Ponderosa Designs

Duncan, Arizona Flood Mitigation Project:

Final Design Report

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Acknowledgments

The Duncan, Arizona Floodplain Feasibility Study is part of the Northern Arizona University's Senior Capstone Projects for the Civil and Environmental Engineering program. The 2017 capstone team Ponderosa Designs would like to thank the following individuals for making this project possible. Ponderosa Design would like to personally thank the Town of Duncan, Arizona for hosting the public forum and showing the team wonderful hospitality.

Tom Loomis P.E., RLS, CFM

Technical Advisor outside of NAU

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Client, Greenlee County Engineer

2015 Senior Capstone Team, NAU Crown Engineering

2016 Senior Capstone Team, Hydro Engineering

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1.0 Project Understanding

1.1 Project Purpose

Duncan is a town located in Southeastern Arizona along the Gila River (Figure 1.1). The current population is about 800 residents, according to the 2015 census. This town has experienced consequential flooding events throughout its history due to its geographic location on the floodplain. The floods that occurred in 2005 and 1978 caused significant damage to infrastructure, agricultural crops, and property [1]. Therefore, the Duncan Flood Mitigation Team has been contracted to protect the town from future flooding. The purpose for this study is to define and evaluate possible mitigation measures in order to alleviate flooding in the town and the surrounding area.

Figure 1.1: Project Location: Duncan, Arizona [2]

1.2 Project Background

The project is located in Duncan, Arizona which lies three miles east of the New Mexico border in Greenlee County (Figure 1.1). The climate in this area is semiarid with much of the total rainfall coming during the months of July and August. The soil in this area is fertile silt and clay, which supports the agricultural industry in the area [1]. The Duncan Floodplain project is in its third stage with the first stage of this project consisting of floodplain analysis and conceptual levee alignment design. This was done by the NAU Crown Engineering Team in partnership with Philip Ronnerud, P.E. This stage provided insight to Duncan's current flood risks. These studies were all based on updated topographic maps and a new estimate of the 100 year peak discharge or 1% annual chance of this storm event happening. The software of Autodesk Civil 3D and HEC-RAS were used to model the one-dimensional flow for this stage of the project. Based on these results, the alignment of a flood protection levee was proposed to extend 1.9 miles along the 3.5 mile Gila River reach. Next, the second stage of this project was conducted by the NAU Hydro Engineering Team in partnership with Philip Ronnerud, P.E to create a two-dimensional model of the floodplain. This model was achieved by using a software called FLO-2D. The team used the data from these modeling programs to conduct a more accurate levee analysis to meet the goals for the project. The current stage of this project will be done by the Ponderosa Design Team. The team will refine the models that the previous two capstone teams developed and create flood mitigation scenarios that will help protect the town from future flooding. This encompasses about 18,000 foot section of the along the Gila River as seen in Figure 1.2 and 1.3.

1.3 Area of Interest

In the town of Duncan, the two major roadways that pass through the area are the Highway 70 which leads to Safford, Arizona and the Highway 75 which heads to Clifton, Arizona. The area of focus for this study is along the Highway 75 bridge which spans over the Gila River (next to Duncan).The bridge restricts the flow of the Gila River during flooding events and leads to water flooding into the town. The major highways, along with the river orientation can be seen in Figure 1.2 below.

Figure 1.2: Aerial View of Project Site [2]

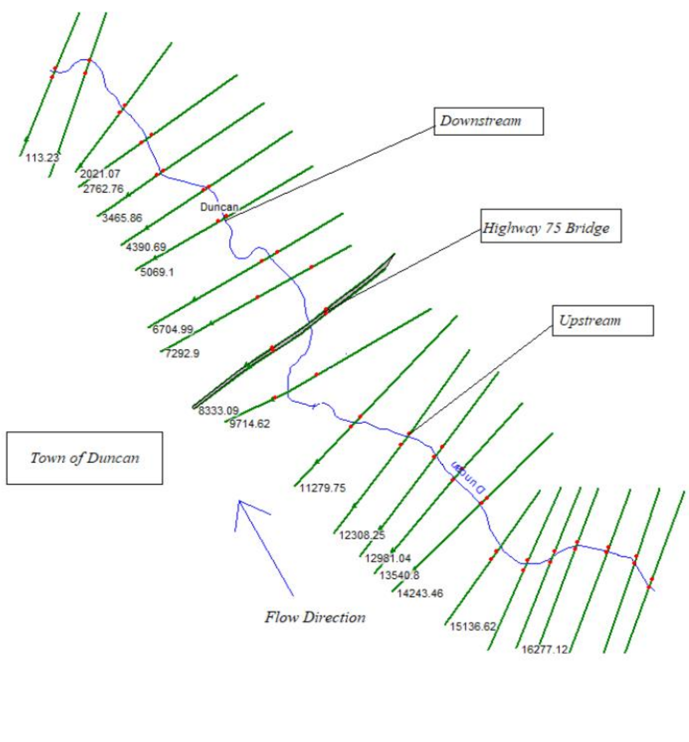

Figure 1.3: Area of Interest along Gila River

1.4 Technical Considerations

Ponderosa Designs has to consider the hydraulic effects that flooding has on hydraulic structures, buildings, and vegetation in the area (Figure 1.2). This also includes the current state of the Gila River channel and overbank floodplain. A combination of the computer programs including; QGIS, ESRI ArcMap, HEC-RAS, and FLO-2D will be utilized to model the effects that the proposed mitigation measures have on the floodplain. The developed modeling systems will be used to determine the existing and future conditions during a 100 year storm event. Lastly, these models will provide the client with a possible structural solution that could be implemented in the area of study.

1.5 Constraints

For this project, the design team will learn how to use all of the modeling programs listed above. However, the team had no prior experience with these modeling programs so problems occurred due to lack of experience. In addition, the team experienced problems corresponding with the technical advisor because he works in a different town. The technical advisor has the ability to process all the QGIS data for FLO-2D. The team is only able to communicate over email, telephone, and through a few meetings with the technical advisor. This made it problematic to troubleshoot modeling errors efficiently. In addition, the team must design for the 100 year storm (48,000 cfs), make no alterations to the shape of the channel, and keep the economic and environmental needs of the public in mind [3]. All of these constraints will impact the team's final design solution.

1.5.1 Site Visit and Public Forum

Ponderosa Designs conducted one site visit to Duncan, Arizona in the spring of 2017. This was done in order to obtain further information on the site and conduct a public forum. There will not be another site visit for the team. All information needed for analysis can be obtained through the client, GIS, or online reports. In addition, Ponderosa Designs (along with Phillip Ronnerud and Tom Loomis) held a public forum in an attempt to further understand the project from the perspective of the citizens of Duncan. The representatives from the Duncan city council and members from the County Board of Supervisors were present. This meeting informed the team that they need to design flood protection that will withstand the 100 year storm event. According to the USGS upstream gage, the 100 year flood is 48,000 cfs. The team used a flow rate of 50,000 cfs for all modeling alternatives.

1.5.2 Technical Advisor Guidance

The Ponderosa Design team coordinated meetings with Wilbert Odem, the grading instructor, to insure course requirements were being met. The team also maintained steady dialogue with the technical advisor, Tom Loomis, to keep focused on the client needs, Phillip Ronnerud. QGIS and HEC-RAS Models are sent to Tom Loomis to be processed through FLO2D. Tom follows up by sending the results back to the team and they adjusted their models accordingly.

1.6 Stakeholders

The main stakeholder involved with the project were Greenlee County residents because they are directly impacted by a flooding event in the area. The residents of Greenlee County will be asked to provide additional support in allowing the accommodation of roadway expansion within the community. This design could have the possibility to influence ADOT to support and fund the corresponding project to ensure proper implication of infrastructure elements. The construction of a levee would require the design to be up to the specifications of the US Army Corps of Engineers. These flood mitigation measures could prompt FEMA to produce a new flood risk map for insurance carriers. ADOT is a stakeholder for the project as well because the highway relocation would impact their roadways in the area. The local farmers are impacted because Duncan has an agriculture-based economy and flooding puts farming land at risk and their land might have to purchase in order to construct the flood mitigation measures. Lastly, FEMA sets the certification and accreditation standards for the levee-based mitigation measures. This could change the towns FEMA Special Flood Hazard designation.

2.0 Modeling Approach

2.1 Abbreviations and Applications

In order to meet the objectives of the project, the design team utilized

QGIS, HEC RAS, and FLO-2D. QGIS stands for Quantum Geographic Information System and was used to evaluate if the levee placements were effective for protection against the 100 year storm. The team used QGIS and ArcGIS to develop FLO-2D input data files designed to simulate the overall effects of the proposed mitigation measures on the flooding in Duncan. In addition, HEC-RAS stands for Hydrologic Engineering Center's River Analysis System and was used to improve the hydraulics at the Highway 75 Bridge, model the flood in one dimension, and obtain a water depth against the proposed levee alignment. Lastly, FLO-2D stands for Two-dimensional Flood Routing Model and was used to analyze how effective the combined input data from HEC-RAS and QGIS were (Figure 2.1) The team used the FLO-2D model to simulate the overall Gila River floodplain hydraulics.

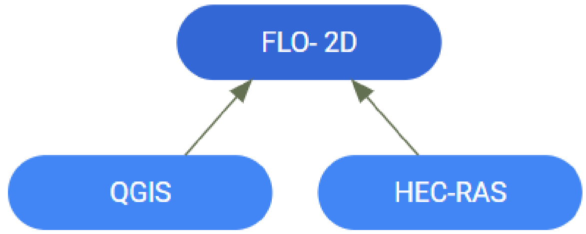

Figure 2.1: Modeling Application Flow Chart

2.2 Software Rights

Ponderosa Designs acquired user manuals, instructions, and programming rights' through Northern Arizona University and the technical advisor Tom Loomis. HEC-RAS and QGIS are public domain software's. A no cost FLO-2D license was granted to the team be FLO-2D software,Inc. for educational purposes.

3.0 Testing and Analysis

Ponderosa Designs came up with 6 different HEC-RAS alternatives and 14 of FLO-2D alternatives in order to meet the project goals and stay within the constraints of the project (outlined above). Each alternative was created, analyzed, and considered for the final design solution. A description of each flood mitigation alternative is described below. The final design solution was chosen from these alternatives and explained in Section 4.0.

3.1 HEC-RAS Modeling

The Highway 75 Bridge is a major obstruction in the Gila River next to the town of Duncan. Ponderosa Designs will utilize HEC-RAS to try to improve the hydraulics at the bridge as well as obtain a water depth against the proposed levee alignment. All HEC-RAS alternatives that were created are based on the base model derived by the 2016 Capstone team. This baseline model was called the, "Corrective Effective Model," and additional simulations were created by Ponderosa Designs. These alternatives expressed changes to the bridge hydraulics and were compared to the existing model. The alternative models are as outlined in the following sections (Section 3.1.2-3.1.7). These are based on a subcritical flow regime, steady flow analysis, and specific set boundary conditions. A subcritical flow regime was used because this flow state addresses the natural condition of the Gila River. The previous capstone team modeled the upstream boundary condition and set the normal depth (S) to 0.0195 ft. The downstream boundary condition was set to critical depth. The bridge fence is only on the upstream side of the Highway 75 (Figure 3.1-3.2) so this is the only side that changes in HEC-RAS simulations. Flow is allowed to overtop the existing levees and be conveyed in the overbank floodplain. The following sections will outline all the characteristics of the HEC-RAS simulations and their corresponding results.

Figure 3.1: Upstream Side of Bridge (Fence) [4]

Figure 3.2: Downstream Side of Bridge [4]

3.1.1 Corrective Effective (Base Model)

The base model of the Highway 75 Bridge created from the previous capstone team includes the roadway, fence, and piers (Figure 3.3). However, the team previous did not add a pier shape coefficient value or debris to the piers. These features, along with removing the bridge fence, and setting the left bank to the proposed levee alignment will be modified to see if the hydraulics at the upstream side of the bridge improve. This base model has a velocity of 2.28 ft/s and a water surface elevation of 3654.93 feet (Appendix A.1). In addition, the Froude number has a minimum value of 0.13 (right before the bridge) and a max value of 0.74 (downstream of the bridge). The importance of this value will be explained in Section 3.1.8)

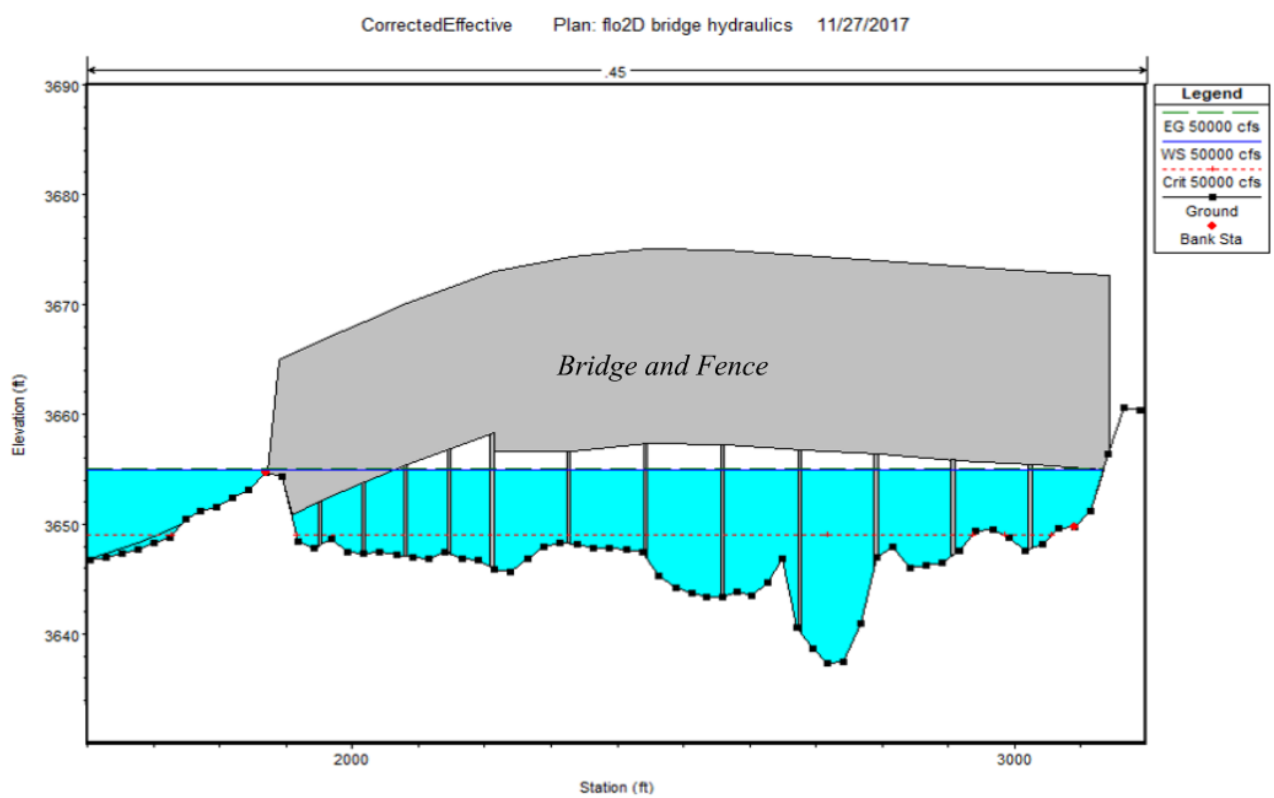

Figure 3.3: HEC-RAS Base Model

3.1.2 Debris Only Model

The first model that the team came up with was called the "Debris Only" model. This represents the inclusion of debris buildup upon the individual piers. This model keeps the fence in place and keeps the pier coefficient or K value at zero. The piers in the highest flow area (Pier 7-10) received a debris blockage of 10 feet by 10 feet. This value was chosen based on river debris reports that the client sent to the team [5,6]. The reports show that the average debris build up height for large floating debris on a large rivers is about 10 feet. The Gila River debris is considered large floating debris because of the Cottonwood, Salt Cedar, and Willow trees that are being washed downstream. A width of 10 feet was chosen because the county engineer advised the team that the debris buildup width is about double the pier width. In addition, the debris blockage dimensions are about 7 feet (pier width plus 2 feet) wide and 2-4 feet high for the piers closer to the banks. This blockage area is smaller because less debris typically gets caught on the piers within lower flow areas of a large river. The bridge piers with debris blockage applied are shown on Figure 3.4 (brown rectangles on piers). It is apparent that pier blockage is small in relation to the entire bridge conveyance area. As a result, when the team ran the 100 year storm event on this model, the hydraulics did improve. The water surface lowered height is 3.24 feet and the velocity increased to 1.59 ft/s (Table 3.1). This is due to the smaller cross sectional area for water to moving under the bridge (Table 3.1, Appendix A.2). Therefore, the team will apply debris to the piers to model a realistic scenario of flooding but will not focus heavily on debris removal as a primary design solution.

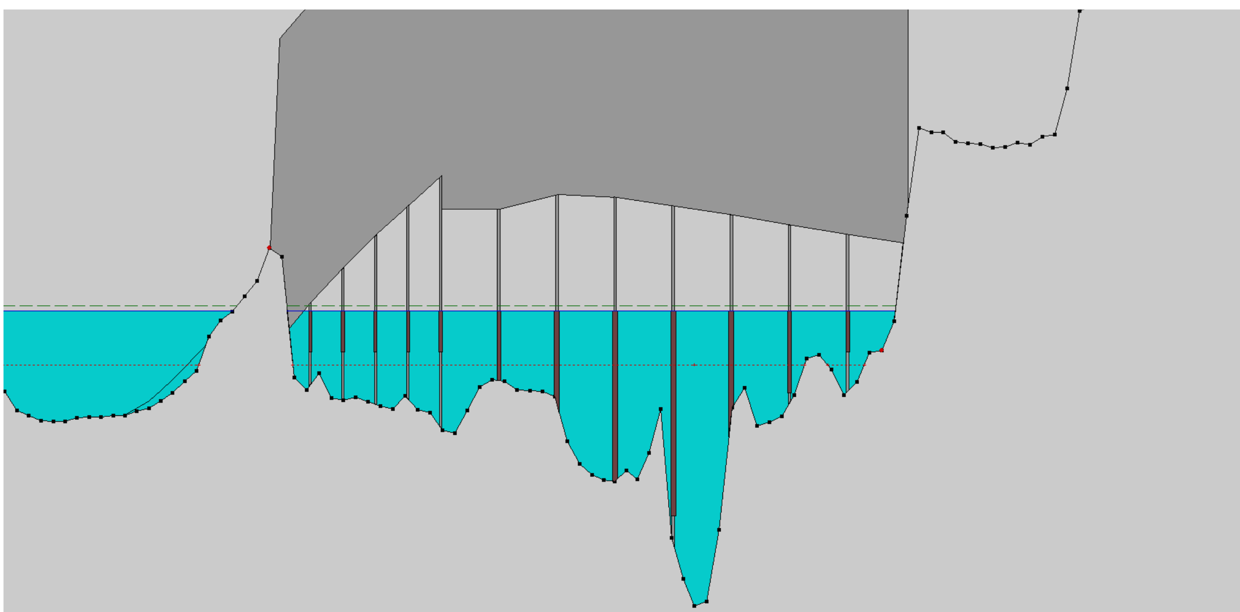

Figure 3.4: Bridge Piers with Debris Blockage

3.1.3 Piers Only Model

This model will highlight the impacts of only changing the pier shape. The model keeps the bridge fence on and debris applied to piers. The team choose a pier coefficient (K) of 0.9 because that is the pier shape that is subject to less friction losses. This shape represents a semi-circle nose and tail pier shape and the current bridge has circular piers that are not connected. In addition, the K value came from the HEC-RAS manual [7] and was suggested for use by the technical advisor (Table 3.0). When a K value of 0.9 was applied to piers, the hydraulics in and around the bridge did not improve (Table 3.1, Appendix A.3). Therefore, the team will apply the K value to the following models to reduce friction losses around the piers but will not focus on this standalone model for hydraulic improvement.

Table 3.0: Yarnell's Pier Coefficient (K) [7]

Table 5-4

Yarnell's pier coefficient, K, for various pier shapes

Pier Shape	Yarnell K Coefficient
Semi-circular nose and tail	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90 degree triangular nose and tail	1.05
Square nose and tail	1.25
Ten pile trestle bent	2.50

3.1.4 Pier Coefficient/ Debris Blockage/Fence in Place Model

The team wanted to see what would happen if they combined the previous two models. Therefore, the "Improved K w/ Fence" model shows the hydraulic effects of widening the piers to account for debris and improving the pier shape coefficient (K). This model keeps the fence on the bridge. The results of this simulation compared to the base model show neutral effects because the water surface elevation and velocity did not change (Table 3.1, Appendix A.4). This is expected because the previous model had neutral impacts so combining the two had the same effect.

3.1.5 Pier Coefficient/ Debris Blockage/ No Fence Model

The Fence Model expresses the changes of the hydraulics pertaining to the removal/reconfiguration of the pedestrian fence (Figure 3.5). This model keeps the debris on the piers and the K value at 0.9 because both options model a realistic scenario. When the fence was removed, the hydraulics did not improve.

The fence removal was done by using the Highway 75 Bridge as-builds provided by ADOT [4]. These results are outlined in Appendix A.5.

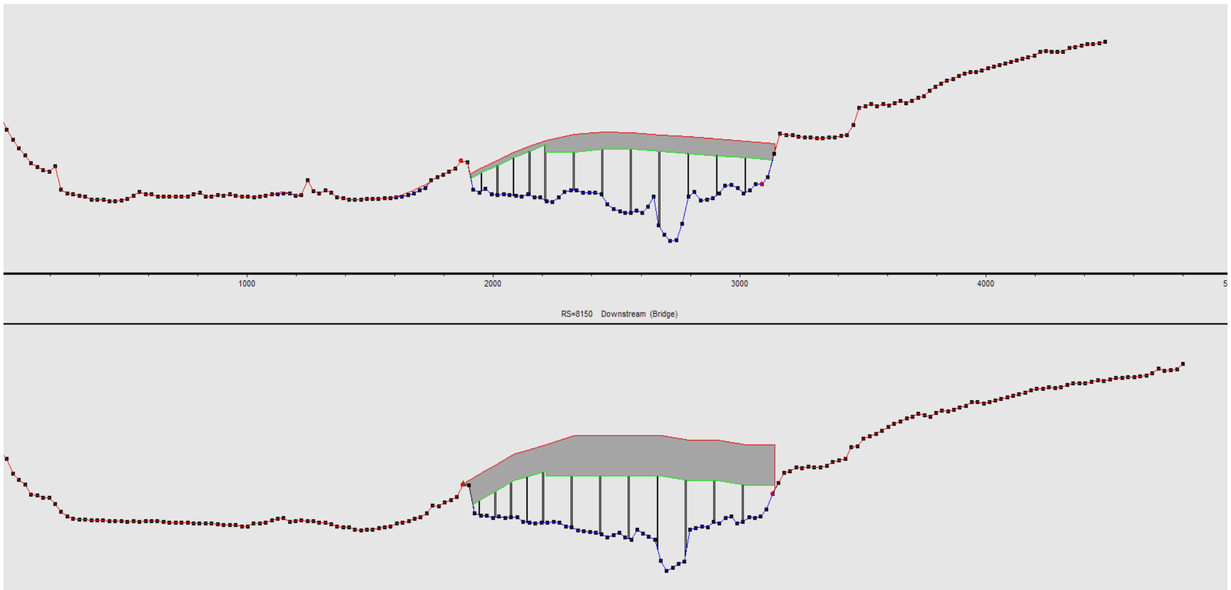

Figure 3.5: Bridge without Fence (Upstream)

3.1.6 Highway Relocation with Fence Model

Since the changes done to the previous models did not significantly change the hydraulics, the team decided to simulate the left bank (left of the bridge) being set to the same station and elevation of the proposed highway alignment levee that is being modeled in FLO-2D (Figure 3.6). The team set the left bank to the height of 4000’ by going through an option in HEC RAS called, “ineffective flow areas”. This would force water over the bridge and prevent water from pooling up in the town. Doing this actually improved the hydraulics at the bridge because the water surface elevation decreased 2 feet and the velocity increased 2.73 ft/s in the model (Table 3.1, Appendix A.6). This is important because the team did not need to design the levee 2 by feet higher during the 100 year flood event. Therefore, the team will focus on this model as a possible design solution because the hydraulics improved. This model also takes into account debris blockage and pier shape.

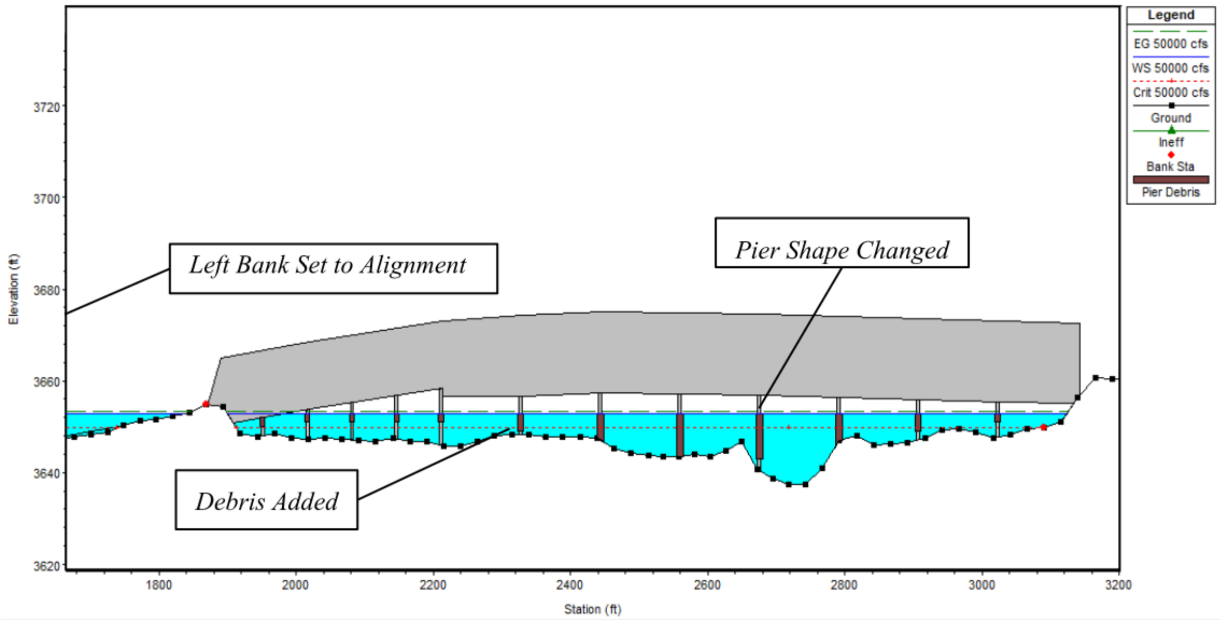

Figure 3.6: Bridge with Left Bank Set to Proposed Highway Alignment

3.1.7 Highway Relocation without Fence Model

The team now knows it is important to set the left bank to the proposed highway alignment (2.7 miles levee along the left bank). The 4,000 foot high levee will allow the bridge to overtop and keep water from pooling up on the left side into the town. Therefore, the team modeled the impacts of previous model only they took away the fence to see the impacts. The velocity increased but not as much as keeping the fence on (up 1.59 ft/s from base). However, the water surface elevation decreased 3.24 ft/s (Table 3.1, Appendix A.7). That being said, removing the fence is positively impacting the hydraulics. However, it is required by ADOT to keep the pedestrian fence intact so the next phase of this project could look at different hinge options in order to drop down the fence during a flooding event (Figure 3.7)

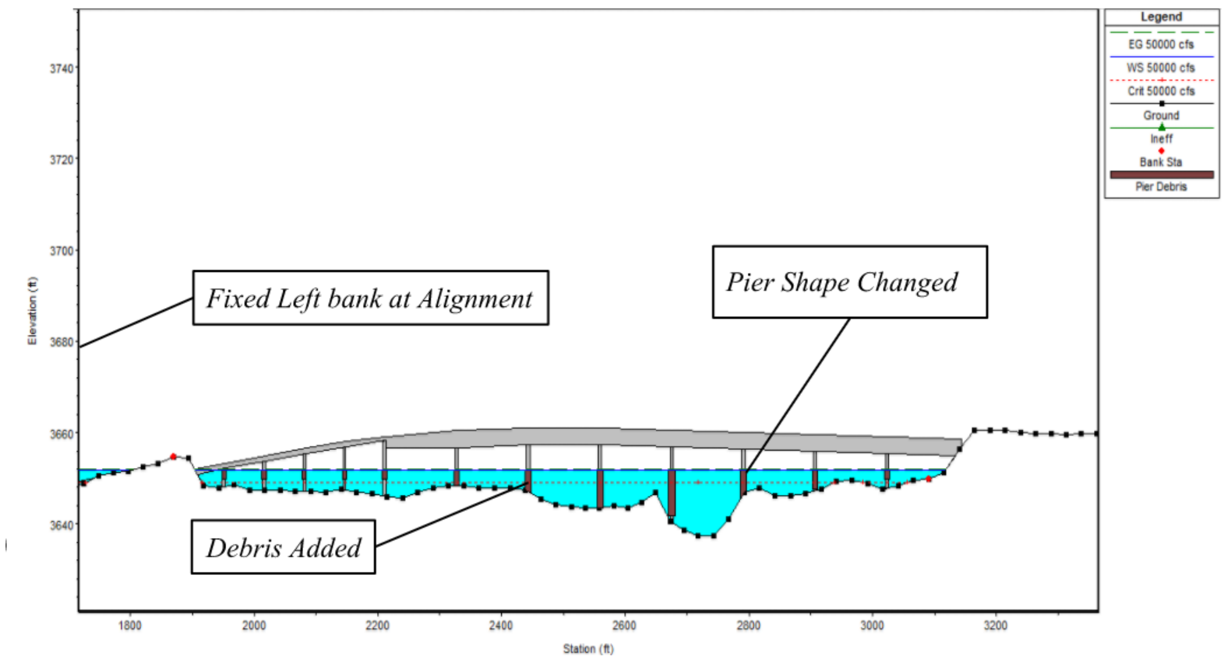

Figure 3.7:Left Bank Set to Proposed Highway Alignment and No Fence

3.1.8 Evaluation of HEC-RAS Alternatives

The following table organized all of the results in a table to see which model changed in hydraulics (compared to the 2016 base model). Table 3.1 shows that the highway alignment models (bottom two models) lowered the water surface elevation and increased the velocity flowing over the bridge. An increased velocity at the bridge and lowering the water surface elevation is desired. However, in the desired (bottom two) models the Froude number increased from 0.13 (base model) to 0.23 at the bridge. This is concerning for the next stage of the project because as the Froude number increases, the team needs to be aware of erosion control. This is because water that becomes too turbulent can wear away the bridge piers and river banks. All and all, the bottom two models (highlighted in red; Section 3.16-3.17) were selected for the final design in comparison to the other alternatives. These were chosen because they best met the goals and constraints of the project. This includes improving hydraulics during the 100 year flood and maintaining the shape of the channel. This model best meets economic constraints because the highway relocation can create the potential for ADOT funding.

Table 3.1: Bridge Hydraulics (Upstream)

	Bridge Hydraulics (Upstream)				
	Model	**Description**	**Δ Velocity (ft/s)**	**Δ W.S. Elevation (ft)**	**Impact (+ or -)**
Base	Corrective Effective	k=0, No Debris, Fence	N/A	N/A	N/A
1	Debris Only	k=0, Add Debris, Fence	1.59	-3.24	Good
2	Piers Only	K=0.9, No debris, Fence	0	0	Neutral
3	Improve K/ Fence	k=0.9, Add Debris, Fence	0.02	0.01	Neutral
4	Improve K/ noFence	k=0.9, Add Debris, No Fence	0.02	0.01	Neutral
5	Highway Relocation w/ Fence	k=0.9, Add Debris, Fence, Left Bank	2.73	-2	Good
6	Highway Relocation w/ out Fence	k=0.9, Add Debris, No Fence, Left Bank	1.59	-3.24	Good

3.2 FLO-2D Modeling and Levee Alternatives

The FLO-2D model from the previous capstone group was revised, removing the agricultural levee system that is currently in the existing model. These levees affect the flow distribution within the channel and overbank floodplains. Once the levee system was removed from the models, Ponderosa Designs then used the adapted model to simulate levee-based flood mitigation scenarios. The levee heights for these models were set to an elevation of 4000 feet above the existing surface to determine the depth of the flood water for each of the flooding events. The team will provide a recommendation for an appropriate new levee alignment and/or existing levee modification scenario.

3.2.1 Enhanced Bridge Levee

This proposed solution model a large levee that would be built along the bridge only. The purpose of this model is to determine how much water would accumulate upstream of the bridge, this information is then used in other levee models to help create a feasible model. A processed image of this alignment can be found in the Appendix. This model shows the elevation of the water depth in Duncan being approximately four to seven feet in depth with velocity of 3.5 feet per second. From this model the team concluded that this levee is necessary to help convey the flow, but more protection is required to prevent water from building up around the levee and flooding the town. A processed ARC-GIS image of critical depth can be found in Figure B.6 in the appendix.

3.2.2 Highway Alignment Working

This model is meant to simulate a raised roadway that parallels the railroad. This model does not have the bridge levee attached. The alignment is setback from the railroad to avoid right of way conflict along with giving additional right of way for the highway easement. This model simulated water crossing up and over the bridge on the lower section to increase how much water can pass through the bridge. A processed image of this model can be found under figure B.2 in the appendix.

3.2.3 Reinforced Existing Working

For this model all previous existing levees were reinforced, raising the height dramatically. This increased the water depths significantly, which translated to a levee height that is not feasible. A processed image can be found in appendix B.3

3.2.4 Levee/Highway Alignment

This model is a combination of Enhanced Bridge Levee and the Highway alignment. This model would be a redesign of the Highway 70 and would incorporate the existing levee along the Highway 75 Bridge. This model uses a levee that would be 2.75 miles in length with a maximum levee height of 8.5 feet above existing ground elevation at the bridge. A processed image can be found in appendix B.5

3.2.5 Evaluation of FLO-2D Alternatives

After evaluating the different alternatives Ponderosa Designs Selected the Levee/Highway Alignment as the final solution. Table 3.2 depicts the criteria and constraints that were considered in determining the final solution.

Table 3.2: Decision matrix for selecting the FLO-2D model

	3.2.2 Highway Alignment Working	**3.2.3 Reniforced Existing Working:**	**3.2.4 Levee /Highway Alignment**
Economically Achievable	no	no	yes
Positive Ecological Impacts	yes	yes	yes
Positive Social Impacts	no	yes	no
Protection from 100 Year Flood	yes	no	yes

4.0 Final Design

The final design that was chosen for this project was a combination of the most effective HEC-RAS and FLO-2D models. As for HEC-RAS data, the “Highway Relocation without Fence” model was chosen and imported into FLO-2D. This alternative was chosen due to water surface elevation drop as well as simulated a realistic scenario of hydraulics at the bridge, including the debris blockage and corrected pier shape. In addition, the fence that was situated along the bridge was removed so it could allow the water to overtop. The water surface elevation was around 5 feet with and extra 3 feet of freeboard, giving the design a total height of 8.5 feet at its maximum elevation near the bridge. The QGIS model, Levee/Highway Alignment was imported into the same FLO-2D model to create the final design. This will be a combination of improving hydraulics at the Highway 75 Bridge and creating a raised levee highway alignment. This will be a levee that the Highway 70 will move over. All in all, this highway/levee alignment is 8.5 feet above existing ground surface, has a width of Approximately 101 feet, and a length of 2.75 miles seen in Figure 4.0. This flood mitigation alternative could help with protecting the town against 100 year flood event as well as have the potential for ADOT funding. The dimensions and characteristics of the raised levee highway are described in Figure 4.1-4.2.

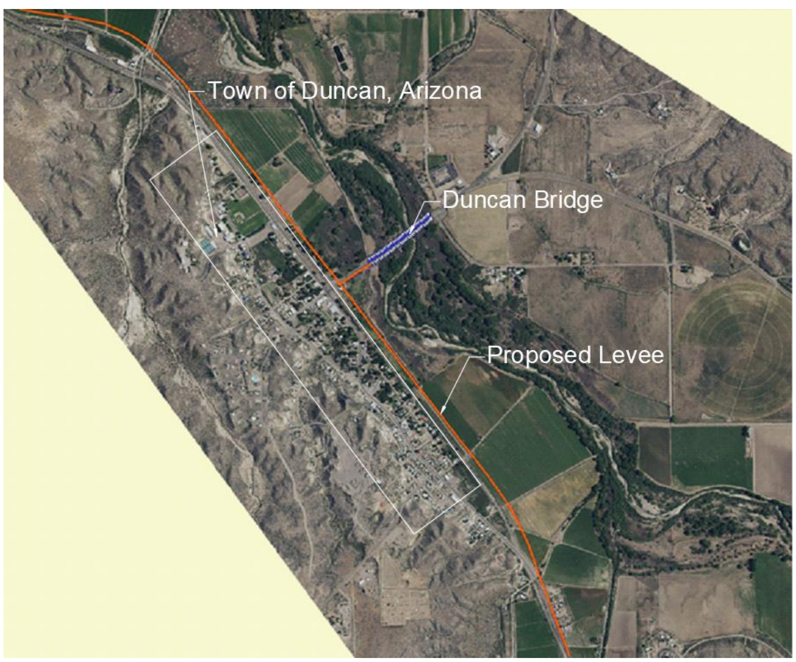

Figure 4.0: Proposed Solution: Highway Alignment

The proposed cross section of the raised highway levee was designed in accordance to ADOT specifications and regulations (figure 4.1). The roadway will be a two lane roadway with one lane in each direction. The lane widths were selected to be 12 feet, a minimum shoulder of 6 feet was utilized, and the 4:1 side slope was selected to ensure stability [9].

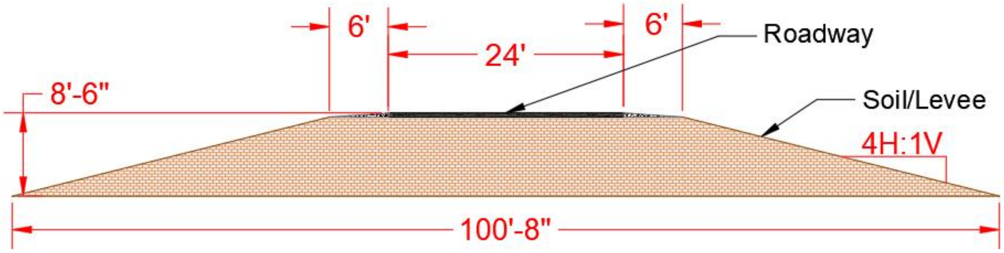

Figure 4.1: AutoCAD Representation of Roadway Cross Section

The anticipated offset of the proposed highway/levee alignments from the existing railroad system shown in figure 4.2 According to Arizona Department of Transportation (ADOT) regulations, a minimum right of way of 50 feet is required for the railroad. ADOT requires an additional amount of right of way regarding fill slopes, per ADOT the toe of the fill slope must not be located within 10 feet of the actual railroad right of way. Referring to the location along the highway/levee alignment, the offset will range from 60 feet to 94 feet from railroad to edge of the fill slope toe [9].

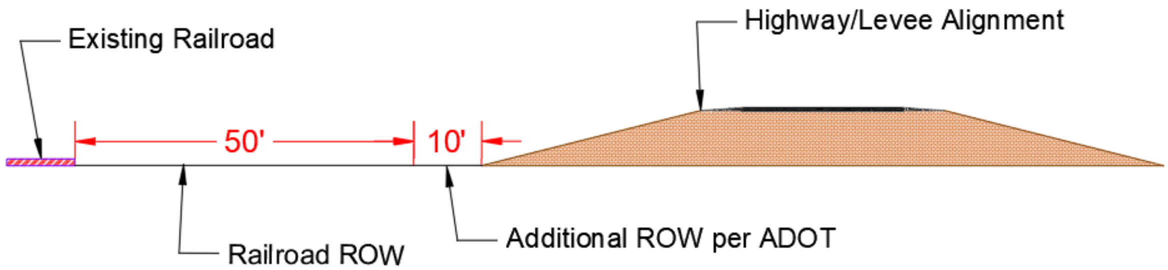

Figure 4.2: AutoCAD representation of ROW Offset from Railroad to Roadway

5.0 Cost of Implement the Design

The construction of the 2.75 miles of elevated roadway levee will cost around $13.7 million, with about $21,000 a year in maintenance fees [10-16]. This estimate was determined by taking past roadway cost estimates and using them to determine a cost of the Duncan project. The costs of purchasing the land around the proposed highway was determined using the cost per acre in the town of Duncan and the cost of fill. The cost of fill was determined by using the necessary roadway height and the current ground elevation. The construction management costs was simply a percentage of the total cost of implementing the design.

Table 5.1: Design Costs

Component	Quantity		Unit Cost		Total Cost
Highway	2.75	miles	$2,500,000	per mile	$6,875,000
Land Acquisition	45.1	Acre	$5,755	per Acre	$259,551
Erosion Control	247,197	ft^2	$3	per ft^2	$741,591
Levee Cost					$4,308,124
Staffing					$65,917
Construction Management					$1,523,033
Total Cost					$13,773,216

6.0 Project Management

6.1 Schedule

There were no changes to the schedule as the dates were meant to expanse the entire semester. As predicted, the modeling took the allotted time because it took many iterations and trials and was accounted for in the project schedule (Table 6.1)

Table 6.1: Project Schedule

Task	Start Date	End Date
1.0 Modeling Parameters		
1.1 Learning Softwares	8/29/2017	11/17/2017
1.2 Bridge/Levee Alternatives	9/12/2017	9/12/2017
1.3 Regulation Research	3/2/2016	12/5/2017
2.0 Modeling		
2.1 HEC-RAS	9/8/2017	11/17/2017
2.2 Q GIS	9/8/2017	11/17/2017
2.3 FLO 2D	10/4/2017	11/17/2017
3.0 Modeling Analysis		
3.1 Recommended Solutions	9/14/2017	11/29/2017
3.2 Impacts	11/30/2017	12/6/2017
3.3 Cost Analysis	11/30/2017	12/6/2017
4.0 Data Collection		
4.1 Site Visit (Travel)	3/3/2017	3/4/2017
4.2 Public Forum	3/4/2017	3/4/2017
4.3 Explore Data Files	5/8/2017	11/17/2017
5.0 Project Management		
5.1 Coordination	1/25/2016	12/15/2017
5.2 50% Report	10/1/2017	10/16/2017
5.3 Impacts Report	11/1/2017	11/13/2017
5.4 Final Proposal	12/12/2017	12/12/2017

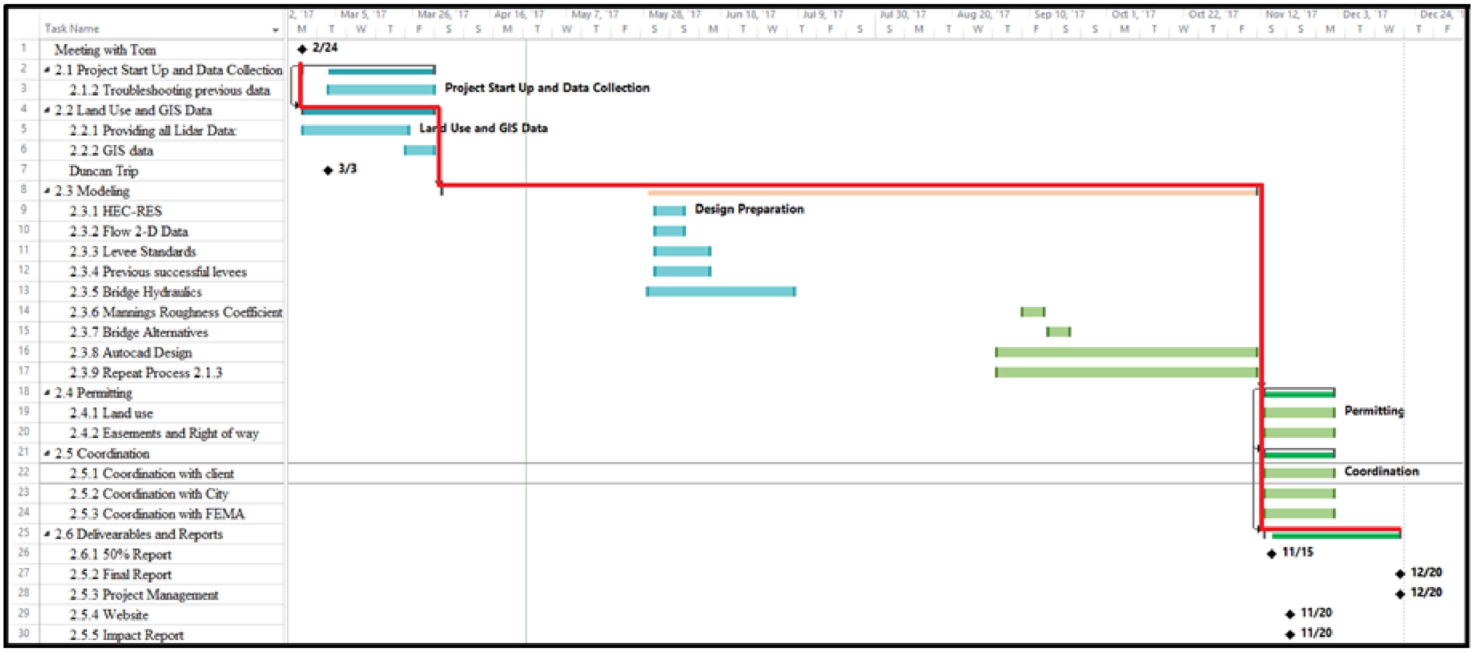

Figure 5.1: Gantt Chart

6.2 Staffing Cost

The costs of the different positions were determined using the billing rates from past proposals. The overall staffing expenses went down from the predicted costs (Table 6.2-6.3). This was primarily due to the team over estimating the overall time it would take for each part of the design due to not having any experience with these programs. The hourly costs for each of the individuals stayed the same but the hours varied based on the different programs that were being run. The actual hours cumulated to over 700 which resulted in a staffing cost of $64,807.

Table 6.2: Staffing Cost Proposed

Classification	Billing Rate ($)/hr	Projected Hours	Total Projected Cost ($)
Senior Engineer	130	189.5	24635
Project Engineer 1	90	301.5	27135
EIT	75	259	19425
Intern	25	184	4600
Total Staff Cost		934	75795

Table 6.3: Actual Staffing Costs

Classification	Billing Rate ($)/hr	Actual Hours	Total Actual Cost ($)
Senior Engineer	130	207.5	26975
Project Engineer 1	90	195.5	17595
EIT	75	217	16275
Intern	25	158.5	3962.5
Total Staff Cost		778.5	64807.5

7.0 Impacts

Ponderosa Designs conducted an impact analysis based upon the final solution. The team will analyze the impacts of the project on the three major categories; environmental, economical, and social. The impacts reports will determine if the suggested final solution promotes the general living conditions set forth by the existing conditions of Duncan, Arizona.

7.1 Environmental Impacts

The proposed design could impact the migratory bird watching, local vegetation, and river flow characteristics. The bird watching might be impacted in a due to changes in the vegetation as the river is allowed to run its own course. The local vegetation will be impacted, because the design will allow the river to flood which may cause erosion and uprooting of local plants. The river flow characteristics could be impacted positively, by allowing the river to take a more natural course of action. This would impact the slopes of the river and the channel width.

7.2 Economic Impacts

This design has an estimated cost of 13.8 million dollars with 21 thousand per year in maintenance costs. The cost of implementation will eventually reflect a positive impact, because of its ability to reduce costs of damages during a flood event. This impact was determined by comparing the 13.8 million dollar implementation cost against the damage cost of 35.5 million dollars for the 1978 flood event. The design will also impact the economics of the town in a positive manner by allowing the traffic to bypass the local businesses.

7.3 Social Impacts

The design will impact the livelihood of the community due to the required land that must be acquired from surrounding landowners. The project will require an estimated amount of 45.1 acres along the Gila River and could affect the property owners. Much of the land that would have to be purchased is farmland, which is much of the identity of Duncan and could make this design not politically feasible.

8.0 Next Steps

The next steps involved with this project would include the refinement of the modeling applications and incorporation of a multidisciplinary team. The team would be focused on roadway geometrics, profile, and analysis for erosion control. This would require a Highway 75 intersection redesign right at the bridge, and transitional design to prevent backwatering on the northern end of the highway/levee design. The following team would need to design these roadway and levee structures with respect to the codes that are set in place as well as the constraints that come with building between a railroad and a river.

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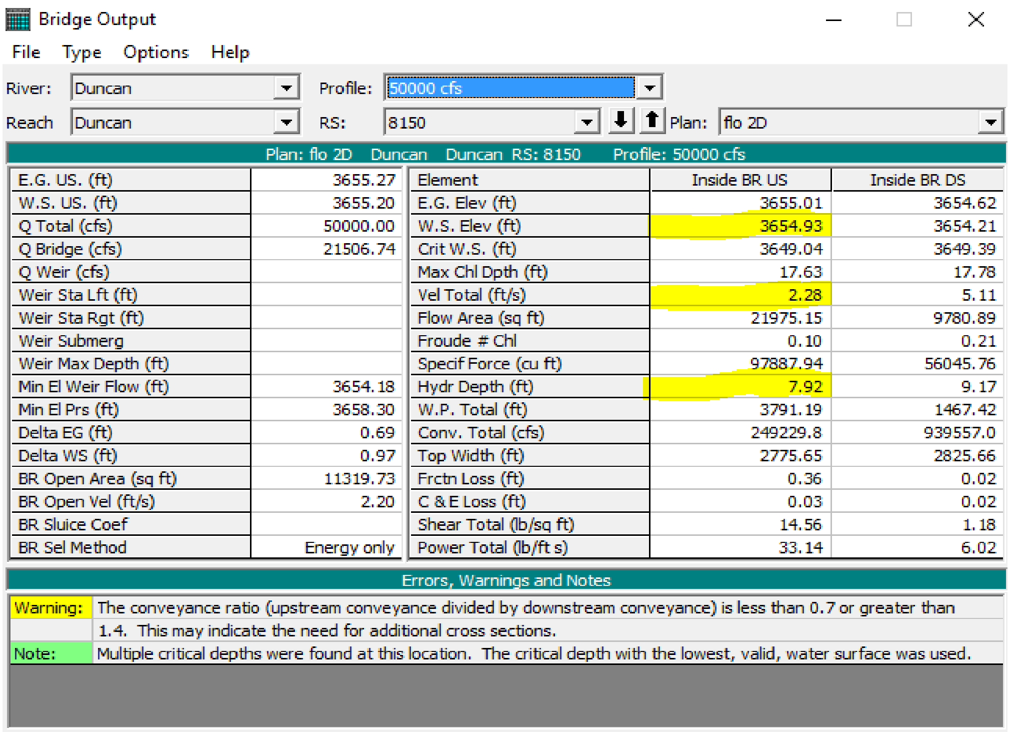
Appendix

Appendix A. HEC RAS Results

A.1 Hydraulic Results of Corrective Effective Model


Bridge Output

File Type Options Help

River: Duncan Profile: 50000 cfs

Reach: Duncan RS: 8150 Plan: flo 2D

Plan: flo 2D Duncan Duncan RS: 8150 Profile: 50000 cfs

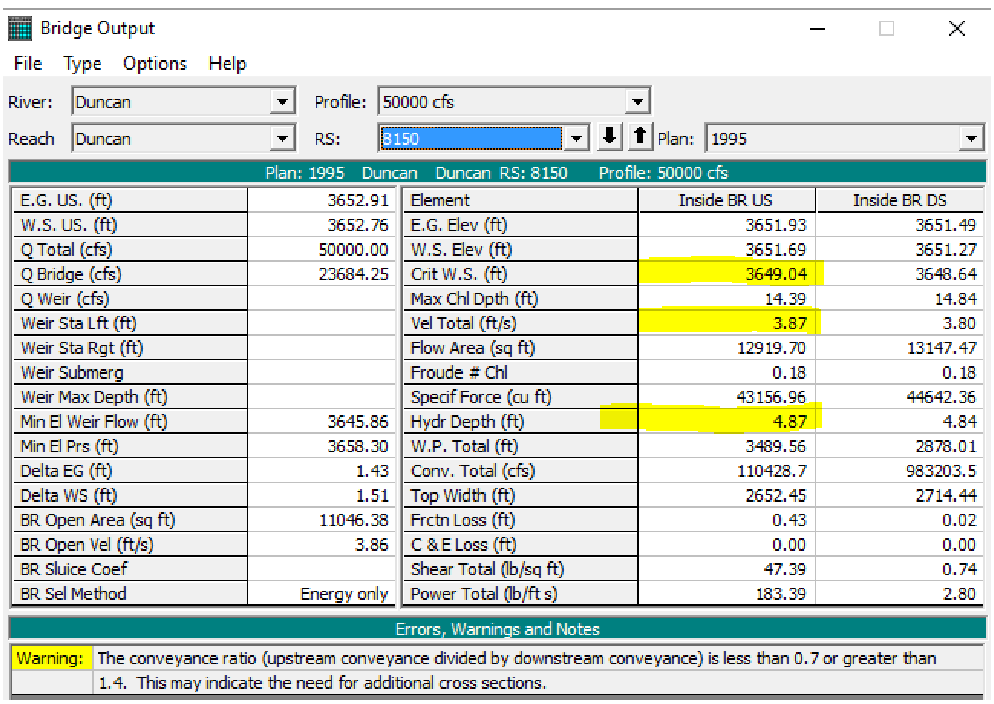
		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	3655.27	E.G. Elev (ft)	3655.01	3654.62
W.S. US. (ft)	3655.20	W.S. Elev (ft)	3654.93	3654.21
Q Total (cfs)	50000.00	Crit W.S. (ft)	3649.04	3649.39
Q Bridge (cfs)	21506.74	Max Chl Dpth (ft)	17.63	17.78
Q Weir (cfs)		Vel Total (ft/s)	2.28	5.11
Weir Sta Lft (ft)		Flow Area (sq ft)	21975.15	9780.89
Weir Sta Rgt (ft)		Froude # Chl	0.10	0.21
Weir Submerg		Specif Force (cu ft)	97887.94	56045.76
Weir Max Depth (ft)		Hydr Depth (ft)	7.92	9.17
Min El Weir Flow (ft)	3654.18	W.P. Total (ft)	3791.19	1467.42
Min El Prs (ft)	3658.30	Conv. Total (cfs)	249229.8	939557.0
Delta EG (ft)	0.69	Top Width (ft)	2775.65	2825.66
Delta WS (ft)	0.97	Frctn Loss (ft)	0.36	0.02
BR Open Area (sq ft)	11319.73	C & E Loss (ft)	0.03	0.02
BR Open Vel (ft/s)	2.20	Shear Total (lb/sq ft)	14.56	1.18
BR Sluice Coef		Power Total (lb/ft s)	33.14	6.02
BR Sel Method	Energy only			

Errors, Warnings and Notes

Warning: The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.

Note: Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.

A.2 Hydraulic Results of Debris Only Model


Bridge Output

File Type Options Help

River: Duncan Profile: 50000 cfs

Reach: Duncan RS: 8150 Plan: 1995

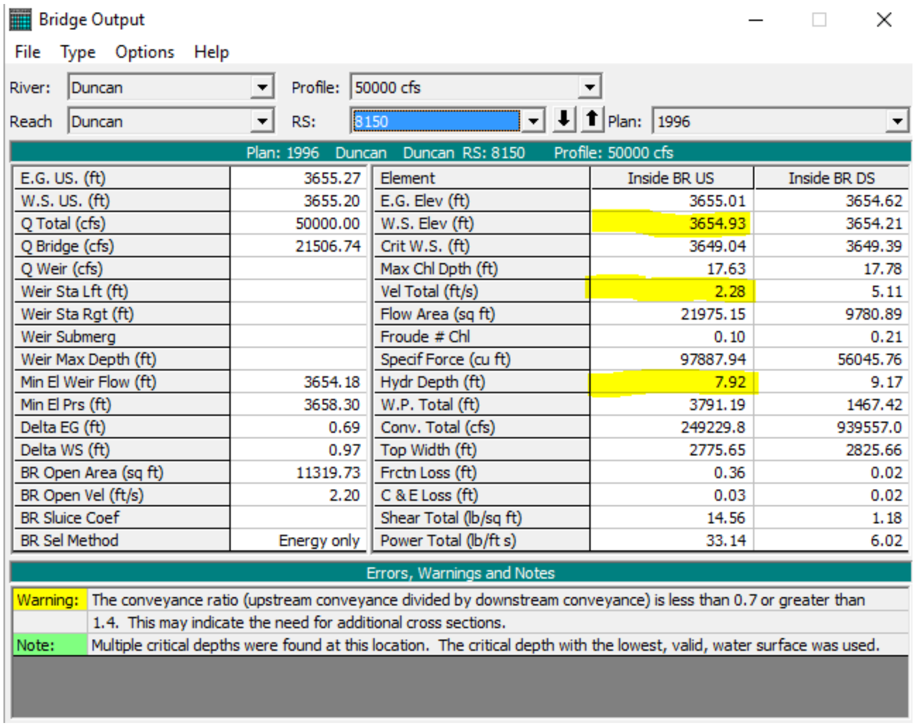
Plan: 1995 Duncan Duncan RS: 8150 Profile: 50000 cfs

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	3652.91	E.G. Elev (ft)	3651.93	3651.49
W.S. US. (ft)	3652.76	W.S. Elev (ft)	3651.69	3651.27
Q Total (cfs)	50000.00	Crit W.S. (ft)	3649.04	3648.64
Q Bridge (cfs)	23684.25	Max Chl Dpth (ft)	14.39	14.84
Q Weir (cfs)		Vel Total (ft/s)	3.87	3.80
Weir Sta Lft (ft)		Flow Area (sq ft)	12919.70	13147.47
Weir Sta Rgt (ft)		Froude # Chl	0.18	0.18
Weir Submerg		Specif Force (cu ft)	43156.96	44642.36
Weir Max Depth (ft)		Hydr Depth (ft)	4.87	4.84
Min El Weir Flow (ft)	3645.86	W.P. Total (ft)	3489.56	2878.01
Min El Prs (ft)	3658.30	Conv. Total (cfs)	110428.7	983203.5
Delta EG (ft)	1.43	Top Width (ft)	2652.45	2714.44
Delta WS (ft)	1.51	Frctn Loss (ft)	0.43	0.02
BR Open Area (sq ft)	11046.38	C & E Loss (ft)	0.00	0.00
BR Open Vel (ft/s)	3.86	Shear Total (lb/sq ft)	47.39	0.74
BR Sluice Coef		Power Total (lb/ft s)	183.39	2.80
BR Sel Method	Energy only			

Errors, Warnings and Notes

Warning: The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.

A.3 Hydraulic Results of Pier Only Model



Bridge Output

File Type Options Help

River: Duncan Profile: 50000 cfs

Reach Duncan RS: 8150 Plan: 1996

Plan: 1996 Duncan Duncan RS: 8150 Profile: 50000 cfs

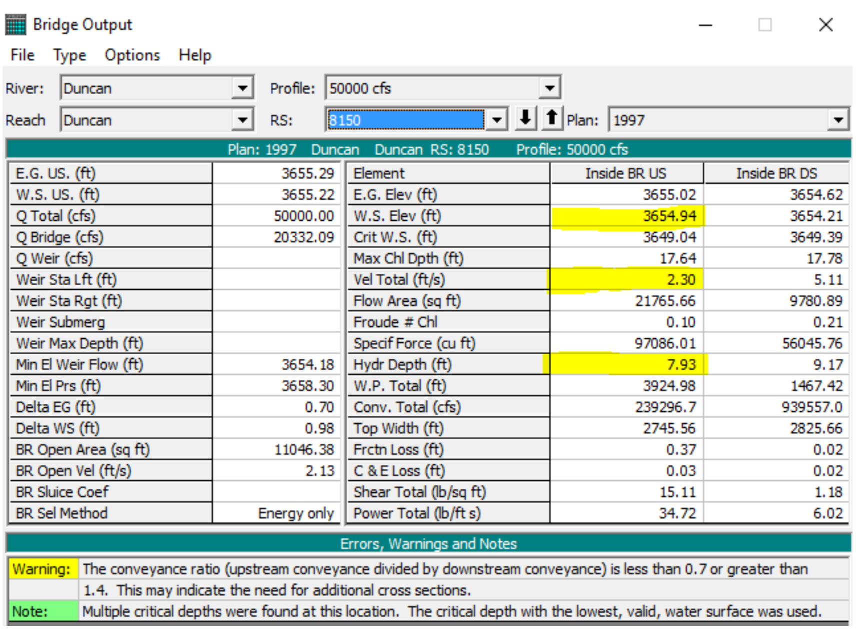
		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	3655.27	E.G. Elev (ft)	3655.01	3654.62
W.S. US. (ft)	3655.20	W.S. Elev (ft)	3654.93	3654.21
Q Total (cfs)	50000.00	Crit W.S. (ft)	3649.04	3649.39
Q Bridge (cfs)	21506.74	Max Chl Dpth (ft)	17.63	17.78
Q Weir (cfs)		Vel Total (ft/s)	2.28	5.11
Weir Sta Lft (ft)		Flow Area (sq ft)	21975.15	9780.89
Weir Sta Rgt (ft)		Froude # Chl	0.10	0.21
Weir Submerg		Specif Force (cu ft)	97887.94	56045.76
Weir Max Depth (ft)		Hydr Depth (ft)	7.92	9.17
Min El Weir Flow (ft)	3654.18	W.P. Total (ft)	3791.19	1467.42
Min El Prs (ft)	3658.30	Conv. Total (cfs)	249229.8	939557.0
Delta EG (ft)	0.69	Top Width (ft)	2775.65	2825.66
Delta WS (ft)	0.97	Frctn Loss (ft)	0.36	0.02
BR Open Area (sq ft)	11319.73	C & E Loss (ft)	0.03	0.02
BR Open Vel (ft/s)	2.20	Shear Total (lb/sq ft)	14.56	1.18
BR Sluice Coef		Power Total (lb/ft s)	33.14	6.02
BR Sel Method	Energy only			

Errors, Warnings and Notes

Warning: The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.

Note: Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.

A.4 Pier Coefficient/ Debris Blockage/Fence In Place Model



Bridge Output

File Type Options Help

River: Duncan Profile: 50000 cfs

Reach Duncan RS: 8150 Plan: 1997

Plan: 1997 Duncan Duncan RS: 8150 Profile: 50000 cfs

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	3655.29	E.G. Elev (ft)	3655.02	3654.62
W.S. US. (ft)	3655.22	W.S. Elev (ft)	3654.94	3654.21
Q Total (cfs)	50000.00	Crit W.S. (ft)	3649.04	3649.39
Q Bridge (cfs)	20332.09	Max Chl Dpth (ft)	17.64	17.78
Q Weir (cfs)		Vel Total (ft/s)	2.30	5.11
Weir Sta Lft (ft)		Flow Area (sq ft)	21765.66	9780.89
Weir Sta Rgt (ft)		Froude # Chl	0.10	0.21
Weir Submerg		Specif Force (cu ft)	97086.01	56045.76
Weir Max Depth (ft)		Hydr Depth (ft)	7.93	9.17
Min El Weir Flow (ft)	3654.18	W.P. Total (ft)	3924.98	1467.42
Min El Prs (ft)	3658.30	Conv. Total (cfs)	239296.7	939557.0
Delta EG (ft)	0.70	Top Width (ft)	2745.56	2825.66
Delta WS (ft)	0.98	Frctn Loss (ft)	0.37	0.02
BR Open Area (sq ft)	11046.38	C & E Loss (ft)	0.03	0.02
BR Open Vel (ft/s)	2.13	Shear Total (lb/sq ft)	15.11	1.18
BR Sluice Coef		Power Total (lb/ft s)	34.72	6.02
BR Sel Method	Energy only			

Errors, Warnings and Notes

Warning: The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.

Note: Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.

A.5 Pier Coefficient/ Debris Blockage/ No Fence Model

Bridge Output

File Type Options Help

River: Duncan Profile: 50000 cfs

Reach Duncan RS: 8150 Plan: 1998

Plan: 1998 Duncan Duncan RS: 8150 Profile: 50000 cfs

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	3655.29	E.G. Elev (ft)	3655.02	3654.62
W.S. US. (ft)	3655.22	W.S. Elev (ft)	3654.94	3654.21
Q Total (cfs)	50000.00	Crit W.S. (ft)	3649.04	3649.39
Q Bridge (cfs)	20332.09	Max Chl Dpth (ft)	17.64	17.78
Q Weir (cfs)		Vel Total (ft/s)	2.30	5.11
Weir Sta Lft (ft)		Flow Area (sq ft)	21765.66	9780.89
Weir Sta Rgt (ft)		Froude # Chl	0.10	0.21
Weir Submerg		Specif Force (cu ft)	97086.01	56045.76
Weir Max Depth (ft)		Hydr Depth (ft)	7.93	9.17
Min El Weir Flow (ft)	3654.18	W.P. Total (ft)	3924.98	1467.42
Min El Prs (ft)	3658.30	Conv. Total (cfs)	239296.7	939557.0
Delta EG (ft)	0.70	Top Width (ft)	2745.56	2825.66
Delta WS (ft)	0.98	Frctn Loss (ft)	0.37	0.02
BR Open Area (sq ft)	11046.38	C & E Loss (ft)	0.03	0.02
BR Open Vel (ft/s)	2.13	Shear Total (lb/sq ft)	15.11	1.18
BR Sluice Coef		Power Total (lb/ft s)	34.72	6.02
BR Sel Method	Energy only			

Errors, Warnings and Notes

Warning:	The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.

A.6 Results of Highway Relocation with Fence Model

Bridge Output

File Type Options Help

River: Duncan Profile: 50000 cfs

Reach Duncan RS: 8150 Plan: 2001

Plan: 2001 Duncan Duncan RS: 8150 Profile: 50000 cfs

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	3654.54	E.G. Elev (ft)	3653.34	3652.81
W.S. US. (ft)	3654.31	W.S. Elev (ft)	3652.94	3652.47
Q Total (cfs)	50000.00	Crit W.S. (ft)	3649.73	3648.93
Q Bridge (cfs)	37519.19	Max Chl Dpth (ft)	15.64	16.04
Q Weir (cfs)		Vel Total (ft/s)	5.01	4.68
Weir Sta Lft (ft)		Flow Area (sq ft)	9976.59	10684.31
Weir Sta Rgt (ft)		Froude # Chl	0.22	0.21
Weir Submerg		Specif Force (cu ft)	44998.42	48281.75
Weir Max Depth (ft)		Hydr Depth (ft)	6.40	6.46
Min El Weir Flow (ft)	3646.28	W.P. Total (ft)	2316.92	1912.72
Min El Prs (ft)	3658.30	Conv. Total (cfs)	94286.7	910448.2
Delta EG (ft)	1.76	Top Width (ft)	2707.24	2789.31
Delta WS (ft)	1.85	Frctn Loss (ft)	0.51	0.02
BR Open Area (sq ft)	11026.98	C & E Loss (ft)	0.02	0.01
BR Open Vel (ft/s)	5.02	Shear Total (lb/sq ft)	75.60	1.05
BR Sluice Coef		Power Total (lb/ft s)	378.87	4.92
BR Sel Method	Energy only			

Errors, Warnings and Notes

Warning:	The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.

A.7 Results of Highway Relocation without Fence Model

Bridge Output

File Type Options Help

River: Duncan Profile: 50000 cfs

Reach: Duncan RS: 8150 Plan: 2000

Plan: 2000 Duncan Duncan RS: 8150 Profile: 50000 cfs

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	3652.91	E.G. Elev (ft)	3651.93	3651.49
W.S. US. (ft)	3652.76	W.S. Elev (ft)	3651.69	3651.27
Q Total (cfs)	50000.00	Crit W.S. (ft)	3649.04	3648.64
Q Bridge (cfs)	23684.25	Max Chl Dpth (ft)	14.39	14.84
Q Weir (cfs)		Vel Total (ft/s)	3.87	3.80
Weir Sta Lft (ft)		Flow Area (sq ft)	12919.70	13147.47
Weir Sta Rgt (ft)		Froude # Chl	0.18	0.18
Weir Submerg		Specif Force (cu ft)	43156.96	44642.36
Weir Max Depth (ft)		Hydr Depth (ft)	4.87	4.84
Min El Weir Flow (ft)	3645.86	W.P. Total (ft)	3489.56	2878.01
Min El Prs (ft)	3658.30	Conv. Total (cfs)	110428.7	983203.5
Delta EG (ft)	1.43	Top Width (ft)	2652.45	2714.44
Delta WS (ft)	1.51	Frctn Loss (ft)	0.43	0.02
BR Open Area (sq ft)	11046.38	C & E Loss (ft)	0.00	0.00
BR Open Vel (ft/s)	3.86	Shear Total (lb/sq ft)	47.39	0.74
BR Sluice Coef		Power Total (lb/ft s)	183.39	2.80
BR Sel Method	Energy only			

Errors, Warnings and Notes

Warning:	The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.

Appendix B: FLO-2D Results

B.1: Highway Alignment with Bridge Levee

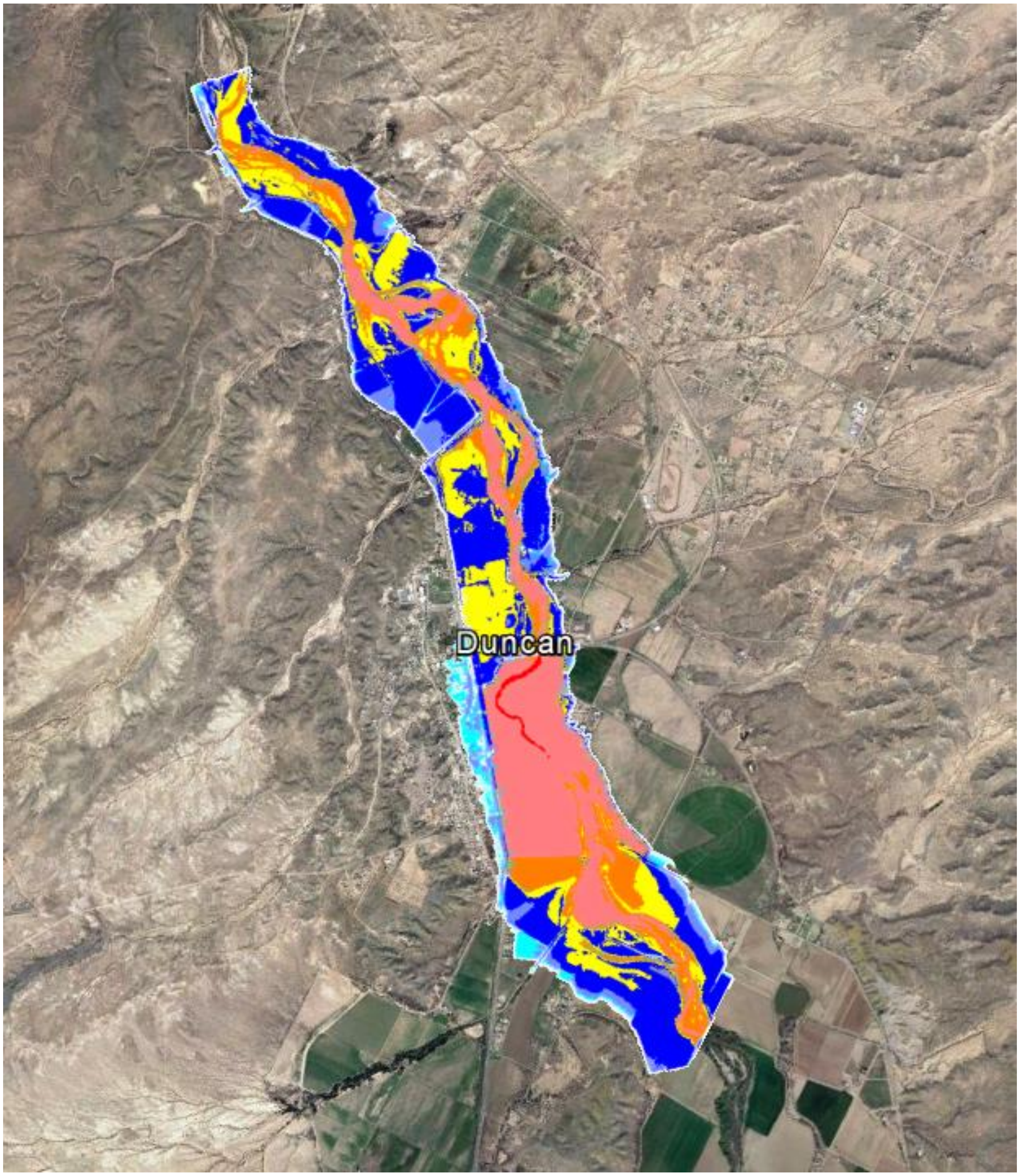


Figure B.1: Highway Alignment with Bridge Levee water depth results

B.2: Enhanced Bridge Levee

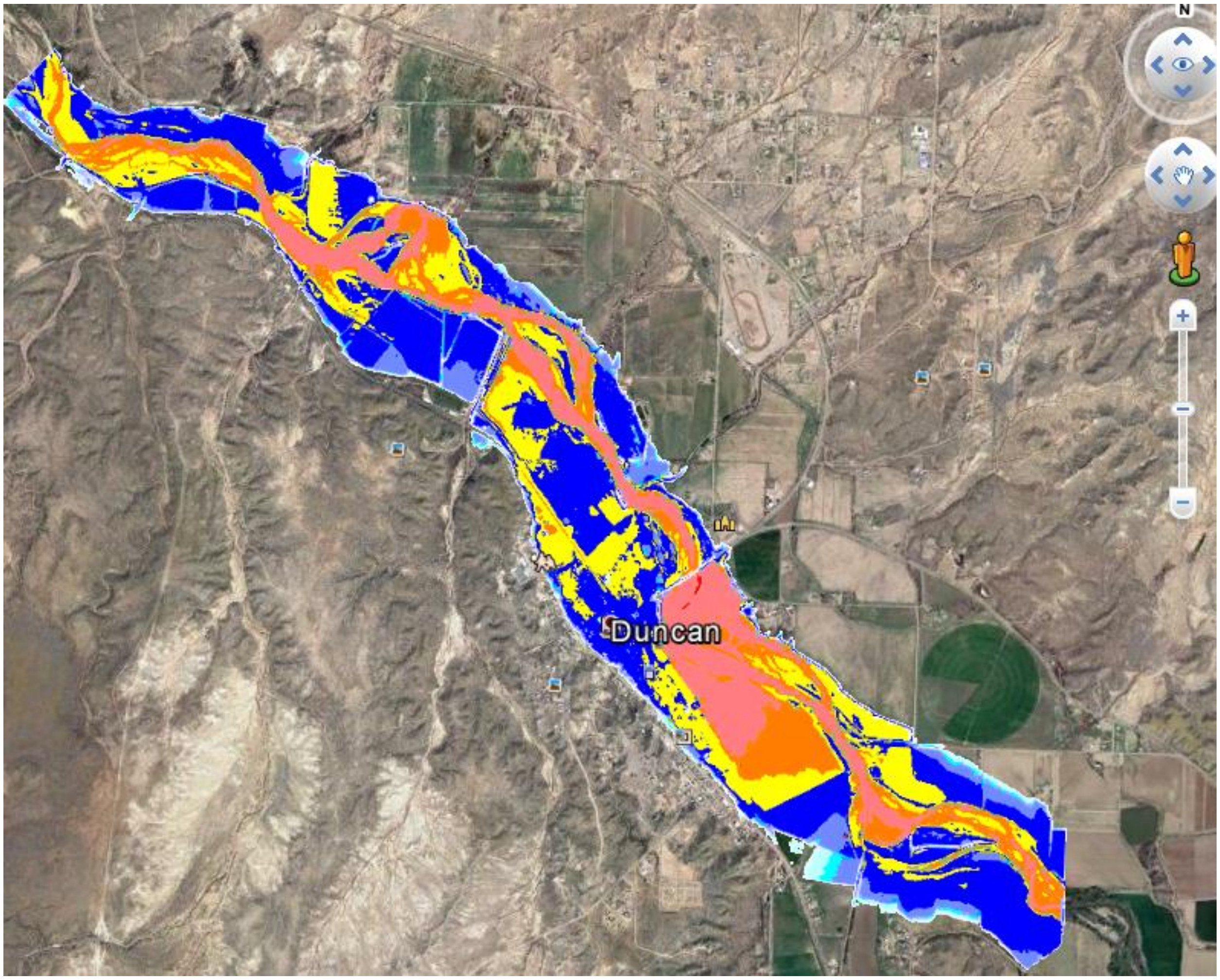


Figure B.2: Enhanced Bridge Levee, animated water depths results

B.3 Reinforced Existing Working

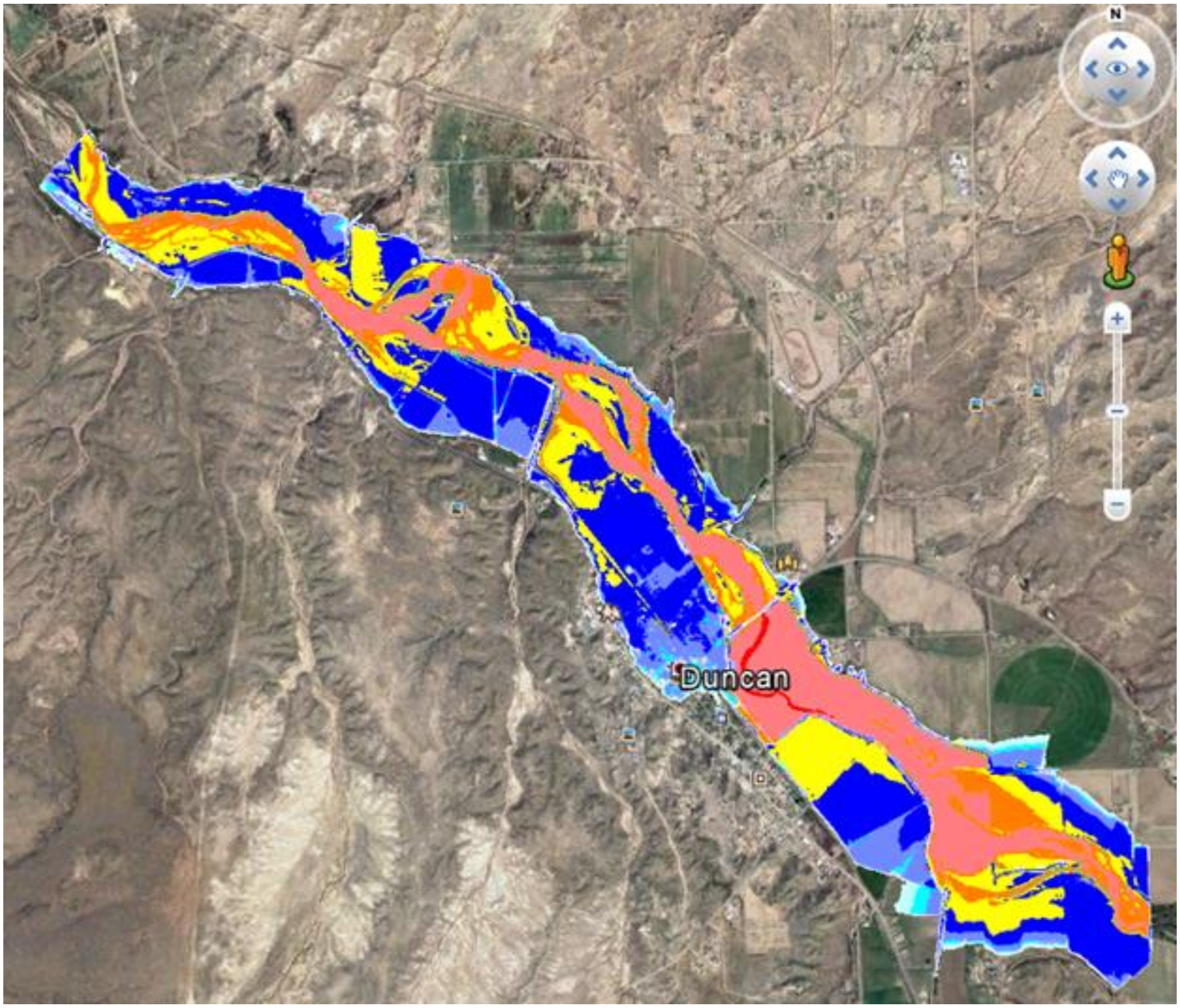


Figure B.3: Reinforce Existing Levees animated water depths results

B.4 QGIS Highway Alignment 100 year flood

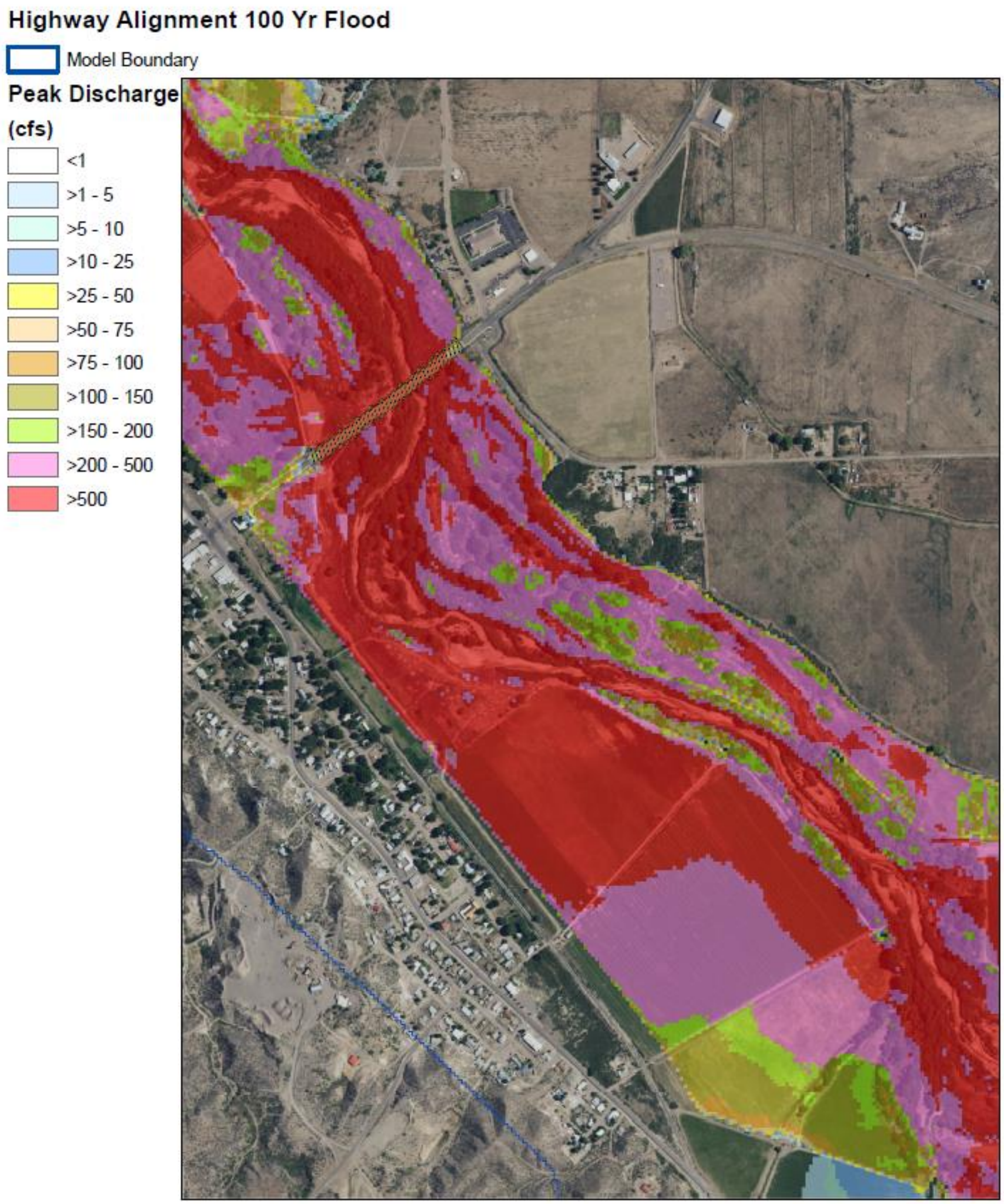


Figure B.4: Highway Alignment Peak Discharge

B.5 QGIS Highway Alignment Corrected Distance

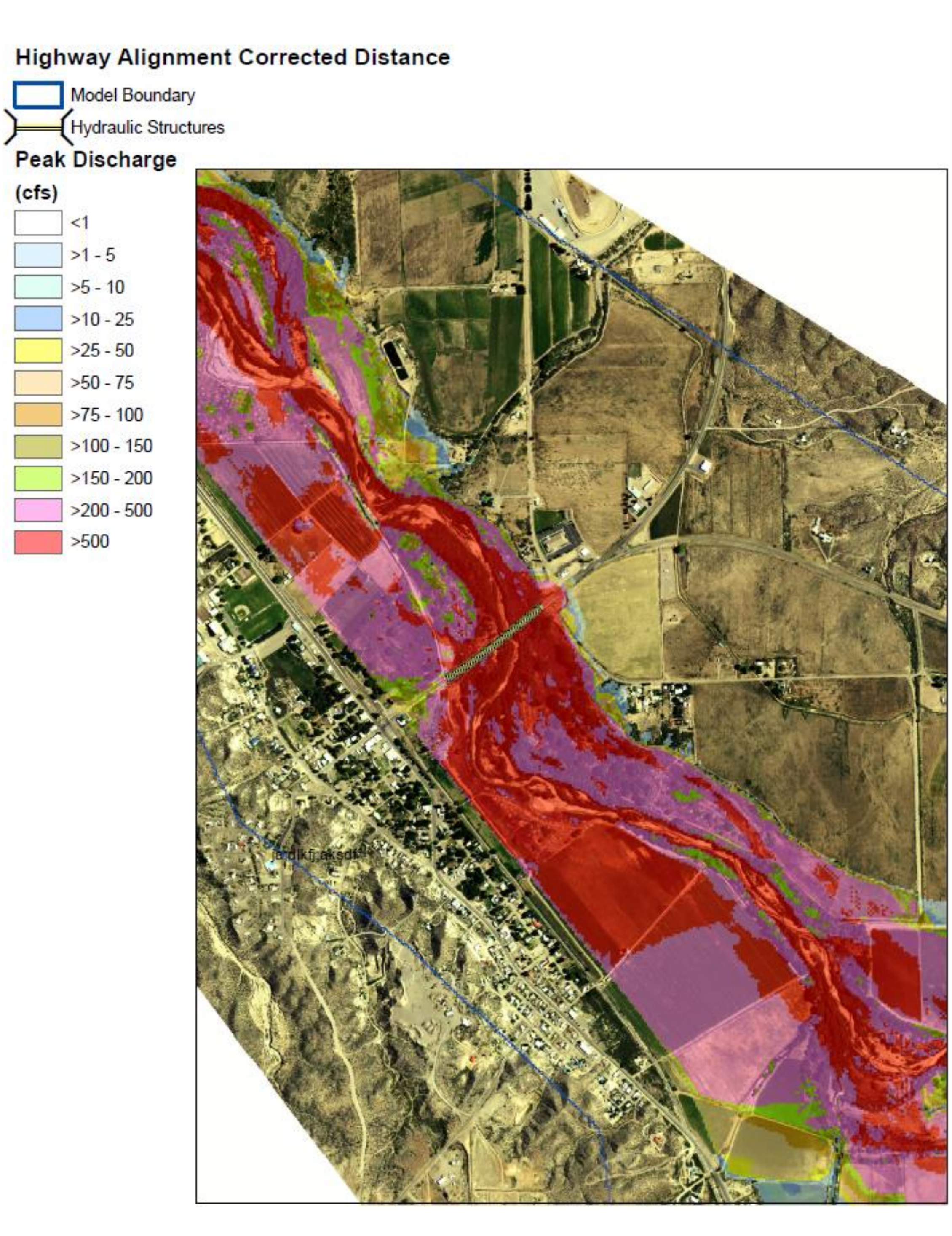


Figure B.5: Highway Alignment Corrected Distance Peak Discharge

B.6 QGIS Enhanced Bridge Levee

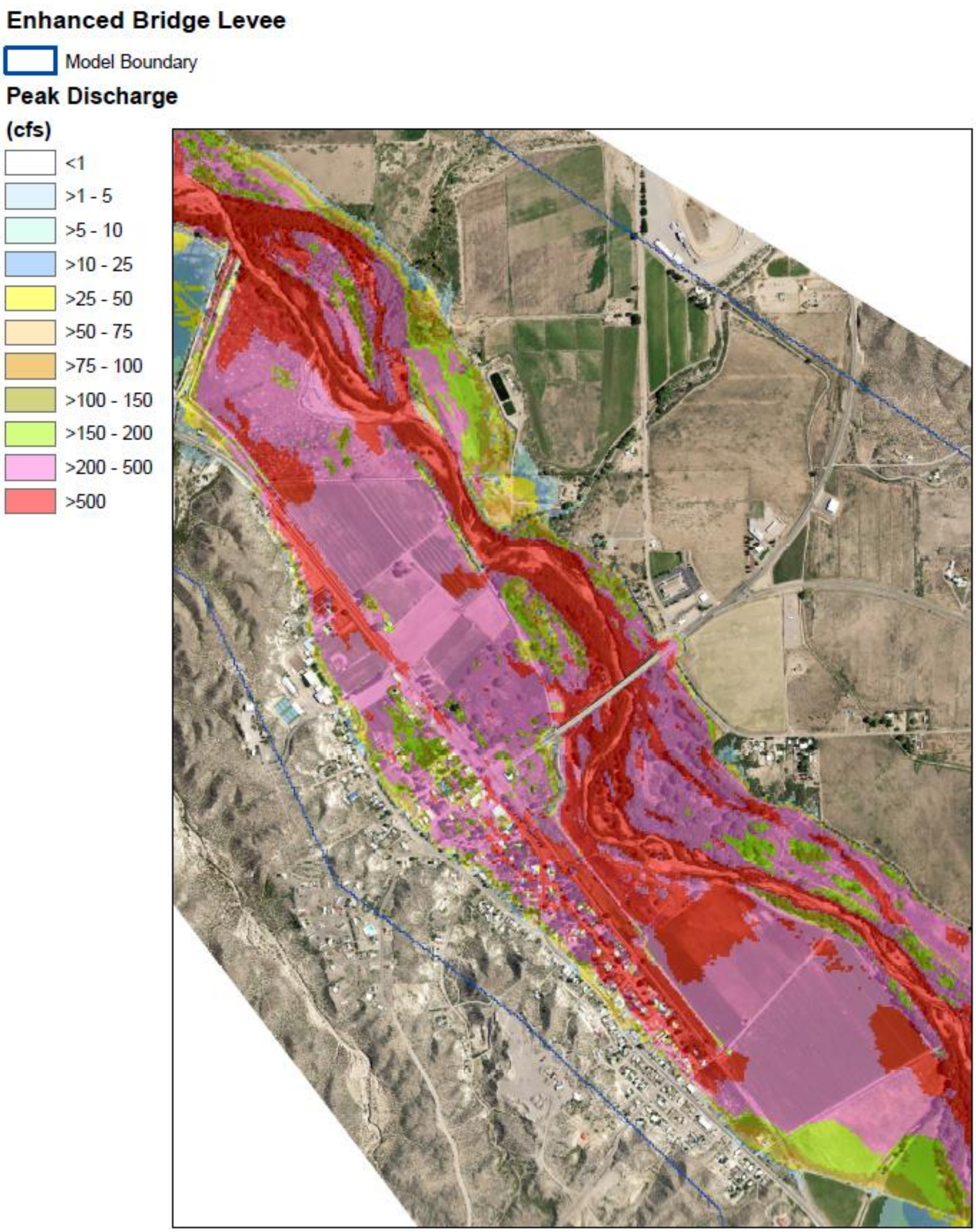


Figure B.6: Enhanced Bridge Levee