		Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L
15	1.85	8	18	2.22	11		14	1.72	7	16	1.97	9
15.50	1.91	8	17.5	2.15	10		14.50	1.79	7	15.50	1.91	8
16.00	1.97	9	17	2.09	10		15.00	1.85	8	15	1.85	8
16.50	2.03	9	16.5	2.03	9		15.50	1.91	8	14.5	1.79	7
17.00	2.09	10	16	1.97	9		16.00	1.97	9	14	1.72	7
17.50	2.15	10	15.5	1.91	8		16.50	2.03	9	13.5	1.66	6
18.00	2.22	11	15	1.85	8		17.00	2.09	10	13	1.60	6

In summary, the pipeline from the McAllen Node to the Mission Node is 29,000 LF of 48" pipe installed in 2020 and an additional 29,000 LF of 48" pipe installed in 2050 to accommodate increased supply from the Hidalgo County Surface Water Treatment Plant.

7.6.10 Mission Node to the West

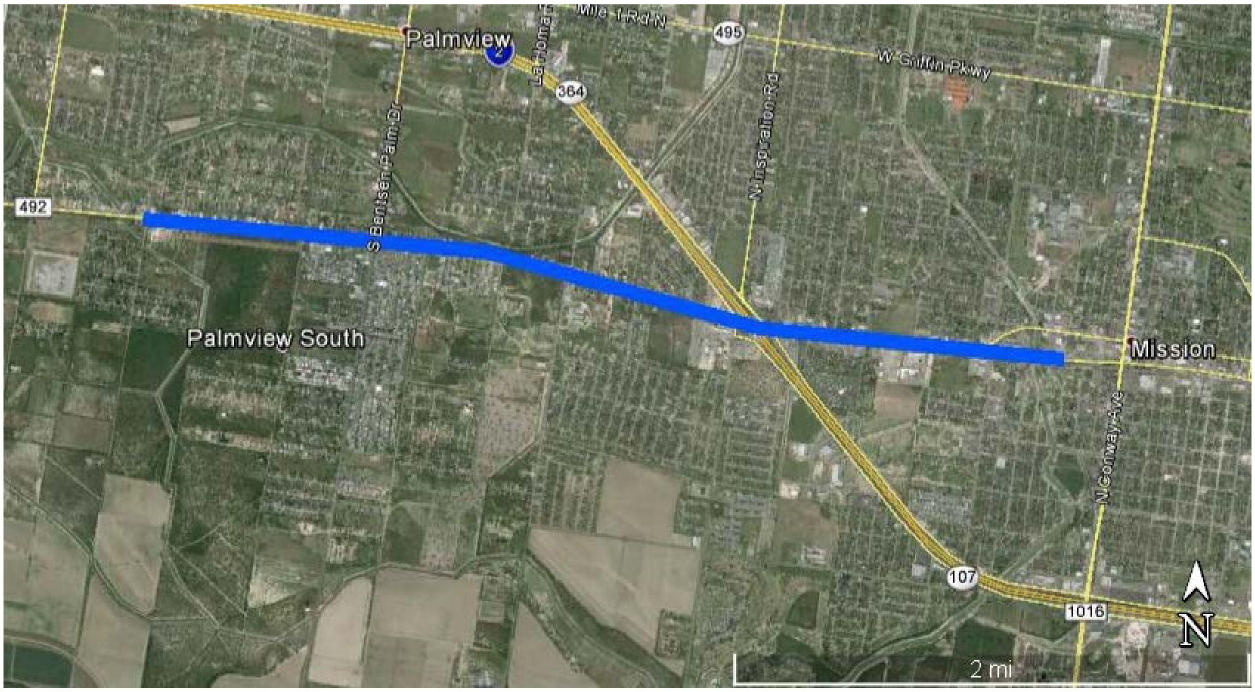


Figure 7-15: Mission Node West Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. The flow quantity and direction is summarized in Table 7-37.

Table 7-37: West Pipeline Flows

	2020	2030	2040	2050	2060	2070
Flow (MGD)	0 West	1 West	1 West	4 West	6 West	8 West

Route Length

The total length of the route from the McAllen Node to the Mission Node is 19,000 LF.

Pipe Size

Table 7-38 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-38: Mission Node West Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
12	0.00	1.00	1.00	4.00	6.00	8.00	0.00	1.97	1.97	7.88	11.82	15.76	0.00	1.5	1.5	19.6	41.5	70.6
16	0.00	1.00	1.00	4.00	6.00	8.00	0.00	1.11	1.11	4.43	6.65	8.86	0.00	0.4	0.4	4.8	10.2	17.4
20	0.00	1.00	1.00	4.00	6.00	8.00	0.00	0.71	0.71	2.84	4.26	5.67	0.00	0.13	0.13	1.63	3.45	5.87
24	0.00	1.00	1.00	4.00	6.00	8.00	0.00	0.49	0.49	1.97	2.95	3.94	0.00	0.05	0.05	0.67	1.42	2.41

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	12	16	20	24	12	16	20	24
Q_{2020}	0.0	0.0	0.0	0.0	-	-	-	-
Q_{2030}	1.5	0.4	0.1	0.1	59	15	5	2
Q_{2040}	1.5	0.4	0.1	0.1	59	15	5	2
Q_{2050}	19.6	4.8	1.6	0.7	765	188	64	26
Q_{2060}	41.5	10.2	3.4	1.4	1,620	399	135	55
Q_{2070}	70.6	17.4	5.9	2.4	2,758	679	229	94

Given the flow criteria, the initial pipeline size is 24" diameter. The 24" diameter pipeline satisfies the flow control rules. Though the 20" diameter would meet the velocity requirements, the 24" diameter reduces headloss in the pipe and reduces the need for a booster pumping station in the 2070 decade. The pipeline will need to be paralleled in 2060 as the demand to the west of the Mission Node increases.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 20" pipe. Table 7-39 below summarizes the headloss calculations.

Table 7-39: Mission Node West Second Pipeline

Second Pipeline											
Q_{2070} =	8	MGD	HL1 =HL2								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	24		D	20		D	24		D	30	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
3.7	1.82	23	4.3	3.05	73	4.3	2.12	30	4.3	1.36	13
3.8	1.87	24	4.2	2.98	70	4.2	2.07	29	4.2	1.32	13
3.9	1.92	25	4.1	2.91	67	4.1	2.02	27	4.1	1.29	12
4.0	1.97	26	4.0	2.84	64	4.0	1.97	26	4.0	1.26	11
4.1	2.02	27	3.9	2.77	61	3.9	1.92	25	3.9	1.23	11
4.2	2.07	29	3.8	2.69	58	3.8	1.87	24	3.8	1.20	10

The table illustrates that a 24" diameter pipe is an appropriate choice for the second pipeline. The velocities are in the acceptable range between 1.0 ft/s and 5.0 ft/s and the pipeline headloss is around 26 ft. Since this reach is paralleled in 2060, it is necessary to verify that flows and headloss in the 24" twin pipe also work for the 2060 flow years. Table 7-40 shows the headloss calculations used to verify the velocities are within the acceptable range.

Table 7-41: Mission Node West Second Pipe Verification

Verify Second Pipe Works for Previous Flow Years					
Q_{2060} =	6	MGD	**HL1 =HL2**		
Original Pipeline					
C	120		C	120	
D	24		D	24	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L
2.8	1.38	14	3.2	1.58	17
2.90	1.43	14	3.1	1.53	16
3.00	1.48	15	3	1.48	15
3.10	1.53	16	2.9	1.43	14
3.20	1.58	17	2.8	1.38	14
3.30	1.63	18	2.7	1.33	13
3.40	1.67	19	2.6	1.28	12
3.50	1.72	20	2.5	1.23	11
3.60	1.77	22	2.4	1.18	10
3.70	1.82	23	2.3	1.13	9
3.80	1.87	24	2.2	1.08	9
3.90	1.92	25	2.1	1.03	8

In summary, the pipeline from the Mission Node to the west is 19,000 LF of 24” pipe installed in 2020 and an additional 19,000 LF of 24” pipe installed in 2060 to accommodate increased demand from the users west of the Mission Node.

7.6.11 Cameron BGD to Military Highway

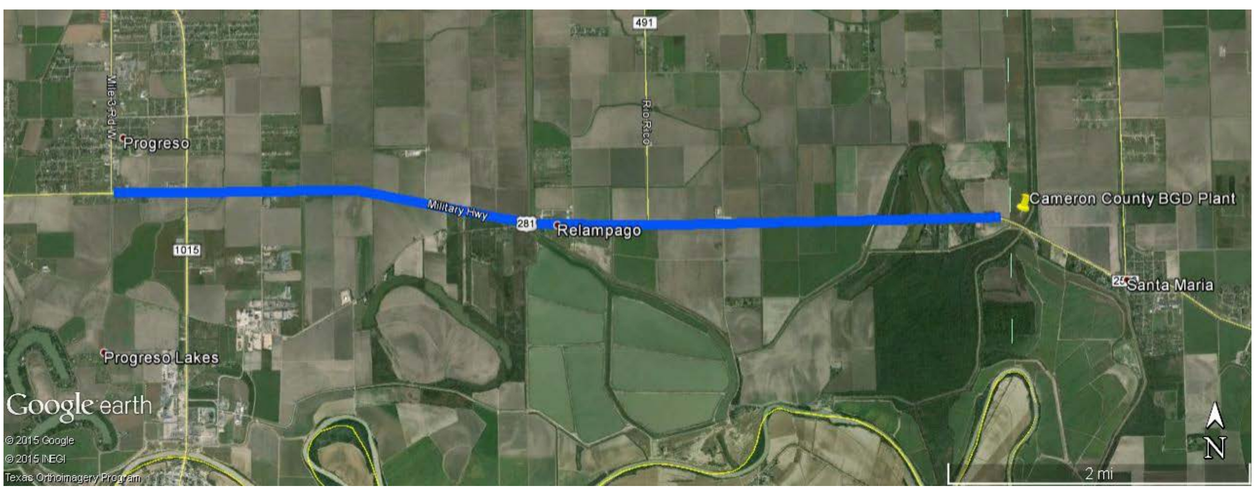


Figure 7-16: Cameron BGD to Military HWY Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2020, the Cameron BGD produces 18 MGD. The Military Highway Node has a demand of 1 MGD. Therefore in 2020, 1 MGD is flowing from the

Cameron County BGD Node to the Military Highway Node. The flow quantity and direction is summarized in Table 7-42.

Table 7-42: Military HWY Pipeline Flows

	2020	2030	2040	2050	2060	2070
Flow (MGD)	1 West	2 West	4 West	5 West	6 West	7 West

Route Length

The total length of the route from the Cameron BGD Node to the Military Highway Node is 31,000 LF.

Pipe Size

Table 7-43 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-43: Cameron BGD to Military Hwy Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
12	0.98	1.83	2.72	3.70	4.69	5.71	1.93	3.61	5.36	7.30	9.23	11.26	1.46	4.61	9.61	16.99	26.25	37.87
16	0.98	1.83	2.72	3.70	4.69	5.71	1.09	2.03	3.02	4.11	5.19	6.33	0.36	1.14	2.37	4.19	6.47	9.33
18	0.98	1.83	2.72	3.70	4.69	5.71	0.86	1.60	2.38	3.24	4.10	5.00	0.20	0.64	1.33	2.36	3.64	5.26
20	0.98	1.83	2.72	3.70	4.69	5.71	0.70	1.30	1.93	2.63	3.32	4.05	0.12	0.38	0.80	1.41	2.18	3.15
24	0.98	1.83	2.72	3.70	4.69	5.71	0.48	0.90	1.34	1.82	2.31	2.81	0.05	0.16	0.33	0.58	0.90	1.30

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	12	16	18	20	12	16	18	20
Q_{2020}	1.46	0.36	0.20	0.12	45	11	6	4
Q_{2030}	4.61	1.14	0.64	0.38	143	35	20	12
Q_{2040}	9.61	2.37	1.33	0.80	298	73	41	25
Q_{2050}	16.99	4.19	2.36	1.41	527	130	73	44
Q_{2060}	26.25	6.47	3.64	2.18	814	200	113	68
Q_{2070}	37.87	9.33	5.26	3.15	1,174	289	163	98

Given the flow criteria, the initial pipeline size is 20" diameter. The 20" diameter pipeline satisfies the flow control rules. The pipeline may need to be paralleled at some point to provide a redundant supply line; however, no parallel pipe will be necessary for purely capacity reasons.

In summary, the pipeline from the Cameron County BGD Node to the Military Highway Node is 31,000 LF of 20" pipe installed in 2020.

7.6.12 Cameron BGD North

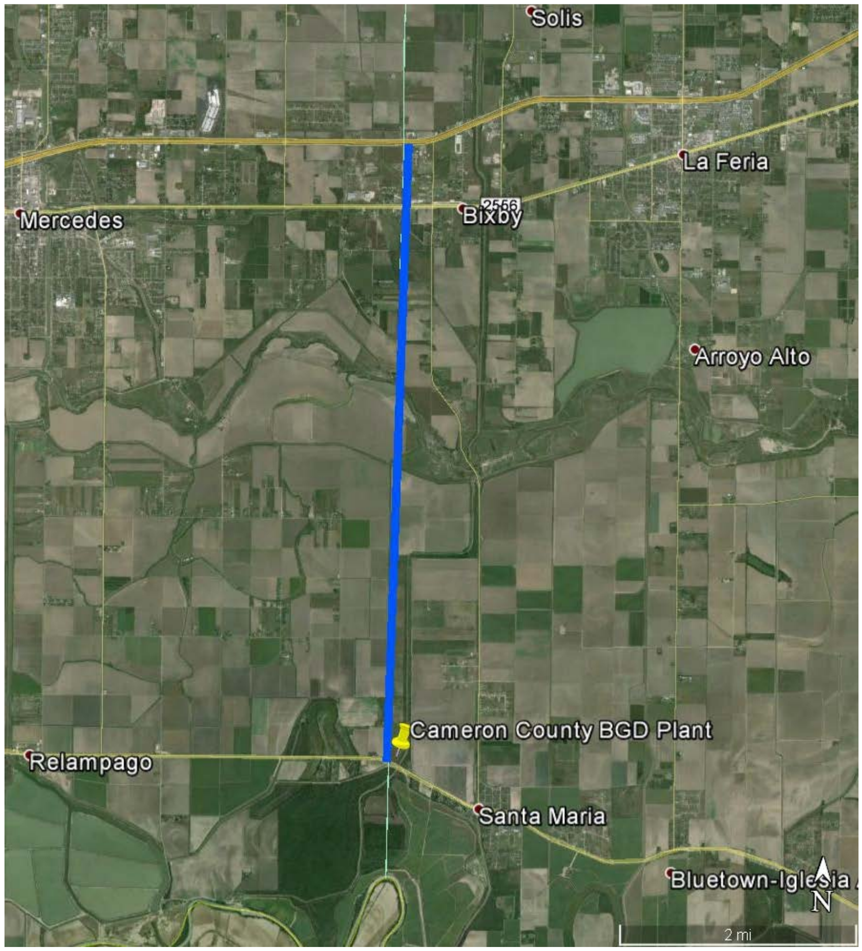


Figure 7-17: Cameron BGD North Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2020, the Cameron BGD produces 18 MGD. The Military Highway Node has a demand of 1 MGD. Therefore in 2020, 17 MGD is flowing from the Cameron County BGD Node to the North. The flow quantity and direction is summarized in Table 7-44.

Table 7-44: Military HWY Pipeline Flows

	2020	2030	2040	2050	2060	2070
Flow (MGD)	17 West	25 West	23 West	22 West	21 West	20 West

Route Length

The total length of the route from the Cameron BGD Node to the Military Highway Node is 30,500 LF.

Pipe Size

Table 7-45 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-45: Cameron BGD North Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
30	17.00	25.00	23.00	22.00	21.00	20.00	5.36	7.88	7.25	6.93	6.62	6.30	3.28	6.70	5.74	5.29	4.85	4.44
36	17.00	25.00	23.00	22.00	21.00	20.00	3.72	5.47	5.03	4.82	4.60	4.38	1.35	2.76	2.36	2.18	2.00	1.83
42	17.00	25.00	23.00	22.00	21.00	20.00	2.73	4.02	3.70	3.54	3.38	3.22	0.64	1.30	1.12	1.03	0.94	0.86
48	17.00	25.00	23.00	22.00	21.00	20.00	2.09	3.08	2.83	2.71	2.59	2.46	0.33	0.68	0.58	0.54	0.49	0.45

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	30	36	42	48	30	36	42	48
Q_{2020}	3.28	1.35	0.64	0.33	100	41	19	10
Q_{2030}	6.70	2.76	1.30	0.68	204	84	40	21
Q_{2040}	5.74	2.36	1.12	0.58	175	72	34	18
Q_{2050}	5.29	2.18	1.03	0.54	161	66	31	16
Q_{2060}	4.85	2.00	0.94	0.49	148	61	29	15
Q_{2070}	4.44	1.83	0.86	0.45	135	56	26	14

Given the flow criteria, the initial pipeline size is 42" diameter. The 42" diameter pipeline satisfies the flow control rules. The pipeline may need to be paralleled at some point to provide a redundant supply line; however, it will not be necessary for purely capacity reasons.

In summary, the pipeline from the Cameron County BGD Node to the Military Highway Node is 30,500 LF of 42" pipe installed in 2020.

7.6.13 Hidalgo County BGD East

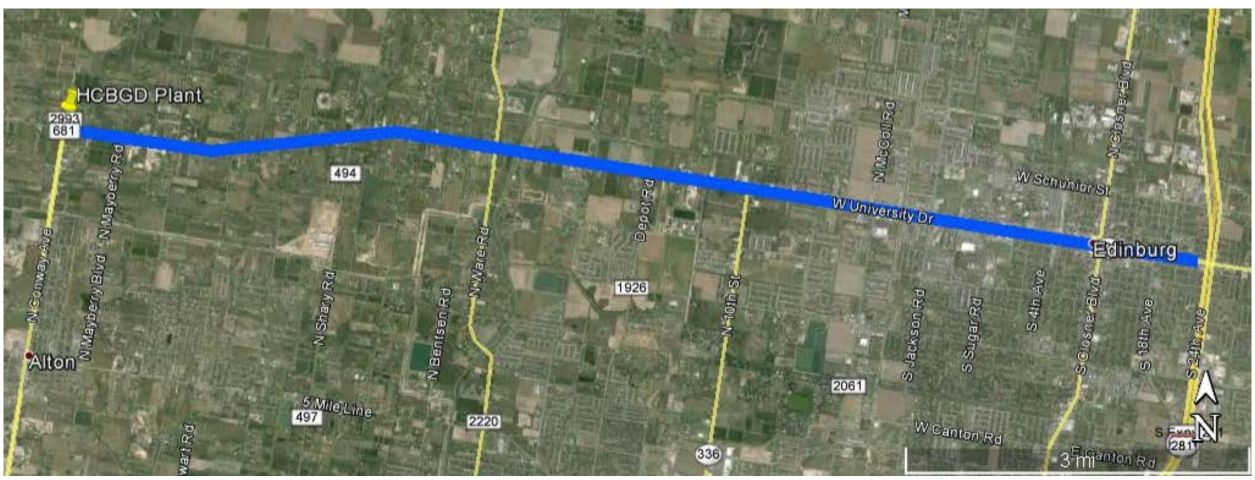


Figure 7-18: Hidalgo County BGD to Edinburg Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2030, the Hidalgo BGD produces 8 MGD. The Edinburg Node has a demand of 9 MGD. The supplies from the Hidalgo BGD are always less than the demand in the Edinburg node, so the flow from the Hidalgo BGD was evenly split between Edinburg and Mission. The flow quantity and direction is summarized in Table 7-46.

Table 7-46: HCBGD to Edinburg Pipeline Flows

	2020	2030	2040	2050	2060	2070
Flow (MGD)	0	4 East	4 East	4 East	4 East	4 East

Route Length

The total length of the route from the Hidalgo BGD Node to the Edinburg Node is 53,000 LF.

Pipe Size

Table 7-47 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-47: Hidalgo BGD to Edinburg Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
12	0.00	4.00	4.00	4.00	4.00	4.00	0.00	7.88	7.88	7.88	7.88	7.88	0.00	19.58	19.58	19.58	19.58	19.58
16	0.00	4.00	4.00	4.00	4.00	4.00	0.00	4.43	4.43	4.43	4.43	4.43	0.00	4.82	4.82	4.82	4.82	4.82
18	0.00	4.00	4.00	4.00	4.00	4.00	0.00	3.50	3.50	3.50	3.50	3.50	0.00	2.72	2.72	2.72	2.72	2.72
20	0.00	4.00	4.00	4.00	4.00	4.00	0.00	2.84	2.84	2.84	2.84	2.84	0.00	1.63	1.63	1.63	1.63	1.63

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	12	16	18	20	12	16	18	20
Q_{2020}	0.00	0.00	0.00	0.00	-	-	-	-
Q_{2030}	19.58	4.82	2.72	1.63	1,038	256	144	86
Q_{2040}	19.58	4.82	2.72	1.63	1,038	256	144	86
Q_{2050}	19.58	4.82	2.72	1.63	1,038	256	144	86
Q_{2060}	19.58	4.82	2.72	1.63	1,038	256	144	86
Q_{2070}	19.58	4.82	2.72	1.63	1,038	256	144	86

Given the flow criteria, the initial pipeline size is 20" diameter. The 20" diameter pipeline satisfies the flow control rules.

In summary, the pipeline from the Hidalgo County BGD Node to the Edinburg Node is 53,000 LF of 20" pipe installed in 2020.

7.6.14 Hidalgo County BGD South

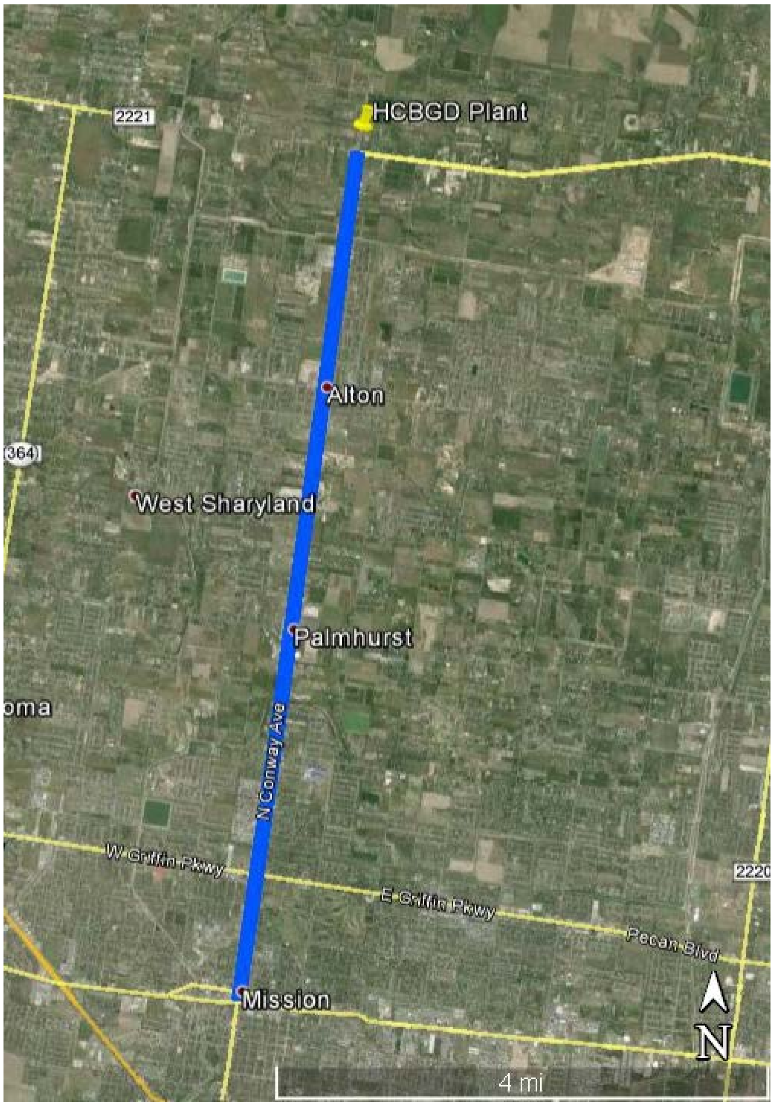


Figure 7-19: Hidalgo County BGD South Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2030, the Hidalgo BGD produces 8 MGD. The Mission Node has a demand of 14 MGD. The supplies from the Hidalgo BGD are always less than the demand in the Mission node, so the flow from the Hidalgo BGD was evenly split between Edinburg and Mission. The flow quantity and direction is summarized in Table 7-48 below.

Table 7-48: HCBGD to Mission Pipeline Flows

	2020	2030	2040	2050	2060	2070
Flow (MGD)	0	4 South	4 South	4 South	4 South	4 South

Route Length

The total length of the route from the Hidalgo BGD Node to the Edinburg Node is 37,000 LF.

Pipe Size

Table 7-49 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-49: Hidalgo BGD to Mission Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
12	0.00	4.00	4.00	4.00	4.00	4.00	0.00	7.88	7.88	7.88	7.88	7.88	0.00	19.58	19.58	19.58	19.58	19.58
16	0.00	4.00	4.00	4.00	4.00	4.00	0.00	4.43	4.43	4.43	4.43	4.43	0.00	4.82	4.82	4.82	4.82	4.82
18	0.00	4.00	4.00	4.00	4.00	4.00	0.00	3.50	3.50	3.50	3.50	3.50	0.00	2.72	2.72	2.72	2.72	2.72
20	0.00	4.00	4.00	4.00	4.00	4.00	0.00	2.84	2.84	2.84	2.84	2.84	0.00	1.63	1.63	1.63	1.63	1.63

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	12	16	18	20	12	16	18	20
Q_{2020}	0.00	0.00	0.00	0.00	-	-	-	-
Q_{2030}	19.58	4.82	2.72	1.63	724	178	101	60
Q_{2040}	19.58	4.82	2.72	1.63	724	178	101	60
Q_{2050}	19.58	4.82	2.72	1.63	724	178	101	60
Q_{2060}	19.58	4.82	2.72	1.63	724	178	101	60
Q_{2070}	19.58	4.82	2.72	1.63	724	178	101	60

Given the flow criteria, the initial pipeline size is 20” diameter. The 20” diameter pipeline satisfies the flow control rules.

In summary, the pipeline from the Hidalgo County BGD Node to the Mission Node is 37,000 LF of 20” pipe installed in 2020.

7.6.15 Hidalgo County SWTP North

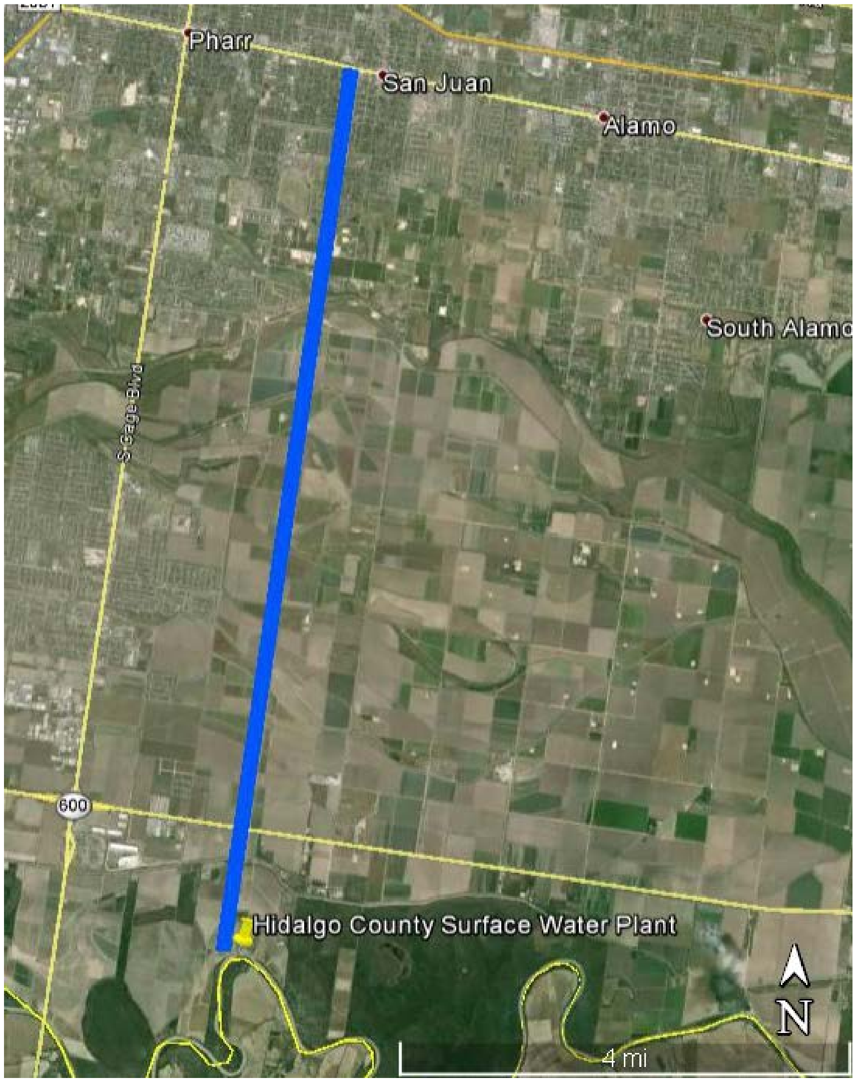


Figure 7-20: Hidalgo County SWTP North Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. For example, in 2030, the Hidalgo SWTP produces 20 MGD. The Hidalgo Node has a demand of 1 MGD. Therefore in 2030, 19 MGD is flowing from the HCSWTP the north. The flow quantity and direction is summarized in Table 7-50 below.

Table 7-50: HCSWTP to the North Pipeline Flows

	2020	2030	2040	2050	2060	2070
Flow (MGD)	0 North	19 North	59 North	108 North	138 North	157 North

Route Length

The total length of the route from the McAllen Node to the Mission Node is 42,000 LF.

Pipe Size

Table 7-51 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-51: HCSWTP North Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
66	0	19	59	108	138	157	0.00	1.24	3.84	7.03	8.99	10.22	0.00	0.09	0.71	2.16	3.40	4.31
72	0	19	59	108	138	157	0.00	1.04	3.23	5.91	7.55	8.59	0.00	0.06	0.46	1.41	2.22	2.82
78	0	19	59	108	138	157	0.00	0.89	2.75	5.04	6.43	7.32	0.00	0.04	0.31	0.96	1.51	1.91
84	0	19	59	108	138	157	0.00	0.76	2.37	4.34	5.55	6.31	0.00	0.03	0.22	0.67	1.05	1.33

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	66	72	78	84	66	72	78	84
Q_{2020}	0.00	0.00	0.00	0.00	-	-	-	-
Q_{2030}	0.09	0.06	0.04	0.03	4	2	2	1
Q_{2040}	0.71	0.46	0.31	0.22	30	19	13	9
Q_{2050}	2.16	1.41	0.96	0.67	91	59	40	28
Q_{2060}	3.40	2.22	1.51	1.05	143	93	63	44
Q_{2070}	4.31	2.82	1.91	1.33	181	119	80	56

Given the flow criteria, the initial pipeline size is 78" diameter. The 78" diameter pipeline satisfies the flow control rules. The pipeline will need to be paralleled in 2050 as the demand to the north increases.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 78" pipe. Table 7-52 below summarizes the headloss calculations.

Table 7-52: HCSWTP North Second Pipeline

Second Pipeline											
Q_{2070} =	157	MGD	**HL1 =HL2**								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	78		D	72		D	78		D	84	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
73.5	3.43	20	83.5	4.57	37	83.5	3.89	25	83.5	3.36	19
78.5	3.66	22	78.5	4.30	33	78.5	3.66	22	78.5	3.16	17
83.5	3.89	25	73.5	4.02	29	73.5	3.43	20	73.5	2.95	15
88.5	4.13	28	68.5	3.75	26	68.5	3.19	17	68.5	2.75	13
93.5	4.36	31	63.5	3.47	22	63.5	2.96	15	63.5	2.55	11

The table illustrates that a 78" diameter pipe is the appropriate choice for the second pipeline. The velocities are in the acceptable range between 1.0 ft/s and 5.0 ft/s and the pipeline headloss is around 22 ft. Since this reach is paralleled in 2050, it is necessary to verify that velocity and headloss in the 78" parallel pipe also work for the 2050 and 2060 flows. Table 7-53 shows the headloss calculations used to verify the velocities are within the acceptable range.

Table 7-54: HCSWTP North Second Pipeline Verification

Verify Second Pipe Works for Previous Flow Years												
Q_{2060} =	138	MGD	**HL1 =HL2**				Q_{2050} =	108	MGD	**HL1 =HL2**		
Original Pipeline							Original Pipeline					
C	120		C	120			C	120		C	120	
D	78		D	78			D	78		D	78	
Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L		Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L
67	3.12	17	71	3.31	19		52	2.42	10	56	2.61	12
68	3.17	17	70	3.26	18		53	2.47	11	55	2.56	12
69	3.22	18	69	3.22	18		54	2.52	11	54	2.52	11
70	3.26	18	68	3.17	17		55	2.56	12	53	2.47	11
71	3.31	19	67	3.12	17		56	2.61	12	52	2.42	10
72	3.36	19	66	3.08	16		57	2.66	12	51	2.38	10

In summary, the pipeline from the HCSWTP to the north is 42,000 LF of 78” pipe installed in 2030 and an additional 42,000 LF of 78” pipe installed in 2050 to accommodate increased production from the HCSWTP.

7.6.16 McAllen Node to Edinburg Node

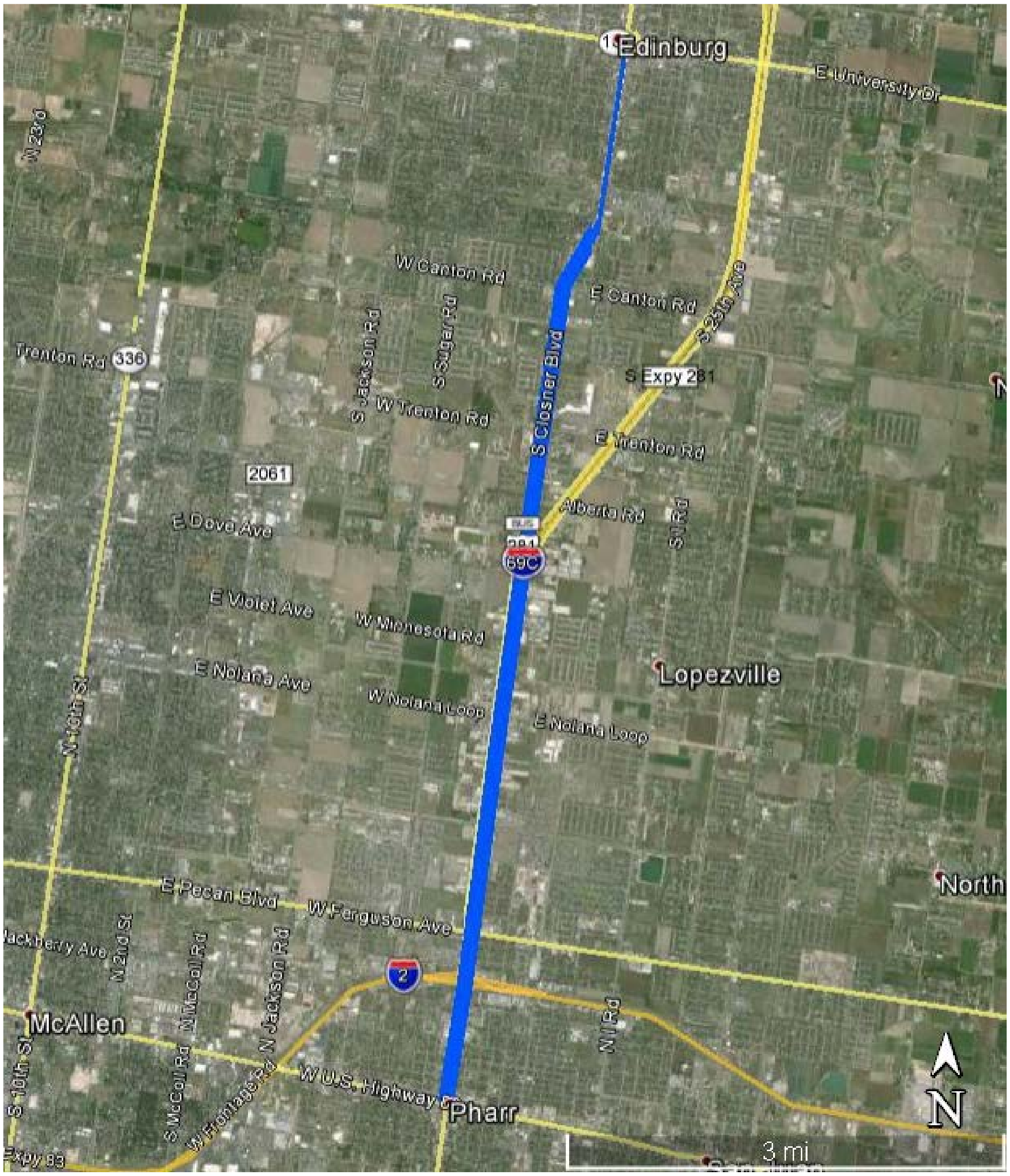


Figure 7-21: McAllen Node to Mission Node Map

Pipeline Flow

The overall flow in the pipeline is calculated by subtracting the node demands from the node supplies for a given flow decade. The flow quantity and direction is summarized in Table 7-55.

Table 7-55: McAllen Node to Mission Node Flow

	2020	2030	2040	2050	2060	2070
Flow (MGD)	4 North	5 North	10 North	20 North	28 North	37 North

Route Length

The total length of the route from the McAllen Node to the Edinburg Node is 42,000 LF.

Pipe Size

Table 7-56 summarizes the flow, velocity, and headloss values for this section of pipe.

Table 7-56: McAllen Node to Edinburg Node Pipeline Velocity & Headloss

Pipeline Diameter Calculation																		
Pipe Size	Flow, MGD						Velocity, Ft/S						Headloss, ft/1000ft					
Inches	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
24	4.0	5.0	10.0	20.0	28.0	37.0	1.97	2.46	4.92	9.85	13.79	18.22	0.67	1.01	3.65	13.15	24.50	41.04
30	4.0	5.0	10.0	20.0	28.0	37.0	1.26	1.58	3.15	6.30	8.83	11.66	0.23	0.34	1.23	4.44	8.27	13.84
36	4.0	5.0	10.0	20.0	28.0	37.0	0.88	1.09	2.19	4.38	6.13	8.10	0.09	0.14	0.51	1.83	3.40	5.70
42	4.0	5.0	10.0	20.0	28.0	37.0	0.64	0.80	1.61	3.22	4.50	5.95	0.04	0.07	0.24	0.86	1.61	2.69

Pipeline Diameter Calculation								
	H_L/1000 ft				H_L Total			
Phase/Dia	24	30	36	42	24	30	36	42
Q_{2020}	0.67	0.23	0.09	0.04	28	10	4	2
Q_{2030}	1.01	0.34	0.14	0.07	43	14	6	3
Q_{2040}	3.65	1.23	0.51	0.24	154	52	21	10
Q_{2050}	13.15	4.44	1.83	0.86	555	187	77	36
Q_{2060}	24.50	8.27	3.40	1.61	1,035	349	144	68
Q_{2070}	41.04	13.84	5.70	2.69	1,733	585	241	114

Given the flow criteria, the initial pipeline size is 42" diameter. The 42" diameter pipeline satisfies the flow control rules and provides a constant pipe diameter from the previous reach. The pipeline will need to be paralleled in 2060 as the capacity of the Hidalgo County Surface Water Treatment Plant increases and provides flows to Edinburg.

In order to determine the proper diameter for the parallel pipe, the Hazen-Williams formula is used again, but this time the pipe is sized based on the ultimate flow in 2070 and setting the headloss in the parallel pipe equal to the headloss in the original 42" pipe. Table 7-57 below summarizes the headloss calculations.

Table 7-57: McAllen to Edinburg Second Pipeline

Second Pipeline											
Q_{2070} =	37	MGD	**HL1 =HL2**								
Original Pipeline			Twin Option 1			Twin Option 2			Twin Option 3		
C	120		C	120		C	120		C	120	
D	42		D	42		D	48		D	54	
Q (MGD)	V (ft/s)	H_L	Q	V	H_L	Q	V	H_L	Q	V	H_L
17	2.73	27	20	3.22	36	20	2.46	19	20	1.95	12
17.50	2.81	28	19.5	3.14	35	19.5	2.40	18	19.5	1.90	12
18.00	2.89	30	19	3.06	33	19	2.34	17	19	1.85	11
18.50	2.98	32	18.5	2.98	32	18.5	2.28	16	18.5	1.80	11
19.00	3.06	33	18	2.89	30	18	2.22	16	18	1.75	10
19.50	3.14	35	17.5	2.81	28	17.5	2.15	15	17.5	1.70	10

The table illustrates that a 42" diameter pipe is the appropriate choice for the second pipeline. The velocities are in the acceptable range between 1.0 ft/s and 5.0 ft/s and the pipeline headloss is around 32 ft. Since this reach is paralleled in 2060, it is necessary to verify that flows and headloss in the 42" parallel pipe also work for the 2060 flow year. Table 7-58 shows the headloss calculations used to verify the velocities are within the acceptable range.

Table 7-59: McAllen to Edinburg Verification

Verify Second Pipe Works for Previous Flow Years					
Q_{2060} =	28	MGD	**HL1 =HL2**		
Original Pipeline					
C	120		C	120	
D	42		D	42	
Q (MGD)	V (ft/s)	H_L	Q (MGD)	V (ft/s)	H_L
12	1.93	14	16	2.57	24
12.50	2.01	15	15.5	2.49	23
13.00	2.09	16	15	2.41	21
13.50	2.17	18	14.5	2.33	20
14.00	2.25	19	14	2.25	19
14.50	2.33	20	13.5	2.17	18
15.00	2.41	21	13	2.09	16
15.50	2.49	23	12.5	2.01	15
16.00	2.57	24	12	1.93	14

In summary, the pipeline from the McAllen Node to the Edinburg Node is 42,000 LF of 42" pipe installed in 2020 and an additional 42,000 LF of 42" pipe installed in 2060 to accommodate increased supply from the Hidalgo County Surface Water Treatment Plant.

7.7 PIPELINE HYDRAULICS

The hydraulic profile of the RGRWA pipeline system will vary throughout the decades depending on the source of water and direction of flow. The pipeline hydraulics go hand in hand with pipeline sizing to determine the location and size of pump stations needed to move water throughout the system.

7.7.1 Assumptions

For the purposes of the Hydraulic Profile the following assumptions are made.

- Maximum allowable pipeline pressure is 250 PSI to match common PSI rating thresholds.
- The minimum allowable pipeline pressure is 85 PSI to allow for distribution into water suppliers systems.
- Each water treatment plant has a high service pump station at the downstream end of the plant to push finished water into the system.

7.7.2 Hydraulic Profile Figure

Figure 7-22 shows the hydraulic profile for the 2020 design flows on the RGRWA regional pipeline. Likewise, Figure 7-22 shows the hydraulic profile for all design flows with hydraulic gradient lines for each decade as indicated in the figure.

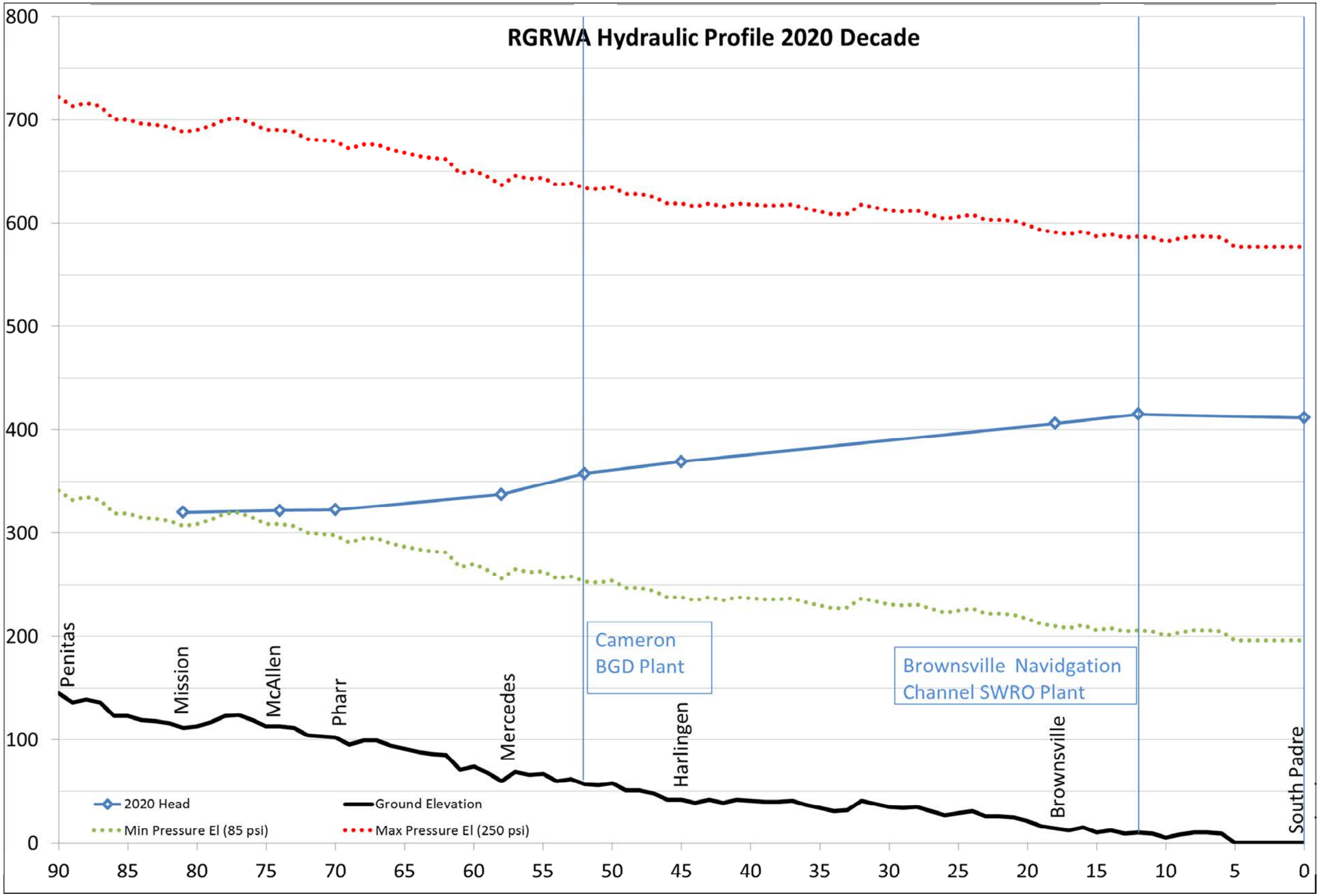


Figure 7-22: RGRWA Hydraulic Profiles for 2020

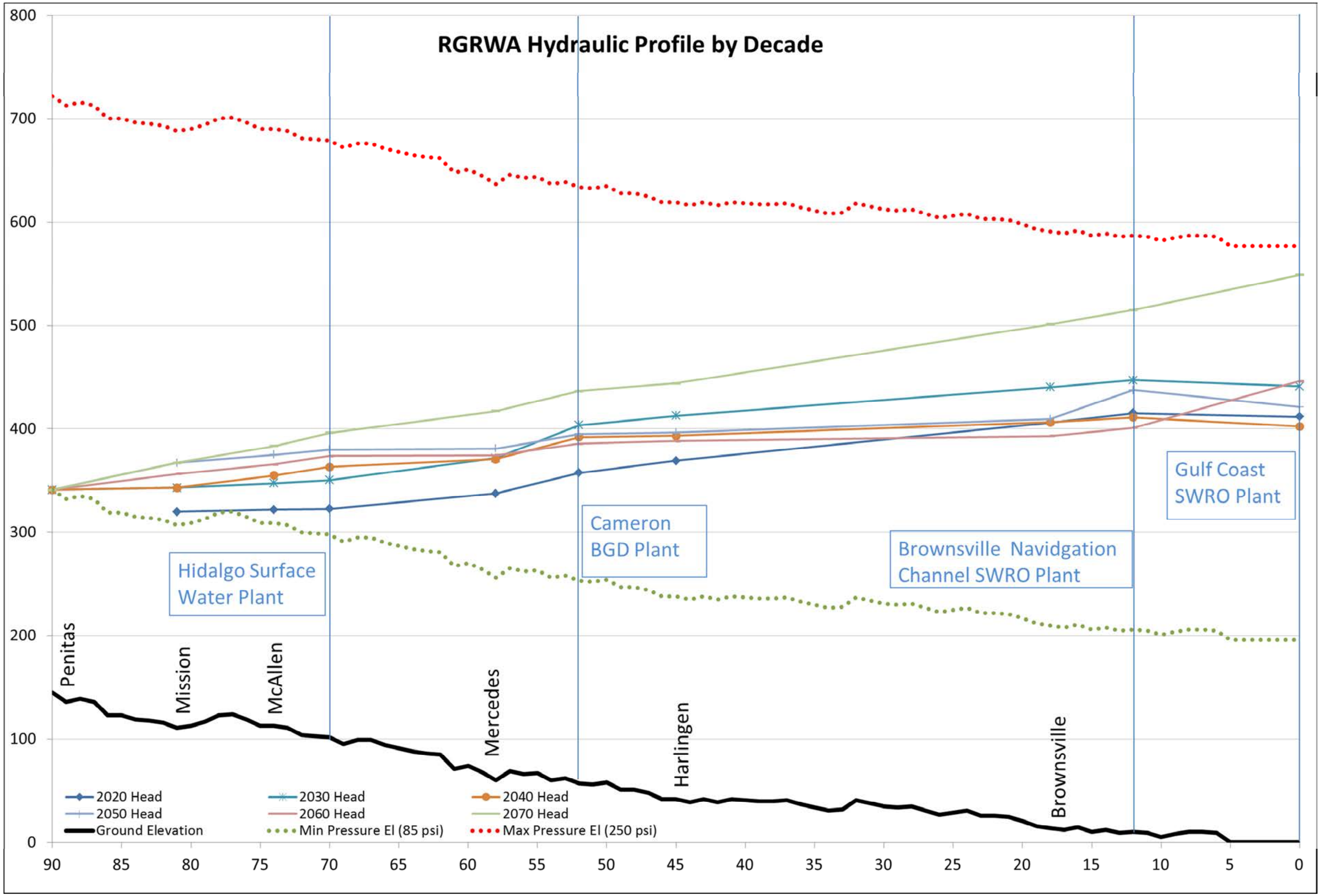


Figure 7-23: RGRWA Hydraulic Profile for All Decades

7.8 COST OPINION

7.8.1 Assumptions

The methodology used to develop the cost estimate for the pipeline project is based on the pipeline diameter, length, crossings, in-line pump stations, and maintenance cost. The costs are broken down by decade for when each pipe segment comes online or is built in the future. The basis for the unit costs in the estimate multiple sources.

Texas Water Development Board (TWDB) Unified Costing Model (UCM) which is provided to each regional planning group to estimate future project costs was used for pipeline costs including the per linear foot installed cost for various diameters, the boring and tunneling costs, land acquisition costs, and pipeline maintenance costs. The UCM is based on cost curves that were developed from construction throughout Texas and allow for adjustment based on both construction date and location. The UCM does not distinguish between pipe material, construction depths, soil considerations other than soil vs rock excavation, and surface repair other than the urban vs. rural distinction.

7.8.2 Capital Costs

Pipeline cost per foot estimates assume the pipeline is installed in rural areas in excavatable soil. Pipeline and crossing lengths were estimated based on the windshield surveys and desktop analysis of maps that were reviewed of the area.

Pumping Station estimates were developed based on the installed HP as determined by the hydraulic profile and the max capacities of the various water supplies. This assumes a peaking factor of 1.3. The cost per HP was determined based on bid prices from the previous projects and equated to $3,052/HP for pumping stations in the range of 500 to 4000 HP. It is assumed that Pumping capacity will installed each decade to match water treatment plant capacity phasing.

Engineering, Legal, Administrative and Permitting costs were estimated at 30% of the infrastructure construction cost.

The 2020 decade costs are shown in Table 7-60: 2020 Decade Pipeline Cost. This decade has the highest cost because it includes the construction and engineering cost for the majority of the pipeline and two pumping stations.

Table 7-60: 2020 Decade Pipeline Cost

2020 Decade Project Cost			
Pipeline			
Pipe Diameter (In.)	Length (ft.)	Cost/Ft	Pipeline Total
20"	31,000	$ 82	$ 2,542,000
36"	286,000	$ 173	$ 49,478,000
42"	194,500	$ 208	$ 40,456,000
48"	61,000	$ 242	$ 14,762,000
84"	26,000	$ 582	$ 15,132,000
		Sub-Total:	$ 122,370,000
Crossings			
	Length (ft.)	Cost/Ft	Crossing Total
Major (Main Roads, Creeks)	15,000	$ 600	$ 9,000,000
Minor (Residential Streets)	10,000	$ 400	$ 4,000,000
Bridge	13,000	$ 800	$ 10,400,000
		Sub-Total:	$ 23,400,000
Pumping Stations			
	HP	Cost/HP	Total
BNC SWRO	1052	$ 3,052	$ 3,209,224
Cameron BGD Plant	1498	$ 3,052	$ 4,573,145
		Sub-Total:	$ 7,782,369
Land Acquisition			
	Acres	Cost/Acre	Total
Pipeline	1374	$ 3,003	$ 4,126,023
		Sub-Total:	$ 4,126,023
Engineering/Permitting			
30% of Construction			$ 46,066,000
		2020 Total:	$ 203,744,392

The 2030 decade cost includes both pipelines and pumping stations. During this decade the Hidalgo County SWTP and Hidalgo Brackish Groundwater Desalination (BGD) plant come online. The Cameron BGD Plant is also expanded during this decade. Pipelines to the new sources as well as to the west of Mission are included in this decade. Table 7-61: 2030 Decade Pipeline Cost shows the 2030 project cost estimates.

Table 7-61: 2030 Decade Pipeline Cost

2030 Decade Project Cost			
Pipeline			
Pipe Diameter (In.)	Length (ft.)	Cost/Ft	Pipeline Total
20"	90,000	$ 82	$ 7,380,000
24"	19,000	$ 104	$ 1,976,000
78"	42,000	$ 498	$ 20,916,000
		Sub-Total:	$ 20,916,000
Crossings			
	Length (ft.)	Cost/Ft	Crossing Total
Major (Main Roads, Creeks)	1,000	$ 600	$ 600,000
Minor (Residential Streets)	750	$ 400	$ 300,000
Bridge	800	$ 800	$ 640,000
		Sub-Total:	$ 1,540,000
Pumping Stations			
	HP	Cost/HP	Total
Cameron BGD Plant	666	$ 3,052	$ 2,032,509
Hidalgo SWTP	1533	$ 3,052	$ 4,680,119
Hidalgo BGD Plant	613	$ 3,052	$ 1,872,047
		Sub-Total:	$ 8,584,675
Land Acquisition			
	Acres	Cost/Acre	Total
Pipeline	96	$ 3,003	$ 289,545
		Sub-Total:	$ 289,545
Engineering/Permitting			
30% of Construction			$ 6,824,000
		2030 Total:	$ 38,154,220

The 2040 decade cost includes only the additional pumping capacity from the Hidalgo SWTP Pumping Station. Table 7-62: 2040 Decade Pipeline Cost shows the 2040 project cost estimates.

Table 7-62: 2040 Decade Pipeline Cost

2040 Decade Project Cost			
Pipeline			
Pipe Diameter (In.)	Length (ft.)	Cost/Ft	Pipeline Total
		Sub-Total:	$ -
Crossings			
	Length (ft.)	Cost/Ft	Crossing Total
		Sub-Total:	$ -
Pumping Stations			
	HP	Cost/HP	Total
Hidalgo SWTP	3593	$ 3,052	$ 10,964,849
		Sub-Total:	$ 10,964,849
Land Acquisition			
	Acres	Cost/Acre	Total
		Sub-Total:	$ -
Engineering/Permitting			
30% of Construction			$ 3,289,000
		2040 Total:	$ 14,253,849

The 2050 decade is when parallel pipelines begin installation. The costs include pipeline and crossing costs, and increased pumping capacity at both the BNC SWRO and the Hidalgo SWTP pumping stations. Table 7-63: 2050 Decade Pipeline Cost shows the 2050 pipeline cost.

Table 7-63: 2050 Decade Pipeline Cost

2050 Decade Project Cost			
Pipeline			
Pipe Diameter (In.)	Length (ft.)	Cost/Ft	Pipeline Total
48"	29,000	$ 242	$ 7,018,000
78"	42,000	$ 498	$ 20,916,000
84"	26,000	$ 582	$ 15,132,000
		Sub-Total:	$ 43,066,000
Crossings			
	Length (ft.)	Cost/Ft	Crossing Total
Major (Main Roads, Creeks)	2,000	$ 600	$ 1,200,000
Minor (Residential Streets)	2,000	$ 400	$ 800,000
Bridge	2,000	$ 800	$ 1,600,000
		Sub-Total:	$ 3,600,000
Pumping Stations			
	HP	Cost/HP	Total
BNC SWRO	1052	$ 3,052	$ 3,209,224
Hidalgo SWTP	4491	$ 3,052	$ 13,706,062
		Sub-Total:	$ 16,915,286
Land Acquisition			
	Acres	Cost/Acre	Total
Pipeline	223	$ 3,003	$ 668,712
		Sub-Total:	$ 668,712
Engineering/Permitting			
30% of Construction			$ 19,074,000
		2050 Total:	$ 83,323,998

Parallel pipelines continue installation in the 2060 decade, the costs include pipeline and crossing costs, increased pumping capacity at the Hidalgo SWTP and a new pumping station conveying water from the GC SWRO. Table 7-64: 2060 Decade Pipeline Cost shows the 2060 project cost estimate.

Table 7-64: 2060 Decade Pipeline Cost

2060 Decade Project Cost			
Pipeline			
Pipe Diameter (In.)	Length (ft.)	Cost/Ft	Pipeline Total
24"	19,000	$ 104	$ 1,976,000
42"	42,000	$ 208	$ 8,736,000
60"	175,000	$ 311	$ 54,425,000
		Sub-Total:	$ 65,137,000
Crossings			
	Length (ft.)	Cost/Ft	Crossing Total
Major (Main Roads, Creeks)	6,000	$ 600	$ 3,600,000
Minor (Residential Streets)	4,000	$ 400	$ 1,600,000
Bridge	5,000	$ 800	$ 4,000,000
		Sub-Total:	$ 9,200,000
Pumping Stations			
	HP	Cost/HP	Total
GC SWRO	2103	$ 3,052	$ 6,418,448
Hidalgo SWTP	2695	$ 3,052	$ 8,223,637
		Sub-Total:	$ 14,642,086
Land Acquisition			
	Acres	Cost/Acre	Total
Pipeline	542	$ 3,003	$ 1,626,970
		Sub-Total:	$ 1,626,970
Engineering/Permitting			
30% of Construction			$ 26,694,000
		2060 Total:	$ 117,300,055

The 2070 decade is when all pipelines are completed to give redundancy for a large portion of the conveyance capacity. The costs in 2070 include pipeline, crossing, and additional pumping capacity at the GC SWRO and Hidalgo SWTP pumping stations. Table 7-65: 2070 Decade Pipeline Costs shows the 2070 project cost estimate.

Table 7-65: 2070 Decade Pipeline Costs

2070 Decade Project Cost			
Pipeline			
Pipe Diameter (In.)	Length (ft.)	Cost/Ft	Pipeline Total
48"	32,000	$ 242	$ 7,744,000
60"	111,000	$ 311	$ 34,521,000
		Sub-Total:	$ 42,265,000
Crossings			
	Length (ft.)	Cost/Ft	Crossing Total
Major (Main Roads, Creeks)	4,000	$ 600	$ 2,400,000
Minor (Residential Streets)	2,500	$ 400	$ 1,000,000
Bridge	3,000	$ 800	$ 2,400,000
		Sub-Total:	$ 5,800,000
Pumping Stations			
	HP	Cost/HP	Total
GC SWRO	4206	$ 3,052	$ 12,836,897
Hidalgo SWTP	1796	$ 3,052	$ 5,482,425
		Sub-Total:	$ 18,319,322
Land Acquisition			
	Acres	Cost/Acre	Total
Pipeline	328	$ 3,003	$ 985,833
		Sub-Total:	$ 985,833
Engineering/Permitting			
30% of Construction			$ 19,915,000
		2070 Total:	$ 87,285,155

7.8.3 Operations & Maintenance Costs

The total O&M costs per decade are shown in Table 7-66. Operations and Maintenance costs were estimated for both the pipelines and the pumping stations. For pipeline repair it was assumed at 1% of the installed construction cost. Electrical Costs were estimated for the pumping based on electrical rages of $0.05/kwh, 90% motor efficiency and annual average pumping rates.

Table 7-66: O&M Cost by Decade

Summary Project O&M Cost by Decade							
Decade	Accumalative Construction Cost	Maintenance Cost per year (in 2020$)*	Accumalative Installed HP	Motor Efficiency	Power Use	Unit Cost	Operation Cost per year (in 2020$)**
2020	$ 154,000,000	$ 1,540,000	2550	0.9	1625	0.05	$ 712,000
2030	$ 185,000,000	$ 1,850,000	5363	0.9	3418	0.05	$ 1,497,000
2040	$ 196,000,000	$ 1,960,000	8956	0.9	5708	0.05	$ 2,500,000
2050	$ 260,000,000	$ 2,600,000	14498	0.9	9240	0.05	$ 4,047,000
2060	$ 349,000,000	$ 3,490,000	19296	0.9	12298	0.05	$ 5,387,000
2070	$ 415,000,000	$ 4,150,000	25298	0.9	16124	0.05	$ 7,062,000

* Includes pipeline and pumpstation repairs = 1% annually of all construction costs to date

** Includes pumping costs = HP(installed)/1.3* 0.7457/0.9*(24*365)*$0.05 - with 1.3 peaking, 0.9 motor efficiency, and $0.05/kwl

7.8.4 Cost Summary

A cost summary per decade is provided in Table 7-60: 2020 Decade Pipeline Cost.

Table 7-67: Total Project Cost by Decade

Summary Project Cost by Decade				
Decade	Capital Cost per decade	Maintenance Cost per year	Operation Cost per year	Total O&M Cost per year
2020	**$ 203,744,392**	$ 1,540,000	$ 712,000	**$ 2,252,000**
2030	**$ 38,154,220**	$ 1,850,000	$ 1,497,000	**$ 3,347,000**
2040	**$ 14,253,849**	$ 1,960,000	$ 2,500,000	**$ 4,460,000**
2050	**$ 83,323,998**	$ 2,600,000	$ 4,047,000	**$ 6,647,000**
2060	**$ 117,300,055**	$ 3,490,000	$ 5,387,000	**$ 8,877,000**
2070	**$ 87,285,155**	$ 4,150,000	$ 7,062,000	**$ 11,212,000**
Total	**$ 544,061,670**			

CHAPTER 8 BRACKISH GROUNDWATER INFRASTRUCTURE

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

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8.0 Brackish Groundwater Infrastructure

8.1 PURPOSE

The purpose of this chapter is to describe the infrastructure required to desalinate groundwater from the identified brackish water aquifers. Process and conveyance infrastructure methods were chosen and sized based on current water quality data, treatment technologies and limitations, regulations and source availability. It is the intent of this chapter to provide sufficient detail to provide preliminary engineering opinions of probable costs for the construction and operation of the proposed facilities. The proposed project includes brackish groundwater well fields, two treatment plants, raw water blending, concentrate disposal and drinking water disinfection and storage, and is described in the following sections.

- Well Field Collection Infrastructure
- Source Water Quality
- Treated Water Quality
- Capacity
- Treatment Process
- Space Requirements
- Cost Opinions

8.2 WELL FIELD COLLECTION INFRASTRUCTURE

The regional project will treat brackish water from two well fields located in Hidalgo and Cameron Counties. The Western Well Field includes 14 wells generally located northwest of McAllen, and the Eastern Well Field includes 44 wells generally located southeast of Weslaco.

As discussed in Chapter 5, there is approximately 50,000 afy of water available for use in Hidalgo and Cameron Counties that can be used on an annual basis. The 38,000 afy in Cameron County is located adjacent to the Hidalgo County Border south of Express Way 83. The 12,000 afy in Hidalgo County could be withdrawn on the opposite side of the border or in an area northwest of McAllen. Utilizing the proposed well field near McAllen will mitigate risk due to its location on the opposite side of the projects centroid of demand discussed in Chapter 2.

Well field collection and conveyance will be sized 30% larger than the annual average groundwater withdrawal amounts to allow for the system to increase production and meet the daily fluctuation in demands.

The well field conveyance systems collect the brackish groundwater and convey it to two BGD treatment plants located at or near the respective well fields. The following assumptions were used to determine the production well field conveyance system facilities:

- The firm capacities of the conveyance/transmission systems to the BGD treatment plants are 13.9 MGD from the western well field and 44.1 MGD from the eastern well field.
- The brackish production wells will have an average capacity of 620 gpm.

- The Western Well Field includes 18 wells to produce a maximum of 16.1 MGD. This equates to a well field reliability of 86.3%meaning there is approximately 14% more well field capacity than can be utilized on annual basis with consideration for daily fluctuation in production.
- The Eastern Well Field includes 58 wells to produce a maximum of 51.8 MGD. This equates to a well field reliability of 85.1% meaning there is approximately 15% more well field capacity than can be utilized on annual basis with consideration for daily fluctuation in production.

8.2.1 Production Well Field Configuration and Layout

The preliminary well field configuration is based on four rows of 4-5 wells for the Western Well Field and six rows of 6-11 wells for the Eastern Well Field. Because of the required spacing between each production well, the well collection piping is based on central headers that connect two rows of wells.

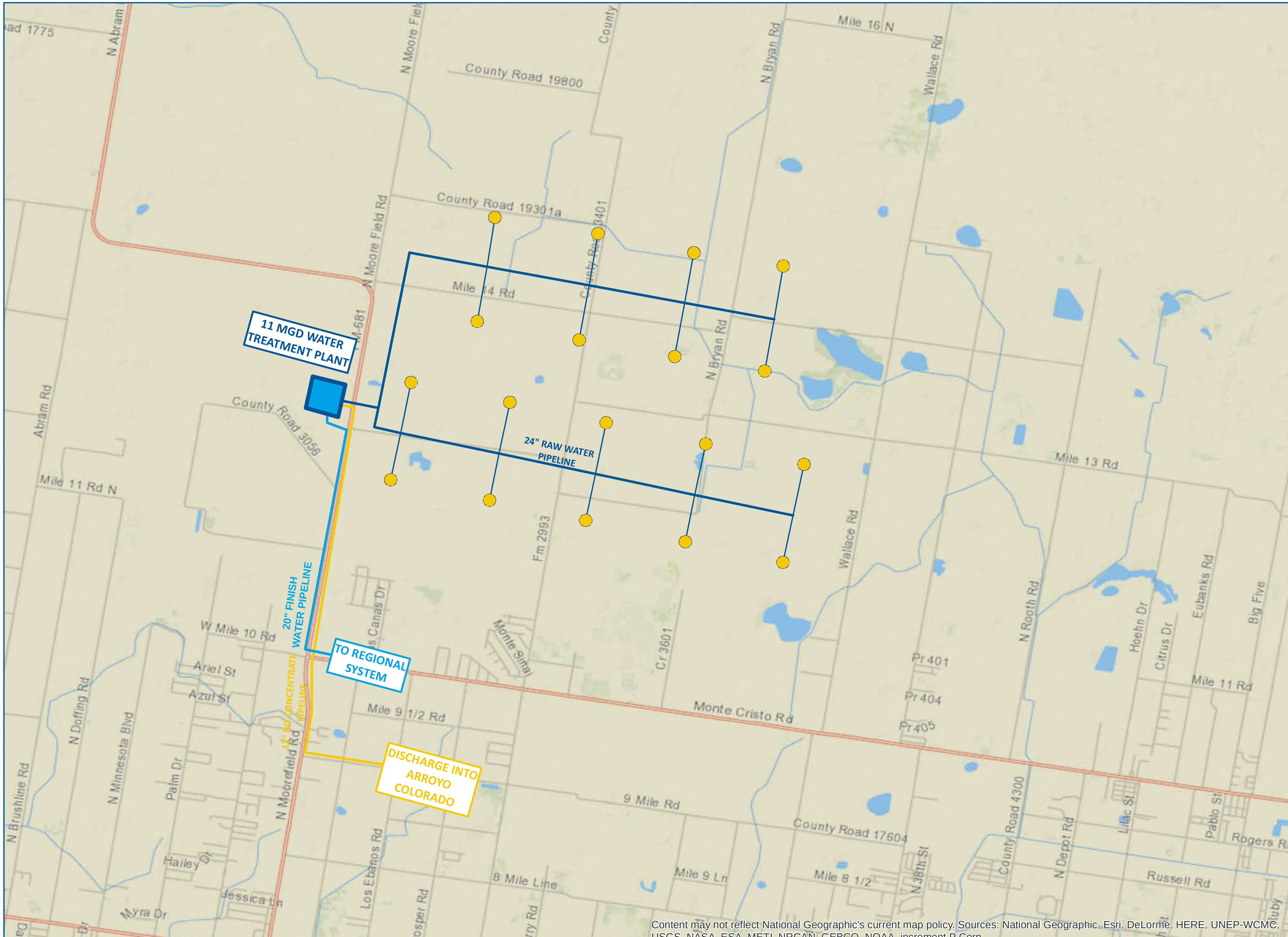
Sizing for the well collection piping was generally based on a maximum velocity of 4 feet per second (fps) with all wells in operation. The collection system piping would range from 10 inches to 54 inches, as shown in Table 8-1. Schematic drawings for the Western and Eastern Well field collection piping are shown on Figures 8-1 and 8-2.

Table 8-1 Well Field Collection Piping Length Summary

DIAMETER (INCHES)	WESTERN WELL FIELD (FEET)	EASTERN WELL FIELD (FEET)	TOTAL LENGTH (FEET)
10	47,340	136,760	184,100
12	10,520	15,780	26,300
18	0	5,260	5,260
24	28,930	31,560	60,090
30	2,630	49,970	52,600
36	2,630	34,190	36,820
42	0	10,520	10,520
54	0	7,890	7,890
Twin 48	0	7,890	7,890
TOTAL	**102,570**	**318,230**	**420,800**

8.2.2 Well Field Storage

Raw water storage is not recommended at this time due to the anticipated iron content in the brackish water. Oxidation of the dissolved iron in a ground storage tank would require pretreatment to remove the iron precipitate.



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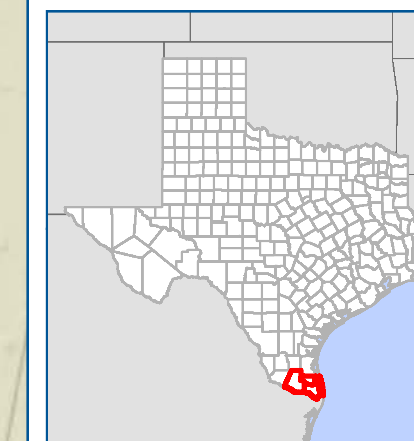
LEGEND

- TREATMENT PLANT
- GROUNDWATER PRODUCTION WELL
- FINISH WATER PIPELINE
- RAW WATER PIPELINE
- RO CONCENTRATE PIPELINE



WESTERN WELLFIELD COLLECTION PIPING

N / W / E / S

0 2,500 5,000
Feet
1 inch = 5,000 feet

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RGRWA RIO GRANDE REGIONAL WATER AUTHORITY

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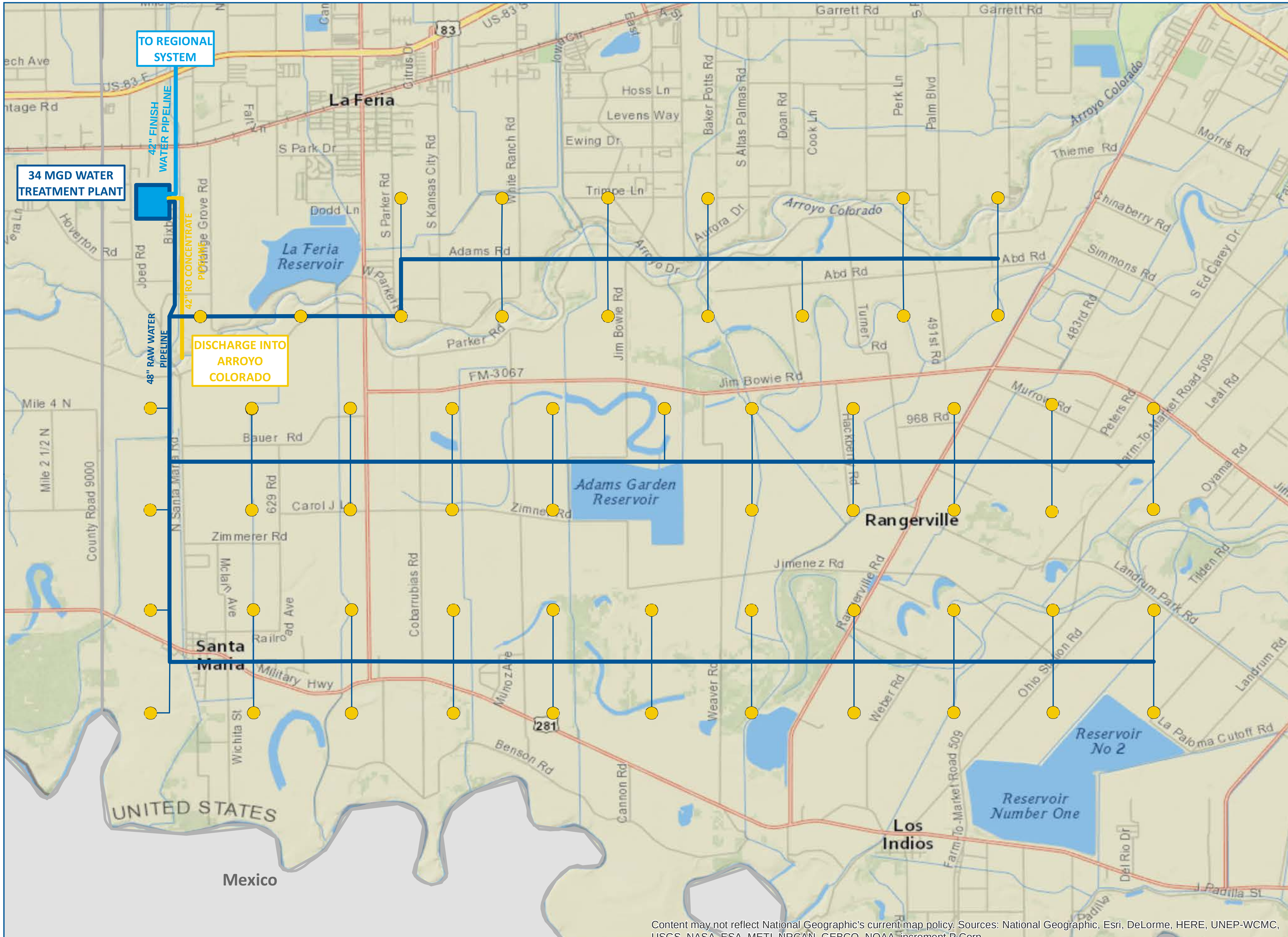

LEGEND

- TREATMENT PLANT
- GROUNDWATER PRODUCTION WELL
- FINISH WATER PIPELINE
- RAW WATER PIPELINE
- RO CONCENTRATE PIPELINE

EASTERN WELLFIELD COLLECTION PIPING

N
W E
S

0 2,500 5,000
Feet
1 inch = 5,000 feet

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8.2.3 Well Pumps

Sizing and selection of production well pumps must consider the ground elevation at each well site, the long-term estimated depth to water, the head loss for the collection piping extending to each well, and the elevation of the GST that each well pump would discharge into. Because these factors will vary somewhat for each well, the total required pump head at each well site will also vary. The assumptions used to determine design criteria for a typical well pump are summarized below.

Ground elevations within the Western Well Field area generally range from about 120 to 140 feet. It is assumed that the average well would be at a ground elevation of 130 feet. As discussed in Section 1.0, the long-term water level including local drawdown is expected to be about 260 feet (below ground surface). This equates to a long-term pumping water level elevation of -130 feet (below msl). The total well depths are estimated to be 400 feet for the Western Well field.

Ground elevations within the Eastern Well Field area generally range from about 40 to 56 feet. For the purposes of this study, it is assumed that the average well would be at ground elevation 48 feet. As discussed in Section 1.0, the long-term pumping water level for the Eastern Well Field is 195 feet (below ground surface). This equates to a long-term pumping water level elevation of -147 (below msl). The total well depths are estimated to be 550 feet for the Western Well field.

Using the assumptions described above, the well pump head and associated motor horsepower design steps are summarized in Table 8-2. The design criteria for the Western Well Field pumps are 545 feet total dynamic head (TDH) and 107 hp. The design criteria for the Eastern Well Field pumps are 555 feet TDH and 109 hp. It is assumed that the pump motors would be 115 hp for the Eastern Well Field and for the Western Well Field.

Table 8-2 Well Pump Design Criteria

WELL FIELD	BGD PLANT ELEVATION (MSL)	AVG. COLLECTION PIPING HEAD LOSS(1) (FT)	REQ'D HGL AT WELL	LONG-TERM WATER ELEVATION (FT)	PUMP HEAD (FT)	MINIMUM PUMP HORSEPOWER(2)
Hidalgo	280	135	415	-130	545	107
Cameron	194	215	270	-147	555	109

(1) Average head loss between GST and production well.
(2) Based on 620 gpm design capacity and 80 percent pump efficiency.

8.3 SOURCE WATER QUALITY

Brackish well water quality measurements from Southmost Regional Water Authority (SRWA) wells were evaluated to determine a source water quality design basis for the conceptual design that is presented in this report. The design basis values are listed in the "Brackish Well Water" column in Table 8-3. The concentration values are based on averages of SRWA well samples collected on 11/4/14, 2/3/15, 5/8/15, and 8/6/15. These values are considered to provide a conservative preliminary design basis because the RGRWA wells that would be developed for the new facilities being considered are located where the aquifer is projected by TWDB to exhibit lower concentrations than the existing SRWA brackish wells.

The temperature design basis values in the table are based on measurements provided by Brownsville Public Utilities Board (BPUB) from wells sampled on 6/4/2015 and 6/25/2015. Well numbers 1-2, 5-8, 10-13, and 16-18 provided 13 temperature readings, averaging 27.1 °C and ranging from 26.4 to 30.1 °C. Other values from the BPUB wells indicate that the design basis for chemical concentrations in the table appears to be conservative. For example, average BPUB values include the following: 3021 mg/L TDS, 0.023 mg/L Arsenic (with a maximum from seven data points of 0.035 mg/L Arsenic), 877 mg/L Chloride, 0.58 mg/L Iron, 0.07 mg/L Manganese, and 1175 mg/L Sulfate, which are generally lower than the SRWA well averages.

As the Cameron and Hidalgo brackish groundwater projects are further developed, additional site-specific testing of the groundwater is recommended for all of the parameters listed in the water quality table as well as a full list of regulated parameters. For example, the design basis for Arsenic in the well water could have a significant impact on the design. For the preliminary conceptual design presented in this report an Arsenic concentration of 0.06 mg/L has been calculated; however, the available data range from 0.0059 to 0.193 mg/L, which is a wider than desired range, especially for such an important parameter. As an additional example, the SRWA well water quality included values for Total Kjeldahl Nitrogen (TKN) ranging from 0.49 to 1.2 mg/L. For the preliminary conceptual design presented in this report TKN has been assumed to be negligible for the new wells, as it has been assumed they would be true groundwater wells and not under the influence of surface water. These and other source water quality parameters should be carefully evaluated for the new wells as the project is developed.

Table 8-3 Comparison of Source Water Quality to Treatment Goals

PARAMETER	UNITS	BRACKISH WELL WATER	TREATED WATER GOAL LIMITS	BASIS FOR GOAL
Total Dissolved Solids (TDS)	mg/L	3200	1000	TCEQ allows 1000 mg/L, rather than EPA SMCL (aesthetic guideline) of 500 mg/L of TDS.
Conductivity	µS/cm	5200	NA	NA
pH	Std. Units	7.8 Design (7.4 – 8.2 Range)	8.0	Detailed design value can be adjusted based on member utility needs.
Temperature	°C	27 Design (24 to 32 Range)	NA	NA
Alkalinity	mg/L as $CaCO_3$	380	NA	Finished water alkalinity to be selected to yield non-corrosive water.
Aluminum	mg/L	0.003	0.05	SMCL = 0.05-0.2
Antimony	mg/L	0.0004	0.006	MCL
Arsenic	mg/L	0.06	0.01	MCL
Barium	mg/L	0.016	2	MCL
Beryllium	mg/L	0.0003	0.004	MCL
Cadmium	mg/L	0.0003	0.005	MCL
Calcium	mg/L	130	NA	NA
Chloride	mg/L	900	300	TCEQ allows 300 mg/L, rather than EPA SMCL (aesthetic guideline) of 250 mg/L Cl.
Chromium	mg/L	0.0003	0.1	MCL

PARAMETER	UNITS	BRACKISH WELL WATER	TREATED WATER GOAL LIMITS	BASIS FOR GOAL
Copper	mg/L	0.003	1.3 Action Level	2.0 SMCL
Fluoride	mg/L	0.8	4.0 EPA MCL	2.0 SMCL
Iron, Diss*	mg/L	2.5	0.3	SMCL
Iron, Total	mg/L	2.5	0.3	SMCL
Lead	mg/L	0.0003	0.015	EPA Action Level
Magnesium	mg/L	50	NA	NA
Manganese	mg/L	0.08	0.05	SMCL
Mercury	mg/L	0.0001	0.002	MCL
Nitrate	mg/L as N	0.18	10	MCL
Selenium	mg/L	0.002	0.05	MCL
Silica (SiO2)	mg/L	36	NA	NA
Sodium	mg/L	780	NA	NA
Sulfate	mg/L	1230	325	TCEQ allows 300 mg/L, rather than EPA SMCL (aesthetic guideline) of 250 mg/L SO_4. A variance by TCEQ up to 325 mg/L part of the time (as an action level as membranes and o-rings age) is proposed to maximize the useful volume of water provided.
Notes: MCL indicates Primary Maximum Contaminant Level, SMCL indicates Secondary MCL (non-enforceable aesthetic guidelines). * For this report the iron is assumed to be dissolved.				

8.4 TREATED WATER QUALITY

The treated water quality goals are listed in Table 8-3. Many of these goals are based on regulatory Primary Maximum Contaminant Levels (MCLs) and Secondary Maximum Contaminant Levels (SMCLs) as well as typical practice in Texas. Considering that the national EPA SMCLs are aesthetic guidelines that are not enforced as MCL's, many utilities in the southwestern United States distribute drinking water at higher concentrations of TDS, Chloride, and Sulfate than the SMCLs. TCEQ generally sets higher Secondary Constituent Levels (SCLs) for these parameters, as summarized in the table. That approach has been incorporated into the preliminary planning of the projects presented in this report.

8.5 CAPACITY

Treatment plant capacity is based on utilizing the available groundwater on a consistent basis with maximum day demands generally being supplied by other sources. The available annual groundwater capacity in the study area totals approximately 50,000 acre feet per year (AFY). The region could support two well field areas. The Eastern Well Field could be located in Cameron County and yield 38,000 AFY of brackish groundwater. The Western Well Field, located in Hidalgo County, could provide 12,000 AFY of brackish groundwater. The Cameron site is planned to have a lower initial capacity in the year 2020 with an expansion in 2030. The Hidalgo BGD facility is planned for the year 2030. The capacities for the planned facilities are summarized in Table 8-4.

Table 8-4 Average Day Capacity of Brackish Groundwater Desalination (BGD) Projects

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO (2030)
Flow from Wells	MGD (AFY)	22.8 (25,500)	34 (38,000)	10.7 (12,000)
Finished Water	MGD	18	26.8	8.4
Finished Water flow rate is based on 75% RO recovery and 20% Bypass / Finished Water ratio.				

8.5.1 Treatment Process

Reverse osmosis (RO) is the most cost effective and widely practiced desalination method for large scale municipal drinking water facilities at these concentrations. Electrodialysis, or the related electrodialysis reversal process, could be technically viable, but in recent years has not been economically competitive with RO. Distillation, such as multistage flash or multi effect, can also be used for desalination, but would not be competitive in this case. In certain situations, distillation may be selected over RO at significantly high source water concentrations (e.g., 35,000 mg/L seawater) and when special conditions apply, such as having a local source of steam or waste heat. Therefore, the flow schematic in Figure 8-3 shows the recommended RO-based treatment process, which is described in the paragraphs below.

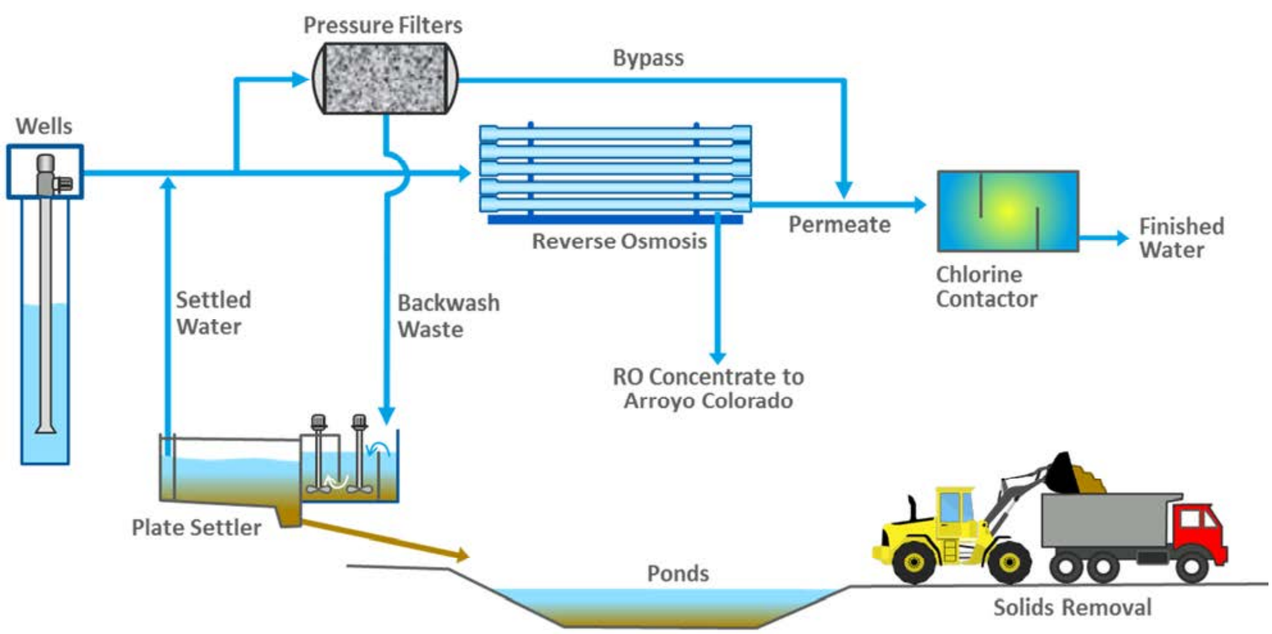


Figure 8-3 Brackish Groundwater Desalination Flow Schematic

Considering the available water quality, the conceptual design is based on 75% RO recovery and a Bypass / Finished Water flow ratio of 20%. The selection of recovery is based on limiting the TDS of the RO Concentrate to 13,000 mg/L to facilitate disposal and the bypass ratio is based on the treated water quality goals. One spare RO train is included at the Cameron site to provide relatively inexpensive redundancy shared between the facilities. On Average Days all of the Duty trains would be running at both sites at typical permeate flow and flux rates. Periodically, on Peak Days all of the Duty trains would be running at Design flux rate, which is higher than the typical Average-Day rate. Less frequently, on special “High-High” days when there are additional system demands, such as when other water plants in the region experience lower production rates, such as due to an equipment servicing event, then all of the trains, including the Spare RO train at Cameron, could be run at Design flows to help the overall system respond to emergency or peaking situations. Flow rates for each of these scenarios are presented in Table 8-5. The Cameron (2020) conceptual design is based on installing fewer RO vessels per train, but including sufficient space and support on each rack to allow increasing the number of vessels in 2030 as part of the planned expansion.

Table 8-5 Flow Rates of Brackish Groundwater Desalination (BGD) Projects

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO (2030)
Average Day (All Duty RO Trains Running at Typical Flow)				
Wells	MGD	22.8	34.0	10.7
RO Feed	MGD	19.2	28.6	9.0
Bypass	MGD	3.6	5.4	1.7
RO Permeate	MGD	14.4	21.5	6.7
RO Concentrate	MGD	4.8	7.2	2.2
Finished Water	MGD	18.0	26.8	8.4
Peak Day (All Duty RO Trains Running at Design Flow)				
Wells	MGD	**25.7**	38.0	12.7
RO Feed	MGD	21.6	32.0	10.7
Bypass	MGD	4.1	6.0	2.0
RO Permeate	MGD	16.2	24.0	8.0
RO Concentrate	MGD	5.4	8.0	2.7
Finished Water	MGD	20.3	30.0	10.0
High-High Day (All Duty & Spare RO Trains Running at Design Flow)				
Wells	MGD	34.2	47.5	12.7
RO Feed	MGD	28.8	40.0	10.7
Bypass	MGD	5.4	7.5	2.0
RO Permeate	MGD	21.6	30.0	8.0
RO Concentrate	MGD	7.2	10.0	2.7
Finished Water	MGD	27.0	37.5	10.0
Number of RO Trains				
Total	Number	4	5	2
Duty	Number	3	4	2
Spare	Number	1	1	0
RO Permeate Flow per Train				
Design	MGD	5.4	6.0	4.0
Typical	MGD	4.8	5.4	3.4
Minimum	MGD	4.3	4.8	3.2

8.5.2 Pretreatment

As currently planned these facilities would have minimal pretreatment, which would consist of antiscalant addition to the RO feed as well as five-micron cartridge filtration. As the project is further developed, the site-specific wells should be evaluated to verify the water quality including iron and manganese concentrations and their associated levels of solubility. For the purpose of the conceptual design, it is assumed that the iron and manganese are fully dissolved without any particulate or colloidal species present. In that case removal of iron and manganese upstream of RO is not required. The well field and associated collection and transmission system would be operated to prevent oxidizing the dissolved iron and manganese to other forms. The RO downstream would reject these constituents as ions. The San Antonio Water System (SAWS) conducted testing and piloting for their brackish groundwater desalination project that showed the iron and manganese could be maintained in dissolved forms and then treated by RO without unwanted scaling. Therefore SAWS has proceeded with a full-scale project without pretreatment providing iron-manganese removal.   That same approach has been assumed for the Cameron and Hidalgo conceptual designs; however, subsequent testing should confirm that aspect of the design. The conceptual pretreatment design is summarized in Table 8-6.

Table 8-6 Pretreatment

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO (2030)
Antiscalant Addition				
Antiscalant Dose	mg/L	3.5	3.5	3.5
Dosed into ROF (Average Day)	MGD	19.2	28.6	9.0
Antiscalant Use	lb/d	560	830	260
Antiscalant Vol	gal/d	56	83	26
Cartridge Filters (Horizontal 316SS)				
Number of Housings – Duty	Number	4	Add 2 for total of 6	2
No of Housings – Spare	Number	1	1	1
Number of 40-in Cartridges per Housing (Min)	Number	340	340	320

8.5.3 Bypass Treatment

Due to the concentrations of Arsenic and Iron in the design basis well water, some treatment of the Bypass stream is needed for the finished water to meet project goals with a margin of safety. If subsequent site-specific well and/or pilot testing indicates the source water has lower concentrations, bypass treatment may not be needed. Finished water concentrations with and without bypass treatment are compared in Table 8-7. This table incorporates the simplifying assumptions of 92% solute rejection by the RO, which is more conservative than exhibited by new RO membrane elements to account for longer-term aging of membranes and o-rings, with the Bypass treatment achieving the goals for Arsenic, Iron, and Manganese, a conservative assumption for the proposed process.

Table 8-7 Comparing Finished Water Quality With and Without Bypass Treatment

PARAMETER	UNITS	WELL WATER	FINISHED WATER (WITHOUT BYPASS TREATMENT)	FINISHED WATER (WITH BYPASS TREATMENT)	TREATED WATER GOAL LIMITS
Total Dissolved Solids	mg/L	3200	845	845	1000
Arsenic	mg/L	0.06	**0.016**	0.006	0.01
Barium	mg/L	0.016	0.004	0.004	2
Calcium	mg/L	130	34	34	None
Chloride	mg/L	900	240	240	300
Fluoride	mg/L	0.8	0.2	0.2	2
Iron	mg/L	2.5	**0.7**	0.2	0.3
Magnesium	mg/L	50	13	13	None
Manganese	mg/L	0.08	0.02	0.015	0.05
Silica, SiO2	mg/L	36	10	10	None
Sodium	mg/L	780	206	206	None
Sulfate	mg/L	1230	325	325	325

Notes: (1) Items marked with **bold, underlined type** exceed treated water limits.
(2) Calculation did not include chemicals added by Post-treatment, such as Calcium and Alkalinity. The focus of this calculation was Arsenic, Iron, and Manganese.

The conceptual design of the bypass treatment system is described in Table 8-8. The bypass treatment would consist of addition of an oxidant, such as sodium hypochlorite, followed by horizontal pressure filters containing a granular media, such as greensand or pyrolusite. The pressure to transport water through the filters would be provided by the well pumps.

Table 8-8 Bypass Treatment

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO (2030)
Filter Design Flow	MGD	4.0	6.0	2.0
Filter Loading Rate (Maximum with 1 cell in Backwash)	gpm/ft2	3.0	3.0	3.0
Configuration	Type	Horizontal	Horizontal	Horizontal
Number of Filter Housings	Number	4	Add 2 = Total of 6	2
Number of Cells per Housing	Number	2	2	2

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO (2030)
Diameter of Filter Housings	ft	10	10	10
Length of Filter Housings, Straight Side	ft	27	27	31
Maximum Pressure Drop, Clean (Dirty)	psi	3 (10)	3 (10)	3 (10)

8.5.4 Reverse Osmosis (RO)

The conceptual design of the RO equipment is described in Table 8-9. Seven-element vessels are applied in this design to allow for higher recovery, if the well water quality allows that. The current design basis was selected to maintain about 13,000 mg/L of TDS in the RO Concentrate to facilitate discharge.

Table 8-9 Reverse Osmosis (RO)

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO (2030)
Number of RO Trains				
Total	Number	4	5	2
Duty	Number	3	4	2
Spare	Number	1	1	0
RO Permeate Flow per Train				
Design	MGD	5.4	6.0	4.0
Typical	MGD	4.8	5.4	3.4
Minimum	MGD	4.3	4.8	3.2
Description of Each Train				
Number of Stages	Number	2	2	2
Number of Elements/Vessel (nominal 8” x 40” elements)	Number	7	7	7
Maximum flux	gfd	13	13	13
Number of Vessels, Stage 1	Number	100	110	76
Number of Vessels, Stage 2	Number	50	55	38
High Pressure Pump (HPP) Flow, Design (Typical)	gpm	5000 (4400)	5600 (5000)	3700 (3100)
HPP Discharge Pressure, Design (Typical)	psi	260 (200)	260 (200)	250 (180)

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO (2030)
HPP Motor size, Design (Typical Consumption)	hp	1000 (650)	1200 (730)	750 (410)
Typical HPP Electrical Consumption, each train	kW	480	540	310
Notes: (1) Cameron (2020) RO units would be built to allow easy expansion of Cameron (2030) configuration. (2) HPP pressure includes 15 psi permeate header pressure plus 15% safety factor for design condition. Energy recovery was not included for the conceptual design, but should be evaluated during future detailed design tasks. For brackish water facilities where power costs tend to be low, energy recovery may not be cost effective.				

8.5.5 Post-Treatment

RO permeate is aggressive to metallic pipe materials. Permeate is aggressive due to low pH, low alkalinity, and low TDS. To avoid corrosion in distribution piping and in customer's homes, permeate needs to be either stabilized via chemical addition and/or blended with less corrosive water. Distributed water that is too corrosive causes aesthetic problems, such as "red" water, as well as the possibility of exceeding acceptable lead and copper levels at points of use.

Utilities maintain a positive Langelier Saturation Index (LSI) and/or a positive Calcium Carbonate Precipitation Potential (CCPP) to prevent corrosion. In some cases, such as when adding a new treated water source to an existing distribution system, additional care is taken to maintain pH and other parameters at or near status quo conditions. Other methods are also employed to verify distributed water quality will avoid corrosion, such as coupon or pipe loop testing and carefully planned flushing programs.

As shown in Table 8-10, for the conceptual design described in this report, addition of sodium hydroxide (NaOH, also known as caustic) is sufficient to increase the pH and alkalinity, as well as yield positive LSI and CCPP. This calculation should be reconsidered after site-specific well water concentration information is available. If the concentrations in the well water were lower, then the Finished Water, which is a blend of RO Permeate and Bypass streams, might require additional treatment for stabilization. For example, that might entail carbon dioxide addition followed by either a calcite bed or lime addition. Either of those methods would increase the pH, alkalinity, and hardness, as well as the LSI and CCPP. However, for the current conceptual design addition of NaOH is sufficient, and certainly less costly.

Table 8-10 Post-treatment

PARAMETER	UNITS	FINISHED WATER BEFORE POST-TREATMENT	FINISHED WATER AFTER ADDITION OF 24 MG/L NAOH (100% BASIS)
pH	Std Units	6.7	8.0
Alkalinity	mg/L as $CaCO_3$	85	110
Hardness	mg/L as $CaCO_3$	110	110
LSI	Unitless index	-1.2	0.2
CCPP	mg/L	-48	2.0

8.5.6 Disinfection

Primary disinfection would be achieved in the facilities with sufficient free chlorine contact to provide at least a 4-log inactivation of virus, which is a standard requirement for groundwater treatment. The chlorine would be provided by addition of sodium hypochlorite from a chemical feed system located in a building located relatively close to the feed point. After the required concentration time (which is referred to as “CT”) has been provided by the free chlorine residual, liquid ammonium sulfate (LAS) would be added to convert the chlorine residual to chloramine, the secondary disinfectant that is used in many of the member utilities’ distribution systems. At the minimum well water design basis temperature (24 ^{o}C) the CT to achieve 4-log virus inactivation is 2.2 mg/L min; to be conservative a CT requirement of at least 3.0 mg/L min (the value at 20 ^{o}C) has been assumed.

Table 8-11 Disinfection

PARAMETER	UNITS	CAMERON (2020)	CAMERON (2030)	HIDALGO
Design Flow (“High-High” Flow)	MGD	20.3 (27.0)	30.0 (37.5)	10.0 (10.0)
Free Chlorine Residual, minimum basis for CT	mg/L	0.5	0.5	0.5
Sodium Hypochlorite Dosage, Typical (Maximum)	mg/l as Cl_2	1.0 (4.0)	1.0 (4.0)	1.0 (4.0)
LAS Dosage, Typical (Maximum)	mg/l as NH_3	0.3 (1.0)	0.3 (1.0)	0.3 (1.0)

8.5.7 Residuals (Including Concentrate)

Managing the concentrate generated is often the most challenging aspect of an inland desalination project. Methods for managing concentrate are as follows:

- Discharge to sewer.
- Discharge to surface water.

- Evaporation pond.
- Deep well injection.
- Treatment to achieve zero liquid discharge (ZLD).

Residual pressure from the RO units would provide sufficient energy to transport the RO Concentrate stream to discharge via the Arroyo Colorado or another surface drain. The RO recovery was selected for the conceptual design to meet a discharge limitation of 13,000 mg/L of TDS for the Arroyo Colorado. Projects without a relatively low cost, local discharge point frequently have significant additional costs for concentrate disposal. For example, deep injection wells have a capital cost of about $4 to 5 million for 400-500 gpm discharge capacity as well as ongoing energy consumption.

Residuals from the bypass treatment and pressure filters will be conventionally thickened, and stored outside in multi-cell ponds that will periodically be cleaned with the resulting residuals transported off-site.

8.5.8 Finished Water

Finished water storage facilities will be provided at the two water treatment plants with a minimum of 5% of total daily plant capacity. Treated water from the RO process mixed with the bypass would be transferred to the storage facility using a transfer pump station. Six 5.5 MGD, 125 hp pumps would be installed to provide a firm capacity of 27.5 MGD at the Cameron plant. Three 4.5 MGD, 100 hp pumps would be installed to provide a firm capacity of 9 MGD at the Hidalgo plant. A circular clearwell with baffle walls is proposed for storage to ensure adequate disinfection contact time. Table 8-12 below summarizes the design parameters of the clearwell at the two plants.

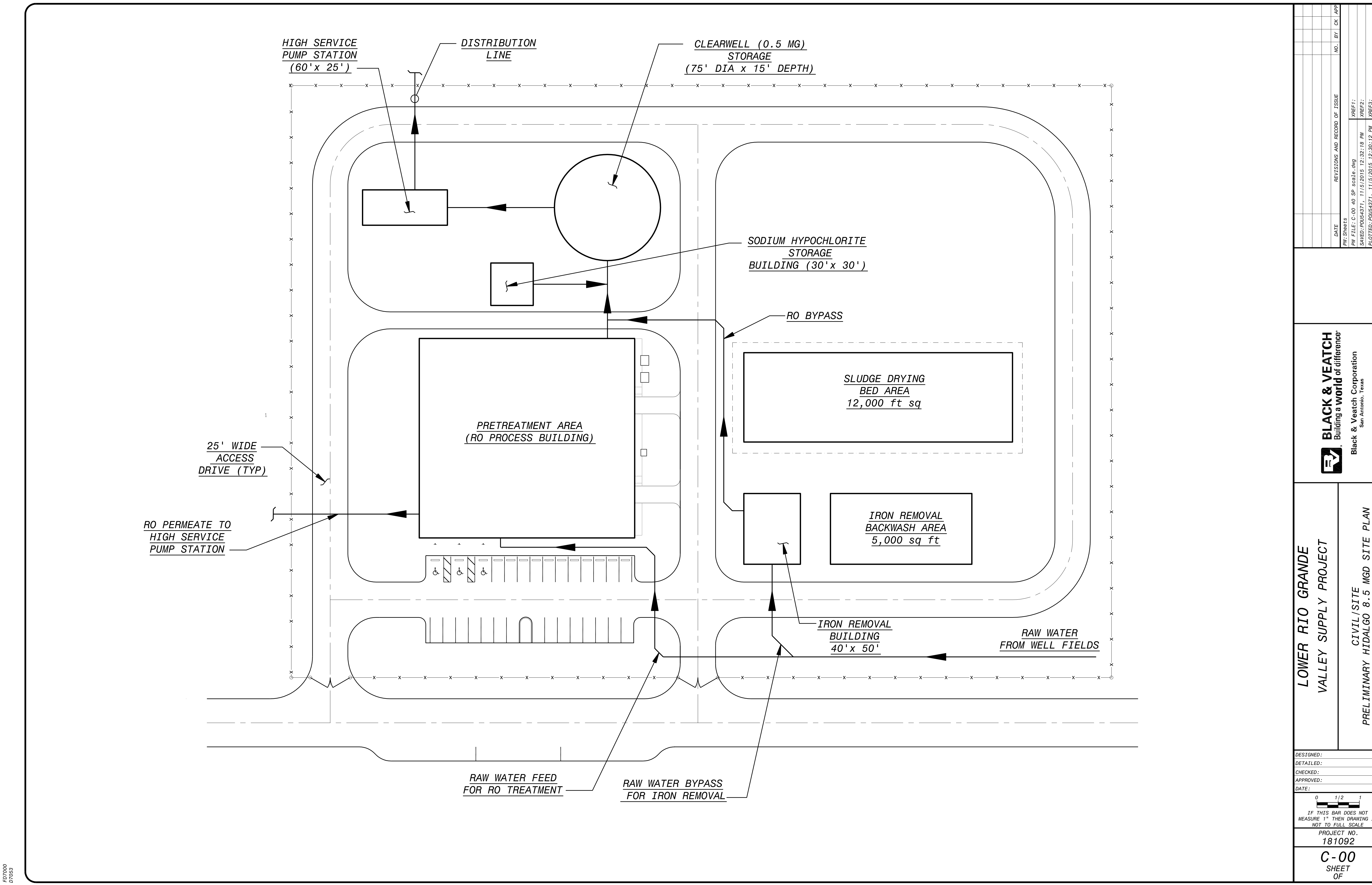
Table 8-12 Finished Water Storage

PARAMETER	CAMERON (2030)	HIDALGO (2030)
Plant Capacity (MGD)	26.85	8.43
5% Storage Volume (MG)	2.00	0.50
Diameter (ft)	110.00	75.00
Depth (ft)	25.00	15.00

8.6 SPACE REQUIREMENTS

A conceptual treatment site plan for the Hidalgo and Cameron treatment plants are shown in Figure 8-4 and Figure 8-5. A portion of raw water from the production well fields is bypassed for iron removal. The bypass treatment described in Section Bypass Treatment 8.5.3 includes six horizontal pressure filters for Cameron plant and two for Hidalgo plant. The spent backwash waste from the pressure filters would be reclaimed by a package plate settler, with the supernatant water returned to the pressure filter header and the residual stream would flow to sludge drying beds for further concentration and subsequent off-site disposal.

The raw water flow for RO treatment process includes chemical addition of antiscalant to the feed, cartridge filters, high pressure pumps and RO elements. In addition to these process units, space is required for support facilities (e.g., laboratory, offices, lobby for visitors, conference rooms, personal needs, control room, disinfection and storage). Simplified floor plan for the treatment facilities are shown on Figures 8-6 and 8-7 below.



HIGH SERVICE PUMP STATION (60'x 25')
DISTRIBUTION LINE
CLEARWELL (0.5 MG) STORAGE (75' DIA x 15' DEPTH)
SODIUM HYPOCHLORITE STORAGE BUILDING (30'x 30')
RO BYPASS
SLUDGE DRYING BED AREA 12,000 ft sq
PRETREATMENT AREA (RO PROCESS BUILDING)
25' WIDE ACCESS DRIVE (TYP)
RO PERMEATE TO HIGH SERVICE PUMP STATION
IRON REMOVAL BACKWASH AREA 5,000 sq ft
IRON REMOVAL BUILDING 40'x 50'
RAW WATER FROM WELL FIELDS
RAW WATER FEED FOR RO TREATMENT
RAW WATER BYPASS FOR IRON REMOVAL
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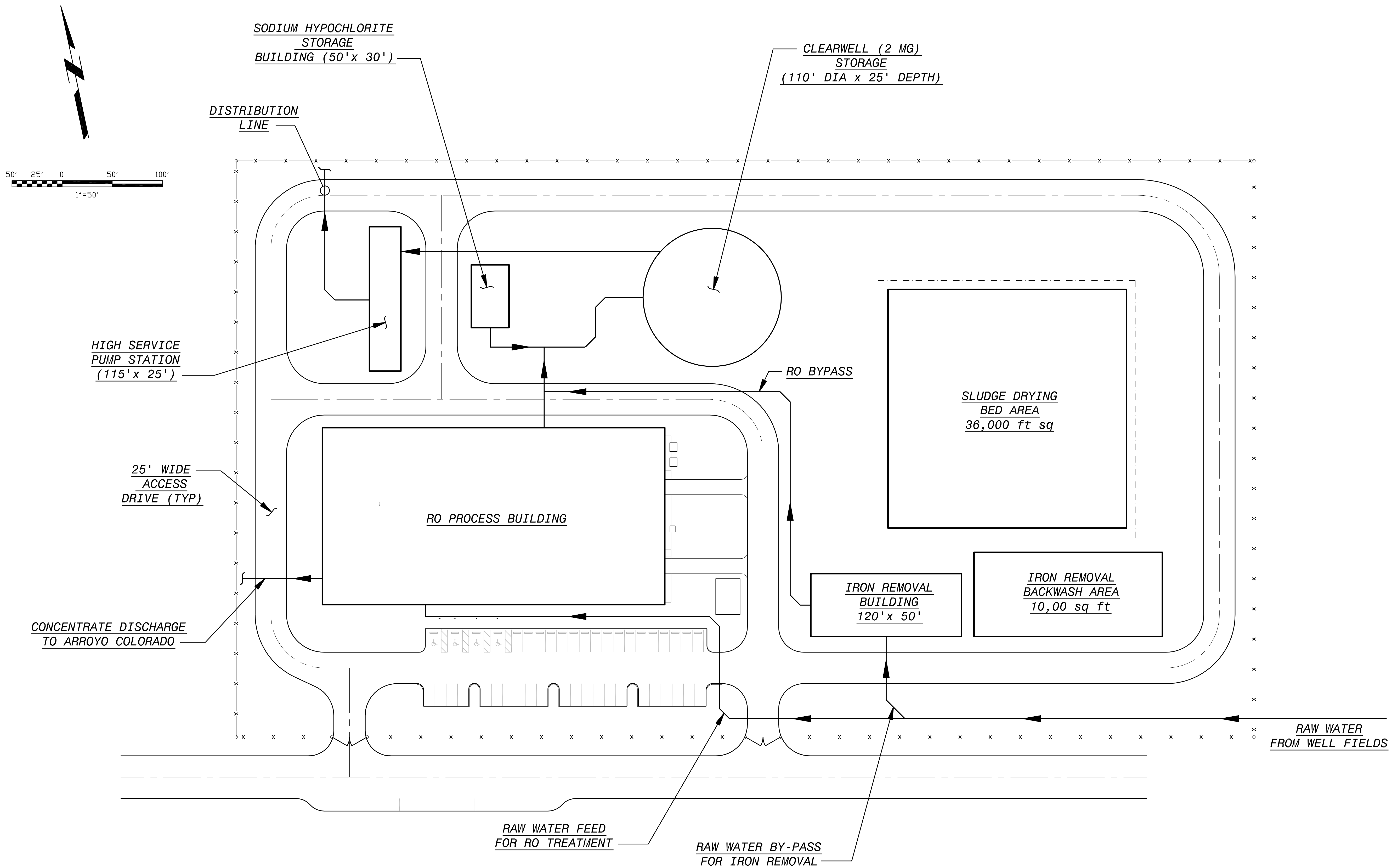

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CIVIL/SITE PRELIMINARY CAMERON 26.8 MGD SITE PLAN

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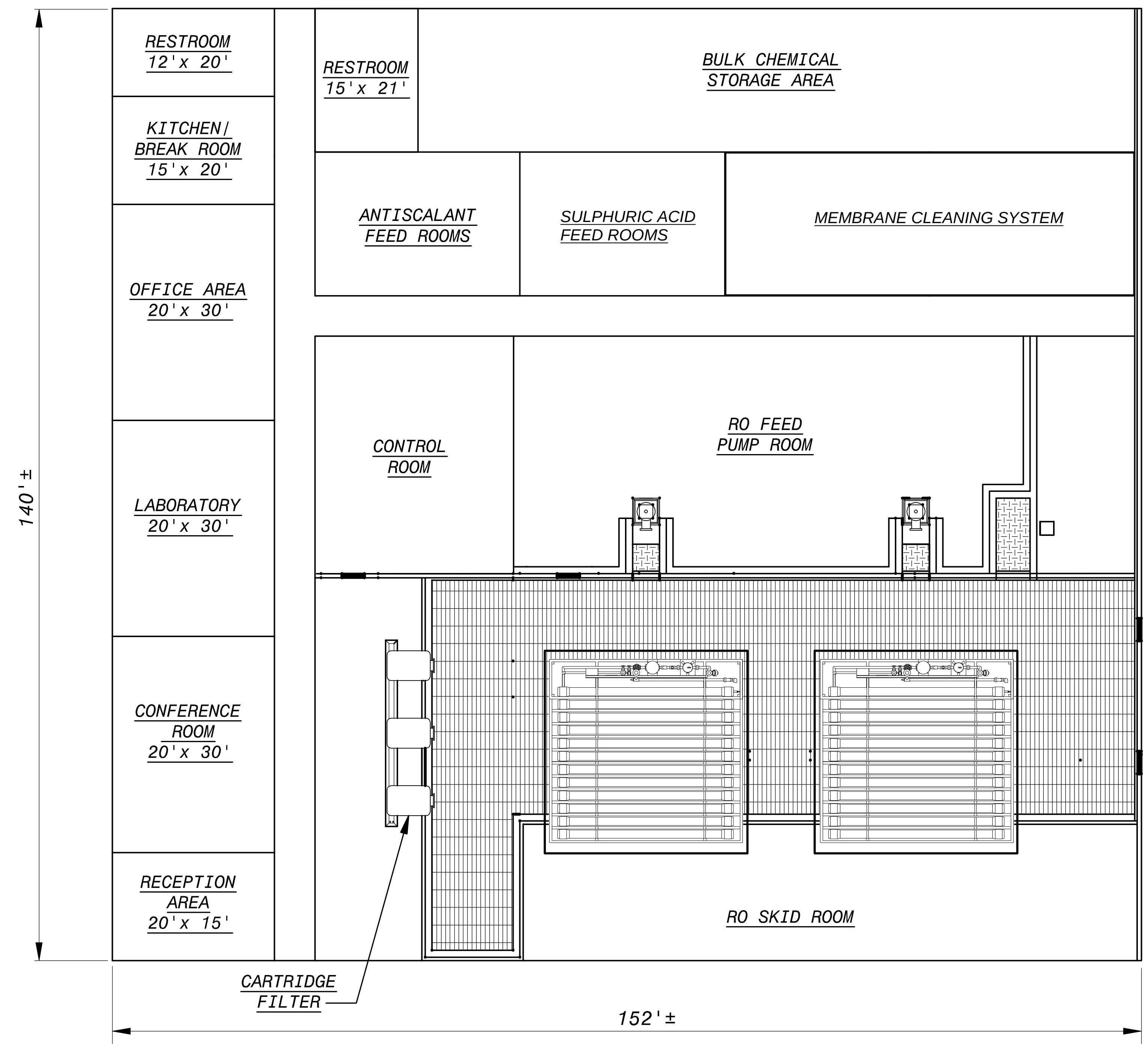

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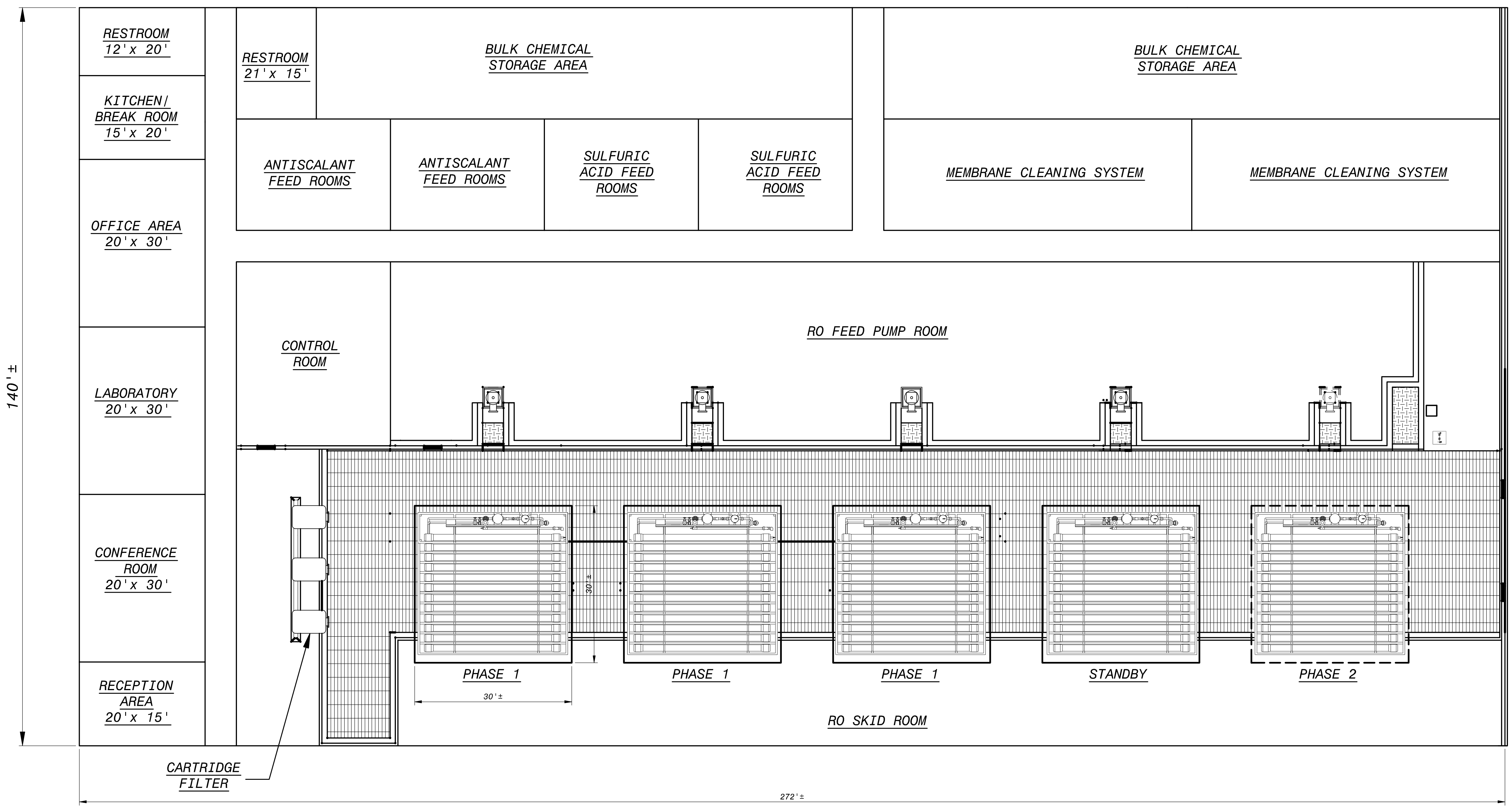

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PRELIMINARY CAMERON 26.8 MGD FLOOR PLAN

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8.7 COST OPINIONS

Planning level Engineers Opinion of Probable Costs (EOPCs) were developed for the two plants utilizing previous projects of similar size and with similar treatment processes.

8.7.1 Description and Methodology

Standard procedures were used to estimate cost on a cost per unit basis. Previous project experience was utilized in obtaining and verifying costs included in the estimates. Costs shown in the report, unless described otherwise are in 2015 dollars. An inflation rate of 3 percent was utilized to project costs to 2020 dollars as needed. For future projections, the Construction Cost Index as reported by Engineering Review in November 2015 is 10092.

8.7.2 Professional Services

Estimates for Pre-Design Phase, Design and Construction Phase, Program Management and Construction Management, and Permitting costs were combined into a professional services category and were calculated to total 25 percent of the infrastructure cost. This is in line with standard estimating procedures of a cost estimate at this level.

8.7.3 Water Supply and Treatment

The estimated cost includes costs associated with the well field, raw water conveyance to the treatment plant, treatment facility, storage, and disposal of wastes from the concentrate and pretreatment process. Costs for land acquisition required for this portion of the project are also included.

8.7.3.1 Well Field

The estimated cost for well construction includes drilling of all production, as well as the pumps, site development, electrical work, collection piping, and access roads within the well field. A unit cost per well was determined from previous Black & Veatch projects. Daniel B. Stephens & Associates also provided prices on the well construction for comparison. As shown in Table 8-13 and Table 8-14 these unit prices were multiplied by the number of wells in the Eastern (Cameron) and Western (Hidalgo) Well Fields to produce a total cost.

Table 8-13 Eastern Well Field (Cameron) Costs

	COST PER UNIT	NO. OF UNITS	TOTAL
Production Well (115 HP, 400 ft deep)	$ 909,600	58	$ 52,756,200
Well Field Pipe (6 inch to 48 inch HDPE)	--	--	$ 50,589,600
Contractor Markup (10 percent Including Insurance/Bond)	--	--	$ 10,334,600
Total Well Field Costs			**$ 113,680,400**

HDPE = High density polyethylene.

Table 8-14 Western Well Field (Hidalgo) Costs

	COST PER UNIT	NO. OF UNITS	TOTAL
Production Well (115 HP, 500 ft deep)	$ 947,600	18	$ 17,056,600
Well Field Pipe (6 inch to 48 inch HDPE)	--	--	$ 12,553,300
Contractor Markup (10 percent Including Insurance/Bond)	--	--	$ 2,961,000
Total Well Field Costs			**$ 32,570,900**

HDPE = High density polyethylene.

Prices per linear foot were developed for the well field pipelines and multiplied by the length of each pipeline. The unit costs were based on the pipeline diameter and incorporate costs for installation, fittings, trench excavation and safety protection, erosion and sedimentation controls, hydrostatic testing, restoration, and other items typically required to install a transmission main. These unit prices were developed from similar projects.

8.7.3.2 Treatment Facilities

In order to estimate the total cost for Cameron and Hidalgo treatment plant, the costs were broken out into costs associated with the building and process equipment. All treatment costs are summarized in Table 8-15 and Table 8-16. The costs for the buildings are based on unit prices per square foot obtained from previous projects. Process and storage costs were developed by comparing flows of previous projects and utilizing the ratios for each process stream. The ratios were tempered from a linear characterization by a modularity exponent. It was assumed that traditional treatment processes were less modular in nature and therefore less linear for cost escalation. Contrarily, RO process equipment scales almost linearly.

Table 8-15 Cameron Treatment Facility Costs

CAMERON TREATMENT FACILITY (26.45 MGD)	
ITEMS	**TOTAL**
Iron Removal System (includes cost for oxidation, pressure filters, backwash equalization basin/tank, backwash clarifier, sludge handling)	$ 2,900,000
Pretreatment Building	$ 1,530,000
Transfer Pumps	$ 1,831,800
Process Building	$ 3,360,000
RO Process Equipment	$ 14,000,000
Post Treatment Area	$ 472,500

CAMERON TREATMENT FACILITY (26.45 MGD)		
ITEMS	TOTAL	
Transfer Pump Station	$	2,289,750
Disinfection (Chlorine Feed)	$	396,000
Clear Well (2 MG @$0.55/gal)	$	1,100,000
Subtotal	$	27,880,000
Mobilization (3%)	$	836,000
Yard Piping (5%)	$	1,394,000
Sitework (10%)	$	2,788,000
Electrical and I&C (10%)	$	2,788,000
SUBTOTAL	$	35,700,000
Contractor Markup @ 10% (Including Insurance/Bond)	$	3,570,000
TOTAL	**$**	**39,270,000**

Table 8-16 Hidalgo Treatment Facility Costs

HIDALGO RO TREATMENT FACILITY (8.43 MGD)		
ITEMS	TOTAL	
Iron Removal System (includes cost for oxidation, pressure filters, backwash equalization basin/tank, backwash clarifier, sludge handling)	$	1,500,000
Pretreatment Building	$	660,000
Transfer Pumps	$	610,600
RO Process Building	$	1,638,000
RO Process Equipment	$	6,200,000
Post Treatment Area	$	262,500
Transfer Pump Station	$	915,900
Disinfection (Sodium Hypochlorite Feed System)	$	198,000
Clearwell (0.5 MG @$0.55/gal)	$	275,000

HIDALGO RO TREATMENT FACILITY (8.43 MGD)	
ITEMS	**TOTAL**
Subtotal	$ 12,260,000
Mobilization (3%)	$ 367,800
Yard Piping (5%)	$ 613,000
Sitework (10%)	$ 1,226,000
Electrical and I&C (10%)	$ 1,226,000
SUBTOTAL	$ 15,700,000
Contractor Markup @ 10% (Including Insurance/Bond)	$ 1,570,000
TOTAL	**$ 17,270,000**

8.7.4 Concentrate Disposal

Pipeline costs were developed similar to the pipeline for raw water conveyance and are shown in Table 8-17 and Table 8-18. A price per linear foot based on the SAWS BGD 90% EOPCC was developed and multiplied by the required quantity. Lengths for the discharge lines were routed along existing roadways and were estimated to be approximately 5 miles for the Hidalgo Plant and 2 miles for the Cameron Plant.

Table 8-17 Cameron Concentrate Disposal Capital Cost

DISPOSAL FACILITIES	TOTAL
Concentrate Pipe (2-miles of 24 inch HDPE)	$ 1,928,000
Contractor Markup (including Insurance/Bond)	$ 192,800
Total Concentrate Disposal Costs	**$ 2,121,000**

Table 8-18 Hidalgo Concentrate Disposal Capital Cost

DISPOSAL FACILITIES	TOTAL
Concentrate Pipe (5 miles of 12 inch HDPE)	$ 2,359,000
Contractor Markup (including Insurance/Bond)	$ 235,900
Total Concentrate Disposal Costs	**$ 2,595,000**

8.7.5 Operations

Operations and maintenance costs for the well field development and production portion of this project were estimated using typical costs for similar applications. The electrical usage, staffing requirements, chemical dosage, and miscellaneous consumables were projected, and approximate costs associated were calculated.

Annual electrical estimates were determined for the production wells, well field pumping stations, and RO plant. Pumping station and well electrical usage and costs were estimated using the flow and head required for each application. The ratio of required power to total flow from a similar desalination plant was multiplied by the total flow for this project in order to estimate the annual power requirement for the WTP. $0.05/KWh was used as the unit cost for electricity for both BGD well fields and treatment locations. This cost is based on input from project stakeholders.

Staffing projections were made utilizing the staffing estimates for similar size plant. Reasonable approximations were used to estimate the amount of staff that would be assigned to each facility. It was assumed that the operators, the plant manager, maintenance mechanics and I&C staff would support the operations. Typical hourly wages for personal at the managerial and various staff levels were used, and a 10% percent annual overtime amount and 40% burden rate were taken into account. Refer to Table 8-19 for water supply and treatment operations and maintenance costs.

Table 8-19 Water Supply and Treatment Staffing Costs

	CAMERON	HIDALGO
Plant Manager(1)	$ 116,500	$ 58,200
Operator Tech 1	$ 63,300	$ 89,100
Operator Tech 2	$ 79,500	$ 98,700
Operator Tech 3	$ 89,100	$ -
Operator Tech 4	$ 98,700	$ -
I&C	$ 66,200	$ 66,200
Maintenance Mechanic	$ 72,900	$ 72,900
Total	**$ 586,100**	**$ 385,100**

(1)Assumed 50 percent of the plant manager's time for the Hidalgo plant.

Chemical consumption was projected using information from similar RO facilities. Typical dosages and concentrations were applied to the treatment plant flow rates in order to calculate the annual usage of each chemical. Annual chemical consumption was multiplied by prices obtained from actual vendors to determine the total cost per year.

Additional operations and maintenance costs were estimated such as replacement equipment for the RO treatment and consumables which include other miscellaneous needs of the facilities. Approximate costs for the RO treatment replacement equipment are annual costs for replacing RO membranes, cartridge filters, pumps, and valves. Even though individual replacement rates vary, this is an estimate of the annual cost to replace each of them at their respective end of life. The costs and life expectancy were based on current knowledge of the RO facilities. Refer to Table 8-20 for water supply and treatment operations and maintenance costs.

Table 8-20 Water Supply and Treatment O&M Costs

	COST FOR CAMERON	COST FOR HIDALGO
Energy Cost	**$ 3,009,000**	**$ 837,000**
RO Replacement Equipment	**$ 523,000**	**$ 164,000**
Consumables	**$ 130,000**	**$ 41,000**
Chemicals	**$ 2,033,000**	**$ 597,000**

8.7.6 Land Acquisition

Easement and property acquisition costs were shown separately from capital costs. Easement costs were calculated based on area requirements for the well field conveyance pipeline, and concentrate conveyance pipeline. Property acquisition costs were calculated based on estimated area needed for production well, and treatment facilities. A unit cost of $4,500/Acre was used for easements and $5,000 for property acquisition and multiplied by the area required for easements and property. Estimated costs are shown in Table 8-21 and Table 8-22.

Table 8-21 Land Acquisition Costs for Cameron Well Field and Plant

PROPERTY	UNIT	QUANTITIES	UNIT COST	TOTAL COST
	ROW Width*	Length (LF)	($/Acre)	
Well Field Conveyance Pipeline	50 feet	318,230	$4,500	$ 1,644,000
Concentrate Disposal Pipeline	25 feet	10,560	$4,500	$ 27,000
	Area Per Unit (AC)	No. of Wells	($/Acre)	
Production Well	0.7	58	$5,000	$ 204,000
Treatment Facility	12.2		$5,000	$ 61,000
				$ 477,000
Total				**$ 2,387,000**

*ROW = Right-of-Way.

Table 8-22 Land Acquisition Costs for Hidalgo Well Field and Plant

PROPERTY	UNIT	QUANTITIES	UNIT COST	TOTAL COST
	ROW Width*	Length (LF)	($/Acre)	
Well Field Conveyance Pipeline	50 feet	102,570	$4,500	$ 530,000
Concentrate Disposal Pipeline	25 feet	24,400	$4,500	$ 55,000
	Area Per Unit (AC)	No. of Wells	($/Acre)	
Production Well	0.7	18	$5,000	$ 63,000
Treatment Facility	5.4		$5,000	$ 27,000
Contingency (25%)				$ 169,000
Total				**$ 845,000**

*ROW = Right-of-Way.

8.7.7 Cost Summary

The final costs for the Cameron and Hidalgo RO Treatment plants are summarized as shown in Table 8-23 and Table 8-24.

Table 8-23 Cameron Treatment Plant Cost Summary

Cost at Full Build Out (26.45 MGD)		2015 Present Worth	2020 Inflation Rate: 3%
Construction Costs			
Well Field		$ 113,680,000	$ 131,787,000
Treatment Facilities		$ 39,270,000	$ 45,525,000
Concentrate Discharge		$ 2,121,000	$ 2,459,000
Electrical Infrastructure		$ 4,037,000	$ 4,680,000
Contingency	25%	$ 38,768,000	$ 44,943,000
Total Construction Cost		$ **197,876,000**	**$ 229,394,000**
Engineering Cost			
Pre-Design Phase	1%	$ 1,979,000	$ 2,294,000
Design and Construction Phase	15%	$ 29,682,000	$ 34,410,000
Program Mgt./Construction Mgt.	8%	$ 15,831,000	$ 18,352,000
Permitting	1%	$ 1,979,000	$ 2,294,000
Total Engineering Cost	25%	**$ 49,471,000**	**$ 57,350,000**
Land Acquisition		$ 2,387,000	$ 2,768,000
Total Capital Costs		**$ 249,734,000**	**$ 289,512,000**
Annual Operating Costs		**$ 7,999,000**	**$ 9,278,000**

Table 8-24 Hidalgo Treatment Plant Cost Summary

Cost at Full Build Out (8.43 MGD)		2015 Present Worth	2020 Inflation Rate: 3%
Construction Costs			
Well Field		$ 32,571,000	$ 37,759,000
Treatment Facilities		$ 17,270,000	$ 20,021,000
Concentrate Discharge		$ 2,595,000	$ 3,009,000
Electrical Infrastructure		$ 870,000	$ 1,009,000
Contingency	25%	$ 13,109,,000	$ 15,197,000
Total Construction Cost		$ **197,876,000**	**$ 76,995,000**
Engineering Cost			
Pre-Design Phase	1%	$ 1,979,000	$ 770,000
Design and Construction Phase	15%	$ 29,682,000	$ 11,550,000
Program Mgt./Construction Mgt.	8%	$ 15,831,000	$ 6,160,000
Permitting	1%	$ 1,979,000	$ 770,000
Total Engineering Cost	25%	**$ 49,471,000**	**$ 19,250,000**
Land Acquisition		$ 2,387,000	$ 845,000
Total Capital Costs		**$ 86,867,000**	**$ 97,227,000**
Annual Operating Costs		**$ 2,605,000**	**$ 3,027,000**

CHAPTER 9 - SEAWATER DESALINATION

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

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Building a world of difference.

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9.0 Seawater Desalination

9.1 PURPOSE

Seawater use in the Lower Rio Grande Valley (LRGV) provides a drought proof supply that is does not have considerable fluctuations in water quality or is reliant on international agreements. This chapter will evaluate seawater from the Gulf of Mexico as a regional water resource for the study area and will provide a conceptual seawater project to provide municipal drinking water for the valley.

9.2 INTRODUCTION

Seawater desalination involves the treatment of saline seawater through one of several desalting technologies to remove minerals, salts, and other materials and produce high quality water for municipal or industrial use. The most commonly used seawater desalination processes are either thermal or membrane based processes. Although thermal desalination is the oldest desalination technology, seawater desalination using reverse osmosis (RO) membranes began to take hold 20 years ago and has dominated the market as the preferred technology for the last decade. Seawater RO (SWRO) currently accounts for about 55 percent of water produced from seawater desalination during the past 10 years, followed by thermal processes, such as multi-effect desalination (MED) and multi-stage flash desalination (MSF). Among thermal processes, MSF was historically favored, although, with larger trains now possible, MED is gaining market share among total number of thermal facilities.

Thermal processes require large amounts of thermal energy, usually in the form of steam. They are more energy intensive, have higher capital costs, and require larger footprints than SWRO. Thermal processes also require comparatively large amounts of electrical energy. Total energy consumption for thermal processes, accounting for equivalent energy in steam, ranges from 20-45 kW·h/kgal. The energy required for SWRO ranges from 12-17 kW·h/kgal. Thermal processes are widely implemented in the Arabian Gulf where energy costs are low, and the reliability of the thermal processes is preferred. For this project, reverse osmosis is the preferred desalination technology as SWRO systems offer several advantages over other available desalting technologies, including higher recovery rate, lower capital cost and lower energy consumption.

Like all potential water sources, site location and water quality will determine the overall cost of producing the water; however, residuals disposal can have a large impact on project costs as well.

9.3 CURRENT USE

Seawater desalination plants are common internationally. In the US, there are two large scale facilities – the Tampa Bay desalination facility in Florida that is in operation and the Carlsbad seawater desalination plant in California that is planned to be commissioned in October, 2015.

To date, there have been no full scale seawater desalination facilities constructed along the Texas Gulf Coast. However, TWDB has funded three seawater desalting pilot and feasibility studies in the LRGV since 1997. Two seawater desalination plants are recommended as alternatives in the 2016 Region M Regional Water Plan for Brownsville Public Utility Board (BPUB) and Laguna Madre Water District.

9.4 RULES AND REGULATIONS

As is the case with any water source, to minimize environmental impacts due to construction and operation several regulatory agencies require permits be obtained. The majority of required permits are dictated by local and state agencies. However, more comprehensive permits will be required by the federal government if certain environmental impact thresholds are exceeded.

9.4.1 Federal Permits

Likely federal permits for a seawater desalination plant include US Army Corps of Engineers Section 10 permit for construction of seawater inlet structure in the Laguna Madre, Brownsville Navigation Channel or Gulf Coast. A Section 404 permit from USACE may also be required depending on the plant siting due to wetlands or use of fill or dredged material.

Coordination with the US Fish and Wildlife Service would be needed to mitigate or avoid potential impacts to endangered species and other wildlife resources. Similarly a National Marine Fisheries Service consultation would be needed to evaluate the impacts on essential fish habitat in the Gulf of Mexico and potential impacts due to impingement and entrainment.

The US Environmental Protection Agency requires permits for fuel storage that is typically used for alternative power generators if the volume exceeds 1,320 gallons as required in the Oil Pollution Prevention Rule.

9.4.2 State Permits

In order to protect both the public and environment the State of Texas has created several rules and regulations contained in the Texas Administrative Code Title 30 Environmental Quality (30 TAC). They are administered through various state agencies.

The Texas Commission on Environmental Quality (TCEQ) requires several permits to construct and operate water treatment facilities. Specifically, a Texas Pollution Discharge Elimination System (TPDES) permit will be required if design of the plant requires any discharge of waste from the system into a water body. This waste could come from stormwater collection and discharge, brine concentrate, cleaning solutions, backwash water or tank overflows. Public Water System Permit by Rule (PBR) is also required as described in 30 TAC Section 290 and is administered by the TCEQ.

The Texas Parks and Wildlife Department (TPWD) requires consultation and reports to determine the potential impacts of the project on any state listed threatened or endangered species. Specifically, the purpose of this permit is to prevent the loss of or damage to wildlife resources and may require an evaluation by the TPWD. Also, a Sand and Gravel Permit may be required in certain situation where materials are disturbed or removed.

The Texas Historical Commission will require an Antiquities Permit and further consultation if federal funds, permits or lands are impacted by the project.

The Texas Land Office requires conformance with Texas Coastal Management Program and easement acquisition on state lands. This requires previously mentioned permits be reviewed for consistency.

Any alterations to state owned roads or right-of-ways will require coordination with Texas Department of Transportation to the extent needed for the alterations.

9.4.3 Local Permits

Local authorization for a seawater project may include the following:

- Building/Occupancy Permits
- Zoning Permits
- Noise Requirements
- Conditional Use Permit
- Beach and Dune Permit
- Floodplain Letters
- Road Construction Permits

9.5 WATER QUALITY

The design of the SWRO plant is very much dependent on the raw seawater quality and the treated water quality goals. These are discussed below.

9.5.1 Raw Water Quality Data

The seawater quality will have considerable impact on the pretreatment upstream of RO. If the pretreatment system is not appropriately designed to address the water quality, the performance of downstream RO process is impacted because of fouling of the RO membranes. Indeed, inadequate pretreatment is often the most cited cause of desalination plants that fail to meet treatment goals.

Given the importance of the seawater quality and selection of pretreatment processes, often the location of the intake is selected based on the anticipated water quality. There are primarily two types of intakes possible – open intakes and subsurface intakes, which are discussed in Section 9.6.1.

There is limited raw seawater quality data available near the location of the proposed desalination plant. The available data is limited to the following studies:

- Report completed for the TWDB and Laguna Madre Water District in 2010: Water quality was assessed in both the Lower Laguna Madre Bay and the Gulf of Mexico east of South Padre Island.
- Report funded by TWDB: The City of Brownsville analyzed seawater quality at the Port of Brownsville.
- Texas Seawater Desalination Pilot Study Report: has data in the ship channel and the Gulf of Mexico.

The water quality data for very few parameters were presented in most of these reports and are summarized in Table 9-1. Parameters include temperature, salinity, chloride, boron, turbidity and pH. In general, the available data suggests that the water quality in the Lower Laguna Madre Bay and the Gulf of Mexico is similar and are representative of typical seawater quality with respect to inorganic parameters. While these parameters provide some of the necessary data for the design of the desalination process, the complete water quality data necessary for the design of the pretreatment systems was not included in these reports. Data needed for final pretreatment process selection will include parameters such as total suspended solids, chlorophyll-a, and algal cells.

Table 9-1: Expected Water Quality of Seawater in the LRGV at Various Locations per TWDB Reports

PARAMATER	LOWER LAGUNA MADRE BAY	GULF OF MEXICO	LMWD STAFF SAMPLE (GULF OF MEXICO)	BROWNSVILLE SHIP CHANNEL (BPUB)
Temperature (C)	22.8 (9.5-30.6)	22.4 (11.7-32.2)	N/A	23.9 (7.9-31.1)
Salinity (ppm)	34k (23k – 41k)	34k (22k – 39k)	35k	32k (16k – 40k)
Conductivity (µS/cm)	52k (37k – 57k)	50k (36k – 60k)	62.5k	50k (27k – 60k)
pH (SU)	8.1 (6.5-11.0)	8.1 (6.9-8.9)	8.19	8.1 (7.3 – 11)
Chlorides (mg/L)	18.8k (7.8k – 27.1k)	19.0k (13.3k – 23.4k)	19.3k	18.7k (9.1k – 29.0)k
Total Boron (mg/L)	7.0	7.32 (3.35 – 21.1)	N/A	4.1
Iron (mg/L)	N/A	N/A	N/A	0.109 (0.003 – 0.215)
Arsenic (µg/L)	N/A	10	N/A	N/A
Turbidity (NTU)	N/A	4.89 (0.062 – 20.7)	N/A	44.7 (0.305 – 2,745)

9.5.1.1 Brownsville Ship Channel Water Quality

Water quality monitoring in the Brownsville Ship Channel was conducted by both the BPUB laboratory and an independent laboratory for various parameters. The collective raw water quality data is summarized in Table 9-2. Water quality in the Brownsville Ship Channel is less favorable than water quality from the Gulf of Mexico, and will impact pretreatment process selection, as well as solids handling. When suspended solids are present at high concentrations, clarification followed by effective pretreatment process is required to protect the RO membranes. With clarification comes the need for a solids handling system to treat the waste flows derived from the clarification process. However, in the case of the initial10 MGD plant, the economic benefits of locating the intake at the Brownsville Ship Channel outweigh the process benefits of drawing water directly from the Gulf of Mexico. Even considering the additional pretreatment and solids handling facilities, the cost can be significantly reduced because of the material and construction savings from a shorter intake pipeline.

Table 9-2 presents a summary of the expected raw water quality from the Brownsville Ship Channel, which will dictate the process selection for the initial SWRO plant located within the Port of Brownsville. Further sampling should be conducted for parameters like algae and total suspended solids.

Table 9-2: Summary of Raw Water Quality in Brownsville Ship Channel from Texas Seawater Desalination Pilot Study Report

PARAMETER	UNITS	NO. OF DATA POINTS	MAXIMUM	MINIMUM	AVERAGE	95^{TH} PERCENTILE
Turbidity	NTU	54,651	2,745	0.305	44.7	121.8
Total Organic Carbon (TOC)	mg/L	403	7.768	2.029	3.525	4.517
Dissolved Organic Carbon (DOC)	mg/L	403	6.351	1.664	3.252	4.117
UV254	cm-1	404	0.13	0.019	0.047	0.07
Alkalinity (as CaCO3)	mg/L	404	318.5	109.4	140.96	155.2
pH	-	448	8.66	7.12	8.01	8.27

PARAMETER	UNITS	NO. OF DATA POINTS	MAXIMUM	MINIMUM	AVERAGE	95TH PERCENTILE
Oil & Grease	mg/L	3	ND	ND	ND	N/A
Boron[1]	mg/L	13	19.3	3.02	7.75	17.8
Strontium	mg/L	14	7.98	2.23	5.69	7.73
Calcium	mg/L	14	434	357	386	418
Magnesium	mg/L	14	1,330	911	1,135	1,310
Potassium	mg/L	13	684	417	487	661
Sodium	mg/L	14	10,500	6,390	8,468	10,175
Silica	mg/L	9	116	ND	24	29.5
Barium	mg/L	14	0.318	ND	0.086	0.242
Sulfate	mg/L	14	6,380	1,850	2,642	4,365
Fluoride	mg/L	14	ND	ND	ND	ND
Nitrate-Nitrogen, Total	mg/L	13	2.62	ND	2.62	1.048
Chloride	mg/L	13	25,500	13,900	17,083	24,360
SOCs	mg/L	6	ND	ND	ND	ND
VOCs	mg/L	6	ND	ND	ND	ND
HAA5	mg/L	1	ND	ND	ND	ND
Bicarbonate (as CaCO3)	mg/L	10	433	144	171	313
Carbonate (as CaCO3)	mg/L	10	6.46	2.49	3	5.99
Color, True	PCU	9	10	ND	8	10
Color, Apparent	PCU	9	25	ND	12	25
Total Dissolved Solids[2]	mg/L	445	34,400	17,600	29,800	33,300
Total Dissolved Solids[3]	mg/L	14	46,800	28,100	30,515	39,585

[1] Note that some of the values were abnormal. For example, minimum concentration of sodium, and maximum and 95th percentile concentrations of boron are quite abnormal.
[2] TDS calculated using a conversion factor of 0.62 to covert conductivity to TDS
[3] TDS measured from periodic grab samples from the Brownsville Ship Channel

9.5.1.2 Gulf of Mexico Water Quality

Water quality monitoring in the Gulf of Mexico was conducted by both the BPUB laboratory and an independent laboratory for various parameters during the desalination pilot study. The collective raw water quality data from the BPUB study is summarized in Table 9-3. Since water quality samples were taken at a depth of 10 ft, it may not be entirely representative of the expected water quality at the proposed depth of the intake structure (the proposed intake location may be deeper than 10 ft). Also, the exact distance of the sampling site from the shore was not indicated. It was also reported that for safety reasons, the samples were not collected during stormy conditions and suggested that the water quality can be worse than this during storms.

Table 9-3: Summary of Raw Water Quality in Gulf of Mexico from Texas Seawater Desalination Pilot Study Report

PARAMETER	UNITS	NO. OF DATA POINTS	MAXIMUM	MINIMUM	AVERAGE	95TH PERCENTILE
Turbidity	NTU	27	20.7	0.062	4.89	11.95
Total Suspended Solids (TSS)	mg/L	10	206	4.5	41.2	145
Total Organic Carbon (TOC)	mg/L	27	4.12	1.36	2.08	3.56

PARAMETER	UNITS	NO. OF DATA POINTS	MAXIMUM	MINIMUM	AVERAGE	95TH PERCENTILE
Dissolved Organic Carbon (DOC)	mg/L	27	3.19	1.41	1.99	2.96
UV254	cm-1	27	0.056	0.008	0.0231	0.0514
Alkalinity (as CaCO3)	mg/L	27	133.1	118.5	124.8	131.3
pH	-	11	8.29	7.86	8.14	8.28
Oil & Grease	mg/L	3	ND	ND	ND	ND
Boron[1]	mg/L	10	21.1	3.35	7.32	20.16
Strontium	mg/L	10	8.92	2.22	5.73	8.37
Calcium	mg/L	10	460	336	387	456
Magnesium	mg/L	10	1,400	1,010	1,227	1,395
Potassium	mg/L	10	684	394	539	673
Sodium[2]	mg/L	10	-	7,750	9,221	11,040
Silica	mg/L	7	12.3	0.387	2.78	9.26
Barium	mg/L	10	0.0424	0.0101	0.0197	0.035
Sulfate	mg/L	10	5010	2280	2830	4160
Fluoride	mg/L	10	5.42	ND	0.542	2.98
Nitrate-Nitrogen, Total	mg/L	10	ND	ND	ND	ND
Chloride	mg/L	10	25,300	14,700	19,450	23,545
SOCs	mg/L	3	ND	ND	ND	ND
VOCs	mg/L	4	ND	ND	ND	ND
HAA5	mg/L	1	ND	ND	ND	ND
Bicarbonate (as CaCO3)	mg/L	10	148	107	125.3	144
Carbonate (as CaCO3)	mg/L	10	2.53	1.4	1.893	2.39
Color, True	PCU	10	ND	ND	ND	N/A
Color, Apparent	PCU	10	20	ND	4	15.5
Total Dissolved Solids (TDS)	mg/L	10	38,200	26,000	34,170	37,930

[1]Note that some of the values were abnormal. For example, minimum concentration of sodium, and maximum and 95th percentile concentrations of boron are quite abnormal.
[2] Maximum concentration of Sodium from available reference sources had a typo and is not confirmed.

9.5.1.3 Critical Water Quality Parameters

Total Suspended Solids & Turbidity: The concentration of colloidal and particulate material is typically characterized by measured concentration of total suspended solids and turbidity. Effective removal of colloidal and particulate material during pretreatment is necessary in order to minimize fouling of the RO membrane elements. Hence, the concentration of total suspended solids and turbidity has a direct impact on the pretreatment process design and selection. The water quality data indicates potential for high concentrations of suspended solids. However, the corresponding turbidity values are low. Because of these inconsistencies, further sampling is recommended. If suspended solids are indeed present at high concentrations, a clarification process is required upstream of filtration process. For the purposes of the preliminary design, it is assumed that the turbidity values are more representative than TSS, given that there are more turbidity samples available.

Algae: One primary constituent in seawater that may have significant impact on the design of the pretreatment is potential for occurrence of algal blooms. Depending on the environmental conditions, these organisms can increase in number considerably, resulting in high concentrations of cells during blooms. If there is possibility of occurrence of algal blooms, the pretreatment should include processes such as dissolved air flotation to remove algae. Otherwise, algae will clog up the downstream filtration process and in worst case, would require shutdown of the plant until the bloom ends. Typically, the concentration of chlorophyll-a is used as surrogate for measurement of algal activity. There is no data available from previous studies on algal cell concentrations of chlorophyll-a. Because of the lack of information on these parameters, it is uncertain whether additional pretreatment measures such as dissolved air flotation (DAF) will be required.

Oil & Grease: RO membranes can be irreversibly fouled by oil and grease in feed water and pre-treatment processes need to be designed to reduce their concentration. Typically, flotation processes such as dissolved air flotation are used for their removal. The limited water quality data currently available does not indicate high concentrations of oil and grease in the raw seawater.

Organics: High concentration of organics in the seawater could result in fouling of the RO membranes and may also result in biofouling of the RO membranes. The limited water quality data currently available does not indicate high concentrations of total or dissolved organic carbon.

9.5.2 Finished Water Quality Goals

Finished water quality goals were developed as part of the initial pilot study for the desalination demonstration project. Finished water quality goals as well as TCEQ and EPA maximum contaminant limits are presented in Table 9-4.

Table 9-4: Finished Water Quality Goals from BPUB Desalination Pilot Study Report

PARAMETER	UNITS	DESIGN GOALS	TCEQ OR EPA MAX CONTAMINANT LIMITS
pH	-	7.0 to 8.5	≥ 7.0
Alkalinity (as CaCO3)	mg/L	75 to 150	N/A
Total Hardness (as CaCO3)	mg/L	<250	N/A
Chlorides	mg/L	<300	<300
Turbidity	NTU	<0.3	<0.5
Dissolved Organic Carbon (DOC)	mg/L	<2	N/A
Color	color units	<5.0	<15.0
Total Dissolved Solids (TDS)	mg/L	<500	<500
Sulfates	mg/L	<300	<300
Boron	mg/L	1.5	No limit
TTHMs	mg/L	<.040	<.08
HAA5	mg/L	<.030	<.06
Giardia removal and inactivation	log	>3	>3
Virus removal and inactivation	log	>4	>4
Cryptosporidium removal and inactivation	log	>2	>2

Because there is no limit set for boron either by TCEQ or USEPA, the water quality goals previously established during the pilot study did not have any limit for boron. World Health Organization had a provisional guideline of 0.5 mg/L historically which was recently revised to 2.4 mg/L, based on health impacts. However, boron can have impact on several plants and crops. The concentration at which boron can impact the plants varies with some plants sensitive at concentrations less than 0.5 mg/L, while others can tolerate more than 2 mg/L. The treated water boron concentration should be evaluated based on the potential agricultural use of the water. For this level of study, boron concentration of 1.5 mg/L is targeted in the permeate. This is achieved through partial treatment of SWRO permeate with brackish water RO at elevated pH.

Another parameter that needs to be considered is the concentration of bromide in the treated water. Depending on the water with which the desalinated water is blended with and the concentration of organic carbon, there is potential for formation of brominated disinfection byproducts. This was not evaluated during the pilot study but should be evaluated during the next phase of the study. Based on the testing results, effluent bromide concentrate would need to be established.

9.6 DESALINATION PROCESS PARAMETERS

9.6.1 Seawater Intake

As discussed, the intake location has a major impact on seawater quality and selection of pretreatment processes. Typically, the location of the intake is selected based on the anticipated water quality. The two major types of intakes, open intakes and subsurface intakes, are described below.

9.6.1.1 Subsurface Intakes

These intakes involve beach wells or infiltration galleries. Since the seawater obtained through this type of intake passes through subsurface media, it is naturally filtered because transport of water through the subsurface is similar to filtration, with the porous subterranean geological formations acting as filter medium. The water quality is not influenced by tidal motion, ship traffic or seasonal variations. Beach wells are essentially wells drilled along a coast, with the goal of abstracting the seawater that flows through the subsurface media into the well, as shown in Figure 9-1. These wells can be either vertical, horizontal or slant/angle wells. The feasibility of this type of intake is very much dependent on the local geology and permeability of subsurface media. There are very few subsurface intake systems in the world for seawater desalination because of the large area needed for the wells for large plants. An alternative is infiltration gallery, which basically consists of a network of perforated/screened pipes (or laterals) buried at a shallow depth in coarse grained sand or gravel deposits beneath a surface water body, as shown in Figure 9-2. A pump draws the surface water downward through the thin layer of sand and into the laterals. The permeability of the sand at the location would affect the efficiency of the system. Sometimes engineered sand can be used to replace the natural sand available to improve permeability. This also requires large area as the loading rates are typically very low.

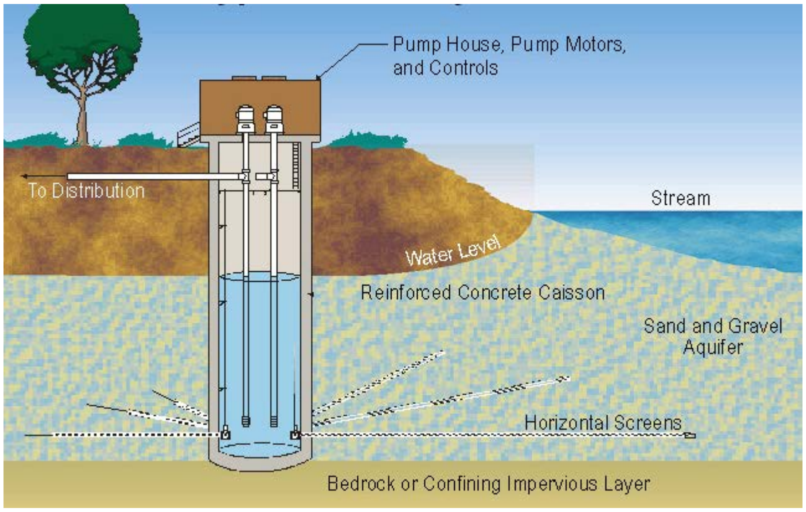


Figure 9-1: Radial Collector Well

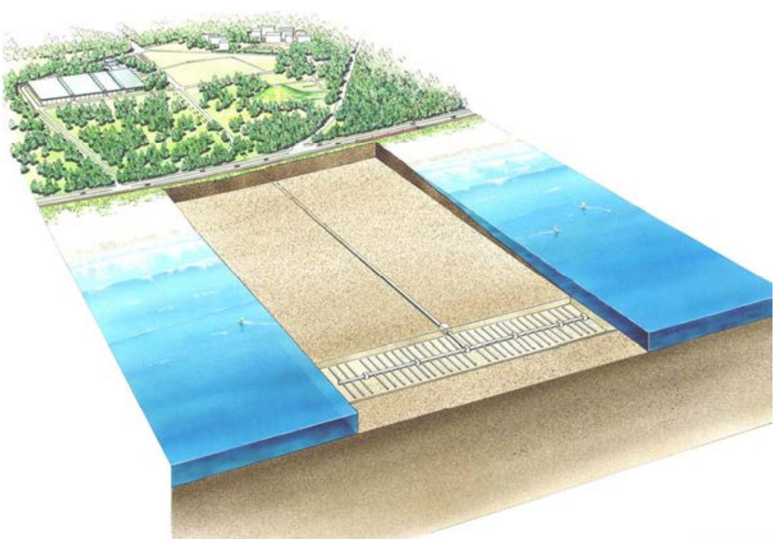


Figure 9-2: Infiltration Gallery

9.6.1.2 Open intakes

Open intakes can be onshore or offshore intakes. Onshore intakes include a simple structure such as a jetty, channel or lagoon to take in seawater. The major advantage of near-shore extraction is the lower construction cost due to the proximity and shallow depths generally required. The disadvantage is water quality is expected to be poor in comparison to water withdrawn from offshore intakes. Common contaminants associated with near-shore intakes include surface debris, biological growth, silt, oil and grease and algae, all of which will adversely affect treatment. The impact of the tidal activity is also significant for this intake option, compared to any other intake systems. This is because of the high penetration of the sunlight periodically during low tides, and increased temperatures that influence biological growth. Further, the tidal activity in the shallow area could also agitate the sediments, resulting in poor water quality. Figure 9-3 shows a near-shore intake that includes a diversion wall.

Figure 9-3: Infiltration Gallery

An offshore open intake includes pipeline(s) extending to a location far away (typically 1 to 2 miles) from the shore and at certain depth, as shown in Figure 9-4. It offers significantly better water quality than onshore intake and as the depth of submergence increases, the water quality improves further. Because of this, offshore intakes remain the predominant intake methods used worldwide.

Figure 9-4: Off-shore Open Intake

9.6.1.3 Seawater Intake Selection

Although subsurface intakes will provide high quality seawater to the desalination plant, they are not considered at this time because of the lack of information regarding subsurface geology and the amount of area required. Open intake systems will be considered for both plant designs. Two levels of screening will be implemented – coarse screens and fine screens. The primary purpose of coarse screens is to prevent large debris from damaging the fine screens, while the fine screens capture smaller contaminants such as shell fragments, sea weed fragments, etc.

Brownsville Intake: Based on the pilot testing done with water from the ship channel, it appears that onshore intake system will not yield optimal raw water quality. However, because BPUB has an existing pilot plant and additional property along the ship channel, the initial 10 MGD plant (expandable to 20 MGD) will be located along the ship channel with intake water drawn from the channel. The intake structure will be located along the bank of the channel, and will be provided with an inlet weir that allows for seawater from the channel to flow into an intake basin.

Gulf Coast Intake: In the case of future Gulf Coast Plant, which is planned for phased expansion from an initial capacity of 20 MGD in 2050 to an ultimate capacity of 80 MGD in 2070, an intake located in the Gulf of Mexico will be considered for the basis of design. The intake structure would consist of an offshore open intake system in the Gulf to achieve sufficient depth of submergence and also better quality than the water abstracted from the Ship Channel. It is assumed that it will be sited so that tides will not affect the intake of raw water and the intake head can be at a depth of 10 m or lower.

9.6.2 Pretreatment System

9.6.2.1 Coagulation/Flocculation

Use of coagulation is very common in seawater applications using either clarification and media filtration process or direct filtration using media filters. The coagulant alternatives include aluminum or iron salts. Typically iron salts are preferred because of the wider range of optimal pH and potential for RO membrane fouling by aluminum and also because of stringent discharge limitations for aluminum. Use of coagulant aids such as polymers is also practiced, particularly during challenging water quality periods. Since polymers have potential to foul the RO membranes irreversibly, their use is often limited. Flocculation is used when clarification is practiced. When direct filtration using granular media is implemented, flocculation is most commonly done in-line.

When membrane filters are used, use of coagulant is not always practiced. If water quality permits, direct filtration of the seawater with membrane filters is possible. If seawater has periods of high concentrations of small colloidal particles, high concentrations of organics or algae, coagulation can be practiced and the coagulated water can be directly treated by membranes. However, if the concentrations of suspended solids, organics, or algae are very high, then clarification will be required.

Based on the pilot testing conducted on the seawater from the Shipping Canal, which is of worse quality than that from the Gulf of Mexico, coagulant addition is considered at this level upstream of the membrane filters.

9.6.2.2 Clarification

Clarification is typically implemented only when the feed water has high concentrations of suspended material. When the suspended material is characterized by denser particles, then gravity settling processes such as plate settlers are implemented. When the suspended material is of low density, such as algae or high concentrations of organic compounds, or if oil and grease are a concern, then Dissolved Air Flotation (DAF) is practiced.

At this stage in the design, for the initial facility at the Shipping Canal, clarification by lamella plates is considered, because of the higher turbidity observed during movement of ship traffic in the channel. For the future facility that draws water from Gulf of Mexico, no clarification is provided. However, because there is limited data with regard to algae, a long term sampling program be initiated to determine the potential for algal blooms. Depending on the results, the need for dissolved air flotation can be re-evaluated.

9.6.2.3 Filtration process

Conventional Filtration: Pretreatment upstream of SWRO can include gravity media filters (single or two stages) or membrane filtration. Gravity filters are designed to treat relatively high quality seawater (low turbidity and suspended solids), with inline coagulation upstream. Typically, these are dual media filters with sand and anthracite. The number of stages is typically determined based on site specific water quality data. Single stage filtration is more common, although for challenging water sources, the need for two stages is established through pilot testing. The single stage media filtration on clarified water pilot tested previously appeared to fail to provide high quality feed water to the SWRO process, therefore membrane filtration is recommended.

Membrane Filtration: Microfiltration (MF) and ultrafiltration (UF) processes remove colloidal particles from water by straining it through hollow fiber polymeric membranes with microscopic pores. Because they provide a well-defined barrier, these membranes consistently produce treated water (filtrate) with very low turbidity. In recent years, the use of MF/UF systems as pre-treatment to RO membranes in seawater applications has increased. MF/UF systems have advantage of not requiring coagulant for significant amount of time, which eliminates or minimizes sludge production and also have smaller footprint compared to media filtration, particularly if two stage media filtration is deemed necessary. However, if there is potential for algal blooms, additional clarification process such as DAF will be required.

The primary issues associated with the use of MF/UF membrane filtration as pretreatment to SWRO is their limited experience and the proprietary nature of the systems. Although use of MF/UF for SWRO pretreatment has increased significantly, not all systems have extensive experience in seawater applications. And since the MF/UF membranes are not interchangeable, careful selection the MF/UF system is critical and should be based on extensive pilot testing. For instance, during the pilot testing, the results indicated that while one system performed well, the other one did not.

In recent years, universal MF/UF rack systems have been gaining traction. Universal rack systems are membrane skids built to accommodate more than one type of MF/UF elements, thereby allowing the owner/operator to choose from a wider variety of membrane elements. This can provide flexibility during design in terms of vendor selection, as well as during operations in membrane replacement. Although these systems cannot be provided currently with the elements of established membrane suppliers such as GE, Evoqua and Pall, the systems will accept elements from Toray, Dow, Inge, etc. Another advantage of using these systems for this project is that during the life of the Brownsville Channel Plant, different membranes can be used if needed and based on the experience gained, highly compatible MF/UF membrane elements can be picked for Plant 2.

9.6.3 Seawater Reverse Osmosis System

9.6.3.1 Cartridge Filters

Cartridge filters are disposable barriers of defined cutoff size, ranging from 1 µm to >20 µm, with 5 µm being the most commonly used. These elements are housed in a vessel and provide a safety barrier to prevent fouling of the RO membranes from poor filtered water quality resulting from any upsets in the pretreatment processes.

9.6.3.2 Reverse Osmosis

Reverse Osmosis (RO) is the heart of desalination process, where removal of the dissolved constituents occurs. The process uses semi-permeable membranes that inhibit passage of dissolved inorganic and organic substances, such as ions and total organic carbon (TOC) and allow passage of water. Water is pressurized above the osmotic pressure of the seawater to push a percentage of the feed water through the membrane. The rejected dissolved constituents are removed from the system through the remaining portion of the feed stream that exits the system as concentrate.

The fraction of the feed water that is converted to low salinity permeate is referred to as recovery and typically ranges from 40 to 50%. The basic unit of the process is a spirally wound RO membrane element. Typically 6 to 8 elements are connected in series and housed in a pressure vessel. Several pressure vessels are connected to the feed manifold in parallel to achieve the treatment capacity.

The feed pressure needed to overcome the osmotic pressure of seawater is high because of the high concentration of total dissolved solids in raw seawater. For instance, within the RO system, the TDS increases to 70,000 mg/L at 50% recovery. The driving pressure needed to overcome the osmotic pressure, concentration polarization at the surface of the membrane, and hydraulic losses; and any foulant layer can range from 800 to >1000 psi. The permeate stream exits the system at pressures required to convey it to the permeate storage tank. The concentrate exits the system at pressures close to the feed pressure, as hydraulic losses are typically in the range of 15 to 45 psi. The hydraulic energy in the concentrate stream is therefore very high and can be recovered and reused within the system by using energy recovery devices.

The overall energy consumption and operating costs of the RO system can be reduced through use of energy recovery devices to pressurize the RO feed water. Typically, the pressurized stream is split, with half of the flow going to the high pressure pumps (one dedicated for each RO unit) and the other half going to the energy recovery devices and booster pump (one dedicated for each RO unit). Both high pressure RO feed pumps and ERD booster pumps will be equipped with adjustable frequency drives (AFDs) to vary the RO feed pressure and produce the required permeate flow in response to changes in operating conditions such as water quality (salinity and temperature) and membrane age. Variations of these designs include high pressure pumps, RO trains and energy recovery systems arranged on common headers - the so called three- center design. The advantage of such configuration is lower footprint and cost. However, the turndown capability of such system is very limited and also the system would need to operate at varying operation conditions (such as recovery and flux).

9.6.3.3 Energy Recovery

Two types of devices can be used to recover the energy – centrifugal force or positive displacement based devices. Centrifugal devices such as a Pelton wheel involve conversion of hydraulic energy into mechanical energy and then back to hydraulic energy, resulting in lower efficiency. Use of positive displacement type devices involves direct transfer of energy from the concentrate stream to the feed stream, achieving very high efficiencies (>95%). During the last 10 years, the use of positive displacement devices has increased exponentially and is recommended for this project.

9.6.3.4 High Pressure RO Feed

As mentioned earlier, the RO feed pressure can be as high as 1000 psi. To reduce total energy demand at the RO feed pumps, energy recovery devices are typically used to transfer energy from the concentrate stream to the feed stream. Typically, feed flow equivalent to the permeate flow is conveyed to high pressure RO feed pumps and the remaining feed flow is conveyed to the energy recovery system. The energy recovery system received the concentrate from the RO and transfers the hydraulic energy in the concentrate to the feed flow entering the energy recovery device. After RO feed water has been pressurized through the energy recovery device, a booster pump is required to supply the incremental pressure required to achieve the feed pressure same as that of high pressure pump discharge. Both streams are then combined and sent through the RO modules. Within the RO train, the pressurized feed is evenly distributed among the pressure vessels. Part of the feed water passes through the membrane elements and exits the system as RO permeate. The remaining water, which is now concentrated with all the rejection ions, will be manifolded and routed to the Energy Recovery System (ERS). The energy in this stream is transferred to the low pressure feed flow entering the ERS.

9.6.3.5 Clean-In-Place System

Periodically, the RO membranes will need to be cleaned by means of a chemical clean-in-place (CIP) system. Cleaning will be triggered based on three operating set points - decrease in permeate, increase in permeate salinity or increase in pressure loss along the length of membrane elements. Cleaning solutions consist of dilute acidic and/or basic solutions, typically made up from citric acid, sodium hydroxide, detergents or proprietary chemicals. Cleaning is done as a batch process, where the CIP solution is heated and recirculated through RO trains. Spent solution is neutralized and blended with RO brine prior to discharge.

9.6.3.6 Second Pass RO

To meet the finished water quality targets assumed in this plan, approximately 30% of the RO permeate would need to be further treated by a second pass RO. During the next phase of design further evaluation of the source water and water quality targets is necessary to determine the percentage of 1st Stage RO permeate that should be passed through the Second Pass RO. This evaluation should consider potential uses of treated water. Specifically, if the treated water is to be used to irrigate certain types of plants, lower boron concentration will need to be targeted. See Appendix A for a list of common plants and their boron concentration sensitivities. Likewise, the potential second pass percentage may be affected by targeting a specific sulfate:chloride ratio or bromide concentration to limit the formation of brominated disinfection byproducts. The limit for the concentration of bromide is dependent on the quality of water it will blend with and would need to be determined based on testing with anticipated blending ratios with other water in the distribution system. This will be determined during next phase of design.

The size of the second pass BWRO can be greatly reduced by selectively treating only permeate from the lag elements of the SWRO train, as this stream has higher concentrations of all constituents as shown in Figure 9-5. The pH of the feed water to the second pass is increased to allow higher rejection of boron.

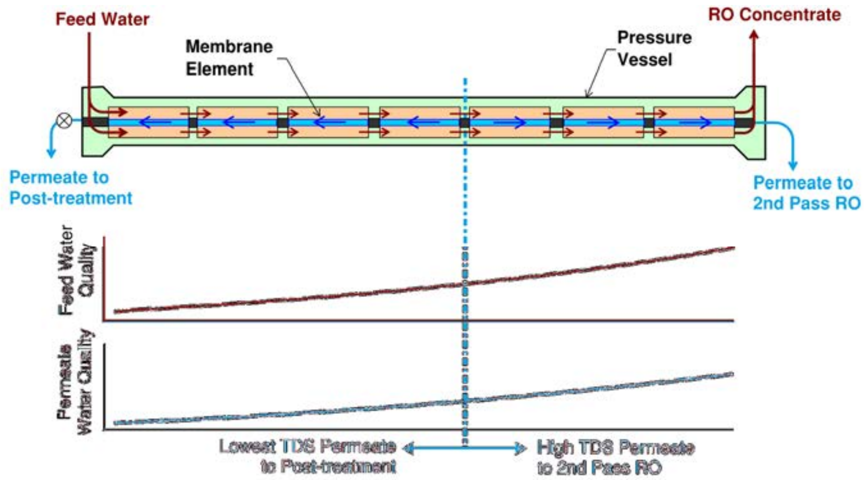


Figure 9-5: Split Partial Second Pass RO Configuration

The split partial second pass RO system configuration collects permeate from both ends of the pressure vessel and utilizes the lower TDS permeate from the front membranes of the 1st pass RO system for blending with 2nd pass RO permeate. The RO desalination will produce two streams: a product water stream referred to as permeate and a waste stream referred to as concentrate. Concentrate is further discussed under the Brine Disposal section.

9.6.4 Disinfection

Inactivation of pathogenic microorganisms can be achieved through use of chlorine compounds, chlorine dioxide, ozone or ultraviolet (UV) radiation. Advantages and disadvantages associated with each method of disinfection/oxidation are described in Chapter 8. While ozone and UV radiation are very effective disinfectants, they are relatively expensive, energy intensive, and must still be used in combination with chlorine or chloramines to maintain chlorine residual.

For the purposes of this study, chlorine disinfection is considered it has been used successfully in other regional facilities, chemical agents are readily available and the process is relatively low-cost to implement. In order to be compatible with other water supplies in the system, chloramines will be used to meet the chlorine residual requirements.

9.6.5 Brine disposal

Discharge pipeline to convey concentrated seawater or brine out to the ocean. The salinity of the brine is expected to be approximately twice that of the raw seawater, with total dissolved solids concentration ranging from 65,000 to 75,000 mg/L. Typically, brine disposal consists of a discharge pipeline with multiport diffusers to allow for proper dispersion of higher salinity water with seawater.

9.6.6 Solids Handling

Solids handling may involve a number of thickening or dewatering techniques to effectively manage solids in the waste streams produced by the pretreatment process of the SWRO desalination plant. Because of the proposed pretreatment process of the Gulf Coast Plant, waste streams are not expected to contain significant concentration of suspended solids, and consequently solids handling infrastructure is not required. However, solids handling will be required at the BNC Plant due to the expected concentration of suspended solids in the waste flows from the additional clarification process. A few solids handling methods are considered below.

9.6.6.1 Thickening Techniques

There are several methods available for thickening the solids prior to dewatering. These include gravity thickening and different mechanical devices such as gravity belt thickeners. Gravity thickening has been used predominantly in similar applications because of the simplicity of the process and limited operator attention required. In addition, the gravity thickener can provide thickened sludge storage, which will eliminate the need for additional downstream storage prior to dewatering.

Gravity belt thickeners would likely achieve a higher thickened sludge concentration, but the process would require greater operator attention and a thickened storage tank following the process to equalize the feed to the dewatering process. For these reasons, gravity thickening was assumed as the basis for this evaluation.

9.6.6.2 Dewatering

As with thickening, there are several technologies that could be used for dewatering the solids. These include mechanical systems such as belt filter presses, centrifuges, and pressure filters, as well as natural dewatering systems such as sand drying beds and evaporation beds. Both mechanical and natural dewatering systems are commonly used for these types of sludge, with site-specific constraints being a key factor in the final process selection.

Natural dewatering systems require a substantial footprint, have a high manual labor demand for removal, and are susceptible to weather constraints.

Mechanical systems have a small footprint and can be automated, but they are generally more expensive. Pressure filters can achieve the highest total solids concentration but are less commonly used because of their high capital cost and greater operator attention requirements. Centrifuges will generally achieve a higher total solids concentration than belt presses and require less operator attention, but they have greater power and polymer conditioning demands. In order to conserve footprint, mechanical dewatering using centrifuge technology was assumed for the solids handling system design at the BNC Plant.

9.7 INFRASTRUCTURE CONCEPT

Major components of the Brownsville Channel (BNC) Desalination Plant will include a seawater intake structure, raw water conveyance pipeline, pretreatment with flocculation, plate settlers and membrane filtration, 1st and 2nd pass reverse osmosis membrane treatment, post treatment and conditioning, disinfection, finished water storage, brine disposal and solids handling. The conceptual location for the seawater desalination plant is near Brownsville, because it is the largest population center closest to the coast. Desalinated water would also be pumped into the regional water supply line.

Major components of the Gulf Coast Seawater Desalination Plant will include a seawater intake structure, pretreatment with membrane filtration, 1st and 2nd pass reverse osmosis membrane treatment, post treatment and conditioning, disinfection, finished water storage and brine disposal. The conceptual location for this facility will be along the coast to reduce infrastructure required for the seawater intake and brine discharge systems.

9.7.1 Capacity

The availability of seawater is effectively unlimited for the purpose of this study. In order to provide a regional plant, it is assumed that the plant would initially be sized for a treated capacity of 10

MGD and expanded to 20 MGD in the year 2050. This plant is referred to as the Brownsville Navigation Channel (BNC) Plant. The design of the BNC Plant should be optimized for these capacities. It was assumed that future BNC Plant capacities may be limited based on the channel hydraulics and available land within the Port of Brownsville.

It is anticipated in 2050 a new facility will be built on the island. The new facility, referred to as the Gulf Coast Desalination Plant will initially be built for a treated capacity of 20 MGD (Phase I), expanded to 40 MGD (Phase II) and ultimately to 80 MGD by 2070 (Phase III).

Given the large size of this plant, the optimal train sizes will be considerably different than that of the BNC plant. Because the location of the intake structure, and consequently the expected variations in the raw water quality between the BNC Desalination Plant and the Gulf Coast Plant, the overall treatment process for each plant will also vary. The 10 MGD BNC Plant would demonstrate and optimize the treatment processes, performance and operations to be incorporated into the larger plants. The following table (Table 9-5) summarizes the planned phasing of drinking water supply to be produced from seawater desalination from 2020 to 2070.

Table 9-5: Seawater Desalination Plant Capacities under Phased Expansion

YEAR	BNC PLANT CAPACITY	GULF COAST PLANT CAPACITY
2020	10 MGD	-
2050	20 MGD	20 MGD
2060	20 MGD	40 MGD
2070	20 MGD	80 MGD

9.7.2 Intake Structure

In order to produce 20 MGD of drinking water a 45 MGD intake structure would be constructed for the BNC Plant. Likewise, to produce 80 MGD of drinking water, a separate 175 MGD intake structure would be constructed for ultimate capacity of the Gulf Coast Plant. An exact location for either plant has not been officially determined and is assumed for the purposes of this study. The BNC Plant intake will be located along the Brownsville Navigation Channel near the Gulf of Mexico and south of Port Isabel. The Gulf Coast Plant intake will be located on the north side of the barrier island with the intake located directly in the Gulf of Mexico. Each plant intake will be specifically positioned and located so that the changing of the tides will not affect the intake structures. The overall intake systems will consist of:

- BNC Plant Intake will be similar to a River intake with a weir, concrete basin, coarse screens, underground piping, fine screens and seawater pumping station.
- The Gulf Coast Plant Intake will have a submerged open intake with coarse screens in the Gulf of Mexico, a submerged pipeline, fine screens, and a seawater pumping station.

Further analysis of the anticipated site location will be required in order to determine the best application to be utilized for this project. In order to properly design the intake structure and intake pump station the following information would need to be gathered:

- Coastal tide information and water quality data
- Regulatory requirements
- Meteorological and Oceanographic data

- Fish/Marine Habitat and Biology Requirements

9.7.2.1 Intake & Coarse Screens

BNC Plant Intake: The intake at the BNC Plant will be located along the bank of the Brownsville Navigation Channel. Seawater from the channel will enter the concrete intake basin through a weir. The primary purpose of the weir is to limit the TSS concentration of influent water, since there are notably higher concentrations of TSS in the navigation channel. From the intake basin, seawater will pass through coarse bar screens and be conveyed by a concrete channel to the fine screening and seawater intake pumping station.

Gulf Coast Intake: For the offshore intake at the Gulf Coast Plant, the capture of seawater and coarse screening will occur by means of an intake structure located at the extremity of the seawater intake pipeline. The intake structure will be constructed of pre-cast concrete, round or octagonal in shape and fitted with corrosion-resistant metal alloy bar screens. At the Gulf Coast Plant, two intake pipelines of approximately 1.5 miles in length will be used to extend out into the ocean. Each intake pipeline will be 80-inches in diameter to meet the total intake capacity. The gravity pipeline will be constructed of fiberglass because of the corrosive nature of the seawater environment.

Both intake structures will be designed for an intake velocity of 0.33 feet per second, which is below the Environmental Protection Agency (EPA) impingement requirements of 0.5 feet per second. Spacing between bar screens shall not exceed 6 inches, to prevent the entrance of large debris or marine animals. The design parameters for the screens are included in Table 9-6.

Table 9-6: Design Parameters for Seawater Intake Screens

PARAMETER	BNC PLANT	GULF COAST PLANT
Number of units	1 duty	2 duty
Type	Bar screen	Bar screen
Capacity per screen	45 MGD	58 MGD
Screen spacing	3-4 inches	3-4 inches
Screen Material	Copper-nickel metal alloy	Copper-nickel metal alloy

9.7.2.2 Onshore Seawater Screening & Pump Station

The fine screens and seawater intake pump station will be located onshore. The primary purpose of the fine screens is to capture smaller contaminants such as shell fragments, sea weed fragments, etc. and eliminate them from the SWRO Plant feed water. The design parameters for the fine screens and intake pump station are included in Tables 9-7 and 9-8.

Table 9-7: Design Parameters for Fine Screens

PARAMETER	BNC PLANT (20 MGD)	GULF COAST PLANT (80 MGD ULTIMATE)
Number of units	1 duty + 1 standby	3 duty + 1 standby
Type	Traveling band screen	Traveling band screen
Capacity per screen	45 MGD	58 MGD
Screen size, mm	3	3
Wetted Parts Material	Super Duplex Stainless Steel	Super Duplex Stainless Steel

The BNC Plant intake pump station will provide a total capacity of 45 MGD and will be composed of three (3) 22 MGD (15,277 gpm) vertical turbine pumps, two duty and one standby, with an assumed total dynamic head (TDH) of 50 feet. Assuming the pumps are spaced 10 feet apart, the overall dimensions of the intake pumping station would be about 60 feet long and 45 feet wide.

Similarly, the Gulf Coast Plant intake will be located onshore and will meet a total capacity of 175 MGD. The pump station will be composed of eight (8) 25 MGD (15,625 gpm) vertical turbine pumps, seven (7) duty, and one (1) standby, with an assumed TDH of 50 feet. Assuming the pumps are spaced 10 feet apart, the overall dimensions of the intake pump station would be about 60 feet long and 110 feet wide.

Table 9-8: Design Parameters for Seawater Intake Pump Station

PARAMETER	BNC PLANT		GULF COAST PLANT		
	10 MGD	20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
Pump Station Firm Capacity	50 MGD		175 MGD		
Number of units (additional)	1 duty, 1 standby	1 duty	2 duty, 1 standby	2 duty	3 duty
Pump Capacity	22 MGD		25 MGD		
Rated Head	25 psi		25 psi		
Material of construction	Super Duplex Stainless Steel		Super Duplex Stainless Steel		

9.7.3 SWRO Desalination Plant

The treatment process illustrated in Figure 9-6 is recommended for the Brownsville Navigation Channel seawater desalination plant based on raw water quality data, finished water quality goals and evaluations from previous pilot study.

Based on expected raw water quality from the Gulf of Mexico, the treatment process for the 80 MGD Gulf Coast seawater desalination will be significantly different from that of the BNC Plant. As illustrated in Figure 9-7, the recommended treatment process for the seawater desalination plant does not include additional pretreatment steps upstream of membrane filtration, nor does it require solids handling to further separate solids from the liquids.

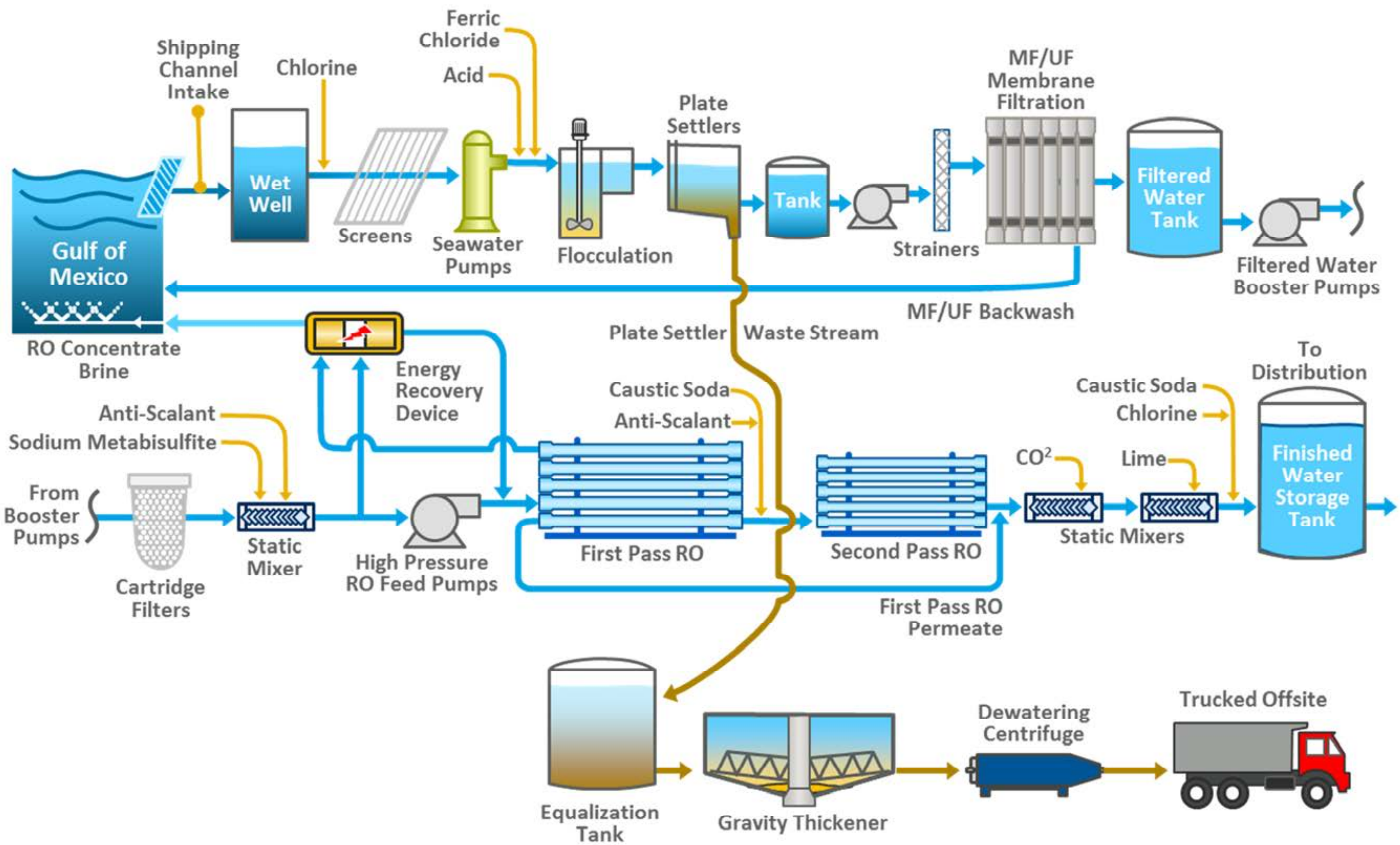


Figure 9-6: BNC Plant Desalination Plant Process Schematic

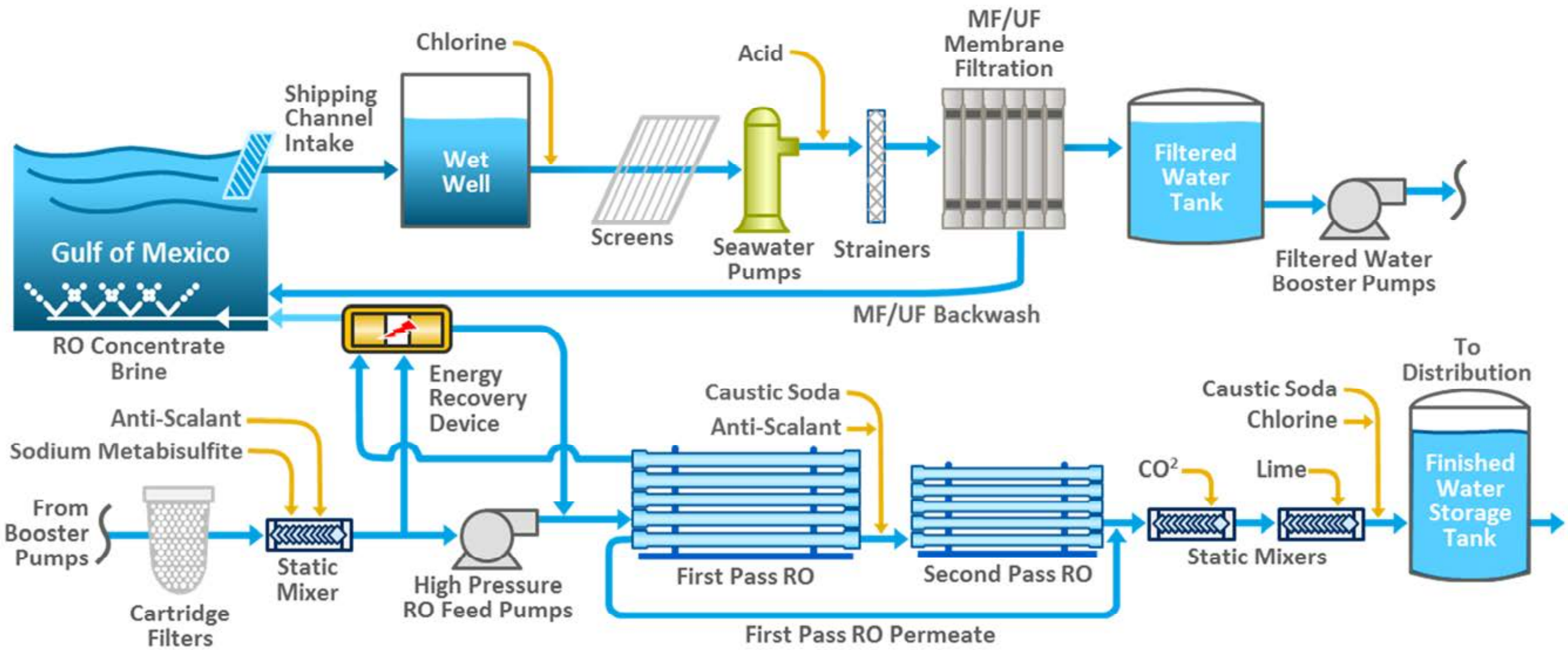


Figure 9-7: Gulf Coast Desalination Plant Process Schematic

9.7.3.1 Biofouling Control

Control of biological growth within the entire SWRO facility is critical. The most common practice is to shock chlorinate raw water periodically, which is recommended for this facility. The duration and frequency of chlorination are dependent on the site specific conditions (nutrients in the water, temperature, etc.). Upstream of the RO system, the RO feed water will be dechlorinated with sodium bisulfite, since the polyamide RO membranes are not tolerant to chlorine. In recent years, other oxidants such as chlorine dioxide have been tested as alternative chemicals for biofouling control. Use of non-oxidizing biocides such as DBNPA is also being considered, although its continuous use while system is online has not been approved. At this stage of design, shock chlorination is considered as shown in Table 9-9.

Table 9-9: Design Parameters for Biofouling Control

PARAMETER	VALUE
Biofouling Control	
Chemical	Sodium hypochlorite
Dosage	5-10 mg/L
Duration	5 hours
Frequency	Once/week

9.7.3.2 Pretreatment System

The pretreatment systems used in seawater desalination vary considerably depending on the raw water quality. The extent of pretreatment and the type of processes selected are highly dependent on the type and concentrations of various constituents.

In light of the water quality analysis and combined with available intake options, the pre-treatment processes to be considered include biofouling control, coagulation/flocculation, clarification, and filtration. The pretreatment system parameters and the various components of the pretreatment system are discussed below.

Coagulation, Flocculation & Clarification

Due to high suspended solids concentration in the Brownsville Navigation Channel, the BNC Plant will be equipped with clarification upstream of membrane filtration. Clarification process will consist of coagulation, flocculation and plate settlers for sedimentation. During periods of low TSS concentration, it may be possible to bypass the clarification process and send screened seawater directly to membrane filtration. Infrastructure to support the bypass will be included as part of the design. The following design parameters (Table 9-10) have been considered for the clarification process.

Table 9-10: Design Parameters for Clarification

PARAMETER	BNC PLANT
Coagulation	
Chemical	Ferric Chloride
Dosage rate (as FeCl3)	30 mg/L (based on the pilot study results. Needs to be verified)
Flocculation	
Number of parallel trains	2
Total Residence Time	30 min
Number of stages	3
Mixer type	Vertical mixers with Super Duplex wetted parts
Clarification	
Mechanism	Plate Settlers
Loading Rate	0.6 gpm/ft2, derated 80%

Membrane Filtration

Clarified water from the BNC Plant will be further treated through MF/UF membrane filtration. In the case of the Gulf Coast Plant, upstream pretreatment processes are not required and direct membrane filtration of seawater is included in the preliminary design. Design parameters for membrane filtration system are presented in the following table (Table 9-11).

Table 9-11: Design Parameters for Membrane Filtration (to be confirmed based on the MF/UF supplier selected)

PARAMETER	BNC PLANT		GULF COAST PLANT		
	PHASE I 10 MGD	PHASE II 20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
Number of units	5 + 1	5 + 1	10 + 2	10 + 2	20 + 2
Filtrate Capacity per train	4.11 MGD		4.11 MGD		
Operating Flux (gallon/foot-day)	25 (to be confirmed during detailed design based on pilot testing)		25 (to be confirmed during detailed design based on pilot testing)		
Strainer screen size	< 200 micron		< 200 micron		
Minimum recovery	92%		92%		
Minimum run time between maintenance washes	72 hours		72 hours		
Minimum run time between CIP cleanings	30 days		30 days		

Filtered Water Storage

The MF/UF filtrate will be stored in a filtered water tank. The backwash water for the MF/UF is supplied from this tank and also serves as the RO feed tank. The filtrate is pumped by a Low Pressure SWRO feed pump station through cartridge filters to the SWRO system. System components are presented in Table 9-12.

Table 9-12: Design Parameters for Filtered Water Tank & Low Pressure Pumps

PARAMETER	BNC PLANT		GULF COAST PLANT		
	PHASE I 10 MGD	PHASE II 20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
Tank Detention Time	20 – 30 min		20 – 30 min		
Tank Diameter	65 ft	65 ft	150 ft (Phase I and II)		150 ft
Number of Pumps	2 + 1	2 + 1	4 + 1		4 + 1
Rated Capacity	10 MGD	10 MGD	20 MGD		
Rated Head	55-70 psi	55-70 psi	55-70 psi		

Cartridge Filters

The MF/UF filtrate is pumped through cartridge filters to the SWRO plant. When using MF/UF membranes, cartridge filters may not be needed. However, given their low cost, and the added security they provide, these are included in the design. Cartridge filters will prevent contamination of the filtered water after MF/UF (such as contamination in tanks or corrosion and breakaway parts of appurtenances). Design parameters of the cartridge filters are provided in the following table (Table 9-13).

Table 9-13: Design Parameters for Cartridge Filters

PARAMETER	BNC PLANT		GULF COAST PLANT		
	PHASE I 10 MGD	PHASE II 20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
Influent Flow Rate	20.3 MGD	41.3 MGD	41.3 MGD	82.5 MGD	162.5 MGD
Number of cartridge filter vessels	5 vessels	5 vessels	10 vessels	10 vessels	20 vessels
Loading Rate (all units in service)	3 gpm/10” equivalent length		3 gpm/10” equivalent length		
Vessel material	Rubber-lined carbon steel, FRP or super duplex stainless steel		Rubber-lined carbon steel, FRP or super duplex stainless steel		
Cartridge Filter element pore size	5 micron		5 micron		

9.7.3.3 Reverse Osmosis System

As discussed in Section 9.6.3, the RO system uses spirally wound, semi-permeable RO membrane elements to remove dissolved solids from the filtered seawater. Typically, 6-8 elements are housed in a single pressure vessel and several pressure vessels make up the RO Rack. RO system components and design parameters are presented in Table 9-14 and Table 9-15.

Table 9-14: Design Parameters for 1st Pass SWRO System

PARAMETER	BNC PLANT		GULF COAST PLANT		
	PHASE I 10 MGD	PHASE II 20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
SWRO Racks					
Number of trains	3 + 1	3 + 1	4 + 1	4 + 1	8 + 2
Permeate Capacity per train	3.44 MGD		5.16 MGD		
Permeate Flux	<8 gfd		<8 gfd		
Element Size	8” diameter, 40” long		8” diameter, 40” long		
1st Pass RO Recovery	50%		50%		
Energy Recovery Device					
Type	Positive displacement		Positive Displacement		
ERS Booster Pump					
Rated Head	30-60 psi		30-60 psi		
Rated Capacity	3.43 MGD		5.14 MGD		
RO High Pressure Feed Pump					
Rated Head	870-1015 psi		870-1015 psi		
Rated Capacity	3.45 MGD		5.18 MGD		

Table 9-15: Design Parameters for 2nd Pass BWRO System

PARAMETER	BNC PLANT		GULF COAST PLANT		
	PHASE I 10 MGD	PHASE II 20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
BWRO Racks					
Number of trains	1 + 1	1 + 1	1 + 1	1 + 1	1 + 1
Permeate Capacity per train	2.86 MGD		5.7 MGD		
Permeate Flux	<20 gfd		<20 gfd		
Element Size	8” diameter, 40” long		8” diameter, 40” long		
2nd Pass RO Recovery	90%		90%		
2nd Pass RO Feed Pump					
Rated Capacity	2.86 MGD		5.7 MGD		
Rated Head	170 psi		170 psi		

9.7.3.4 Post-Treatment

Permeate stabilization is required because of the corrosive and aggressive nature of RO permeate. Post-treatment stabilization includes alkalinity and pH adjustment through addition of lime and CO_2. Caustic will be used for trimming pH as needed. After discharge from the RO membranes, the permeate will go through post treatment and conditioning to remove corrosivity and add alkalinity so that it will be compatible with water from other sources it may blend with in the distribution system and any installed infrastructure. During next phase of design, potential for use of calcite filters will be evaluated.

9.7.3.5 Disinfection

Following post-treatment stabilization, desalinated water will be disinfected using sodium hypochlorite. Desalinated water will run through a chlorine contact tank to provide sufficient contact time for 0.5 log inactivation of giardia. The following table (Table 9-16) summarizes the design conditions for chlorine disinfection.

Table 9-16: Design Parameters for Chlorine Contact Tank

PARAMETER	BNC PLANT	GULF COAST PLANT
Chlorine Contact Time	30 min	30 min
Chlorine residual	1.5 mg/L	1.5 mg/L

9.7.3.6 Brine Disposal

Waste flows from the SWRO plant will be composed primarily of high salinity RO concentrate. Other waste flows will include backwash waste from MF/UF systems, neutralized spent CIP solution from MF/UF and RO membranes. Waste flows will be collected and discharged to the Gulf of Mexico in a location that will not interfere with the raw water intake. During the next phase of design, the permitting process would need to be imitated to obtain permit for disposal of MF/UF backwash waste (Gulf Coast Plant) with RO concentrate. Since the backwash wastewater essentially consists of same constituents as in seawater, but at approximately 1.8 times the concentration in the seawater, it is not expected to cause any environmental impacts. Brine disposal will consist of a concentrate discharge pump station and a buried/submerged pipeline to convey the brine out in the ocean. Diffuser nozzles will be installed along the end of the submerged pipeline to disperse the brine and reduce potential environmental impacts to marine life around the outfall discharge. Treatment and handling of waste flows produced by the clarification process are described in the following section. The table below (Table 9-17) details the design parameters for Brine Disposal at both plants.

Table 9-17: Design Parameters for Brine Disposal

PARAMETER	BNC PLANT		GULF COAST PLANT (80 MGD ULTIMATE)
	10 MGD	20 MGD	
Brine Discharge Pipeline			
Diameter	42-inch		60-inch
Approximate Length	74,000 feet		8,000 feet
Material	HDPE		HDPE
Brine Discharge Pumps			
Number of units (additional)	1 duty, 1 standby	1 duty	Not required (discharged by gravity)
Capacity (additional)	10.4 MGD	10.4 MGD	N/A
Material of construction	Super duplex stainless steel		N/A

9.7.4 Solids Handling Facility

Waste flows from the clarification process at the BNC Plant will need to be further treated through a solids handling system. The average solids production is projected to be approximately 6,000 kg/day with a peak load of 12,900 kg/day based on the use of the 95th percentile statistical evaluation of the raw water turbidity data. Solids management facilities were sized to handle the peak load production. The proposed solids handling system consists of flow equalization, gravity thickening, and dewatering.

9.7.4.1 Gravity Thickening

Upstream of the gravity thickener, waste flows from the clarification process will be conveyed to an equalization tank sized for 10,000 gal. Sizing of the equalization tank is based on a preliminary estimate and further detailed analysis is needed during future design phases to consider site specific operating conditions. Waste flows received by the gravity thickener will vary in consistency, but were assumed to average 0.5% total solids. Gravity thickening will consist of a circular sedimentation tank 95 ft. in diameter. The thickened sludge flow, which is expected to contain approximately 2% total solids, will be conveyed to the dewatering area for further treatment. Overflow from the gravity thickener will be combined with RO concentrate and discharged through the outfall. A polymer conditioning system will be required to optimize thickening performance.

9.7.4.2 Dewatering

Mechanical dewatering through a centrifuge has been considered for the solids handling system design at the BNC Plant. Thickened sludge from the gravity thickener will be conveyed to a centrifuge for dewatering. Two centrifuge units will be required and will operate continuously throughout the work week (Monday to Friday) as 1 duty, 1 standby. Dewatered sludge will be trucked off site to a landfill.

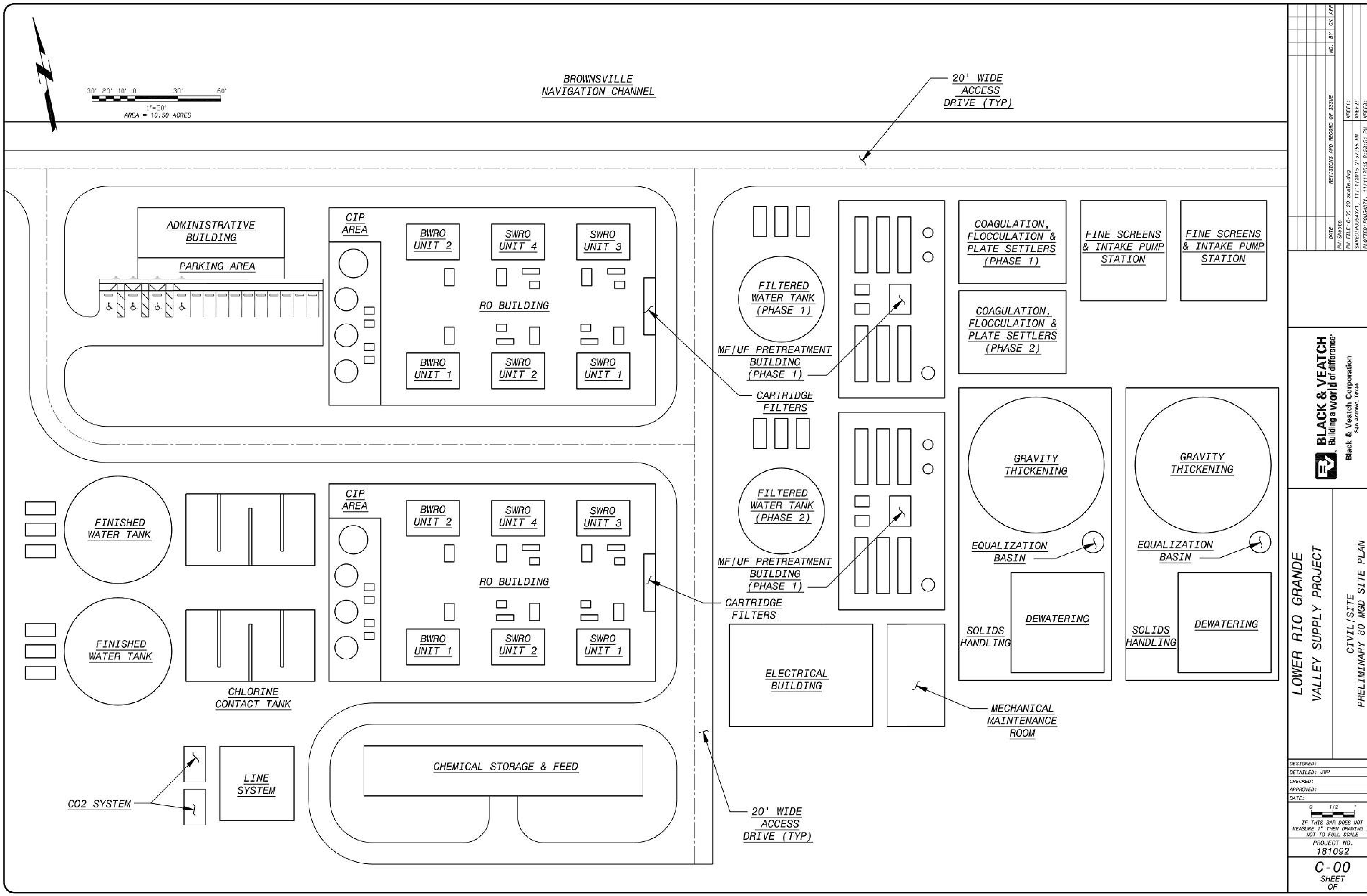
9.7.5 Treated Water Storage & Conveyance

Finished water storage will be provided for system operation purposes. Initially, a 1 MG clearwell would be built for storage at the BNC plant. At the Gulf Coast Plant, a clearwell would be constructed during Phase 1 for total storage capacity of 8 MG to sustain the ultimate plant capacity.

A high service pump station located at the SWRO plants would pump finished water to the regional conveyance line. Appropriate conveyance infrastructure is detailed in Chapter 7.

9.7.6 Site Layouts

The proposed site layouts for the 20 MGD BNC SWRO Plant and the future 80 MGD Gulf Coast SWRO Plant are shown in Figure 9-8 and Figure 9-9, respectively.


BROWNSVILLE NAVIGATION CHANNEL
1"=30'
AREA = 10.50 ACRES
20' WIDE ACCESS DRIVE (TYP)
ADMINISTRATIVE BUILDING
PARKING AREA
CIP AREA
BWRO UNIT 2
SWRO UNIT 4
SWRO UNIT 3
RO BUILDING
BWRO UNIT 1
SWRO UNIT 2
SWRO UNIT 1
FILTERED WATER TANK (PHASE 1)
MF/UF PRETREATMENT BUILDING (PHASE 1)
CARTRIDGE FILTERS
COAGULATION, FLOCCULATION & PLATE SETTLERS (PHASE 1)
COAGULATION, FLOCCULATION & PLATE SETTLERS (PHASE 2)
FINE SCREENS & INTAKE PUMP STATION
FINE SCREENS & INTAKE PUMP STATION
GRAVITY THICKENING
GRAVITY THICKENING
EQUALIZATION BASIN
EQUALIZATION BASIN
SOLIDS HANDLING
DEWATERING
SOLIDS HANDLING
DEWATERING
FINISHED WATER TANK
FINISHED WATER TANK
CHLORINE CONTACT TANK
CIP AREA
BWRO UNIT 2
SWRO UNIT 4
SWRO UNIT 3
RO BUILDING
BWRO UNIT 1
SWRO UNIT 2
SWRO UNIT 1
FILTERED WATER TANK (PHASE 2)
MF/UF PRETREATMENT BUILDING (PHASE 1)
CARTRIDGE FILTERS
ELECTRICAL BUILDING
MECHANICAL MAINTENANCE ROOM
CHEMICAL STORAGE & FEED
LINE SYSTEM
CO2 SYSTEM
20' WIDE ACCESS DRIVE (TYP)
REVISIONS AND RECORD OF ISSUE
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LOWER RIO GRANDE VALLEY SUPPLY PROJECT
CIVIL/SITE
PRELIMINARY 80 MGD SITE PLAN
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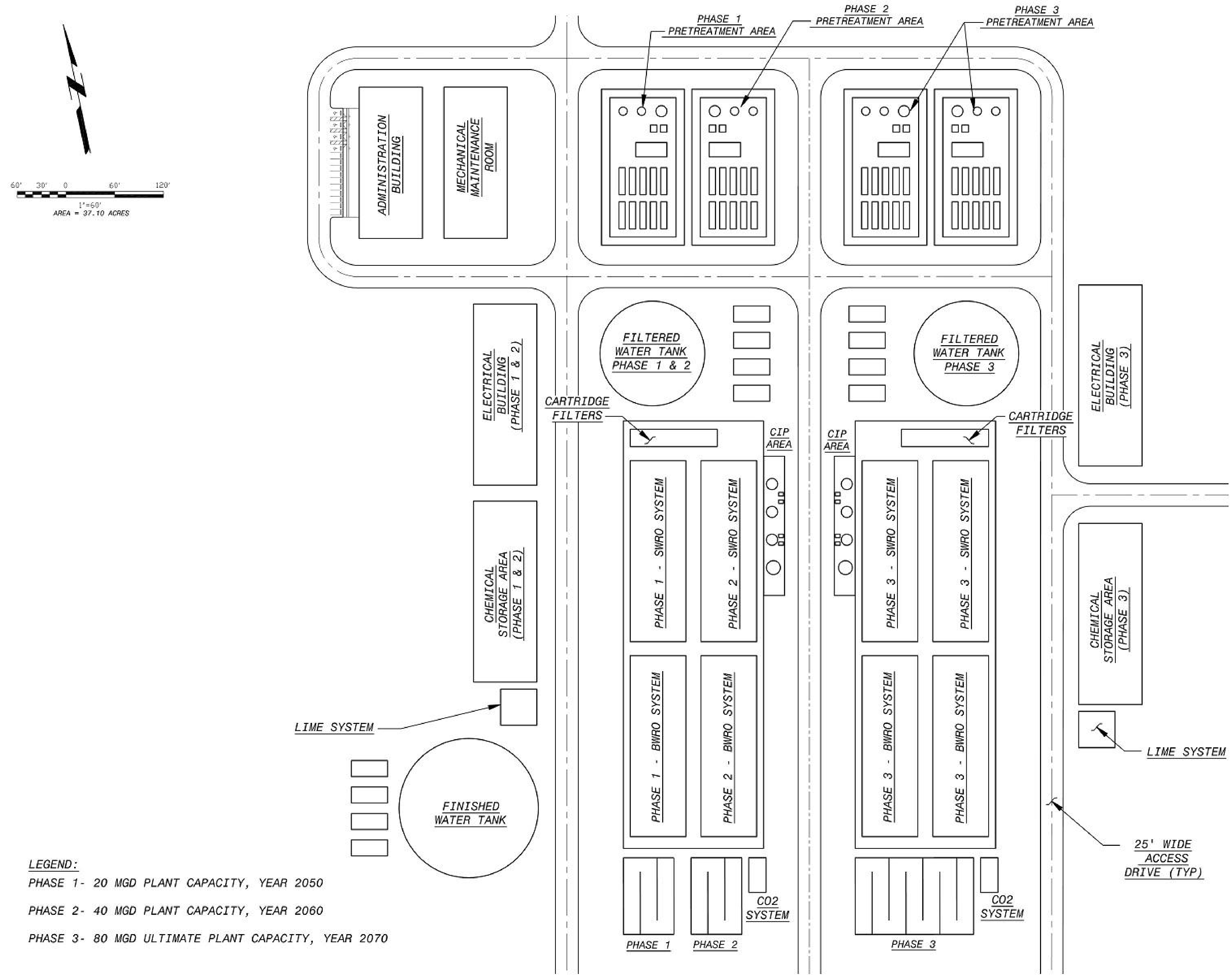

LEGEND:

PHASE 1- 20 MGD PLANT CAPACITY, YEAR 2050

PHASE 2- 40 MGD PLANT CAPACITY, YEAR 2060

PHASE 3- 80 MGD ULTIMATE PLANT CAPACITY, YEAR 2070

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USER: POU54371 | DWG VER: 1000 | XREF4:

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LOWER RIO GRANDE
VALLEY SUPPLY PROJECT

CIVIL/SITE
PRELIMINARY 80 MGD SITE PLAN

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9.8 COSTS

An engineer's opinion of probable cost (EOPC) was developed for each phase of the proposed projects. Table 9-18 presents the EOPC for each construction period. Below the summary table is a description of each cost category. The EOPC relied on previous projects of similar size and treatment technologies, along with local information obtained from project partners. Summarized in Table 9-19 are the EOPC for annual operations of the projects at their increasing total capacities.

Table 9-18: Detailed Cost Summary for BNC and Gulf Coast SWRO Plant Capital Expenses

FACILITY / PROCESS SYSTEM	COST FOR BNC PLANT ($M USD)		COST FOR GULF COAST PLANT ($M USD)		
	10 MGD	20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
Land Acquisition	0.09	-	0.72	-	-
Intake and Pipeline	3.98	3.98	15.66	1.95	5.19
Pretreatment	15.40	15.40	21.31	21.31	39.55
RO Treatment	32.31	32.31	60.48	59.38	111.23
Post Treatment	3.83	3.83	6.69	0.38	8.31
Solids Handling System	3.50	3.50	-	-	-
Administration Building	0.88	-	1.76	-	-
Mechanical	8.39	8.25	13.54	12.16	23.86
Elect & Instrumentation	6.71	6.60	10.83	9.73	19.09
Sitework	2.80	2.75	4.51	4.05	7.95
General requirements	2.80	2.75	4.51	4.05	7.95
TOTAL DIRECT COSTS	80.68	81.26	140.02	113.02	223.13
Contingency	20.17	20.31	35.01	28.25	55.78
Contractor Overhead and Profit	8.07	7.94	14.00	11.30	22.31
Electrical Infrastructure Allowance	4.71	-	18.97	-	-
ELA	12.10	12.19	21.00	16.95	33.47
TOTAL INDIRECT COSTS	45.04	39.69	88.98	56.51	111.57
TOTAL CAPITAL COST	**125.72**	**119.07**	**229.00**	**169.53**	**334.70**

Land Acquisition	The cost includes purchase of land for plant at $18,000 per acre on the island and $9,000 per acre on the channel, where cost of land was ascertained from local stakeholders. It is assumed the land required for full plant capacity is acquired in the initial phase of the project.
Intake and Pipeline	Raw water intake, pipeline, pumping station, coarse screens
Pretreatment	Coagulant, fine screens, plate settler, MF/UF, pumps, pipelines and backwash lines.

RO Treatment	RO feed tank and pumping station, cartridge filters, antiscalant, RO process equipment, Energy Recovery System.
Post Treatment	Lime and CO2 systems, chemical feed systems, chlorine contact basin, finished water reservoir
Mechanical	Major yard piping and mechanical appurtenances not included above.
Electrical, Instrumentation, and Control	All transformers, motor control equipment, electrical and instrumentation duct banks, SCADA programming, and instruments not provided as part of mechanical equipment are included in this category.
Sitework	This includes site clearing and grading
General Requirements	This includes the contractor's general requirements such as project management and commissioning, temporary facilities, etc.
Contingency	A contingency of 25 percent is appropriate given the information available and project requirements.
Construction Overhead and Profit	Construction overhead and profit of 10% of the Direct Capital Costs is assumed.
Electrical Infrastructure Allowance	An allowance to cover costs for needed power distribution costs to the plants. This allowance is calculated utilizing a ratio of horse-power to impact fees assessed for previous large water supply projects.
Engineering, Legal, and Administrative	A value of 15 percent was assumed for detailed design, construction phase services and legal and administrative activities that will be required to execute the project.

Table 9-19: Detailed Cost Summary for BNC and Gulf Coast SWRO Plant Operating Expenses

ANNUAL OPERATING COST SUMMARY ($)	BNC PLANT		GULF COAST PLANT		
	10 MGD	20 MGD	PHASE I (20 MGD)	PHASE II (40 MGD)	PHASE III (80 MGD)
Electricity	1,729,000	3,432,000	3,405,000	6,707,000	13,145,000
Chemicals	1,088,000	2,177,000	1,776,000	3,552,000	7,104,000
Membrane Replacement	575,000	1,150,000	1,150,000	2,300,000	4,600,000
Maintenance	807,000	1,601,000	1,400,000	2,530,000	4,762,000
Labor	1,657,000	1,788,000	1,789,000	2,303,000	3,043,000
Sludge Trucking	314,000	628,000			
Miscellaneous	172,000	297,000	345,000	690,000	1,380,000
Contingency	967,000	1,716,000	1,933,000	3,867,000	7,732,000
Cumulative Total Annual O&M Costs ($/year)	**7,309,000**	**12,789,000**	**11,798,000**	**21,949,000**	**41,766,000**

The following are a list of considerations for the development of the annual operational expenses.

Electricity	Assumes cost of electricity to be $0.05/kWh. Considers energy consumption of SWRO Plant equipment for raw water pumps, MF/UF system, RO feed pumps and ERS booster pumps, 2nd pass RO feed pumps, brine discharge pumps, as well as miscellaneous use.
Chemicals	Cost per pound varies by chemical and total cost considers all chemicals used as part of the SWRO process (ferric chloride, sulfuric acid, hypochlorite, antiscalant, sodium bisulfite, etc.
Membrane Replacement	Considers 15% of 1st Pass RO membrane elements and 5% of 2nd Pass RO membrane elements being replaced annually. Also considers replacement of cartridge filter elements.
Maintenance Costs	Considers cost to maintain plant equipment such as pumps, valves, instruments, etc., excluding membrane replacement
Labor Costs	Accounts for various levels of plant staffing, including management, senior operators, plant operators and technicians, maintenance staff and administrative staff.
Sludge Trucking Costs	Considers trucking cost of $125 per dry ton

9.9 SUMMARY

Seawater desalination is a viable option for regional water supply to the LRGV. Providing desalinated water to the major municipal users in Cameron County will allow other water sources to be available for the rest of the region. The largest drawback to seawater desalination is the operational cost of treating the water and conveying it across the counties. However, as demands increase, available resources are depleted, and technology improves, seawater desalination will become more economical for the region.

If a seawater desalination system is to be implemented in the LRGV the next steps would include additional raw water quality testing, SWRO system design, discussions with TCEQ and other regulatory agencies, and continued pilot testing with the aim of receiving the necessary TCEQ permit.

Appendix A. Crop Sensitivity to Boron

VERY SENSITIVE (<0.5 MG/L)	
Lemon	*Citrus limon*
Blackberry	*Rubus spp.*
Sensitive (0.5 – 0.75 mg/l)	
Avocado	*Persea americana*
Grapefruit	*Citrus X paradisi*
Orange	*Citrus sinensis*
Apricot	*Prunus armeniaca*
Peach	*Prunus persica*
Cherry	*Prunus avium*
Plum	*Prunus domestica*
Persimmon	*Diospyros kaki*
Fig, kadota	*Ficus carica*
Grape	*Vitis vinifera*
Walnut	*Juglans regia*
Pecan	*Carya illinoiensis*
Cowpea	*Vigna unguiculata*
Onion	*Allium cepa*
Sensitive (0.75 – 1.0 mg/l)	
Garlic	*Allium sativum*
Sweet potato	*Ipomoea batatas*
Wheat	*Triticum eastivum*
Barley	*Hordeum vulgare*
Sunflower	*Helianthus annuus*
Bean, mung	*Vigna radiata*
Sesame	*Sesamum indicum*
Lupine	*Lupinus hartwegii*
Strawberry	*Fragaria spp.*
Artichoke, Jerusalem	*Helianthus tuberosus*
Bean, kidney	*Phaseolus vulgaris*
Bean, lima	*Phaseolus lunatus*
Groundnut/Peanut	*Arachis hypogaea*

MODERATELY SENSITIVE (1.0 – 2.0 MG/L)	
Pepper, red	*Capsicum annuum*
Pea	*Pisum sativa*
Carrot	*Daucus carota*
Radish	*Raphanus sativus*
Potato	*Solanum tuberosum*
Cucumber	*Cucumis sativus*
Moderately Tolerant (2.0 – 4.0 mg/l)	
Lettuce	*Lactuca sativa*
Cabbage	*Brassica oleracea capitata*
Celery	*Apium graveolens*
Turnip	*Brassica rapa*
Bluegrass, Kentucky	*Poa pratensis*
Oats	*Avena sativa*
Maize	*Zea mays*
Artichoke	*Cynara scolymus*
Tobacco	*Nicotiana tabacum*
Mustard	*Brassica juncea*
Clover, sweet	*Melilotus indica*
Squash	*Cucurbita pepo*
Muskmelon	*Cucumis melo*
Tolerant (4.0 – 6.0 mg/l)	
Sorghum	*Sorghum bicolor*
Tomato	*Lycopersicon lycopersicum*
Alfalfa	*Medicago sativa*
Vetch, purple	*Vicia benghalensis*
Parsley	*Petroselinum crispum*
Beet, red	*Beta vulgaris*
Sugarbeet	*Beta vulgaris*
Very Tolerant (6.0 – 15.0 mg/l)	
Cotton	*Gossypium hirsutum*
Asparagus	*Asparagus officinalis*

From: Food and Agricultural Organization of the United Nations (http://www.fao.org/docrep/003/T0234E/T0234E05.htm#ch4.1.3)

CHAPTER 10 - SURFACE WATER INFRASTRUCTURE

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

4 DECEMBER 2015

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10.0 Surface Water Infrastructure

10.1 PURPOSE

This chapter describes the design basis for a regional surface water treatment plant (SWTP) and expected capital and operating costs for the facility. Surface water availability and raw water sources are evaluated in other chapters of the report. Source water for the SWTP will initially be from the Rio Grande and in future phases advanced treated wastewater effluent will be blended with raw surface water prior to treatment in this facility.

10.2 RAW WATER QUALITY

Raw water data was analyzed from Brownsville Public Utilities Board Rio Grande and Resaca sources and the McAllen Southwest WTP. The resultant range of water quality is summarized in Table 10-1.

Table 10-1 Source Water Data Characterized for a Regional SWTP

PARAMETER	UNITS	MIN.	MAX.
Arsenic	mg/L	0.0037	0.0096
Bromide	mg/L	0.159	0.566
Calcium	mg/L	84	123
Chromium	mg/L	0.001	0.0018
Copper	mg/L	0.0028	0.005
Lead	mg/L	0.001	0.0017
Magnesium	mg/L	25.6	40.8
Manganese	mg/L	0.078	0.224
Sodium	mg/L	128	232
Total Hardness Ca/Mg Eq. $CaCO_3$	mg/L	326	475
Total Iron	mg/L	0.59	1.44
Fluoride	mg/L	0.50	0.69
Nitrate-Nitrogen Total	mg/L	0.09	0.30
Chloride	mg/L	162	341
Sulfate	mg/L	253	492
Total Alkalinity (as $CaCO_3$)	mg/L	126	191
Total Suspended Solids	mg/L	34	216
Total Dissolved Solids	mg/L	620	1,350
Lab pH	SU	8.0	8.4

10.2.1 Regional SWTP Infrastructure

A regional SWTP could provide a large portion of the future demands in LRGV. This Regional SWTP could utilize both purchased and converted water rights from the Rio Grande River along with wastewater effluent that had been sufficiently treated prior to being blended in a proposed raw water reservoir upstream of the proposed Regional SWTP. This Section will outline the necessary infrastructure based on the quantity and quality of the raw water sources and the quantity and quality targets of the regional system.

10.2.1.1 Drinking Water Quality Regulations

10.2.1.1.1 Existing Drinking Water Regulations

The first step in establishing the treated water quality objectives is the determination of the applicable water quality regulations. Table 10-2 provides a regulatory framework for designing the Regional SWTP.

Table 10-2 Drinking Water Regulations

REGULATION	PERFORMANCE REQUIREMENT
CURRENT REGULATIONS	
Total Coliform Rule (TCR)	• TCR Revisions in 2008 • Biologically stable finished water • Revised TCR published February 2013. Maintains MCL for *E.coli* and eliminates MCL for total coliforms
Surface Water Treatment Rule (SWTR)	• Credit for 3-log Giardia reduction • 4-Log virus reduction • CT credit for disinfection
Lead and Copper Rule (LCR)	• Optimized corrosion control
Interim Enhanced Surface Water Treatment Rule (IESWTR)	• 2 Log crypto filtration credit • Turbidity < 0.3 NTU CFE
Stage 1 Disinfectant/Disinfection By-product Rule (D/DBPR)	• TOC removal by enhanced coagulation • THMs < 80 ug/L running annual average (RAA) • HAA5 < 60 ug/L RAA • Bromate < 10 ug/L RAA • Chlorite < 1 mg/L
Filter Backwash Rule (FBR)	• Recycle streams to front of plant
Long-term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)	• Bin classification for additional *Cryptosporidium* treatment requirements • Toolbox of treatment processes to meet bin requirements

REGULATION	PERFORMANCE REQUIREMENT
CURRENT REGULATIONS	
Stage 2 DBPR	• THMs < 80 ug/L locational running annual average (LRAA) • HAAs < 60 ug/L LRAA
Arsenic Rule	• MCL of Arsenic < 10 ug/L
Radionuclides Rule	• MCLs for radionuclides in drinking water
Secondary Standards	• Manganese < 50 ug/L • TDS < 500 mg/L (Federal) < 1,000 mg/L (State) • Chloride < 250 mg/L (Federal) < 300 mg/L (State)

Potential future regulations are summarized in Table 10-3 below.

Table 10-3 Pending/Potential Regulations

REGULATION	PERFORMANCE REQUIREMENT
Long-term Revisions to Lead and Copper Rule	• Intended to publish in 2016 • Sampling, measurements and public education modifications • Definition of "Lead-free" changes January 2014 (applies to all meters and parts *installed* after January 4, 2014).
Regulation in Progress for Unregulated Contaminant	• Chromium-6 • Nitrosamines (NDMA) • Chlorinated VOCs
Review of Fluoride MCL	• Potential fluoride MCL of 0.7 mg/L
Revised Total Coliform Rule (April 1, 2016)	• Removes the MCLG and the MCL for total coliform • Goal is to determine cause of coliform presence through Level 1 and 2 assessments.
6-Year Review	• Primary MCLs reviewed every 6 years. Round 3 to be completed in 2016.

10.2.1.2 Water Treatment Objectives

The treatment goals for the Regional SWTP are influenced by a combination of federal and state drinking water quality regulations, treatability of the raw water, and the basic objective to provide a safe, aesthetically pleasing, and economical water supply. The recommended water quality objectives for the Regional SWTP are as follows and are in line with Texas Commission on Environmental Quality (TCEQ).

- Meet all primary MCL requirements.
- Turbidity – less than 0.1 NTU 95% of the time
- Bromate – Less than 10 ppb
- *Giardia* inactivation (through disinfection) – 1.0 Log (90.0%)
- *Giardia* removal (through treatment and turbidity removal) – 2.5 Logs (99.7%)
- Virus inactivation (through disinfection) – 4.0 Logs (99.0%)
- Virus removal (through treatment and turbidity removal - 2.0 Logs (99.0%)
- TOC – reduction through enhanced coagulation to meet regulatory requirements and produce a biologically stable water
- TTHMs – less than 80 ppb
- HAA5 – less than 60 ppb
- TDS – less than 1000 ppm
- Manganese – less than 25 ppb
- Threshold odor number less than 3.
- Corrosion indices to provide stable water quality:
  - Langelier Saturation Index of 0.2 to 0.8;
  - Calcium Carbonate Precipitation Potential of 4.0 to 10 mg/L as $CaCO_3$
  - Chloride Sulfate Mass Ratio (CSMR) less than 2.0 or unchanged from existing ratio

Pathogen inactivation goals are more rigorous than the minimum required by TCEQ. Inactivation goals achieve a performance ratio of 2.0. The corrosion index values are typical and a detailed corrosion evaluation would be required to confirm water compatibility in the distribution system. For this evaluation, chemicals and their dosages were calculated to meet the corrosion index values stated in this section.

10.2.1.3 Raw Water Collection

10.2.1.3.1 Intake Structure and Pump Station

A total capacity 80 MGD raw water intake structure will be provided on the bank of the Rio Grande River. This intake will be composed of a concrete structure integrated into the river bank with either a vertical or inclined inlet with plate screens or coarse/fine self-cleaning trash racks, a conventional rectangular pump station designed per Hydraulic Institute (HI) Standards, and vertical column pumps with vertical induction pumps.

The entire raw water intake structure will be constructed to meet the total capacity of 80 MGD, but the actual pump station would only provide 20 MGD during Phase I. The total capacity of the pump station will be composed of four (4) 20 MGD (13,888 gpm) pumps and two (2) 10 MGD (6,944 gpm) pumps each rated for an assumed Total Dynamic Head (TDH) of 50 feet. For each phase a redundant pump is being provided except during Phase IV, during this phase only one 10 MGD pump will be provided, so the pump station will either be able to put out 80 MGD or 60 MGD if the smaller 10 MGD pump is out of commission. A summary of the phasing and capacities are provided in Table 10-4 below.

Table 10-4 Phasing and Capacity Breakdown

Phase	Capacity (MGD)	Pumps
I	20	20 MGD (1 duty, 1 standby)
II	40	20 MGD (2 duty, 1 standby)
III	60	20 MGD (3 duty, 1 standby)
IV	70	20 MGD (3 duty, 1 standby); 10 MGD (1 duty, 0 standby)
V	80	20 MGD (3 duty, 1 standby); 10 MGD (1 duty, 1 standby)

Assuming the pumps are spaced 10 feet apart, the overall dimensions of the pump station would be about 90 feet long by 60 feet wide. Each pump intake would be about 10 feet wide by 15 feet long. Without an adequate river profile the depth of the Rio Grande River was assumed to be 10-15 feet deep. To be conservative the pumps would have a shaft length of 20 feet.

Further analysis of the Rio Grande River and the raw water intake will be required in order to determine the best application to be utilized for this project. The information provided above is based on the most typical and reliable applications utilized in the United States. Detailed information would need to be collected on the following in order to properly design the intake structure and pump station:

- River data: flows, levels, velocities, silt/sedimentation, and profiles
- Orientation and location of structure relative to the body of water
- Requirement for divergent walls in the body of water
- Regulatory requirements
- Environmental factors: floodplain delineation, vegetation load, species of interest, wetland delineation and turbidity

10.2.1.3.2 Raw Water Pipeline

The raw water will be conveyed from the intake pump station via a 54-inch diameter steel pipeline approximately 6,000 linear feet to the proposed new raw water reservoirs. A 54-inch diameter pipeline was chosen because this diameter pipeline will be able to handle a range of flows from 21 MGD to 82 MGD, while maintaining a minimum velocity of 2 feet per second and a maximum velocity of 8 feet per second. Headloss due in the pipe is expected to be less than 25 feet. Multiple assumptions were made in regards to the pipeline. Further detail will be needed on the following items to design the pipeline.

- Soil conditions
- Location and routing
- Permitting and environmental requirements
- Boring locations, length, and type

10.2.1.3.3 Raw Water Storage Reservoir

Raw water storage will be provided near the SWTP site to "balance" the variable daily plant production and available raw water supply. Terminal storage would also provide short-term emergency storage in the event of interruptions to the raw water conveyance systems during high demand periods. The recommended volume of terminal raw water storage is 2 days production capacity and additional reservoirs would be constructed with each expansion.

10.2.2 Summary of Available Treatment Technologies

Various treatment alternatives were identified for study and consideration in meeting the treatment goals for this project. Table 10-5 provides a summary of the advantages and disadvantages of the various technologies for the Regional SWTP.

Table 10-5 Treatment Technology Summary

Technology	Advantages	Disadvantages
Clarification		
Ballasted Clarification	• Smallest footprint • Quick process response	• High dependence on polymer • Sole source equipment • Higher operating cost than conventional treatment • More operator intensive
Conventional Sedimentation	• Well understood • Simple operation • Familiar to regional staff	• Largest footprint
Dissolved Air Flotation	• Small footprint • Effective for TSS that are not easily settleable	• Highest operating cost • Less effective for high turbidity source waters
Inclined Plates/Tubes	• Smaller footprint • Similar operation to conventional sedimentation	• Solids collection equipment is under plates so access is limited and basin is deeper • Routine cleaning of tubes
Solids Contact Clarifiers	• High solids concentration in blow down • Smaller footprint	• Startup period long • Does not respond well to changes in flow or water quality
TOC Reduction		
Enhanced Coagulation	• Familiar to regional staff • Effective for TOC removal	• Increased solids quantities • Reduced filter run times
Enhanced Softening	• Less effective for TOC removal • Removes hardness	• Increased solids quantities • Removes alkalinity
Granular Activated Carbon (GAC)	• Can provide filtration and TOC removal	• Recurring cost to replace spent GAC results in higher O&M cost
Magnetic Ion Exchange (MIEX)	• Reduces quantity of coagulant	• Resin regeneration brine stream requires disposal • Sole source equipment

Technology	Advantages	Disadvantages
Filtration		
Granular Media Filtration	• Familiar to regional staff • Low operating cost • Allows biologically active filtration	• Turbidity can be higher than 0.1 NTU
Membrane Filtration (microfiltration/ultrafiltration)	• Turbidity always less than 0.1 NTU if membrane integrity is intact • Generally provides higher overall water quality compared to conventional treatment • Can provide direct physical barrier to pathogens • May allow reduced coagulant usage compared to conventional treatment	• Higher replacement cost relative to conventional filtration • Higher operating cost • Does not allow biologically active filtration • Membrane integrity issues with polymer membranes can increase O&M cost • Cleaning chemicals often cannot be recycled, if no sewer, off-hauling will increase O&M cost
Desalination		
Dilution	• Reduces levels of hardness, sodium, and chloride • Low capital and operating cost	• Limited by availability and water quality of diluting water source
Distillation	• Removes hardness, TOC, sodium and chloride	• High capital and operating cost • Not cost competitive for brackish water
Electrodialysis Reversal	• Recovery not impacted by silica removal	• Sole source equipment • EDR generally not used in large scale facilities
Nanofiltration	• Lower operating pressure than reverse osmosis • Removes hardness and TOC	• Chloride and sodium removal is low
Reverse Osmosis	• Removes hardness, TOC, sodium and chloride	• Higher operating pressure and cost
Disinfection/Oxidation		
Chlorine/chloramines	• Persistent residual • Effective for iron oxidation • Inactivates *Giardia* and viruses	• Regulated disinfection byproducts (THMs and HAA5) • Corrosive and highly toxic if in gaseous form
Chlorine Dioxide	• Effective oxidant for iron and manganese • Inactivates *Giardia* and viruses	• Higher chemical cost than chlorine • More complicated delivery since generated onsite • Regulated disinfection byproduct (chlorite)

Technology	Advantages	Disadvantages
Ozone	• Powerful oxidant for iron, manganese, and taste and odor compounds • Inactivates *Giardia* and viruses	• Unstable residual • More complicated delivery since generated onsite • Regulated disinfection byproduct (bromate) • Energy intensive • Must be used in combination with chlorine or chloramines for residual
Ultraviolet Disinfection	• Effective for *Giardia* and *Cryptosporidium inactivation* • No regulated disinfection byproducts • Can be combined with hydrogen peroxide for advanced oxidation that is effective for taste and odor control	• Ineffective for viruses • No residual • T&O control requires much higher UV dosages than applied for *Giardia* and *Cryptosporidium* inactivation • Must be used in combination with chlorine or chloramines for residual • Energy intensive
Taste & Odor Control		
GAC Roughing Filters	• Can provide filtration and TOC removal	• Very expensive to replace spent GAC • Continuous operation regardless need to remove taste and odor compounds
Post-filter GAC Absorbers	• Operate as needed so GAC life is extended	• Additional filtration complex must be constructed downstream from filters
Powdered Activated Carbon	• Operate as needed	• Added solids • Feed system operation is challenging
Ozone	• Provides oxidation and disinfection as well • Most cost effective for long duration events • Effective for low level taste and odor events without dosage change	• Onsite generation • Disinfection byproduct (bromate)
UV and Peroxide	• Provides disinfection, as well as oxidation • Effective for periodic short duration taste and odor events	• UV reactors much larger than those required for disinfection • Must increase UV dosage over the dosage applied for disinfection and dose peroxide to deal with even low level taste and odor events

Technology	Advantages	Disadvantages
Manganese Control		
Manganese Absorbers	• Highly effective for removal of manganese	• Additional filtration complex must be constructed
Multimedia Filtration with Chlorine	• Can provide filtration and manganese removal	• Does not allow biological filtration
Enhanced Biofiltration	• Can provide biological filtration and manganese removal	• Will not provide as strong, consistent, and controllable a barrier as other control methods
Note: Advantages and disadvantages are relative to conventional treatment		

10.2.3 Water Treatment Process

The base design for the Regional SWTP treatment process will include conventional surface water treatment plant consisting of rapid mixing, flocculation, sedimentation, intermediate ozonation, biologically active filters, pH adjustment, addition of disinfectant residual (chlorine and ammonia), and finished water storage.

This treatment process has proven to be reliable on similar source waters. The unit processes were selected to provide treatment for suspended solids, total organic carbon removal (TOC), iron and manganese oxidation, disinfection, and removal of taste-and-odor causing compounds. Ozone was also included to deal with emerging contaminants such as algal toxins, endocrine disrupting compounds (EDCs), and pharmaceutical and personal care products (PPCPs). The specific purpose of each unit process is listed in Table 10-6.

Table 10-6 Purpose of Each Treatment Process

TREATMENT TECHNOLOGY	PURPOSE
Aluminum sulfate & polymer (coagulation)	Particle and TOC removal (enhanced coagulation)
Conventional sedimentation	Particle removal
Intermediate ozonation	Oxidation of iron and manganese, taste- and odor control, primary disinfection, and to enhance biological filtration
Deep bed biological filtration	Particle removal (TOC removal with biological filtration enabled by chlorine-free settled and backwash water)
Ultraviolet (UV) disinfection (future)	Inactivation of *Cryptosporidium* and potential UV/peroxide enhanced oxidation
Caustic	Provide stable (non-corrosive) water
Clearwells	Storage
Chlorine	Provide residual disinfection
Ammonia	Form chloramines

Conventional sedimentation was selected as it will provide reliable solids removal and is used at other SWTPs in the area. Large rectangular basins were assumed that included circular solids removal equipment. For master planning level studies, it is desirable to include larger processes to ensure adequate footprint is available for the treatment plant. Once the treatment site and source water are known, the type of sedimentation process can be revisited and a high-rate process utilized. Coagulation is assumed to be performed with aluminum sulfate and polymer. Aluminum sulfate was selected as the coagulant as the addition of sulfate lowers the rate of distribution system piping corrosion.

Intermediate ozonation was selected to provide oxidation of organic and inorganic compounds and lower the concentration of chlorination disinfection byproducts (TTHMs and HAA5s). Ozone is generally added at the head of the plant or between sedimentation and filtration. Ozone applied downstream from sedimentation has lower ozone dosages because a large quantity of solids and organic compounds that would exert ozone demand has been removed during sedimentation.

Previous work indicates that bromate formation in the water can be quite high and may prevent the use of ozone. Sources with elevated bromide have been treated successfully with ozone but require bromate mitigation strategies that include:

- Ozone Dosage Control. Limiting the dosage and the residual is the single most effective control method. Overdosing ozone results in more bromate formation and careful operations to meet process goals and minimize operating cost are successful at many locations.
- Source Water Blending. Blending sources with high and low levels of bromide is used in California to limit the formation of bromate. The reclaimed water will contain much lower levels of bromide and will reduce the plant influent bromide concentration.
- pH Depression. Applying ozone at a pH less than 7.0 is highly effective (30% or more reduction). The dominant oxidized bromide species at pH 7.0 is hypobromous acid, which does not react with ozone to form bromate.
- Ammonia Process. Applying 0.3 to 0.5 mg/L-N ammonia upstream from ozonation to block the reaction of bromide to bromate.
- Chlorine + Ammonia Process. Applying a dosage of chlorine less that the amount required to meet demand and then applying 0.3 or less ammonia to block the reaction of bromide to bromate.

During conceptual design, the bromide concentration must be better characterized and the bromate formation potential and mitigation strategies evaluated to determine the feasibility of implementing ozone into the treatment process. Given the potential for high levels of bromide in the source water, a bromate mitigation strategy will likely be needed to implement ozonation.

It is advantageous to have ozone upstream from biological filtration to assist with the removal of assimilable organic carbon (AOC) formed as a result of oxidation of TOC during ozonation. If AOC is allowed to enter the distribution system, it can result in biofilm growth, loss of disinfectant residual, taste and odor problems, and red water complaints. The TOC is removed by biofilms in the filters rather than in the distribution system resulting is improved TOC removal and water that is biologically stable. Deep bed biological filters are included to maximize the removal of TOC, turbidity, and filter run time.

One additional technology was included for future consideration: ultraviolet disinfection. UV could be added if it is later found that the watershed contains *Cryptosporidium* at a level that requires additional treatment. Ozonation could also be effective for *Cryptosporidium* inactivation but much higher ozone dosages are generally required and UV likely has a lower life-cycle cost if 0.5 log or more *Cryptosporidium* inactivation is required.

During coagulation, the pH is depressed by aluminum sulfate. Caustic would be applied to the finished water to increase the pH and provide stable water with regard to calcium carbonate precipitation. Other caustic chemicals such as quicklime o hydrated lime were not considered since adding those downstream from filtration would result in higher turbidity in the tap sample.

The design includes a short period, 2 to 5 minutes, of free chlorine exposure following biological filtration to inactivate bacteria that may slough from the filters. At the outlet of that zone, ammonia is added to form chloramines, the secondary disinfectant. Primary disinfection would be achieved with ozonation, and utilities that use this approach generally have TTHM and HAA5 values less than 15 ug/L at the distribution system point of entry.

Solids would be conveyed from the clearwells, filter backwash, and filter-to-waste to an equalization tank, sludge pump station, earthen bank lagoons and decant return pump station. Solids would be allowed to accumulate in the lagoons and then taken allowed to dewater for 1 year prior to removal.

The process flow diagram for the treatment process is shown on Figure 10-1 with the option of including UV disinfection after the Deep-Bed Biological Filtration.

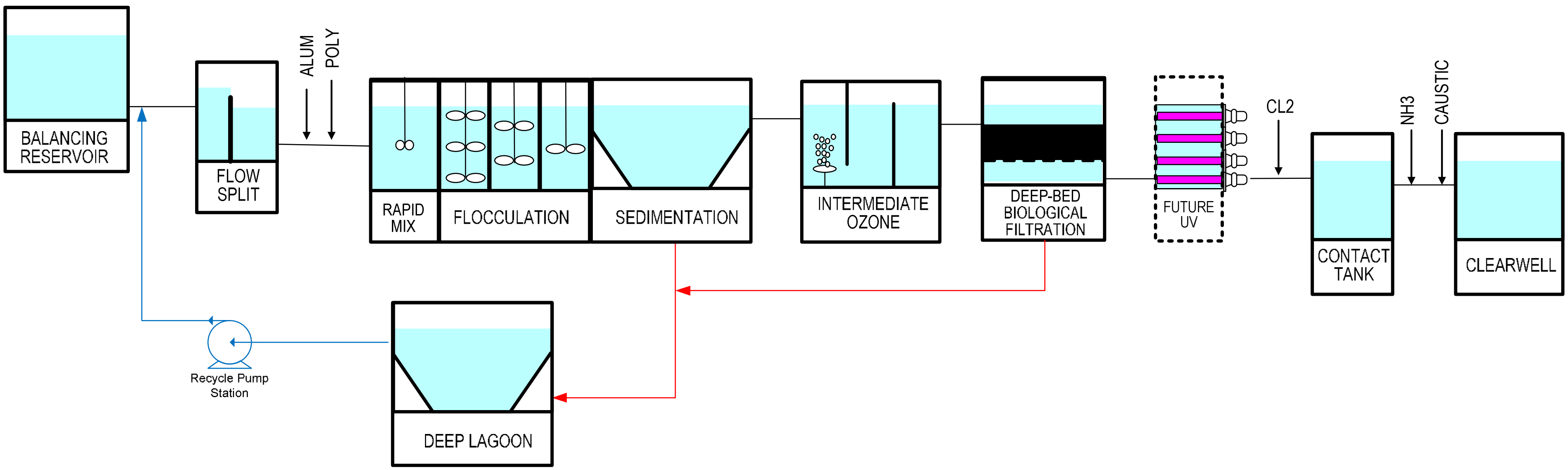


Figure 10-1 Regional SWTP Process Flow Diagram with optional UV Disinfection

10.2.4 Conceptual Site Layout

Conceptual site layouts were developed based on the selected processes sized to 10 and 20 MGD treatment trains. The conceptual site layouts for the Regional SWTP are shown on Figure 10-2 and 10-3. The figures present the initial 20-MGD phase, as well as the built-out facilities for an ultimate capacity of 160-MGD. The site layout includes a general residuals treatment area as well. Table 10-7 describes the planned expansions.

Table 10-7 Regional SWTP Capacity and Expansion Schedule

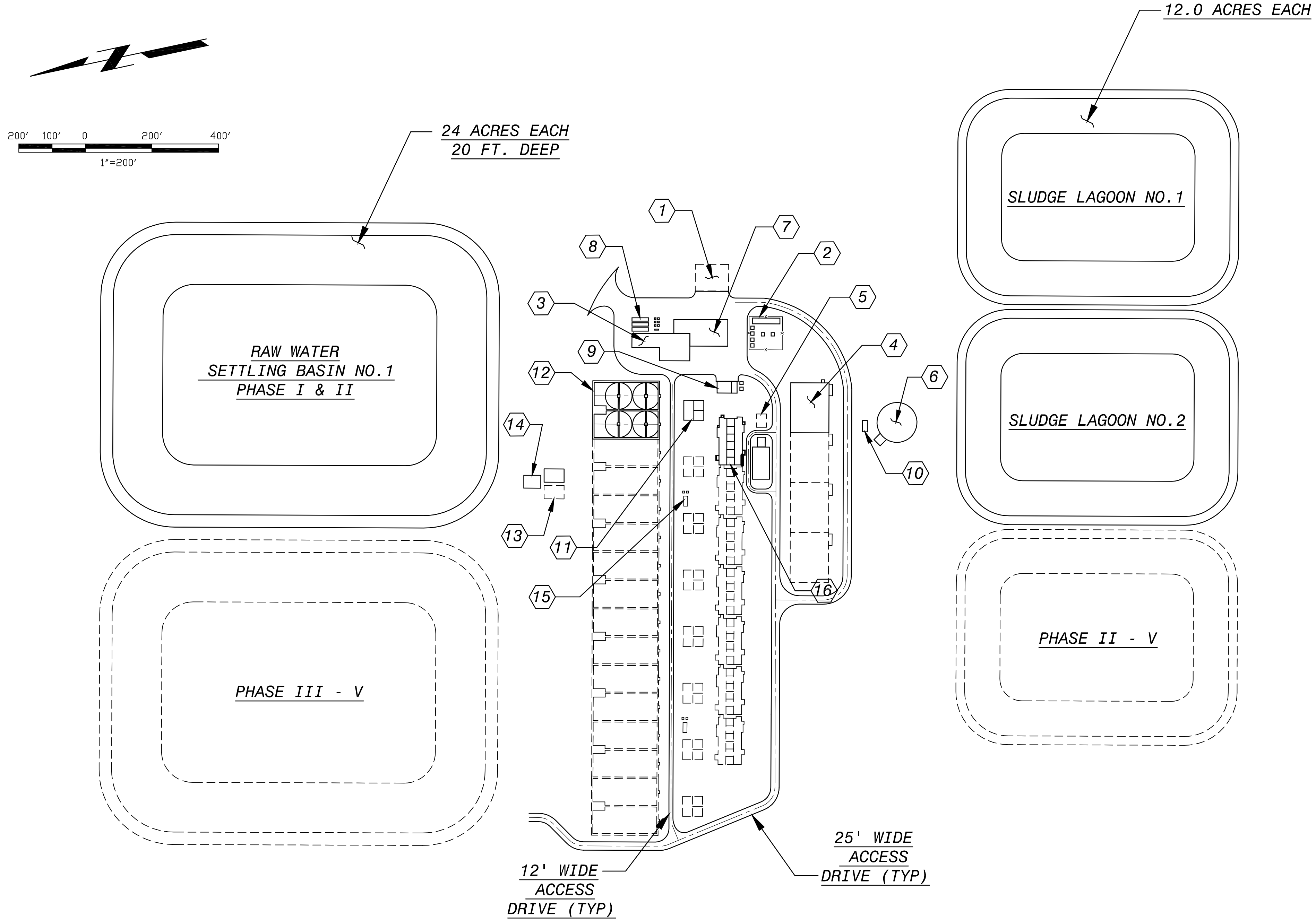
YEAR	MAXIMUM TREATMENT CAPACITY	MODULE SIZE
2030 (Phase I – 20 MGD initially)	20 MGD	2 x 10 MGD trains
2040 (Phase II – 40 MGD expansion)	60 MGD	2 x 20 MGD trains
2050 (Phase III – 50 MGD expansion)	110 MGD	1 x 10 MGD train 2 x 20 MGD trains
2060 (Phase IV – 30 MGD expansion)	140 MGD	1 x 10 MGD train 1 x 20 MGD train
2070 (Phase V – 20 MGD expansion)	160 MGD	1 x 20 MGD train

Planning level conceptual design criteria for the unit process and facilities sizing is summarized below in Table 10-8. It should be noted that the listed conceptual process design criteria were used to preliminary size unit processes and operations for the purposes of providing a conceptual plant layout and developing preliminary engineer's opinions of probable construction cost. These conceptual process design criteria will likely be revised or refined in future engineering phases based on pilot testing results and other engineering and regulatory considerations.

Table 10-8 Regional SWTP Unit Process Sizing Criteria

UNIT PROCESS	CONCEPTUAL PROCESS DESIGN CRITERIA	PHASE I (20 MGD) CAPACITY	PHASES II THROUGH V CAPACITY
Raw Water Intake and Pump Station	Sized for ultimate capacity of 80 MGD	80 MGD intake and pipeline	Addition of pump capacity
Raw Water Storage Reservoir	2 days storage at design flow at Build Out	160 MG (24 acres x 20' deep)	160 MG (24 acres x 20' deep)
Flow Distribution Structure	Split flow equally	1 x 80 MGD	1 x 80 MGD in Phase III
Conventional Sedimentation Basins	0.6 gpm/sq ft	2 x 10 MGD	10 or 20 MGD basin trains
Ozone Contact Basin	10 min HRT	2 x 10 MGD	10 or 20 MGD basin trains
Filters	4.0 gpm/sq ft (with 1 filter out of service)	6 x 4.0 MGD	4 or 8 MGD filters (with 2 filters out of service)

UNIT PROCESS	CONCEPTUAL PROCESS DESIGN CRITERIA	PHASE I (20 MGD) CAPACITY	PHASES II THROUGH V CAPACITY
UV Reactors (future)	3-log *Cryptosporidium* 12 mJ/cm^2	Future	If required, to be determined by design
Reclaimed Water Basin	2.5 backwashes + Filter to Waste	1 x 0.5 Mgal	
Clearwell *-Backwash Compartment* *-Finished Water Storage* *- Chlorine Contact Time Compartment*	 2.25 filter washes Operational flexibility (10%) 0.5 log *Giardia* inactivation (10 °C and pH 8.0)	 1 x 0.4 Mgal 1 x 4.0 Mgal 1 x 0.5 Mgal	 1 x 0.4 Mgal (Phase III) 1 x 4 Mgal (Phase II ,III, & IV) 1 x 0.5 Mgal (Phase III & IV)
Lagoons	10% solids consolidation	2 x 6 acre cells	1 x 6 acre cell (Phase II & III)



1 KEY LEGEND:

1. FUTURE MAINTENANCE BUILDING
2. ELECTRICAL AREA (100'x100')
3. CHEMICAL FEED OPERATIONS & LAB
4. CLEARWELL (150'x 115')
5. FUTURE UV STRUCTURE
6. RECLAIMED WATER BASIN & PUMP STATION
7. CHEMICAL STORAGE AREA
8. LOX: 60x80 OUTDOORS
9. BLOWER BUILDING (60'x35')
10. MCC BUIDING
11. OZONE CONTACT BASIN (60'x 60')
12. FLOCCULATION AND SEDIMENT BASIN (220'x 175')
13. RAW WATER DISTRIBUTION STRUCTURE
14. RAW WATER FLOW CONTROL STRUCTURE
15. FUTURE MCC BUILDING
16. FILTERS (150'x70')

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CIVIL/SITE PRELIMINARY SITE PLAN

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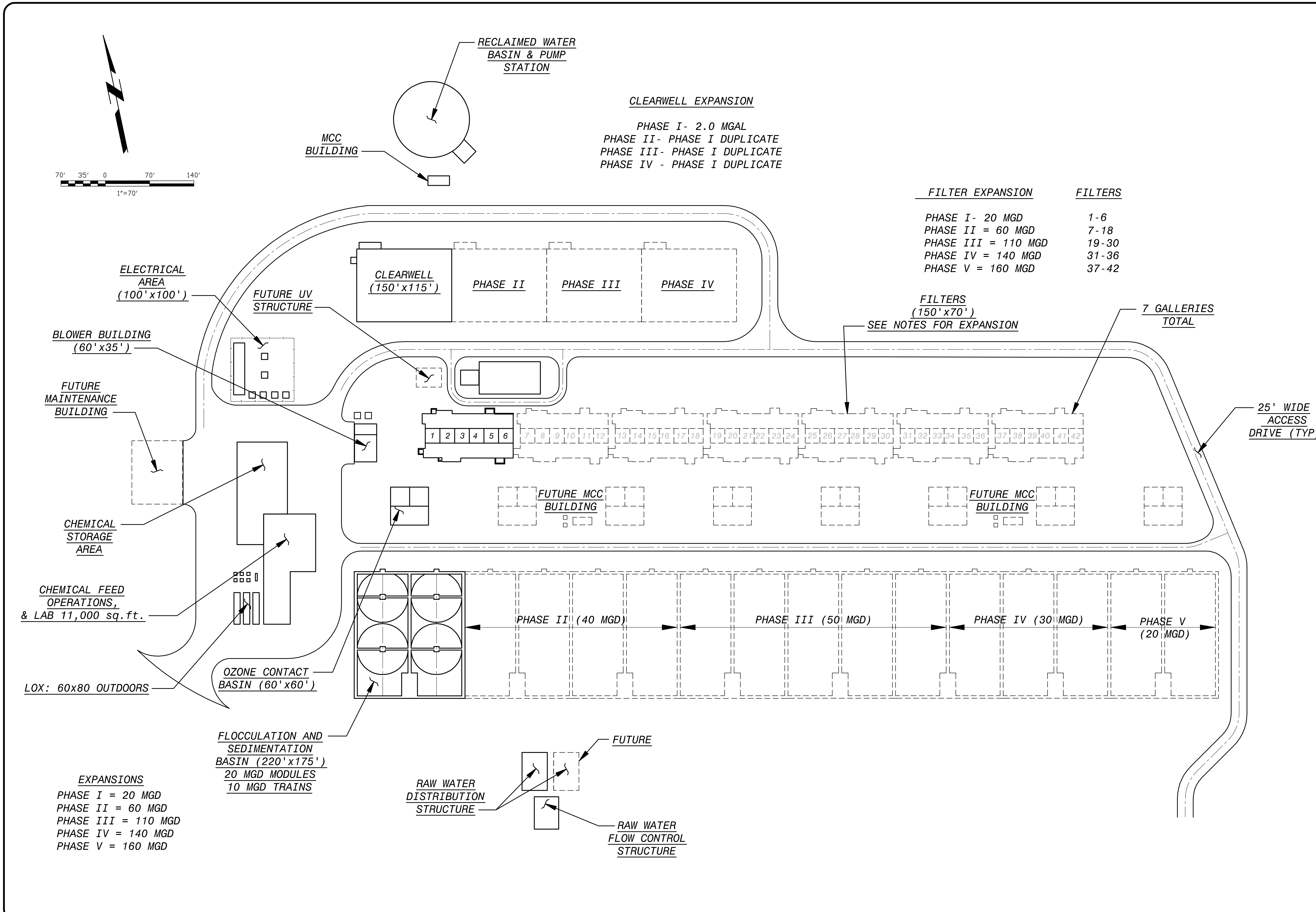

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10.2.5 Solids Quantities and Characteristics

The quantity of residual solids generated at Regional SWTP was calculated based on the amount of water treated, raw water quality, and dosage and type of chemicals used. Based on the water quality data, the majority of the solids produced at the Regional SWTP will be coagulant solids, which can be calculated using the equation below. Assumed dosages and chemicals are as stated.

$$S = 8.34*Q*(b*Tu + aC + A) \qquad \text{(Equation 1)}$$

Where
S = Solids produced (ppd)
Q = plant flow (MGD)
b = Suspended solids/Turbidity ratio (mg/L TSS / NTU)
Tu = raw water turbidity (NTU)
a = solids produced/coagulant addition (mg/mg)
C = coagulant addition (mg/L)
MgNCH = non-carbonaceous Mg hardness (mg/L)
A = additional solids-producing chemicals added (mg/L)

The Regional SWTP will treat raw water from two water sources; the Rio Grande River and collected wastewater effluent that has bas been treated with advanced treatment processes as described in Chapter 11. Raw water characteristics and the parameters used for average solids production calculations are listed in Table 10-9. These values were developed based on average influent water quality and chemical dosages.

Table 10-9 Water Characteristics and Parameters for Solids Production

PARAMETER	UNIT	RIO GRANDE	DPR EFFLUENT
Tu	NTU	88	0
b	mg/L TSS / NTU	1.6	1.6
C (alum)	mg/L	40	5
a (alum)	mg/mg	0.44	0.44
A (polymer)	mg/L	1.0	1.0
Solids Production	lb/MGAL	1,320	27
Solids Volume(5% wt)	Cyd/MGAL treated	16	3

Dewatering lagoons provide solids storage and thickening. Lagoons are typically sized to provide several months or years of storage between cleaning cycles. Decant would be returned to the head of the SWTP and solids would be removed from lagoons through dredging or by allowing the solids to dry in-situ. Since dewatering lagoons have long cycle times, weekly and monthly peaks in solids are attenuated, allowing the lagoons to be sized to support average solids production. The solids concentration of sludge produced by lagoons varies depending on the depth, drying time, and weather conditions, but they are typically in the range of 10 percent total solids for alum sludge with high concentration of solids in the raw water. The number of lagoon cells and area was selected to allow 2 years for drying and cleaning.

The lagoons for the first phase consist of 2 x 6 acre cells, each 15 ft SWD and consolidated to 10 ft after allowed to dry to an average solids concentration of 10 percent by weight. The total area required at complete build out is 24 acres. Solids processing could become more efficient as the plant increases in size and mechanical dewatering could eliminate the need to add lagoons in subsequent expansions.

The cost for removal, hauling, and disposal of water treatment residuals was $125/dry ton and based on recent bid prices for lagoon cleaning and land application of enhanced coagulation residuals.

10.2.6 Chemical and Electrical Use

10.2.6.1 Chemical Summary

A number of chemicals will be required at the Regional SWTP for achieving the desired treatment goals. This section identifies these chemicals with the main purposes of providing a basis for performing a life-cycle cost evaluation for the Regional SWTP.

The chemical feed rates used for the development of operating cost and the chemical feed system range are listed in Table 10-10. The values were estimated based on raw water quality and operating experience. Because the source water for the Regional SWTP will be a blend of surface water and direct potable reuse effluent treated by reverse osmosis, chemical dosages were estimated for the two sources. Chemical dosage ranges should be confirmed during detailed design.

Table 10-10 Chemical Dosage Summary

CHEMICAL	MINIMUM DOSAGE, MG/L	SURFACE WATER AVERAGE DOSAGE, MG/L	DPR EFFLUENT AVERAGE DOSAGE, MG/L	MAXIMUM DOSAGE, MG/L
Hydrofluorosilic Acid, mg/L as F^-	0.2	0.4	0.4	0.8
Aluminum Sulfate, mg/L	20.0	40	5	60
Coagulant Aid Polymer, mg/L	0.5	1.0	1.0	2.0
Ozone, mg/L	2.5	4.0	1.0	5.0
Caustic, mg/L	2	5.0	2.0	15
Chlorine (Hypochlorite), mg/L	3.0	5.0	3.0	8.0
Liquid Anhydrous Ammonia, mg/L as N	0.20	0.6	0.6	1.3

10.2.6.2 Electrical Use Summary

The power required for the Regional SWTP was calculated based on reference projects and unit power costs for operating equipment. Because the site has not been selected, power to pump to the raw water storage reservoirs and to the distribution system have been assumed. Power required for treatment inclusive of mixer motors, ozone generation and injection, filter backwash blowers, etc. is 200 kW-hr/Mgal treated. The remaining energy, nearly 80 percent, is attributed to raw and finished water pumping. The average annual electrical usage is shown in Table 10-11.

Table 10-11 Electricity Use

UNIT PROCESS	HEAD, FT	POWER, KW-HR/MGAL
Raw Water Pump Station	30	127
Rapid Mix	0.9	5
Flocculation	0.0	8
Sedimentation	1.2	5
Ozonation	2.8	195
Granular Media Filtration	16	74
HSPS Power	149	632
Total kW-hr/Mgal		1,047

10.2.7 Finished Water Quality Compatibility Analysis

The treated water from the Regional SWTP must be stabilized in order to prevent degradation of water quality, leaching of pipe materials and scale from releasing into the distributed water, and long-term pipe corrosion. In addition to being compatible with the existing distribution system materials, the finished water must be compatible with treated water from other existing WTPs, which will be blended with finished water from the Regional SWTP in the distribution system. The corrosion indices outlined in the water quality goals section of this document will result in providing a stable water that is non-corrosive. As a general rule, the mixing of two stable waters results in a water that is also stable. Less stable water conditions only occur if one of the blend waters is unstable. Typical raw and finished water conditions for the Regional SWTP are listed in Table 10-12.

Table 10-12 Typical Raw and Finished Water Quality for the Regional SWTP

PARAMETERS	UNITS	RAW (RIO GRANDE)	FINISHED
TDS	mg/L	890	940
pH	Std pH Units	8.2	7.7
Total Alkalinity	mg/L as CaCO3	145	140
Chloride	mg/L	250	255
Sulfate	mg/L	385	390
CSMR	-	0.7	0.7
Total Hardness	mg/L as CaCO3	400	400
Calcium	mg/L	107	107
Magnesium	mg/L	35	35
CCPP	mg/L	-	10
LSI	Std pH Units	-	+0.4

The water served in the distribution systems will change from 100% from the existing WTPs effluent to a blend with the Regional SWTPs finished waters. Changing the water quality in the distribution system can cause concern for materials release from the pipe walls, even if the change results in theoretical improvements to the water quality. It is always recommended that a distribution system water quality monitoring program be developed and implemented following any changes in water quality resulting from bringing a new water source online.

Despite the general stability from the standpoint of solubility chemistry, a potential risk for galvanic corrosion and lead release in customer premise plumbing exists due to the high chloride concentration in the water originating from the Rio Grande River. The chloride sulfate mass ratio (CSMR) remains low, but a distribution system water quality monitoring program should be developed because of the high chloride content of the finished water.

10.3 COST EVALUATION

Budget-level engineer's opinion of probable construction costs (EOPCC), capital, and O&M costs were developed for treatment process, including the planned five development phases. The EOPCC were developed in year 2015 dollars. The EOPCC includes a 25 percent contingency factor to account for uncertainties at this stage of the project including lack of detailed design information and future market conditions. Capital costs were calculated using an additional 15 percent factor on top of the EOPCC for Engineering, Legal, and Administration (ELA) costs associated with the project. The construction contingency and ELA percentages used are in line with industry practices for estimating costs at the conceptual/preliminary stages of a project. O&M costs include chemicals, power, labor, solids disposal, and equipment maintenance.

10.3.1 EOPCC and Capital Cost

Construction costs were developed based on actual construction cost data from available contractor's schedule of values from previous reference projects. The Engineering News Record (ENR) construction cost index was used to adjust the cost Table 10-13 below is a summary of the budget-level EOPCC (including contingency) and capital costs (including ELA) in 2015 dollars.

Table 10-13 Treatment Alternatives - EOPCC* and Capital** Cost

	PHASE I 20 MGD	PHASE II 40 MGD EXPANSION	PHASE III 50 MGD EXPANSION	PHASE IV 30 MGD EXPANSION	PHASE V 20 MGD EXPANSION
Land Acquisition	$1,330,992	$0	$0	$0	$0
Intake and Pipeline	$6,800,000	$600,000	$600,000	$500,000	$500,000
Raw Water Storage Reservoir	$1,300,000	$2,700,000	$3,400,000	$2,000,000	$1,300,000
Sitework/Piping, etc.	$2,500,000	$2,700,000	$3,100,000	$2,200,000	$1,700,000
Process Structures	$9,900,000	$9,200,000	$14,600,000	$9,200,000	$5,700,000
Equipment	$18,000,000	$29,000,000	$34,000,000	$24,000,000	$18,000,000
Chemical, Maintenance, and Operations Buildings	$4,100,000	$0	$1,500,000	$0	$0
General Requirements, Misc.	$5,300,000	$8,100,000	$9,700,000	$6,600,000	$5,000,000

	PHASE I 20 MGD	PHASE II 40 MGD EXPANSION	PHASE III 50 MGD EXPANSION	PHASE IV 30 MGD EXPANSION	PHASE V 20 MGD EXPANSION
Lagoons and residuals pump station	$7,000,000	$3,000,000	$3,500,000	$200,000	$200,000
Electrical, Instrumentation, and Control	$5,700,000	$9,200,000	$10,800,000	$7,500,000	$5,700,000
High Service Pump Station	$4,100,000	$2,000,000	$4,100,000	$1,500,000	$1,000,000
Subtotal EOPCC	**$66,000,000**	**$67,000,000**	**$86,000,000**	**$54,000,000**	**$39,000,000**
Contingency (25%)	$17,000,000	$17,000,000	$21,000,000	$13,000,000	$10,000,000
Engineering, Legal, and Administrative (15%)	$12,400,000	$12,500,000	$16,000,000	$10,100,000	$7,300,000
Water Rights	$20,900,000	$24,800,000	$17,000,000	$15,800,000	$15,500,000
Capital Cost	**$116,000,000**	**$121,000,000**	**$140,000,000**	**$93,000,000**	**$72,000,000**
Cost/gal (with Contingency & ELA)	$5.80 for 20 MGD	$3.00 for 40 MGD expansion	$2.80 for 50 MGD expansion	$3.10 for 30 MGD expansion	$3.60 for 20 MGD expansion
Cumulative Cost/gal (with Contingency & ELA)	$5.80 for 20 MGD capacity	$3.90 for 60 MGD capacity	$3.40 for 100 MGD capacity	$3.40 for 140 MGD capacity	$3.40 for 160 MGD capacity

A brief description of the facilities included in each category is described below:

Land Acquisition	The cost includes purchase of 10 riverfront acres, right of way for the raw water pipeline, and a 240 acre site to accommodate the raw water reservoirs, treatment facility, and lagoons. The 240 acres includes 30 percent excess space.
Intake and Pipeline	A new raw water river intake on piers and 6,000 ft raw water pipeline was included. A single54-inch pipeline would be constructed in Phase I and pumps added in subsequent phases.
Raw Water Reservoirs	Earthen bank reservoirs would be constructed to allow flow by gravity to the Regional SWTP.
Sitework/piping, etc.	This includes site clearing and major yard piping that connects liquid and solids treatment trains.

Process Structures	Process structures includes the concrete liquid train treatment and water storage structures. These include the flow splitting box, sedimentation basins, ozone contact basins, filters, and clearwells.
Equipment	Equipment is inclusive of all mechanical process equipment, valves, and actuators. This would include gate valves, chemical feed pumps, filter underdrains, etc.
Chemical, Maintenance, and Operations Buildings	This includes the structures and finishes of the buildings.
General Requirements, Misc.	This includes the contractor's general requirements such as project management and commissioning, temporary facilities, etc.
Lagoons and residuals pump station	The solids processing components of the project are captured in this category. This includes the reclaimed water basin, sludge and decant water pumps stations, and lagoons.
Electrical, Instrumentation, and Control	All transformers, motor control equipment, electrical and instrumentation duct banks, SCADA programming, and instruments not provided as part of mechanical equipment are included in this category.
High Service Pump Station	The structure, pumps, and pipes within the footprint of the high service pump station.
Contingency	A contingency of 25 percent is appropriate given the information available and project requirements.
Engineering, Legal, and Administrative	A value of 15 percent was assumed for detailed design, construction phase services and legal and administrative activities that will be required to execute the project.
Unit Cost	The unit cost is the total capital cost divided by the treatment capacity of the expansion. The unit cost for Phase I is high in comparison to subsequent phases because several structures (administrative building, raw water flow splitter, raw water pipelines, etc.) are built in Phase I and utilized in subsequent expansions. E.g. the administration building includes space to house future chemical storage tanks and chemical feed pumps.
Cumulative Unit Cost	The cumulative unit cost is the total capital cost for the expansion and previous phases divided by the Regional SWTP capacity. The cumulative unit cost goes down with

each expansion since the project cost of the common facilities constructed Phase I are dispersed over subsequent phases.

10.3.2 Operation & Maintenance Cost

The engineer's opinions of probable O&M cost were developed based on the following categories:

- Chemical Costs. These costs were determined based on the chemicals and dosages described in this chapter and the anticipated average daily flows from surface water and direct potable reuse effluent supplies. The chemical unit prices were established based on regional chemical supply contracts and vendor quotes.
- Power Costs. These costs were determined based on the anticipated plant loads and an average power cost of $0.05/kW-hr.
- Equipment Repair and Miscellaneous O&M. These costs were assumed to represent 1.5% each year of the project's equipment cost.
- Staffing costs are included and the value was based on staffing levels for similar facilities.
- Solids disposal is annualized although disposal may occur every second year.

Approximate operational costs are summarized for each treatment phase Table 10-14. The largest operational cost is chemicals.

Table 10-14 Annual Operational Cost Summary*

	AVERAGE DAILY FLOW MGD	ELECTRICITY $/YR	CHEMICALS $/YR	LABOR & MAINTENANCE $/YR	SOLIDS DISPOSAL $/YR	TOTAL ANNUAL COST $/YR	UNIT COST $/1,000 GAL
Phase I	11	$200,000	$800,000	$500,000	$300,000	$1,800,000	$0.43
Phase II	40	$800,000	$2,000,000	$1,200,000	$800,000	$4,800,000	$0.33
Phase III	69	$1,300,000	$3,000,000	$1,800,000	$1,100,000	$7,200,000	$0.29
Phase IV	86	$1,600,000	$3,800,000	$2,100,000	$1,300,000	$8,800,000	$0.28
Phase V	103	$2,000,000	$4,500,000	$2,400,000	$1,600,000	$10,500,000	$0.28

*These numbers correspond to the initial year of phase operation and are in 2015 dollars

10.4 CONCLUSION

Costs for this project will vary with changes in design decisions, project element locations, raw water quality, water quality targets, and other items still to be considered. Based on the considerations for this cost summary, significant cost variations may arise from several key decisions or findings. In particular, the location, variations in raw water quality, and pipeline routing could lower or raise the costs of this project.

The final capital and operational costs for the Regional SWTP is summarized as shown in Table 10-15.

Table 10-15 Cost Summary

	PHASE I (20 MGD)	PHASE II (40 MGD EXPANSION)	PHASE III (50 MGD EXPANSION)	PHASE IV (30 MGD EXPANSION)	PHASE V (20 MGD EXPANSION)
Project Cost	$83,000,000	$82,000,000	$105,000,000	$66,000,000	$48,000,000
Annual Operations and Maintenance Costs	$1,800,000	$4,800,000	$7,200,000	$8,800,000	$10,500,000

CHAPTER 11 - REUSE WATER SUPPLY AND INFRASTRUCTURE

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

11 FEBRUARY 2016

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Building a world of difference.

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11.0 Reuse Water Supply and Infrastructure

The purpose of this chapter of the report is to provide an assessment of opportunities to beneficially use reclaimed water in the Lower Rio Grande Valley (LRGV). Current uses of reclaimed water in the area and availability of reclaimed wastewater treatment plant effluent are described. The chapter provides a conceptual base case design as well as preliminary opinions of probable costs for the construction and operation of the facilities described.

11.1 INTRODUCTION

Wastewater reclamation is becoming increasingly applied around the world to augment water supplies, especially in drought-prone areas. Major projects have been implemented in the United States (including in Texas, California, and Arizona), Singapore, and Australia. According to the 2012 State Water Plan, water reuse will provide about 1.5 million acre-feet per year of Texas' water supply by the year 2060, which is projected to meet about 18% of water needs statewide (TWDB, 2012).

The two basic types of water reuse projects are non-potable reuse (NPR) and potable reuse. In the majority of cases, both types require additional treatment processes beyond what is practiced for typical municipal wastewater treatment. NPR is the use of reuse water for purposes other than drinking, such as industrial uses, fire protection, cooling towers, and irrigation for agricultural or other landscaping (e.g., golf courses). Typically, NPR requires tertiary treatment including filtration and disinfection. NPR reclaimed water is conveyed directly from the waste water treatment plant (WWTP) to the end users via transmission piping dedicated to that purpose, which is sometimes called "purple pipe" since color coding is frequently used to avoid misapplication of the NPR water.

Regarding potable reuse, according to the TWDB's April 2015 report, "Direct Potable Reuse Resource Document," there are three basic classifications: De facto, Indirect Potable Reuse (IPR), and Direct Potable Reuse (DPR), (See Figure 11-1). In the TWDB report, De facto Water Reuse is defined as "a drinking water supply that contains a significant fraction of treated wastewater, typically from wastewater discharges, although the water supply has not been permitted as a water reuse project." IPR is defined as "the use of reclaimed water for potable purposes by discharging to a water supply source, such as a surface water or groundwater. The mixed reclaimed and natural water then receive additional treatment at a water treatment plant before entering the drinking water distribution system." DPR is defined as "the introduction of advanced-treated reclaimed water either directly into the potable water system or into the raw water supply entering a water treatment plant."

De facto reuse would be a situation in which a WWTP effluent discharge stream constituted a significant portion of the water flowing into a drinking WTP. Examples include some surface WTPs that draw water from the Mississippi River or the Occoquan Reservoir near Washington, D.C. Some examples of IPR include Orange County Water District's 100-MGD Ground Water Replenishment System (GWRS) and its predecessor facility, the Water Factory 21 (WF21), which commenced operation in 1975. GWRS and WF21 are RO-based advanced facilities that spread treated, reclaimed water over land to recharge potable aquifer. Part of the water is also injected in to wells to form a hydraulic barrier to reduce seawater intrusion in California. Projects in Wichita Falls and Big Spring, Texas are examples of DPR. The two projects utilize MF/UF membranes ahead of RO membranes to treat wastewater effluent prior to blending in their raw water supply line or onsite reservoir.

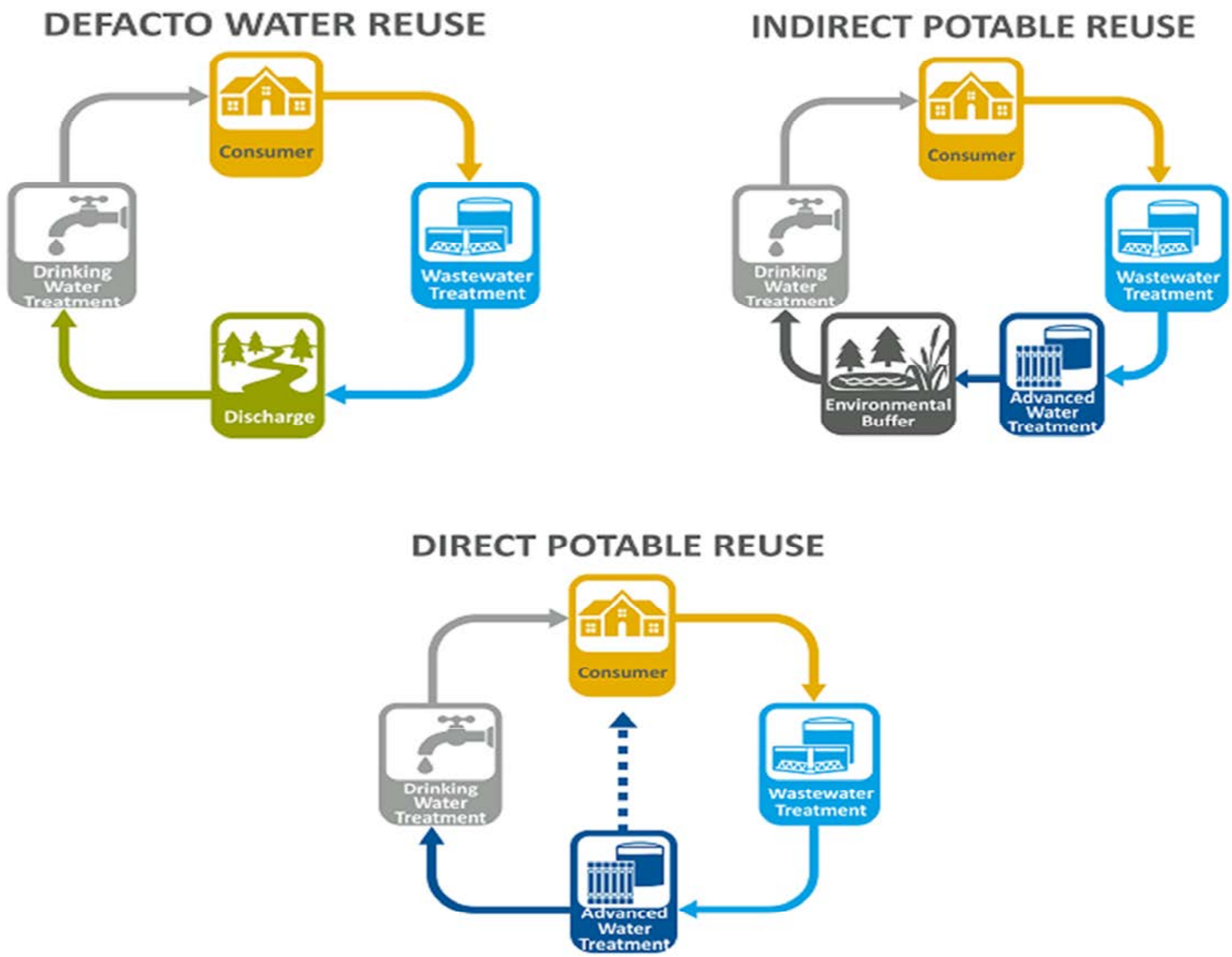
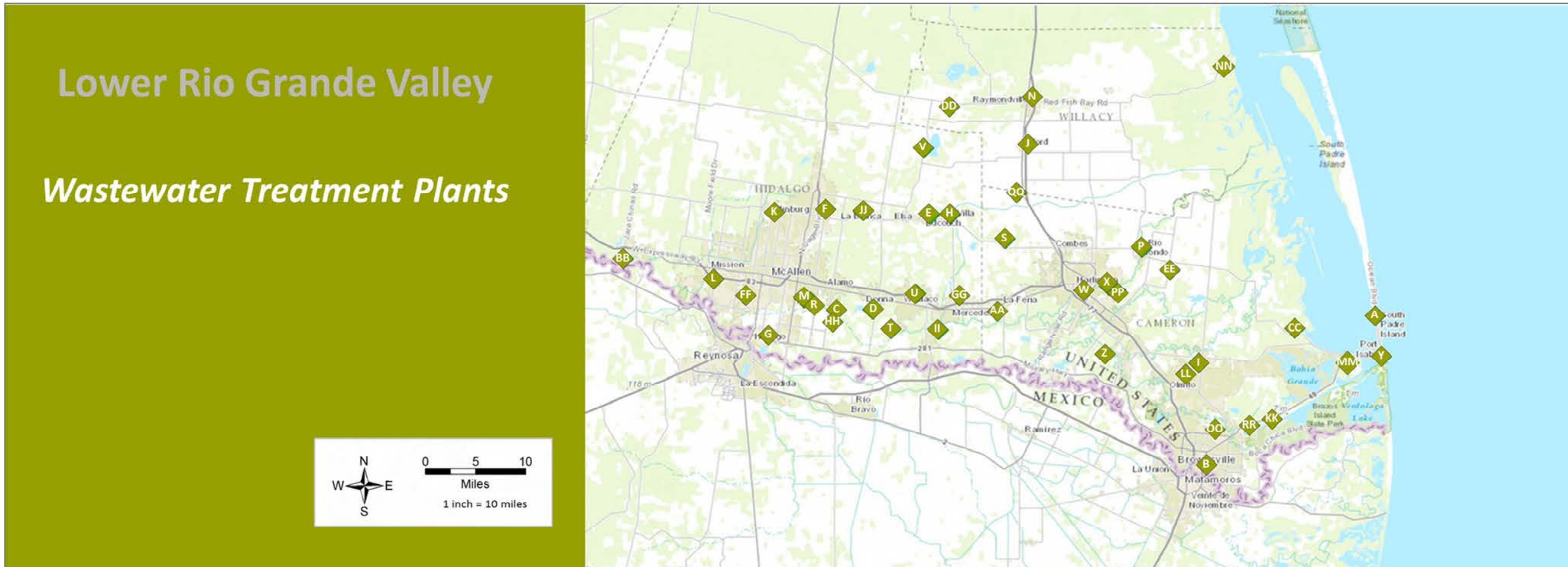


Figure 11-1 Different Types of Potable Reuse from the TWDB DPR Resource Document (April 2015)

While the reuse industry is still developing treatment standards, for purposes of this study, IPR refers to the practice of augmenting a natural drinking water supply with highly treated reclaimed water by discharging into an environmental buffer (such as groundwater aquifer, reservoir, lake or river) where dilution and natural treatment of contaminants can occur. Also, for purposes of this study, DPR refers to the use of highly treated reclaimed water as a drinking water supply without an intermediate environmental buffer prior to drinking water treatment and distribution. Based on this definition, blending highly treated reclaimed water with other natural water supplies, in any proportion, at the intake to a drinking water plant would constitute DPR. Both IPR and DPR require advanced treatment using a multiple-barrier approach. Advanced water treatment (AWT) systems for potable reuse are typically designed to meet target pathogen log reduction values (LRV), meeting the primary and secondary maximum contaminant levels, and also contaminants of emerging concern such as pharmaceuticals and personal care products. For AWTs there is also concern regarding disinfection byproducts (DBPs), including standard water treatment DBPs as well as some more specialized within reclamation applications, such as nitrosodimethylamine (NDMA).

There are currently 43 wastewater treatment plants located in the LRGV, ranging in max capacity from 0.005 MGD to 10 MGD. As expected, they are distributed near population centers in the area. The locations and capacities of the facilities are shown in Figure 11-2.



Label	Name	Capacity (MGD)
A	LMWD Andy Bowie WWTP	0.9
B	BPUB Southside WWTF	7.8
C	City of Alamo WWTP	2.0
D	City of Donna WWTP	2.3
E	City of Edcouch WWTP	0.3
F	City of Edinburg WWTP	7.6
G	City of Hidalgo WWTP	1.2
H	City of La Villa WWTP	0.2
I	City of Los Fresnos WWTP	1.0
J	City of Lyford WWTP	0.3
K	McAllen North WWTP	8.0
L	City of Mission WWTP	9.0
M	City of Pharr WWTP	6.0
N	City of Raymondville WWTP	1.5
P	City of Rio Hondo WWTF	0.4

Label	Name	Capacity (MGD)
R	City of San Juan WWTP	4.0
S	City of Santa Rosa WWTP	0.4
T	City of Weslaco South WWTF	2.0
U	City of Weslaco North WWTP	5.5
V	Hidalgo County Delta Lake Park WWTF	.005
W	Harlingen WWTP 1	6.0
X	Harlingen WWTP 2	6.0
Y	LMWD Isla Blanca WWTP	2.6
Z	MHWSC Joines Road WWTP	0.5
AA	La Feria WWTF	0.5
BB	La Joya WWTP	0.4
CC	LMWD Laguna Vista WWTP	0.7
DD	NAWSC Lasara WWTP	0.1
EE	ERHWSC Lozano WWTP	0.1
FF	McAllen South WWTP	10

Label	Name	Capacity (MGD)
GG	Mercedes WWTP	5.0
HH	MHWSC Bali Road WWTF	0.5
II	MHWSC Progresso WWTP	0.8
JJ	NAWSC Monte Alto WWTP	0.3
KK	Brownsville Navigational District Northside WWFT	0.1
LL	Olmito WSC WWTP	0.8
MM	LMWD Port Isabel WWTP	1.1
NN	Port Mansfield PUD WWTF	0.2
OO	BPUB Robindale WWTP	10
PP	San Benito WWTF	3.8
QQ	Sebastian MUD WWTF	0.2
RR	Brownsville Navigational District Turning Basin WWTP	0.1

Figure 11-2 Wastewater Treatment Plants in the Lower Rio Grande Valley

11.2 CURRENT USE

There are currently eight municipalities in Cameron and Hidalgo Counties that use reclaimed water for NPR, Table 11-1. There are no potable reuse facilities in the LRGV to-date.

Table 11-1 Current Reuse Water Usage in the Lower Rio Grande Valley

MUNICIPALITY	WWTP	AVERAGE REUSE (MGD)	MAXIMUM REUSE CAPACITY (MGD)	INTENDED USE
Laguna Madre Water District	Isla Blanca Park WWTP	0.06	0.40	Irrigation
Laguna Madre Water District	Laguna Vista WWTP	0.30	0.40	Golf Course Irrigation and Lagoons
City of Harlingen	Harlingen WWTP No. 2	1.00	3.00	Irrigation; Watering Ponds; Industrial
Valley MUD No. 2	Rancho Viejo WWTP	0.10	0.21	Golf Course Pond
City of McAllen	McAllen South WWTP	2.00	6.00	Golf Course Irrigation Power Supply
City of Pharr	Pharr WWTP	1.20	8.00	Golf Course Irrigation
City of Weslaco	Weslaco South WWTP	0.94	1.00	Golf Course Irrigation
Brownsville PUB	Robindale WWTP	6.0	10	Power Supply

11.3 LIMITATIONS

11.3.1 Wastewater Treatment Plant Effluent Flow and Water Quality

The total capacity of wastewater treatment plants in the study region is approximately 110 MGD. However, the amount of reclaimed water that can be utilized is limited by the actual flow of the wastewater treatment plants that supply the effluent. It is assumed that half of a wastewater treatment plant's average effluent is available on a consistent basis to produce reclaimed water. It is important to note that for potable reuse applications there must also be sufficient capacity in the water treatment plants that will receive the reclaimed effluent. At this time, no environment flows are required to be maintained in receiving streams for the wastewater treatment plants, so it is not a limitation on the availability of reclaimed water.

The typical Total Dissolved Solids (TDS) concentration of surface water diverted from the Rio Grande is between 700 and 920 mg/L. It should be noted that TDS concentrations in the Rio Grande have been considerably higher for short durations, which can further increase TDS concentrations in the wastewater, and this has led some municipalities to discontinue using reclaimed wastewater for irrigation purposes temporarily or permanently. It is anticipated that a 25% increase in TDS occurs between the water and wastewater treatment plants (WWTP). Therefore, local WWTP effluent will have a TDS in the range of 875 to 1,150 mg/L, and sometimes significantly higher. High TDS concentrations limit the use of reclaimed water for agricultural and some industrial purposes. Specific industrial customers may opt to treat the water for their individual processes.

11.3.2 Rules and Regulations

11.3.2.1 Non-Potable Reuse

In order to implement a reclaimed water system for municipal use, an entity must receive written approval from TCEQ. TCEQ enforces set regulations for NPR. According to Chapter 210 of the Texas Administration Code, NPR water is divided into two categories based on water quality parameters that can only be used for specific purposes. Tables 3 and 4 outline the requirements for each type of NPR water.

Table 11-2 Type I Reclaimed Water Requirements

PARAMETER	LIMIT	TIMEFRAME
BOD_5 or $CBOD_5$	5 mg/l	30-day average
Turbidity	3 NTU	30-day average
Fecal Coliform or *E. Coli*	20 CFU/100 ml	30-day average
Fecal Coliform or *E. Coli*	75 CFU/100 ml	Maximum single grab
Enterococci	4 CFU/100 ml	30-day average
Enterococci	9 CFU/100 ml	Maximum single grab

Table 11-3 Type II Reclaimed Water Requirements

PARAMETER	LIMIT	TIMEFRAME
BOD_5 or $CBOD_5$	20 mg/l or 15 mg/l	30-day average
Fecal Coliform or *E. Coli*	200 CFU/100 ml	30-day average
Fecal Coliform or *E. Coli*	800 CFU/100 ml	Maximum single grab
Enterococci	35 CFU/100 ml	30-day average
Enterococci	89 CFU/100 ml	Maximum single grab

The allowable uses for Types I and II Reclaimed Water are based on the likelihood of human contact. Type I allowable uses include:

- Residential irrigation.
- Irrigation of public parks, golf courses, school yards, and athletic fields.
- Fire protection (either internal sprinkler systems or external fire hydrants).
- Food crops where the reclaimed water may have direct contact with the edible part of the crop.
- Irrigation of pasture for milking animals.

- Maintenance of water bodies where recreational activities, such as wading or fishing, are anticipated.
- Toilet or urinal-flush water.
- Other similar activities where the potential for unintentional human exposure may occur.

Type II allowable uses are listed below. It is important to note that Type I reuse water may be used for any Type II application.

- Irrigation of sod farms, silviculture, limited access highway rights of way, and other areas where human access is restricted or unlikely to occur.
- Irrigation of food crops where the reclaimed water is unlikely to have direct contact with the edible part of the crop, or where the food crop undergoes pasteurization prior to distribution for consumption.
- Irrigation of animal feed crops other than pasture for milking animals.
- Maintenance of water bodies where direct human contact is not likely.
- Soil compaction or dust control in construction areas.
- Cooling tower makeup water.
- Irrigation or other non-potable uses of reclaimed water at a wastewater treatment facility.

11.3.2.2 Potable Reuse

TCEQ is currently undergoing the process to develop written regulations for potable reuse; however, the agency has given authorization for potable reuse systems in Texas on a case by case basis. TCEQ has published draft information on the topic and developed guidelines for review of potable reuse projects[1]. Select information on potable reuse follows:

- WWTP effluent is to undergo multiple physical and chemical barriers (removal and/or inactivation) prior to being discharged to a public water system for public consumption. Treatment via a conventional WTP, operating within state and federal requirements, may be considered one set of minimum barriers, however, treatment processes at the WWTP will not be counted.
- DPR projects must provide two separate and redundant sets of treatment barriers in series to provide total minimum microbial inactivation and removal as follows:
  - 8 log virus removal
  - 6 log *Giardia* removal
  - 5.5 log *Cryptosporidium* removal (WWTPs with sufficient Cryptosporidium monitoring results may be subject to lower requirements)
- Existing IPR plants generally specify that reverse osmosis membranes provide a minimum of 99.5% salt rejection because membranes that provide lower rejection are sometimes considered by regulators to achieve lower rejection of chemicals that may pose potential or actual threats to human health.

In general, it is anticipated that a pilot test would be required for any future potable reuse authorizations whether it is IPR or DPR.

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[1] Texas Water Development Board, Direct Potable Reuse Resource Document, 2015, page 3-10.

11.4 WATER QUALITY

Table 11-4 gives an assumption of the water quality in both the untreated wastewater and that of the water after treatment with both primary and secondary treatment. Specifically, it is assumed that that existing plants will have at the minimum bar screens, grit removal primary clarification, conventional activated sludge treatment and chlorination. This table is derived from one found in the Framework for Direct Potable Reuse by the Water Reuse Association.

Table 11-4 Expected Wastewater Water Quality Before and After Conventional Activated Sludge Treatment

CONSTITUENT	UNIT	UNTREATED WASTEWATER	AFTER CONVENTIONAL ACTIVATED SLUDGE TREATMENT (BEFORE DISINFECTION)
Total Suspended Solids	mg/L	130-389	5-25
Turbidity	NTU	80-150	2-15
Biochemical Oxygen Demand	mg/L	133-400	5-25
Chemical Oxygen Demand	mg/L	339-1016	40-80
Total Organic Carbon	mg/L	109-328	20-40
Ammonia Nitrogen	mg N/L	14-41	1-10
Nitrate Nitrogen	mg N/L	0-trace	5-30
Nitrite Nitrogen	mg N/L	0-trace	0-trace
Total Nitrogen	mg N/L	23-69	15-35
Total Phosphorus	mg P/L	3.7-11	3-10
Volatile Organic Compounds	µg/L	<100->400	10-40
Iron and Manganese	mg/L	1-2.5	1-1.5
Surfactants	mg/L	4-10	0.5-2
Total Dissolved Solids	mg/L	374-1121	374-1121
Trace Constituents	µg/L	10-50	5-40
Total Coliform	No./ 100 mL	10^6-10^{10}	10^4-10^5
Protozoan Cysts and Oocysts	No./ 100 mL	10^1-10^5	10^1-10^2
Virus	PFU/100 mL	10^1-10^8	10^1-10^4

Target water quality goals are listed in 11.3 for each reuse application. The disinfected secondary effluent water quality may meet Type II reuse water standards but will likely require a filtration process to remove turbidity to be permitted for Type I uses. To meet potable water standards advanced treatment processes such as advanced oxidation (such as UV and hydrogen peroxide) and Reverse Osmosis will be required.

11.5 POTENTIAL REUSE STRATEGIES

11.5.1 Non-Potable Reuse

Standard treatment for NPR involves a tertiary treatment filter at the end of the typical municipal wastewater treatment process, such as activated sludge and clarification. Disinfection with chlorine is also typically required to provide a residual in the reclaimed water during storage and transport to the end user.

NPR is a feasible replacement for potable water users who do not require drinking water quality. However, the focus of this study is to augment municipal water use, which is principally potable water. Although irrigation of public landscaped areas is a typical application of NPR, as previously discussed, the TDS of wastewater effluent in the region is higher than desired to make NPR suitable for agricultural use. Therefore, outside customers of NPR water would need to be identified in order to develop an infrastructure plan that includes special distribution piping for this type of reclaimed water. Additionally, NPR demands are typically seasonal and the cost to install and operate the reuse system may be cost prohibitive for seasonal uses. Therefore NPR will not be evaluated further as an application for this study. At the present time, it doesn't appear to be enough industrial demand in the region to offset a significant amount of potable water use.

11.5.2 Potable Reuse

The use of advanced water treatment (AWT) processes would be required to meet TCEQ's requirements for DPR, including anticipated requirements for microbial log removals and emerging contaminants. The product water from the AWT Facilities would then be blended with raw surface water and diluted at a maximum ratio of 1:1 prior to entering a traditional surface water treatment plant. The 1:1 ratio is assumed for this study because it is the value that TCEQ allowed for the DPR system in Wichita Falls, TX. The AWT Facilities are assumed to be located at the water treatment plant site to consolidate AWT operations.

11.5.2.1 Treatment

The industry standard for accomplishing AWT for potable reuse includes the following major process steps: membrane filtration applying either microfiltration or ultrafiltration (MF/UF), reverse osmosis (RO), and an advanced oxidation process (AOP) applying ultraviolet light (UV) and hydrogen peroxide (H2O2) or other chemical oxidation. In addition this process generally incorporates a chloramine residual to control membrane fouling, which helps to maintain operating pressures and lowers operating costs. Overall, this process approach provides a well-proven, multiple-barrier approach. It is the basis for potable water treatment projects in Texas as well as projects around the world. A long-term study of full-scale AWT facilities in Australia demonstrated consistent results at meeting a full range of drinking water quality parameters. A schematic of the process is shown in Figure 11-3. The TCEQ allowable log removal per selected process is shown Table 11-5. The purpose of each of the major treatment steps is summarized in Table 11-6.

Table 11-5 Allowable Log Removal from Proposed Treatment Process[2]

TREATMENT	CRYPTO LOG REDUCTION	GIARDA LOG REDUCTION	VIRUS LOG REDUCTION
MF/UF	4	4	0
Reverse Osmosis	0	0	0
Advanced Oxidation/ UV	4	4	4
Water Treatment Plant	3	3	4
Total	**11**	**11**	**8**
TCEQ Goal	**5.5**	**6**	**8**

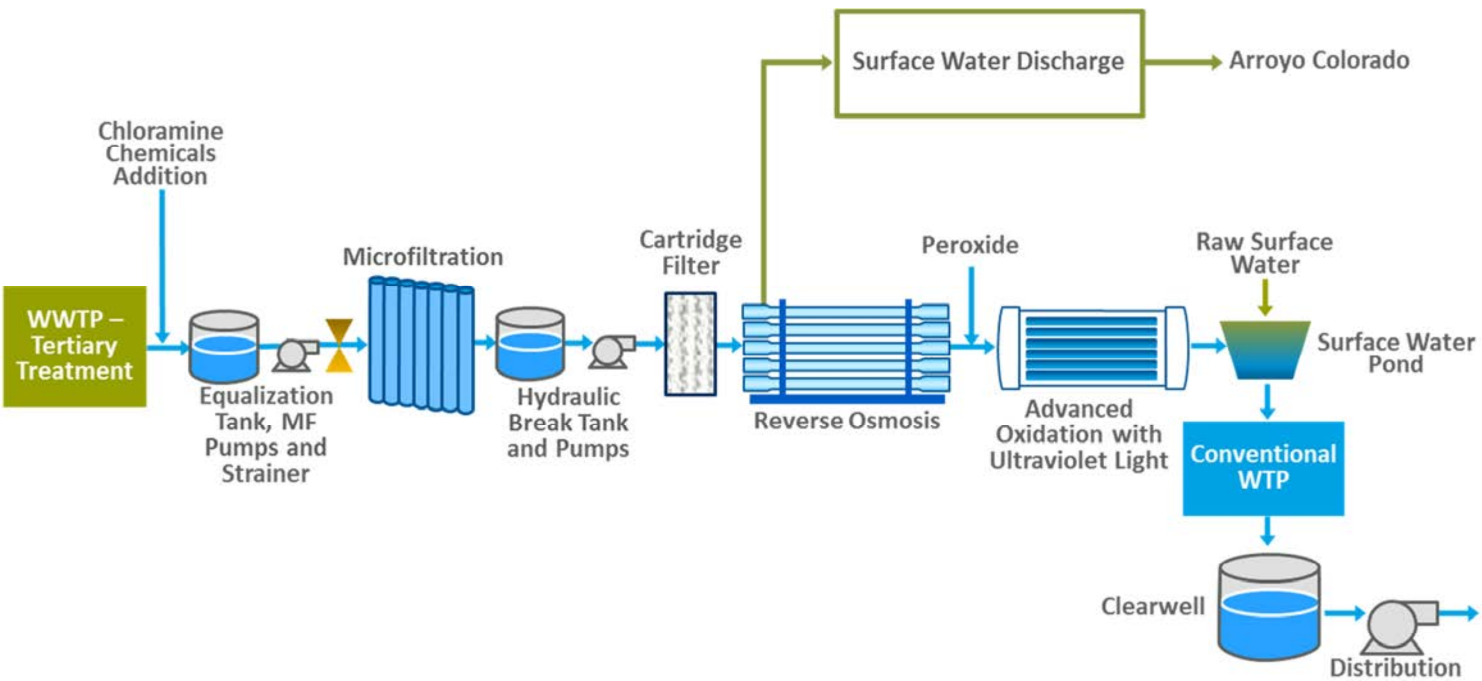


Figure 11-3 Schematic of Anticipated Direct Potable Reuse Treatment

[2] Texas Water Development Board, Direct Potable Reuse Resource Document, 2015, pages 3-10, 5-15

Table 11-6 Summary of Treatment Steps

TREATMENT STEP	SUMMARY
Disinfection - Chloramine Residual	A low chloramine residual (1 to 4 mg/l) is maintained in the water to limit biological activity and protect the MF/UF and RO processes from biofouling.
Storage/Equalization Tankage	The equalization storage tank provides a location to collect the water that is to be treated.
Fine Screen/Strainer	A fine screen, nominally rated at 100 to 400 micron (0.4 mm), protects the MF/UF from damaging particles.
Microfiltration/Ultrafiltration (MF/UF)	MF/UF membrane filtration protects the RO from damaging particles and provides a significant barrier to microbial material, such as at least 4-log (99.99%) removal of Giardia and Cryptosporidium.
RO Pretreatment	Cartridge filtration, nominally 5 micron, as well as antiscalant addition are included as part of the RO system to protect the membrane from damage due to particles in the water as well as inorganic scaling due to the precipitation of sparingly soluble solutes.
Reverse Osmosis (RO)	RO is a cross-flow filtration process that removes dissolved materials (e.g., DOC, TDS, and individual ions, such as nitrate and phosphate) from the water.
Advanced Oxidation Process (AOP), Generally with Ultraviolet Light and Hydrogen Peroxide (UV/H2O2)	UV/H2O2 provides disinfection and oxidation of trace quantities of residual constituents.
Stabilization	Adding controlled amounts of hardness, alkalinity, and/or pH adjustment in the stabilization step controls the corrosiveness of the RO permeate.
Clean In Place (CIP) Waste Neutralization	Spent chemical cleaning solution waste streams from the MF/UF and RO processes are blended and pH adjusted, as needed, before being returned to the WWTP.

Throughout the treatment process, different processes are used to separate out the containments in the water. Each process removes different sizes of materials from the water as it passes through. Figure 11-4 shows different containments and their sizes, as well as what process of separation is used to remove the containment. The figure shows the various particle cutoff points for the membrane processes. MF/UF serves as an initial filtration step to remove larger particles that may damage or shorten the RO run time. RO offers the advantage that it can remove particle as well as dissolved salts (or TDS).

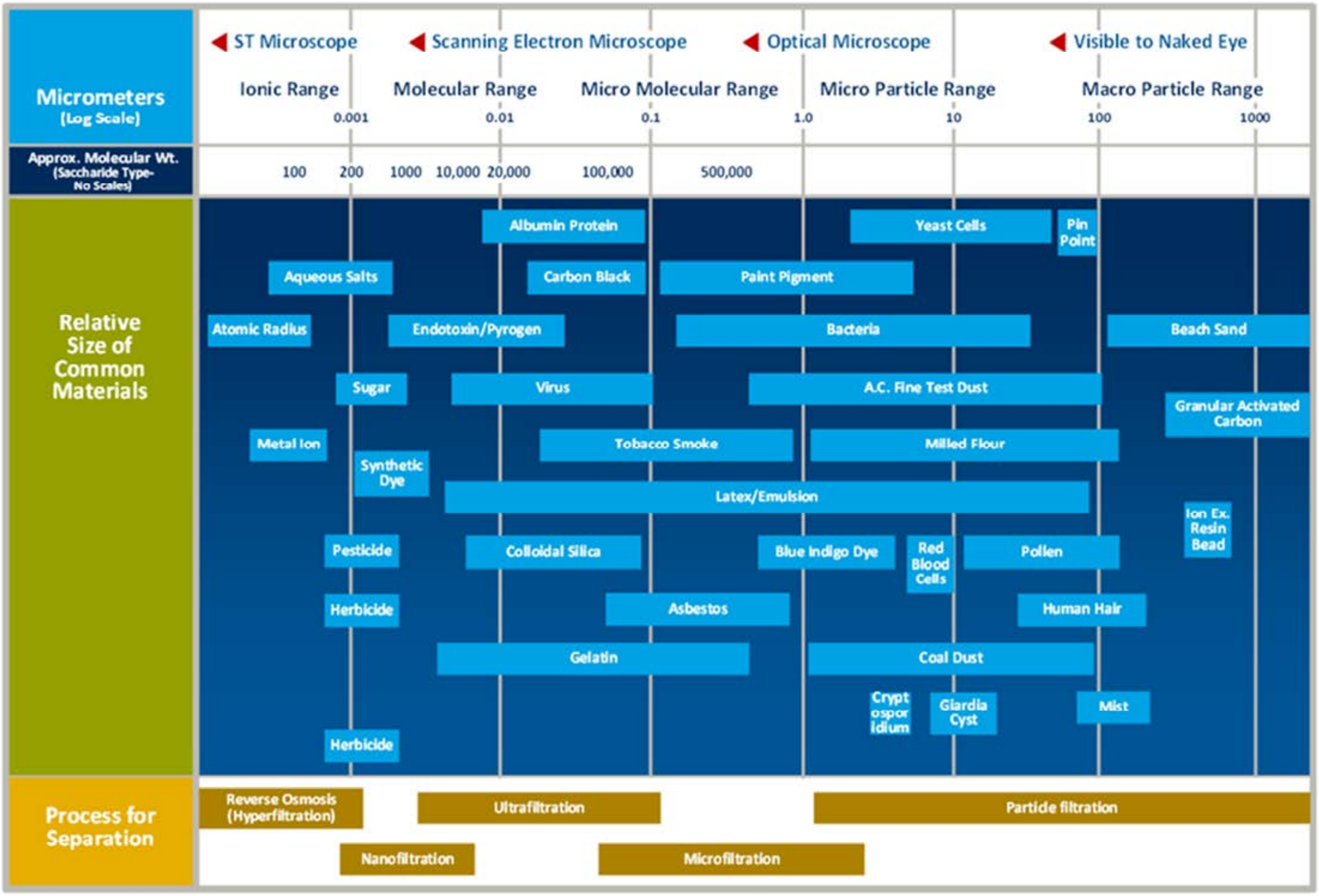


Figure 11-4 Particle Size Removal for Filtration Techniques

Comparing this overall treatment approach (MF/UF + RO + AOP) to alternative methods, other process sequences have been suggested for DPR, including methods that do not incorporate RO. Examples of each are provided in Figure 11-5. The figure provides 6 potential treatment trains as identified in the TWDB DPR Resource Document completed in April of 2015. The second treatment method (MF/UF + RO + AOP) is the basis for the RGRWA conceptual design presented in this chapter for a variety of reasons and is highlighted in the figure. A major reason for its selection is that this is currently the most frequently applied and well-proven method for potable reuse. Alternatives without RO are sometimes considered, but not widely practiced. In some cases treatment approaches without RO concentrate, and hence without RO, are considered due to high disposal costs or other problems, such as limitations in securing high-TDS discharge permits. Neither of these is anticipated in this case, since RO concentrate disposal via the Arroyo Colorado is straightforward and relatively inexpensive. In addition, in this case there is a need to reduce the concentration of dissolved materials, including TDS, in the reclaimed water before reuse, and the selected process including RO is the most practical way to accomplish that. Therefore, the treatment approach on which the cost opinions are based includes RO. An RO recovery (ratio of permeate/feed) of 85% is assumed, since this is commonly practiced and will yield a concentration lower than the anticipated TDS limit of 13,000 mg/L for discharge into the Arroyo Colorado.

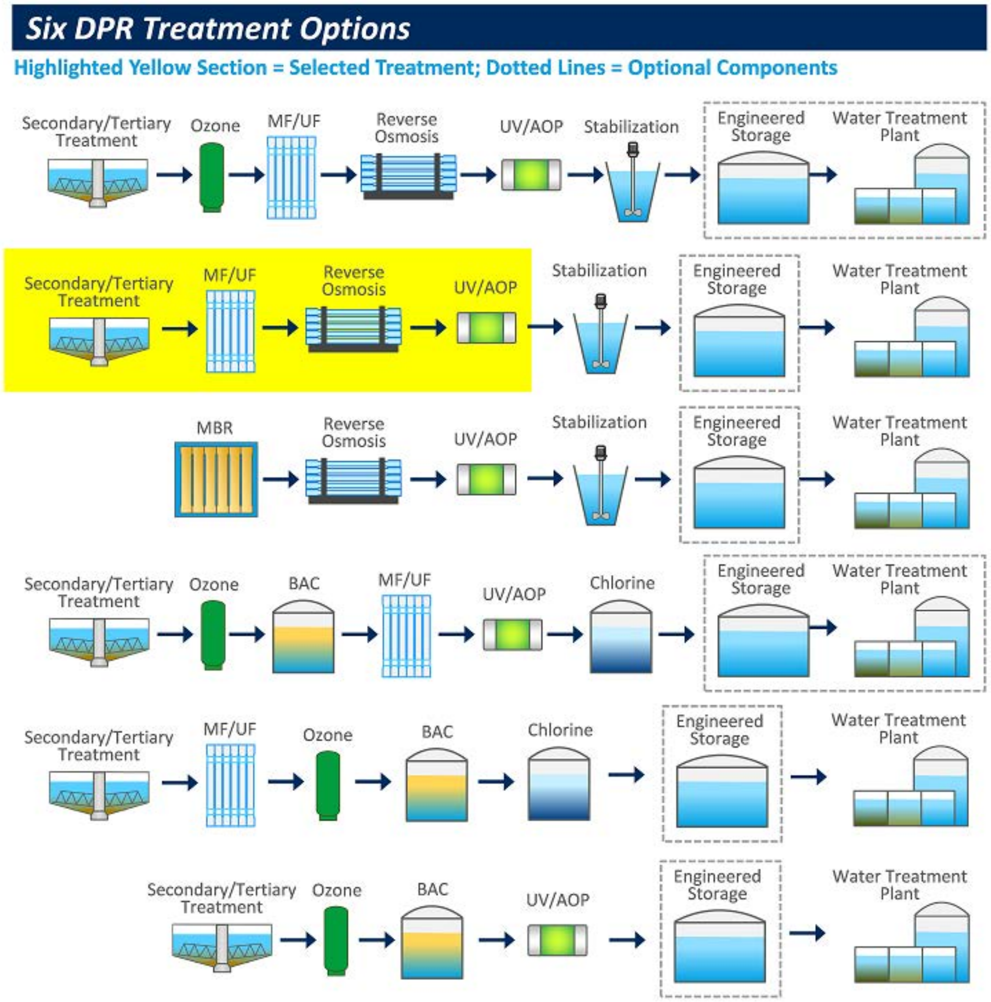


Figure 11-5 Particle Size Removal for Filtration Techniques

Downstream from the RO step there are various methods that could provide advanced oxidation. AOP is typically done using the ultraviolet light (UV)/peroxide (H_2O_2) method. In this process, water is exposed to UV light in the presence of H_2O_2 which results in the formation of hydroxyl radicals (OH^-). The hydroxyl radical is the most powerful oxidant used in water treatment, and it reacts rapidly with most substances. The resulting oxidizing environment breaks chemical bonds, removing or breaking down many organic compounds including many of the CECs that are being examined in research today and that may be regulated.

11.5.2.2 Coupling of local WWTP and WTP for dispersed systems

A strategy for direct potable reuse in the study area would be to couple the WWTP and WTP for certain cities. An evaluation of wastewater treatment plants in the study area was performed in order to determine the entities that could benefit from potable reuse. All of the wastewater treatment plants with an average effluent flow greater than 2.0 MGD were considered suitable to potentially provide a cost effective yield of reuse water. It was assumed that half of the average daily effluent flow would be available for reuse on a consistent basis with storage for diurnal fluctuations. The reclaimed water available for reuse was developed by reducing the average effluent flow by half and subtracting any current or planned use. Only plants with more than 1 MGD of available effluent have been further evaluated for DPR strategies. The proximity of wastewater treatment plants to water treatment plants and available capacities of the water treatment plants were also taken into consideration. DPR projects which do not require expansion of existing water treatment facilities are more cost competitive with other potential supplies.

Wastewater treatment plant average effluent flow data and water treatment plant capacities were obtained from TCEQ and the service provider websites. Information about current water treatment plant flows came from the Region M Regional Water Plan data. In order to determine the available capacity of the water treatment plants, the average amount of surface water that each plant currently treats was multiplied by a peaking factor of 1.7 and then subtracted from the plant's total treatment capacity.

Table 11-7 presents the municipalities that are considered potentially feasible to install new potable reuse systems. The amount of effluent available for a DPR system from each wastewater treatment plant is given. Each water treatment plant is listed with its total capacity and the capacity it has available to treat potable reuse. The DPR system capacity is sized based on the lesser of available WWTP effluent or available WTP capacity. If there is adequate WTP capacity, the potable reuse system is shown to provide 80% of the available WWTP effluent, under the assumption that 20% of the WWTP effluent would be discharged as waste after the treatment processes.

Table 11-7 Potentially Feasible Potable Reuse Systems

ENTITY	WWTP	AVAILABLE WWTP EFFLUENT* (MGD)	WTP	WTP CURRENT CAPACITY (MGD)	AVAILABLE WTP CAPACITY (MGD)	POTABLE REUSE SYSTEM CAPACITY (MGD)
Brownsville PUB	Southside WWTF	3.57	WTP #2	20.00	4.84	2.85
City of Harlingen	Harlingen WWTP #2	1.63	Downtown WTP	18.70	9.34	1.30
City of McAllen	South WWTP	1.12	South WTP	47.25	23.73	0.90
City of Mission	Mission WWTP	3.41	South WTP	19.50	1.17	1.17
City of Mercedes	Mercedes WWTP	1.49	Mercedes WTP	5.50	3.18	1.19
					Total	**7.41**

**Available WWTP Effluent is equal to 0.5*Average Effluent Flow minus existing reuse.*

Utilizing this strategy would provide a limited amount of raw water to local systems and reduce or delay the future water supply projects. As development occurs in the valley WWTPs will increase their capacities and more effluent will become available. If municipalities plan future SWTP infrastructure for both raw surface water and treated WWTP effluent, municipal supplies could reliably be augmented through this application. There are a number of disadvantages to this approach. Namely, finding qualified operators, increase in capital costs to construct many smaller AWT systems, and complexity of system operations. Further, for DPR applications, it is critical that the plants are well maintained and have rigid and extensive monitoring of the performance of various processes within the AWT. This will be difficult to accomplish with numerous smaller plants compared to one centralized large scale facility.

11.5.2.3 Collect WWTP Effluent to Centralized SWTP

Another strategy is to collect the effluent from the WWTPs in the study area and pump all of the water to the centralized SWTP that is proposed near the Rio Grande in Hidalgo County. The collection line follows the same route as the proposed potable water distribution line, allowing for any WWTP along the path to connect to the line and feed its effluent all the way to the SWTP. The collection line is sized to convey all available flow from the WWTPs that could be treated for raw water blending and will be expanded as needed. Prior to blending the wastewater effluent would be equalized in a storage basin and processed through the AWT system.

Table 11-8 shows the available amount of effluent along the proposed pipelines at each WWTP as well as the projected effluent flow for each decade. The projected flows were calculated by taking the current flow and dividing it by the population and then multiplying the ratio by the projected population for each decade. If a city had two WWTPs, a ratio was first found by dividing one WWTP flow by the total combined flow of both plants, then multiplying it by the population growth and the projected population for that decade. The table also removes any current non-potable reuse that may already be implemented by each entity.

Table 11-8 WWTP Effluent in the Lower Rio Grande Valley

ENTITY	WWTP	WWTP CAPACITY (MGD)	WWTP AVERAGE EFFLUENT FLOW (MGD)	WWTP AVERAGE EFFLUENT FLOW PER DECADE (MGD)					
				2020	2030	2040	2050	2060	2070
Brownsville PUB	Robindale WWTP	10	6.7	1.8	3.3	4.8	6.4	8.0	9.7
Brownsville PUB	Southside WWTF	7.8	7.1	8.2	9.8	11.4	13.1	14.9	16.7
Edinburg	WWTP	10	6.2	7.5	9.3	11.1	12.9	14.8	16.5
Harlingen	WWTP #1*	NA	5.0	5.4	5.9	6.4	6.9	7.4	8.0
Harlingen	WWTP #2	6	5.3	5.7	6.3	6.8	7.4	8.0	8.6
Mercedes	WWTP	5	2.9	3.7	4.6	5.5	6.3	7.2	8.1
Mission	WWTP	9	6.8	8.2	10.2	12.2	14.2	16.1	18.1
Pharr	WWTP	8	5.2	5.1	6.6	8.1	9.7	11.2	12.7
Weslaco	North WWTP	5.5	3.1	3.6	4.3	5.0	5.7	6.5	7.1
Weslaco	South WWTP*	NA	0.92	1.0	1.0	1.1	1.2	1.2	1.3

ENTITY	WWTP	WWTP CAPACITY (MGD)	WWTP AVERAGE EFFLUENT FLOW (MGD)	WWTP AVERAGE EFFLUENT FLOW PER DECADE (MGD)					
				2020	2030	2040	2050	2060	2070
McAllen	South WWTP	10	6.2	5.5	7.3	9.1	10.9	12.7	14.5
McAllen	North WWTP	8	5.7	6.9	8.5	10.2	11.9	13.5	15.2
Donna	WWTP*	NA	1.3	1.5	1.9	2.2	2.6	3.0	3.3
San Juan	WWTP*	NA	1.1	1.3	1.6	1.9	2.2	2.5	2.8
Alamo	WWTP*	NA	0.7	0.8	1.0	1.2	1.5	1.7	1.9
Hidalgo	WWTP*	NA	1.0	1.1	1.4	1.6	1.9	2.1	2.4
La Feria	WWTP*	NA	0.6	0.7	0.8	0.9	1.0	1.1	1.3
San Benito	WWTP	3.75	2.2	2.2	2.6	2.9	3.3	3.8	4.2
	TOTAL	83.05	68.02	70.2	86.4	102.4	119.1	135.7	152.4

*WWTP data not provided. Values calculated by using average effluent to population ratio and multiplying by current city population

The benefit of using a collection system would be the ability to provide a much larger volume of potable reuse water than the previous strategy. Since the WTP is being built and expanded as a part of the plan for the valley, the additional reuse water can be accounted for in its planned expansions and the treatment necessary would only have to be done at one location rather than at each city's WTP. This method was selected for further evaluation due to these advantages. Conversely, the capital cost of building the collection line and having to expand it would be very large as well.

11.6 REGIONAL DIRECT POTABLE REUSE INFRASTRUCTURE

11.6.1 WWTP Effluent Collection Pipeline

In order to start creating the route for the effluent collection line, decisions need to be made on which WWTPs need to be used and which decade they need to be added in. The addition of reuse begins in 2040. Table 11-9 shows the amount of DPR water that is needed each decade and effluent collected to meet that need.

Table 11-9 WWTP Effluent in the Lower Rio Grande Valley

DPR Collection	2040	2050	2060	2070
Target DPR produced for Raw Water Blending (AFY)	17,100	38,400	47,700	57,300
WTP Effluent Required with 85% recovery (AFY)	20,100	45,100	56,100	67,500
DPR Contribution to Municipal Supply after SWTP efficiency of 95% (AFY)	16,300	36,500	45,300	54,500

In order to avoid installing the entire piping infrastructure by 2040, the collection of the effluent has been phased by decade. Moving from the SWTP along the proposed route, pumping stations and force mains are installed at various WWTPs to connect to the conveyance line. In addition, only half of the average daily flow is assumed available to account for the variation of effluent flow from the plant. Table 11-10 shows the available effluent each decade that can be collected from each different WWTP and the amount of effluent that is collected each decade in order to meet the DPR requirement.

Table 11-10 WWTP Effluent Collected by Decade

ENTITY	WWTP	AVAILABLE WWTP EFFLUENT (MGD)						COLLECTED WWTP EFFLUENT (MGD)					
		2020	2030	2040	2050	2060	2070	2020	2030	2040	2050	2060	2070
Brownsville PUB	Robindale WWTP	0.9	1.6	2.4	3.2	4.0	4.9	0.0	0.0	0.0	0.0	0.0	0.0
Brownsville PUB	Southside WWTP	4.1	4.9	5.7	6.6	7.4	8.3	0.0	0.0	0.0	0.0	0.0	0.0
Edinburg	WWTP	3.7	4.7	5.6	6.5	7.4	8.3	0.0	0.0	0.0	6.5	7.4	8.3
Harlingen	WWTP #1*	2.7	2.9	3.2	3.4	3.7	4.0	0.0	0.0	0.0	0.0	3.7	4.0
Harlingen	WWTP #2	2.9	3.1	3.4	3.7	4.0	4.3	0.0	0.0	0.0	0.0	0.0	4.3
Mercedes	WWTP	1.8	2.3	2.7	3.2	3.6	4.1	0.0	0.0	0.0	3.2	3.6	4.1
Mission	WWTP	4.1	5.1	6.1	7.1	8.1	9.0	0.0	0.0	6.1	7.1	8.1	9.0
Pharr	WWTP	2.5	3.3	4.1	4.8	5.6	6.3	0.0	0.0	4.1	4.8	5.6	6.3
Weslaco	North WWTP	1.8	2.2	2.5	2.9	3.2	3.6	0.0	0.0	0.0	2.9	3.2	3.6
Weslaco	South WWTP*	0.5	0.5	0.5	0.6	0.6	0.6	0.0	0.0	0.0	0.0	0.0	0.6
McAllen	South WWTP	2.7	3.6	4.5	5.5	6.4	7.2	0.0	0.0	4.5	5.5	6.4	7.2
McAllen	North WWTP	3.4	4.3	5.1	5.9	6.8	7.6	0.0	0.0	5.1	5.9	6.8	7.6
Donna	WWTP*	0.8	0.9	1.1	1.3	1.5	1.7	0.0	0.0	0.0	1.3	1.5	1.7
San Juan	WWTP*	0.6	0.8	0.9	1.1	1.2	1.4	0.0	0.0	0.0	1.1	1.2	1.4
Alamo	WWTP*	0.4	0.5	0.6	0.7	0.8	0.9	0.0	0.0	0.0	0.7	0.8	0.9
Hidalgo	WWTP*	0.5	0.7	0.8	0.9	1.1	1.2	0.0	0.0	0.0	0.9	1.1	1.2
La Feria	WWTP*	0.3	0.4	0.4	0.5	0.6	0.6	0.0	0.0	0.0	0.5	0.6	0.6
San Benito	WWTP	1.1	1.3	1.5	1.7	1.9	2.1	0.0	0.0	0.0	0.0	0.0	0.0
	TOTAL	35.1	43.1	51.2	59.5	67.9	76.1	0.0	0.0	19.8	40.4	49.9	60.8

From Table 11-11, a phasing plan can be derived to show when different sections of the effluent collection pipeline need to be installed. The size of each section of the collection pipe can be determined by using the flow through each section along the pipeline and using 4 feet per second as a maximum velocity for the pipeline. Table 9 conveys the size of each section of pipe, what decade it is installed in, and if the pipe is twinned or not. Figure 11-6 gives a map of the pipeline as well.

Table 11-11 WWTP Effluent Collected by Decade

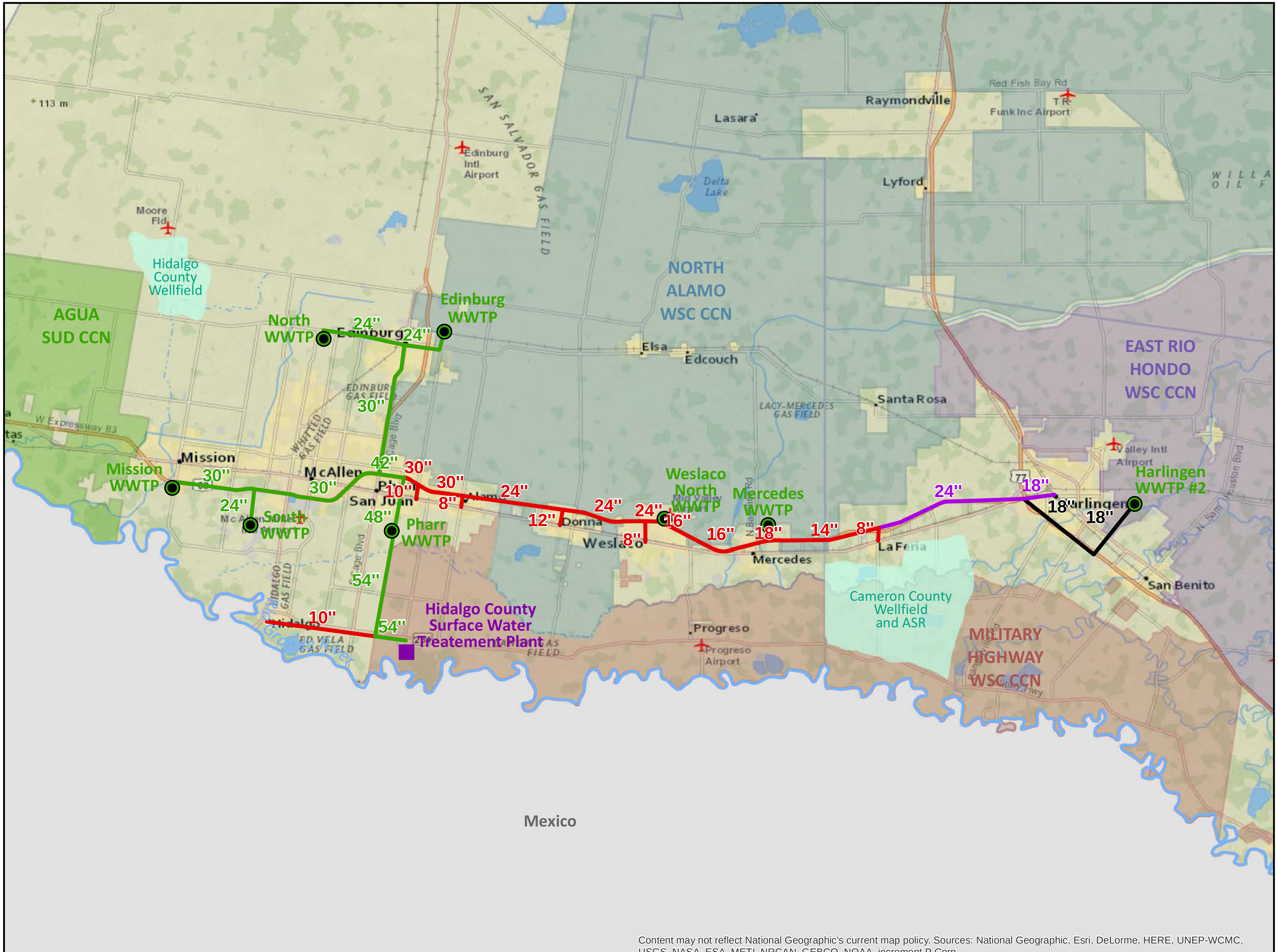
ENTITY	WWTP	LENGTH (FT)	TOTAL PIPE INSTALLED BY DECADE							
			2040		2050		2060		2070	
			PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE
Edinburg	WWTP	13,500	0	0	24	1	24	1	24	1
Harlingen	WWTP #1*	7,500	0	0	0	0	18	1	18	1
Harlingen	WWTP #2	17,800	0	0	0	0	0	0	18	1
Mercedes	WWTP	4,500	0	0	18	1	18	1	18	1
Mission	WWTP	21,500	30	1	30	1	30	1	30	1
Pharr	WWTP	15,000	24	1	24	1	24	1	24	1
Weslaco	North WWTP	2,000	0	0	16	1	16	1	16	1
Weslaco	South WWTP*	16,600	0	0	0	0	0	0	8	1
McAllen	South WWTP	23,000	24	1	24	1	24	1	24	1
McAllen	North WWTP	21,000	24	1	24	1	24	1	24	1
Donna	WWTP*	10,000	0	0	12	1	12	1	12	1
San Juan	WWTP*	14,000	0	0	10	1	10	1	10	1
Alamo	WWTP*	12,000	0	0	8	1	8	1	8	1
Hidalgo	WWTP*	5,000	0	0	10	1	10	1	10	1

ENTITY	WWTP	LENGTH (FT)	TOTAL PIPE INSTALLED BY DECADE							
			2040		2050		2060		2070	
			PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE
La Feria	WWTP*	5,900	0	0	8	1	8	1	8	1
San Benito	WWTP	18,000	0	0	0	0	0	0	0	0
Segment 1	Robin + South	99,000	0	0	0	0	0	0	0	0
Segment 2	Seg 1+San Benito	24,700	0	0	0	0	0	0	0	0
Segment 3	Seg 2 + Har #2	19,900	0	0	0	0	0	0	18	1
Segment 4	Seg 3 + Har #1	41,000	0	0	0	0	24	1	24	1
Segment 5	Seg 4 + La Feria	29,000	0	0	14	1	14	2	14	2
Segment 6	Seg 5 + Mercedes	35,800	0	0	16	1	16	2	16	2
Segment 7	Seg 6 + Weslaco	31,500	0	0	24	1	24	2	24	2
Segment 8	Seg 7 + Donna	26,500	0	0	24	1	24	2	24	2
Segment 9	Seg 8 + Alamo	12,000	0	0	30	1	30	2	30	2
Segment 10	Seg 9 + San Juan	12,700	0	0	30	1	30	2	30	2
Segment 11	Seg 13 + Pharr	5,280	48	1	48	1	48	2	48	2
Segment 12	North +Edinburg	35,000	30	1	30	1	30	2	30	2

ENTITY	WWTP	LENGTH (FT)	TOTAL PIPE INSTALLED BY DECADE							
			2040		2050		2060		2070	
			PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE	PIPE SIZE (IN)	PIPE
Segment 13	Seg 14 + Seg 12	30,000	42	1	42	1	42	2	42	2
Segment 14	Mission + South	10,800	30	1	30	1	30	2	30	2
Segment 15	Seg 11 + Seg 10	34,000	54	1	54	1	54	2	54	2
Segment 16	Seg 15 + Hildago	3,400	54	1	54	1	54	2	54	2

11.6.2 WWTP Effluent Collection Pumping

In order to convey all of the effluent to the centralized SWTP, pump stations will be installed at each separate WWTP. These pump stations will be sized individually based on the quantity of effluent that needs to be conveyed at the needed velocity and also on the amount of headloss that has to be overcome. For this report, pumping stations were not phased to match potential flows from each entity. Each pumping station would require hydraulic calculations and storage calculations based on the WWTP elevations and effluent flows in the final design process. For costing purposes, the average head and flows were calculated at build out (2070) and pumping stations were given a unit cost based on the HP required to supply the maximum design capacity. The pumping station costs were allocated to the decade that the pump station is to be built.



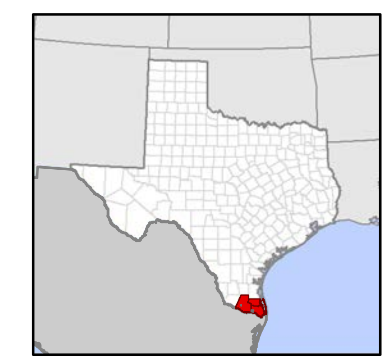
Legend

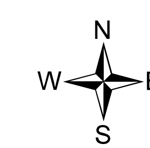
Water Sources
- Reuse Water
- Sea Water

Facilities
- Surface WTP
- Surface Water Source

Reuse Projects Routing Plan
- 2020 Project
- 2030 Project
- 2040 Project
- 2050 Project
- 2060 Project
- 2070 Project
- Wellfields

Water Utility Service Areas
- Agua SUD
- East Rio Hondo WSC
- Military Highway WSC
- North Alamo WSC





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Miles
1 inch = 4.5 miles

Lower Rio Grande Valley Supply Project

Reuse Pipeline Routing

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11.6.3 Treatment

A conceptual design of the base case treatment approach (MF/UF + RO + AOP) is described in this section of the report. Since the first phase of implementing DPR is currently not planned until the year 2040, and since treatment for potable reuse is an on-going, developing field, aspects of the treatment described herein may be different two decades in the future. The conceptual design presented herein is based on currently practiced treatment methods to facilitate planning and associated activities, such as the development of cost opinions and considerations for land acquisition. A summary of the conceptual design is presented below with additional details listed in Table 11-12.

To control biological growth and what is called 'biofouling' within the MF/UF and RO membrane processes, the source water would be disinfected with chloramines and a residual would be maintained through the membrane systems. A free chlorine residual cannot be practiced, since the currently used RO membranes cannot tolerate chlorine. While it may be more detail than needed for a planning-level study, care is advised in how the chloramine residual is added to the water. Currently the best method is to form the chloramine in a clean water side-stream, rather than the main process stream to minimize formation of unwanted disinfection byproducts, such as nitrosodimethylamine (NDMA).

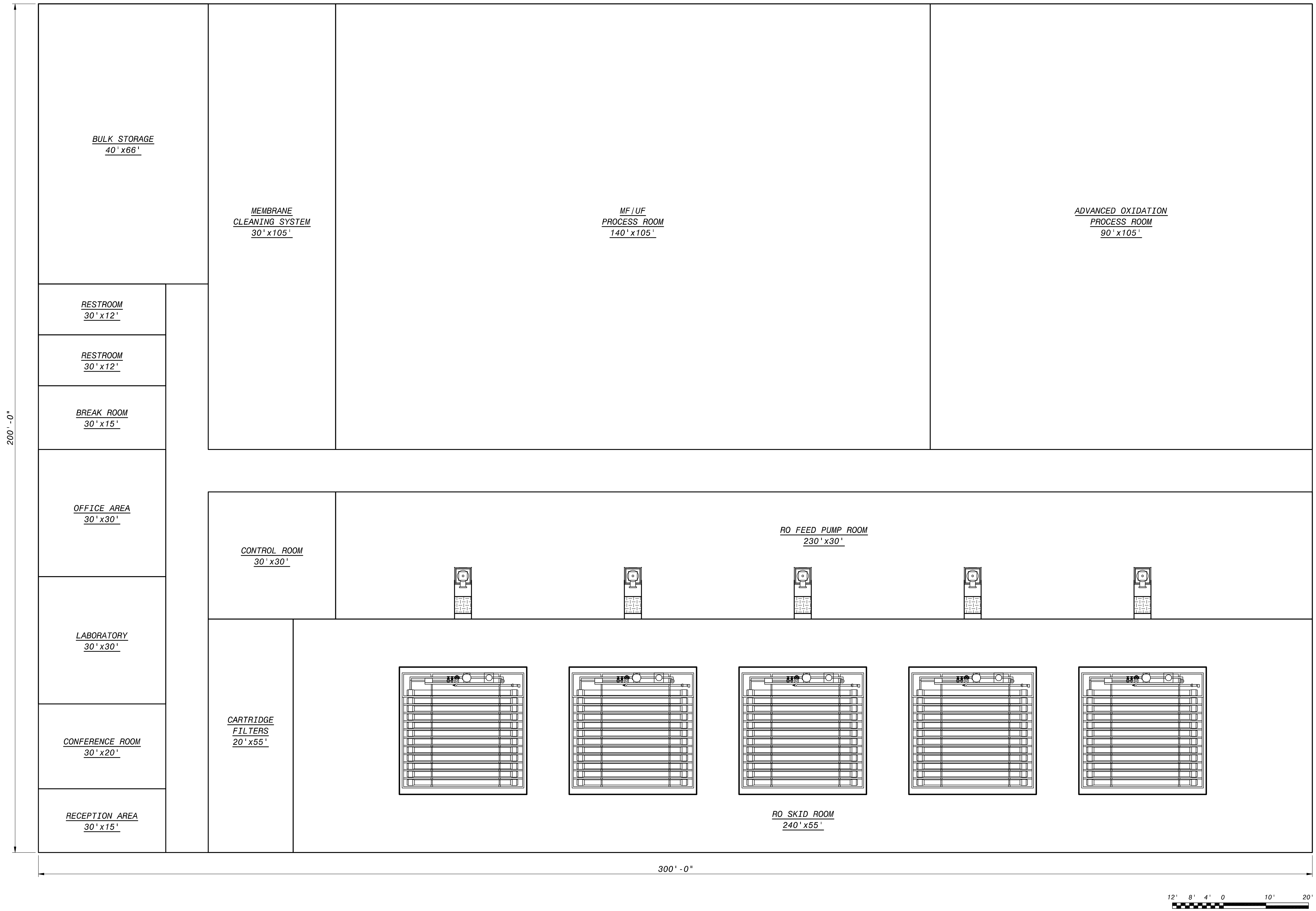
After chloramination, the source water, which is WWTP effluent, would be stored in an equalization tank/basin to allow for variation in the flow rates from the WWTPs while operating the AWT facility independently. A pump station would transport water from the equalization tank/basin through the MF/UF membrane filtration units, including associated process steps, such as the protective prescreens/strainers. Two streams leave the MF/UF units: a portion of the flow becomes spent backwash and cleaning wastes while most of the flow becomes MF/UF filtrate. The filtrate is stored in an intermediate storage tank to allow flexibility in plant operations, and the backwash and cleaning wastes are combined with the RO Concentrate and discharged to the Arroyo Colorado.

A low pressure pump station transports MF/UF filtrate through the cartridge filters and to the suction side of the RO feed pumps, which are also called High Pressure Pumps (HPPs). The HPPs provide the pressure required by the RO process, which is a cross-flow filtration process that yields two effluent streams. Most of the RO feed water becomes the purified water stream, which is called permeate. The rejected constituents are concentrated into the RO concentrate stream.

The permeate would be further treated by advanced oxidation. This process injects hydrogen peroxide (H_2O_2) into the permeate stream and then applies UV light at specific levels to create strong oxidants (hydroxyl radicals) to react with contaminants in the product stream.

If needed, caustic or lime or other chemicals are added to reduce the treated water corrosivity. To reduce the amount of lime or caustic, a degassifier may be used to strip some of the carbon dioxide from the water. The product water would be blended with surface water and subsequently further treated by the surface water treatment plant. Further analysis of treatment process will be necessary to determine if raw water blending will be sufficient to reduce water corrosivity.

Figure 11-7 depicts the building layout for the treatment facilities to be located on the proposed regional SWTP site.



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The DPR AWT Treatment Plant will be divided into four phases. Phase I will be built in 2040 and will have a raw feed design flow of 26 MGD. Phase II is built the following decade with same capacity. Phases III and IV are planned for 13 MGD each in 2060 and 2070. Table 11-12 describes the required equipment for the 26 MGD treatment trains in 2040 and 2050. Later phases will be proportionately adjusted based on the feed flows.

Table 11-12 DPR Advanced Water Treatment Conceptual Design for Phase I

PARAMETER	UNITS	PHASE I VALUES / DESCRIPTION
Equalization Tank		
Volume	Gallons	2,600,000
MF/UF Feed Pumps		
Total Design Flow	MGD (gpm)	26 (18,000)
Number of Pumps	No.	5 Duty + 1 Spare (3600 gpm each, nominal discharge 50 psi)
MF/UF Units		
Total Design Flow Influent Filtrate Wastewater	MGD (gpm)	26 (18,000) 24.5 (17,000) 1.5 (1040)
Number of Units (This can vary depending on the supplier)	No.	10 Duty + 1 Spare
Hydraulic Break Tank		
Usable Volume	Gallons	400,000
Low Pressure Pumps		
Total Design Flow	MGD (gpm)	24.5 (17,000)
Number of Pumps	No.	5 Duty + 1 Spare (3400 gpm each, nominal discharge 40 psi)
RO Units		
Total Design Flow Influent Permeate Concentrate	MGD (gpm)	24.5 (17,000) 20.8 (14,450) 3.7 (2250)

PARAMETER	UNITS	PHASE I VALUES / DESCRIPTION
Number of Units	No.	5 Duty + 1 Spare
AOP Units (UV w/H2O2 injection)		
Total Design Flow	MGD (gpm)	20.8 (14,450)
Number of Units	No.	5 Duty + 1 Spare

11.7 COST OPINIONS

Planning level Engineers Opinion of Probable Costs (EOPCs) were developed for the centralized direct potable reuse system utilizing previous projects of similar size and with similar treatment processes.

11.7.1 Description and Methodology

Standard procedures were used to estimate cost on a cost per unit basis. Previous project experience was utilized in obtaining and verifying costs included in the estimates. Costs shown in the report are in 2015 dollars. For future projections, the Construction Cost Index as reported by Engineering Review in November 2015 was 10092.

11.7.2 Professional Services

Estimates for Pre-Design Phase, Design and Construction Phase, Program Management and Construction Management, and Permitting costs were combined into a professional services category and were calculated to total 25 percent of the infrastructure cost. This is in line with standard estimating procedures of a cost estimate at this level.

11.7.3 Wastewater Effluent Collection

Wastewater Effluent Collection includes the EOPC for the pipelines and pumping stations required to convey the treated wastewater to the treatment facility located at the proposed regional surface water plant. These costs are estimated in the Table 11-13 below.

Table 11-13 Wastewater Effluent Collection Cost

	2040	2050	2060	2070
Pumping Stations	$ 3,250,000	$ 6,501,000	$ 813,000	$ 1,625,000
Collection Pipe (6 inch to 54 inch)	$ 51,319,000	$ 31,479,000	$ 69,094,000	$ 5,040,000
Contractor Markup (10%) Includes Bonding and Insurance	$ 5,457,000	$ 3,798,000	$ 6,991,000	$ 667,000
Total Collection Costs	**$ 60,026,000**	**$ 41,778,000**	**$ 76,898,000**	**$ 7,332,000**

Prices per linear foot were developed for the pipelines and multiplied by the length of each pipeline. The unit costs were based on the pipeline diameter and incorporate costs for installation, fittings, trench excavation and safety protection, erosion and sedimentation controls, hydrostatic testing, restoration, and other items typically required to install a transmission main. These unit prices were developed from similar projects.

11.7.4 Treatment Facilities

In order to estimate the total cost for AWT plant, the costs were broken out into costs associated with the building and process equipment. All treatment costs are summarized in Table 11-14. The costs for the buildings are based on unit prices per square foot obtained from previous projects. Process and storage costs were developed by comparing flows of previous projects and utilizing the ratios for each process stream. The ratios were tempered from a linear characterization by a modularity exponent. It was assumed that traditional treatment processes were less modular in nature and therefore less linear for cost escalation. Contrarily, RO process equipment scales almost linearly.

Table 11-14 AWT Treatment Facility Costs

AWT TREATMENT FACILITY (20.8 MGD)				
ITEMS	**PHASE I 20.8 MGD**	**PHASE II 20.8 MGD**	**PHASE III 10.4 MGD**	**PHASE IV 10.4 MGD**
Equalization Tank	$14,300,000.00	$14,300,000	$7,150,000	$7,150,000
MF/UF Feed Pumps	$3,664,000	$3,664,000	$1,832,000	$1,832,000
MF/UF Units	$14,333,000	$14,333,000	$7,167,000	$7,167,000
Hydraulic Break Tank	$2,200,000	$2,200,000	$1,100,000	$1,100,000
Transfer Pumps	$3,664,000	$3,664,000	$1,832,000	$1,832,000
Process Building	$6,300,000	$-	$-	$-
RO Process Equipment	$17,100,000	$17,100,000	$8,550,000	$8,550,000
AOP	$10,816,000	$10,816,000	$5,408,000	$5,408,000
Miscellaneous Equipment	$5,808,000	$5,808,000	$2,904,000	$2,904,000
SUBTOTAL	$78,185,000	$71,885,000	$35,943,000	$35,943,000
Mobilization (3%)	$2,346,000	$2,157,000	$1,078,000	$1,078,000
Yard Piping (5%)	$3,909,000	$3,594,000	$1,797,000	$1,797,000
Sitework (10%)	$7,819,000	$7,819,000	$7,819,000	$7,819,000
Electrical and I&C (10%)	$7,819,000	$7,819,000	$7,819,000	$7,819,000
SUBTOTAL	$100,078,000	$93,274,000	$54,456,000	$54,456,000
Contractor Markup @ 10% (Including Insurance/Bond)	$10,008,000	$9,327,000	$5,446,000	$5,446,000
TOTAL	**$110,110,000**	**$110,110,000**	**$55,055,000**	**$55,055,000**

11.7.5 Concentrate Disposal

Pipeline costs were developed similar to the pipeline for raw water conveyance and are shown in Table 11-15. A price per linear foot based on the SAWS BGD 90% EOPCC was developed and multiplied by the required quantity. Lengths for the discharge lines were routed along existing roadways and were estimated to be approximately 5 miles for the Hidalgo Plant and 2 miles for the Cameron Plant.

Table 11-15 Concentrate Disposal Capital Cost

DISPOSAL FACILITIES	TOTAL
Concentrate Pipe (2-miles of 24 inch HDPE)	$ 4,819,000
Contractor Markup (including Insurance/Bond)	$ 482,000
Total Concentrate Disposal Costs	**$ 5,301,000**

11.7.6 Land Acquisition

Easement acquisition costs were estimated based on area required to construct and maintain the collection and concentrate conveyance pipelines. It is assumed that the effluent collection pipeline will parallel the regional transmission line were possible and utilize the proposed easements obtained during the early phases of that project. Property acquisition costs were calculated based on estimated area needed for the treatment facility that is to be collocated at the SWTP, It is estimated that the AWT facility will require an additional 10 acres of land for the building and equalization basin. A unit cost of $4,500/Acre was used for easements and $5,000 for property acquisition and multiplied by the area required for easements and property. Estimated costs are shown in Table 11-16.

Table 11-16 Land Acquisition Costs for DPR System

PROPERTY	UNIT	QUANTITIES	UNIT COST	TOTAL COST
Easements	**ROW Width***	**Length (LF)**	**($/Acre)**	
WWTP Effluent Conveyance Pipeline	50 feet	189,300	$4,500	$ 978,000
Concentrate Disposal Pipeline	25 feet	26,400	$4,500	$ 68,000
Purchase	**Width**	**Length**	**($/Acre)**	
Treatment Facility	900	500	$5,000	$ 51,000
			Total	**$ 1,098,000**

*ROW = Right-of-Way.

11.7.7 Electrical Infrastructure Allowance

The proposed treatment facilities have a large demand on the existing electrical infrastructure and will likely require extensive augmentation near the treatment facilities. Without an evaluation of the existing facilities, an allowance for budgetary purposes is suggested based on similar projects with similar electrical loads in remote locations. This allowance is shown in the cost summary table below.

11.7.8 Operations

Operations and maintenance costs for the collection and treatment facilities were estimated using typical costs for similar applications. The electrical usage, staffing requirements, chemical dosage, and miscellaneous consumables were projected, and approximate costs associated were calculated.

Annual electrical estimates were determined for the conveyance pumping stations, low head feed pumping stations for the MF/UF and RO skids, the high pressure booster and intermediate booster pumps utilized in the RO process and the UV consumption in the AOP equipment. $0.05/KWh was used as the unit cost for electricity for both the collection system and treatment facilities. This cost is based on input from project stakeholders.

Staffing projections were made utilizing the staffing estimates for a similar size plant. Reasonable approximations were used to estimate the amount of staff that would be assigned to each facility. It was assumed that the operators, the plant manager, maintenance mechanics and I&C staff would support the operations. Typical hourly wages for personal at the managerial and various staff levels were used, and a 10% percent annual overtime amount and 40% burden rate were taken into account. Refer to Table 11-17 for collection and treatment operations and maintenance costs.

Chemical consumption was projected using information from similar AWT facilities. Typical dosages and concentrations were applied to the treatment plant flow rates in order to calculate the annual usage of each chemical. Annual chemical consumption was multiplied by prices obtained from vendors and other similar projects to determine the total cost per year.

Additional operations and maintenance costs were estimated such as replacement equipment for the RO treatment equipment and consumables which include other miscellaneous needs of the facilities. Approximate costs for the RO treatment replacement equipment are annual costs for replacing RO and MF/UF membranes, cartridge filters, pumps, and valves. Even though individual replacement rates vary, this is an estimate of the annual cost to replace each of them at their respective end of life. The costs and life expectancy were based on current knowledge of the facilities.

Table 11-17 Collection and Treatment O&M Costs

CATEGORY	2040	2050 (ADDITIONAL)	2060 (ADDITIONAL)	2070 (ADDITIONAL)
Energy	$ 1,794,000	$ 1,794,000	$ 897,000	$ 897,000
Replacement Equipment	$ 567,000	$ 567,000	$ 284,000	$ 284,000
Chemicals	$ 763,000	$ 763,000	$ 382,000	$ 382,000
Staffing	$ 665,000	$ 665,000	$ 333,000	$ 333,000
AOP O&M	$ 876,000	$ 876,000	$ 438,000	$ 438,000
Subtotal	**$ 4,665,000**	**$ 4,665,000**	**$ 2,334,000**	**$ 2,334,000**
Contingency (25%)	$ 1,166,000	$ 1,166,000	$ 584,000	$ 584,000
Total	**$5,831,000**	**$5,831,000**	**$2,918,000**	**$2,918,000**

11.7.9 Cost Summary

The final costs for the DPR collection and treatment are summarized as shown in Table 11-18.

Table 11-18 DPR Collection and Treatment Cost Summary

ITEM	2040 (20.8 MGD)	2050 (20.8 MGD)	2060 (10.4 MGD)	2070 (10.4 MGD)
Construction Costs				
Collection Lines	$56,451,000	$34,627,000	$76,003,000	$5,544,000
Pumping Stations	$3,575,000	$7,151,000	$894,000	$1,788,000
Treatment Facilities	$110,110,000	$103,810,000	$51,905,000	$51,905,000
Concentrate Discharge	$5,301,000	$-	$-	$-
Land Acquisition	$1,098,000	$-	$-	$-
Contingency (25%)	$44,133,750	$36,397,000	$32,200,500	$14,809,250
Total Construction Cost	$220,668,750	$181,985,000	$161,002,500	$74,046,250
Engineering Cost (25%)	$55,167,188	$45,496,250	$40,250,625	$18,511,563
Electrical Infrastructure Allowance	$10,531,000	$-	$-	$-
Total Capital Costs	**$286,366,938**	**$227,481,250**	**$201,253,125**	**$92,557,813**
Annual O&M Costs (Additional)	**$5,831,000**	**$5,831,000**	**$2,918,000**	**$2,918,000**

The AWT effluent is required to be treatment at the SWTP prior to entering the distribution line. SWTP infrastructure expansions required to incorporate the produced water were sized and estimated previously. Capital costs for the necessary SWTP expansions are calculated to provide an overall cost of for all infrastructure necessary. For the purpose of this evaluation, the infrastructure and O&M costs were calculated using the ratio of the flow to the total costs. The total DPR and O&M costs are summarized in Table 11-19.

Table 11-19 DPR Cost Summary with SWTP Cost

ITEM	2040 (20.8 MGD)	2050 (20.8 MGD)	2060 (10.4 MGD)	2070 (10.4 MGD)
Capital Cost				
Collection and AWT	$286,367,000	$227,481,000	$201,253,000	$92,558,000
SWTP Expansion (1:1 flow ratio)	$62,920,000	$58,240,000	$32,240,000	$37,440,000
Total	**$349,287,000**	**$285,721,000**	**$233,493,000**	**$129,998,000**
Annual O&M Cost				
Collection and AWT	$5,831,000	$11,662,000	$14,580,000	$17,498,000
SWTP Expansion	$1,664,000	$2,723,000	$3,269,000	$4,095,000
Total	**$7,495,000**	**$14,385,000**	**$17,849,000**	**$21,593,000**

CHAPTER 12 AQUIFER STORAGE AND RECOVERY

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

25 FEBRUARY 2016

BLACK & VEATCH
Building a world of difference.

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12.0 Aquifer Storage and Recovery

12.1 PURPOSE

With growing population and the associated increase in water demands, it is critical for utilities to manage their water resources. The highest demands often occur during times with the lowest water availability. Storing excess water when it is available for seasonal needs or long-term drought situations would help manage these peak demands. Although surface water is typically stored in surface reservoirs, it may also be possible to store excess surface water below ground in an aquifer. Aquifer Storage and Recovery (ASR) may be an effective water management technique for storing excess flows in Rio Grande during wet periods, and recovered during dry periods to meet the peak demands. Based on the hydrologic simulations described in Chapter – 5 Groundwater Hydrology, ASR appears to be a viable option to help meet some of water demands in the valley. The purpose of this chapter is to evaluate ASR feasibility and conceptualize ASR solutions for the study area.

12.2 POTENTIAL WATER SUPPLIES

The waters of the Rio Grande are stored in the Amistad-Falcon Reservoir System and released based on requests from downstream water users. The TCEQ Rio Grande Watermaster's Office is responsible for allocating, monitoring, and controlling the release of surface water in the Rio Grande Basin from Ft. Quitman to the Gulf of Mexico. The water utilities within LRGV region have an annual allocation of municipal priority water rights from the Texas Commission of Environmental Quality (TCEQ) which is met by water stored in the Falcon and Amistad Reservoirs. Water rights on the river are divided into two major types: Domestic, Municipal, and Industrial (DMI) rights and irrigation and mining rights (which are sub-divided into Class A and B). Because the water rights exceeds the available supply in a drought year, only the highest priority water rights receive the full amount of their allocations in a drought. The first priority goes to DMI, the second goes to a minimum volume required for reservoir operations, and the third priority goes to the irrigation and mining accounts. In drought years, irrigation and mining water right holders may not have access to all of their water rights.

To store water in ASR, a source of drinking water is required. Our assumption is that Rio Grande is that source of the water to be stored. Excess water is assumed to be available from the following pools:

- Unused DMI water rights during a wet year.
- Surplus flow from a permit similar to BPUB's 1838 permit.
- Run of river flows (or non-permit rights)

Drinking water utilities own and purchase water rights in sufficient amounts to meet their annual water demands of surface water for their system. Water rights may be owned outright, they may be leased under a long term water agreement, or they may be leased annually. Since it is virtually impossible to estimate the exact amount of water needed for a given year down to the acre-ft, every year there are unused DMI Water Rights at the end of the year that could be treated and stored. Since these are owned or leased for use by the water purveyor, there is no restriction on the use for an ASR.

In addition to their DMI rights, the Brownsville Public Utilities Board also holds Permit No. 1838 entitling it to 40,000 acre-feet of surplus water. This permit allows Brownsville to intermittently divert water when flow in the Rio Grande River is above 25 cfs. Further detail of operations, discharge and water rights are explained in the Surface Water Availability chapter. This permit allows BPUB to utilize water that would otherwise flow to the Gulf of Mexico unused. There are no constraints on the actual rate at which the water is withdrawn from the Rio Grande as long as the minimum flows are met. This type of permit is ideal as a methodology to obtain rights to capture excess flow for storage in an ASR facility. While further work with TCEQ Watermaster's Office is necessary to acquire a permit to withdraw surplus Rio Grande water, for our analysis it is assumed availability of flows above 55 cfs up to 100 cfs as measured near Brownsville could be a potential water source.

Run of the river flows occur when excess flow is in the river, usually due to mandatory releases from the reservoirs due to rain events that are filling, or overfilling the reservoirs. During these periods of time water can be withdrawn from the Rio Grande at "No Charge", meaning that the water taken does not count against their water rights. Based on information received from the TCEQ Watermaster, there have been no-charge events in 28 of the last 30 years. In the years with No Charge events, the annual diversions range from as little as 1,300 acre-ft to a maximum of 689,000 acre-ft. The actual periods of time that No Charge events occur however, is not available.

As a methodology to determine if excess surface water (from one of the three methods described above) we analyzed the river flow below Brownsville as an indicator of available water to treat and store. Historical flow data (1992-2012) in the Rio Grande recorded at IBWC station 08-4750.00 near Brownsville shows a median flow of 205 cfs and average flow of 550 cfs. From Figure 12-1 below, it can be seen that even during the drought period between 1995-2003, excess flows in the river could be diverted, and stored for recovery at a later time to meet demands during dry periods. The amount of water available from this source is variable and depends on both reservoir management and the climate. An estimated 555,000 Ac-ft of water could be diverted and stored in the aquifer over the 20 year period of assuming aquifer recharge capacity of 33,000 ac-ft/yr.

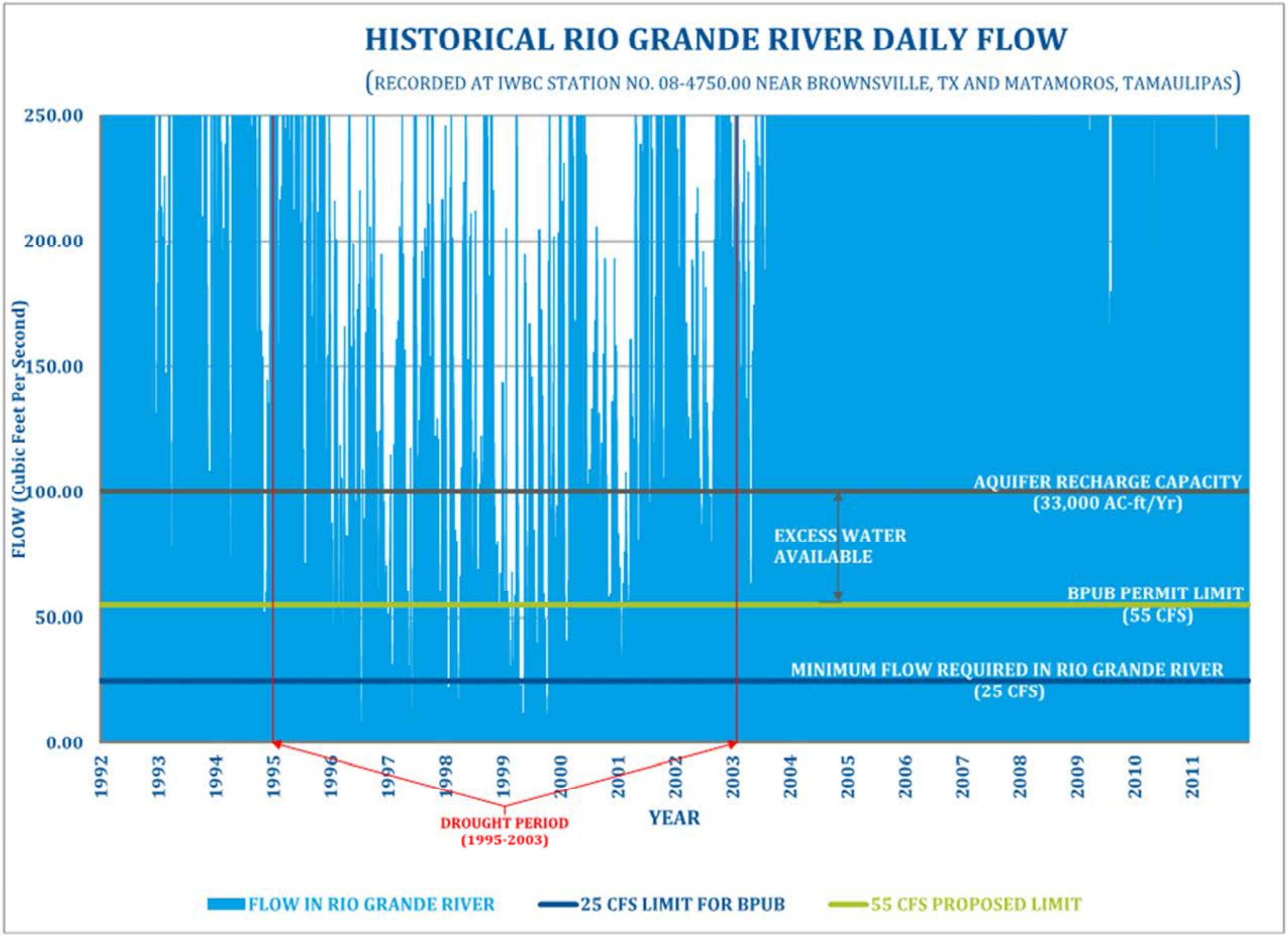

Figure 12-1 Historical Rio Grande River Daily Flow

Municipal water demands are seasonal in nature and systems often encounter a periodic increase in water demands during summer and low demands during winter months.

Figure 12-2 below shows historical monthly and annual water demands. The water demands are calculated using typical consumption data from the same time period (1992-2012). Demands were estimated by multiplying the per capita consumption data by the population in the study area. The data suggests that 80% of total water demands in the LRGV area are met by surface water because total surface water rights of 227,440 ac-ft/yr is approximately 80% of the net demand in the peak year (2011). The other 20% is met by other water sources (groundwater, leased water rights, etc...).

The water treatment plants are designed to produce water to meet the maximum day demand. During wet periods, when the demands are low the treatment plant is operated at lower rate than design capacity. If the treatment plants were operated to treat all available water rights, the excess water produced during low demands would be available for potential storage. It is estimated a total of 570,000 ac-ft of water is unused in the 20 year period. This is an average of 27,000 Ac-ft/yr of unused water rights costing over $74 million a year at current market value of $2,750/ Ac-ft.

Figure 12-3 shows the comparison between annual and average water demands. The peak demand over the last 20 years is around 106,000 Ac-ft. Since water use is variable based on rain, temperature and costumer profile, it is virtually impossible to accurately predict both available water and need for stored water in an ASR.

For the purposes to estimate cost it is assumed that 33,000 Ac-ft/Yr of ASR supply come through either full utilization of annual surface water rights or obtaining an excess flows permit to divert water during periods of higher flows.

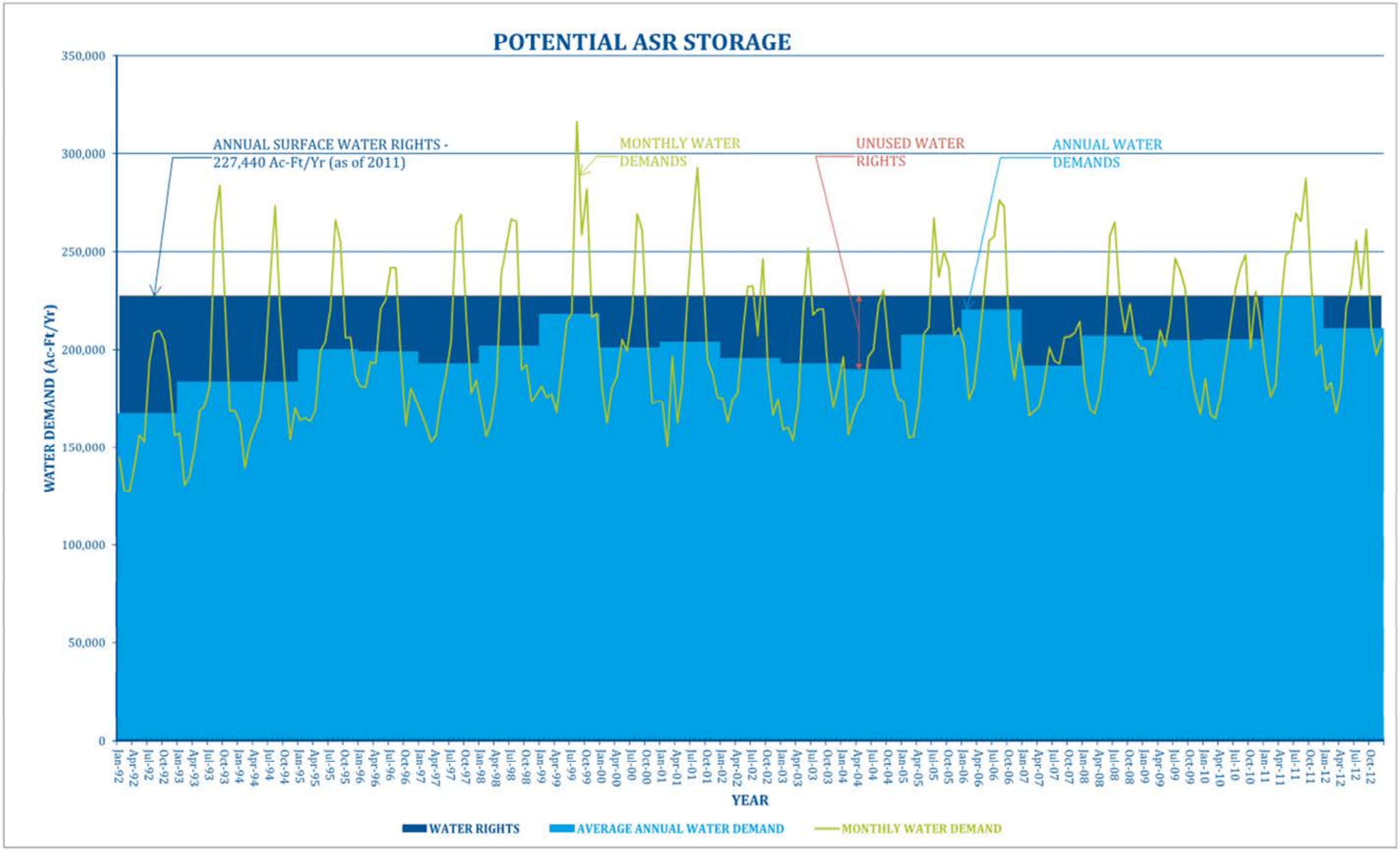

Figure 12-2 Potential ASR Storage

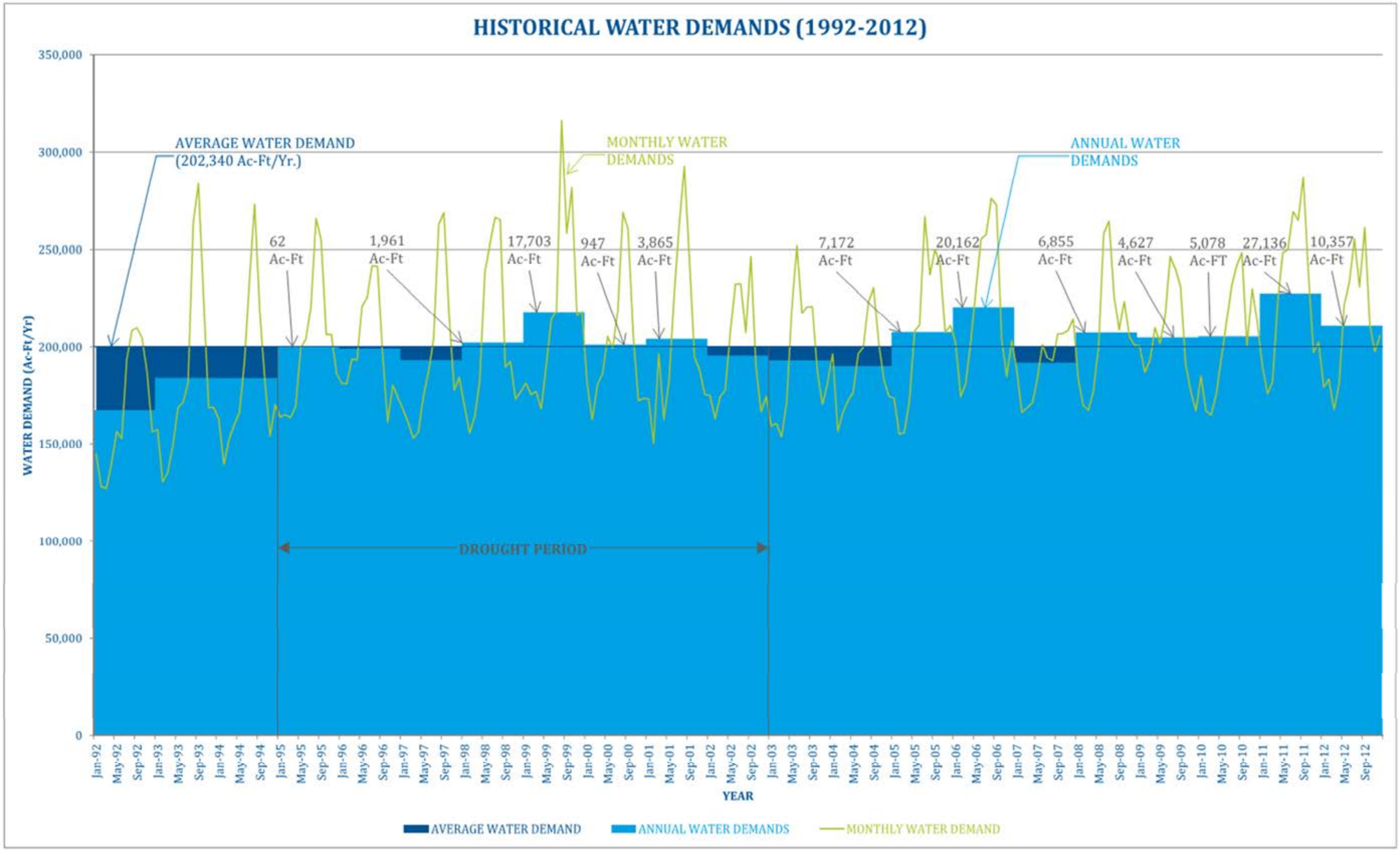

Figure 12-3 Historical Water Demands

12.3 RULES AND REGULATIONS

The Environmental Protection Agency's (EPA) Underground Injection Control program (title 40 CFR, part 144-147) outlines the minimum regulatory requirement for ASR injection wells. The rules were established under the Safe Drinking Water Act. State and local regulatory agencies also have other requirements in addition to the EPA. Provisions for aquifer storage and recovery are included in Texas Water Code (TWC) and were passed by the 74th Texas Legislature with House Bill 1989. Recent legislation (House Bill 655) passed in June of 2015 amended the Texas Water Code (TWC) chapters 11, 27 and 36 to address requirements for authorization to inject and recover water as part of an ASR project . Under the revised rules the requirements for a pilot project, followed by a final authorization, is changed to a single authorization for an ASR project. The bill (HB 655) also granted the TCEQ jurisdiction over the regulation and permitting of ASR wells requires reporting of injection and recovery volumes and water quality data to TCEQ by the project operator. This section briefly describes the statutes and rules that govern aquifer storage and recovery systems.

- Underground Injection Wells: The Texas Commission for Environmental Quality (TCEQ) Underground Injection Program administers injection of water for ASR project. ASR injection wells are classified as Class V injection wells and the authorization, construction, operations, monitoring and closure of the injection wells are regulated by TCEQ's 30 TAC Chapter 331, Subchapters H and K. Water quality requirements are described in § 331.184 and states "Water injected into an ASR injection well must be of a quality that does not result in pollution of native groundwater or an underground source of drinking water. If the injected water comes from a source other than groundwater, such as surface water or treated wastewater, the project operator must demonstrate that the water to be injected has been processed using appropriate treatment techniques to remove pathogens and other organisms that are not present in the native groundwater. Water recovered from an ASR project that is provided to a public water system is subject to all applicable requirements, maximum contaminant levels, and treatment techniques under Chapter 290 of this title (relating to Public Drinking Water)." Applications for injection well permits are regulated by TAC 30 Chapter 39, Subchapter L. The Groundwater Rule may require that the water be disinfected upon withdrawal unless the water meets the requirement for "natural disinfection", or if the system qualifies for variance.
- Water rights: TWC 30, Chapter 295, Subchapter A is responsible for regulations pertaining to the storage and recovery of appropriated water. A water right holder or an applicable water user can proceed with an aquifer storage and recovery project so long as they comply with the terms of the applicable water right and other required authorizations.

There are also other requirements specific to ASR projects within Groundwater Conservation District (GCD) boundaries. Groundwater Management Areas were created "in order to provide for the conservation, preservation, protection, recharging, and prevention of waste of the groundwater, and of groundwater reservoirs or their subdivisions, and to control subsidence caused by withdrawal of water from those groundwater reservoirs or their subdivisions, consistent with the objectives of Section 59, Article XVI, Texas Constitution, groundwater management areas may be created..." (Texas Water Code §35.001). The proposed ASR project is not located in an existing GCD; however, RGRWA has considered creating a GCD to manage groundwater in the region. Figure 12-4 shows groundwater management area 16. When located in a GCD ASR project operator need an application or notification submitted to the GCD in accordance with TAW 30, Chapter 295.21

reflecting consent to cooperate in the development of, and abidance with the rules governing the injection, storage, or retrieval of appropriated water.

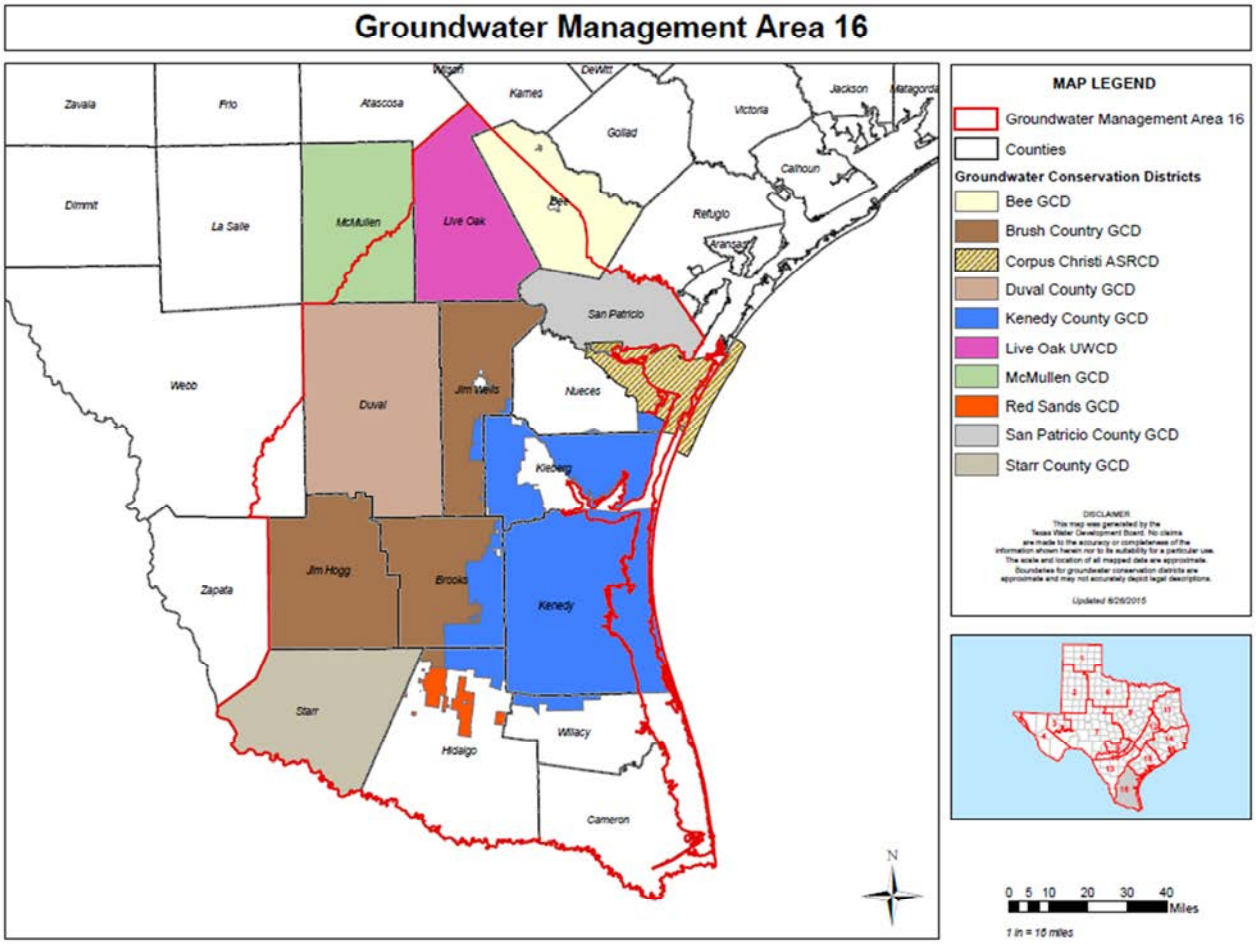

Figure 12-4 Groundwater Management Area 16 and Groundwater Conservation Districts (GCDs)

12.4 AQUIFER GEOLOGY

A general characterization of geological formations and groundwater recharge analysis is described in Chapter – 5 Groundwater Hydrology. A groundwater recharge analysis was conducted based on a recharge rate of 30 MGD (about 33,000 ac-ft/yr) of water for a five year period.

Two locations were identified for ASR Wellfields: one location is the eastern brackish aquifer well-field location, and the second location is southeast of McAllen.

Preliminary hydraulic simulations estimated a maximum water level rise after 5 years of aquifer recharge of over 30 feet near the center of the eastern well field, and about 50 feet for the recharge site southeast of McAllen. Significant water level rises attributable to the assumed aquifer recharge extend approximately 10 to 15 miles from the center of each well field. Approximately 20% of recharged water exits the aquifer through irrigation infrastructure like canal and drains. The potential drift was simulated for a period of 50 years using an effective porosity of 10 percent. Modeling indicated that there is not significant drift in the groundwater and that most of the water could be retrieved. As with an ASR system, it is recommended that a pilot well be drilled and ASR productivity and efficiency should be tested.

Based on the hydrologic simulations, the well spacing is expected to be a 1-mile radius. Well recharge capacities of approximately 500-750 gpm are anticipated. Well depths in western recharge zone are 600-800 feet below ground surface. The eastern recharge well depths are 400-500 feet as simulated. The simulation results for recharge are shown in Figure 12-5, Figure 12-6 and Figure 12-7 below.

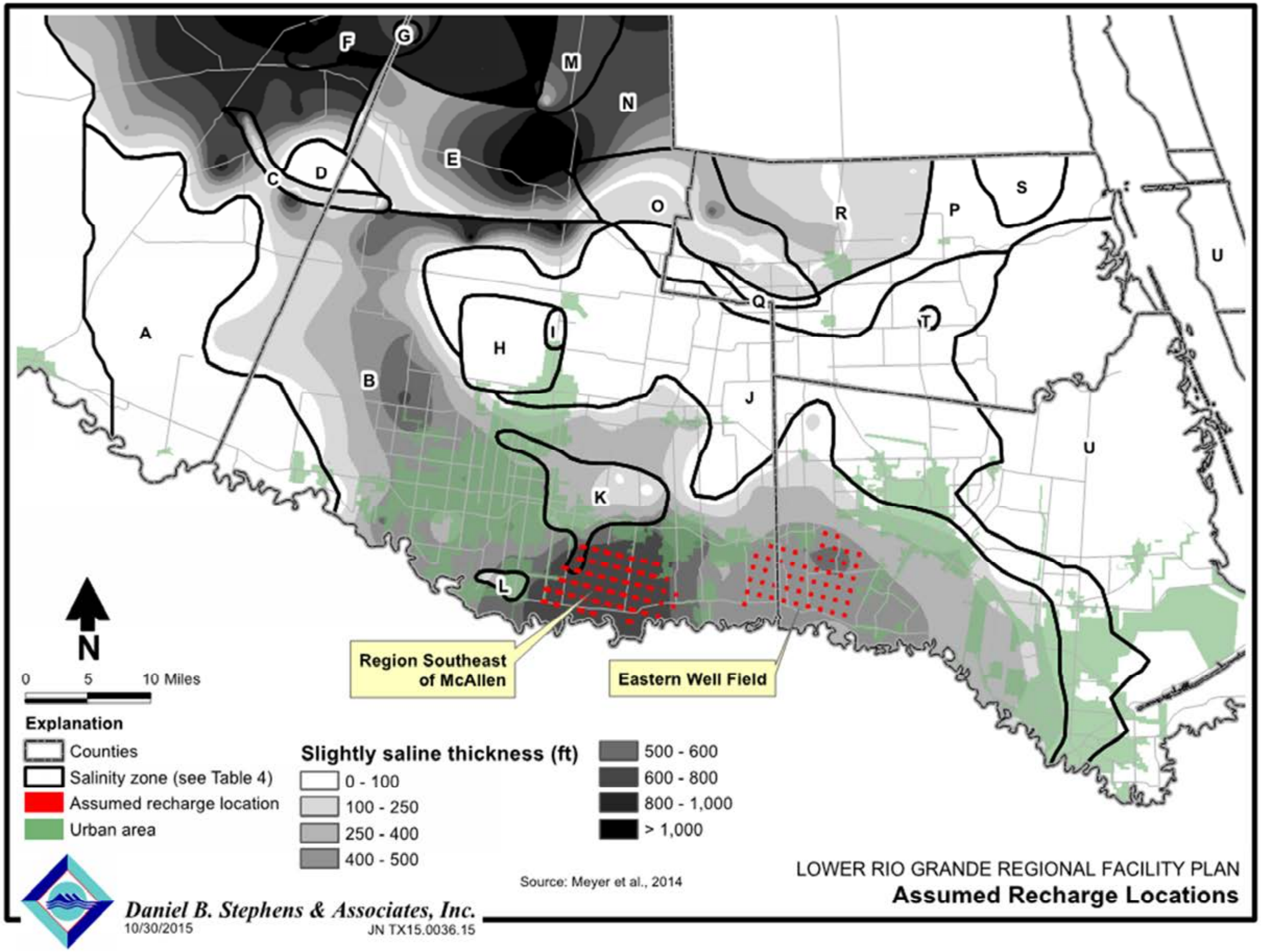

Figure 12-5 ASR Well Fields Recharge Locations

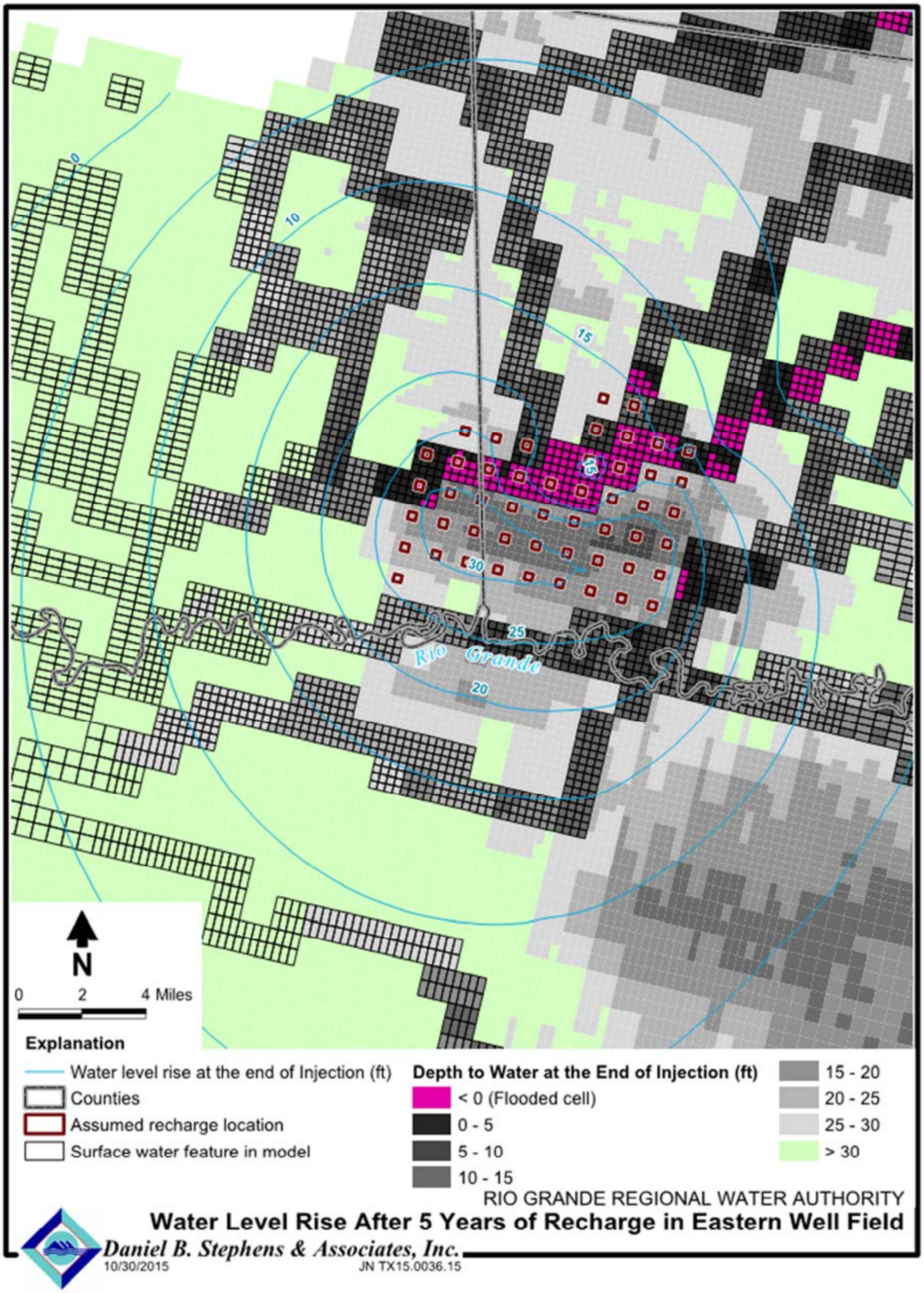

Figure 12-6 Recharge in Eastern Well Fields

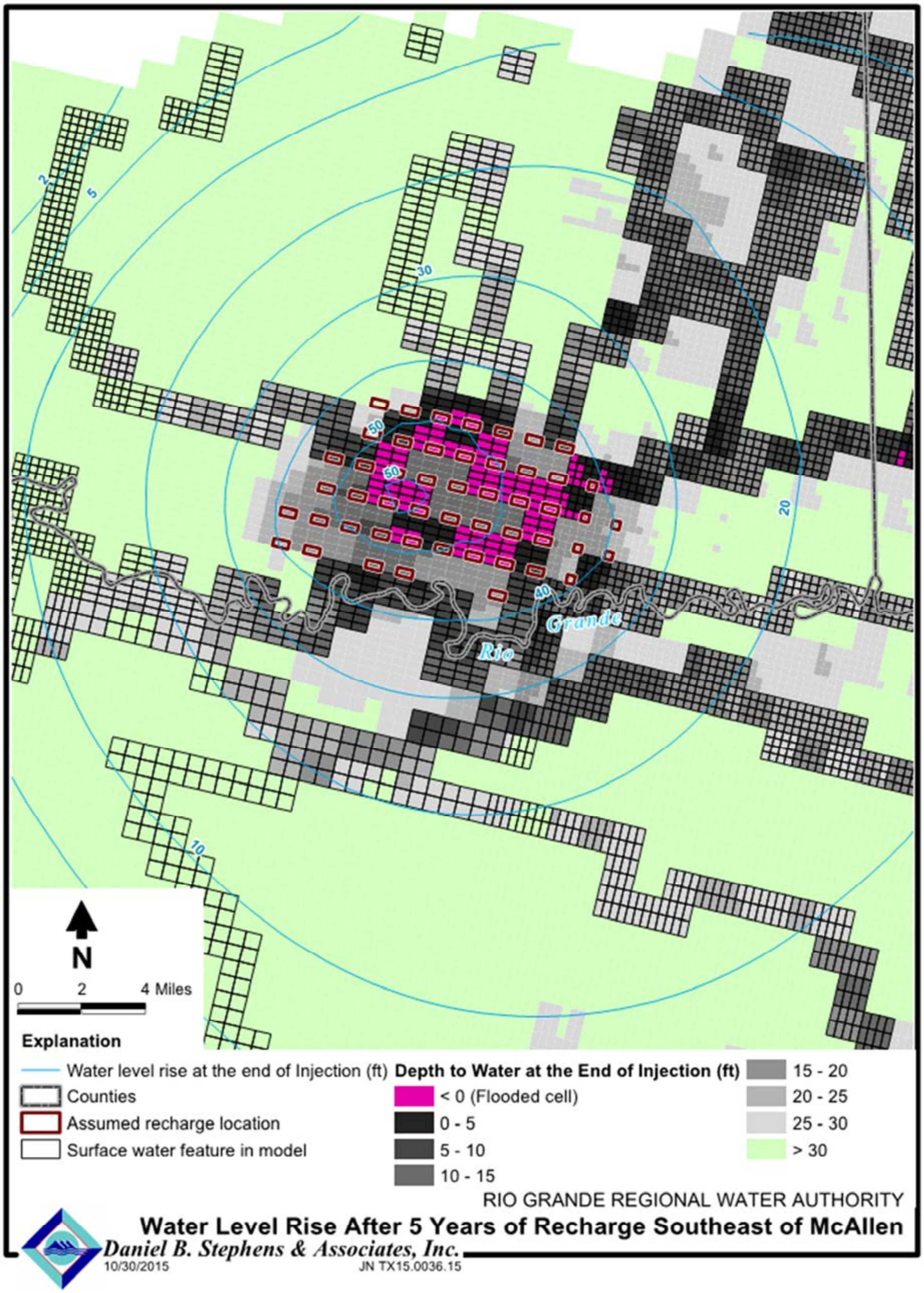

Figure 12-7 Recharge in Western Well Fields, Southeast of McAllen

12.5 AQUIFER RECHARGE INFRASTRUCTURE CONCEPT

12.5.1 Purpose

A conceptual design of the proposed facilities for ASR systems is described in this section. For the purpose of this evaluation, it is assumed that the ASR system will have a production and recharge capacity of 30 MGD, assuming that the ASR stores water for 5 consecutive years, the total stored volume would be in excess of 150,000 Ac-ft. Preliminary hydrogeological models show that the aquifer may be able to store this amount of water. Excess water from Rio Grande River and unused water rights, identified as potential water sources for storage during low demand periods, could be pumped using the proposed raw water intake pump stations and treated at the surface water treatment plant described in Chapter 10 – Surface Water Infrastructure. Depending on the systems water demand, excess treated water could be pumped to recharge area using the finish water pump station at the surface water treatment plant. It is assumed that the finish water pumps will provide necessary pressure for injection of treated water into the aquifer.

12.5.2 Flow Monitoring

Excess surface water will be monitored with a flow monitoring system near the Brownsville gaging station in Cameron County. The proposed intake structure located in Hidalgo County will operate when the flows are above the threshold of 55 cfs or when utilizing annual water rights.

12.5.3 Well Field Infrastructure

The excess finished water will be conveyed from the finished pump station via a 36-inch diameter HDPE pipeline approximately 25,000 linear feet to the recharge area. A 36-inch diameter pipeline was chosen because this diameter pipeline will be able to handle a range of flows from 10 MGD to 30 MGD, while maintaining a minimum velocity of 2 feet per second. Headloss in the pipe is expected to be less than 150 feet.

Of the two recharge locations identified, the area southeast of McAllen, because it is near to the proposed surface water treatment plant is selected for the conceptual design. Although recharges through either recharge wells or infiltration basins appear to be viable, a system of recharge wells is assumed for this evaluation. A total of 36 wells spaced a mile apart will be designed for both recharge and recovery. An 80% water recovery efficiency is assumed based on 20% loss in stored water. The preliminary well field configuration is based on three rows of 12 wells, each equipped with vertical turbine pumps with 630 gpm injection and a 700 gpm recovery rate. Well depths in the recharge zone are assumed to be 500-600 feet below ground surface.

Sizing for the well piping was generally based on a maximum velocity of 4 feet per second (fps) with all wells in operation. The well system piping would range from 10 inches to 30 inches as shown in Table 12-1 below. Recovered water will be pumped back to treatment plant for chlorination and distribution during high water demands.

Table 12-1 ASR Well Field Piping Length Summary

DIAMETER (INCHES)	WELL FIELD COLLECTION PIPELINE(FEET)
10	94,680
12	15,780
18	15,780
24	31,560
30	52,600
TOTAL	210,400

The information provided above is based on the most typical and reliable applications utilized in the United States. Detailed information would need to be collected on the following in order to properly design the pipeline and well fields:

- Regulatory requirements
- Environmental factors: floodplain delineation, vegetation load, species of interest, wetland delineation and turbidity
- Soil conditions
- Location and routing
- Boring locations, length, and type

12.5.4 Well Pumps

Sizing and selection of ASR well pumps must consider the ground elevation at each well site, the long-term estimated depth to water, the head loss for the collection piping extending to each well, and the elevation at which each well pump would discharge into. Because these factors will vary somewhat for each well, the total required pump head at each well site will also vary. The assumptions used to determine design criteria for a typical well pump are summarized below.

Ground elevations within the Well Field area generally range from about 120 to 140 feet. It is assumed that the average well would be at a ground elevation of 130 feet. The total well depths are estimated to be 600 feet for the Well field.

Using the assumptions described above, the well pump head and associated motor horsepower design steps are summarized in Table 12-2. The design criteria for the Well Field pumps are 560 feet total dynamic head (TDH) and 124 hp. It is assumed that the pump motors would be 150 hp for the Well Field. A schematic drawing of ASR facility is shown in Figure 12-8.

Table 12-2 ASR Well Pump Design Criteria

WELL FIELD	TREATMENT PLANT ELEVATION (MSL)	AVG. COLLECTION PIPING HEAD LOSS[1] (FT)	REQ'D HGL AT WELL	LONG-TERM WATER ELEVATION (FT)	PUMP HEAD (FT)	MINIMUM PUMP HORSEPOWER[2]
ASR Wellfields	280	150	430	-130	560	124

(1) Average head loss between treatment plant and ASR well.
(2) Based on 700 gpm design capacity and 80 percent pump efficiency.

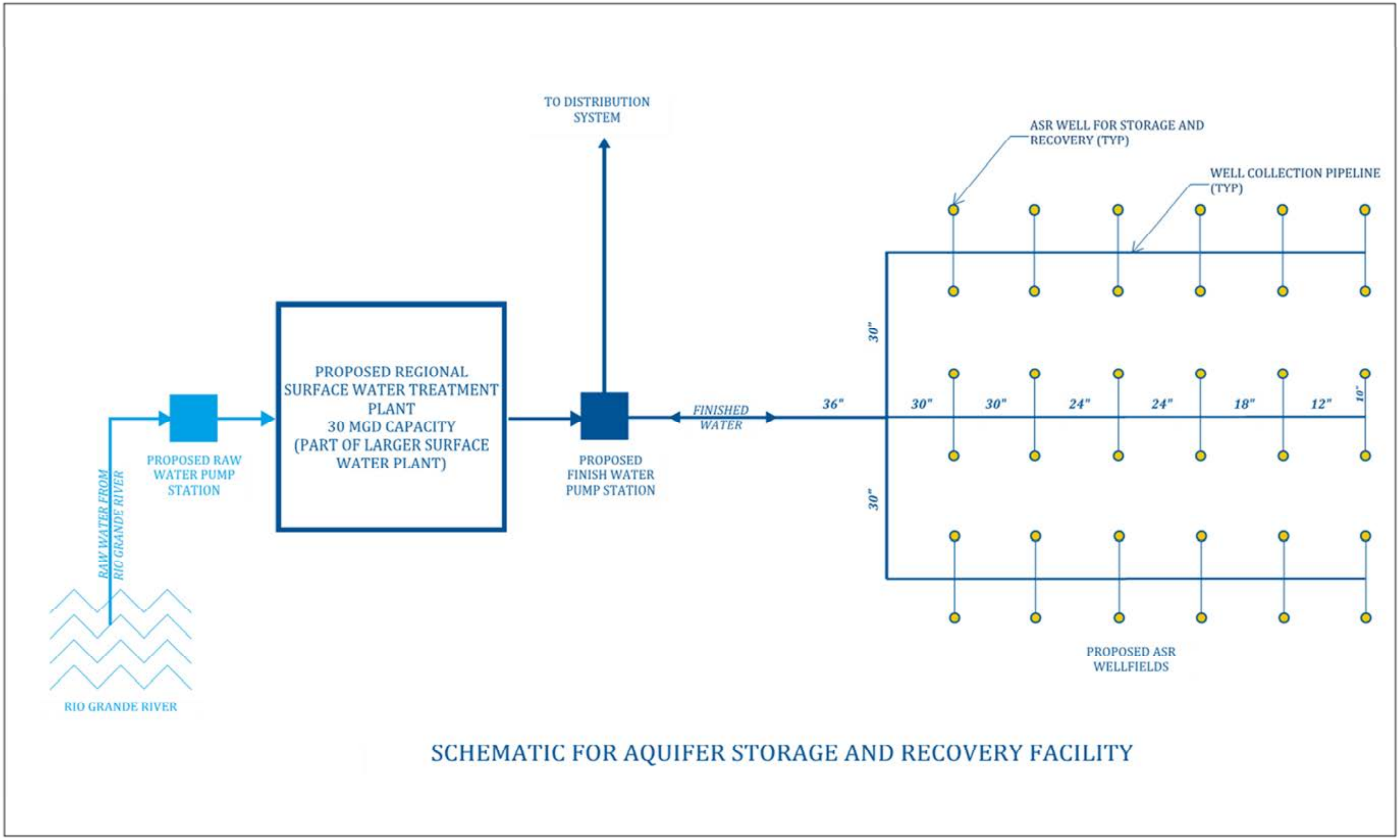

Figure 12-8 Schematic for ASR Facility

12.6 COST EVALUATION

Budget-level engineer's opinion of probable construction costs (EOPCC), capital, and O&M costs were developed for proposed ASR facility. The EOPCC includes a 25 percent contingency factor to account for uncertainties at this stage of the project. Capital costs were calculated using an additional 25 percent factor on top of the EOPCC for Engineering, Legal, and Administration (ELA) costs associated with the project. The construction contingency and ELA percentages used are in line with industry practices for estimating costs at the conceptual/preliminary stages of a project. O&M costs include chemicals, power, labor, and equipment maintenance.

12.6.1 EOPCC and Capital Cost

Construction costs were developed based on actual construction cost data from available contractor's schedule of values from previous reference projects. The Engineering News Record (ENR) construction cost index was used to adjust the cost. Table below is a summary of the budget-level EOPCC (including contingency) and capital costs (including ELA) in 2015 dollars.

The estimated cost for well construction includes drilling of all ASR wells, pumps, site development, electrical work, collection piping, and access roads within the well field. A unit cost per well was determined from previous Black & Veatch projects. Daniel B. Stephens & Associates also provided prices on the well construction for comparison. As shown in Table 12-3, these unit prices were multiplied by the number of wells to produce a total cost.

Prices per linear foot were developed for the well field pipelines and multiplied by the length of each pipeline. The unit costs were based on the pipeline diameter and incorporate costs for installation, fittings, trench excavation and safety protection, erosion and sedimentation controls, hydrostatic testing, restoration, and other items typically required to install a transmission main. These unit prices were developed from similar projects.

Table 12-3 ASR Well Field Costs

	COST PER UNIT	NO. OF UNITS	TOTAL
ASR Well (150 HP, 600 ft deep)	$1,003,200	36	$ 36,116,000
Well Field Pipe (10 inch to 36 inch HDPE)	--	--	$ 35,334,000
Contractor Markup (10 percent Including Insurance/Bond)	--	--	$ 7,145,000
Total Well Field Costs			**$ 78,595,000**

12.6.2 Operation & Maintenance Cost

Operations and maintenance costs for the ASR facility were calculated for storage and recovery period. For this evaluation, O&M cost during storage period were calculated based on the assumption that 30 MGD of water will be stored for a period of five years. O&M cost during storage period include cost for the excess water that was captured, treated and pumped, and are based on the cost calculated in Chapter 3 – Surface Water Infrastructure.

The electrical usage, staffing requirements, chemical dosage, and miscellaneous consumables were projected for recovery of stored water based on seven year recovery period.

The engineer's opinions of probable O&M cost were developed based on the following categories:

- **Chemical Costs:** These costs were determined based on the chemicals required for treatment of excess water during storage. During recovery, water withdrawn from aquifer may require disinfection. O&M cost during recovery mode are calculated on the 30 MGD of daily flows assuming chloramine is used for disinfection. The chemical unit prices were established based on regional chemical supply contracts and vendor quotes.
- **Energy Costs:** These costs were determined for ASR well fields electrical usage based on the anticipated flow and head during recovery period. . A power cost $0.05/KWh was used for electricity based on input from project stakeholders.
- **Solid Disposal:** Solids disposal is annualized although disposal may occur every second year.
- **Labor & Maintenance:** Maintenance costs were assumed to represent 1.5% each year of the project's equipment cost. Labor costs are included and the value was based on staffing levels for similar facilities.

Approximate operational costs are summarized for in Table 12-4.

Table 12-4 Annual ASR Facility O&M Costs Per Operation Mode

	COST FOR ASR STORAGE	COST FOR ASR RECOVERY
Chemicals	$ 1,063,000	$ 517,000
Energy Cost	$ 266,000	$ 1,336,000
Solid Disposal	$ 399,000	-
Labor & Maintenance	$ 665,000	$ 391,000
TOTAL	$ 2,393,000	$ 2,244,000

12.6.3 Land Acquisition

Easement and property acquisition costs were shown separately from capital costs. Easement costs were calculated based on area requirements for the well field conveyance pipeline. Property acquisition costs were calculated based on estimated area needed for ASR well. A unit cost of $4,500/Acre was used for easements and $5,000 for property acquisition and multiplied by the area required for easements and property. Estimated costs are shown in Table 12-5.

Table 12-5 Land Acquisition Costs for ASR Well Fields and Piping

PROPERTY	UNIT	QUANTITIES	UNIT COST	TOTAL COST
	ROW Width*	Length (LF)	($/Acre)	
Well Field Conveyance Pipeline	50 feet	236,700	$4,500	$ 1,223,000
	Area Per Unit (AC)	No. of Wells	($/Acre)	
ASR Well	0.7	36	$5,000	$ 127,000
Total				**$ 1,350,000**

12.6.4 Cost Summary

The final costs for the ASR system are summarized as shown in Table 12-6.

Table 12-6 ASR Facility Cost Summary

		2015	2020
		Present Worth	*Inflation Rate:
Construction Costs			
ASR Wellfield		$ 39,758,400.00	$ 46,091,000.00
Conveyance Pipeline		$ 38,867,000.00	$ 45,058,000.00
Electrical Infrastructure		$ 1,995,000.00	$ 2,313,000.00
Contingency	25%	$ 19,657,000.00	$ 22,788,000.00
Total Construction Cost		$ 100,277,400.00	$ 116,250,000.00
Engineering Cost			
Pre-Design Phase	1%	$ 1,003,000.00	$ 1,163,000.00
Design and Construction Phase	15%	$ 15,042,000.00	$ 17,438,000.00
Program Mgt./Construction Mgt.	8%	$ 8,023,000.00	$ 9,300,000.00
Permitting	1%	$ 1,003,000.00	$ 1,163,000.00
Total Engineering Cost	25%	$ 25,071,000.00	$ 29,064,000.00
Land Acquisition			
Wellfield		$ 127,000.00	$ 148,000.00
Pipeline		$ 1,223,000.00	$ 1,418,000.00
Contingency	25%	$ 338,000.00	$ 392,000.00
Total Land & Easement Cost		$ 1,688,000.00	$ 1,958,000.00
Water Rights Fees		$ -	$ -
Total Capital Costs		**$ 127,036,400.00**	**$ 147,272,000.00**
		2015	**2020**
		Present Worth	*Inflation Rate:
Operations and Maintenance Costs			
ASR Storage w/contingency	25%	$ 2,992,000.00	$ 3,470,000.00
ASR Recovery w/contingency	25%	$ 2,225,000.00	$ 3,227,000.00
Total O&M Costs		**$ 5,774,000.00**	**$ 6,697,000.00**

*Assuming 3% inflation rate

CHAPTER 13 - FUNDING AND FINANCE ALTERNATIVES

Regional Facility Plan

B&V PROJECT NO. 181082

PREPARED FOR

Rio Grande Regional Water Authority

9 MARCH 2016

BLACK & VEATCH
Building a world of difference.

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13.0 Funding and Finance Alternatives

The purposes of this chapter are to outline potential funding strategies and to estimate monthly rates and fees per connection for water users. This chapter utilizes research performed previously on funding strategies for the RGRWA. This more detailed information on the various funding programs is included as an appendix to this chapter.

13.1 FUNDING STRATEGY

13.1.1 Local and State Resource Opportunities

The State of Texas and the federal government have made funding resources available for a variety of utility projects. These projects include water supply, water quality, and protection of wildlife. The funds are used to assist the state in meeting established goals such as water conservation, expansion of drought-proof water supplies and restoration of habitats. The Texas Water Development Board has available resources for projects that are recommended in adopted regional water plans and subsequently in statewide water management plans. Elements of the RGRWA water supply project are eligible expenses. These include, planning, design, desalinating seawater-brackish water and building new pipelines.

It is recommended that RGRWA implement multiple funding options to lower costs and ultimately maintain reasonable costs and/or rates. The initiatives identified below include grants, low-interest loans, and combinations of the two offerings. The proposed applications incorporate both municipal connections that meet the rural criteria and unincorporated areas of the county that meet the eligibility requirements for grant/loan opportunities.

Section 13.2 describes each available funding opportunity and probable inclusion to the program via eligibility and percentages of available resources. The following Section 13.3 describes federal funding options to include legislative strategy. Finally, Section 13.4 provides insight to grant percentages, loan percentages and match requirements by program. Also, included in Section 13.4 are funding strategy scenarios that may assist in the evaluation of potential programs.

13.2 TEXAS WATER DEVELOPMENT BOARD (TWDB)

13.2.1 Drinking Water State Revolving Loan Fund Program (SRF)

The Drinking Water SRF program provides loans and principal forgiveness for eligible projects. The loans are offered at a subsidy rate of 125 basis points off the underlying credit rating of the borrower. The terms of the program were expanded to 20-30 years depending on the life cycle of the improvement. Principal forgiveness is a limited option for eligible systems. The option is available for projects that meet standards established by SRF. It appears the eligible “Green” elements of the project may qualify if funding is available by the department. Subsidies for “Green” qualified projects are up to 15% depending on the eligible components of the project exceeding 30% of the total project costs.

13.2.2 Green Project Reserve (GPR)

State Revolving Fund (SRF) programs generally include provisions to promote Green principles and technologies, and require States to establish a Green Project Reserve (GPR). The GPR provision generally requires States to reserve not less than 20% of the annual federal allocation for SRF capitalization grants to address green infrastructure, water or energy efficiency improvements, or other environmentally innovative activities.

13.2.3 Rural Water Assistance Fund (RWAF)

The State of Texas Rural Water Assistance Fund (RWAF) program exists to provide assistance to rural areas that offer utility expansion and service to customers. The program offers loan proceeds to assist in developing water utilities within municipal areas of 10,000 or less or counties with no urban area exceeding 50,000 in population. The program advantages include a lower cost of borrowing and expanded terms up to 40 years.

13.2.4 State Water Implementation Fund for Texas (SWIFT)

The State Water Implementation Fund for Texas (SWIFT) provides funds for projects included in the State Water Plan. 10% of the funds are reserved for rural initiatives, and 20% are reserved for conservation projects. Projects are prioritized based on the description of need, feasibility, viability, sustainability, and cost-effectiveness.

Three options exist for borrowing from the SWIFT program:

- The first option is a low-interest loan with fixed terms at below-market rates. The loan maturities vary from 20-30 years. Available subsidy options also vary from 20 year loans at 35% to 30 year loans at 20%. These subsidies provide principal forgiveness or grant allowances based on affordability and underwriting review.
- The second option is a deferred loan with the same type of standards that defers principal and interest for up to 8 years from the delivery date or end of construction.
- Finally, the third option is a board participation offer that includes a temporary ownership option. The TWDB would have interest in the excess capacity of the project with a limit of up to 80% of total project costs. The program allows for the sponsor to repurchase the TWDB interest via a schedule of repayment. This allows deferral of both the principal and interest with terms of 30-35 years.

13.3 FEDERAL FUNDING

13.3.1 United States Department of Agriculture (USDA) Rural Development

United States Department of Agriculture (USDA) Rural Development provides funding opportunities in the form of payments, grants, loans, and loan guarantees, for the development and commercialization of vital utility services. These programs revitalize rural communities with a variety of infrastructure improvements, and create sustainable opportunities for wealth, new jobs, and increased economic activity in rural America.

13.3.1.1 Direct Loans and Grants

Direct Loans and Grants can be used to develop water and waste disposal systems in rural areas and towns with a population not in excess of 10,000 or qualified unincorporated areas of a county. The funds are available to public bodies, non-profit corporations and Indian tribes. The program allows unincorporated areas of county governments to present applications for utility improvements. Grant percentages are based on underwriting outcomes and loans are currently being offered at 3.5% for 40 year terms.

13.3.1.2 Guaranteed Loans

Guaranteed loans provide funding for the construction or improvement of eligible projects serving the financially needy communities in rural areas. This purpose is achieved through bolstering the existing private credit structure through the guarantee of quality loans which will provide lasting benefits. The water and waste disposal guarantee loans are to serve a municipal government with a population less than 10,000 or unincorporated area within a county jurisdiction.

13.3.2 Bureau of Reclamation

The Bureau of Reclamation provides grant programs to assist in development of new water supply infrastructure and associated facilities. These grants begin at $200,000 and increase depending on the project need. The pledged match ranges from 25% to 50% of the total requested funding.

13.3.3 Border Environment Cooperation Commission

13.3.4 North American Development Bank (NADB)

13.3.4.1 NADB Loan Program

NADB was established to finance the development, execution and operation of environmental infrastructure along the US-Mexico border region. The NADB is authorized to loan any project, of any size, of any demographic or of any project cost at a maximum of 85% of the capital cost. The program is a loan only. Municipal agencies should expect a capital financial plan review to determine affordability and maximum debt allowance. NADB will present an offer of terms and conditions based on this assessment for review and acceptance by RGRWA. Grant assistance is available for project development and design through other related programs with a maximum of $500,000 of matching funds.

13.3.4.2 Border Environment Infrastructure Fund (BEIF)

The U.S.-Mexico Border Water Infrastructure Program, funded by Congress through EPA, has awarded grants to water and wastewater systems in the border region through the Project Development Assistance Program (PDAP) for project development and design. The Border Environment Infrastructure Fund (BEIF) provides funding for construction, programs administered by NADB with BECC approval.

Applications are for a maximum of $30M and project sponsors are encouraged to complete final design for analysis of eligibility. The analysis shall include a comprehensive financial review of the project and eligible project costs. The agency will work with RGRWA to determine a maximum debt capacity and work from that point to a final determination of grant eligibility. The BEIF program shall not exceed $8M on any one project in grant funding. The remainder of the eligible project will be funded by a loan.

13.3.5 Federal Appropriation

Federal appropriation requests have experienced a delay over the past couple of years due to limited funding and budget issues. The allocations process has received scrutiny due to the overall selection process. Due to the attention given to the process, federal officials have discussed how to revise the prior process and still be able to make a difference for critical projects across the nation. It is expected that the ongoing discussions will lead to an opportunity to request federal funds under this umbrella in FY2016/FY2017.

The United States Army Corp of Engineers (USACE) offers multiple programs that may also provide for the federal opportunity to support regional water supply in south Texas. The USACE plans, designs, and constructs projects that reduce flood risk and conducts emergency management when the need arises. Since none of the projects recommend infrastructure that could be used to mitigate flood risk, this program isn't considered further. The program is subject to federal funding allocation annually.

13.3.6 Water Infrastructure Financing Innovation Act (WIFIA)

The 2014 Water Resources Reform and Development Act (WRRDA) included language pertaining to the Water Infrastructure Financing Authority that described changes to the State Revolving Loan Fund process. The change directly affects the low interest loan terms. The language includes the ability for an eligible applicant to apply for a 30 year loan instead of the typical 20 year loan request.

13.4 FUNDING SCENARIOS

The following Table 13-1 provides a summary of the various funding scenarios and programs that are believed to be applicable for the proposed project elements, community demographics, and the associated health, safety and environmental issues. In addition, there are both federal and state agency funding opportunities that are not included in the overall alternative finance scenarios

All grant applications are subject to evaluation, ranking and potential award. The grant opportunities described below, as well as the opportunities described throughout this document, are subject to annual budget allowances, application, evaluation and agency participation. The federal and state agencies determine eligibility and are the sole decision makers regarding funding award.

The agencies and associated data provided within this document are for planning purposes only. Should the board elect to pursue funding through any one agency, there must be a completed application and they must compete for funding as described by the target agency. Funding is not guaranteed. Applications may be denied or may simply not compete to a level that receives approval.

Table 13-1 Funding Agencies Analysis

FUNDING AGENCIES	FUNDING BY ELIGIBILITY		
	MAXIMUM LOAN	MAXIMUM GRANT	REQUIRED MATCH
Drinking Water State Revolving Loan Fund Program (SRF)	90%	10%	0%
Green Project Reserve (GPR)	0%	20%	0%

FUNDING AGENCIES	FUNDING BY ELIGIBILITY		
	MAXIMUM LOAN	MAXIMUM GRANT	REQUIRED MATCH
Rural Water Assistance Fund (RWAF)	100%	0%	0%
State Water Implementation Fund for Texas (SWIFT)	100%	35%	0%
US Department of Agriculture (USDA) Rural Development	65%	35%	0%
Bureau of Reclamation	0%	50%	50%
North American Development Bank (NADB)	85%	50%	15%
Federal Appropriation	0%	100%	25%
Water Infrastructure Financing Innovation Act (WIFIA)	100%	0%	50%

13.4.1 Funding Alternative 1

The following funding scenarios 1-1, 1-2, 1-3 and 1-4 in Table 13-2 represents funding available via the Texas Water Development Board programs to include deferred payment and board participation options. TWDB has available eight (8) year deferred payment schedule offering and TWDB participation in ownership of any unused portion of the supply or disbursement:

Table 13-2 Funding Alternatives 1 – Federal and State

RGRWA	PHASE I		FUNDING ALTERNATIVE 1 – TWDB EXAMPLE			
			SCENARIO 1-1	SCENARIO 1-2	SCENARIO 1-3	SCENARIO 1-4
CUSTOMERS (EDU)			200,000	200,000	200,000	200,000
TOTAL PROJECT COST			$480,000,000	$480,000,000	$480,000,000	$480,000,000
TOTAL ANNUAL OMR			$1,000,000	$1,000,000	$1,000,000	$1,000,000
FINANCING						
Principal Forgiveness			$15,000,000	-	-	$15,000,000
SWIFT Direct			$68,000,000	-	-	-
GPR	30	1.00%	$50,000,000	-	-	$50,000,000
SRF	30	2.80%	$30,000,000	-	$480,000,000	$415,000,000
SWIFT	30	2.00%	$317,000,000	$480,000,000	-	-
Total Financing			$480,000,000	$480,000,000	$480,000,000	$480,000,000
ANNUAL DEBT						
GPR			$1,937,406	-	-	$1,937,406
SRF			$1,491,275	-	$23,860,395	$20,629,300

RGRWA	PHASE I	FUNDING ALTERNATIVE 1 – TWDB EXAMPLE			
		SCENARIO 1-1	SCENARIO 1-2	SCENARIO 1-3	SCENARIO 1-4
SWIFT		$14,154,025	$21,431,963	-	-
ANNUAL DEBT & OMR		$16,645,300	$22,431,963	$24,860,395	$21,629,300
Total Future Av. Mo. Cost Per Customer		$6.94	$9.35	$10.36	$9.01
GPR		$58,122,170	-	-	$58,122,170
SRF		$44,738,241	-	$715,811,859	$618,879,003
SWIFT		$424,620,761	$642,958,881	-	-

**Loan terms and interest rates can be changed and payments/payback will change accordingly.
**Column A depicts a fair market value based on current conditions but have not been accurately provided by a bond agent. The figures are for example only.

13.4.1.1 TWDB Funding Alternative Descriptions

Funding Scenario 1-1 –

The first scenario describes options for Phase I project costs. The total amount of funding required for Phase I is estimated at $480,000,000. Available resources include grant and low-interest loans. In Scenario 1 the approach would be a multi-application approach to the Texas Water Development Board for both the traditional direct loan program of the State Revolving Loan Program (SRF) and participation in the State Water Implementation Fund for Texas (SWIFT). These programs have both a principal forgiveness and long term loan associated with each application. The goal would be to incorporate elements of the project in each application that would allow for the maximum amount of grant funding. The described scenario integrates both a request for approximately $100,000,000 of the required funding to be applied to the direct program including a Green Project Reserve (GPR) business case describing the eligible green components and decreasing the interest rates based on acceptance by the program. Also, the scenario involves application to SWIFT for the remaining program costs.

Funding Scenario 1-2 –

The second scenario describes applying only to the SWIFT program for the entire project and receiving no principal forgiveness element. The $480,000,000 Phase I total project cost is amortized over a 30 year schedule at 2.00% interest. It would be unlikely that the request would not receive some element of principal forgiveness.

Funding Scenario 1-3 –

The third scenario describes the traditional direct SRF loan option with a 30 year term and a 2.8% interest rate. This scenario would not include principal forgiveness or GPR qualification to lower interest rates.

Funding Scenario 1-4 –

In the fourth scenario RGRWA would apply for the direct loan including principal forgiveness. The remainder of the program would be a traditional SRF loan.

Funding Scenario 1-5 –

In each of the funding scenarios that include SWIFT funding, two additional options exist and are shown in Table 13-3 as scenario 1-5. The first option is to have amortized payments deferred for 8 years. The second is TWDB participation through capital investment in additional infrastructure capacity. The recommended approach would be to utilize both of these options. This would allow RGRWA to purchase the required amount of project capacity for Phase I, while constructing the project entirely and buying back the unused portion of treatment or conveyance during the remaining phases when the infrastructure is needed. The purchase would be on similar terms to SWIFT current rates. The payment deferral option allows for the RGRWA to finish the projects and start collecting revenue prior to their first annual debt service payment. A more detailed analysis with the capital cost deferral due to TWDB ownership is included in Table 13-3.

Table 13-3 Funding Scenario 1-5 Including Deferred Payment and Partnership Option

RGRWA	PHASE I		FUNDING ALTERNATIVE 1-5 – TWDB EXAMPLE
			SCENARIO 1-5
CUSTOMERS (EDU)			200,000
TOTAL PROJECT COST			$480,000,000
FINANCING			
Principal Forgiveness			$15,000,000
SWIFT Direct			$68,000,000
SRF	30	2.80%	-
SWIFT	30	2.00%	$397,000,000
Total Financing			$480,000,000
ANNUAL DEBT			
SRF			-
SWIFT			$17,726,019
25% Borrowing		2017	$90,000,000
25% Borrowing		2018	$120,000,000
25% Borrowing		2019	$150,000,000
25% Borrowing		2020	$120,000,000
TOTAL			**$480,000,000**
Deferred 2017			$3,991,000
Deferred 2018			$9,314,000
Deferred 2019			$15,967,000
Deferred 2020			$21,289,000
Deferred 2021			$21,289,000

RGRWA	PHASE I	FUNDING ALTERNATIVE 1-5 – TWDB EXAMPLE
		SCENARIO 1-5
Deferred 2022		$21,289,000
Deferred 2023		$21,289,000
Deferred 2024		$21,289,000
TOTAL		$135,717,000
SRF		-
SWIFT		$531,780,575
TOTALS		$531,780,575
Est. Partnership Option SWIFT (25%)		$132,945,143
Est. Deferred Annual Payment SWIFT		$141,808,153

**Loan terms and interest rates can be changed and payments/payback will change accordingly.
**Column A depicts a fair market value based on current conditions but have not been accurately provided by a bond agent. The figures are for example only.

13.4.2 Funding Alternative 2

The following funding scenarios 2-1, 2-2, 2-3 and 2-4 are modeled in Table 13-4 and represent participation by rural areas that are to be served by the utility. The table also shows the percentages of funds available through the Border Control Commission.

Table 13-4 Funding Alternatives 2 – Local, Rural, and State

RGRWA	PHASE I		FUNDING ALTERNATIVE 2 – OTHER SOURCES EXAMPLE			
			SCENARIO 2-1	SCENARIO 2-2	SCENARIO 2-3	SCENARIO 2-4
CUSTOMERS (EDU)			200,000	200,000	200,000	200,000
TOTAL PROJECT COST			$480,000,000	$480,000,000	$480,000,000	$480,000,000
TOTAL ANNUAL OMR			$1,000,000	$1,000,000	$1,000,000	$1,000,000
FINANCING						
USDA			$33,000,000	-	$33,000,000	$33,000,000
BECC			$168,000,000	-	-	-
USDA	40	2.50%	$59,000,000	-	$59,000,000	$59,000,000
SRF	30	2.80%	$253,000,000	-	$388,000,000	-
WIFIA	20	3.00%	-	$480,000,000	-	$388,000,000
Total Financing			**$480,000,000**	**$480,000,000**	**$480,000,000**	**$480,000,000**
ANNUAL DEBT						
USDA			$2,350,338	-	$2,350,338	$2,350,338

RGRWA	PHASE I	FUNDING ALTERNATIVE 2 – OTHER SOURCES EXAMPLE			
		SCENARIO 2-1	SCENARIO 2-2	SCENARIO 2-3	SCENARIO 2-4
SRF		$377,292,602	-	$578,614,586	-
WIFIA		-	$625,270,793	-	$521,593,891

**Loan terms and interest rates can be changed and payments/payback will change accordingly.
**Column A depicts a fair market value based on current conditions but have not been accurately provided by a bond agent. The figures are for example only.

13.4.2.1 Other Funding Alternative Descriptions

Funding Scenario 2-1 -
The first scenario describes using United States Department of Agriculture (USDA) funding for eligible project components that are demographically eligible. These may include areas within the unincorporated county boundaries and within city limits of areas that do not exceed population and income limits. The proposed scenario would allow for grant and loan combinations that include 40 year terms on loan packages at an estimated 2.5% interest rate (the interest rate changes quarterly). Also, the RGRWA would work closely with the Border Environmental Cooperation Commission (BECC) to receive funding via federal processes. These include appropriations, the Border Environmental Infrastructure Fund and the banking system used by the BECC agency. It is expected that all available resources of the BECC agency and federal legislative assistance would be required to fund such a large project. The remainder of the project would then be requested to the SRF program through the deferred loan program allowing an eight year window prior to debt service.

Funding Scenario 2-2 -
The second scenario describes applying to the Water Infrastructure Fund for a 20 year loan at 3% which does not seem to be a reasonable financial model.

Funding Scenario 2-3 -
The third scenario describes both USDA and SRF being used to fund the program.

Funding Scenario 2-4 -
Finally, the fourth option describes a combination of USDA and WIFIA programs.

13.5 FINANCIAL BREAKDOWN

In evaluating the financial impacts on ratepayers over a 50 year period, a simplified cash flow analysis was conducted to determine: 1) costs per connection for the conveyance projects, 2) costs per 1,000 gallons for treatment and storage projects, and 3) overall cost per acre foot delivered. In the cash flow analysis, four financing scenarios were examined based on the funding opportunities previously discussed. The scenarios are as follows:

- Scenario 1: Use of Revenue Bonds
- Scenario 2: Use of SWIFT deferred option with SWIFT loans
- Scenario 3: Use of SWIFT state participation option with SWIFT loans

- Scenario 4: Use of a combination of SWIFT deferred and state participation options with SWIFT loans

In the development of all scenarios, there are two major costs components that are considered: capital and operation and maintenance (O&M).

13.5.1 Capital Financing

Capital costs represent the total investment to build the facilities which include planning, design and construction. In determining how to finance the capital costs the four scenarios were examined. The following is a brief description of the scenarios and associated assumptions:

- Scenario 1: RGRWA obtains capital funding from private financial institutions in the form of revenue bonds. These types of bonds usually demand a risk premium and therefore have a higher interest rate than state and federal loans. Assumptions are:
    - Interest rate of 5.5% with a 20 year payback period.
- Scenario 2: RGRWA obtains funding from the State of Texas through their SWIFT program. The SWIFT program provides RGRWA an option to defer repayment for up to 8 years. The deferment allows RGRWA to build up revenues before repayment begins. Assumptions are:
    - Deferment of repayment for 8 years. Interest accrued over the 8 year period. The loan amount in year 1 is the full capital costs of the project.
    - Interest rate of 2% with a 30 year payback period after year 8. Principal includes original loan plus accrued interest.
- Scenario 3: RGRWA obtains funding from the State of Texas through their SWIFT program. The SWIFT program provides RGRWA an option for the State to participate in ownership of the assets, thus deferring the capital costs until a future date. By having the State co-own the assets, RGRWA only pays for the assets it needs and defers capital costs to a later date. The partial deferment of capital costs allows RGRWA time to build up revenues for future acquisition of assets. Assumptions are:
    - State of Texas owns a stake in conveyance assets. Percentages vary over the 50 years period. At the end of the 50 years, RGRWA owns all assets. Treatment assets are fully owned by RGRWA.
    - Interest rate of 2% with a 30 year payback period starting year 1 after the acquisition of the assets.
- Scenario 4: RGRWA uses a combination of the deferment and State participation options described in Scenarios 2 and 3. Assumptions are:
    - Deferment of repayment for 8 years. Interest accrued over the 8 year period. The loan amount in year 1 is the full capital costs of the project.
    - State of Texas owns a stake in conveyance assets. Treatment assets are fully owned by RGRWA.
    - Interest rate of 2% with a 30 year payback period after year 8. Principal includes original loan plus accrued interest.

Shown in Tables 13-5 and 13-6 are the average annual costs by decade for conveyance projects and treatment and storage projects based on the cash flow analysis.

Table 13-5 Amortized Capital Costs - Conveyance

	Amortized Conveyance Capital Costs- Equivalent Annual Cost*					
	2020	2030	2040	2050	2060	2070
Scenario 1	$17,070,583	$19,932,416	$5,941,232	$7,740,338	$9,759,338	$10,120,784
Scenario 2	$2,359,219	$10,814,241	$12,666,922	$12,692,943	$7,436,743	$11,516,803
Scenario 3	$5,246,544	$6,791,521	$8,199,601	$6,486,026	$11,094,523	$15,374,665
Scenario 4	$1,358,910	$6,919,137	$8,208,936	$9,484,891	$7,942,691	$14,052,581

**Cost represents average cost for 10 year timeframe of amortized capital costs. For example, 2020 amount represents the average annual cost from 2020 to 2029.*

Table 13-6 Amortized Capital Costs – Treatment & Storage

	Amortized Treatment & Storage Capital Costs- Equivalent Annual Cost*					
	2020	2030	2040	2050	2060	2070
Scenario 1	$24,484,405	$55,542,975	$67,637,341	$91,423,024	$62,175,141	$48,983,567
Scenario 2	$3,383,836	$17,373,469	$38,809,320	$60,090,092	$80,166,685	$83,075,029
Scenario 3	$11,758,030	$29,636,823	$47,848,210	$66,660,624	$70,632,422	$74,568,625
Scenario 4	$3,045,453	$17,373,469	$38,809,320	$60,090,092	$80,166,685	$83,075,029

**Cost represents average cost for 10 year timeframe of amortized capital costs. For example, 2020 amount represents the average annual cost from 2020 to 2029.*

13.5.2 Operation and Maintenance

Operation and maintenance costs represent annual costs associated with program administration, plant and pumping station operation and maintaining the facilities in working order. The annual O&M costs are developed in the Organizational Structure and individual infrastructure chapters. It is assumed that O&M costs are the same for all scenarios. As projects are completed and facilities come online, O&M costs are incurred on an annual basis. Under the State participation scenarios, it is assumed that RGRWA will still be responsible for the O&M associated with all assets. Shown in Table 13-7 is the average uninflated annual O&M by decade for conveyance projects and treatment and storage projects.

Table 13-7 Operations and Maintenance Costs – Administration & Conveyance/Treatment & Storage

	O&M Costs- Equivalent Annual Cost*					
	2020	2030	2040	2050	2060	2070
Conveyance	$2,665,000	$3,927,700	$6,431,760	$8,858,000	$11,105,400	$13,429,900
Tmt/Storage	$12,700,000	$19,000,000	$20,902,400	$37,812,982	$49,805,995	$69,537,643

**Cost represents average cost for 10 year timeframe.*

13.5.3 Ratepayer Impacts

The basis for developing the cash flow analysis was to determine 1) the costs per connection for the administration and conveyance projects, 2) the costs per 1,000 gallons for treatment and storage projects, and 3) the overall cost per acre foot delivered. Therefore it was important to combine the capital and O&M costs as shown in Tables 13-8 and 13-9.

Table 13-8 Total Costs – Administration & Conveyance

	O&M and Capital Costs- Total Equivalent Annual (Administration & Conveyance) Cost*					
	2020	2030	2040	2050	2060	2070

Scenario 1	$19,735,583	$23,860,116	$12,372,992	$16,598,338	$20,864,738	$23,550,684
Scenario 2	$5,024,219	$14,741,941	$19,098,682	$21,550,943	$18,542,143	$24,946,703
Scenario 3	$7,911,544	$10,719,221	$14,631,361	$15,344,026	$22,199,923	$28,804,565
Scenario 4	$4,023,910	$10,846,837	$14,640,696	$18,342,891	$19,048,091	$27,482,481

**Cost represents average cost for 10 year timeframe. For example, 2020 amount represents the average annual cost from 2020 to 2029.*

Table 13-9 Total Costs – Treatment & Storage

	O&M and Capital Costs- Total Equivalent Annual (Treatment & Storage) Cost*					
	2020	2030	2040	2050	2060	2070
Scenario 1	$37,184,405	$74,542,975	$88,539,741	$129,236,005	$111,981,136	$118,521,210
Scenario 2	$16,083,836	$36,373,469	$59,711,720	$97,903,074	$129,972,680	$152,612,672
Scenario 3	$24,458,030	$48,636,823	$68,750,610	$104,473,606	$120,438,417	$144,106,268
Scenario 4	$15,745,453	$36,373,469	$59,711,720	$97,903,074	$129,972,680	$152,612,672

**Cost represents average cost for 10 year timeframe. For example, 2020 amount represents the average annual cost from 2020 to 2029.*

Based on the total costs, the projected connections, and projected capacity, the unit costs were calculated for both capital and O&M costs by decade as shown in Tables 13-10 and 13-11. Table 13-12 shows the total costs per acre-foot delivered.

Table 13-10 Equivalent Monthly Cost per Connection – Administration & Conveyance

	Equivalent Monthly Cost per Connection					
	2020	2030	2040	2050	2060	2070
No. of Customers	412,614	503,542	595,080	688,606	782,859	875,890
Scenario 1	$3.99	$3.95	$1.73	$2.01	$2.22	$2.24
Scenario 2	$1.01	$2.44	$2.67	$2.61	$1.97	$2.37
Scenario 3	$1.60	$1.77	$2.05	$1.86	$2.36	$2.74
Scenario 4	$0.81	$1.80	$2.05	$2.22	$2.03	$2.61

Table 13-11 Equivalent Annual Cost per 1,000 gals – Treatment & Storage

	Equivalent Annual Cost per 1,000 gals					
	2020	2030	2040	2050	2060	2070
Water Produced	10,220,000	22,141,068	33,371,838	58,360,299	74,083,376	94,298,761
Scenario 1	$3.64	$3.37	$2.65	$2.21	$1.51	$1.26
Scenario 2	$1.57	$1.64	$1.79	$1.68	$1.75	$1.62
Scenario 3	$2.39	$2.20	$2.06	$1.79	$1.63	$1.53
Scenario 4	$1.54	$1.64	$1.79	$1.68	$1.75	$1.62

Table 13-12 Equivalent Annual Cost per Acre-Foot - All

	Equivalent Annual Cost per Acre-Feet					
	2020	2030	2040	2050	2060	2070

Water Produced	31,364	67,948	102,414	179,101	227,353	289,392
Scenario 1	$1,815	$1,448	$985	$814	$584	$491
Scenario 2	$673	$752	$770	$667	$653	$614
Scenario 3	$1,032	$874	$814	$669	$627	$597
Scenario 4	$630	$695	$726	$649	$655	$622

Based on the cash flow analysis conducted and its associated assumptions, if state funding strategies are utilized (scenarios 2-4), the average monthly bill in 2020 would increase in the range of $0.81 to $1.01/month and costs per 1,000 gallons delivered would range from $1.54 to $2.39. The fluctuation between decades is largely attributed to repayment of capital costs.

State funding strategies, if successful, will improve the cash flow by lowering upfront debt service payments and ease impacts on rate payers and distributors. It should be noted that the cost to integrate new water supplies into individual systems is additional and is assumed to borne by the distributors. Some of these same water supply funding strategies would be available to help with integration costs as well.

Appendix A

DRAFT

LOWER RIO GRANDE VALLEY STRATEGIC WATER MANAGEMENT PLAN

FINANCIAL INITIATIVE PLAN (FIP)

B&V PROJECT NO. 177723

PREPARED FOR

RIO GRANDE REGIONAL WATER AUTHORITY

RIO GRANDE REGIONAL WATER AUTHORITY

28 OCTOBER 2013

BLACK & VEATCH
Building a world of difference.

Table of Contents

1 Contents

Executive Summary

The Rio Grande Regional Water Authority (RGRWA) is seeking funding to prepare a Strategic Water Management Program (SWMP) for the Lower Rio Grande Valley (LRGV). The project goal is to determine the most efficient methodologies to manage the water resources available to the Lower Rio Grande Valley to reliably meet the future agricultural and municipal water demands.

Black & Veatch has analyzed available alternative revenue resources to support the execution of the SWMP, including both infrastructure improvements and conservation measures. Funding programs that may be applicable have been characterized based on their priority and summarized in the Funding Program Summary. Each of the programs is described in more detail in Section 3, and the process for application and administration is outlined in Section 4.

It is the recommendation of Black & Veatch that the RGRWA pursue the following finding alternatives:

- **Federal Legislative Grant Appropriations** can be applied for in the spring, and it is recommended that a submittal be prepared or submittal by the end of the year (2013).
- The **TWDB** efforts, specifically the **Facilities Planning Grant** and ongoing positioning for **SWIFT** funding, are ongoing and high-priority.
- The **BECC** request has been submitted and requires follow through.
- The **USDA** application should move forward now. These dollars are allocated via the Farm Bill and are on a 'first come-first served' basis. The application can include planning, design, legal, and could potentially be applied toward upfront costs to expedite planning and design.

Black & Veatch would like to offer assistance in the pursuit of these and any other funds identified for pursuit by RGRWA.

Funding Program Summary

A summary of funding programs pertinent to this project follows and is provided in greater detail in section 2.

AGENCY	PROGRAM	FUNDS AVAILABLE	PRIORITY
Texas Water Development Board (TWDB)	i. Drinking Water State Revolving Fund, Green Project Reserve ii. Regional Facility Planning Grant iii. Economically Distressed Area Program iv. SWIFT	i. \$107 Million ii. \$500,000 iii. \$50 Million iv. \$2 Billion (not available until Spring 2015)	High Priority
US Department of Agriculture	i. Technical Assistance Grant (TAT) ii. Water and Waste Disposal Direct Loans and Grants iii. Natural Resource Conservation Service: EQIP	i. TAT – up to \$1,000,000 ii. WWD - Based upon Application iii. \$300,000 max (over 6 years)	High Priority
US Economic Development Agency	Cooperative Agreement Grant	\$111,640,000 for Public Works	High Priority
Border Environment Cooperation Commission	i. Border Environment Infrastructure Fund (BEIF) ii. Community Assistance Program (CAP)	i. \$8 Million ii. \$500,000	i. High Priority ii. Medium Priority (Expect no awards until 2015/2016)
Federal Legislative Grant Appropriation	Legislative Appropriations	\$1,000,000 - \$5,000,000 Proposed	High Priority (time sensitive)
Texas Department of Agriculture	i. Texas Capital Fund (Infrastructure Development) ii. Community Development Fund (Rural focus)	i. Between \$50,000 and \$1,500,000 ii. \$55,000 max	Medium Priority
NA	Public – Private Partnerships	Unlimited	Medium Priority
Federal Emergency Management Association (FEMA)	Water Infrastructure Financing Innovation Act (WIFIA)	Unlimited Cap Request start at \$20M	Low Priority (loan only and not complete)

1 Background Information

1.1 PROJECT INFORMATION

The Rio Grande Regional Water Authority (RGRWA) is looking to develop a Strategic Water Management Program (SWMP) which will include diversification of supplies and optimization of existing systems in the Lower Rio Grande Valley (LRGV). The project goal is to determine the most efficient methodologies to manage the water resources available to the Lower Rio Grande Valley to reliably meet the future agricultural and municipal water demands. In addition RGRWA is looking to partner with other entities to operate one or more regional groundwater desalination plants to provide drinking water to residents of the LRGV. The fast growing region of Cameron, Willacy, and Hidalgo Counties currently relies on the Rio Grande for just over 90% of its water, making it extremely vulnerable to drought. Brackish Groundwater Desalination technology has been selected specifically to provide a highly reliable supply to meet the member cities' increasing demands and will supplement continued use of Rio Grande River water. While the RGRWA may have ownership of the treatment facility/facilities, the user base and funding will come from the regional entities that distribute and consume the water. By developing systems that serve more than one or two communities, there will be cost savings from the economy of scale and increased resilience due to interconnectivity.

The SWMP would also evaluate the conveyance systems on a regional scale so that targeted improvements can be made to minimize losses and unreliable infrastructure. Efficiency of some of these delivery systems is as low as 60%, and significant gains could be made with a regional review of the systems. Individual systems that have been viewed piecemeal up to now will need to be reviewed on a regional scale in order to meet the growing demands of both municipal and agricultural users.

The next step toward implementation is a feasibility report, which will review options for meeting current and potential future water shortages in the area. The Regional Water Plan and other applicable studies will be evaluated for feasibility. Different configurations of treatment plant size and service area will be considered. In addition, an analysis of the current water delivery network would be required to determine potential interconnects needed to transport water to the regional water providers. The selected configuration will be developed, with stakeholder feedback, into a preliminary facility design.

1.2 FUNDING APPROACH

This report is an evaluation of alternative funding programs for the proposed water supply improvements. The alternatives evaluated herein focused on grants and principal forgiveness programs where possible, with some discussion of low interest financing options.

Gather Funding Agency Information

Black & Veatch has reviewed active funding programs, agency drivers and initiatives and the associated financial resources related to federal and state program to fund utility improvements. Black & Veatch staff members have discussed project elements with appropriate agency officials in an effort to provide the most current program information, as specific funding mechanisms are selected for further review.

Investigation and Financial Feasibility

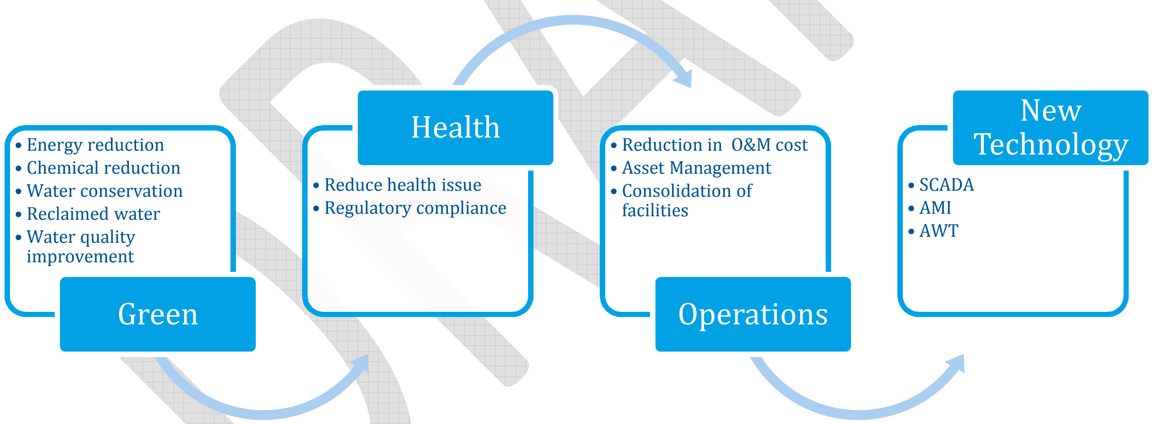
The analysis of fundable capital improvement elements includes available funds within each appropriate program, timelines associated with the availability of funding, proposed level of effort for application purposes, leverage requirements, percentage of match required, expected percentage of probability of success with each. These components are included under Funding Options and will be further developed as specific funds are selected.

Funding Scenario

Black & Veatch has created a Funding Scenario spreadsheet (Appendix A) that evaluates a range of likely options for both grant and low-interest financing and demonstrates local impact. This funding scenario spreadsheet will be developed in further detail as the scope and cost of the Program become defined. A comparison of the scenarios for funding, along with the full fund descriptions described later in this report, can be used as a decision making tool for the RGRWA to select programs to pursue.

1.3 AGENCY PRIORITIES

In our evaluation of funding sources, it is critical to review current initiatives of potential funding agencies, and discuss components of the SWMP that align with these initiatives. Funding agencies recognize the following four components as critical elements of capital improvement planning, design and construction. The four categories of Green, Health, Operations and New Technology are considerations for planning capital projects that may be eligible for alternative financial resources.



These aspects of the project will be discussed in terms of these guiding principles which may qualify for funding set aside specifically to meet these initiatives. The following elements may qualify this Program for funding focused on the above initiatives:

- A regional utility supply solution with potential for cooperative operation and management,
- Potential to benefit to a range of users including municipal, industrial, and agricultural, either by direct access to supply or an opportunity for reallocation of Rio Grande river water
- Increased access to a reliable source of high quality potable water for an economically disadvantaged region,
- Long term planning for sustainable growth,

- Inclusion of new technology, e.g. SCADA and advanced membrane filtration,
- Opportunity for optimization of energy resources by colocation with power generating facility or utilization of alternative energy sources,
- Increased reliability by increasing interconnectivity,
- Educational opportunities, specifically technical training programs for operations and maintenance of membrane filtration technology operated by one of the regional educational institutions,
- Potential improvements in Rio Grande ecosystem by diversification of sources.

The LRGV Strategic Water Management Program has been conceived on the basis of these same values, and will align with many funding programs that emphasize sustainability, resilience, and development in rural and/or economically disadvantaged areas.

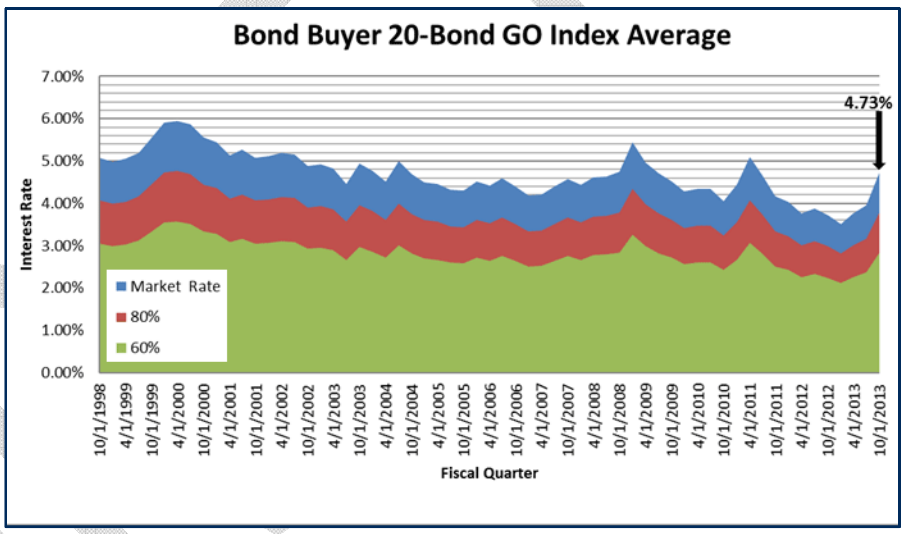
2 Funding Options

2.1 TEXAS WATER DEVELOPMENT BOARD (TWDB)

2.1.1 Drinking Water State Revolving Loan Fund Program (SRF)

An anchor funding program for utility projects within Texas is the Drinking Water State Revolving Loan Fund Program (DWSRF). The SRF program provides low-interest loans for planning, acquisition, design, and construction of water supply infrastructure projects. If RGRWA chooses to engage the SRF program, a Request for Inclusion (RFI) will be compiled and submitted to TWDB for consideration that includes a description of existing water facilities, facility needs, the nature of the project being considered, and project cost estimates. Loan applicants are ranked by the TCEQ using this and other data to establish project priority list for the state's annual Intended Use Plan. TWDB Staff will review the RFI and determine the projects "Project Priority Score" which is based on the Utilities ability to repay debt service.

The DWSRF Program provides funding for Preconstruction and Construction Loans. The Loan Terms include a 20-year amortization and low-interest rates. Financing rates vary based on the median household income, the poverty index, and the unemployment index, but average just over 50 percent of the market rate.



The DWSRF financing rate reported in the Bond Buyer 20-Bond GO Index for the full weeks occurring during the three months in the preceding fiscal quarter is determined by multiplying the market rate times the affordability index divided by 200. The maximum financing rate shall be limited to 80% of the market rate. The DWSRF financing rate is 60% of the market rate.

Eligible Project Sponsors

The RGRWA should apply as a regional utility provider for eligibility for a SRF loan for the proposed improvements. Projects eligible for loans include new construction of and improvements to eligible drinking water projects.

Green Project Reserve (GPR)

State Revolving Fund (SRF) programs generally include provisions to promote Green principles and technologies and require States to establish a Green Project Reserve (GPR). The GPR provision generally requires States to reserve not less than 20% of the annual federal allocation for SRF capitalization grants to address green infrastructure, water or energy efficiency improvements, or other environmentally innovative activities.

Projects meeting GPR criteria are subject to all SRF program requirements. Criteria for determining eligibility can be found in GPR Guidance Document. Projects clearly eligible for GPR are known as 'categorically eligible' projects. A list of 'categorically eligible' projects can be found in the GPR guidance document mentioned above. However, some traditional projects that are not determined to be 'categorically eligible' may have benefits that can be counted toward the 20% Green Project requirement. For traditional projects (or portions thereof) to be counted towards the 20% GPR requirement, the project files must contain documentation that satisfies the 'business case' criteria established by USEPA which includes projections of identifiable and measurable benefits associated with the use of 'green' technologies in the construction of the project.

GPR Business Case

A 'business case' needs to provide a well-documented justification for a project to be considered a GPR project. The required documentation could be a simple memo but must indicate the basis on which this project was judged to qualify to be counted toward the 20% requirement. Such a memo would typically include direct reference to a preliminary engineering or other planning document that makes clear that the basis upon which the project (or portion) was undertaken included identifiable and substantial benefits qualifying for the Green Project Reserve. For the further detail on how to develop a 'business case', see Part A, Section 5 of the GPR Guidance Document mentioned above. EPA requires States to post the approved business cases on the program website so that they are available to the public.

2.1.2 Regional Facility Planning Grant

The TWDB offers grants to political subdivisions of the State of Texas for studies and analyses to evaluate and determine the most feasible alternatives to meet regional water supply and wastewater facility needs, estimate the costs associated with implementing feasible regional water supply and wastewater facility alternatives, and identify institutional arrangements to provide regional water supply and wastewater services for areas in Texas. For FY2014, the total available is $500,000.

The proposed planning must be regional in nature by inclusion of more than one service area or more than one political subdivision. Grants for regional facility planning are generally limited to 50% of the total cost of the project, except that the board may supply up to 75% of the total cost to political subdivisions which have unemployment rates exceeding the state average by 50% or more, and which have per capita income which is 65% or less of the state average for the last reporting period available. In-kind services may be substituted for any part of the local share, if such services are directly in support of the planning effort, are properly documented, and approved in advance by the board. The application for this grant is underway.

2.1.3 Economically Distressed Area Program

The Economically Distressed Areas Program (EDAP) provides financial assistance to provide water and wastewater services to economically distressed areas where services do not exist or systems do not meet minimum state standards. Eligible applicants for the EDAP include cities, counties, water districts, nonprofit water supply corporations, and all other political subdivisions. The city or county where the project is located must adopt Model Subdivision Rules for the regulation of subdivisions prior to application for financial assistance. Projects must also be located in an economically distressed area where the median household income that is not greater than 75% of the median state household income. Financial assistance from the EDAP can be utilized for:

- planning,
- land acquisition,
- design, and
- construction of first-time service or improvements to water supply and wastewater collection and treatment works

The EDAP program provides financial assistance in the form of a grant or a combination grant/loan depending on the project's phase (planning, acquisition and design (PAD) or construction). Applicants seeking funding for the PAD phase can obtain 50% - 100% of the financial assistance in the form of a grant. Applicants seeking funding for the construction phase of a project may obtain a combination grant/loan. The amount of the loan is determined by a grant-to-loan calculation which is based on either the applicant's existing capital component or on regional benchmarks.

State law requires a determination of an existing health and safety nuisance issued by the Texas Department of State Health Services for grant funding greater than 50% from the EDAP. Board staff will process request for nuisance surveys for EDAP applicants once eligibility determinations have been made.

2.1.4 Agricultural Water Conservation Grant

The Agricultural Water Conservation Grants Program offers grants to state agencies and political subdivisions for technical assistance, demonstration, technology transfer, education, and metering projects that conserve water. Grant topics vary from year to year to address current issues and topics in agricultural water conservation. The goal of these projects is to implement designated agricultural irrigation conservation strategies in the state water plan and to demonstrate best management practices that may save water or improve water use efficiency.

2.2 FEDERAL FUNDING

Federal funding requests associated with the Clean Water Act of 1977 can be exercised. The federal act supports water projects associated with language provided in Section 201 of PL 92-500; Section 214 and Section 313. These sections provide for funding if the project has been studied and evaluated but also provides funding for public information, education and information.

Other federal resources include the United States Department of Environmental Protection; United States Department of Agriculture; Rural Utilities Service; and United States Department of Housing and Urban Development. These resources may be included in projects that meet the associated income limits and population requirements. Individual project aspects may be included in applications if census tracks qualify for funding.

2.2.1 Technical Assistance Grant (USDA-TAT)

This program provides contracts with a nonaffiliated organization for not more than 49 percent of the grant to provide the proposed assistance. Eligible purposes include: Grant funds must be used to capitalize a Technical Assistance and Training program for the purpose of: a. Identifying and evaluating solutions to water problems of associations in rural areas relating to source, storage, treatment, or distribution; b. Identifying and evaluating solutions to waste problems of associations in rural areas relating to collection, treatment, or disposal; c. Assisting associations in the preparation of water and/or waste loan and/or grant applications; d. Providing technical assistance and/or training to association personnel that will improve the management, operation and maintenance of water and waste disposal facilities; or e. Paying expenses associated with providing technical assistance and/or training.

2.2.2 Water and Waste Disposal Direct Loans and Grants (USDA-WWD)

The purpose of the USDA-WWD grant is to develop water and waste disposal systems in rural areas and towns with a population not in excess of 10,000. The funds are available to public bodies, non-profit corporations and Indian tribes.

To qualify, applicants must be unable to obtain the financing from other sources at rates and terms they can afford and/or their own resources. Funds can be used for construction, land acquisition, legal fees, engineering fees, capitalized interest, equipment, initial operation and maintenance costs, project contingencies, and any other cost that is determined by the Rural Development to be necessary for the completion of the project. Projects must be primarily for the benefit of rural users.

The rates that are used to calculate these loans are subject to change quarterly. Loans are made based on the applicant's authority and the life expectancy of the system's project, which may be up to the maximum of 40 years.

The material submitted with the application should include an application SF 424.2, two copies of the Preliminary Engineering Report, Environmental Report, population and median household income of the area to be served, current audits or financial information for the past three years, evidence of outstanding indebtedness, organizational documents, the applicant's IRS tax identification number, DUNS number, a proposed operating budget, and some certification forms. This loan program is based on repayment ability. These loans are calculated on similar systems rates, median household income, financial status of the system, and outstanding indebtedness. There are some systems that qualify for grant funding; however, grant funding availability is limited. Applicant contributions show ownership in the projects and are often recommended. These applicant contributions are the first money spent in any project.

2.2.3 US Economic Development Administration (USEDA)

The United States Economic Development Administration provides financial assistance to projects that foster job creation and attract private investment to support economic development or growth. The programs are designed as leverage of critical assets that support the economy and strategic economic drivers. The process is a competitive grant process where projects are evaluated for overall compliance with the USEDA initiatives.

USEDA's investment priorities provide goals and initiatives to guide the decisions of how the agency determines investment decisions and/or strategies. Applications should align with one of the multiple investment priorities:

- ***Public/Private Partnerships.*** Projects that use both public- and private-sector resources and leverage complementary investments by other government/public entities and/or nonprofits.

- ***Environmentally-Sustainable Development.*** Projects that promote job creation and economic prosperity through enhancing environmental quality and developing and implementing green products, processes, places, and buildings as part of the green economy. This includes support for energy-efficient green technologies.

Through the competitive grant process outlined in this funding opportunity, all proposed projects are evaluated to determine the extent to which they align with EDA's investment priorities, create or retain jobs, leverage public and private resources, demonstrate the ability to start the proposed project promptly and use funds quickly and effectively, and provide a clear scope of work and specific, measureable outcomes.

2.3 BORDER ENVIRONMENT COOPERATION COMMISSION

2.3.1 Border Environment Infrastructure Fund (BEIF)

Through the Border Environment Infrastructure Fund (BEIF), a maximum of 8 million dollars (for each project) is made available annually for the improvement of water and wastewater infrastructure needs in the U.S. –Mexico border region critical to health and environmental needs. Projects selected to receive a BEIF grant must complete project development activities, including obtaining environmental clearances and finalizing design, as well as obtain project certification from BECC and sign the grant agreement with North American Development Bank (NADB) within two and a half (2.5) years of receiving notification of project selection. Moreover, the project must be able to complete construction within three (3) years following the signing of the BEIF grant agreement for construction funding. Project sponsors are generally expected to finance part of the project with a debt component and must be able to confirm the commitment of other funding sources to complement the BEIF grant prior to certification.

2.3.2 Community Assistance Program (CAP)

CAP grants are available for public projects in all environmental sectors eligible for NADB financing, provided that they meet the following criteria:

- The project must be located in the U.S.-Mexico border region, defined as the area within 100 kilometers north and 300 kilometers south of the international boundary between the United States and Mexico.
- The project sponsor must have little capacity to incur debt.
- The project must benefit communities (i) in the United States with median household income (MHI) at or below the average of the MHI of U.S. communities in the border region or (ii) in Mexico with average household income at or below the average household income of Mexican communities in the border region.

Priority will be given to drinking water, wastewater and solid waste infrastructure. Projects that receive grants from the Border Environment Infrastructure Fund (BEIF) are NOT eligible for grants from the CAP.

General Financing Requirements

Projects selected to receive a CAP grant must comply with the following funding requirements:

- Projects must obtain certification from the Border Environment Cooperation Commission (BECC).
- The project sponsor must contribute at least 10% of the total project cost in the form of cash. On a case-by-case basis, in-kind contributions such as land, equipment, or other tangible assets or cost components of a project may be considered towards fulfilling this contribution.

Grant Amount and Uses

Projects may receive a CAP grant for up to $500,000. The grant proceeds may be used for project construction and related costs, including final design, project management and supervision, as well as other project components, such as equipment.

2.4 FEDERAL LEGISLATIVE GRANT APPROPRIATION (FLGA)

The Federal Legislative Earmark Request is available to county, cities and towns throughout the nation. The process begins with a written document that describes the project scope and detail. Black & Veatch recommends that following submittal of the written request, we meet with representatives and make a formal request for project support.

The model of packaging a legislative request and submitting directly thru the legislative delegation is a proven capital funding technique in which the project and it's unique features are highlighted, explained, analyzed and presented for consideration. The presentation is developed in such a way that it shows similarities to past grant or legislative funded projects, area wide commitment and important strategic project information that allows easier approvals. The packaging process results in providing agencies or legislatures with the answers to all possible questions that could be asked about the project.

A funding request package does all the necessary background leg work for the agency personnel, legislative staff or legislative member. It becomes the link between the project's financial reality and its engineering details. It makes defending the funding request easier and therefore, more likely to happen. As an agency funding cycle moves ahead or a legislative calendar advances, requests for project information that can differentiate your project from its competitors, come at unpredictable times. The package provides the details needed to make a quick, favorable impression on those forced to pick between a number of worthy projects.

2.5 LEVERAGE FUNDING

For many communities, funding is provided through federal, state and regional opportunities that may provide only a portion of the necessary funds to complete an environmental program. In many cases, communities choose to accumulate funding over multiple years, allocating these funds as pledged revenue to federal opportunities. Eligible projects may include multiple layers of leveraged funds to off-set pledged revenue requirements. Projects that are considered multi-jurisdictional may claim leverage from eligible funding programs offered by independent jurisdictions but earmarked for the same overall common goal. The creative leveraging techniques used for federal funding often lessen the overall financial burden to the owner's annual budget.

2.5.1 Public Private Partnerships (P3)

The contract operator for the facility is a private group; the customer base includes large agricultural operations, etc. Public Private Partnership (P3) benefits can be broken into three categories:

- Source of stable capital
  - Address bonding capacity issues
  - Doesn't increase Debt Coverage requirements
  - Provides ability to smooth rate increases over time
- Risk mitigation opportunity - Improved certainty of future performance around
  - Regulatory Compliance
  - Asset Management and Operation & Maintenance practices
  - Interest rate mitigation
  - Capital project delivery cost overruns
- Other benefits
  - Asset Ownership does not necessarily need to change
  - Allows another avenue of funding for a utility's green initiatives associated with the asset.

2.6 ADDITIONAL FUNDING RESOURCES

The following Agencies may provide viable funding programs which match this drinking water regionalization program. These programs should be evaluate once the planning document is complete and subsequently identifies critical elements of the overall program.

2.6.1 State Water Implementation Fund for Texas (SWIFT)

The SWIFT will not be available until March of 2015 at the earliest, when there will be $2 Billion made available for low interest financing for water infrastructure and conservation projects across Texas. Projects funded must be included in the State Water Plan, and 10% of the funds are reserved for each rural initiative, and 20% reserved for conservation projects. All projects must be included in the state Water Plan, and projects will be prioritized based on the decade of need, feasibility, viability, sustainability, and cost-effectiveness.

2.6.2 Water Infrastructure Financing Innovation Act (WIFIA)

The Water Resources Development Act of 2013 under Senate Bill 601 passed by a vote of 83-14 and now resides in the House of Representatives. The bill includes the following language:

"Water Infrastructure Financing Innovation Act (WIFIA): This new five-year pilot program would allow water and wastewater utilities to apply for low-interest financing via the federal government to construct or improve local water and wastewater infrastructure. The Environmental Protection Agency has estimated that the shortfall in funding for water and wastewater needs will exceed $540 billion in the next 20 years. Modeled after the successful federal transportation (TIFIA) loan program, WIFIA will lower the cost of borrowing for local drinking water and wastewater management entities with major

projects. According to the American Water Works Association the program will allow these entities to leverage $10 for every $1 of federal investment."

WIFIA is expected to have competitive interest rates with the State Revolving Loan Fund programs and terms that reach 35 years where the SRF programs reach 20 years. These flexible terms and conditions make the program attractive as it lessens the annual debt service. Another attractive feature is that WIFIA does not require the same planning and administration requirements as the SRF program. These requirements take time and add costs to the overall capital project. The WIFIA process is expected to eliminate the need for the extended schedule or added cost of both the planning and administration.

3 Next Steps

The funding alternatives described here and the initial ranks given should be reviewed by the Authority and used to determine the course of action. The funds selected by RGRWA will be further evaluated, and the AFS team may demonstrate funding agency requirements, notice of funding availability expectations, administrative requirements (red flags) and process requirements, as agreed upon with the Authority. The AFS team will provide insight to the action plan based upon findings and best selected alternative for project financing

3.1 APPLICATION AND ADMINISTRATION

An evaluation of funding programs as it related to the Strategic Water Management Program is included in this report. The following is a list of ongoing and future activities associated with the funding component of this project:

3.1.1 Capital Funding Evaluation

The funding evaluation process is ongoing, including detailed data investigation, supporting documentation, project schedules, potential legislative agendas, discussion of agencies known to participate in utilities, agency requirements, agency milestones, and expected follow-up items.

3.1.2 Funding Matrix

As the Program details and more specific Capital Funding information become available, those data will be used to add further detail to the Funding Matrix. Desired utility capital improvements as identified by Owner will be aligned with available Funding Agencies to participate in specific project elements.

3.1.3 Preconstruction Funding Activities

RGRWA staff will identify desirable funding options for pursuit of funding approval through formal application based on the alternatives presented in this document. Black and Veatch will prepare the required funding application to each Agency as identified by the Owner on a contract basis.

3.1.4 Funding Management Description

Black and Veatch may provide funding services to include preliminary funding source identification, application development and funding administration. Black and Veatch will prepare and submit funding applications as selected by RGRWA. Proposed applications are identified in the recommendations section of this document.

It is anticipated RGRWA will provide support and collaboration for funding application development including required approvals and financial information necessary for application. RGRWA will designate a funding lead to serve as the point of contact for funding identification and development activities. Planning Services will be performed as required by Funding Programs.

State and Federal funding programs require facilities planning or a preliminary engineering report in support of any funding request. Also required may be an environmental repost which describes environmental effects as a result of implementing said project.

The Program Manager will provide funding administration services for any approved program funds (loans and/or grants) aside from Program Owner general financing through bonds or other Program Owner resources. Funding administration service requirements will be identified during the conceptual design phase and included in the future construction phase services contract. Example administration tasks:

Appendix A Available Funding Scenarios

The following chart depicts infrastructure projects, as required. Estimated on $10,000,000 project cost:

RGRWA			FUNDING SCENARIOS			
			Project:	Rio Grande Regional Water Supply		
		Funding Options	*Scenario 1*	*Scenario 2*	*Scenario 3*	*Scenario 4*
CUSTOMERS (EDU)			100,000	100,000	100,000	100,000
TOTAL PROJECT COST			$ 10,000,000	$10,000,000	$10,000,000	$10,000,000
TOTAL ANNUAL OMR			$ 50,000	$ 50,000	$ 50,000	$ 50,000
FINANCING						
USDA RD			$ -	$ -	$ 2,000,000	$ 2,000,000
BEIF			$ -	$ -	$ -	$ 700,000
FLGA			$ -	$ -	$ -	$ 500,000
SEED			$ -	$ -	$ -	$ 50,000
GREEN			$ -	$ -	$ -	$ -
USEDA			$ -	$ -	$ -	$ -
Local Funds (Cash, Tap Fees, Etc.)			$ -	$ -	$ -	$ -
TWDB	20	2.60%	$ -	$10,000,000	$ -	$ 5,250,000
GREEN	20	2.00%	$ -	$ -	$ -	$ 1,500,000
COMM	20	6.70%	$ -	$ -	$ -	$ -
BOND	20	5.50%	$ 10,000,000	$ -	$ -	$ -
USDA	40	2.40%	$ -	$ -	$ 8,000,000	$ -
Total Financing			$ 10,000,000	$10,000,000	$10,000,000	$10,000,000
ANNUAL DEBT						
Annual TWDB Payment			$ -	$ 647,546	$ -	$ 339,962
Annual GREEN Payment			$ -	$ -	$ -	$ 91,735
Annual COMM Payment			$ -	$ -	$ -	$ -
Annual BOND Payment			$ 836,793	$ -	$ -	$ -
Annual USDA Payment			$ -	$ -	$ 313,346	$ -
Reserve			$ -	$ -	$ 31,335	$ -
ANNUAL DEBT & OMR			$ 886,793	$ 697,546	$ 394,681	$ 481,697
Total Future Av. Mo. Cost Per Customer			$ 0.74	$ 0.58	$ 0.33	$ 0.40
Total TWDB Payback			$ -	$12,950,927	$ -	$ 6,799,237
Total GREEN Payback			$ -	$ -	$ -	$ 1,834,702
Total COMM Payback			$ -	$ -	$ -	$ -
Total BOND Payback			$ 16,735,866	$ -	$ -	$ -
Total USDA Payback			$ -	$ -	$12,533,848	$ -
*Blue font indicates formula cell.						
** Loan terms and interest rates can be changed and payments/payback will change accordingly.						
***Example Funding Scenarios						

Appendix B Funding Application Requirements

FEDERAL APPLICATION EXAMPLES

RGRWA must submit a complete application package including all required documents necessary based on proposed project request. Applications must be submitted electronically before the closing deadline. Applications may be submitted for pre-construction and/or construction elements based on project need. Examples of required documentation for application completion are as follows:

I. Form SF-424: Application for Federal Assistance

II. Form SF-424C: Budget; pre-construction allowances; special studies; legal; and other costs associated with project execution

III. Form SF-424D: Assurances

IV. Form CD-511: Lobbying Certification

V. Form SF-LLL : Lobbying Disclosure

VI. Non-Federal Documentation – may include match or shared cost documentation

VII. Form ED-900 Financial Documentation

VIII. Compliance with Executive Order 12372 – Clearinghouse Review

IX. Project Site Maps

X. Commitment and Compliance Assurances

XI. Preliminary Engineering Report

XII. Environmental Reports

XIII. All Federal Compliance Approvals

XIV. Pre-Application Consultation Review

XV. Any other documentation required or requested by individual agencies.

Prepare for administrative requirements to include Davis-Bacon and Buy American. These processes may require additional costs and should be included in all front end documents prior to release for bid.

STATE APPLICATION EXAMPLES

Planning

- Project Identification and Information
- Need Identification
- Technical Planning Requirements
- Environmental Analysis
- Benefit Cost Analysis
- Life Cycle Cost Analysis
- Match or Leverage Requirements
- Expenditure Based Budget
- Capital Budget
- Work Plan
- Supporting Documentation (Public Participation)
- Clearinghouse Review

Administration Compliance

- Request for Inclusion
- Application/Agreement
- Bidding (Funding Requirements)
- Construction Award
- Construction Cost Eligibility Review
- Change Order Review
- Davis-Bacon Act (Payroll Review, Labor Interviews, Additional Job Classification)
- Disadvantage Business Enterprise
- Technical Services
- EEO Requirements
- E-Verify
- Disbursement Requests
- Audit Review
- Closeout Documents

CHAPTER 14
ORGANIZATIONAL STRUCTURE

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

9 MARCH 2016

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14.0 Organizational Structure

The goal of this project is to identify an affordable regional water system to meet the growing needs of the Lower Rio Grande Valley through the year 2070. The technical portions of this analysis have described water resources, treatment and conveyance facilities to meet the goal. The purpose of this chapter is to explore alternatives to meet the needs for administration, ownership and operations of those facilities. Within this chapter an organizational structure and staffing plan is recommended to meet the goals of the program.

14.1 CURRENT RGRWA ROLE

The Rio Grande Regional Water Authority was created by the Texas Legislature in 2003 as a conservation and reclamation district "to serve a public use and benefit" by bringing together regional water interests to accomplish projects and services within Willacy, Cameron, Hidalgo, Starr, Zapata, and Webb counties (excluding the City of Laredo). The RGRWA is governed by an 18 member board representing irrigation districts, the public, municipal class entities, water supply corporations and counties. They have the ability to contract for, fund, own and operate water treatment facilities. The RGRWA does not currently have infrastructure for water supply but acts as a planning organization to facilitate regional planning for inclusion in the state water plan.

The organization has recently created an Infrastructure Improvement Council through House Bill 3545 in the 2015 Texas legislative cycle. This Council will allow the RGRWA partners to form under the RGRWA a subgroup of stakeholders for participation in a regional program as described in this report. The Council would be able to utilize the RGRWA's authority to own and operate water supply infrastructure and limit the voting authority and be governed by the members of the RGRWA who are members of the Council.

It is assumed that the RGRWA will be in a leadership role in the execution of this regional system since that is the stated function of the organization.

14.2 ORGANIZATIONAL OPTIONS TO MEET REGIONAL WATER NEEDS

The overarching options to meet the overall water needs of the region include:

- Do nothing
- Increase sub-regional activity, proactively:
- Privatization
- Regionalization

The "Do Nothing" approach will leave the market to drive solutions for each individual water supplier. This will be accomplished through privatization, augmentation of sub regional systems, or state mandated solutions. This uncoordinated approach may lead to conflicts between water suppliers, costly development of water resources for entities not in close proximity to resources, and require expensive solutions as resources become limited.

Increasing sub regional activity proactively through existing stakeholders requires utilizing the regional planning group for Region M to encourage sub regional water suppliers to consolidate water supply projects. Sub regional suppliers SRWA, McAllen, Harlingen, NAWSC and others fill this role; however, as supplies become limited and further from their use, the cost of water conveyance will become burdensome on their customers.

The privatization of water resources can bring needed water to the area, and transfer the project development and operations to a private investment firm(s). The disadvantages of privatization include: higher costs due to funding limitations and profit expectations, and general business risks associated with private organizations. Privatization would require an entity to contract with. Presumably the RGRWA could fill that role as well.

Regionalization of water resources may provide the benefits of consolidating resource infrastructure and operation along with supply planning while spreading the conveyance costs across an entire region of water users. This could be accomplished through the RGRWA and its Infrastructure Improvement Council or a new entity. It is recommended that the RGRWA fill this role because it is already in place with governance and creating a new entity with this same role isn't necessary.

The RGRWA's role is recommended to expand to provide drinking water, water conservation support, water rights management, and contract operations. RGRWA's internal operations would include developing and operating water supply projects. The RGRWA would be able to contract with regional stakeholders to provide drinking water, reuse water, water rights, and O&M for regional facilities. Figure 14-1 illustrates the recommended role of the RGRWA in the regionalization of the water resource projects.

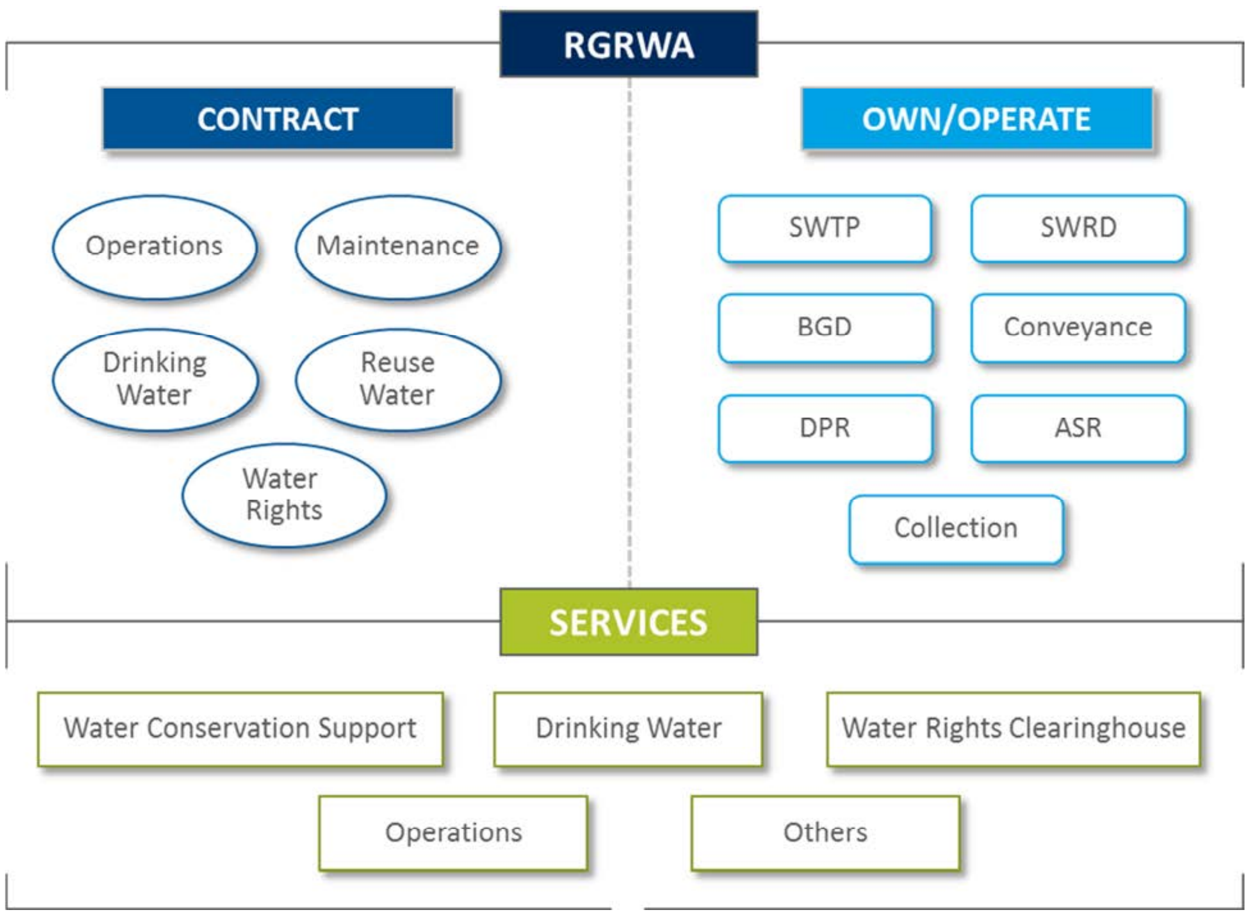

Figure 14-1 RGRWA Roles

The recommended approach will ultimately end in the RGRWA owning and operating the treatment and conveyance facilities, purchasing water, and/or contracting the operations of the facilities.

There are a couple of models for dividing ownership and operations of each regional facility.

- Purchase water contracts
- Own/Operate
- Own/Contract Operations
- Operate regional plants for others

Table 14-1 describes the advantages and disadvantages of each assuming RGRWA serves as the program manager.

Table 14-1 Operations and Ownership Analysis

	ADVANTAGES	DISADVANTAGES
Purchase Water Contracts RGRWA purchases water from existing water suppliers	• Infrastructure location is closer to existing staff • Treatment staff in place	• Multiple contracts • Water quality/pressure • Large project funding will negatively affect debt ratios for potential owners
Own/Operate Infrastructure is owned and operated by RGRWA	• Single contract • Water quality/pressure standardization • Consolidation of resources/staff • Advanced treatment operations knowledge transfer • Consolidate debt to regional water supplier	• Infrastructure spread out and staffing spread out
Own/Contract Operations Infrastructure would owned by RGRWA while operations would be contracted to others	• Infrastructure location is closer to existing staff • Water quality/pressure standardization • Consolidate debt to regional water supplier	• Multiple contracts
Operate Regional Plants for Others The infrastructure would be owned by regional stakeholders, but operated by RGRWA Staff	• Water quality/pressure standardization • Consolidation of resources/staff • Advanced treatment operations knowledge transfer	• Multiple contracts • Infrastructure spread out and staffing spread out • Large project funding will negatively affect debt ratios for potential owners

If the identified regional supply projects are managed under the RGRWA, as recommended, water quality and pressure requirements can be standardized. The RGRWA will have the ability to analyze each supply project and determine which ownership/operation model to utilize.

In early phases of the program, the RGRWA should consider purchasing water contracts, and contracting operations for facilities to allow for knowledge transfer and organic staff augmentation.

14.3 AGENCY ANALYSIS

The following entities are potential owners or operators of regional water supply infrastructure because of their proximity to regional water supplies and current ability to operate the proposed treatment facilities.

- Brownsville Public Utility Board
- East Rio Hondo Water Supply Corporation
- Harlingen
- McAllen
- Military Highway Water Supply Corporation
- North Alamo Water Supply Corporation
- Southmost Regional Water Authority

Table 14-2 provides the current functions of the current water suppliers evaluated.

Table 14-2 Current Status of Water Suppliers

SUPPLY	OWN	OPERATE	FUND	SUPPLY
Brownsville PUB	●	●	●	●
East Rio Hondo Water Supply Corporation	●	●	●	●
Harlingen	●	●	●	●
McAllen	●	●	●	
Military Highway Water Supply Corporation	●	●	●	●
North Alamo Water Supply Corporation	●	●	●	●
Southmost Regional Water Authority	●	●	●	

Regional suppliers and operators should be selected based on both their proximity to the facilities and their existing capabilities to fund and operate them.

14.4 RGRWA STAFFING

In order to fulfill the recommended role in the regional supply program, the RGRWA will need to increase staffing levels. Current staffing includes an empty executive director position, Board of

Directors made up of municipal water suppliers and irrigation districts, and administrative staffing donated by the Rio Grande Valley Partnership.

The proposed increase in RGRWA staffing is based on similar water supply organizations and increases as water supply and infrastructure is constructed. Figure 14-2 shows the proposed organizational structure in 2070. In 2016, it is assumed that an executive director, operations director, a senior administrator and a grant writer would be necessary in addition to plant staffing.

Plant staffing depicted in earlier chapters was developed with the assumption that plant staff would not be shared amongst the facilities. It is anticipated that some reduction in staffing would be expected if the operations of nearby facilities are conducted by the same organization but is not taken into account in this analysis because it is unknown at this time what facilities may be contract operated. The recommended staffing required for operations and maintenance of the infrastructure is consolidated in Table 14-3 below.

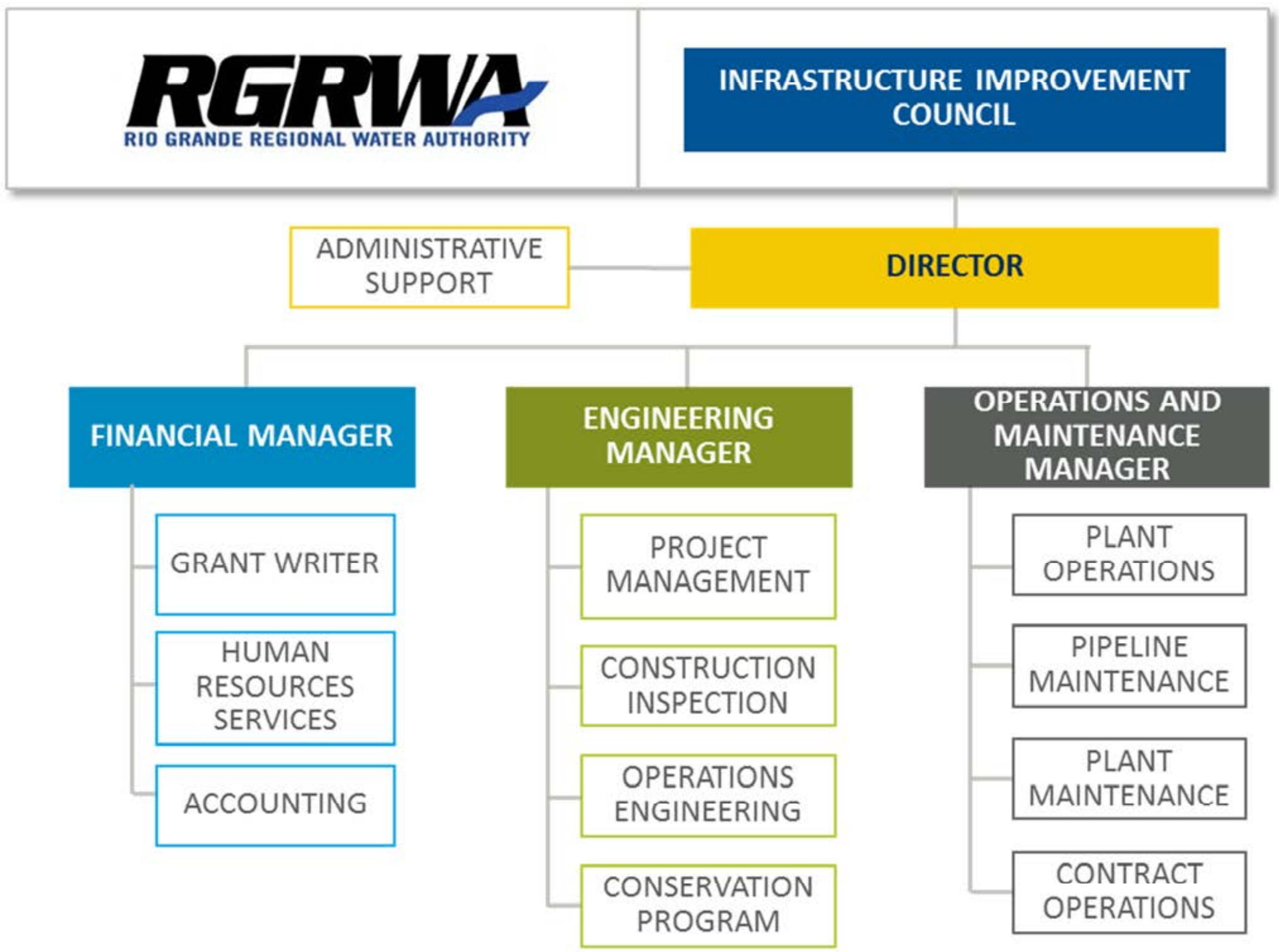

Figure 14-2 RGRWA Organizational Structure

Table 14-3 Plant and Conveyance Operations and Maintenance Staff

	FACILITY	Plant Manager	Operator Tech 1	Operator Tech 2	Operator Tech 3	Operator Tech 4	I&C	Maintenance Mechanic	Assistant Plant Manager	Operations Manager	Laborer	Admin Assistant	TOTAL STAFF
2020	Cameron BGD	1	1	1	1	1	1	1	0	0	0	0	**7**
	BNC SWRO	1	2	6	1	1	2	3	1	1	2	1	**21**
	Total	3	4	8	2	2	4	5	1	1	2	1	33
2030	Cameron BGD	1	1	1	1	1	1	1	0	0	0	0	**7**
	Hidalgo BGD	1	1	1	0	0	1	1	0	0	0	0	**5**
	BNC SWRO	1	2	6	1	1	2	3	1	1	2	1	**21**
	SWTP	1	1	1	1	1	1	1	0	0	0	0	**7**
	Aquifer Storage	1	1	1	1	0	1	1	0	0	0	0	**6**
	Total	5	6	10	4	3	6	7	1	1	2	1	46
2040	Cameron BGD	1	1	1	1	1	1	1	0	0	0	0	**7**
	Hidalgo BGD	1	1	1	0	0	1	1	0	0	0	0	**5**
	BNC SWRO	1	2	6	1	1	2	3	1	1	2	1	**21**
	SWTP	1	2	2	2	2	1	2	0	0	0	0	**12**
	Aquifer Storage	1	1	1	1	0	1	1	0	0	0	0	**6**
	DPR	1	2	2	2	2	1	2	0	0	0	0	**12**
	Total	6	9	13	7	6	7	10	1	1	2	1	63
2050	Cameron BGD	1	1	1	1	1	1	1	0	0	0	0	**7**
	Hidalgo BGD	1	1	1	0	0	1	1	0	0	0	0	**5**
	BNC SWRO	1	3	8	1	1	2	3	1	1	2	1	**24**
	GC SWRO	1	3	8	1	1	2	3	1	1	2	1	**24**
	SWTP	1	3	3	2	2	2	2	0	0	0	0	**15**
	Aquifer Storage	1	1	1	1	0	1	1	0	0	0	0	**6**
	DPR	1	4	4	4	4	2	4	0	0	0	0	**23**
	Total	7	16	26	10	9	11	15	2	2	4	2	104
2060	Cameron BGD	1	1	1	1	1	1	1	0	0	0	0	**7**
	Hidalgo BGD	1	1	1	0	0	1	1	0	0	0	0	**5**
	BNC SWRO	1	3	8	1	1	2	3	1	1	2	1	**24**
	GC SWRO	1	4	10	1	1	3	4	1	1	3	2	**31**
	SWTP	1	3	3	2	2	2	2	0	0	0	0	**15**
	Aquifer Storage	1	1	1	1	0	1	1	0	0	0	0	**6**
	DPR	2	5	5	5	5	3	5	0	0	0	0	**30**
	Total	8	18	29	11	10	13	17	2	2	5	3	118
2070	Cameron BGD	1	1	1	1	1	1	1	0	0	0	0	**7**
	Hidalgo BGD	1	1	1	0	0	1	1	0	0	0	0	**5**
	BNC SWRO	1	3	8	1	1	2	3	1	1	2	1	**24**
	GC SWRO	1	6	14	1	1	3	5	1	1	4	2	**39**
	SWTP	1	3	3	2	2	2	2	0	0	0	0	**15**
	Aquifer Storage	1	1	1	1	0	1	1	0	0	0	0	**6**
	DPR	2	6	6	6	6	3	6	0	0	0	0	**35**
	Total	8	21	34	12	11	13	19	2	2	6	3	131

14.5 ADMINISTRATIVE COSTS

The administrative cost by decade is described in Table 14-4. All salaries are approximate 2016 dollars and increases were included based on the number of staff managed. All benefits and overhead is included in the assumed 40% burden. Office space was assumed to reside within the

Cameron County BGD Plant Offices initially and would then transfer to the Hidalgo Regional SWTP once constructed. The cost for this space was included in the infrastructure costs already detailed in previous chapters. The administrative costs are summarized in Table 14-5.

Table 14-4 Administration Costs by Decade

Position	Qty	Salary	Burden	Cost
2020				
Executive Director	1	$115,000	$46,000	$161,000
Operations Director	1	$90,000	$36,000	$126,000
Accountant/Grant Writer	1	$50,000	$20,000	$70,000
Administrator	1	$40,000	$16,000	$56,000
Total	**4**			**$413,000**
2030				
Executive Director	1	$130,000	$52,000	$182,000
Operations Director	1	$105,000	$42,000	$147,000
Engineer Manager	1	$100,000	$40,000	$140,000
Accountant	2	$55,000	$44,000	$154,000
Administrator	2	$45,000	$36,000	$126,000
Total	**7**			**$749,000**
2040				
Executive Director	1	$150,000	$60,000	$210,000
Operations Director	1	$140,000	$56,000	$196,000
Facility Manager	1	$140,000	$56,000	$196,000
Engineering Manager	1	$120,000	$48,000	$168,000
Engineer	2	$100,000	$80,000	$280,000
Finance Director	1	$110,000	$44,000	$154,000
Human Resource Director	1	$110,000	$44,000	$154,000
Accountant	2	$75,000	$60,000	$210,000
Administrator	1	$75,000	$30,000	$105,000
Total	**11**			**$1,673,000**
2050				
Executive Director	1	$150,000	$60,000	$210,000
Operations Director	1	$140,000	$56,000	$196,000
Facility Manager	1	$140,000	$56,000	$196,000
Engineering Manager	1	$120,000	$48,000	$168,000
Engineer	2	$100,000	$80,000	$280,000
Finance Director	1	$110,000	$44,000	$154,000
Human Resource Director	1	$110,000	$44,000	$154,000
Accountant	3	$75,000	$90,000	$315,000
Administrator	2	$75,000	$60,000	$210,000
Total	**13**			**$1,883,000**

Position	Qty	Salary	Burden	Cost
2060				
Executive Director	1	$150,000	$60,000	$210,000
Operations Director	1	$140,000	$56,000	$196,000
Facility Manager	1	$140,000	$56,000	$196,000
Engineering Manager	1	$120,000	$48,000	$168,000
Engineer	3	$100,000	$120,000	$420,000
Finance Director	1	$110,000	$44,000	$154,000
Human Resource Director	1	$110,000	$44,000	$154,000
Accountant	4	$75,000	$120,000	$420,000
Administrator	3	$75,000	$90,000	$315,000
Total	**16**			**$2,233,000**
2070				
Executive Director	1	$150,000	$60,000	$210,000
Operations Director	1	$140,000	$56,000	$196,000
Facility Manager	1	$140,000	$56,000	$196,000
Engineering Manager	1	$120,000	$48,000	$168,000
Engineer	4	$100,000	$160,000	$560,000
Finance Director	1	$110,000	$44,000	$154,000
Human Resource Director	1	$110,000	$44,000	$154,000
Accountant	5	$75,000	$150,000	$525,000
Administrator	4	$75,000	$120,000	$420,000
Total	**19**			**$2,583,000**

Table 14-5 Administrative Cost Summary by Decade

DECADE	QTY	COST
2020	4	$413,000
2030	7	$749,000
2040	11	$1,673,000
2050	13	$1,883,000
2060	16	$2,233,000
2070	19	$2,583,000

CHAPTER 15 PROGRAM IMPLEMENTATION

Regional Facility Plan

B&V PROJECT NO. 181092

PREPARED FOR

Rio Grande Regional Water Authority

9 MARCH 2016

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15.0 Program Implementation

15.1 PURPOSE

The objective of this plan is to facilitate timely and effective program implementation in order to meet the regions water demands by 2020 and the next 50 years. The purpose of this chapter is to layout a program implementation plan for the various water treatment plant and conveyance projects recommended in the previous chapters.

15.2 ASSUMPTIONS

The implementation plan includes major tasks required for project execution, from planning and construction to startup. The schedule is setup into four major components: pilot studies, preliminary and final design, construction and operation and maintenance. The times allotted for each task are based on general industry standards and B&V's experience on similar projects. The schedule assumes one year for pilot studies, two years for preliminary and final design and three years for construction, commissioning and startup activities. These estimates are conservative for the types of projects defined in the plan; however, they will allow schedule float for extended permitting, ROW acquisition or legal processes if necessary. It is assumed most of the permitting and funding activities follow concurrently during pilot studies and preliminary design phase. Permit processes for federal, state and government agencies typically take one to two years to complete. Other permits may also be required; however they may have less time impact on the schedule. Real estate, easements and water rights acquisitions activities will follow prior to design. The schedule assumes a traditional design-bid-build delivery method. Other delivery methods, such as design-build were not evaluated at this time.

15.3 INFRASTRUCTURE PHASING SUMMARY

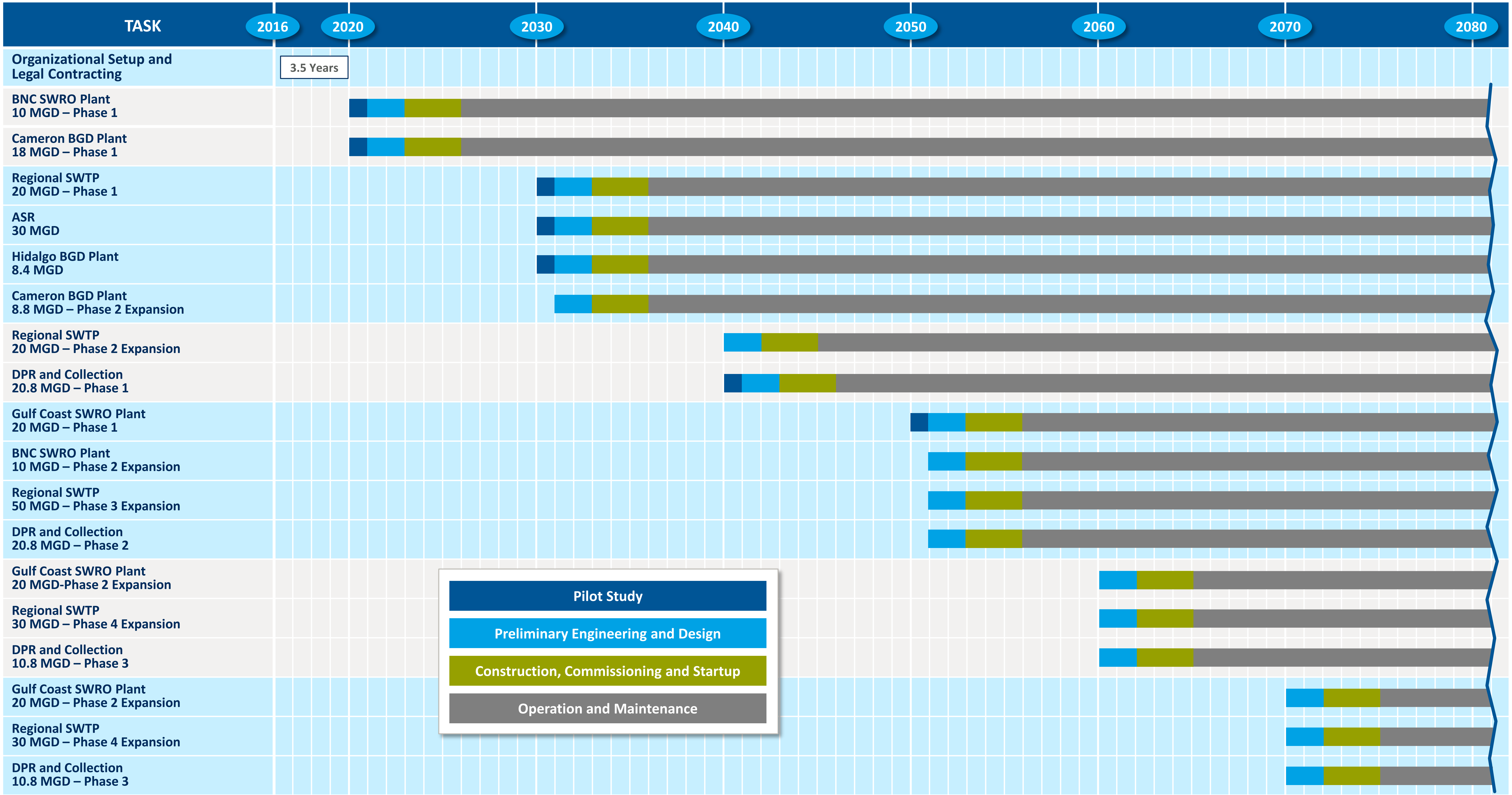
The regional facility plan recommends various water infrastructure projects. These projects were phased by the project team to meet the region's 50-year water needs. Table 15-1 below provides the summary of these projects.

Table 15-1 Project Description Summary

PROJECT DESCRIPTION	UNIT	EXPANSION					
		2020	2030	2040	2050	2060	2070
Cameron Brackish Groundwater Desalination (BGD) Plant	MGD	18	9				
Hidalgo Brackish Groundwater Desalination (BGD) Plant	MGD		10				
Brownsville Navigation Channel (BNC) SWRO Plant	MGD	10			10		
Gulf Coast SWRO Plant	MGD				20	20	40
Regional Surface Water Treatment Plant (SWTP)	MGD		20	40	50	30	20
Direct Potable Reuse (DPR) and Collection	MGD			21	21	10	10
Aquifer Storage & Recovery (ASR)	MGD		30				
Pipeline Conveyance (24" to 84")	L.F.	598,500	151,000	0	97,000	236,000	143,000

The implementation schedule for water treatment and conveyance facilitates are presented in Figure 15-1 and Figure 15-2 below. Since water infrastructure projects typically require several years to complete, it is recommended for the local and regional planning groups to prioritize organization and legal setup and establish policies and procedures for ownership and operations of the projects over the next few years. During this phase it is necessary to perform pre-implementation activities, such as negotiating contracts, applying for funding and securing finances, water rights, etc.

Lower Rio Grande Valley Regional Facility Implementation Master Plan



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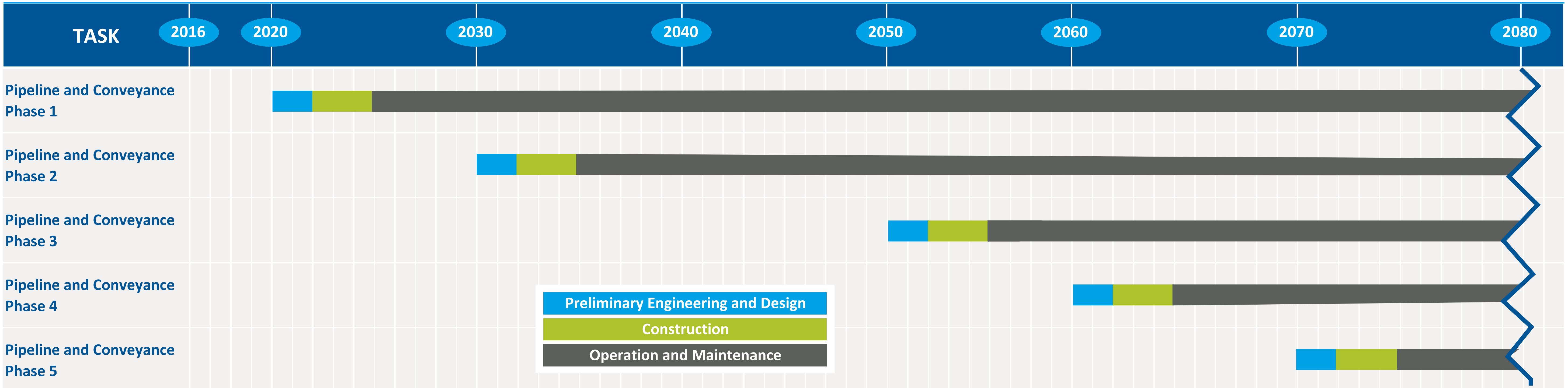
Lower Rio Grande Valley Regional Facility Pipeline Conveyance Implementation Schedule



PIPELINE CONVEYANCE SUMMARY							
PIPE SIZE	UNIT	2020	2030	2040	2050	2060	2070
20 inch	L.F.	31,000	90,000				
24 inch	L.F.		19,000			19,000	
36 inch	L.F.	286,000					
42 inch	L.F.	194,500				42,000	
48 inch	L.F.	61,000			29,000		32,000
60 inch	L.F.					175,000	111,000
78 inch	L.F.		42,000		42,000		
84 inch	L.F.	26,000			26,000		
	TOTAL	598,500	151,000	0	97,000	236,000	143,000

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15.4 PHASE I IMPLEMENTATION SCHEDULE

A typical phase 1 project implementation will have the following phases:

- Organizational Development
- Preliminary and detailed design
- Permits and Regulatory Approvals
- Bidding Period
- Construction

The Organizational Structure chapter discusses alternative organizational arrangements that could be used as the overseer for the development of the facility plan. It is assumed that the Infrastructure Improvement Council will be created under the RGRWA in accordance with House Bill 3545 which enabled its creation. If the RGRWA decides to move forward with the Regional Facility Plan, it is assumed that it will take approximately two years to form the Council, and develop the fees and structure necessary to implement the project. Full time staff are anticipated to lead the design and operational efforts and provide oversight for all of the contracts.

Pilot studies provide the opportunity to evaluate the performance of proposed treatment system under site-specific conditions. Data gathered from the pilot studies are used in the planning and design process and adjustments are made accordingly. Pilot testing is typically performed for a period of 6 to12 months. Historically, pilot testing has been required for permitting approval by Texas Commission of Environmental Quality (TCEQ) for implementation of membrane treatments of brackish groundwater, sea water, and reuse water. Recent changes in TCEQ regulations may allow for desktop analysis of membrane treatment; however, pilot testing also reduces project risk. For this reason it is recommended for all treatment projects. Aquifer storage and recovery will also be pilot tested to evaluate extent of storage and recovery, groundwater quality and address other technical uncertainties. Pilot testing setup depends on the source water quality and the size of the plant. For brackish groundwater desal, one pilot train per membrane manufacturer is typically standard, however desalination may require more than one treatment train. Various RO membranes can be tested during this phase.

Final water quality goals, plant capacity, and all design parameters are established during preliminary design phase of the project. Alternative analysis, conceptual design, desktop cultural and environmental investigations, cost evaluations, survey, and geotechnical investigations are performed in the preliminary phase. During the detailed design phase technical processes are clearly defined. Drawings and specifications are developed to include equipment, materials, systems, quality and performance goals. Typically these activities require up to two years to complete, but may vary depending up on the size and complexity of each project.

In addition to the technical details, it is important to include field cultural and environmental assessments, permitting, regulatory approvals, and funding applications concurrently with the design phase. Environmental assessments are required to identify and mitigate potential environmental risk associated with the construction and operation of the project. Multiple permits from federal, state and local agencies are anticipated depending upon the source water, type of facility, environmental discharge, extent of disturbance, and historic significance of the area. It is necessary to work with relevant regulatory agencies like EPA, TCEQ, USACE, US Fish and Wildlife, etc., to obtain needed permits. Other permits like, building permits, site work, roadway crossing, etc., are required after final design drawings are available or during construction.

Bid documents are developed and contract delivery method is defined in the design stage. It is assumed all projects shall be delivered using a traditional design-bid-build method. The bid documents are advertised and contract is awarded. Typically advertising and awarding contract may range anywhere from 2 to 6 months depending on contract and purchasing requirements. Once the contract is awarded and notice to proceed is given to the contractor, the construction phase begins. The construction phase involves project construction and reporting, quality inspections and testing, submittal reviews, payment processing and as-built and O&M developments. The construction phase is completed after successfully demonstration of plant start-up and commissioning. Typical construction projects may require two to three years for completion, depending upon the size and complexity of the project. Project equipment and performance normally carry a one year warranty after the project is completed. Figure 15-3 provides a more detailed schedule for the first decade of the program.

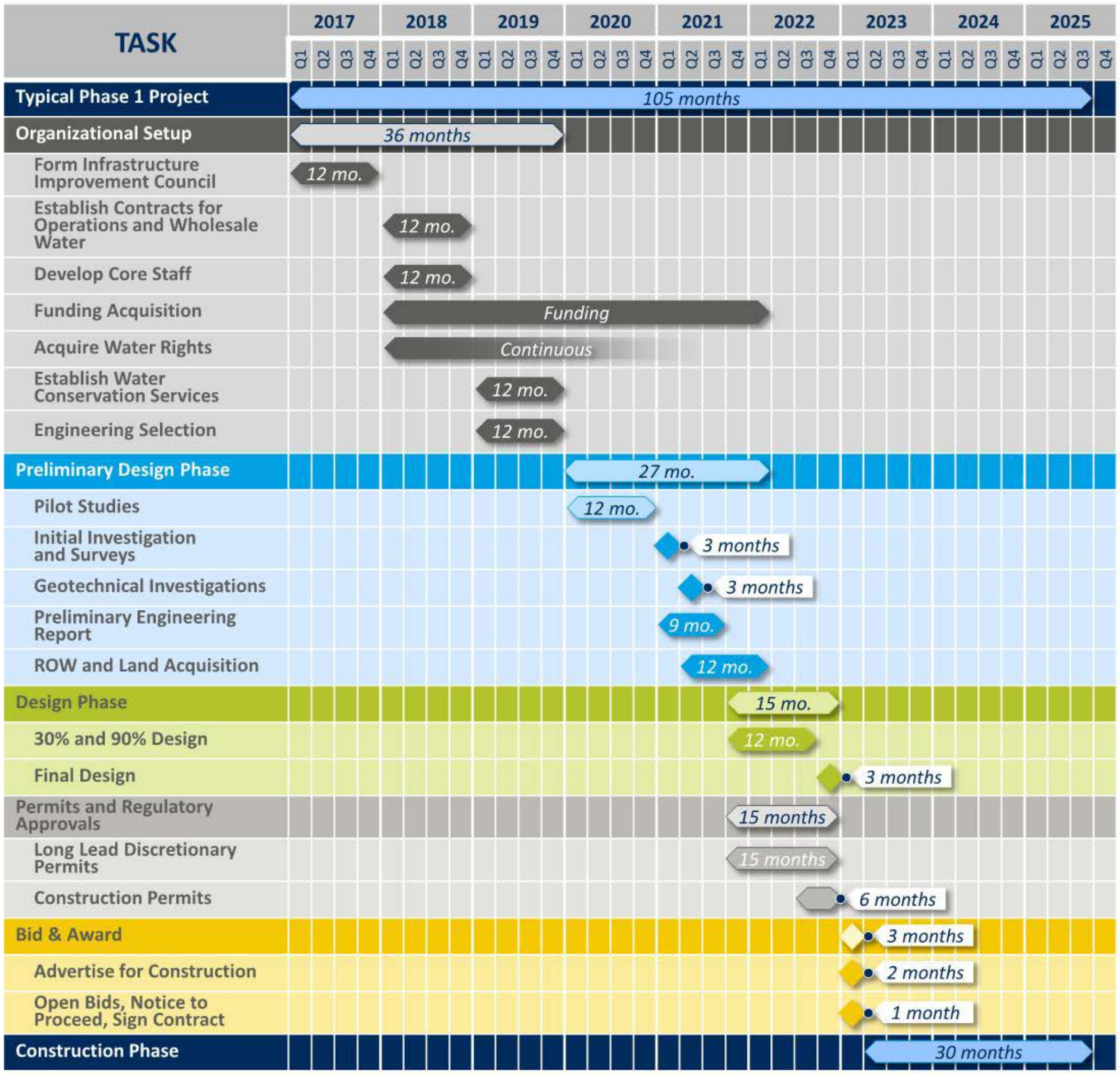

Figure 15-3 Typical Project Schedule