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SECTION 1. GENERAL PROJECT INFORMATION

1.1 Description and Purpose of Project
The Doña Ana County Flood Commission (DACFC) authorized Smith Engineering Company (Smith) to prepare a drainage master plan for the community of Chamberino. The purpose is to analyze existing drainage conditions in the watershed of Chamberino, determine deficiencies and propose improvements. Figure A below shows the Chamberino vicinity map.

Figure A: Chamberino Vicinity Map

1.2 Field Observation
Smith conducted field observations in August, 2015. The annotated photographs are presented in Appendix 1.

SECTION 2. EXISTING HYDROLOGIC AND HYDRAULIC ANALYSIS

2.1 Existing Flood Control Structures
Chamberino has several existing dams within the watershed. Ownership of the dams could not be determined based on GIS data provided by EBID and DACFC. All dams within the study area are breached and do not appear to be engineered.
The dams and their condition are listed on the drainage basin map, Figure 1. At the top of the watersheds south western quadrant there are five abandoned treatment ponds that still act as full retention basins.

Due to the enormous storage volume of these retention ponds, the basins contributing to the ponds were modeled as closed basins. For the purposes of this drainage master plan, the dams in the major arroyos were not modeled under existing conditions.

2.2 Drainage Basin Description and Basin Delineation

Drainage Basin Description

The Chamberino watershed is predominantly undeveloped range land with mild to steep topography. The community of Chamberino is located at the bottom of the watershed. The Chamberino community could be classified as a medium density, residential development. The roads within the community are a mix of paved and unpaved with no curb and gutter. The rest of the watershed east of Chamberino is comprised of straight row crops on agricultural fields that are interlaced with irrigation channels.

FEMA Floodplains

DACFC supplied Smith with various GIS shape files for the area that included the Preliminary Flood Zones for 2014. Based on this data, parts of the Chamberino watershed floodplains are classified as FEMA Zone A (approximate). Figure 4 shows the limits of these floodplains. Figure 4 is included in Appendix 2.

Drainage Basin Delineation

Drainage basins were delineated from the digital elevation models (DEM) provided by DACFC. The DEM’s resolution was at a 2-ft interval. The basins and basin characteristics were determined using the Geo-HMS extension with Arc Hydro using Arc Map 10.1. Once the parameters are computed digitally, the data is exported out to create a HEC-HMS model. The HEC-HMS model created has all hydrologic elements automatically generated and connected in correct hydrologic order along with most of the pertinent hydrologic data. The user then has the flexibility to add, reconnect or delete elements as necessary. Figure 1 shows the subbasins delineated for the Chamberino Watershed. This process eliminates the need to manually delineate subbasins, flow paths and compute parameters such as areas, lengths and slopes.

The flow chart on the following page illustrates the process involved to generate a HEC-HMS model using the Geo-HMS extension within Arc-Hydro.
Arc Hydro Process

**Terrain Pre Processing to Generate Basins, T_c Flowpaths and routing reaches**
- DEM Reconditioning
- Flow Direction
- Flow Accumulation
- Stream Definition
- Stream Segmentation
- Catchment Grid Delination
- Catchment Polygon Processing
- Drainage Line Processing
- Watershed Aggregation

**HEC HMS Project Set Up**
- Setting Target Location to save digital data
- Defining Outlet of project
- Basin Generation and Editing
- Stream and Subbasin characteristics
- Hydrologic Parameter Estimation
- Basin and River Autoname
- TR-55 Flow Segments

**Exporting Project from Arc-HYDRO to HMS**
- Determine Map Units
- Data Check
- Generate HMS Legend
- Add coordinates
- Exporting background Shapefiles
2.3 Drainage and Analysis Criteria

The “Storm Drainage Criteria “per DACFC requires that the hydrologic analysis be based on the 5-year, 10-year, and 100-year return period storms of 24-hour duration.

Hydrologic Computer Program

The US Army Corps of Engineers “Hydrologic Modeling System” program or commonly called “HEC-HMS” (Version 4.0) program was selected for simulation of basin storm rainfall – runoff for existing and proposed options in conjunction with its GIS based extension called Geo-HMS.

2.4 Rainfall Data

Rainfall Distribution

The study basin is located within the USDA Natural Resources Conservation Service (NRCS) (previously the Soil Conservation Service (SCS)) Type II rainfall distribution area as defined by the SCS. Please refer to Appendix 4 for Type II boundaries.

However, the DACFC dictated that the 25% Frequency Storm Distribution be adopted. That distribution is available in the HEC-HMS program. It places most of the rainfall in a short period at 25% of the storm duration, or at 6 hours for a 24-hour storm.

Areal Reduction Factors

No areal reduction factors were necessary due to the small watershed.

Point Rainfall Data

Point rainfall data for the 5-yr, 10-yr, and 100-yr return period storms for various durations were obtained from NOAA Atlas 14 website. Appendix 4 contains the printouts from the NOAA Atlas 14 point rainfall data results. Table 1 (Appendix 3) contains the point rainfall depth data.

2.5 Soils Data and Runoff Curve Numbers (CNs)

Soils data used to determine Runoff Curve Numbers (CNs) were provided by the DACFC. The data was checked against available data from the Natural Resources Conservation Service (NRCS) internet site Web Soil Surveys as follows:


Appendix 4 contains a soils map generated from the data provided by the DACFC. Also included is the comparison data from the NRCS. The Hydrologic Soil Group distribution was predominantly uniform across the watershed. Therefore, no CN weightings were necessary. Refer to Figure 3 in Appendix 4.
The following assumptions were applied in order to approximate initial abstractions using CNs:

1. Antecedent Runoff Condition II (AMR II) is defined as the soil Average runoff condition (moisture condition) by the NRCS. AMR III is defined as saturated runoff conditions. To be conservative with runoff rates and volumes for basins classified as semi arid rangelands, an average curve number between AMR II and AMR III was used.

2. Hydrologic Soil Group (A, B, C, or D) was determined by the soils data provided by DACFC and compared with NRCS Web Soil Survey. The Chamberino soils were largely Hydrologic Soil Group A in 85% of the watershed, and Hydrologic Soil Group B in the agricultural fields.

3. Land Use Type is either – semi-arid rangeland (most subbasins), urban (within developed Chamberino area) or cultivated agricultural land. The orthophotography as presented on the Drainage Basin Map (map pocket) was used to make the land use type determinations. The CN tables are obtained from “Urban Hydrology for Small Watersheds, US Dept of Agricultural Soil Conservation Service, Technical Release 55 (TR-55), June 1986. *

4. The TR-55 CN tables are listed here:
   - Table 2-2a Runoff Curve Numbers for Urban Areas*
   - Table 2-2b Runoff Curve Numbers for Cultivated Agricultural Land *
   - Table 2-2c Runoff Curve Numbers for Other Agricultural Lands *
   - Table 2-2d Runoff Curve Numbers for Arid and Semiarid Rangelands*
   *Copies included in Appendix 4

5. Cover Type, Hydrologic Condition and Percent Imperviousness
   - Semi-Arid Rangeland - Desert Shrub, etc., poor hydrologic condition. Curve number of 72 was applied for all undeveloped subbasins. (Table 2-2d applies)
   - Urban – Due to the mixture of medium to low density development, an Average curve number of 85 was applied to account for unpaved right of way, paved roads and impervious roofs and ¼ acre lots. Curve Number of 94 was applied to the commercial processing plant in subbasin W1110 at the west end of San Francisco De Assisi Rd. (Table 2-2a applies)
   - Cultivated Agricultural Land – Assumed straight row crops with good hydrologic conditions. Curve number of 78 was applied. (Table 2-2b applies).

6. CN selections were based on the previous data, assumptions and NRCS soils data / and guidelines in the TR-55 Urban Hydrology for Small Watershed Handbook.
2.6 Split hydrographs for Subbasins

Purpose

When subbasins are mostly homogeneous in terms of land use type and runoff curve numbers, an areal weighted CN approach may be acceptable. When non-homogeneous land use types occur and a greater range of CN’s occur between those land used types, the subbasin runoff is more accurately simulated with split hydrographs as described here. This method was applied to Subbasin W1110.

Hydrograph 1 will simulate the pervious or undeveloped subbasin area and will have a subbasin name such as W1110- P ("P" for pervious). Hydrograph 2 will simulate the impervious or developed subbasin area and will have a subbasin name such as W1110 - I ("I" for impervious). The pervious and impervious hydrographs are then computed and added together at a junction before being routed downstream. This is particularly important when the impervious part of the subbasin is close to the outlet of the subbasin as in the case of subbasin W1110 as the impervious area will respond to rainfall much faster than the pervious portion.

Impervious Area Assumptions and Computations for Subbasin W1110

1. Measure the impervious area including the approximate graded limits
2. Because the impervious area is small relative to the overall basin, assume a minimum time of concentration of 12 minutes
3. Assume CN of 94 as prescribed by Table 2-2a for a commercial site that is 85% impervious
4. The pervious part of the subbasin is assigned the computed T_c and assigned a curve number of 72 per Table 2-2d
5. Within HEC-HMS two separate hydrographs are computed based on the above parameters and then added at a junction

2.7 Travel Time (T_t), Time of Concentration (T_c), Unit Hydrograph Lag Time (T_L) Computations and Unit Hydrograph

A water course may have up to three sub-reaches that comprise the longest flow path. These are an upper overland flow reach, then a shallow concentrated flow reach, followed by a channel reach. Geo-HMS uses the NRCS TR-55 to compute T_t and T_c for each water course. The time of concentration (T_c) for the watercourse equals the summation of travel times (T_t) from each sub-reach. Appendix 4 contains the TR-55 description and procedures. The various reaches and their physical characteristics are computed by the program directly from the DEM and an Excel table is generated and stored within the Geo-HMS geo-database. Table 4 was saved separately to document the parameters generated by Geo-HMS in Appendix 3. By default, Geo-HMS allocates the first 100 ft. for sheet flow. This was appropriate for the Chamberino watershed because the upper basin slopes are very steep with slopes on the order of 15-25 %. Engineering judgment had to be exercised when determining the T_c parameters through the lower Chamberino watershed. This is because the lack of curb and gutter on the streets and the lack of any storm drain infrastructure means that runoff still largely follows the topography. The T_c flow paths generated by Arc Hydro through the agricultural fields are extremely subjective. In the absence of better topographic data, the flow paths interpolated by Arc Hydro over the DEM were adopted.
Subbasins consisting of agricultural fields were not allowed to have any channel flow because the physical characteristics of the subbasins would prevent that from occurring. As a result, $T_c$ computations were only limited to sheet flow and shallow concentrated flow.

NRCS Unit Hydrograph Lag Time Method ($T_L$) was applied to the $T_c$ to compute the unit hydrograph Time to Peak ($T_p$). Note that Lag Time = 0.6 $T_c$. Appendix 4 contains the reference pages from the National Engineering Handbook, May 2015. Chapter 15 describes the lag time concept and method.

Manning’s Roughness Coefficients “$n$” assumptions were obtained from TR-55, by experience and by review of “$n$” value tables by Chow, 1959 (copies include in Appendix 4).

Tables 4 (Appendix 3) summarizes the travel time, time of concentration, and lag time data and results. Table 2 also presents the lag time results that were used as input in HEC-HMS.

### 2.8 Channel Routing

The Muskingum-Cunge channel routing method was applied to route hydrographs. Figure 2 in Appendix 3 illustrates the routing reaches. Manning’s “$n$” values were assumed based on field observation, experience, and the Manning’s “$n$” values from Chow, 1959. Bottom width assumptions were determined as the typical channel width from the drainage basin maps. Table 3 (Appendix 3) presents the Muskingum-Cunge channel routing input data summary.

Note that runoff losses to channel bed infiltration and percolation were assumed to be small and were therefore not simulated.

### 2.9 Sediment Bulking

The HEC-HMS models simulate clear water hydrographs unless a “Flow Ratio” is applied to simulate sediment volume within hydrographs. This is also called sediment bulking. Note that a sediment bulking value of about 17% is considered the limit before mud flow would occur. Due to lack of site specific data, a sediment bulking factor of 1.10 or 10% was assumed for all undeveloped subbasin hydrographs while the urban subbasins were allocated a factor of 1.05 or 5%. That assumption is based on review of information presented in the Sediment and Erosion Design Guide, Nov. 2008; Mussetter Engineering, Inc. Appendix 4 contains a copy of relevant pages from that document.

### 2.10 Computation Time Increment for HEC-HMS Models

The computation increment assumed within a HEC-HMS model may make a large difference in model peak discharge results particularly for large drainage basins. Guidance on computation intervals was found in a Digital Engineering Library (McGraw-Hill, a copy included in Appendix 4) and summarized here. The computation time increment is typically based on Time of Concentration ($T_c$) and the following equation:

$$\frac{T_c}{5} \leq \text{computation time increment} \leq \frac{T_c}{3}$$
Due to the small lag time observed for the smaller urban basins, the computation time increment was set to 1 minute.

2.11 Modeling Results

The results for the 5yr, 10yr, and 100yr–24 Hour Storms are presented in Appendix 5. The unit peak discharges for the 100 year – 24 hour event for undeveloped basins ranged from 1.1 - 1.5 cfs per acre while the more urban subbasins were in the range of 3.5 cfs per acre.

These numbers are well within the acceptable unit peak discharges observed in other similar studies.

SECTION 3. OPTIONS HYDROLOGIC AND HYDRAULIC ANALYSES

3.1 Proposed Options Hydrologic and Hydraulic Data

Two options were simulated within HEC-HMS to improve drainage conditions in Chamberino. The idea behind the two options was to divert off-site flows around town and to redirect flows within town where possible. The redirection of flows in town was handled with extreme caution. The only streets that were considered were those that had low/no risk of downstream damage. Since many of the houses are below street grade, concentrating flows in location could prove to be hazardous in large events where the street flow capacity is exceeded and water could drain into yards and houses through the driveway.

Several detention and retention ponds were simulated within HEC-HMS. Pond routings were performed based on conceptual level grading plans and elevation-storage-discharge curves derived from topographic data. Typical side slopes for ponds were assumed to 1V:3H and where applicable, principal spillways were simulated as a 24 inch CMP pipe. Hydraulic capacity calculations for the diversion channel and street flow were performed with the Flowmaster software and culvert calculations were performed with Culvertmaster.

3.2 Overview of Option 1

Sheet Opt1.1 provides an overview for all the flood control elements proposed in Option 1. These consist of the following:

Pond 1: Pond 1 is a rehabilitation of Dam D4, as labeled on Figure 1 (map Pocket). This dam is located east of the Chamberino Mutual Domestic water facility at the outlet of basin W1160. Refer to Sheet OPT1.2 for grading plan and pond routing summary. The existing structure is of unknown ownership based on EBID and Doña Ana Flood Commission GIS records. The current structure is currently breached. However this structure can be rehabilitated as follows: the existing embankment can be to be lowered to elevation 3901. This would provide a total storage volume of 23.7 ac-ft. The spillway will have to be designed to handle the 100 Yr-24 Hr flows.
Based on the routing results, this dam will control the 100 Yr-24Hr storm without activating the emergency spillway.

Pond 1 would be a non-jurisdictional dam due to its embankment height being less than 6 ft from top of dam (Elev-3901) to the lowest downstream toe elevation (Elev-3896). The detailed routing summary is provided below and detailed rating curve data is provided in Appendix 6. The cost of demolishing the existing embankment and building Pond 1 is approximately $1,160,236. A detailed cost estimate table is presented with Sheet OPT1.2.

Table 1

<table>
<thead>
<tr>
<th>Detention Pond Name</th>
<th>Principal Spillway Pipe Diameter</th>
<th>Return Period/year</th>
<th>Peak Inflow</th>
<th>Peak Outflow</th>
<th>Inflow Runoff Volume</th>
<th>Outflow Runoff Volume</th>
<th>Maximum Design Storage Volume (top of embank)</th>
<th>Peak Storage Volume During Design Storm</th>
<th>Peak Water Surface Elevation</th>
<th>Emergency Spillway Elevation</th>
<th>Pond Invert Elevation</th>
<th>Max Pond Depth</th>
<th>Peak Water Depth</th>
<th>Top of Pond Embankment Elevation</th>
<th>Free Board to Emergency Spillway Elevation</th>
<th>Free Board to top of Pond Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pond 1</td>
<td>24 100 / 24 258 30 23.7 23.7 23.1 11.3</td>
<td>24 102/24 72 7 8.6 8.6 8.6 23.1 3.8</td>
<td>24 102/24 72 7 8.6 8.6 8.6 23.1 3.8</td>
<td>24 102/24 72 7 8.6 8.6 8.6 23.1 3.8</td>
<td>24 102/24 72 7 8.6 8.6 8.6 23.1 3.8</td>
<td>24 102/24 72 7 8.6 8.6 8.6 23.1 3.8</td>
<td>24 102/24 72 7 8.6 8.6 8.6 23.1 3.8</td>
<td></td>
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<td></td>
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</tr>
</tbody>
</table>

Diversion Channel: The diversion channel would be located on the west side of the community. Refer to Sheet OPT 1.3. This channel would run south to north at a slope of 1.5%. The channel would be trapezoidal in shape with a bottom width of 15 ft. with 1V:3H side slopes and a total length of approximately 3450 ft. Based on preliminary hydraulic calculations, the channel capacity would be 110 cfs at a normal depth of 1 ft. The diversion channel will divert offsite runoff that currently flows into the Chamberino community. In some locations, the channel does not entirely divert a 100% of a subbasin. In that case, a flow divide element was used in HEC-HMS to split hydrographs. Visual inspection was used to determine what percent of the subbasin was diverted by the channel. The maximum diversion flow rate was based on multiplying the 100 Yr-24Hr flow rate by the diversion percent. In HEC-HMS, the diversion is sent to the channel and a main branch is added to the downstream element. Table D in Appendix 6 summarizes the percentages of basins diverted and the maximum flow values.

Culverts are required to convey the flow under the private road that leads to the Chamberino Mutual Domestic water facility and a construction easement will be required.

The flow at this location (HEC-HMS junction element J.CHNL6) for the 10Yr-24Hr storm is 29 cfs while the 100 Yr-24Hr flow is 69 cfs. Preliminary culvert calculations show the need for two 30 inch diameter CMPS that would pass 67 cfs. Alternatively, smoother materials such as corrugated HDPE will increase capacity to 83 cfs. Appendix 6 has the output for the channel and culvert calculations. The diversion channel will outfall into the proposed Pond 2. The cost of building the diversion channel is approximately $264,494. A detailed cost estimate table is presented with Sheet OPT1.3.
**Pond 2:** Pond 2 acts as the outfall to the diversion channel. This pond is located at the very north end of Chamberino. Refer to **Sheet OPT1.4** for grading. Pond 2 would serve as a sediment control and detention facility. Pond 2 has a design volume of 5 ac-ft. This would be a non-jurisdictional facility. The detailed pond routing summary is provided below.

The routing results show that Pond 2 will fully control the 10Yr-24Hr storm while the emergency spillway will be activated during the 100 Yr-24 Hr storm.

**Table 2**

<table>
<thead>
<tr>
<th>Detention Pond Name</th>
<th>Principal Spillway Pipe Diameter</th>
<th>Return Period/Year</th>
<th>Peak Inflow</th>
<th>Peak Outflow</th>
<th>Inflow Runoff Volume</th>
<th>Outflow Runoff Volume</th>
<th>Maximum Design Storage Volume</th>
<th>Peak Storage Volume During Design Storm</th>
<th>Peak Water Surface Elevation</th>
<th>Emergency Spillway Elevation</th>
<th>Pond Invert Elevation</th>
<th>Max Pond Depth</th>
<th>Peak Water Depth</th>
<th>Top of Pond Embankment Elevation</th>
<th>Free board to Emergency Spillway Elevation</th>
<th>Free board to top of Pond Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pond 2</td>
<td>24</td>
<td>100 / 24</td>
<td>175</td>
<td>117</td>
<td>30.9</td>
<td>39.9</td>
<td>5.0</td>
<td>3.6</td>
<td>3920.90</td>
<td>3920.00</td>
<td>3914.00</td>
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<td>6.3</td>
<td>3922.00</td>
<td>-3.3</td>
<td>1.7</td>
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<td></td>
<td>24</td>
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<td>25</td>
<td>31.3</td>
<td>40.1</td>
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<td>8</td>
<td>2.7</td>
<td>3922.00</td>
<td>3.3</td>
<td>0.3</td>
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</table>

Footnotes for **Detention Pond Routing Summary Table**

a. Option 1
b. Conceptual Design Outfall Pipe
c. Results summarized from the HEC-HMS model
d. See elev-areas-capacity-discharge data table and sources in Appendix 6
f. Negative number indicates the flow depth exceeds referenced elevation - no freeboard available therefore cell highlights
i. Footnotes include the pond routing method and referenced elevation

The cost of building Pond 2 is approximately **$173,657**. A detailed cost estimate table is presented with **Sheet OPT1.4**.

**Pond 3:** Pond 3 would act as the outfall for Subbasin W720 well as a sediment control and detention facility. Pond 3 will be a non-jurisdictional pond. **Sheet OPT1.5** shows the conceptual layout for the pond. Pond 3 will fully control the 10Yr-24 Hr storm. The emergency spillway will be activated during the 100Yr-24 Hr storm. Routing results are provided below.

**Table 3**

<table>
<thead>
<tr>
<th>Detention Pond Name</th>
<th>Principal Spillway Pipe Diameter</th>
<th>Return Period/Year</th>
<th>Peak Inflow</th>
<th>Peak Outflow</th>
<th>Inflow Runoff Volume</th>
<th>Outflow Runoff Volume</th>
<th>Maximum Design Storage Volume</th>
<th>Peak Storage Volume During Design Storm</th>
<th>Peak Water Surface Elevation</th>
<th>Emergency Spillway Elevation</th>
<th>Pond Invert Elevation</th>
<th>Max Pond Depth</th>
<th>Peak Water Depth</th>
<th>Top of Pond Embankment Elevation</th>
<th>Free board to Emergency Spillway Elevation</th>
<th>Free board to top of Pond Embankment</th>
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<tr>
<td>Pond 3</td>
<td>24</td>
<td>160 / 24</td>
<td>81</td>
<td>50</td>
<td>40.7</td>
<td>48.7</td>
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</table>
The cost of building Pond 3 is approximately $308,662. A detailed cost estimate table is presented with Sheet OPT1.5.

**Roadway Improvements on Padre Pio Ave.**: Roadway improvements are recommended with an inverted crown section and curb and gutter along Padre Pio Ave. This will help convey runoff from subbasin W720 to Pond 3 on the east side of N Saucedo Ave. Street conveyance capacity with the proposed section was computed to be 109 cfs. Sheet OPT1.6 shows the typical roadway section and limits of proposed improvement and general assumptions made for hydraulic calculations. The subbasin runoff is 62 cfs for the 100 Yr-24 Hr storm.

The runoff from Subbasin W720 would outfall into Pond 3. Hydraulic calculations from FlowMaster are summarized in Appendix 6. The cost of roadway improvements is approximately $480,217. A detailed cost estimate table is presented with Sheet OPT1.6.

**Pond 4**: Pond 4 is located on the east side of the processing factory. The impervious area from the site generates a considerable amount of runoff as evidenced by the severe gullying at the east property boundary. Pond 4 would act as a detention facility that would control the discharge from the site.

Sheet OPT1.7 shows the conceptual layout for Pond 4. Pond 4 will fully control the 100 Yr-24 Hr storm and will be a non-jurisdictional facility. Pond routing summary is shown below.

**Table 4**

<table>
<thead>
<tr>
<th>Proposed Ponds Detention Pond Routing Summary</th>
<th>Option 1</th>
<th>Chamberino Drainage Master Plan</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detention Pond Name</td>
<td>Principal Spillway Pipe Diameter</td>
<td>Return Period</td>
</tr>
<tr>
<td>inches</td>
<td>cfs</td>
<td>cfs</td>
</tr>
<tr>
<td>-------</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Pond 4</td>
<td>24 / 24</td>
<td>129</td>
</tr>
<tr>
<td>24</td>
<td>10/24</td>
<td>42</td>
</tr>
<tr>
<td>24</td>
<td>6/24</td>
<td>26</td>
</tr>
</tbody>
</table>

Footnotes for Detention Pond Routing Summary Table

- Option 1
- Conceptual Design Outfall Pipe
- c: Results summarized from the HEC-HMS model
- d: See area-specific discharge data table and sources in Appendix 6
- e: Negative number indicates the low depth exceeds referenced elevation - no freeboard available therefore cell highlights
- f: Negative number indicates the free depth exceeds referenced elevation - no freeboard available therefore cell highlights (Spills through emergency spillway or top of dam by this depth)

The cost of building Pond 4 is approximately $281,523. A detailed cost estimate table is presented with Sheet OPT1.7.
The cost estimate tables for all the proposed facilities are provided in Appendix 7.

3.3 Overview of Option 2

Option 2 attempts to address drainage issues in the southern part of Chamberino primarily south San Jacinto Rd. This part of town is much more complicated because houses are built at the flow line of the historic arroyos. With the majority of houses lacking hard boundaries such as brick walls, any considerable rainfall will result in flooding. Due to the density of development and the nature of development, redirecting and concentrating flows through streets was not considered for the risk of flooding a property.

Option 2 assumes that Option 1 has been built therefore it attempts to deal with the local drainage in south Chamberino. Sheet OPT 2 shows the overall improvements being proposed along with a detailed layout of the various drainage improvements.

Berm Construction: Part of San Luis Ave. south of San Jacinto Rd. has an earth berm that has been constructed and runs north to south on east side of San Luis Ave. The berm stops one lot north of the intersection of Convent Rd. and San Luis Ave. This lot is the low point for subbasins south of Jan Jacinto Rd. to Monte Alto St. which would see most of the runoff pass through the lot and house. On the south west corner of Convent Rd. and Lopez Rd., there is an enclosed facility that is owned by the Board County Commission that would be compromised in a big event. Extending the existing berm south along San Luis Rd. and east along Convent Rd. would keep surface runoff within the limits of the street.

Discussion on Ponds 5 through 6-3: In prior meetings with the DACFC, several sites for ponds along Lopez Rd. from Convent Rd. to San Bernardo St were discussed.

Pond 5 south of Convent Rd. and Lopez Rd. was examined first. While this pond can provide the necessary storage, it would be a retention pond because there is no way to drain this pond by gravity. A pump station/forcemain system would be required to drain the pond. At four feet deep, the pond would store the entire 10 Yr-24Hr volume while the 100 Yr-24 Hr events would over top the pond. However, there would be a 4 ft. deep standing body of water that would drain over a prolonged period of time.

At two feet deep, the pond would have insufficient volume to contain both the 10Yr and the 100 yr volumes.

Either way, both scenarios would create a standing body of water that would not drain without a pump.

The same scenario is true for Pond 6-2. The pond would have to be very large and deep to store the 8.7 ac-ft of water for the 10 Yr – 24Hr storm. Without a pump station, there would be no way of draining the pond. As a result, any storms of significance would leave a body of standing water that would create a breeding ground for mosquitoes or a drowning hazard if not fenced off given both ponds proximity to houses.

Pond 6-1 and 6-3 were also examined, however no results are reported due the inefficiency of the ponds.
From a flood control standpoint, Pond 6-3 would make the most sense as it would detain the most area. However, that would require DACFC to acquire the property. Both ponds 6-1 and 6-2 are high enough to where they could be drained completely by gravity.

The other issue for these proposed ponds is cost-benefit. Ponds 5 and 6-1 through 6-3 are basically at the outfall of the Chamberino watershed.

The construction of these ponds does not provide a great downstream flood control benefit other than keeping the sediment out the agricultural fields on the east side of Lopez Rd.

Only Pond 6-3 was simulated in the Option 2 HEC-HMS model. The routing summary is provided below.

**Table 5**

<table>
<thead>
<tr>
<th>Proposed Ponds Detention Pond Routing Summary</th>
<th>Option 2</th>
<th>Chamberino Drainage Master Plan</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Detention Pond Name</strong></td>
<td><strong>Principal Spillway Pipe Diameter</strong></td>
<td><strong>Return Period</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Inflow</strong></td>
<td><strong>Peak Outflow</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Outflow Runoff Volume</strong></td>
<td><strong>Maximum Design Storage Volume (top of embank)</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Peak Storage Volume During Design Storm</strong></td>
<td><strong>Peak Water Surface Elevation</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Emergency Spillway Elevation</strong></td>
<td><strong>Pond Inlet Elevation</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Max Pond Depth</strong></td>
<td><strong>Peak Water Depth</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Top of Pond Embankment Elevation</strong></td>
<td><strong>Free board to Emergency Spillway Elevation</strong></td>
</tr>
<tr>
<td><strong>Free board to top of Pond Embankment</strong></td>
<td><strong>Footnotes for Detention Pond Routing Summary Table</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>a. Option 1</strong></td>
<td><strong>b. Conceptual Design Outlet Pipe</strong></td>
</tr>
<tr>
<td></td>
<td><strong>c. Results summarized from the HEC-HMS model</strong></td>
<td><strong>d. See subarea capacity discharge table and sources in Appendix A</strong></td>
</tr>
<tr>
<td>24 100 / 24 187 112 20.4 19.6 5.0 4.9 3810.30 3810.00 3804.00 7 6.3 3810.59</td>
<td>-2.3</td>
<td>0.2</td>
</tr>
<tr>
<td>24 1024 65 21 8.7 7.9 5.0 2.6 3808.00 3810.00 3804.00 7 4.0 3810.59</td>
<td>2.0</td>
<td>2.1</td>
</tr>
<tr>
<td>24 524 41 12 6.1 5.3 5.0 1.8 3807.00 3810.00 3804.00 7 3.0 3810.59</td>
<td>3.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

**Channel Improvements:** It is quite clear from field work and aerial imagery that there used to be a drainage facility in between Pond 6-1 and 6-3 (**Sheet Opt2**). In fact there is still evidence of an emergency spillway and remnants of a channel on the west side of Lopez Rd. The subbasins draining south east from San Francisco de Asis Ave. to San Bernardo Rd. drain to this point. The old channel is currently full of trash and debris. The construction of a rectangular open channel section with vertical walls to contain the water would help drainage issues that would arise at this point. See the conceptual channel section on **Sheet OPT2**.

DACFC determined that facilities proposed in Option 2 provided insufficient benefit for Chamberino therefore were not considered any further.
3.3 Conclusion

The facilities proposed in Option 1 will improve drainage conditions in Chamberino. Figure B summarizes the reduction in bypass flows after the implementation of Option 1 facilities. However the facilities will have to be phased out in order of most effective to least effective in terms of flood mitigation.

Smith recommends phasing the Option 1 proposed facilities in the following order:

**Pond 1:** Smith recommends the demolition of the existing embankment and construction of Pond 1 for two reasons.

1 - With Pond 1 in place, the risk to the proposed downstream facilities (Pond 2 and the Diversion Channel) will be lower.

2 - The existing dam embankment does not appear to be engineered. Based on field observation, there appears to be no compaction of the embankment. This poses as a significant risk to the north part of Chamberino should the dam fail catastrophically in a large storm event.

**Pond 2:** Pond 2 needs to be constructed to act as the outfall for the diversion channel.

**Diversion Channel:** The diversion channel will divert most of the offsite runoff to Pond 2.

**Pond 4:** Pond 4 controls the impervious discharge that is generated from the bean processing factory at the west end of San Francisco De Asis Rd.

**Pond 3:** Pond 3 would act as the outfall for the runoff captured and concentrated by the roadway improvements on Padre Pio Ave.

**Roadway Improvements on Padre Pio Ave:** Proposed improvements to keep runoff within ROW. Flows will discharge into Pond 3.

Table 6 below summarizes the facility costs recommended for Chamberino.

<table>
<thead>
<tr>
<th>Option 1 Facility Name In Order of Importance</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pond 1</td>
<td>$1,160,236</td>
</tr>
<tr>
<td>Pond 2</td>
<td>$264,494</td>
</tr>
<tr>
<td>Diversion Channel</td>
<td>$173,657</td>
</tr>
<tr>
<td>Pond 4</td>
<td>$281,523</td>
</tr>
<tr>
<td>Pond 3</td>
<td>$308,662</td>
</tr>
<tr>
<td>Roadway Improvements</td>
<td>$480,217</td>
</tr>
<tr>
<td><strong>Total Cost of Facilities</strong></td>
<td><strong>$2,668,789</strong></td>
</tr>
</tbody>
</table>
FIGURES / OPTION MAPS:

OPT1.1  Option 1 Overview of Proposed Facilities
OPT1.2  Pond 1
OPT1.3  Diversion Channel
OPT1.4  Pond 2
OPT1.5  Pond 3
OPT1.6  Padre Pio Roadway Improvements
OPT1.7  Pond 4
Figure B  Option 2 Overview Results Summary
OPT2  Option 2 Overview of Proposed Facilities
Option 1: Pond 1

- Pond Invert: 3894
- Pond Top: 3901
- Emergency Spillway: 3900
- Design Volume: 23.7 AC-FT
- Pond Side Slopes: 1V:3H
- 479.88' 340.74'
- 24" CMP Outfall Pipe

HEC-HMS Junction
10 YR - 24 HR Flow = 65 CFS

2 x 30 Inch CMP Culverts

Proposed Improvements
**Diversions Channel to be designed at 1.5% slope.

Channel Design Based on Normal Depths of 1.0 ft.

Channel Design Deficit to be provided by the Contractor.

See Appendix 6 for Preliminary Flowmaster Calculations.

**Typical Channel Section**

<table>
<thead>
<tr>
<th>WIDTH</th>
<th>DEPTH</th>
<th>SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>24 inch CMP</td>
<td>3.00'</td>
<td>0.025</td>
</tr>
</tbody>
</table>

**Proposed Diversion Channel to Outfall into Pond 2**

**Legend**

- **W** = Wastewater
- **P** = Stormwater
- **D** = Diversion

**Option 1:**

- **Pond Top:** 2822
- **Emergency Spillway:** 3620
- **Design Volume:** 5 AC-Ft
- **Pond Side Slopes:** 1V:3H

**Table Opt.1**

<table>
<thead>
<tr>
<th>Description</th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaning and Grubbing</td>
<td>1 each</td>
<td>$2,500.00</td>
<td>$2,500.00</td>
</tr>
<tr>
<td>Excavation for Diversion Channel</td>
<td>1 each</td>
<td>$2,000.00</td>
<td>$2,000.00</td>
</tr>
<tr>
<td>Removal of Existing Materials</td>
<td>1 each</td>
<td>$1,000.00</td>
<td>$1,000.00</td>
</tr>
<tr>
<td>Grading</td>
<td>2,000 cu yds</td>
<td>$2,000.00</td>
<td>$2,000.00</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>$7,500.00</td>
</tr>
</tbody>
</table>

**Notes:**

- The proposed design is subject to engineering review by the Contractor.
- The proposed design includes the following:
  - Diversion channel to outfall into Pond 2.
  - Channel design based on normal depths of 1.0 ft.
  - Channel design deficit provided by the Contractor.

**For Planning Purposes Only and Shall Not Be Used for Construction, Bidding, or Permitting Purposes.**

---

**Solutions For Today... Vision For Tomorrow**

**Dona Ana County Chamberino Drainage Master Plan**

**June, 2016**

24 inch CMP TO OUTFALL INTO POND 2.
OPTION 1: POND 3
POND INVERT: 3810
POND TOP: 3816.5
EMERGENCY SPILLWAY: 3816
DESIGN VOLUME: 8.4 AC-FT
POND SIDE SLOPES: 1V:3H

24 INCH CMP
TABLE 1-1

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>RATE 1</th>
<th>RATE 2</th>
<th>RATE 3</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>6&quot; GRANULAR BASE</td>
<td>$12.30</td>
<td>$12.30</td>
<td>$12.30</td>
<td>$12.30</td>
</tr>
<tr>
<td>10&quot; SUBBASE PREPARATION</td>
<td>$15.00</td>
<td>$15.00</td>
<td>$15.00</td>
<td>$15.00</td>
</tr>
<tr>
<td>2&quot; PCC AND GUTTER</td>
<td>$23.00</td>
<td>$23.00</td>
<td>$23.00</td>
<td>$23.00</td>
</tr>
</tbody>
</table>

PROJECTED ROAD SECTION
ASSUMPTIONS:
2% CROSS SLOPE
1.1% LONGITUDINAL SLOPE
MANNING'S N = 0.015 (ASPHALT)
TOP OF CURB CAPACITY = 109 CFS
10 YR - 24 HR FLOW BASIN W720 = 29 CFS
100 YR - 24 HR FLOW W720 = 62 CFS
SEE APPENDIX 6 FOR FLOWMASTER OUTPUT
Proposed Ponds Detention Pond Routing Summary: Option 1

Chamberino Drainage Master Plan

Pond 4

<table>
<thead>
<tr>
<th>JOB NO:</th>
<th>SHEET NO:</th>
<th>DATE:</th>
</tr>
</thead>
</table>

24 Inch CMP

POND 4

24 1004 40 11 3.3 2.3 4.7 1.2 3000.40 3000.40 7 7.4 3000.50 3.6 4.1

24 1024 20 9 3.3 2.3 4.7 1.2 3000.40 3000.40 7 7.4 3000.50 4.1 4.6

Legend:
- MAJOR BASIN BOUNDARY
- SUB-BASIN NUMBER
- SUB-BASIN NAME
- PRINCIPAL PIPE OUTLET
- PROPOSED IMPROVEMENTS
- BASE-LEVEL SURFACETOP
- PROPOSED SPILLWAY

Table 1:

<table>
<thead>
<tr>
<th>ITEM NO.</th>
<th>FIELD DESCRIPTION</th>
<th>LIMIT LIMIT DESIGNSPREAD</th>
<th>LIMIT LIMIT DESIGNSPREAD</th>
<th>LIMIT LIMIT DESIGNSPREAD</th>
<th>LIMIT LIMIT DESIGNSPREAD</th>
<th>LIMIT LIMIT DESIGNSPREAD</th>
<th>LIMIT LIMIT DESIGNSPREAD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CLEARANCE AND DRUMS COMPLETE IN PLACE</td>
<td>LENS 2800</td>
<td>1</td>
<td>2,000.00</td>
<td>2,000.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>CLEARANCE AND DRUMS COMPLETE IN PLACE</td>
<td>CY 4,000</td>
<td>1</td>
<td>4,000.00</td>
<td>4,000.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>CLEARANCE OF EXISTING POND INFILTRATION</td>
<td>LENS 2800</td>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>CLEARANCE OF EXISTING POND INFILTRATION</td>
<td>CY 4,000</td>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>CLEARANCE OF EXISTING POND INFILTRATION</td>
<td>LENS 2800</td>
<td>3</td>
<td>3,000.00</td>
<td>3,000.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Option 1: POND 4

DONA ANA COUNTY
CHAMBERINO DRAINAGE MASTER PLAN

JUNE, 2016

SOLUTIONS FOR TODAY...
VISION FOR TOMORROW

FOR PLANNING PURPOSES ONLY AND SHALL NOT BE USED FOR CONSTRUCTION, BIDDING, OR PERMITTING PURPOSES
OPTION 1: POND 1

- **POND INVERT:** 3894
- **POND TOP:** 3901
- **EMERGENCY SPILLWAY:** 3900
- **DESIGN VOLUME:** 23.7 AC-FT
- **POND SIDE SLOPES:** 1V:3H
- **479.88' 340.74'**
- **24" CMP OUTFALL PIPE**
- **DIVERSION CHANNEL**
- **DESIGN CAPACITY 110 CFS**
- **HEC-HMS JUNCTION J.CHNL6**
- **100 YR - 24 HR FLOW = 69 CFS**
- **2 X 30 INCH CMP CULVERTS**

**TOTAL 10 YR Q = 42 CFS**
**TOTAL 100 YR Q = 131 CFS**

OPTION 1: POND 2

- **POND INVERT:** 3810
- **POND TOP:** 3816.5
- **EMERGENCY SPILLWAY:** 3816
- **DESIGN VOLUME:** 5 AC-FT
- **POND SIDE SLOPES:** 1V:3H
- **400.53' 168.54'**

OPTION 1: POND 3

- **POND INVERT:** 3810
- **POND TOP:** 3816.5
- **EMERGENCY SPILLWAY:** 3816
- **DESIGN VOLUME:** 8.4 AC-FT
- **POND SIDE SLOPES:** 1V:3H
- **24 INCH CMP**
Proposed Ponds Detention Pond Routing Summary  Option 2

<table>
<thead>
<tr>
<th>Pond</th>
<th>Principal Return Period</th>
<th>Duration</th>
<th>Volume</th>
<th>Flow Rate</th>
<th>Surface Area</th>
<th>Drainage Area</th>
<th>Top of Pond</th>
<th>Bottom of Pond</th>
<th>Elevation</th>
<th>Elevation</th>
<th>Proposed Flood Elev.</th>
<th>Pro-coned Flood Elev.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pond 6-1</td>
<td>200'</td>
<td>7</td>
<td>10.4</td>
<td>1.8</td>
<td>0.8</td>
<td>0</td>
<td>3815.00</td>
<td>3810.00</td>
<td>8</td>
<td>3815.00</td>
<td>43.2</td>
<td>4.8</td>
</tr>
<tr>
<td>Pond 6-2</td>
<td>200'</td>
<td>7</td>
<td>10.4</td>
<td>1.8</td>
<td>0.8</td>
<td>0</td>
<td>3815.00</td>
<td>3810.00</td>
<td>8</td>
<td>3815.00</td>
<td>43.2</td>
<td>4.8</td>
</tr>
<tr>
<td>Pond 6-3</td>
<td>200'</td>
<td>7</td>
<td>10.4</td>
<td>1.8</td>
<td>0.8</td>
<td>0</td>
<td>3815.00</td>
<td>3810.00</td>
<td>8</td>
<td>3815.00</td>
<td>43.2</td>
<td>4.8</td>
</tr>
</tbody>
</table>

**Proposed Improvements:**
- **Option 1:** POND 3
- **POND 6-1/6-3 Channel Rehabilitation**

**Major Basins:**
- **Principal Pipe Outlet:** Proposed improvements
- **Proposed Improvements:**

**Design Volume:**
- **Pond Top:** 3816.5

**Legend:**
- Major Basins Boundary
- Minor Basin Boundary
- Minor Basin Number
- Principal Pipe Outlet Protection
- Design Property Line
- Proposed Improvements

**Note:** Design, orthophotography, and layout completed in 2015 by DORA. Engineering - T. Conner, A. Stover, J. F. Bivens.