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1.0 INTRODUCTION

The purpose of this Hydraulic Report is to assess the hydraulic impacts of an approximately 3-mile long bollard fence constructed by a private firm Fisher Industries in early 2020 on private land along the bank of the Rio Grande floodplain in Hidalgo County, Texas, and to determine if these impacts are minor and consistent with Article IV B(1) of the 1970 Boundary Treaty. To ensure impacts are minor, the U.S. Section of the International Boundary and Water Commission (USIBWC) has criteria stating that the design flood Water Surface Elevations (WSE) for the proposed (with project/fence) condition shall not increase more than 6-inches in rural areas or 3-inches in urban areas compared to the existing condition WSE and have no more than +5% increase in flow deflection towards either the U.S. or to Mexico. In areas where levees exist, there shall be no WSE increase as such increase would decrease the existing freeboard requirements by the U.S. Federal Emergency Management Agency (FEMA) regulation (44CFR 65.10). Adherence to these regulations is required for levees to be accredited by FEMA and remove areas on the landside from the floodplain resulting in residents not having to purchase flood insurance.

The U.S. Section had developed a hydraulic modeling methodology for a detailed analysis of the hydraulic impacts. This methodology was shared with the project proponent, who conducted the analysis and submitted a report (Fisher, 2020). This report documents additional analysis conducted by the USIBWC to analyze the hydraulic impacts.

Figure 1: Area map of Hidalgo County and Tamaulipas State at Rio Grande/Rio Bravo

2.0 LOCATION

The bollard fence was constructed by the private firm Fisher Industries along the bank of the floodplain on the U.S. side of the Rio Grande upstream from the Anzalduas Dam in Hidalgo County,
Texas. The exact project location starts at 3.36 miles upstream of Anzalduas Dam and extends to 6.36 miles upstream. Therefore, the length of the bollard fence is approximately three (3) miles. The fence alignment runs parallel to the river with an offset of 35 feet from the bank. The location is shown with the green place-mark in Figure 1.

Figure 2 shows the exact location of the bollard fence close to the left bank (looking downstream) of the Rio Grande. This project falls approximately within the following overall limits: Upstream (26° 10’ 15.90” N and 98° 21’ 28.21” W), and downstream end (26° 10’ 1.80” N and 98° 20’ 30.84” W).

![Figure 2](image)

**Figure 2**: Bollard Fence Location on the Left Bank of Rio Grande (US Side)

### 3.0 HYDRAULIC ANALYSIS

Hydraulic modeling was conducted to assess the hydraulic impacts of the constructed bollard fence on the Rio Grande floodplain. The hydraulic impacts were evaluated using the design flow of 235,000 cfs for this reach of the Rio Grande (Figure 3). The hydraulic model was developed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center - River Analysis System (HEC-RAS) software Version 5.0.7. A 1D/2D HEC-RAS model developed by Fisher Industries was modified to calibrate the model. The calibrated model was then used for the existing and proposed conditions modeling to assess the hydraulic impacts.

The HEC-RAS software supports simultaneous 1D channel and 2D floodplain components in a single model, performing calculation iterations between 1D and 2D areas. The 1D and 2D components are connected using lateral structures. These structures use the weir equation to exchange flows between 1D component and 2D mesh. A total of 16 lateral structures was used to exchange flows between river channel and floodplains.
Figure 3: Design Flow Hydrograph with Peak of 235,000 cfs

4.0 INPUT DATA

4.1 Topographic/Bathymetric Data

LiDAR data (2011) was used to model the terrain. Surveyed cross-section by Fisher Industries, and cross-sections from the S&B (2008) model were used to construct the 1D channel cross-section at every 500 feet for the full length of the project site. Using channel geometry data and LiDAR dataset, a single terrain was created by the Fisher Industries. The terrain model developed by Fisher Industries was used to complete this hydraulic analysis. The horizontal datum is NAD 1983, State Plane Texas South 4205 Feet, and the vertical datum is NAVD88.

4.2 Manning’s Roughness Values

Manning’s roughness values were refined during calibration with observed stage near the downstream of the project site. Manning’s roughness values for different regions of the modeled area are shown in Table 1.
4.3 Boundary Conditions

Boundary conditions were applied at the upstream project extent (RS 47798) with an inflow hydrograph with peak flow equal to the design flow of 235,000 cfs with hydrograph shape of the Hurricane Beulah of September 1967. The downstream boundary condition was normal depth with slope 0.01 foot/foot.

5.0 MODEL CALIBRATION

A detailed model calibration was conducted in this study. There are no measured water surface elevations within the project reach for past major floods. The nearest gaging station is at Los Ebanos, Texas, about 24 miles upstream from the project site. However, this gaging station malfunctioned during Hurricane Beulah and there was no recorded data once the peak flow was observed. Hurricane Beulah was recorded at the gaging station located in Rio Grande City which is about 57 miles upstream from the project site. The peak discharge of 220,000 cfs was recorded at midnight of September 22\textsuperscript{nd}-23\textsuperscript{rd}, 1967 at this gaging station.

![Image of Hurricane Beulah Hydrograph](image_url)

**Figure 4:** Hurricane Beulah Hydrograph (September 1967) at Half-Mile Downstream of Anzalduas Dam
During Hurricane Beulah of 1967, there was no flow diversion at Banker weir. Instead, flood waters were diverted from the Rio Grande through the Mission Inlet, located approximately 6.5 miles upstream of Anzalduas Dam. The remaining flow continues downstream. On September 24, 1967, the gage about 0.5-mile downstream of Anzalduas Dam had a maximum gage height of 112.82 feet (NAVD 88) with a peak flow of 131,000 cfs. Since there is no full hydrograph for the event, a hydrograph for model calibration was designed with a peak flow of 135,000 cfs with the shape of hydrograph adopted from Hurricane Beulah hydrograph observed at gaging station of Rio Grande City, Texas. Figure 4 shows the hydrograph used for model calibration.

Several calibration runs were conducted to match the WSE upstream of Anzalduas Dam extrapolated from the value measured at the gage 0.5 mile downstream of the dam. The finalized roughness coefficient values based on calibration model run are shown in Figure 5. Channel roughness value is 0.035 with 0.04 for both overbanks. Floodplain comprises of row crop (corn), forest and cleared area. Roughness for various forest areas vary from 0.08 to 0.3, including some areas with 0.10. Roughness values have to be increased to 0.3 at the downstream areas to match observed WSE.

Figure 5: Final Manning’s Roughness Values after Model Calibration
Based on 1D steady state S&B (2008) model, the WSE slope is about 0.000331 near the Anzalduas Dam. At this WSE slope, a point 445 feet upstream of the dam would have a WSE of 1.02 feet more than the WSE measured at the gage located 0.5 mile downstream of the dam. Thus, the peak measured WSE at 445 feet upstream of the dam would be 113.84 (112.82+1.02) feet. The model has simulated 113.45 feet WSE at this point which is very close to the observed WSE (113.84 feet). Maximum WSE obtained during calibration run is provided in Appendix A.

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<th>Channel</th>
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<th>Overbanks</th>
<th>0.04</th>
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<td>Forest 1</td>
<td>0.10</td>
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<tr>
<td>Forest 2</td>
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<td>Forest 3</td>
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</tr>
<tr>
<td>Field 4</td>
<td>0.06</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 1: Manning’s Roughness Values used in Simulation

5.0 EXISTING CONDITION MODEL

The existing condition model was used as a baseline to compare with the proposed condition model results and evaluate the hydraulic impacts.

The existing condition model includes:

- 2D computational grid with 25-foot grid cell size for the left and right overbanks
- 1D channel river reach
- 1D/2D lateral weir structures (16 of them) for flow movement from channel to overbanks
- downstream Anzalduas Dam as an inline structure

The inflow hydrograph with peak flow of 235,000 cfs and boundary conditions are as described above.

6.0 PROPOSED CONDITION MODEL

The proposed condition model for this project includes the constructed 18-foot high bollard fence along the U. S. side bank of the Rio Grande. The bollard wall consists of 6-inch square bollards with a 4-inch clearance between bollards. The bollards are rotated 45-degrees to have 5-inch clearance, as shown in Figure 6. For the proposed fence condition, to account for debris blockage during the design flood in the model, the clear space between adjacent bollards is reduced by 30%. That distance is added to the bollard widths to keep the center to center spacing between adjacent bollards the same. Therefore, in the 45-degrees rotated configuration, the open space becomes 3.5 inches and the bollard width becomes 7.5 inches (increased from 6 inches).
The proposed condition model includes the elements in the existing condition model and:

- 18-foot tall bollard modeled as 18 feet gate with 3.5 feet width
- 2D left floodplain with 10 lateral structures, while 2D right floodplain with 8 lateral structures
- Bollard wall modeled with 59 gate groups with 1427 identical gate openings

Eighteen (18) lateral weir structures are used to connect the 1D channel with 2D left (USA) and right (Mexico) overbanks. The constructed bollard fence along the Rio Grande’s left bank (35 feet offset) is modeled as a lateral gate on top of the 1D/2D weir structure as shown in Figure 7. To avoid having a large number of lateral structures to represent the bollard fence in the model, adjacent bollards were combined. The spacings between the bollards were also similarly combined. Twelve (12) adjacent bollards were combined. Therefore, using the multiplier factor of 12 resulted in a bollard thickness of 7.5 feet and spacing of 3.5 feet. This also took into account the 30% blockage.
The USIBWC levee at the project site was designed and constructed to have three (3) feet of freeboard above the design flood WSE as estimated by the USIBWC (2003) model. The constructed bollard wall falls between levee stations 130+00 and 190+00.00 (Figure 8). WSEs from both existing and proposed 1D/2D models are compared with design flood WSE values to determine if levee freeboard is encroached. Existing condition model WSE values are close to the design flood WSE values in the USIBWC (2003) model. In the Fisher Industries modeling, the WSE values obtained were two (2) to three (3) feet lower than the WSE values in S&B (2008). The more detailed calibration conducted in this study resulted in a better match with the design flood WSE values from USIBWC (2003) and S&B (2008) models.

**Figure 7:** 18 Feet High 3.5 Feet Wide Gates (Gaps Between Bollards) With 7.5 Feet Bollard Wall

### 7.0 EVALUATION OF HEC-RAS OUTPUT

The USIBWC levee at the project site was designed and constructed to have three (3) feet of freeboard above the design flood WSE as estimated by the USIBWC (2003) model. The constructed bollard wall falls between levee stations 130+00 and 190+00.00 (Figure 8). WSEs from both existing and proposed 1D/2D models are compared with design flood WSE values to determine if levee freeboard is encroached. Existing condition model WSE values are close to the design flood WSE values in the USIBWC (2003) model. In the Fisher Industries modeling, the WSE values obtained were two (2) to three (3) feet lower than the WSE values in S&B (2008). The more detailed calibration conducted in this study resulted in a better match with the design flood WSE values from USIBWC (2003) and S&B (2008) models.
Maximum water surface elevations for the entire simulation period for both existing and proposed condition models are shown in Appendix B. With bollard fence placed in the floodplain close to the riverbank, it was expected that proposed condition model WSEs would be higher than the existing WSEs in all areas. But based on model simulation, proposed condition model WSE values are slightly lower than the existing condition WSE values at most locations (Table 2). This can be explained through flow patterns for the two conditions in the project site. Initially, the bollard fence provides resistance to flows and the flow is diverted around the wall as shown in Figure C-1 through Figure C-3 in Appendix C. As time passes, the flow pattern for proposed condition model becomes similar to that of the existing condition model (Figure C-4 through Figure C-6). Flows in proposed condition model pass through the bollard fence gaps (3.5 feet) with slightly higher velocity than the existing
condition model where there is no flow resistance due to the fence. Thus, this slight increase in velocity can result in slightly lower WSE in the proposed condition model. Velocity distribution for both existing and proposed condition models are shown in Appendix C.

Although velocity distribution is similar in both existing and proposed condition models, the river bank modified with 5H:1V slope has to be monitored for erosion occurrences and any observed erosion should be repaired by the proponent in a timely manner.

**8.0 ANALYSIS OF DEFLECTION CALCULATIONS**

Flow deflections were evaluated at a limited number of cross sections. Profile lines for flow deflection calculations developed by Fisher Industries were used. The deflection calculations were repeated using the modeling results from this study. The highest flow deflection is 10.32% towards the U.S. at profile line 24260. **Figure 9** shows the deflection values to both the U.S. and Mexico at the location indicated.

![Flow Deflection Map](image)

**Figure 9**: Flow Deflection along Selected Profile Lines at the Project Site

The hydraulic impacts for this 3-mile long bollard fence project were found to be minor. WSE elevations were found to be mostly lower in the proposed condition than in the existing condition, and therefore did not decrease existing levee freeboard. Percent deflection calculations exceeded the
+5% threshold of the USIBWC at one location in a limited number of cross sections analyzed. Flow deflections at other locations appears to be minor because flow is diverted to the left bank to the U.S. side by the bollard fence alignment. Also, the site is within the backwater area of Anzalduas Dam, resulting in lower flow velocities in the floodplain.

12.0 CONCLUSIONS

There is significant deflection towards the U.S. approximately near the middle of the constructed bollard fence which needs to be mitigated. This can be done, for example, by including a gate in the location of the observed impact. If the bollard fence is extended, there may be hydraulic impact which needs to be verified through new hydraulic analysis.

Although velocity distribution is similar in both existing and proposed condition models, the riverbank modified with 5H:1V slope must be monitored for erosion occurrences and any observed erosion should be repaired by the proponent in a timely manner.

11.0 REFERENCES


Appendix A

Screen shots of Hurricane Beulah Hydrograph September 1967 Calibration

Figure A-1: Maximum Water Surface Elevation for Calibration Model

Figure A-2: Simulation of 113.45 Feet WSE 445 Feet Upstream of Anzalduas Dam During Calibration
Figure A-3: Manning’s Roughness Values for River Channel and Overbanks

Figure A-4: Manning’s Roughness Values for 2D Flow Areas
Appendix B: Maximum Water Surface Elevations

Figure B-1: Maximum water surface elevations for existing (top) and proposed model (bottom), proposed model elevations were slightly lower.
Appendix C: Flow Pattern-Velocity Distribution

**Figure C-1:** Flow pattern on September 22, 1967 at 11:50 hours, existing (top), and proposed model (bottom); flow is blocked by fence at the bottom, so no flow in floodplain near the fence compared to existing condition model.
Figure C-2: Flow pattern on September 22, 1967 at 13:00 hours, existing (top), and proposed model (bottom); flow is blocked by fence at the bottom, so very little flow in floodplain near the fence compared to existing condition model. Flow in proposed model is going around the upstream end of the fence.
Figure C-3: Flow pattern on September 22, 1967 at 13:50 hours, existing (top), and proposed model (bottom); flow patterns are slightly different; more flow is in the floodplain near fence location in existing model compared to proposed model where there is resistance to flow from the fence (bottom).
Figure C-4: Flow pattern on September 22, 1967 at 15:40 hours, existing (top), and proposed model (bottom); with high flows, flow patterns are similar for existing and proposed model near the fence location.
**Figure C-5**: Flow distribution on September 22, 1967 at 20:10 hours, existing (top), and proposed model (bottom); with high flows, flow patterns are similar near the fence location for existing and proposed model; flow patterns are slightly different at the downstream areas.
Figure C-6: Flow pattern on September 23, 1967 at 3:10 hours, existing (top), and proposed model (bottom); with peak flow, flow patterns are similar for both existing and proposed model; flow can go through, around and over the fence for proposed model.
Figure C-7: Maximum flow velocities for existing (top), and proposed model (bottom); with high flow, both models exhibited erosive velocities along the river channel.