North East Corridor Project

Muscatine, IA

Designed By

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Executive Summary

Hawkeye Engineering analyzed an existing land use plan that was proposed by Snyder & Associates against the actual development that has taken place in the Northeast Corridor (Figure 1-1) of the City of Muscatine. Commercial development has increased considerably along US-61, and the city intends to facilitate continued development with the implementation of Hawkeye Engineering's design. As a result of the development analysis, an arterial road connecting IA-38 to US-61 and an arterial road connecting US-61 to 180th St. served as the focal point to encourage development in the NE Corridor. An additional route that would have extended University Dr. to the north to connect the E-W arterial connecting IA-38 to US-61 is no longer a plausible option for development due to the construction of a hotel in its path. Following route selection, five new intersections in the NE Corridor were analyzed using HCS2010 Warrants to determine if traffic signals were necessary at any of the intersections. It was determined that four of the five intersections did not require any traffic signals due the lack of traffic flow at full build out. The E-W arterial/US 61 intersection required an actuated traffic signal to allow safe through and left turning movements. The E-W arterial will have a 90 ft. right-of-way with a shared left turn lane along the length due to the large amount of commercial zoning in the area. The N-S arterial will have a 100 ft. right of way with a raised median due to the surrounding residential population. This allowed safe and easy access to the neighborhoods on either side of the road.

Other design considerations for the development plan included a sanitary sewer demand analysis, 4 Mad Creek crossings, and a stormwater management plan for the modifications to the watershed resulting from the two new roads. Following a land use analysis to determine the amount of usable area that is included in each zoning type, the necessary size for a sanitary sewer trunk line was estimated to be 18 in. A 15 in. sanitary sewer extension is already in place that would be satisfactory to handle the majority of the flows that are expected to be generated after development.

Hawkeye Engineering considered the design of a slab-girder bridge according to the AASHTO LRFD design strip method. After evaluating the design strength of PCC slab using the ACI building code and computing the girder strength using the AISC steel manual, the bridge will consist of 7 W21x122 steel girders equally spaced a distance of 7.5 ft. The overall span of the ridge will be 130 ft. covered with a central span of 50 ft. and 40 ft. spans on either end of the bridge.

Hawkeye Engineering also considered the use of a culvert. The design recommends three culverts, one culvert along the E-W arterial with a 10 ft. pipe diameter, the other two culverts will be along the N-S arterial, one with an 8 ft. diameter and the other with a 15 ft. diameter. Along with the proposed road, a curb inlet stormwater system was selected as the most desirable option to handle the excess runoff due to the newly proposed road.

The total cost estimate for the design of the road, storm sewer, sanitary sewer, culverts, and bridge is approximately $8.6 million.

I. Introduction

Hawkeye Engineering analyzed an existing land use plan and proposed improvements to the design for a 1400 acre area located in the northeast part of Muscatine, IA (Figure 1-1). Elements of the design included two arterial roads connecting Highways 38 & 61 and Highway 61 to 180th Street, a stormwater management plan for the roadways, several Mad Creek crossings, additional utility system considerations, and a general land use plan. Any additional considerations regarding stormwater management were not part of the scope of this project. Design of several detention basins were performed as a part of the Mad Creek Regional Water Detention Project.

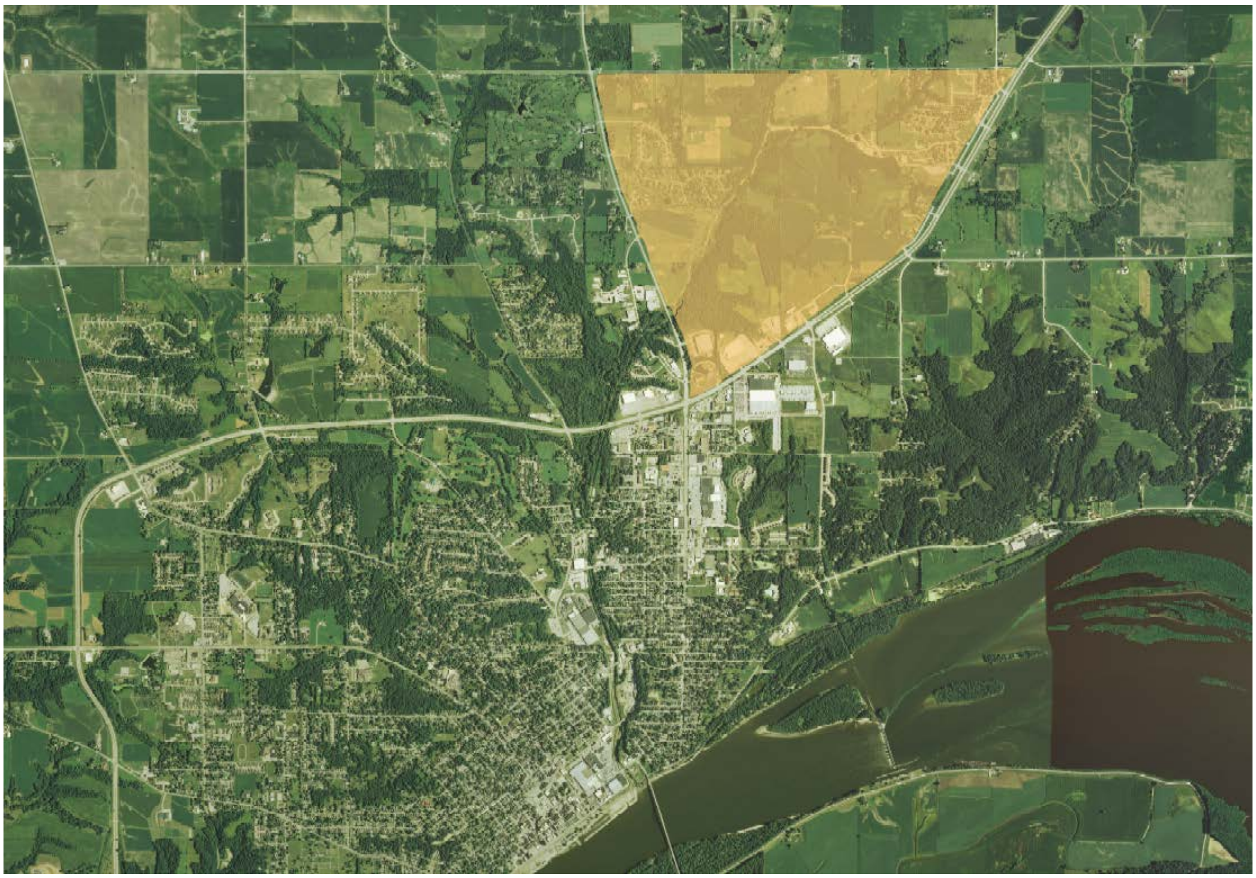


Figure 1-1: Northeast Corridor Project Area

II. Problem Statement

2-1 Design Objectives

Considerations were taken for future growth in the Northeast Corridor of the City of Muscatine, Hawkeye Engineering evaluated the current conditions in the Northeast Corridor along U.S. Highway 61 and continuing north to 180th St. The intent to connect Park Ave. to New Era Rd was investigated by utilizing current land ownership and existing elevation data to determine the feasibility of the route proposed in Figure 2-1(taken from the Comprehensive Plan of the City of Muscatine). Two intersections will be designed where the proposed E-W arterial road intersects Highways 38 and 61. Along this new road, several crossings over Mad Creek were evaluated. The option of either using a culvert or a bridge were considered. A cost/benefit analysis was performed for each option to aid in the selection process. Along with an east/west connector, a north/south route to connect University Dr. to 180th St. was also designed. The two proposed arterial roads resulted in a new intersection near the center of the project location, which was analyzed as a part of the traffic analysis. A traffic impact analysis was performed to determine the best option for the location of the arterial roads (Figure 2-1).

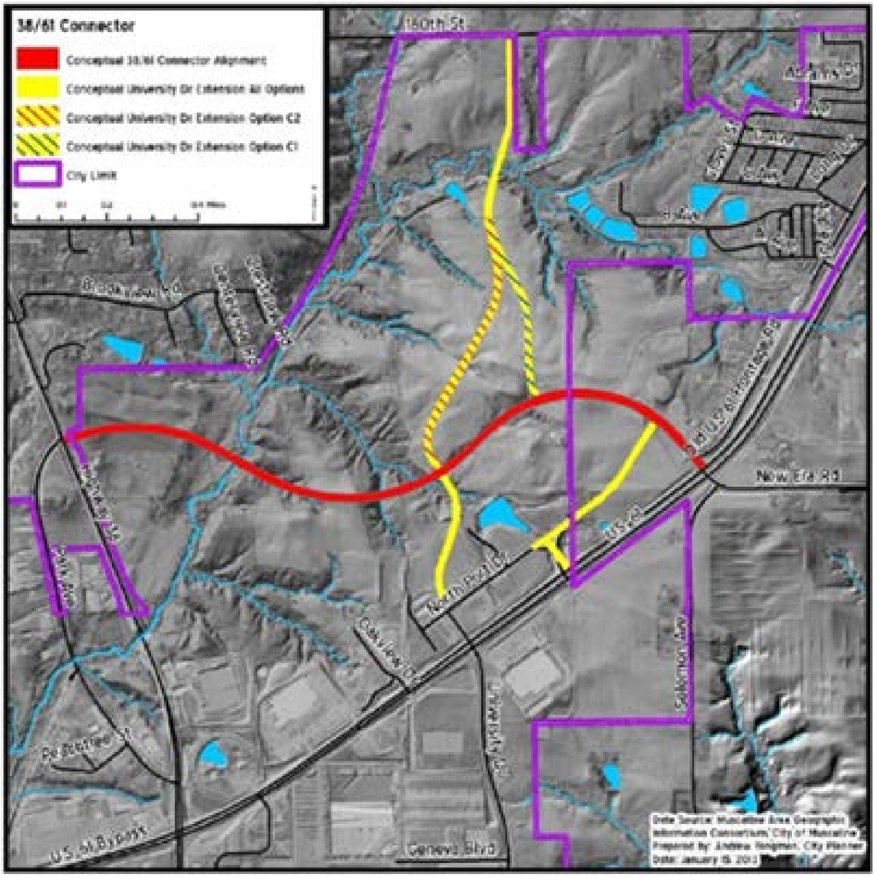


Figure 2-1: 38/61 Connector Study for the City of Muscatine

Along with transportation considerations, Hawkeye Engineering investigated utility extensions to serve the future development that will take place in the northeast corridor. When considering land use and utility modifications, additional runoff will not be created when converting farm fields and other pervious surfaces to impervious surfaces. The current state of Mad Creek will not be altered by the design that is proposed in the later sections of this report.

2-2 Approaches

To begin considerations for future growth in the northeast corridor of Muscatine, Hawkeye Engineers analyzed several different route possibilities for the arterial roads that serve as the center of the design for this area. Existing elevation data obtained from the Iowa DNR GIS Library were utilized to select routes that minimized the grading necessary. Along with minimizing the amount of earthwork necessary, the number of creek crossings was minimized during the route selection process. The overall objective was still to provide access to commercial and residential development that will take place in this area of town in the future.

Using the most desirable route for the arterial roads, an area analysis was performed to be used for trip generation and sanitary sewer demand calculations. These area calculations were performed starting with the parcel lines that were obtained from the City of Muscatine. Any area that was deemed to be unusable was then subtracted out of the total project area (Figure 2-2). Unusable area included an assumed 100' right of way for the arterial roads, the outline of Mad Creek with a 10' buffer on either side, and any area with a slope of 20% or greater. Other area that was currently outside of the corporate limits of the City of Muscatine was not considered for development. Along with the areas that lie outside of the corporate limits, any already developed land was differentiated from proposed development and was not factored into the demand for utility systems. The results of the usable area analysis can be found in Table 3-10.

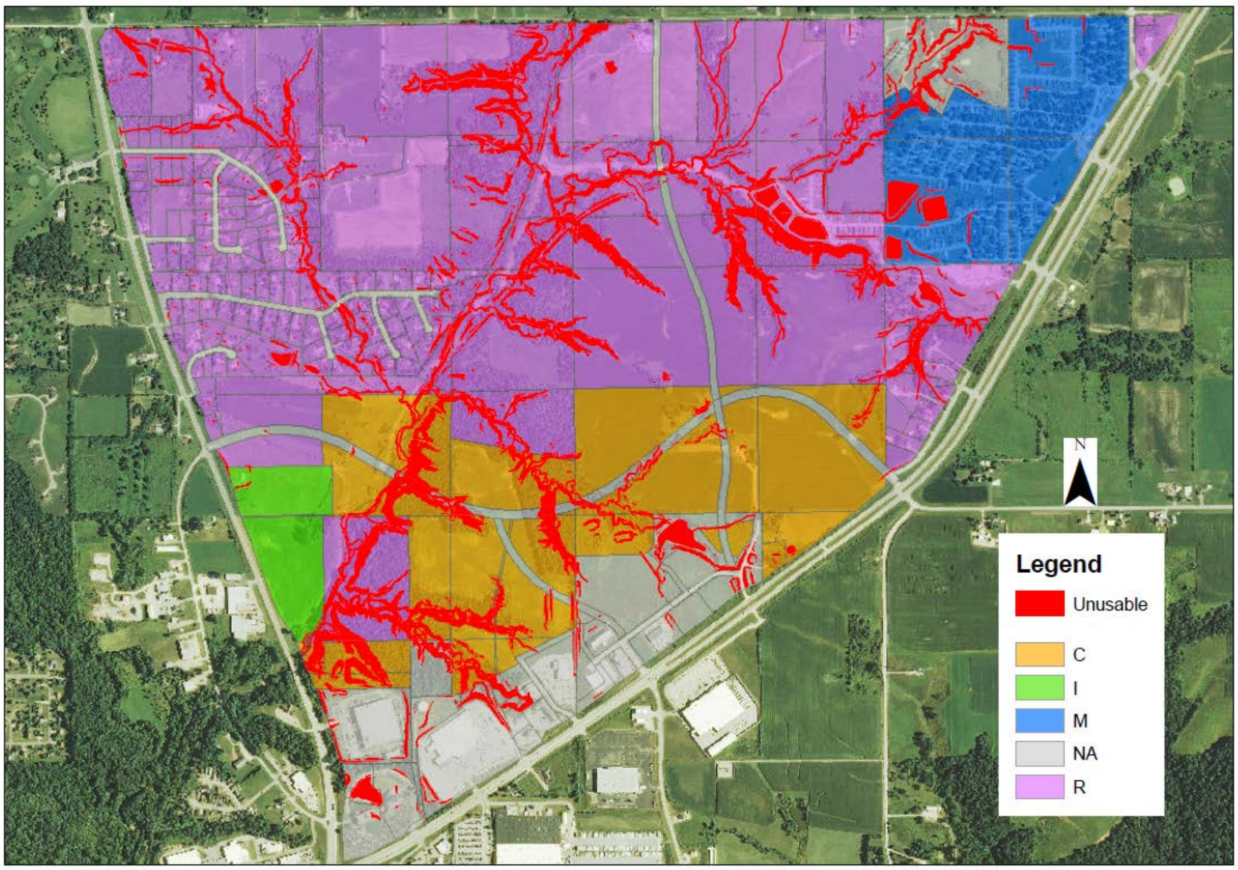


Figure 2-2: Land use plan for the NE Corridor.
(R = residential, C = commercial, I = Industrial, M = mobile home park, NA = any land that was not considered)

Traffic Flow Rate Calculations

The traffic flow rate was calculated by taking into considerations a number of factors as well as making some assumptions. The zoning map in Figure 2-2 along with the area values in Table A-1 were used to estimate the amount of building area that would be included in the future development of Northeast Corridor. The office building land use area was based on the assumption that it accounted for 60% of the total commercial area. This assumption was made by evaluating the City of Muscatine's zoning map that indicated a majority of the area is intended to be zoned for light commercial as well as office buildings. After reviewing the City of Muscatine's comprehensive plan, the maximum floor-to-area ratio for all uses whether commercial or residential was determined to be 4:1. A 4:1 floor-to-area ratio is very large compared to the existing buildings in the City of Muscatine. After some trial and error calculations, a floor-to-area ratio of 2:1 and an open space ratio of 0.5 were determined to be the most reasonable for the Northeast Corridor.

Combining the open space ratio and the floor-to-area ratio with the number of trips generated per zone type. These numbers were obtained from the ITE Trip Generation Manual, and the calculations are summarized in Table A-1. It was assumed that the development in the Northeast Corridor would be spread out over the course of 30 years. In order to determine the rate of growth of the area, the Muscatine County Census data was consulted which showed a relatively steady growth rate of approximately 3%, as shown in Figure 2-3. The steady growth rate allowed the equal division of the 30 year trip generation numbers into thirds to get both 10 and 20 year trip generation numbers.

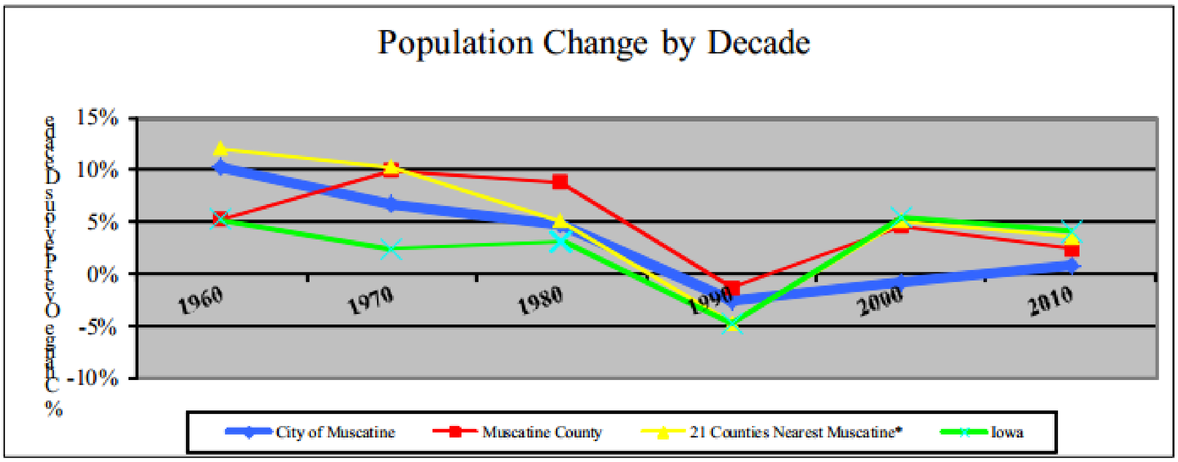


Figure 2-3: Population change courtesy of the City of Muscatine Comprehensive Plan

Table 2-1: Trips Generated for each zone type in 10 year increments until full 30 year build out

	10 Year	20 Year	30 Year
Mobile Home	280	280	280
Residential	445	890	1334
Industrial	91	181	272
Commercial/ Office	77	153	230

The zoning map shown in Figure 2-2 was re-analyzed and the zoning map was divided into four quadrants as shown in Figure 2-4. This was used to distribute the generated trips onto the surrounding existing and proposed roads for further analysis.

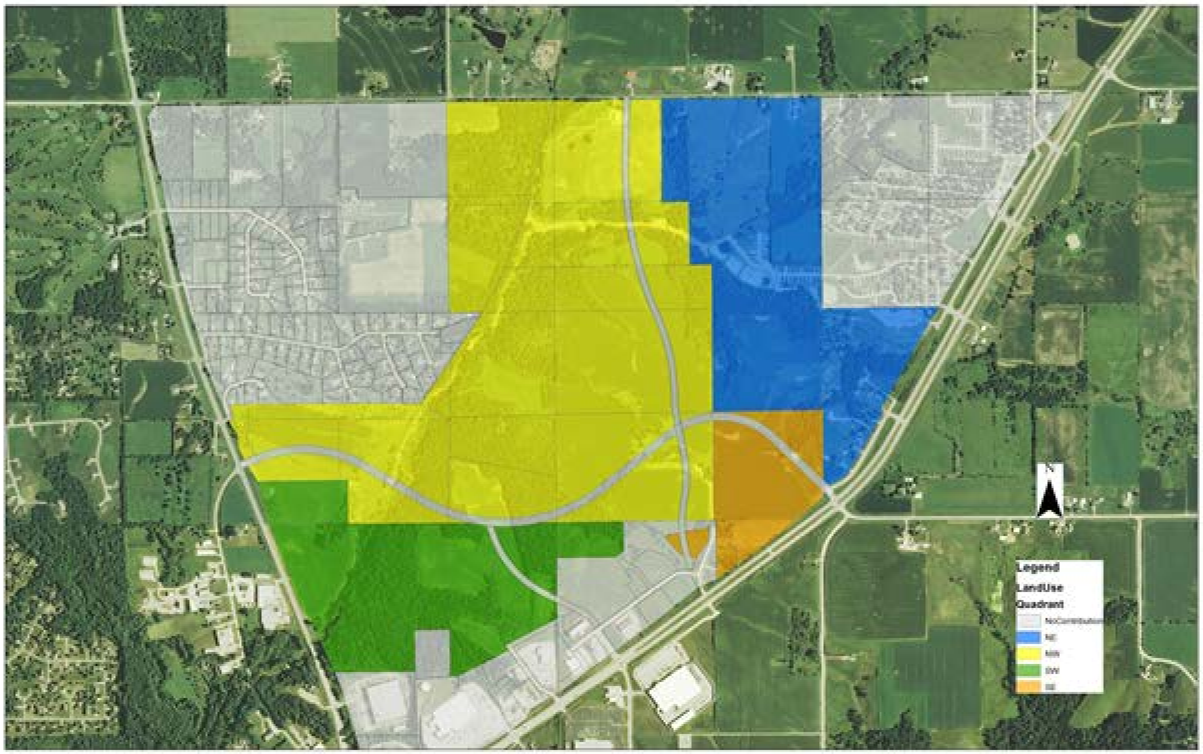


Figure 2-4: Four Quadrants of Project Area

On a quadrant by quadrant basis, the areas for each zone type were calculated as shown in Table A-1 in Appendix A. The areas for each zone type in each quadrant were divided by the total zone area in all quadrants to give a percentage of the total. This is summarized in Table 2-2 below. For example, the northwest quadrant contains 57% of the total residential zoning in the project area.

Table 2-2: Percentage of each zone type in each quadrant.

NW			NE	
RES	57%		RES	39%
COM	46%		COM	0%
IND	0%		IND	0%
SW			**SE**	
RES	4%		RES	0%
COM	31%		COM	23%
IND	100%		IND	0%

Table 2-3: 10 year trip generation per quadrant by zone type

Quadrant	Zone	Cars Generated
NW	Residential	254
	Commercial	35
	Industrial	0
NE	Residential	172
	Commercial	0
	Industrial	0
SW	Residential	19
	Commercial	24
	Industrial	91
SE	Residential	0
	Commercial	17
	Industrial	0

The trips generated per zone per quadrant were then calculated for 10, 20, and 30 year build outs by multiplying the total trip generation shown in Table 2-1 by the percentage shown in Table 2-2. The results for the 10 year (one-third build out) are shown above in Table 2-3. The 20 and 30 year build out numbers can be found in Appendix A. With the total number of trips generated per zone type per quadrant, the number of cars generated onto each road surrounding each quadrant was calculated. This was done by looking at traffic flow around Muscatine around the peak analysis hours of 7:00 to 8:00 AM. It was found that many cars drive south towards the city itself and therefore the percentages used are divided likewise. These numbers were then subdivided again as to whether they would turn left or right onto their designated street. Again, the percentages used were gathered from current Muscatine traffic information. The results for the northwest quadrant for 10 year build out are seen below in Table 2-4. The results for the rest of the quadrants during 20 and 30 year build outs can be found in Appendix A.

Table 2-4: Trips generated and turning movements during 10 year build out onto surrounding roads.

NW

Type	Road	%	Cars
RES	NS Art	80%	203
	180	10%	25
	EW Art	5%	13
	38	5%	13
COM	NS Art	5%	2
	180	0%	0
	EW Art	95%	33
	38	0%	0
IND	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	38	0%	0

NS Art	205	Left	40%	82
		Right	60%	123
180	25	Left	50%	13
		Right	50%	13
EW Art	46	Left	30%	14
		Right	70%	32
38	13	Left	80%	10
		Right	20%	3

Existing traffic data was then combined with the trips generated in each direction on each road to calculate the turning movements for each intersection in the area of analysis. There were four intersections in the area with existing turning movement diagrams. These were IA 38 & Park Ave W, US 61 & IA 38, US 61 & New Era Road, and US 61 & Taylor Ave. The turning movement diagrams for these four intersection can be found in Appendix A. The trips generated onto each road in each direction were then added onto this existing data to obtain total turning movements for each intersection in the area of analysis and for all three stages of build out. The number of turning movements created by trips generated due to development were calculated using the same assumptions as above where a majority of the populace will be driving south towards the inner city during the peak hour.

Table 2-5: Turning movements for intersection of proposed East-West arterial and Park Ave W with IA 38.

IA 38 & Park Ave W/EW Arterial

Existing								
			L	T	R			
			0	351	4			
L	T	R				L	T	R
5	0	3				0	0	0
			L	T	R			
			1	146	0			

IA 38 & Park Ave W/EW Arterial

Additional		
	Incoming	
	10	
Incoming		Incoming
0		60
	Incoming	
	30	

IA 38 & Park Ave W/EW Arterial

Total								
			L	T	R			
			3	338	4			
L	T	R				L	T	R
5	2	3				48	6	6
			L	T	R			
			4	156	18			

Table 2-5 above shows the existing turning movements, additional trips generated per approach, and total divided turning movements for the intersection of IA 38 with Park Ave W and the designed EW Arterial during 10 year build out. The turning movements for the rest of the intersections during each stage of build out can be found in Appendix A. These numbers along with the lane configurations for each intersection approach were then used to determine the signal warrants at each intersection. The signal warrants were determined for every intersection at all three stages of build out using HCS 2010 Warrants. The five intersection warrant results summaries for full (30 year) build out can be found in Appendix A. Results for these intersections at the 10 and 20 year build out marks are available upon request.

Runoff analysis

Several runoff analyses were performed to ensure the proper handling of stormwater that will be generated by rain events in the northeast corridor. A runoff analysis for the existing state of the land was performed to determine the flow rate in Mad Creek for a 100 year rain event to be used when determining a size for any of the proposed culverts. The rational method was used to perform a runoff analysis for the new arterial roads that was used for the design of a stormwater management system along the roadway. According to the Iowa Stormwater Management Manual (ISMM), the rational method is suitable for estimating the runoff of a small and highly impervious area such as parking lots and roads. The maximum area eligible in order to use the rational method is 160 acres. The largest area for the proposed roadway is 20.1 acres, which falls within the appropriate range for the rational method. In order to follow the City of Muscatine City Code, a runoff analysis for a 2-year and 100-year return period was performed because more than 5 acres were developed in the Northeast Corridor. The rainfall intensity for zone 6 (Figure 2-5) from the ISMM was used for analysis.

Figure 2-5: Rainfall intensity zone map of Iowa

The NRCS unit hydrograph method was used to estimate the entire area's runoff as part of the Mad Creek Regional Water Detention Project. The NRCS method was conducted under the condition of both the 2-year and 100-year return period for a 6 hour rainfall duration. This runoff analysis was used in the design of all culverts because the area contributing to Mad Creek is too big for the rational method to be a viable option for runoff analysis. Since all of the culverts were designed for a 100-year return period, a backwater analysis was not necessary for Mad Creek.

Creek Crossing Design

Due to the nature of the terrain in the northeast corridor, four creek crossings were necessary along the arterial roads. The location of creek crossing 1, as shown in Figure 2-6, was considered to be the most extreme location due to the high flow experienced at that location. Also, the elevation difference between the creek bed and proposed road elevation on either side of Mad Creek was evaluated at that location. As a result of the large elevation difference and high flow rate, a bridge was considered as an alternative creek crossing at this location. A detailed discussion of the alternatives for the creek crossing can be found in Section III of this report. Creek crossings 2, 3, and 4 were less critical than the location of culvert 1 because the flows experienced here are much less because they are on smaller branches of the creek or located further upstream. Creek crossing 2 serves as another creek crossing for Mad Creek. Creek crossings 3 and 4 primarily serve to allow runoff to reach the detention basins that have been proposed as a part of the Mad Creek Regional Water Detention Plan shown in Figure 3-2. Section 2-N of the Iowa Stormwater Management Manual was used to design culverts.

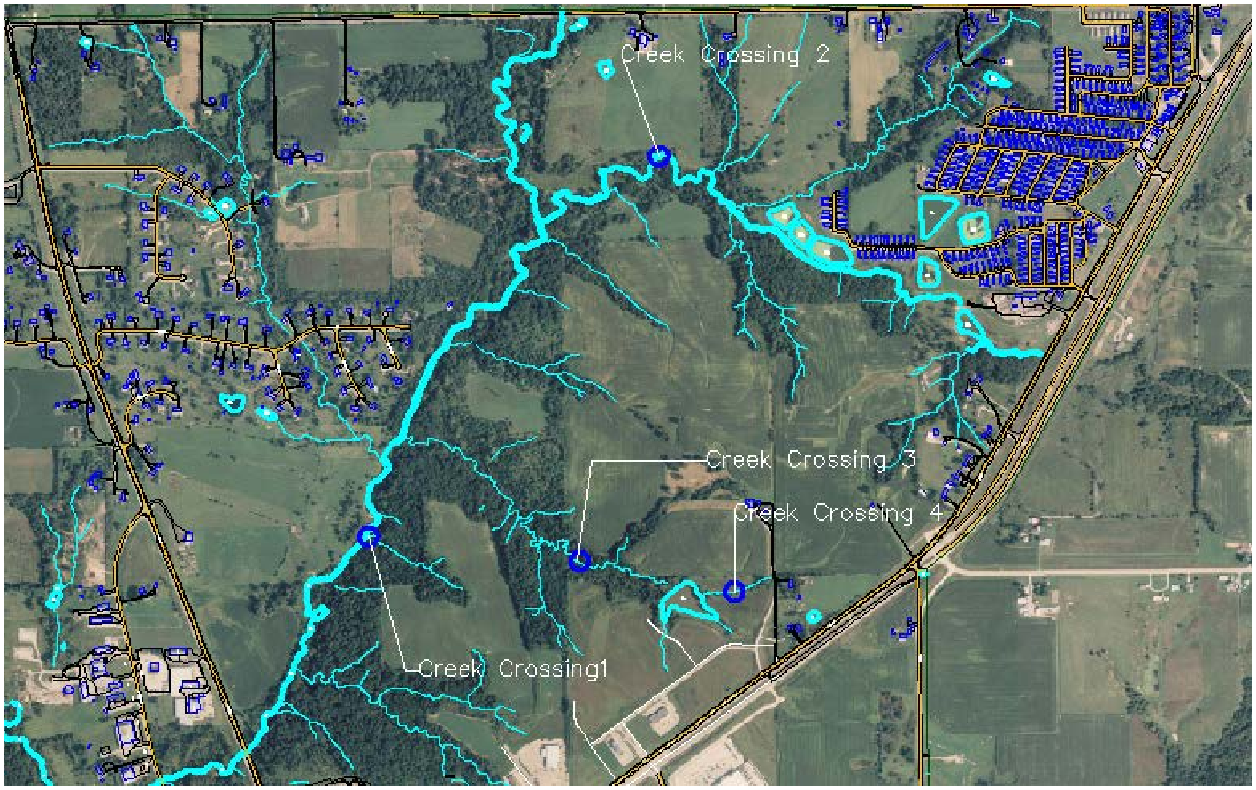


Figure 2-6: Creek Crossing Locations

Storm Sewer System Design

In order to handle runoff created from the construction of the new roadways, a pipe system with curb inlets and a vegetative swale were considered by Hawkeye Engineering. These two sewer systems were considered because they each have their own advantages and disadvantages, which will be further discussed in Section III.

The approach used to design a curb inlet and pipe size estimation was to follow the ISMM and Water-Resources Engineering 3rd edition's recommendations. The calculation and selection of the curb inlet was made using Chapter 2M-3 of the ISMM.

The vegetative swale was designed using the Water-Resources Engineering 3rd edition. This swale was designed to retain a fixed volume of runoff with a triangular cross-section. The Manning equation was also used to calculate the required length of the swale.

Bridge Design

As an alternative to culvert 1, a slab-girder bridge was designed that would not modify the cross-section of Mad Creek. Since the cross-section of the creek was not modified, a backwater analysis was not performed. The AASHTO LRFD bridge design strip method was used to determine the required strength of the girder and slab. A detailed analysis of the bridge substructure was not investigated as a part of this project, but it would be necessary for the final design. The load in Figure 2-7 was applied to the slab design strip widths as a moving load to determine the worst loading case for positive bending moment, negative bending moment, and shear force in the slab. All moving load analyses were performed using Autodesk's Robot; the resulting influence functions can be found in Appendix B. The vehicular live load was applied along with the self-weight of the bridge and a load resulting from a 2" asphalt overlay that would simulate a potential wearing surface replacement later in the life of the bridge.

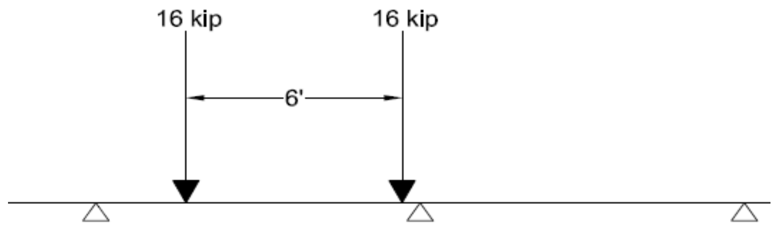


Figure 2-7: Cross-section with vehicular point loads

The (American Concrete Institute) ACI building code was used when computing the design strength of the concrete slab. The results of the design strength calculations can be found are presented in Section III. To determine the required strength of a girder, the AASHTO HL-93 truck load (Figure 2-8) was applied as a moving load to determine the worst loading case and the load placement that causes maximum positive bending moment, maximum negative bending moment, and maximum shear force in a girder. As with the slab, other loads that were applied to determine the required strength of the girders include the self-weight of the bridge along with the load that would result from a 2" asphalt overlay. The American Institute of Steel Construction (AISC) steel construction manual was consulted to calculate the design strength of a steel girder.

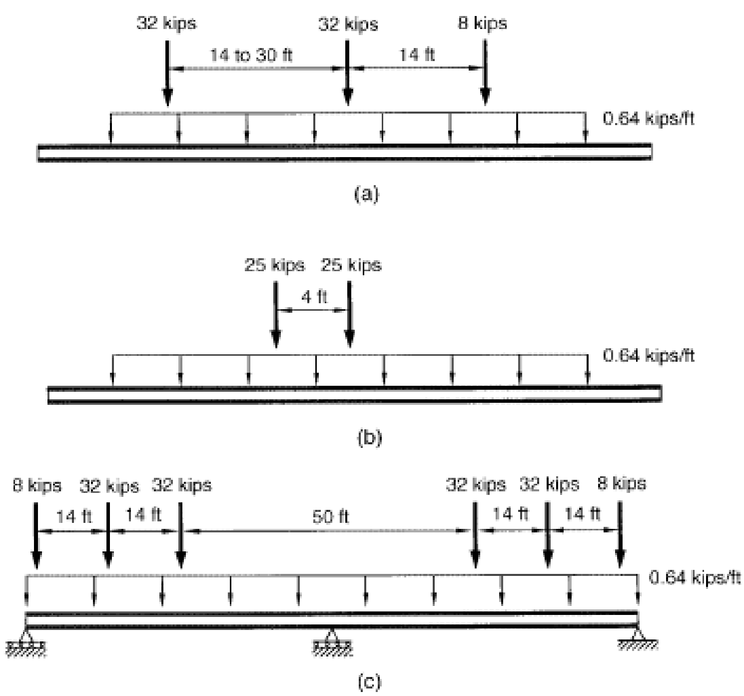


Figure 2-8: AASHTO HL-93 truck loading side view

Image courtesy of Design of Highway Bridges: An LRFD Approach by Barker and Puckett

Sanitary Sewer Approach

Using the usable area values for several land use types in Table 3-1, the expected sanitary sewer flows that would be generated after development for each land use were estimated based on the Iowa DNR Design Manual. With the expected daily flow, a diameter and slope of the sanitary sewer pipe was assumed and iterated together in order to calculate the size of the sanitary sewer. Using Manning's equation, the flow rate in the pipe was estimated and confirmed to be less than the maximum velocity in the pipe to prevent scouring.

2.3 Constraints

Several constraints became evident upon further analysis of the terrain in the Northeast Corridor. Most of the constraints served to confine the location of the arterials to a small area of the land available for development. The location of the proposed arterial roads were constrained by the existing intersections along U.S. 61 and IA-38 which provided natural access points to the proposed roads without significant alterations to the existing road network, as shown in Figure 2-9 and Figure 2-10.

Figure 2-9: Location of the intersection of the E-W arterial with IA-38

Figure 2-10: Location of the intersection of the E-W arterial with US-61

The N-S arterial had a similar set of constraints when considering the location of the intersection with surrounding roads. The north intersection was constrained to be between two branches of Mad Creek near 180th St. (Figure 2-11).

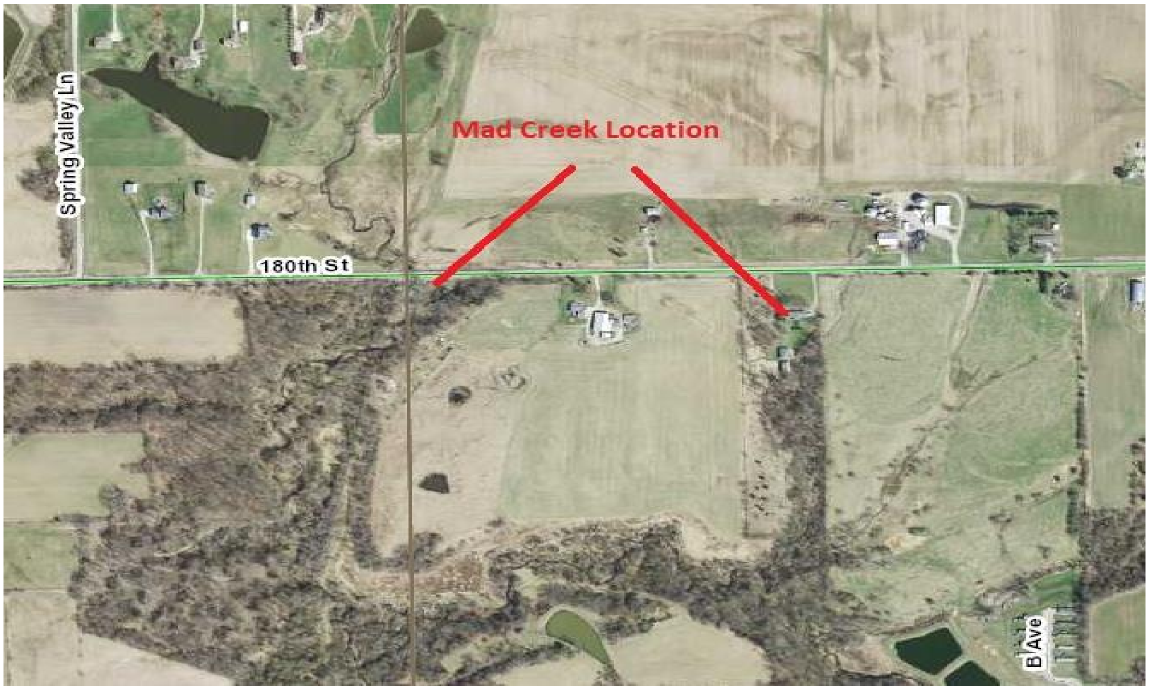


Figure 2-11: General Location of the intersection of the N-S arterial with 180th St.

The willingness of the current landowners to sell their land for development could become a major hindrance in the progression of the development plan in the Northeast Corridor and is the biggest potential delay in the project.

When considering whether to use a bridge for each of the creek crossings discussed in Section 2-2, the creek cross-section provided a constraint. If a bridge were to be utilized for creek crossing 1, it would span a distance of 130 ft. to connect the two peaks as shown in Figure 2-12.

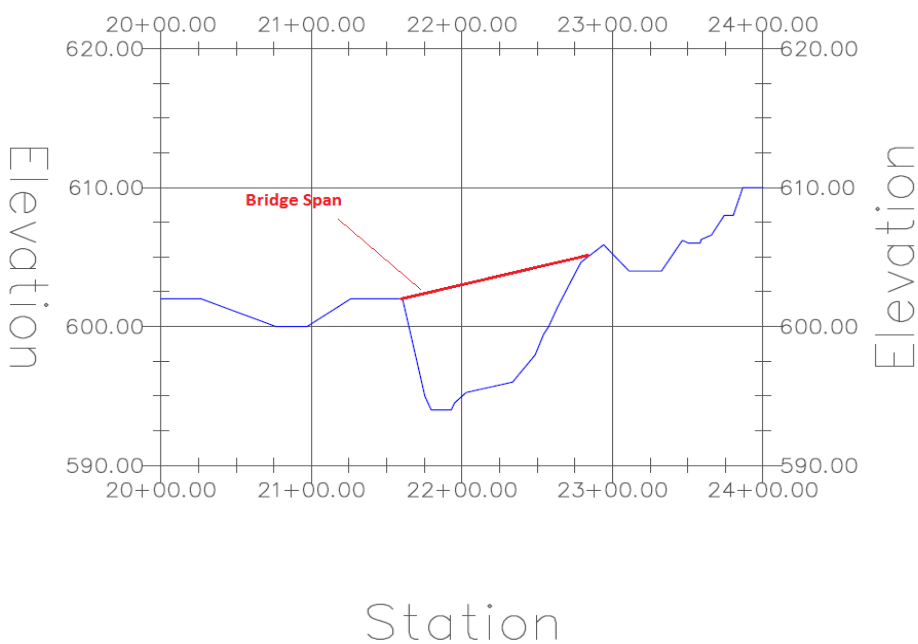


Figure 2-12: Mad Creek cross-section at creek crossing 1 looking north

2-4 Challenges

Many challenges related to the terrain in the Northeast Corridor influenced the design of the roads and necessary creek crossings. Some of these challenges were discussed in detail in Section 2.3. Aside from the constraints discussed previously, the steep terrain dictated the vertical alignment of the proposed roadways to minimize the necessary earthwork. However, the sudden changes in elevation required fairly significant alterations to the terrain in certain areas to maintain a reasonable longitudinal slope for the arterial roads. Along with the terrain, new development along US 61 (Figure 2-13) prevented one of the designed road segments initially proposed by Snyder & Associates, Inc. from being implemented. Hawkeye Engineering was required to develop an alternate plan to connect US 61 with the E-W arterial.

Figure 2-13: Development conflicting with the proposed extension of University Dr.

2-5 Selection Process

<u>Creek Crossing Selection</u>

Using the runoff calculated for a 100-year return period in Table 3-2, a culvert with a diameter of 25 ft. was determined to be necessary at creek crossing 1. A bridge was determined to be the superior option for creek crossing 1 due to the excessively large 25 ft. diameter culvert that was determined to be necessary to handle the flow in Mad Creek. The cross-sectional view of Mad Creek at creek crossing 1 is shown in Figure 2-14. Also, constructing a culvert in this location would require a large amount of floodplain to be filled around the culvert which is undesirable. The other three creek crossings have a much lower flow rate than creek crossing 1 and are more suitable for a culvert than a bridge.

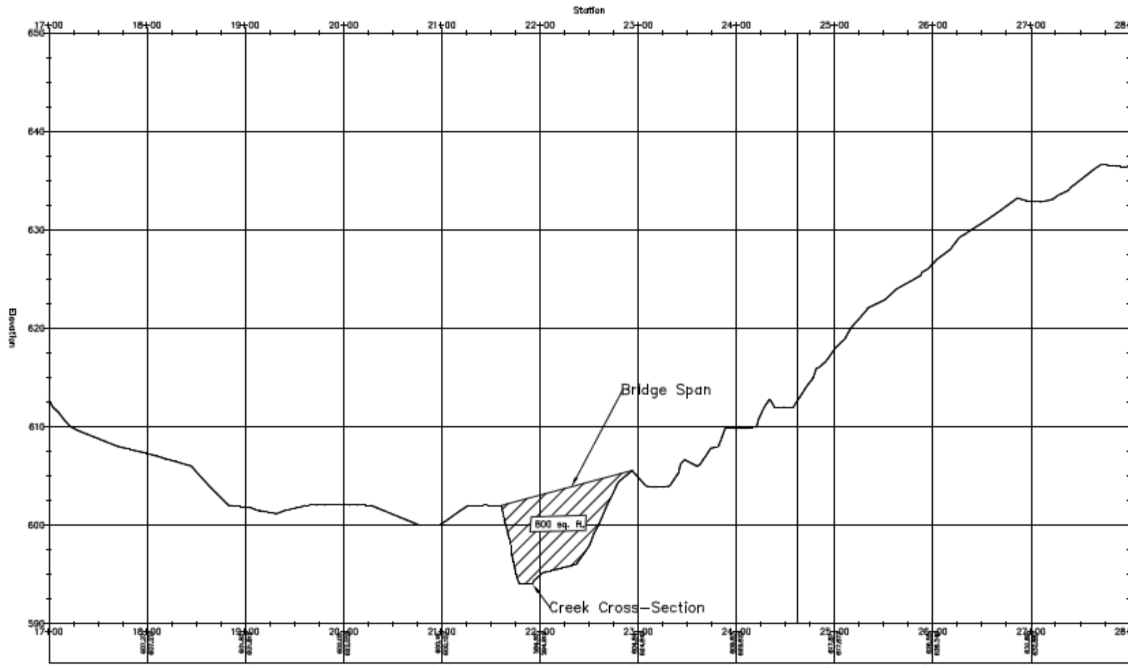


Figure 2-14: Cross-Section Mad Creek at culvert 1 location looking north

Lane Options

Snyder & Associates, Inc. proposed two different lane options which are shown in Figures 2-15 and 2-16. The first option includes a median along the road, with a left turning lane at the intersections. The second option has a shared left turn lane along the entire length of the roadway. Due to the large amount of commercial zoning in the area, it was determined the E-W arterial will utilize option 1. It will have a 90 ft. right-of-way with a shared left turn along the length of the road as shown in Figure 2-15 below. This allows for easy access to the multitude of stores and businesses located alongside the road and will slow traffic down without causing major interruptions. The N-S arterial will utilize option 2 and have a 100 ft. right-of-way with a raised median like that shown in Figure 2-16 below. The road serves a mainly residential population and will allow safe access to neighborhoods on either side. The right-of-ways associated with each cross section are typical of similar arterial roads and will provide a small amount of room for road expansion and improvements if deemed necessary in the future.

Figure 2-15: Typical Cross Section

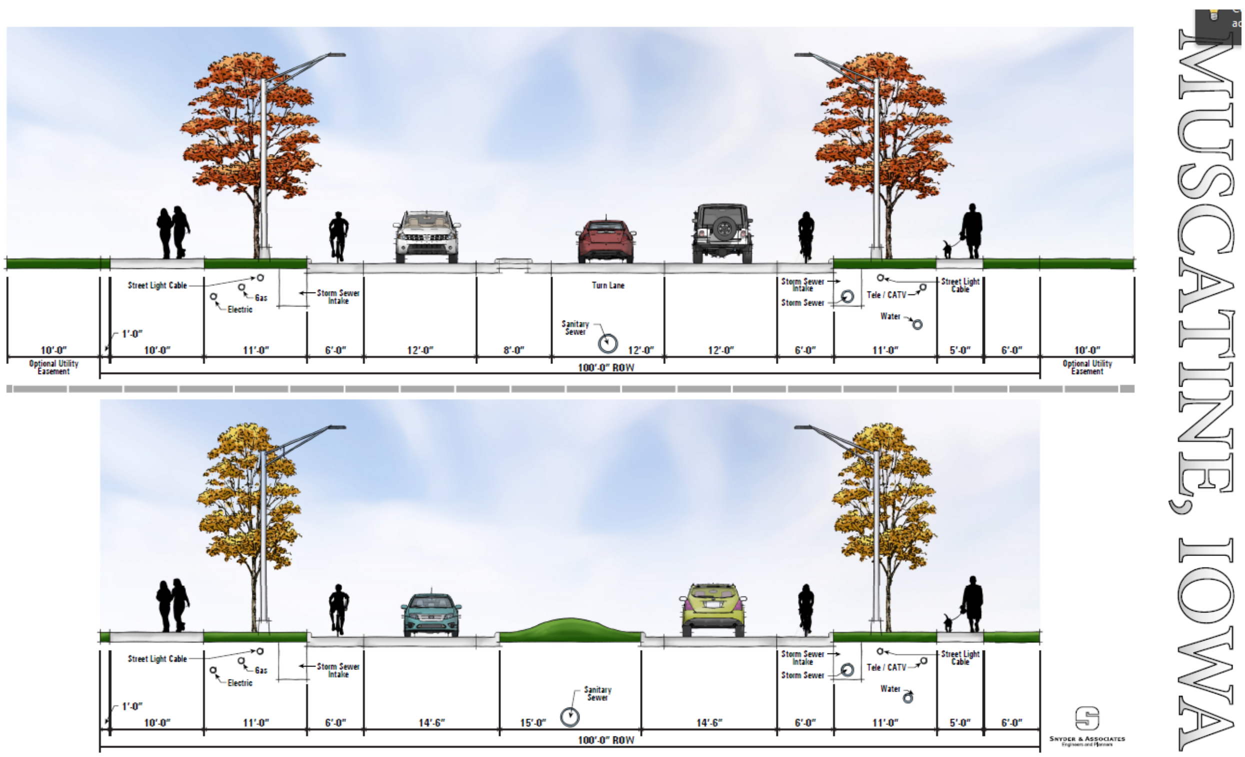


Figure 2-16: Typical Cross Section

2-6 Societal Impacts

The development in the northeast corridor in the City of Muscatine has been planned by the city to encourage growth in the area. US-61 and IA-22 have been built to increase accessibility to the properties in this area, this means that the economy around this neighborhood will have a greater possibility for business development. More commercial stores are planned to be built in order to offer convenience to the neighborhood, and thus attracting more residents.

The stormwater structures in our plan will reduce the contamination in the runoff going to the river to eliminate negative health effects on residents.

The population in the city of Muscatine has been steadily increasing at a rate of approximately 3%. The expansion will provide sufficient new space for the increasing populace. A low unemployment rate is an important factor in maintaining a strong, durable, and diverse economy. From Chapter 9, Figure 1 of Muscatine Comprehensive Plan, the unemployment rate has been below the average for the state of Iowa and the US. It can be deduced that with our expansion of the city, more jobs will be created. The categories include but are not limited to: the construction of the roads, utilities, facilities and the employees in the new retailer outlets. There is a potential need for government employees within utility services. All of the jobs created will support economic growth.

The project will also allow more interactions between the City of Muscatine and other cities in the state of Iowa, given a development in the transportation, and an expansion in industry. The efficient design of intersections will lead to minimal traffic delays and therefore reduce the economic costs associated with congestion.

Hawkeye Engineering strives to uphold all guidelines in order to practice under the fundamental canons of ethics. All team members of Hawkeye Engineering perform services only in areas of their competence. Working alongside the Van Allen Design Group, all team members act in a professional manner while avoiding any conflicts of interest. Most importantly, the City of Muscatine was under strict eye of Hawkeye Engineering in order to ensure the safety, health and welfare of the public throughout the duration of this project. Hawkeye Engineering is an honorable firm upholding the dignity of the engineering profession with no tolerance for bribery, fraud or corruption.

III. Preliminary Development of Alternative Solutions

Route Design

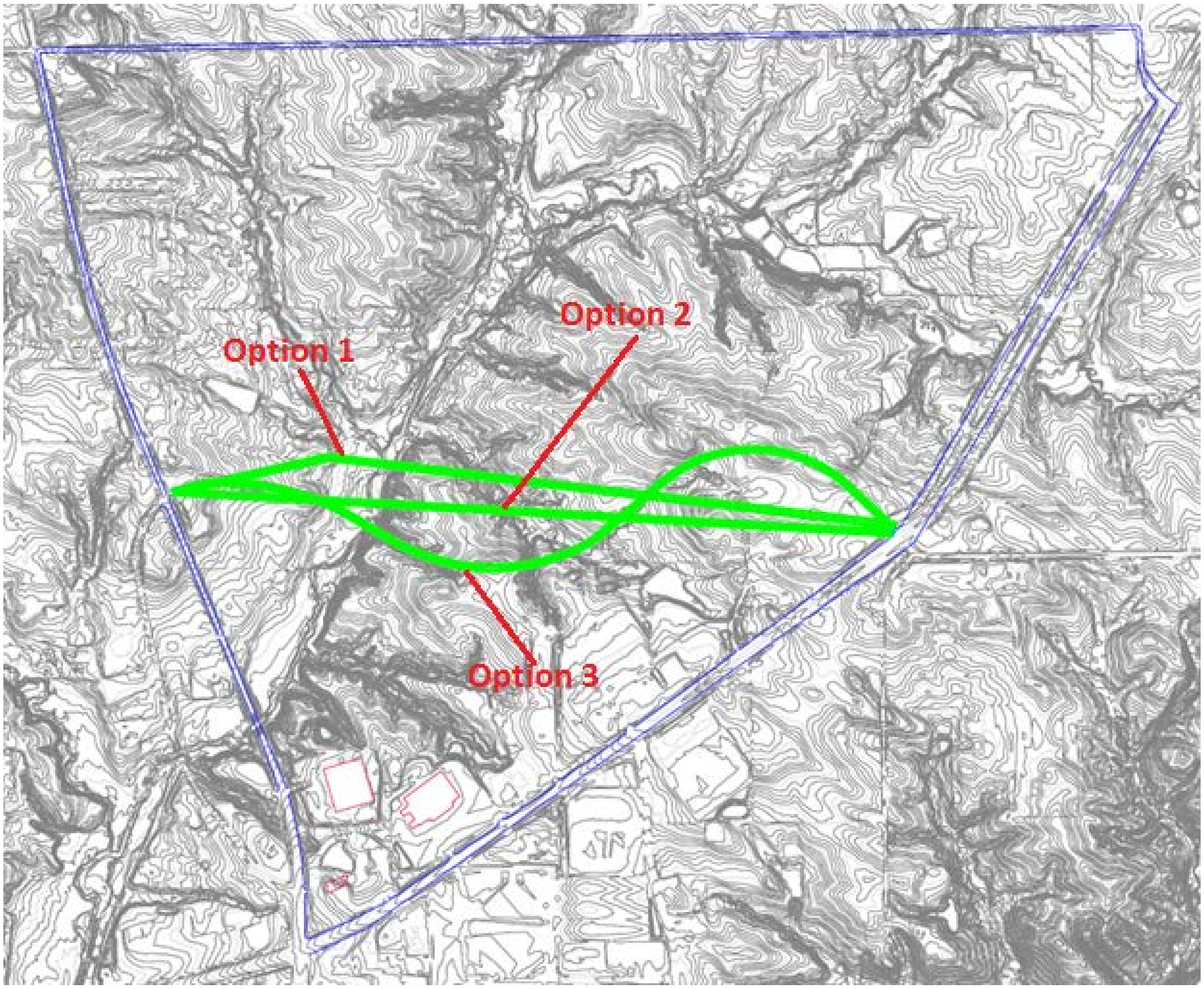


Figure 3-1: Potential EW Arterial Options

Potential layouts for the East-West arterial are shown above in Figure 3-1. Option one consists of a relatively linear road with only curved sections at the ends to provide perpendicular attachments to US-61 and IA-38. Option two is an entirely linear road with non-perpendicular connections at each end. Option 3 consists of a much curvier road than the previous two with perpendicular connections at each end.

Runoff Analysis

The results from runoff analyses for the proposed N-S and E-W roads are shown in Table 3-1 for the final cross-section selections. Analyses for the pre-development state with a runoff coefficient of 0.15 and the post-developed state with a runoff coefficient of 0.90 (typical for impervious pavement) were performed and later used in the design of a stormwater management system for the new roads.

Table 3-1: Final proposed route runoff analysis

	A (ft^2)	I $\left(\frac{in}{d}\right)$	Q = CiA 2 Year Pre (cfs)	Q = CiA 100 Year Pre (cfs)	Q = CiA 2 Year Post (cfs)	Q = CiA 100 Year Post (cfs)	Difference Between Pre & Post
E-W	877140	0.13	0.0165	0.1142	0.0935	0.2157	0.1102
N-W	694400	0.13	0.0131	0.0904	0.074	0.1708	0.1222
Total			0.0296	0.2046	0.1675	0.3865	0.2190

Curb Inlet Design Analysis

A curb inlet system was designed using the ISMM with following parameters to handle the runoff that was calculated using the rational method.

- Longitudinal slope: 4%
- Transverse slope: 2%
- Manning's coefficient: 0.013

With the design flow rate for a 100-year storm, the total runoff rate was estimated to be 0.219 cfs. According to intake standards, the Curb-Grate SW-501 was selected. The capacity of the curb inlet was estimated to be 0.161 cfs. Five intakes along the N-S road and four intakes along the E-W road were designed. The curb inlets were located at low points along the path of the proposed roads. All design calculations for the curb inlet system can be found in Appendix D.

Swale

All of the calculations and formula used in this section can be found in Appendix D. A swale was designed to treat the same amount of runoff as the curb inlet sewer system, which is 0.219 cfs for 100-year return period storm. An infrequently mowed swale was selected with a Manning roughness coefficient of 0.24. The length of the swale was estimated to be 1600 ft. from the Water-Resource Engineering 3rd edition.

Stormwater System

The curb inlets along with pipes and the vegetative swale were considered in our design. A curb inlet system is one of the most popular storm sewer systems. The advantage of using a curb inlet and pipe system to collect extra runoff is that the pipe can be used for future development runoff collection by simply adding branch pipes and other curb inlets, as long as the pipe has enough capacity. A curb inlet system can also provide redundancy during the high flows preventing flash flooding. The disadvantage is that future maintenance could be expensive, and the capital cost is high. As for the swale, it cannot handle as much water as the pipe system can take, due to the requirement of large collecting area. A swale cannot be connected for future development either. However, it is much cheaper and easier to be built and can provide a green space, while there is no precipitation. The curb inlets were selected in our final design as the stormwater management method due to the expected growth in the NE Corridor. The final design details for the stormwater system are presented in Section IV.

Culvert Design

The overall peak discharge data was analyzed using the NRCS unit hydrograph method as a part of the Mad Creek Regional Water Detention Project for the 6 rainfall subcatchments in Figure 3-2. Both 2-year and 100-year return period storm conditions were estimated. The peak discharge used in the culvert design process was the 100-year return period for pre-developed state.

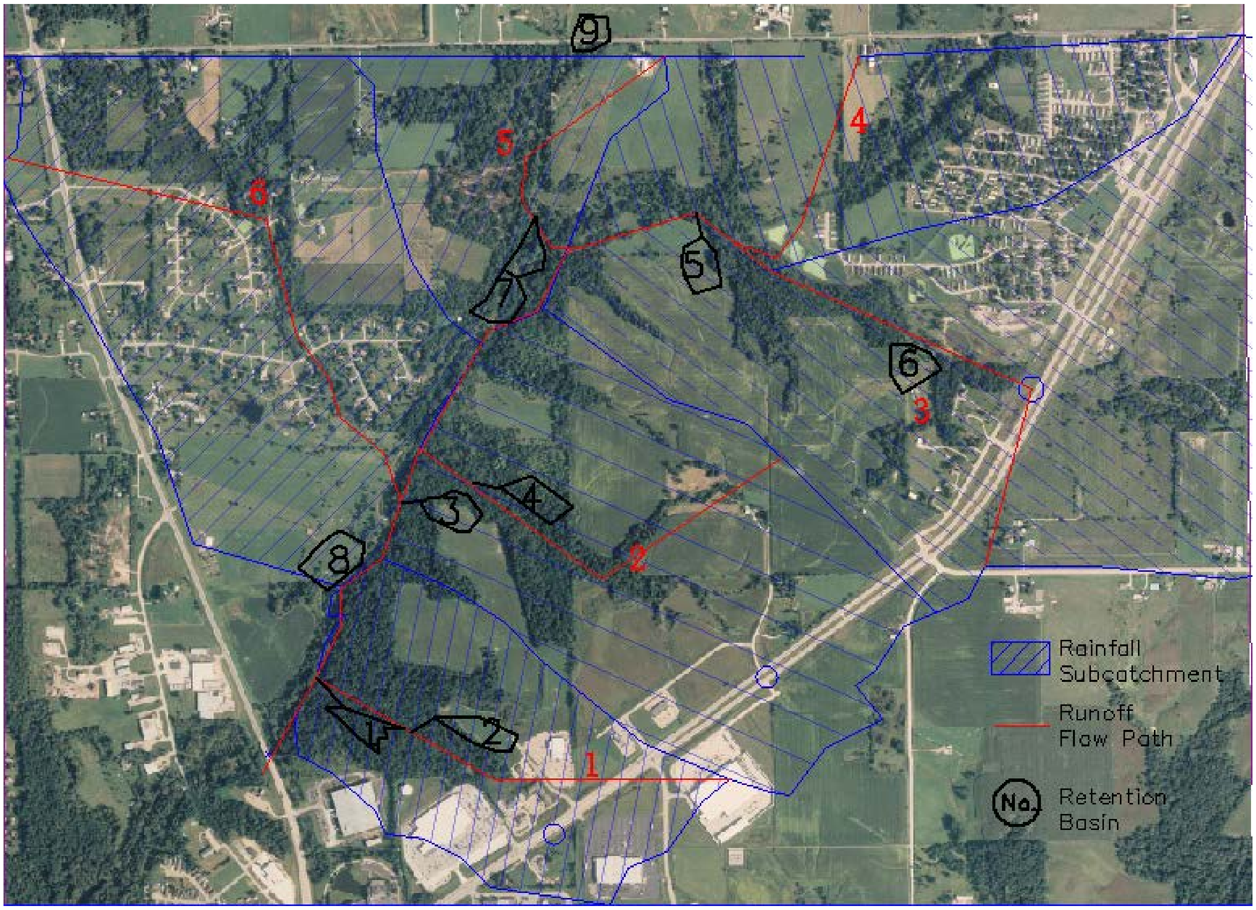


Figure 3-2: Subcatchment Arrangement

All of the calculations and formulas used for the culvert design can be found in Appendix C. The fixed-flow method was used in the culvert designs. The peak discharge of an individual culvert was estimated by the percentage of each water subcatchment and the runoff flow path. The peak discharge was estimated by summing the contributing areas' runoff, which was the worst scenario that can happen in real life. Adding all of the peak discharge of the contributing areas together means that there are storm delays for every contributing area and the delays accumulate runoff together. This is a very conservative peak discharge to use in culvert design. In Figure 3-2, the shaded area represents different subcatchments. Culvert 1 located in the main branch of Mad Creek, thus the peak discharge for culvert 1 was almost the overall peak discharge. Because culverts 2, 3, and 4 are not located on the main branch of Mad Creek, the runoff at these locations is significantly less. The calculated peak discharge for each culvert is shown Table 3-2.

Table 3-2: Peak Discharge for each Culvert

	Q (cfs)
Culvert 1	1957.5
Culvert 2	720.9
Culvert 3	336.5
Culvert 4	224.3

During the design process, the culvert length was assumed to be a little bit larger than the road width, which is 75 ft. The slope of the culvert was assumed to be 1%. All of the culverts are concrete grooved pipe culverts with different sizes. The Manning constant for a concrete pipe with good joints and smooth walls is 0.013. The culvert entrance loss coefficient for groove end with wing walls is 0.2. A rendered picture of the proposed culverts is shown in Figure 3-3.

Figure 3-3: Concrete grooved pipe culvert

According to the ISMM, the flow velocity inside the culvert should be between 3 ft/s to 5 ft/s. During the design process, 5 ft/s of the flow velocity within the culvert was used. Type 2 flow (inlet control) was assumed in the first. To determine whether the culvert was inlet control, the ratio of the head water depth and the diameter of the pipe culvert had to be calculated. The ratio of head water depth to the diameter of the pipe culvert was found by using the inlet control nomograph in the ISMM Section 2N-2 found in Appendix C. Once the ratio is known, the head water depth was calculated. If the head water depth is greater than the pipe diameter, the culvert will be inlet controlled. Otherwise, the culvert will be outlet control. Both Type 2 and Type 3 flows were considered. The difference of the head water depth and the diameter of the culvert pipe was always positive, thus inlet submergence was sustained. Since the length of the culvert is smaller than ten times the culvert pipe diameter, the culvert was hydraulically short. Thus, Type 3 flow can also be a possibility of this culvert. The ratio of headwater depth and the diameter of the culvert pipe was smaller than 1, which means that the inlet was not submerged. All Type 3 flow should be inlet control, therefore the proposed culverts should not be type 3 flow. Thus Type 2 flow was the designed flow type for all of the proposed culverts. The typical headwater depth and culvert diameter ratio in the United States is 1 to 1.5. All of the culvert designs in this project were within this range. The design summaries of four culverts are shown below, from Table 3-3 to Table 3-6.

Table 3-3: Culvert 1 design results and parameters

Culvert 1	
Calculated D (ft)	22.33
Integer D (ft)	25.00
R	6.25
Assume Type 2	
H/D	1.05
H	26.25
Δh=H-D	1.25
L < 10 D	Hydraulically Short
Assume Type 3 (Not Applicable)	
Fr	0.176
Cb	1
Cc	0.729
H/D	0.758

Table 3-4: Culvert 2 design results and parameters

Culvert 2	
Calculated D (ft)	13.55
Integer D (ft)	15.00
R	3.75
Assume Type 2	
H/D	1.08
H	16.27
Δh=H-D	1.27
L < 10 D	Hydraulically Short
Assume Type 3 (Not Applicable)	
Fr	0.228
Cb	1
Cc	0.729
H/D	0.778

Table 3-5: Culvert 3 design results and parameters

Culvert 3	
Calculated D (ft)	9.26
Integer D (ft)	10.00
R	2.50
Assume Type 2	
H/D	1.13
H	11.31
Δh=H-D	1.31
L < 10 D	Hydraulically Short
Assume Type 3 (Not Applicable)	
Fr	0.279
Cb	1
Cc	0.729
H/D	0.802

Table 3-6: Culvert 4 design results and parameters

Culvert 4	
Calculated D (ft)	7.56
Integer D (ft)	8.00
R	2.00
Assume Type 2	
H/D	1.17
H	9.34
Δh=H-D	1.34
L < 10 D	Hydraulically Short
Assume Type 3 (Not Applicable)	
Fr	0.312
Cb	1
Cc	0.729
H/D	0.820

<u>Bridge Design</u>

The moving load analyses performed using Robot resulted in the required strength of the PCC slab in Table 3-7, which summarizes the data found in Figures B-1, B-2, B-3, B-7, B-8, and B-9 (Appendix B). Following the ACI building code, the design strength computation results are shown in Table 3-8. The 12 in. slab thickness that is recommended was controlled by the applied shear force. If shear reinforcement were provided, the slab thickness could be reduced if desired.

Table 3-7: Required strength of the PCC slab for 2 different girder spacings

Girder Spacing (ft)	Shear (kips/ft)	Positive Moment (kip-ft/ft)	Negative Moment (kip-ft/ft)
9	7.5	5.3	12.0
7.5	5.2	4.4	11.4

Table 3-8: Design strength of a concrete slab for varying girder spacing

Girder Spacing (ft)	Slab Thickness (in)	Shear (kips/ft)	Moment (kip-ft/ft)
9	16	16	47.7
7.5	12	11.4	33.5

Following a moving load analysis for the three load cases in Figure 2-8, the required strength of an interior girder is shown in Table 3-9, which summarizes the data found in Figures B-4, B-5, and B-6 (Appendix B).

Table 3-9: Required strength of an interior girder for 2 different girder spacings

Girder Spacing (ft)	Shear (kips)	Positive Moment (kip-ft)	Negative Moment (kip-ft)
9	132	1041	1124
7.5	163	993	1095

The bending strength of a W21x122 section provides bending moment strength of 1151 kip-ft. This strength was computed with lateral truss braces spaced at 10 ft. to prevent lateral torsional buckling from occurring when the section is under negative bending moment. The shear strength of the section was computed to be 351 kips. The strength provided by the W21x122 section is adequate for the applied loading. Detailed results and calculations for the structural design of the slab-girder bridge can be found in Appendix B.

Sanitary Sewer Analysis

Using the results from the land use analysis, the expected sanitary flows were estimated using the Iowa Wastewater Facilities Design Standards from the Iowa DNR. For the residential zones, the US Census Bureau estimates the number of people per home to be 2.41 for the state of Iowa. The average size of a single residential lot was estimated to be 6500 ft^2. Besides the residential wastewater production, the commercial wastewater flow was also estimated. The commercial wastewater flow was related to type and size of the business, and the number employees using the DNR standard. An open space ratio of 0.5 combined with a floor to area ratio of 2 (low end estimate) and 4 (high end estimate), the overall commercial area was estimated. The industrial wastewater flow depends largely on the type and size of the industry, operational techniques, and methods of on-site wastewater treatment. Therefore, a more detailed analysis of a particular industry would be required to more accurately predict the discharge coming from an individual plant. Unit flows obtained from the Iowa DNR were used to estimate the total flows that are expected for the given area, and the results from these calculations are presented in Table 3-10.

Table 3-10: Land use and corresponding expected flow

Type of Building	Total Usable Acrea (acre)	Total Area Percentage (%)	Unit Flow (gpd/acre)	Expected Flow (gpd)	Flow Percentage (%)
Mobile Home	99	12	100	160000	5
Residential	474	58	100	770000	26
Industrial	31	4	10000	310000	11
Total Commercial	217	26	5000	1630000	56
Total	821	100	-	2870000	100

Besides the overall wastewater flow, there may be fluctuations of flow for the residential and commercial sector. The infiltration/inflow were also taken into consideration at the beginning and end of the design period. Since the infiltration/inflow are not the key factor in our design and can be considered constant, a conventional number of 0.029m^3/s was used in our design. The peak factor had been applied to the residential and commercial sector, which yields a total of 0.302 m^3/s as peak discharge, and a minimum of 0.181m^3/s. The calculations for the peak factors were demonstrated in Appendix E.

Based on a few iterations, the diameter and slope were calculated to be 0.5 m and 1% respectively. With several attempts in iteration, the slope of 1% would be considered appropriate with a design diameter of 0.5 meter. The iteration process can be found in Appendix E.

From Manning's equation, the flow velocity in the pipe under the full flow condition was estimated to be 2.95 m/s, which was below the limit of 3.5 m/s to 4.5 m/s. The velocity when the flow rate was equal to Q_{min} was also estimated and was above the limit of 2 ft/s to ensure self-cleansing. Detailed calculations are shown in Appendix E.

The depth of the sanitary is designed to be 10 ft. under the ground in order to prevent freezing and along the contour lines for a steady slope. The sewer pipe is to be constructed under all utilities and is to be connected to the wastewater treatment facilities outside our design boundary.

IV. Final Design Details

Creek Crossings

As discussed in Section 2.5, culvert 1 was replaced by a bridge due to the enormous size and possible backwater effect (if designed with 25 return period storm). The locations of the bridge and other three culverts are shown in Figure 4-1.

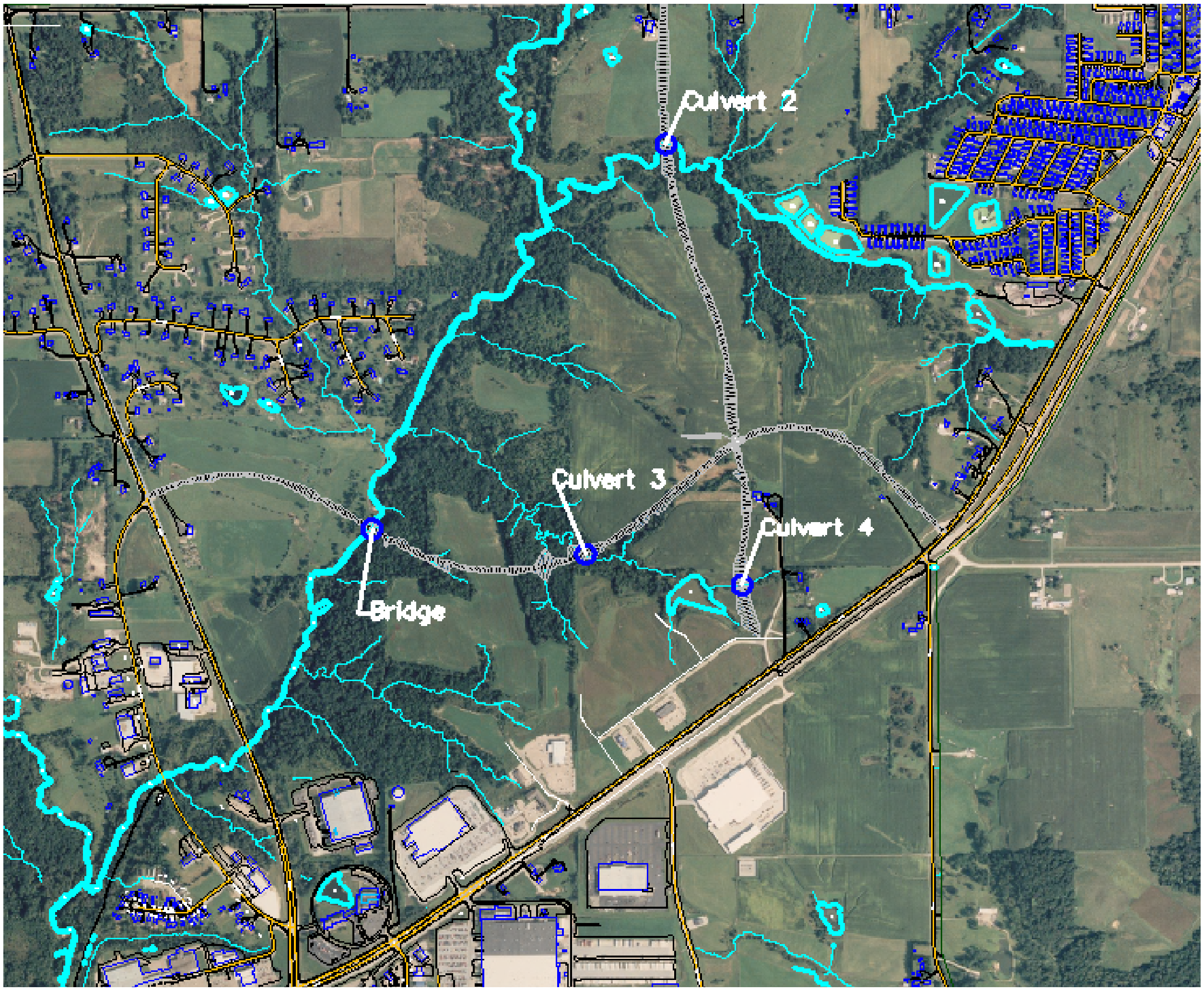


Figure 4-1: Final Creek Crossing Location

Bridge Design

The final bridge design consists of 7 W21x122 steel girders equally spaced at a distance of 7.5 ft., as shown in Figure 4-2. Along the span of the bridge, truss structures consisting of L6x6x1/2 members will be provided between the girders at a longitudinal spacing of 10 ft. The overall 130 ft. span will be covered by 40 ft. spans on either end and a 50 ft. central span (Figure 4-3).

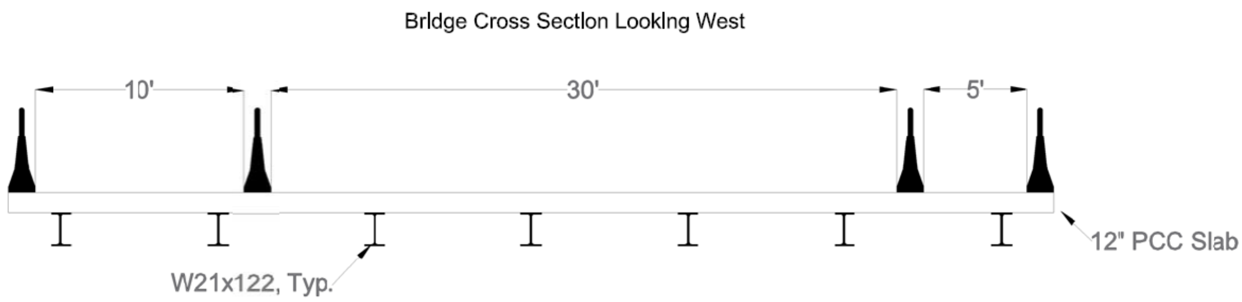
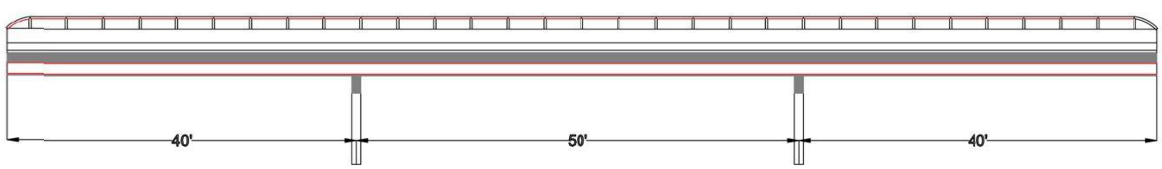


Figure 4-2: Bridge superstructure cross section looking west



Figures 4-3: Bridge side view

Culvert Designs

Based on the previous calculations and analyses, the following AutoCAD sketches of the designed culverts were generated. The culvert sketches are from Figure 4-4 to Figure 4-6. All of the units used in these sketches are ft.

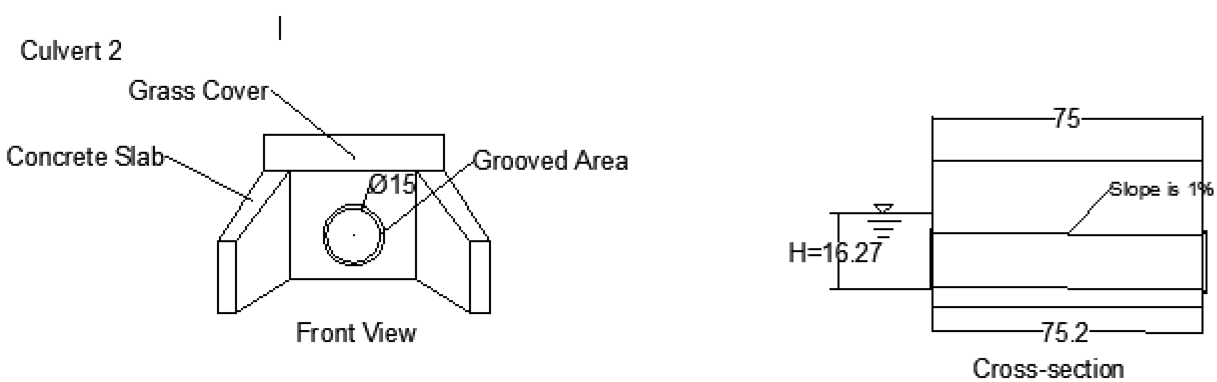


Figure 4-4: Culvert 2 Design Details

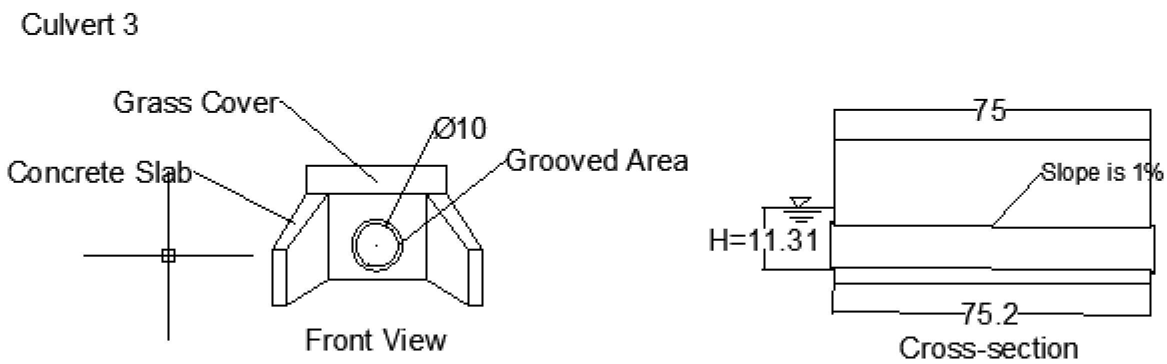


Figure 4-5: Culvert 3 Design Details

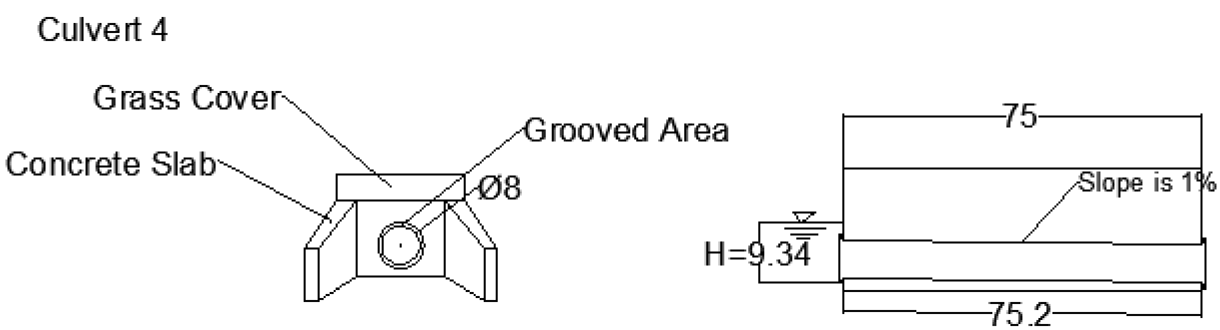


Figure 4-6: Culvert 4 Design Details

The proposed culvert sizes were calculated under the most severe storm situation, which is a 100-year return period for 6 hour duration. By using 100-year return period runoff, it can be almost guaranteed that there will be no flood in that area, which is a very attractive characteristic to those people who are planning to open a business or move their homes to this area. Normally, culverts are designed for a 25-year return period storm. Thus, these culverts are much bigger than a typical design. In the future, taking into account the Mad Creek Regional Water Detention Project, the peak discharge of this area would be reduced significantly. Thus, the culvert sizes can be decreased, which would be more affordable.

Storm Sewer System Design

The final design for the storm sewer structure of our choice was the Curb-Grate SW-501. The locations of the inlets are denoted in Figure 4-7 by the blue dots. The inlets were located at the lower elevation part of the road for maximum intake of the runoff from the road.

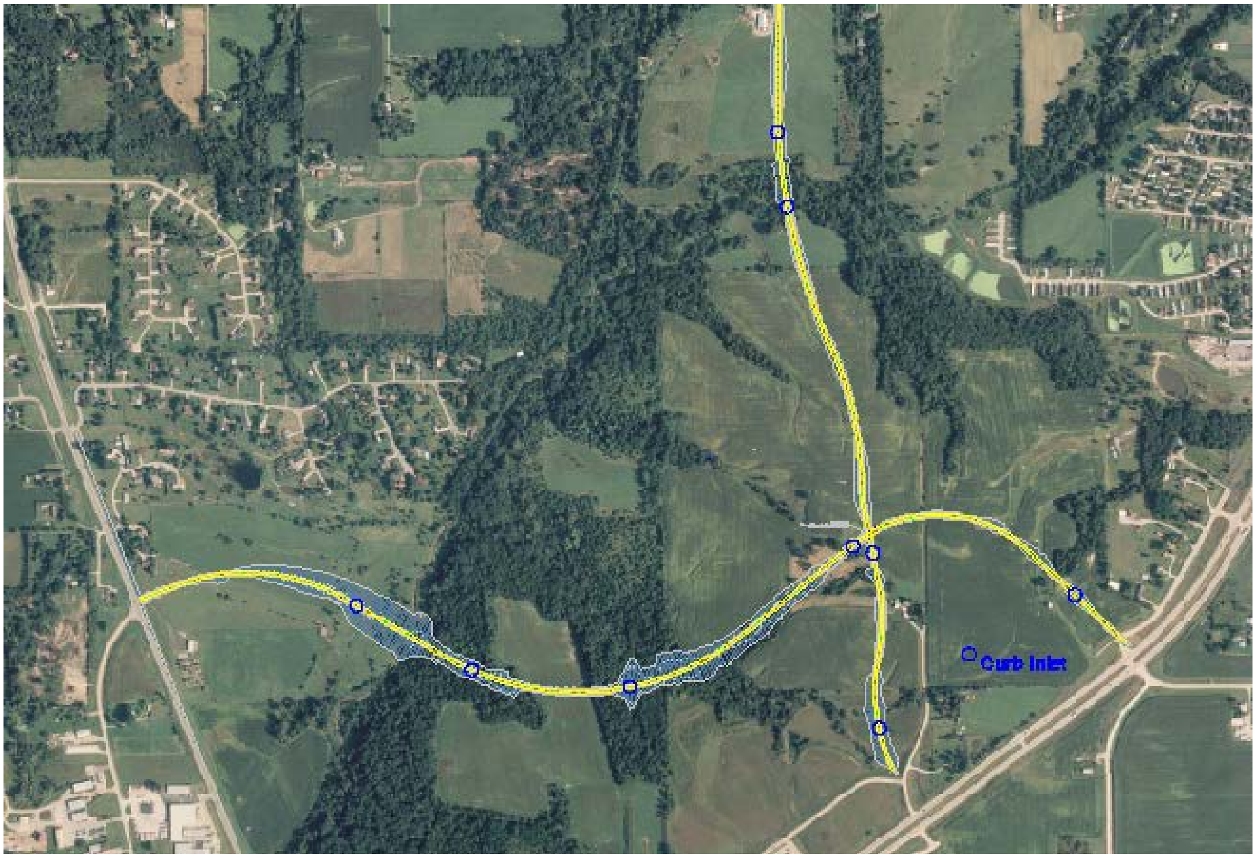


Figure 4-7: Curb Inlet Locations

Sanitary Sewer Design

The final design for the sanitary sewer pipes were selected with an 18 in. diameter reinforced concrete pipe. The alignment of the pipes were to be designed along the contour lines (in Figure 4-8) in order to take advantage of gravity.

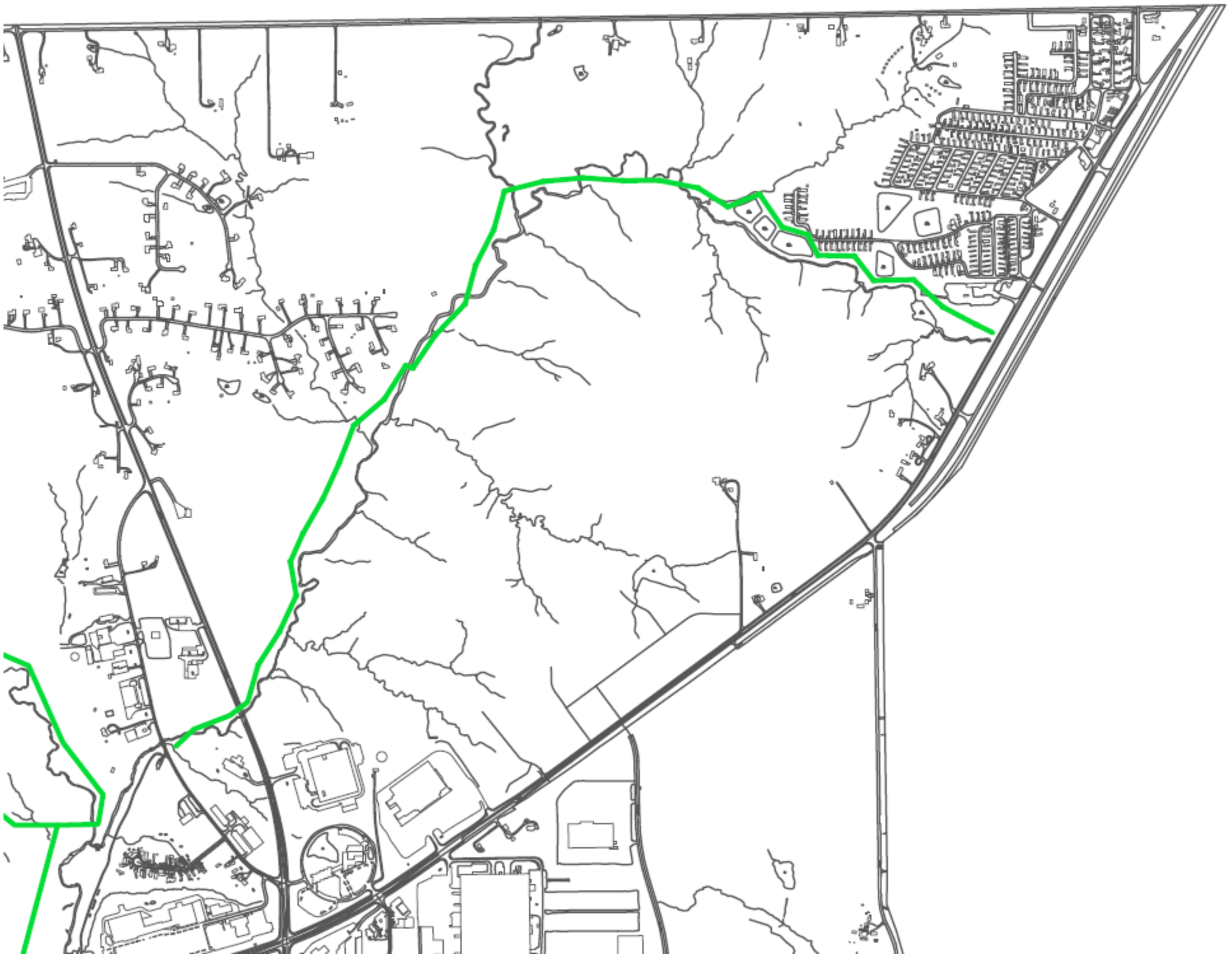


Figure 4-8: The position of the gravity sewer is shown in dark green.

Roadway and Traffic Design

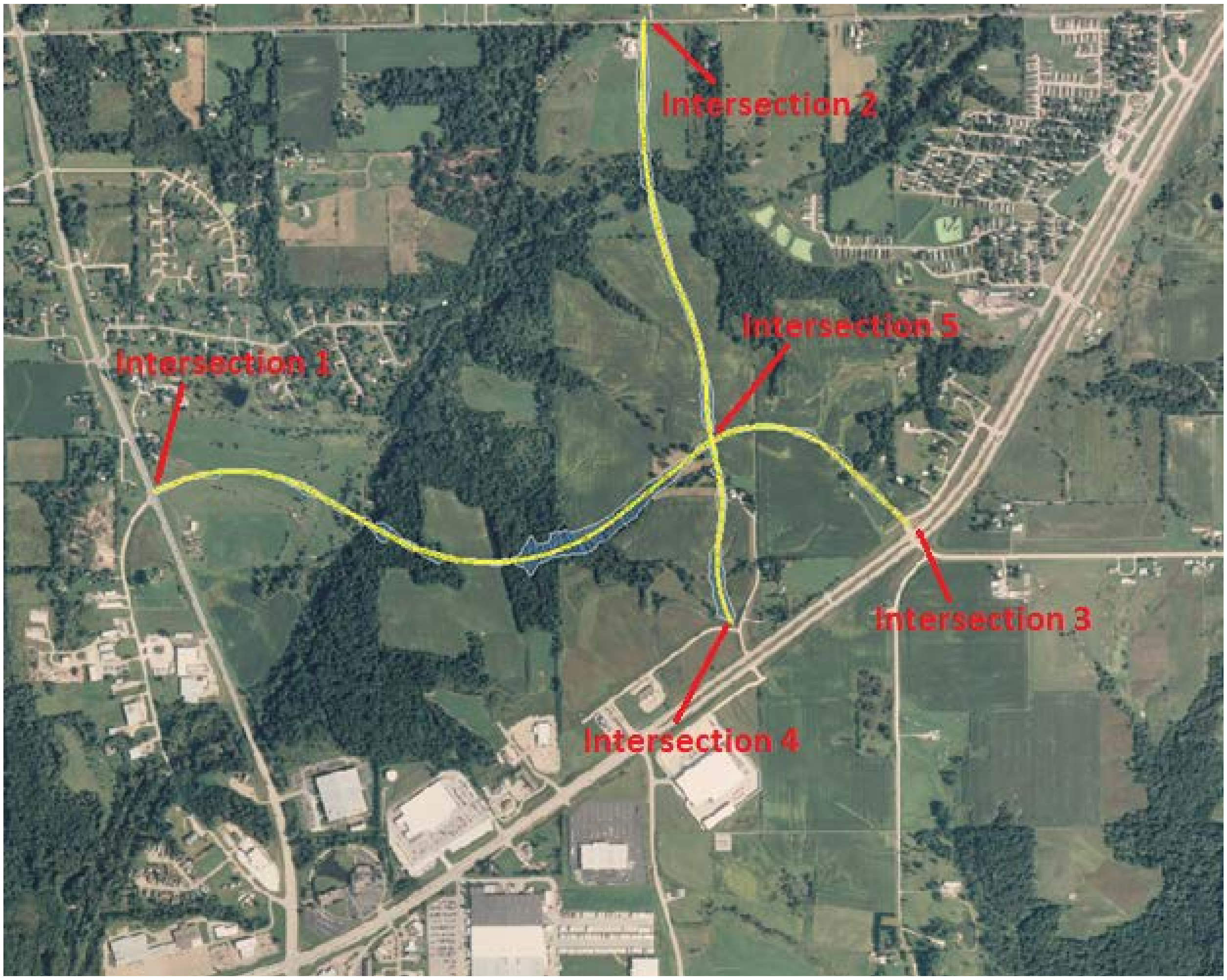


Figure 4-9: Roadway Layout

The final roadway layout can be seen in Figure 4-9. Option 3 from Figure 3-1 was determined to be the most appropriate option for the area. While the curvy layout is naturally longer and consequently more expensive than the linear options, the layout of the route follows the natural contours of the existing ground and therefore requires the least amount of grading along the roadway. This option also provides the added benefit of slowing traffic and making the entire area safer for the general populace.

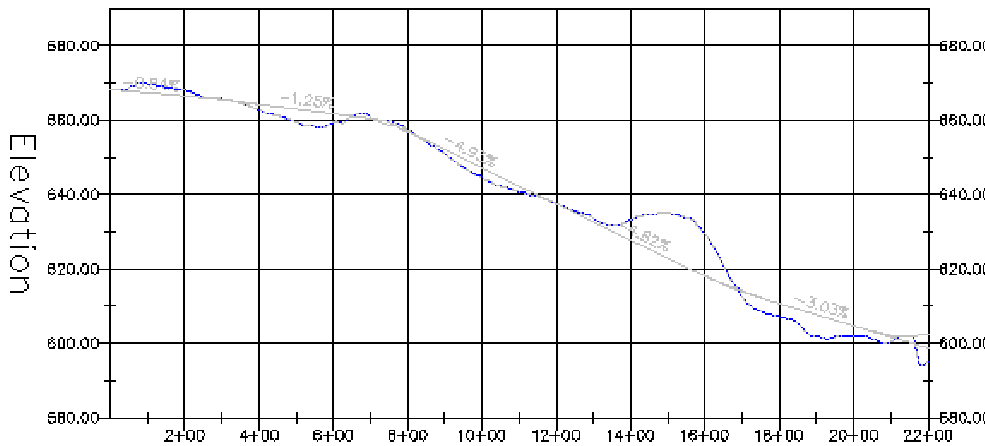


Figure 4-10: First quarter of E-W arterial profile

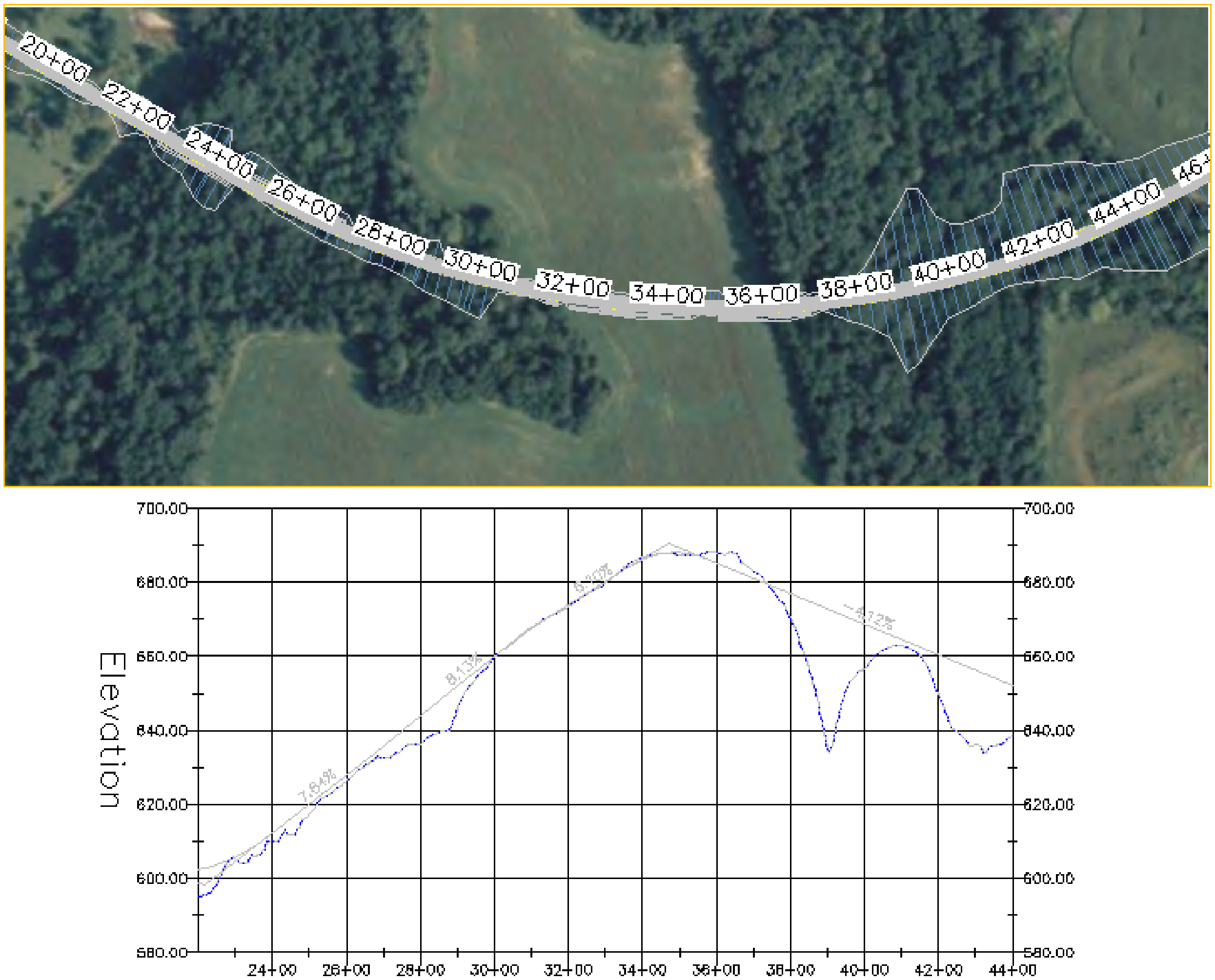
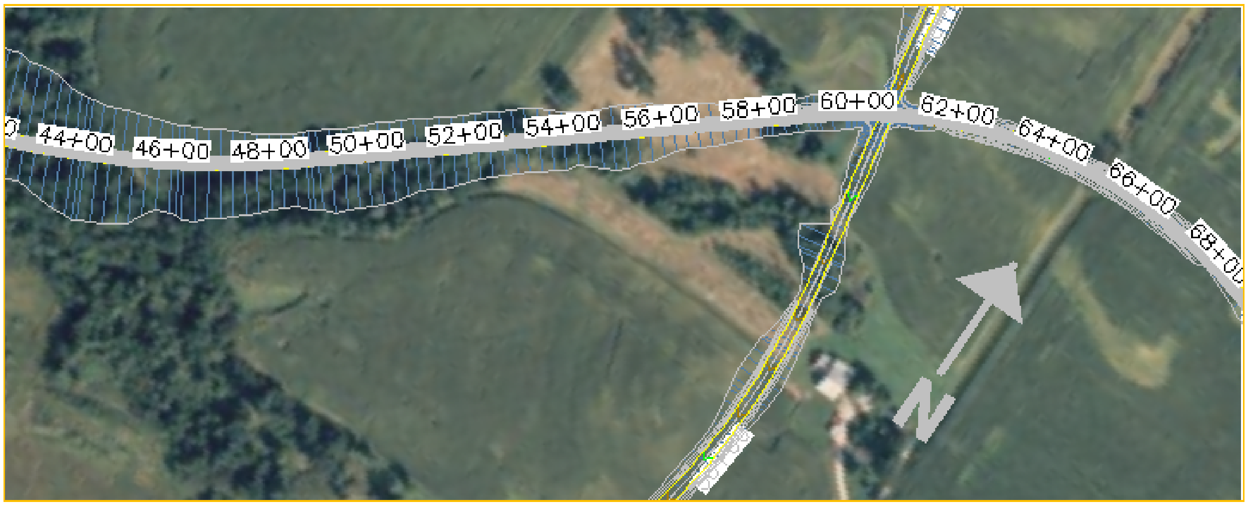


Figure 4-11: Second quarter of E-W arterial profile



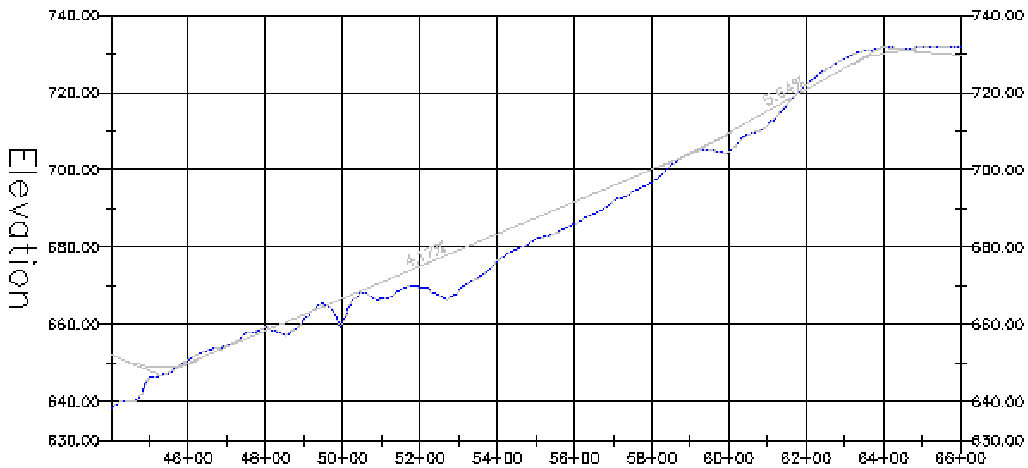


Figure 4-12: Third quarter of E-W arterial profile

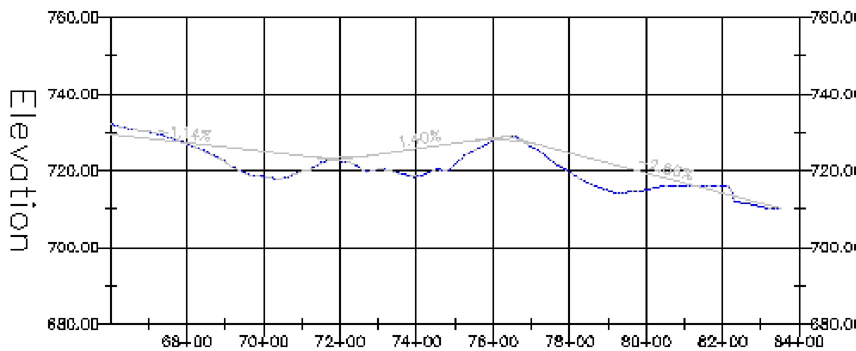


Figure 4-13: Final quarter of E-W arterial profile

Figures 4-9 through 4-13 show the elevations of the E-W arterial at different stations. The slopes of the roads were designed following the recommended guidelines shown in Figure A-1 in Appendix A.

Table 4-1: Intersection Traffic Control Devices

Intersection	Traffic Control Device
1	All-Way Stop
2	All-Way Stop
3	2-Way Stop Along Minor Streets
4	Semi-Actuated Traffic Lights
5	All-Way Stop

The five main intersections created by the two arterials will be controlled by stop signs with the exception of the intersection of the E-W arterial and US 61 as seen in Table 4-1 above. While there were no signal warrants for this intersection at any stage of build out, it was determined it would require a signal for drivers to use safely. The signal will be actuated along the E-W arterial and New Era Road since the traffic along these two roads is too low to warrant a non-actuated signal and the traffic along US 61 is too heavy to safely allow the increased number of left hand turns and through movements generated by development at peak hours.

V. Cost and Construction Estimates

For the pipe system, 15 inch diameter reinforced concrete pipe was selected. The unit price of this pipe is $ 33. The unit length is about 2 ft. The total length of the proposed the route is 14974 ft. Thus, the cost of the pipe system of this project is to be about $250,000.

The total cost estimation for road construction was calculated to be $7.1M. The cost per foot of road is about $500. The N-S arterial road has total length of 6030 ft., which results in a cost of $3.0M. The E-W arterial road is a total length of 8350 ft., resulting in a cost of $4.1M.

The unit cost for the reinforced concrete pipe used for the culverts was estimated from source of a company called "Con Cast Pipe." The total cost of the three proposed culverts was estimated to be $400,000. Considering this is the result of the overdesign 100 year flow rate, the actual cost may be subjected to a lower change.

The length of the sanitary sewer was estimated to be 6300 ft. The unit cost for the sanitary sewer was to be estimated with the project information sheet for Coralville. The unit cost for the sanitary pipe was estimated to be $60 for each 2 ft. Therefore the total cost was estimated to be $190,000.

According to the Iowa DOT Preliminary Bridge Design Manual, the average cost for a three span rolled steel beam bridge is $90/ft^2. The total surface area for the 130 ft. x 50 ft. bridge deck is 6500 ft^2, which brings the total cost estimate of the bridge to $585,000.

The total cost has been approximated to be $8.6M, shown below in Table 5-1.

Table 5-1: Cost Estimate

Storm Sewer	Unit	price/unit	Length (ft)	Cost of pipe system
	2ft	$33	14974	$247,071
Transportation Road Estimation				
E-W	1ft	$500	8350	$4,175,000
N-S	1ft	$500	6030	$3,015,000
Cost of Culvert				
Culvert #2	1ft	$1,328.32	99.2	$131,769.34
Culvert #3	1ft	$1,328.32	110	$146,115.20
Culvert #4	1ft	$1,328.32	99.2	$131,769.34
Sanitary Sewer Cost				
	2ft	$60	6300	$189,000
Bridge Cost	1ft^2	$90	6500	$585,000
Total Cost				$8,620,724.89

VI. Conclusions

With the implementation of the two new arterial roads, the City of Muscatine will experience significant growth in the Northeast Corridor. Other considerations were made to encourage developers to build in this area. Some of the considerations include a stormwater management system, a sanitary sewer, along with other necessary infrastructure.

The roadway design proposed by Hawkeye Engineering features four Mad Creek crossings, 3 of which will be RCP culverts and the fourth will be a slab-girder bridge. The total cost of the project is estimated to be approximately $8.6 million. The positive outcomes as a result of this project will far outweigh the initial cost of the project.

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Appendix A: Transportation Data, Calculations and Analysis

Table A-1: The figure below shows the spreadsheet of the calculations for the traffic flow rate.

		Mobile Home	Residential	Industrial	Office Buildings	Commercial
	Land Area	99.43	473.85	31.44	129.92	86.61
	Land Area*(floor area ratio)	397.73	1895.38	125.75	519.68	346.44
Average Trip Rate per Acre						
Residental-Weekday	General Light Industry-Weekday					
26.45	51.803	10520.44	50134.70	6514.09		
Residental- Weekday- AM-Peak	General Light Industry- Weekday- A M Peak					
2.17	8.002	864.67	4120.56	1006.23		
Residental -Weekday- P M peak	General Light Industry- Weekday- P M Peak					
2.81	8.64	1119	5334	1086		
Residental- Saturday	General Light Industry -Saturday					
30.95	8.729	12309.04	58658.22	1097.65		
Residental -Sunday	General Light Industry-Sunday					
25.82	4.42	10271.06	48946.29	555.80		
Residental -Sunday-Peak hours	General Light Industry-Sunday-Peak					
2.90	0.641	1153.03	5494.71	80.60		
Retail-General Merchandise						
0.4						138.7
General Office Building-Weekday						
8.16					4240.59	
General Office Building- Weekday-AM Peak						
1.5					779.5	
General Office Building-Weekday- P M Peak						
1.38						
					717.16	

CONNECTOR ROAD - DESIGN GUIDE		
Cross-Section Element	**Design Value**	
	Desirable	**Minimum**
Design Speed, mph (a)	40	35
Right of Way, ft	100	90
Access Spacing (c)		
Full Access, ft	600	600
Partial Access, ft	600	300
Travel Lane Width		
Thru Lanes, ft	12	11
Left/Right Turn Lane, ft (d)	12	11
Curb and Gutter Width, ft (e)	2.5	1.5
Median Width, ft	16 (b)	0
Raised Curb at Narrowest Point (Face to Face), ft		4
Two Way Left Turn Lanes, ft	14	12
Trail Width, ft	10	8
Sidewalk Width, ft	5	4
Bike Lane Width, ft	5	4
Vertical Alignment (f)		
Curve Length, ft		120
Crest: k	70	44
Sag: k		64
Maximum Gradient, percent	6	
Minimum Gradient, percent	0.5	
Horizontal Alignment (Radius), ft	1000 or greater	675
Stopping Sight Distance, ft (g)		305
Vertical Clearance, ft		14.5
Clear Zone		
Roadway, ft (h)	10	5.5
Trail, ft	3	2
Object Setback, ft (i)		3
Border Area (ROW, from back of curb), ft	22	14
Bridge Roadway Width, ft	Total lane width + 3 ft clearance each side	
Trail Width, ft	10	8
Sidewalk Width, ft	5	4

a. Design speed should be equal to or greater than posted speed.
b. Width allows for left turn plus a 4' raised median but not positive offset left turns.

c. Access spacing coordination with the multiple property owners will be a key development element.
d. Turn lane widths are to face of curb. No additional curb offset is required.
e. No offset is required to median curb for design speeds less than 45 mph.
f. Based on design speed.
g. Based on design speed.
h. Measured from outside edge of vehicular lane.
i. Measured from back of curb.

Figure A-1: Road Design Guide

Traffic Data

10 year (⅓ build out):

Trip Generation Data:

Table A-2: Trips Generated per Zone Type from Each Quadrant

Quadrant	Zone	Cars Generated
NW	Residential	254
	Commercial	35
	Industrial	0
NE	Residential	172
	Commercial	0
	Industrial	0
SW	Residential	19
	Commercial	24
	Industrial	91
SE	Residential	0
	Commercial	17
	Industrial	0

Table A-3: Trips generated and turning movements onto surrounding roads.

NW

Type	Road	%	Cars
RES	NS Art	80%	203
	180	10%	25
	EW Art	5%	13
	38	5%	13
COM	NS Art	5%	2
	180	0%	0
	EW Art	95%	33
	38	0%	0
IND	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	38	0%	0

NS Art	205	Left	40%	82
		Right	60%	123
180	25	Left	50%	13
		Right	50%	13
EW Art	46	Left	30%	14
		Right	70%	32
38	13	Left	80%	10
		Right	20%	3

NE

Type	Road	%	Cars
RES	NS Art	75%	129
	180	10%	17
	EW Art	5%	9
	61	10%	17
COM	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	61	0%	0
IND	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	61	0%	0

NS Art	129	Left	60%	77
		Right	40%	52
180	17	Left	25%	4
		Right	75%	13
EW Art	9	Left	75%	6
		Right	25%	2
61	17	Left	25%	4
		Right	75%	13

SW

Type	Road	%	Cars
RES	NS Art	0%	0
	61	0%	0
	EW Art	95%	18
	38	5%	1
COM	NS Art	10%	2
	61	0%	0
	EW Art	80%	19
	38	10%	2
IND	NS Art	0%	0
	61	0%	0
	EW Art	20%	18
	38	80%	72

NS Art	2	Left	25%	1
		Right	75%	2
61	0	Left	-	0
		Right	-	0
EW Art	55	Left	50%	28
		Right	50%	28
38	76	Left	60%	45
		Right	40%	30

SE

Type	Road	%	Cars
RES	NS Art	0%	0
	61	0%	0
	EW Art	0%	0
COM	NS Art	33%	6
	61	33%	6
	EW Art	33%	6
IND	NS Art	0%	0
	61	0%	0
	EW Art	0%	0

NS Art	6	Left	75%	4
		Right	25%	1
61	6	Left	25%	1
		Right	75%	4
EW Art	6	Left	50%	3
		Right	50%	3

Turning Movement Data:

Table A-4: Turning Movements for Intersection 1 (US 61 & IA 38)

US 61 & IA 38 — Existing

			L	T	R			
			54	143	79			
L	T	R				L	T	R
105	338	84				138	352	40
			L	T	R			
			57	96	123			

US 61 & IA 38 — Additional

	Incoming	
	45	
Incoming		Incoming
0		0
	Incoming	
	0	

US 61 & IA 38 — Total

			L	T	R			
			63	166	92			
L	T	R				L	T	R
105	338	84				138	352	40
			L	T	R			
			57	96	123			

Table A-5: Turning Movements for Intersection 2 (IA 38 & Park Ave W/EW Arterial)

IA 38 & Park Ave W/EW Arterial — Existing

			L	T	R			
			0	351	4			
L	T	R				L	T	R
5	0	3				0	0	0
			L	T	R			
			1	146	0			

IA 38 & Park Ave W/EW Arterial — Additional

	Incoming	
	10	
Incoming		Incoming
0		60
	Incoming	
	30	

IA 38 & Park Ave W/EW Arterial — Total

			L	T	R			
			3	358	4			
L	T	R				L	T	R
5	2	3				48	6	6
			L	T	R			
			4	155	18			

Table A-6: Turning Movements for Intersection 3 (IA 38 & 180th St.)

IA 38 & 180th Street — Existing

			L	T	R			
			2	355	8			
L	T	R				L	T	R
15	15	30				18	6	6
			L	T	R			
			10	130	6			

IA 38 & 180th Street — Additional

	Incoming	
	0	
Incoming		Incoming
0		13
	Incoming	
	3	

IA 38 & 180th Street — Total

			L	T	R			
			2	355	8			
L	T	R				L	T	R
15	15	30				25	9	9
			L	T	R			
			10	132	7			

Table A-7: Turning Movements for Intersection 4 (NS Arterial & 180th St.)

NS Arterial & 180th Street — Existing

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	0	0				0	0	0
			L	T	R			
			0	0	0			

NS Arterial & 180th Street — Additional

	Incoming 0	
Incoming 13		Incoming 4
	Incoming 134	

NS Arterial & 180th Street — Total

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	9	4				2	2	0
			L	T	R			
			80	0	54			

Table A-8: Turning Movements for Intersection 5 (US 61 & Taylor Ave.)

US 61 & Taylor Ave. — Existing

			L	T	R			
			0	463	11			
L	T	R				L	T	R
8	0	54				0	0	0
			L	T	R			
			19	274	0			

US 61 & Taylor Ave. — Additional

	Incoming 0	
Incoming 13		Incoming 0
	Incoming 4	

US 61 & Taylor Ave. — Total

			L	T	R			
			0	463	11			
L	T	R				L	T	R
12	0	63				0	0	0
			L	T	R			
			20	277	0			

Table A-9: Turning Movements for Intersection 6 (US 61 & EW Arterial)

US 61 & EW Arterial — Existing

			L	T	R			
			2	572	0			
L	T	R				L	T	R
0	1	2				23	1	1
			L	T	R			
			1	326	7			

US 61 & EW Arterial — Additional

	Incoming 6	
Incoming 9		Incoming 0
	Incoming 0	

US 61 & EW Arterial — Total

			L	T	R			
			2	570	8			
L	T	R				L	T	R
2	2	8				23	1	1
			L	T	R			
			1	326	7			

Table A-10: Turning Movements for Intersection 7 (EW Arterial & NS Arterial)

EW Arterial & NS Arterial

Existing

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	0	0				0	0	0
			L	T	R			
			0	0	0			

EW Arterial & NS Arterial

Additional

	Incoming	
	200	
Incoming		Incoming
42		5
	Incoming	
	2	

EW Arterial & NS Arterial

Total

			L	T	R			
			50	100	50			
L	T	R				L	T	R
8	17	17				2	2	2
			L	T	R			
			1	1	1			

Table A-11: Turning Movements for Intersection 8 (US 61 & NS Arterial)

US 61 & NS Arterial

Existing

			L	T	R			
			0	569	3			
L	T	R				L	T	R
0	0	2				0	0	1
			L	T	R			
			0	334	2			

US 61 & NS Arterial

Additional

	Incoming	
	4	
Incoming		Incoming
6		0
	Incoming	
	0	

US 61 & NS Arterial

Total

			L	T	R			
			0	572	4			
L	T	R				L	T	R
0	0	8				0	0	1
			L	T	R			
			0	334	2			

20 year (⅔ Buildout):

Trip Generation Data:

Table A-12: Trips Generated per Zone Type From Each Quadrant

Quadrant	Zone	Cars Generated
NW	Residential	507
	Commercial	71
	Industrial	0
NE	Residential	344
	Commercial	0
	Industrial	0
SW	Residential	38
	Commercial	48
	Industrial	181
SE	Residential	0
	Commercial	35
	Industrial	0

Table A-13: Trips generated and turning movements onto surrounding roads.

NW

Type	Road	%	Cars
RES	NS Art	80%	406
	180	10%	51
	EW Art	5%	25
	38	5%	25
COM	NS Art	5%	4
	180	0%	0
	EW Art	95%	67
	38	0%	0
IND	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	38	0%	0

NS Art	409	Left	40%	164
		Right	60%	246
180	51	Left	50%	25
		Right	50%	25
EW Art	92	Left	30%	65
		Right	70%	28
38	25	Left	80%	20
		Right	20%	5

NE

Type	Road	%	Cars
RES	NS Art	75%	258
	180	10%	34
	EW Art	5%	17
	61	10%	34
COM	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	61	0%	0
IND	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	61	0%	0

NS Art	258	Left	60%	155
		Right	40%	103
180	34	Left	25%	9
		Right	75%	26
EW Art	17	Left	75%	13
		Right	25%	4
61	34	Left	25%	9
		Right	75%	26

SW

Type	Road	%	Cars
RES	NS Art	0%	0
	61	0%	0
	EW Art	95%	36
	38	5%	2
COM	NS Art	10%	5
	61	0%	0
	EW Art	80%	38
	38	10%	5
IND	NS Art	0%	0
	61	0%	0
	EW Art	20%	36
	38	80%	145

NS Art	5	Left	25%	4
		Right	75%	1
61	0	Left	-	0
		Right	-	0
EW Art	110	Left	50%	55
		Right	50%	55
38	151	Left	60%	91
		Right	40%	61

SE

Type	Road	%	Cars
RES	NS Art	0%	0
	61	0%	0
	EW Art	0%	0
COM	NS Art	33%	12
	61	33%	12
	EW Art	33%	12
IND	NS Art	0%	0
	61	0%	0
	EW Art	0%	0

NS Art	12	Left	75%	9
		Right	25%	3
61	12	Left	25%	9
		Right	75%	3
EW Art	12	Left	50%	6
		Right	50%	6

Intersection Data:

Table A-14: Turning Movements for Intersection 1 (US 61 & IA 38)

US 61 & IA 38 — Existing

			L	T	R			
			54	143	79			
L	T	R				L	T	R
105	338	84				138	352	40
			L	T	R			
			57	96	123			

US 61 & IA 38 — Additional

	Incoming	
	90	
Incoming		Incoming
0		0
	Incoming	
	0	

US 61 & IA 38 — Total

			L	T	R			
			72	189	104			
L	T	R				L	T	R
105	338	84				138	352	40
			L	T	R			
			57	96	123			

Table A-15: Turning Movements for Intersection 2 (IA 38 & Park Ave W/EW Arterial)

IA 38 & Park Ave W/EW Arterial

Existing

			L	T	R			
			0	351	4			
L	T	R				L	T	R
5	0	3				0	0	0
			L	T	R			
			1	146	0			

IA 38 & Park Ave W/EW Arterial

Additional

	Incoming	
	20	
Incoming		Incoming
0		120
	Incoming	
	60	

IA 38 & Park Ave W/EW Arterial

Total

			L	T	R			
			5	365	4			
L	T	R				L	T	R
5	2	3				96	12	12
			L	T	R			
			7	164	36			

Table A-16: Turning Movements for Intersection 3 (IA 38 & 180th St.)

IA 38 & 180th Street

Existing

			L	T	R			
			2	355	8			
L	T	R				L	T	R
15	15	30				18	6	6
			L	T	R			
			10	130	6			

IA 38 & 180th Street

Additional

	Incoming	
	0	
Incoming		Incoming
0		26
	Incoming	
	6	

IA 38 & 180th Street

Total

			L	T	R			
			2	355	8			
L	T	R				L	T	R
15	15	30				32	12	12
			L	T	R			
			12	133	8			

Table A-17: Turning Movements for Intersection 4 (NS Arterial & 180th St.)

NS Arterial & 180th Street

Existing

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	0	0				0	0	0
			L	T	R			
			0	0	0			

NS Arterial & 180th Street

Additional

	Incoming	
	0	
Incoming		Incoming
26		8
	Incoming	
	268	

NS Arterial & 180th Street

Total

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	18	8				4	4	0
			L	T	R			
			161	0	107			

Table A-18: Turning Movements for Intersection 5 (US 61 & Taylor Ave.)

US 61 & Taylor Ave.

Existing

			L	T	R			
			0	463	11			
L	T	R				L	T	R
8	0	54				0	0	0
			L	T	R			
			19	274	0			

US 61 & Taylor Ave.

Additional

	Incoming	
	0	
Incoming		Incoming
26		0
	Incoming	
	8	

US 61 & Taylor Ave.

Total

			L	T	R			
			0	463	11			
L	T	R				L	T	R
16	0	72				0	0	0
			L	T	R			
			22	282	0			

Table A-19: Turning Movements for Intersection 6 (US 61 & EW Arterial)

US 61 & EW Arterial

Existing

			L	T	R			
			2	572	0			
L	T	R				L	T	R
0	1	2				23	1	1
			L	T	R			
			1	326	7			

US 61 & EW Arterial

Additional

	Incoming	
	12	
Incoming		Incoming
18		0
	Incoming	
	0	

US 61 & EW Arterial

Total

			L	T	R			
			3	574	10			
L	T	R				L	T	R
5	2	14				23	1	1
			L	T	R			
			1	326	7			

Table A-20: Turning Movements for Intersection 7 (EW Arterial & NS Arterial)

EW Arterial & NS Arterial

Existing

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	0	0				0	0	0
			L	T	R			
			0	0	0			

EW Arterial & NS Arterial

Additional

	Incoming	
	400	
Incoming		Incoming
84		10
	Incoming	
	4	

EW Arterial & NS Arterial

Total

			L	T	R			
			100	200	100			
L	T	R				L	T	R
17	34	34				4	4	4
			L	T	R			
			2	2	2			

Table A-21: Turning Movements for Intersection 8 (US 61 & NS Arterial)

US 61 & NS Arterial

Existing

			L	T	R			
			0	569	3			
L	T	R				L	T	R
0	0	2				0	0	1
			L	T	R			
			0	334	2			

US 61 & NS Arterial

Additional

	Incoming	
	8	
Incoming		Incoming
12		0
	Incoming	
	0	

US 61 & NS Arterial

Total

			L	T	R			
			0	575	5			
L	T	R				L	T	R
0	0	14				0	0	1
			L	T	R			
			0	334	2			

30 year (Full Buildout):

Trip Generation Data:

Table A-22: Trips Generated per Zone Type From Each Quadrant

Quadrant	Zone	Cars Generated
NW	Residential	761
	Commercial	106
	Industrial	0
NE	Residential	517
	Commercial	0
	Industrial	0
SW	Residential	57
	Commercial	72
	Industrial	272
SE	Residential	0
	Commercial	52
	Industrial	0

Table A-23: Trips generated and turning movements onto surrounding roads.

NW

Type	Road	%	Cars
RES	NS Art	80%	609
	180	10%	76
	EW Art	5%	38
	38	5%	38
COM	NS Art	5%	5
	180	0%	0
	EW Art	95%	100
	38	0%	0
IND	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	38	0%	0

Road	Cars	Turn	%	Cars
NS Art	614	Left	40%	246
		Right	60%	368
180	76	Left	50%	38
		Right	50%	38
EW Art	139	Left	30%	97
		Right	70%	42
38	38	Left	80%	30
		Right	20%	8

NE

Type	Road	%	Cars
RES	NS Art	75%	387
	180	10%	52
	EW Art	5%	26
	61	10%	52
COM	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	61	0%	0
IND	NS Art	0%	0
	180	0%	0
	EW Art	0%	0
	61	0%	0

Road	Cars	Turn	%	Cars
NS Art	387	Left	60%	232
		Right	40%	155
180	52	Left	25%	13
		Right	75%	39
EW Art	26	Left	75%	19
		Right	25%	6
61	52	Left	25%	13
		Right	75%	39

SW

Type	Road	%	Cars
RES	NS Art	0%	0
	61	0%	0
	EW Art	95%	54
	38	5%	3
COM	NS Art	10%	7
	61	0%	0
	EW Art	80%	57
	38	10%	7
IND	NS Art	0%	0
	61	0%	0
	EW Art	20%	54
	38	80%	217

Road	Cars	Turn	%	Cars
NS Art	7	Left	25%	5
		Right	75%	2
61	0	Left	-	0
		Right	-	0
EW Art	166	Left	50%	83
		Right	50%	83
38	227	Left	60%	136
		Right	40%	91

SE

Type	Road	%	Cars
RES	NS Art	0%	0
	61	0%	0
	EW Art	0%	0
COM	NS Art	33%	17
	61	33%	17
	EW Art	33%	17
IND	NS Art	0%	0
	61	0%	0
	EW Art	0%	0

Road	Cars	Turn	%	Cars
NS Art	17	Left	75%	13
		Right	25%	4
61	17	Left	25%	13
		Right	75%	4
EW Art	17	Left	50%	9
		Right	50%	9

Intersection Data:

Table A-24: Turning Movements for Intersection 1 (US 61 & IA 38)

US 61 & IA 38								
Existing								
			L	T	R			
			54	143	79			
L	T	R				L	T	R
105	338	84				138	352	40
			L	T	R			
			57	96	123			

US 61 & IA 38		
Additional		
	Incoming	
	135	
Incoming		Incoming
0		0
	Incoming	
	0	

US 61 & IA 38								
Total								
			L	T	R			
			90	235	129			
L	T	R				L	T	R
105	338	84				138	352	40
			L	T	R			
			57	96	123			

Table A-25: Turning Movements for Intersection 2 (IA 38 & Park Ave W/EW Arterial)

IA 38 & Park Ave W/EW Arterial								
Existing								
			L	T	R			
			0	351	4			
L	T	R				L	T	R
5	0	3				0	0	0
			L	T	R			
			1	146	0			

IA 38 & Park Ave W/EW Arterial		
Additional		
	Incoming	
	30	
Incoming		Incoming
0		180
	Incoming	
	90	

IA 38 & Park Ave W/EW Arterial								
Total								
			L	T	R			
			7	372	4			
L	T	R				L	T	R
5	2	3				144	18	18
			L	T	R			
			10	173	54			

Table A-26: Turning Movements for Intersection 3 (IA 38 & 180th St.)

IA 38 & 180th Street								
Existing								
			L	T	R			
			2	355	8			
L	T	R				L	T	R
15	15	30				18	6	6
			L	T	R			
			10	130	6			

IA 38 & 180th Street		
Additional		
	Incoming	
	0	
Incoming		Incoming
0		39
	Incoming	
	9	

IA 38 & 180th Street								
Total								
			L	T	R			
			2	355	8			
L	T	R				L	T	R
15	15	30				39	15	15
			L	T	R			
			13	234	9			

Table A-27: Turning Movements for Intersection 4 (NS Arterial & 180th St.)

NS Arterial & 180th Street

Existing

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	0	0				0	0	0
			L	T	R			
			0	0	0			

NS Arterial & 180th Street

Additional

	Incoming	
	0	
Incoming		Incoming
39		12
	Incoming	
	402	

NS Arterial & 180th Street

Total

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	27	12				6	6	0
			L	T	R			
			241	0	161			

Table A-28: Turning Movements for Intersection 5 (US 61 & Taylor Ave.)

US 61 & Taylor Ave.

Existing

			L	T	R			
			0	463	11			
L	T	R				L	T	R
8	0	54				0	0	0
			L	T	R			
			19	274	0			

US 61 & Taylor Ave.

Additional

	Incoming	
	0	
Incoming		Incoming
39		0
	Incoming	
	12	

US 61 & Taylor Ave.

Total

			L	T	R			
			0	463	11			
L	T	R				L	T	R
20	0	81				0	0	0
			L	T	R			
			24	284	0			

Table A-29: Turning Movements for Intersection 6 (US 61 & EW Arterial)

US 61 & EW Arterial

Existing

			L	T	R			
			2	572	0			
L	T	R				L	T	R
0	1	2				23	1	1
			L	T	R			
			1	326	7			

US 61 & EW Arterial

Additional

	Incoming	
	18	
Incoming		Incoming
27		0
	Incoming	
	0	

US 61 & EW Arterial

Total

			L	T	R			
			5	574	10			
L	T	R				L	T	R
7	4	19				23	1	1
			L	T	R			
			1	326	7			

Table A-30: Turning Movements for Intersection 7 (EW Arterial & NS Arterial)

EW Arterial & NS Arterial

Existing

			L	T	R			
			0	0	0			
L	T	R				L	T	R
0	0	0				0	0	0
			L	T	R			
			0	0	0			

EW Arterial & NS Arterial

Additional

	Incoming	
	600	
Incoming		Incoming
126		15
	Incoming	
	6	

EW Arterial & NS Arterial

Total

			L	T	R			
			150	300	150			
L	T	R				L	T	R
25	50	50				5	5	5
			L	T	R			
			3	3	3			

Table A-31: Turning Movements for Intersection 8 (US 61 & NS Arterial)

US 61 & NS Arterial

Existing

			L	T	R			
			0	569	3			
L	T	R				L	T	R
0	0	2				0	0	1
			L	T	R			
			0	334	2			

US 61 & NS Arterial

Additional

	Incoming	
	12	
Incoming		Incoming
18		0
	Incoming	
	0	

US 61 & NS Arterial

Total

			L	T	R			
			0	578	1			
L	T	R				L	T	R
0	0	20				0	0	1
			L	T	R			
			0	334	2			

Warrant Summary

Intersection warrants for all five created intersections for full (30 year) build out

Warrants Summary

Information			
Analyst	Zach Wilson	Intersection	EW Art & 38
Agency/Co		Jurisdiction	
Date Performed	4/25/2014	Units	U.S. Customary
Project ID		Time Period Analyzed	AM Peak
East/West Street	EW Arterial	North/South Street	38
File Name	EW Art and 38	Major Street	North-South
Project Description			

General				Roadway Network	
Major Street Speed (mph)	35	☐	Population < 10,000	Two Major Routes	☐
Nearest Signal (ft)	4700	☐	Coordinated Signal System	Weekend Count	☐
Crashes (per year)	0	☐	Adequate Trials of Alternatives	5-yr Growth Factor	0

Geometry and Traffic	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N	0	1	0	1	1	0	0	1	0	0	1	0
Lane usage		LTR		L	TR			LTR			LTR	
Vehicle Volume Averages (vph)	0	0	0	12	1	1	0	14	4	0	31	0
Peds (ped/h) / Gaps (gaps/h)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--
Delay (s/veh) / (veh-hr)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--

Warrant 1: Eight-Hour Vehicular Volume	☐
1 A. Minimum Vehicular Volumes (Both major approaches --and-- higher minor approach) --or--	☐
1 B. Interruption of Continuous Traffic (Both major approaches --and-- higher minor approach) --or--	☐
1 80% Vehicular --and-- Interruption Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 2: Four-Hour Vehicular Volume	☐
2 A. Four-Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 3: Peak Hour	☐
3 A. Peak-Hour Conditions (Minor delay --and-- minor volume --and-- total volume) --or--	☐
3 B. Peak- Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 4: Pedestrian Volume	☐
4 A. Pedestrian Volumes (Four hours --or-- one hour) --and--	☐
4 B. Gaps Same Period (Four hours --or-- one hour)	☐
Warrant 5: School Crossing	☐
5. Student Volumes --and--	☐
5. Gaps Same Period	☐
Warrant 6: Coordinated Signal System	☐
6. Degree of Platooning (Predominant direction or both directions)	☐
Warrant 7: Crash Experience	☐
7 A. Adequate trials of alternatives, observance and enforcement failed --and--	☐
7 B. Reported crashes susceptible to correction by signal (12-month period) --and--	☐
7 C. 80% Volumes for Warrants 1A, 1B --or-- 4 are satisfied	☐

Warrant 8: Roadway Network	☐
8 A. Weekday Volume (Peak hour total --and-- projected warrants 1, 2 or 3) --or--	☐
8 B. Weekend Volume (Five hours total)	☐

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Figure A-2: Signal warrants for intersection #1

Warrants Summary			
Information			
Analyst	Zach Wilson	Intersection	NS Art and 180
Agency/Co		Jurisdiction	
Date Performed	4/25/2014	Units	U.S. Customary
Project ID		Time Period Analyzed	AM Peak
East/West Street	180	North/South Street	NS Arterial
File Name	NS Art and 180	Major Street	North-South
Project Description			

General				Roadway Network	
Major Street Speed (mph)	35	☐	Population < 10,000	Two Major Routes	☐
Nearest Signal (ft)	0	☐	Coordinated Signal System	Weekend Count	☐
Crashes (per year)	0	☐	Adequate Trials of Alternatives	5-yr Growth Factor	0

Geometry and Traffic	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N	0	1	0	0	1	0	1	0	1	0	0	0
Lane usage		TR			LT		L		R			
Vehicle Volume Averages (vph)	0	2	1	0	0	0	20	0	13	0	0	0
Peds (ped/h) / Gaps (gaps/h)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--
Delay (s/veh) / (veh-hr)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--

Warrant 1: Eight-Hour Vehicular Volume	☐
1 A. Minimum Vehicular Volumes (Both major approaches --and-- higher minor approach) --or--	☐
1 B. Interruption of Continuous Traffic (Both major approaches --and-- higher minor approach) --or--	☐
1 80% Vehicular --and-- Interruption Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 2: Four-Hour Vehicular Volume	☐
2 A. Four-Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 3: Peak Hour	☐
3 A. Peak-Hour Conditions (Minor delay --and-- minor volume --and-- total volume) --or--	☐
3 B. Peak- Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 4: Pedestrian Volume	☐
4 A. Pedestrian Volumes (Four hours --or-- one hour) --and--	☐
4 B. Gaps Same Period (Four hours --or-- one hour)	☐
Warrant 5: School Crossing	☐
5. Student Volumes --and--	☐
5. Gaps Same Period	☐
Warrant 6: Coordinated Signal System	☐
6. Degree of Platooning (Predominant direction or both directions)	☐
Warrant 7: Crash Experience	☐
7 A. Adequate trials of alternatives, observance and enforcement failed --and--	☐
7 B. Reported crashes susceptible to correction by signal (12-month period) --and--	☐
7 C. 80% Volumes for Warrants 1A, 1B --or-- 4 are satisfied	☐

Warrant 8: Roadway Network	☐
8 A. Weekday Volume (Peak hour total --and-- projected warrants 1, 2 or 3) --or--	☐
8 B. Weekend Volume (Five hours total)	☐

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Figure A-3: Signal warrants for intersection #2

Warrants Summary			
Information			
Analyst	Zach Wilson	Intersection	EW Art & 61
Agency/Co		Jurisdiction	
Date Performed	4/25/2014	Units	U.S. Customary
Project ID		Time Period Analyzed	AM Peak
East/West Street	EW Arterial	North/South Street	38
File Name	EW Art and 38	Major Street	North-South
Project Description			

General				**Roadway Network**	
Major Street Speed (mph)	35	☐	Population < 10,000	Two Major Routes	☐
Nearest Signal (ft)	4700	☐	Coordinated Signal System	Weekend Count	☐
Crashes (per year)	0	☐	Adequate Trials of Alternatives	5-yr Growth Factor	0

Geometry and Traffic	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N	0	1	0	1	1	0	0	1	0	0	1	0
Lane usage		LTR		L	TR			LTR			LTR	
Vehicle Volume Averages (vph)	0	0	0	12	1	1	0	14	4	0	31	0
Peds (ped/h) / Gaps (gaps/h)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--
Delay (s/veh) / (veh-hr)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--

Warrant 1: Eight-Hour Vehicular Volume	☐
1 A. Minimum Vehicular Volumes (Both major approaches --and-- higher minor approach) --or--	☐
1 B. Interruption of Continuous Traffic (Both major approaches --and-- higher minor approach) --or--	☐
1 80% Vehicular --and-- Interruption Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 2: Four-Hour Vehicular Volume	☐
2 A. Four-Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 3: Peak Hour	☐
3 A. Peak-Hour Conditions (Minor delay --and-- minor volume --and-- total volume) --or--	☐
3 B. Peak- Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 4: Pedestrian Volume	☐
4 A. Pedestrian Volumes (Four hours --or-- one hour) --and--	☐
4 B. Gaps Same Period (Four hours --or-- one hour)	☐
Warrant 5: School Crossing	☐
5. Student Volumes --and--	☐
5. Gaps Same Period	☐
Warrant 6: Coordinated Signal System	☐
6. Degree of Platooning (Predominant direction or both directions)	☐
Warrant 7: Crash Experience	☐
7 A. Adequate trials of alternatives, observance and enforcement failed --and--	☐
7 B. Reported crashes susceptible to correction by signal (12-month period) --and--	☐
7 C. 80% Volumes for Warrants 1A, 1B --or-- 4 are satisfied	☐

Warrant 8: Roadway Network	☐
8 A. Weekday Volume (Peak hour total --and-- projected warrants 1, 2 or 3) --or--	☐
8 B. Weekend Volume (Five hours total)	☐

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Figure A-4: Signal warrants for intersection #3

Warrants Summary

Information			
Analyst	Zach Wilson	Intersection	NS Art & 61
Agency/Co		Jurisdiction	
Date Performed	4/25/2014	Units	U.S. Customary
Project ID		Time Period Analyzed	AM Peak
East/West Street	61	North/South Street	NS Arterial
File Name	NS Art and 61	Major Street	North-South

Project Description

General				Roadway Network	
Major Street Speed (mph)	55	☐	Population < 10,000	Two Major Routes	☐
Nearest Signal (ft)	0	☐	Coordinated Signal System	Weekend Count	☐
Crashes (per year)	0	☐	Adequate Trials of Alternatives	5-yr Growth Factor	0

Geometry and Traffic	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N	0	0	1	0	0	1	0	2	1	0	2	0
Lane usage			R			R		T	R		TR	
Vehicle Volume Averages (vph)	0	0	1	0	0	0	0	48	0	0	27	0
Peds (ped/h) / Gaps (gaps/h)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--
Delay (s/veh) / (veh-hr)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--

Warrant 1: Eight-Hour Vehicular Volume	☐
1 A. Minimum Vehicular Volumes (Both major approaches --and-- higher minor approach) --or--	☐
1 B. Interruption of Continuous Traffic (Both major approaches --and-- higher minor approach) --or--	☐
1 80% Vehicular --and-- Interruption Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 2: Four-Hour Vehicular Volume	☐
2 A. Four-Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 3: Peak Hour	☐
3 A. Peak-Hour Conditions (Minor delay --and-- minor volume --and-- total volume) --or--	☐
3 B. Peak- Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 4: Pedestrian Volume	☐
4 A. Pedestrian Volumes (Four hours --or-- one hour) --and--	☐
4 B. Gaps Same Period (Four hours --or-- one hour)	☐
Warrant 5: School Crossing	☐
5. Student Volumes --and--	☐
5. Gaps Same Period	☐
Warrant 6: Coordinated Signal System	☐
6. Degree of Platooning (Predominant direction or both directions)	☐
Warrant 7: Crash Experience	☐
7 A. Adequate trials of alternatives, observance and enforcement failed --and--	☐
7 B. Reported crashes susceptible to correction by signal (12-month period) --and--	☐
7 C. 80% Volumes for Warrants 1A, 1B --or-- 4 are satisfied	☐

Warrant 8: Roadway Network	☐
8 A. Weekday Volume (Peak hour total --and-- projected warrants 1, 2 or 3) --or--	☐
8 B. Weekend Volume (Five hours total)	☐

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Figure A-5: Signal warrants for intersection #4

Warrants Summary

Information			
Analyst	Zach Wilson	Intersection	EW Art & NS Art
Agency/Co		Jurisdiction	
Date Performed	4/25/2014	Units	U.S. Customary
Project ID		Time Period Analyzed	AM Peak
East/West Street	EW Arterial	North/South Street	NS Arterial
File Name	EW Art and NS Art	Major Street	East-West
Project Description			

General				Roadway Network	
Major Street Speed (mph)	35	☐	Population < 10,000	Two Major Routes	☑
Nearest Signal (ft)	0	☐	Coordinated Signal System	Weekend Count	☐
Crashes (per year)	0	☐	Adequate Trials of Alternatives	5-yr Growth Factor	0

Geometry and Traffic	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Number of lanes, N	1	1	0	1	1	0	1	1	0	1	1	0
Lane usage	L	TR		L	TR		L	TR		L	TR	
Vehicle Volume Averages (vph)	2	5	5	0	0	0	16	33	16	0	0	0
Peds (ped/h) / Gaps (gaps/h)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--
Delay (s/veh) / (veh-hr)	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--	--	0 / 0	--

Warrant 1: Eight-Hour Vehicular Volume	☐
1 A. Minimum Vehicular Volumes (Both major approaches --and-- higher minor approach) --or--	☐
1 B. Interruption of Continuous Traffic (Both major approaches --and-- higher minor approach) --or--	☐
1 80% Vehicular --and-- Interruption Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 2: Four-Hour Vehicular Volume	☐
2 A. Four-Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 3: Peak Hour	☐
3 A. Peak-Hour Conditions (Minor delay --and-- minor volume --and-- total volume) --or--	☐
3 B. Peak- Hour Vehicular Volumes (Both major approaches --and-- higher minor approach)	☐
Warrant 4: Pedestrian Volume	☐
4 A. Pedestrian Volumes (Four hours --or-- one hour) --and--	☐
4 B. Gaps Same Period (Four hours --or-- one hour)	☐
Warrant 5: School Crossing	☐
5. Student Volumes --and--	☐
5. Gaps Same Period	☐
Warrant 6: Coordinated Signal System	☐
6. Degree of Platooning (Predominant direction or both directions)	☐
Warrant 7: Crash Experience	☐
7 A. Adequate trials of alternatives, observance and enforcement failed --and--	☐
7 B. Reported crashes susceptible to correction by signal (12-month period) --and--	☐
7 C. 80% Volumes for Warrants 1A, 1B --or-- 4 are satisfied	☐

Warrant 8: Roadway Network	☐
8 A. Weekday Volume (Peak hour total --and-- projected warrants 1, 2 or 3) --or--	☐
8 B. Weekend Volume (Five hours total)	☐

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Figure A-6: Signal warrants for intersection #5

Appendix B: Bridge Load Analysis Results and Strength Calculations

Figure B-1: Bending moment (kip-ft) in the slab at mid-span vs. vehicle load position from left end (position 1 corresponds to the starting point of x = 0 ft)

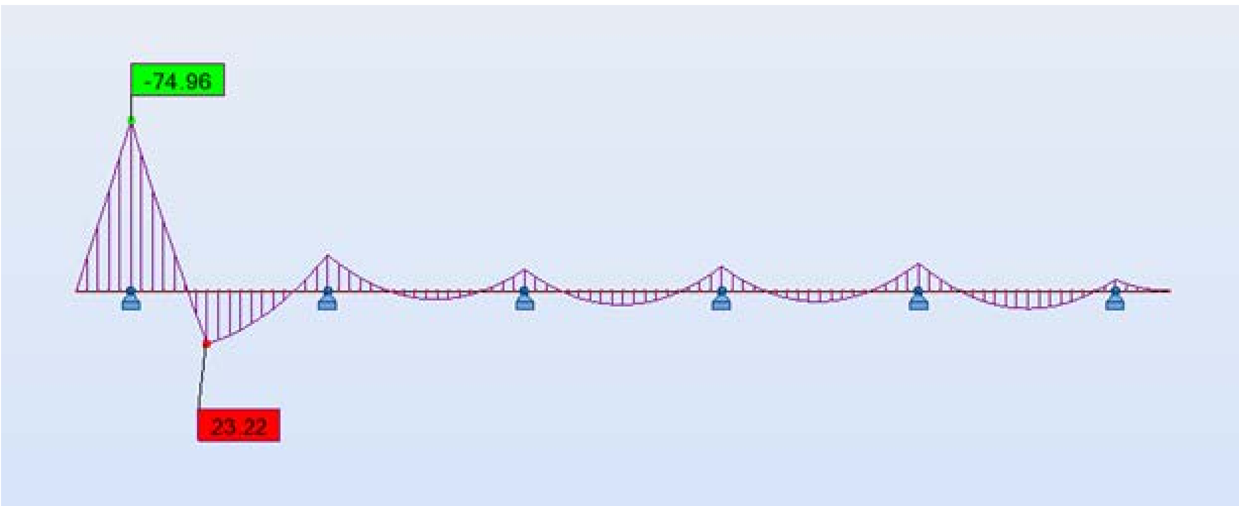


Figure B-2: Bending moment (kip-ft) in the slab at the first girder vs. vehicle load position from left end (position 1 corresponds to the starting point of x = 0 ft)

Figure B-3: Shear force (kips) in the slab at the first interior girder vs. vehicle load position from left end (position 1 corresponds to the starting point of x = 0 ft)

Slab Design (6 girders)

Design Data

In[6]:= `S = 9;  P = 16;`

Strip width for positive moment

In[7]:= `Swpos = (26 + 6.6 * S) / 12`

Out[7]= 7.11667

Required positive bending moment strength

In[17]:= `Mpos = 38;`
`Mpos / Swpos kipft / ft`

Out[18]= $\frac{5.33958\,\text{kipft}}{\text{ft}}$

Strip width for negative moment

In[10]:= `Swneg = (48 + 3 * S) / 12 // N`

Out[10]= 6.25

Required negative bending moment strength

In[21]:= `Mneg = 75;`
`Mneg / Swneg kipft / ft`

Out[22]= $\frac{12.\,\text{kipft}}{\text{ft}}$

Required Shear Strength

In[24]:= `Vu = 45 / 6 kip / ft // N`

Out[24]= $\frac{7.5\,\text{kip}}{\text{ft}}$

Shear Strength of 16 in slab

In[26]:= `λ = 1; d = 16 - 2; bw = 12; fcp = 4000;`

Design shear resisitance of 16 in concrete slab

In[27]:= `ϕVc = 0.75 * 2 * λ * Sqrt[fcp] * bw * d / 1000 // N`

Out[27]= 15.9379

Shear strength of concrete must be twice the applied shear to not require shear reinforcement.

Moment Capacity (#8 bar spaced at 12 in)

```
In[45]:= b = 12; h = 16; cover = 2; d = h - cover;
         As = 0.79; fcp = 4000; wc = 150; fy = 60000;
         Es = 29000000;
         Ec =
           If[fcp ≤ 6000, 33 * wc^1.5 * fcp^0.5, (wc / 145)^1.5 * (40000 * fcp^0.5 + 1.0 * 10^6)];
         Clear[a, c];
         ecu = 0.003; es = ecu * (d - c) / c;
         B1 = If[fcp < 4000, 0.85, Max[0.65, 0.85 - 0.05 * ((fcp - 4000) / 1000)]];
         c = a / B1;
         Fs = fs * As; fs = Min[Es * es, fy];
         Fc = 0.85 * fcp * b * a;
         sol = FindRoot[Fs == Fc, {a, d / 2}];
         a = sol[[1, 2]];
         Mn = Fs * (d - a / 2);
         et = es;
         ey = fy / Es;
         phi = If[et ≤ ey, 0.65, If[et ≥ 0.005, 0.9, 0.65 + (et - ey) * (250 / 3)]];
```

Design moment resistance for 16 in. concrete slab with #8 bars spaced at 12" apart

```
In[51]:= Mr = (Mn * phi) / (12 * 1000)
Out[51]= 47.705
```

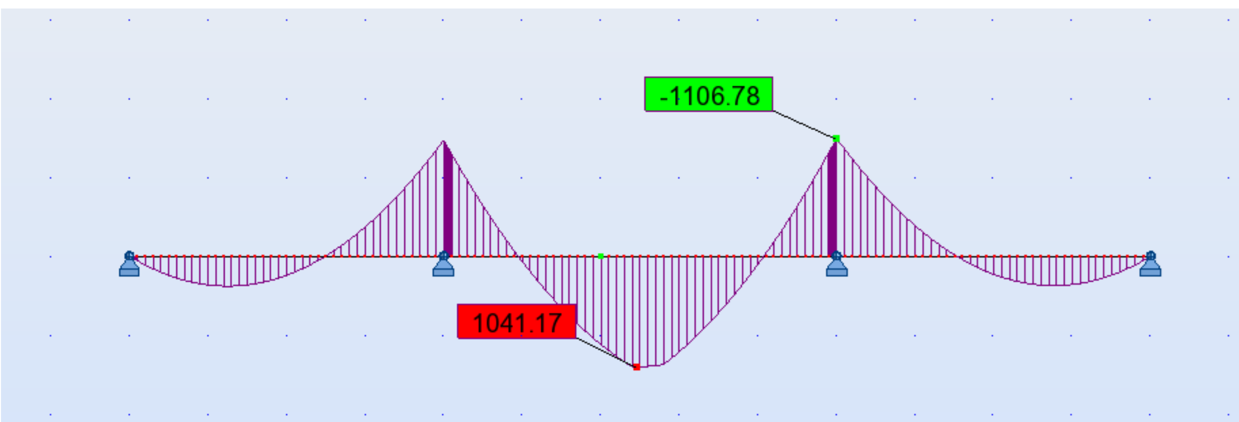


Figure B-4: Bending moment diagram from load case B applied at a distance of 64' from the left end

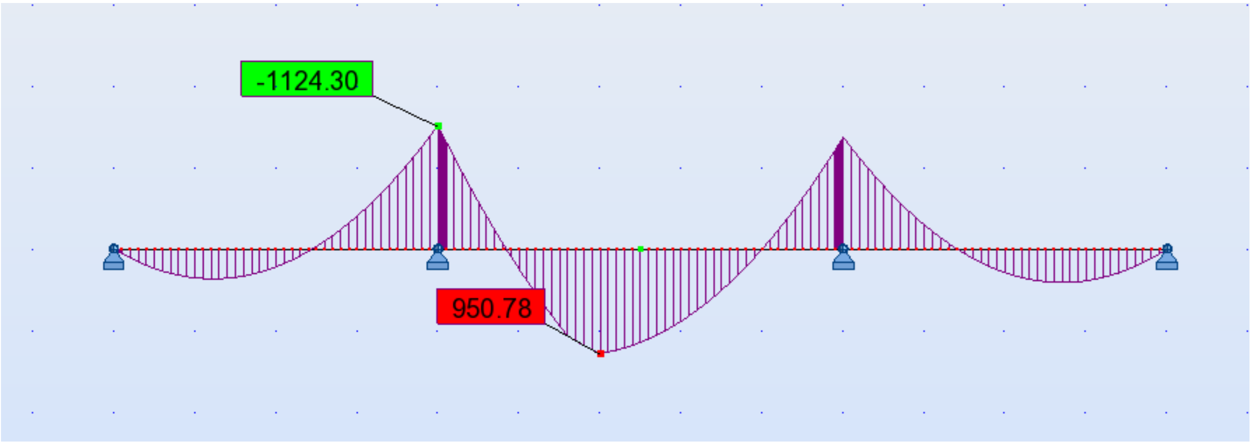


Figure B-5: Bending moment diagram from load case B applied at a distance of 56' from the left end

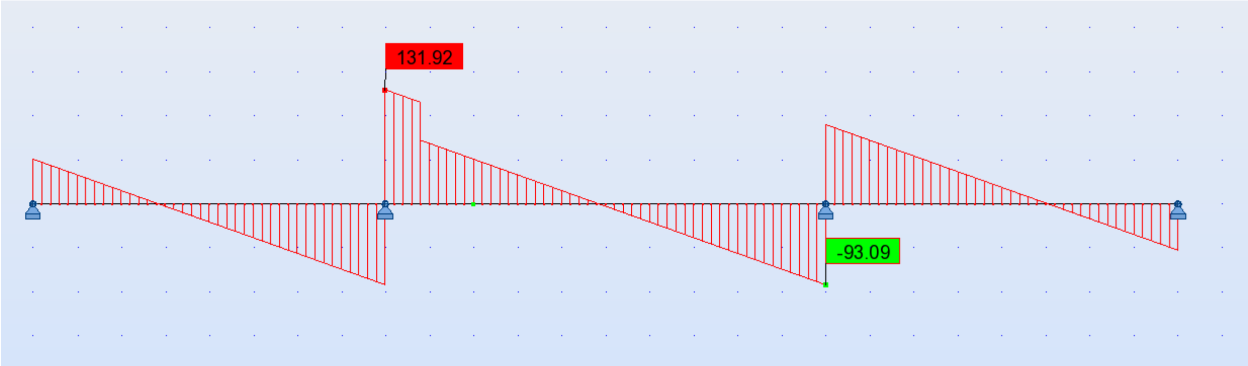


Figure B-6: Shear force diagram from load case B applied at a distance of 40' from the left end

Compute necessary plastic section modulus assuming fully plastic section

In[86]:=
```
Fy = 50; Clear[Zx];
ϕMn = 0.9 * Zx * Fy / 12;
FindRoot[ϕMn == 1120, {Zx, 500}]
```

Out[88]= {Zx → 298.667}

Bending moment strength for a W21x122 section

Plastic Yield Strength

In[89]:=
```
Fy = 50; Zx = 307;
Mn1 = Zx * Fy / 12 // N
```

Out[90]= 1279.17

LTB strength with unbraced length of 50 ft

In[109]:=
```
Jc = 8.98; rts = 3.4; ho = 20.7; Sx = 273; ry = 2.92; Es = 29 000;
```

In[118]:=
```
Lb = 50 * 12;
Lp = 1.76 * ry * Sqrt[Es / Fy];
Lr = 1.95 * rts * Es / (0.7 * Fy) *
   +Sqrt[Jc / (Sx * ho) + Sqrt[(Jc / (Sx * ho)) ^2 + 6.76 * (0.7 * Fy / Es) ^2]];
```

Lateral torsional buckling factor for 50 ft span between columns

In[103]:=
```
Mmax = 1122; Ma = 424; Mb = 814; Mc = 225;
Cb = 12.5 * Mmax / (2.5 * Mmax + 3 * Ma + 4 * Mb + 3 * Mc)
```

Out[104]= 1.75137

In[110]:=
```
Fcr = Cb * Pi^2 * Es / (Lb / rts) ^2 * Sqrt[1 + 0.078 * Jc / (Sx * ho) * (Lb / rts) ^2];
Mn2 = Fcr * Sx / 12
```

Out[111]= 807.29

Positive bending moment strength

In[112]:=
```
ϕMn = 0.9 * Mn1
```

Out[112]= 1151.25

Negative bending moment strength

In[113]:=
```
ϕMn = 0.9 * Min[Mn1, Mn2]
```

Out[113]= 726.561

Negative bending moment strength is not enough, provide lateral bracing to improve Lateral Torsional Buckling Strength

LTB (Lb=10 ft)

```
Lb = 10 * 12
```

120

In[121]:= Lp

Out[121]= 123.768

Lb<Lp. LTB does not apply

Positive and negative bending moment strength

```
ϕMn = 0.9 * Mn1
```

1151.25

This strength is greater than the applied maximum of 1124 kip-ft

Shear Strength

```
In[122]:= Es = 29000; tw = 0.6; d = 21.7; htw = 31.3
Out[122]= 31.3

In[123]:= 1.1 * Sqrt[5 * Es / Fy]
Out[123]= 59.2368

In[124]:= Cv = 1; Aw = tw * d;
          ϕVn = 0.9 * 0.6 * Fy * Aw * Cv
Out[125]= 351.54
```

This strength is greater than the applied shear of 132 kips

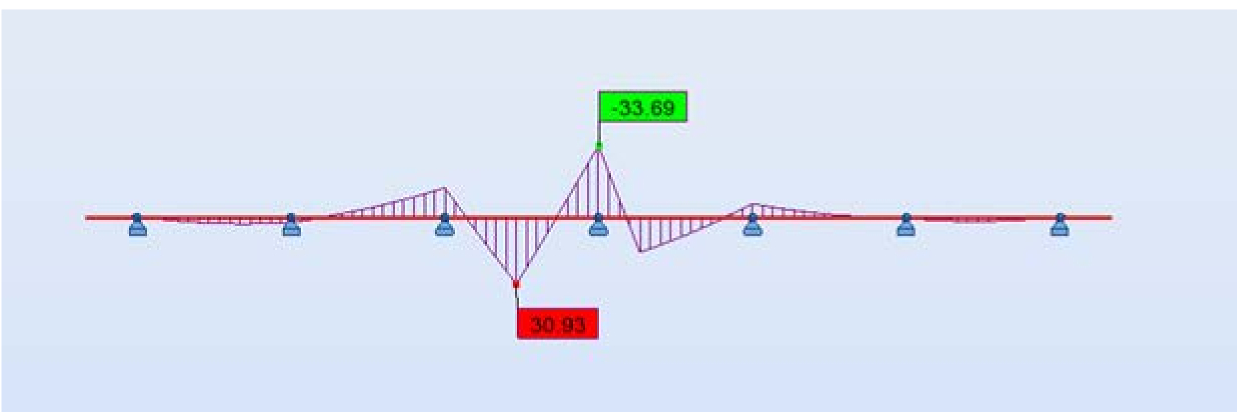


Figure B-7: Bending moment diagram for the slab with the vehicle load placed at a distance of 21' from

Figure B-8: Bending moment diagram for the slab with the vehicle load placed right at the left end

Figure B-9: Shear force diagram for the slab with the vehicle load placed at a distance of 11' from the left end

Slab Design (7 girders)

Design Data

In[6]:=
```
S = 9;  P = 16;
```

Strip width for positive moment

In[7]:=
```
Swpos = (26 + 6.6 * S) / 12
```

Out[7]= 7.11667

Required positive bending moment strength

In[62]:=
```
Mpos = 31;
Mpos / Swpos kipft / ft
```

Out[63]= $\frac{4.35597\,\text{kipft}}{\text{ft}}$

Strip width for negative moment

In[10]:=
```
Swneg = (48 + 3 * S) / 12 // N
```

Out[10]= 6.25

Required negative bending moment strength

In[64]:=
```
Mneg = 71;
Mneg / Swneg kipft / ft
```

Out[65]= $\frac{11.36\,\text{kipft}}{\text{ft}}$

Required Shear Strength

In[66]:=
```
Vu = 31 / 6 kip / ft // N
```

Out[66]= $\frac{5.16667\,\text{kip}}{\text{ft}}$

Shear Strength of 12 in slab

In[67]:=
```
λ = 1; d = 12 - 2; bw = 12; fcp = 4000;
```

Design shear resisitance of 16 in concrete slab

In[68]:=
```
ϕVc = 0.75 * 2 * λ * Sqrt[fcp] * bw * d / 1000 // N
```

Out[68]= 11.3842

Shear strength of concrete must be twice the applied shear to not require shear reinforcement.

Moment Capacity (#8 bar spaced at 12 in)

In[69]:=
```
b = 12; h = 12; cover = 2; d = h - cover;
As = 0.79; fcp = 4000; wc = 150; fy = 60 000;
Es = 29 000 000;
Ec =
  If[fcp ≤ 6000, 33 * wc^1.5 * fcp^0.5, (wc / 145) ^1.5 * (40 000 * fcp^0.5 + 1.0 * 10^6)];
Clear[a, c];
ecu = 0.003; es = ecu * (d - c) / c;
B1 = If[fcp < 4000, 0.85, Max[0.65, 0.85 - 0.05 * ((fcp - 4000) / 1000)]];
c = a / B1;
Fs = fs * As; fs = Min[Es * es, fy];
Fc = 0.85 * fcp * b * a;
sol = FindRoot[Fs == Fc, {a, d / 2}];
a = sol[[1, 2]];
Mn = Fs * (d - a / 2);
et = es;
ey = fy / Es;
phi = If[et ≤ ey, 0.65, If[et ≥ 0.005, 0.9, 0.65 + (et - ey) * (250 / 3)]];
```

Design moment resistance for 12 in. concrete slab with #8 bars spaced at 12" apart

In[85]:=
```
Mr = (Mn * phi) / (12 * 1000)
```

Out[85]= 33.485

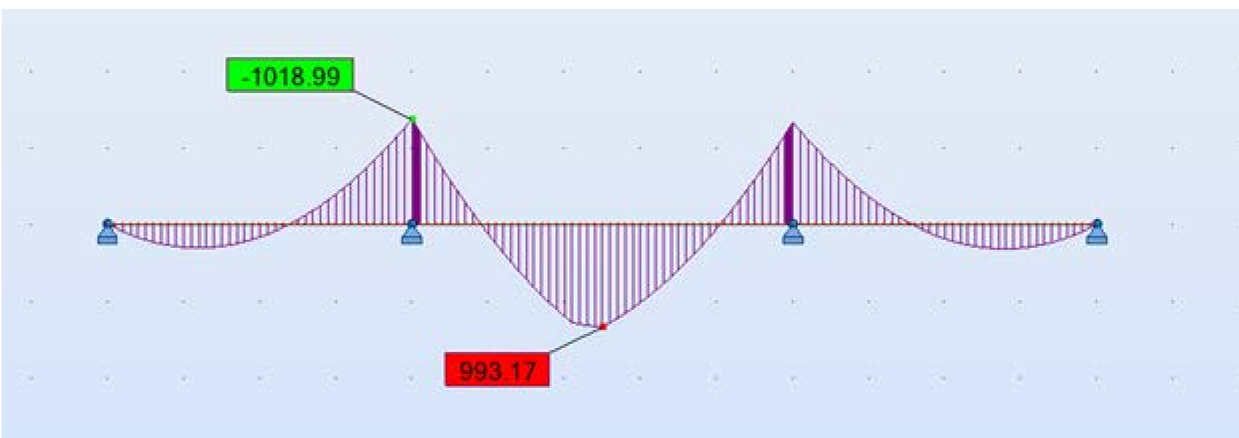


Figure B-10: Bending moment diagram from load case B applied at a distance of 65' from the left end

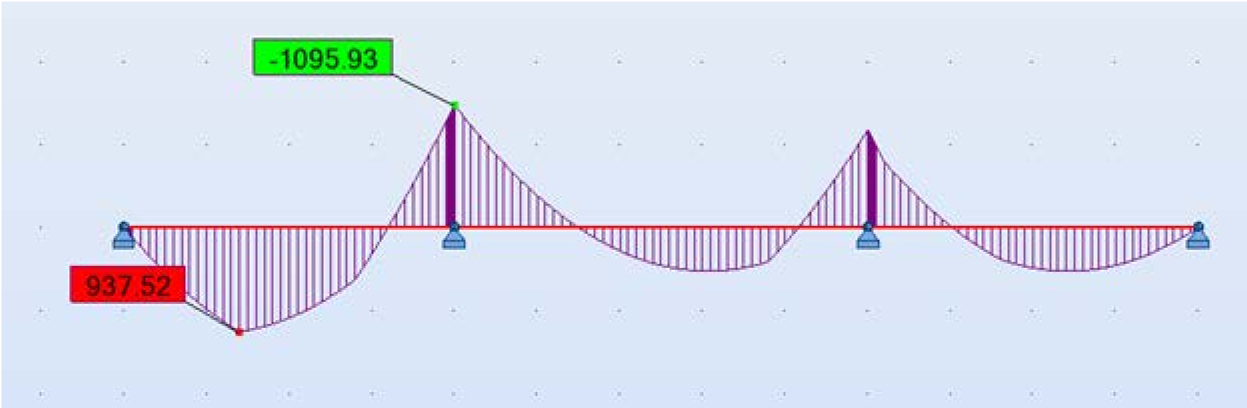


Figure B-11: Bending moment diagram from load case C applied at a distance of 0’ from the left end

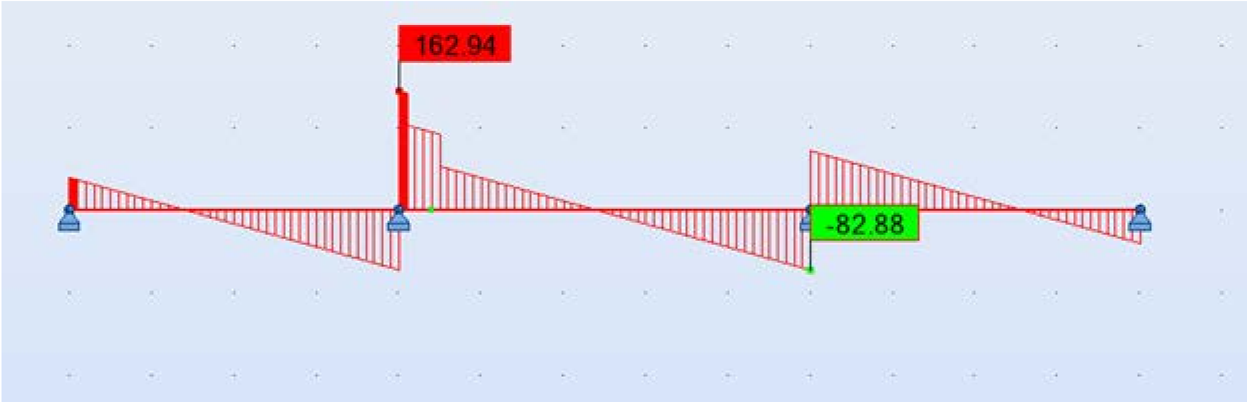


Figure B-12: Shear force diagram from load case B applied at a distance of 41’ from the left end

Appendix C Culvert Design Process

Table C-1: Runoff calculation for each culvert location

Runoff from Contribute Area of Subcatchment (cfs)						
	1/2 x A2	**A3**	**A4**	**A5**	**A6**	**Total**
Q1	336.47	709.46	247.92	193.49	470.12	1957.46
	7/8 x A3	**A4**				**Total**
Q2	472.97	247.92				720.89
	1/2 x A2					**Total**
Q3	336.47					336.47
	1/3 x A2					**Total**
Q4	224.31					224.31

Table C-2: Design parameters for culvert 1

Calculated D	22.33 ft		
Integer D	25 ft	**Q**	1957.5 cfs
L (ft)	75.2 ft	**R = D/4**	6.25 ft
n (Table 7.3) good joints small walls	0.013	**ke (Table 7.4) projecting from fill socket end (groove end)**	0.2
v	5 ft/s	**g**	32.2 ft/s^2
height difference between inlet and outlet			**0.752 ft**

A=Q/v---------Eq. C1
R=D/4---------Eq. C2

Table C-3: Calculation to confirm culvert flow type for Culvert 1

Assume Type 2				
Δh = H-D	1.245 ft		L = 75.2 ft	
H	26.245 ft		10xD = 250 ft	
H/D	1.050		L<10D hydraulically short	
Assume Type 3			**Inlet Shape:** circular concrete groove end w/ headwall (Table 7.2)	
Fr	0.176			
H/D	0.758		Cb = 1	Cc = 0.729
			Not Applicable for Type 3	

$$\Delta h = \frac{n^2 v^2 L}{R^{\frac{4}{3}}} + ke\ \frac{V^2}{2g} + \frac{V^2}{2g}$$ ---------Eq. C3

$$Fr = \frac{V}{\sqrt{g\ D}}$$ ------------Eq. C4

$$\frac{H}{D} = \frac{1}{2(CbCc)^2} Fr^2 Cc$$ ---------Eq. C5

Table C-4: Design parameters for culvert 2

Calculated D	13.55 ft		
Integer D	15 ft	**Q**	720.89 cfs
L (ft)	75.2 ft	**R = D/4**	3.75 ft
n (Table 7.3) good joints small walls	0.013	**ke (Table 7.4) projecting from fill socket end (groove end)**	0.2
v	5 ft/s	**g**	32.2 ft/s^2
height difference between inlet and outlet			**0.752 ft**

Table C-5: Calculation to confirm culvert flow type for Culvert 2

Assume Type 2				
Δh = H-D	1.272 ft		L = 75.2 ft	
H	16.272 ft		10xD = 150 ft	
H/D	1.080		L<10D hydraulically short	
Assume Type 3			**Inlet Shape:** circular concrete groove end w/ headwall (Table 7.2)	
Fr	0.228			
H/D	0.778		Cb = 1	Cc = 0.729
			Not Applicable for Type 3	

Table C-6: Design parameters for culvert 3

Calculated D	9.26 ft		
Integer D	10 ft	**Q**	336.47 cfs
L (ft)	75.2 ft	**R = D/4**	2.5 ft
n (Table 7.3) good joints small walls	0.013	**ke (Table 7.4) projecting from fill socket end (groove end)**	0.2
v	5 ft/s	**g**	32.2 ft/s^2
height difference between inlet and outlet			**0.752 ft**

Table C-7: Calculation to confirm culvert flow type for Culvert 3

Assume Type 2				
Δh = H-D	1.311 ft		L = 75.2 ft	
H	11.311 ft		10xD = 100 ft	
H/D	1.131		L<10D hydraulically short	
Assume Type 3			**Inlet Shape:** circular concrete groove end w/ headwall (Table 7.2)	
Fr	0.279			
H/D	0.802		Cb = 1	Cc = 0.729
			Not Applicable for Type 3	

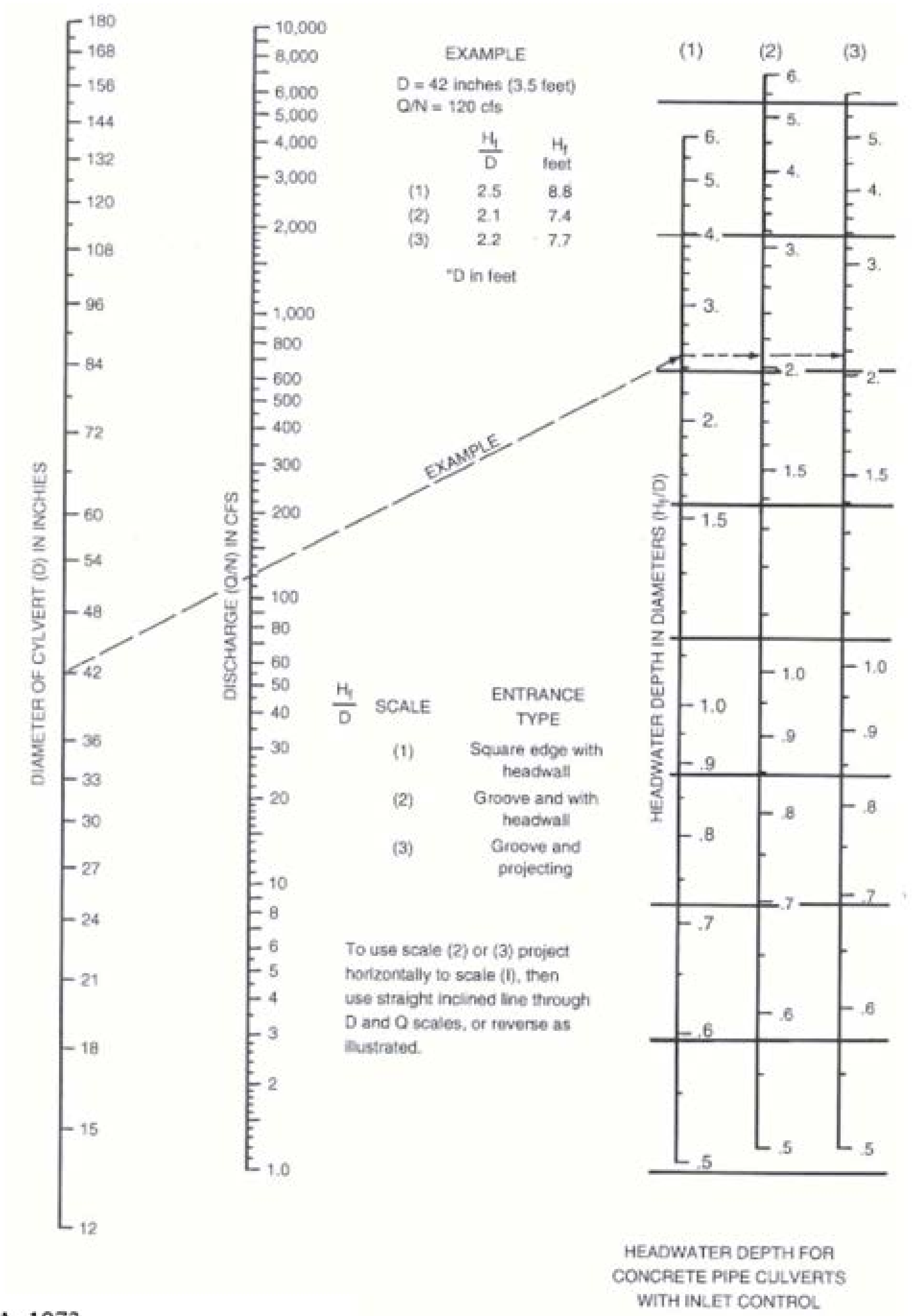
Table C-8: Design parameters for culvert 4

Integer D	8 ft	**Q**	224.32 cfs
L (ft)	75.2 ft	**R = D/4**	2 ft
n (Table 7.3) good joints small walls	0.013	**ke (Table 7.4) projecting from fill socket end (groove end)**	0.2
v	5 ft/s	**g**	32.2 ft/s^2
height difference between inlet and outlet			**0.752 ft**

Table C-9: Calculation to confirm culvert flow type for Culvert 4

Assume Type 2				
Δh = H-D	1.344 ft		L = 75.2 ft	
H	9.344 ft		10xD = 80 ft	
H/D	1.168		L<10D hydraulically short	
Assume Type 3			**Inlet Shape:** circular concrete groove end w/ headwall (Table 7.2)	
Fr	0.312			
H/D	0.820		Cb = 1	Cc = 0.729
			Not Applicable for Type 3	

Figure 5: Inlet Control Nomograph



FHWA, 1973.

Figure C-1: Inlet control Nomograph

Appendix D Curb Inlet & Swale Sample Calculation

Curb Inlet Sample Calculation

$$S_L = 4\%$$

$$S_T \ = 2\%$$

$$n = 0.013$$

$$Q = 0.291\ cfs$$

$$Z = \frac{1}{S_T} = 50$$

$$\frac{Z}{n} = \frac{50}{0.013} = 3846$$

$$According\ to\ figure\ of\ nomograph\ for\ capacity\ of\ gutter, d = 0.054ft = 0.65\ in$$

$$T = 50 \times 0.65 = 2.7\ ft$$

$$Accroding\ to\ intake\ standard, use\ Curb - Grate\ SW - 501$$

$$According\ to\ the\ figure\ of\ \text{"k" values for grate inlet}, k = 23.9$$

$$Reduction\ factor\ for\ standard\ inake\ on\ continuous\ gradeis\ 90\%$$

$$Q_I = k\ \left(d^{\frac{5}{3}}\right)\left(R_f\right) = 23.9 \times \left(0.054^{\frac{5}{3}}\right)(0.9) = 0.166\ cfs$$

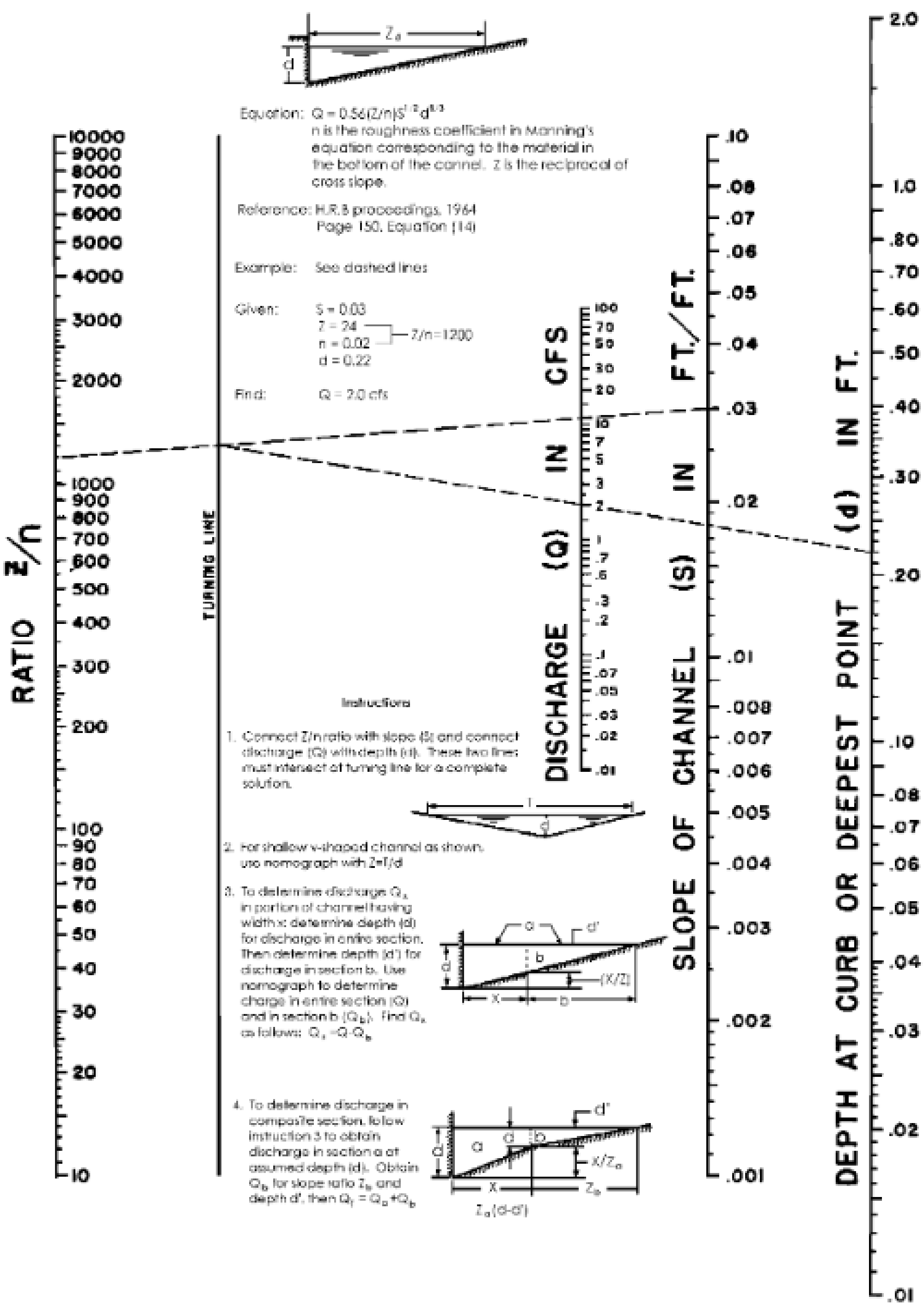


Figure D-1: Nomograph for Capacity of the Gutter for Straight Crown

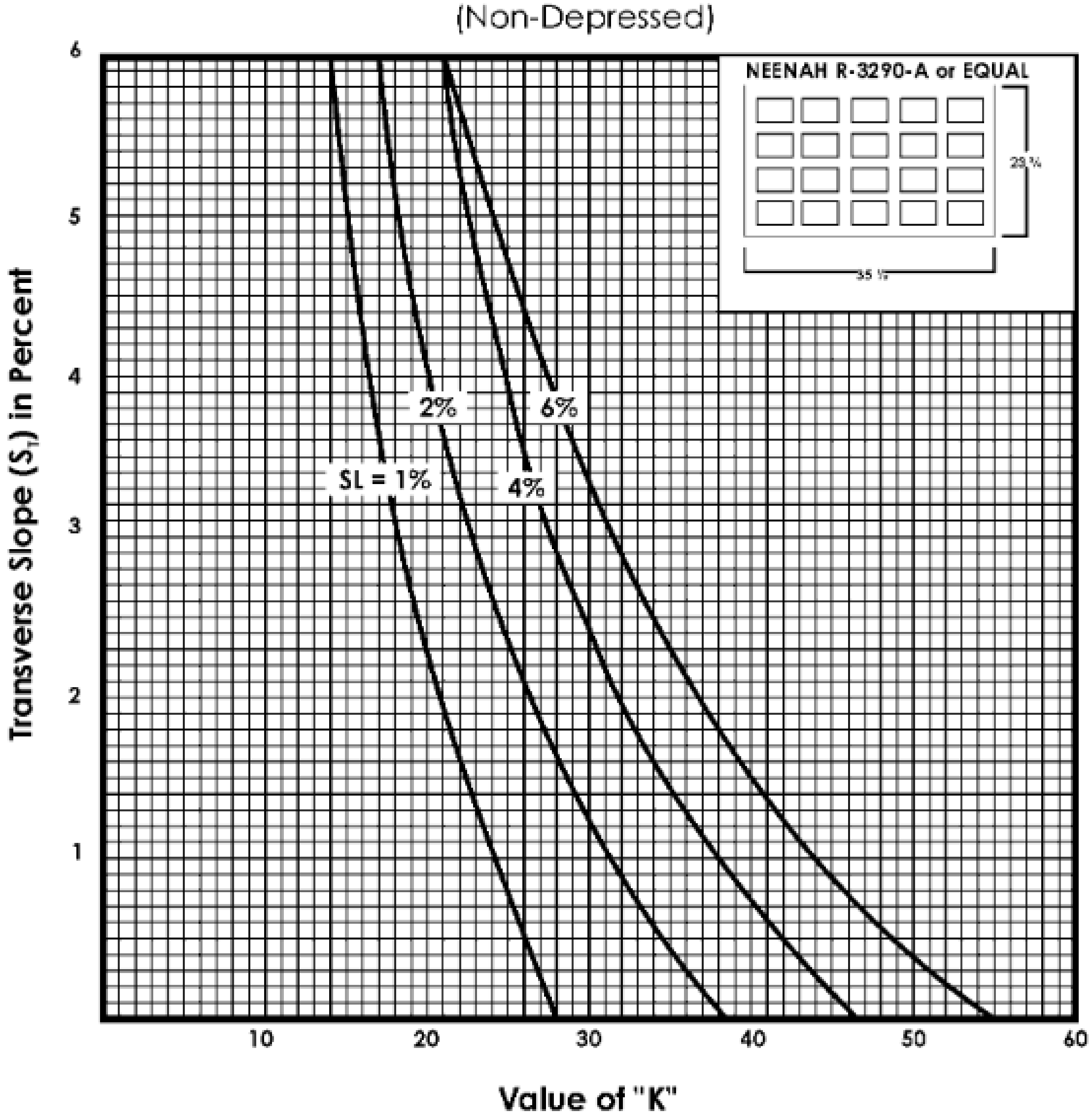


Figure D-2: "K"Values for Driveway Grate Intake

Swale

$$L = \frac{(151{,}400Q^{\frac{5}{8}}m^{\frac{5}{8}}S^{\frac{3}{16}})}{n^{\frac{3}{8}}(1+m^2)^{5/8}f}$$

Appendix E: Sanitary Sewer Design Calculations

Table E-1: The estimation process of the expected daily wastewater flow for each type of land use.

Residential						
Total Usable Area (acre)	(ft^2)	Avg. Person Per Household	Avg. Size of Single Home (ft^2)	People in Residential Area	Avg. Use (gpcd)	Residential Daily Use (gpd)
473.8	20638093	2.41	6429	7736	100	770000

Mobile Home					
(ft^2)	Avg. Person Per Household	Avg. Size of Single Home (ft^2)	People in Residential Area	Avg. Use (gpcd)	Expected Flow (gpd)
4331314	2.41	6429	1624	100	160000

Commercial (Office Combined)				
	Area (acre)	Design Flow (gal/acre)	Floor-Space Ratio	Expected Daily Flow (gpd)
Low End	216.53	5000	2	1083000
High End	433.06	5000	4	2165000
Average				1600000

Industrial		
Area (acre)	(gpd/acre)	Expected Daily Flow (gpd)
31.4	10000	310000

Q_{avg} 2 = Residential + Commericial	2551537.3 gpd	**0.1117896 m^3/s**
Q_{avg} 1 = 0.5 x Q_{avg}	1275768.7 gpd	**0.0558948 m^3/s**

$$PF = \begin{cases} 1.88Q_{avg}^{-0.095} & Qavg \geq 0.0368m3/s \\ 0.281Q_{avg}^{-0.44} & Q_{avg} < 0.0368m3/s \end{cases}$$

$$PFmax = 1.88(0.11^{-0.095}) = 2.32$$

$$PFmin = 1.88(0.056^{-0.095}) = 2.47$$

$$Qmin = PFmin \times Qavg1 + Qind1 + \frac{QI}{I1} = 2.47(0.056) + 0.014 + 0.029 = 0.181\ m^3/s$$

$$Qmax = PFmax \times Qavg2 + Qind2 + \frac{QI}{I2} = 2.32(0.112) + 0.014 + 0.029 = 0.302\ m^3/s$$

Q_{max}	A typical self-cleaning velocity is 2 ft/s (0.6 m/s), (3.5 ft/s sometimes), not greater than 10-15 ft/s (3.5-4.5 m/s)								
θ_{max}	h/D	R_{max}	D	A_{max}	Re_{max}	ϵ/R_{max}	f_{max}	n_{max}	S_{max}
4.189	0.75	0.3017	1	0.6319	144360	0.000331	0.014021	0.010948	0.002028
4.189	0.75	0.3771	1.25	0.9873	115488	0.000265	0.014203	0.011436	0.001168
4.189	0.75	0.2263	0.74	0.3354	192480	0.000442	0.013968	0.010417	0.004155
4.189	0.75	0.1508	**0.5**	0.1580	288720	0.000663	0.014255	0.009835	**0.011566**
4.189	0.75	0.1207	**0.4**	0.1011	360900	0.000829	0.014585	0.009585	**0.020438**
4.189	0.75	0.1056	0.35	0.0774	412457	0.000947	0.014838	0.009455	0.028783
4.189	0.75	0.0754	0.25	0.0395	577440	0.001326	0.015638	0.010659	0.079599
							n_{ave} =	0.010335	

$$Q = VA = \frac{1}{n} A R^{\frac{2}{3}} \sqrt{S} = \frac{1}{0.0103} \frac{\pi}{4}\ 0.5^2 \left(\frac{D}{4}\right)^{\frac{2}{3}} \sqrt{S_o} = 0.302 \frac{m^3}{s}\ (SI\ unit)$$

$$v = \frac{Q}{A} = 2.95 \frac{m}{s}$$