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APPENDIX A-1 ENGINEERING

A1 – SECTION 1 INTRODUCTION

1.1 GENERAL HISTORY

This appendix primarily documents the methodology used to prepare the hydrologic and hydraulic analysis for the Raymondville Drain (RD) project, and calibration and validation processes. It also documents the Geotechnical investigations accomplished for the study, analysis of changing conditions, and civil design and construction considerations. The technical work for this study was done by RRP Consulting Engineers, Ltd. (RRP), previously known and operating as S&B Infrastructure (S&B), on behalf of the sponsor, Hidalgo County Drainage District #1 (HCDD1).

The study area analyzed in this appendix includes the Raymondville Drain watershed and the North Main Drain system watershed. For the purposes of this report, the "North Main Drain system" (NMD) is the drainage network south of the RD, consisting of the West Main Drain, the North Main Drain, and the South Main Drain, all of which flow into the Main Floodwater Channel, which discharges into the back bays of the Laguna Madre. The recommended RD improvement limits are from the area east of Edinburg Lake to two miles upstream of where the drain crosses state highway 186 near Port Mansfield, feeding into the back bays of the Laguna Madre. The North Main Drain system limits are from the West Main Drain southwest of Edinburg to the outfall into the back bays of the Laguna Madre through the Main Floodwater Channel. This report includes descriptions of the hydrologic and hydraulic methodologies, computer models utilized, data used in the preparation of these models, and outputs for both the hydrologic and hydraulic analysis. There are a total of three (3) scenarios presented in this appendix: a without-project model and two with-project models. The without-project (no project improvements) model represents the existing conditions without any modifications. For the with-project condition, separate models were prepared to determine the effects of the final project alternatives (Alternative 1 & Alternative 2).

The results from this analysis were utilized in the Flood Damage Assessment (FDA) and Economics portions of this study found in Appendix A-5. The two (2) refined alternative plans in the final array were evaluated on the effectiveness on flood damage reduction and overall cost effectiveness in accordance with USACE guidelines.

A1 – SECTION 2 GENERAL WATERSHED DESCRIPTIONS

2.1 WATERSHED MODEL SEGMENTATION AND SUB-BASIN DELINEATION

The project study area analyzed in this Appendix encompasses the areas within Hidalgo County and Willacy County ranging from areas west of Edinburg Lake and spanning east to the Laguna Madre. There are two main watersheds: The RD system, and the NMD system (Table A1). For modeling purposes in this study, we have identified four watersheds: one for the NMD system; and three sub-watersheds within the greater RD watershed: Raymondville (Hidalgo and Willacy), Delta Lake, and Willacy (Table A2). The NMD watershed area is south of the Raymondville Drain and encompasses approximately 592 square miles. The RD watershed encompasses approximately 625 square miles.

Table A1 Watersheds

Watersheds	Acres	SqMi
North Main Drain (NMD)	378,672	592
Raymondville Drain (RD)	400,116	625
Total	**778,788**	**1,217**

Table A2 RD Subwatersheds

Raymondville Drain Sub Watersheds	Acres	SqMi
RVD Hidalgo & Willacy	222,594	348
RVD Willacy	86,624	135
Delta Lake	90,898	142
Total	**400,116**	**625**

The sub-watershed and sub-watersheds are depicted in Figure A1, and in more detail in Attachment A to this Appendix, (Project Drainage Area Map). For the purposes of the report, the Willacy sub-watershed refers to the portion of the RD that is downstream of the La Sal Vieja connection and lies entirely in Willacy

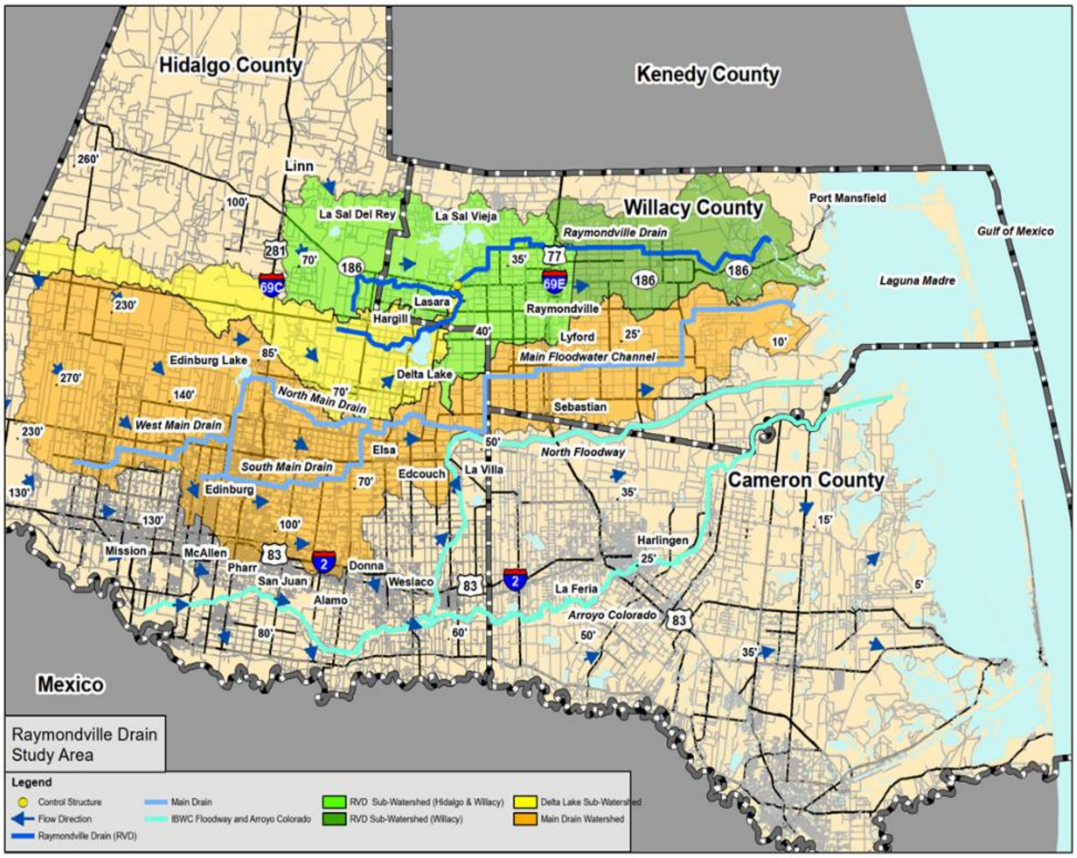


Figure A1 Project Drainage Area Map

County. For this study, the portion of the RD watershed that drains toward the existing drains around Delta Lake (an off-channel storage reservoir to the Rio Grande, owned and operated by the Delta Lake Irrigation District for water supply, irrigation, and recreation purposes) is referred to as the Delta Lake sub-watershed. The hydrologic modeling for the Delta Lake sub-watershed was done in conjunction with the RD hydrologic model; however, the hydraulic model for this watershed is separate from the RD model. Generally, the stormwater runoff within all the watersheds flows from west to east to the back bays of the Laguna Madre.

2.2 WATERSHED CHARACTERISTICS

The developed portion of the study area primarily lies within Hidalgo County. This developed area consists of residential and commercial uses. The remainder of the total area consists of agricultural and native rangeland. The agricultural land consists of irrigated cropland, dry cropland, and improved pasture. The topography of the project study area is generally flat, as described below. The primary drainage system consists of man-made drains constructed to convey storm runoff from the developed areas and agricultural lands eastward toward the Laguna Madre.

The horizontal datum used for this study was North American Datum (NAD) 83, South Zone. The vertical datum used was North American Vertical Datum (NAVD) 88. All additional survey data used to supplement the existing hydrologic and hydraulic models was based on the above horizontal and vertical data.

2.2.1 North Main Drain (Hidalgo and Willacy Counties)

Of the four primary watersheds, the NMD watershed contains most of the existing urban development. This watershed contains the communities of Edinburg, Pharr, San Juan, Edcouch, La Villa, Lyford, and the northern portions of Alamo, Donna, and McAllen. These communities contain both residential and commercial development. The undeveloped portion of the watershed is a combination of farmland and vacant rangeland. Additionally, there are numerous elevated canals and depressions within the watershed that needed to be accounted for in the hydrologic model. The existing slope in this watershed varies from approximately .003 ft/ft within the western portion of the watershed to 0.0006 ft/ft in the eastern portion of the watershed in Willacy County. This watershed also contains Edinburg Lake which is located on the northwest corner of Edinburg. Edinburg Lake is not a typical lake; it is an elevated storage basin surrounded by earthen berms. Stormwater must be pumped into the storage basin instead of draining to it naturally via overland flow. There is also an outlet structure that drains to the NMD west of US 281. Edinburg Lake is not part of the flood damage reduction plan.

2.2.2 Raymondville Drain Watershed

2.2.2.1 Raymondville Drain Sub-watershed (Hidalgo and Willacy Counties)

The RD watershed is located north of the NMD system watershed and only encompasses two (2) urbanized areas, La Sara and Raymondville. This watershed extends from US 281 on the west to US 77 on the east. The remainder of the watershed consists of farmland and vacant rangeland. Within this watershed are two (2) large waterbodies - Sal del Rey and La Sal Vieja located north of the RD. These waterbodies do not receive flows from the existing RD unless water is purposely directed through the existing drain from RD to La Sal Vieja. The existing slope in this watershed varies from 0.007 ft/ft at the northwest portion of the watershed to 0.001 ft/ft near the community of Raymondville.

A2.2.2.2 Delta Lake Sub-watershed (Hidalgo and Willacy Counties)

The Delta Lake portion of the RD watershed is located between the NMD system watershed and the upstream portion of the Raymondville Drain. This watershed encompasses two (2) urbanized areas; Hargill and Monte Alto. It also includes the Edinburg Airport within its limits. The remainder of the watershed consists of farmland and vacant rangeland. Within this watershed are the two (2) reservoirs associated with Delta Lake. These reservoirs are elevated storage facilities and thus do not receive overland flow. Runoff is instead pumped to the reservoirs. The existing slope in this watershed varies from 0.006 ft/ft at the upstream portion of the watershed to 0.001 ft/ft near the Delta Lake confluence with the RD near the community of La Sara.

2.2.3 Raymondville Willacy Sub- watershed (Willacy County)

The Willacy watershed constitutes the portion of the RD in Willacy County that is downstream of US 77. This area is primarily vacant ranchland and some farmland (with 171 wind turbines and supporting distribution equipment). There is little development within this watershed. The only urbanized community within this watershed is the town of San Perlita. The existing slope in this watershed is very flat, with an average approximate slope of 0.001 ft/ft.

A1 - SECTION 3 ANALYSIS METHODOLOGY & PROCEDURE

3.1 GENERAL BACKGROUND

The methodology and computer modeling techniques for this study were based on an initial coordinated effort between USACE and HCDD1. RRP worked closely with USACE staff over a span of several years to jointly prepare the methodology that was utilized in this analysis, then validated by RRP. For this analysis, eight flood frequency events and the Beulah storm event were modeled to determine the peak flow rates and magnitude of flooding for the various storm events. The analysis methodology is provided in further detail below in Section 3.2.

There have been several hydrologic and hydraulic studies completed for this region in previous years. This study considered the Hurricane Beulah storm event to be equivalent to the 100-year storm event. Additionally, the USACE, Galveston District, conducted several studies of this region that were documented in the technical document entitled, "Lower Rio Grande Basin, Texas, Flood Control and Major Drainage Project General Design Memorandum." This technical document was originally published by the USACE in January 1982. A limited re-evaluation report (LRR) was completed in September 1997 by the USACE to update the cost estimate and verify the feasibility of the recommended alternative plan from the previous document. These studies document the history and the need to address flooding in the region.

There are also supporting studies for this region in the aftermath of Hurricane Harvey (a devastating Category 4 hurricane that made landfall on Texas and Louisiana in August 2017, causing catastrophic flooding and more than 100 deaths). Principally, these reports are the 2021 Coastal Texas Protection and Restoration Feasibility Study (Texas Coastal) and the 2023 Region 15 Lower Rio Grande Regional Flood Plan.

The USACE and the State of Texas General Land Office (GLO) partnered to identify and recommend solutions to threats to vital resources critical to the social, economic, and environmental welfare of the nation. In August 2021, the Texas Coastal Study was published. This Study presents the findings and recommendations of the multi-year study effort by the USACE and GLO and it supersedes the 2018 Draft Integrated Feasibility Report and Environmental Impact Statement (DIFR-EIS) and the October 2020 Draft Feasibility Report and Draft EIS and represents the most current and complete findings. The study provides context for large-scale coastal storm risk management (CSRM) and ecosystem restoration actions that provide coastal communities of Texas with multiple lines of defense to reduce impacts from a wide array of coastal hazards. Specifically, this study emphasizes the need to provide storm protection for the communities of south Texas.

Additionally, in 2023, the Texas Water Development Board (TWDB) published the draft Region 15 Lower Rio Grande Regional Flood Plan (RFP). The Lower Rio Grande River Basin, also known as Flood Planning Region 15, covers the southern half of the Rio Grande River Basin within Texas. This region begins at International Amistad Reservoir in Val Verde County. It extends along the Rio Grande River to the Gulf of Mexico, encompassing all or parts of 14 counties. The report provides a floodplain "quilt" that offers additional data for consideration in Hidalgo, Willacy, and Cameron Counties - including existing conditions flood risk analysis, flood areas, structures, storms, damages, and social and economic characteristics of the counties.

The hydrologic and hydraulic models in previous studies for this region were developed using the best available data at the time. The region remains in need of comprehensive modeling. Development continues along the border with Mexico and changes how the watersheds and existing drainage systems respond to precipitation changes. The hydrologic and hydraulic computer models used in the previous reports (the 2020 Draft Integrated Feasibility Report and Environmental Impact Statement (DIFR-EIS) and 2017 General

Reevaluation Report (GRR)) provide the accuracy necessary for feasibility-level analysis. In this report, existing bridge, culvert and inline weir structures along the NMD system and RD were added to the hydraulic model to accurately model the existing conditions. These revised models are used to create the base condition models. The condition models served as the base information for proposed measures, alternatives analysis, and selection of the recommended plan.

3.2 ANALYSIS PROCEDURE

RRP utilized the USACE program HEC-HMS for all hydrologic analysis to determine the peak flow rates for the individual frequency events detailed in this study. The USACE program HEC-RAS was used for hydraulic modeling to determine water surface elevations for the NMD system and RD watersheds for each flood frequency event. The hydrologic and hydraulic models were used to determine the base conditions and with-project conditions for the two (2) alternative plans to determine the effectiveness of each alternative plan.

A detailed report entitled "*Final Technical Memorandum, Summary of Quality Assurance Review, Hydrology and Hydraulics Base Models, Regarding Raymondville Drain Project, Project for Flood Control*," is included as Attachment C to this Appendix. Attachment C documents the Quality Assurance process for the H&H base models developed for this Feasibility study. Attachment D contains additional model calibration and validation completed in response to Independent Technical Review (ITR) and Independent External Peer Review (IEPR) comments.

3.2.1 Hydrology Design Task Protocol

RRP prepared a design task protocol which detailed the procedure used for the preparation of the hydrologic models in this study. The hydrologic design task protocol is found in Attachment A to this Appendix. The original hydrologic models were prepared using HEC-HMS.

The hydrologic models were prepared using the methodology that was agreed upon between RRP and the USACE during the initial phases of the project in the 2006-2007 timeframe, and validated by RRP. The design task protocol dictated the means of calculating the rainfall depths, loss methods, times of concentration, and preparing the Natural Resources Conservation Service (NRCS) unit hydrographs. The design task protocol presented the methodology for converted NRCS curve numbers from Antecedent Moisture Condition (AMC) II to AMC I conditions, detailed the number flood frequency events to be analyzed, and provided methodology and standardization for the preparation of all the hydrologic models.

3.2.2 Watershed Boundaries

The watersheds and sub-basins were previously delineated based on surface topography using HEC-GeoHMS Version 1.1. HEC-GeoHMS enabled the PDT to delineate each watershed and sub-basin using digital terrain data. LIDAR survey data was used in combination with Digital Elevation Models (DEMs) representing USGS quad maps to develop a 60-ft X 60-ft grid. The relatively flat topography required the watershed to be reconditioned to ensure that HEC-GeoHMS identifies the main drain within each watershed. A line file was created to delineate and identify the main drainage channel. This line file was subsequently used to modify the existing grid in order to more easily identify the main drainage channel within the flat topography.

3.2.3 Hydraulic Design Task Protocol

RRP also prepared a design task protocol that details the procedure used for the preparation of the hydraulic models used in this study. This document is found in Attachment B to this Appendix. The original hydraulic model for Raymondville Drain was created using HEC-RAS.

The hydraulic design task protocol detailed the methodology and selection of variables for preparation of the geometry data and calculation methods. A bridge modeling approach for both low-flow and high-flow scenarios at each bridge crossing was also established, as well as geometric variables such Manning's "n" values and expansion/contraction coefficients.

A1 - SECTION 4 FORMULATION OF ALTERNATIVES

4.1 GENERAL

As documented in the main report, the project delivery team (PDT) evaluated possible approaches to accomplish the goal of alleviating flooding along the developed and urbanized portions of the NMD. Early iterations eliminated northern and southern routes, buyouts, and improvements only to the NMD system or only the RD. Subsequent iterations focused on improvements to the RD and the NMD system. As many as 17 alternatives were initially generated from combinations of structural measures including 11 potential detention basins, 9 alternate conveyance drains, 10 drain improvement options, and multiple weir control structures. Figure A2 depicts the array of measures initially considered, and detailed documentation of these 17 alternatives can be found in RRP technical files.

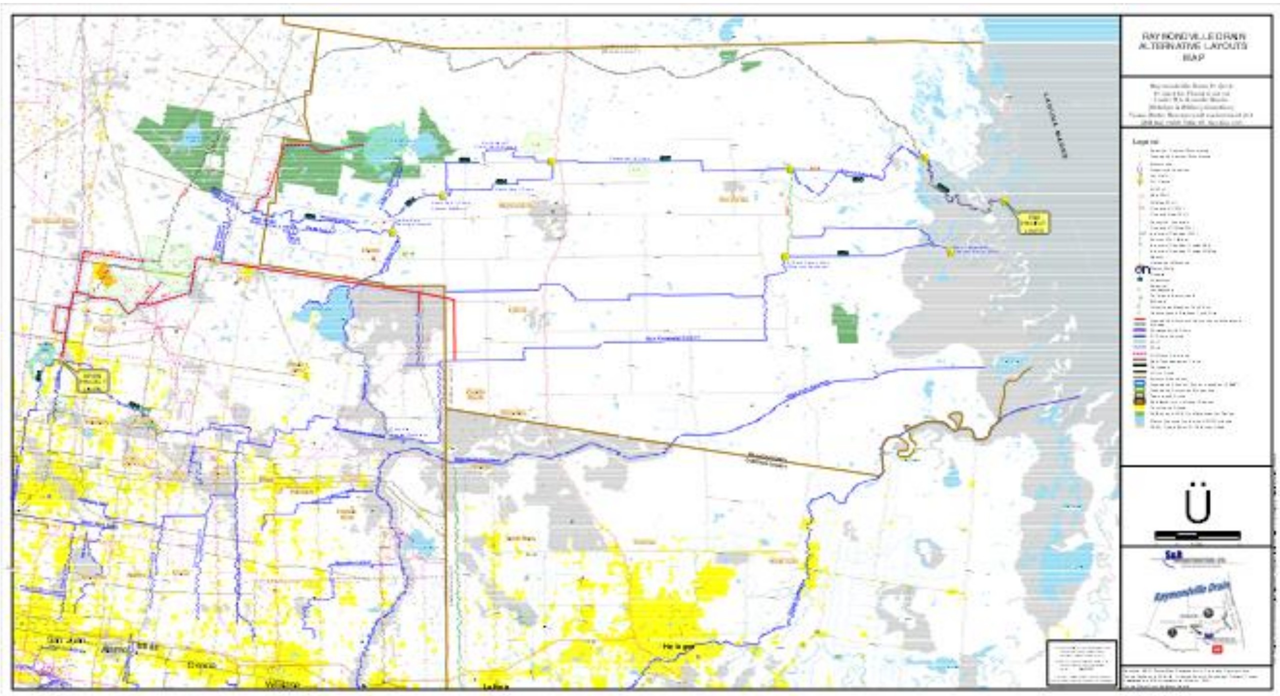


Figure A2 Measures Initially Considered

After several iterations which ruled out infeasible or impractical solutions or combinations, and ruled out alternatives that did not accomplish the objective of improving the NMD and RD systems, the PDT identified four hydraulic alternatives associated with the existing RD that could potentially accomplish this flood reduction goal. Hydraulic models were run for the four alternatives. However, consistent with SMART Planning principles, two alternatives were eliminated on a technical basis because they provided limited conveyance improvement while causing additional new land disturbance and additional construction and real estate costs. A brief description of the two eliminated alternatives is found in Section 4.4. The proposed alternatives forwarded for consideration have been designated as “Alternative 1” and “Alternative 2.” Sections 4.2 and 4.3 provide a general description of these proposed alternatives.

Multiple drain configurations were considered by the PDT during the development of possible Alternative Conveyance Channels (AC - new drains), and Channel Improvement (CH – existing drain modifications) measures. One of the major considerations for the cross-section configuration was to develop a single profile for consistency throughout the system, which met operational, hydraulic design, and environmental considerations. The typical drain cross-section used by the Local Sponsor consists of a pilot channel, maintenance benches, raised access roads, and field drains. The purpose of the pilot channel is to keep the drains wet, even during low flow conditions; the maintenance bench is for the characteristically large drains present in the study area to be maintained properly; the access roads enable the drain to be inspected and operated during flood events; and the field drains exist so the raised access roads do not impact overland sheet flow into the channel, and so flows entering the drain do not erode the slide slopes.

During this process, the preliminary H&H analysis determined that a channel width of 350-foot wide was needed to contain the 1% annual chance exceedance flood. For evaluation purposes, all CH measures proposed as a part of this analysis were designed to this channel width. Furthermore, AC measures which are typically downstream would inherit this channel dimension as well to prevent flooding downstream of any new channels. Ultimately, the 350-foot wide channel was incorporated into a 450-foot wide Right of Way (ROW), which allowed addition of a 100-foot wide spoil berm / bank to reduce hauling of excavated material. The change to a 450-foot wide ROW did not impact the hydraulic calculations.

The proposed drain configuration (Figure A3) consists of a 20-foot flat bottom width and 4H:1V side slopes. There are benches, which vary in lengths, up to 110-feet wide at approximately mid-depth within the drain section to provide flood condition volume and facilitate maintenance of the drain. Additionally, 20-foot access roads at the top of the drain allow for maintenance access. Drain improvements will be required through Willacy County, to a point upstream of its eventual discharge into the Laguna Madre. The drain improvement geometry is the same as the proposed diversion drain.

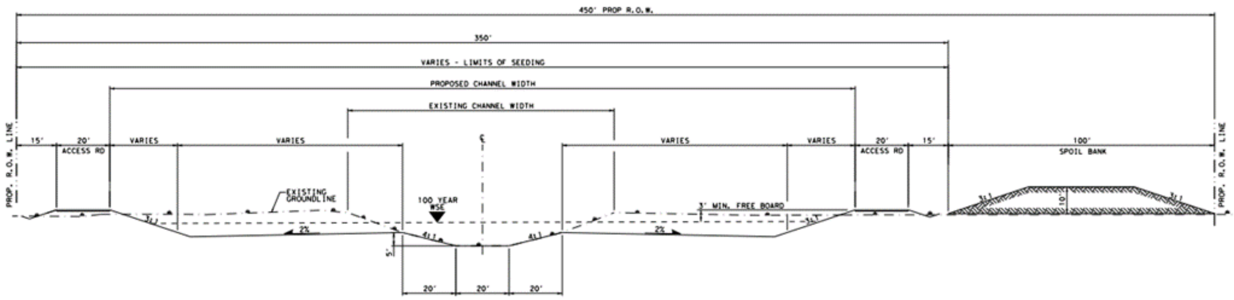


Figure A3 Proposed Drain Configuration

4.2 DESCRIPTION OF ALTERNATIVE 1

Alternative 1 consists of a new diversion drain that connects the existing NMD system and the RD, improvements along the RD, a detention basin located in Hidalgo County, and five control structures to regulate the flow of water. The proposed typical section of the new drain and existing drain improvements generally includes a 450’ ROW consisting of 15’ of vegetative buffers and 20’ of access roads on both sides, in-channel maintenance benches, and a 60’ wide 5’ deep pilot channel, and 100’ spoil berms. In environmentally sensitive downstream areas, the spoil berms would be omitted, and the ROW would be limited to 350’.

The diversion channel would start approximately 0.4 miles east of Edinburg Lake with a control structure along the North Main Drain and proceed north before crossing I-69C/US 281 approximately 0.7 miles north of El Cibolo Road. From there, the drain proceeds eastward toward Brushline Road, with the detention pond and control structure located in line with the pilot channel; from Brushline Road the drain proceeds north until it is 0.4 miles north of FM 490 where it turns east and connects to the existing Raymondville Drain (locally known as West Hargill Drain Lateral 5); a second control structure is designed south of 12th Street before FM 490 to divert water toward Delta Lake for the irrigation purposes (Delta Lake Irrigation District). The proposed new drain from the start to this point is approximately 13.8 miles long, with an approximately 270-acre detention basin.

The proposed project continues with drain widening to match the proposed diversion drain's cross section and continues from the start of the RD (West Hargill Lateral 5) approximately 2.7 miles north; and then proceeds west along the RD (West Hargill Drain) approximately 7.8 miles, and then flows into the RD (North Hargill Drain), with a control structure located at the county line between Hidalgo and Willacy Counties. The drain improvements continue along the existing drain in a northeasterly, then east, direction past I-69E/US 77 approximately 30.2 miles to an unnamed private bridge, approximately 2 miles north of the drain's intersection with SH 186. An existing control structure between the drain and its connection to La Sal Vieja, north of SH 186, would be replaced due to the channel widening. The length of the channel improvements is approximately 43 miles, and the overall length of the proposed project is approximately 56.8 miles. This Alternative would replace an estimated 56 bridge structures or culvert crossings, and construct 13 new bridges so that existing roadways can cross the proposed diversion drain. The Alternative 1 alignment is shown in Figure A4, and the typical channel profile is shown in Figure A3. A more detailed map of the alignment of the proposed diversion drain is in Attachment A to this appendix, (Project Drainage Area Map Alternative 1).

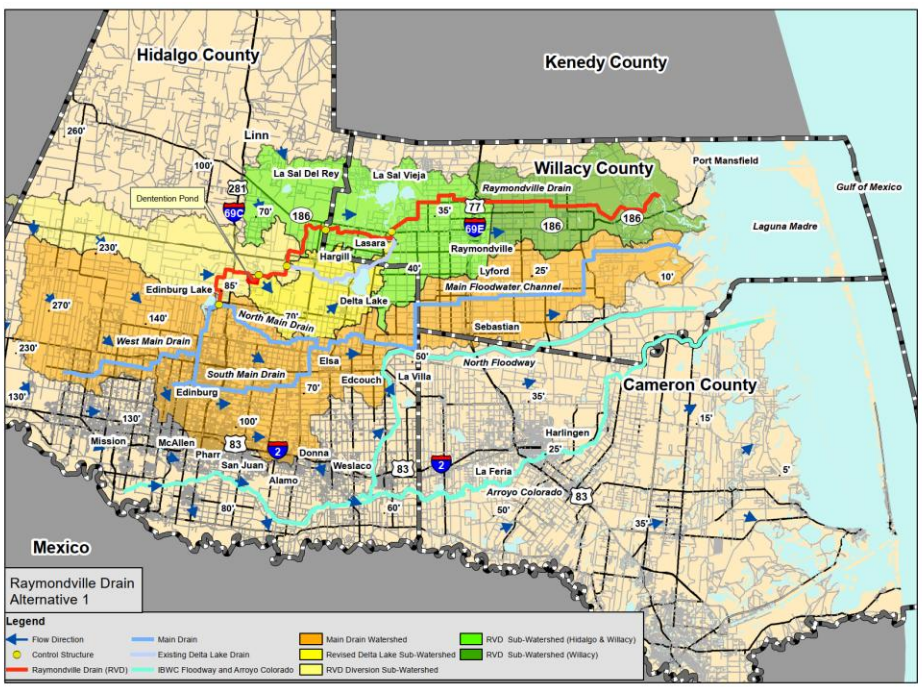


Figure A4 Alternative 1

4.3 DESCRIPTION OF ALTERNATIVE 2

Alternative 2 consists of a new diversion channel that connects the existing North Main Drain and the Raymondville Drain, improvements along the Raymondville Drain, a detention basin located in Hidalgo County, and four control structures to regulate the flow of water. The proposed typical section of the new channel and channel improvements would generally include a 450’ ROW consisting of 15’ of vegetative buffers and 20’ of access roads on both sides, in channel maintenance benches, and a 60’ wide 5’ deep pilot channel, and 100’ spoil berms. A 350’ ROW without spoil berms would be used in sections of the drain along the Delta Lake reach due to space constraints, and in environmentally sensitive downstream areas.

The diversion channel would start approximately 0.4 miles east of Edinburg Lake with a control structure along the North Main Drain and proceed north before crossing I-69C/US 281 approximately 0.7 miles north of El Cibolo Road. From there, the project would proceed eastward toward Brushline Road, with the detention pond and control structure located in line with the pilot channel, from Brushline Road the channel would proceed north and then turn towards the existing Delta Lake South Main Drain south of 12th Street. The proposed new diversion channel from the start to this point is approximately 11.5 miles long, with an approximately 270-acre detention basin.

The proposed project would continue with drain widening to match the proposed diversion channel’s cross section, and would continue from the start of the Delta Lake South Main Drain east around the existing Delta Lake, widening to the north to accommodate existing irrigation channels, to its junction with the North Hargill Drain northeast of Lasara, approximately 13.3 miles, with a control structure located at county line between Hidalgo and Willacy Counties. This Delta Lake reach would include some areas excluding the 100’ spoil berm. The channel improvements would continue along the existing channel in a north easterly then east direction past I-69E/US 77 approximately 30.2 miles to an unnamed private bridge, approximately 2.0 miles north of the channel’s intersection with SH 186. An existing control structure between the drain and its connection to La Sal Vieja, north of SH 186, would be replaced due to the channel widening. The length of the channel improvements is approximately 43.9 miles, and the overall length of the proposed project is approximately 55.4 miles. This Alternative would replace an estimated 51 bridge structures or culvert crossings, and construct 13 new bridges so that existing roadways can cross the proposed diversion drain.

The difference between the two alternatives is that starting downstream of the proposed detention basin and east of the airport, this alternative conveys flow into the Delta Lake Drain, a more southerly route passing along the north side of Delta Lake, while alternative 1 continues northward to connect to the West Hargill Drain, with both alternatives connecting to Raymondville Drain northeast of Lasara. Alternative 2 is depicted in Figure A5. A more detailed map of the alignment of the proposed diversion drain is in Attachment A to this Appendix, (Project Drainage Area Map Alternative 2).

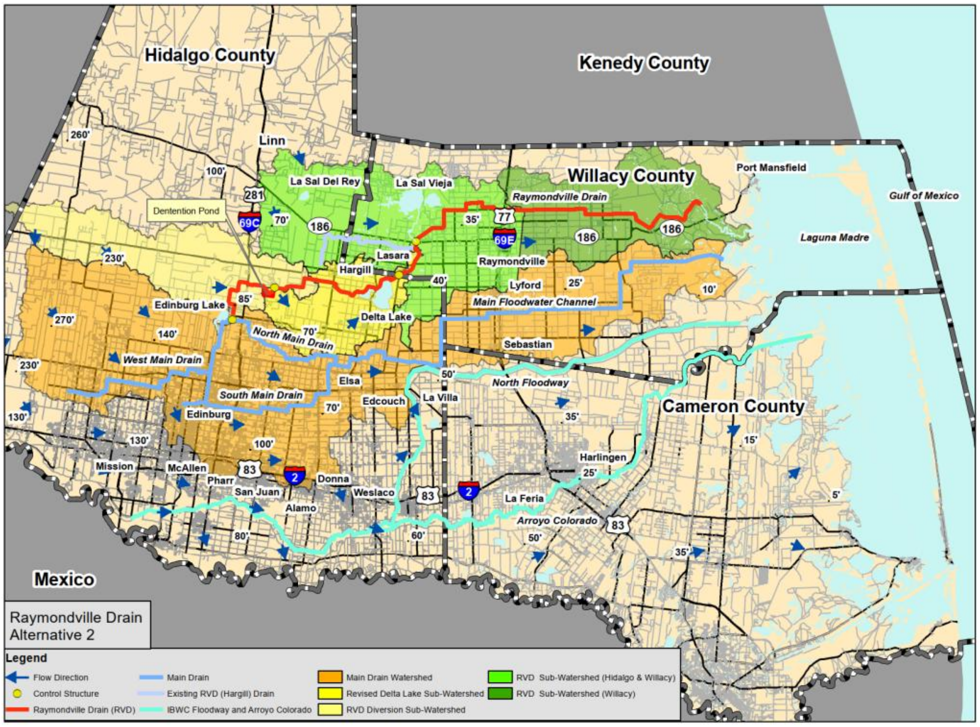


Figure A5 Alternative 2

4.4 ELIMINATED ALTERNATIVES

The eliminated alternatives are modifications to Alternative 1 described above, which were modeled, but did not provide significant hydraulic benefits, while requiring additional construction and land acquisition. Consistent with SMART Planning, additional details on these eliminated alternatives are not included in this Appendix nor the main report, however they are briefly described below and shown in Figure A6. Each of the following alternatives also use the proposed diversion drain (new drain and widened existing drains North of Hargill).

The first eliminated alternative proposed a second diversion drain (AC-7) in Willacy County to divert storm water runoff back to the North Main Drain. This proposed second diversion drain is downstream of the Raymondville Drain crossing at San Andres County Road, east of the City of San Perlita. The diversion required the use of side weir with a variable control structure on the south bank of the Raymondville Drain, approximately 9,500 linear feet downstream of the San Andres County Road crossing in Willacy County. From this point, the proposed second diversion drain proceeds south, crossing SH 186, and connecting to the North Main Drain (CH-9) just downstream of Weir #2 in Willacy County. The diversion drain would be approximately 22,000 linear feet with the same cross section characteristics as the proposed diversion drain utilized in Alternative 1. The hydraulics ultimately required pumping to the North Main Drain, which made this option technically infeasible.

The second eliminated alternative included the realignment of a portion of drain along the Raymondville Drain within Willacy County. The purpose of this is to straighten an existing portion of the Raymondville Drain within the vicinity of the El Sauz Ranch (AC-8). This alternative includes approximately 11,300 linear feet of new drain alignment. The typical cross section of the proposed drain will be identical to the cross-section geometry of the drain found in Alternative 1. This alternative also includes the abandoning of approximately 17,000 linear feet of existing drain. The "straightening" of the RD alignment did not provide substantial benefits, required new ROW, and cost more to excavate than widen the existing drain.

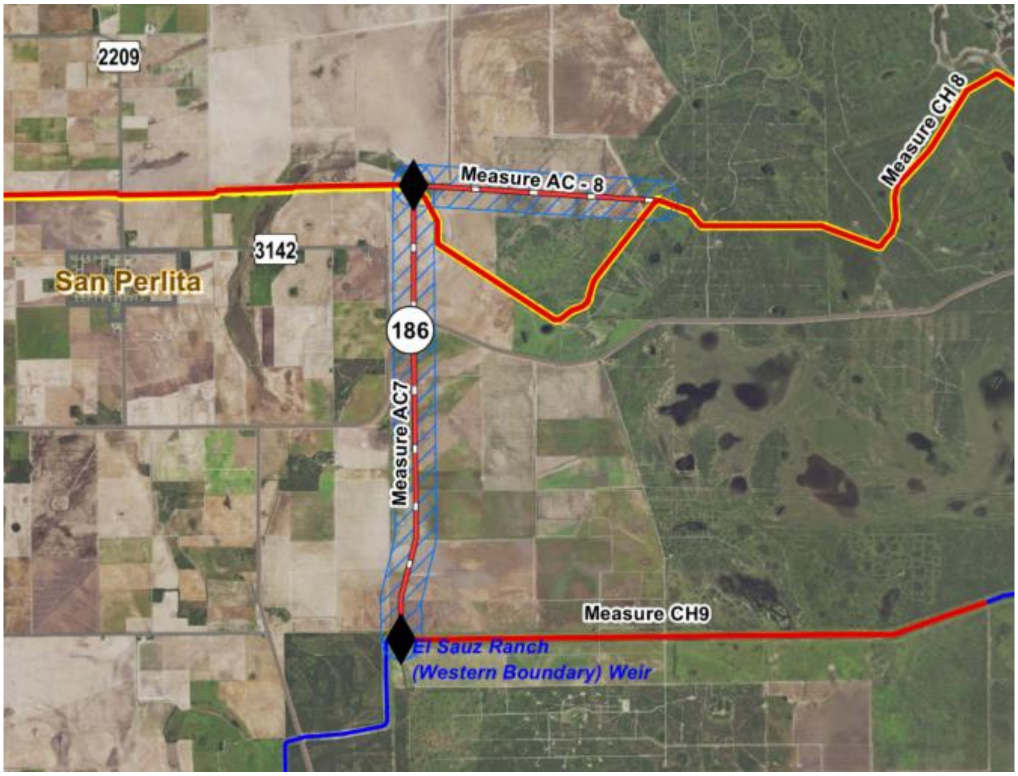


Figure A6 Eliminated Alternatives

A1 – SECTION 5 HYDROLOGY

5.1 GENERAL

For the hydrologic modeling used in this study, the first step was to prepare models that would adequately model the watershed characteristics for each of the four (4) watersheds. This was accomplished by preparing three (3) separate HEC-HMS models. These three (3) HEC-HMS models separated the NMD system, RD (Hidalgo County) and the RD (Willacy County) watersheds. The connection between the Delta Lake and Raymondville models is represented by a time-series dataset that was input as a source at the upstream junction of the RD (Willacy County) model. This time-series dataset was extracted from the RD (Hidalgo County) model at the outlet of this model. This connection is labeled on the exhibit in Attachment A to this Appendix, (ALT 1 Inflow Locations). (The time-series dataset can be provided upon request). These three (3) HEC-HMS models represented the current watershed conditions, prior to any watershed improvements associated with the proposed alternatives. For each alternative, a separate HEC-HMS model was prepared with the appropriate modifications. A matrix of the flow changes in the final hydrologic models and a hydrologic model summary is available upon request.

The hydrology modeling for this feasibility study was developed using TP40/TP49 rainfall data. New rainfall data is now available through NOAA Atlas 14. An analysis was performed to determine the impacts of the updated Atlas 14 data on the project. In the study area, the rainfall estimates in Atlas 14 have only increased by a minimal amount from TP40/TP49. The analysis indicated that the new NOAA Atlas 14 rainfall has minimal impact on resulting runoff, so RRP will continue the use of TP40/TP49 rainfall data for the RD feasibility study. The difference is within the margin of error for this Feasibility level analysis and would not impact the plan selection, analysis, or project recommendations. Additionally, the hydraulics and hydrology will be updated in the future utilizing Atlas 14 for the plans and specifications effort, as appropriate. This approach is consistent with SMART Planning. This analysis is included in Section 9.1 of this Appendix and is documented in the project's Risk Register.

5.2 WATERSHED BOUNDARIES (HIDALGO AND WILLACY COUNTIES)

The watersheds and sub-basins were previously delineated based on surface topography using HEC-GeoHMS. HEC-GeoHMS enabled RRP to delineate each watershed and sub-basin using digital terrain data. Light Detection and Ranging (LIDAR) survey data was used in combination with Digital Elevation Models (DEMs) representing United States Geological Service (USGS) quad maps to develop a 60-ft X 60-ft grid. The relatively flat topography required the watershed to be reconditioned to ensure that HEC-GeoHMS identifies the main drain within each watershed. A line file was created to delineate and identify the main drainage drain. This line file was subsequently used to modify the existing grid to identify the main drainage drain more easily, within the flat topography.

5.2.1 NORTH MAIN DRAIN (HIDALGO AND WILLACY COUNTIES)

The NMD system watershed includes four (4) existing system drains: West Main, South Main, North Main, and Main Floodwater drains. These four drains convey the runoff through populated areas in Hidalgo County and rural areas in Willacy County.

5.2.1.1 Hydrologic Soil Groups (HSG)

The NMD watershed consists primarily of Type B soils. Type B soils typically consist of shallow loess and sandy loam and have a moderate infiltration rate when wet with a moderate rate of water flow. These types of soils have a typical loss rate of 0.15-0.30 inches/hour. The Type B soils found in the NMD watershed primarily consist of Brennan fine sandy loam and Hidalgo sandy clay loam. Detailed HSG maps for the NMD watershed can be provided upon request.

5.2.1.2 Vegetative Cover

The characteristics of the vegetative cover for the NMD watershed were determined by available aerial photography. Undeveloped portions of the NMD watershed are primarily brush, pasture, grassland and/or range. The majority of the watershed vegetative cover was classified as "good" per guidelines found in the United States Department of Agriculture (USDA) Technical Release 55 (TR-55) "Urban Hydrology for Small Watersheds". This classification is for areas with greater than 75% ground cover.

5.2.1.3 Land Use

The characteristics of the existing land uses for the NMD watershed were determined by available aerial photography. The developed portions of the NMD watershed consisted primarily of residential and commercial development. Additionally, there are large areas of farmland outside of the incorporated areas. Residential areas were classified based on average lot size (i.e., ¼ acre lot, ½ acre lot, etc.). Farmlands were primarily designated as straight row, small grain crops. Details of the individual land uses per sub-basin can be provided upon request.

5.2.1.4 Terrain Slope and Depression Storage

The NMD watershed is characterized by very flat terrain with numerous depressions, elevated storage basins, and elevated canals. The general terrain slope for the NMD watershed varies from 0.003 ft./ft. to 0.00017 ft./ft. The natural depressions, elevated storage basins (Edinburg Lake) and elevated canals inhibit the flow of runoff due to impoundment and/or redirection prior to entering the main drain. This depression storage was accounted for in the precipitation losses for each sub-basin, which reduced to overall peak flow rate for the watershed. The methodology for modeling the effects of the depression storage is further detailed in Section A5.5.

5.2.2 RAYMONDVILLE DRAIN SUB-WATERSHED (HIDALGO AND WILLACY COUNTIES)

The RD consists of a primary drainage drain, with several smaller lateral drains, draining eastward from Hidalgo County to Willacy County. Below is a general description of the watershed with characteristics that were used to develop the hydrologic model for the without-project condition.

5.2.2.1 Hydrologic Soil Groups (HSG)

The RD watershed consists primarily of Type B soils. Type B soils typically consist of shallow loess and sandy loam and have a moderate infiltration rate when wet with a moderate rate of water flow. These types of soils have a typical loss rate of 0.15-0.30 inches/hour. The Type B soils found in the RD watershed primarily consist of Brennan fine sandy loam and Delfina loamy fine sand sandy clay loam. Detailed HSG maps for the RD watershed can be provided upon request.

5.2.2.2 Vegetative Cover

The characteristics of the vegetative cover for the RD watershed were determined by available aerial photography. Undeveloped portions of the RD watershed are primarily brush, pasture, grassland and/or range. There were also areas within this watershed that were classified as woods-grass combination due to the large presence of trees in some of the undeveloped areas. Most of the watershed vegetative cover was classified as "good" per guidelines found in the USDA TR-55 "Urban Hydrology for Small Watersheds". This classification is for areas with greater than 75% ground cover.

5.2.2.3 Land Use

The characteristic of the existing land uses for the RD watershed were determined by available aerial photography. The developed portions of the RD watershed consisted primarily of residential and commercial development with a small amount of industrial development. Additionally, there are large areas of farmland outside of the incorporated areas. Residential areas were classified based on average lot size (i.e., ¼ acre lot, ½ acre lot, etc.). Farmlands were primarily designated as straight row, small grain crops and/or legumes or rotation meadow. Details of the individual land uses per sub-basin can be provided upon request.

5.2.2.4 Terrain Slope and Depression Storage

The RD watershed is characterized by very flat terrain with numerous depressions, elevated storage basins, and elevated canals. The general terrain slope for the RD watershed varies from 0.004 ft./ft. to 0.001 ft./ft. Like the NMD watershed, the natural depressions, elevated storage basins and elevated canals inhibit the flow of runoff due to impoundment and/or redirection prior to entering the main drain. This depression storage was accounted for in the precipitation losses for each sub-basin, which reduced to overall peak flow rate for the watershed. The methodology for modeling the effects of the depression storage is further detailed in Section 5.5.

5.2.3 DELTA LAKE SUB-WATERSHED (HIDALGO AND WILLACY COUNTIES)

The Delta Lake watershed consists of a primary drain that conveys runoff along the north side of Delta Lake, eventually connecting the main drain of the RD near the town of La Sara. This watershed is included in the HEC-HMS model for the RD watershed. Below is a general description of the watershed with characteristics that were used to develop the hydrologic model for the without-project condition.

5.2.3.1 Hydrologic Soil Groups (HSG)

The Delta Lake watershed consists primarily of Type B soils. Type B soils typically consist of shallow loess and sandy loam and have a moderate infiltration rate when wet with a moderate rate of water flow. These types of soils have a typical loss rate of 0.15-0.30 inches/hour. The Type B soils found in the Delta Lake watershed primarily consist of Brennan fine sandy loam and Delfina loamy fine sand sandy clay loam. Detailed HSG maps for the Delta Lake watershed are included in the Raymondville Drain watershed maps and can be provided upon request.

5.2.3.2 Vegetative Cover

The characteristics of the vegetative cover for the Delta Lake watershed are similar to the RD watershed and were determined by available aerial photography. Undeveloped portions of the Delta Lake watershed are primarily brush, pasture, grassland and/or range. There were also areas within this watershed that were classified as woods-grass combination due to the large presence of trees in some of the undeveloped areas. The majority of the watershed vegetative cover was classified as "good" per guidelines found in the USDA TR-55 "Urban Hydrology for Small Watersheds." This classification is for areas with greater than 75% ground cover.

5.2.3.3 Land Use

The characteristic of the existing land uses for the Delta Lake watershed were determined by available aerial photography. The developed portions of the Delta Lake watershed consisted primarily of residential and commercial development with a small amount of industrial development. Additionally, there are large areas of farmland outside of the incorporated areas. Residential areas were classified based on average lot size (i.e., ¼ acre lot, ½ acre lot, etc.). Farmlands were primarily designated as straight row, small grain crops and/or legumes or rotation meadow. Details of the individual land uses per sub-basin can be provided upon request.

5.2.3.4 Terrain Slope and Depression Storage

The Delta Lake watershed is characterized by very flat terrain with numerous depressions, elevated storage basins, and elevated canals. The general terrain slope for the Delta Lake watershed varies from 0.004 ft./ft. to 0.001 ft./ft. Like the RD watershed, the natural depressions, elevated storage basins and elevated canals inhibit the flow of runoff due to impoundment and/or redirection prior to entering the main drain. This depression storage was accounted for in the precipitation losses for each sub-basin, which reduced to overall peak flow rate for the watershed. The methodology for modeling the effects of the depression storage is further detailed in Section 5.5.

5.2.4 WILLACY SUB-WATERSHED (WILLACY COUNTY)

The Willacy watershed is the downstream portion of the RD as it progresses eastward through Willacy County to the Laguna Madre. Below is a general description of the watershed with characteristics that were used to develop the hydrologic model for the without-project condition. Note that the HEC-HMS model for the Willacy watershed was initially provided by the USACE in 2006 (*Draft Raymondville Drain Without-project Conditions Report, April 2006, U.S. Army Corps of Engineers Galveston District – "USACE, 2006"*) and updated by RRP. RRP incorporated the results of the USACE-provided HEC-HMS model into the overall study results.

5.2.4.1 Hydrologic Soil Groups (HSG)

The Willacy watershed consists primarily of Type B soils. Type B soils typically consist of shallow loess and sandy loam and have a moderate infiltration rate when wet with a moderate rate of water flow. These types of soils have a typical loss rate of 0.15-0.30 inches/hour.

5.2.4.2 Vegetative Cover

The characteristics of the vegetative cover for the Willacy watershed are like the RD watershed and consist primarily of brush, pasture, grassland and/or range. Curve numbers found in the USACE-provided HEC-HMS model were based on guidelines found in the USDA TR-55 "Urban Hydrology for Small Watersheds".

5.2.4.3 Land Use

The developed portions of the Willacy watershed consist primarily of residential and commercial development. Additionally, there are large areas of farmland outside of the incorporated areas. The type of land use dictated the curve number utilized by the USACE in the preparation of their HEC-HMS model.

5.2.4.4 Terrain Slope and Depression Storage

The Willacy watershed is characterized by very flat terrain with numerous depressions, elevated storage basins, and elevated canals. The general terrain slope for the Delta Lake watershed varies from 0.003 ft./ft. to 0.0005 ft./ft. Like the RD watershed, the natural depressions, elevated storage basins and elevated canals inhibit the flow of runoff due to impoundment and/or redirection prior to entering the main drain. This depression storage was accounted for in the precipitation losses for each sub-basin, which reduced to overall peak flow rate for the watershed. The methodology for modeling the effects of the depression storage is further detailed in Section 5.5.

5.3 TIME OF CONCENTRATION AND LAG TIME

The time of concentration is defined as the travel time along the longest flow path within any given watershed. For time of concentration calculations of the individual drainage areas, USACE provided the following recommendation in *USACE, 2006.*

> *New … [subarea time of concentrations] were computed using a method for estimating time of concentration. Travel time along the longest flow path in each subbasin is computed based on flow velocity. The method assumes the first 500-ft length is sheet flow, the next length is represented as shallow concentrated flow and is equal to 15% of the total length, and the remaining length is assumed to be channel flow (USACE, 2006, p. 10).*

In previous models, the USACE assumed velocities in their downstream models as 0.05 fps for sheet flow, 0.1 fps for shallow concentrated flow, and 0.6 fps for open channel flow. However, the velocity assumptions were determined by RRP to be too broad based to be used for all sub-basins. In lieu of the assumed velocities, RRP utilized the velocity method, as required by the USACE, and stated by the *"National Engineering Handbook (NEH), Part 630 – Hydrology"* as "the best method for calculating time of concentration for an urbanizing watershed." Additionally, RRP utilized the best available LIDAR data to extract the slope and topographic data used to prepare these calculations for each sub-basin within the project area.

The time of concentration (T_c) was calculated by adding the sheet flow, shallow concentrated flow, and channel flow travel times. The travel path through each subarea was created in HEC-GeoHMS and exported as a line file to a MicroStation file. This was utilized to determine the total travel distance through each subarea of the watershed.

i. For Sheet Flow, the following equation was used:

$$T_{t1} = \frac{0.007(nL_1)^{0.8}}{(P_2)^{0.5}s^{0.4}}$$

where:

T_{t1} = travel time (hr),
n = Manning's roughness coefficient (n = 0.10 as per USACE, p. 13)
L_1 = flow length (500 ft)
P_2 = 2-year, 24-hour rainfall (4.375 in)
s = slope of hydraulic grade line (land slope, ft/ft)

ii. Shallow Concentrated Flow – Fifteen percent of the longest flow path was used to determine shallow concentrated flow. The travel time was calculated as follows:

$$T_{t2} = \frac{0.15L_T}{3600V_1}$$

where:

T_{t2} = travel time (hr)
L_T = longest flow path (ft)
V_1 = velocity (fps)

1. For slopes greater than 0.005 ft/ft, the flow path was designated as either paved or unpaved and use Figure 3-1, "Average Velocities for Estimating Travel Time for Shallow

Concentrated Flow", from Urban Hydrology for Small Watersheds, TR-55, June 1986 to obtain the velocity for use in the above equation.

2. The existing slopes along the RD and NMD are predominantly less than 0.005 ft/ft. For this condition, the velocity was calculated from the following equations:

For Unpaved $V_1 = 16.1345\,(s)^{0.5}$, or
For Paved $V_1 = 20.3282\,(s)^{0.5}$
where "s" = slope (ft/ft)

iii. Channel Flow – The channel flow travel time was determined using the remaining length of the longest flow path, and a cross section within each sub-basin area. This cross section was obtained from the digital terrain model along the channel flow path within each subbasin. The channel slope of the main drainage channel was also obtained from the DTM of the area. The equation to determine the travel time is shown below:

$$T_{t3} = \frac{L_2}{3600V_2}$$

T_{t3} = travel time (hr)
L_2 = longest flow path (ft)
V_2 = velocity (fps)

The average flow velocity for bank full elevation was determined using the following equation (Manning's):

$$V_2 = \frac{1.49r^{2/3}s^{1/2}}{n}$$

where:
V_2 = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow (n = 0.10)

The times of concentration calculations and summary tables for the RD and NMD watersheds are available upon request. For the Willacy watershed, these values were provided by the USACE in their HEC-HMS model. The sub-basins within the Delta Lake watershed are included in the calculations for the RD watershed since both watersheds are combined into one HEC-HMS model.

5.4 USER SPECIFIED UNIT HYDROGRAPH

The hydrologic models found in this report utilized the NRCS Unit Hydrograph (UHG) method. The UHG is a discharge hydrograph generated from one (1) inch of rainfall over a watershed for a specific duration. RRP utilized a design spreadsheet to calculate the ordinates of the UHG for each sub-basin. The individual NRCS Unit Hydrograph spreadsheets for the without-project conditions can be provided upon request. For the proposed alternative plans, individual NRCS Unit Hydrograph spreadsheets can be provided upon request. The input variable for this spreadsheet is the catchment area in square miles, Time of Concentration (T_c), and a Peak Rate Factor (PRF) to be entered as input values for each sub-basin. The standard NRCS PRF is 484. This is a national average of watersheds studied across the nation and was dictated by the NRCS. However, this value does not properly reflect the flat terrain within the project area and would result in higher peak flow rates than would be expected due to the flat terrain. To model the flat terrain more accurately within the project area, all unit hydrographs were modified to utilize a PRF of 150.

5.5 NRCS CURVE NUMBER (AMC I & II) AND INITIAL/CONSTANT LOSS METHOD

The initial loss method utilized in the hydrologic model was the NRCS Curve Number (CN) method. However, the NRCS Curve Number Method assumes that after initial infiltration, all precipitation losses approach zero. Because the hydrologic models in this study utilized a ten (10) day storm duration, the NRCS Curve Number Method was not an appropriate loss method to use. In coordination with USACE staff, the loss method for the hydrologic modeling was revised to the Initial / Constant Loss Method. Since a constant loss is applied to the 10-day duration, there will always be some precipitation losses throughout the duration of the 10-day storm.

The Weighted Curve Number was computed using TR-55. The input values for both Land Use and Hydrologic Soil Group (HSG) were obtained from the aerial photography and USDA Soil Surveys for Hidalgo and Willacy Counties. The percentage of HSG Group A, B, C, and D obtained from the web soil survey and land use were developed based on observations from the aerial photo of the subject area. Note that the maximum area per file in TR-55 is 25 square miles. Once this limit is reached, a new TR-55 file was created to document the Weighted Curve Number for each subwatershed. The soils maps and TR-55 calculations can be provided upon request.

The Weighted Curve Numbers calculated using TR-55 were based on Antecedent Moisture Condition II (AMC II). However, it was determined from past documentation that the soils in this region should be classified as Antecedent Moisture Condition I (AMC I) according to NRCS Engineering Technical Note 210-18-TX5. Using the original AMC II values, the individual weighted curve numbers were adjusted to represent AMC I soil conditions based on equations from the Texas Department of Transportation Hydraulic Design Manual. The individual calculation sheets for adjusting the AMC II curve numbers to AMC I curve numbers can be provided upon request. According to the NRCS Engineering Technical Note 210-18-TX5, this reduction should not allow for curve numbers to be reduced below a value of 60. However, since found that there are numerous minor depressions and storage areas, no minimum value was set, and the actual calculated CN was used.

In coordinated agreement with the USACE, the Initial Loss was derived using the following NRCS equation, which relates the initial loss to the soil curve number. For this study, CN value used was the AMC I adjusted value described above. The conversion calculations for each sub-basin can be provided upon request.

$$I = 0.2S$$

$$S = (1000/CN) - 10$$

where:
I = Initial loss (in)
S = Potential Maximum Retention
CN = Curve Number

The constant loss parameter was based on the NRCS recommendations for specific hydrologic soil groups. The soil loss rate values shown in Table A3 were assigned to each soil group and calculate composite

Constant Loss Rate based on proportion of Hydrologic Soil Group as determined using the Web Soil Survey, and the average of the following ranges:

Table A3 NRCS Soil Groups and Infiltration (Loss) Rates

Soil Group	Description	Range of Loss Rates (in/hr)
A	Deep sand, deep loess, aggregated silts	0.30-0.45
B	Shallow loess, sandy loam	0.15-0.30
C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05-0.15
D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0.00-0.05

The highest allowable loss rate for each soil group was utilized, based on the presence of natural depressions, storage areas, and elevated canals impeding the natural flow of the storm runoff.

5.6 METEOROLOGIC MODELS (RAINFALL AND STORM DATA)

A meteorological model was prepared to represent the rainfall and storm data to be utilized in the hydrologic analysis of the without-project condition and the proposed alternative plans. The following sections detail the methodology and procedures for preparing the meteorological models.

5.6.1 STORM EXCEEDANCE PROBABILITIES, STORM DURATION, AND POINT RAINFALL DATA

The point rainfall data used for the base conditions hydrologic modes was obtained from data found in the National Weather Service (NWS) Technical Paper 40 (TP-40) and Technical Paper 49 (TP- 49) documents for the individual storm events for storm durations ranging from one (1) hour to ten (10) days. Additional data was obtained from HYDRO-35 to determine the rainfall depths for the 5-minute and the 15-minute storm durations. An HMS Meteorological Model for each storm frequency was created based on these rainfall depths and durations. The Exceedance Probability is the chance that a storm will occur in a single year. The relationship between Storm Frequency and Exceedance Probability is described in Table A4. For each frequency listed above, the rainfall depths in Hidalgo/Willacy counties were obtained from HYDRO-35, TP-40, and TP-49.

Table A4: Storm Frequency and Probability

STORM FREQUENCY	EXCEEDANCE PROBABILITY
2-year	50%
5-year	20%
10-year	10%
25-year	4%
50-year	2%
100-year	1%
250-year	0.4%
500-year	0.2%

TP-49 states that in frequency analysis there are:

"Two types of series - Frequency analyses of precipitation data are based on one of two types of data series. The annual series consists only of the highest value for each year. The partial-duration series recognizes that the second highest of some year occasionally exceeds the highest of some other year and utilizes all items above a base value which is selected to yield n-items for

n-years. The highest value of record, of course, is the top value of either series, but the lower values in the partial-duration series tend to be higher than those of the annual series" (p. 2).

In addition to the rainfall amounts obtained from TP-40 and TP-49, rainfall data from two historical storm events was utilized to provide a means to calibrate the HEC-HMS model to an actual storm event. Rainfall data for the Hurricane Beulah event was obtained from the USACE Hurricane Beulah report prepared in September 1968. The data available entailed incremental rainfall depths in one (1) hour increments for a total of five (5) days. The Beulah Incremental Rainfall Data can be provided upon request. This recorded data was entered into the HEC-HMS meteorological model to determine the peak flow rate from this event. Two (2) separate models were prepared using the two (2) loss rates (CN I & II) in order to calibrate the HEC-HMS models with the observed water surfaces and with available as-built structure plans.

5.6.2 CONVERSION OF POINT RAINFALL DATA FROM PARTIAL TO ANNUAL DURATION

The TP-49 uses partial-duration series to determine rainfall depths. For the 2-yr, 5-yr, and 10-yr storm events, the rainfall depths were converted to annual series as described in Table A5. For storm frequencies equal to or greater than 25-year, the annual and partial duration series data converge, thus, no adjustment was performed.

Table A5: Empirical Factors for Converting Partial-Duration Series to Annual Series

RETURN PERIOD	CONVERSION FACTOR
2-yr.	0.88
5-yr.	0.96
10-yr.	0.99

For the base condition HEC-HMS models, no manual conversion was used for converting the partial-duration series information to annual series. The HEC-HMS model performs this conversion internally for the required return periods.

5.6.3 PEAK RAINFALL INTENSITY LOCATION ALONG HYETOGRAPH

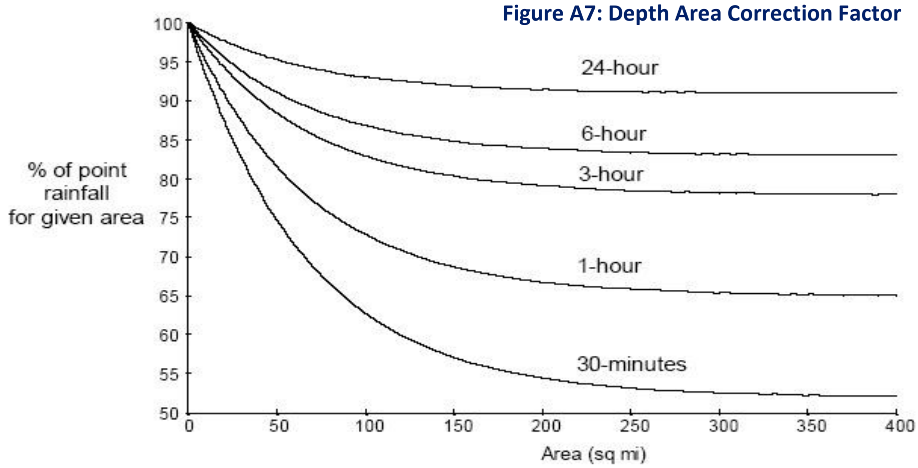
The peak center for the hyetograph was set to the default value of 50%. This will cause the location of the peak rainfall amount to occur at the center of the hyetograph. As stated in TP-49,

"Any value read from an isopluvial map for a point is an average depth for the location, for a given return period and duration. The depth-area curve attempts to relate this average point value, for a given duration and frequency and within a given area, to the average depth over that area for the same duration used frequency" (p. 4).

5.6.4 STORM AREA REDUCTIONS AT POINTS OF INTEREST (POI)

Because the watershed areas for the RD and the NMD systems encompass several hundred square miles, it is unlikely that the point rainfall values obtained from TP-40 and TP-49 will fall uniformly over the entire watershed. As a result, it is necessary to use a depth-area correction factor (also known as an Areal Reduction Factor – ARF) for each storm event. This correction factor is applied internally in the HEC-HMS program. By entering the watershed area in the meteorological model, HEC-HMS will correct the point rainfall totals based on Figure A7.

Figure A7: Depth Area Correction Factor



In order to accurately calculate the peak flow rates at the multiple flow change locations, several iterations were computed for each storm event, while varying the storm area within the meteorological model. Each HEC-HMS model was run at a fifty (50) square mile interval, up to the maximum basin area. The peak flow rate for each storm event was interpolated from these results based upon the actual storm area at each point of interest. By doing this, accurate peak flow rates at each flow change in the hydraulic model were calculated. These calculations for the without-project conditions can be provided upon request.

5.7 HYDROGRAPH ROUTING

The HEC-HMS models used in for the RD and NMD systems utilized the Muskingum Cunge 8-point routing method and the Modified Puls method. For smaller reaches, the Muskingum Cunge 8-point routing method was utilized. The geometry for the routing reaches was created by generating 8-point cross sections from LIDAR data available for the project area. The roughness coefficient for these reach lengths were adjusted to use a Manning's "n" value of 0.10, as recommended by USACE *"to account for the relatively flat terrain, as well as the numerous roads and elevated irrigation drains that crisscross the watershed" (USACE, 2006, p. 11)*. Documentation for the 8-point cross sections can be provided upon request.

Portions of the main drains for both the RD and NMD were modeled using the Modified Puls method. This method is also known as storage routing or level-pool routing. The procedure involves extracting the storage-outflow data for each reach from the HEC-RAS model and entering this information back into the HEC-HMS model. The HEC-HMS calculations are repeated, and the revised peak flow rates are entered into the HEC-RAS model again. This process is repeated until the storage-outflow data between the two models converges to a point where the difference is negligible.

5.8 BASE HYDROLOGIC MODEL RESULTS (BASE WITHOUT PROJECT)

The results from the base hydrologic models were used as the basis to analyze the effectiveness of the two (2) alternative plans, and to provide the necessary data to input into FDA model to determine the base flood damage for each stream. The base hydrologic models provided the peak flow rates for the current conditions (base) and are summarized in Section 5.8.1. The FDA model also required projected conditions to account for future growth. The methodology used to determine the projected future peak flow rates is described in further detail in Section 5.8.2, and additional details are in the Future Flows Memorandum, Section 9.2.

5.8.1 WITHOUT-BASE WITHOUT PROJECT CONDITIONS (BASE)

The without-project current year HEC-HMS models were prepared to provide peak flow rates to be used in the without-project hydraulic model (HEC-RAS). A total of three (3) HEC-HMS models were prepared for the current year without-project conditions. A total of nine (9) storm events were used in the HEC-HMS models, which yielded nine (9) peak flow rates at each point of interest. As discussed in Section 5.6.3, the HEC-HMS results were adjusted at each POI based on the total catchment area at the specific point. It is these peak flow rates that were entered into the hydraulic models. Tables A6 through A9 summarize base without project peak flow rates for the four (4) watersheds. Detailed base Flow Comparison tables for the base without Project condition is available upon request.

Table A6: North Main Drain Base Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=R2630W2610	258	397	518	669	784	914	1111	1260	956
Q=Userpoint5	626	969	1266	1639	1918	2241	2721	3084	2309
Q=JR1920	71	110	143	185	216	253	307	349	281
Q=JR1870	1010	1698	2324	3048	3644	4692	6270	7417	6505
Q=JR1670	1020	1575	2096	2856	3547	4462	5979	7155	6115
Q=JR1490	942	1674	2415	3377	4058	4925	6179	7227	7335
Q=JR1420	1341	1948	2337	2855	3230	3714	4399	5650	8680
Q=JR2690	1354	1990	2374	3533	4575	6016	8091	9806	12409
Q=UserPoint11	1232	1910	2252	3372	4294	5551	7581	9304	12307
Q=JR2700	1127	1821	2131	3252	4096	5214	7104	8859	12313
Q=UserPoint12	1109	1769	2079	3184	3977	5012	6745	8522	12283
Q=UserPoint16	1104	1687	2037	3076	3784	4694	6146	7759	12203
Q=UserPoint17	1077	1676	2030	3023	3706	4606	5821	7078	11985
Q=JR1780	1097	1487	1861	2653	3185	3855	5078	6472	11874
Q=JR2780	1107	1489	1860	2652	3184	3856	5070	6457	11947
Q=UserPoint18	1096	1487	1859	2651	3182	3854	5048	6403	11890
Q=Userpoint5	626	969	1266	1639	1918	2241	2721	3084	2309
Q=JR2210	1485	2205	2823	3597	4183	4955	5951	6765	5483
Q=JR2190	1420	2220	2891	3732	4377	5246	6341	7230	6021
Q=JR2180	1461	2139	2752	3541	4142	4949	6085	7010	5748
Q=JR2150	1617	2373	2980	3783	4396	5215	6314	7113	6465
Q=JR2090	1684	2289	2715	3298	3748	4368	5173	5786	5965

Q=JR2060	1741	2531	3030	3687	4188	4871	5769	6451	6589
Q=JR1840	1994	3185	3902	4820	5512	6490	7734	8741	8929
Q=JR1630	1815	3104	4134	5337	6218	7421	8806	9584	16045
Q=JR1800	1575	2682	3428	4176	4717	5444	6459	7287	13961
Q=JR1790	1785	3005	3796	4596	5217	6064	7252	8160	14463
Q=JR1450	1745	2907	3482	4170	4773	5626	6705	7573	13961
Q=JR1280	1722	2938	3376	3973	4478	5305	6312	7134	13454
Q=JR1290	1628	2706	3131	3659	4080	4721	5615	6571	12176
Q=JR1190	1655	2767	3183	3711	4136	4788	5694	6638	12266
Q=OutletNMD	1629	2602	3030	3554	3982	4581	5399	6387	11586
Q=JR3	1654	2767	3180	3686	4108	4762	5599	6561	11877
Q=JR4	1674	2860	3269	3771	4193	4870	5725	6675	12056
Q=JR5	1680	2856	3268	3771	4195	4873	5722	6671	12043
Q=JR7	1680	2855	3269	3770	4189	4863	5713	6663	12016
Q=JR8	1701	2907	3308	3798	4213	4892	5744	6685	11774
Q=JR10	1685	2845	3271	3771	4173	4790	5633	6583	11315
Q=JR12 OUTLET	1629	2761	3214	3726	4131	4741	5582	6522	11135

Table A7: Raymondville Drain Base Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
R660W660	344	546	713	920	1093	1273	1573	1797	1462
R660W660	344	546	713	920	1093	1273	1573	1797	1462
R660W660	344	546	713	920	1093	1273	1573	1797	1462
R750W750	181	284	369	474	561	651	799	908	641
R750W750	181	284	369	474	561	651	799	908	641
R750W750	181	284	369	474	561	651	799	908	641
JR770	492	784	1025	1324	1574	1834	2267	2588	1967
JR780	646	976	1207	1468	1684	1945	2355	2686	2368
JR720	1330	2435	3288	4335	5187	6081	7622	8720	10991
JR630	0	0	0	0	2	134	566	960	1811
JR1570	1278	2372	3214	4254	5099	5990	7525	8679	11264

Table A8: Delta Lake Base Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1060	283	447	581	747	885	1027	1265	1444	1038
JR 1120	452	738	968	1267	1509	1757	2171	2479	1763
JR 1080	634	1248	1926	2735	3437	4238	5512	6468	7340
JR 1620	819	1439	1944	2718	3424	4227	5522	6502	7771
USERPOINT 8	817	1435	1940	2715	3421	4222	5518	6496	7770
JR 790	1039	1871	2551	3431	4192	4996	6324	7335	8925
JR 720	1330	2435	3288	4335	5187	6081	7622	8720	10991
JR 1570	1278	2372	3214	4254	5099	5990	7525	8679	11264

Table A9: Willacy Base Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1570	1278	2372	3214	4254	5099	5990	7525	8679	11264
USERPOINT 3	1246	2245	3012	3975	4811	5671	6960	8377	10777
JR 1560	1540	2790	3750	4933	6008	7105	8675	9808	12485
USERPOINT 2	1452	2574	3483	4605	5638	6695	8448	9645	12448
JR 390	1495	2511	3369	4360	5207	6255	8099	9506	12769
JR 400	1362	2180	2887	3871	4668	5569	7743	9387	12903
JR 420	1529	2400	3144	4150	4930	5847	8122	9859	13900
OUTLET 35/ SOURCE1	1594	2502	3275	4304	5103	6021	8345	10133	14393
JR 490	1354	2238	3040	4156	5078	6013	8065	9795	14353
JR 500	1370	2289	3122	4293	5246	6190	8319	10110	15111
JR 530	1183	1962	2755	3870	4929	5893	7768	9529	14468
JR 540	1196	1970	2771	3882	4960	5936	7832	9602	14657
JR 460	1148	1906	2703	3791	4865	5880	7749	9494	14574
JR 370	1136	1889	2686	3770	4853	5884	7757	9509	14701
JR 590	1210	1982	2829	3991	5155	6259	8277	10161	16128
OUTLET	1197	1935	2701	3758	4859	5957	7802	9498	15411

5.8.2 FUTURE YEAR WITHOUT-PROJECT CONDITIONS

In addition to preparing hydrologic calculations to determine peak flows for current year development conditions, projected future year peak flows must also be calculated to determine the peak flow rate at the end of the analysis period for each storm event. The specific future year is a projected value used as the end of the analysis period. For this study, an analysis period of fifty (50) years was utilized to determine the flood damage reductions over the service life of the improvements. The methodology for determining the future year without project hydrologic conditions will be based on the population projections available from the U.S. Census Bureau.

An Analysis of Future Flows was conducted by RRP which confirmed and justified the population growth and changes in land use assumptions in this report. The methodology and analysis are described in the sections below. This analysis confirms that the assumptions and conclusions utilized are valid, are appropriate for this feasibility study, and do not adversely impact the study results or conclusions. These assumptions ensure proper project formulation and analysis of benefits. The RRP Future Flows memorandum is included in section 9.2, and its impacts have also been considered in the project's Risk Register.

The scope of this study encompasses areas in Hidalgo County and Willacy County. According to the 2021 U.S. Census, the approximate population of Hidalgo County was 981,890. For the same year, Willacy County had an approximate population of 25,264. As these numbers illustrate, most of the population within the study limits reside in Hidalgo County. As a result, most of the urbanized and developed acreage is within Hidalgo County. The four largest urbanized areas within the project limits are the cities of McAllen, Edinburg, Pharr, and Mission. To determine the future peak flows, it is necessary to approximate the impact the projected population will have on the amount of development. Due to the presence of existing development, the focus will be on the drainage basins that include the cities of McAllen, Edinburg, Pharr, and Mission. These areas drain to both the South Main Drain and North Main Drain. For these future conditions analysis, all sub-basins up to the point where the South Main Drain and North Main Drain combine to the Main Floodwater channel east of the city of Edinburg will be included (63 sub-basins).

The first step is to determine the level of population growth over the project study life. The population projections for the four municipalities are detailed in Table A10.

Table A10: South Texas Population Projections

City Name	P2000 Census	P2010 Census	P2020 Census	P2030	P2040	P2050	**P2060**
McAllen	106,414	129,877	142,210	179,586	209,386	241,933	**275,322**
Edinburg	48,465	77,100	100,243	105,237	128,358	153,611	**179,517**
Pharr	46,660	70,400	79,715	91,553	109,836	129,805	**150,291**
Mission	**45,408**	**77,058**	**86,635**	**100,157**	**122,454**	**146,807**	**171,790**

Source: U.S. Census Bureau

The projections shown above illustrate a substantial amount of population growth over the 60-year period from the Year 2000 to the Year 2060. Population census values for Year 2000 and Year 2010 were obtained from publicly available U.S. Census Bureau data. Population estimate for subsequent years were also obtained from the U.S. Census Bureau. For the 50-year project life associated with this study, the percentage of population growth from the year 2010 to the year 2060 was calculated. Table A11 illustrates the population growth factors for the cities of McAllen, Edinburg, Pharr, and Mission.

Table A11: South Texas Population Growth Factors

CITY NAME	P2010 CENSUS	P2060	% GROWTH	GROWTH FACTOR
McAllen	129,877	275,322	112%	2.12
Edinburg	77,100	179,517	133%	2.33
Pharr	70,400	150,291	113%	2.13
Mission	77,059	171,790	123%	2.23

Source: U.S. Census Bureau

With this population growth, additional development will occur in the form of additional residential subdivisions, commercial developments, and infrastructure improvements. All of these items will increase the overall impervious cover within any given watershed. For this analysis, areas closer to the existing municipalities would increase to 70% impervious cover and outlying areas would increase to 60% impervious in the future year. Note that there are two sub-basins with an existing impervious cover percentage of 85%. These were raised to 95% to account for minimal future development. Attachment A to this Appendix, (Projected Future Area of Development Map) illustrates the revised sub-basins used in this analysis. For this analysis, the existing land use pattern remained the same proportion throughout the watershed. This was documented with the USACE concerning the methodology to be used for future without project conditions. A summary table illustrating the change in curve numbers for the sub-basins is available upon request.

As stated previously, the focus of this analysis will be the drainage basins within the NMD system basin that drains to the North and South Main Drain. During the preparation of the without project hydrologic models, land use maps and aerial photographs were utilized to determine the land use characteristics of the individual sub-basins. Additionally, zoning maps were obtained from the City of McAllen Planning Department and the Edinburg Planning and Zoning Department to confirm the extent of the existing development. The developed land uses corresponded to an impervious cover percentage based on the density of residential structures and/or commercial classification. These impervious cover percentages for the residential and commercial land uses were obtained from TR-55, Table 2-2a (Runoff Curve Numbers for Urban Areas). According to the tables above, the population of McAllen and Edinburg are expected to more than double over the 50-year analysis period. As the existing amount of impervious cover accommodates the existing population, it is reasonable to estimate that a similar, although smaller amount of additional impervious cover would be needed to accommodate this increase in population. To determine the effect that this additional development would have on the hydrologic conditions of this area, a separate base conditions hydrologic model was prepared which included additional impervious cover for the future conditions.

To accommodate the previously referenced population increases, additional impervious cover amounts were estimated and added to the hydrologic model for this portion of the study. This will estimate the future without-project hydrologic conditions for the watersheds. In order to account for routing effects, present within the watershed, several inflow points were selected along the NMD to compare the present and future conditions without-project peak flow rates. Note that for this comparison, the storm area reduction calculations were not performed, since only the relative difference in peak flow rates is needed. The relative increases in the peak flow rates for the 100-year storm event for the selected project inflow locations are tabulated in Table A12.

Table A12: Selected Project Inflow Locations

Project Inflow Location	Base Q_{100} (cfs)	Future Q_{100} (cfs)	Increase Factor
JR1630	8857.3	12203.7	1.38
User Point 18	5129.9	9026.2	1.76
JR2780	5151.8	9090.2	1.76
JR1780	5155.9	8998.0	1.75
User Point 17	5874.8	9113.3	1.55
User Point 16	6223.9	9327.2	1.50
User Point 12	6845.2	9772.1	1.43
JR2700	7204.2	9929.4	1.38
User Point 11	7668.6	10137.7	1.32
JR 2690	8179.5	10527.9	1.29
JR1420	4377.4	7283.0	1.66
JR1490	6034	7615.1	1.26
JR 1670	5591.8	7146.9	1.28
JR 1870	5830.2	7165.7	1.23
JR 1920	255.1	420.7	1.65
		Weighted Average	1.37

Table A12 illustrates the weighted average based on individual sub-basin areas to determine the difference in peak flow rates. The future peak flow rates for the NMD Reach 2 are on average approximately 1.37 times (37% higher) than the peak flow rates for the current year. This analysis was also completed for the South Main Drain Reach 3. It was found that the weighted average of both the North Main Drain Reach 2 and the South Main Drain Reach 3 increase factor was 1.35 (35% higher). Since it has been previously documented that the existing land use pattern is estimated to continue in the same proportion throughout the watershed, it is appropriate that this factor be applied uniformly to the current year peak flow rates throughout the entire watershed. The population growth obtained from Census Bureau projections is likely low when consideration is given that US 281 and US 77 are being readied to convert to interstate roadway facilities and new ports of entries/additional bridge construction is slated to occur between Mexico and the United States throughout the Rio Grande Valley area. The future peak flow rates (1.35 x Year base peak flow rates) were utilized in the hydraulic models to calculate future water surface profiles for each damage reach. These future conditions water surface profiles were subsequently entered into HEC-FDA for use in determining the expected annual damages for the without project conditions and for each proposed alternative. Tables A13 through A16 summarize the without-project future peak flow rates for the four watersheds.

Table A13: North Main Drain Base Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=R2630W2610	348	536	699	903	1058	1234	1499	1701	956
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR1920	96	148	193	250	292	342	415	471	281
Q=JR1870	1364	2292	3138	4115	4920	6334	8465	10013	6505
Q=JR1670	1377	2126	2829	3856	4789	6024	8072	9660	6115
Q=JR1490	1271	2260	3260	4559	5478	6648	8342	9756	7335
Q=JR1420	1811	2630	3155	3854	4360	5014	5938	7627	8680
Q=JR2690	1828	2687	3206	4770	6176	8122	10923	13239	12409
Q=UserPoint11	1663	2578	3040	4552	5796	7494	10235	12561	12307
Q=JR2700	1521	2459	2877	4391	5529	7038	9590	11960	12313
Q=UserPoint12	1497	2389	2807	4298	5369	6767	9105	11504	12283
Q=UserPoint16	1490	2277	2750	4152	5108	6336	8298	10475	12203
Q=UserPoint17	1454	2262	2741	4081	5003	6219	7859	9555	11985
Q=JR1780	1481	2007	2513	3582	4299	5204	6856	8737	11874
Q=JR2780	1494	2009	2512	3581	4298	5205	6845	8716	11947
Q=UserPoint18	1480	2008	2509	3578	4296	5202	6815	8643	11890
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR2210	2005	2977	3811	4856	5647	6689	8034	9133	5483
Q=JR2190	1918	2997	3903	5039	5909	7082	8561	9761	6021
Q=JR2180	1973	2888	3716	4781	5592	6681	8215	9464	5748
Q=JR2150	2183	3203	4022	5107	5934	7040	8523	9603	6465
Q=JR2090	2274	3090	3666	4452	5060	5897	6984	7811	5965
Q=JR2060	2351	3416	4091	4978	5654	6576	7788	8709	6589
Q=JR1840	2692	4300	5267	6507	7441	8761	10441	11801	8929
Q=JR1630	2451	4190	5581	7205	8395	10018	11889	12938	16045
Q=JR1800	2127	3620	4628	5638	6368	7350	8720	9838	13961
Q=JR1790	2409	4057	5124	6205	7043	8187	9790	11016	14463
Q=JR1450	2356	3924	4701	5630	6444	7595	9051	10223	13961
Q=JR1280	2325	3966	4558	5363	6045	7161	8522	9630	13454
Q=JR1290	2198	3652	4227	4939	5508	6373	7581	8871	12176
Q=JR1190	2234	3735	4298	5010	5584	6463	7687	8961	12266
Q=OutletNMD	2200	3513	4090	4797	5376	6185	7289	8623	11586

Q=JR3	2233	3735	4293	4976	5546	6429	7558	8857	11877
Q=JR4	2260	3861	4413	5091	5661	6575	7729	9011	12056
Q=JR5	2268	3855	4412	5091	5663	6578	7725	9006	12043
Q=JR7	2268	3855	4413	5090	5655	6565	7712	8995	12016
Q=JR8	2297	3925	4466	5128	5688	6604	7755	9024	11774
Q=JR10	2274	3841	4416	5091	5634	6467	7604	8886	11315
Q=JR12 OUTLET	2198	3727	4338	5030	5577	6400	7536	8805	11135

Table A14: Raymondville Drain Base Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
JR770	664	1058	1383	1788	2126	2476	3060	3494	1967
JR780	873	1318	1630	1981	2273	2626	3179	3627	2368
JR720	1795	3287	4439	5852	7002	8210	10289	11772	10991
JR630	0	0	0	0	3	181	765	1296	1811
JR1570	1726	3202	4339	5743	6883	8086	10158	11717	11264

Table A15: Delta Lake Base Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1060	382	604	784	1008	1195	1386	1708	1949	1038
JR 1120	610	996	1307	1710	2038	2372	2931	3347	1763
JR 1080	856	1685	2600	3692	4640	5722	7441	8731	7340
JR 1620	1105	1943	2625	3669	4622	5706	7455	8778	7771
USERPOINT 8	1103	1937	2620	3665	4619	5700	7449	8770	7770
JR 790	1403	2526	3443	4632	5660	6745	8538	9903	8925
JR 720	1795	3287	4439	5852	7002	8210	10289	11772	10991
JR 1570	1726	3202	4339	5743	6883	8086	10158	11717	11264

Table A16: Willacy Base Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1570	1726	3202	4339	5743	6883	8086	10158	11717	11264
USERPOINT 3	1682	3030	4066	5367	6495	7655	9395	11309	10777
JR 1560	2079	3766	5062	6659	8110	9592	11712	13241	12485
USERPOINT 2	1960	3475	4702	6216	7611	9038	11405	13021	12448
JR 390	2018	3390	4548	5886	7029	8445	10934	12834	12769
JR 400	1839	2943	3897	5226	6301	7518	10453	12672	12903
JR 420	2064	3240	4245	5603	6656	7894	10965	13309	13900
OUTLET 35/ SOURCE1	2152	3378	4421	5811	6889	8129	11266	13680	14393
JR 490	1757.4	3051.2	4173.9	5590	6780.6	8092.3	10916.5	13198.6	14353
JR 500	1762.6	3098.3	4279.6	5719.8	6947.4	8291.3	11175.3	13515.1	15111
JR 530	1450.3	2640.6	3777.4	5310.7	6542.7	7915	10641	12790.7	14468
JR 540	1458.7	2651.2	3784.5	5335.3	6577.3	7955.3	10699.6	12859.2	14657
JR 460	1400.5	2567.8	3693.6	5217.1	6518.5	7900.4	10591.5	12753.4	14574
JR 370	1384.7	2542.3	3668.9	5198	6515.2	7912.6	10598.9	12767.2	14701
JR 590	1446.6	2651.2	3839.5	5449.5	6835.8	8327.5	11166.4	13447.3	16128
OUTLET	1425.	2504.4	3582.3	5078	6455.9	7869.4	10445.1	12619.7	15411

Findings:

US Census data shows actual population numbers have been increasing along with RRP projected values for the last 10 years, from 2010 to 2020. Population growth in the area has shown to be most increased in areas outside of the most urbanized towns. More people have been moving into areas previously dominated by farm and rangeland. Population growth in these historically pervious areas will increase flows in the local floodways and future Raymondville drain system.

As shown in the exhibits and tables above, these numbers further establish the validity of assuming the 35% increase in impervious land area, especially in the more rural areas. The 35% increase has been validated above, as accurate for the purposes of this feasibility study, and the flow computations are not sensitive to the assumptions of growth in the rural eastern portion of Willacy County.

Conclusion:

Based on findings in the analysis performed, RRP will continue to use future flows estimated using the US Census Data and our resulting development assumptions for this feasibility study. It is not reasonable to assume that future ordinances will significantly control increases in flows, since that is beyond our control, and historically this has not been the case in this rural part of the state. RD is not considered a FEMA floodplain, so options to control future development are limited. The flow is not sensitive to assumptions of growth in less developed downstream areas of the basin, because the majority of flow originates in the more developed upstream areas. Furthermore, the projected data calculated for recent years is accurate based on actual population growth in the areas, and overall projected numbers are slightly less than actual population numbers, based on the table below, representing population growth in the major urban areas from 2010 to 2020. Models will be updated, as appropriate, for the design phase.

5.9 VALIDATION OF HYDROLOGIC MODEL

A detailed report entitled "*Final Technical Memorandum, Summary of Quality Assurance Review, Hydrology and Hydraulics Base Models, Regarding Raymondville Drain Project, Project for Flood Control*," is included as Attachment C to this Appendix. Attachment C documents the Quality Assurance process for the hydrologic models developed for this Feasibility study. Attachment D contains additional model calibration completed in response to Independent Technical Review (ITR) and Independent External Peer Review (IEPR) comments.

The Beulah storm event was utilized to validate the without-project hydrologic conditions. Based upon a comparison of the HEC-HMS results, the peak flow rates from the Beulah storm event are close to the peak flow rates for the 250-year storm event at most flow junction locations. This is in-line with previous documentation that the Beulah storm event was widely considered to be between a 100-year and 500-year event, depending on the location. Additionally, validation was performed by comparing various sources available in Hidalgo County and Willacy County. In Hidalgo County, the results were compared to available data found on Texas Department of Transportation (TxDOT) as-built plans, the USACE Hurricane Beulah report, Hidalgo County gage data, IBWC documentation and observed highwater marks that pre-date 1967. For the portion of the project in Willacy County, only TxDOT as-built documentation and the USACE Hurricane Beulah report were available for validation.

The storm used for this study was the 10-day 100-year event. In addition to validating the model to historic storm data, the models were run based on a 1-day, 4-day, and 10-day 100-year storm event to verify the runoff used for the large watershed was in fact the peak discharge for the study area. The RD watershed was analyzed by changing the storm event hydrologic data to reflect a 24-hour storm and a 4-day storm to compare with the 10-day storm event output used for this report. The 10-day storm event yielded the highest flows for each event, validating that the 10-day event rainfall is a worst-case scenario, verifying safe, reliable, and resilient channel capacity. Tables A17 through A19 summarize the comparison of different duration storms at various flow locations.

Table A17: Raymondville Drain Storm Duration Flow Comparison

HEC HMS JUNCTION	10-YR 1 DAY	10-YR 4 DAY	10-YR 10 DAY			100-YR 1 DAY	100-YR 4-DAY	100- YR 10 DAY	
R660W660	344	546	713			1262	1272	1273	
JR1570	1278	2372	3214			4041	5987	5990	

Table A18: Delta Lake Storm Duration Flow Comparison

HEC HMS JUNCTION	10-YR 1 DAY	10-YR 4 DAY	10-YR 10 DAY	100- YR 1 DAY	100- YR 4-DAY	100- YR 10 DAY
JR 1080	634	1248	1926	2781	3620	4238
JR 1570	1278	2372	3214	4041	5987	5990

Table A19: Willacy Storm Duration Flow Comparison

HEC HMS JUNCTION	10- YR 1 DAY	10- YR 4 DAY	10- YR 10 DAY	100- YR 1 DAY	100- YR 4-DAY	100- YR 10 DAY
JR 1570	1278	2372	3214	4041	5987	5990
OUTLET 35/ SOURCE1	1594	2502	3275	3848	5587	6021
OUTLET	205	976	2701	4845	5066	5957

5.10 HYDROLOGIC RESULTS (WITH PROJECT)

The alternative plans included proposed drain to divert runoff from the upstream portion of the NMD system. These diversions also intercepted overland flow and changed the base hydrology for the four watersheds. As a result, the HEC-HMS hydrologic models were modified to account for both diversion and interception of stormwater runoff. For all modifications, additional calculations for sub-basin delineations, NRCS curve numbers, times of concentration, and routing reach data were performed. Detailed calculations

for each alternative plan can be provided upon request. Further details of the required hydrologic modifications are described in Sections 5.10.1 and 5.10.2.

5.10.1 CURRENT YEAR WITH PROJECT CONDITIONS BASE (BASE)

For the current year (Base) with-project conditions hydrologic models, the without-project HEC-HMS models were used as template for the alternative plans. Each alternative plan uses a separate HEC-HMS model that is consistent with changes in the watersheds due to the proposed improvements. The following sections detail the modifications to the without-project HEC-HMS model and the corresponding results.

5.10.1.1 Alternative 1

As described in Section 4.2, Alternative 1 consists of a new diversion drain, which would divert a portion of the runoff in the NMD starting just east of Edinburg Lake and connecting to Lateral 5 of the RD. Although a portion of the flow within this new diversion drain will originate from the NMD, much of the flow in this new diversion drain will be from overland flow that is intercepted due to the alignment of the proposed diversion drain. This required modifications to the sub-basin delineation due to the flow interception - see Attachment A to this Appendix, (Project Drainage Area Map Alternative 1). Due to the revised sub-basins, this required further calculations pertaining to the NRCS Curve Numbers, Tc's, and routing reaches for RD, NMD and Willacy HEC-HMS models. These calculations can be provided upon request. Once the HEC-HMS models were completed, the storm area adjustment calculations were modified to obtain the peak flow rates that were to be used the corresponding HEC-RAS hydraulic models. Tables A20 through A24 summarize the Alternative 1 base peak flow rates for the four watersheds.

Table A20: North Main Drain Alternative 1 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=R2630W2610	258	397	518	669	784	914	1111	1260	956
Q=Userpoint5	626	969	1266	1639	1918	2241	2721	3084	2309
Q=JR1920	71	110	143	185	216	253	307	349	281
Q=JR1870	1010	1698	2324	3048	3644	4692	6273	7419	6507
Q=JR1670	1020	1575	2096	2856	3547	4462	5980	7155	6115
Q=JR1490	942	1674	2415	3377	4058	4925	6179	7227	7335
Q=JR1420	1341	1948	2337	2855	3230	3714	4399	5650	8680
Q=JR2690	1345	1951	2316	2775	3121	3571	4408	6103	10041
Q=UserPoint11	1217	1868	2192	2602	2915	3373	4379	5913	9976
Q=JR2700	999	1381	1533	1723	1891	2273	2996	4169	8052
Q=UserPoint12	991	1363	1511	1698	1883	2258	2965	4023	7968
Q=UserPoint16	987	1344	1469	1672	1863	2223	2894	3799	7689
Q=UserPoint17	968	1340	1464	1670	1860	2216	2855	3711	7157
Q=JR1780	995	1301	1397	1608	1769	2045	2563	3146	5777
Q=JR2780	1006	1329	1398	1608	1769	2044	2561	3144	5778
Q=UserPoint18	999	1322	1398	1608	1768	2043	2559	3142	5728
Q=Userpoint5	626	969	1266	1639	1918	2241	2721	3084	2309
Q=JR2210	1485	2205	2823	3597	4183	4955	5951	6765	5483
Q=JR2190	1420	2220	2891	3732	4377	5246	6341	7230	6021
Q=JR2180	1461	2139	2752	3541	4142	4949	6085	7010	5748
Q=JR2150	1617	2373	2980	3783	4396	5215	6314	7113	6465
Q=JR2090	1684	2289	2715	3298	3748	4368	5173	5786	5965
Q=JR2060	1741	2531	3030	3687	4188	4871	5769	6451	6589
Q=JR1840	1994	3185	3902	4820	5512	6490	7734	8741	8929
Q=JR1630	1815	3105	4128	5326	6203	7407	8811	9589	10647
Q=JR1800	1572	2667	3405	4151	4697	5436	6459	7287	9611
Q=JR1790	1780	3003	3782	4575	5194	6052	7247	8157	11015
Q=JR1450	1739	2899	3465	4154	4751	5615	6697	7564	10305
Q=JR1280	1710	2931	3362	3958	4454	5287	6297	7101	9947
Q=JR1290	1597	2687	3116	3636	4047	4687	5543	6236	8843
Q=JR1190	1625	2750	3171	3692	4105	4759	5630	6331	8995
Q=OutletNMD	1597	2583	3012	3512	3922	4525	5245	5860	8492
Q=JR3	1623	2753	3168	3661	4061	4715	5489	6138	8874
Q=JR4	1643	2849	3260	3751	4152	4829	5635	6304	9099
Q=JR5	1651	2845	3259	3751	4153	4830	5634	6303	9097
Q=JR7	1650	2845	3259	3750	4149	4820	5619	6286	9086
Q=JR8	1672	2899	3303	3782	4178	4852	5657	6329	9042
Q=JR10	1654	2833	3262	3752	4137	4746	5524	6189	8862
Q=JR12 OUTLET	1597	2744	3201	3701	4091	4693	5465	6122	8767

Table A21: Raymondville Drain Alternative 1 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR1380A1	150	443	611	837	988	1124	1342	1549	2004
JR1380A1	150	443	611	837	988	1124	1342	1549	2004
JR1230A1	181	438	603	814	972	1163	1341	1548	2034
JR1240A1	666	1291	1798	2471	2988	3602	4668	5433	4639
JR1160A1	680	1346	1872	2561	3119	3745	4813	5605	4861
JR1030A1	728	1396	1948	2657	3302	3921	5082	5830	5164
JR1360A1	993	2234	3356	4733	5919	6967	8856	10411	9838
JR1040A1	995	2173	3136	4218	5045	5800	8036	9909	9535
JR1060A1	995	2173	3135	4217	5044	5800	8036	9905	9537
JR940A1	1013	2206	3192	4299	5148	5923	8225	10194	9906
JR660A1	1017	2218	3217	4339	5204	5992	8312	10380	10260
JR660A1	1017	2218	3217	4339	5204	5992	8312	10380	10260
R660W660	346	547	712	917	1087	1262	1555	1773	1291
JR770	1022	2251	3278	4433	5386	6030	8224	10412	11085
R750W750	181	284	369	474	561	651	798	907	641
R750W750	181	284	369	474	561	651	798	907	641
R750W750	181	284	369	474	561	651	798	907	641
JR770	1022	2251	3278	4433	5386	6030	8224	10412	11085
JR780	1085	2288	3339	4525	5517	5680	8355	10639	11637
JR720	1779	3344	4646	6278	7647	9014	11602	14229	16079
JR630	0	0	0	0	2	134	566	960	1811
JR1570	1746	3301	4591	6221	7582	8938	11152	14080	16202

Table A22: Delta Lake Alternative 1 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1120	225	369	480	616	729	845	1037	1176	783
JR 1120	225	369	480	616	729	845	1037	1176	783
JR 1080	581	955	1260	1644	1960	2279	2819	3275	2937
JR 1620	808	1358	1797	2374	2853	3345	4174	4730	3692
USERPOINT 8	804	1353	1795	2356	2835	3335	4168	4723	3702
JR 790	1019	1790	2422	3235	3908	4613	5818	6699	5401
JR 720	1779	3344	4646	6278	7647	9014	11602	14080	16079
JR 1570	1746	3301	4591	6221	7582	8938	11152	14229	16202

Table A23: Willacy Alternative 1 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1570	1746	3301	4591	6221	7582	8938	11152	14080	16202
USERPOINT 3	1708	3158	4489	6186	7354	7788	10513	13364	15763
JR 1560	1995	3672	5095	6946	8339	9199	11347	12904	15609
USERPOINT 2	1882	3505	4970	6840	8225	8989	11195	12828	15607
JR 390	1849	3449	4751	6547	7820	8638	11296	12753	15579
JR 400	1705	3116	4297	5627	6930	7768	11147	12770	15052
JR 420	1809	3237	4442	5821	7203	8160	11589	13236	15662
OUTLET 35/ SOURCE1	1861	3304	4529	5934	7362	8371	11838	13508	16019
JR 490	1940	3421	4694	5902	7079	7975	11326	13350	16489
JR 500	2014	3521	4794	6032	7230	8182	11561	13614	17076
JR 530	2052	3566	4846	6106	7295	8198	11312	13359	16962
JR 540	2090	3537	4837	6137	7316	8246	11332	13383	17090
JR 460	2072	3534	4831	6139	7303	8243	11310	13338	17113

Table A23: Willacy Alternative 1 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 370	2092	3560	4860	6178	7345	8297	11359	13390	17231
JR 590	2285	3830	5153	6526	7751	8802	11813	13863	18218
OUTLET	1710	3201	4560	6017	7230	8227	10863	12744	17588

5.10.1.2 Alternative 2

As described in Section 4.3, Alternative 2 is similar to Alternative 1 in that it includes a new diversion drain that would divert a portion of the runoff in the North Main Drain starting just east of Edinburg Lake, but instead of connecting to Lateral 5 of the Raymondville Drain, it instead connects to an existing drainage drain that flows along the north edge of Delta Lake. This drain eventually ties back to the Raymondville Drain near the town of La Sara. Similar to the previous alternatives, modifications to the sub-basin delineation were needed due to the proposed drain alignment. The drainage area map for Alternative 2 is found in Attachment A to this Appendix, (Project Drainage Area Map Alternative 2). Due to the revised sub-basins, this required further calculations pertaining to the NRCS Curve Numbers, Tc's, and routing reaches for Raymondville Drain, North Main Drain and Willacy HEC-HMS models. These calculations can be provided upon request. Once the HEC-HMS models were completed, the storm area adjustment calculations were modified to obtain the peak flow rates that were to be used the corresponding HEC-RAS hydraulic models. Tables A24 through A27 summarize the Alternative 2 base peak flow rates for the four watersheds.

Table A24: North Main Drain Alternative 2 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=R2630W2610	258	397	518	669	784	914	1111	1260	956
Q=Userpoint5	626	969	1266	1639	1918	2241	2721	3084	2309
Q=JR1920	71	110	143	185	216	253	307	349	281
Q=JR1870	1010	1698	2324	3048	3644	4692	6273	7419	6507
Q=JR1670	1020	1575	2096	2856	3547	4462	5980	7155	6115
Q=JR1490	942	1674	2415	3377	4058	4925	6179	7227	7335
Q=JR1420	1341	1948	2337	2855	3230	3714	4399	5650	8680
Q=JR2690	1345	1951	2316	2775	3121	3571	4408	6103	10041
Q=UserPoint11	1217	1868	2192	2602	2915	3373	4379	5913	9976
Q=JR2700	999	1381	1533	1723	1891	2273	2996	4169	8052
Q=UserPoint12	991	1363	1511	1698	1883	2258	2965	4023	7968
Q=UserPoint16	987	1344	1469	1672	1863	2223	2894	3799	7689
Q=UserPoint17	968	1340	1464	1670	1860	2216	2855	3711	7157
Q=JR1780	995	1301	1397	1608	1769	2045	2563	3146	5777
Q=JR2780	1006	1329	1398	1608	1769	2044	2561	3144	5778
Q=UserPoint18	999	1322	1398	1608	1768	2043	2559	3142	5728
Q=Userpoint5	626	969	1266	1639	1918	2241	2721	3084	2309
Q=JR2210	1485	2205	2823	3597	4183	4955	5951	6765	5483
Q=JR2190	1420	2220	2891	3732	4377	5246	6341	7230	6021
Q=JR2180	1461	2139	2752	3541	4142	4949	6085	7010	5748
Q=JR2150	1617	2373	2980	3783	4396	5215	6314	7113	6465
Q=JR2090	1684	2289	2715	3298	3748	4368	5173	5786	5965
Q=JR2060	1741	2531	3030	3687	4188	4871	5769	6451	6589
Q=JR1840	1994	3185	3902	4820	5512	6490	7734	8741	8929
Q=JR1630	1815	3105	4128	5326	6203	7407	8811	9589	10647
Q=JR1800	1572	2667	3405	4151	4697	5436	6459	7287	9611

Table A24: North Main Drain Alternative 2 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=JR1790	1780	3003	3782	4575	5194	6052	7247	8157	11015
Q=JR1450	1739	2899	3465	4154	4751	5615	6697	7564	10305
Q=JR1280	1710	2931	3362	3958	4454	5287	6297	7101	9947
Q=JR1290	1597	2687	3116	3636	4047	4687	5543	6236	8843
Q=JR1190	1625	2750	3171	3692	4105	4759	5630	6331	8995
Q=OutletNMD	1597	2583	3012	3512	3922	4525	5245	5860	8492
Q=JR3	1623	2753	3168	3661	4061	4715	5489	6138	8874
Q=JR4	1643	2849	3260	3751	4152	4829	5635	6304	9099
Q=JR5	1651	2845	3259	3751	4153	4830	5634	6303	9097
Q=JR7	1650	2845	3259	3750	4149	4820	5619	6286	9086
Q=JR8	1672	2899	3303	3782	4178	4852	5657	6329	9042
Q=JR10	1654	2833	3262	3752	4137	4746	5524	6189	8862
Q=JR12 OUTLET	1597	2744	3201	3701	4091	4693	5465	6122	8767

Table A25: Raymondville Drain Alternative 2 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
R660W660	344	546	713	920	1093	1273	1573	1797	1462
R660W660	344	546	713	920	1093	1273	1573	1797	1462
R660W660	344	546	713	920	1093	1273	1573	1797	1462
R750W750	181	284	369	474	561	651	798	907	641
R750W750	181	284	369	474	561	651	798	907	641
R750W750	181	284	369	474	561	651	798	907	641
JR770	492	784	1024	1324	1574	1834	2267	2588	1967
JR780	646	976	1207	1468	1684	1945	2355	2686	2368
JR720	1752	3341	4686	6555	7985	9452	12011	15097	16280
JR630	0	0	0	0	2	134	566	960	1811
JR1570	1717	3299	4632	6489	7923	9380	11882	14951	16412

Table A26: Delta Lake Alternative 2 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR1380A1	150	443	611	837	988	1124	1341	1549	2004
JR1380A1	150	443	611	837	988	1124	1341	1549	2004
JR1230A1	181	438	603	814	972	1163	1341	1548	2034
JR1240A1	666	1291	1798	2471	2988	3602	4668	5433	4639
JR1160A1	680	1346	1872	2561	3119	3745	4813	5605	4861
JR1030A1	728	1396	1948	2657	3302	3921	5082	5830	5164
JR1360A1	993	2234	3356	4733	5919	6967	8856	10411	9838
JR1040A1	1006	2193	3169	4265	5105	5872	8146	10080	9745
JR 1040A1	1006	2193	3169	4265	5105	5872	8146	10080	9745
JR 1120	1024	2231	3234	4363	5233	6028	8291	10312	10166
JR 1080	1200	2530	3682	4987	6006	6952	9400	11812	12042
JR 1620	1213	2553	3724	5054	6099	7091	9516	12032	12660
USERPOINT 8	1212	2553	3723	5053	6098	7089	9508	12021	12707
JR 790	1374	2809	4060	5523	6692	7892	10272	13040	13951
JR 720	1752	3341	4686	6555	7985	9452	12011	15097	16280
JR 1570	1717	3299	4632	6489	7923	9380	11882	14951	16412

Table A27: Willacy Alternative 2 Peak Flow Rates (cfs) Base

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1570	1717	3299	4632	6489	7923	9380	11882	14951	16412
USERPOINT 3	1681	3237	4595	6342	7637	8967	11430	14172	16089
JR 1560	1974	3698	5135	7026	8422	9910	12518	15498	18220
USERPOINT 2	1868	3566	5042	6925	8353	9824	12403	15205	18134
JR 390	1849	3486	4781	6607	8153	9730	12346	14414	17903
JR 400	1714	3141	4313	5651	7456	9208	12034	13464	16811
JR 420	1816	3263	4455	5835	7707	9552	12519	13969	17707
OUTLET 35/ SOURCE1	1867	3329	4540	5943	7851	9743	12779	14268	18158
JR 490	1946	3448	4703	5901	7384	9085	11917	14071	18308
JR 500	2020	3548	4801	6025	7512	9254	12153	14338	18941
JR 530	2058	3593	4852	6094	7554	9128	11813	13842	18474
JR 540	2096	3561	4841	6119	7554	9135	11823	13869	18588
JR 460	2080	3557	4835	6119	7527	9096	11791	13811	18571
JR 370	2099	3583	4862	6154	7562	9135	11838	13863	18689
JR 590	2293	3851	5152	6493	7916	9526	12269	14328	19660
OUTLET	1713	3213	4566	6002	7376	8836	11177	13046	18579

5.10.2 FUTURE YEAR WITH PROJECT CONDITIONS

For the current year With-project conditions hydrologic models, the results from the With-project HEC-HMS models were modified based on the analysis presented in Section 5.8.2. As shown in Section 5.8.2, a multiplier of 1.35 will be applied to the peak flow rates found at each junction. These new peak flow rates will be input into the With-project HEC-RAS hydraulic models to determine the projected water surface elevations for the alternative plans. The following sections detail the modifications to the previous With-project HEC-HMS model results and the resulting projected peak flow rates.

5.10.2.1 Alternative 1

Projected peak flow rates for Alternative 1 were calculated using the flow rates found in Table A20 through Table A23. A multiplier of 1.35 was applied to these peak flow rates to determine the projected future year flow rates as previously described in Section 5.8.2. Table A28 through Table A31 summarizes the Alternative 1 future peak flow rates for the four watersheds. Detailed future Flow Comparison tables for Alternative 1 are available upon request.

Table A28: North Main Drain Alternative 1 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=R2630W2610	348	536	699	903	1058	1234	1499	1701	956
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR1920	96	148	193	250	292	342	415	471	281
Q=JR1870	1364	2292	3138	4115	4920	6334	8469	10016	6507
Q=JR1670	1377	2126	2829	3856	4789	6024	8073	9659	6115
Q=JR1490	1271	2260	3260	4559	5478	6648	8342	9756	7335
Q=JR1420	1811	2630	3155	3854	4360	5014	5938	7627	8680
Q=JR2690	1816	2635	3126	3746	4213	4820	5951	8239	10041
Q=UserPoint11	1643	2522	2959	3512	3935	4553	5912	7983	9976
Q=JR2700	1349	1864	2069	2327	2553	3069	4044	5629	8052
Q=UserPoint12	1337	1840	2040	2292	2541	3049	4003	5431	7968
Q=UserPoint16	1332	1814	1983	2257	2516	3000	3907	5129	7689
Q=UserPoint17	1307	1809	1976	2254	2511	2992	3854	5010	7157
Q=JR1780	1343	1756	1885	2171	2388	2760	3459	4247	5777

Table A28: North Main Drain Alternative 1 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=JR2780	1359	1794	1887	2171	2388	2759	3458	4245	5778
Q=UserPoint18	1348	1784	1887	2170	2387	2758	3455	4242	5728
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR2210	2005	2977	3811	4856	5647	6689	8034	9133	5483
Q=JR2190	1918	2997	3903	5039	5909	7082	8561	9761	6021
Q=JR2180	1973	2888	3716	4781	5592	6681	8215	9464	5748
Q=JR2150	2183	3203	4022	5107	5934	7040	8523	9603	6465
Q=JR2090	2274	3090	3666	4452	5060	5897	6984	7811	5965
Q=JR2060	2351	3416	4091	4978	5654	6576	7788	8709	6589
Q=JR1840	2692	4300	5267	6507	7441	8761	10441	11801	8929
Q=JR1630	2450	4192	5573	7191	8373	9999	11894	12945	10647
Q=JR1800	2122	3600	4597	5604	6340	7339	8719	9837	9611
Q=JR1790	2403	4054	5106	6176	7012	8171	9784	11011	11015
Q=JR1450	2348	3913	4677	5607	6414	7580	9041	10212	10305
Q=JR1280	2309	3956	4539	5343	6014	7138	8500	9587	9947
Q=JR1290	2156	3627	4207	4909	5464	6328	7483	8419	8843
Q=JR1190	2194	3712	4281	4985	5542	6424	7600	8547	8995
Q=OutletNMD	2156	3487	4066	4742	5295	6109	7081	7911	8492
Q=JR3	2191	3717	4277	4942	5482	6365	7411	8287	8874
Q=JR4	2219	3846	4401	5064	5605	6519	7607	8510	9099
Q=JR5	2229	3841	4400	5064	5607	6521	7606	8509	9097
Q=JR7	2227	3840	4400	5062	5602	6507	7586	8486	9086
Q=JR8	2257	3913	4460	5106	5640	6551	7636	8544	9042
Q=JR10	2233	3825	4404	5065	5585	6407	7458	8355	8862
Q=JR12 OUTLET	2155	3704	4321	4996	5522	6335	7377	8265	8767

Table A29: Raymondville Drain Alternative 1 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR1380A1	203	598	825	1131	1334	1518	1811	2090	2004
JR1380A1	203	598	825	1131	1334	1518	1811	2090	2004
JR1230A1	244	592	815	1099	1312	1571	1811	2090	2034
JR1240A1	899	1743	2428	3335	4033	4863	6302	7335	4639
JR1160A1	918	1817	2528	3457	4210	5056	6498	7566	4861
JR1030A1	982	1885	2630	3586	4457	5294	6861	7871	5164
JR1360A1	1340	3016	4531	6390	7991	9405	11955	14055	9838
JR1040A1	1343	2934	4233	5694	6811	7830	10849	13377	9535
JR1060A1	1343	2933	4233	5693	6810	7830	10849	13372	9537
JR940A1	1367	2979	4309	5803	6950	7996	11104	13762	9906
JR660A1	1373	2994	4343	5858	7025	8089	11221	14013	10260
JR660A1	1373	2994	4343	5858	7025	8089	11221	14013	10260
R660W660	468	739	961	1238	1467	1704	2100	2394	1291
JR770	1380	3038	4426	5984	7270	8141	11103	14057	11085
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
JR770	1380	3039	4426	5984	7270	8141	11103	14057	11085
JR780	1464	3088	4508	6109	7448	7668	11279	14362	11637
JR720	2401	4514	6273	8476	10324	12169	15663	19209	16079
JR630	0	0	0	0	3	181	765	1296	1811
JR1570	2357	4456	6197	8398	10235	12066	15055	19008	16202

Table A30: Delta Lake Alternative 1 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1120	303	499	648	832	985	1141	1399	1587	783
JR 1120	303	499	648	832	985	1141	1399	1587	783
JR 1080	784	1289	1701	2220	2646	3076	3805	4421	2937
JR 1620	1091	1834	2426	3205	3851	4516	5635	6385	3692
USERPOINT 8	1085	1826	2423	3180	3828	4502	5626	6376	3702
JR 790	1375	2417	3270	4367	5276	6227	7854	9043	5401
JR 720	2401	4514	6273	8476	10324	12169	15663	19008	16079
JR 1570	2357	4456	6197	8398	10235	12066	15055	19209	16202

Table A31: Willacy Alternative 1 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1570	2357	4456	6197	8398	10235	12066	15055	19008	16202
USERPOINT 3	2305	4264	6060	8351	9928	10514	14192	18042	15763
JR 1560	2693	4958	6879	9377	11257	12418	15319	17420	15609
USERPOINT 2	2540	4732	6709	9235	11104	12135	15113	17318	15607
JR 390	2496	4657	6414	8838	10557	11661	15249	17217	15579
JR 400	2301	4206	5801	7597	9355	10487	15048	17240	15052
JR 420	2442	4370	5997	7858	9724	11016	15645	17869	15662
OUTLET 35/ SOURCE1	2512	4460	6115	8011	9938	11301	15981	18236	16019
JR 490	2619	4619	6337	7967	9557	10767	15291	18022	16489
JR 500	2719	4753	6472	8143	9761	11046	15607	18379	17076
JR 530	2770	4814	6541	8244	9848	11068	15271	18035	16962
JR 540	2821	4775	6530	8285	9876	11133	15299	18067	17090
JR 460	2797	4770	6522	8288	9859	11127	15269	18006	17113
JR 370	2824	4806	6561	8340	9916	11201	15334	18077	17231
JR 590	3085	5170	6956	8810	10464	11882	15947	18716	18218
OUTLET	2308	4321	6156	8123	9761	11106	14665	17204	17588

5.10.2.2 Alternative 2

Projected peak flow rates for Alternative 2 were calculated using the flow rates found in Table A24 through Table A27. A multiplier of 1.35 was applied to these peak flow rates to determine the projected future year flow rates as previously described in Section 5.8.2. Table A32 through Table A35 summarize the Alternative 2 future peak flow rates for the four watersheds. Detailed Future Flow Comparison tables for Alternative 2 are available upon request

Table A32: North Main Drain Alternative 2 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=R2630W2610	348	536	699	903	1058	1234	1499	1701	956
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR1920	96	148	193	250	292	342	415	471	281
Q=JR1870	1364	2292	3138	4115	4920	6334	8469	10016	6507
Q=JR1670	1377	2126	2829	3856	4789	6024	8073	9659	6115
Q=JR1490	1271	2260	3260	4559	5478	6648	8342	9756	7335

Q=JR1420	1811	2630	3155	3854	4360	5014	5938	7627	8680
Q=JR2690	1816	2635	3126	3746	4213	4820	5951	8239	10041
Q=UserPoint11	1643	2522	2959	3512	3935	4553	5912	7983	9976
Q=JR2700	1349	1864	2069	2327	2553	3069	4044	5629	8052
Q=UserPoint12	1337	1840	2040	2292	2541	3049	4003	5431	7968
Q=UserPoint16	1332	1814	1983	2257	2516	3000	3907	5129	7689
Q=UserPoint17	1307	1809	1976	2254	2511	2992	3854	5010	7157
Q=JR1780	1343	1756	1885	2171	2388	2760	3459	4247	5777
Q=JR2780	1359	1794	1887	2171	2388	2759	3458	4245	5778
Q=UserPoint18	1348	1784	1887	2170	2387	2758	3455	4242	5728
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR2210	2005	2977	3811	4856	5647	6689	8034	9133	5483
Q=JR2190	1918	2997	3903	5039	5909	7082	8561	9761	6021
Q=JR2180	1973	2888	3716	4781	5592	6681	8215	9464	5748
Q=JR2150	2183	3203	4022	5107	5934	7040	8523	9603	6465
Q=JR2090	2274	3090	3666	4452	5060	5897	6984	7811	5965
Q=JR2060	2351	3416	4091	4978	5654	6576	7788	8709	6589
Q=JR1840	2692	4300	5267	6507	7441	8761	10441	11801	8929
Q=JR1630	2450	4192	5573	7191	8373	9999	11894	12945	10647
Q=JR1800	2122	3600	4597	5604	6340	7339	8719	9837	9611
Q=JR1790	2403	4054	5106	6176	7012	8171	9784	11011	11015
Q=JR1450	2348	3913	4677	5607	6414	7580	9041	10212	10305
Q=JR1280	2309	3956	4539	5343	6014	7138	8500	9587	9947
Q=JR1290	2156	3627	4207	4909	5464	6328	7483	8419	8843
Q=JR1190	2194	3712	4281	4985	5542	6424	7600	8547	8995
Q=OutletNMD	2156	3487	4066	4742	5295	6109	7081	7911	8492
Q=JR3	2191	3717	4277	4942	5482	6365	7411	8287	8874
Q=JR4	2219	3846	4401	5064	5605	6519	7607	8510	9099
Q=JR5	2229	3841	4400	5064	5607	6521	7606	8509	9097
Q=JR7	2227	3840	4400	5062	5602	6507	7586	8486	9086
Q=JR8	2257	3913	4460	5106	5640	6551	7636	8544	9042
Q=JR10	2233	3825	4404	5065	5585	6407	7458	8355	8862
Q=JR12 OUTLET	2155	3704	4321	4996	5522	6335	7377	8265	8767

Table A33: Raymondville Drain Alternative 2 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
JR770	664	1058	1383	1788	2125	2476	3060	3494	1967
JR780	872	1318	1630	1981	2273	2626	3179	3626	2368
JR720	2366	4510	6326	8849	10780	12760	16215	20380	16280
JR630	0	0	0	0	3	181	765	1296	1811
JR1570	2318	4453	6253	8760	10696	12662	16041	20184	16412

Table A34: Delta Lake Alternative 2 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR1380A1	203	598	825	1131	1334	1518	1811	2090	2004
JR1380A1	203	598	825	1131	1334	1518	1811	2090	2004
JR1230A1	244	592	815	1099	1312	1571	1811	2090	2034
JR1240A1	899	1743	2428	3335	4033	4863	6302	7335	4639
JR1160A1	918	1817	2528	3457	4210	5056	6498	7566	4861
JR1030A1	982	1885	2630	3586	4457	5294	6861	7871	5164
JR1360A1	1340	3016	4531	6390	7991	9405	11955	14055	9838
JR1040A1	1358	2961	4278	5758	6892	7927	10997	13608	9745
JR 1040A1	1358	2961	4278	5758	6892	7927	10997	13608	9745
JR 1120	1383	3012	4366	5890	7065	8138	11193	13921	10166
JR 1080	1619	3415	4971	6733	8108	9386	12690	15946	12042
JR 1620	1638	3447	5027	6823	8234	9572	12846	16243	12660
USERPOINT 8	1637	3446	5026	6821	8232	9571	12836	16228	12707
JR 790	1855	3792	5482	7457	9035	10654	13867	17605	13951
JR 720	2366	4510	6326	8849	10780	12760	16215	20380	16280
JR 1570	2318	4453	6253	8760	10696	12662	16041	20184	16412

Table A35: Willacy Alternative 2 Peak Flow Rates (cfs) Future

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1570	2318	4453	6253	8760	10696	12662	16041	20184	16412
USERPOINT 3	2270	4369	6204	8561	10309	12106	15431	19132	16089
JR 1560	2665	4993	6932	9485	11370	13378	16900	20922	18220
USERPOINT 2	2522	4814	6806	9349	11276	13263	16744	20527	18134
JR 390	2496	4706	6454	8920	11007	13135	16667	19460	17903
JR 400	2313	4240	5823	7629	10066	12431	16246	18176	16811
JR 420	2452	4405	6015	7878	10405	12895	16901	18858	17707
OUTLET 35/ SOURCE1	2521	4495	6129	8023	10598	13153	17252	19262	18158
JR 490	2627	4655	6349	7966	9968	12264	16088	18996	18308
JR 500	2727	4790	6481	8134	10141	12493	16406	19356	18941
JR 530	2778	4850	6550	8227	10197	12323	15948	18687	18474
JR 540	2830	4808	6536	8261	10197	12333	15961	18723	18588
JR 460	2808	4802	6527	8260	10161	12279	15918	18645	18571
JR 370	2834	4837	6564	8308	10209	12332	15981	18715	18689
JR 590	3095	5199	6955	8765	10686	12860	16563	19343	19660
OUTLET	2313	4338	6163	8103	9958	11928	15089	17612	18579

A1 – SECTION 6 HYDRAULICS

6.1 GENERAL

Once the peak flow rates for the various storm events and alternative plans were calculated, hydraulic models were prepared to determine the water surface elevation and width of inundation for the without and With-project conditions. The results of the hydraulic models were utilized to determine the flood damages associated for each storm event for the base conditions and alternative plans. This information was used to evaluate the effectiveness of each alternative plan.

6.2 WITHOUT PROJECT (WITHOUT-PROJECT/BASE) CONDITIONS MODEL DEVELOPMENT

The base hydraulic models for the RD and NMD systems were originally created using HEC-RAS. The hydraulic modeling option used for this analysis was the steady state option to determine the base conditions flood depths for use in the alternatives analysis. The flood depths were used to prepare the flood damage assessments for the surrounding structures and agricultural land as detailed in Appendix A-5. The peak flows calculated in the base HEC-HMS models were input into the HEC-RAS model to determine the base water surface elevations and base flood depths for each storm event. The without-project condition was the basis for all further hydraulic analysis of the alternative plans.

6.3 WITHOUT-PROJECT CROSS SECTION DEVELOPMENT

Both the RD and NMD systems are man-made structures designed convey stormwater runoff away from the towns of Raymondville, San Perlita, La Sara, and Hargill. The drain is typically trapezoidal with varying bottom widths throughout the reaches. Cross section data was obtained from both LIDAR data and on-the-ground field survey along both drainage drains. The Project Cross Section Map, The Without-project Conditions map is found in Attachment B to this Appendix, (Project Cross Section Map, Pre-Project Conditions).

6.4 WITHOUT-PROJECT INFLOW LOCATIONS

As shown in the without-project HEC-HMS hydrologic model, the peak flow rates within the watershed vary along the main drains. To accurately model this in the HEC-RAS hydraulic model, inflow locations were selected along the existing drainage drain alignment to model the addition of these flows along the alignment. The Project Inflow Locations Map for the without-project conditions is included in Attachment A to this Appendix, (Project Inflow Locations, Pre-Project Conditions).

6.5 WITHOUT-PROJECT BRIDGES, CULVERTS, WEIRS, AND GATED STRUCTURES MODELING

During the preparation of the HEC-RAS models, additional structures were added to the models based on as-built plans provided by TxDOT for on-system bridge structures found within the TxDOT system. For all other structures (off-system), a structure inventory study was conducted to determine the required geometric data to input into the updated HEC-RAS model. Data collected for the structure inventory study can be provided upon request. Once the additional structures were input into the models, the water surface elevations were calculated for the nine storm events. These water surface elevations were then compared to the as-built plans to determine the flood frequency that favorably compares to the documented high-water marks. The without-project HEC-RAS models will be used to study the proposed measures and alternatives for flood reduction and flood damage control. This was then duplicated for each of the two considered alternatives in order to compare them and aid in determining which was the most effective.

6.6 WITHOUT-PROJECT MANNING'S "N" VALUES

The Manning's roughness coefficients ("n"-values) used in the analysis were based on observations from the aerial photographs and site inspections for both the RD and NMD systems, utilizing reference tables from the HEC-RAS hydraulic reference manual. The Manning's roughness coefficient for the main drain was selected to be 0.06. The overbank roughness coefficient was selected as 0.10. This high value was chosen due to the flat overbanks found along these drainage drains, as well as structural obstructions and crops present in these areas. The selection of these Manning's roughness coefficients was also coordinated with the USACE during the early portion of this project and validated through calibration.

6.7 WITHOUT-PROJECT EXPANSION/CONTRACTION COEFFICIENTS

During standard step backwater calculations, additional losses occur during expansion and contraction of flow between each cross section. For gradual expansions and contractions, values of 0.1 and 0.3 respectively, are generally accepted to adequately model these losses. However, at bridge and culvert crossing, there are more significant and rapid contractions and expansions of flow. For these areas, the expansion and contraction coefficients have been increased to 0.3 and 0.5, respectively.

6.8 WITHOUT-PROJECT INEFFECTIVE FLOW AREAS, BLOCKED OBSTRUCTIONS, AND LEVEES

Ineffective flow areas are used in hydraulic modeling to accurately reflect those areas along the stream that do not contribute to the overall drain flow. Typically, these areas are located in the portions of the overbanks where the water velocity approaches zero. This commonly occurs at bridge and culvert crossings where the structure width is significantly less than the overall floodplain width. Ineffective flow areas have been included at all existing structure crossings for both the RD and NMD systems. Additionally, the spoil banks along the RD and NMD required the use of ineffective flow designations. Although it may appear that these spoil banks should be modeled as levees, there are numerous utility and roadway crossings that allow water into the overbanks. As a result, for these models, it was determined that the areas behind the spoil banks be modeled using the ineffective flow designation.

6.9 WITHOUT-PROJECT HYDRAULIC STARTING CONDITIONS

For HEC-RAS models, a starting water surface elevation is necessary to begin the calculations for the water surface profile. The models in this report were calculated using only a subcritical flow regime. These types of models only require a starting water surface elevation at the farthest downstream point. For the hydraulic models that outfall to the Laguna Madre (NMD and Willacy), the starting water surface elevation was determined by calculating the normal depth of the drain at the farthest downstream cross section. This method requires RRP to determine the energy slope of the drain at this point. With the energy slope, it was possible to compute the normal depth at this location using Manning's Equation. The energy slope was approximated by using the average slope of the drain. For the RD model, the calculated upstream water surface elevation for each storm event from the Willacy model was used as the known starting water surface elevation. For the Delta Lake model, the water surface elevation calculated in the RD model at the connection point was utilized as the known starting water surface elevation for each storm event.

6.10 WITHOUT-PROJECT (BASE) HYDRAULIC MODEL RESULTS (WITHOUT PROJECT)

HEC-RAS results are provided for the RRP rainfall models and for the Hurricane Beulah event. The RRP model provides results for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year storm events. The peak flow rates generated from the HEC-HMS models were entered as steady state flow data into the base conditions HEC-RAS models for the Raymondville Drain, North Main Drain system, Willacy, and Delta Lake reaches. The output from the HEC-RAS models can be provided upon request.

6.10.1 CURRENT YEAR WITHOUT-PROJECT CONDITIONS (BASE)

The current year without-project conditions HEC-RAS model provided the water surface elevations for the nine storm events that were to be used in the Flood Damage Assessment. This model and the corresponding results would be utilized as the starting point to determine the efficiency of the alternative plans. The efficiency would be measured by the reduction in water surface elevation, which would result in lower flood damages for each storm event. Tables A36 through A39 detail the water surface elevations found during the base conditions for each project inflow location. Complete HEC-RAS output and water surface profiles can be provided upon request.

Table A36: North Main Drain Base Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
McAllen Lateral	333707	105.07	107.06	108.52	110.14	110.82	111.37	112.12	112.67	111.54
	317000	101.04	103.05	104.33	105.58	106.36	107.21	107.61	107.97	107.31
South Drain	315796	100.20	102.13	103.32	104.47	105.18	105.99	106.54	107.10	106.03
	298000	94.84	95.93	96.19	96.81	97.25	97.77	98.33	98.72	98.20
	292000	92.80	94.63	95.41	96.30	96.83	97.43	98.02	98.39	97.87
	286897	88.35	89.54	90.00	90.11	90.10	89.93	90.64	91.68	90.26
	275122	80.11	80.85	81.26	81.75	82.16	82.79	83.77	84.54	83.91
	262000	72.96	74.56	75.68	76.64	77.06	75.35	77.20	77.51	77.52
	250000	71.68	74.28	75.60	76.59	77.01	74.88	77.11	77.42	77.43
	248746	71.54	74.21	75.57	76.56	76.98	74.63	77.05	77.41	77.42
North Drain	109000	93.80	95.88	96.87	97.45	97.94	98.58	99.53	100.41	99.26
	95350	93.22	95.24	95.93	96.38	96.55	96.76	97.01	97.41	97.18
	92296	92.18	94.21	94.37	95.84	96.40	96.58	96.76	97.20	96.97
	85781	90.33	92.54	90.94	94.13	94.06	94.38	95.64	96.81	96.63
	71558	86.34	87.90	88.64	89.67	90.22	90.85	91.69	92.97	95.58
	66413	83.61	85.49	86.09	87.68	88.18	88.60	89.21	89.74	90.97
	64591	83.53	85.48	86.09	87.68	88.18	88.59	89.19	89.72	90.95
	62591	83.32	85.46	86.08	87.67	88.17	88.58	89.18	89.71	90.94
	59655	83.05	85.42	86.06	87.66	88.16	88.56	89.16	89.68	90.92
	55237	82.33	84.80	85.82	87.61	88.11	88.51	89.09	89.61	90.84
	46091	80.60	83.04	84.01	86.19	86.92	87.29	87.89	88.36	89.71
	31191	75.97	77.52	77.52	78.37	78.91	79.64	73.47	80.58	78.10
	26091	72.22	72.84	73.75	75.35	75.44	76.29	76.66	77.02	78.06
	19091	70.77	72.08	72.97	74.61	74.33	75.07	75.86	75.76	74.81
Main Flood Water Drain	233824	64.96	67.76	69.33	70.10	70.77	71.27	71.87	71.96	74.19
	192000	47.00	49.99	51.49	52.86	53.74	54.47	55.62	56.27	60.53
	188000	46.09	49.13	50.58	51.92	52.77	53.38	54.44	54.95	59.62
	168000	43.45	46.31	47.25	48.28	49.06	49.71	50.09	50.79	54.12
	153231	39.99	41.88	42.15	42.49	42.71	43.05	43.31	43.50	45.01
	141154	37.50	39.52	39.92	40.37	40.74	41.16	41.78	42.32	45.09
	134000	36.08	38.65	39.17	39.69	40.16	40.72	41.49	42.08	45.01
	124185	34.18	36.89	37.31	37.51	37.94	38.65	39.50	40.52	44.70
	84000	24.22	25.44	25.65	25.86	26.17	26.78	27.43	28.42	29.83
	68373	20.78	22.16	22.53	23.24	23.86	24.16	25.01	25.86	29.36
	52275	15.18	17.91	18.63	19.43	20.12	20.98	21.46	21.84	23.44
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	42466	12.63	15.11	15.78	16.53	17.13	17.98	18.78	19.48	21.58
	29585	11.45	13.57	14.09	14.68	15.16	15.87	16.66	17.43	19.59
	10000	8.21	9.78	10.17	10.48	10.70	11.03	11.50	12.05	14.67
	2000	4.12	4.85	5.11	5.37	5.57	5.86	6.23	6.61	8.23

Table A37: Raymondville Drain Base Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Lateral 5	267939.7	55.84	58.90	59.36	59.65	59.98	60.38	60.37	60.66	60.19
Lateral 4	5524.022	55.60	56.69	57.40	58.19	58.76	59.30	60.16	60.33	59.85
Lateral 3	254087.2	50.30	51.88	52.93	54.05	54.66	55.09	56.02	56.63	55.73
Trib 1	1713.911	48.96	49.38	49.64	49.83	49.87	49.90	49.99	50.05	49.97
Trib 2	3939.517	49.14	49.59	49.92	50.20	50.35	50.50	50.77	50.97	50.53
FM 88	12056.29	48.96	49.37	49.63	49.81	49.83	49.86	49.93	49.98	49.93
West Hargill Drain	222187.5	42.08	43.91	45.01	46.05	46.73	47.25	48.26	48.73	49.54
	209621.7	39.09	41.15	42.29	43.49	44.37	45.26	46.58	47.51	49.21
	205314.5	37.90	40.34	41.75	42.97	43.83	44.68	46.08	46.99	48.59
La Sal Vieja	19393.5	26.64	26.64	26.64	26.64	27.23	31.53	36.91	40.16	42.70
North Hargill	202200.2	36.75	38.31	39.02	39.62	40.01	40.35	40.98	41.36	42.40
	201934.8	36.61	38.10	38.77	39.29	39.60	39.82	40.27	40.41	40.74

Table A38: Willacy Base Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Raymondville Drain (Willacy)	191551.8	34.99	35.61	35.85	36.10	36.28	36.45	36.69	36.89	37.26
	187949	34.33	35.03	35.34	35.61	35.82	36.00	36.26	36.42	36.77
	181079.2	33.47	33.97	34.26	34.55	34.76	34.96	35.27	35.46	35.82
	165775.1	32.19	32.96	33.28	33.57	33.78	34.01	34.35	34.58	35.00
	145744.9	29.34	30.35	30.32	31.11	31.57	31.90	32.57	32.92	33.24
	139999.4	27.15	28.13	28.31	28.69	28.96	29.16	29.62	29.88	30.31
	137297.8	26.61	27.26	27.63	28.02	28.23	28.46	28.95	29.16	29.69
	107273.5	20.53	21.07	21.37	21.65	21.83	22.09	22.39	22.61	23.17
	104167.2	20.35	20.87	21.19	21.44	21.60	21.89	22.15	22.36	22.90
	72996.3	14.53	15.34	15.56	15.84	16.04	16.19	16.42	16.63	17.10
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
	65979.24	11.70	12.29	12.68	13.15	13.47	13.72	14.13	14.46	15.22
	55505.96	11.21	11.84	12.25	12.67	13.00	13.26	13.63	13.92	14.64
	47254.57	10.26	11.13	11.62	12.04	12.33	12.56	12.90	13.16	13.81
	43401.41	9.24	10.24	10.84	11.30	11.62	11.86	12.19	12.45	13.13
	126.337	-0.31	0.23	0.37	0.54	0.69	0.82	0.99	1.16	1.61

Table A39: Delta Lake Base Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Delta/South Drain	72438	65.97	66.13	66.26	66.37	66.45	66.48	66.61	66.71	66.49
	63910	59.61	59.88	62.16	62.75	62.83	62.89	62.99	63.11	62.91
	48794	49.73	53.28	53.69	54.07	54.56	55.09	55.72	56.26	56.70
	29898	42.75	45.53	46.86	49.02	49.62	49.99	50.47	49.52	50.85
	26297.8	41.10	44.37	46.01	48.75	49.35	49.67	50.08	47.45	50.19
	21382.71	40.65	43.98	45.67	48.62	49.23	49.52	49.87	42.80	49.77
	3499.257	37.39	39.41	40.42	41.38	42.06	42.66	43.55	44.10	44.85
	196.869	36.75	38.31	39.02	39.62	40.01	40.35	40.98	41.36	42.40

6.10.2 FUTURE YEAR WITHOUT-PROJECT CONDITIONS

In addition to the current year HEC-RAS model, the project flows were input into the without-project HEC-RAS model as a separate flow file. This was done to ensure that the geometry and other variables would remain consistent with the current year without-project model. Of particular use of this model was the 500-year flood envelope. This flood envelope was used to determine the maximum inundation area, to delineate the number of structures that would be damaged. Tables A40 through A43 detail the water surface elevations found during the base conditions for each project inflow location for the projected Future Year. Complete HEC-RAS output and water surface profiles can be provided upon request.

Table A40: North Main Drain Base Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
McAllen Lateral	333707	106.40	108.72	110.37	111.33	111.92	112.58	113.49	114.15	111.54
	317000	102.40	104.49	105.79	107.13	107.36	107.87	108.39	108.65	107.31
South Drain	315796	101.51	103.46	104.65	105.92	106.24	106.99	108.05	108.32	106.03
	298000	95.76	96.26	96.90	97.65	98.12	98.66	99.24	99.64	98.20
	292000	94.23	95.54	96.45	97.30	97.81	98.34	98.85	99.18	97.87
	286897	89.29	90.03	90.11	89.98	89.71	91.29	93.20	91.48	90.26
	275122	80.69	81.40	81.90	82.70	83.41	84.47	85.48	86.49	83.91
	262000	73.97	76.15	76.99	75.38	77.24	77.53	77.95	77.94	77.52
	250000	73.41	76.08	76.95	74.90	77.17	77.44	77.86	77.83	77.43
	248746	73.32	76.05	76.92	74.64	77.15	77.43	77.85	77.81	77.42
North Drain	109000	95.34	96.81	97.53	98.55	99.13	100.04	101.90	102.23	99.26
	95350	94.74	96.07	96.38	96.78	96.76	96.93	98.03	97.86	97.18
	92296	93.48	95.26	95.79	94.57	96.55	96.65	97.89	97.62	96.97
	85781	91.35	94.11	94.07	94.44	95.09	96.17	97.63	95.63	96.63
	71558	87.57	89.15	89.98	90.96	91.58	92.31	93.26	94.75	95.58
	66413	84.82	86.53	87.36	88.35	88.80	89.28	89.92	90.50	90.97
	64591	84.81	86.52	87.36	88.34	88.79	89.27	89.91	90.48	90.95
	62591	84.79	86.51	87.35	88.33	88.78	89.26	89.89	90.47	90.94
	59655	84.73	86.49	87.34	88.32	88.76	89.24	89.87	90.44	90.92
	55237	84.12	86.42	87.29	88.27	88.71	89.18	89.79	90.35	90.84
	46091	82.33	84.62	85.79	86.98	87.53	87.98	88.58	89.13	89.71
	31191	77.51	77.72	78.21	79.34	80.16	73.58	80.82	82.16	78.10
	26091	72.79	74.08	75.14	75.96	76.33	76.85	77.26	77.55	78.06
	19091	72.00	73.28	74.43	74.73	75.49	76.04	75.91	73.93	74.81
Main Flood Water Drain	233824	66.56	69.40	70.33	70.82	71.70	72.02	72.18	72.49	74.19
	192000	48.81	52.09	53.62	54.63	55.42	56.31	57.76	58.06	60.53
	188000	48.02	51.24	52.66	53.49	54.20	54.98	56.27	57.34	59.62
	168000	45.21	48.00	48.98	49.72	49.92	50.81	52.03	52.98	54.12
	153231	41.29	42.49	42.75	43.09	43.25	43.50	43.71	43.96	45.01
	141154	38.95	40.38	40.85	41.29	41.73	42.22	42.79	43.43	45.09
	134000	37.72	39.70	40.30	40.93	41.44	41.97	42.59	43.27	45.01
	124185	36.02	37.48	38.05	38.88	39.48	40.31	41.29	42.39	44.70
	84000	25.03	25.90	26.25	26.85	27.38	28.30	29.34	30.54	29.83
	68373	21.50	23.36	23.60	24.39	24.95	25.77	26.57	27.55	29.36
	52275	16.72	19.57	20.46	21.15	21.44	21.81	22.23	22.63	23.44
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	42466	14.02	16.69	17.45	18.22	18.74	19.42	20.08	20.60	21.58
	29585	12.68	14.83	15.43	16.10	16.61	17.37	18.11	18.71	19.59
	10000	9.11	10.52	10.83	11.19	11.50	11.98	12.65	13.39	14.67
	2000	4.51	5.37	5.67	5.99	6.23	6.57	7.00	7.46	8.23

Table A41: Raymondville Drain Base Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Lateral 5	267939.7	57.07	59.39	59.71	60.31	60.24	60.62	61.80	62.88	60.19
Lateral 4	5524.022	56.30	57.50	58.34	59.21	59.89	60.16	60.83	61.28	59.85
Lateral 3	254087.2	51.30	53.07	54.27	54.99	55.71	56.38	57.36	58.01	55.73
Trib 1	1713.911	49.22	49.68	49.84	49.90	49.97	50.03	50.51	50.71	49.97
Trib 2	3939.517	49.40	49.97	50.24	50.49	50.70	50.91	51.41	51.70	50.53
FM 88	12056.29	49.21	49.66	49.81	49.86	49.91	49.96	50.43	50.62	49.93
West Hargill Drain	222187.5	43.27	45.07	46.20	47.16	47.95	48.54	49.66	49.86	49.54
	209621.7	40.22	42.34	43.64	45.05	46.10	47.09	48.72	48.65	49.21
	205314.5	39.05	41.75	43.08	44.47	45.52	46.57	48.12	47.84	48.59
La Sal Vieja	19393.5	26.64	26.64	26.64	26.64	27.42	32.30	38.58	42.42	42.70
North Hargill	202200.2	37.54	39.02	39.67	40.26	40.68	41.17	41.90	42.62	42.40
	201934.8	37.37	38.77	39.33	39.76	40.04	40.35	40.56	40.78	40.74

Table A42: Willacy Base Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Raymondville Drain (Willacy)	191551.8	35.36	35.86	36.12	36.39	36.60	36.81	37.10	37.34	37.26
	187949	34.70	35.34	35.64	35.93	36.16	36.37	36.66	36.85	36.77
	181079.2	33.71	34.26	34.57	34.88	35.12	35.35	35.69	35.87	35.82
	165775.1	32.68	33.29	33.62	33.93	34.16	34.41	34.77	35.00	35.00
	145744.9	29.98	30.38	31.14	31.79	32.13	32.50	33.05	33.21	33.24
	139999.4	27.84	28.36	28.80	29.12	29.37	29.59	30.00	30.23	30.31
	137297.8	27.05	27.67	28.06	28.41	28.64	28.91	29.33	29.60	29.69
	107273.5	20.85	21.36	21.63	22.09	22.22	22.39	22.74	23.02	23.17
	104167.2	20.69	21.18	21.43	21.91	21.99	22.15	22.49	22.74	22.90
	72996.3	15.06	15.54	15.81	16.09	16.28	16.44	16.72	16.96	17.10
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	65979.24	12.06	12.64	13.10	13.54	13.89	14.16	14.61	14.99	15.22
	55505.96	11.56	12.22	12.62	13.07	13.40	13.66	14.07	14.40	14.64
	47254.57	10.77	11.55	12.00	12.38	12.69	12.92	13.29	13.58	13.81
	43401.41	9.85	10.75	11.25	11.67	11.98	12.21	12.57	12.86	13.13
	126.337	0.11	0.36	0.53	0.72	0.88	1.01	1.22	1.43	1.61

Table A43: Delta Lake Base Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Delta/South Drain	72438	66.08	66.26	66.36	66.49	66.62	66.67	66.80	66.96	66.49
	63910	59.76	62.50	62.78	62.91	62.96	63.06	63.17	63.20	62.91
	48794	51.34	53.57	54.03	54.75	55.28	55.86	56.76	57.38	56.70
	29898	44.17	46.81	49.02	49.80	50.21	50.77	49.87	51.33	50.85
	26297.8	42.78	45.97	48.77	49.53	49.89	50.44	47.91	50.71	50.19
	21382.71	42.34	45.63	48.65	49.39	49.72	50.13	43.80	50.01	49.77
	3499.257	38.40	40.42	41.47	42.51	43.20	43.84	44.80	45.16	44.85
	196.869	37.54	39.02	39.67	40.26	40.68	41.17	41.90	42.62	42.40

6.11 HYDRAULIC MODEL CALIBRATION (WITHOUT-PROJECT/BASE)

A detailed report entitled "*Final Technical Memorandum, Summary of Quality Assurance Review, Hydrology and Hydraulics Base Models, Regarding Raymondville Drain Project, Project for Flood Control*," is included as Attachment C to this Appendix. Attachment C documents the Quality Assurance process for the base hydraulic models developed for this Feasibility study. Attachment D contains additional model calibration completed in response to Independent Technical Review (ITR) and Independent External Peer Review (IEPR) comments.

The water surface elevations for the eight storm events and for Hurricane Beulah were compared to the data found on the as-built plans available for some of the structures crossing the NMD system, to calibrate the HEC-HMS models for these watersheds. For example, observed water surface elevations were obtained for the Alamo Road bridge (RRP Bridge #14) and the Val Verde Road bridge (RRP Bridge #16) from available as-built construction plans. No flood frequency event was indicated on the plans for the observed water surface elevations. However, since the water surface elevations shown on the construction plans are below the bank-full elevations, it was determined that the observed water surface elevations are due to the smaller, more frequent events such as the 2-year event. Table A44 summarizes the peak flow rates and calculated water surface elevations for the nine storm events and for the data obtained from the available as-built construction plans. From the data shown in Table A44, the observed water surface elevations found on the as-built construction plans appear to correspond to the smaller events, such as the 2-year storm event.

Table A44: Comparison of Observed Water Surface

North Main Drain

Storm Event	S&B Bridge #14				S&B Bridge #16			
	HEC-RAS XS 284245				HEC-RAS XS 269855			
	Q (cfs)	WSE	Observed Water Surface Elevation*	Elevation Difference (ft)	Q (cfs)	WSE	Observed Water Surface Elevation*	Elevation Difference (ft)
2 YR	1461	84.22	85.00	-0.78	1617	77.05	77.00	0.05
5 YR	2139	85.13	85.00	0.13	2373	77.90	77.00	0.90
10 YR	2752	85.89	85.00	0.89	2980	78.71	77.00	1.71
25 YR	3541	86.83	85.00	1.83	3783	79.99	77.00	2.99
50 YR	4142	87.54	85.00	2.54	4396	80.83	77.00	3.83
100 YR	4949	88.50	85.00	3.50	5215	81.88	77.00	4.88
250 YR	6085	90.25	85.00	5.25	6314	83.16	77.00	6.16
500 YR	7010	91.52	85.00	6.52	7113	84.02	77.00	7.02
Beulah	5748	89.79	85.00	4.79	6465	83.33	77.00	6.33

* Data obtained from as-built construction plans

Other observed high-water marks, IBWC historical data, available Hidalgo County data, and documentation from the USACE for the Hurricane Beulah event were utilized in the overall validation of the without-project hydraulic models.

6.12 WITH PROJECT (ALTERNATIVES) CONDITIONS MODEL DEVELOPMENT

The With-project hydraulic models for the RD and NMD systems were prepared using HEC-RAS. These models were based on the base conditions models discussed earlier. The flow rates used in these hydraulic models were determined from the with-project HEC-HMS models discussed in previous sections of this report. These hydraulic models were used to determine the effects of the proposed improvements on both the RD and NMD systems. These models were utilized further in the preparation of a with-project flood damage assessment for the surrounding structures and agricultural land.

The effects of the proposed diversion drain, detention pond and drain improvements along RD are utilized to reduce the water surface elevations found in the NMD and to reduce potential effects to the downstream portions of the RD.

6.13 WITH PROJECT CROSS SECTION DEVELOPMENT

The geometric data used in the base models were modified to include the proposed improvements for the individual alternative plans. For instance, the alignment for the proposed diversion drain at Edinburg Lake was input into the RD HEC-RAS model and connected to the upstream end of Lateral 5. The cross sections data was also used to confirm the adequacy of the proposed right-of-way throughout the project alignment.

6.14 WITH PROJECT INFLOW LOCATIONS

The without-project conditions Project Inflow Locations Maps included in Attachment A of this Appendix, and were utilized as the starting template for each of the alternative plans. Each plan required minor modifications to the inflow locations, depending on the proposed improvements and locations of flow diversions. Project Inflow Locations Maps have also been prepared for each alternative plan and are also included in Attachment A of this Appendix. These modified project inflow locations were utilized to prepare the flow data information for each alternative plan HEC-RAS model.

6.15 WITH PROJECT BRIDGES, CULVERTS, WEIRS, AND GATED STRUCTURES MODELING

After inputting the With-project peak flow rates and widened drain geometry into the HEC-RAS model, it was discovered that many of the existing bridge and culvert crossings were inadequately sized for the drain geometry and hydrology. The With-project HEC-RAS model for RD includes improvements to the existing bridge and culvert crossings along the improved portion of RD. All culvert crossings and existing bridge crossings were improved to a bridge structure that could span a trapezoidal drain with a 160-foot bottom width. Each crossing was classified as a city, county, state Farm to Market (FM) or state highway (HWY) roadway. For the purposes of consistency between the alternative plans, fifty (50) spans were utilized with TY A prestressed concrete beams for each bridge structure. These variables were input into the geometric data files for each HEC-RAS model. However, the existing profile of the crossing was not changed for this analysis. A further study would be required to determine the extent that the profile of each road crossing would need to be modified to ensure that the design storm passes below the low chord of the bridge crossing. Figures A8 (typical roadway bridges) and A9 (railroad bridge) provide an illustration of typical proposed bridge geometry for the improved crossings.

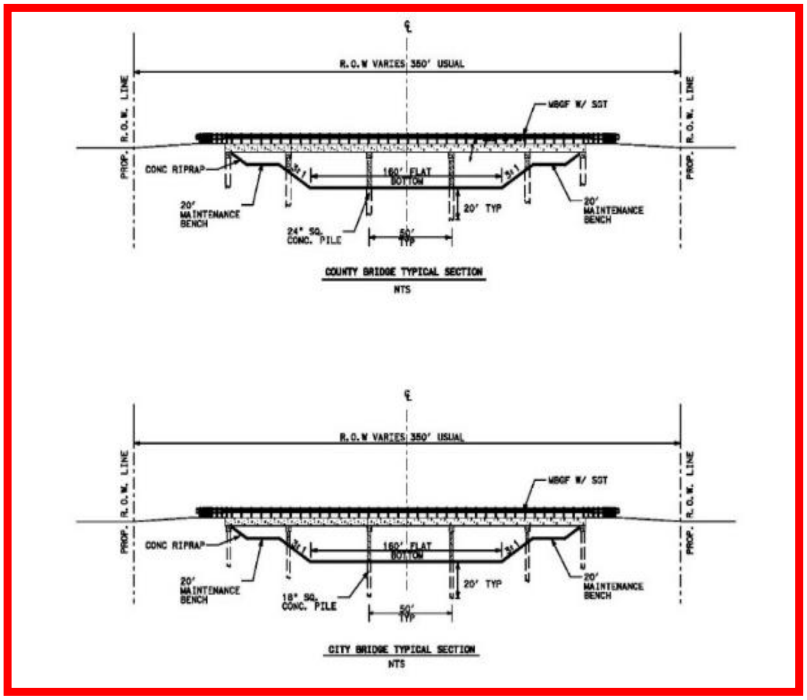


Figure A8 Typical Roadway Bridges

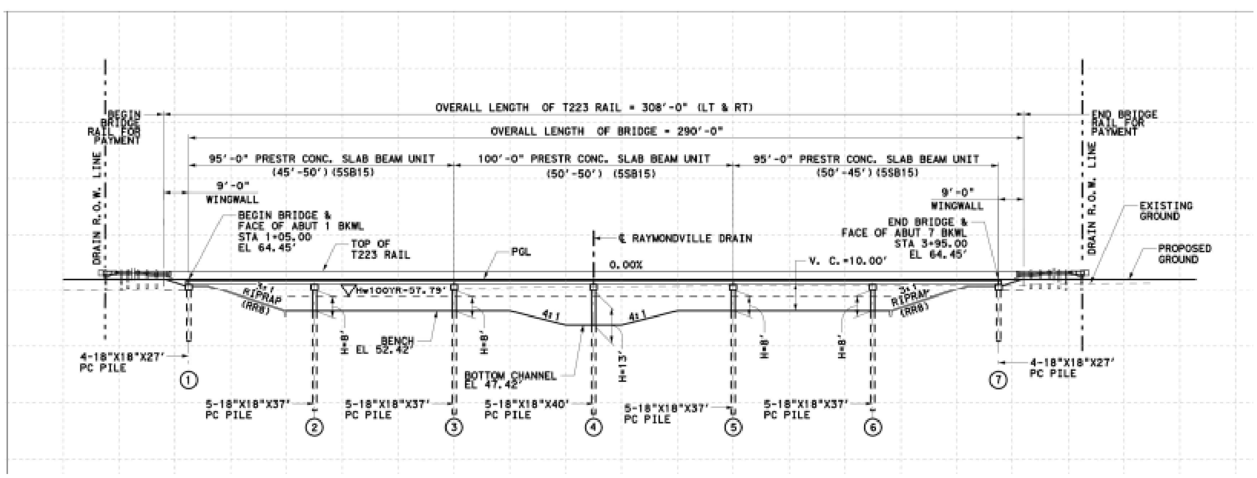


Figure A9 Railroad Bridge

6.16 WITH PROJECT MANNING'S "N" VALUES

The Manning's roughness coefficients ("n"-values) used in the base conditions analysis were based on observations from the aerial photographs and site inspections for both the RD and NMD systems. However, for the proposed diversion drain and for the portion of the RD to be improved, a Manning's roughness coefficient of 0.04 was selected. This is due to the level of maintenance that is expected for the proposed improvements. The overbank roughness coefficient of 0.10 that was utilized in the base conditions models remains unchanged in the With-project models.

6.17 WITH PROJECT EXPANSION/CONTRACTION COEFFICIENTS

The expansion and contraction coefficients for gradual expansions and contractions used in the base condition models were also used in the With-project models. Additionally, the coefficients for rapid contractions and expansions used at bridge and culvert crossings were also left unchanged from the base models previously discussed.

6.18 WITH PROJECT INEFFECTIVE FLOW AREAS, BLOCKED OBSTRUCTIONS, AND LEVEES

The ineffective flow areas used in the portion of the existing RD were modified to account for the widening of the existing drain. Additionally, the ineffective flow areas at the proposed bridge crossings were revised to account for the widened structures.

6.19 WITH PROJECT HYDRAULIC STARTING CONDITIONS

As previously described for the base conditions HEC-RAS models, the starting water surface elevation was selected by calculating the normal depth at the farthest downstream cross sections for both the NMD system and Willacy hydraulic models. The calculated water surface elevation at the upstream end of the Willacy model was used as the starting water surface elevation for the RD model. Like the base models, the water surface elevation at the Delta Lake connection with the Raymondville Drain was used as the starting water surface elevation for the Delta Lake hydraulic model.

6.20 CURRENT YEAR WITH PROJECT MODEL RESULTS (BASE)

The current year With-project conditions HEC-RAS models provided the water surface elevations for the nine storm events that were used in the Flood Damage Assessment. HEC-RAS results are provided for the RRP rainfall models and for the Hurricane Beulah event. The RRP rainfall models provide results for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year storm events. Separate HEC-RAS hydraulic models were prepared for each alternative plan. An Overall Project Matrix and an Overall Project Model File Summary that catalogs the various HEC-RAS models and files can be provided upon request. Similarly, complete output for each alternative plan can be provided upon request.

6.20.1 ALTERNATIVE 1

HEC-RAS models for the NMD system, RD, Willacy, and Delta Lake reaches were prepared using the With-project Alternative 1 flow rates previously reported Section 5.10.1.1. Table A45 through Table A48 detail the water surface elevations at each project inflow location within the project area. Complete HEC-RAS Calculations and Water Surface Profiles can be provided upon request.

Table A45: North Main Drain Alternative 1 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
McAllen Lateral	333707	105.07	107.06	108.52	110.14	110.82	111.37	112.12	112.67	111.54
	317000	101.04	103.05	104.33	105.58	106.36	107.21	107.61	107.97	107.31
South Drain	315796	100.20	102.13	103.32	104.47	105.18	105.99	106.54	107.10	106.03
	298000	94.84	95.93	96.19	96.81	97.25	97.77	98.33	98.72	98.20
	292000	92.80	94.63	95.41	96.30	96.83	97.43	98.02	98.39	97.87
	286897	88.35	89.54	90.00	90.11	90.10	89.93	90.64	91.68	90.26
	275122	80.11	80.85	81.26	81.75	82.16	82.79	83.77	84.54	83.91
	262000	72.96	74.56	75.68	76.64	77.05	75.35	77.20	77.51	77.53
	250000	71.68	74.28	75.60	76.59	77.01	74.88	77.11	77.42	77.45
	248746	71.54	74.21	75.57	76.56	76.98	74.63	77.05	77.41	77.43
North Drain	109000	93.80	95.87	96.87	97.44	97.94	98.57	99.52	100.39	99.25
	95350	93.22	95.23	95.93	96.37	96.55	96.74	96.99	97.40	97.17

Table A45: North Main Drain Alternative 1 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
	92296	92.18	94.20	94.37	95.82	96.40	96.55	96.74	97.18	96.96
	85781	90.32	92.52	90.94	94.08	94.06	94.33	95.60	96.79	96.62
	71558	86.30	87.78	88.51	89.41	89.98	90.66	91.54	92.92	95.56
	66413	83.04	84.34	84.82	85.42	85.99	86.42	87.54	88.20	89.72
	64591	82.95	84.32	84.80	85.40	85.97	86.41	87.53	88.19	89.71
	62591	82.71	84.29	84.77	85.38	85.95	86.40	87.52	88.18	89.69
	59655	82.40	84.21	84.71	85.33	85.92	86.38	87.51	88.16	89.67
	55237	81.73	83.55	84.08	84.74	85.35	86.30	87.46	88.12	89.61
	46091	80.02	81.81	82.32	82.97	83.59	84.52	85.99	86.93	88.38
	31191	75.48	76.85	77.28	77.07	77.41	77.77	78.26	78.88	80.16
	26091	72.00	72.42	72.72	73.42	73.98	74.50	74.81	75.59	76.79
	19091	70.54	71.65	72.01	72.76	73.35	73.94	73.99	74.57	75.76
Main Flood Water Drain	233824	64.96	67.76	69.32	70.08	70.75	71.26	71.87	71.96	72.04
	192000	46.98	49.98	51.45	52.83	53.71	54.46	55.62	56.27	58.06
	188000	46.06	49.12	50.55	51.89	52.74	53.36	54.43	54.95	57.38
	168000	43.41	46.29	47.23	48.26	49.03	49.70	50.08	50.78	53.07
	153231	39.93	41.87	42.14	42.49	42.70	43.04	43.30	43.50	43.96
	141154	37.40	39.50	39.91	40.35	40.71	41.14	41.73	42.10	43.39
	134000	36.00	38.63	39.17	39.66	40.13	40.69	41.44	41.84	43.23
	124185	34.07	36.85	37.32	37.48	37.87	38.58	39.37	39.94	42.29
	84000	24.12	25.43	25.65	25.85	26.12	26.71	27.34	28.00	30.55
	68373	20.71	22.15	22.52	23.21	23.79	24.11	24.92	25.53	27.57
	52275	15.09	17.89	18.61	19.40	20.03	20.94	21.42	21.70	22.66
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	42466	12.55	15.09	15.77	16.51	17.08	17.94	18.71	19.22	20.63
	29585	11.38	13.56	14.08	14.66	15.12	15.83	16.58	17.14	18.72
	10000	8.15	9.76	10.16	10.47	10.68	11.01	11.44	11.82	13.37
	2000	4.09	4.84	5.10	5.36	5.55	5.84	6.18	6.45	7.45

Table A46: Raymondville Drain Alternative 1 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Diversion Drain	69000.00	70.20	71.37	71.96	72.81	73.66	74.43	75.59	76.46	76.33
	60000.00	67.83	69.32	70.29	71.53	72.76	73.65	74.99	75.94	75.36
	58000.00	67.62	69.11	70.13	71.41	72.66	73.57	74.93	75.89	75.24
	54000.00	67.23	68.71	69.75	71.05	72.35	73.28	74.66	75.65	74.93
	45300.00	64.55	66.26	67.52	69.00	70.42	71.51	72.98	74.05	73.46
	36000.00	62.02	64.12	65.67	67.33	68.95	69.85	71.38	72.53	72.10
	31000.00	60.95	63.28	64.92	66.58	68.21	69.10	70.61	71.78	71.40
	13000.00	55.50	57.57	59.05	60.21	61.02	61.71	63.73	65.33	65.09
	11000.00	54.82	56.82	58.23	59.40	60.21	60.90	62.91	64.32	64.14
	7000.00	53.16	54.97	56.30	57.41	58.20	58.86	60.78	62.19	62.10
	0.00	48.52	51.14	52.51	53.86	54.80	55.55	57.49	59.03	59.07
Lateral 5	267939.7	48.47	51.14	52.51	53.85	54.78	55.52	57.45	58.99	59.03
Lateral 4	5524.022	55.60	56.68	57.35	58.06	58.58	59.08	59.86	60.42	59.25
Lateral 3	254087.2	42.39	45.21	46.68	48.11	49.14	49.79	51.70	53.40	53.87
Trib 1	1713.911	48.98	49.37	49.67	49.83	49.87	49.90	49.99	50.05	49.90
Trib 2	3939.517	49.15	49.58	49.94	50.20	50.35	50.50	50.77	50.96	50.49
FM 88	12056.29	48.97	49.35	49.65	49.81	49.83	49.86	49.93	49.97	49.86
West Hargill	222187.5	34.80	37.58	39.19	40.63	41.62	42.29	43.90	45.21	45.70
	209621.7	33.22	35.82	37.47	38.87	39.76	40.43	41.75	42.69	43.21

Table A46: Raymondville Drain Alternative 1 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Drain	205314.5	32.97	35.50	37.09	38.41	39.23	39.92	41.09	41.92	42.40
La Sal Vieja	19393.5	26.64	26.64	26.64	26.64	27.23	31.53	36.91	40.16	42.70
North Hargill	202200.2	32.47	35.00	36.51	37.65	38.27	38.76	39.48	40.04	40.40
	201934.8	32.42	34.95	36.46	37.59	38.21	38.70	39.45	40.05	40.45

Table A47: Willacy Alternative 1 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Raymondville Drain (Willacy)	191551.8	30.96	33.45	34.80	35.59	35.91	36.05	36.46	36.75	37.07
	187949	30.47	32.95	34.21	35.01	35.40	35.58	35.97	36.21	36.57
	181079.2	29.30	31.76	33.10	33.97	34.39	34.60	35.07	35.32	35.71
	165775.1	26.86	29.26	30.52	31.75	32.58	33.05	33.86	34.17	34.61
	145744.9	23.71	26.04	27.46	28.57	29.32	29.61	31.02	31.54	31.91
	139999.4	22.86	25.14	26.47	27.41	28.07	28.34	29.19	29.49	29.83
	137297.8	22.45	24.73	26.01	26.83	27.38	27.69	28.51	28.82	29.16
	107273.5	17.91	20.04	20.85	21.33	21.64	21.81	22.39	22.62	22.97
	104167.2	17.41	19.51	20.53	21.07	21.40	21.56	22.14	22.37	22.69
	72996.3	12.47	13.95	14.83	15.39	15.71	15.90	16.34	16.57	16.93
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
	65979.24	11.19	12.17	12.72	13.16	13.48	13.69	14.29	14.61	15.14
	55505.96	9.88	11.38	12.06	12.53	12.85	13.10	13.68	13.99	14.51
	47254.57	8.51	10.18	11.03	11.60	11.96	12.21	12.80	13.09	13.62
	43401.41	8.13	9.68	10.48	11.01	11.36	11.60	12.18	12.46	13.02
	126.337	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.76

Table A48: Delta Lake Alternative 1 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Delta/South Drain	72438	65.88	66.04	66.14	66.28	66.33	66.42	66.52	66.57	66.35
	63910	59.37	59.53	59.64	59.80	59.89	60.67	62.66	62.76	60.23
	48794	49.34	52.22	53.30	53.57	53.71	53.90	54.11	54.39	54.17
	29898	42.55	45.27	46.63	48.70	49.45	49.76	50.17	48.63	50.08
	26297.8	40.66	44.14	45.84	48.43	49.24	49.54	49.91	46.36	49.86
	21382.71	40.15	43.75	45.52	48.31	49.14	49.42	49.78	41.82	49.76
	3499.257	35.81	38.98	40.75	42.22	43.19	43.96	45.05	45.58	46.23
	196.869	32.47	35.00	36.51	37.65	38.27	38.76	39.48	40.04	40.40

6.20.2 ALTERNATIVE 2

HEC-RAS models for the NMD system, RD, Willacy, and Delta Lake reaches were prepared using the With-project Alternative 2 flow rates previously reported Section 5.10.1.4. Table A49 through Table A52 detail the water surface elevations at each project inflow location within the project area. Complete HEC-RAS Calculations and Water Surface Profiles can be provided upon request.

Table A49: North Main Drain Alternative 2 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
McAllen	333707	105.07	107.06	108.52	110.14	110.82	111.37	112.12	112.67	111.54
Lateral	317000	101.04	103.05	104.33	105.58	106.36	107.21	107.61	107.97	107.31
South	315796	100.20	102.13	103.32	104.47	105.18	105.99	106.54	107.10	106.03

Table A49: North Main Drain Alternative 2 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Drain	298000	94.84	95.93	96.19	96.81	97.25	97.77	98.33	98.72	98.20
	292000	92.80	94.63	95.41	96.30	96.83	97.43	98.02	98.39	97.87
	286897	88.35	89.54	90.00	90.11	90.10	89.93	90.64	91.68	90.26
	275122	80.11	80.85	81.26	81.75	82.16	82.79	83.77	84.54	83.91
	262000	72.96	74.56	75.68	76.64	77.05	75.35	77.20	77.51	77.53
	250000	71.68	74.28	75.60	76.59	77.01	74.88	77.11	77.42	77.45
	248746	71.54	74.21	75.57	76.56	76.98	74.63	77.05	77.41	77.43
North Drain	109000	93.80	95.87	96.87	97.44	97.94	98.57	99.52	100.39	99.25
	95350	93.22	95.23	95.93	96.37	96.55	96.74	96.99	97.40	97.17
	92296	92.18	94.20	94.37	95.82	96.40	96.55	96.74	97.18	96.96
	85781	90.32	92.52	90.94	94.08	94.06	94.33	95.60	96.79	96.62
	71558	86.30	87.78	88.51	89.41	89.98	90.66	91.54	92.92	95.56
	66413	83.04	84.34	84.82	85.42	85.99	86.42	87.54	88.20	89.72
	64591	82.95	84.32	84.80	85.40	85.97	86.41	87.53	88.19	89.71
	62591	82.71	84.29	84.77	85.38	85.95	86.40	87.52	88.18	89.69
	59655	82.40	84.21	84.71	85.33	85.92	86.38	87.51	88.16	89.67
	55237	81.73	83.55	84.08	84.74	85.35	86.30	87.46	88.12	89.61
	46091	80.02	81.81	82.32	82.97	83.59	84.52	85.99	86.93	88.38
	31191	75.48	76.85	77.28	77.07	77.41	77.77	78.26	78.88	80.16
	26091	72.00	72.42	72.72	73.42	73.98	74.50	74.81	75.59	76.79
	19091	70.54	71.65	72.01	72.76	73.35	73.94	73.99	74.57	75.76
Main Flood Water Drain	233824	64.96	67.76	69.32	70.08	70.75	71.26	71.87	71.96	72.04
	192000	46.98	49.98	51.45	52.83	53.71	54.46	55.62	56.27	58.06
	188000	46.06	49.12	50.55	51.89	52.74	53.36	54.43	54.95	57.38
	168000	43.41	46.29	47.23	48.26	49.03	49.70	50.08	50.78	53.07
	153231	39.93	41.87	42.14	42.49	42.70	43.04	43.30	43.50	43.96
	141154	37.40	39.50	39.91	40.35	40.71	41.14	41.73	42.10	43.39
	134000	36.00	38.63	39.17	39.66	40.13	40.69	41.44	41.84	43.23
	124185	34.07	36.85	37.32	37.48	37.87	38.58	39.37	39.94	42.29
	84000	24.12	25.43	25.65	25.85	26.12	26.71	27.34	28.00	30.55
	68373	20.71	22.15	22.52	23.21	23.79	24.11	24.92	25.53	27.57
	52275	15.09	17.89	18.61	19.40	20.03	20.94	21.42	21.70	22.66
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	42466	12.55	15.09	15.77	16.51	17.08	17.94	18.71	19.22	20.63
	29585	11.38	13.56	14.08	14.66	15.12	15.83	16.58	17.14	18.72
	10000	8.15	9.76	10.16	10.47	10.68	11.01	11.44	11.82	13.37
	2000	4.09	4.84	5.10	5.36	5.55	5.84	6.18	6.45	7.45

Table A50: Raymondville Drain Alternative 2 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Diversion Drain	69000.00	70.20	71.37	71.96	72.81	73.67	74.44	75.60	76.45	76.32
	60000.00	67.83	69.32	70.29	71.53	72.76	73.65	74.99	75.93	75.35
	58000.00	67.62	69.11	70.13	71.41	72.66	73.57	74.93	75.88	75.24
	54000.00	67.23	68.71	69.75	71.05	72.36	73.28	74.67	75.64	74.92
	45300.00	64.55	66.26	67.52	69.00	70.44	71.52	72.98	74.03	73.45
	36000.00	62.02	64.11	65.66	67.32	68.97	69.87	71.40	72.51	72.08
	31000.00	60.94	63.28	64.90	66.58	68.25	69.12	70.62	71.74	71.38
	13000.00	55.10	57.32	58.73	60.13	61.34	62.07	63.86	65.08	64.93
	11000.00	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	7000.00	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	0.00	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table A50: Raymondville Drain Alternative 2 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Lateral 5	267939.7	55.84	58.86	59.36	59.64	59.98	60.38	60.37	60.66	60.19
Lateral 4	5524.022	55.60	56.69	57.40	58.19	58.76	59.30	60.15	60.32	59.84
Lateral 3	254087.2	50.29	51.87	52.92	54.04	54.65	55.08	56.00	56.60	55.64
Trib 1	1713.911	48.97	49.38	49.65	49.83	49.87	49.90	49.99	50.05	49.90
Trib 2	3939.517	49.15	49.58	49.92	50.20	50.35	50.50	50.77	50.96	50.49
FM 88	12056.29	48.97	49.36	49.63	49.81	49.83	49.86	49.93	49.97	49.86
West Hargill Drain	222187.5	41.78	43.70	44.57	45.60	46.28	46.92	47.48	48.00	47.09
	209621.7	37.51	39.32	40.26	41.28	41.60	42.13	42.85	43.37	43.42
	205314.5	32.91	35.53	37.14	38.56	39.42	40.21	41.26	42.14	42.44
La Sal Vieja	19393.5	26.64	26.64	26.64	26.64	27.23	31.53	36.91	40.16	42.70
North Hargill	202200.2	32.41	35.03	36.55	37.75	38.41	38.94	39.63	40.15	40.42
	201934.8	32.37	34.99	36.51	37.70	38.36	38.90	39.61	40.18	40.47

Table A51: Willacy Alternative 2 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Raymondville Drain (Willacy)	191551.8	30.93	33.52	34.84	35.62	35.95	36.22	36.61	36.98	37.29
	187949	30.44	33.00	34.23	35.04	35.43	35.73	36.15	36.53	36.86
	181079.2	29.28	31.82	33.15	34.00	34.45	34.82	35.26	35.61	35.97
	165775.1	26.86	29.30	30.54	31.78	32.82	33.51	34.08	34.43	34.94
	145744.9	23.72	26.07	27.47	28.58	29.57	30.15	31.27	31.67	32.14
	139999.4	22.87	25.17	26.48	27.41	28.24	28.73	29.37	29.61	30.06
	137297.8	22.47	24.76	26.02	26.83	27.53	28.07	28.68	28.95	29.43
	107273.5	17.92	20.06	20.85	21.33	21.71	21.98	22.46	22.70	23.15
	104167.2	17.42	19.54	20.53	21.07	21.47	21.72	22.21	22.44	22.87
	72996.3	12.48	13.97	14.83	15.39	15.78	16.05	16.40	16.62	17.06
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
	65979.24	11.20	12.18	12.72	13.16	13.53	13.87	14.37	14.68	15.32
	55505.96	9.89	11.39	12.06	12.52	12.91	13.27	13.75	14.05	14.68
	47254.57	8.52	10.19	11.03	11.59	12.01	12.37	12.87	13.16	13.79
	43401.41	8.14	9.68	10.48	11.00	11.40	11.74	12.24	12.52	13.19
	126.337	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.82

Table A52: Delta Lake Alternative 2 Water Surface Elevations Base

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Delta/South Drain	72438	55.11	57.33	58.77	60.19	61.40	62.13	63.95	65.19	65.04
	63910	50.89	53.15	54.64	56.21	57.45	58.15	59.83	61.15	61.10
	48794	44.18	46.82	48.60	50.51	51.64	52.51	54.18	55.17	55.25
	29898	36.00	39.34	41.46	43.40	44.61	45.61	47.36	48.78	49.11
	26297.8	35.26	38.75	40.84	42.77	43.96	44.96	46.63	48.06	48.40
	21382.71	34.91	38.29	40.33	42.22	43.40	44.40	46.02	47.42	47.75
	3499.257	33.10	35.84	37.55	39.01	39.86	40.60	41.64	42.62	43.00
	196.869	32.41	35.03	36.55	37.75	38.41	38.94	39.63	40.15	40.42

6.21 FUTURE YEAR WITH PROJECT MODEL RESULTS

The future year With-project conditions HEC-RAS models provided the water surface elevations for the nine storm events that were used in the Flood Damage Assessment. HEC-RAS results are provided for the RRP rainfall models and for the Hurricane Beulah event. Separate flow data files and plan files were created in each HEC-RAS hydraulic model to calculate the future year water surface elevations for each alternative plan. Complete output for each alternative plan can be provided upon request.

6.21.1 ALTERNATIVE 1

HEC-RAS models for the NMD system, RD, Willacy, and Delta Lake reaches were prepared using the With-project, future year Alternative 1 flow rates previously reported Section 5.10.2.1. Table A53 through Table A56 detail the water surface elevations at each project inflow location within the project area. Complete HEC-RAS Calculations and Water Surface Profiles can be provided upon request.

Table A53: North Main Drain Alternative 1 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
McAllen Lateral	333707	106.40	108.72	110.37	111.33	111.92	112.58	113.49	114.15	111.54
	317000	102.40	104.49	105.79	107.13	107.36	107.87	108.39	108.65	107.31
South Drain	315796	101.51	103.46	104.65	105.92	106.24	106.99	108.05	108.32	106.03
	298000	95.76	96.26	96.90	97.65	98.12	98.66	99.24	99.64	98.20
	292000	94.23	95.54	96.45	97.30	97.81	98.34	98.85	99.18	97.87
	286897	89.29	90.03	90.11	89.98	89.71	91.29	93.20	91.48	90.26
	275122	80.69	81.40	81.90	82.70	83.41	84.47	85.48	86.49	83.91
	262000	73.97	76.15	77.04	75.38	77.24	77.53	77.95	77.94	77.53
	250000	73.41	76.08	76.99	74.90	77.17	77.44	77.86	77.83	77.45
	248746	73.32	76.05	76.96	74.64	77.15	77.43	77.85	77.81	77.43
North Drain	109000	95.34	96.81	97.57	98.55	99.13	100.03	101.89	102.26	99.25
	95350	94.73	96.06	96.42	96.78	96.75	96.92	98.02	97.86	97.17
	92296	93.47	95.25	95.86	94.57	96.53	96.63	97.88	97.62	96.96
	85781	91.33	94.10	94.22	94.40	95.05	96.15	97.62	95.63	96.62
	71558	87.50	89.07	89.88	90.84	91.48	92.24	93.20	94.72	95.56
	66413	84.26	85.84	86.14	86.51	87.06	87.64	88.23	88.82	89.72
	64591	84.24	85.83	86.13	86.50	87.05	87.62	88.22	88.80	89.71
	62591	84.22	85.81	86.12	86.49	87.04	87.62	88.21	88.79	89.69
	59655	84.13	85.78	86.09	86.47	87.03	87.60	88.19	88.77	89.67
	55237	83.47	85.20	85.67	86.40	86.97	87.56	88.15	88.72	89.61
	46091	81.72	83.43	83.88	84.63	85.31	86.17	86.88	87.53	88.38
	31191	77.01	77.35	77.59	77.89	78.13	78.46	79.22	80.05	80.16
	26091	72.44	73.61	74.06	74.67	75.13	75.15	76.13	76.38	76.79
	19091	71.63	72.84	73.36	74.07	74.49	74.27	75.06	75.80	75.76
Main Flood Water Drain	233824	66.55	69.40	70.32	70.81	71.69	72.02	72.19	72.49	72.04
	192000	48.79	52.07	53.63	54.59	55.38	56.29	57.75	58.05	58.06
	188000	48.00	51.22	52.69	53.46	54.17	54.96	56.26	57.32	57.38
	168000	45.19	47.98	48.95	49.70	49.90	50.79	52.02	52.96	53.07
	153231	41.27	42.48	42.74	43.08	43.24	43.50	43.69	43.86	43.96
	141154	38.89	40.35	40.84	41.27	41.70	42.19	42.72	43.13	43.39
	134000	37.61	39.67	40.28	40.90	41.40	41.94	42.51	42.94	43.23
	124185	35.91	37.46	38.02	38.85	39.42	40.23	41.10	41.79	42.29
	84000	24.98	25.89	26.23	26.81	27.32	28.23	29.21	30.04	30.55
	68373	21.46	23.34	23.58	24.36	24.89	25.72	26.48	27.19	27.57
	52275	16.63	19.55	20.44	21.13	21.41	21.79	22.19	22.48	22.66
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	42466	13.94	16.67	17.44	18.20	18.69	19.38	20.02	20.39	20.63
	29585	12.62	14.82	15.42	16.08	16.57	17.32	18.04	18.52	18.72
	10000	9.05	10.51	10.83	11.18	11.47	11.94	12.56	13.08	13.37
	2000	4.48	5.36	5.66	5.97	6.20	6.54	6.94	7.27	7.45

Table A54: Raymondville Drain Alternative 1 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Diversion Drain	69000.00	70.45	71.91	72.76	74.16	75.10	75.93	77.42	78.52	76.33
	60000.00	68.34	70.18	71.46	73.26	74.28	75.27	76.96	78.14	75.36
	58000.00	68.17	70.01	71.33	73.16	74.20	75.20	76.90	78.08	75.24
	54000.00	67.82	69.62	70.96	72.86	73.92	74.94	76.65	77.82	74.93
	45300.00	65.19	67.31	68.88	71.00	72.24	73.29	75.05	76.32	73.46
	36000.00	62.74	65.29	67.15	69.39	70.67	71.72	73.53	74.85	72.10
	31000.00	61.69	64.47	66.38	68.65	69.92	70.94	72.72	74.06	71.40
	13000.00	56.19	58.81	60.23	61.61	62.65	63.57	65.93	67.42	65.09
	11000.00	55.49	57.99	59.41	60.80	61.88	62.76	64.91	66.61	64.14
	7000.00	53.74	56.07	57.42	58.76	59.69	60.64	62.71	64.18	62.10
	0.00	49.25	52.22	53.87	55.43	56.49	57.33	59.56	61.07	59.07
Lateral 5	267939.7	49.25	52.22	53.85	55.41	56.46	57.30	59.52	61.06	59.03
Lateral 4	5524.022	56.30	57.45	58.20	59.02	59.63	60.23	60.66	61.08	59.25
Lateral 3	254087.2	43.23	46.36	48.10	49.74	50.93	51.64	53.87	55.55	53.87
Trib 1	1713.911	49.23	49.70	49.84	49.90	49.97	50.43	50.16	50.26	49.90
Trib 2	3939.517	49.41	49.98	50.24	50.49	50.70	51.11	51.27	51.52	50.49
FM 88	12056.29	49.22	49.68	49.82	49.86	49.91	50.37	50.06	50.14	49.86
West Hargill Drain	222187.5	35.83	38.93	40.61	42.18	43.29	43.93	45.62	46.96	45.70
	209621.7	34.33	37.30	38.85	40.27	41.29	41.88	43.05	44.01	43.21
	205314.5	34.07	36.95	38.39	39.70	40.67	41.31	42.25	43.04	42.40
La Sal Vieja	19393.5	26.64	26.64	26.64	26.64	27.42	32.30	38.58	42.42	42.70
North Hargill	202200.2	33.58	36.38	37.62	38.64	39.25	39.66	40.17	40.74	40.40
	201934.8	33.53	36.33	37.56	38.58	39.21	39.63	40.19	40.82	40.45

Table A55: Willacy Alternative 1 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Raymondville Drain (Willacy)	191551.8	32.08	34.68	35.56	36.11	36.41	36.55	36.98	37.30	37.07
	187949	31.58	34.11	34.98	35.62	35.94	36.12	36.53	36.77	36.57
	181079.2	30.41	32.96	33.93	34.65	35.01	35.19	35.65	35.88	35.71
	165775.1	27.96	30.45	31.72	33.12	33.67	33.91	34.57	34.84	34.61
	145744.9	24.83	27.38	28.70	29.60	30.25	30.73	31.91	32.19	31.91
	139999.4	23.98	26.41	27.50	28.22	28.80	29.08	29.82	30.07	29.83
	137297.8	23.57	25.95	26.90	27.58	28.12	28.41	29.15	29.44	29.16
	107273.5	19.00	20.83	21.44	21.80	22.06	22.33	22.82	23.10	22.97
	104167.2	18.47	20.50	21.19	21.56	21.79	22.09	22.55	22.81	22.69
	72996.3	13.27	14.81	15.51	15.90	16.16	16.31	16.77	17.02	16.93
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	65979.24	11.75	12.70	13.28	13.70	14.02	14.25	14.89	15.25	15.14
	55505.96	10.83	12.05	12.64	13.10	13.43	13.66	14.25	14.60	14.51
	47254.57	9.47	11.03	11.73	12.22	12.54	12.79	13.36	13.69	13.62
	43401.41	9.02	10.49	11.15	11.61	11.90	12.19	12.73	13.08	13.02
	126.337	1.50	1.50	1.50	1.50	1.50	1.50	1.56	1.74	1.76

Table A56: Delta Lake Alternative 1 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Delta/South Drain	72438	66.00	66.18	66.30	66.40	66.48	66.58	66.68	66.77	66.35
	63910	59.47	59.65	59.83	60.51	62.58	62.69	62.80	62.82	60.23
	48794	50.82	53.31	53.60	53.86	54.03	54.29	54.82	55.21	54.17
	29898	44.00	46.65	48.78	49.67	50.02	50.33	49.16	50.90	50.08
	26297.8	42.47	45.82	48.52	49.44	49.77	50.05	47.10	50.50	49.86
	21382.71	41.99	45.49	48.39	49.33	49.65	49.90	43.30	50.12	49.76
	3499.257	37.23	40.61	42.21	43.71	44.53	44.88	46.06	47.15	46.23
	196.869	33.58	36.38	37.62	38.64	39.25	39.66	40.17	40.74	40.40

6.21.2 ALTERNATIVE 2

HEC-RAS models for the NMD system, RD, Willacy, and Delta Lake reaches were prepared using the With-project, future year Alternative 2 flow rates previously reported Section 5.10.2.4. Table A57 through Table A60 detail the water surface elevations at each project inflow location within the project area. Complete HEC-RAS Calculations and Water Surface Profiles can be provided upon request.

Table A57: North Main Drain Alternative 2 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
McAllen Lateral	333707	106.40	108.72	110.37	111.33	111.92	112.58	113.49	114.15	111.54
	317000	102.40	104.49	105.79	107.13	107.36	107.87	108.39	108.65	107.31
South Drain	315796	101.51	103.46	104.65	105.92	106.24	106.99	108.05	108.32	106.03
	298000	95.76	96.26	96.90	97.65	98.12	98.66	99.24	99.64	98.20
	292000	94.23	95.54	96.45	97.30	97.81	98.34	98.85	99.18	97.87
	286897	89.29	90.03	90.11	89.98	89.71	91.29	93.20	91.48	90.26
	275122	80.69	81.40	81.90	82.70	83.41	84.47	85.48	86.49	83.91
	262000	73.97	76.15	77.04	75.38	77.24	77.53	77.95	77.94	77.53
	250000	73.41	76.08	76.99	74.90	77.17	77.44	77.86	77.83	77.45
	248746	73.32	76.05	76.96	74.64	77.15	77.43	77.85	77.81	77.43
North Drain	109000	95.34	96.81	97.57	98.55	99.13	100.03	101.89	102.26	99.25
	95350	94.73	96.06	96.42	96.78	96.75	96.92	98.02	97.86	97.17
	92296	93.47	95.25	95.86	94.57	96.53	96.63	97.88	97.62	96.96
	85781	91.33	94.10	94.22	94.40	95.05	96.15	97.62	95.63	96.62
	71558	87.50	89.07	89.88	90.84	91.48	92.24	93.20	94.72	95.56
	66413	84.26	85.84	86.14	86.51	87.06	87.64	88.23	88.82	89.72
	64591	84.24	85.83	86.13	86.50	87.05	87.62	88.22	88.80	89.71
	62591	84.22	85.81	86.12	86.49	87.04	87.62	88.21	88.79	89.69
	59655	84.13	85.78	86.09	86.47	87.03	87.60	88.19	88.77	89.67
	55237	83.47	85.20	85.67	86.40	86.97	87.56	88.15	88.72	89.61
	46091	81.72	83.43	83.88	84.63	85.31	86.17	86.88	87.53	88.38
	31191	77.01	77.35	77.59	77.89	78.13	78.46	79.22	80.05	80.16
	26091	72.44	73.61	74.06	74.67	75.13	75.15	76.13	76.38	76.79
	19091	71.63	72.84	73.36	74.07	74.49	74.27	75.06	75.80	75.76
Main Flood Water Drain	233824	66.55	69.40	70.32	70.81	71.69	72.02	72.19	72.49	72.04
	192000	48.79	52.07	53.63	54.59	55.38	56.29	57.75	58.05	58.06
	188000	48.00	51.22	52.69	53.46	54.17	54.96	56.26	57.32	57.38
	168000	45.19	47.98	48.95	49.70	49.90	50.79	52.02	52.96	53.07
	153231	41.27	42.48	42.74	43.08	43.24	43.50	43.69	43.86	43.96
	141154	38.89	40.35	40.84	41.27	41.70	42.19	42.72	43.13	43.39
	134000	37.61	39.67	40.28	40.90	41.40	41.94	42.51	42.94	43.23
	124185	35.91	37.46	38.02	38.85	39.42	40.23	41.10	41.79	42.29
	84000	24.98	25.89	26.23	26.81	27.32	28.23	29.21	30.04	30.55

Table A57: North Main Drain Alternative 2 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
	68373	21.46	23.34	23.58	24.36	24.89	25.72	26.48	27.19	27.57
	52275	16.63	19.55	20.44	21.13	21.41	21.79	22.19	22.48	22.66
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	42466	13.94	16.67	17.44	18.20	18.69	19.38	20.02	20.39	20.63
	29585	12.62	14.82	15.42	16.08	16.57	17.32	18.04	18.52	18.72
	10000	9.05	10.51	10.83	11.18	11.47	11.94	12.56	13.08	13.37
	2000	4.48	5.36	5.66	5.97	6.20	6.54	6.94	7.27	7.45

Table A58: Raymondville Drain Alternative 2 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Diversion Drain	69000.00	70.45	71.91	72.76	74.16	75.10	75.93	77.41	78.48	76.32
	60000.00	68.34	70.18	71.45	73.26	74.28	75.28	76.94	78.09	75.35
	58000.00	68.17	70.01	71.33	73.16	74.20	75.21	76.89	78.04	75.24
	54000.00	67.82	69.62	70.96	72.86	73.92	74.94	76.63	77.77	74.92
	45300.00	65.19	67.30	68.87	71.01	72.25	73.30	75.03	76.24	73.45
	36000.00	62.74	65.27	67.14	69.41	70.69	71.73	73.49	74.75	72.08
	31000.00	61.68	64.45	66.37	68.67	69.94	70.96	72.67	73.93	71.38
	13000.00	55.85	58.45	60.15	61.96	62.92	63.72	65.54	66.70	64.93
	11000.00	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	7000.00	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	0.00	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Lateral 5	267939.7	57.07	59.38	59.71	60.31	60.21	60.62	61.80	62.88	60.19
Lateral 4	5524.022	56.30	57.50	58.34	59.21	59.88	60.16	60.83	61.27	59.84
Lateral 3	254087.2	51.29	53.06	54.25	55.00	55.69	56.36	57.35	58.00	55.64
Trib 1	1713.911	49.23	49.69	49.84	49.90	49.97	50.03	50.16	50.38	49.90
Trib 2	3939.517	49.40	49.98	50.25	50.48	50.70	50.91	51.27	51.56	50.49
FM 88	12056.29	49.21	49.67	49.82	49.86	49.91	49.96	50.06	50.26	49.86
West Hargill Drain	222187.5	42.94	44.73	45.80	46.82	47.28	47.86	48.56	49.04	47.09
	209621.7	38.67	40.53	41.45	42.06	42.75	42.98	43.81	44.56	43.42
	205314.5	34.02	36.96	38.43	39.88	40.85	41.49	42.41	43.25	42.44
La Sal Vieja	19393.5	26.64	26.64	26.64	26.64	27.42	32.30	38.58	42.42	42.70
North Hargill	202200.2	33.52	36.39	37.65	38.75	39.37	39.78	40.38	40.80	40.42
	201934.8	33.48	36.35	37.60	38.70	39.34	39.76	40.43	40.89	40.47

Table A59: Willacy Alternative 2 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Raymondville Drain (Willacy)	191551.8	32.04	34.72	35.59	36.14	36.45	36.72	37.16	37.58	37.29
	187949	31.55	34.14	35.00	35.64	35.97	36.27	36.72	37.13	36.86
	181079.2	30.39	33.02	33.96	34.68	35.06	35.38	35.84	36.20	35.97
	165775.1	27.97	30.49	31.75	33.15	33.77	34.23	34.76	35.11	34.94
	145744.9	24.84	27.42	28.71	29.61	30.50	31.46	32.06	32.31	32.14
	139999.4	23.99	26.44	27.51	28.23	28.94	29.42	29.95	30.16	30.06
	137297.8	23.59	25.98	26.91	27.58	28.26	28.73	29.29	29.55	29.43
	107273.5	19.01	20.85	21.44	21.80	22.11	22.50	22.91	23.20	23.15
	104167.2	18.48	20.52	21.19	21.55	21.84	22.25	22.63	22.91	22.87
	72996.3	13.27	14.83	15.52	15.90	16.20	16.46	16.84	17.07	17.06
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	65979.24	11.76	12.71	13.28	13.69	14.07	14.45	14.98	15.32	15.32
	55505.96	10.84	12.06	12.64	13.10	13.48	13.83	14.34	14.67	14.68
	47254.57	9.48	11.05	11.73	12.21	12.58	12.95	13.43	13.76	13.79

Table A59: Willacy Alternative 2 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
	43401.41	9.03	10.51	11.15	11.60	11.94	12.33	12.81	13.15	13.19
	126.337	1.50	1.50	1.50	1.50	1.50	1.50	1.59	1.76	1.83

Table A60: Delta Lake Alternative 2 Water Surface Elevations Future

RIVER NAME	XS STATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Delta/South Drain	72438	55.85	58.48	60.20	62.03	63.00	63.81	65.65	66.82	65.04
	63910	51.64	54.33	56.21	58.03	58.92	59.74	61.64	63.05	61.10
	48794	45.12	48.22	50.48	52.31	53.39	54.18	55.43	56.25	55.25
	29898	37.21	41.09	43.32	45.28	46.50	47.54	49.11	50.30	49.11
	26297.8	36.57	40.49	42.68	44.62	45.81	46.85	48.38	49.54	48.40
	21382.71	36.20	40.01	42.13	44.05	45.23	46.27	47.72	48.86	47.75
	3499.257	34.27	37.37	38.88	40.32	41.19	41.90	42.95	44.00	43.00
	196.869	33.52	36.39	37.65	38.75	39.37	39.78	40.38	40.80	40.42

6.22 HYDRAULIC MODEL CALIBRATION (WITH PROJECT)

The With-project hydraulic models were based on the without-project hydraulic models for each of the four (4) main reaches. These without-project hydraulic models were calibrated using observed water surface elevations from TxDOT as-built drawings, IBWC historical data, available Hidalgo County data, and observations by the USACE during the Hurricane Beulah event. Since all the With-project hydraulic models were based on the calibrated base models, the With-project HEC-RAS models are also considered calibrated.

6.23 INDUCED FLOODING

There will be no induced flooding from the construction or operation of the proposed project. Diversion from the North Main Drain will not occur until the Raymondville Drain channel has the additional capacity to carry the diverted flow. The additional channel capacity throughout the project will ensure that flows will stay in the channel through a range of flows, and will not create adverse impacts. Once the construction is complete, diversion from the North Main Drain to the Raymondville Drain would only occur when North Main Drain flows approach damaging levels, and the project would be operated to avoid induced flooding downstream in Hidalgo and Willacy Counties. This would be accomplished by controlled operation of the gated control structures at the headwaters of the project, and at the Hidalgo-Willacy line.

A1 – SECTION 7 GEOTECHNICAL ENGINEERING

7.1 GENERAL

Significant geotechnical investigations have been undertaken, ensuring a high-level of confidence in the feasibility investigation. The engineer and the sponsor are familiar with soil conditions in the region and the proposed project corridor. Soil properties within the area vary between Sandy Clay Loams, Loamy Sands, Sandy Loams. The native soils are nearly level, varying in slope from 0 to 3 percent. The risk related to geotechnical conditions remains low.

The recommended alternative primarily consists of expanding an existing channel that is approximately 43 miles long (approximately 75% of the proposed project length) with existing bridge crossings. These drains were successfully constructed remain stable without additional intervention. Similar drains are being constructed by the sponsor throughout the region, without issue. There is no evidence of erosion or sedimentation in any of the well-maintained existing channels in the region.

The remaining approximately 14 miles (25%) of the project will be new drains, being constructed in areas of known and consistent soil conditions. Ongoing projects, like the Faysville Drain along the proposed alignment, continue to indicate suitable soil conditions for the project. Bridge and channel stability is a low risk, as channel stability issues would have come up earlier with the implementation and maintenance of the existing channel, and existing bridges have been in-place for many years.

As part of the planning efforts for the feasibility of the preferred alternative, the sponsor has conducted geotechnical investigations along the proposed channel alignment. Borings at the proposed bridge crossings are deeper than general channel borings, and the initial bridge borings were collected in pairs.

In Hidalgo County, investigations completed to date included 34 borings to a depth of 75 feet below the top of natural ground. Two borings were done at each bridge location in accordance with Texas Department of Transportation standards for proposed bridge structures. The geotechnical report for Hidalgo County is in Attachment E to this Appendix.

In Willacy County, 114 channel borings were collected along the channel alignment to a depth of 35 feet below the top of natural ground. An additional 75 borings to a depth of approximately 75 feet below the top of natural ground were made at location of proposed structures (including two at each bridge location). The geotechnical report for Willacy County is in Attachment F to this Appendix.

Both reports included deep foundation analysis for bridges, and provided general considerations during construction. The analysis indicates that the soils along the proposed project are suitable for the project. Groundwater is not expected to be encountered during channel excavation.

The borings evaluation included the Texas Cone Penetration Test (CPT) to determine soil strength parameters, recording of initial water strike depth, moisture content, plasticity index, sulfate content, along with a Particle Size Analysis for the Determination of Fines Content and Gradation Curves D50 & D90. Drilling logs were also plotted.

The majority of this project is an expansion of existing stable channels with relatively low flow velocities due to the flat terrain. All of the road crossings to be constructed in Willacy County will replace existing structures due to expansion of the channel, and there is no evidence of inadequate design or adverse foundation conditions at any existing road crossing location. The geotechnical information has been considered in the project Risk Register and the Cost and Schedule Risk Analysis (CSRA). Additional geotechnical investigations would be accomplished during the design phase if determined to be necessary.

A1 – SECTION 8 CIVIL AND CONSTRUCTION CONSIDERATIONS

8.1 GENERAL

Plans and Specifications for the upstream Hidalgo County portion have been completed. These documents contain full consideration of Civil and Construction disciplines, including channel alignment conditions, curvature design of the channel, excavation and haul for material movement, channel slopes, temporary and permanent environmental considerations (also included in section x of the main report) including Storm Water Pollution Prevention considerations, and traffic control designs. These items have been considered in the cost estimate and design maturity documents (Appendix A-2). Similar design considerations will be taken into account for Willacy County during the PED phase.

A1 – SECTION 9 TECHNICAL JUSTIFICATIONS

9.1 ATLAS 14 MEMORANDUM

The National Oceanic and Atmospheric Administration (NOAA), in conjunction with the U.S. Army Corps of Engineers released Atlas 14 version 11 in 2018 to update precipitation-frequency estimates for Texas. Atlas 14 vastly improves precipitation-frequency data, as it is based on historical rainfall data, stream gauge data, and more recent large tropical storms and hurricane events compared to the TP 40/TP 49 data that was computed in the 1950s and early 1960s. NOAA's Atlas 14 data is therefore considered a better rainfall dataset to be used in flood studies across central Texas. However, as shown in Figure A10 and summarized in Table A62, Atlas 14 rainfall will have little to no effect around the Raymondville Drain Feasibility study area, therefore re-analysis of TP40/TP49-based computations is not necessary for this feasibility study.

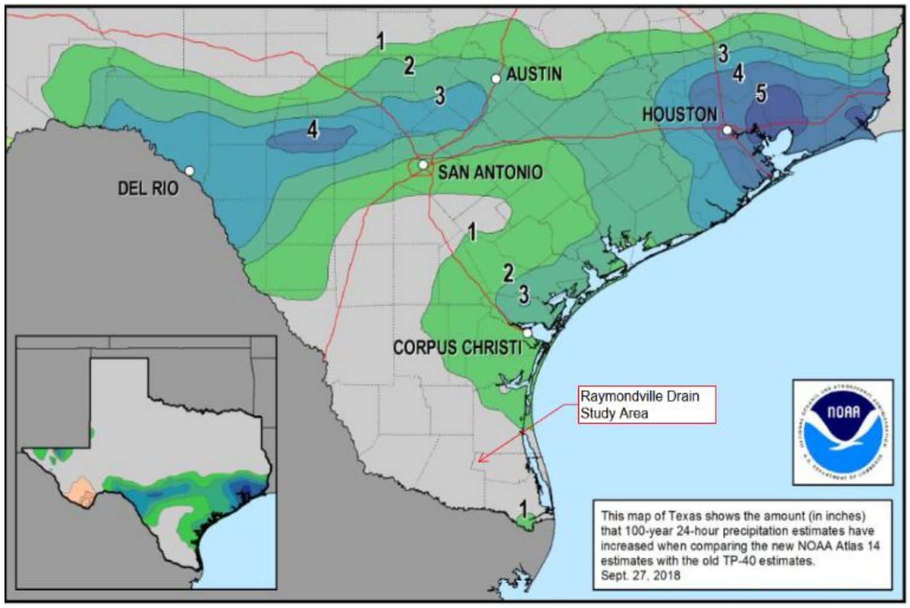


Figure A10 Atlas 14 Rainfall Increases

Table A62 - Rainfall Data Comparison at City of Edinburg

100-Year Flood Frequency (inches)			
Duration	Atlas 14	TP40/49	Difference
5 min	1.2	0.9	0.3
15 min	2.4	2	0.4
1 hour	4.4	4.6	-0.2
2 hours	5.8	5.8	0
3 hours	6.6	6.3	0.3
6 hours	8.1	7.8	0.3
12 hours	9.4	9.5	-0.1
1 day	10.7	11.2	-0.5
2 days	12.2	12.8	-0.6
4 days	13.4	15.5	-2.1
7 days	14.3	16.5	-2.2
10 days	15	18	-3

To further evaluate impacts, Atlas 14 100-year rainfall runoff was generated in HEC-HMS using existing base conditions. Only the meteorologic model was revised in the HMS model to incorporate Atlas 14 rainfall data at the city of Edinburg in Hidalgo County, a major damage center. Comparisons of the resulting flows for the RD and the NMD system Watersheds from the model showed minimal impact or increase in overall flows. In review of the 46 junction points, the average flow increase is slightly over 3% with flow increases less than 5% in the majority of the junctions. The hydrologic output from the HEC-HMS analysis is presented in Table A63 below:

Table A63 - Flow Comparisons

System	HEC HMS Junction	ATLAS 14 (CFS)	TP 40/TP49 (CFS)	Difference (CFS)	% Change
Upper RVD Watershed	R660W660	1296.3	1272.6	23.7	1.9%
	R750W750	658.8	650.5	8.3	1.3%
	JR770	1867.6	1834.2	33.4	1.8%
	JR780	1984.6	1945.0	39.6	2.0%
	JR720	6221.7	6081.2	140.5	2.3%
	JR630	141.1	134.1	7.1	5.3%
	JR1570	6121.8	5989.7	132.1	2.2%
North Main Drain Watershed	Q=R2630W2610	965.4	914.0	51.4	5.6%
	Q=Userpoint5	2360.9	2241.0	119.9	5.3%
	Q=JR1920	266.2	253.0	13.2	5.2%
	Q=JR1870	4984.2	4692.0	292.2	6.2%
	Q=JR1670	4710.2	4462.0	248.2	5.6%
	Q=JR1490	5162.7	4925.0	237.7	4.8%
	Q=JR1420	3840.1	3714.0	126.1	3.4%
	Q=JR2690	6362.2	6016.0	346.2	5.8%
	Q=UserPoint11	5846.6	5551.0	295.6	5.3%
	Q=JR2700	5477.9	5214.0	263.9	5.1%
	Q=UserPoint12	5256.5	5012.0	244.5	4.9%
	Q=UserPoint16	4907.4	4694.0	213.4	4.5%
	Q=UserPoint17	4819.7	4606.0	213.7	4.6%
	Q=JR1780	4009.2	3855.0	154.2	4.0%
	Q=JR2780	4009.1	3856.0	153.1	4.0%
	Q=UserPoint18	4007.1	3854.0	153.1	4.0%
	Q=Userpoint5	2360.9	2241.0	119.9	5.3%
	Q=JR2210	5093.3	4955.0	138.3	2.8%
	Q=JR2190	5380.3	5246.0	134.3	2.6%
	Q=JR2180	5082.5	4949.0	133.5	2.7%
	Q=JR2150	5354.4	5215.0	139.4	2.7%
	Q=JR2090	4465.7	4368.0	97.7	2.2%
	Q=JR2060	4986.5	4871.0	115.5	2.4%

Table A63 - Flow Comparisons

System	HEC HMS Junction	ATLAS 14 (CFS)	TP 40/TP49 (CFS)	Difference (CFS)	% Change
	Q=JR1840	6651.9	6490.0	161.9	2.5%
	Q=JR1630	7633.3	7421.0	212.3	2.9%
	Q=JR1800	5569.2	5444.0	125.2	2.3%
	Q=JR1790	6195.4	6064.0	131.4	2.2%
	Q=JR1450	5751.9	5626.0	125.9	2.2%
	Q=JR1280	5418.9	5305.0	113.9	2.1%
	Q=JR1290	4814.9	4721.0	93.9	2.0%
	Q=JR1190	4882.3	4788.0	94.3	2.0%
	Q=OutletNMD	4660.1	4581.0	79.1	1.7%
	Q=JR3	4846.1	4762.0	84.1	1.8%
	Q=JR4	4958.5	4870.0	88.5	1.8%
	Q=JR5	4961.1	4873.0	88.1	1.8%
	Q=JR7	4950.8	4863.0	87.8	1.8%
	Q=JR8	4981.0	4892.1	88.9	1.8%
	Q=JR10	4876.8	4790.0	86.8	1.8%
	Q=JR12 OUTLET	4826.4	4741.0	85.4	1.8%

Conclusion:

Since the new NOAA Atlas 14 rainfall has minimal impact on resulting runoff, RRP will continue the use of TP40/TP49 rainfall data for the Raymondville Drain feasibility study. The difference is within the margin of error for this Feasibility level analysis and would not impact the project recommendations. Additionally, the resilience assessment (Attachment G) confirms no expected change in future runoff from changing conditions over time. Additionally, the hydraulics and hydrology will be updated utilizing Atlas 14 for the plans and specifications effort for the RD Project. This approach is consistent with SMART Planning.

9.2 FUTURE FLOWS MEMORANDUM

In response to an ASA(CW) comment on the 2020 report submission, RRP conducted a comprehensive review of assumptions and performed computations to justify the future flows used in this Feasibility. This section was originally prepared as a stand-alone justification document. Many of the tables in this section and some of the explanatory information is also included in section 5.8.2.

A thorough understanding of the area, surrounding communities, and previous coordination with HCDD1 and USACE is critical to properly analyze the anticipated future flows for the subject area. RRP used the available local models, along with most recent RRP models, TP40/TP49 rainfall data (for a 10-day 100-year storm), and local knowledge and input related to flood levels and future probability of occurrence along the North Main Drain system and Raymondville Drain study areas. Population growth exceeding government estimates, and limited regulation of development in the study area support our previous future flows conclusions.

RRP projected the future year peak flows based on a calculation of the anticipated impacts of population growth in the area. The methodology for determining the future year without project hydrologic conditions is based on the population projections available from the U.S. Census Bureau. The four (4) largest urbanized areas within the project limits are the cities of McAllen, Edinburg, Pharr, and Mission. RRP analyzed population data from the US Census and projected populations in and around these major urban areas to get a better understanding for future conditions analysis. See Table A64 for US Census recent history, 2020 actual population data versus RRP original projections, and projections to 2060.

The project base year, for the purposes of this feasibility study will be 2034 and project improvements will be computed to provide community flood benefits through 2084.

TABLE A64: RRP 2020 Overall Project Population Estimates

CITY NAME	P2000 CENSUS	P2010 CENSUS	PROJECTED P2020	P2020 CENSUS	P2030	P2040	P2050	P2060
McAllen	106,414	129,877	152,045	142,210	179,586	209,386	241,933	275,322
Edinburg	48,465	77,100	83,869	100,243	105,237	128,358	153,611	179,517
Pharr	46,660	70,400	74,656	79,715	91,553	109,836	129,805	150,291
Mission	45,408	77,059	79,551	85,778	100,157	122,454	146,807	171,790

TABLE A65: RRP 2020 Overall Project Population Detail

CITY NAME	PROJECTED P2020	P2020 CENSUS	POPULATION DIFFERENCE	% DIFFERENCE
McAllen	152,045	142,210	-9,835	-6.92%
Edinburg	83,869	100,243	16,374	16.33%
Pharr	74,656	79,715	5,059	6.35%
Mission	79,551	85,778	6,227	7.26%
Total	**390,121**	**407,946**	**17,825**	**4.37%**

As shown in Table A65, overall population projected by RRP for the year 2020 is accurate (within 5%), and actual Census numbers are slightly higher than overall projected. Census numbers also exceeded Government estimates for population growth.

The community has developed significantly with the expansion of the University of Texas Rio Grande Valley, Doctor's Hospital at Renaissance, and the Bert Ogden Arena in Edinburg. Hidalgo County has drastically changed with the introduction of the North American Free Trade Agreement (NAFTA) in 1994, transforming the primarily agricultural communities into a diverse economy which thrives off international trade due to its multiple International Land Ports of Entry along the boundary with Mexico and its position relative to I 69-C and US 281.

With this population growth, additional development will occur in the form of additional residential subdivisions, commercial developments, and Consulting Engineers improvements. These areas around Edinburg are currently heavily developed, and are expected to continue to develop. This anticipated development would increase the overall impervious cover within any given watershed. For this analysis, areas closer to the existing municipalities would increase to 70% impervious cover, and outlying areas would increase to 60% impervious in the future years. This is because we anticipate that as outlying areas develop, additional local ordinances will be implemented to somewhat limit flow increases. However, HCDD1 currently requires design to only account for analysis and mitigation of the proposed 10-year flow after development, while the Raymondville drain modeling and feasibility study is based on much larger storm events. Note that there are two sub-basins with an existing impervious cover percentage of 85%. These were raised to 95% to account for minimal future development. A map indicating the projected future extent of development is Figure A11, and a more detailed version of the map is in Attachment A (Projected Future Area of Development Map, Year 2061). For this analysis, the existing land use pattern remained at the same proportions throughout the watershed. This was documented with the USACE concerning the methodology to be used for future without project conditions. A summary table illustrating the change in curve numbers for the revised sub-basins is available upon request.

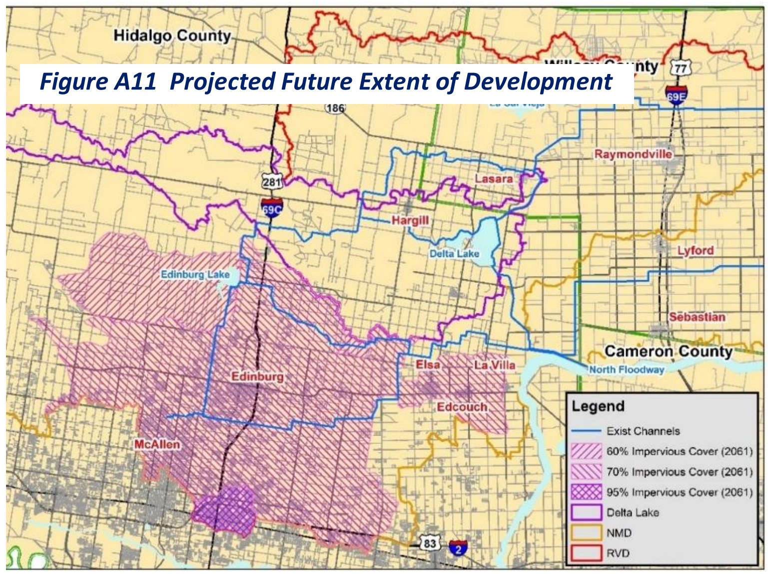


Figure A11 Projected Future Extent of Development

During the preparation of the without project hydrologic models, land use maps and aerial photographs were utilized to determine the land use characteristics of the individual sub-basins. Additionally, zoning maps were obtained from the City of McAllen Planning Department and the Edinburg Planning and Zoning Department to confirm the extent of the existing development. The developed land uses corresponded to an impervious cover percentage based on the density of residential structures and/or commercial classification. These impervious cover percentages for the residential and commercial land uses were obtained from TR-55, Table 2-2a (Runoff Curve Numbers for Urban Areas). According to the tables above, the population of McAllen and Edinburg are expected to more than double over the 50-year analysis period. As the existing amount of impervious cover accommodates the existing population, it is reasonable to estimate that a similar, although lower amount of additional impervious cover would be needed to accommodate this increase in population. To determine the effect that this additional development would have on the hydrologic conditions of this area, a separate base conditions hydrologic model was prepared which included additional impervious cover for the future conditions.

To accommodate the previously referenced population increases, additional impervious cover amounts were estimated and added to the hydrologic model for this portion of the study. This will estimate the future pre-project hydrologic conditions for the watersheds. To account for routing effects, present within the watershed, several inflow points were selected along the NMD system to compare the present and future conditions pre-project peak flow rates. Note that for this comparison, the storm area reduction calculations were not performed, since only the relative difference in peak flow rates is needed. The relative increases in the peak flow rates for the 100-year storm event for the selected project inflow locations are tabulated in Table A66.

Table A66: Selected Project Inflow Locations

PROJECT INFLOW LOCATION	BASE WOP Q100 (CFS)	FUTURE WOP Q100 (CFS)	INCREASE FACTOR
JR1630	8857.3	12203.7	1.38
User Point 18	5129.9	9026.2	1.76
JR2780	5151.8	9090.2	1.76
JR1780	5155.9	8998.0	1.75
User Point 17	5874.8	9113.3	1.55
User Point 16	6223.9	9327.2	1.50
User Point 12	6845.2	9772.1	1.43
JR2700	7204.2	9929.4	1.38
User Point 11	7668.6	10137.7	1.32
JR 2690	8179.5	10527.9	1.29
JR1420	4377.4	7283.0	1.66
JR1490	6034	7615.1	1.26
JR 1670	5591.8	7146.9	1.28
JR 1870	5830.2	7165.7	1.23
JR 1920	255.1	420.7	1.65
		Weighted Average	1.37

Table A66 illustrates the weighted average based on individual sub-basin areas to determine the difference in peak flow rates. The future peak flow rates for the NMD Reach 2 increased on average by a factor of approximately 1.37 (37% higher) than the peak flow rates for the current year. This analysis was also completed for the South Main Drain Reach 3. A detailed summary table comparing the difference in peak flow rates and the weighted averages is available upon request. The weighted average of both the NMD

Reach 2 and the South Main Drain Reach 3 increase factor was determined to be 1.35 (35% higher). Since it has been previously documented that the existing land use pattern is estimated to continue in the same proportion throughout the watershed, it is appropriate that this factor be applied uniformly to the current year peak flow rates throughout the entire watershed. The population growth obtained from Census Bureau projections is likely low when considering that US 281 and US 77 are being readied to convert to interstate roadway facilities and new ports of entries/additional bridge construction is slated to occur between Mexico and the United States throughout the Rio Grande Valley area. The future peak flow rates (1.35 x Year 2011 peak flow rates) were utilized in the hydraulic models to calculate future water surface profiles for each damage reach. These future conditions water surface profiles were subsequently entered into HEC-FDA for use in determining the expected annual damages for the without project conditions and for each proposed alternative. Table A67 through Table A70 summarize the future without project peak flow rates for the four watersheds.

Table A67: North Main Drain Base Peak Flow Rates (cfs) (FUTURE WITHOUT PROJECT)

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=R2630W2610	348	536	699	903	1058	1234	1499	1701	956
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR1920	96	148	193	250	292	342	415	471	281
Q=JR1870	1364	2292	3138	4115	4920	6334	8465	10013	6505
Q=JR1670	1377	2126	2829	3856	4789	6024	8072	9660	6115
Q=JR1490	1271	2260	3260	4559	5478	6648	8342	9756	7335
Q=JR1420	1811	2630	3155	3854	4360	5014	5938	7627	8680
Q=JR2690	1828	2687	3206	4770	6176	8122	10923	13239	12409
Q=UserPoint11	1663	2578	3040	4552	5796	7494	10235	12561	12307
Q=JR2700	1521	2459	2877	4391	5529	7038	9590	11960	12313
Q=UserPoint12	1497	2389	2807	4298	5369	6767	9105	11504	12283
Q=UserPoint16	1490	2277	2750	4152	5108	6336	8298	10475	12203
Q=UserPoint17	1454	2262	2741	4081	5003	6219	7859	9555	11985
Q=JR1780	1481	2007	2513	3582	4299	5204	6856	8737	11874
Q=JR2780	1494	2009	2512	3581	4298	5205	6845	8716	11947
Q=UserPoint18	1480	2008	2509	3578	4296	5202	6815	8643	11890
Q=Userpoint5	846	1308	1709	2212	2589	3025	3673	4163	2309
Q=JR2210	2005	2977	3811	4856	5647	6689	8034	9133	5483
Q=JR2190	1918	2997	3903	5039	5909	7082	8561	9761	6021
Q=JR2180	1973	2888	3716	4781	5592	6681	8215	9464	5748
Q=JR2150	2183	3203	4022	5107	5934	7040	8523	9603	6465
Q=JR2090	2274	3090	3666	4452	5060	5897	6984	7811	5965
Q=JR2060	2351	3416	4091	4978	5654	6576	7788	8709	6589
Q=JR1840	2692	4300	5267	6507	7441	8761	10441	11801	8929
Q=JR1630	2451	4190	5581	7205	8395	10018	11889	12938	16045
Q=JR1800	2127	3620	4628	5638	6368	7350	8720	9838	13961
Q=JR1790	2409	4057	5124	6205	7043	8187	9790	11016	14463
Q=JR1450	2356	3924	4701	5630	6444	7595	9051	10223	13961
Q=JR1280	2325	3966	4558	5363	6045	7161	8522	9630	13454
Q=JR1290	2198	3652	4227	4939	5508	6373	7581	8871	12176
Q=JR1190	2234	3735	4298	5010	5584	6463	7687	8961	12266
Q=OutletNMD	2200	3513	4090	4797	5376	6185	7289	8623	11586
Q=JR3	2233	3735	4293	4976	5546	6429	7558	8857	11877

Table A67: North Main Drain Base Peak Flow Rates (cfs) (FUTURE WITHOUT PROJECT)

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
Q=JR4	2260	3861	4413	5091	5661	6575	7729	9011	12056
Q=JR5	2268	3855	4412	5091	5663	6578	7725	9006	12043
Q=JR7	2268	3855	4413	5090	5655	6565	7712	8995	12016
Q=JR8	2297	3925	4466	5128	5688	6604	7755	9024	11774
Q=JR10	2274	3841	4416	5091	5634	6467	7604	8886	11315
Q=JR12 OUTLET	2198	3727	4338	5030	5577	6400	7536	8805	11135

Table A68: Raymondville Drain Base Peak Flow Rates (cfs) (FUTURE WITHOUT PROJECT)

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R660W660	464	737	962	1243	1476	1718	2123	2426	1462
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
R750W750	244	383	498	640	757	878	1078	1225	641
JR770	664	1058	1383	1788	2126	2476	3060	3494	1967
JR780	873	1318	1630	1981	2273	2626	3179	3627	2368
JR720	1795	3287	4439	5852	7002	8210	10289	11772	10991
JR630	0	0	0	0	3	181	765	1296	1811
JR1570	1726	3202	4339	5743	6883	8086	10158	11717	11264

Table A69: Delta Lake Base Peak Flow Rates (cfs) (FUTURE WITHOUT PROJECT)

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1060	382	604	784	1008	1195	1386	1708	1949	1038
JR 1120	610	996	1307	1710	2038	2372	2931	3347	1763
JR 1080	856	1685	2600	3692	4640	5722	7441	8731	7340
JR 1620	1105	1943	2625	3669	4622	5706	7455	8778	7771
USERPOINT 8	1103	1937	2620	3665	4619	5700	7449	8770	7770
JR 790	1403	2526	3443	4632	5660	6745	8538	9903	8925
JR 720	1795	3287	4439	5852	7002	8210	10289	11772	10991
JR 1570	1726	3202	4339	5743	6883	8086	10158	11717	11264

Table A70: Willacy Base Peak Flow Rates (cfs) (FUTURE WITHOUT PROJECT)

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 1570	1726	3202	4339	5743	6883	8086	10158	11717	11264
USERPOINT 3	1682	3030	4066	5367	6495	7655	9395	11309	10777
JR 1560	2079	3766	5062	6659	8110	9592	11712	13241	12485
USERPOINT 2	1960	3475	4702	6216	7611	9038	11405	13021	12448
JR 390	2018	3390	4548	5886	7029	8445	10934	12834	12769
JR 400	1839	2943	3897	5226	6301	7518	10453	12672	12903
JR 420	2064	3240	4245	5603	6656	7894	10965	13309	13900
OUTLET 35/ SOURCE1	2152	3378	4421	5811	6889	8129	11266	13680	14393

Table A70: Willacy Base Peak Flow Rates (cfs) (FUTURE WITHOUT PROJECT)

HEC HMS JUNCTION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR	BEULAH FLOOD
JR 490	1828	3021	4104	5610	6855	8118	10887	13223	14353
JR 500	1849	3090	4214	5795	7082	8357	11230	13649	15111
JR 530	1597	2649	3720	5224	6654	7956	10486	12864	14468
JR 540	1614	2659	3741	5241	6695	8013	10573	12963	14657
JR 460	1550	2572	3648	5118	6567	7938	10461	12816	14574
JR 370	1534	2550	3626	5090	6551	7944	10471	12837	14701
JR 590	1633	2676	3820	5387	6959	8450	11174	13717	16128
OUTLET	1616	2612	3646	5073	6560	8042	10533	12822	15411

Areas in eastern Willacy County (Figure A12), nearest to the Laguna Madre were also modeled with the proposed 35% increase in flows, even though the populations and development here are not expected to grow at this rate. Hydraulic calculations were performed using this blanket 35% increase to analyze the resulting flows within the channel, and to simplify modeling where reducing the project growth would still be considered negligible, for uses in this feasibility study. Table A71 indicates flows are estimated to be well within acceptable percentage of difference to justify leaving the proposed flows, due to the population growth, as estimated with the 35% increase. The model is not sensitive to assumptions of growth in the eastern portion of the basin since the majority of development and runoff will occur in western portion of the basin. Because of the storage available in the channel and HEC-HMS modeling with flow increases upstream, resulting flows in the east Willacy County portion of the channel remain within 2% of the adjusted future flows model regardless of growth assumptions in the rural areas.

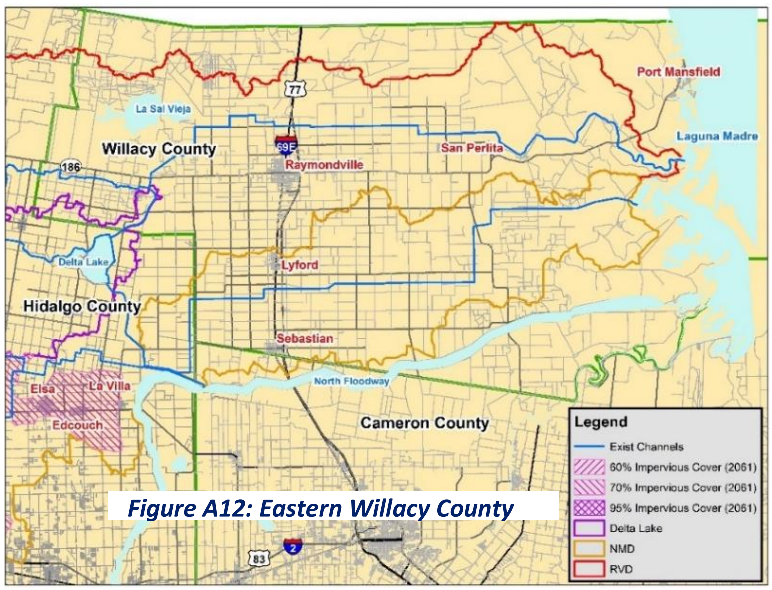


Figure A12: Eastern Willacy County

Comparison of Report Future Flows and Assuming No Growth in Willacy County									
HEC HMS JUNCTION	WOP 50 Yr Willacy Co. 1.35% Increase	WOP 50 Yr Willacy Co. No Impervious Increase	% Difference	WOP 100 Yr Willacy Co. 1.35% Increase	WOP 50 Yr Willacy Co. No Impervious Increase	% Difference	WOP 50 Yr Willacy Co. 1.35% Increase	WOP 50 Yr Willacy Co. No Impervious Increase	% Difference
OUTLET 35/ SOURCE1	6889	**6889**	**0.00%**	8129	8129	**0.00%**	11266	**11266**	**0.00%**
JR 490	6855	**6781**	**-1.09%**	8118	**8092**	**-0.32%**	10887	**10917**	**0.27%**
JR 500	7082	**6947**	**-1.90%**	8357	**8291**	**-0.79%**	11230	**11175**	**-0.49%**
JR 530	6654	**6543**	**-1.67%**	7956	**7915**	**-0.52%**	10486	**10641**	**1.48%**
JR 540	6695	**6577**	**-1.76%**	8013	**7955**	**-0.72%**	10573	**10700**	**1.20%**
JR 460	6567	**6519**	**-0.74%**	7938	**7900**	**-0.47%**	10461	**10592**	**1.25%**
JR 370	6551	**6515**	**-0.55%**	7944	**7913**	**-0.40%**	10471	**10599**	**1.22%**
JR 590	6959	**6836**	**-1.77%**	8450	**8328**	**-1.45%**	11174	**11166**	**-0.07%**
OUTLET	6560	**6889**	**5.01%**	8042	**7869**	**-2.15%**	10533	**11266**	**6.96%**

Table A71: Comparison of Willacy Future Flows

Because of the limited area available to develop, and with most population increases anticipated to be around currently populated areas, land use changes and increased runoff in these areas will be anticipated to peak around the 35% increase as discussed above.

Local ordinances, future land use regulations, redevelopment policies, and other non-structural methods of flood control will not adequately address larger storm events such as those used for the purposes of this feasibility study. Local policies and ordinances are mainly designed to control and mitigate flood effects due to much smaller storm events. For example, the HCDD1 currently requires design to only account for analysis and mitigation of the proposed 10-year flow, after development. The Raymondville drain modeling and feasibility study is based on the 10-day, 100-year storm event. This event is much larger and, based on future growth in the area, will be comparable to the major hurricane Beulah, of 1967. When performing calculations and flood analysis using rainfall of this magnitude, RRP is focused on sizing realistic drainage capacity associated with the anticipated new construction, and uncertain future non-structural policies or building restrictions within potential floodways will not be analyzed, as it will not be a factor considering overall flood levels and potential damages. This approach also provides additional resilience against future changing conditions and extreme weather.

Findings:

US Census data shows actual population numbers have been increasing along with RRP projected values for the 10 years from 2010 to 2020. Population growth in the area has shown to be most increased in areas outside of the most urbanized towns. More people have been moving into areas previously dominated by farm and rangeland. Population growth in these historically pervious areas will increase flows in the local floodways and future RD drain system.

As shown in the exhibits and tables above, these numbers further establish the validity of assuming the 35% increase in flow due to the increase of impervious land area, especially in the more rural areas. The 35% increase has been validated above, as accurate for the purposes of this feasibility study, and the flow computations are not sensitive to the assumptions of growth in the rural eastern portion of Willacy County.

Conclusion:

Based on findings in the analysis performed, RRP will continue to use future flows estimated using the US Census Data and our resulting development assumptions for this feasibility study. It is not reasonable to assume that future ordinances will significantly control increases in flows, since that is beyond our control, and historically this has not been the case in this rural part of the state. For example, HCDD1 currently requires design to only account for analysis and mitigation of the proposed 10-year flow after development, while the Raymondville drain modeling and feasibility study is based on much larger storm events. Raymondville Drain is not currently considered a FEMA floodplain, so options to control future development are limited. The flow is not sensitive to assumptions of growth in less developed downstream areas of the basin, because the majority of flow originates in the more developed upstream areas. Furthermore, the projected data calculated for recent years is accurate based on actual population growth in the areas, and overall projected numbers are slightly less than actual population numbers, based on the Tables A64 and A65, representing population growth in the major urban areas from 2010 to 2020. Models will be updated, as appropriate, for the design phase.

9.3 RESILIENCE ASSESSMENT

An assessment was performed to highlight existing and future challenges facing the study area due to changing conditions over time. This assessment was done to demonstrate that the project will be resilient to future changes and uncertainty.

This evaluation identifies potential vulnerabilities for the Raymondville Drain Feasibility Study. This analysis indicated that predicted changes are not expected to significantly increase flows, nor impact study recommendations. The additional channel capacity provided by the project provides additional resilience for the communities within the study area, reducing vulnerability. The assessment document is included as Attachment G to this Appendix.

A1 – SECTION 10 REFERENCES

10.1 HYDROLOGY REFERENCES

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1 ATTACHMENTS

1.1 ATTACHMENT A – HYDROLOGY

Attachment A contains drainage area maps, project inflow locations maps, and a projected area of future development map. It also includes the Design Task Protocol - 1, Hydrology.

1.2 ATTACHMENT B – HYDRAULICS

Attachment B contains a pre-project structure crossing map, project structure crossing maps, and project cross section maps. It also includes Design Task Protocol -2 , Hydraulics.

1.3 ATTACHMENT C – H&H QUALITY ASSURANCE REPORT

Attachment C contains the Final Technical Memorandum, Summary of Quality Assurance Review, Hydrology & Hydraulics Base Models.

1.4 ATTACHMENT D – ADDITIONAL H&H MODEL VALIDATION

Attachment D contains additional calibration comparisons for the Hydrology and Hydraulics base models, including Nash-Sutcliffe Equivalency (NSE) computations, developed as a response to the IEPR and ATR reviews.

1.5 ATTACHMENT E – GEOTECHNICAL REPORT – HIDALGO COUNTY

Attachment E contains the Hidalgo Geotechnical Report.

1.6 ATTACHMENT F – GEOTECHNICAL REPORT – WILLACY COUNTY

Attachment F contains the Willacy Geotechnical Report.

1.7 ATTACHMENT G – RESILIENCE ASSESSMENT

Attachment F contains an assessment of the project for resilience against changing conditions.