Galesburg Autonomous Transshipment Corridor (GATC)

CEE 415 Geometric Design of Roads

and CEE 453 Urban Hydrology and Hydraulics

Spring 2020

Team Number: 11

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Executive Summary

A project team from the University of Illinois' Civil and Environmental Engineering course 415/453 has designed the Galesburg Autonomous Transshipment Corridor (GATC). The GATC, located near Galesburg, IL, is a roadway which connects a shipping facility to a receiving facility. The roadway is designed to service level 5 autonomous vehicles that ship goods and people to and from each facility. Future economic expansion has also been factored into the design.

An alignment was selected based on the minimization of length, control points, stream crossings, interchanges, home demolitions, forest area, and elevation change. The roadway will be classified as a Two- Lane Rural Minor Arterial with approximately 50% of the 8,000 annual average daily traffic predicted to be WB-65 autonomous trucks. A design speed of 50 miles per hour has been selected. Geometric design criteria from the IDOT BLSR were used as guideline and adjusted for autonomous vehicles. Due to automated communication, perception reaction time is decreased from 2.5 seconds in humans to 0.5 seconds in autonomous vehicles. Considering the heightened perception abilities of automated vehicles, the geometric design parameters can be altered significantly in order to optimize traffic flow.

Enhanced perception abilities decrease the minimum radius (Rmin) in horizontal curves from 1330 feet to 833.33 feet for a superelevation of 6%. The stopping sight distance is the determining criterion for vertical curves (sag or crest) and is altered due to greater visual acuity. For this reason, the stopping sight distance is reduced from 274.85 feet to 570 feet and the vertical clearance is reduced from 11.31 feet to 11.03 feet. The vertical alignment was developed through consideration of the existing topography, minimum and maximum longitudinal grade requirements, intersection locations, and total cut and fill values. Approximately 197,302 CCY of fill has been calculated for the earthwork volumes using the average end area method.

Cross sectional elements for a 20-year design life have also be optimized. The width of the each of the two travel lanes is 11 feet as opposed to 12 feet. A combined shoulder and bike lane of 10 feet with a protective six-inch barrier is proposed. The median has been completely removed due to the low chance of errors with automate vehicles. Primary factors used to select pavement were soil composition, traffic, cost comparison, and weather. After considering these factors, asphalt was chosen as the surface material. From traffic factor and subgrade analysis, the final pavement design is calculated to be 12 inches of improved subgrade layer and 16.5 inches of HMA thickness.

Drainage aspects to mitigate storm water were also considered as the proposed alignment transgresses on a FEMA 100 year floodplain at the crossing over Haw Creek. The proposed

structure at the Haw Creek crossing is a 115’ single span bridge with a concrete deck on steel plate girder beams. The alignment chosen contains 13 culverts, 22 swales, and numerous ditches. Design details for 5 stream crossings, 2 roadside swales, and 1 detention basin are included. Culverts were designed using the HEC HMS and NRCS methods. Material, shape and size of the culverts varies for each site location to ensure the best fit for each environment. Swales were designed with a bottom width of 4 feet, back slopes of 1V:3H, and front sloped of 1V:6H. Mixed growth form and good cover with stem heights of 0.75 feet were chosen for less upkeep once vegetation is established. Due to the topography of the GATC, a single detention basin to serve the entire roadway has been made infeasible and the longest swale was used to predict floodplain storage lost. The area for the basin can then be calculated as the relationship between the area and infiltration rate equaling the drainage value. The area of the drainage basin is 23,463 square feet.

A preliminary economic analysis factors in the cost of pavement structure, land acquisition, slope stabilization, and earthwork. Drainage factors such as culverts, detentions/wetlands, and scour protection for swales, are also included in the cost analysis. Finally, a consulting fee rate of $200 per hour per person is added. The predicted cost of the GATC is $35,539,452.23.

The project team is pleased to present this alignment for consideration.

1. Introduction

1.1. Project Background and Location

We have been retained for the design and analysis of a new roadway corridor located near the city of Galesburg IL, in Knox County. New agricultural facilities are proposed to be constructed southeast of Galesburg, with a shipping facility (Lat: 40º 52’ 35” N, Long: 090º 22’ 48” W) located approximately 7 mi west of a receiving facility (Lat: 40° 53' 11.436" N, Long: -90° 15' 20.376" W). The roadway will provide connection between the two facilities, for the shipment of goods and people. The roadway will also be designed to service level 5 autonomous vehicles as it is anticipated that this will be the primary form of transport between facilities. Because of this last accommodation, this project has been dubbed the Galesburg Autonomous Transshipment Corridor, or the GATC.

The extent of the project area can be seen below in figure 1, along with the proposed locations of the shipping and processing facilities. The project area is predominantly agricultural, with some residential neighborhoods and also some forested areas which are found near local streams and tributaries.

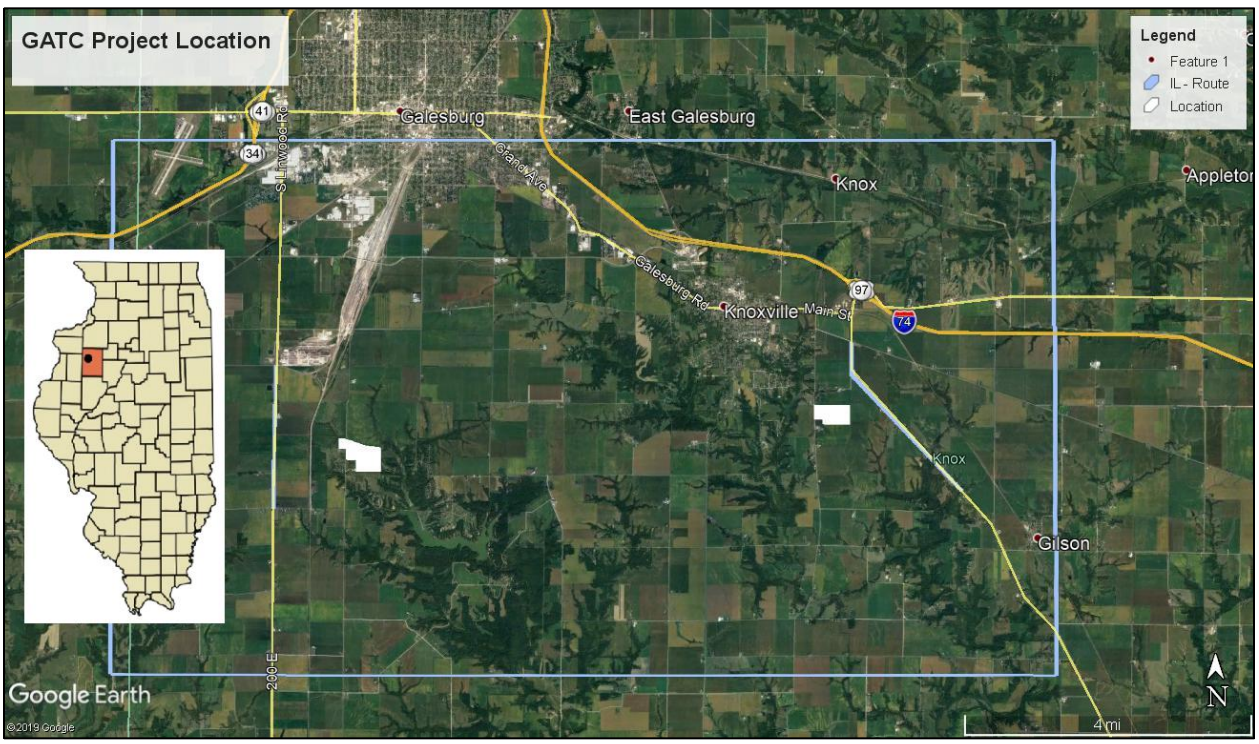

Figure 1-1: Project location map

The topography of the project area has an average slope of 2.6% with a maximum elevation difference of 118 feet along the approximately 9-mile-long corridor, which is consistent with the flat land characteristics of Illinois. However, there will be streams crossings and drainage design to contend with. For example, Haw Creek, just south of the city of Knoxville, has a 100-year floodplain, designated by the Federal Emergency Management Agency, which will have to be crossed without adverse upstream effects to the 100-year flood elevation.

1.2. Geometric Assumptions and Project Constraints

Based on the Illinois Bureau of Local Roads and Streets Manual, the functional classification of the GATC will be Rural Minor Arterial. It is anticipated that approximately 50% of the projected 8,000 AADT will be WB-65 autonomous trucks, which will have an impact on the geometric design of the new roadway.

Alignment of a bridge over a BNSF railroad to the east of the project location is also included in the scope of this project. Besides this crossing, however, all roadway intersections are to be designed at grade and at 90° ± 10° to facilitate access from other areas within the vicinity of the project location. The construction of the new facilities and the GATC is expected to encourage economic growth within the area, so future access to the new roadway is important.

Additionally, the GATC will be constructed using a design speed of 50 mph, a minimum longitudinal grade of 0.5%, and a maximum longitudinal grade of 5.0%. Overall, the project will operate within several constraints, which are listed below:

- Shipping and processing facilities must be connected by the new roadway, either by direct connection or by spurs from the main roadway
- The roadway must be designed for service of level 4/5 autonomous vehicles as well as bicyclists
- The western terminus of the new roadway will be on Illinois Route 41 between Knox County Road 1050N on the south and the BNSF railroad on the north
- The eastern terminus of the new roadway will be on Illinois Route 97 between Knox County Road 1080N on the south and Knox County Road 1290N (County Highway 17) on the north

1.3. Impact of Autonomous Vehicles and Bicyclists on Geometric Design

The GATC will accommodate bicyclists, which will have an influence on the geometric design of the roadway. Including a bikeway will widen the road cross section and will introduce more safety concerns which will be considered in the roadway design. Some of these concerns will be addressed by signage and bikeway markings, sufficient separation and protection from vehicle traffic, and appropriate grade of the bikeway for drainage purposes and user comfort.
Autonomous vehicles would assist in altering traditional geometric design features because of the precise driving capabilities associated with self-driving cars. The visual acuity obtained from the cameras, sensors, with light detection and ranging (LiDAR) technologies would be reason enough to decrease the lane width because this machinery helps limit transversal wander and increases the perception reaction time of the vehicle (PRT). As a result, precise vertical and horizontal geometric curves help instill more strict parameters without the repercussion of unsafe driving practices.

2. Project Objectives

Our main objective is to design a transportation corridor between two potential agricultural facility locations near Galesburg, IL while honoring the project constraints outlined in 1.2. The design will accommodate level 5 autonomous cars and trucks, as well as bicyclists along a roadside bike path. The roadway must also be designed with future economic expansion in mind, which will dictate the design of intersections with existing roads.

Proper drainage design of the corridor is another project objective. These designs will be able to efficiently move water from roadway features into drainage and conveyance structures and to contend with any potential existing streams or defined floodplains.

The economics of this project will be important when considering the project's feasibility, so minimizing costs is an important objective. As autonomous roadway corridors are a new project type there is little to no ability to compare project costs to past proposals and business ventures, therefore the criteria and design considerations used to construct this corridor will require a proper understanding of the industrial use of autonomous vehicles. Some ways to reduce total project costs are to minimize the amount of private land acquisitions, which will also minimize the impact to local communities, minimize forest clearings, and minimize the amount of earthwork, which will be dictated by vertical alignments and surface topography. Many of these project attributes will help guide the ranking of alternative designs, in order to choose the most effective one. This decision will help set a precedent to organize the project standards as technology advances to automated transport.

Lastly, developing a safe design for both vehicles and bicyclists is an essential project objective. In fulfillment of this objective, roadway geometric designs will be guided by design standards published by the Illinois Department of Transportation. These standards, however, may be adjusted to accommodate the service of autonomous vehicles, given that proper justification exists.

3. Design Team Members

3.1. Shain Hennessy

I am a senior in CEE major with a primary in Water Resource Engineering Science and a secondary a Construction Management. I have previously worked for two civil engineering firms. At both of these firms my focuses were on construction and reconstruction of roadways as well as underground work. I worked on projects ranging from $30,000 to $8,000,000. I hope from this class to learn gain more design experience on roadways and earth work.

3.2. Heli Shah

I am a senior CEE major with a primary in Environmental Engineering and secondary in SRIS. Currently, I am interning with the University's Capital Programs Division developing preliminary deferred maintenance project scope and budgets, in addition to performing take-offs from CAD and Revit documents. Last summer, I interned with the Village of Wilmette where I brought their sewer repair project to bid. I am currently deciding where to work after I graduate.

3.3. Mihir Thakar

I am a junior in the CEE department with a primary in Transportation Engineering and a secondary + minor combination in Informatics on the Data Science track. Last summer I interned at Wight and Company as a Sustainability Engineering intern and was exposed to the entire civil industry because the firm followed a design-bid-build practice. I chose transportation because I am interested in traffic optimization and the infrastructure involved in developing smart cities.

3.4. Matt Kuechenberg

I am a senior CEE major who will be graduating this spring with a primary emphasis in Water Resources Engineering Science and a secondary emphasis in Geotechnical Engineering. I spent last summer interning with the engineering consulting firm Hanson Professional Services Inc., and I will be returning to them full time after my graduation. In my internship I worked in Hanson's infrastructure department as a part of the water resources technical group. I gained experience in hydraulic and hydrologic modeling using programs such HEC-RAS and HEC-HMS and hope to apply this knowledge to the semester project.

A log of hours worked by each team member is included in **Appendix A.**

4. Alignment Alternatives

Three alternative alignments were developed and ranked against each other in order to select the best preliminary route. These proposed route alternatives are shown in figure 4-1 and will be referred to in this section as the following:

Route #1: Purple
Route #2: Blue
Route #3: Yellow

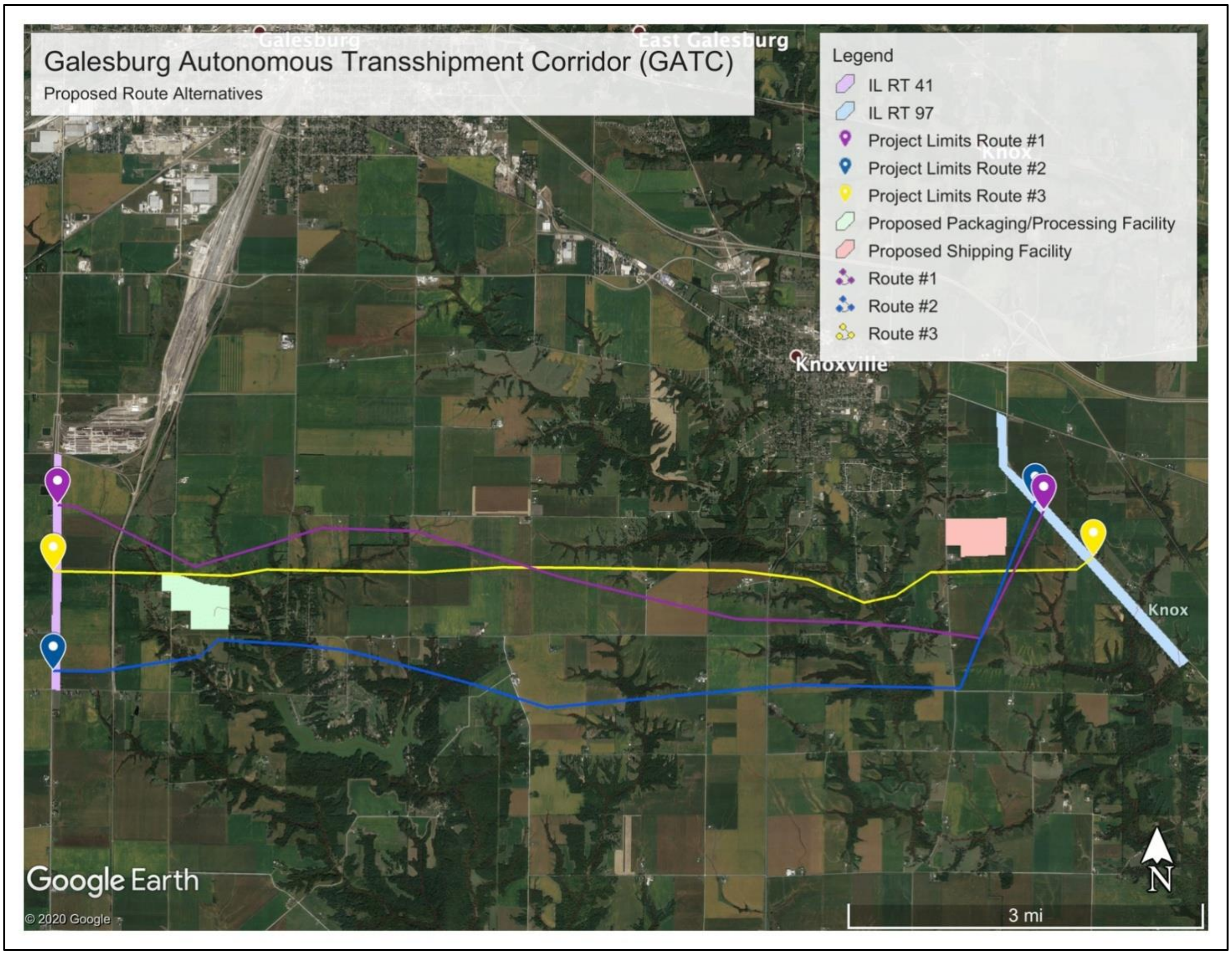

Figure 4-1: Alignment Alternatives

4.1. Alignment Descriptions

Route #1 starts on the northern limits of the Illinois Rout 41 terminus. **Route #1** angles down toward the proposed packaging/shipping facility with respect to figure 4.1's horizontal. This keeps **Route #1** close to the shipping facility and makes **Route #1** perpendicular to the railroad tracks where a bridge is to be installed. From here **Route #1** angles north to avoid running along a stream. **Route #1** continues approximately 20 degrees south of east to avoid running parallel to another stream and to avoid residential homes. **Route #1** then continues on to cross Haw Creek and angles north to approach the receiving facility. This would avoid creating a longer additional road from the shipping facility back to **Route #1. Route #1** keeps this angle until it connects within the eastern terminus limits on Illinois Route 97.

Route #2 starts on the southern limits of the IL 41 terminus. First **Route #2** extends perpendicular to IL 41 to cross the railroad tracks where a bridge is be installed. **Route #2** angle towards up toward the proposed packaging/shipping facility with respect to figure 4.1's horizontal. **Route #2** continues eastward at a few degrees south of east. This keep **Route #2** from running very close and parallel to Knox Highway 26. To note to make this path possible and entry to a subdivision will need to be changed. **Route #2** continues east and eventually crosses Haw Creek. **Route #2** must change direction to run northeast to pass near the proposed receiving facility. **Route #2** keeps this path until it connects within the eastern terminus limits on IL route 97.

Route #3 starts at approximately in the center of the western terminus limits of IL Route 41. **Route #3** extends perpendicular to IL 41 to cross the existing railroad, where a bridge is be installed. Due to **Route #3**'s location of connection with IL 41, **Route #3** does not need to change direction to pass near the proposed packaging/shipping facility. To avoid complicated intersections with existing east-west roads, **Route #3** continues east on a fairly straight path. Near Knox Highway 8, **Route #3** angles south to create a perpendicular intersection and to avoid more forest removal. **Route #3** continues eastward where it crosses Haw Creek and its 100-year floodplain, then, to avoid home acquisitions, angles north to approach the proposed receiving facility. Finally, **Route #3** angles back south, passes the proposed receiving facility, and connects to IL 97 within the terminus limits.

4.2. Alignment Rankings

To assess and compare the alternatives, 7 different route attributes were quantified or described. These attributes are total route length, route complexity, number of stream crossings, number of interchanges, approximate number of residential home acquisitions, amount of forested area to be removed, and the maximum elevation change along each route. Table 4-1 summarizes and compares each route in these categories.

Table 4-1 Alignment Alternative Rankings

Route	Length (mi)	Control Points (Including Beginning & End)	Stream Crossings	Interchanges	Approximate # home demolitions	Forest Areas (ac)	Max Change Elevation%
#1	9.24	11	13	9	0	126	17.70%
#2	9.47	9	13	10	4	383	21.30%
#3	8.85	12	9	8	0	111	17.30%

Above categories for our analysis are equally weighted meaning 1st place receives one point, 2nd place receives 2 points, and 3rd receives 3 points for here respected categories. Therefore Route #1 total is 2+2+2+2+1+2+2= 13. Route #2 total is 3+1+2+3+3+3+3=18. Route #3 total is 1+3+1+1+1+1+1=9.

4.2.1. Economics

The economics of the GATC will determine its feasibility, so it is essential to compare the estimated cost of each route. In general, the shortest route would require the least amount of materials to build and therefore reduce cost. Route complexity drives up the engineering cost because a more complex route will take longer to design, and consequentially would be more expensive to build. More stream crossings and intersections will require the design and construction of culverts and interchanges respectively, which will also increase costs. Demolition of residential homes will require the upfront cost to buy the property, and also the cost of the demolition itself; forest clearings are expensive and more removed trees result in increased project costs. Finally, routes with greater elevation changes will require more cut and fills, further increasing the total cost.

4.2.2. Social Impacts

There are also social impacts that must be considered when assessing the feasibility of alternative routes. A longer and more complex route will require more private land acquisitions, which will not only increase project costs but may also provoke resistance from landowners and local governments. Additionally, routes which intersect residential homes or neighborhoods will cause complications as persuading residents to leave their homes may be difficult.

4.2.3. Environmental Impacts

The environmental impacts are accounted for in mainly two categories of this ranking system. Keeping the number of stream crossing low is important. A low number is cheaper as stated in section 4.2.1 and less construction over streams will result in less impact on the stream ecosystems. Also, deforestation has environmental impacts. Deforestation is costly both economically as mentioned in section 4.2.1 and environmentally as it would destroy some ecosystems as well.

4.3. Route #3 - Selected Alignment

Route #3 performs the best in most of the comparison categories, as illustrated in **Table 4-2**, and therefore it is expected that this proposed alignment would be the cheapest alternative and have the least social and environmental impacts. These considerations also lead to us to conclude that **Route #3** would be the easiest to design and construct. Route #3 will now be referred to as the proposed route, and its preliminary alignment is shown in **Figure 4-3.** The coordinates for intersection locations and control points can be found in **Appendix B.**

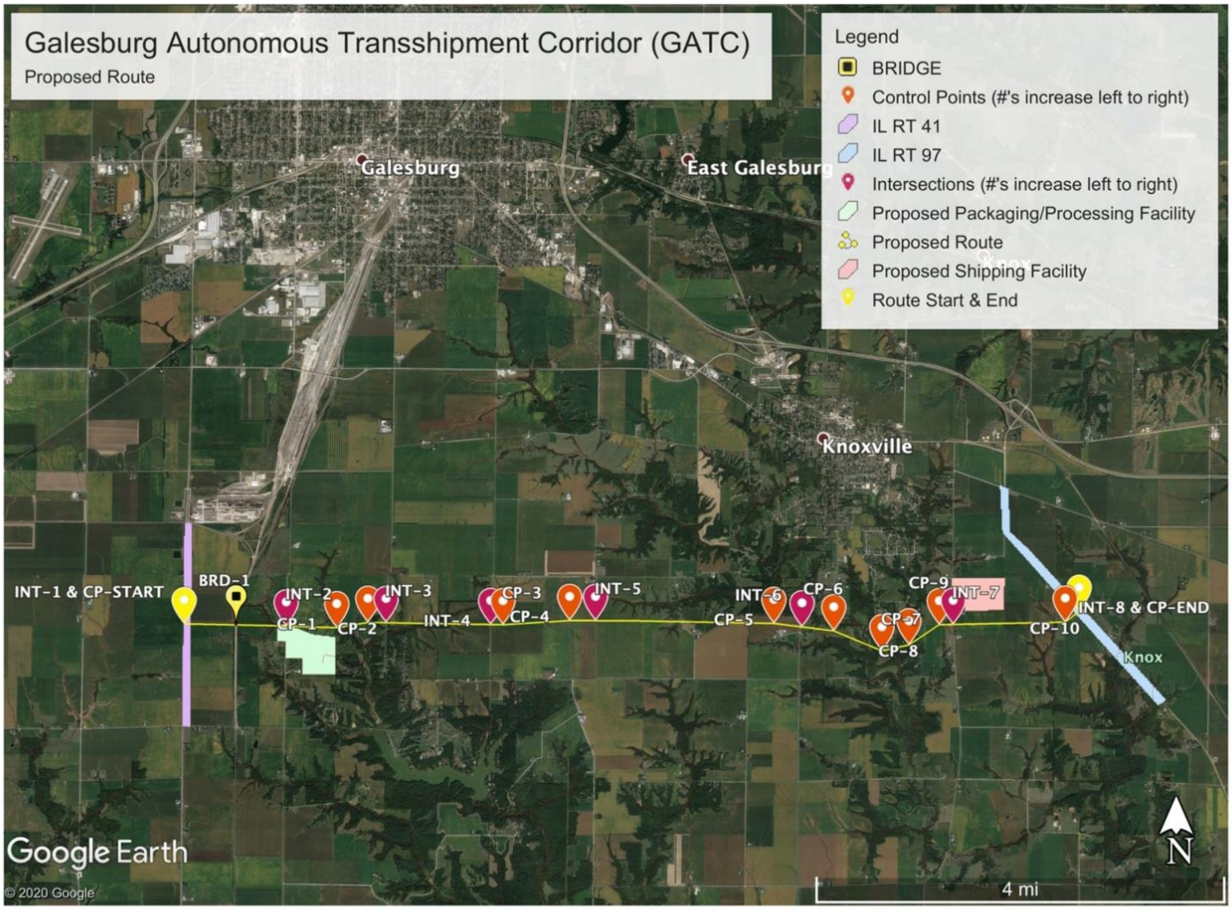

Figure 4-2: Proposed route alignment including control points, bridge, and intersection locations

5. Autonomous Vehicles Overview

5.1. Autonomous Vehicles Background

In order to effectively and efficiently implement the construction of a roadway corridor between Knox County and the BNSF railway, the path will be geometrically designed for autonomous and connected vehicles. The roadway would be designed for level 5 design vehicles; a fully automatic standard under all conditions as defined by the NHTSA [1]. Connected vehicle communication and technology would aid in travel time, mileage, and safety amongst the trucks that would primarily account for the AADT of this road. Measurement precautions during construction must be taken seriously because of the significantly different parameters involved in designing a full-fledged system for autonomous vehicles. The presence of automation would promote stricter standards geometric design standards for larger traffic flow as opposed to the liberal designs for human controlled counterparts.

The latest advances in AV technology have helped reimagine the properties of geometric design, simultaneously allowing for a greater efficiency within all travel related costs. With regard to this project, the concept of connected vehicles will help govern the physical standards of the road as the design vehicle will be an autonomous fleet of WB-65 connected trucks. The vehicles would use digital short-range communication in order to platoon together and reduce air flow so that successive trucks could “draft off one another with reduced aerodynamic drag, reducing operating costs” [2]. This data would also be populated in “cloud database...to process the data to determine the beginning and end of congestion zones” so that the platoon could drive accordingly [5]. Tesla, a company at the forefront of autonomous driving stated their “vehicles are equipped with the hardware needed for full self-driving capability...will generally have eight surround cameras, twelve updated ultrasonic sensors, and a forward-facing radar to allow it to see through the heavy rain, fog, dust, and other vehicles ahead.” [3] These innovative methods of automated communication would work in synchronization much faster than a human driver who would have an average perception reaction time (PRT) of 2.5 seconds compared to a level 5 autonomous vehicle which has a PRT of approximately 0.5 seconds [4].

Considering the heightened perception abilities of automated vehicles, the geometric design parameters can be altered significantly in order to optimize traffic flow. Enhanced perception abilities would aid in decreasing horizontal curves especially if the route data is pre-loaded into the GPS system of the vehicle. With a reduction in the minimum radius (R_{min}) the exact value of a curve could be input into the routing system and the car could anticipate the turn by optimizing its fuel usage and adjust its turning radius in real time preparation. The main criterion for determining the vertical curvature of a road (either sag or crest) is the stopping sight distance of a vehicle, but with greater visual acuity and the possibility of pre-loaded route data the degree of design can be significantly decreased.

Besides the vertical and horizontal components of geometric design which represent changes in infrastructure; the accurate and precise driving capabilities of autonomous vehicles allow roadway designers to consider decreasing the width of lanes because of the minimized transversal wander. Additionally, shoulder widths could be significantly scaled down in case of emergency maintenance procedures, medians and barriers could be made obsolete in certain areas because of the exact driving, and drainage can be intelligently placed in other areas.

With regard to pedestrians, the shoulder could be split and shared as a bike lane when necessary due to the environment. The presence of significant signal locators and communication devices would also make rumble strips outdated. The cutdown of these roadway features would significantly decrease the cost of the roadway corridor and allow for increased V2V and V2I infrastructure expenses. Infrastructure wise, the concept of road signage could inexorably change in AV restricted area by using QR codes or different visual flags to decrease the manual labor of roadway design. Strategic lighting could also be pursued so that sustainable best method practices (BMP) are used in construction. Overall, the construct of automated vehicle roadway corridors would grant opportunities to pursue unforeseen driving and infrastructure standards for optimized travel.

5.2. Autonomous Vehicles Design Criteria

As previously explained, the advent of autonomous vehicles provides significant impacts on the design criteria chosen with respect to roadway construction. Provided constraints on the design speed, super elevation, and side friction were used as constants to determine the R_{min}, lateral clearance, and stopping sight distance for autonomous vehicles while the normal roadway geometric design parameters were obtained from the BLRS manual for a Rural Two-Way Open Arterial Road.

The K values for the AV roadway were obtained from the Civil 3D vertical curves report and can be found in the appendices along with sample calculations for the AV roadway.

Design Criteria	Rural Two-Way Open Road Arterial*	AV Roadway
R_{min}	1330’	833.33’
Lateral Clearance	11.03’	11.31’
Super elevation	6%	6%
Side Friction	.12	.14
Design Speed	60mph	50 mph
Stopping Sight Distance	570’	274.85’
K Values	151’	HSD-K: 96’ SSD-K: 84’ PSD-K: 229’

*BLRS Manual

6. Preliminary Cross-Section Elements

6.1. Design Considerations

The following parameters were considered for the design of the GATC:

- Design Speed: 50 mph
- Projected Average Annual Daily Traffic (AADT): 8,000 vehicles
    - 50% WB-65 Autonomous Trucks: 4,000 autonomous trucks
- Design Hourly Volume (15% AADT): 1200 vehicles per hour
- Minimum Longitudinal Grade: 0.5%
- Maximum Longitudinal Grade: 5.0%

Along with these parameters, the Illinois Department of Transportation Bureau of Local Roads and Street manual was also consulted to find the preliminary design criteria for human-driven vehicles. There are no current regulations for automated vehicles. Engineering judgment was used to decide if and how the criteria could be adjusted when the vehicles are level 5 autonomous vehicles.

6.2. Cross Section Elements

The GATC is comprised of two types of cross sections. The first cross section is for new rural minor arterial road construction. The second cross section is for bridges utilized in the railroad and stream crossings. The elements considered at the cross sections are travel way, shoulders, cross slope, median, side slope, and median slope.

6.2.1. Geometric Design for New Rural Two-Lane Arterial Road Construction

The majority of the GATC will be compromised of the cross-sectional elements summarized in the table below. Some of these criteria differ from the BLRS manual because the roadway is designed for autonomous vehicles. See Section 6.3 for the justification of the criteria selected and the identification of the differing criteria.

Table 6-1: Geometric Design Criteria for New Rural Two-Lane Arterial Road Construction

Traveled Way	Number of Travel Lanes	2
	Travel Lane Width	11 ft
	Bicycle Lane Width	3ft
Shoulder	Right Shoulder Width	7ft
	Left Shoulder Width	7ft
Cross Slope	Traveled Lane	1.5%
	Shoulder	4%
Median	None	N/A
Side Slope	Front Slope	1V:6H, 16.7%
	Ditch Bottom Width	4 ft
	Back Slope	1V:3H, 33%

The shoulders are chosen to be paved in case it may need to be in service one day. For this reason, a cross slope of 4% was chosen, as opposed the cross slope of 6% if the shoulder was composed of aggregate. The shoulders on each side are 7 feet to allow just enough room for a vehicle in the unlikely event of an emergency.

The cross-sectional elements from **Table 6-1** are illustrated below in **Figure 6-1**.

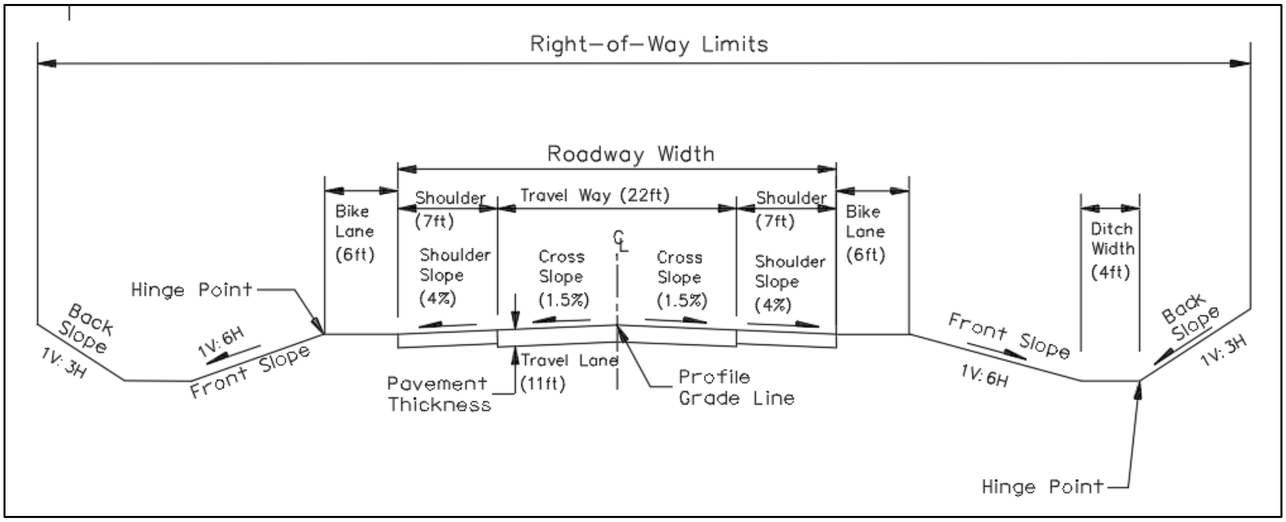

Figure 6-1: Cross Section of New Rural Two-Lane Arterial Road

6.2.2. Geometric Design for Bridges

The cross-sectional design for bridges is similar to the cross section used for the rest of the arterial road. This cross section will be utilized at the railroad and stream crossings encountered along the alignment. **Table 6-2** summarizes cross-sectional elements at bridges.

Table 6-2: Geometric Design Criteria for Bridge Crossing

Traveled Way	Number of Travel Lanes	2
	Travel Lane Width	11 ft
	Bicycle Lane Width	5 ft
Shoulder	Right Shoulder Width	0 ft
	Left Shoulder Width	0 ft
Cross Slope	Traveled Lane	1.5%
	Shoulder	N/A
Median	None	0 ft

The cross-sectional elements from Table 6.2 are illustrated below in **Figure 6-2**.

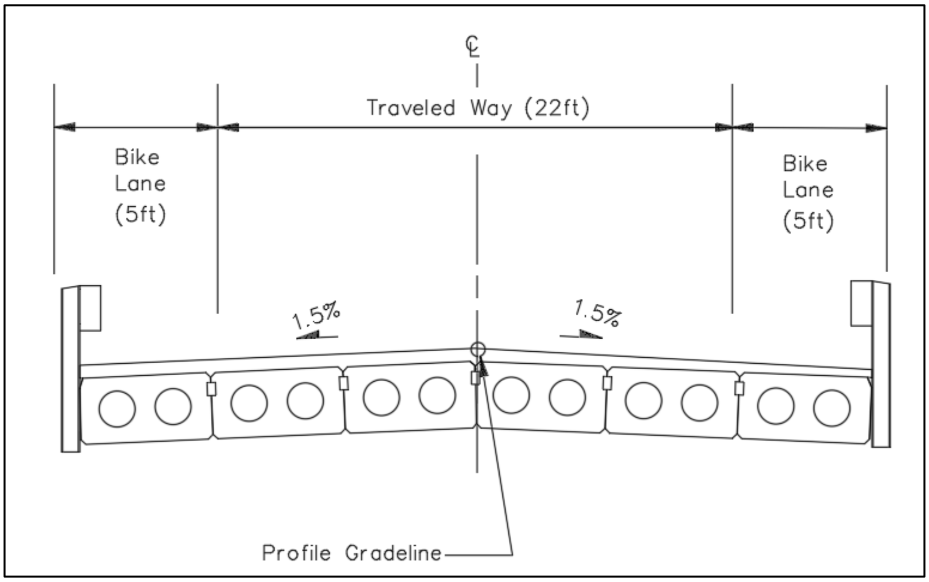

Figure 6-2: Cross Section of Bridge Crossing (Typical, edited in design)

6.3. Cross Section Element Justification

The cross-sectional elements were designed according to the IDOT BLSR for Rural Two-Lane Minor Arterials. Figures 32-2A and 34-1B were consulted. The figures contained criteria for rural two-lane minor arterials and for the construction of a rural highway, respectively. The GATC has a 20-year design life. There are a few cross-sectional elements that differ from the IDOT BLSR criteria to accommodate the level 5 automated vehicles.

As stated earlier, fully automated vehicles have cameras that surround the vehicle, ultrasonic sensors, and a radar to detect oncoming obstructions. The synchronization of these devices allows for faster communication and reaction time. The average perception reaction time of an autonomous vehicle is only 0.5 seconds, compared to 2.5 seconds for a human. The enhanced communication decreases the probability for the vehicles straying transversally and departing form the roadway. The decreased reaction time allows for the vehicles to drive much closer to each other and decreases the chances of collisions. Medians and barriers can be scaled down, or even removed completely, due to the low chance of errors with automate vehicles.

With this knowledge of autonomous vehicles, the travel lane width was reduced from 12 feet to 11 feet. The shoulder width was also reduced and combined with the bike lane for an overall width of 10 feet. In addition, the median removed due to the low probability of automated vehicles straying transversely. In order to protect the bicyclist and pedestrians, a six-inch barrier will be placed between the roadway shoulders and bicycle lane. This is mostly to provide a buffer for potential mistakes by the bicyclists/pedestrians and to prevent them from entering into the roadway.

6.4. Pavement Design

The primary factors used to select pavement were soil composition, traffic, cost comparison, and weather. A secondary factor considered was the adjacent existing pavements. After considering these factors, asphalt was chosen as the surface material. Asphalt has a relatively low capital cost when compared to concrete; however, asphalt does require more maintenance. Minor arterials are characterized by relatively lower travel speeds and traffic volume. This means that there will be less degradation and thus maintenance require on the roadway, lowering the overall cost. The GATC will be constructed in central Illinois, indicating that it will experience a wide range of temperatures from winter to summer months. Asphalt performs well in freeze-thaw cycles. In addition, the GATC has a western terminus on Illinois Route 41 and an eastern terminus on Illinois Route 97. Route 41 and 97 both have asphalt as the surface material, making asphalt the best option in order to have continuity.

6.4.1. Traffic Factor Analysis

A major decision factor in determining pavement design is the expected traffic on the proposed roadway. The pavement thickness is specifically determined by the volume of truck traffic that is expected over the design life of the roadway. For the GATC, 50% of 8,000 AADT is expected to be WB-65 automated trucks, the design vehicle which is an interstate semitrailer. The gross vehicle weight limit is 80,000 lbs.; however, most trucks do not reach this limit as the volume is usually filled before it reaches the weight limit. WB-65 are multi-unit (MU) trucks, which means there is a high loading condition on the proposed roadway.

A 20-year lifetime was assumed for the calculations for the GATC. Class II roads and streets are characterized by two or three lane streets with structural design traffic greater than 2,000 ADT. The GATC follows these guidelines and will thus be classified as a Class II road. As part of the design procedure for the roadway, the actual traffic factor (TF) must be calculated as seen in **Appendix C**. This value is characterized as “The total number of 18-kip equivalent, single-axle load applications anticipated in the design lane during the design period, expressed in millions” and is used for the pavement structure design. The actual TF value is 19.96.

6.4.2. Subgrade Discussion

Soil properties and behavior are important to understand during construction of the roadway and will influence the structural design of GATC pavements. An ideal subgrade will have a high strength and low compressibility to be able to carry design loads and minimize pavement distresses. An approximate distribution of soil textures in Knox County was obtained from the USDA National Resources Conservation Center’s Web Soil Survey. The soil survey reports that the project area is dominated by fine grained soils, with low plasticity clays being found near streams and their floodplains, and high plasticity clays being found elsewhere. The AASHTO classification system classifies these soils as A-7-6 and A-6, which are described as clayey soils which score fair to poor ratings as subgrades. The Subgrade Support Rating (SSR) will be rated as fair due to the presence of clayey soils. AASHTO’s classification system and the area’s soil map can be found in **Appendix D**.

Clays are known for their low hydraulic conductivities, which produces poor soil infiltration and drainage, and also have lower shear strengths than granular soils. In-situ soil improvements such as compaction, pre-loading, or lime treatment can be conducted to reach sufficient design strengths but will be costly and dependent on geotechnical subsurface investigations.

To ensure the performance of the surface pavement, a modified layer of granular material, or subbase, is used to improve the subgrade layer. The purpose of a subbase is to provide a platform to construct pavement on. In most cases, a 12-inch layer is adequate. For a Class II road a 12-inch improved subbase layer is required.

6.4.3. Pavement Section Selection

Once the TF and SSR values were found for GATC, the pavement thickness could be derived. TF is 19.96 and SSR is fair. From the flow chart in IDOT BSLR Figure 44-1E, it is determined that a Full-Depth HMA pavement design is needed. At minimum, this consists of HMA binder and surface courses. **Appendix E** contains the figures used for the details of the pavement selection. After all the calculations the HMA thickness is 16.5 inches. The final pavement design is 12 inches of improved subgrade layer and 16.5 inches of HMA thickness. The cross-section of the pavement is shown below in **Figure 6-3**.

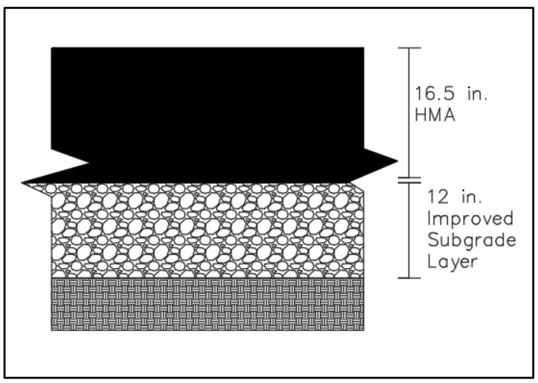

Figure 6-3: Structural cross section design of pavement

6.5. Superelevation

According to BLSR section 29-3.01 the maximum allowable rate of superelevation for rural areas is 8.0%. The design speed of GATC is low, so the need to actually use this rate is unlikely. Automated vehicles have the capability to drive safely on higher superelevation rates; however, a higher superelevation rate leads to higher construction costs. After considering these factors, a decision was made to use a maximum superelevation rate of 6%.

GATC is an undivided, crowned two-lane roadway. The proposed method of superelevation attainment from normal crown to full super is shown on the diagram below in **Figure 6-4**.

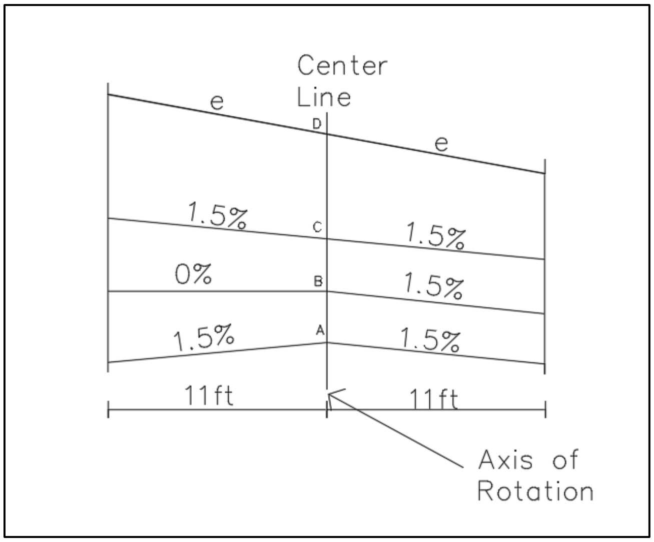

Figure 6-4: Attainment of superelevation from normal cross slope

Superelevation runoff (L) is the distance needed to change the cross slope of the roadway from the end of the tangent runout to the design superelevation rate. The tangent runout (TR) is the distance needed to change from a normal crown section to a point where the adverse cross slope of the outside lane is removed. Sample calculations and typical figures of both these values for two lane roadways are displayed in A**ppendix F**. **Table B-4** in **Appendix B** gives a summary of all the control points of the ten curves in the alignment.

7. Stream Crossings

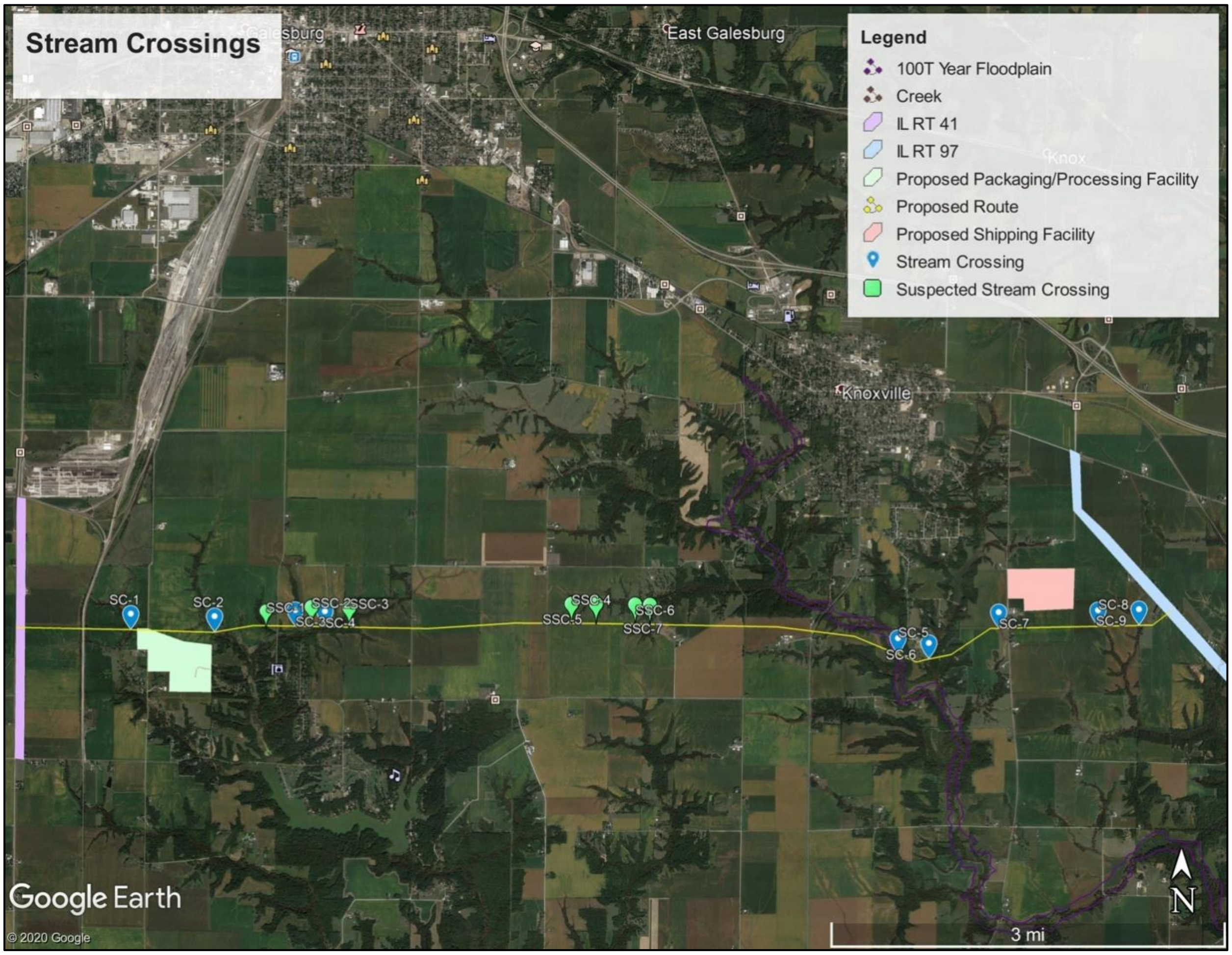

Figure 7-1: Stream Crossing Locations & 100T Year Floodplain

7.1. Background

Stream crossings are a major aspect of this project. Each stream crossing will require drainage design and an installation of a hydraulic structure such as a culvert. Along the proposed route there is one major stream crossing across Haw Creek and eight other minor stream crossings. Haw creek, as mentioned above, has a defined 100-year floodplain. The FEMA flood insurance rate map for Haw Creek in Knox County shows this designated 100-year flood plain and is provided in **Appendix D**. Below in **Table 7-1** is list of all stream crossings, their ID names, and their coordinate locations.

Table 7-1: Stream Crossing ID

ID	Description	Latitude	Longitude
SC-1	Unidentified	40°52'54.9"N	90°23'16.1"W
SC-2	Unidentified	40°52'53.9"N	90°22'32.4"W
SC-3	Unidentified	40°52'56.4"N	90°21'50.3"W
SC-4	Unidentified	40°52'55.9"N	90°21'34.9"W
SC-5	Haw Creek	40°52'45.2"N	90°16'35.8"W
SC-6	Unidentified	40°52'43.3"N	90°16'19.7"W
SC-7	Unidentified	40°52'55.5"N	90°15'43.3"W
SC-8	Unidentified	40°52'56.2"N	90°14'51.4"W
SC-9	Unidentified	40°52'56.6"N	90°14'30.0"W
SSC-1	Unidentified	40°52'55.4"N	90°22'5.6"W
SSC-2	Unidentified	40°52'56.2"N	90°21'41.9"W
SSC-3	Unidentified	40°52'55.9"N	90°21'22.2"W
SSC-4	Unidentified	40°52'57.5"N	90°19'26.3"W
SSC-5	Unidentified	40°52'57.5"N	90°19'13.4"W
SSC-6	Unidentified	40°52'57.3"N	90°18'53.1"W
SSC-7	Unidentified	40°52'57.1"N	90°18'45.4"W

7.2. Drainage Details

Properly sized culverts will be designed for all the stream crossing along the **proposed route**, along with roadside ditches, water retention areas, and storm sewers as needed. Extra attention will be given to stream crossing **SC-5** and its 100-year floodplain. Below, **Figure 7-3** illustrates the proposed location of all culverts and the direction of stream flow at these locations.

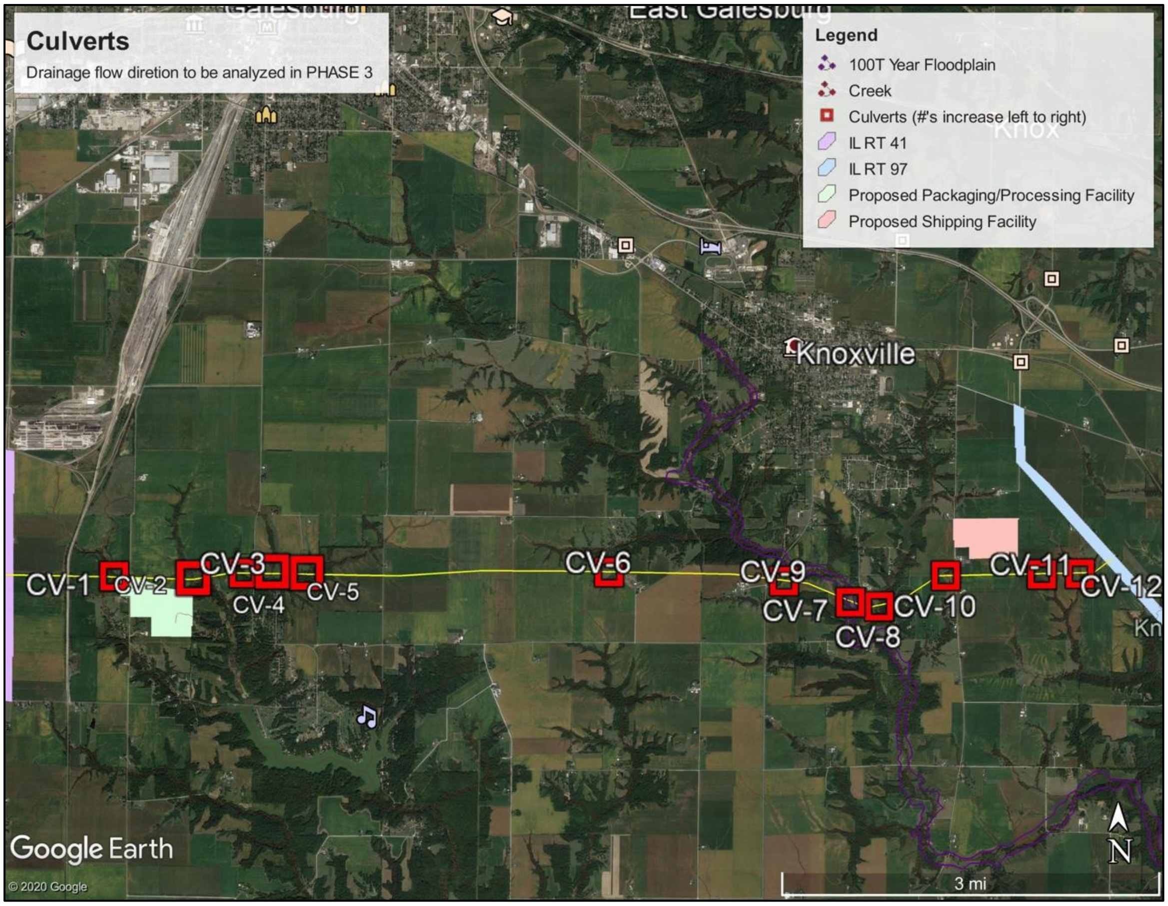

Figure 7-2: Culvert Locations and Stream Flow Direction

8. Horizontal and Vertical Alignment Details

The stationing for the horizontal and vertical alignments of the proposed route begins at the western most control point, **CP-1**, at location 40° 52' 53.9'' N, 90° 22' 33.0'' W. The beginning station is 0+00, and ending station 469+95.59, with each station representing 100' of road following the horizontal alignment. All elevations are defined by The North American Datum of 1983.

Horizontal curves were designed in accordance with the BLRS design constraints, fulfilling the minimum curve radii related to our chosen superelevation and design speed. Below, in **Table 8-1**, all control points for each horizontal curve are given, along with each curve radius, curve length, and delta.

Table 8-1: Horizontal curve information

Curve No.	PI	PC	PT	Radius	Curve Length	Delta
1	78+90.57	77+93.69	79+86.85	1000’	193.16’	11° 04' 03"
2	95+18.35	94+25.91	96+10.27	1000’	184.36’	10° 33' 47"
3	165+0.56	164+50.41	165+50.66	1250’	100.25’	4° 35' 43"
4	199+79.36	199+29.88	200+28.79	1250’	98.91’	4° 32' 01"
5	305+10.88	304+58.32	305+63.34	1000’	105.02’	6° 01' 02"
6	336+21.1	334+84.06	337+56.45	1000’	137.04’	15° 36' 24"
7	363+13.65	360+39.57	365+70.79	875’	532.22’	34° 47' 06"
8	377+27.27	375+64.10	378+86.74	875’	163.17’	21° 07; 36"
9	395+99.93	393+10.37	398+74.08	1000’	563.71’	32° 17' 53"
10	460+98.95	457+97.2	463+79.51	900’	582.3’	37° 04' 14"

The vertical alignment was developed while considering the existing topography, minimum and maximum longitudinal grade requirements, intersection locations, and total cut and fill values. The vertical curves also fulfill BLRS design criteria, with curve lengths meeting drainage, comfort, headlight and stopping sight distance criterion. **Table 8-2** below reports the minimum K values for stopping sight distance, passing sight distance, and headlight sight distance, as compared to the actual K value of the vertical curve. Having a K value greater than the minimum value indicates that the sight distance is fulfilled, as sight distance/curve length = A*K, where A is the grade change between longitudinal slopes in percent.

Table 8-2: Vertical curve sight distances

Curve No.	Profile Curve Type	K Value	Min K for SSD	Min K for PSD	Min K for HSD
1	Sag	139.469	-	-	96
2	Crest	235.783	84	229	-
3	Sag	128.119	-	-	96
4	Crest	257.476	84	229	-
5	Sag	130.528	-	-	96
6	Crest	440.788	84	229	-
7	Sag	153.072	-	-	96
8	Crest	2827.58	84	229	-
9	Crest	384.598	84	229	-
10	Sag	125.108	-	-	96
11	Crest	744.849	84	229	-
12	Sag	114.33	-	-	96

Table B-5 in **Appendix B** incontains the geometric parameters of all vertical curves, such as control point stations and elevations, longitudinal grades and curve lengths. It is noted that a vertical clearance of 25’ is obtained for curve number 2, which is an overpass above the existing railroad. According to the chapter 36 of BLRS minimum clearance shall be 23’.

Figures of the proposed horizontal and vertical alignments are included in **Appendix G**, and sample calculations pertaining to horizontal and vertical curves can be found in **Appendix F**.

Table B-1 in **Appendix B** provides all intersection locations and acute angles of these intersection. Two intersection do not follow the general rule that the acute angle only be +/- five degrees from 90. These intersections are INT-6 and INT-8. In the case of INT-6 our best alignment option has an acute angle of 70.11 degrees. The intersection also occurs on the curve of Co Hwy 8. To avoid any dangerous situation this intersection will be controlled by a stop light and signs will be posted for no turn on red. The angle for INT-8 is 85 degrees. In this case is quite different. This southwest corner of intersection at 95 degrees. Rather than adding additional horizontal curve to meet the +/- five degree rule, our team decided on leaving it as is. Almost all trucks leaving the east end of the corridor will turn right onto IL RT 97, and enter the corridor on the east end by turning left off of IL RT 97. Our team knows this because the corridor is a bypass

9. Drainage Analysis

9.1. Drainage Overview

Proper accounting of storm water is an important objective for the GATC, and this section documents the details of its drainage design. These details include the documentation of all drainage infrastructure and the design results for 5 stream crossings, 2 roadside swales, and 1 detention basin for storm water routing.

14 locations where a conveyance structure, such as a culvert or bridge, were identified and the contributing areas draining to these locations were delineated. Additionally, 22 roadside swales were incorporated into the GATC drainage system, which mostly were defined by the roadway’s vertical alignment. Figure 9-1 below shows the overall drainage map of the GATC while **Figures H1 – H4**, included in **Appendix H**, show more details such as the ID names of proposed culverts, bridges and swales, along with their flow directions.

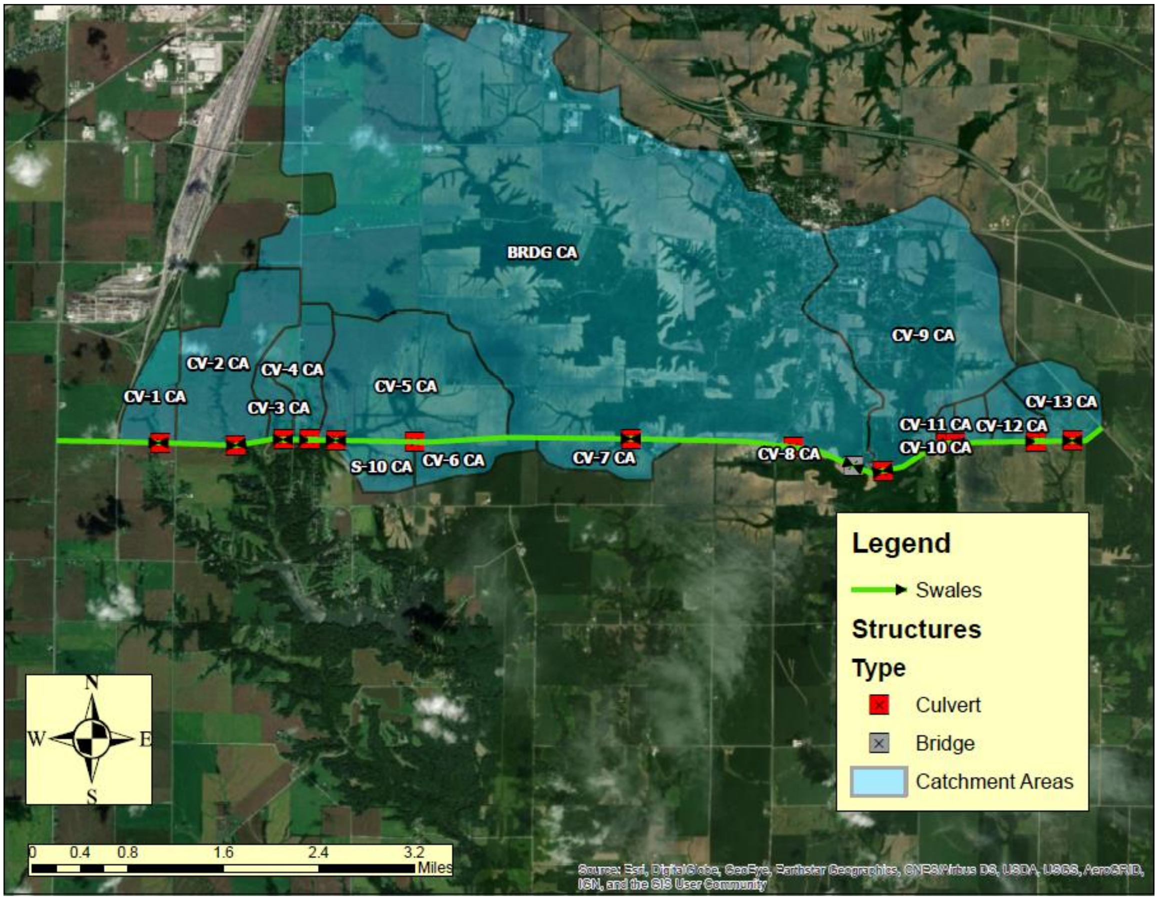

Figure 9-1: GATC Drainage Map

The conveyance structures and swales mentioned above are included in an identification table below, which reports each item's ID and station along the proposed route.

Structures		Swales		
ID	**Station**	**ID**	**Low Station**	**High Station**
CV-1	45+66	S-1	45+66	0+00
CV-2	78+94	S-2	45+66	52+52
CV-3	100+32	S-3	78+94	52+52
CV-4	111+82	S-4	78+94	92+92
CV-5	123+64	S-5	100+32	95+18
CV-6	158+63	S-6	100+32	104+68
CV-7	254+25	S-7	111+82	104+68
CV-8	326+37	S-8	111+82	115+45
BRDG	354+39	S-9	123+64	111+82
CV-9	368+99	S-10	123+64	226+42
CV-10	399+32	S-11	254+25	226+42
CV-11	403+46	S-12	254+25	275+87
CV-12	439+01	S-13	354+39	275+87
CV-13	455+43	S-14	354+39	363+26
		S-15	368+99	363+26
		S-16	368+99	398+87

		S-17	399+32	398+87
		S-18	399+32	419+44
		S-19	439+01	419+44
		S-20	439+01	443+20
		S-21	455+43	443+20
		S-22	455+43	470+52

Table 9-1: Conveyance Structure and Swale ID Names and Stations

Several methods were used to obtain discharges for the design of culverts and swales, such as the NCRS peak discharge method, the Rational Method, a HEC-HMS model which utilized the Curve Number method to calculate runoff, and the USGS Streamstats regression method, when it was available. Each of these methods were applied to each culvert catchment, the resulting discharge values were compared to one another, and the most appropriate discharge values were chosen. A table containing the hydrologic data for 5 culverts is included in **Appendix H**, along with the equations used in calculation.

9.2. Floodplain Encroachment

The proposed alignment encroaches on a FEMA defined 100 year floodplain at the crossing over Haw Creek. In order to mitigate upstream flooding, the proposed crossing must limit the raise in water surface elevation to 1' from the existing conditions for a flood with an annual exceedance probability of 1%. Additionally, in order to keep the roadway structurally sound, it is required that water surface elevations for transverse floodplain encroachments are a minimum of 2' below the low chord of the bridge for the 1% A.E.P. flood. Similarly, water surface elevations are to be a minimum of 3' below pavement elevations for the 1% A.E.P. flood and are not to overtop the pavement for the 0.2% A.E.P. flood.

The U.S. Army Corps of Engineer's Hydrologic Engineering Center River Analysis System, known as HEC-RAS, was used to calculate water levels for different frequency floods and to document that design criteria are satisfied. LiDAR data describing overbank and channel topography was used to develop cross-sectional geometries for the Haw Creek HEC-RAS model, and HEC-HMS was utilized to develop design discharges for the analysis. These peak discharges were calculated according to the NRCS Curve Number approach, as mentioned earlier in this report, and are listed along with their associated return periods in **Table 9-2**, below.

Return Period (1/P) in years	Discharge (cfs)
2	515.20
5	786.10
10	1014.90
25	1332.50
50	1593.30
100	1864.00
500	2525.10

Table 9-2: Design Discharge

These frequency floods were input into two versions of the model, one which captures the existing conditions of Haw Creek near the roadway crossing, pre-GATC, and another which incorporated the proposed structure. By comparing the outputs of these two versions, the existing and proposed conditions at Haw Creek can be compared, and therefore the hydraulic impact of the proposed structure can be quantified.

Figure 9-2 below shows a map of the proposed crossing and indicates the surrounding cross sections used to create the HEC-RAS model. The crossing falls in-between cross sections 5254.50* and 5052, so cross section 5254.50* will be used to compare the existing and proposed conditions, since it is just upstream of the GATC.

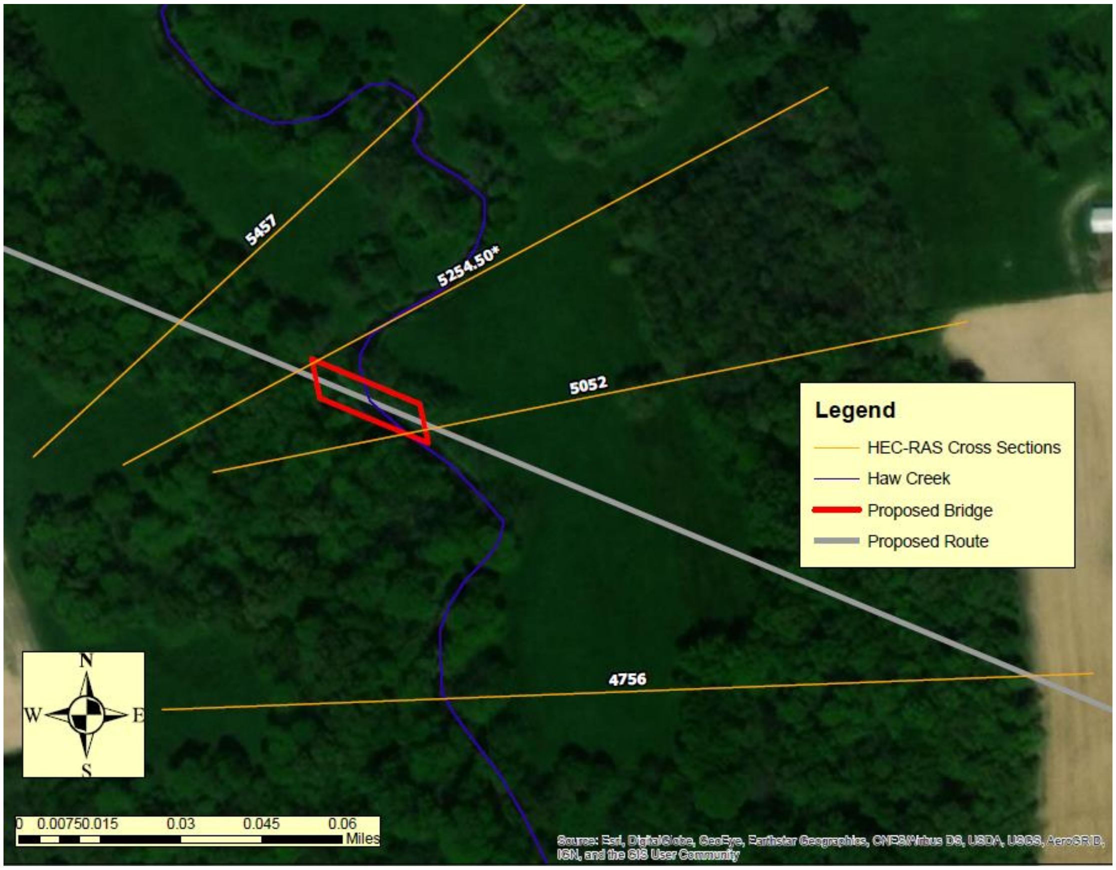

Figure 9-2: Map of GATC Crossing Over Haw Creek and HEC-RAS Cross Sections

The proposed structure at the Haw Creek crossing is a 115' single span bridge with a concrete deck on steel plate girder beams. The crossing occurs just after the PVT of a sag vertical curve so its high and low chord elevations are variable, but they exhibit a constant difference of 7'. The proposed bridge also features 2H:1V sloped gravel fill on either side of the bridge along its vertical abutments. The stations and elevations of the bridge's high and low chords are listed below in **Table 9-3**, with stations beginning on the east abutment and increasing towards the west abutment.

Station (ft)	High Chord Elevation (ft)	Low Chord Elevation (ft)
0	691.06	684.06
155	689.5	882.5

Table 9-3: Proposed Bridge Deck Geometry

After running the model, it is found that the 100 year water surface elevation for the existing conditions, just upstream of the roadway crossing at cross section 5254.50*, is 662.76’. When the proposed bridge and vertical alignment is included in the model, the 100 year water surface elevation takes a value of 663.25’. This difference is less than the maximum 1’ allotted to the raise in the 1% A.E.P. flood elevation. Table 9-4 below documents this information and presents the hydraulic outputs at cross section 5254.50* for both the existing and proposed conditions.

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 3	5254.50*	2 year	prop_no_p	515.20	654.05	659.60	657.08	659.78	0.001685	3.44	149.55	42.06	0.32
Reach 3	5254.50*	2 year	existing_cond	515.20	654.05	659.65		659.83	0.001628	3.40	151.50	42.33	0.32
Reach 3	5254.50*	5 year	prop_no_p	786.10	654.05	660.81	657.87	661.03	0.001764	3.83	205.04	51.60	0.34
Reach 3	5254.50*	5 year	existing_cond	786.10	654.05	660.83		661.06	0.001738	3.81	206.40	55.08	0.33
Reach 3	5254.50*	10 year	prop_no_p	1014.90	654.05	661.62	658.45	661.86	0.001896	3.98	273.98	118.07	0.35
Reach 3	5254.50*	10 year	existing_cond	1014.90	654.05	661.63		661.87	0.001880	3.97	275.24	118.98	0.35
Reach 3	5254.50*	25 year	prop_no_p	1332.50	654.05	662.39	659.09	662.63	0.001710	4.08	392.17	167.23	0.34
Reach 3	5254.50*	25 year	existing_cond	1332.50	654.05	662.20		662.48	0.002003	4.35	360.85	163.41	0.37
Reach 3	5254.50*	50 year	prop_no_p	1593.30	654.05	662.94	659.58	663.17	0.001580	4.06	488.24	180.74	0.33
Reach 3	5254.50*	50 year	existing_cond	1593.30	654.05	662.61		662.90	0.002087	4.52	429.82	174.62	0.38
Reach 3	5254.50*	100 year	prop_no_p	1864.00	654.05	663.25	660.03	663.50	0.001671	4.29	545.10	184.94	0.34
Reach 3	5254.50*	100 year	existing_cond	1864.00	654.05	662.76		663.11	0.002524	5.04	455.97	178.05	0.41
Reach 3	5254.50*	500 year	prop_no_p	2525.10	654.05	664.17	661.38	664.42	0.001527	4.47	719.79	198.29	0.33
Reach 3	5254.50*	500 year	existing_cond	2525.10	654.05	663.27		663.72	0.003023	5.78	548.34	185.12	0.46

Table 9-4: Hydraulic Outputs for Existing and Proposed Conditions

The proposed bridge also satisfies all other water surface elevation criteria as water levels are well below the minimum requirements of 1’ below low chord elevation and 3’ below pavement elevation for the 1% A.E.P. flood, and are far from overtopping the structure during the 0.2% A.E.P. flood. The satisfaction of these criteria is documented through **Figure 9-3** and **Tables 9-5 and 9-6**, which are included below.

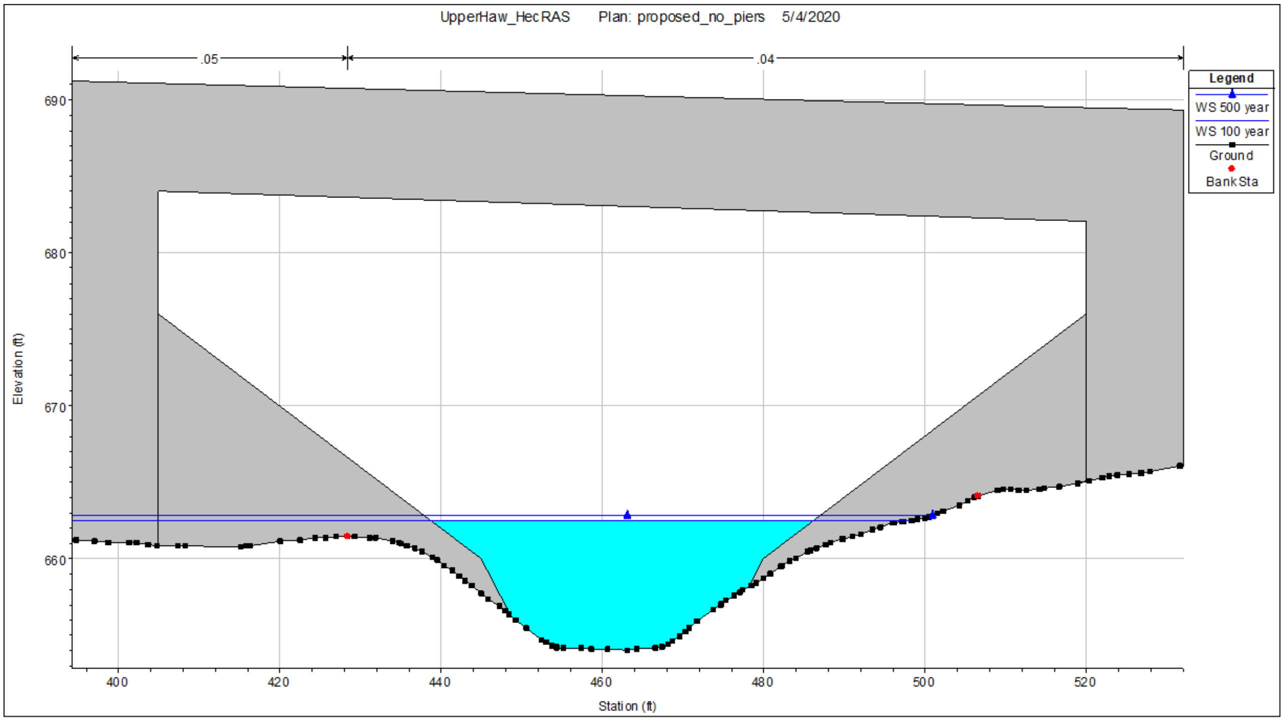

Figure 9-3: Proposed Bridge at Haw Creek Crossing with 100 and 500 year WSELs

Plan: prop_no_p Upper Haw Reach 3 RS: 5250 Profile: 100 year				
E.G. US. (ft)	663.50	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	663.25	E.G. Elev (ft)	663.32	663.11
Q Total (cfs)	1864.00	W.S. Elev (ft)	662.49	662.60
Q Bridge (cfs)	1864.00	Crit W.S. (ft)	660.10	659.26
Q Weir (cfs)		Max Chl Dpth (ft)	8.44	9.16
Weir Sta Lft (ft)		Vel Total (ft/s)	7.29	5.76
Weir Sta Rgt (ft)		Flow Area (sq ft)	255.60	323.63
Weir Submerg		Froude # Chl	0.55	0.34
Weir Max Depth (ft)		Specif Force (cu ft)	1310.26	1530.70
Min El Weir Flow (ft)	689.29	Hydr Depth (ft)	5.39	5.90
Min El Prs (ft)	684.06	W.P. Total (ft)	51.80	63.84
Delta EG (ft)	1.19	Conv. Total (cfs)	27518.0	37205.7
Delta WS (ft)	1.10	Top Width (ft)	47.45	54.89
BR Open Area (sq ft)	2164.12	Frctn Loss (ft)	0.11	0.70
BR Open Vel (ft/s)	7.29	C & E Loss (ft)	0.09	0.11
BR Sluice Coef		Shear Total (lb/sq ft)	1.41	0.79
BR Sel Method	Energy only	Power Total (lb/ft s)	10.31	4.58

Table 9-5: Hydraulic Outputs at Proposed Bridge for 100 year Flood

Plan: prop_no_p Upper Haw Reach 3 RS: 5250 Profile: 500 year				
E.G. US. (ft)	664.42	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	664.17	E.G. Elev (ft)	664.18	663.86
Q Total (cfs)	2525.10	W.S. Elev (ft)	662.84	663.04
Q Bridge (cfs)	2525.10	Crit W.S. (ft)	661.29	660.20
Q Weir (cfs)		Max Chl Dpth (ft)	8.79	9.60
Weir Sta Lft (ft)		Vel Total (ft/s)	9.26	7.24
Weir Sta Rgt (ft)		Flow Area (sq ft)	272.81	348.56
Weir Submerg		Froude # Chl	0.69	0.41
Weir Max Depth (ft)		Specif Force (cu ft)	1708.23	1915.96
Min El Weir Flow (ft)	689.29	Hydr Depth (ft)	5.54	6.11
Min El Prs (ft)	684.06	W.P. Total (ft)	53.72	67.10
Delta EG (ft)	1.62	Conv. Total (cfs)	29940.5	41471.2
Delta WS (ft)	1.54	Top Width (ft)	49.23	57.09
BR Open Area (sq ft)	2164.12	Frctn Loss (ft)	0.16	0.87
BR Open Vel (ft/s)	9.26	C & E Loss (ft)	0.15	0.19
BR Sluice Coef		Shear Total (lb/sq ft)	2.25	1.20
BR Sel Method	Energy only	Power Total (lb/ft s)	20.87	8.71

Table 9-6: Hydraulic Outputs at Proposed Bridge for 500 year Flood

9.3. Culvert Design

The culvert design was a very important aspect to this project. As stated previously the culverts chosen in the design were selected to ensure the client would receive a design that focused on the complexity at each location. The chosen culverts provide many differences in sizing, shape, and material. Our team chose to design culvert 2, bridge (100-year flood plain), culvert 6, culvert 9, and culvert 12. For all culverts, besides the one that passes over the flood plain, HY-8 was used for sizing, material, and shape. The culverts were designed for a 50-year storm, but also provide flow conditions were no roadway over topping for a 100-year storm. Our method of design is outline in the chapter 6 of IDOTs drainage manual. We started the culvert areas be equal to 1/5 the flow design discharge, we then manipulated the sizing at regular sizing intervals until a solution was met. Our team is confident that we designed suitable culvert devices for the roadway.

The first culvert our team would like to highlight is culvert 12. This culvert was stands out from the others, because of shape and number of barrels. The roadway elevation here was much closer to the existing elevation than others. The team designed 2 3’ x 3’ concrete box culvert to be installed side by side. This insured at designed flow conditions that there would be no overtopping, and for there still be adequate cover for corridor construction. Another culvert that is different than the others is culvert 6. This culvert connects our longest swale to the other swale designed. The culvert is be installed along the ditch path, allowing the water to convey underneath INT-4 at STA 158+62.92. When the intersection is replaced. Table 9-7 provides a summary of the designed culverts, and Appendix H has all calculation data, graphic from HY-8.

Name	Peak Discharge Method	50yr Flow (cfs)	100yr Flow (cfs)	Material	Shape	Number of Barrels	Size
Culvert 2	Stream Stats	622	728	Concrete	Circular	1	6' Diameter
Culvert 6	NRCS	62	74	Aluminum	Circular	1	2.5' Diameter

Culvert 9	Stream Stats	1320	1560	Concrete	Box	1	11' wide x 8 ft tall
Culvert 12	HEC HMS	70	83	Concrete	Box	2	3' wide x 3 ft tall

Table 9-7: Summary of Culvert Design

When choosing design conditions for said storms. If the stream stats were available our team used them. Our team believes this is the most accurate method because it is from actual measurements. Culvert 12 we used HEC HMS since its number was very similar to other methods, but was slightly higher. Finally, for culvert 6 NRCS method was used because it was the same method used in swale designed, and this culvert is connected to them.

9.4. Swale Design

Our design team focused on two swales. The first East of culvert 5 and the second west of culvert 5. Our team choose the longest swale as requested by the client. The reason we choose the other swale to be close was the special case of this swale. The other swale not only accepted the water heading out of culvert 5, and additional area. Our team believed that this special case should also be designed for. In the road design phase, we designed the front slope of the corridor to 1V:6H this insured, in the case of an accident, the vehicle would not be falling off a steep slope. The back slope designed was 1V:3H. These two slopes were brought into the design for our trapezoidal swales. The team focused designing the bottom width and ground cover type to ensure the swale could provide conveyance of the water during a design storm with an A.E.P. of 2% or a 50-year return interval. The designs were also checked for 100-year, and 500-year storms. For the swale the peak discharge method used was from the NRCS.

Table 9-8: Design Storms Swale Design

	50 year (fps)	100 year (fps)	500 year (fps)
Other swale	128	153	218
Longest swale	62	74	106

Table 9-9: Overview of Swales

	Station begin	Station end	Flow direction	back slope	front slope	bottom width
Swale 1 (other)	122+69	158+60.26	West	1V:3H	1V:6H	4ft
Swale 2 (longest)	158+90	212+92.87	West	1V:3H	1V:6H	4ft

This region of the project was primarily a silty clay loam. The soil type has a porosity n=0.42 (Tarboton 2003). The USGS soil code used was SC and the soil was considered to have a PI between 10 and 20. We chose the swales to have a mixed growth form and good cover. We also chose a stem height of 0.75 ft. These parameters insured that the max shear was smaller than the max permissible. Our chosen design parameters will provide the client with less upkeep once vegetation is established. A summary of the table to used and sample calculation are provided in appendix H. Our team would like to comment that before design we expected the swales to be quite different, but with construction limitation we choose that there be a minimum of 4 ft bottom width. It happened that this minimum provided an acceptable shear all the way up to a storm with a 500-year return interval. Swale 1 starts at the existing ground elevation of the stream it empties to at STA 121+86.08 at the minimum slope of 0.5% until STA 158+47.92. Here is the culvert 6 outlet. The longest swale then begins at Culvert 6 inlet at STA 158+77.92 and ends at 212+92.87. The closest either swales' bottom is to the edge of pavement elevation is 6.61 ft. Therefore, as illustrated in the appendix H there is no overtopping of the corridor even for a 500-year storm. It civil3d was still available our team would update the profile sheets to illustrate the swale designs.

9.5. Detention Design

Due to the topography of the GATC corridor a single detention basin to serve the entire roadway has been made entirely infeasible and would only produce inaccurate results. In order to accurately design a functional detention center for the GATC corridor, it was decided to use the longest swale as previously mentioned to come up with accurate values for the floodplain storage lost. The culvert at this location will be east of CV-5 with a catchment area of 165.44 acres (Table 10-1) as mentioned earlier which feeds to the downstream area of the detention basin. The criteria for the detention design are as followed:

- Storage Volume ≥ Floodplain Storage lost to construction
- Dry basin drained within 24 hours
- Infiltration basin drained within 48 hours
- Low-Flow outlet at least 1-foot above water level of receiving stream
- Outlet limit peak of 100-year event to no more than 0.2 cfs/acre of impervious area created by construction of the GATC.
- Maximum embankment height is 10 feet
- Side Slopes 3H:1V
- Wet basin 6 foot deep
- Wetland Length:Width = 5:1, alternating depths

Based on the chosen swale the maximum outflow for the design storm according to the given constraints would be 33.09 cfs. Using HEC-HMS the peak discharge for the culvert were found to be as followed.

Table 9-10: Detention Basin Peak Discharge

	50 year (cfs)	100 year (cfs)	500 year (cfs)
Longest swale	69	99	104

The peak outflow charge corresponds to the intersection of the falling limb of the inflow charge found in the table above. The volume of water stored in the basin can be found using a numeric integration between as the area between the outflow and inflow graphs.

In turn the points of introspection on the hydrograph will be the outflow max (33.1 cfs) and inflow max (69 cfs), and the area between these two curves represents the volume of storage for the detention basin. A decision was made to use a low flow outlet orifice due to the small size of the detention basin. Two pipes would be sufficient enough to direct water out of the detention basin when necessary. Based on the parameters and discharge values we do not expect the basin to undergo heavy flows and therefore decided that a drop inlet structure would best be used in conjunction with 2 low flow outlet controls.

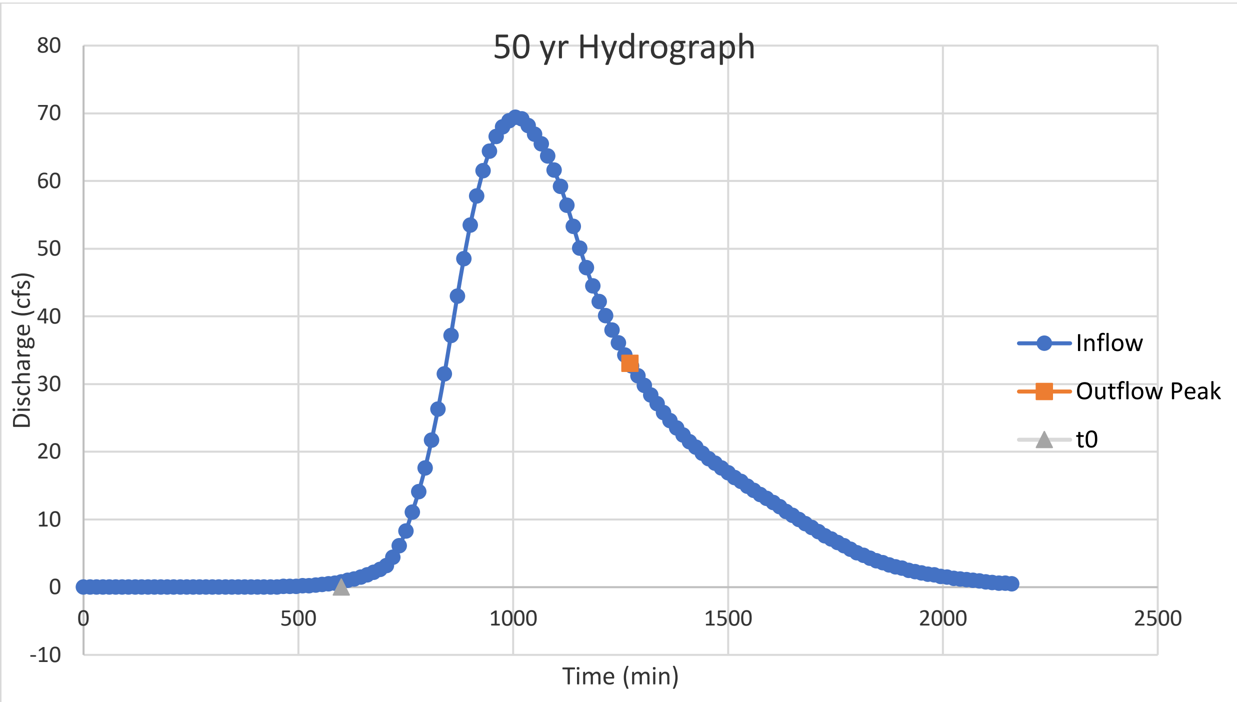

Figure 9-4: 50-year Hydrograph

Taking the area under the curve using integration the water storage volume turns out to be about 32,849 cubic feet. Given this volume and a 48-hour time frame to drain for the basin it would take 684.35 cfh. The infiltration rate of the soil which has a PI of 19 can be approximated to be about .35 in/hr (Tables, H-0-5, H-0-6, H8). The area for the basin can then be calculated as the relationship between the area and infiltration rate equaling the previously mentioned drainage value. The area turns out to be 23,463 square feet. Considering all the parameters 2 low flow outlet orifices would also be assisted by the fact that the area would be an infiltration basin that has the ability to percolate water.

9.6. Site Runoff

To make sure that the runoff volume from a 2 year 24 hour storm for the entire area of the catchment is not greater than the runoff volume for the pre-development conditions the difference in the discharge values multiplied by the area of the basin obtained earlier provides a value of X. The post runoff value is 2.37 inches as found by the NRCS method and the pre runoff value is found to by multiplying the intensity by 24 hours. According to NOAA the

Galesburg predevelopment precipitation frequency is 2.99 inches which contributed to a 1.37-inch pre runoff value. Therefore, the volume to store is 1,955 cubic feet, supporting the constraint that the post construction runoff is less than the pre-construction. The calculations for these values can be found in **Appendix H**.

10. Earthwork

Earthwork volumes were calculated using the average end area method which calculates the average area of cut and fill between two cross sections and multiplies by the distance between them to obtain a volume. This is done along the entire roadway and the cumulative net volume, in compacted cubic yards, is reported at station intervals of 0+50.

A shrinkage of 10% is assumed for all areas of fill, meaning that each cubic yard of cut removed will equal 0.90 cubic yards of fill, due to the compaction of the in-situ soil. Different soils exhibit different amounts of swell and shrinkage, and for Illinois, 10% shrinkage is typical. In addition to the shrinkage factor, a free haul distance of 500' is defined, meaning that there are no costs associated with moving cut for distances less than 500' in our estimations. A cumulative mass haul diagram for the proposed corridor is shown below in Figure 10-1.

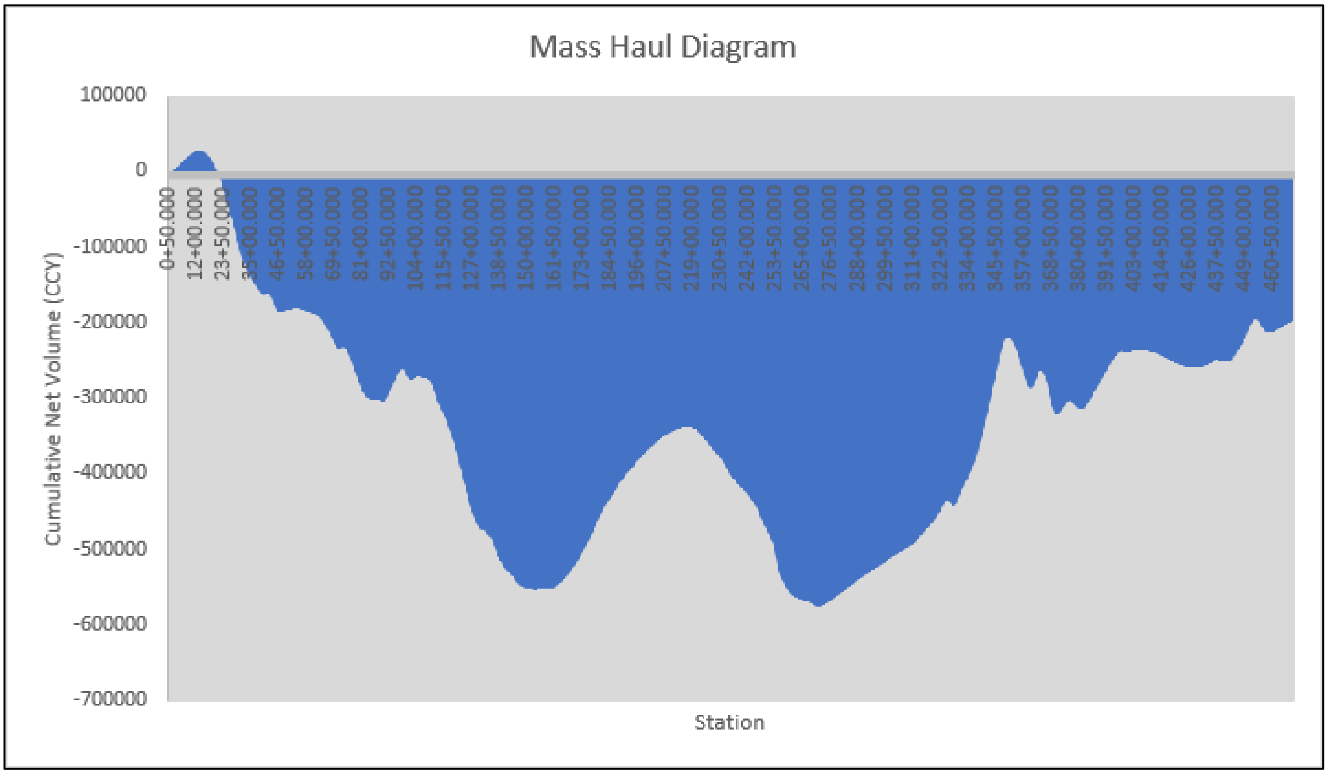

Figure 10-1: Cumulative Mass Haul Diagram

At end station 469+50 of the diagram a cumulative net volume of -197302.23 CCY is reported, indicating that approximately 197,302 CCY of fill is needed for this project. This value is an overestimation however, as it was calculated disregarding the bridge opening over the railroad and areas which will be open for culverts. At a bid price of $85.00/CY, the total cost for furnished excavation is estimated to be $16,770,670.00.

11. Economic Analysis

A preliminary cost estimate of the proposed alignment consists of the cost of pavement structure, land acquisition, slope stabilization, and earthwork. Drainage factors including culverts, detentions/wetlands, and scour protection for swales, are also included in the cost analysis. A consulting fee rate of $200 per hour per person is added to the preliminary cost estimate. Table 11-1 below shows a summary of the economic analysis. All values were obtained from previous projects archived by the IDOT. The estimated cost of the GATC is $35,539,452.23.

Table 11-1: Economic Analysis

Item	Units Needed	Unit Price	Item Cost
Land Acquisition (Acre)	254.55	$ 6,970.00	$ 1,774,213.50
HMA SC "C" N50 (TON)	111804.1	$ 66.00	$ 7,379,070.60
Lime (TON)	77064.28	$ 95.00	$ 7,321,106.60
Furnished Excavation (CU YD)	197302	$ 85.00	$ 16,770,670.00
Box Culvert (CU YD)	2840	$ 604.00	$ 1,715,360.00
Circular Culvert (CU YD)	58.86	$ 239.84	$ 14,116.98
Detention Basin (CU YD)	1216.63	$ 85.00	$ 103,413.55
Seeding (SQ FT)	1698804	$ 0.25	$ 424,701.00
Consulting fee (HRS)	184	$ 200.00	$ 36,800.00

TOTAL COST	**$ 35,539,452.23**

12. Conclusion

Team 11 came up with various roadway options for a new autonomous vehicle highway in order to better optimize the delivery of a hemp and produce processing and shipping facility between Knox County and the BNSF railway in the wake of a booming alternative crop and vertical farming industry.

The highway will be primarily designed for level 5 autonomous vehicles (0 human control) which will take the form of WB-65 trucks. Using V2V communication in conjunction with the visual and radar-based sensor technology embedded in the vehicles, the trucks would platoon together to optimize their mileage and movement. These factors have a direct impact on the geometric design of the road compared to that of traditional roadways because the design parameters significantly change in the presence of the faster PRT attributed to autonomous vehicles.

Alternative alignments were considered for the path taken between the processing and shipping facilities and ranked based upon a myriad of economic, environmental, and social impact factors. The elevation profile also helped narrow down the final decision as the one with the least variation provided an opportunity for having the lowest construction expenses. The-mile designed road for the Galesburg Southeast bypass corridor will work on a 11 foot 2-lane operation with 1 bridge connection over a railroad using a shared-use shoulder and bike path absent of a median. Additionally, the roadway corridor will undergo 10 horizontal curves and 12 vertical curves on its 8.85-mile path. With the lowest number of stream crossings at (9) and interchanges (8) and the lowest max elevation change of 17.3% alongside the environmental and geographic factors the decision to choose Route 3 presented itself as the best option.

Likewise, pavement design depended on numerous factors such as traffic, cost comparison, weather, and the presence of existing paths. Ideally, asphalt presented itself as the best material choice for the surface because of its strength in freeze-thaw cycles that dominate the weather pattern. A 16.5 HMA thickness and 19.96 Truck Factor value were proper design parameters because of the load count from AV on this path.

Drainage considerations and stream crossings were the final parameter considered in the design of this roadway because culverts are necessary to properly facilitate water movement. According to floodplain data, the culverts will be strategically placed at stream crossings, while different tactics such as water detention areas and roadside ditches will address water problems along the highway. This holistic process will guarantee a properly designed highway that minimizes the effect on the surrounding community but also provides economic opportunity for growing businesses within a large area with more access.

After calculating the cost for individual items, the total estimated cost of the GATC is $35,539,452.23.

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Appendices

Appendix A – Log of Work Hours

Table A-0-1: Log of Work Hours

Team Member	Phase 1 Hours	Phase 2 Hours	Phase 3 Hours
Shain Hennessy	**Total:** 13 **Map Design:** 7 **Route Analysis:** 3 **Written Report:** 3	**Total:** 31 **CAD:** 26 **Map Updates:** 3 **Comment Resolution:** 2	**Total:** 21 **Drainage Written:** 4 **Drainage Analysis:** 17
Heli Shah	**Total:** 16 **Written Report:** 10 **Geometric Constraints:** 4 **Pavement Design:** 2	**Total:** 11 **Cross Sections:** 3 **Superelevation:** 4 **Economic Analysis:** 2 **Written Report:** 2	**Total: 10** **Drainage Written: 6** **Drainage Analysis: 4**
Mihir Thakar	**Total:** 11 **Written Report**: 11	**Total:** 12 **CAD Report Analysis:** 2 **Sample Calculation**s: 2 **Written Report:** 7	**Total: 12** **Drainage Written: 11** **Drainage Analysis: 1**
Matthew Kuechenberg	**Total:** 11 **Alignment Design:** 2 **Soil Analysis:** 3 **Written Report:** 6	**Total:** 12 **CAD:** 3 **Earthwork:** 2 **Written Report:** 7	**Total:** 25 **Drainage Written:** 4 **Drainage Analysis:** 21
Total	50	66	68

Appendix B – Alignment Control Points and Features

Table B-0-1: Intersection Locations

Intersection	Description	Latitude	Longitude	Type of Road	Station	Elevation	Acute Angle
INT-1	IL RT 41	40°52'55.8"N	90°24'15.7"W	Primary	0+00	772	91.10
INT-2	Knox Rd 300 E	40°52'54.8"N	90°23'07.0"W	Secondary	52+69.82	767.79	88.99
INT-3	Knox Hwy 25	40°52'56.5"N	90°21'59.8"W	Secondary	104+74.08	744.06	88.48
INT-4	Knox Rd 500 E	40°52'55.6"N	90°20'49.5"W	Secondary	158+62.92	766.4	90.78
INT-5	600 E	40°52'57.6"N	90°19'38.9"W	Secondary	212+92.87	771.26	89.07
INT-6	Co Hwy 8	40°52'54.4"N	90°17'20.0"W	Primary	320+02.37	729.66	70.11
INT-7	Knox Rd 900 E	40°52'55.6"N	90°15'38.3"W	Secondary	403+10.77	728.82	87.59
INT-8	IL RT 97	40°53'02.2"N	90°14'13.5"W	Primary	469+95.59	723.19	85
BRD-1	BNSF Railway	40°52'55.3"N	90°23'40.9"W	Railroad	26+77.34	TOP: 799.48 Railroad: 771.52	90.62

Table B-0-2: Control Point Locations

ID	Latitude	Longitude
CP-1	40°52'53.9"N	90°22'33.0"W
CP-2	40°52'56.6"N	90°22'12.1"W
CP-3	40°52'55.5"N	90°20'41.2"W
CP-4	40°52'57.7"N	90°19'56.0"W
CP-5	40°52'56.1"N	90°17'38.9"W
CP-6	40°52'52.4"N	90°16'58.7"W
CP-7	40°52'42.2"N	90°16'26.3"W
CP-8	40°52'45.2"N	90°16'08.1"W
CP-9	40°52'55.4"N	90°15'47.7"W
CP-10	40°52'56.6"N	90°14'22.9"W

Table B-0-3: Superelevation Control Point Location

Superelevation Region	Station	Description	Left Outside Shoulder	Left Outside Lane	Right Outside Shoulder	Right Outside Lane
	0+00.00'	Begin Alignment	-4.00%	-1.50%	-4.00%	-1.50%
Curve 1	75+89.69'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	76+53.44'	End Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	76+91.69'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	77+29.94'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	77+93.69'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	78+44.69'	Begin Full Super	-6.00%	-6.00%	6.00%	6.00%
	79+35.86'	End Full Super	-6.00%	-6.00%	6.00%	6.00%
	79+86.86'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	80+50.61'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	80+88.86'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	81+27.11'	Begin Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	81+90.86'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
Curve 2	92+21.91'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	92+85.66'	End Normal Crown	-1.50%	-1.50%	-4.00%	-1.50%
	93+23.91'	Level Crown	0.00%	0.00%	-4.00%	-1.50%
	93+62.16'	Reverse Crown	1.50%	1.50%	-4.00%	-1.50%
	94+25.91'	Low Shoulder Match	4.00%	4.00%	-4.00%	-4.00%
	94+76.91'	Begin Full Super	6.00%	6.00%	-6.00%	-6.00%
	95+59.28'	End Full Super	6.00%	6.00%	-6.00%	-6.00%
	96+10.28'	Low Shoulder Match	4.00%	4.00%	-4.00%	-4.00%
	96+74.03'	Reverse Crown	1.50%	1.50%	-4.00%	-1.50%
	97+12.28'	Level Crown	0.00%	0.00%	-4.00%	-1.50%
	97+50.53'	Begin Normal Crown	-1.50%	-1.50%	-4.00%	-1.50%
	98+14.28'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
Curve 3	162+52.93'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	163+16.77'	End Normal Crown	-4.00%	-1.50%	-4.00%	-1.50%
	163+55.08'	Level Crown	-4.00%	-1.50%	-4.00%	-1.50%
	163+93.38'	Reverse Crown	-4.00%	-1.50%	-4.00%	-1.50%
	164+57.22'	Low Shoulder Match	-4.00%	-4.00%	-4.00%	-4.00%
	164+98.08'	Begin Full Super	-5.60%	-5.60%	-6.00%	-6.00%
	165+03.00'	End Full Super	-5.60%	-5.60%	-6.00%	-6.00%
	165+43.86'	Low Shoulder Match	-4.00%	-4.00%	-4.00%	-4.00%
	166+07.70'	Reverse Crown	-4.00%	-1.50%	-4.00%	-1.50%
	166+46.00'	Level Crown	-4.00%	-1.50%	-4.00%	-1.50%
	166+84.31'	Begin Normal Crown	-4.00%	-1.50%	-4.00%	-1.50%
	167+48.15'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
Curve 4	197+32.39'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	197+96.23'	End Normal Crown	-1.50%	-1.50%	-4.00%	-1.50%
	198+34.54'	Level Crown	0.00%	0.00%	-4.00%	-1.50%
	198+72.84'	Reverse Crown	1.50%	1.50%	-4.00%	-1.50%
	199+36.68'	Low Shoulder Match	4.00%	4.00%	-4.00%	-4.00%
	199+77.54'	Begin Full Super	5.60%	5.60%	-6.00%	-6.00%
	199+81.12'	End Full Super	5.60%	5.60%	-6.00%	-6.00%
	200+21.98'	Low Shoulder Match	4.00%	4.00%	-4.00%	-4.00%
	200+85.82'	Reverse Crown	1.50%	1.50%	-4.00%	-1.50%
	201+24.12'	Level Crown	0.00%	0.00%	-4.00%	-1.50%
	201+62.43'	Begin Normal Crown	-1.50%	-1.50%	-4.00%	-1.50%
	202+26.27'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%

Curve 7	358+35.57'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	358+99.32'	End Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	359+37.57'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	359+75.82'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	360+39.57'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	360+90.57'	Begin Full Super	-6.00%	-6.00%	6.00%	6.00%
	365+19.80'	End Full Super	-6.00%	-6.00%	6.00%	6.00%
	365+70.80'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	366+34.55'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	366+72.80'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	367+11.05'	Begin Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	367+74.80'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
Curve 8	373+60.10'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	374+23.85'	End Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	374+62.10'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	375+00.35'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	375+64.10'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	376+15.10'	Begin Full Super	-6.00%	-6.00%	6.00%	6.00%
	378+35.74'	End Full Super	-6.00%	-6.00%	6.00%	6.00%
	378+86.74'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	379+50.49'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	379+88.74'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	380+26.99'	Begin Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	380+90.74'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
Curve 9	391+06.37'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	391+70.12'	End Normal Crown	-1.50%	-1.50%	-4.00%	-1.50%
	392+08.37'	Level Crown	0.00%	0.00%	-4.00%	-1.50%
	392+46.62'	Reverse Crown	1.50%	1.50%	-4.00%	-1.50%
	393+10.37'	Low Shoulder Match	4.00%	4.00%	-4.00%	-4.00%
	393+61.37'	Begin Full Super	6.00%	6.00%	-6.00%	-6.00%
	398+23.09'	End Full Super	6.00%	6.00%	-6.00%	-6.00%
	398+74.09'	Low Shoulder Match	4.00%	4.00%	-4.00%	-4.00%
	399+37.84'	Reverse Crown	1.50%	1.50%	-4.00%	-1.50%
	399+76.09'	Level Crown	0.00%	0.00%	-4.00%	-1.50%
	400+14.34'	Begin Normal Crown	-1.50%	-1.50%	-4.00%	-1.50%
	400+78.09'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
Curve 10	455+93.20'	End Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	456+56.95'	End Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	456+95.20'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	457+33.45'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	457+97.20'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	458+48.20'	Begin Full Super	-6.00%	-6.00%	6.00%	6.00%
	463+28.51'	End Full Super	-6.00%	-6.00%	6.00%	6.00%
	463+79.51'	Low Shoulder Match	-4.00%	-4.00%	4.00%	4.00%
	464+43.26'	Reverse Crown	-4.00%	-1.50%	1.50%	1.50%
	464+81.51'	Level Crown	-4.00%	-1.50%	0.00%	0.00%
	465+19.76'	Begin Normal Crown	-4.00%	-1.50%	-1.50%	-1.50%
	465+83.51'	Begin Normal Shoulder	-4.00%	-1.50%	-4.00%	-1.50%
	469+95.59'	End Alignment	-4.00%	-1.50%	-4.00%	-1.50%

Table B-0-4: Vertical Curve Alignment

No.	Grade In	Grade Out	Profile Curve Type	PVC Station	PVC Elevation	PVI Station	PVI Elevation	PVT Station	PVT Elevation	Profile Curve Length	Curve Radius
1	-1.30%	3.90%	Sag	7+65.08	762.45'	11+27.41'	757.75'	14+98.75	771.88'	724.67'	13946.92'
2	3.90%	-4.05%	Crest	17+39.95	781.63'	26+77.34'	818.19'	36+14.74	780.21'	1874.79'	23578.33'
3	-4.05%	0.75%	Sag	38+23.13	771.76'	41+30.72'	759.30'	44+38.31	761.61'	615.18'	12811.89'
4	0.75%	-1.19%	Crest	55+82.85	770.19'	58+32.92'	772.06'	60+82.98	769.08'	500.13'	25747.59'
5	-1.19%	0.72%	Sag	89+12.98	735.33'	90+37.72'	733.84'	91+62.45	734.74'	249.47'	13052.77'
6	0.72%	-0.56%	Crest	142+94.33	771.62'	145+77.18'	773.65'	148+60.04	772.05'	565.71'	44078.77'
7	-0.56%	0.51%	Sag	178+84.52	754.97'	179+66.42'	754.51'	180+48.32	754.92'	163.80'	15307.23'
8	0.51%	-0.51%	Crest	222+97.20	776.39'	237+29.11'	783.63'	251+61.03	776.36'	2863.83'	282757.99'
9	-0.51%	-1.63%	Crest	306+95.49	748.27'	309+11.70'	747.17'	311+27.92	743.64'	432.43'	38459.80'
10	-1.63%	0.80%	Sag	346+06.60	686.87'	347+58.72'	684.39'	349+10.84	685.61'	304.24'	12510.76'
11	0.80%	-1.33%	Crest	412+43.15	736.26'	420+35.66'	742.60'	428+28.18	732.07'	1585.02'	74484.93'
12	-1.33%	1.01%	Sag	448+70.83	704.95'	450+04.22'	703.18'	451+37.61	704.52'	266.78'	11432.96'

Appendix C – Traffic Factor Calculations

GATC is classified as a Class II road due to the AADT of 8,000. The design period (DP) is 20 years because it is standard value for the design of a Class II roadway. It is assumed that 50% of the traffic will be multi-unit (MU) trucks, so the MU is 4,000. GATC connects two shipping facilities and will be used for the shipment of goods and people. The exact percentage of single-unit (SU) trucks and passenger vehicles (PV) is unknown so the table below is referenced. Since GATC is 2 lanes, 50% of the traffic can be assumed as SU and 50% of the traffic can be assumed as SU.

Number of Facility Lanes	Percent of Total Vehicular Class Volume (ADT) in Design Lane					
	Rural			Urban		
	PV	SU	MU	PV	SU	MU
2 or 3 *	50%	50%	50%	50%	50%	50%
4 or 5	32%	45%	45%	32%	45%	45%
≥ 6	20%	40%	40%	8%	37%	37%

* *2 or 3 lane facilities include all one-way roads and streets.*

DESIGN LANE DISTRIBUTION FACTORS FOR STRUCTURAL DESIGN TRAFFIC

Figure 44-1B

The following equation is used to calculate TF for Class II vehicles (IDOT BLSR Figure 44-4A). The load limit is 80,000 pounds which aligns with the characteristics of WB-65 trucks.

Class II Roads and Streets	
2 or 3 Lane Pavements	$TF = DP\left[\frac{(0.073PV + 56.030SU + 192.720MU)}{1{,}000{,}000}\right]$

TF = 20 [(0.073*4,000 + 56.030*4,000 + 192.720*4,000) / (1,000,000)] = 19.96

Appendix D – Soil Information and FEMA FIRM

Table D-0-1: AASHTO Soil Classification System

AASHTO Soil Classification System (from AASHTO M 145 or ASTM D3282)

General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	...	...	...	...	...	...	...	...	...	...
0.425 (No. 40)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	...		...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[1]
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30.

Appendix E – Pavement Selection

PG Binder Grade Selection [1]			
	Traffic Loading Rate (Adjustment)		
Districts 1 – 4	Standard Traffic [2]	Slow Traffic [3]	Standing Traffic [4]
Surface[5]	PG 58-28 [6][7]	PG 64-28 or SBS PG 64-28	SBS PG 70-28
Remaining Lifts[5]	PG 64-22 or PG 58-22	PG 64-22 or PG 58-22	PG 64-22 or PG 58-22
Districts 5 – 9			
Surface[5]	PG 64-22 [6][7]	PG 70-22 or SBS PG 70-22	SBS PG 76-22
Remaining Lifts[5]	PG 64-22	PG 64-22	PG 64-22

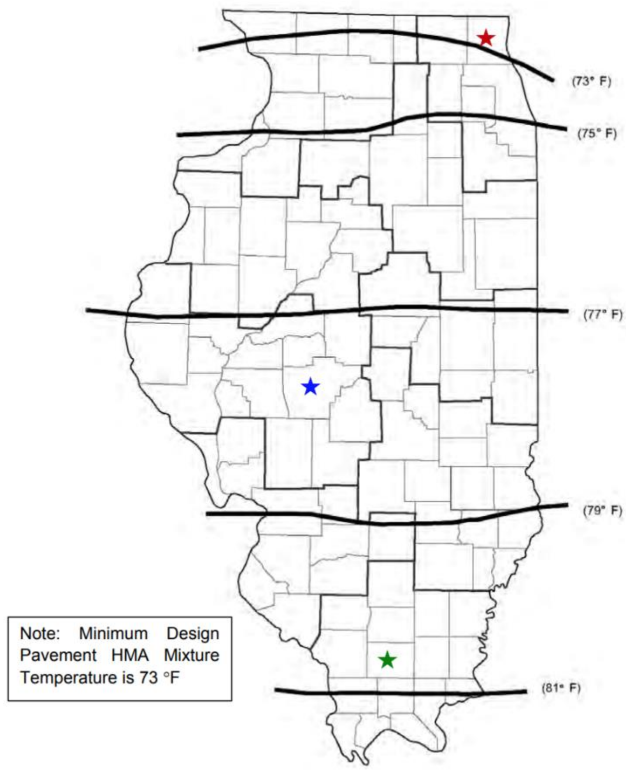
Notes:

1. *The binder grades provided in this table are based on the recommendations given in Illinois-Modified AASHTO* MP-2, Table 1, “Binder Selection on the Basis of Traffic Speed and Traffic Level.”
2. *Standard traffic is used where the average traffic speed is greater than 43 mph (70 km/h).*
3. *Slow traffic is used where the average traffic speed ranges from 12 to 43 mph (20 to 70 km/h).*
4. *Standing traffic is used where the average traffic speed is less than 12 mph (20 km/h).*
5. *Consideration should be given to increasing the high temperature grade by one grade equivalent when 10 ≤ T.F. ≤ 30. For example, if use of a PG 64-22 is specified for standard traffic, a PG 70-22 or a SBS PG 70-22 should be specified.*
6. *Surface includes the top 2 in. (50 mm) of HMA. The remaining lifts of HMA may be the same PG binder grade as surface; however, this may increase or decrease the pavement design thickness. If multiple PG Binder grades are used in a HMA design, the predominant PG Binder grade should be used for determining HMA Modulus on Figure 44-4H.*
7. *The high temperature grade should be increased by one grade equivalent when T.F. > 30. For example, if use of a PG 64-22 is specified for standard traffic, a PG 70-22 or a SBS PG 70-22 should be specified.*

PG BINDER GRADE SELECTION FOR FULL-DEPTH HMA PAVEMENTS

Figure 44-4C

Galesburg is in IDOT district 4 and standard traffic is assumed. The surface PG Binder Grade is PG 58-28.



DESIGN PAVEMENT HMA MIXTURE TEMPERATURE
(Full Depth)

Figure 44-4E

Galesburg is located in the second section from the top of the map. The design pavement HMA mixture temperature will be 75 degrees Fahrenheit.

44-4.02(g) Design Reliability

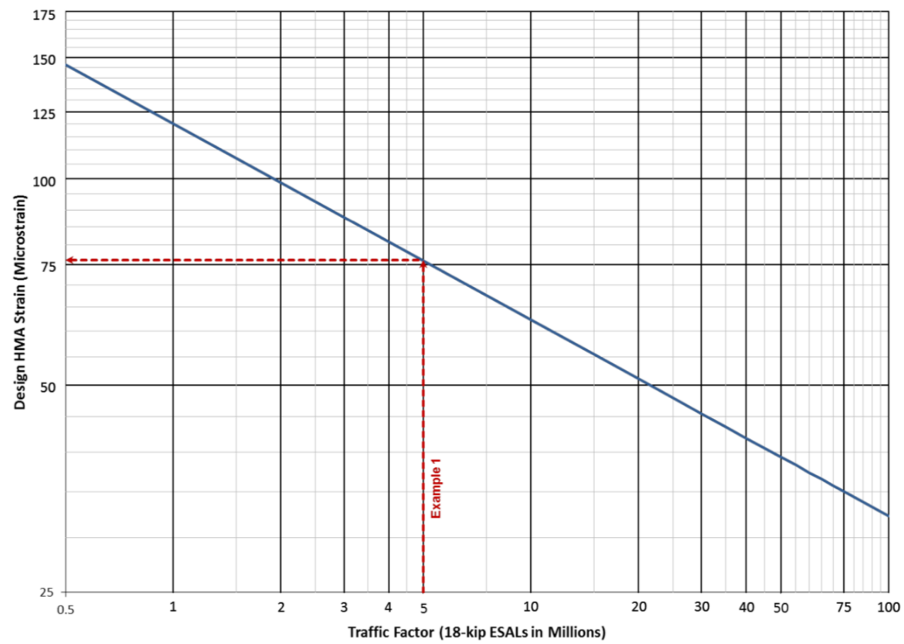
Design reliability is considered through traffic factor multipliers applied to the design TF. These traffic multipliers are built into the design HMA strain curves in Figures 44-4F and 44-4G. The minimum reliability levels by class of road are given in Figure 44-4D.

Road Class	Minimum Reliability Level	Reliability (%)
Class I, II, III, and IV	High	90's

DESIGN RELIABILITY

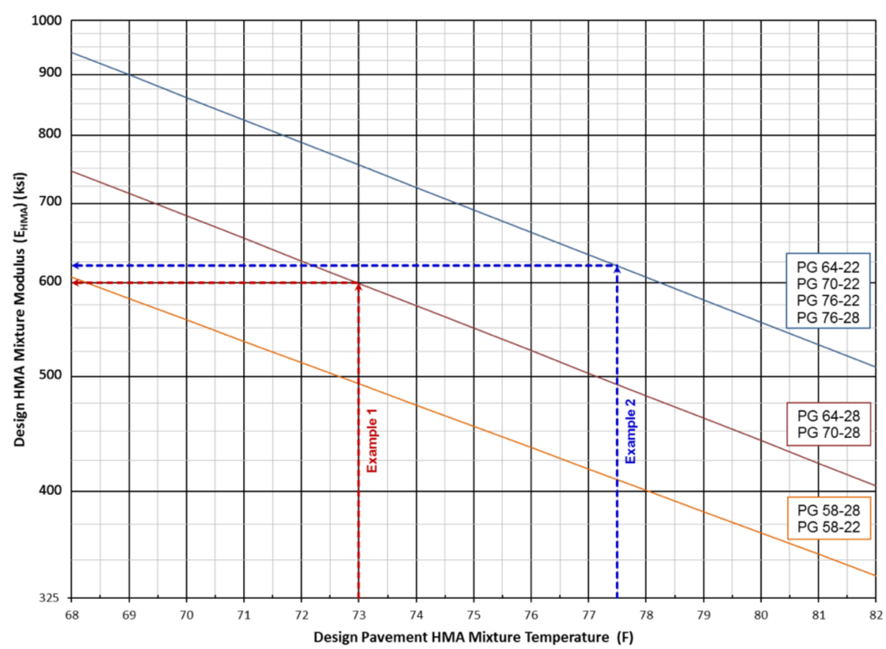
Figure 44-4D

The design reliability is high in the 90% range.



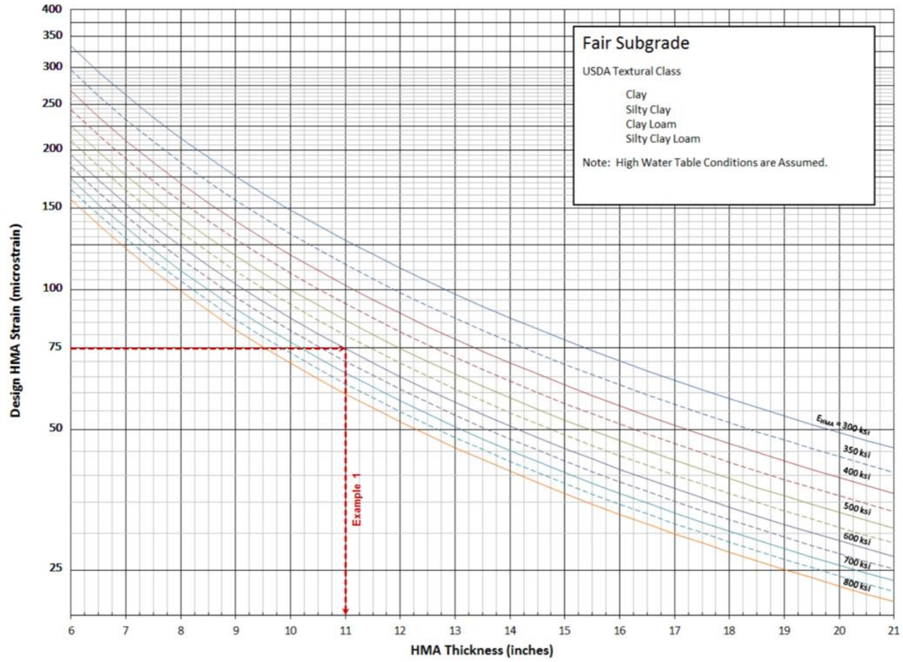
DESIGN HMA STRAIN
(Traffic Factor Relation for Traffic Factor ≥ 0.5)
Figure 44-4G

With a TF of 19.96, the Design HMA Strain is about 51 MicroStation.



HMA MIXTURE MODULUS (E_{HMA})
Figure 44-4H

After the selection of PG 58-28 and the assumption of 75 degrees HMA mixture temperature, the EHMA is about 450 ksi.



FAIR SUBGRADE DESIGN CHART

Figure 44-4J

EHMA is calculated to be around 450 ksi and using the previously calculated Design HMA Strain, the HMA thickness is 16.5 inches.

Appendix F – Sample Calculations

Superelevation Runoff
L = (e)(W)(RS) BLSR Equation 29-3.1

L = superelevation runoff length for a two-lane roadway (assuming the axis of rotation is about the roadway centerline), ft – Typical values can be found in BLSR Figures 29-3B, 29-3C, 29-3D
e = design superelevation rate (0.08 ft/ft), decimal
W = width of rotation for one lane (11 ft)
RS = reciprocal of relative longitudinal gradient between the profile grade and outside edge of two-lane roadway – Value can be found in BLSR Figure 29-3A

Sample Calculation for GATC:
L = (0.08)(11ft)(200) = 176 ft

Tangent Runout
TR = (NC)(W)(RS) BLSR Equation 29-3.2

TR = tangent runout length for a two-lane roadway, (assuming the axis of rotation is about the roadway centerline), ft
NC = normal crown slope (0.015 ft/ft), decimal
W = width of rotation for one lane (11 ft)
RS = reciprocal of relative longitudinal gradient between the profile grade and outside edge of two-lane roadway– Value can be found in BLSR Figure 29-3A

Sample Calculation for GATC:

TR = (0.015)(11ft)(200) = 33 ft

US Customary			Metric		
Design Speed (mph)	Maximum Relative (G) Gradient (%)	Reciprocal (RS)	Design Speed (km/h)	Maximum Relative (G) Gradient (%)	Reciprocal (RS)
20	0.74	135	30	0.75	133
25	0.70	143	40	0.70	143
30	0.66	152	50	0.65	150
35	0.62	161	60	0.60	167
40	0.58	172	70	0.55	182
45	0.54	185	80	0.50	200
50	0.50	200	90	0.47	213
55	0.47	213	100	0.44	227
60	0.45	222			

MAXIMUM RELATIVE LONGITUDINAL GRADIENTS

Figure 29-3A

Horizontal Curve Calculations

The 3rd horizontal curve values were chosen to support the geometric design parameters chosen for the roadway corridor. The consistent super elevation value of .06 was used for the horizontal curves while a side friction of .14 was used in accordance with BLRS values for a 50-mph roadway. Additionally, the deflection angle of 37.29 degrees between the two tangents provided the necessary data to find the tangent length and curve length. The PI station value of 457 + 97.2' helped obtain the PC and PT stations.

$$R_{min} = \frac{u^2}{15(e+f)} = \frac{50^2}{15(.06+.14)} = 833.33\ ft$$

R_{min} = radius of curve
u = design speed
e = super elevation
f = side friction

$$T = R * tan\frac{\Delta}{2} = 833.33 * \tan\left(\frac{37.29}{2}\right) = 281.18\ ft$$

T = Tangent length
u = design speed
Δ = deflection angle

$$PC = PI - T = 457 + 97.2' - 281.18 = 455 + 16.02'$$

PC = Point of Curve
PI = Point of Intersection

$$L = \frac{2\pi R\Delta}{360} = \frac{2 * \pi * 833.33 * 37.29}{360} = 542.36\ ft$$

L = Curve Length

$$PT = PC + L = 455 + 16.02' + 542.36 = 460 + 58.38'$$

PT = Point of Tangent

Horizonal Curve Calculations:

Curve Radius	833.33
Curve Length	542.36
Tangent Length	281.18
PI Station	457+97.2
PC Station	455+16.02
PT Station	460+58.38

Vertical Curve Calculations

The first sag vertical curve values were chosen to support the geometric design parameters chosen for the roadway corridor. The constants of Perception Reaction Time used were .5 seconds according to AV abilities and the grade found at that curve to find the stopping sight distance. The algebraic difference in grades ‘A’ used the different grades on the crest or sag of the curve, while the lengths of the curves were taken using various situational parameters. The point of vertical curve and vertical tangent stations were found using the known point of vertical intersection station using the minimum length found by the Civil 3D program, while the elevations of each station were found given the known PVI elevation. Lastly, the K value was determined by length and the grade differential.

$$S = SSD = 1.47ut + \frac{u^2}{30(\frac{a}{g} + G)} = 1.47 * 50 * .5 + \frac{50^2}{30(.35 + .039)} = 250.97\ ft$$

SSD = S = Stopping Sight Distance
u = Design Speed
t = Perception Reaction Time
a/g = car deceleration rate/gravity
G = Road Grade

$$A = |3.90 - -1.30| = 5.2\%$$

A = Algebraic grade difference

Headlight Requirement:
Assume S < L

$$L_{min} = \frac{AS^2}{400 + 3.5S} = 256.2\ ft$$

L_{min} = Minimum Length of Sag Curve

Assume S > L

$$L_{min} = 2S - \frac{400 + 3.5S}{A} = 256.09\ ft$$

L_{min} = Minimum Length of Sag Curve

Comfort Criterion:

$$L_{min} = \frac{Au^2}{46.5} = 279.57\ ft$$

Appearance:

$$L_{min} = 100A = 520\ ft$$

Civil 3D Value:

$$L_{min}(Civil\ 3D) = 724.67ft$$

Stations:

$$PVC = PVI - \frac{L}{2} = 11 + 27.41 - \frac{724.67}{2} = 7 + 65.08\ ft$$

PVC = Point of Vertical Curve
PVI = Point of Vertical Intersection

$$PVT = PVI + \frac{L}{2} = 11 + 27.41 + \frac{724.67}{2} = 14 + 89.75$$

PVT = Point of Vertical Tangent

Station Elevations:

$$PVC = 757.75 + \frac{1.3\%}{100} * 0.5L = 762.46$$

$$PVT = 757.75 + \frac{3.9\%}{100} * 0.5L = 771.88$$

K Value:

$$K = \frac{L}{A} = \frac{724.67}{5.2} = 19.36$$

Vertical Curve Calculations:

Stopping Sight Distance	250.97’
Algebraic Grade Difference	5.2%
Headlight Curve Length Minimum	256.2’
Headlight Curve Length S > L	256.09’
Comfort Curve Length Minimum	279.57’
Appearance Curve Length Minimum	520’
PVI Station	11 + 27.41
PVC Station	7 + 65.08
PVT Station	14 + 89.75
PVI Station	757.75’
PVC Elevation	762.46’
PVI Elevation	771.88’
PVT Elevation	19.36

Appendix G – Horizontal and Vertical Alignment Plan

Appendix H- Drainage Appendix

Catchment	Area (ac)	Tc (min)	50 yr Flow (cfs)	100 yr Flow (cfs)	500 yr Flow (cfs)
CV-2 CA	637.8	753.3	622	728	-
CV-6 CA	165.4	356.1	62	74	-
CV-9 CA	1534.0	418.5	1320	1560	-
BRDG CA	7690.0	873.5	1593	1864	2525
CV-12 CA	113.6	349.4	70	83	-

Table X: Hydrologic Data used for Design

Hydrologic Equations Used to Calculate Design Discharges:

<u>NRCS Method</u>
Qp = qu*A*Q (cfs)
Where qu = unit peak discharge (cfs/sq mi/in of runnoff)
A = area (sq mi)
Q = runoff (in)

<u>HEC HMS (NRCS Curve Number Method)</u>
S = 1000/CN - 10 (in)
Ia = 0.2*S (in)
Q = (P- Ia)^2/(P+0.8*S) (in)
P50yr,24hr = 5.94 (in)
P100yr,24hr = 6.7 (in)
P500yr, 24hr = 8.63 (in), SCS Type 2 Storm

<u>Rational Method</u>
Q = c*i*A (cfs)
Where c = rational runoff coefficient
i=rainfall intensity (in/hr)
A = drainage area (acres)

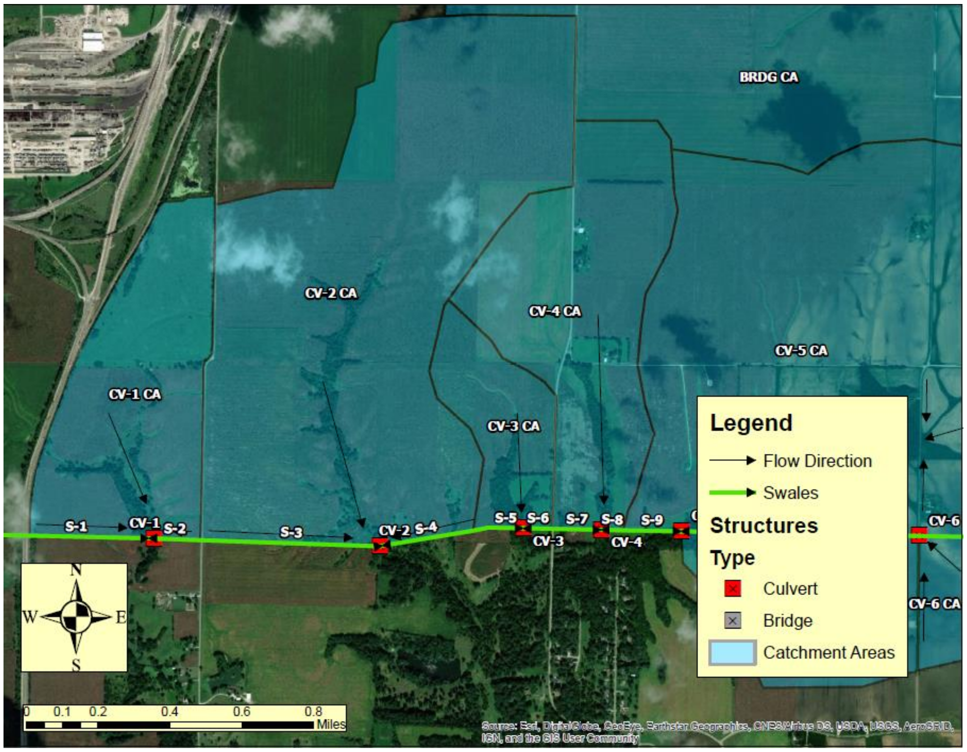


Figure H01: Drainage Map for Culverts 1-4

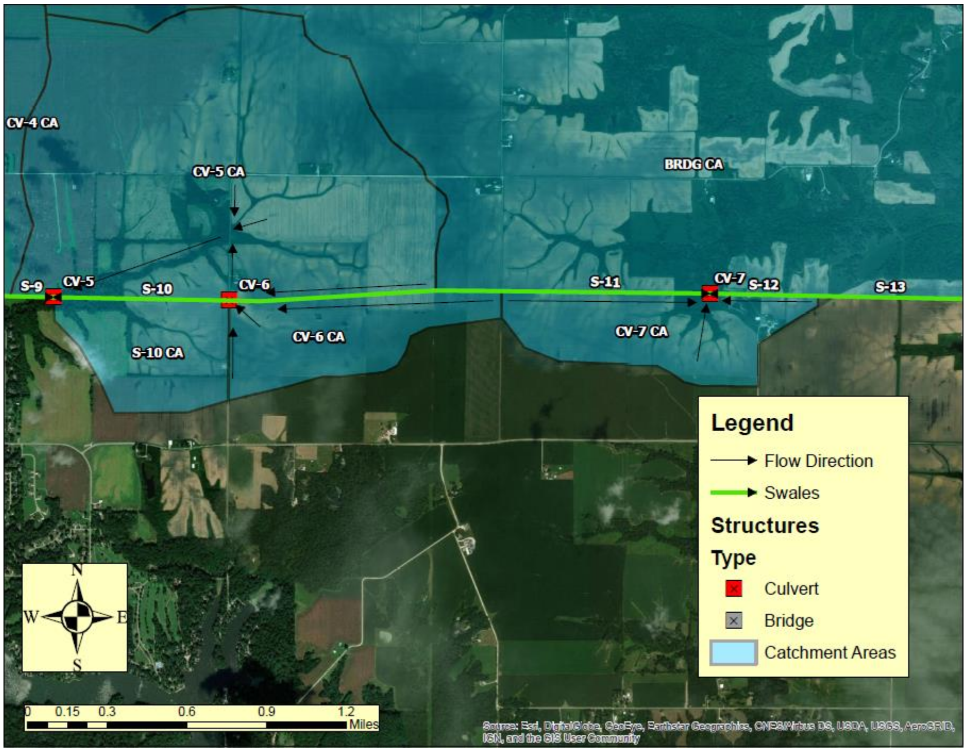


Figure H02: Drainage Map for Culverts 5 – 7

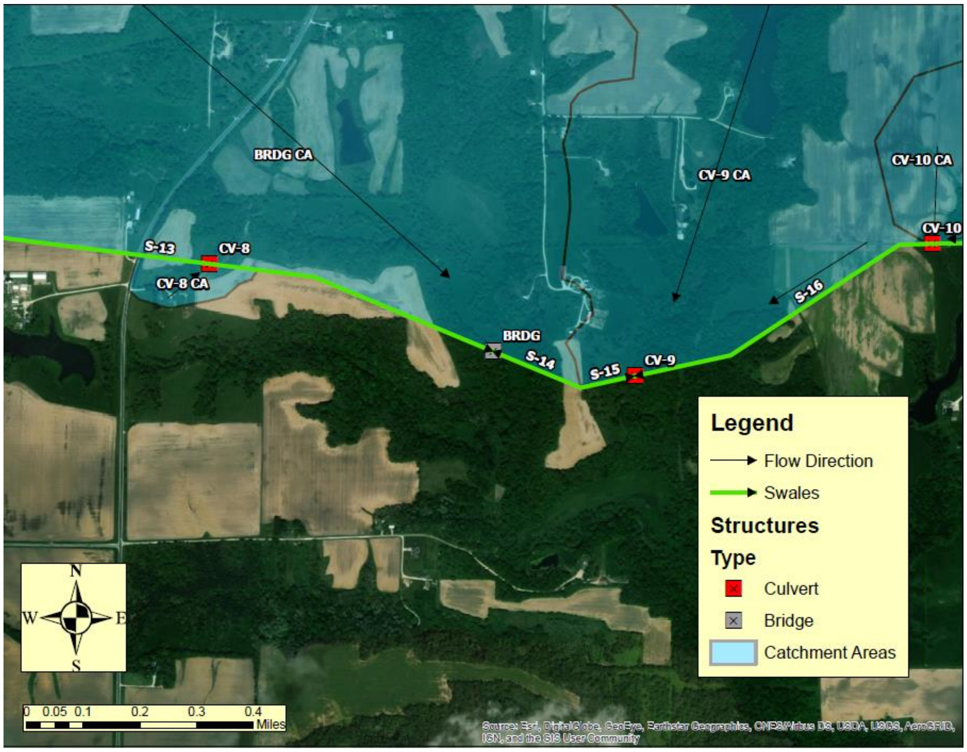


Figure H03: Drainage Map for Culverts 8 and 9 and Bridge over Haw Creek

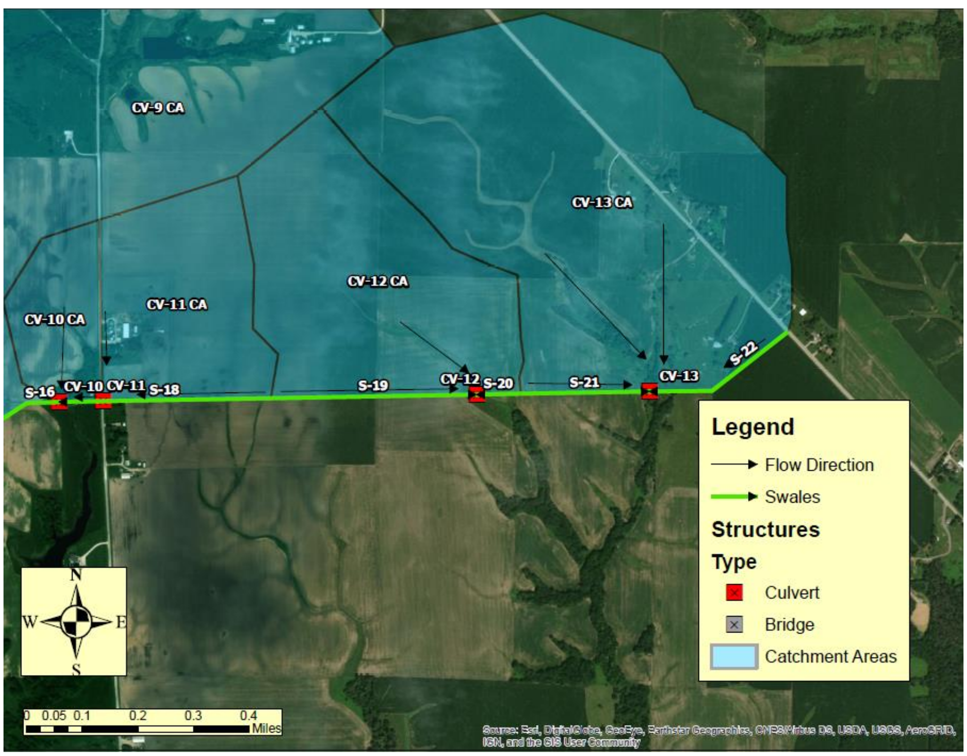


Figure H04: Drainage Map for Culverts 10 - 13

Table H-0-1: Culvert 2 Discharge Data

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (fps)	Tailwater Velocity (fps)
622	585.31	738.7	21.22~	16.85	7-M2c	6	5.53	5.53	3.08	21.48	11.7
632.6	586.39	738.77	21.29~	16.9	7-M2c	6	5.18	5.18	3.11	22.6	11.75
643.2	587.35	738.83	21.35~	16.94	7-M2c	6	5.32	5.32	3.13	22.15	11.8
653.8	588.22	738.88	21.4	16.98	6-FFc	6	6	6	3.16	20.8	11.85
664.4	589.03	738.93	21.45	17.01	6-FFc	6	6	6	3.18	20.83	11.9
675	589.78	738.98	21.5	17.05	6-FFc	6	6	6	3.21	20.86	11.95
685.6	590.46	739.02	21.54	17.08	6-FFc	6	6	6	3.23	20.88	12
696.2	591.13	739.06	21.58	17.11	6-FFc	6	6	6	3.25	20.91	12.05
706.8	591.74	739.1	21.62	17.13	6-FFc	6	6	6	3.28	20.93	12.1
717.4	592.33	739.14	21.66	17.16	6-FFc	6	6	6	3.3	20.95	12.14
728	592.9	739.18	21.7	17.18	6-FFc	6	6	6	3.32	20.97	12.19

Table H-0-2: Culvert 6 Discharge Data

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (fps)	Tailwater Velocity (fps)
62	62	751.17	10.42	13.57	7-M2c	2.5	2.28	2.28	1.73	13.2	3.04
63.2	63.2	751.6	10.78	14	7-M2c	2.5	2.23	2.23	1.74	13.69	3.06
64.4	64.4	752.05	11.15	14.45	7-M2c	2.5	2.11	2.11	1.76	14.54	3.07
65.6	65.6	752.47	11.52	14.87	7-M2c	2.5	2.42	2.42	1.77	13.49	3.09
66.8	66.8	752.96	11.9	15.36	6-FFc	2.5	2.5	2.5	1.79	13.61	3.1
68	68	753.43	12.29	15.83	6-FFc	2.5	2.5	2.5	1.8	13.85	3.12
69.2	69.2	753.91	12.68	16.31	6-FFc	2.5	2.5	2.5	1.82	14.1	3.13
70.4	70.4	754.4	13.08	16.8	6-FFc	2.5	2.5	2.5	1.83	14.34	3.15
71.6	71.6	754.89	13.48	17.29	6-FFc	2.5	2.5	2.5	1.84	14.59	3.16
72.8	72.8	755.4	13.9	17.8	6-FFc	2.5	2.5	2.5	1.86	14.83	3.17
74	74	755.91	14.32	18.31	6-FFc	2.5	2.5	2.5	1.87	15.08	3.19

Table H-0-3: Culvert 9 Discharge Data

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (fps)	Tailwater Velocity (fps)
0	0	657.39	0	0	0-NF	0	0	0	0	0	0
156	156	660.52	3.13	0.27	1-S2n	1.2	1.84	1.25	1.59	11.36	7.67
312	312	662.34	4.95	1.6	1-S2n	1.89	2.92	2.05	2.27	13.81	9.3
468	468	663.84	6.45	2.94	1-S2n	2.49	3.83	2.77	2.77	15.36	10.37
624	624	665.22	7.83	4.34	1-S2n	3.04	4.64	3.43	3.18	16.52	11.19
780	780	666.62	9.23	5.85	5-S2n	3.55	5.38	4.06	3.53	17.46	11.87
936	936	668.11	10.72	8.44	5-S2n	4.05	6.08	4.66	3.85	18.27	12.45
1092	1092	669.79	12.4	9.88	5-S2n	4.53	6.74	5.23	4.13	18.98	12.96
1320	1320	672.65	15.26	12.26	5-S2n	5.21	7.65	6.03	4.51	19.89	13.61
1404	1404	673.85	16.46	13.22	5-S2n	5.46	7.97	6.32	4.64	20.19	13.82
1560	1560	676.28	18.89	14.85	5-S2n	5.91	8	6.81	4.86	20.82	14.2

Table H-0-4: Culvert 12 Discharge Data

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (fps)	Tailwater Velocity (fps)
0	0	710.52	0	0	0-NF	0	0	0	0	0	0
8.3	8.3	711.17	0.65	0.0*	1-S2n	0.2	0.39	0.21	0.35	6.64	2.62
16.6	16.6	711.55	1.03	0.0*	1-S2n	0.32	0.62	0.32	0.52	8.72	3.32
24.9	24.9	711.87	1.35	0.0*	1-S2n	0.41	0.81	0.42	0.66	9.77	3.79
33.2	33.2	712.16	1.64	0.0*	1-S2n	0.5	0.98	0.53	0.77	10.47	4.15
41.5	41.5	712.42	1.9	0.0*	1-S2n	0.58	1.14	0.6	0.88	11.45	4.45
49.8	49.8	712.66	2.14	0.0*	1-S2n	0.66	1.29	0.69	0.97	12.06	4.71
58.1	58.1	712.89	2.37	0.0*	1-S2n	0.73	1.43	0.77	1.05	12.5	4.93
70	70	713.21	2.69	0.0*	1-S2n	0.83	1.62	0.89	1.17	13.07	5.22
74.7	74.7	713.34	2.82	0.0*	1-S2n	0.87	1.69	0.94	1.21	13.31	5.32
83	83	713.55	3.03	0.0*	5-S2n	0.94	1.81	1.01	1.28	13.67	5.48

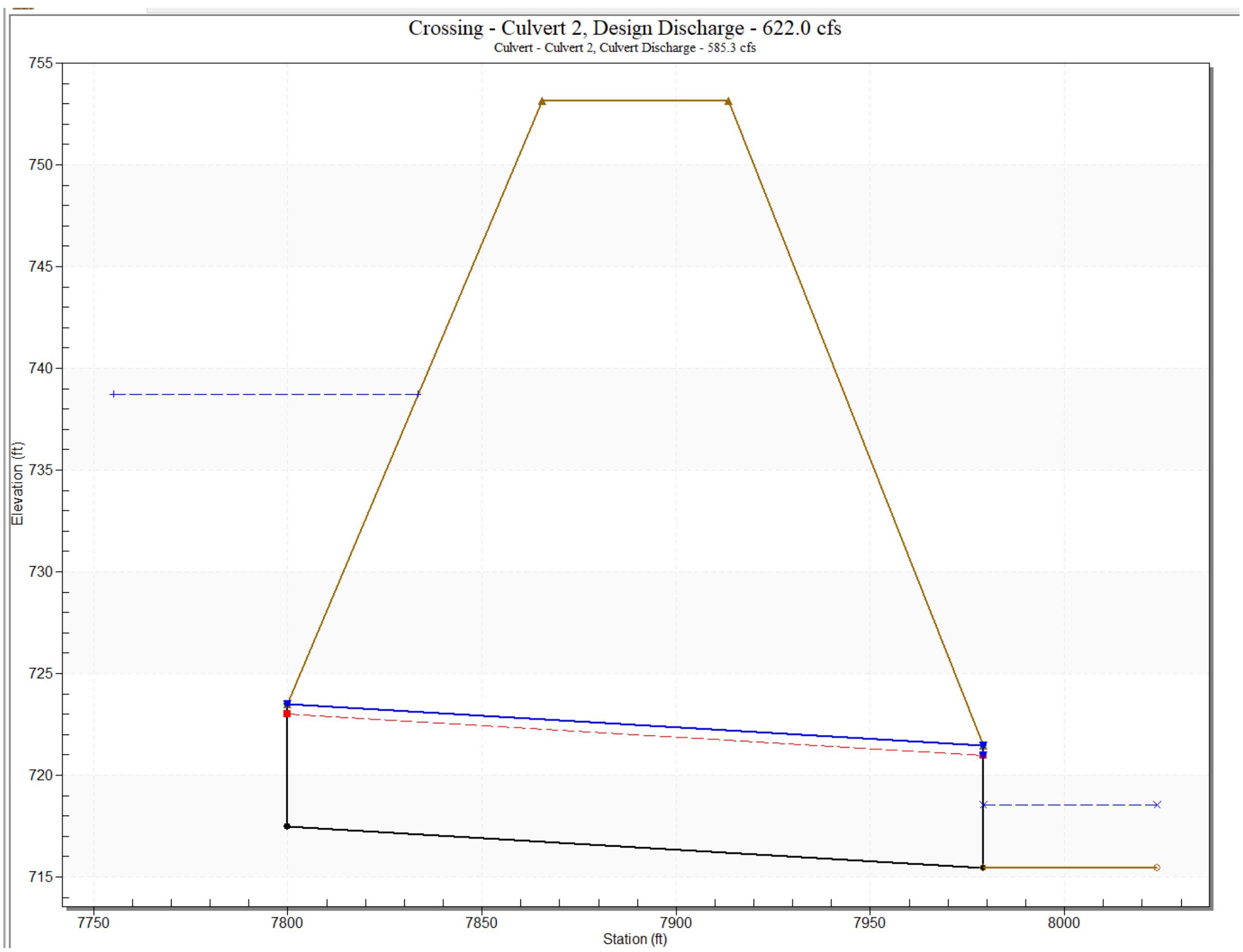


Figure H-0-1: Culvert 2 Profile

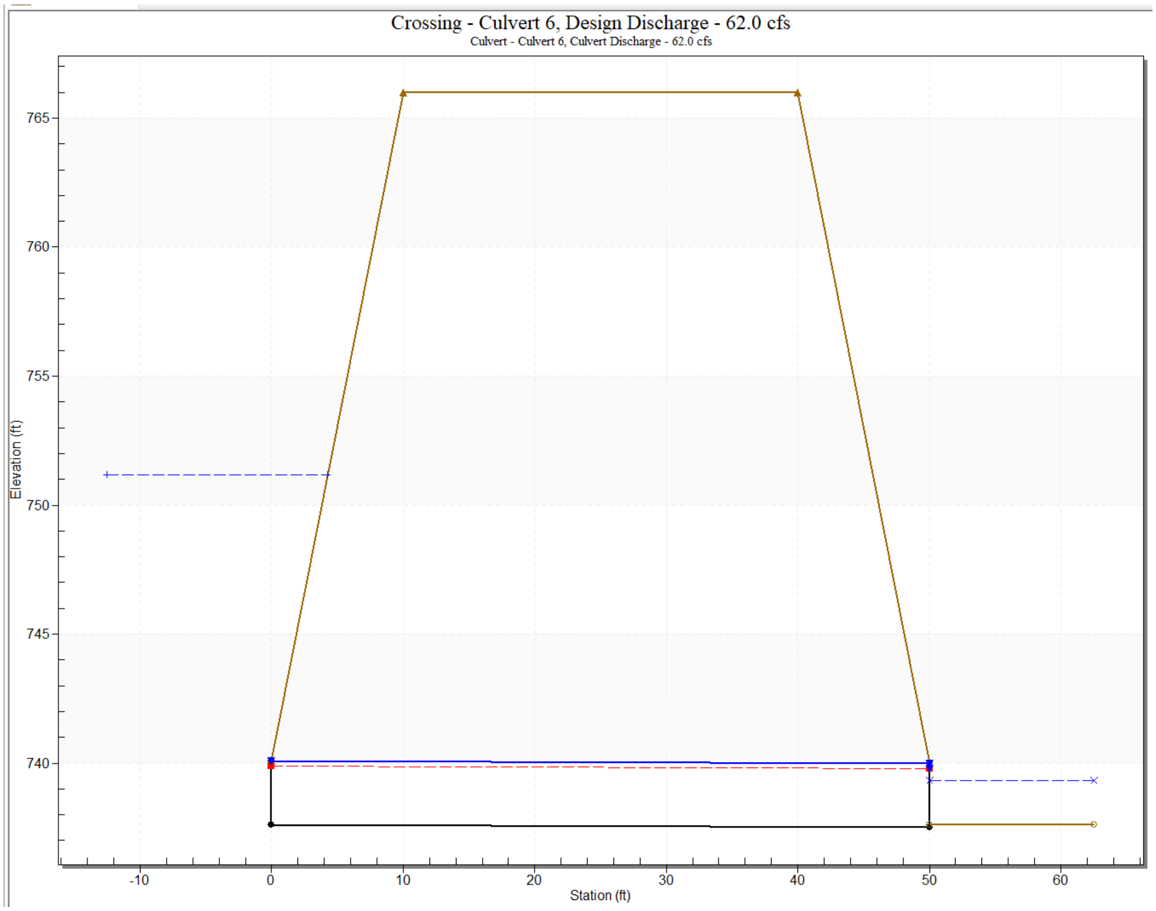


Figure H-0-2: Culvert 6 Profile

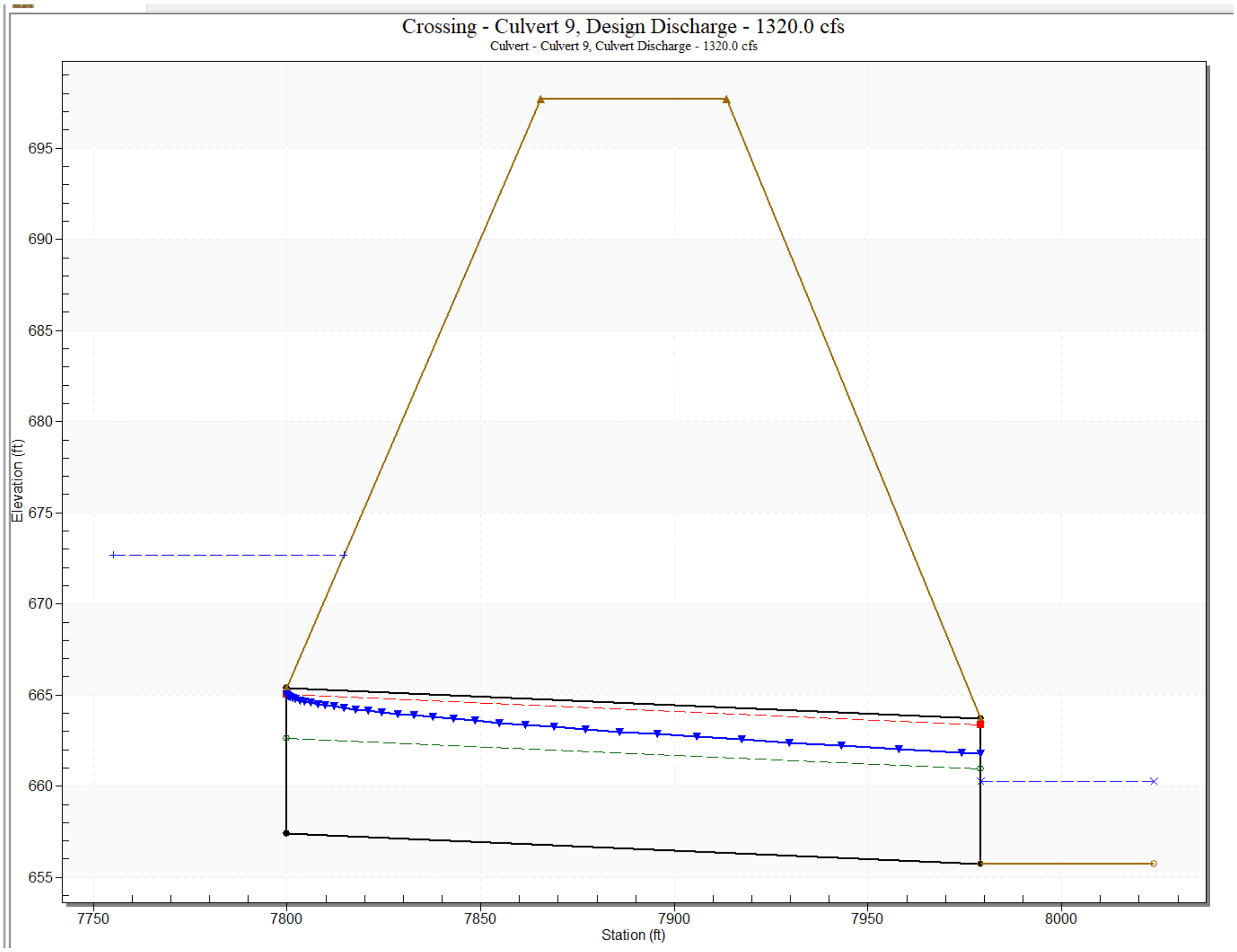


Figure H-0-3: Culvert 9 Profile

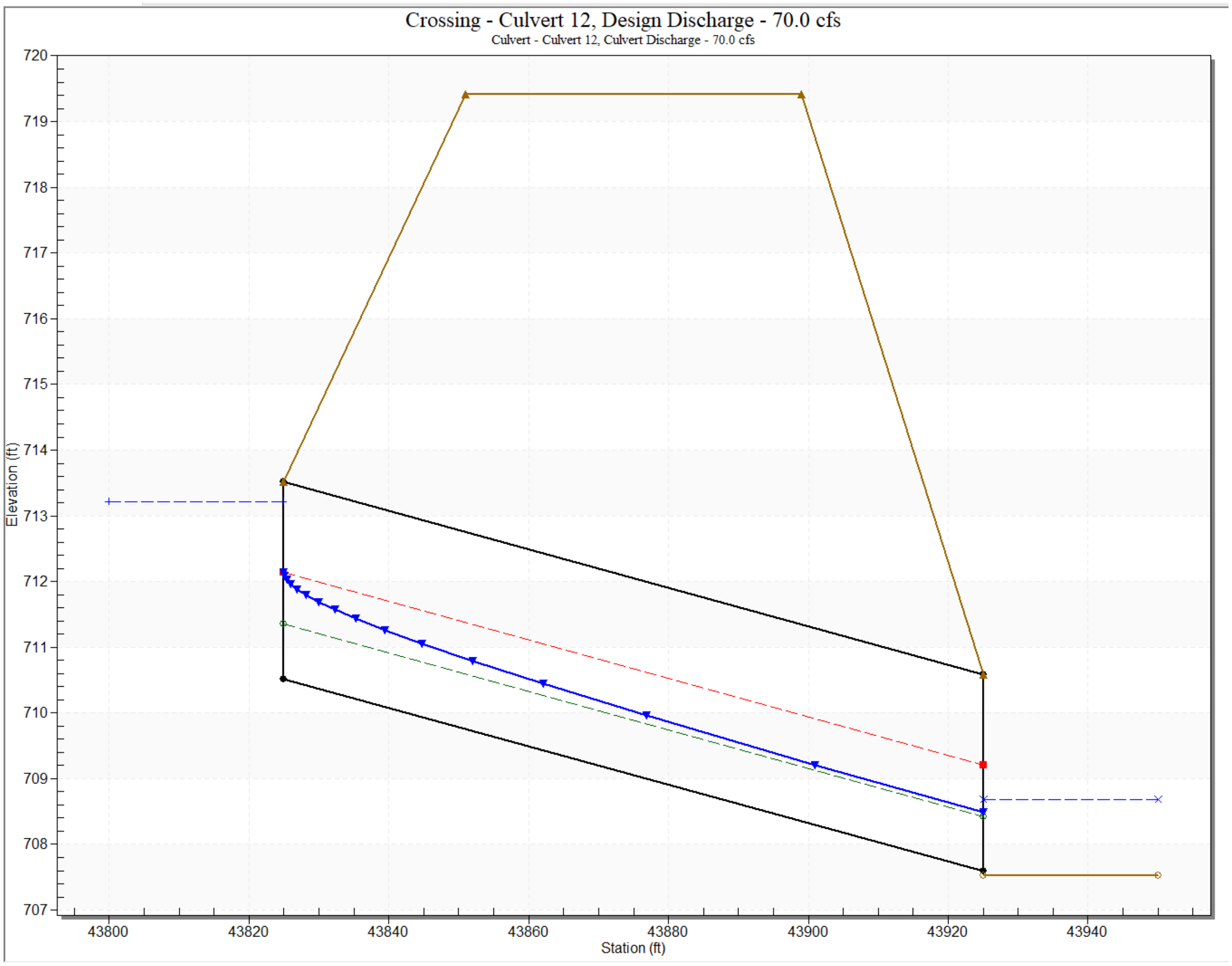


Figure H-0-4: Culvert 12 Profile

Table H-0-5: Plasticity DATA from FAO

Examples
Plasticity of various silt/clay soils

Category	Soil	PI (percentage)	Degree of plasticity
I	Sand or silt • traces of clay • little clay	0-1	Non-plastic
		1-5	Slight plasticity
		5-10	Low plasticity
II	Clay loam	10-20	Medium plasticity
III	Silty clay Clay	20-35	High plasticity
		>35	Very high plasticity

Figure 0-5

Table H-0-6: Porosity Data (Tarboton)

Table 1. Clapp and Hornberger (1978) parameters for equation (27) based on analysis of 1845 soils. Values in parentheses are standard deviations.

Soil Texture	Porosity n	K_{sat} (cm/hr)	$\lvert\psi_a\rvert$ (cm)	b
Sand	0.395 (0.056)	63.36	12.1 (14.3)	4.05 (1.78)
Loamy sand	0.410 (0.068)	56.16	9 (12.4)	4.38 (1.47)
Sandy loam	0.435 (0.086)	12.49	21.8 (31.0)	4.9 (1.75)
Silt loam	0.485 (0.059)	2.59	78.6 (51.2)	5.3 (1.96)
Loam	0.451 (0.078)	2.50	47.8 (51.2)	5.39 (1.87)
Sandy clay loam	0.420 (0.059)	2.27	29.9 (37.8)	7.12 (2.43)
Silty clay loam	0.477 (0.057)	0.612	35.6 (37.8)	7.75 (2.77)
Clay loam	0.476 (0.053)	0.882	63 (51.0)	8.52 (3.44)
Sandy clay	0.426 (0.057)	0.781	15.3 (17.3)	10.4 (1.64)
Silty clay	0.492 (0.064)	0.371	49 (62.0)	10.4 (4.45)
Clay	0.482 (0.050)	0.461	40.5 (39.7)	11.4 (3.7)

(Tarboton)

Table 5.1. Grass Roughness Coefficient, C_n.					
Stem Height m (ft)	Excellent	Very Good	Good	Fair	Poor
0.075 (0.25)	0.168	0.157	0.142	0.122	0.111
0.150 (0.50)	0.243	0.227	0.205	0.177	0.159
0.225 (0.75)	0.301	0.281	0.254	0.219	0.197

Table 5.2. Coefficients for Permissible Soil Shear Stress.								
ASTM Soil Classification[1]	Applicable Range	C_1	C_2	C_3	C_4	C_5	C_6 (SI)	C_6 (English)
GM	$10 \le PI \le 20$	1.07	14.3	47.7	1.42	-0.61	4.8×10^{-3}	10^{-4}
	$20 \le PI$			0.076	1.42	-0.61	48.	1.0
GC	$10 \le PI \le 20$	0.0477	2.86	42.9	1.42	-0.61	4.8×10^{-2}	10^{-3}
	$20 \le PI$			0.119	1.42	-0.61	48.	1.0
SM	$10 \le PI \le 20$	1.07	7.15	11.9	1.42	-0.61	4.8×10^{-3}	10^{-4}
	$20 \le PI$			0.058	1.42	-0.61	48.	1.0
SC	$10 \le PI \le 20$	1.07	14.3	47.7	1.42	-0.61	4.8×10^{-3}	10^{-4}
	$20 \le PI$			0.076	1.42	-0.61	48.	1.0
ML	$10 \le PI \le 20$	1.07	7.15	11.9	1.48	-0.57	4.8×10^{-3}	10^{-4}
	$20 \le PI$			0.058	1.48	-0.57	48.	1.0
CL	$10 \le PI \le 20$	1.07	14.3	47.7	1.48	-0.57	4.8×10^{-3}	10^{-4}
	$20 \le PI$			0.076	1.48	-0.57	48.	1.0
MH	$10 \le PI \le 20$	0.0477	14.3	10.7	1.38	-0.373	4.8×10^{-2}	10^{-3}
	$20 \le PI$			0.058	1.38	-0.373	48.	1.0
CH	$20 \le PI$			0.097	1.38	-0.373	48.	1.0

[1]NOTE: Typical Names

GM	Silty gravels, gravel-sand silt mixtures
GC	Clayey gravels, gravel-sand-clay mixtures
SM	Silty sands, sand-silt mixtures
SC	Clayey sands, sand-clay mixtures
ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
CH	Inorganic clays of high plasticity, fat clays

Table 5.3. Cover Factor Values for Uniform Stands of Grass.					
Growth Form	Cover Factor, C_f				
	Excellent	Very Good	Good	Fair	Poor
Sod	0.98	0.95	0.90	0.84	0.75
Bunch	0.55	0.53	0.50	0.47	0.41
Mixed	0.82	0.79	0.75	0.70	0.62

Table H-0-7 Other Swale Data Variety of Storms

Design Parameters With Lining		**Other Swale 50 year storm**			
Side Slope Inside (H:V)	6				
Side Slope Outside (H:V)	3	Flow Depth (ft)	3.17	New Manning's n	0.069
Bottom Width (ft)	4	Discharge Calc (cfs)	128	New Mean Shear (psf)	0.543
Area (ft^2)	58.01	Design Discharge (cfs)	128	New Max Shear (psf)	0.990
Wetted Perimeter (ft)	33.34	Squared Error	6.28475E-15		
Hydraulic Radius (ft)	1.74				
Hydraulic Mean Depth (ft)	1.63				
Grass Roughness Coefficient	0.254	Permissible Shear, Soil (psf)	0.0630	Velocity (fps)	2.21
Cover Factor	0.75	Max Permissible Shear (psf)	4.70	Mean Shear (psf)	0.543
Manning's n soil	0.016			Max Shear (psf)	0.990
C1	1.07				
C2	14.3	**Soil Properties**		**Max Shear < Permissible Shear**	
C3	0.076	Porosity	0.42		
C4	1.42	Void Ratio	0.72		
C5	-0.61	Plasticity Index	19		
C6	0.0001	Manning's n	0.04		

Design Parameters With Lining		**Other Swale 100 year storm**			
Side Slope Inside (H:V)	6				
Side Slope Outside (H:V)	3	Flow Depth (ft)	3.39	New Manning's n	0.067
Bottom Width (ft)	4	Discharge Calc (cfs)	153	New Mean Shear (psf)	0.576
Area (ft^2)	65.14	Design Discharge (cfs)	153	New Max Shear (psf)	1.056
Wetted Perimeter (ft)	35.30	Squared Error	2.99157E-15		
Hydraulic Radius (ft)	1.85				
Hydraulic Mean Depth (ft)	1.74				
Grass Roughness Coefficient	0.254	Permissible Shear, Soil (psf)	0.0630	Velocity (fps)	2.35
Cover Factor	0.75	Max Permissible Shear (psf)	4.48	Mean Shear (psf)	0.576
Manning's n soil	0.016			Max Shear (psf)	1.056
C1	1.07				
C2	14.3	**Soil Properties**		**Max Shear < Permissible Shear**	
C3	0.076	Porosity	0.42		
C4	1.42	Void Ratio	0.72		
C5	-0.61	Plasticity Index	19		
C6	0.0001	Manning's n	0.04		

Design Parameters With Lining		**Other Swale 500 year storm**			
Side Slope Inside (H:V)	6				
Side Slope Outside (H:V)	3	Flow Depth (ft)	3.85	New Manning's n	0.064
Bottom Width (ft)	4	Discharge Calc (cfs)	218	New Mean Shear (psf)	0.647
Area (ft^2)	82.00	Design Discharge (cfs)	218	New Max Shear (psf)	1.200
Wetted Perimeter (ft)	39.57	Squared Error	1.0716E-17		
Hydraulic Radius (ft)	2.07				
Hydraulic Mean Depth (ft)	1.97				
Grass Roughness Coefficient	0.254	Permissible Shear, Soil (psf)	0.0630	Velocity (fps)	2.66
Cover Factor	0.75	Max Permissible Shear (psf)	4.08	Mean Shear (psf)	0.647
Manning's n soil	0.016			Max Shear (psf)	1.200
C1	1.07				
C2	14.3	**Soil Properties**			
				Max Shear < Permissible Shear	
C3	0.076	Porosity	0.42		
C4	1.42	Void Ratio	0.72		
C5	-0.61	Plasticity Index	19		
C6	0.0001	Manning's n	0.04		

Table H-8: Longest Swale Data Variety of Storms

Design Parameters With Lining		**Longest Swale 50 year storm**			
Side Slope Inside (H:V)	6				
Side Slope Outside (H:V)	3	Flow Depth (ft)	2.43	New Manning's n	0.076
Bottom Width (ft)	4	Discharge Calc (cfs)	62	New Mean Shear (psf)	0.428
Area (ft^2)	36.25	Design Discharge (cfs)	62	New Max Shear (psf)	0.758
Wetted Perimeter (ft)	26.45	Squared Error	3.53024E-09		
Hydraulic Radius (ft)	1.37				
Hydraulic Mean Depth (ft)	1.26				
Grass Roughness Coefficient	0.254	Permissible Shear, Soil (psf)	0.0630	Velocity (fps)	1.71
Cover Factor	0.75	Max Permissible Shear (psf)	5.68	Mean Shear (psf)	0.428
Manning's n soil	0.016			Max Shear (psf)	0.758
C1	1.07				
C2	14.3	**Soil Properties**			
				Max Shear < Permissible Shear	
C3	0.076	Porosity	0.42		
C4	1.42	Void Ratio	0.72		
C5	-0.61	Plasticity Index	19		
C6	0.0001	Manning's n	0.04		

Design Parameters With Lining		**Longest Swale 100 year storm**			
Side Slope Inside (H:V)	6				
Side Slope Outside (H:V)	3	Flow Depth (ft)	2.59	New Manning's n	0.074
Bottom Width (ft)	4	Discharge Calc (cfs)	74	New Mean Shear (psf)	0.453
Area (ft^2)	40.65	Design Discharge (cfs)	74	New Max Shear (psf)	0.809
Wetted Perimeter (ft)	27.98	Squared Error	3.01678E-16		
Hydraulic Radius (ft)	1.45				
Hydraulic Mean Depth (ft)	1.34				
Grass Roughness Coefficient	0.254	Permissible Shear, Soil (psf)	0.0630	Velocity (fps)	1.82
Cover Factor	0.75	Max Permissible Shear (psf)	5.42	Mean Shear (psf)	0.453
Manning's n soil	0.016			Max Shear (psf)	0.809
C1	1.07				
C2	14.3	**Soil Properties**			
C3	0.076	Porosity	0.42	**Max Shear < Permissible Shear**	
C4	1.42	Void Ratio	0.72		
C5	-0.61	Plasticity Index	19		
C6	0.0001	Manning's n	0.04		

Design Parameters With Lining		**Longest Swale 500 year storm**			
Side Slope Inside (H:V)	6				
Side Slope Outside (H:V)	3	Flow Depth (ft)	2.96	New Manning's n	0.071
Bottom Width (ft)	4	Discharge Calc (cfs)	106	New Mean Shear (psf)	0.510
Area (ft^2)	51.32	Design Discharge (cfs)	106	New Max Shear (psf)	0.924
Wetted Perimeter (ft)	31.38	Squared Error	1.04454E-14		
Hydraulic Radius (ft)	1.64				
Hydraulic Mean Depth (ft)	1.52				
Grass Roughness Coefficient	0.254	Permissible Shear, Soil (psf)	0.0630	Velocity (fps)	2.07
Cover Factor	0.75	Max Permissible Shear (psf)	4.93	Mean Shear (psf)	0.510
Manning's n soil	0.016			Max Shear (psf)	0.924
C1	1.07				
C2	14.3	**Soil Properties**			
C3	0.076	Porosity	0.42	**Max Shear < Permissible Shear**	
C4	1.42	Void Ratio	0.72		
C5	-0.61	Plasticity Index	19		
C6	0.0001	Manning's n	0.04		

Swale Example Calculations: Longest Swale 500 year storm
Step 1: Solver

$Q=A*V$; $V=1.49/n*R^{2/3}*S^{1/2}$
$n=C_n*(\text{mean Shear})^{-0.4}$
mean Shear = S*R*62.4

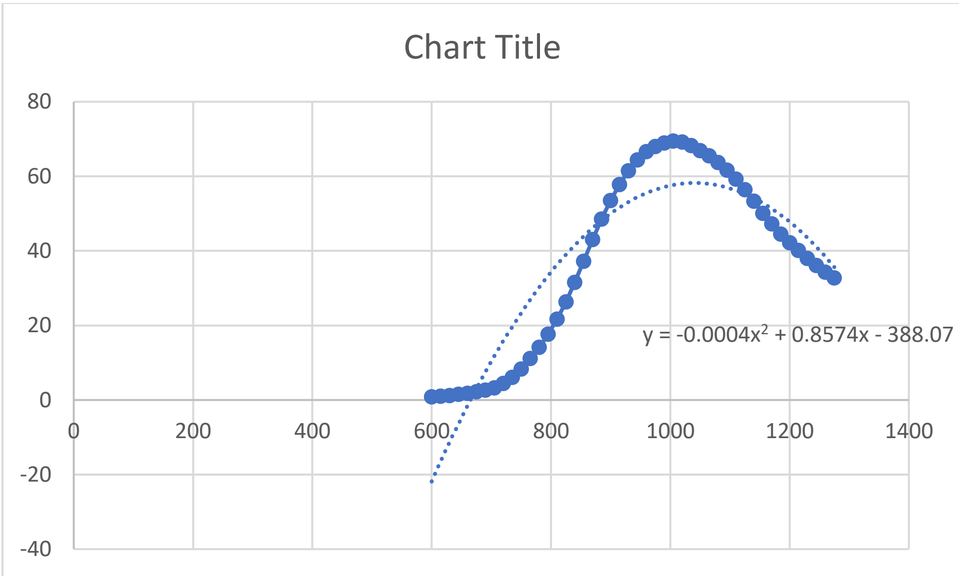
run the solver to minimized squared difference of design Q and manning Q by changing depth

Step 2: Find Soil Shear, Max Shear, and Permissible Shear
Void ratio = Porosity/(1-Porosity)
Soil Permissible Shear = $(C_1*PI^2+C_2*PI+C_3)*(C_4+C_5(\text{void ratio}))2*C6$
Max Permissible Shear = (Soil Permissible Shear)/(1-CoverFactor)*(n/manning soil n)^2
Max Shear = 62.4*Depth*S

Finally check that Max Shear is Less Than Permissible Shear

Detention Basin Calculations:

Graph Equation:



f(x) = -.0004x^2 + .8574x – 388.07

Volume = 32,849.18
Constrain Volume = 684 cfh
PI = 19 Table H-8
Type of Soil: Clay Loam Medium Plasticity
Infiltration Rate: .885 cm/hr = .35 in/hr = .029 ft/hr

$$Infil\ Rate * Area = Constraint\ Volume$$
$$\frac{684.35}{.029} = 23463.57\ ft^2$$

NRCS Post runoff = 2.37 in
CN = 82
S = 2.19
Ia = .439
P = 2.99
Q = $(P-Ia)^2/(P+.8*S)$
Q = 1.37
Pre runoff =1.37 in
Volume Not Released = (Post Runoff – Pre Runoff) * Area
Volume Not Released = 1,955 ft^2

Appendix I – Response to Reviewers

Table H-0-1: Response to Deliverable 1 Comments

Section	Page	Comments	Response
Introduction	2	Introduce figures before presentation	Fixed Format

	2	**Subjective diction and lack of quantitative descriptions**	**Provided route parameters and topographical data**
Project Objectives	**3**	**Address other objectives such as project economics and impact to local areas**	**Included economic factors and other factors into objectives**
Design Team Members	**7**	**NA**	**NA**
Alignment Alternatives	**5** **6-7** **8**	**-Alignment color is hard to see** **-Add clarification to ranking system** **-Add labels to alignment map**	**-Changed alignment color in Figure 4-1** **-Explained ranking system** **-Added labels to Figure 4-2**
Autonomous Vehicles Overview	**13**	**NA**	**NA**
Preliminary Cross-Section Elements	**15**	**Different Criteria than BLRS**	**-Removed median after Civil 3D work** **-Updated Cross-Section diagrams**
Stream Crossings	**21**	**Add labels, indicate potential unlisted streams and culverts**	**-Labels updated in Figure 7-1 and Table 7-1 updated** **-Included culverts on map**
Summary	**27**	**Provide quantitative data for final alternative chosen**	**Agreed, provided geometric and pavement design factors**
References	**28**	**Include references for all data sources and manuals**	**Included missing references**
Appendices	**29**	**Include scale on soil map**	**Updated soil map**