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Mount Elden Lookout Road – Shultz Creek Crossing Final Design Memorandum

1 of 7 pages + Attachments (40 pages)

DATE: May 8, 2023

TO: Nate Reisner, PE
Coconino County Public Works
5600 E Commerce Ave
Flagstaff, Arizona 86004

FROM: Michael Kearly, PE, CFM
Natural Channel Design, Inc

RE: Mount Elden Lookout Road - Schultz Creek Crossing
Flagstaff, Arizona

Registered Professional Engineer (Civil)
Certificate No. 34587
MICHAEL D KEARLY
Date Signed 5/8/2023
Arizona U.S.A.

Project Description

The Schultz Creek Crossing project of Mount Elden Lookout Road is an infrastructure upgrade that was triggered in response to increased flow from the Schultz Creek watershed as a result of the 2022 Pipeline Fire. This project will be part of a broader response to the post-fire increase in runoff from the watershed that includes the construction of large stormwater detention basins by the City of Flagstaff and watershed restoration by the Coconino County Flood Control District and US Forest Service. This project will include upgrading an existing 24 inch cmp culvert crossing with by installation of a three-barrel concrete box culvert, modification of the road profile and channel grading upstream and downstream of the road to transition the flow into and out of the new box culvert.

Project Location & Access

The project is located within Coconino County in area of northwest Flagstaff, approximately 3 miles north of downtown and just east of the intersection of Mount Elden Lookout Road and Shultz Pass Road. See Figure 1.

Access to the project site will be via Mount Elden Lookout Road, north from US Highway 180. Traffic control during construction and a temporary off-road diversion of traffic will be necessary to facilitate construction of the box culverts beneath the road.

The existing roadway at the drainage crossing is located within a Coconino County easement that crosses US Forest Service land. Construction will include disturbance within the actual roadway and within areas adjacent to the road both on USFS land to the north of the road and an adjacent parcel to the south side of the road. The adjacent parcel to the south is owned by the Museum of Northern Arizona and includes an 80 foot wide easement belonging to Kinder Morgan for high pressure, natural gas transmission mains.

Figure 1 - Project Site Overview

Concrete Box Culvert Design

LiDAR data dating from 2019 and obtained from Coconino County, supplemented with a ground-based topographic survey performed by the Coconino County Surveyor in January 2023 were used to create an existing Digital Terrain Model (DTM) as a base map for design.

Sizing of the box culvert was determined by Coconino County Public Works (CCPW) to ensure passage of what has become the default design storm for post-fire flood mitigation within the Pipeline Fire burn scar. The 2” in 45 minute storm event in the immediate post-burn condition has been used for much of the mitigation work implemented by the City of Flagstaff and Coconino County Flood Control District and will be used for this project. To pass the design storm without any overtopping the road, a triple barrel (2 - 4x8 + 1 - 5x8) concrete box culvert is required. Other design storms are also evaluated with this design to check for function and flooding extents during higher flows.

The invert of the center barrel of the box culvert crossing will be depressed one foot lower than the outside two barrels to provide for more shear during low flows to reduce sedimentation within the box culverts and thereby reduce maintenance needs.

On the north, upstream side of the road, a concrete headwall with tapered wing walls will be constructed. Grading will be conducted to intercept the native flow path as well as to intercept pipe and spillway discharge from the recently completed City of Flagstaff detention basins. This grading will include transitioning the native channel down to the new box culvert invert as well as roadside ditch modifications to intercept the outflow from the detention basins. Natural rock and rock riprap will be used for grade control on the north side of the road.

On the south side of the road, the box culvert will terminate with a straight-type headwall to avoid encroachment into the Kinder Morgan easement. Grading will include road shoulder reconstruction and channel widening to accommodate the width of the three-barrel box culvert and transitioning of that width back to the downstream natural geomorphic channel. Erosion control for the new downstream channel will be provided by an articulated concrete mat (ACM) as required by the Kinder Morgan Gas Company.

Road Reconstruction

The horizontal alignment of Mt Elden Lookout Road will remain unchanged. However, the vertical alignment will require adjustment in order to accommodate the new 4 foot high box culvert openings. The finished grade of the road at the box culverts will be approximately 8 inches higher in elevation that the existing road. The road profile will be adjusted by reconstruction of approximately 250 feet of the pavement and installation of vertical curves transitions in and out of the new road elevation.

To avoid significant changes to the road profile at the location where flood flows from Schutlz Creek currently overtops the pavement, vertical curves shorter than AASHTO standard will be used. Pavement will be replaced to match the existing pavement section or as otherwise recommended by geotechnical testing and design by Speedie and Associates, when that report becomes available.

Guardrail

Guardrail will be included across the extent of the concrete headwalls and extend beyond the crossing until embankment side slopes reduce to 3:1 or less. Guardrail is specified per ADOT standard drawings and project-specific details as needed.

A standard W-Beam guard rail with box culvert guardrail post per ADOT Detail C-10.07 is specified across the top of the box culvert. Beyond the ends of the box culvert headwall, standard w-beam guard rail (C-10.03) and Test Level 2 MSKT terminal end sections will be placed to the limits required per the road embankment slopes. Per current MASH guidelines, the low speed of Mt Elden Lookout Road (25 MPH speed limit) allows for the reduced length of a terminal end section as the TL-2 MSKT provides. This shorter length is necessary due to an otherwise conflicting driveway, west of the crossing.

Armoring/Erosion Control

Slope and channel armoring to prevent erosion will be included as part of this project. On the upstream, inlet side of the box culvert crossing, multiple sources of flow from different direction creates a complicated design scenario. Natural rock and rock riprap will be uses to stabilize slopes and channel bottoms. Rock and riprap have been sized with various calculation tools which are explained in a section of the attached appendices to this memo report.

On the downstream side of the box culverts, flow discharges immediately onto the gas line easement. Kinder Morgan only allows articulated concrete mats to be placed for erosion control across their easements to allow for future removal and replacement if maintenance of the gas line is required. “Submar” is the product required by Kinder Morgan to be placed across their easements. The Submar manufacturer considers the design specifications of their product and placement to be proprietary. Other than the approximate extent of their product being shown within the 90% plans, all aspects of that design and installation will be per the manufacturer.

Immediately downstream of the Kinder Morgan easement, where the modified, ACM-lined channel transitions back into a natural geomorphic channel. A rock cross vane weir will be constructed at this location to stabilize the channel bottom at that location and aid in returning the over-widened channel flow to the more narrow, native bankfull channel.

FEMA Floodplain Mapping

The project will be constructed partly within an existing FEMA Zone AE regulatory floodplain and floodway included within FIRM panel 04005-C6806G, effective 9/3/2010. The FEMA flood zone begins at the south/downstream edge of the road at the City of Flagstaff corporate limits and extends downstream into the city's floodplain jurisdiction. The channel through approximately 120 feet of the regulatory floodplain will be modified. The channel shape will be modified significantly from its existing condition and include both excavation of a new thalweg and filling in some portions of the old channel to achieve the final shape.

Permitting of this work within the FEMA floodplain will be through City of Flagstaff Floodplain Management. A separate "Floodplain Modification Report" has been prepared by NCD that provides documentation of no rise in the pre- to post-project maximum water surface elevations at any of the FEMA regulatory channel cross sections. That report has been submitted separately to CCPW and the City of Flagstaff.

Jurisdictional Waters of the US – 404 Permitting

A Nationwide 14 (Linear Transportation) 404 permit will be required for this project through the US Army Corps of Engineers (USACE). NCD has already prepared and submitted a Preliminary Jurisdictional Delineation of jurisdictional waters to the USACE for their review. The next step in the 404 process will be to submit a Preliminary Construction Notification (PCN). NCD has also prepared the biological evaluation of the affected area. At this writing, we are waiting for the archeological survey of the ground to be completed before submitting the PCN to the USACE.

Utilities Impacted

APS electric and CenturyLink/Optimum telephone have underground facilities present in the work area, running east and wet along the north shoulder of the road. This project will require vertical relocation of those utilities to below the new box culvert and associated roadside channel grading.

High pressure natural gas transmission pipelines are located within an 80 foot wide Kinder Morgan easement on the south side of the road. Grading across the top of the gas lines will be required to transition the flow from full width box culvert outlets to the more narrow downstream natural geomorphic channel.

Anticipated Excavation Conditions

Native soils through the extent of the project excavation area are anticipated to be alluvium soils and bedrock is not expected. CCPW has engaged with Speedie and Associates to conduct a geotechnical investigation in support of this design. At the time of this writing, the geotechnical design report has not been completed. When available, the geotechnical report will be reviewed with respect to the plans to be sure both are in agreement and a copy of the report included within the project special provisions.

Hydrologic/Hydraulic Analysis

Hydrologic modeling of the upstream watershed has been provided by JE Fuller Hydrology and Geomorphology (JEF) as part of their analysis of the Pipeline Fire for the Coconino County Flood Control District and the City of Flagstaff.

For the design of the box culvert crossing, NCD was provided with hydrographs of various design storms by JEF. Hydrographs provided by JEF are of the post-burn watershed condition and include the effects of the recently completed stormwater detention basins just upstream of the road crossing. For reference, the FEMA regulatory event was also modeled by NCD. Peak flow from the storm events evaluated for this project include:

Table 1 - Hydrologic Design Summary

Storm Event	Peak Flow (cfs)
2 in/45 minute storm (primary design storm)	295
25-year Type II/6 hr storm (post-burn condition)	496
50-year Type II/6hr storm (post-burn condition)	762
100-year Type II/6hr storm (post-burn condition)	1086
FEMA 100 yr regulatory flood event (pre-burn)	440

Peak flows and hydrographs supplied by JE Fuller Hydrology and Geomorphology
FEMA 100 year flow per the current Flood Insurance Study 04005CV001B, 7/19/2022

NCD has hydraulically modelled the culvert crossing using the two-dimensional capabilities within HEC-RAS. The results show that with the 3-barrel box culvert, the 2in/45min storm will pass completely beneath the road and not inundate the traffic lanes. The 25, 50 and 100 year, flow from the post-burn watershed condition will continue to overtop the road to varying degrees, but to a lesser extent than the per-project condition.

A summary of the existing condition and performance of the proposed box culvert crossing is included in Table 2 below:

Table 2 - Comparison Existing and Proposed Flow Conditions

Storm Event	Existing Condition		Post-Project Condition	
	[a]Culvert Peak Flow (cfs)	Road Overtop Peak Flow (cfs)	Culvert Peak Flow (cfs)	Road Overtop Peak Flow (cfs)
2 in/45 minute	0	293	293	0
25-year Type II/6 hr	0	495	493	2
50-year Type II/6hr storm	0	761	642	119
100-year Type II/6hr storm	0	1084	714	370
FEMA 100 yr regulatory flood	0	440	440	0

[a] *Assumes that the existing 24 inch culvert is clogged with sediment/debris and ineffective during the event.*
Note: Total peak flows may not add up to the peak incoming flow per Table 1 due to a minor amount of attenuation realized in the ponded volume held on the upstream side of the road.

Relative to the effective FEMA floodplain/floodway that begins at the immediate downstream edge of the road, there should be no difference in the pre-post project flow condition in the downstream main stem of the channel as the box culverts will pass the 440 cfs. At the outlet of the box culverts, there will be minor differences in the water surface elevation due to the grading required to open the existing channel thalweg to match the wider box culvert outlet. The grading will result in a net removal of 39 cy of material from the channel/banks within the floodplain limits and pre- to post-project water surface elevations should result in reduced depths accordingly when strictly comparing the regulatory flow within the channel.

The 2D model results do not directly correlate to the effective FEMA model due to differences between the effective model topography and the current terrain. Of note, the effective one-dimensional model (2004) does not account for the upstream overtopping of the road during a 440 cfs flood event that is otherwise demonstrated by the 2D model. This overtopping diverts approximately 73 cfs to a swale located approximately 250 feet to the west of the culvert crossing. That diversion of floodplain flow reduces the in-channel flow (compared to the effective FEMA model) until it rejoins the mainstem at a location approximately 800 feet downstream of the culvert crossing.

With respect to the current conditions and topography, the increased capacity below the road that the box culverts provide will result in an increased amount of flow within the main stem of the channel for approximately 800 feet immediately downstream of the road during large storm events. With the increased capacity of the box culverts, a reduced amount of overtopping flow will be diverted 250 feet to the west and to the natural, broad swale at that location. Downstream of the confluence of the main stem channel and swale impacts from the project will be negligible. Compared to the immediate post-burn condition and prior to the combined mitigation that efforts by the City of Flagstaff, Coconino County and the US Forest Service, the post-project condition will be a significant reduction in flood risk to this reach of Schultz Creek.

Moving forward, Coconino County and the City of Flagstaff will need to determine what will be required to permit the project through floodplain management. With the desired project schedule, it would preferable if floodplain permitting can be conducted on the local level. A FEMA CLORM/LOMR process would inevitably delay the construction of this project beyond the coming monsoon season. Potentially, a limited 1D model comparison of pre- to post-project water surface elevations from the regulatory flood flow (440 cfs) that is “inserted” into the main stem of the channel at the outlet of the box culverts will suffice for local permitting.

Additionally, further refinements of NCD’s 2D model will continue as the design progresses. It is noted at this writing that there as some minor issues with the DEM surface and false flattening of the surface near contour lines that will need to be refined prior to the next submittal. However, the changes to the final model result are anticipated to be minor.

Conclusion

The concrete box culvert will provide for passage of the 2in/45min storm event beneath the road without overtopping or interruption in traffic. Peak flows from storms larger than the design storm will continue to overtop the road as before the project, but with significantly reduced impact to access and traffic.

The increased capacity that the large box culvert crossing provides will result in a greater amount of flow being directed into the channel that is immediately downstream of the road crossing during large runoff events. During those larger events, flows would otherwise overtop the road with a portion of that flow being diverted to the west and away from the main stem of the channel. This result will impact the river reach for approximately 800 feet downstream of the road but will have negligible impact beyond where the overflow returns to the main stem of the channel. The main stem of the channel for that 800 feet will experience more flow than if the project were not constructed, however, this flow will be less than if the combined mitigation of the City, County and USFS had not been implemented.

Relative to the FEMA 100year flow, the increased capacity of the culverts to pass water beneath the road will ensure that the regulatory flood event will be delivered directly into the mainstem of the channel as per the effective regulatory floodplain model.

The project will also include minor reprofiling of the road centerline. To accommodate 4ft high box culverts, the finished grade of the road centerline will be elevated 0.8 feet from the existing condition. The road will be transitioned into and out of the new grade through vertical curves, requiring approximately 250 feet of the road pavement to be reconstructed.

This submittal reflects the 90% progress design delivery for this project. We look forward to the comments and input to the project from Coconino County Public works and other interested stake holders.

Attachments

Included as attachments to this report are supporting data, output and screen shots of the various HEC-RAS 2D model runs for the box culverts, road overtopping and downstream channel.

Appendix Attachments

HEC-RAS 2D Geometry – Boundary Conditions

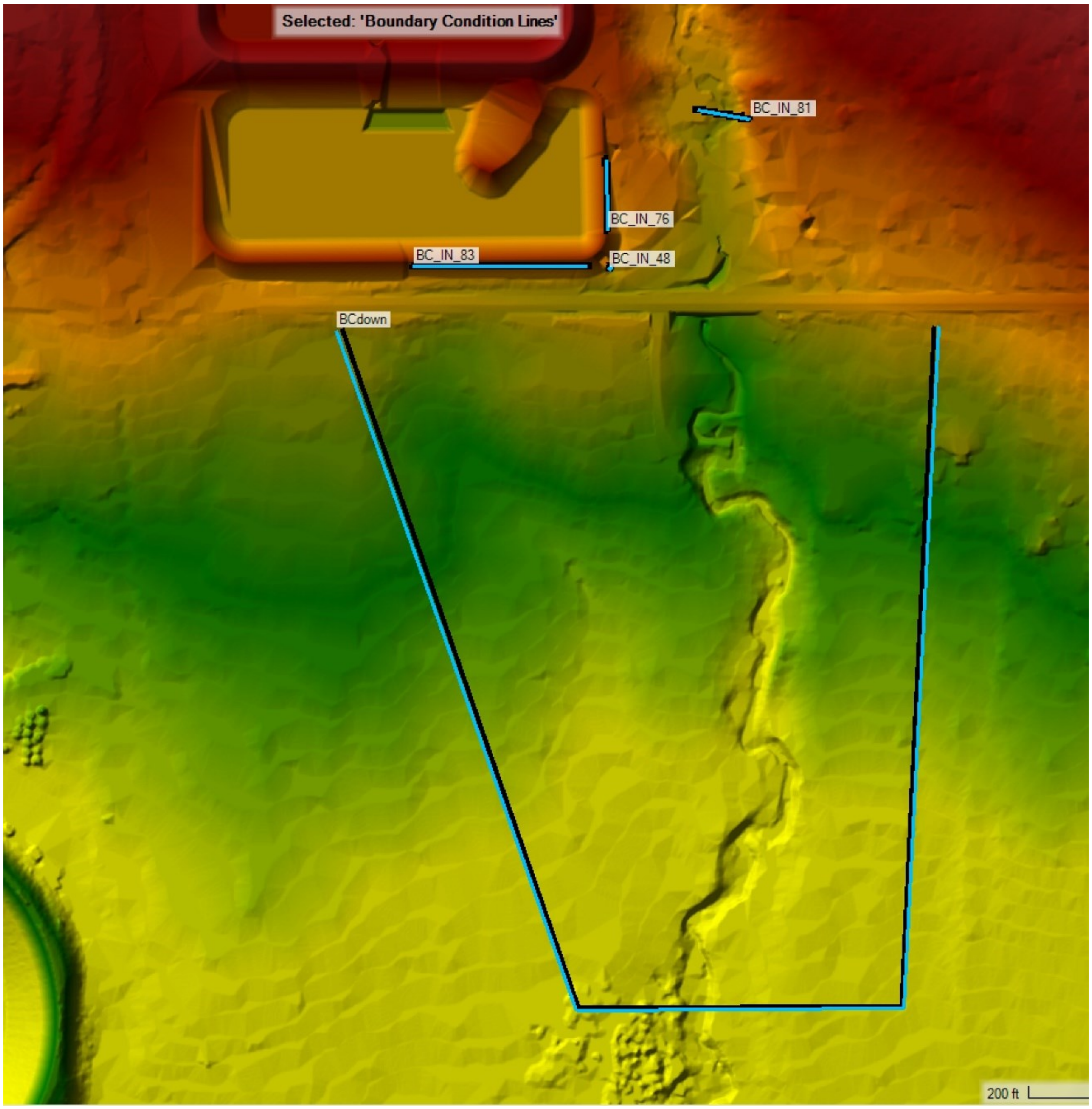
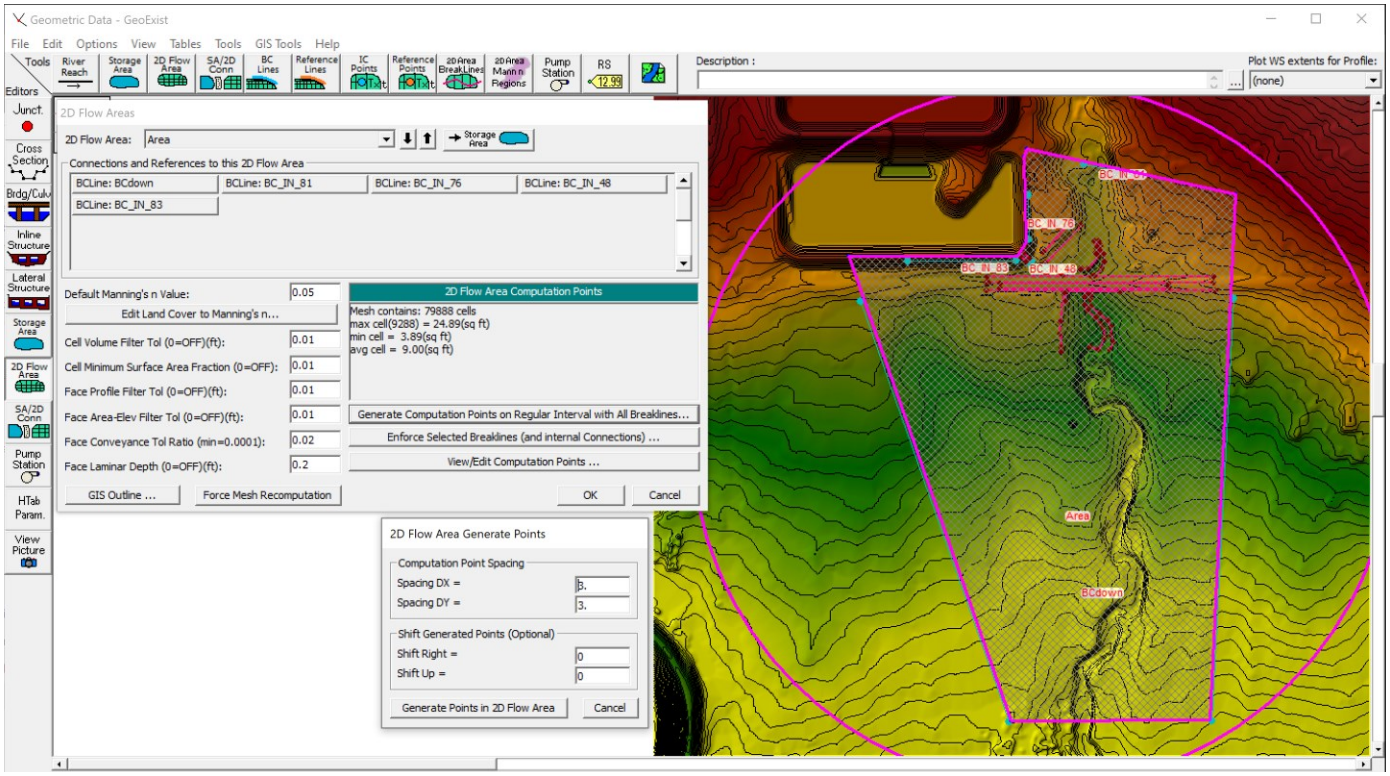


Figure 2: Hydraulic Model Boundary Lines Shown Against Existing Terrain

Each of the upstream "BC_IN" boundary conditions has a corresponding hydrograph provided by JE Fuller. The aggregate total peak flow from the all of the hydrographs to the the road crossing for the following storms is as follows:

2 inch in 45 minutes	Estimated Peak Flow 296 cfs
25-year 6-hour Duration Type-II Storm	Estimated Peak Flow 497 cfs
50-year 6-hour Duration Type-II Storm	Estimated Peak Flow 763 cfs
100-year 6-hour Duration Type-II Storm	Estimated Peak Flow 1086 cfs

The downstream boundary condition is modeled as "Normal Depth" with a friction slope of 1% to approximate the existing valley slope. The downstream boundary condition is placed well away from the project area to minimize error near the project area due to incongruencies between the model and the actual conditions.



2D Flow Areas

2D Flow Area: Area

Connections and References to this 2D Flow Area

BCLine: BCdown | BCLine: BC_IN_81 | BCLine: BC_IN_76 | BCLine: BC_IN_48 | BCLine: BC_IN_83

Default Manning's n Value: 0.05

Edit Land Cover to Manning's n...

Cell Volume Filter Tol (0=OFF)(ft): 0.01

Cell Minimum Surface Area Fraction (0=OFF): 0.01

Face Profile Filter Tol (0=OFF)(ft): 0.01

Face Area-Elev Filter Tol (0=OFF)(ft): 0.01

Face Conveyance Tol Ratio (min=0.0001): 0.02

Face Laminar Depth (0=OFF)(ft): 0.2

2D Flow Area Computation Points

Mesh contains: 79888 cells
max cell(9288) = 24.89(sq ft)
min cell = 3.89(sq ft)
avg cell = 9.00(sq ft)

Generate Computation Points on Regular Interval with All Breaklines...

Enforce Selected Breaklines (and internal Connections) ...

View/Edit Computation Points ...

GIS Outline ... | Force Mesh Recomputation | OK | Cancel

2D Flow Area Generate Points

Computation Point Spacing
Spacing DX = 3.
Spacing DY = 3.

Shift Generated Points (Optional)
Shift Right = 0
Shift Up = 0

Generate Points in 2D Flow Area | Cancel

Figure 3: Existing condition simulation area parameters

HEC-RAS 2D Geometry - Culvert Modeling Data

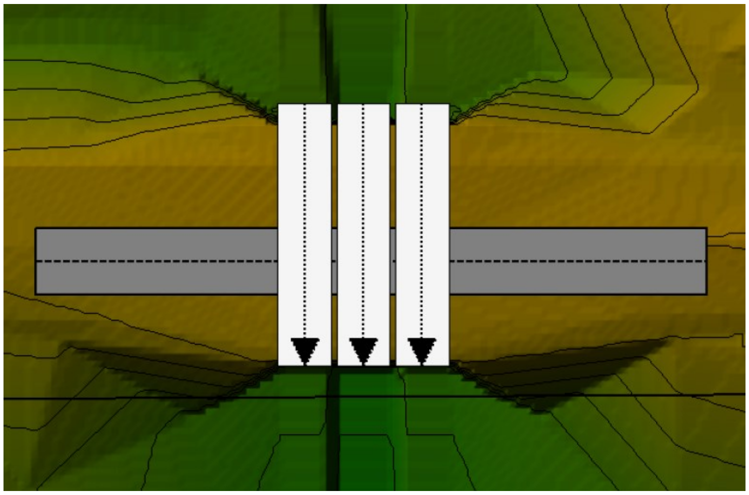
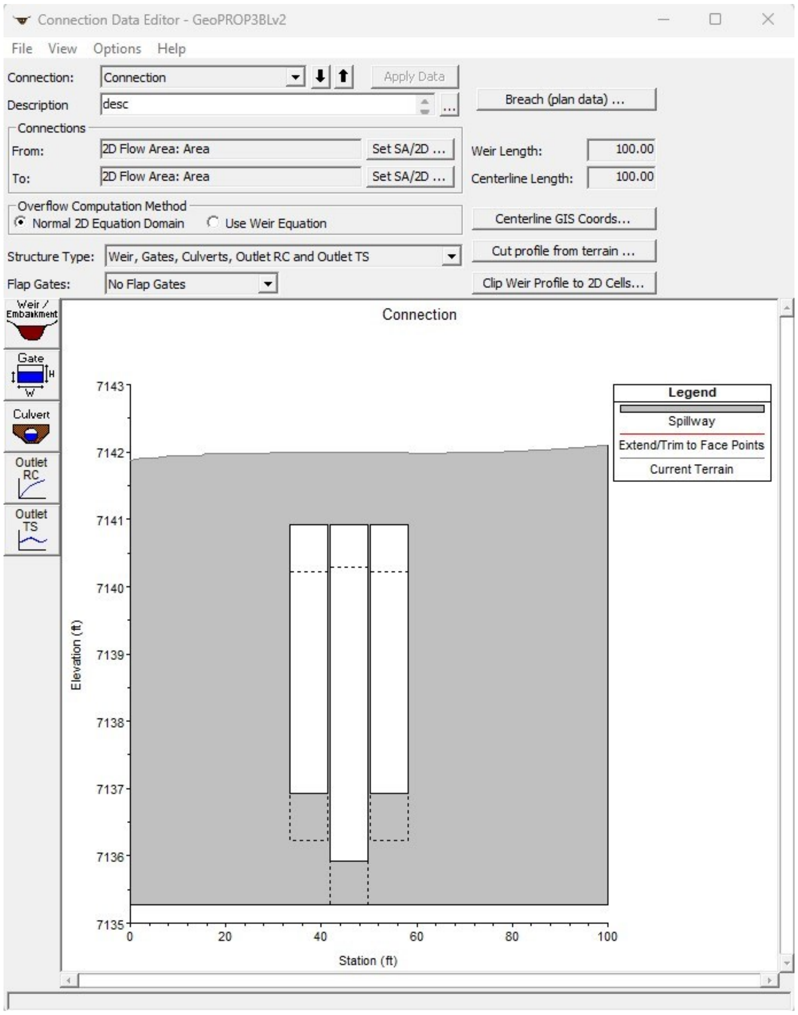


Figure 4: RAS Mapper illustration of culvert configuration.



Connection Data Editor - GeoPROP3BLv2

Connection: Connection

Description: desc

Connections

From: 2D Flow Area: Area

To: 2D Flow Area: Area

Weir Length: 100.00

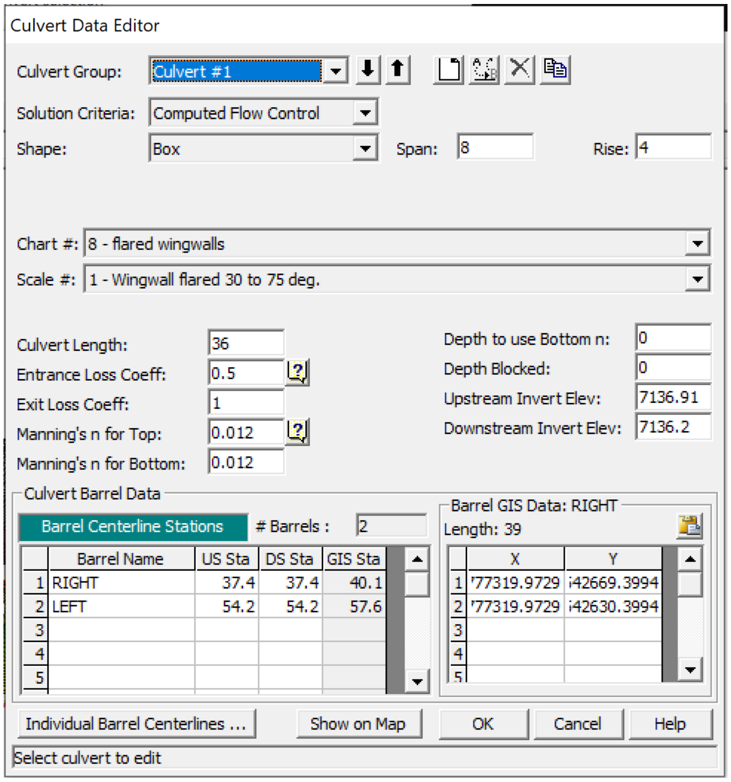
Centerline Length: 100.00

Overflow Computation Method: ● Normal 2D Equation Domain ○ Use Weir Equation

Structure Type: Weir, Gates, Culverts, Outlet RC and Outlet TS

Flap Gates: No Flap Gates

Figure 5: Geometry SA/2D connection data editor illustrates culvert cross section.



Culvert Group: Culvert #1

Solution Criteria: Computed Flow Control

Shape: Box

Span: 8

Rise: 4

Chart #: 8 - flared wingwalls

Scale #: 1 - Wingwall flared 30 to 75 deg.

Culvert Length: 36

Entrance Loss Coeff: 0.5

Exit Loss Coeff: 1

Manning's n for Top: 0.012

Manning's n for Bottom: 0.012

Depth to use Bottom n: 0

Depth Blocked: 0

Upstream Invert Elev: 7136.91

Downstream Invert Elev: 7136.2

Culvert Barrel Data

Barrel Centerline Stations

Barrels : 2

	Barrel Name	US Sta	DS Sta	GIS Sta
1	RIGHT	37.4	37.4	40.1
2	LEFT	54.2	54.2	57.6
3				
4				
5				

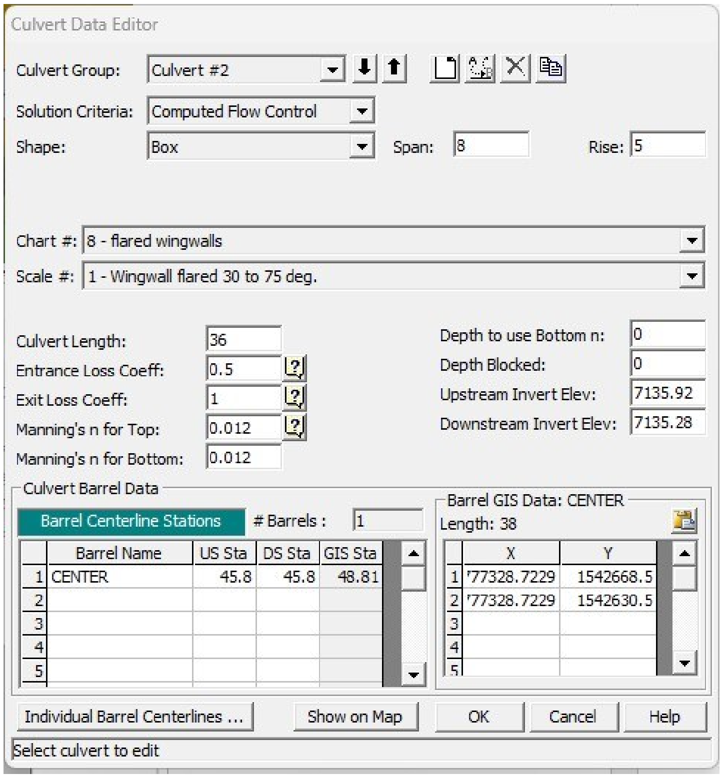
Barrel GIS Data: RIGHT

Length: 39

	X	Y
1	'77319.9729	i42669.3994
2	'77319.9729	i42630.3994
3		
4		
5		

Select culvert to edit

Figure 6: Culvert Data Editor for Culvert #1 pertaining to the left and right barrel



Culvert Group: Culvert #2

Solution Criteria: Computed Flow Control

Shape: Box

Span: 8

Rise: 5

Chart #: 8 - flared wingwalls

Scale #: 1 - Wingwall flared 30 to 75 deg.

Culvert Length: 36

Entrance Loss Coeff: 0.5

Exit Loss Coeff: 1

Manning's n for Top: 0.012

Manning's n for Bottom: 0.012

Depth to use Bottom n: 0

Depth Blocked: 0

Upstream Invert Elev: 7135.92

Downstream Invert Elev: 7135.28

Culvert Barrel Data

Barrel Centerline Stations

Barrels : 1

	Barrel Name	US Sta	DS Sta	GIS Sta
1	CENTER	45.8	45.8	48.81
2				
3				
4				
5				

Barrel GIS Data: CENTER

Length: 38

	X	Y
1	'77328.7229	1542668.5
2	'77328.7229	1542630.5
3		
4		
5		

Select culvert to edit

Figure 7: Culvert Data Editor for Culvert #2 pertaining to the center barrel

DESIGN HYDROGRAPHS

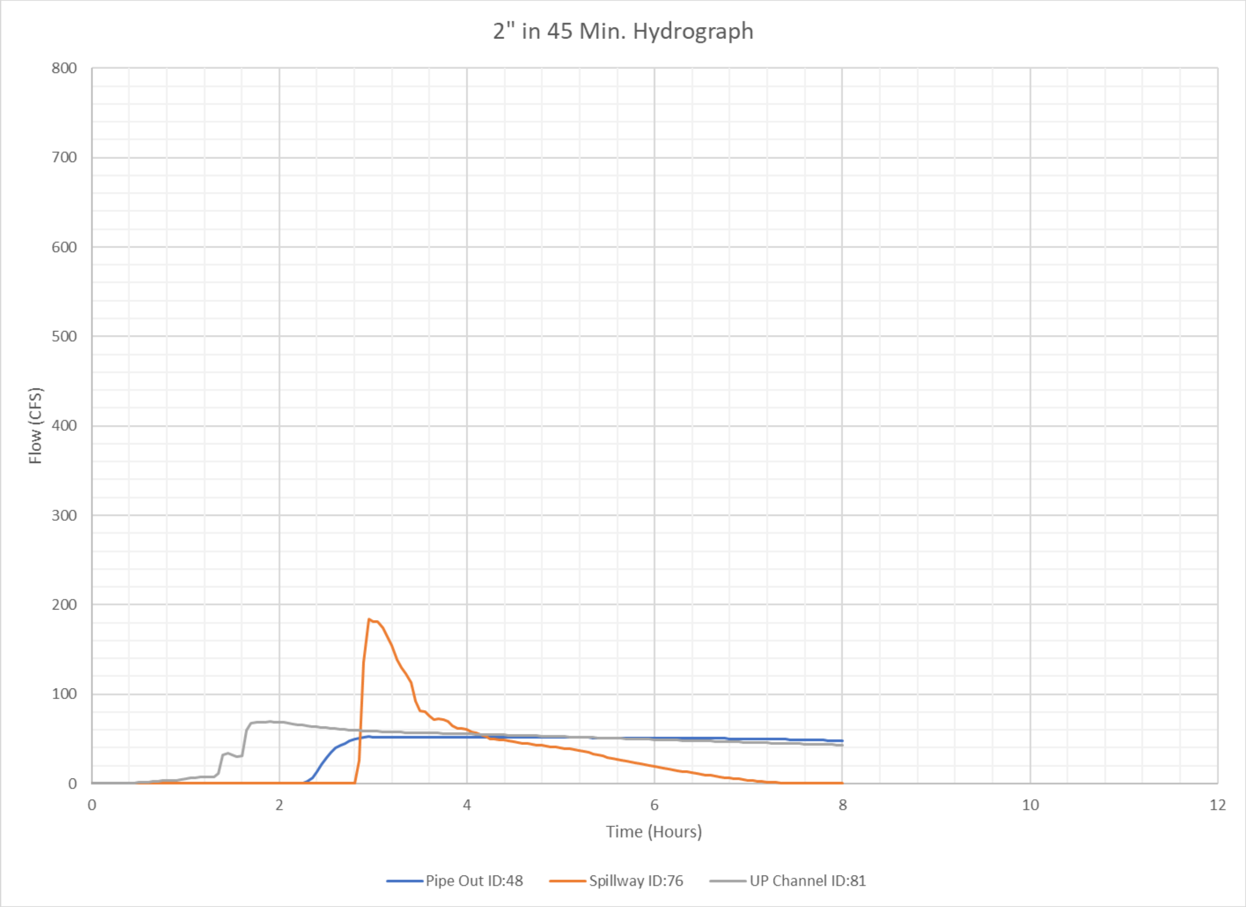


Figure 8: 2" in 45 Min. Hydrograph Chart

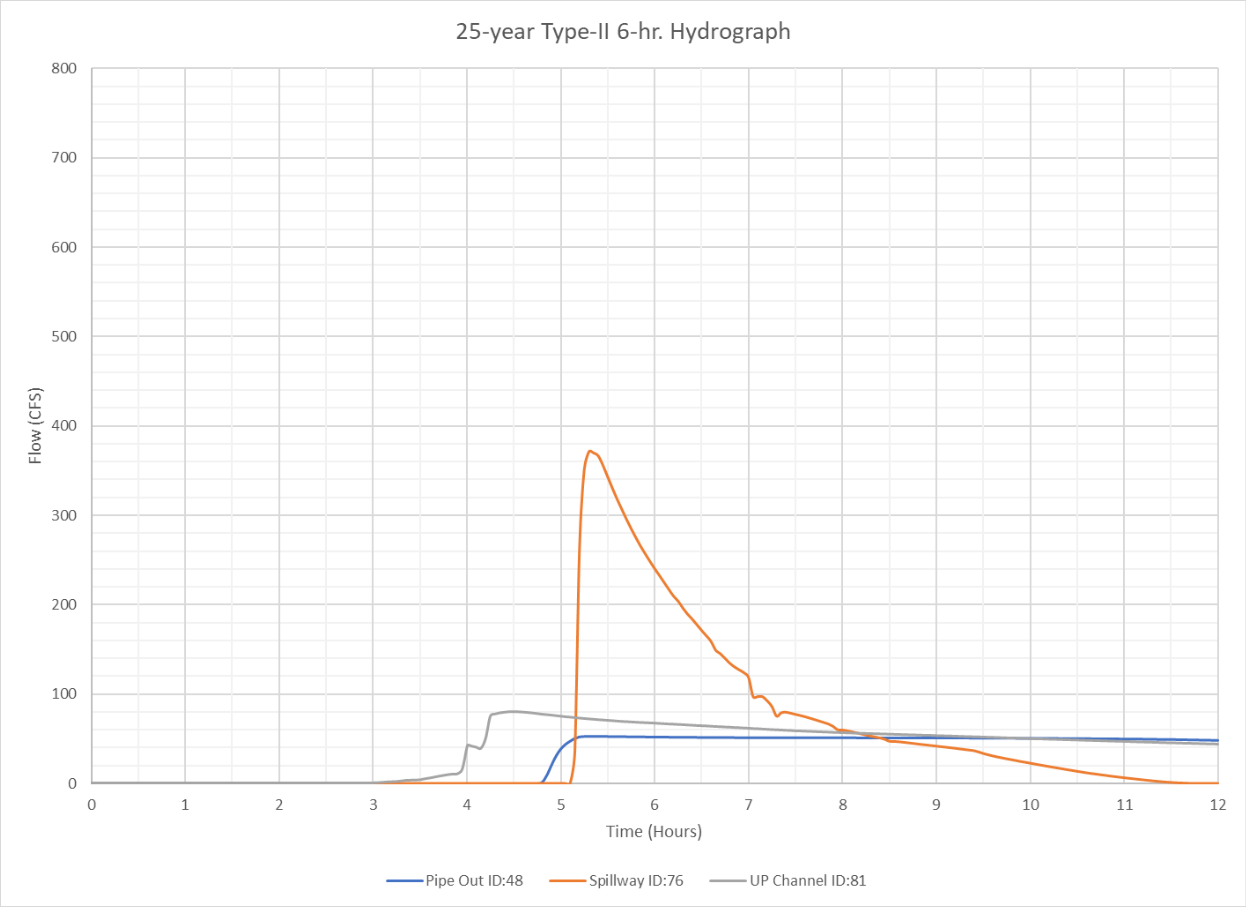


Figure 9: 25-year Type-II 6-hr. Hydrograph Chart

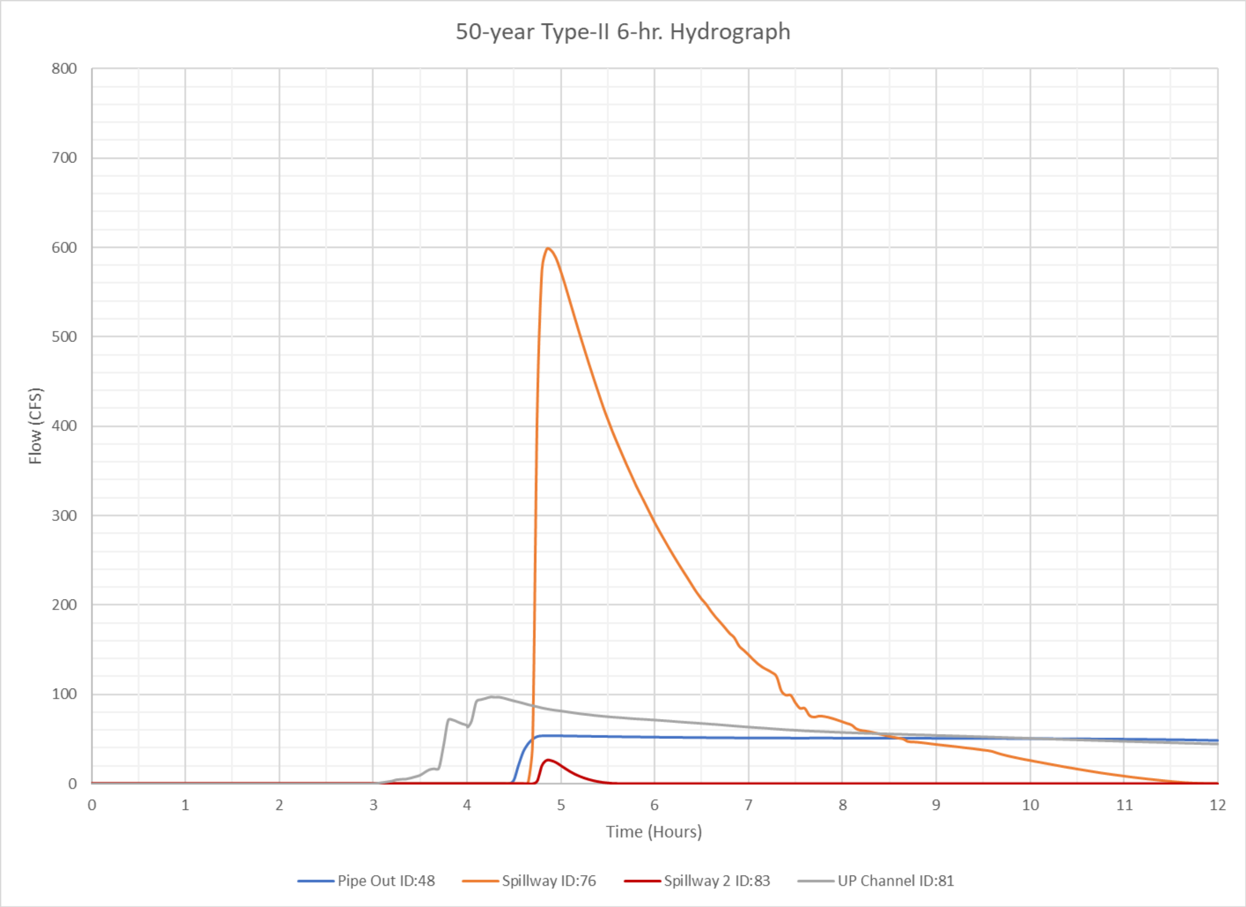


Figure 10: 50-year Type-II 6-hr. Hydrograph Chart

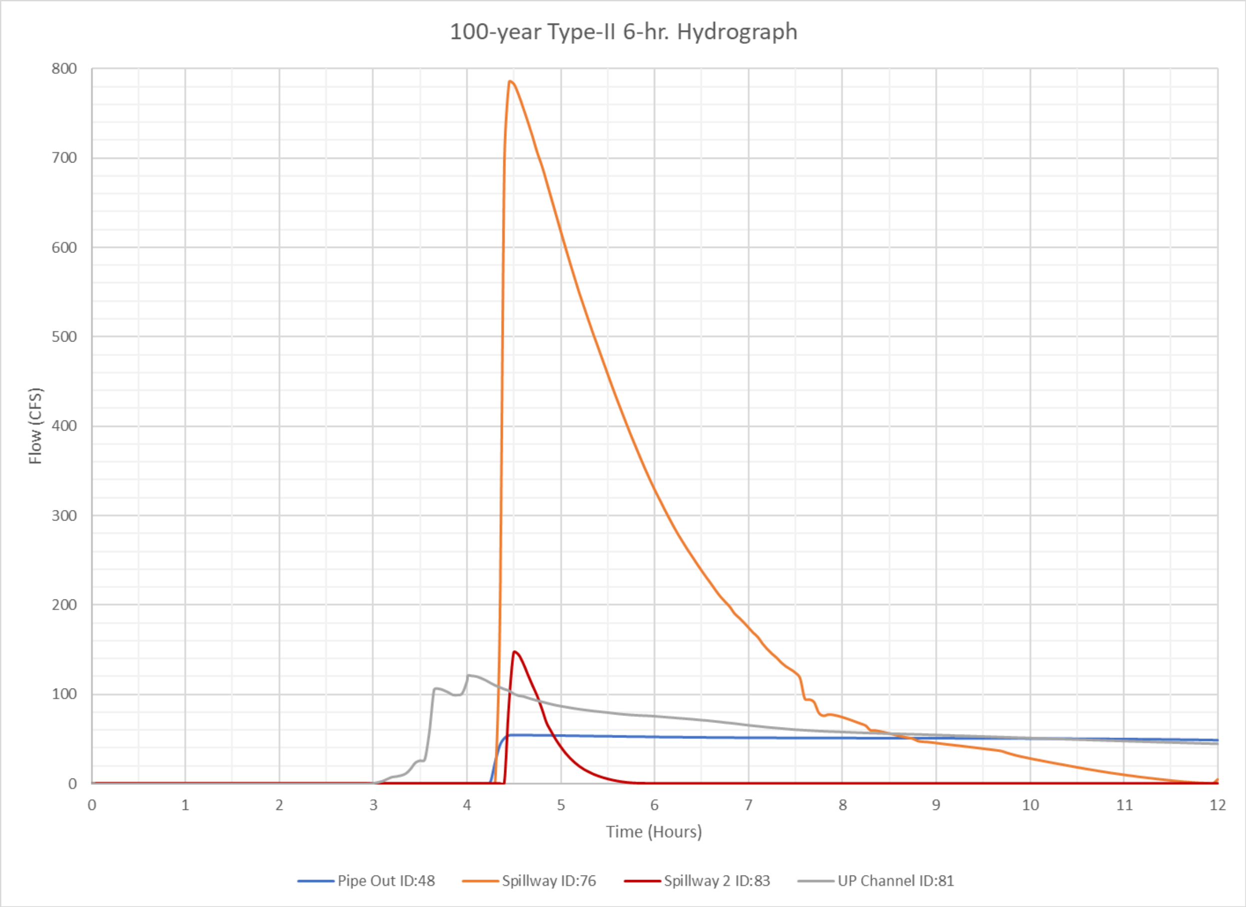


Figure 11: 100-year Type-II 6-hr. Hydrograph Chart

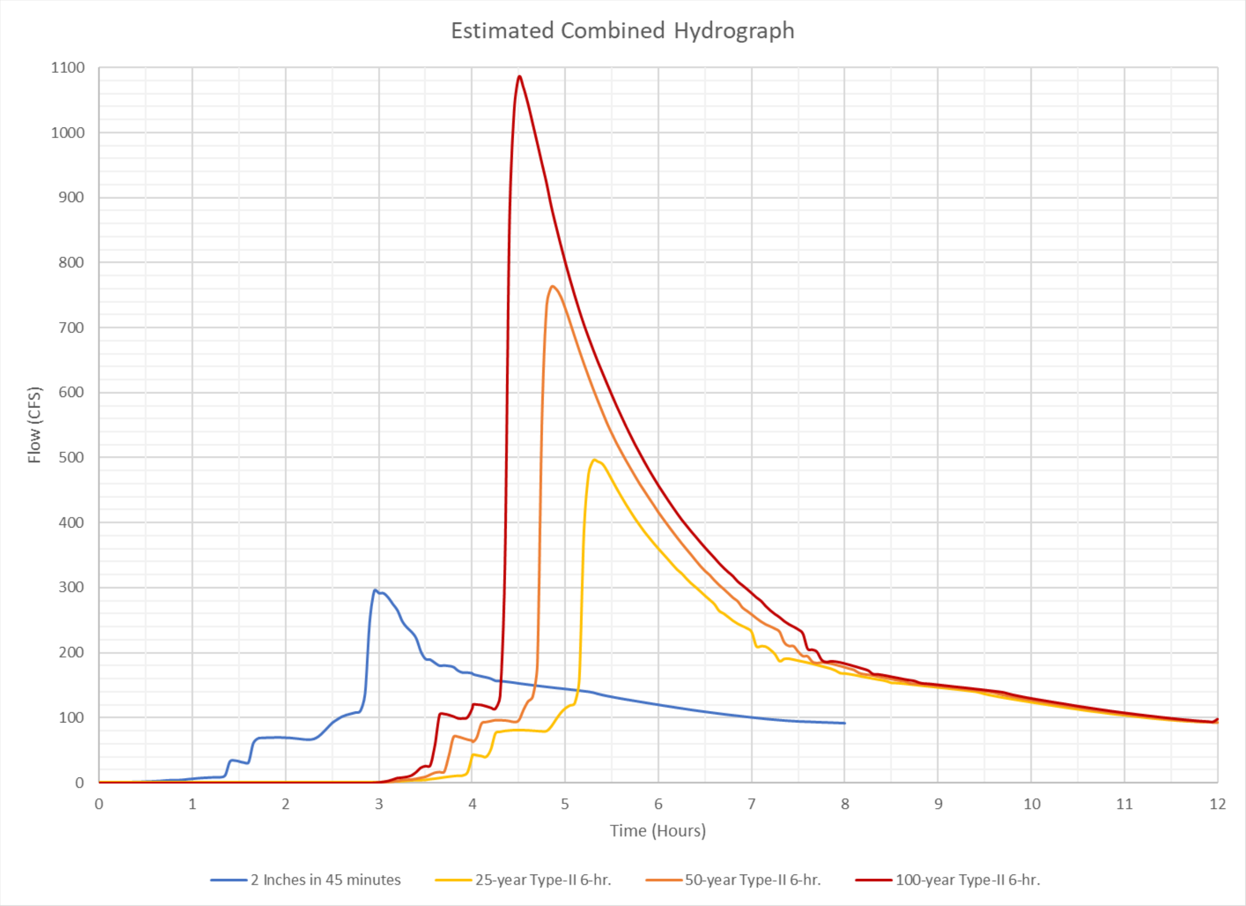


Figure 12: Combined Hydrograph Chart

HEC-RAS Results – Culvert Performance Hydrographs

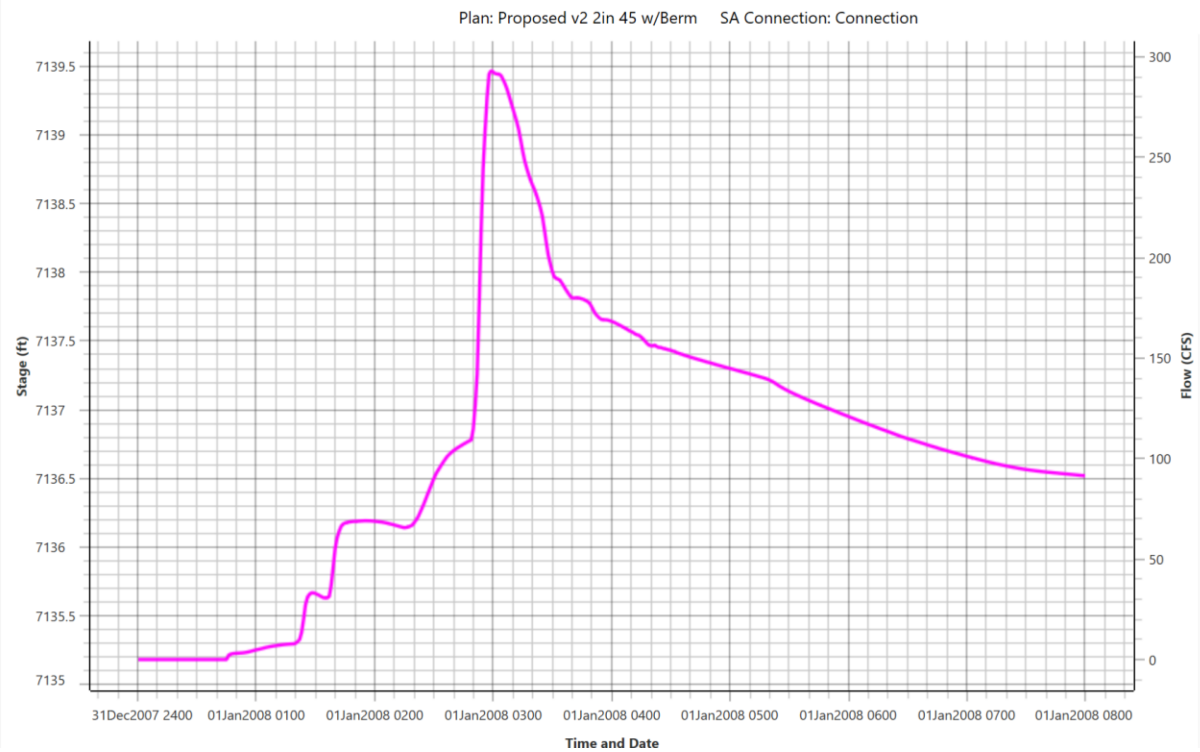


Figure 13: Culvert Stage/Flow Hydrograph from 2 in. in 45-min. simulation (Peak ~293 CFS)

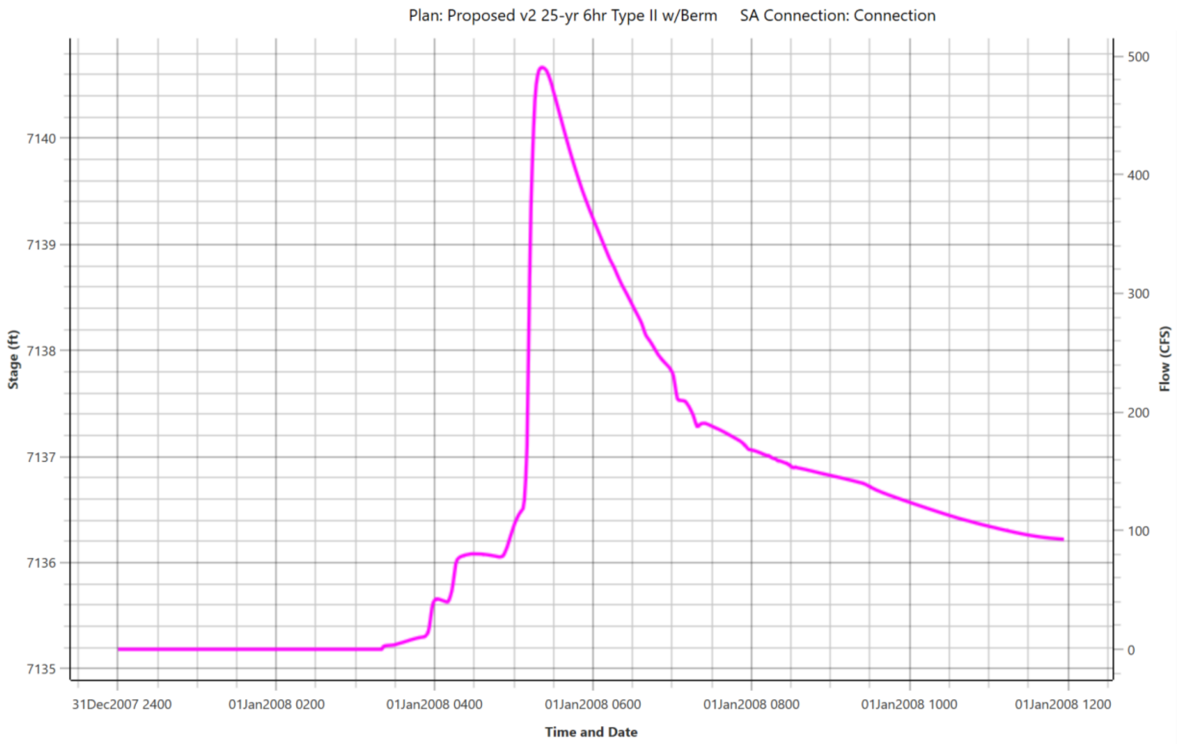


Figure 14:Culvert Stage/Flow Hydrograph from 25-year storm simulation (Peak ~493 CFS)

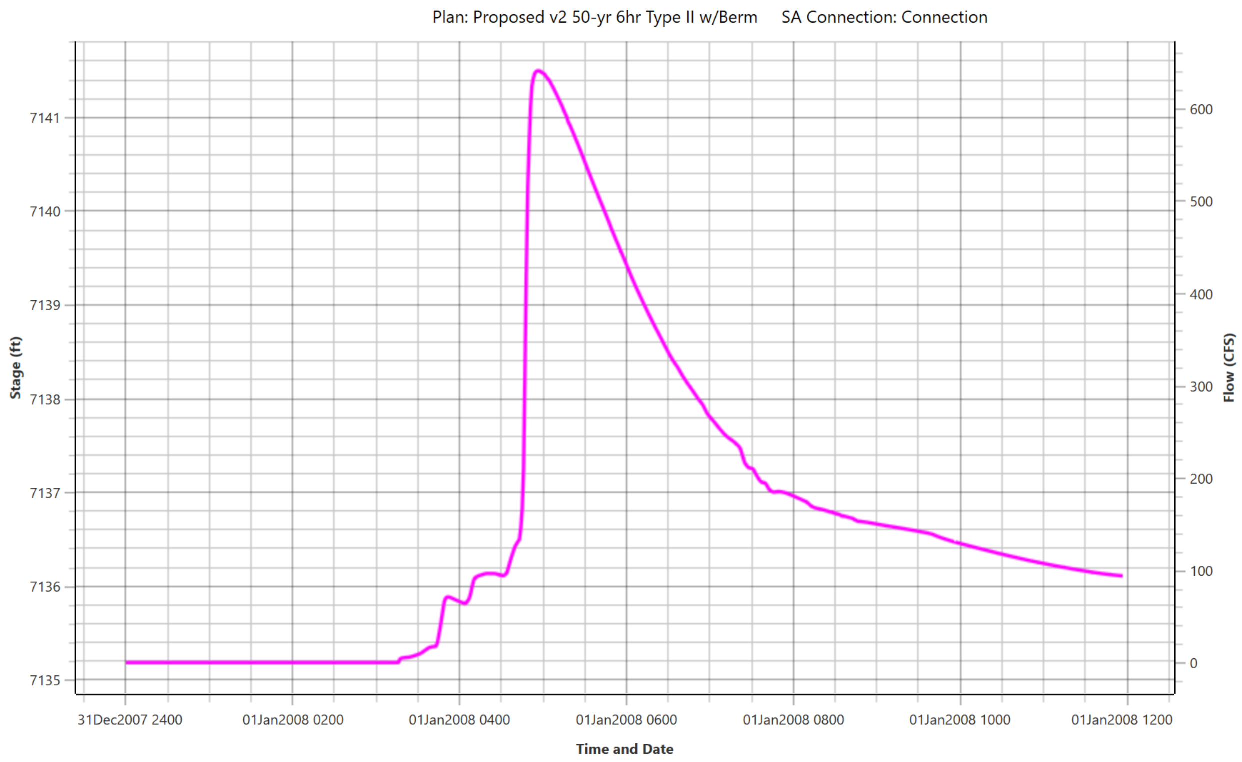


Figure 15:Culvert Stage/Flow Hydrograph from 50-year storm simulation (Peak ~642 CFS)

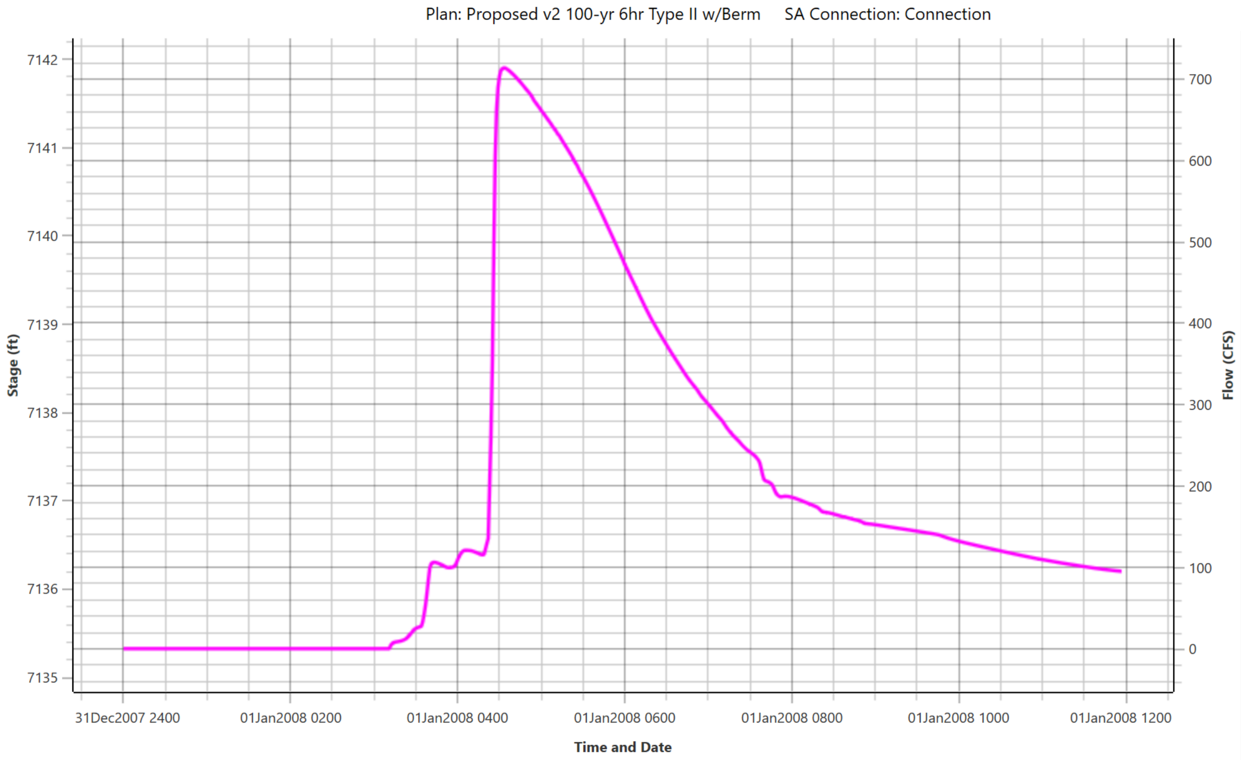


Figure 16: Culvert Stage/Flow Hydrograph from 100-year storm simulation (Peak ~714 CFS)

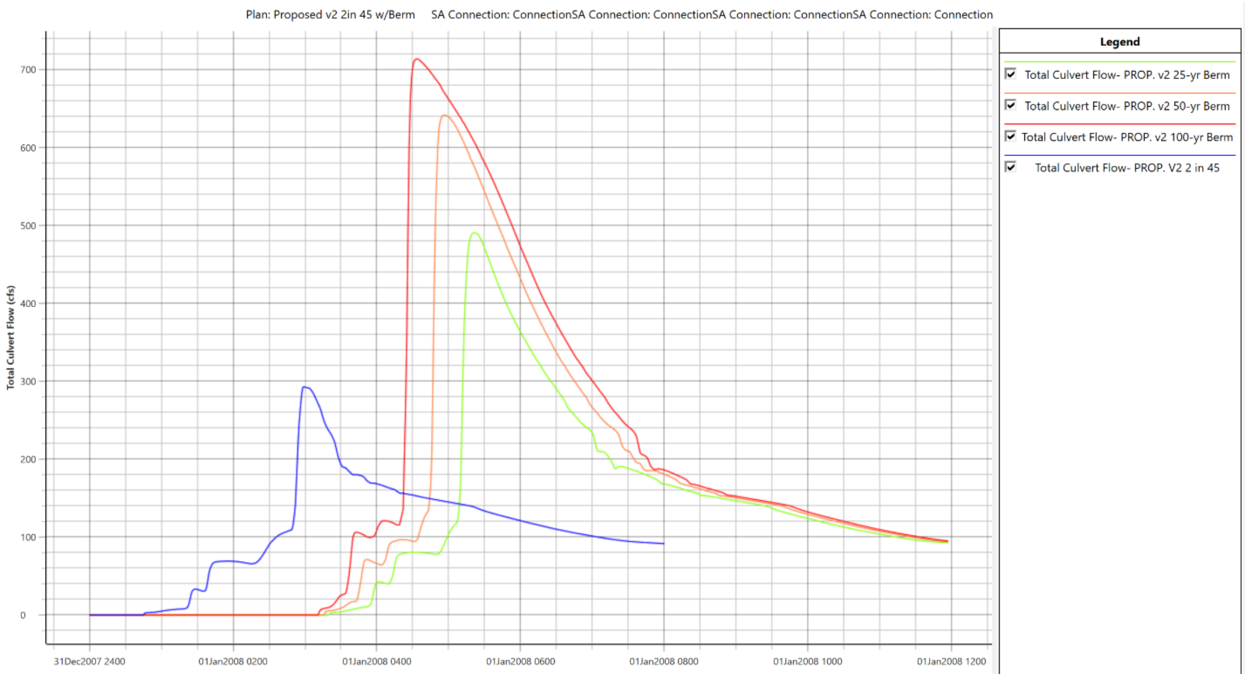


Figure 17: Culvert Flow Hydrograph for 2 in. in 45-min., 25-year, 50-year, and 100-year storms

Note: To model the FEMA regulatory flood event (hydrograph not show above) a simulated hydrograph was created that starts as 0 cfs and quickly ramps up to a steady state 440 cfs flow.

HEC-RAS Results – Screen Captures of Existing Condition Model Results

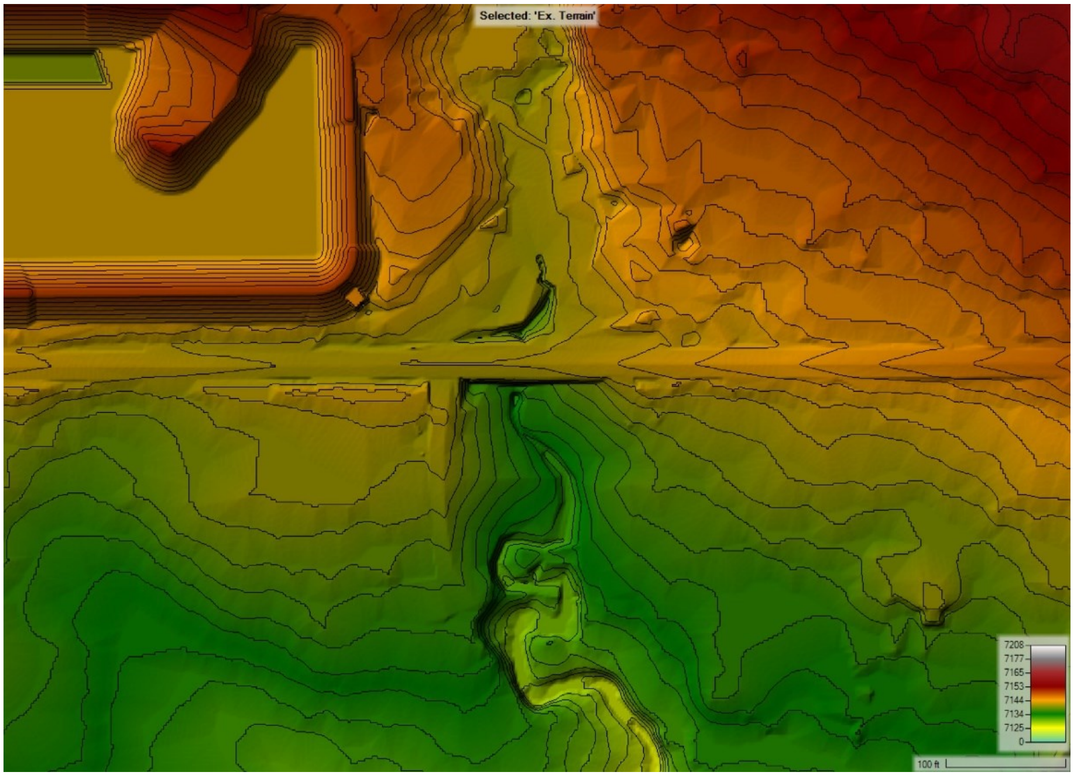


Figure 18: Image Existing Terrain (No Flow)

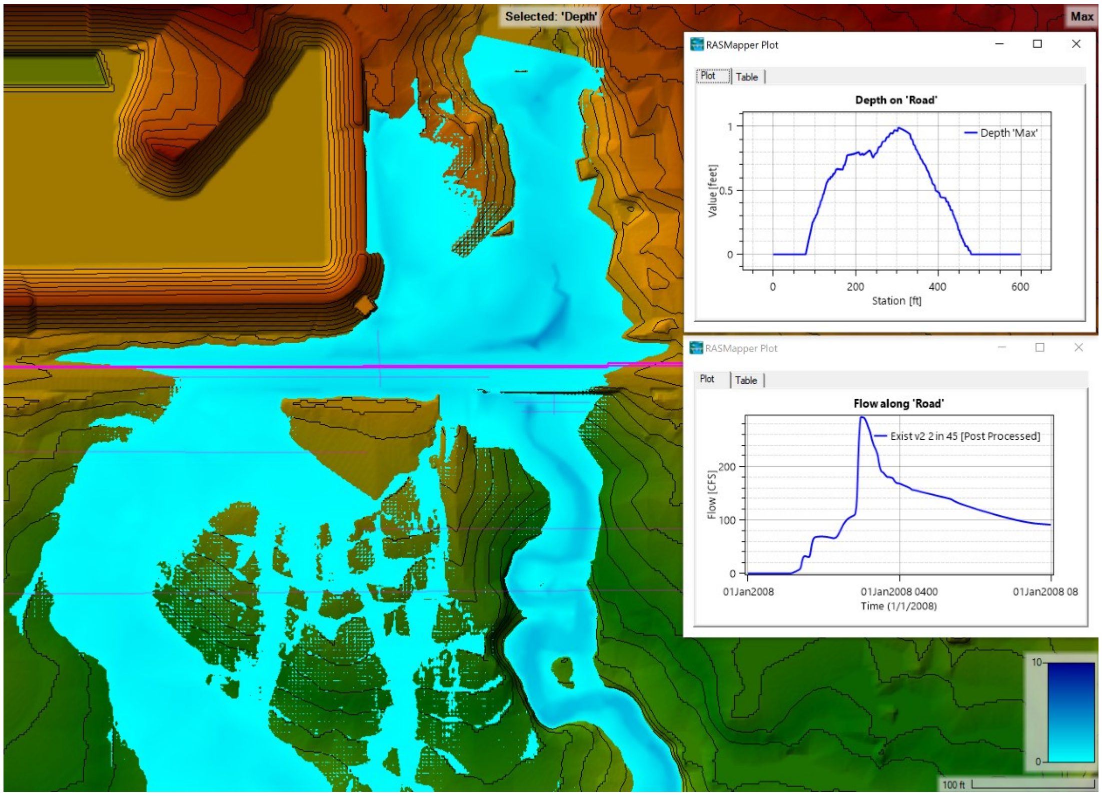


Figure 19: Image Existing Max Depth Over Road at 2" in 45 Min.

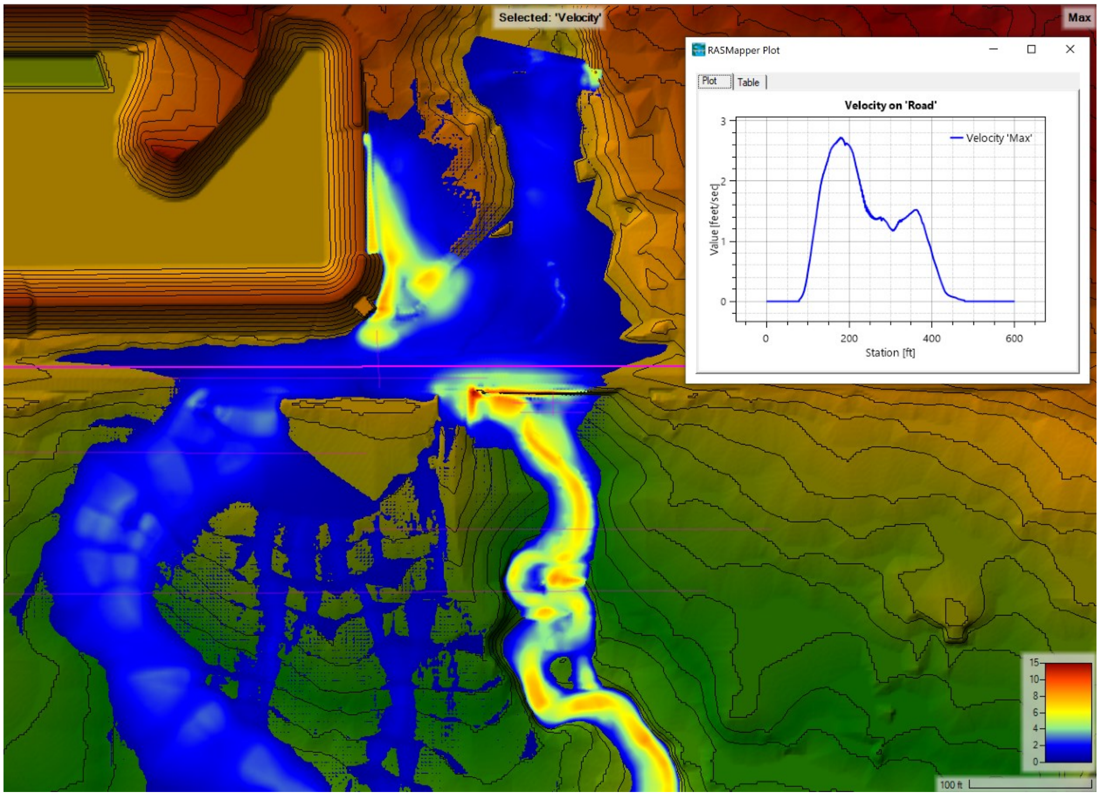


Figure 20: Image Existing Max Velocity Over Road at 2" in 45 Min.

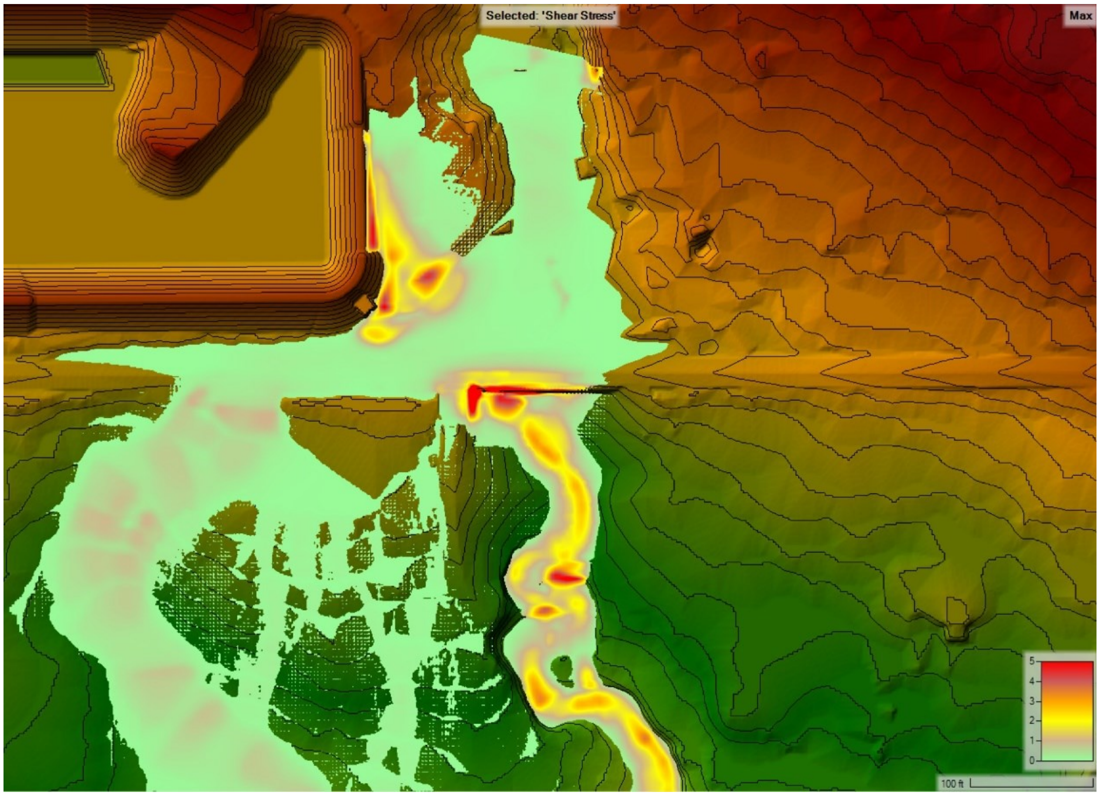


Figure 21 Image Existing Max Shear Stress at 2" in 45 Min.

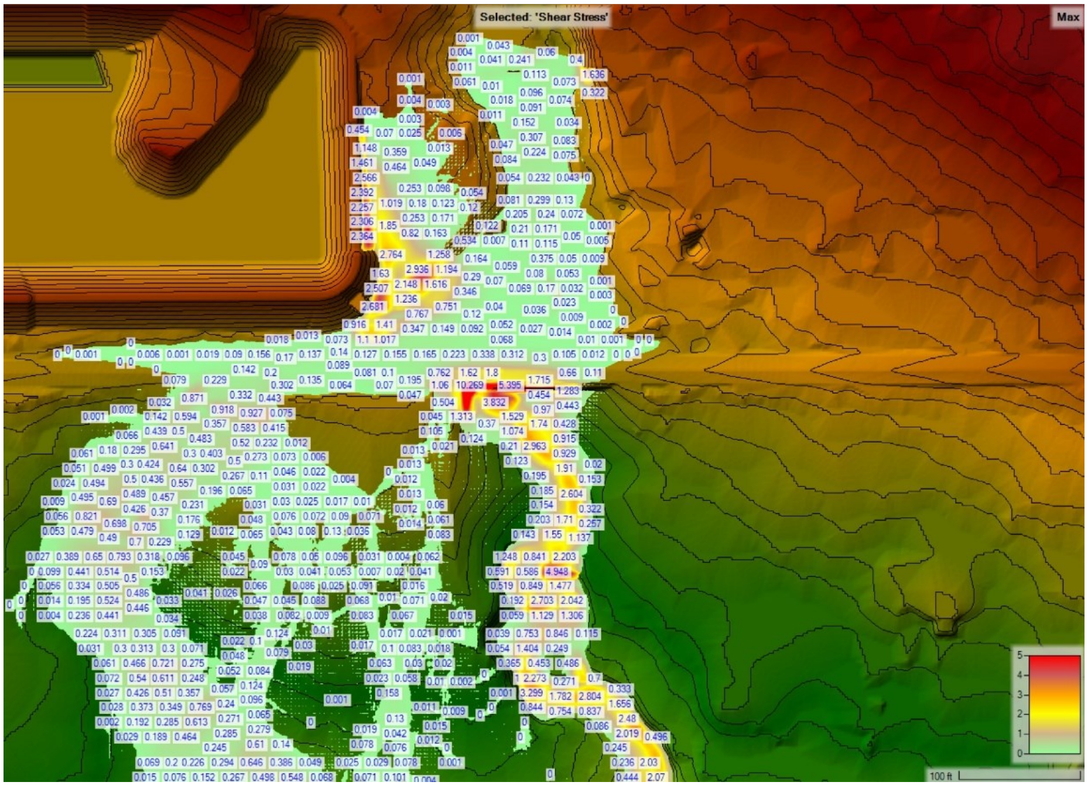


Figure 22 Image Existing Max Shear Stress with Values at 2" in 45 Min.

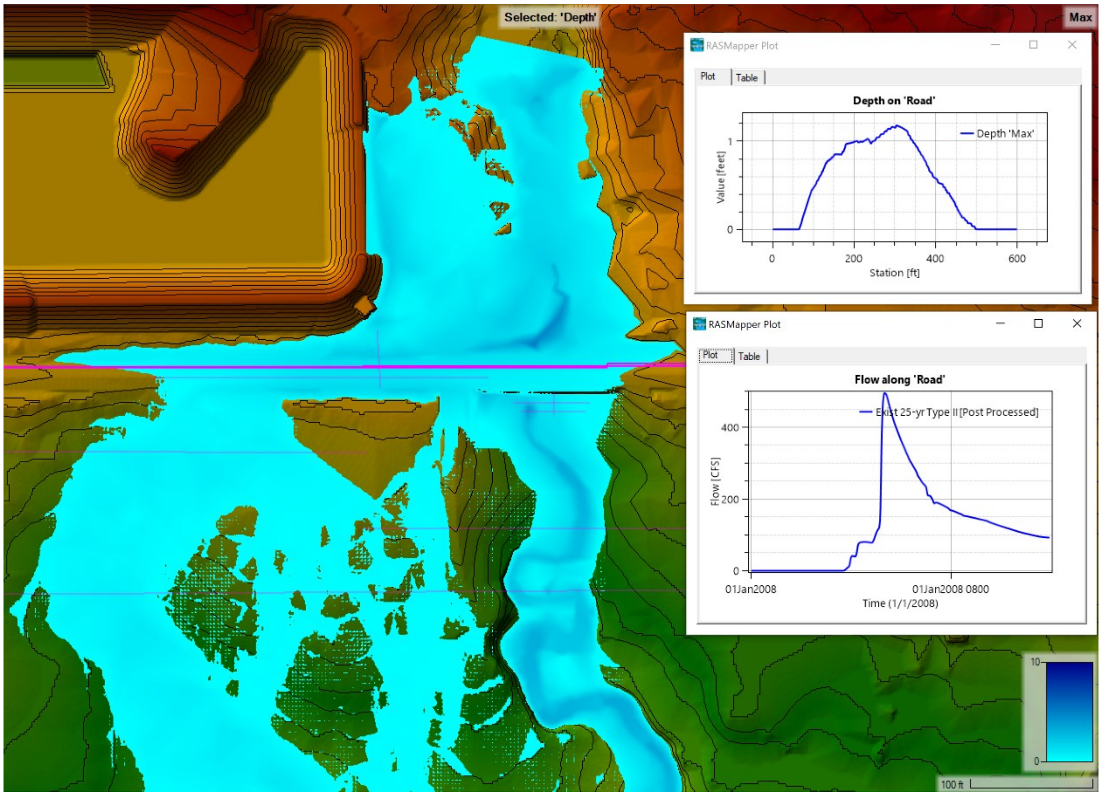


Figure 23 Image Existing Max Depth Over Road at 25-year Type-II

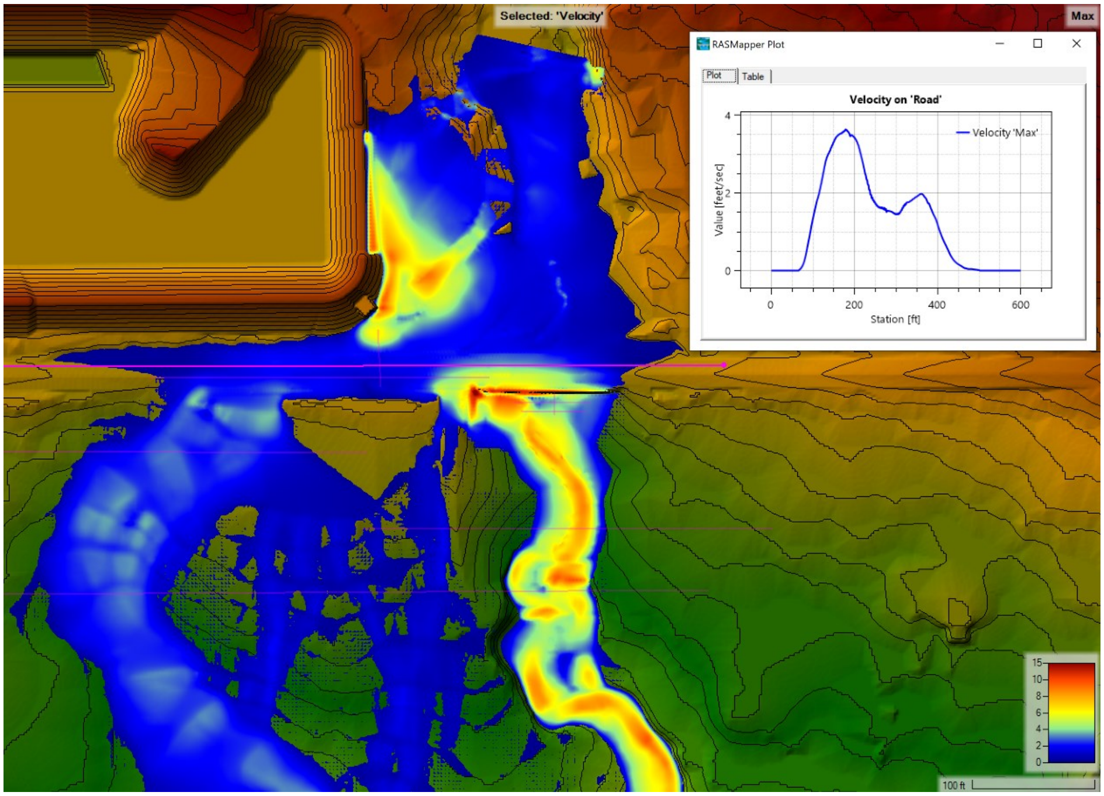


Figure 24 Image Existing Max Velocity Over Road at 25-year Type-II

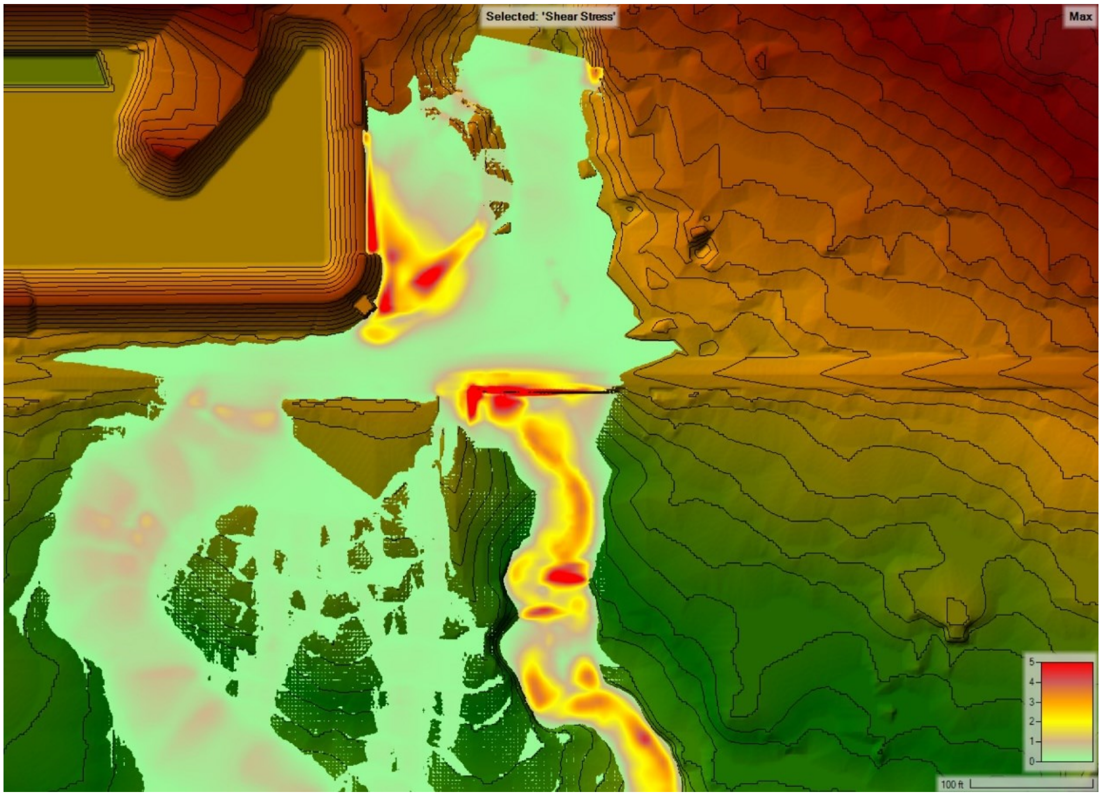


Figure 25 Image Existing Max Shear at 25-year Type-II

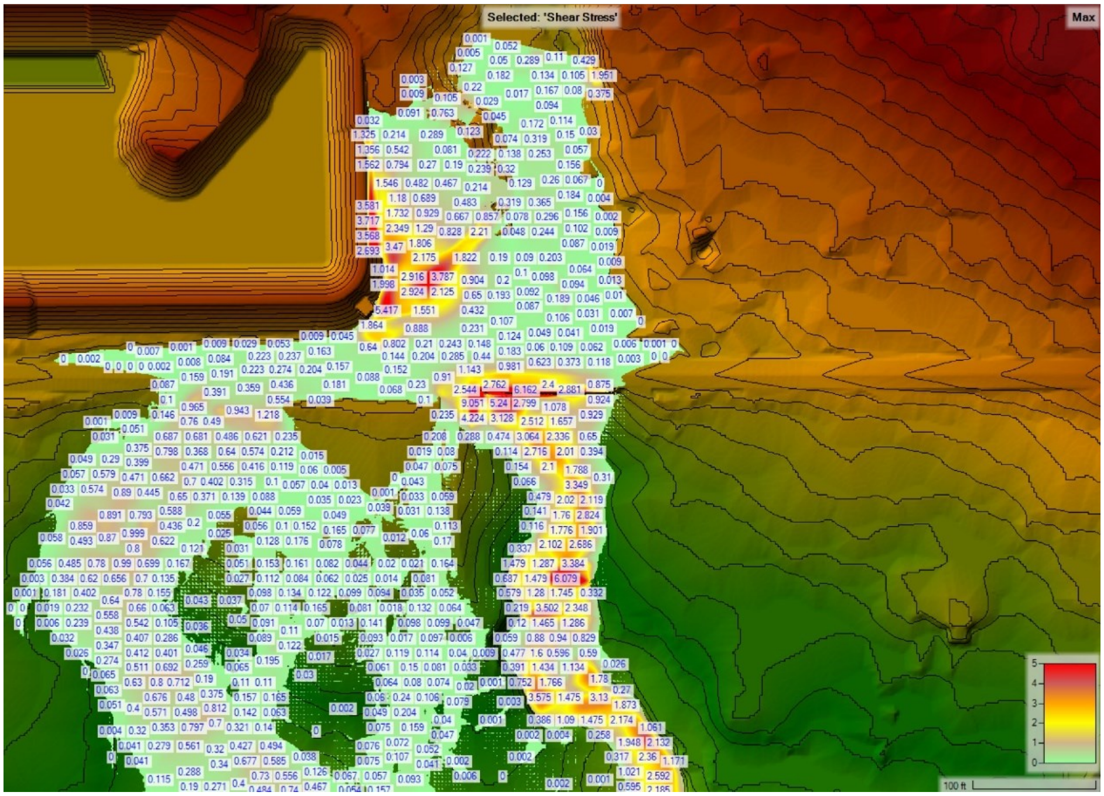


Figure 26 Image Existing Max Shear with Values at 25-year Type-II

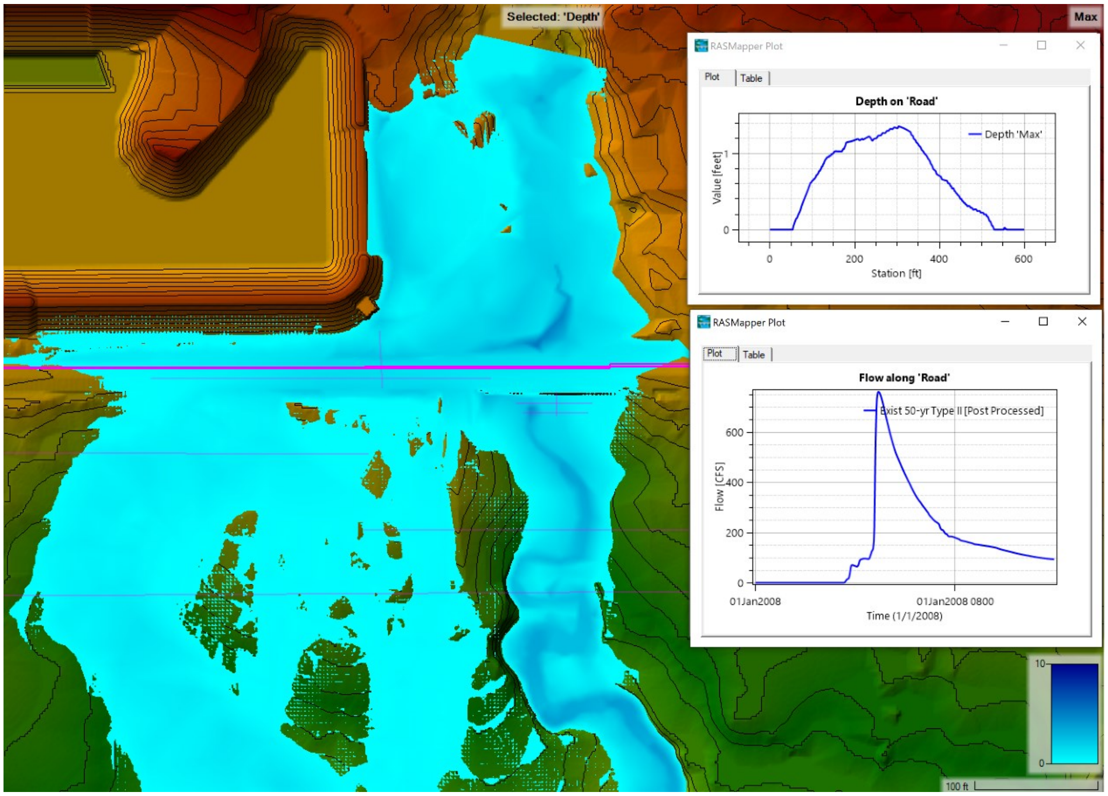


Figure 27 Image Existing Max Depth Over Road at 50-year Type-II

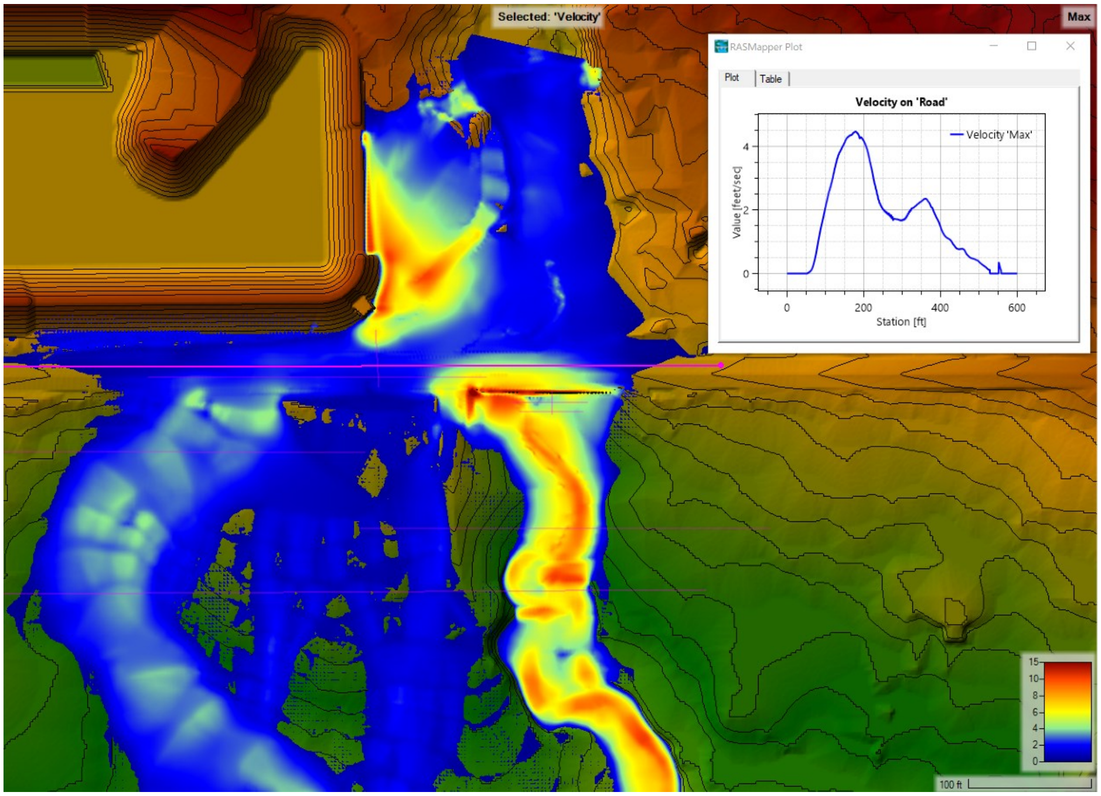


Figure 28 Image Existing Max Velocity Over Road at 50-year Type-II

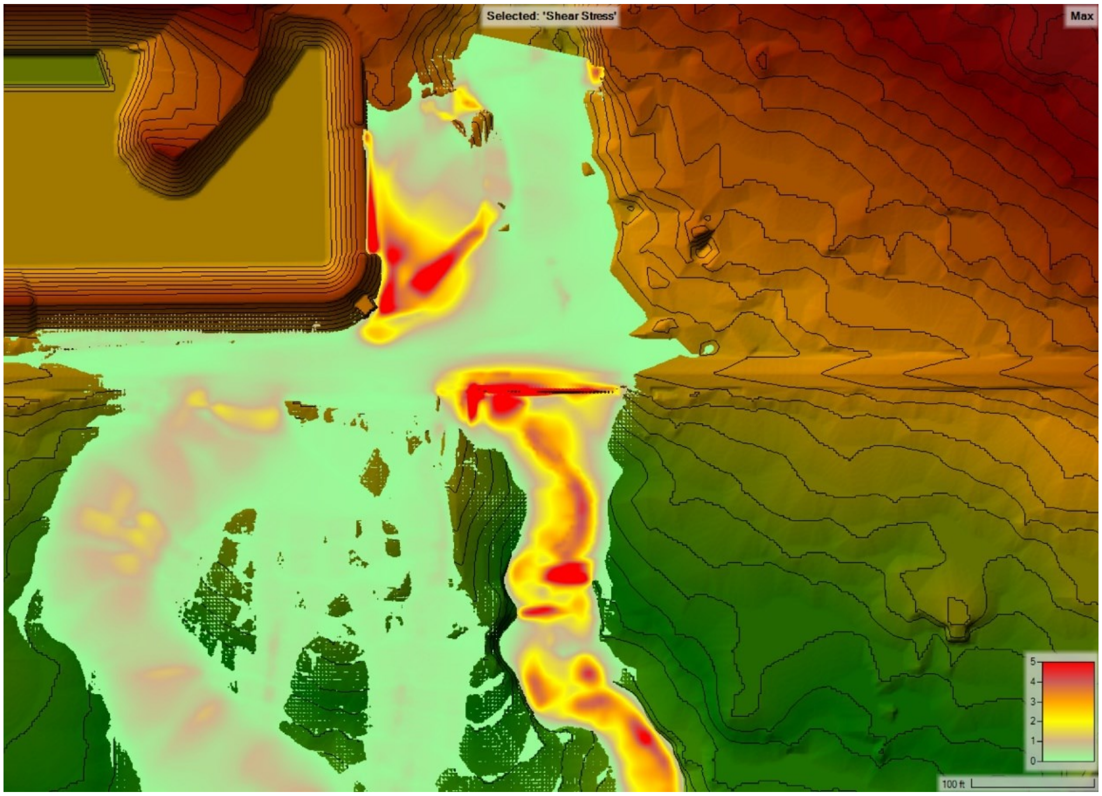


Figure 29 Image Existing Max Shear at 50-year Type-II

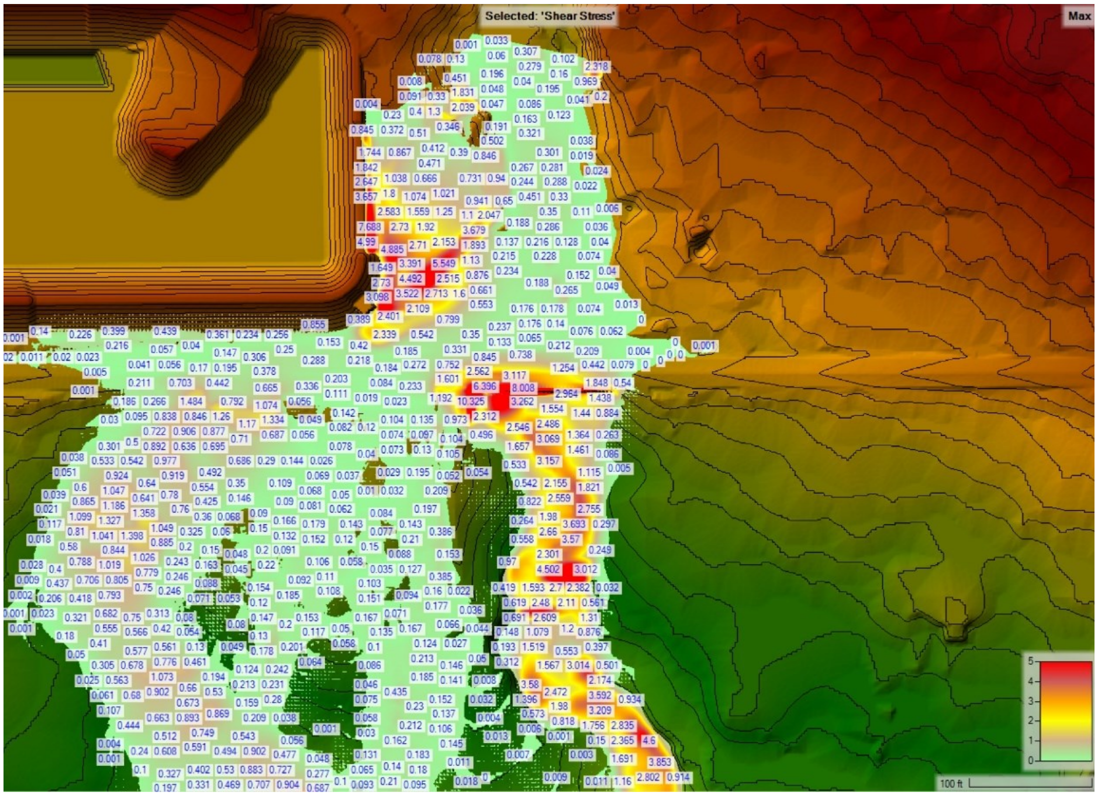


Figure 30 Image Existing Max Shear with Values at 50-year Type-II

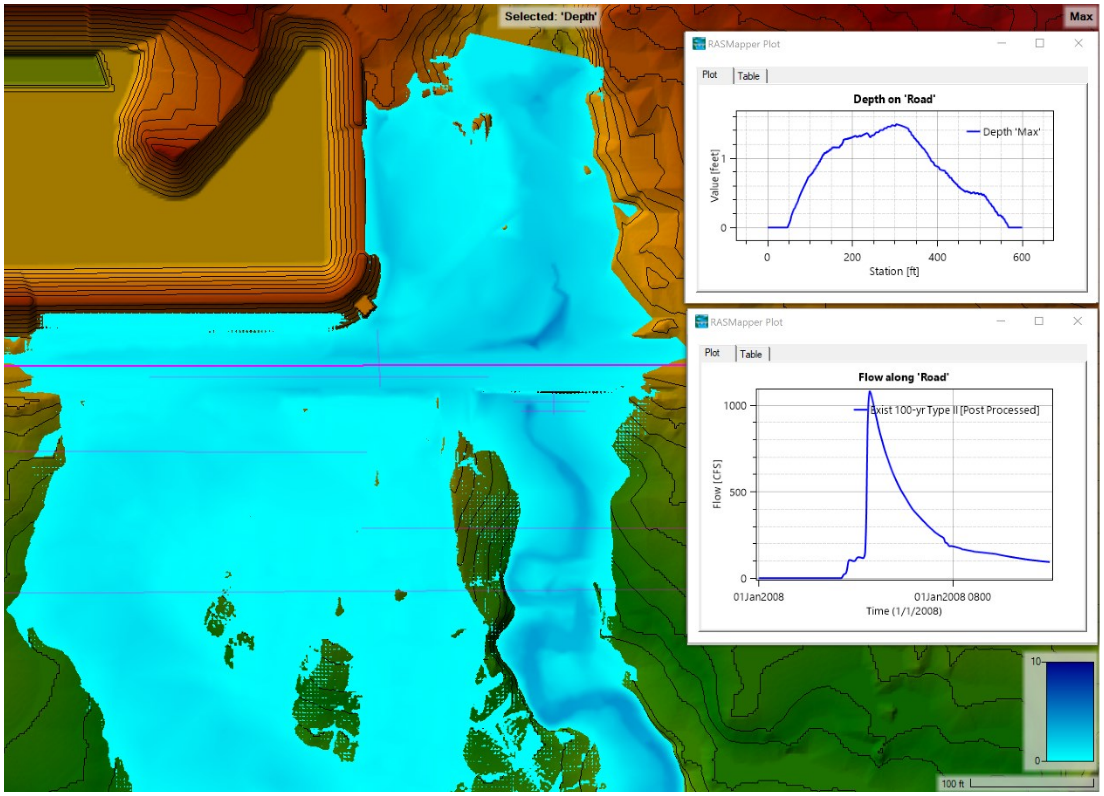


Figure 31 Image Existing Max Depth Over Road at 100-year Type-II

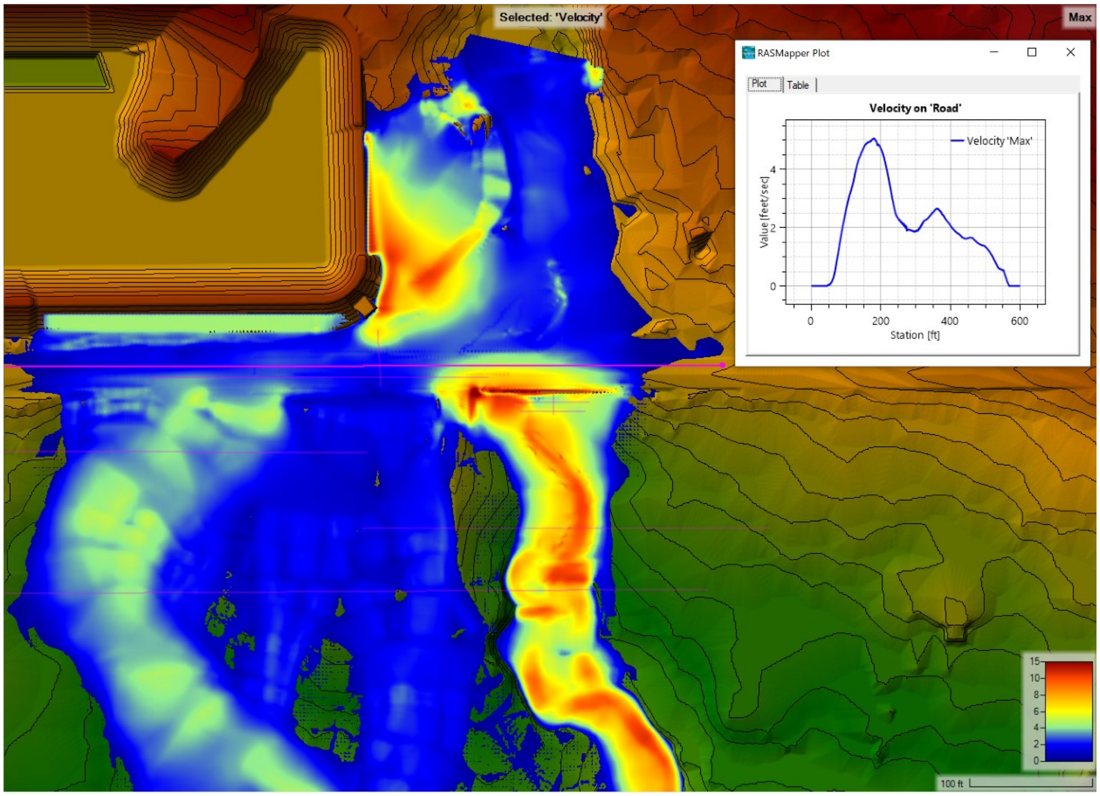


Figure 32 Image Existing Max Velocity Over Road at 100-year Type-II

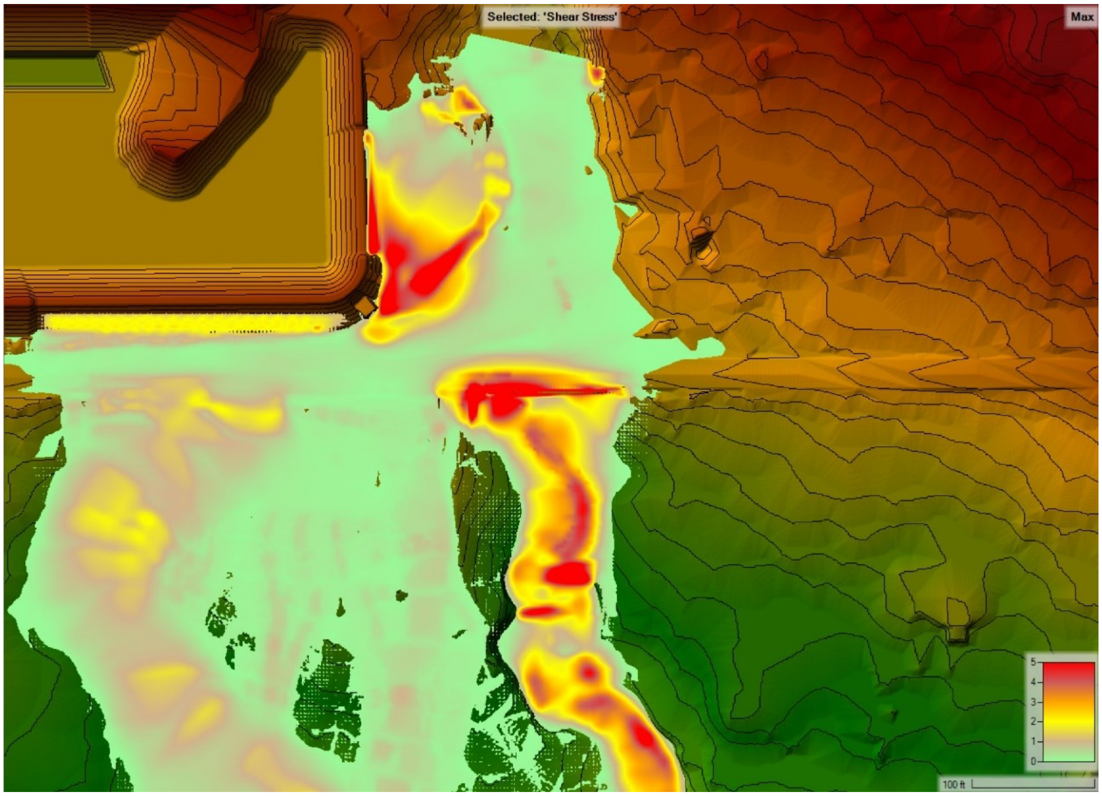


Figure 33 Image Existing Max Shear at 100-year Type-II

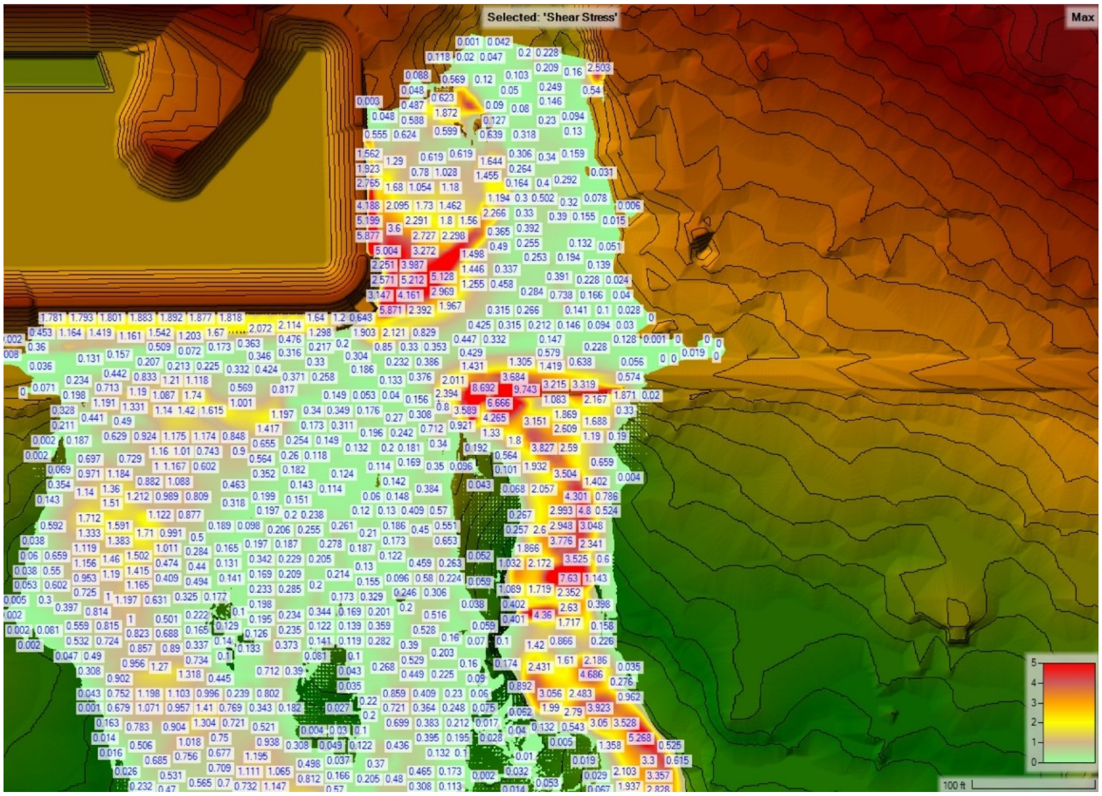


Figure 34 Image Existing Max Shear with Values at 100-year Type-II

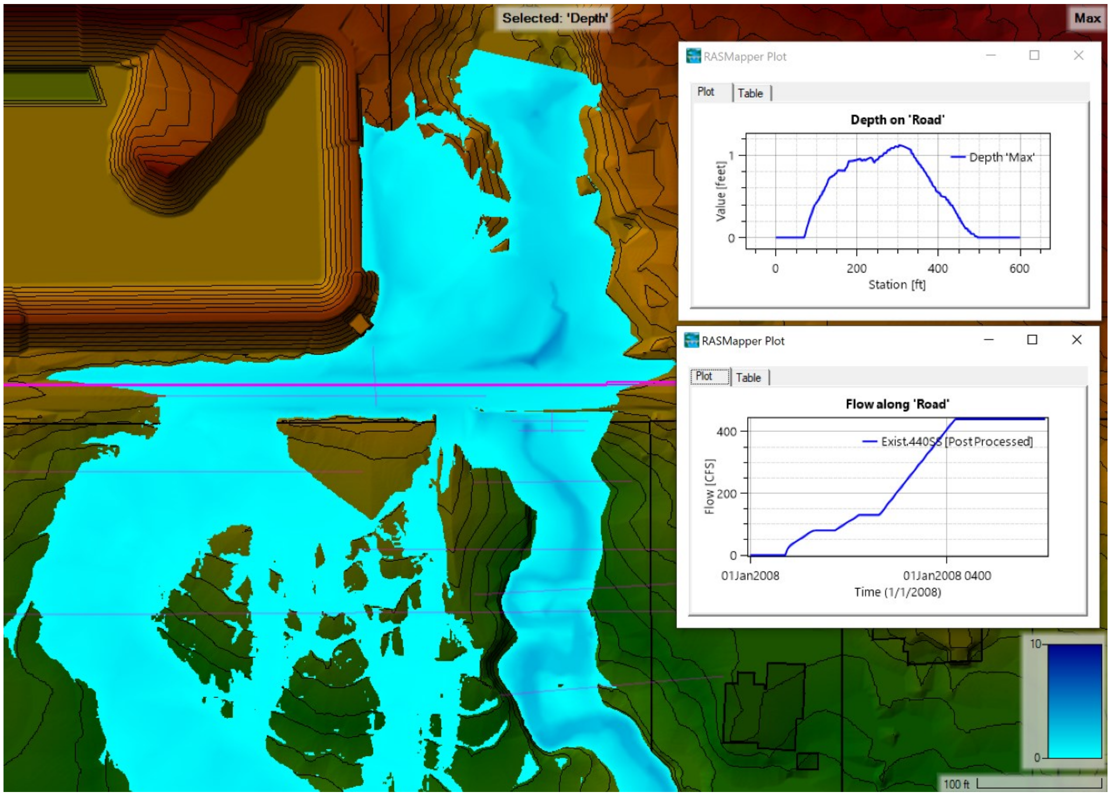


Figure 35: Image existing max depth over road at FEMA 100-yr regulatory flood (440 CFS)

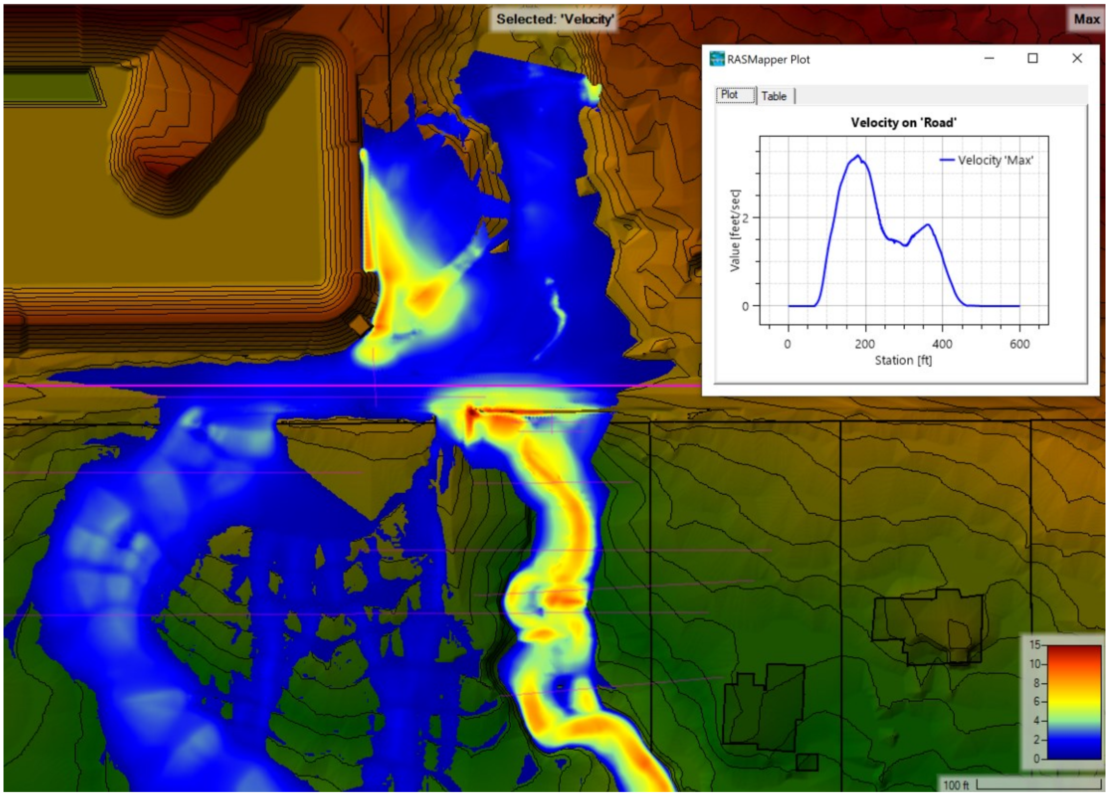


Figure 36: Image existing max velocity at FEMA 100-yr regulatory flood (440 CFS)

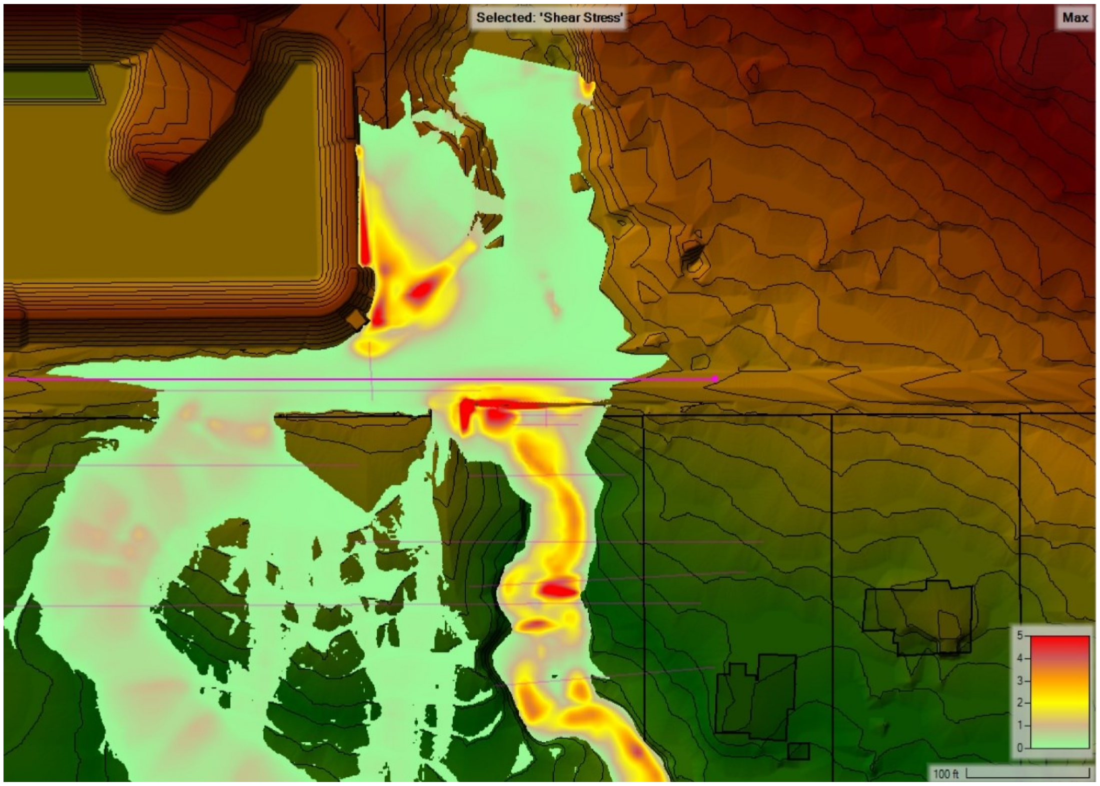


Figure 37: Image existing max shear at FEMA 100-yr regulatory flood (440 CFS)

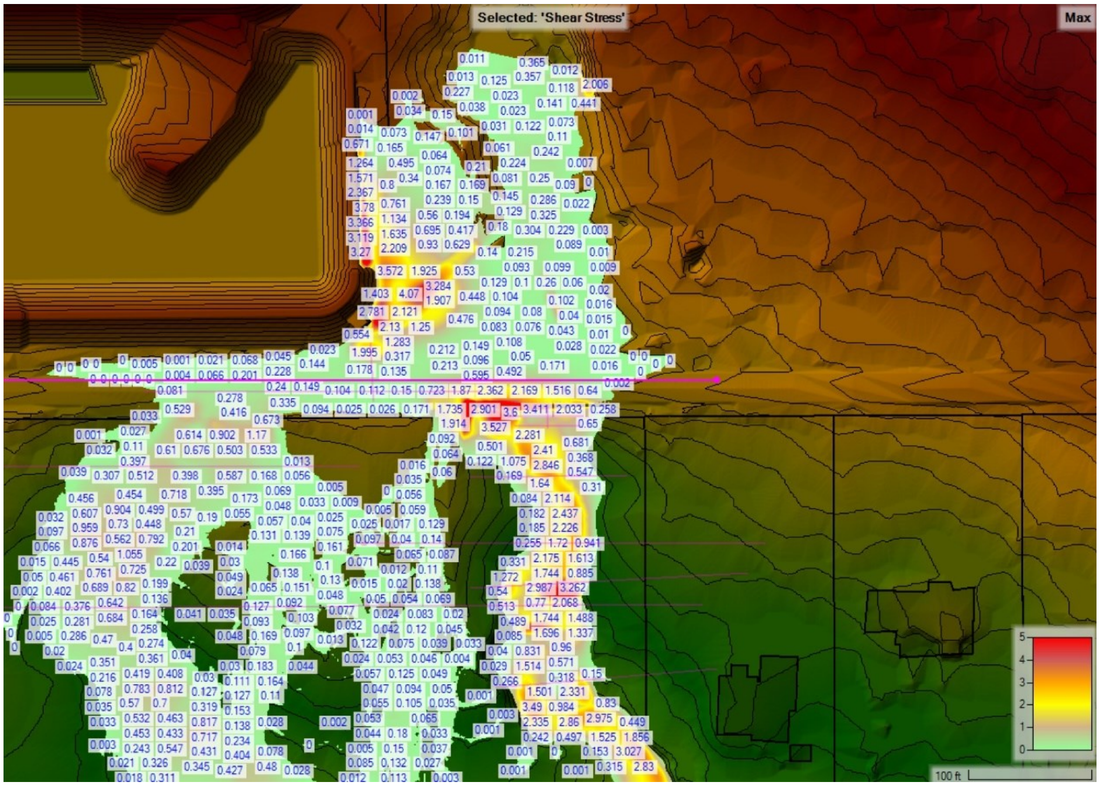


Figure 38: Image existing max shear at FEMA 100-yr regulatory flood (440 CFS) with values

<u>HEC-RAS Results – Screen Captures of Proposed Condition Model Results</u>

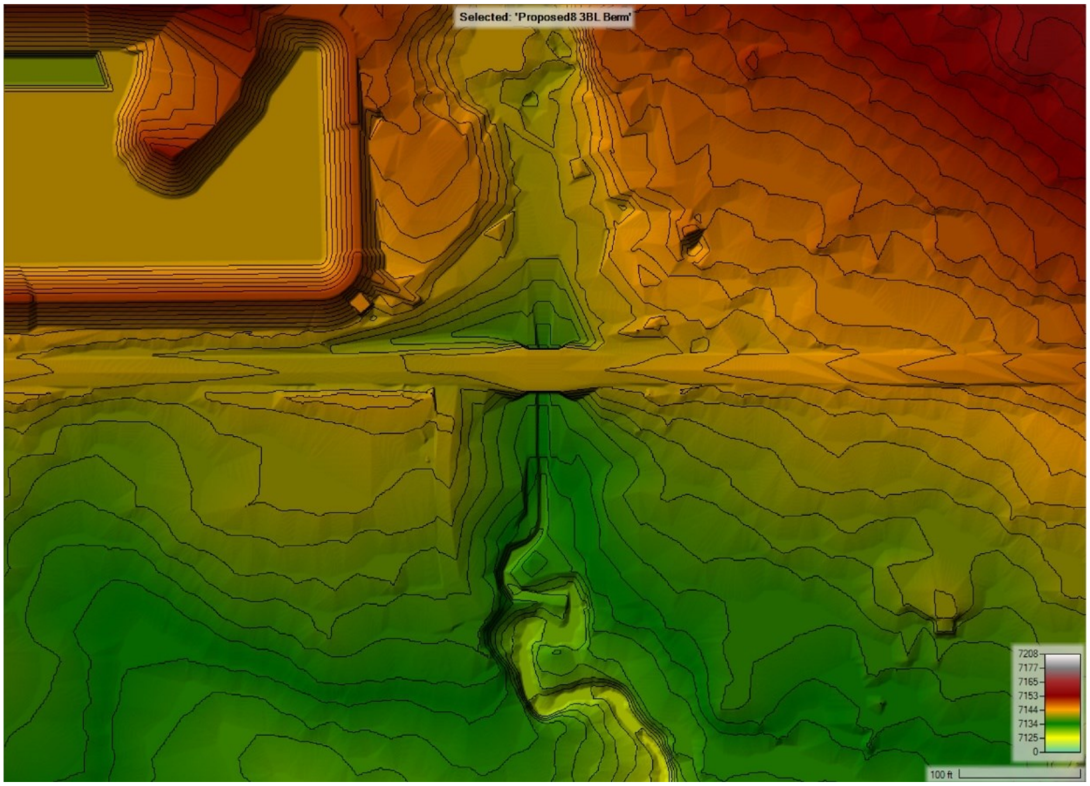


Figure 39 Image Proposed Terrain (No Flow)

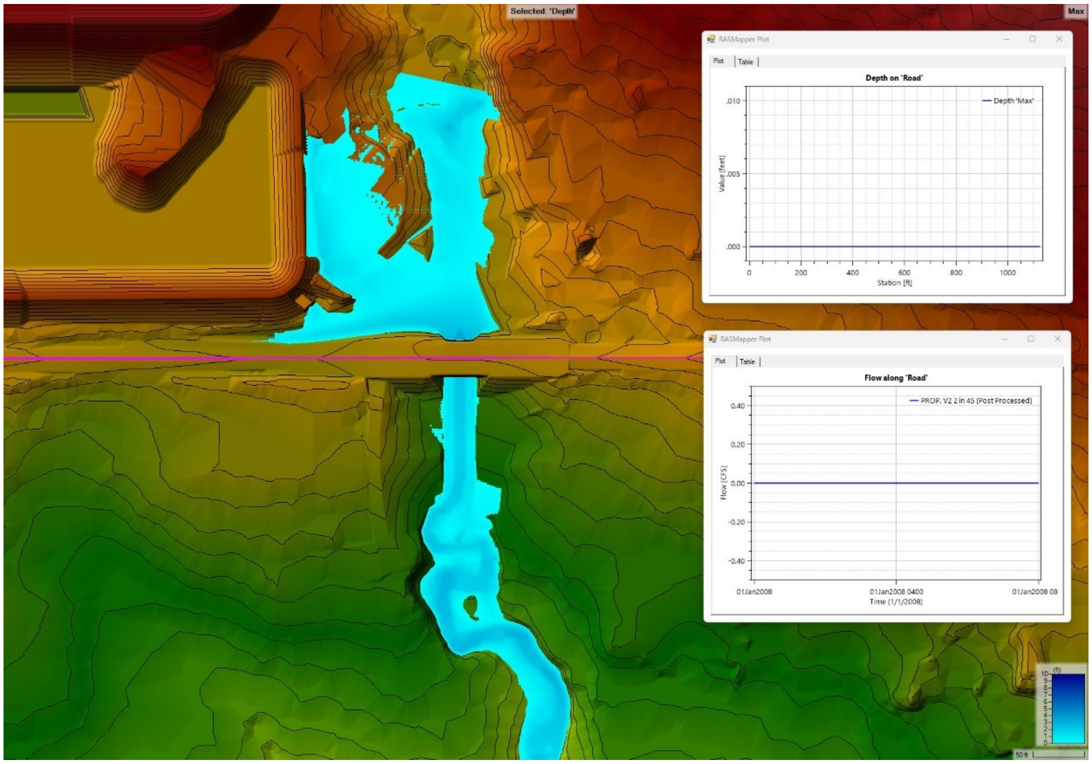


Figure 40 Image Proposed Max Depth Over Road at 2" in 45 Min.

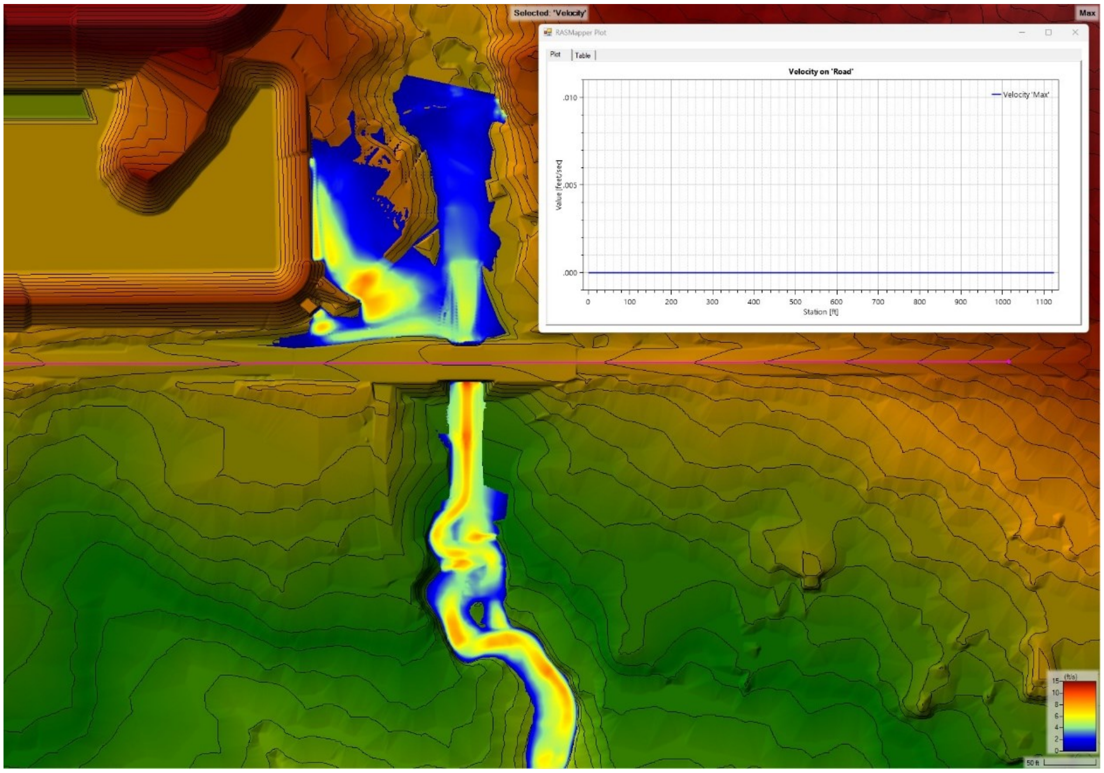


Figure 41 Image Proposed Max Velocity Over Road at 2" in 45 Min

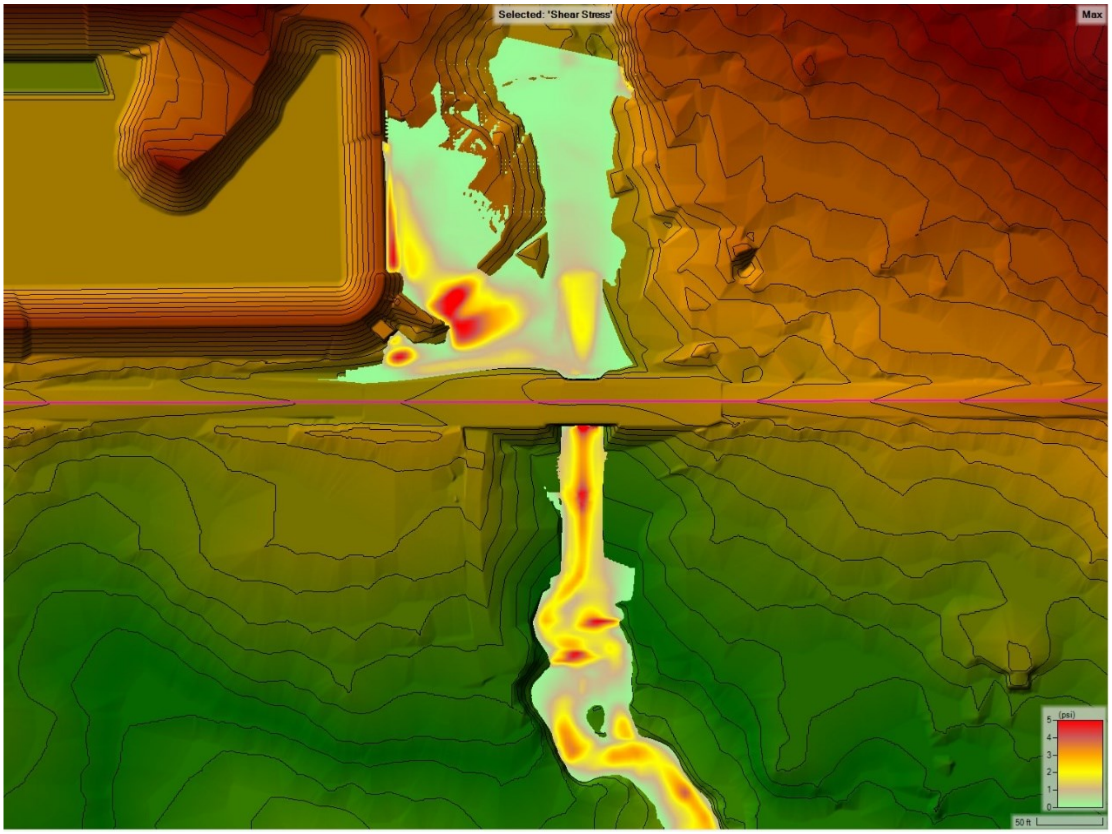


Figure 42 Image Proposed Max Shear at 2" in 45 Min.

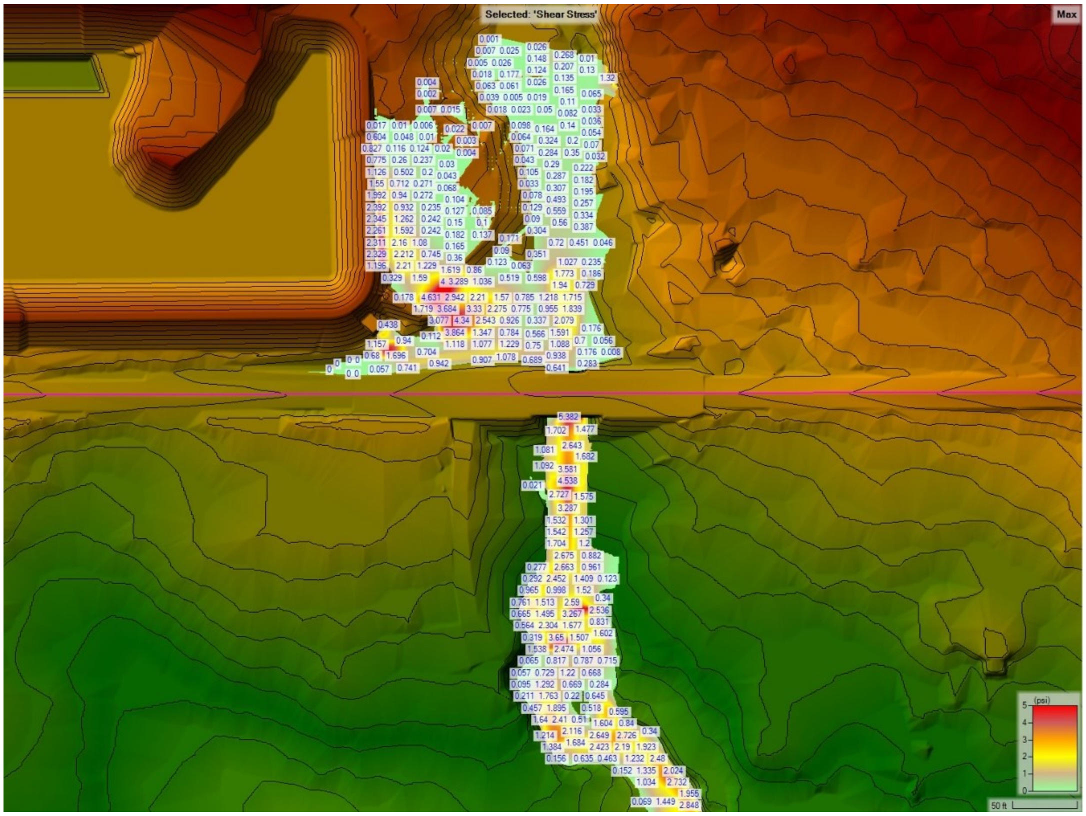


Figure 43 Image Proposed Max Shear with Values at 2" in 45 Min.

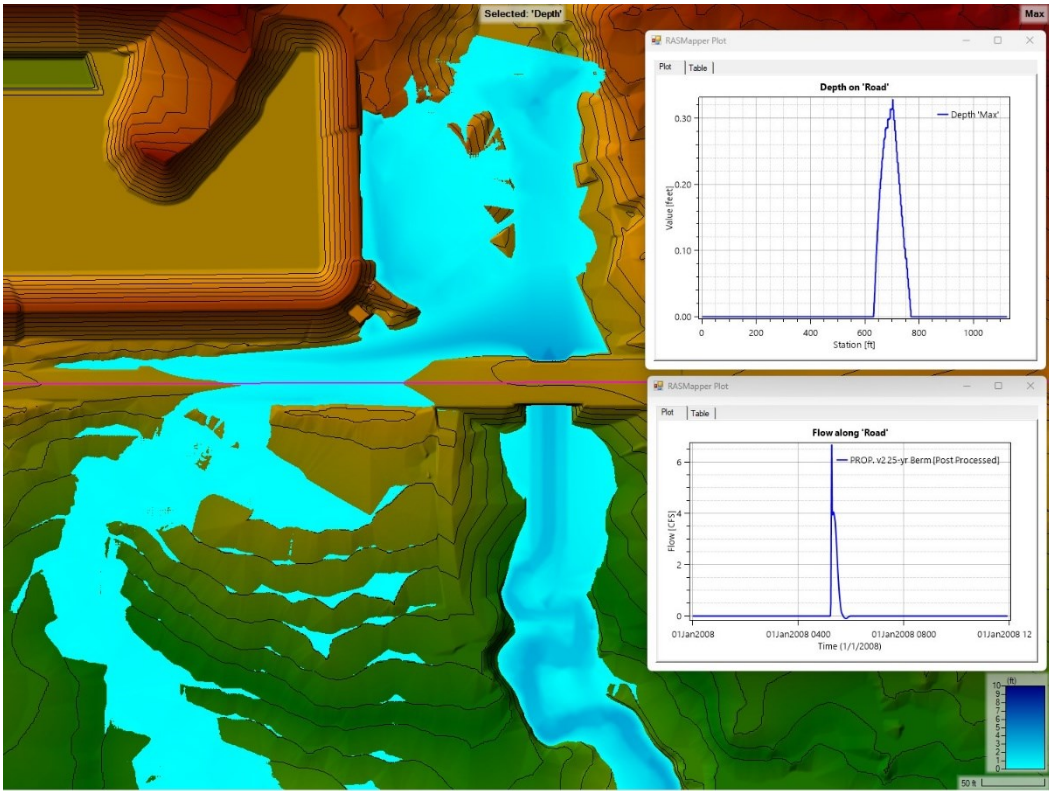


Figure 44 Image Proposed Max Depth Over Road at 25-year Type-II

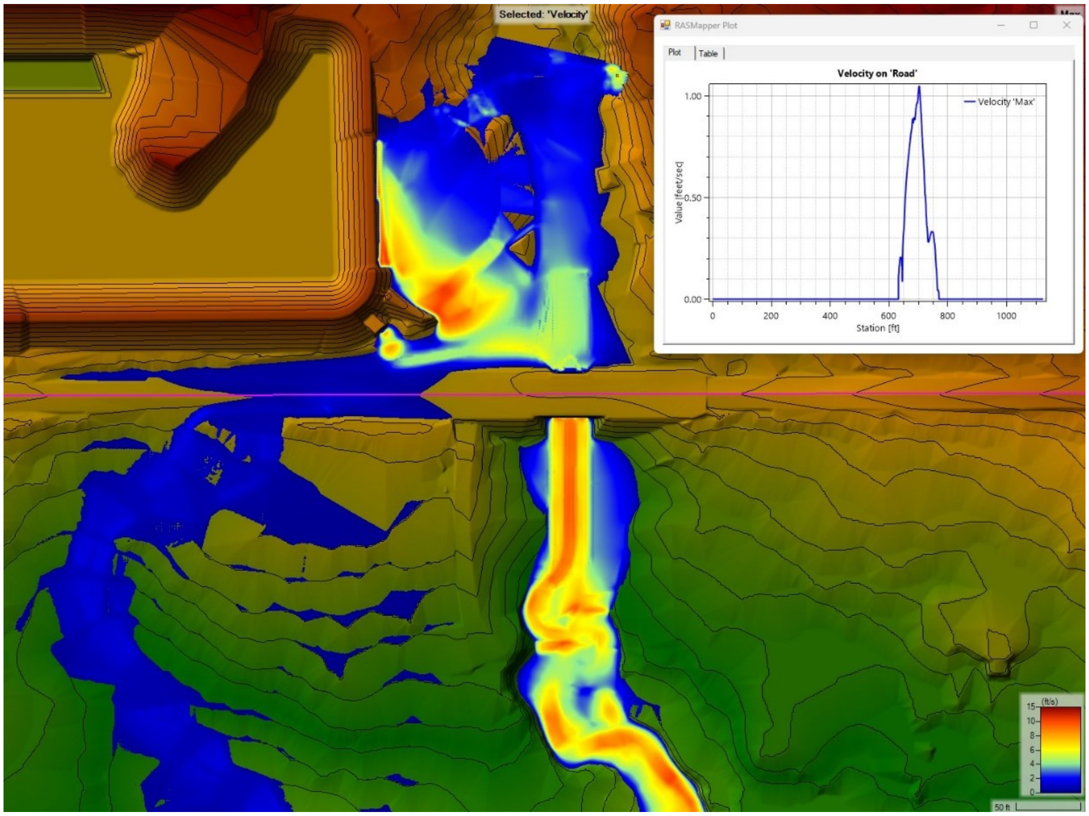


Figure 45 Image Proposed Max Velocity Over Road at 25-year Type-II

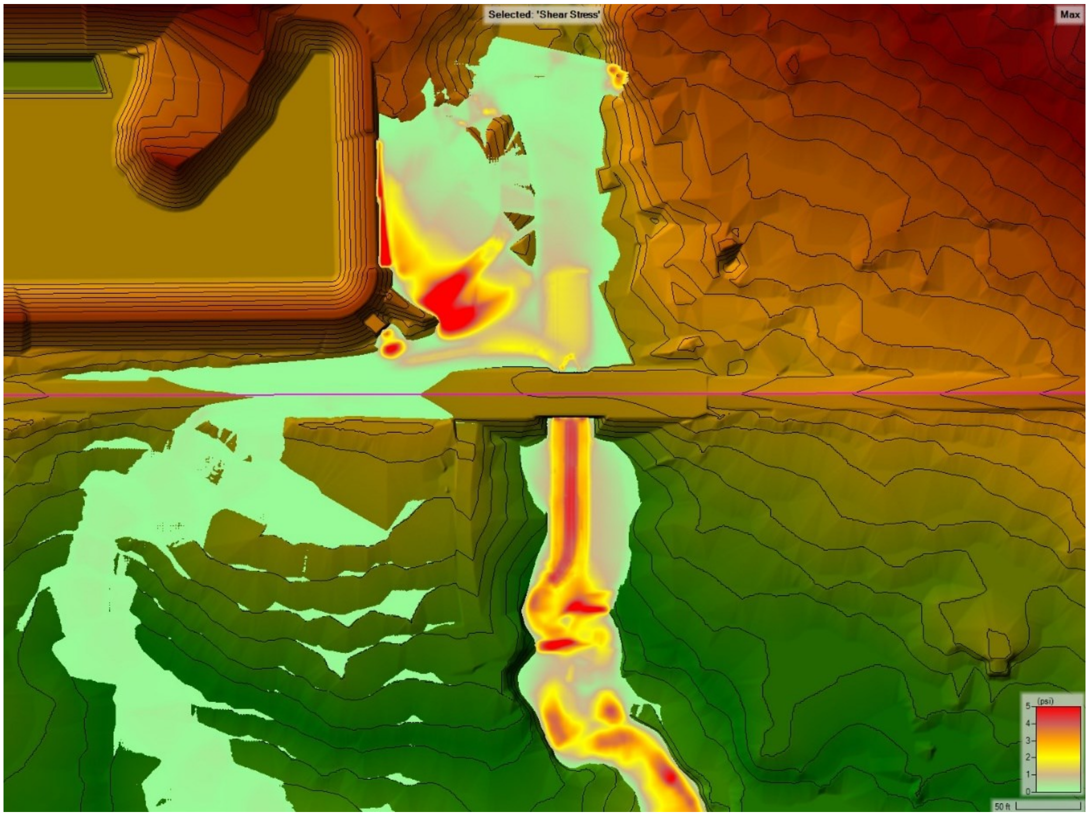


Figure 46 Image Proposed Max Shear at 25-year Type-II

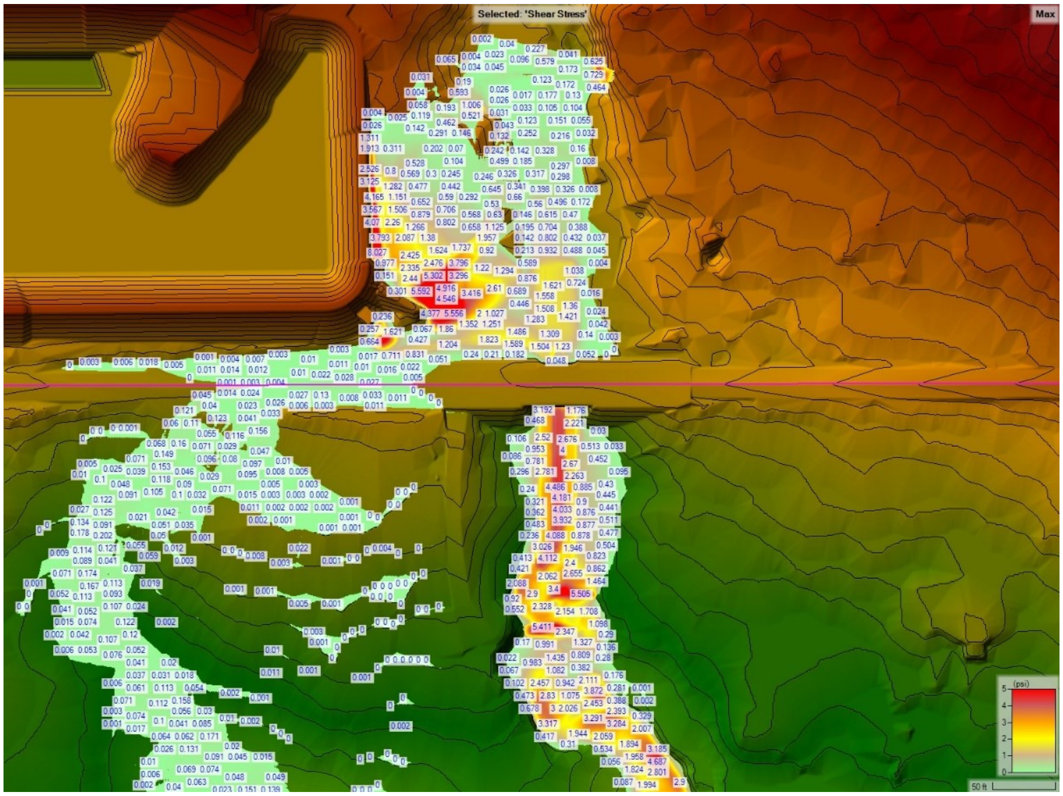


Figure 47 Image Proposed Max Shear with Values at 25-year Type-II

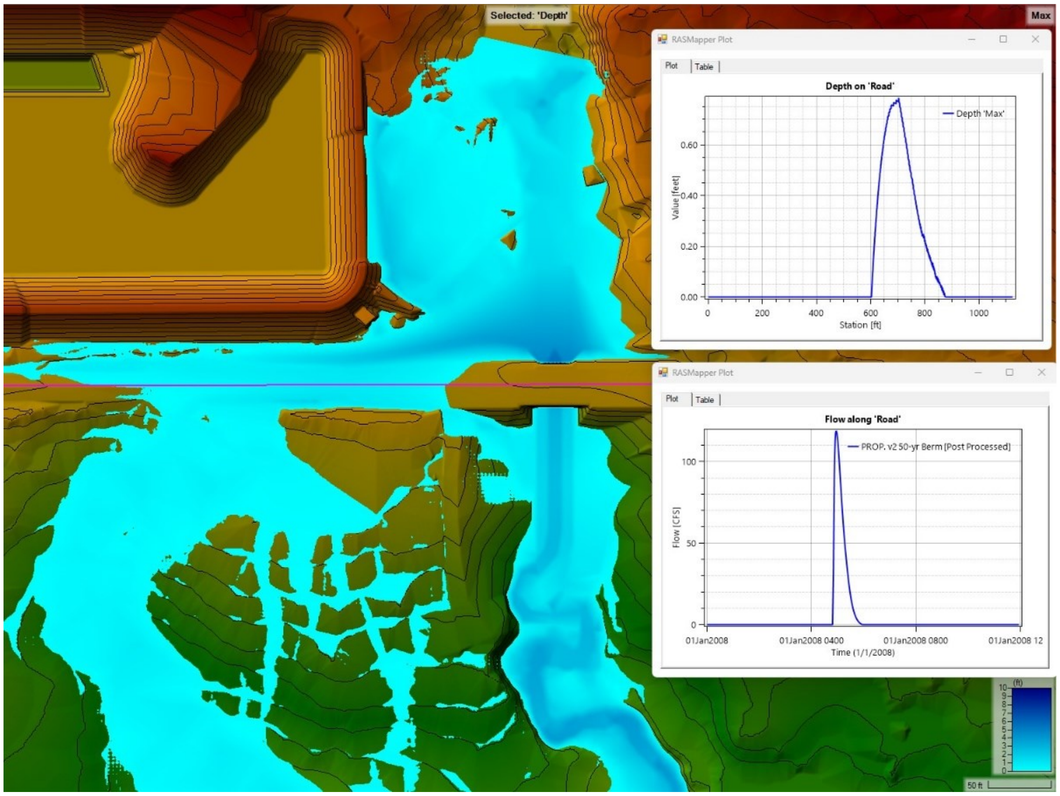


Figure 48 Image Proposed Max Depth Over Road at 50-year Type-II

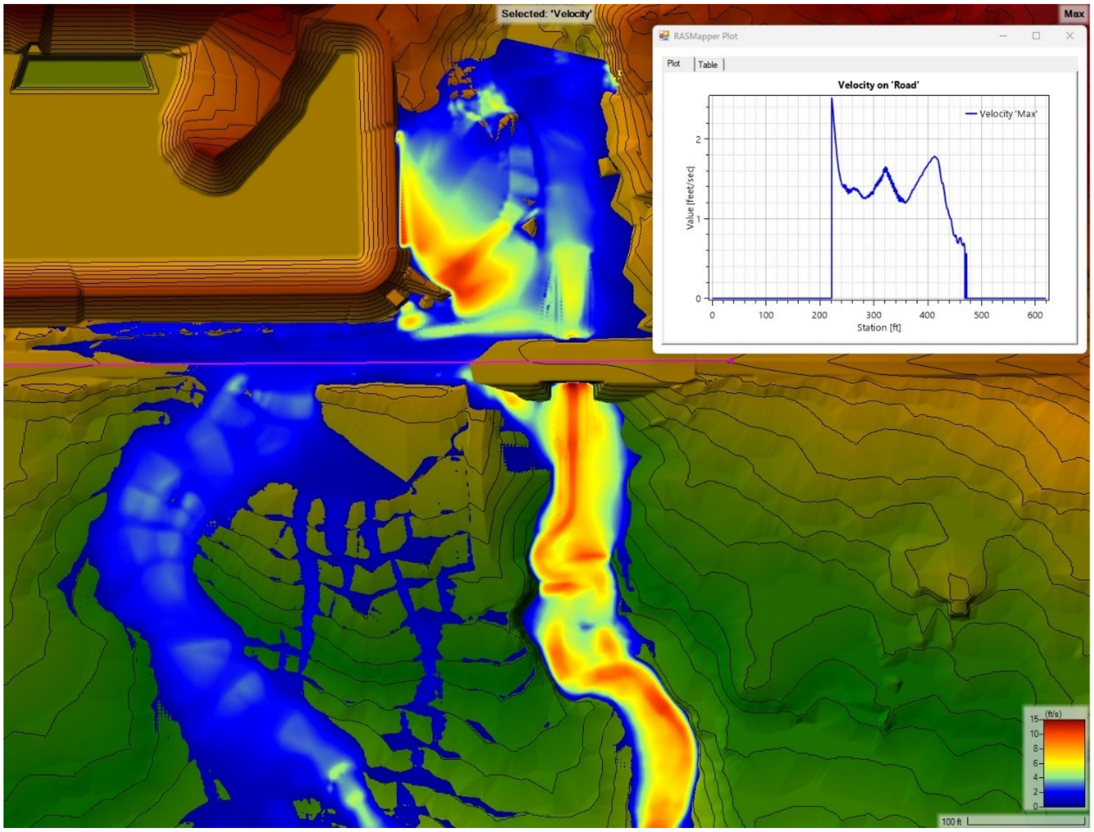


Figure 49 Image Proposed Max Velocity Over Road at 50-year Type-II

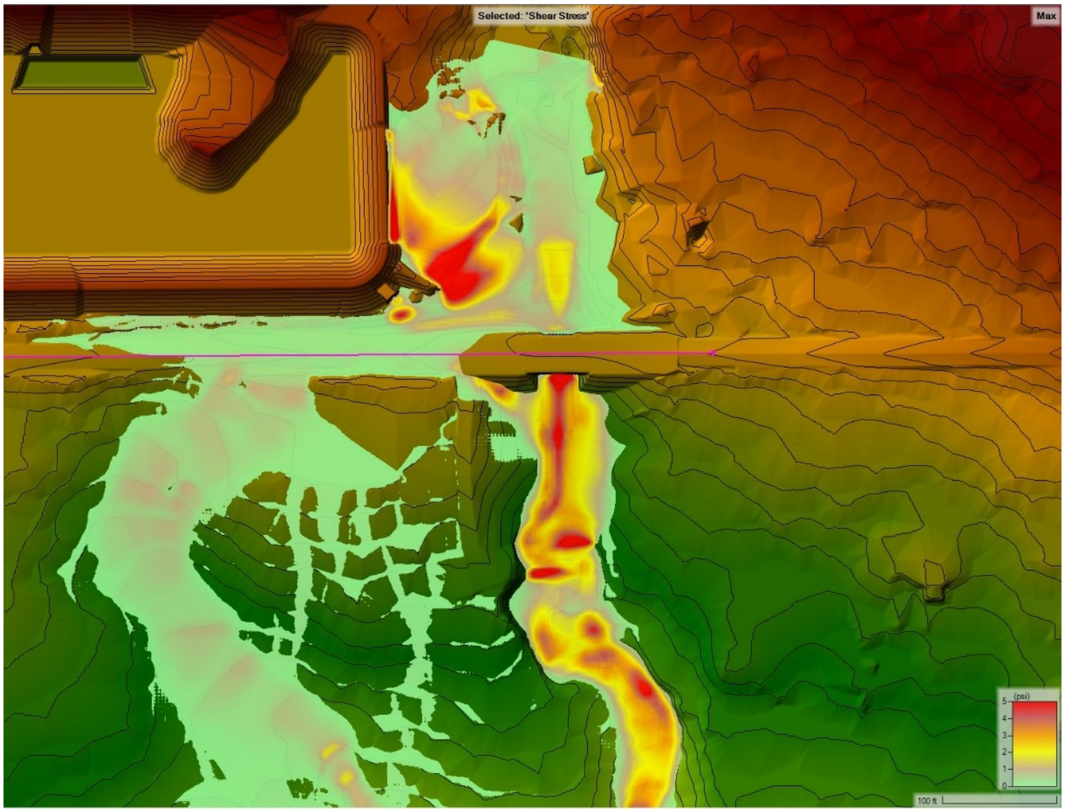


Figure 50 Image Proposed Max Shear at 50-year Type-II

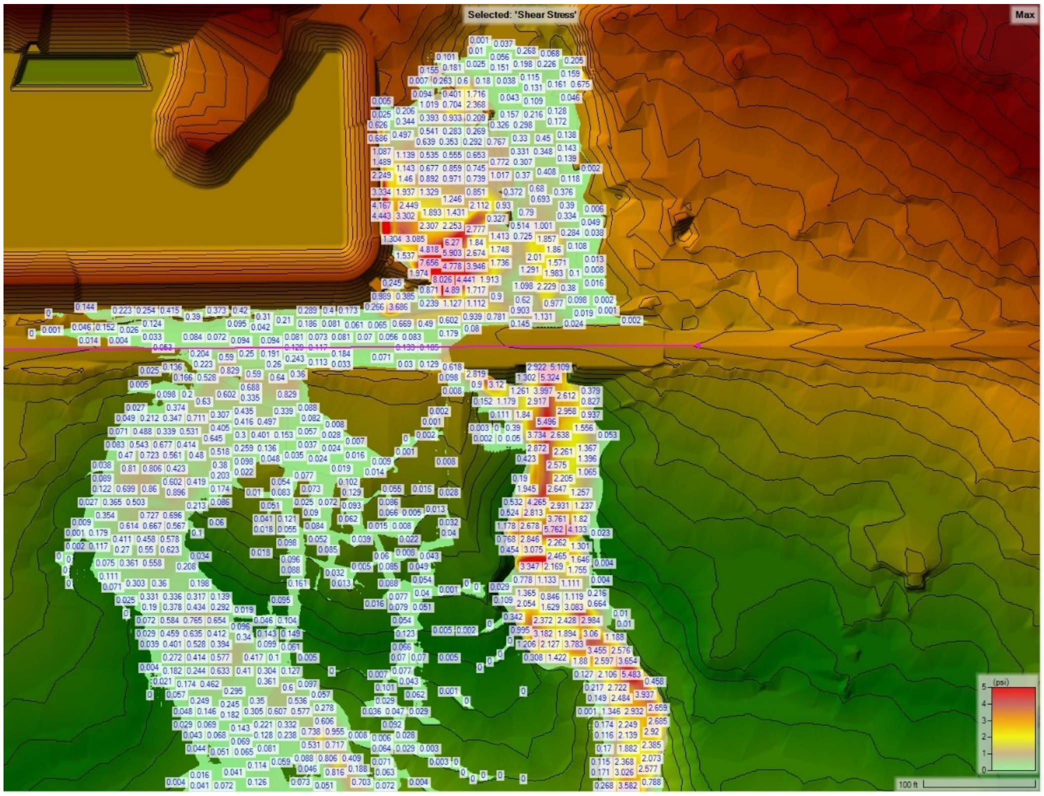


Figure 51 Image Proposed Max Shear with Values at 50-year Type-II

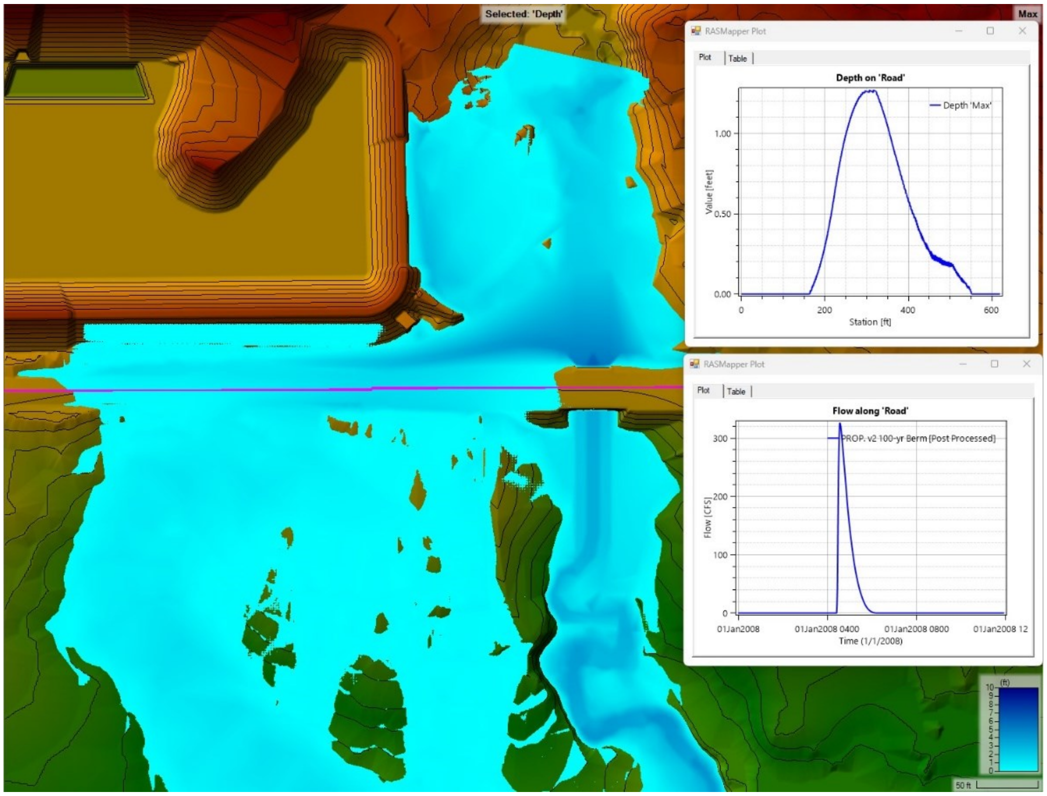


Figure 52 Image Proposed Max Depth Over Road at 100-year Type-II

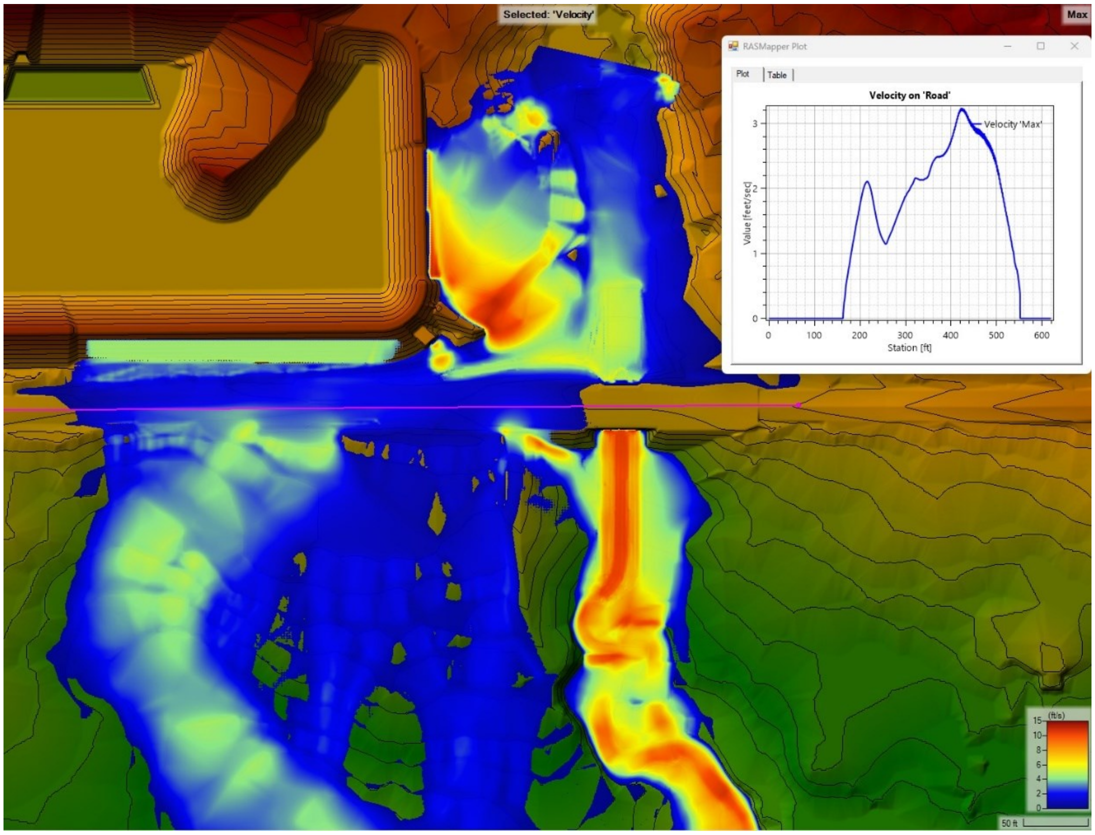


Figure 53 Image Proposed Max Velocity Over Road at 100-year Type-II

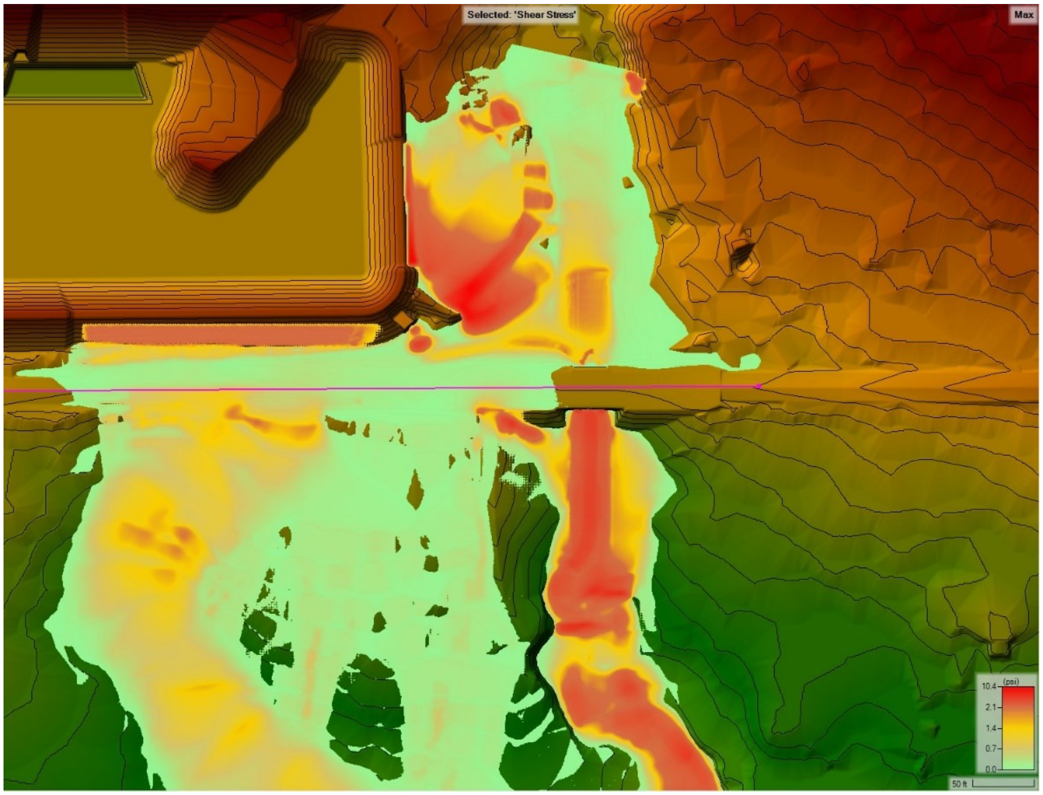


Figure 54 Image Proposed Max Shear at 100-year Type-II

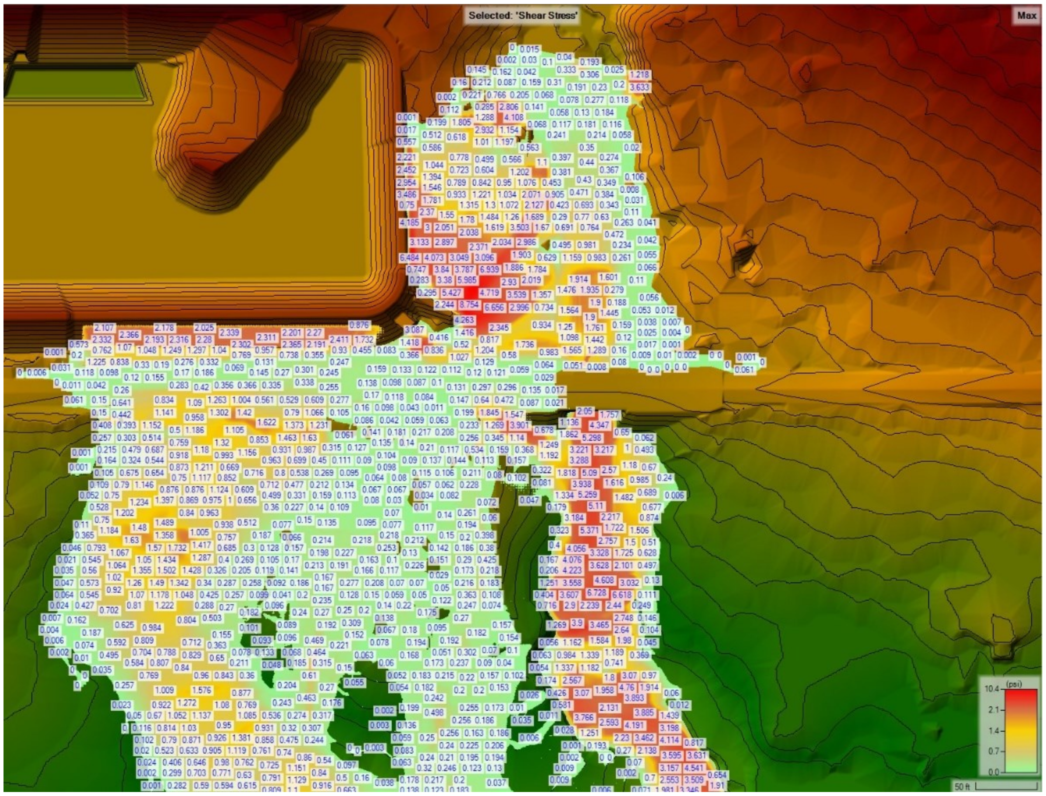


Figure 55 Image Proposed Max Shear with Values at 100-year Type-II

HEC-RAS Results – Comparisons of Road Overtopping Pre- to Post-Project

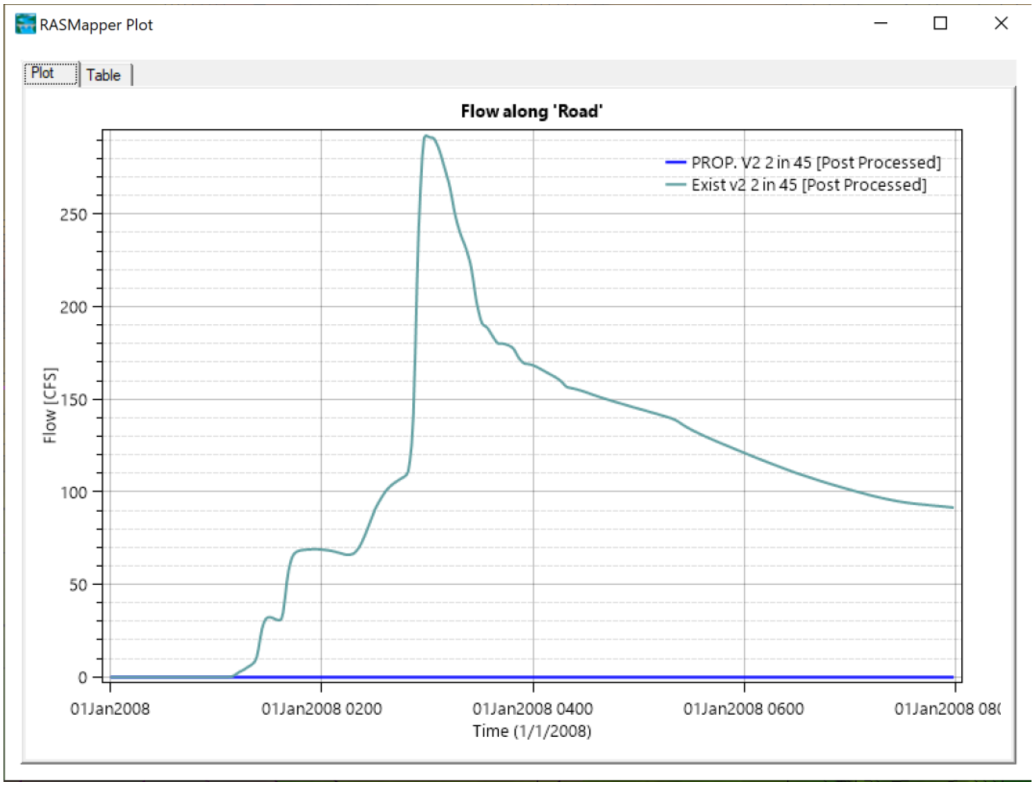


Figure 56 Chart Comparing Flow Over Road at 2" in 45 Min.

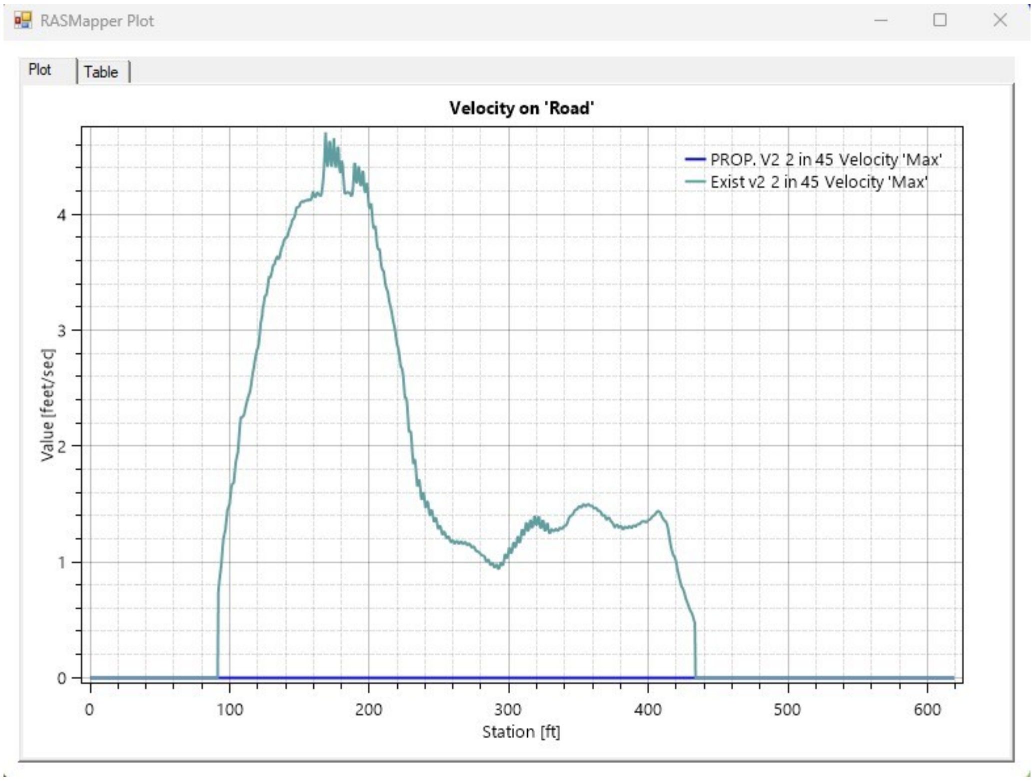


Figure 57 Chart Comparing Velocity Over Road at 2" in 45 Min.

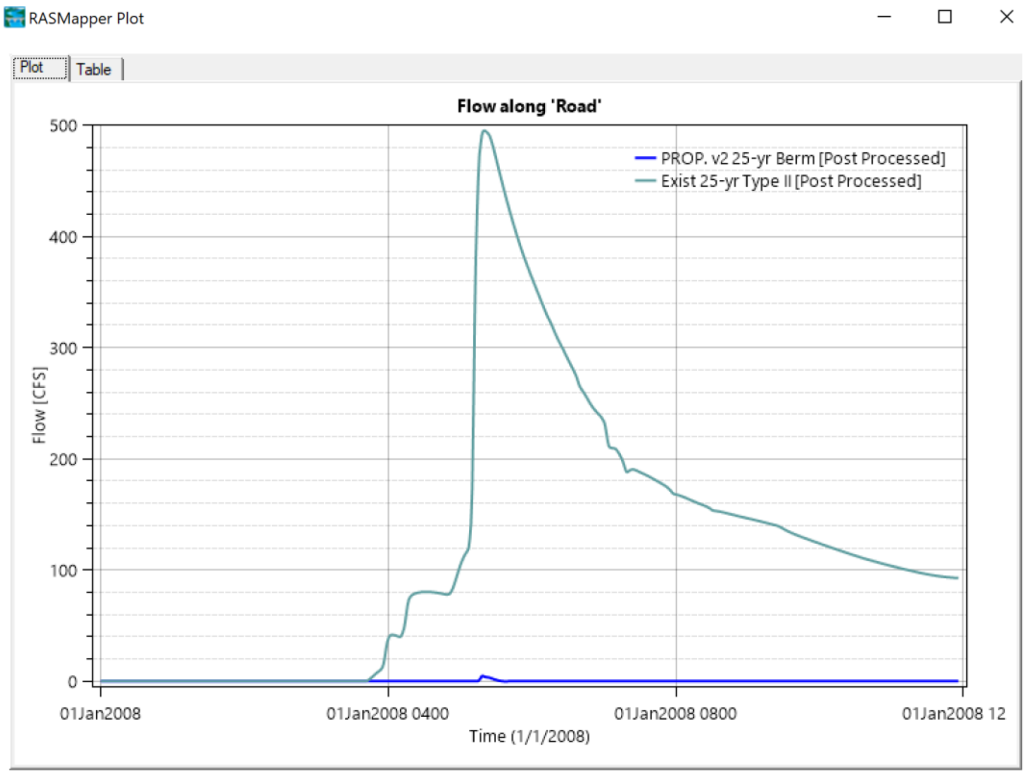


Figure 58 Chart Comparing Flow Over Road at 25-year Type-II

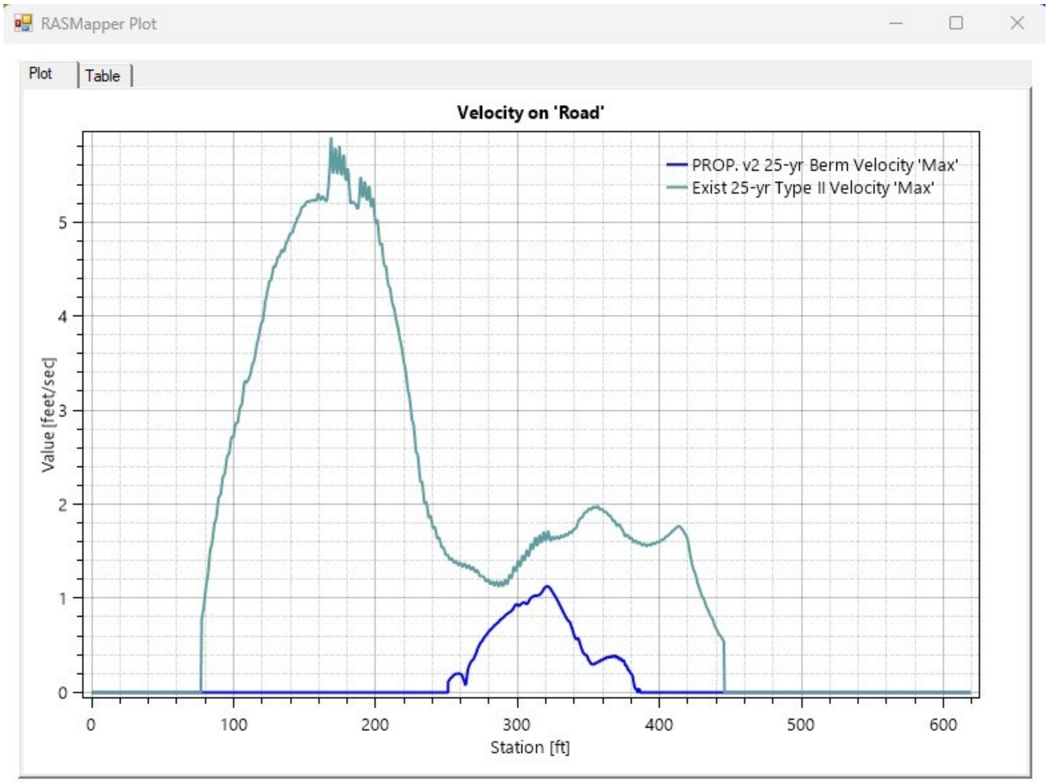


Figure 59 Chart Comparing Velocity Over Road at 25-year Type-II

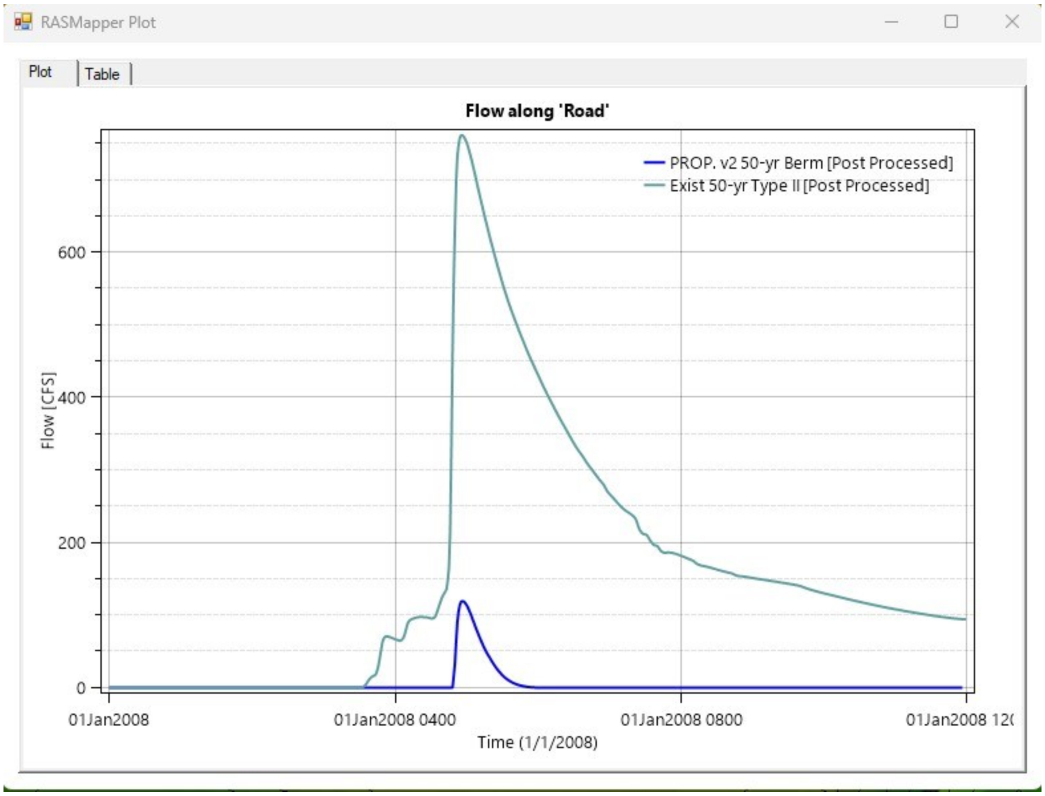


Figure 60 Chart Comparing Flow Over Road at 50-year Type-II

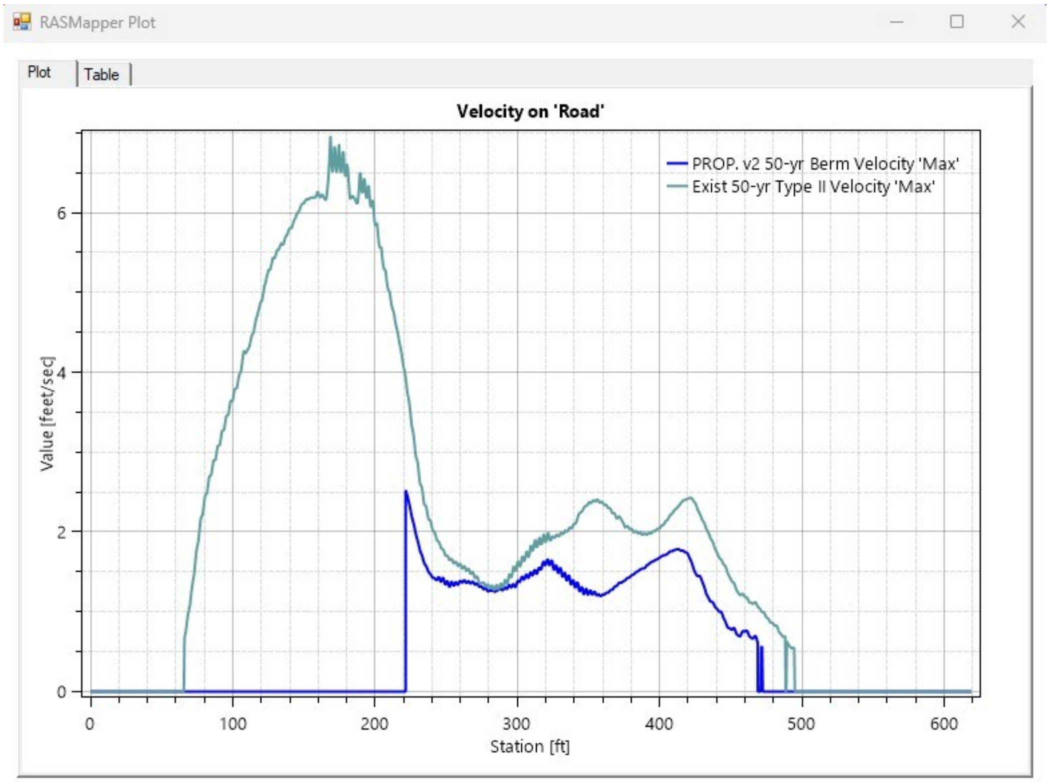


Figure 61 Chart Comparing Velocity Over Road at 50-year Type-II

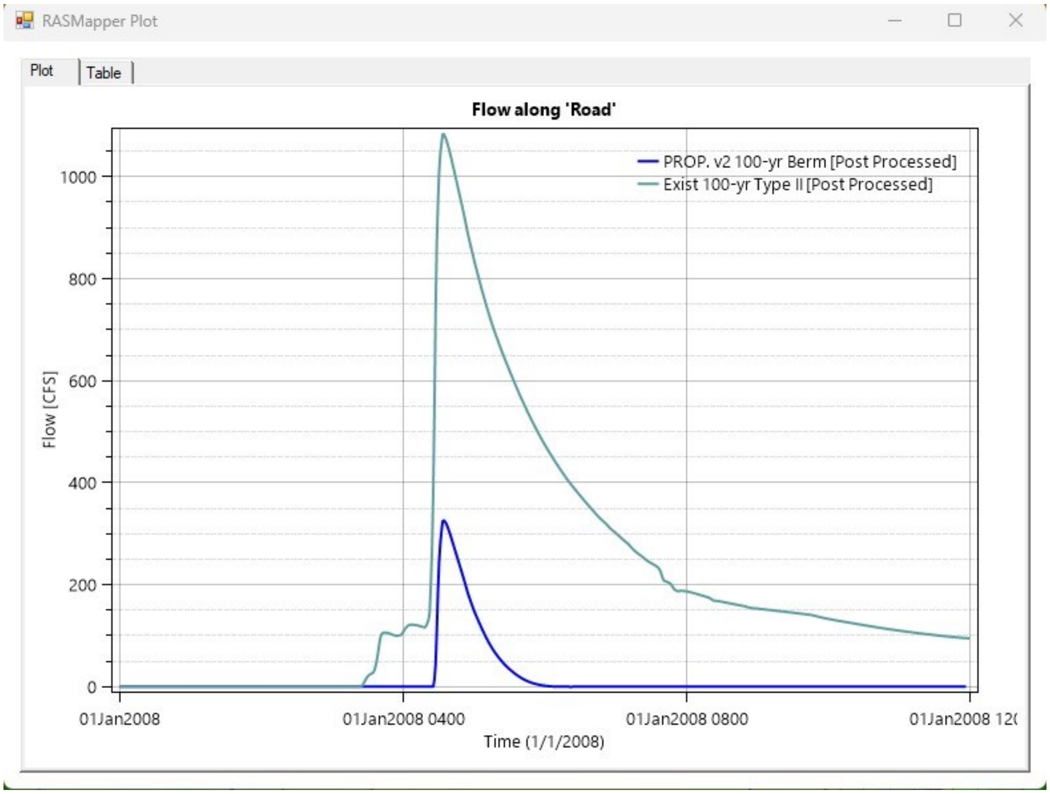


Figure 62 Chart Comparing Flow Over Road at 100-year Type-II

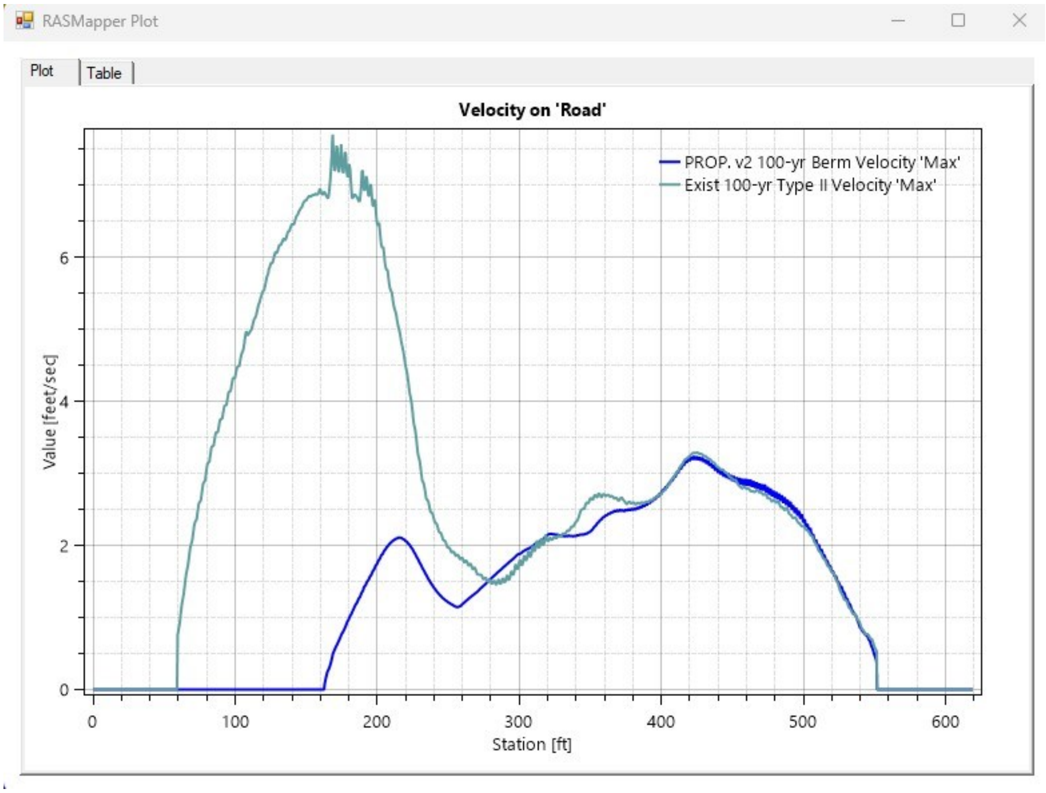


Figure 63 Chart Comparing Velocity Over Road at 100-year Type-II

Rock and Riprap Sizing

Natural rock riprap will be placed within the newly graded channels and bank slopes upstream of the concrete box culvert. The hydraulics of this area are complex with multiple inflow hydrograph locations (see Appendix Pg. A1) such that several methodologies were used to size the feature rocks and riprap that will placed upstream of the culverts.

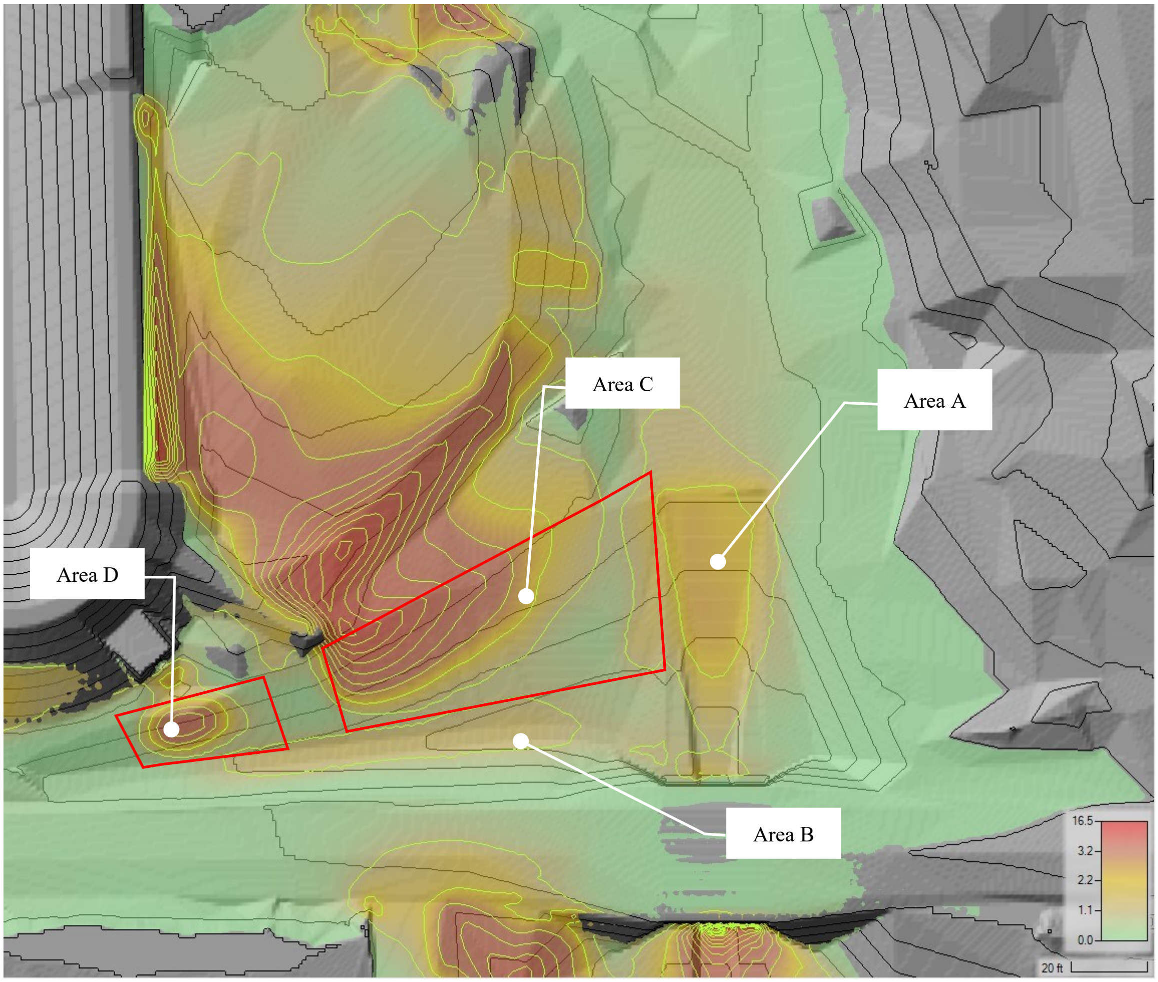


Figure 64: HEC-RAS Model showing Shear Stress During 100yr - Type II event
Riprap in the areas notes were sized per various methodologies noted below

Area A: Rock Structure Sizing

Area A experiences flows from the unburned portion of the Spruce Wash watershed located to the east of the mainstem of the channel that has otherwise been cut off from the main flow by the recent construction of the City of Flagstaff Detention Bains.

Rock sills have been specified for Area A, rather than a full carpet of riprap. Rock sills will allow for large areas of soil between the rock that will revegetate and allow the channel to recover to a more natural state.

Sizing of the large rock sills that will be installed is accomplished with consideration of the maximum observed shear stresses during the various analyzed flow events and a relationship between critical shear stress and initiation of movement in various grain sizes of bedload developed by Wildland Hydrology.

Error! Reference source not found. on the next page provides a relationship between critical s hear stress and initiation of movement in bedload particles. Using the higher curve (Colorado Data) and the shear stresses associated with the modeled flow events (~2.3 lb/ft^2 during the 50yr event), bedload materials of approximately 12” should not experience movement.

If the flow conditions were to be experienced exactly as the clear water flow, 2D model results indicate, 12” rock would likely be sufficient. However, this section of channel is subject to chaotic flow conditions when the spillway of the City of Flagstaff detention basins become active. Floating logs and other debris may be included within such flood flows which may interfere with the flow path across the rock structures and concentrate shear stresses.

Given this uncertainty, we have chosen to apply a factor of safety of 2.5 and have chosen 30” diameter feature rocks with 18” footer rocks for the rock sills in Area A.

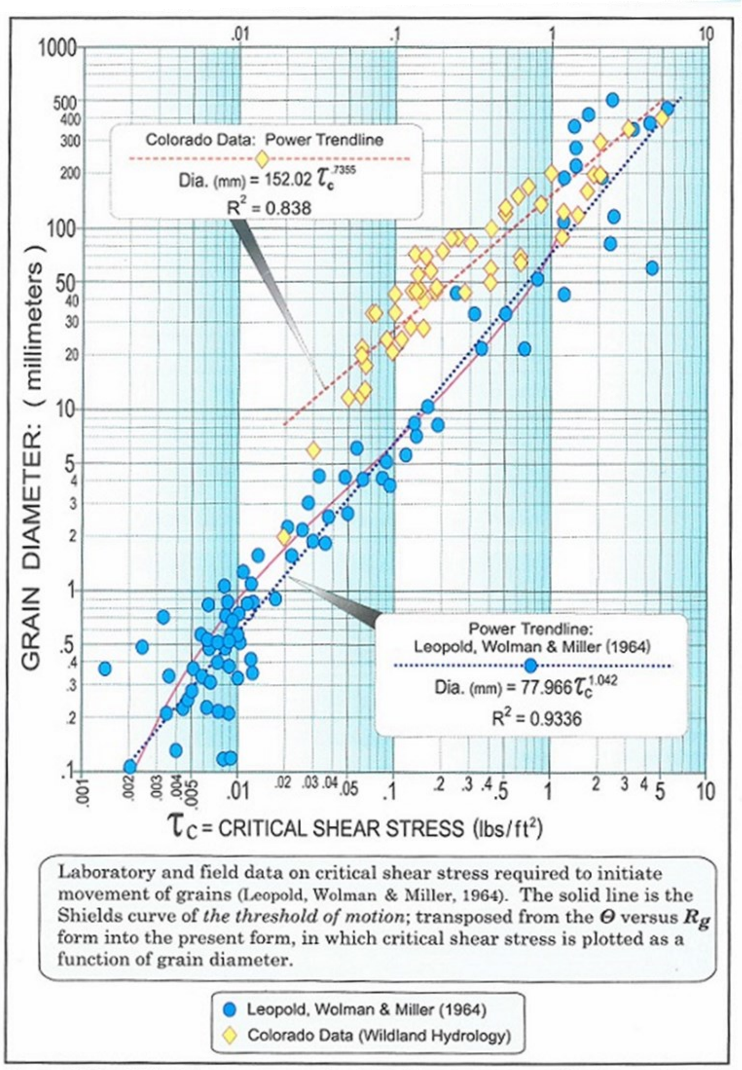
Area B: Riprap Sizing

Area B represents the reconstructed roadside ditch along the north side of the road. This ditch will capture flows from a culvert outlet and an overtopping spillway from the city detention basins. The D_{50} of the riprap was chosen from a relationship developed by Wildland Hydrology that plots shear stress required to initiate movement in bed material grains (Figure65). Maximum shear stresses within Area B indicated by the model results are approximately 1.9 lb/ft^2 which translates on the curve to a particle size of 10 inches. Applying a FS=1.2 to this calculation and using that for the D50 of riprap, results in a 24” thick, D50=12” riprap section necessary for Area B.

Area C and D: Riprap Sizing

Areas C and D are tie slopes of the roadside ditch that will receive flow from the City of Flagstaff detention basin outlets. Area C receives flow from the overflow spillway of the nearest basin and Area D receives flow from a CMP outlet pipe of that same basin. Although the riprap will serve as bank protection for the channel indicated by Area B, when the detention basin spillway becomes active, the slopes of Areas C and D will act similarly to a rock-lined chute.

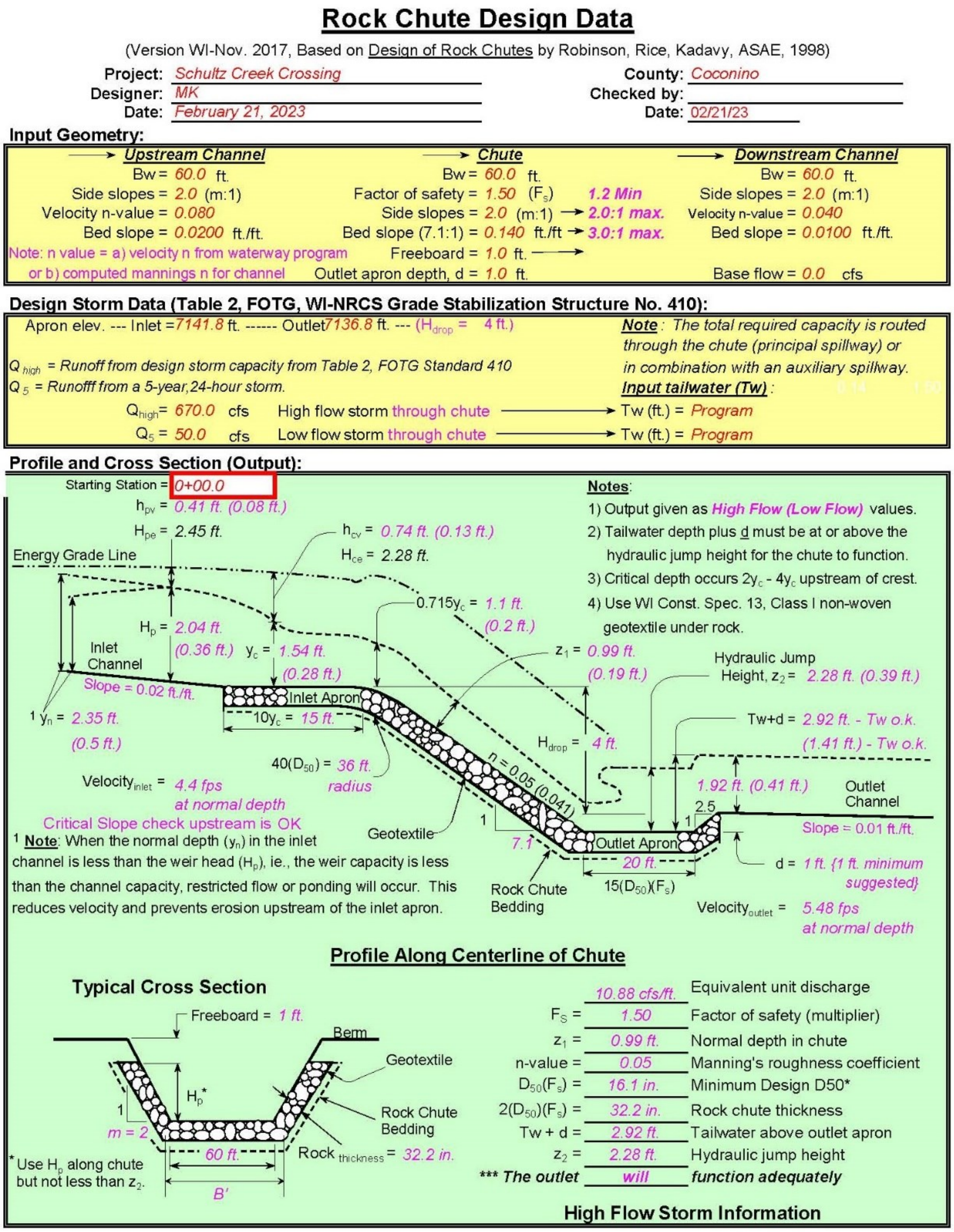
Riprap sizing for these areas was based upon modeling the slope down into the ditch using the NRCS Rock Chute Calculator spreadsheet (Figure 67). The construction of the slope and riprap placement will not be an exact application of the spreadsheet, but it is a good approximation for the purpose of sizing the D_{50} of the riprap for the slopes experiencing flow from top to bottom across them. Areas C and D experience similar shear stress per the HEC-RAS model results. The slope of area C was modeled using the spreadsheet and the resulting D_{50} used for both areas.



Laboratory and field data on critical shear stress required to initiate movement of grains (Leopold, Wolman & Miller, 1964). The solid line is the Shields curve of *the threshold of motion*; transposed from the Θ versus R_g form into the present form, in which critical shear stress is plotted as a function of grain diameter.

Figure 3-11. Critical shear stress required to initiate movement of bed-material grains following the Shields relation (Leopold *et al.*, 1964), as modified by field data from Colorado (Rosgen, 2006b; Rosgen and Silvey, 2007).

Figure 65 - Initiation of Particle Size Movement as a Function of Critical Shear Stress



Rock Chute Design Data

(Version WI-Nov. 2017, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: *Schultz Creek Crossing*
Designer: *MK*
Date: *February 21, 2023*
County: *Coconino*
Checked by:
Date: *02/21/23*

Input Geometry:

Upstream Channel
Bw = *60.0* ft.
Side slopes = *2.0* (m:1)
Velocity n-value = *0.080*
Bed slope = *0.0200* ft./ft.
Note: n value = a) velocity n from waterway program or b) computed mannings n for channel

Chute
Bw = *60.0* ft.
Factor of safety = *1.50* (F_s) *1.2 Min*
Side slopes = *2.0* (m:1) → *2.0:1 max.*
Bed slope (7.1:1) = *0.140* ft./ft → *3.0:1 max.*
Freeboard = *1.0* ft.
Outlet apron depth, d = *1.0* ft.

Downstream Channel
Bw = *60.0* ft.
Side slopes = *2.0* (m:1)
Velocity n-value = *0.040*
Bed slope = *0.0100* ft./ft.
Base flow = *0.0* cfs

Design Storm Data (Table 2, FOTG, WI-NRCS Grade Stabilization Structure No. 410):

Apron elev. --- Inlet = *7141.8* ft. ------ Outlet *7136.8* ft. --- (H_{drop} = 4 ft.)

Q_{high} = Runoff from design storm capacity from Table 2, FOTG Standard 410
Q_5 = Runofff from a 5-year,24-hour storm.

Q_{high} = *670.0* cfs High flow storm through chute → Tw (ft.) = *Program*
Q_5 = *50.0* cfs Low flow storm through chute → Tw (ft.) = *Program*

***Note**: The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.*
***Input tailwater (Tw)**:*

Profile and Cross Section (Output):

Starting Station = *0+00.0*
h_{pv} = *0.41 ft. (0.08 ft.)*
H_{pe} = *2.45 ft.*
h_{cv} = *0.74 ft. (0.13 ft.)*
H_{ce} = *2.28 ft.*
H_p = *2.04 ft. (0.36 ft.)*
y_c = *1.54 ft. (0.28 ft.)*
$0.715y_c$ = *1.1 ft. (0.2 ft.)*
z_1 = *0.99 ft. (0.19 ft.)*
Hydraulic Jump Height, z_2 = *2.28 ft. (0.39 ft.)*
[1] y_n = *2.35 ft. (0.5 ft.)*
$10y_c$ = *15 ft.*
$40(D_{50})$ = *36 ft. radius*
H_{drop} = *4 ft.*
Tw+d = *2.92 ft. - Tw o.k. (1.41 ft.) - Tw o.k.*
1.92 ft. (0.41 ft.)
Velocity$_{inlet}$ = *4.4 fps at normal depth*
Critical Slope check upstream is OK
d = *1 ft. {1 ft. minimum suggested}*
Velocity$_{outlet}$ = *5.48 fps at normal depth*

Notes:
1) Output given as ***High Flow (Low Flow)*** values.
2) Tailwater depth plus d must be at or above the hydraulic jump height for the chute to function.
3) Critical depth occurs $2y_c$ - $4y_c$ upstream of crest.
4) Use WI Const. Spec. 13, Class I non-woven geotextile under rock.

[1] **Note**: When the normal depth (y_n) in the inlet channel is less than the weir head (H_p), ie., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

Profile Along Centerline of Chute

Typical Cross Section

Rock$_{thickness}$ = *32.2 in.*

*Use H_p along chute but not less than z_2.

	10.88 cfs/ft.	Equivalent unit discharge
F_S =	*1.50*	Factor of safety (multiplier)
z_1 =	*0.99 ft.*	Normal depth in chute
n-value =	*0.05*	Manning's roughness coefficient
$D_{50}(F_s)$ =	*16.1 in.*	Minimum Design D50*
$2(D_{50})(F_s)$ =	*32.2 in.*	Rock chute thickness
Tw + d =	*2.92 ft.*	Tailwater above outlet apron
z_2 =	*2.28 ft.*	Hydraulic jump height
****** The outlet***	***will***	***function adequately***

High Flow Storm Information

Figure 66 - NRCS Rock Chute Design Calculator for Area C

Note that the calculator calls for D_{50}=16.1". The plans and details for Areas C and D specify D50=18" to be conservative. Area C experiences 670 cfs flow during the 100yr event per the HEC-RAS model results and coming from the City of Flagstaff Detention Basin overflow spillway.