CITY OF DUBUQUE STORMWATER CLIMATE ACTION PLAN DESIGN REPORT

May 2, 2024

THE CITY OF DUBUQUE

Masterpiece on the Mississippi

Section I Executive Summary

We are pleased to have the opportunity present our findings for this exciting project. The team is comprised of four senior civil engineering students at the University of Iowa. We recommend that the City of Dubuque begins to design their stormwater system for a Climate Change Adjusted Design Storm (CCADS). This design storm is shorter than the SUDAS 100-year 24-hour design storm, but it is much more intense, aligning with the impacts that climate change is having on rainfall patterns.

We have prepared a Stormwater Climate Action Plan that addresses current and future problem areas within the city's stormwater system and provide potential action items to account for climate change effects. There are 51 basins within the city limits; to work within the time constraints for this project, we chose four regional basins within the Catfish Creek Watershed to analyze, and our findings were extrapolated city wide to create the Stormwater Climate Action Plan. One basin is in the North Fork Catfish Creek Watershed and the other three are in the Middle Fork Catfish Creek Watershed. The NW Arterial basin is in the North Fork Catfish Creek Watershed and Dubuque Industrial Center 2 (DIC2), Bergfeld, and Seippel are located in the Middle Fork Catfish Creek Watershed. Bergfeld and Seippel are two separate basins, but Seippel drains into the Bergfeld basin, so the combined analysis of these basins is called Dubuque Industrial Center West (DICW). After analysis of these four basins, DIC2 was found to perform the worst; the design aspect of this project was focused on DIC2.

Bergfeld

With intensifying weather-related climate change issues, it is anticipated that rainfall intensity will increase in the Dubuque area. This means the city's stormwater system needs to be prepared during high-intensity rainfall events to manage increased volumes of rain within stormwater basins and increased outflows through the outlets and other downstream structures. The team's analysis includes an estimation of future rainfall trends based on historical information and predicted climate change effects on rainfall, which we used to analyze potential issues Dubuque's stormwater system may face in the future. We analyzed these basins to see if they could handle current and future increased-intensity rainfall events and developed three adaptation options for issue areas. We took the findings from the team's basin analysis and translated them to city-wide recommendations through the Stormwater Climate Action Plan.

There are constraints, challenges, and societal impacts that were considered for this project. One significant constraint was considering the amount of space available in the basin and the surrounding areas. Depending on the level of development, it limited our ability to increase the volume of basin storage and influenced the development of adaptation options. Another constraint was the existing research on future rainfall trends influenced by climate change was largely qualitative and thus we had to consult a variety of sources and experts to determine quantitative values to analyze the selected basins with. We aimed to develop solutions where basins can support this increased rainfall intensity but aren't too overdesigned. Another challenge the team came to consider is that a higher intensity rainfall means a larger flood peak traveling downstream and interacting with existing and future water resource infrastructure. Possible outcomes from this interaction include the early activation of emergency spillways, aggressive channel erosion, and other degrading actions like property damage and public disruptions. Societal impacts considered was reducing the impact of flooding when Dubuque experiences high intensity rainfall. Since floods can be highly damaging, we found it is important to seek out solutions that decrease the amount of damage high intensity rainfall events might produce. Detention basins and the surrounding areas also become great spaces for new development and provide scenery, wildlife, and recreation opportunities.

NW Arterial

We have developed several strategies for the Stormwater Climate Action Plan, from our analysis of DIC2. The client requested the analysis of detention basins experiencing increased rainfall intensity. In the event of failure of a basin, the team has suggested three solutions for the client. One solution would be to add additional storage volume. The second solution would be to adjust the outlet structure hydraulics by balancing increased flows and attenuation. The third solution would be to redesign the stream corridor to contend with increased velocities and erosion, along with creating a controlled floodplain. Another option would be to reduce the amount of runoff into these basins by incorporating certain design aspects in the drainage area to help limit the runoff volumes. Adding more detention basins upstream could also limit the rate of flow into these basins. Each stormwater basin will need to be evaluated to determine which strategies will work best.


DIC2

Lastly, we have developed cost estimations for each DIC2 adaptation option, which includes construction materials, labor, administration, and contingencies, along with the cost of design services. Altering the outlet structure design comes out to the cheapest option at $181,750, followed by increasing the embankment height costing $213,600, and finally, estimated at $220,750, modifying the stream corridor would be the most expensive option.


Seippel

Section II Organization Qualifications and Experience

Organization and Design Team Description

The project was completed by a team of four students from the University of Iowa enrolled in the senior capstone design class. Members of our group are all well-versed in water resources and several have backgrounds in structures and environmental engineering. Managing the project is Anthony Lamoreux, who has a focus area in civil practice. Anthony was the primary point of contact between the team and the City of Dubuque. In addition, Anthony designed the downstream channel for the third adaptation option. Tate Houser with a pre- architecture focus area and water resources interest. Tate worked on DICW, creating inundation maps, and completing hydrologic analysis. He also designed the first adaptation option—increasing the dam embankment height for DIC2—as well as helped in designing the downstream channel for DIC2. Matthew Kliegl brings experience in his focus area, structures, with an interest in water resources. Matthew was the technical support member for the team; he completed a new design for the outlet structure (adaptation option 2), as well as performed the technical work for the downstream channel redesign (adaptation option 3). In addition, Matthew analyzed the NW Arterial basin and completed inundation maps for two different design storms. Maren Williams brings experience in her student-tailored focus area emphasizing water resources, environmental policy, and development. Maren was the report editor for this project and supported in completing hydraulic analysis for the DICW system.

Section III Design Services

Project Scope

The engineering project team worked with the City of Dubuque to create a Stormwater Climate Action Plan that addresses growing concerns with climate change and its effect on detention basins and their storage. Due to the projected increase in rainfall intensity, the team worked through options to achieve adequacy in stormwater basins in the future. A thorough investigation of selected basins followed the same approach, but as basins vary in shape, size, and location, further research was completed to ensure the development of fitting solutions. A list of the general process can be seen laid out as follows.

1. Gather Relevant Data
    a. Delineate basins via StreamStats.
    b. Determine values for rainfall intensity and storage in selected basins to quantify current condition.
    c. As-built designs for any existing structures related to basins.
    d. Various aerial/contour maps of selected basins.
    e. Investigate upstream and downstream basins for issues.
2. Estimate the potential impact of climate change on Dubuque's rainfall.
    a. Research potential impacts of climate change on Iowa's rainfall patterns.
    b. Estimate the parameters of a future design storm range.
3. Calculate the performance of select stormwater basins.
    a. Calculate the performance using today's design storm (SUDAS recommended).
    b. Calculate the performance using a design storm modified to reflect the estimated impacts of climate change.
    c. Use results to begin finding options for solutions.
4. Evaluate the potential impact of climate change adjusted rainfall on Dubuque's current stormwater systems.
    a. Run simulations on current and future hydraulics of the detention basins.
    b. Identify the success rate of the current system.
        i. Evaluate secondary and emergency spillway activation stage and hydraulics.
        ii. Understand upstream and downstream implications during climate flow events.
5. Prepare the Stormwater Climate Action Plan.
    a. Prepare prioritized recommendations for the four basins that we evaluated. The recommendations may include the following potential actions.
        i. Increase storage of analyzed basins.
        ii. Integrating new redesigned outflow structures to control flow.
        iii. Stream corridor redesign for a controlled floodplain.

b. Examine the findings for the four basins and identify climate change related impacts that can be broadly extrapolated to the rest of Dubuque's stormwater system. Use these extrapolations to form the recommendations of the city-wide Stormwater Climate Action Plan.

Work Plan

Completed first was the climate change impact on rainfall. Each member did individual research before coming together to share the findings. A new design storm was established based off said findings, which would be used in the basin analysis to compare to the current design storm—SUDAS. Next, each person selected one basin to analyze. DIC2 was analyzed by Anthony, and Matthew investigated NW Arterial. Seippel and Bergfeld were evaluated by Tate and Maren, respectively. With information provided by the client, the team was able to complete analysis on four of the 51 detention basins in Dubuque, which included watershed delineation, ground cover characteristics, time of concentration calculations, peak inflow and outflow discharge rates, stage-storage curves, maximum water levels, inundation maps, and channel characteristics. Two hydrologic modeling software, HEC-HMS and HEC-RAS, were used to determine the characteristics listed above. With the findings, the team was able to determine which basins performed well and which basin failed during the new design storm; DIC2 failed. The team determined that adaptation options would be created for DIC2. These options included increasing the dam embankment height, modifying the outlet structure, and redesigning the downstream channel to combat erosion.

Section IV Constraints, Challenges, and Impacts

Constraints

One significant constraint the team faced was considering the amount of space and land surrounding the basins. In a densely populated area, there is an inability to increase the volume of a given basin; in this circumstance, the team had to carefully analyze the selected DIC2 basin that we designed the adaptation options for to develop options that were not heavily reliant on additional capacity within the basin. Another constraint that developed through the team's analysis of climate change patterns was working with the existing research on climate change trends and how the team would determine potential rainfall quantities. A significant portion of the research was focused only on generalizations and qualitative observations rather than quantitative values, so we had to find and combine research from a variety of different sources and experts to develop the quantitative analysis of future climate change trends in Dubuque. Time is another constraint that the team faced in developing the Stormwater Climate Action. Since the senior capstone class is only a semester long, a project including all 51 city basins in Dubuque was not feasible. With the time we had, our group was able to examine four detention basins in the Middle Catfish and North Catfish watersheds and what effects they have on Dubuque's stormwater system. So, determining a method in which the team could analyze select basins to subsequently extrapolate those findings to basins city-wide was a priority within the time constraint.

Challenges

Climate change is the main challenge of the project and its impact on Dubuque's stormwater system is something that was focused on significantly. Estimating the change in rainfall and frequency of rainfall for future storms was a challenge for our team. Research and engineering inference helped us estimate future rainfall amounts and intensities. In the past, Dubuque has seen some very intense rainfall, up to 14 inches at a time in recent history. Another challenge we faced was determining how to design for that much rain. For example, ISWMM Unified Sizing Criteria reflected by SUDAS standards design for a water quality treatment of 1.25" storm, but obviously that design will fail quickly if Dubuque sees the same rainfall it has had in the past. On the other hand, overdesigning was a challenge the team faced as well, having to maintain realistic expectations for the development of adaptation options. To address the challenge of climate change and how to design for the increased rainfall intensity, the team developed a comprehensive Stormwater Climate Action Plan. The team analyzed historical rainfall trends over the past 100 years in northeast Iowa. With help from John Wiley, the industrial pretreatment coordinator for the Water and Resource Recovery Center for the City of Dubuque, and advising from Rick Fosse, Professor Priscilla Williams, and research engineer Humberto Vergara, the team estimated a range of future increased rainfall intensities. Another challenge the team came to consider is that a higher intensity rainfall means a larger flood peak traveling downstream and interacting with existing and future water resource infrastructure. Possible outcomes from this interaction include the early activation of emergency spillways, aggressive channel erosion, and other degrading actions like property damage and public disruptions. Comparing the performance of the basins from the current recommended design to the projected design values gave insight to the adaptation options the team developed to implement within Dubuque's stormwater system to mitigate the possible outcomes discussed.

Societal Impact within the Community and the State of Iowa

Our goal is to reduce the chance of flooding when Dubuque sees high intensity rainfall. Reduced flooding leads to less money spent on water damage, helping the economy. Also, if the stormwater impacts of climate change are not addressed, flooding problems can devalue properties within Dubuque. These detention basins will also provide recreation areas where families can enjoy the outdoors. Detention ponds also offer great spaces for new housing developments or communities in Dubuque. The ponds will provide scenery, wildlife, and recreation opportunities for new developments. Bringing new people into Dubuque will also help boost the economy. The Mississippi River already provides leisure and landscape, but the detention ponds will add to that and help reduce the flooding Dubuque will experience, being so close to the Mississippi River. Our analysis will also help prepare Dubuque for future rainfalls and the stormwater impacts of climate change. We hope that this project can set an example that other communities can follow and integrate in their communities to prepare for climate change.

Section V Adaptations Options That Were Considered

After determining that DIC2 would be used for the development of adaptation options, the project team went through the process of coming up with three options that could be implemented to improve basin storage and flow conveyance during increased rainfall events.

The first option considered was increasing the embankment height of the basin that included the addition of an emergency spillway within the embankment. Increasing embankment height would increase the storage capacity so that the basin could hold more stormwater during increased rainfall intensity events which would reduce the risk of flooding and potential overtopping of the embankment. With the increased capacity, the basin can also better attenuate peak flows, may require less frequent maintenance, and gives more flexibility for future development in the area and climate change that may increase runoff values. Some downsides to this potential option are the high construction cost associated and potential negative environmental impacts from altering the landscape which could lead to habitat disruption or loss of vegetation. Also, if not properly engineered or maintained, there is a potential for failure of higher embankments.

The second option considered is to modify the existing outlet structure in the DIC2 basin. This option would include the removal of the existing structure and pipe, boring underneath the embankment to set the new pipe, and construction of a new custom outlet structure with different dimensions that accommodate more flow and maintain a one-foot freeboard between the maximum water surface and the embankment height. This design would eliminate the waterfall at the downstream end of the outlet structure and can help reduce knickpoints along the downstream channel; the proposed outlet structure would include a drop box that would in turn lower the outlet pipe elevation to the final downstream channel elevation. This option includes a longer and more complicated construction process.

The team's final adaptation option developed involves a downstream corridor redesign. If increased storage and discharge flows are not achievable within the basins, then the city of Dubuque can control the floodplain and decreased flow velocity within the stream. A two-stage natural channel consists of a low-flow, meandering stream within a main channel section, an overbank section, and then a secondary tier that accommodates high flows without flooding outside the corridor. Natural vegetation is encouraged on the overbank and secondary tier because of the increased roughness that slows down stream discharge without eroding the soil. The corridor boundaries are a major restriction for this solution. This is also the most expensive option, as the cut and fill volumes required are substantial. In contrast, water quality would be improved, which would decrease sedimentation of new or existing basins.

Section VI Final Design Details

Through the analysis of the four selected test basins, only one, DIC2, failed during the Climate Change Adjusted Design Storm. In fact, this basin also was the only basin to fail during the SUDAS design storm. This shows that the basin was under designed from the start; it might've been implemented before SUDAS was created. A basin should have enough capacity such that the SUDAS 24-hour 100-year design storm fills the basin to the emergency spillway elevation; for DIC2, the SUDAS storm activates the emergency spillway, hence it is under designed. This is why it is the only basin to fail when introduced to the Climate Change Adjusted Design Storm; this is why DIC2 was chosen for adaptation option developments. Three adaptation options were developed to enhance the performance of DIC2. The rest of the basins performed as shown in Table 1.

Table 1: Performance of test basins

Test Basins		Required Storage (acre-ft)		Available Storage Used (%)		Peak Inflow (cfs)		Peak Discharge (cfs)	
Basin #	Drainage Area (acres)	Iowa SUDAS	CCADS	Iowa SUDAS	CCADS	Iowa SUDAS	CCADS	Iowa SUDAS	CCADS
Basin 1	2656.0	38.0	49.0	73.8%	95.2%	1764.7	2665.3	1763.1	2653.7
Basin 2	115.2	5.8	5.4	111.5%	103.8%	288.7	120.6	285.7	117.6
Basin 3	358.4	71.1	69.1	84.7%	82.3%	806.2	391.8	158.9	153.5
Basin 4	864.0	8.9	13.4	54.0%	81.3%	467.1	886.2	466.7	882.8

Adaptation Option 1:

The first option for the DIC2 basin would be increasing the embankment height and adding an emergency spillway within the embankment. The spillway is designed with enough capacity to carry twice the inflow amount for the 100-year storm event as per SUDAS design manual section 2G-1, and is placed with freeboard height above the simulated high-water levels. After running the design storm in HEC-HMS, a peak flow value of 479 cfs was summarized in the findings. The peak flow value caused the water levels to reach an elevation of 803.9 MSL. From there it was important to make sure the spillway would be activated when any rainfall event just above a 100-year storm occurs to mitigate the stress acting on the basin. Using the 100-year discharge value the model of the basin yielded, the flowrate was related to the area and velocity. From here an iterative process helped to narrow down reasonable dimensions for the emergency spillway to keep the velocity to a permissible amount based on codes in SUDAS section 7E-12.02 general conditions. Making a larger width for the spillway greatly reduced velocity to a value of under 5.5 ft/s, the calculations for spillway size are located in Appendix A. Velocity values around 6 or greater posed a risk and should be used with caution, this is the restraint we kept in mind, also including a little more room for unforeseen large storm events. The embankment elevation is located 3.5 foot above the peak water elevation for the simulated storm, which exceeds guidelines laid out in SUDAS section 2G-1. The embankment of the basin was increased to make it so the spillway dimensions weren't going to be as big. It also gives much more capacity within the basin that can make it sufficient for the foreseen future climate changes. The bulk of this design would be in cut and fill operations and creating the emergency spillway. Volumes for cut and fill are located in Appendix A.

Adaptation Option 2:

The second option is modifying the DIC2 outlet structure, which would increase flow out of the basin and maintain a one-foot freeboard between the maximum water elevation and the top of the embankment. The top of the embankment is 803.5 MSL so the maximum water elevation is 802.5 MSL. An iterative process was used to calculate the stage-storage-discharge functions that were input into HEC-HMS for modeling. Currently, the outlet structure has a circular primary outlet, with a diameter of 3 feet. A secondary rectangular outlet sits on top of the structure, measuring 3 feet by 5.33 feet. The structure connects to a culvert that is 3.5 feet in diameter that transports the water under the embankment to the downstream channel. The downstream outlet leads to an immediate water fall and this creates several knickpoints initially along the channel before the water can reach its final path. The knickpoints are unideal because these will erode more; therefore, the outlet point of the pipe to the existing downstream elevation was lowered. This will eliminate the waterfall at the end of the pipe, remove the knickpoints, and reduce the future potential for any further erosion. The current outlet structure design fails in the 6-hour 100-year design storm; the water level overtops the emergency spillway. This means that the outlet structure needs to allow more flow through it so that the water surface does not get as high. After completing calculations, increasing the circular primary spillway from 3 feet in diameter to 4 feet in diameter allowed enough flow to maintain a one- foot freeboard between the maximum water level and the top of the embankment. The pipe diameter and secondary outlet dimensions remained the same. The outlet structure was also redesigned to include a drop; this lowers the outlet pipe elevation by seven feet, eliminating the waterfall and any knickpoints. Finally, the maximum elevation in DIC2 from a 6-hour 100-year storm with the proposed outlet structure design is 802.5 MSL; this meets the one-foot freeboard requirement. See the calculations in Appendix B. It should be noted that this option will increase the discharge downstream so downstream corridor conditions should be examined for capacity before option 2 is implemented.

Adaptation Option 3:

The third option is a two-stage channel that consists of a lower stage called the Bankfull Channel and an upper stage for flood conveyance. The lower stage has two floodplain benches that reduce the energy in the flow during overbank flood storms. The reduced energy preserves the geometry of the cross-section by controlling excess erosion. Also, other benefits are highlighted within the plan. The design of the two-stage channel contains three subsections that reflect the purpose listed above.

The Bankfull Channel contains three methods to correctly design the lower stage to the correct depth.

1. Regional Curve Development
    - The size of the channel depends on the stream characteristics. For a natural stream, a rating curve that describes the stage-discharge relationship of the reach being studied would help identify the bankfull design discharge.
2. Rapid Regional Curve Development
    - Another method in determining the bankfull channel is to look at similar in characteristics streams that have a fully developed flood plain width and bankfull width and depth. Multiple streams with rating curves should be measured to compare to the channel being analyzed.
3. Reference Reach
    - The final option would be to complete a detailed survey along the same reach or a nearby reach. The reference reach must have a similar climate, history, drainage area, and watershed conditions.

For the DIC2 basin design, all methods mentioned above were not applicable because of the lack of information available. The team decided to model the downstream reaches using the HEC-RAS software to run simulations of the peak discharges from the Iowa SUDAS design storm and the Team design storm. The results of the simulation runs are shown in Figure 1-4 in Appendix C. What is displayed is the large amount of capacity the stream contains during the high peak flow of both storms. The team believes the excess conveyance can be attributed to the significant backwater from the Middle Fork Catfish Creek approximately 1035' from the outfall of the basin. The model simulates a known water surface elevation of 768' which accounts for a bankfull condition of the Middle Fork Catfish Creek. This bankfull condition was extrapolated upstream for the cross-section of the channel.

The Floodplain channel subsection design consists of a range of parameters for the benches to prevent instabilities within the channel. The total width of the benches is less than three times the top width of the bankfull channel otherwise the benches may not develop and are more likely to become unstable. Furthermore, the bench width cannot exceed five times the width of the bankfull channel because the channel will begin to experience a natural meandering behavior that will cut into the banks of the stream.

The Flood Conveyance channel subsection design should accommodate a design-storm event without flooding outside the embankments. The team chose to accommodate the largest peak discharge for the DIC2 basin which is the Iowa SUDAS design storm.

The final design of adaptation option 3 is shown in Figure 5 in Appendix C and the cross-section water levels from the HEC-RAS simulation run with the Iowa SUDAS design storm are shown in Figure 6 and Figure 7. Regarding Appendix C, apparent ponding areas display no visible means of drainage, but in fact there is a constant downstream slope that allows the excess water to either percolate or flow to the Middle Fork Catfish Creek.

Overall, the three adaptation options guided the creation of the Action Plan by displaying sensitivities to recent climate change predictions. The Action Plan contains numerous maintenance tasks due to the clear indication of weakness becoming apparent whether the maintenance is debris or structurally related. Additionally, numerous inspection tasks are listed because identifying excess erosion or hydrological issues (e.g. sediment deposition within the basin) is important for potentially observing changing hydrology within the area. Finally, detention basin models need to be created and tested against the Climate Change Action Design Storm due to the results from the four-basin sample size for Phase 1. Phase 2 of the detention basin model task involves testing all basins against any significant changes in the most recent climate change predictions.

Section VII Engineer's Cost Estimate

Below are the final cost estimates for the three adaptation options. Included in the cost estimate is a construction subtotal, which includes the cost of material, labor, and equipment. A 10% contingency was assumed for any issue in the construction process, along with a 20% fee for administrative and inspection costs during construction. Finally, the design services are estimated to cost an additional $45,250, which includes a 3:1 overhead and profit rate. Another thing to note is the assumption that cut material will be able to be repurposed as fill material, as long as it meets Class II backfill requirements and is sufficiently compacted. Option 2 is the least expensive option, followed by option 1 and then option 3. See Table 1.

Table 1: Cost estimations for each option

	Option 1	Option 2	Option 3
Construction Subtotal	$129,500.00	$105,000.00	$ 135,000.00
10% Contingencies	$ 12,950.00	$ 10,500.00	$ 13,500.00
20% Inspection and Administration	$ 25,900.00	$ 21,000.00	$ 27,000.00
Design Services	$ 45,250.00	$ 45,250.00	$ 45,250.00
Total Project Cost	$213,600.00	$181,750.00	$ 220,750.00

Adaptation Option 1

Table 2: Construction cost estimate for option 1

BID ITEM DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	COST
ENBANKMENT-IN-PLACE	6322.49	CY	$ 18.73	$118,500.00
STABILIZING CROP - SEEDING AND FERTILIZING	0.62	ACRE	$ 335.65	$ 210.00
SILT FENCE	600.00	LF	$ 1.78	$ 1,075.00
PERIMETER AND SLOPE SEDIMENT CONTROL DEVICE, 9 IN. DIA.	600.00	LF	$ 2.67	$ 1,600.00
COMPACTION WITH MOISTURE AND DENSITY CONTROL	6322.49	CY	$ 1.28	$ 8,100.00
TOTAL:				$129,500.00

Adaptation Option 2

Table 3: Construction cost estimate for option 2

BID ITEM DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	COST
CULVERT, CONCRETE PIPE, 2000D, TRENCHLESS, 42 IN. DIA.	68.00	LF	$ 934.51	$ 63,500.00
COMPACTING BACKFILL ADJACENT TO BRIDGES, CULVERTS, OR STRUCTURES	17.63	CY	$ 16.21	$ 285.00
STRUCTURAL CONCRETE (RCB CULVERT)	1.65	CY	$ 710.29	$ 1,175.00
REINFORCING STEEL	614.23	LB	$ 1.72	$ 1,050.00
REMOVAL OF EXISTING STRUCTURE	1.00	LS	$26,411.68	$ 26,400.00
REMOVAL OF RIGID PIPE CULVERT	68.00	LF	$ 176.67	$ 12,000.00
EXCAVATION, CLASS 10, CHANNEL	17.63	CY	$ 8.97	$ 160.00
EXCAVATION, CLASS 10, WASTE	9.33	CY	$ 9.58	$ 89.00
SILT FENCE	60.00	LF	$ 1.78	$ 105.00
PERIMETER AND SLOPE SEDIMENT CONTROL DEVICE, 9 IN. DIA.	60.00	LF	$ 2.67	$ 160.00
MODIFIED SUBBASE	1.58	CY	$ 41.64	$ 66.00
TOTAL:				$105,000.00

Adaptation Option 3

Table 4: Construction cost estimate for option 3

BID ITEM DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	COST
EXCAVATION, CLASS 10, CHANNEL	2295.64	CY	$ 8.97	$ 20,600.00
ENBANKMENT-IN-PLACE	4446.99	CY	$ 18.73	$ 83,500.00
CLEARING AND GRUBBING	2.45	ACRE	$5,434.48	$ 13,300.00
SILT FENCE	1780.84	LF	$ 1.78	$ 3,175.00
PERIMETER AND SLOPE SEDIMENT CONTROL DEVICE, 9 IN. DIA.	1780.84	LF	$ 2.67	$ 4,750.00
STABILIZING CROP - SEEDING AND FERTILIZING	2.45	ACRE	$ 335.65	$ 825.00
COMPACTION WITH MOISTURE AND DENSITY CONTROL	6742.63	CY	$ 1.28	$ 8,625.00
TOTAL:				$135,000.00

Appendices

Appendix A Adaptation Option 1 Details

Designing the emergency spillway started with finding how it could be sized to carry the inflow and still be able to function with non-damaging velocities to keep the channel shape intact. Using discharge equation, Q=VA and rearranging for V, we get V=Q/A. This is where an iterative process was used to find what spillway dimensions would yield the desired velocities as shown below. The spillway ended up being a trapezoidal channel with a height of 3.5 feet at the low section, and a width of 78 ft. After finding the discharge according to SUDAS 2G-1, it was divided by the cross-sectional area of the channels spillway which yielded velocity. Before finding the final values, many widths and heights were tested to give us an optimal result. The channel slope was also modeled from SUDAS design section 2-G1, as it followed the embankment. The maximum value for slope of embankments is 4:1, so the spillway was designed with 4:1 channel down to existing ground with a length of 70 ft.

Spillway Size Calculations

Flow Capacity Calculations

$$Q100 := 479 \, \frac{ft^3}{s}$$

$$DesignCapacity := 2 \cdot Q100 = 958 \, \frac{ft^3}{s}$$

Spillway Dimensions

$$BottomWidth := 50 \, ft$$

$$TopWidth := 78 \, ft$$

$$ChannelHeight := 3.5 \, ft$$

$$TrapezoidalChannelArea := (TopWidth \cdot ChannelHeight) - 2 \cdot \left(\frac{ChannelHeight \cdot 14 \, ft}{2} \right) = 224 \, ft^2$$

Permissible Velocity Calculations

$$V := \frac{DesignCapacity}{TrapezoidalChannelArea} = 4.277 \, \frac{ft}{s}$$

For embankment, it was increased by 3.5 ft at the top and filled over the existing embankment to get to a slope of 4:1 down to existing elevation. The top of the embankment is set at 807.5 which gives plenty of freeboard height, as the design storm only rose to a highwater level of 803.9 MSL. The embankment and spillway were drafted in Civil3D, and the volumes of the fill were found by finding the areas of the cross-section and multiplying by the length. Fill was calculated across the entire embankment and the spillway was taken out from the calculations as it would be open.

Fill Calculations

Spillway Volume Calculations

$Area := 260.0840\ ft^2$

$ChannelVolume := Area \cdot 50\ ft = 13004.2\ ft^3$

$SlopeVolume := \dfrac{Area \cdot 7\ ft}{2} = 910.294\ ft^3$

$$SpillwayVolume := ChannelVolume + SlopeVolume = 13914.494\ ft^3$$

Embankment Volume Calculations

$EmbankmentArea := 561.16\ ft^2$

$EmbankmentVolume := EmbankmentArea \cdot 329\ ft = 184621.64\ ft^3$

Total Fill Calculations

$$TotalFill := EmbankmentVolume - SpillwayVolume = 6322.487\ yd^3$$

Appendix B Adaptation Option 2 Details

The primary and secondary outlets were modeled as weirs initially. Once the water surface reached a certain height, the nature of the outlets reflected that of an orifice, as the outlets became submerged. The flow out of a weir and the flow out of an orifice have different equations, ISWMM C3-S10 and ISWMM C3-S12 respectively. They are as follows:

$$Q_{weir} = C * L * H^{1.5}$$

$$Q_{orifice} = C * A * (2 * g * h)^{0.5}$$

The coefficient, C, varies between a weir and an orifice. C_{weir} was taken as 2.7 and $C_{orifice}$ was taken as 0.2, as recommended by ISWMM. The length, L, for the primary circular outlet was estimated by taking the outlet area and modeling it as a square. This simplified weir calculations because the length would vary for a circular cross section as the water level rose. This allowed the length to remain constant. For the rectangular secondary outlet, the length was set as the outlet's perimeter, since it was on the top of the structure and its opening faced upward. This assumes that the water enters the secondary outlet equally from all sides. The area, A, used in the orifice equation is simply the opening area of the respective outlets. The gravitational constant, g, was taken as 32.2 ft/s^2. The weir height, H, is the difference between the bottom of the outlet and the water elevation. The orifice height, h, is the difference between the vertical middle of the outlet and the water elevation. The question becomes when do the outlets switch from acting as weirs to acting as orifices. To solve this, weir and orifice flow rates were calculated at each 0.5-foot increment, starting at 790 MSL to 806 MSL. At each water level, the minimum value was taken as the governing flow rate. These calculations were performed for both the primary and secondary outlets. The emergency spillway was modeled as a weir, with a length of 300 feet and weir coefficient of 3. Using the weir equation above, the flow rate of the emergency spillway was found at every 0.5-foot increment. Adding the governing flowrates for the primary and secondary outlets to the emergency spillway flowrates, a stage-discharge function was found, see Table 1 below.

Table 1: Excel calculations to find stage-discharge functions for outlet redesign

Water Surface Elevations [ft]	Primary Spillway Weir		Primary Spillway Orifice		CONTROL	Secondary Spillway Weir		Secondry Spillway Orifice		CONTROL	Emergency Spillway		Overall Outflow
	Elevation [ft]	797.2	Elevation [ft]	798.721		Elevation [ft]	802.292	Elevation [ft]	802.292		Elevation [ft]	803.528	
	C_w	2.7	C_d	0.2		C_w	2.7	C_d	0.2		C_w	3	
	L [ft]	3.54	A [ft^2]	7.07		L [ft]	16.66	A [ft^2]	15.99		L [ft]	300	
	h [ft]	Q [cfs]	h [ft]	Q [cfs]	**Q [cfs]**	h [ft]	Q [cfs]	h [ft]	Q [cfs]	**Q [cfs]**	h [ft]	Q [cfs]	Q [cfs]
790	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
790.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
791	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
791.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
792	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
792.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
793	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
793.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
794	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
794.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
795	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
795.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
796	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
796.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
797	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	0	0.0
797.5	0.3	1.4	0.0	0.0	1.4	0.0	0.0	0.0	0.0	0.0	0	0	1.4
798	0.8	6.5	0.0	0.0	6.5	0.0	0.0	0.0	0.0	0.0	0	0	6.5
798.5	1.3	13.6	0.0	0.0	13.6	0.0	0.0	0.0	0.0	0.0	0	0	13.6
799	1.8	22.3	0.3	6.0	6.0	0.0	0.0	0.0	0.0	0.0	0	0	6.0
799.5	2.3	32.3	0.8	10.0	10.0	0.0	0.0	0.0	0.0	0.0	0	0	10.0
800	2.8	43.5	1.3	12.8	12.8	0.0	0.0	0.0	0.0	0.0	0	0	12.8
800.5	3.3	55.8	1.8	15.1	15.1	0.0	0.0	0.0	0.0	0.0	0	0	15.1
801	3.8	69.0	2.3	17.1	17.1	0.0	0.0	0.0	0.0	0.0	0	0	17.1
801.5	4.3	83.2	2.8	18.9	18.9	0.0	0.0	0.0	0.0	0.0	0	0	18.9
802	4.8	98.1	3.3	20.5	20.5	0.0	0.0	0.0	0.0	0.0	0	0	20.5
802.5	5.3	113.9	3.8	22.1	22.1	0.2	4.3	0.2	11.7	4.3	0	0	26.3
803	5.8	130.5	4.3	23.5	23.5	0.7	26.8	0.7	21.6	21.6	0	0	45.1
803.5	6.3	147.8	4.8	24.8	24.8	1.2	59.7	1.2	28.2	28.2	0	0	53.0
804	6.8	165.8	5.3	26.1	26.1	1.7	100.4	1.7	33.5	33.5	0.5	291.8	351.5
804.5	7.3	184.5	5.8	27.3	27.3	2.2	147.6	2.2	38.1	38.1	1.0	862.5	927.9
805	7.8	203.8	6.3	28.4	28.4	2.7	200.5	2.7	42.2	42.2	1.5	1607.3	1678.0
805.5	8.3	223.8	6.8	29.5	29.5	3.2	258.5	3.2	46.0	46.0	2.0	2492.3	2567.8
806	8.8	244.4	7.3	30.6	30.6	3.7	321.2	3.7	49.4	49.4	2.5	3498.0	3578.0

Next, the capacity of the 3.5-foot outlet pipe needed to be checked at every elevation. This was used to determine whether the outlet structure controlled the flow or if the pipe did. The orifice equation was used to calculate the flow the pipe could permit. Only the orifice values were analyzed because the pipe won't start to limit the flow until it becomes full of water, acting as an orifice, see Table 2. Comparing these pipe outflow values to the inflow values above, the inflow values are all lower than outflow values, excluding the values that include the emergency spillway flowrates, as this flow will be overtopping the embankment and not going into the outlet pipe. Hence, the inlet controls all scenarios. That means that the size of the outlet pipe, 3.5 feet in diameter, is sufficient.

Table 2: outlet pipe calculations

Outlet Pipe		
Water Surface Elevation [ft]	Elevation [ft]	801.324
	C_d	1
	A [ft^2]	9.62
	h [ft]	Q [cfs]
797.5	0	1.0
798.0	0	4.8
798.5	0	10.2
799.0	0	6.0
799.5	0	10.0
800.0	0	12.8
800.5	0	15.1
801.0	0	17.1
801.5	0.176	26.4
802.0	0.676	51.8
802.5	1.176	68.4
803.0	1.676	81.6
803.5	2.176	93.0
804.0	2.676	103.1
804.5	3.176	112.3
805.0	3.676	120.9
805.5	4.176	128.8
806.0	4.676	136.3

The design process was iterative, meaning that as the dimensions of the outlets were changed, the stage-storage-discharge functions also changed. Each design had to then be input into HEC-HMS as functions to see what the maximum water level came out to be with the respective outlet dimensions. The final design for the outlet structure that maintained a one-foot freeboard includes a circular primary spillway with a 4-foot diameter, a three-foot by 5.33-foot rectangular secondary spillway, a 12-foot-tall outlet structure that includes a six-foot drop box to lower the outlet pipe elevation, and a 3.5-foot outlet pipe that goes underneath the embankment. The summary table output from HEC-HMS is shown in Figure 1.

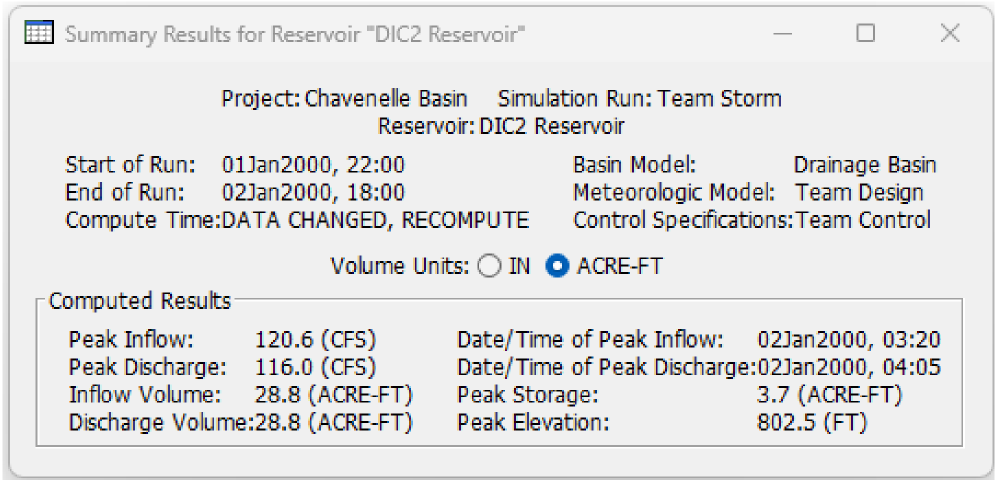


Figure 1: HEC-HMS DIC2 summary table for 6-hour 100-year design storm

The existing pipe has a slope of 4.8%; this was used for the slope of the proposed pipe. The length of the pipe was calculated so that the toe of the embankment would meet the outlet pipe elevation, with the embankment slope at 4:1, which is what it currently is. See Figure 2.1 and Figure 2.2 in Design Drawings for profile and dimension sheets.

Appendix C Adaptation Option 3 Details

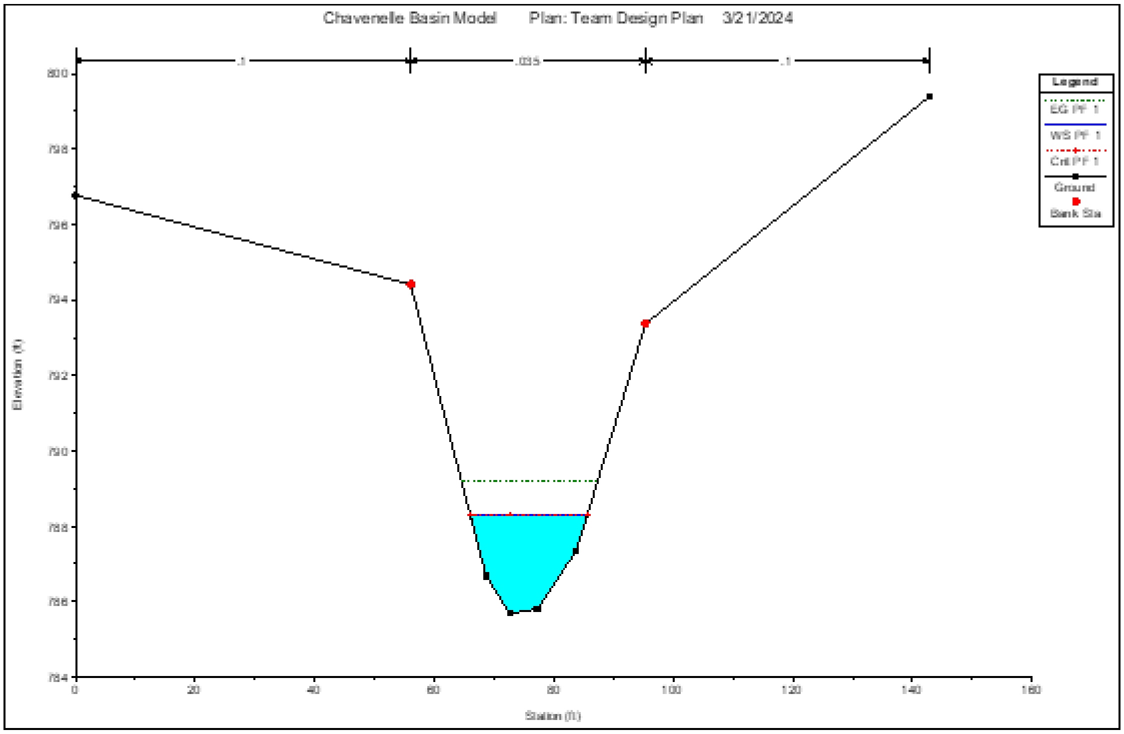


Figure 1: upstream cross-section for the team design storm water level

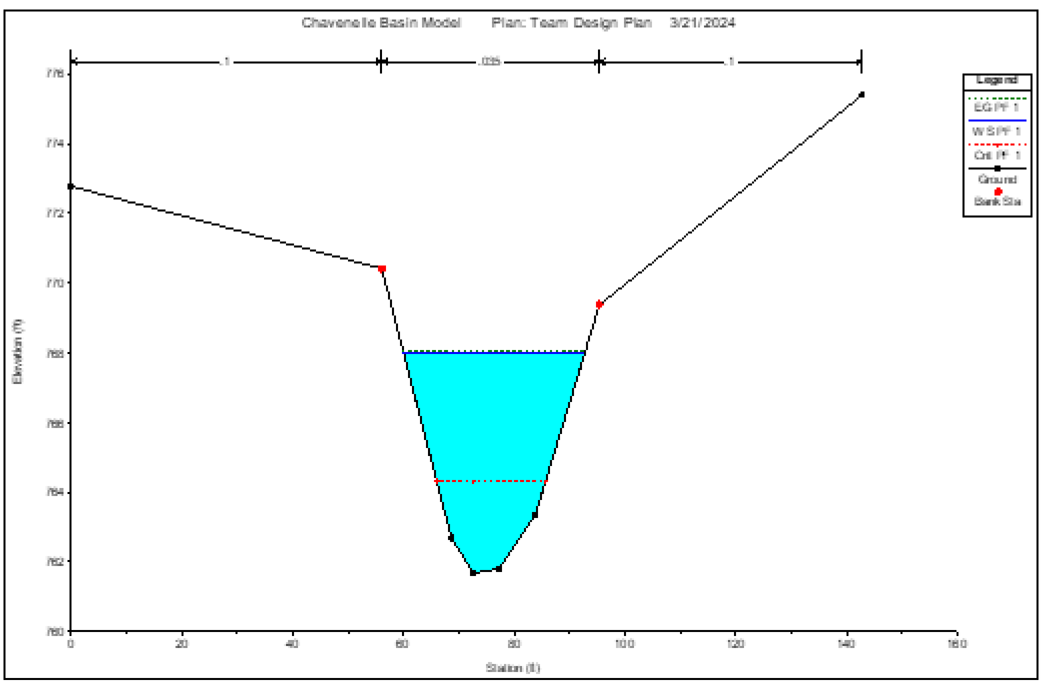


Figure 2: downstream cross-section for the team design storm water level

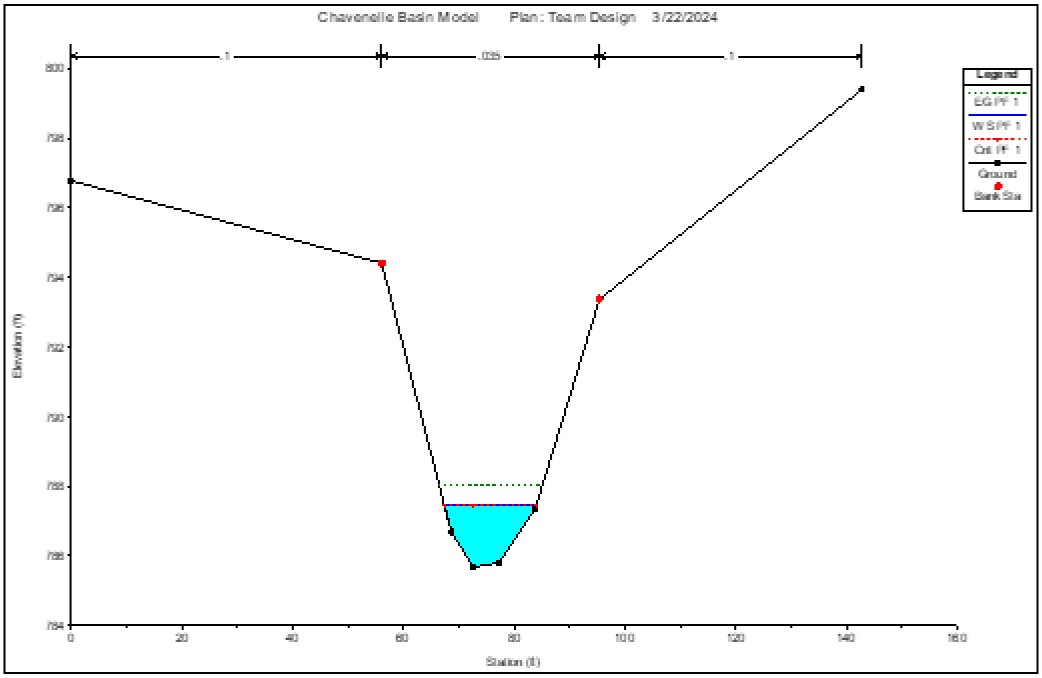


Figure 3: upstream cross-section for the Iowa SUDAS design storm water level

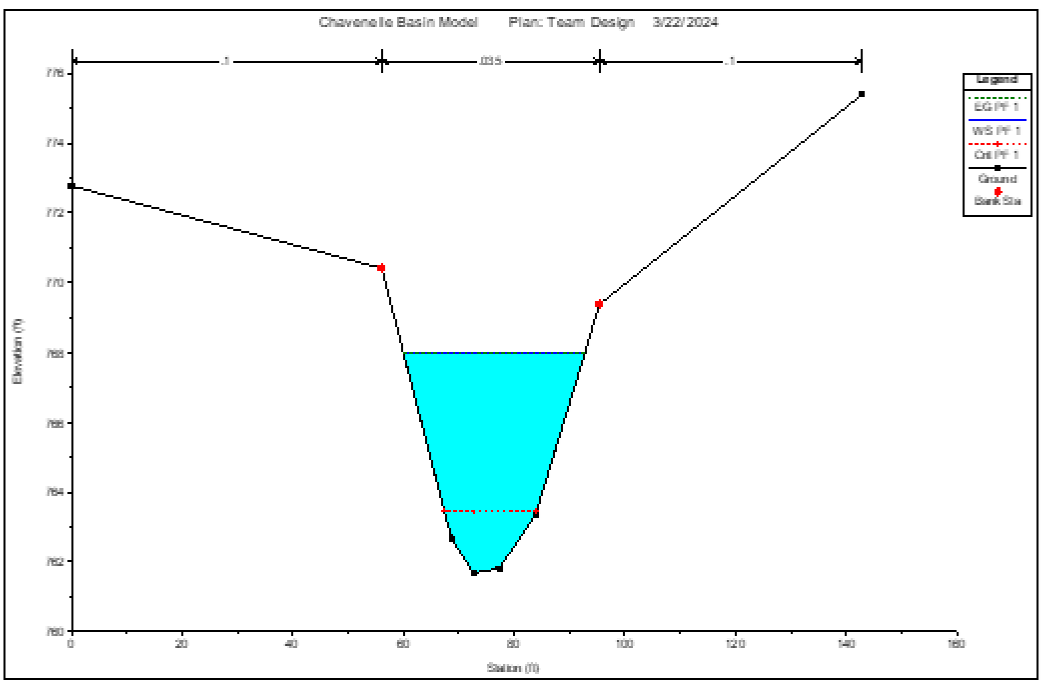


Figure 4: downstream cross-section for the Iowa SUDAS design storm water level

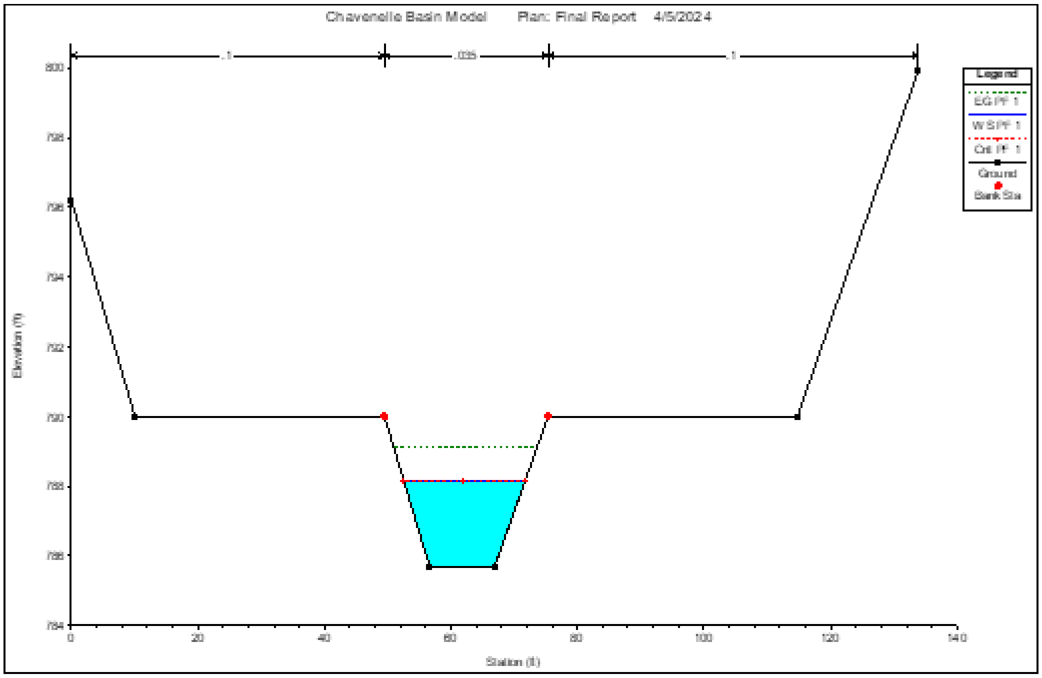


Figure 5: upstream two-stage cross-section for the Iowa SUDAS design storm water level

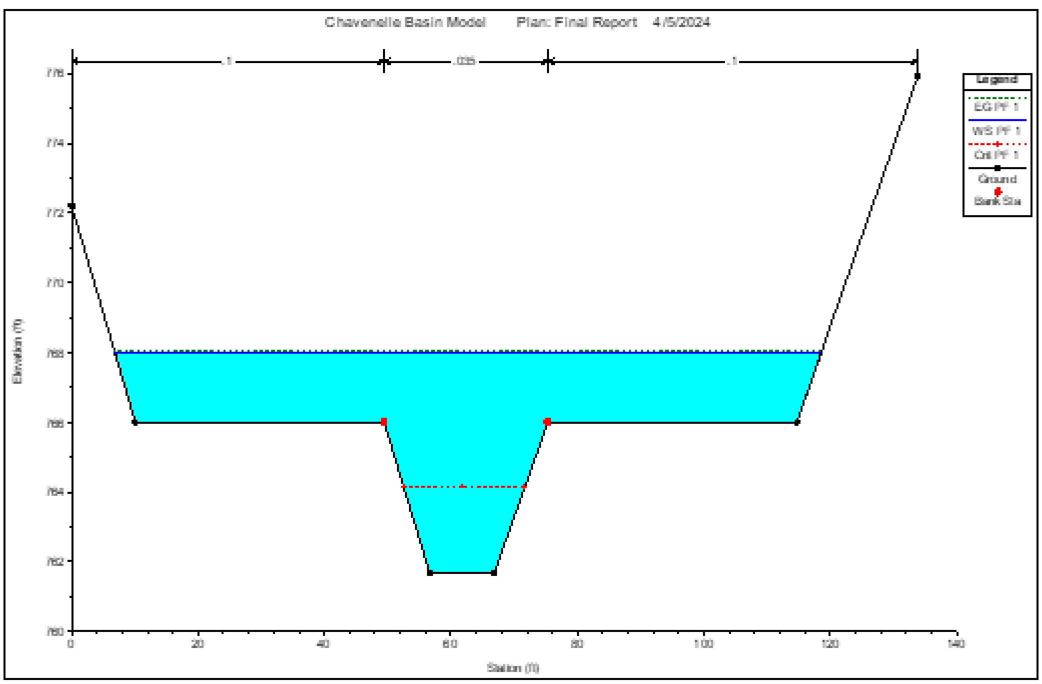


Figure 6: downstream two-stage cross-section for the Iowa SUDAS design storm water level

Appendix D Cost Estimation

The following calculations are for option 2 cost estimations. Steel specifications can be found on Figure 2.4 in Design Drawings. Rebar positioning was estimated with reference to inlet specifications from the Iowa DOT.

Volume Calculations

Steel Calculations

$L_{Ushape} := 6\ ft + 10\ in$ $\quad L_{Lstraight} := 11\ ft + 6\ in$ $\quad L_{Sstraight} := 5\ ft + 9\ in$

$N_{Ushape} := 42$ $\quad N_{Lstraight} := 44$ $\quad N_{Sstraight} := 22$

$$L_{total} := L_{Ushape} \cdot N_{Ushape} + L_{Lstraight} \cdot N_{Lstraight} + L_{Sstraight} \cdot N_{Sstraight} = 919.5\ ft$$

using #4 bars $\quad \gamma_{steel} := 0.668\ \frac{lb}{ft}$

$$W_{steel} := L_{total} \cdot \gamma_{steel} = 614.226\ lb$$

Concrete Calculations

$B_{ext} := 6\ ft$ $\quad H_{ext} := 12\ ft$ $\quad D_{ext} := 6\ ft$ $\quad t := 4\ in$

$B_{int} := B_{ext} - t$ $\quad H_{int} := H_{ext} - t$ $\quad D_{int} := D_{ext} - t$

$D_1 := 4\ ft$ $\quad D_2 := 3.5\ ft$ $\quad base := 5\ ft + 4\ in$ $\quad width := 3\ ft$

$$V_{conc} := B_{ext} \cdot H_{ext} \cdot D_{ext} - B_{int} \cdot H_{int} \cdot D_{int} - \pi \cdot \frac{{D_1}^2}{4} \cdot t - \pi \cdot \frac{{D_2}^2}{4} \cdot t - base \cdot width \cdot t$$

$$V_{conc} = 1.653\ yd^3$$

Cut Calculations

$base_1 := 6\ ft + 2\ ft$ $\quad base_2 := 6\ ft + 6\ ft$ $\quad height := 7\ ft$

$$V_{cut} := \frac{{base_1}^2 + {base_2}^2}{2} \cdot height = 26.963\ yd^3$$

Fill Calculations

$$V_{fill} := V_{cut} - B_{ext} \cdot D_{ext} \cdot height = 17.63\ yd^3$$

Subbase Calculations

$B_{sub} := 6\ ft + 2\ ft$ $\quad t_{sub} := 8\ in$

$$V_{subbase} := {B_{sub}}^2 \cdot t_{sub} = 1.58\ yd^3$$

Appendix E Rainfall Research

Abstract

The research for the Dubuque Stormwater Climate Action Plan involves using creditable and relevant information from governmental and independent research sources such as the Environmental Protection Agency (EPA), the Iowa Department of Natural Resource (DNR), articles found through the University of Iowa Libraries INFOHAWK+ system, and National Climate Assessments completed by the United States Global Change Research Program (USGCRP). Additionally, the team completed a data analysis on the rainfall record using National Weather Service (NWS) records of Dubuque, Iowa. Though multiple locations for recording stations exist within the region of study, the selected station for data is the Dubuque Regional Airport (KDBQ). The summary of the team's findings includes an observation of increase precipitation on the recorded annual rainfall and the decrease of rainfall days in a year. Likewise, the information gathered from resources listed mention the same observation within the northeast region of Iowa allowing the team to quantify a specified range of rainfall intensities for basin simulations.

National Climate Action – 2023 Report

Within the National Climate Action report, the mentioning of increased intense precipitation occurs in multiple sections. In the water quality and quantity section, projections of 0.3% to 1.5% increase and 0.2% to 0.5% increase per decade in the eastern and western part of the Midwest, respectively, are anticipated. Cumulative runoff across the Midwest region is projected to increase causing the level of local channels to increase during rainfall events. Natural channels will be redefined over time as new discharge rates reshape reach geometry. Wider floodplains and deeper channel beds are expected because of increased volume and velocities promoting erosive conditions. If a channel is not well protected with natural vegetation within the floodplain or conformed to the increased runoff, then sediment will be stripped and transported from the upper parts of the watershed to the outlet or deposited somewhere in between. Figure 1 provides a projected change in cumulative runoff for the years 2036-2065 within the Midwest region. Eastern Iowa sees a significant increase within the winter and spring seasons along with a slight increase in the summer and autumn seasons. In particular, the team is focused on the summer season, but the given information about other seasons could be used in the future during an analysis of a basin performance throughout the year.

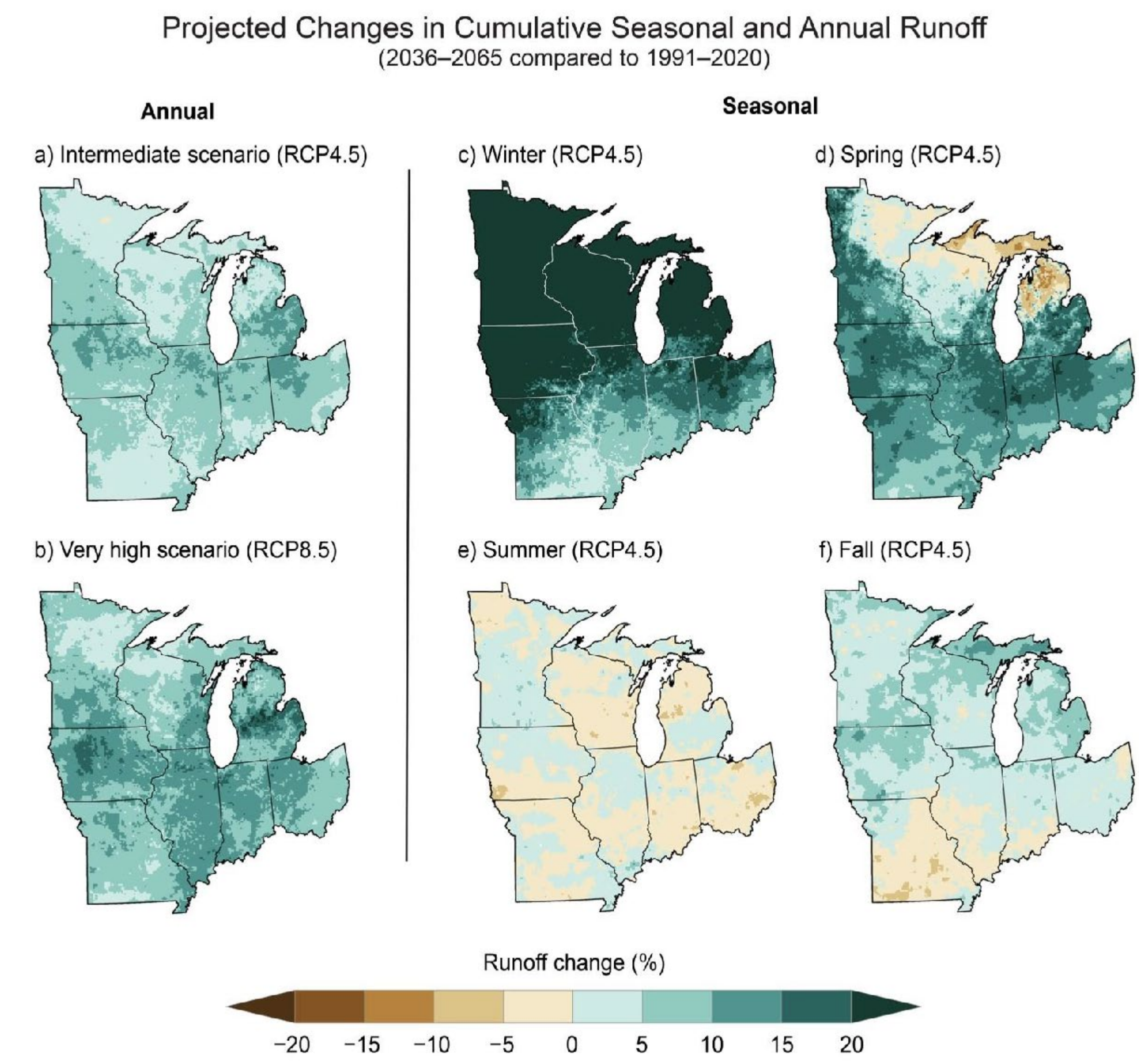


Figure 1: NCA 2023, Cumulative Runoff Change Projection

Iowa Department of Natural Resource – Climate Change

A more literary approach to the team's research is looking at the state natural resource database to record any information on the changing environment. The Iowa DNR limits the use of any statistical trends or observations, but the information does provide a few compelling pieces of information. The highlight of how climate change can affect the state of Iowa results in a more associated understanding from a local level rather than a region level like the Midwest. The first topic covered is precipitation which the Iowa DNR states the increase of 8% from the start of the record in 1873 to 2008. Additionally, the increases in precipitation are seen more in eastern Iowa where Dubuque is located rather than western Iowa. A significant statement made is that humidity increased the dewpoint by three to five degrees Fahrenheit resulting in more summertime thunderstorms that have bring in precipitation. This topic reenforces conversations amongst the team. The Dubuque Stormwater Climate Action Plan could result in a different style of analyzing basin performance. While detention basins are traditionally designed for normal duration storms such as a 24-hour rainfall, the team may suggest testing basins for a short-duration storm driven by climate change to prepare for future events. The seasons of late spring and early summer are the team's focus since the months of June and July bring the most rainfall during the year according to the NWS local data. The general understanding of the team from the current research is that intensity is increasing along with precipitation.

National Weather Service Data – Rainfall Record

To justify the findings, the team completed a rainfall record analysis on the city of Dubuque to make the necessary observations of the area. The data acquired from the NWS involved using a third-party site created by Iowa State University under the Iowa Environment Mesonet which provided the requested information like specific station rainfall data, monthly rainfall amounts, monthly rainfall days, rainfall records, and even a snapshot in time of Dubuque's hyetograph of the record 10" rainfall set in July 27 and July 28 of 2011. The data collected for the month of July showed signs of agreement with what research the team gathered thus far. The Box and Whisker plots displayed in Figure 2 and Figure 3 are the record rainfall period broken up into two time periods of 24 years and one period of 23 years from 1951 to 2023. Focusing on the most recent range, one can notice the outlier of the nearly 15 total inches of rainfall in the month of July in 2011. Otherwise, the interquartile range of the last time period expanded suggesting more variability of precipitation. Additionally, the maximum of the plot and the average rainfall reach a larger precipitation amount when compared to the two time periods prior.

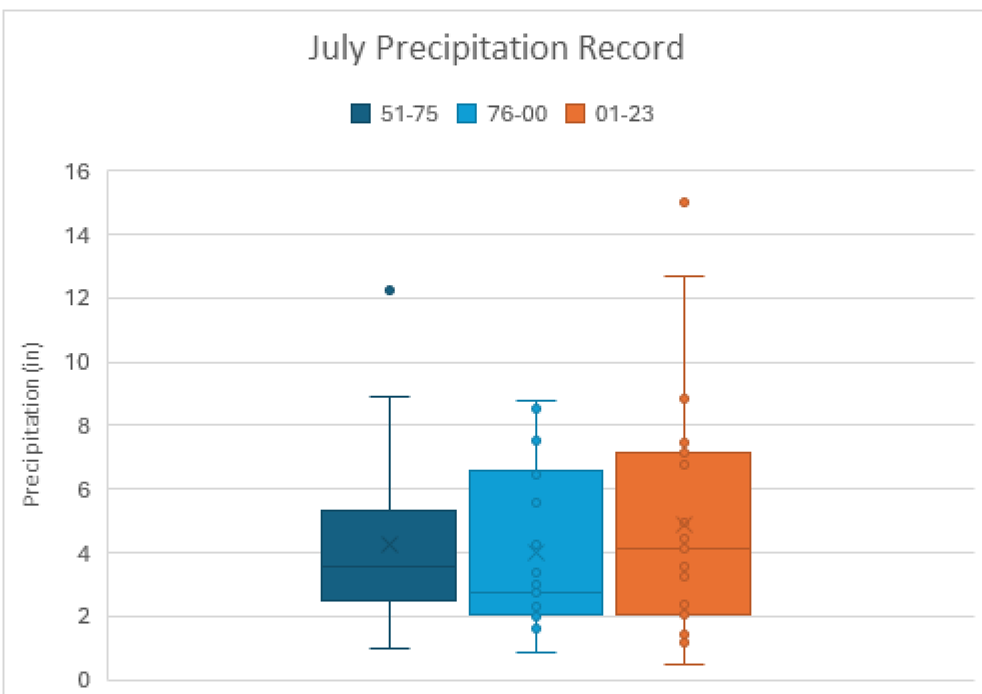


Figure 2: July rainfall record from 1951 to 2023

June precipitation in Figure 3 shows a slight shift of the interquartile range and a significant increase in the maximum with no outliers.

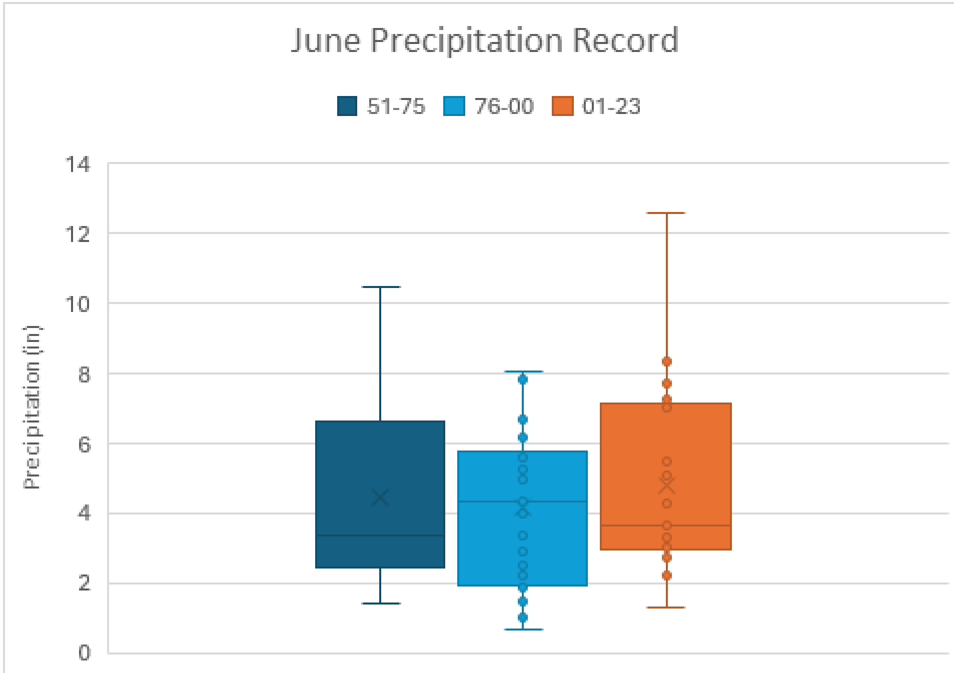


Figure 3: June rainfall record from 1951 to 2023

The rainfall record of Dubuque displays the largest increase in precipitation in the month of July where the highest average temperature and most solar energy provide conditions for intense evapotranspiration from the surrounding environment. As highlighted by the Iowa DNR, evapotranspiration yields more convective thunderstorms. In terms of analysis, paring increased precipitation with a time period gives an intensity, but finding the average intensity increase over a rainfall record is beyond the scope of the team since the data analysis would be daunting to accomplish. Rather, the data collection center has a category for number of rainy days within a given month for the complete rainfall record. The same three time periods were established, and the data for June and July were displayed in Table 1.

Table 1: percent increase from 1951-1975 period to 2001-2023 period

Precipitation Days within the Month					
June			July		
51-75	76-00	01-23	51-75	76-00	01-23
10.6	10.8	11.3	10.2	10.1	9
% Increase		6.2	**% Increase**		-12.1

The increase in June precipitation shown in Figure 3 can be pared with an increase of rainy days, but the decrease of rainy days within the month of July suggests a different narrative. The team indirectly solved for an increase intensity for the month of July by comparing monthly precipitation total to number of days of precipitation. Although crude, the data analysis provides a foothold for proving increased intensity due to climate change.

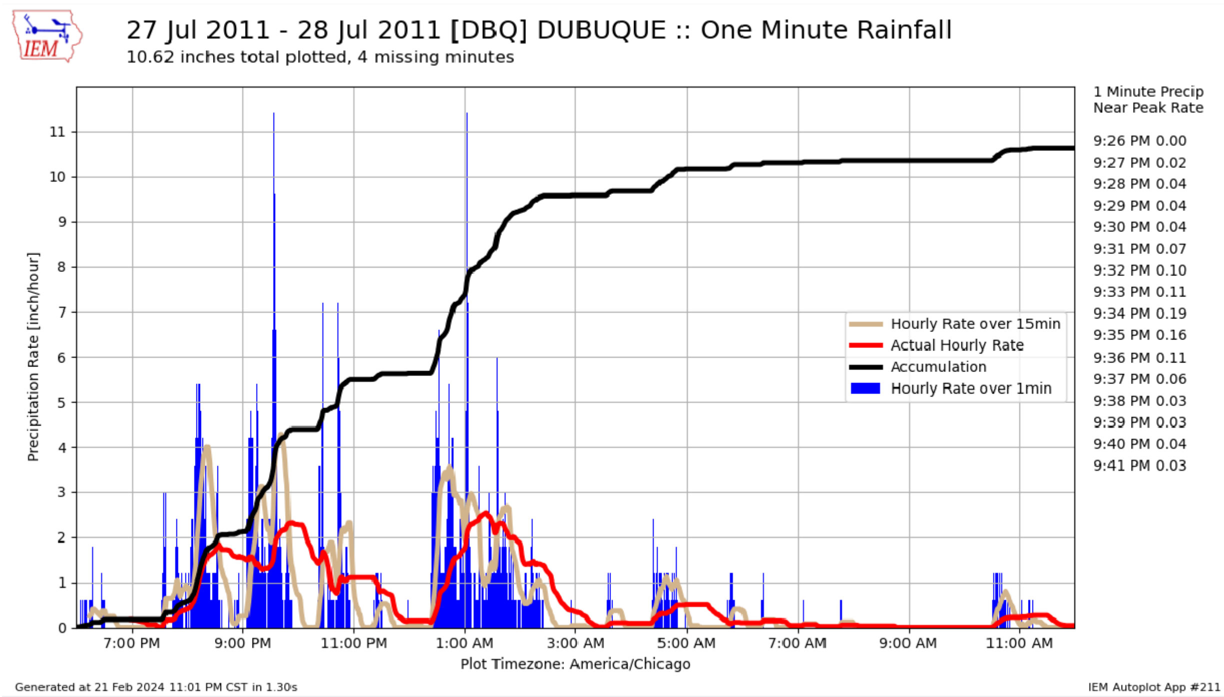


Figure 4: the hyetograph of Dubuque's 10-inch rainfall in July 2011

Table 2: Dubuque's Top 30 Rainfall Records

	day	precip
1	1967-09-14	8.85
2	2011-07-27	7.47
3	1961-07-01	6.28
4	2002-08-21	5.99
5	1961-11-02	4.79
6	2010-07-23	4.59
7	1960-05-06	4.37
8	1961-09-12	4.37
9	1951-07-08	4.36
10	1977-07-17	3.91
11	1993-07-05	3.91
12	2000-06-13	3.84
13	2019-09-12	3.7
14	1961-09-13	3.67
15	1962-05-29	3.55
16	1953-07-05	3.55
17	2016-06-14	3.5
18	2002-06-04	3.5
19	1990-08-17	3.45
20	1963-11-22	3.45
21	1966-06-09	3.45
22	2007-07-18	3.4
23	2011-07-28	3.27
24	2023-08-14	3.2
25	1954-06-15	3.15
26	1960-01-12	3.04
27	2002-08-22	2.97
28	1961-11-16	2.95
29	1972-08-01	2.95
30	2002-06-03	2.92

In Table 2, one can notice the 10-inch rainfall event making two spots on the top 30 rainfall events given that the storm started July 27, 2011 (position 2) in the evening and continue through midnight into July 28, 2011 (position 23).

National Climate Assessment – 2014 Report

The final piece of information used for preliminary research is the Third National Climate Assessment created in 2014. The use of this resource is relevant because the report displays a map and a chart of observed changes over time of heavy precipitation. Figure 6 emulates the narrative of increase heavy precipitation stated by all resources considered in the team's findings while Figure 7 consolidates and shows a percent of increase of heavy precipitation in the recent decades. One will notice more quantifiable evidence within Figure 7 and Figure 8. The team determined that the observed increase of heavy precipitation within the Midwest is 37% while the future change multiplier is between four and five.

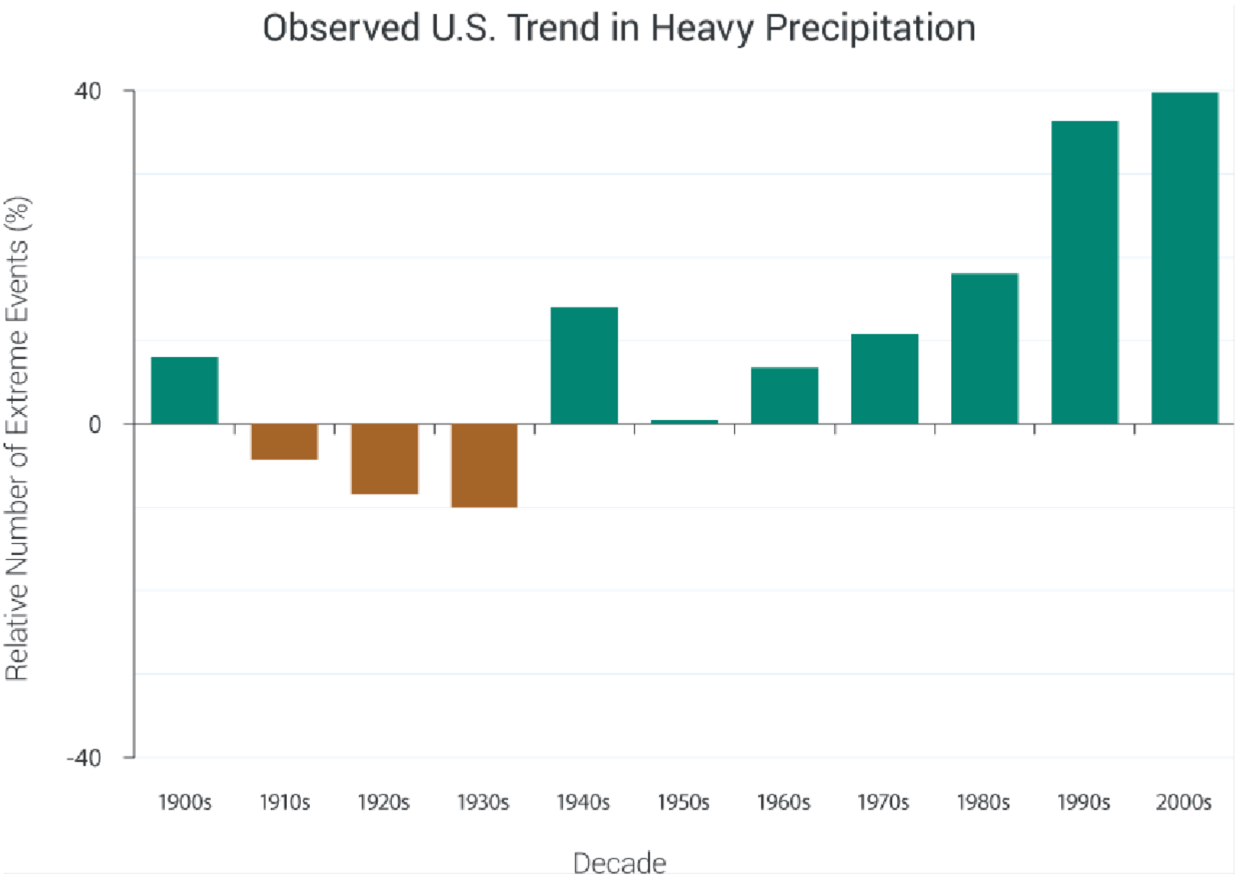


Figure 5: observed heavy precipitation trends in the United States

Figure 2.18: Observed Change in Very Heavy Precipitation

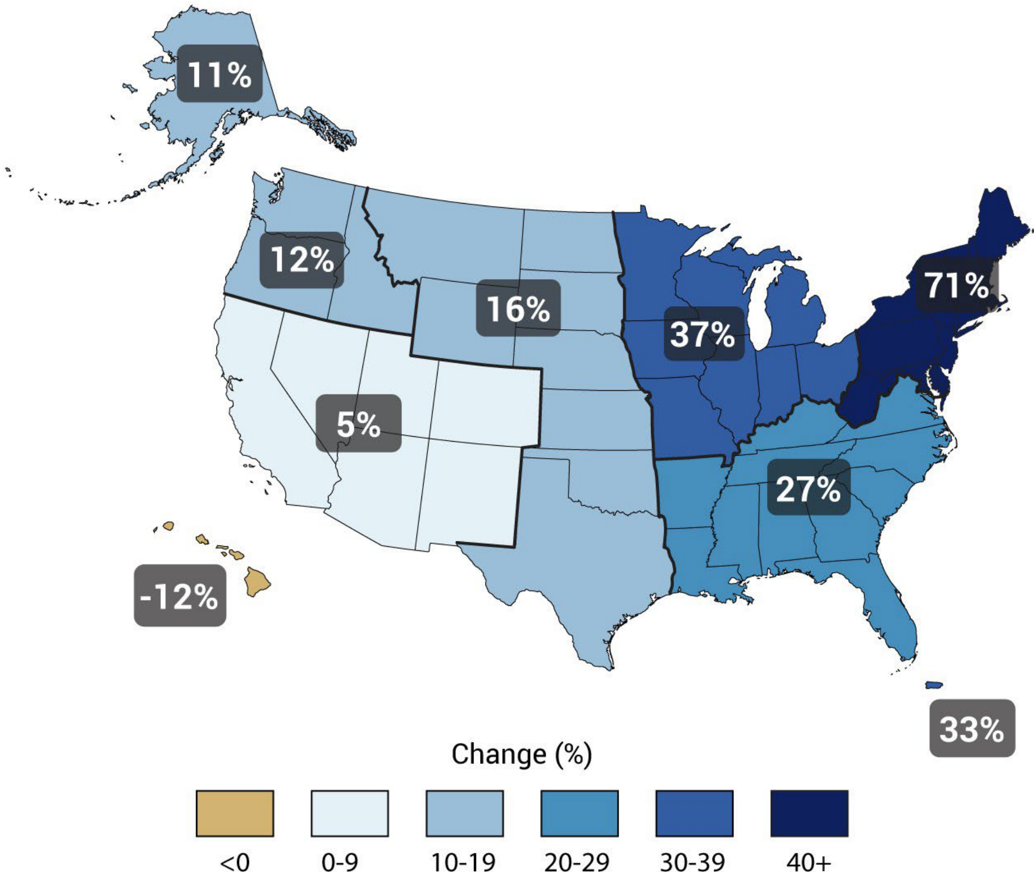


Figure 6: observed Heavy precipitation increase related to average

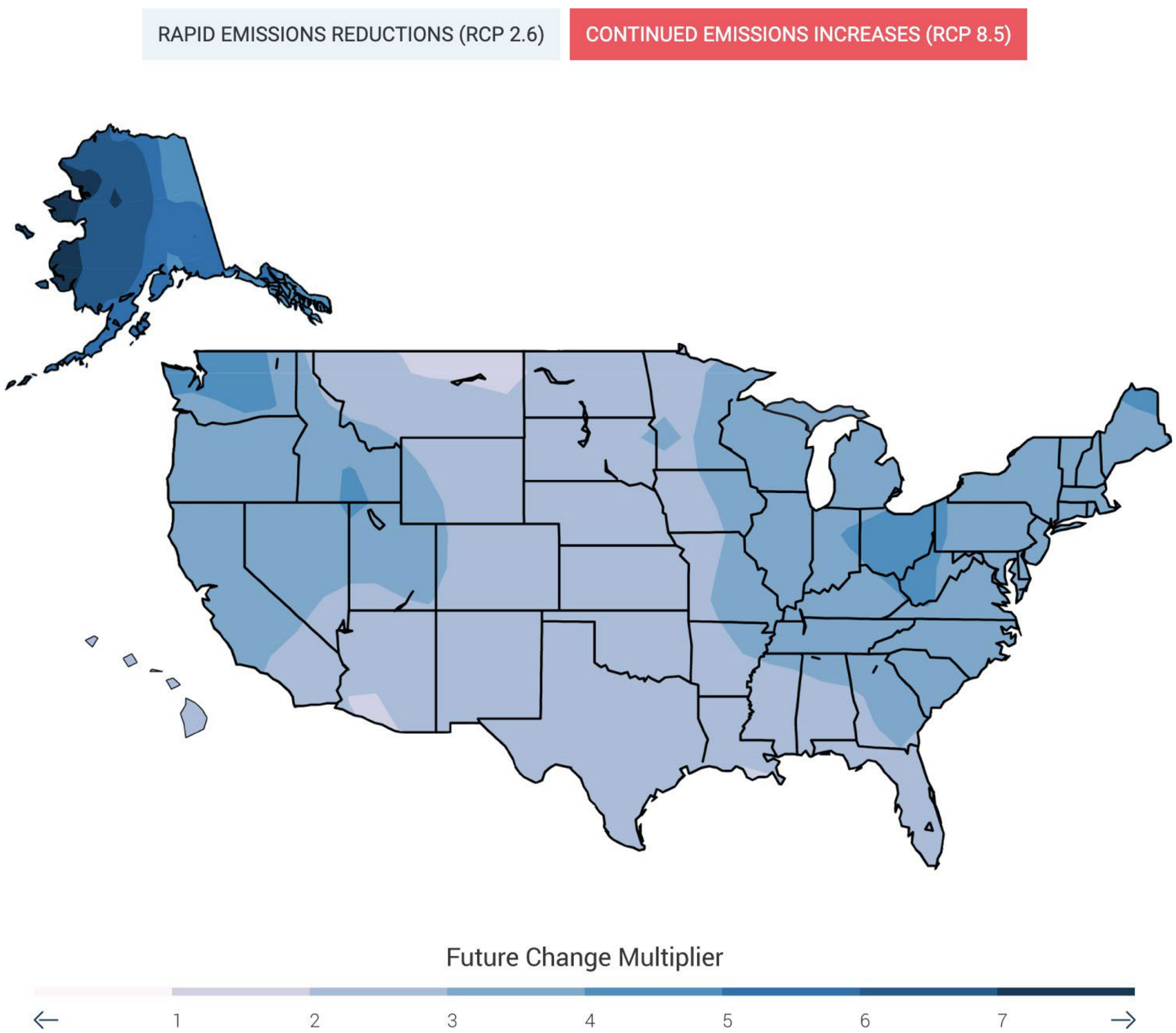


Figure 7: predicted change of heavy precipitation if continued emission trends cease to decrease

Climate Stats

Precipitation	Hist.	Mid-Century RCP 4.5	Mid-Century RCP 8.5	End-Of-Century RCP 4.5	End-Of-Century RCP 8.5
ANNUAL					
Days Without Precipitation	23.67	22.33	22.67	24	18.33
Annual Precipitation (Inches)	30.35	37.18	36.73	33.13	39.58
AUTUMN					
Daily Precipitation (Inches)	0.06	-	0.08	-	0.08
Maximum Daily Precipitation (Inches)	1.02	-	1.46	-	1.43
WINTER					
Daily Precipitation (Inches)	0.06	-	0.07	-	0.07
Maximum Daily Precipitation (Inches)	0.93	-	1.07	-	1.13
SPRING					
Daily Precipitation (Inches)	0.09	-	0.13	-	0.12
Maximum Daily Precipitation (Inches)	1.26	-	1.58	-	1.7
SUMMER					
Daily Precipitation (Inches)	0.12	-	0.11	-	0.14
Maximum Daily Precipitation (Inches)	1.58	-	1.4	-	1.73

Figure 8: climate Risk and Resilience Portal report on the Dubuque, Iowa area

Using the climate projection model centered on the Dubuque, Iowa area the following observations were made and considered in the design:

- Maximum daily precipitation increases in almost every season for mid-century and end of century periods.
- A slight increase in number of rain days is expected over the mid-century and end of century periods.
- The amount of annual rainfall increases from historical in mid-century is 6.38 inches (from 30.35 inches to 36.73 inches), and in end of century is 9.23 inches (from 30.35 inches to 39.58 inches).
- End of Century consecutive days without precipitation is 5.34 days lower than historical (18.33 days from 23.67 days) while mid-century is 1.0 days lower than historical (22.67 from 23.67 days

The data filtered using the RCP 8.5 conditions which is the 'worst case scenario' in terms of climate predictions with continually fossil fuel use. The average rise in global temperature would be 4.9 degrees Celsius.

Conclusion

Using the quantifiable information from the Third National Climate Assessment and the fact that a heavy precipitation is an intensity above 0.30 inches per hour, the team suggests testing selected basins for an intense storm. According to the Iowa Stormwater Management Manual, detention basins are designed for design peak flow attenuations of any given storm below the 100-year rainfall. Typically, the emergency spillway on Iowa detention basins should be activated at the 100-year design. The overarching goal of the Dubuque Stormwater Climate Action Plan is to identify the client's concerns of, "Are we ready for what is coming." A justifiable outcome of basin performance from increasing heavy precipitation in the future decades would be to simulate intense storms above the 0.30-inch per hour threshold.

The National Oceanic and Atmospheric Administration provides design storm precipitation amounts in the Atlas 14 Point Precipitation graphs shown in Figure 9. The team unanimously decided to simulate the 100-year 6-hour design storm producing an intensity of about 1-inch per hour. The idea behind the design storm selection is that the team believes a high intensity storm will prematurely fail a selected basin because of too much attenuation of peak flow. With the 90% confidence intervals NOAA provides, the range of intensities that will be simulated will be 0.45 inches per hour to 1.31 inches per hour. With significantly increased discharge inflow, the outflow structures may attenuate too long resulting in an earlier emergency overflow activation. The quick duration storms with high intensities are becoming more prevalent according to the team's research, which is why the team encourages the "High-Intensity Design Storm" (HIDS) design parameter.

Point Frequency Tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.379** (0.307-0.473)	**0.444** (0.359-0.555)	**0.556** (0.448-0.695)	**0.653** (0.524-0.820)	**0.794** (0.618-1.03)	**0.909** (0.690-1.18)	**1.03** (0.754-1.36)	**1.15** (0.812-1.56)	**1.33** (0.899-1.83)	**1.47** (0.964-2.04)
10-min	**0.555** (0.450-0.693)	**0.650** (0.526-0.812)	**0.814** (0.656-1.02)	**0.956** (0.767-1.20)	**1.16** (0.905-1.50)	**1.33** (1.01-1.74)	**1.50** (1.10-2.00)	**1.69** (1.19-2.28)	**1.95** (1.32-2.68)	**2.15** (1.41-2.98)
15-min	**0.677** (0.548-0.845)	**0.793** (0.642-0.991)	**0.992** (0.800-1.24)	**1.17** (0.935-1.46)	**1.42** (1.10-1.84)	**1.62** (1.23-2.12)	**1.84** (1.35-2.44)	**2.06** (1.45-2.78)	**2.37** (1.60-3.27)	**2.62** (1.72-3.64)
30-min	**0.945** (0.766-1.18)	**1.11** (0.898-1.39)	**1.39** (1.12-1.74)	**1.64** (1.31-2.06)	**2.00** (1.55-2.58)	**2.28** (1.74-2.98)	**2.59** (1.90-3.43)	**2.91** (2.04-3.93)	**3.35** (2.27-4.62)	**3.70** (2.43-5.14)
60-min	**1.22** (0.989-1.52)	**1.43** (1.16-1.79)	**1.81** (1.46-2.26)	**2.14** (1.72-2.69)	**2.64** (2.06-3.43)	**3.05** (2.32-3.99)	**3.49** (2.56-4.64)	**3.96** (2.79-5.37)	**4.62** (3.13-6.38)	**5.15** (3.39-7.15)
2-hr	**1.50** (1.22-1.85)	**1.76** (1.44-2.17)	**2.22** (1.81-2.75)	**2.65** (2.14-3.29)	**3.28** (2.59-4.24)	**3.82** (2.94-4.96)	**4.39** (3.26-5.80)	**5.01** (3.56-6.75)	**5.89** (4.02-8.09)	**6.60** (4.37-9.10)
3-hr	**1.67** (1.38-2.05)	**1.96** (1.61-2.40)	**2.47** (2.02-3.04)	**2.96** (2.41-3.65)	**3.70** (2.94-4.76)	**4.33** (3.35-5.61)	**5.02** (3.74-6.61)	**5.76** (4.12-7.74)	**6.84** (4.70-9.36)	**7.72** (5.13-10.6)
6-hr	**2.00** (1.66-2.42)	**2.30** (1.91-2.80)	**2.88** (2.38-3.51)	**3.45** (2.83-4.21)	**4.34** (3.50-5.56)	**5.12** (4.01-6.59)	**5.98** (4.51-7.83)	**6.93** (5.01-9.26)	**8.32** (5.77-11.3)	**9.47** (6.34-12.9)
12-hr	**2.35** (1.97-2.82)	**2.66** (2.23-3.20)	**3.28** (2.73-3.94)	**3.89** (3.22-4.69)	**4.88** (3.99-6.22)	**5.77** (4.56-7.38)	**6.76** (5.15-8.80)	**7.87** (5.74-10.4)	**9.50** (6.65-12.9)	**10.9** (7.33-14.7)
24-hr	**2.69** (2.28-3.19)	**3.04** (2.57-3.61)	**3.74** (3.15-4.45)	**4.43** (3.71-5.29)	**5.54** (4.56-6.98)	**6.53** (5.21-8.26)	**7.62** (5.86-9.82)	**8.85** (6.50-11.6)	**10.6** (7.50-14.3)	**12.1** (8.25-16.3)
2-day	**3.01** (2.57-3.53)	**3.46** (2.96-4.06)	**4.31** (3.67-5.07)	**5.11** (4.32-6.04)	**6.37** (5.27-7.90)	**7.46** (5.99-9.30)	**8.65** (6.69-11.0)	**9.96** (7.37-13.0)	**11.9** (8.41-15.8)	**13.4** (9.20-17.9)
3-day	**3.30** (2.84-3.84)	**3.74** (3.22-4.37)	**4.59** (3.93-5.36)	**5.40** (4.60-6.34)	**6.68** (5.57-8.24)	**7.80** (6.30-9.67)	**9.02** (7.02-11.4)	**10.4** (7.72-13.4)	**12.4** (8.80-16.3)	**14.0** (9.62-18.5)
4-day	**3.57** (3.08-4.14)	**4.00** (3.45-4.64)	**4.83** (4.15-5.61)	**5.63** (4.81-6.57)	**6.90** (5.78-8.47)	**8.02** (6.51-9.91)	**9.25** (7.23-11.7)	**10.6** (7.93-13.7)	**12.6** (9.02-16.6)	**14.3** (9.85-18.9)
7-day	**4.26** (3.71-4.89)	**4.71** (4.10-5.42)	**5.56** (4.82-6.40)	**6.37** (5.48-7.36)	**7.64** (6.42-9.24)	**8.73** (7.13-10.7)	**9.94** (7.81-12.4)	**11.3** (8.46-14.4)	**13.2** (9.49-17.3)	**14.8** (10.3-19.4)
10-day	**4.86** (4.25-5.54)	**5.36** (4.69-6.13)	**6.29** (5.48-7.20)	**7.14** (6.18-8.21)	**8.44** (7.12-10.1)	**9.55** (7.82-11.6)	**10.7** (8.47-13.3)	**12.0** (9.07-15.3)	**13.9** (10.0-18.1)	**15.4** (10.8-20.2)
20-day	**6.53** (5.78-7.37)	**7.27** (6.42-8.21)	**8.51** (7.49-9.63)	**9.58** (8.38-10.9)	**11.1** (9.39-13.0)	**12.3** (10.1-14.6)	**13.6** (10.8-16.5)	**14.9** (11.3-18.6)	**16.7** (12.1-21.4)	**18.1** (12.7-23.5)
30-day	**8.00** (7.12-8.97)	**8.94** (7.95-10.0)	**10.5** (9.27-11.8)	**11.7** (10.3-13.2)	**13.5** (11.4-15.6)	**14.8** (12.2-17.4)	**16.1** (12.8-19.4)	**17.5** (13.3-21.6)	**19.3** (14.0-24.5)	**20.6** (14.5-26.6)
45-day	**9.98** (8.94-11.1)	**11.2** (9.98-12.4)	**13.0** (11.6-14.6)	**14.5** (12.9-16.3)	**16.5** (14.0-19.0)	**18.0** (14.9-21.0)	**19.4** (15.5-23.2)	**20.8** (15.8-25.5)	**22.5** (16.4-28.4)	**23.8** (16.9-30.6)
60-day	**11.7** (10.6-13.0)	**13.1** (11.8-14.5)	**15.3** (13.7-17.0)	**17.0** (15.1-18.9)	**19.2** (16.3-21.8)	**20.8** (17.3-24.0)	**22.2** (17.8-26.4)	**23.6** (18.0-28.8)	**25.3** (18.5-31.7)	**26.5** (18.8-34.0)

Figure 9: Dubuque, Iowa Precipitation frequency estimates

Appendix F NW Arterial Basin Analysis

NW Arterial HEC-HMS Software Analysis

Basin Models – NW Arterial

NW Arterial

This is the watershed for the NW Arterial Basin.
It has an area of 0.56 mi^2 from USGS StreamStats delineation (Figure 1).

Loss Method
SCS Curve Number – used the lag method Excel (Figure 2) to retrieve curve number: 74.74. Note that the curve number already factors in impervious area so Impervious % is set to zero.

Transform Method
SCS Unit Hydrograph – used the lag method Excel (Figure 2) to retrieve the lag time (37.72 minutes).

Reach

Reach flows from watershed to NW Arterial Basin/outlet structure.

Routing Method
Muskingham-Cunge:
Length, slope, Manning's n, bottom width, and side slope came from provided NW Arterial SC-96 file.
Index Flow was calculated using the rational method (Table 1). Assumed baseflow was zero and found peak flow using Q=C*i*A with the runoff coefficient being 0.35 (provided by NW Arterial SC-96), intensity being 0.99 in/hr, and area being the watershed drainage area (0.56 mi^2). Peak flow calculated to be 125.2 cfs. Averaging the peak flow and base flow gave an index flow of 62.6 cfs.

NW Arterial Basin

This holds the actual storage detention basin characteristics.
Storage method: Elevation-Storage-Discharge
Storage-Discharge and Elevation-Storage function were provided (Table 2).

Meteorologic Models

Met 1

For the Precipitation, Hypothetical Storm was used.

Hypothetical Storm
User-Specified Pattern was used for Method. This way, the storm duration (6 hr) and point depth (5.97 inches) could be input into the software. A SCS Type 2 storm assumes a 24-hr duration which would be incorrect for this design.
Storm Pattern

From NOAA Atlas 14, a 6-hr percentage curve (input into paired data), which relates the percent of the storm that has happened to the percent of cumulative rainfall that has occurred (Table 3). This data can be found in NOAA Atlas 14, under Supplementary Information: Temporal Distributions (Figure 4).

Control Specifications

Control 1

Arbitrary start and end dates/times were selected to see the overall performance of NW Arterial. Since it was a 6-hr rainfall, the total observation time needed to be only 24-hr (01Jan2000 00:00 to 02Jan2000 00:00). Time Interval of 5 minutes was selected to view relatively precise time-series data.

Paired Data

Storage-Discharge Functions

Information was provided by the client and used for the Storage method in Basin Models (Table 2).

Elevation-Storage Functions

Information was provided by the client and used for the Storage method in Basin Models (Table 2).

Percentage Curves

6-hr Percentage Curve:

Data retrieved from NOAA Atlas 14 that gives the cumulative rainfall over the 6-hr rainfall. This data was used in the Meteorologic Models: Hypothetical Storm: Storm Pattern.

Figures

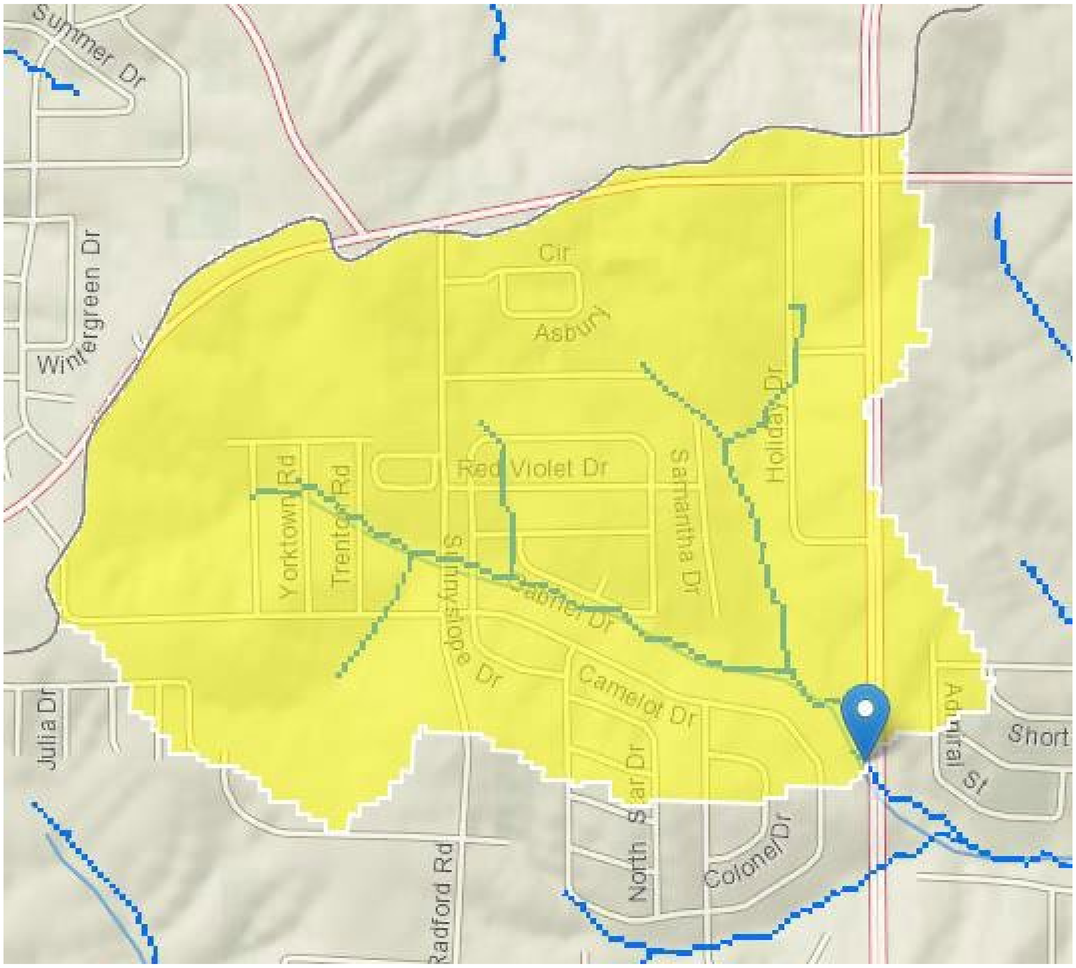


Figure 1: USGS StreamStats delineation tool used for drainage area, curve number, and lag time

NRCS Water Lag Method Using USGS StreamStats for Iowa											
StreamStats Overall Basin Characteristics											
Entire Area											
DRNAREA		BSLDEM10M		LC11CRPHAY	LC11DEV	LC11IMP		SSURGOA	SSURGOB	SSURGOC	SSURGOD
(mi^2)	(acre)	(%)		(%)	(%)	(%)		(%)	(%)	(%)	(%)
0.56	358.4	8.09		0.32	99.6	33.9		0	86.7	13.3	0
Approximate NRCS Weighted Curve Number											
			0	86.7	13.3	0	100				
	Landuse	*Area (%)*	*CN-A*	*CN-B*	*CN-C*	*CN-D*	*CN*	*A*CN*	*Land Use/Land Cover*		
	Crops	0.32	71	80	87	90	80.93	25.90	Row crops: SR+CR: Poor		
	Impervious	33.9	98	98	98	98	98.00	3322.20	Impervious Areas		
	Grassland	65.78	39	61	74	80	62.73	4126.31	Open Space: Good Condition		
	Sum	100					*CN*	74.74			
NRCS Watershed Lag Time of Concentration (Approximate Flow Length)											
Area		*L*	*S*	*Y*	t_l	t_c	t_c				
(ac)		(ft)	(in)	(%)	(h)	(h)	(min)				
358.4		7124.708276	3.37898	8.09	0.63	1.05	62.87				
					37.72						

Figure 2: Excel calculations using info from USGS StreamStats to find curve number and lag time

Table 1: Excel calculations used to find peak flow (Q) and Index Flow

C	0.35	
i	0.99	in/hr
A	0.56	mi^2
Q=	C*i*A	
Q=	125.2205	cfs
Index Flow=	62.61024	cfs

Table 2: stage-storage and storage-discharge functions provided by client

NW Arterial Detention Basin			
Elevation (ft)	Storage (ac-ft)	Storage (ac-ft)	Discharge (cfs)
818	0	0	0
818.5	0.001	0.001	1.48
819	0.01	0.01	5.35
819.5	0.03	0.03	10.53
820	0.06	0.06	15.12
820.5	0.11	0.11	18.52
821	0.18	0.18	21.38
821.5	0.32	0.32	23.91
822	0.56	0.56	26.19
822.5	0.92	0.92	28.29
823	1.41	1.41	30.24
823.5	2.03	2.03	32.08
824	2.78	2.78	33.81
824.5	3.69	3.69	35.46
825	4.8	4.8	37.04
825.5	6.07	6.07	38.55
826	7.49	7.49	40.01
826.5	9.06	9.06	41.41
827	10.8	10.8	42.77
827.5	12.73	12.73	44.08
828	14.88	14.88	45.36
828.5	17.23	17.23	46.6
829	19.8	19.8	47.82
829.5	22.55	22.55	49
830	25.44	25.44	50.15
830.5	28.44	28.44	51.28
831	31.51	31.51	52.38
831.5	34.65	34.65	53.46
832	37.87	37.87	54.52
832.5	41.18	41.18	55.56
833	44.57	44.57	56.58
833.5	48.05	48.05	57.58
834	51.64	51.64	58.56
834.5	55.33	55.33	74.33
835	59.11	59.11	102.33
835.5	63	63	130
836	66.99	66.99	147.88
836.5	71.07	71.07	158.89
837	75.26	75.26	168.91
837.5	79.56	79.56	178.19
838	83.97	83.97	198.67
838.5	88.5	88.5	368.42
839	93.14	93.14	633.17
840	102.78	102.78	1350.73

Note: Storage is repeated as that is how it is input into software

Table 3: Temporal Distribution for 6-hr storm in Dubuque, Iowa for all cases

Time Elapsed	Percent of occurrence
%	90%
0.00	0
8.33	1.47
16.67	3.77
25.00	8.59
33.33	14.06
41.67	21.24
50.00	30.7
58.33	41.37
66.67	53.27
75.00	66.47
83.33	78.82
91.67	89.47
100.00	100

Figure 4: Temporal Distribution location in NOAA Atlas 14

Appendix G DIC2 Basin Analysis

DIC2 HEC-HMS Software Analysis

Basin Models – DIC2

DIC2

The watershed for the DIC2 basin has an area of 0.18 mi^2 which was determined from USGS StreamStats delineation (Figure 1.1)

Loss Method

SCS Curve Number – the approximate NRCS weighted curve number was utilized in Microsoft Excel (Figure 2) and the calculated StreamStats basin characteristics (Figure 1.2) to calculate a curve number of 72.3. Within the Loss Method, impervious area is set to zero because of the curve number already accommodating the impervious area with the watershed.

Transform Method

SCS Unit Hydrograph – the NRCS watershed lag time of concentration (Figure 3) calculated the lag time of 34.1 minutes within the DIC2 basin.

Reach

The upstream reach's length of approximately 3606 feet runs from a commercial development with runoff slopes of 2% to 6% that discharges into the DIC2 Basin. The hydraulic structure within the basin has a primary and secondary inlet, and an emergency spillway. The singular outlet structure discharges into a downstream reach with an approximate 2% bed slope. The downstream reach has an approximate length of 1035 feet before discharging into the Middle Fork Catfish Creek.

Routing Method

Muskingham-Cunge:

The parameters for the downstream Muskingham-Cunge routing method are as follows:

- Length: 1035 feet
- Main Channel Manning's: 0.035 [Look up table (Figure 4), straight channel with stones]
- Left and Right Bank Manning's: 0.10 [Look up table (Figure 5), medium to dense brush]
- Eight Point cross-section (Figure 6): Collected from Field Survey
- Bed Slope: 0.023

The rational method was utilized to calculate the index flow (Figure 7). With an assumed negligible base flow (zero), the peak flow was calculated using Equation 1 where Q is the runoff due to rainfall in cubic feet per second [CFS], C is the weighted runoff coefficient determined using look up tables (Figure 8), i is the rainfall intensity chosen by the team [0.99 inches per hour], and A is the area of the watershed in acres [ac].

$Q=CiA$ (Equation 1)

The values used in Equation 1 and Figure 5 are as follows:

C: 0.89
i: 0.99 in/hr
A: 115.2 ac

Peak flow was calculated to be 101.5 CFS which was averaged with the baseflow to yield and index flow of 50.8 CFS.

DIC2 Basin Characteristics

Storage Method: Elevation-Storage which was calculated (Figure 9) using 2-foot contours from the State of Iowa Open Geospatial Data portal of Dubuque County, Iowa.

Outlets

The outlet structure was required due no current Storage-Discharge information for the DIC2 basin. The method of discharge was a singular circular concrete culvert outlet. The scale was a groove end entrance with the pipe projecting from fill. The following parameters were acquired from the field survey completed by the team:

- Length = 50 feet
- Diameter (Outlet) = 3.5 feet
- Inlet Elevation = 797.221 feet
- Entrance Coefficient = 0.2 [Look up table (Figure 10)]
- Outlet Elevation = 794.824
- Exit Coefficient = 1.0 (assumed)
- Manning's n = 0.012 [Look up table (Figure 11)]

Dam Tops

Additional information is needed for the model to create a Storage-Discharge function. The embankment height recorded during the field survey was 803.5 feet with a length of approximately 300 feet and a weir coefficient estimated at 2.65. The embankment is assumed to give a level overflow during emergency spillway activation.

Meteorologic Models

Met 1

The Hypothetical Storm was used for precipitation.

Hypothetical Storm

User-Specific Pattern was used for the Method due to the control over the storm duration [6 hours] and point depth [5.97 inches]. Traditionally, the SCS Storm Type 2 would be chosen, but the default duration for such an event is a 24-hour storm rather than a 6-hour storm.

Storm Pattern

From NOAA Atlas 14, a 6-hour temporal curve relates the distribution of the point depth to time resulting in a cumulative rainfall event within the software. The temporal curve is a region-specific parameter found in NOAA Atlas 14 under Supplementary Information: Temporal Distributions (Table 1).

Control Specifications

Control 1

Arbitrary start and end dates/times were selected to see the overall performance of the DIC2 Basin. Due to the 6-hour storm being tested, the total observation time needed to be only 24 hours (01Jan2000 00:00 to 02Jan2000 00:00). Time interval of 5 minutes was selected to view higher frequency time-series data since the duration window was relatively small.

Paired Data

Elevation-Area-Storage

Information was calculated using 2-foot contour data from the Iowa Open Geospatial Data portal. The Frustrum of a Period Method was utilized in calculating the volume from the area.

Percentage Curves

6-hour Percentage Curves:

Data retrieved from NOAA Atlas 14 gives the cumulative rainfall over the 6-hour rainfall. The data was used in the Meteorologic Models: Hypothetical Storm: Storm Pattern.

Eight Point Cross-section

Survey data collected by the team included a downstream cross-section that would represent the general shape of the channel.

Figures and Tables

StreamStats Report

Region ID: IA
Workspace ID: [illegible]
Clicked Point (Latitude, Longitude): 42.45873, -90.74294
Time: [illegible]

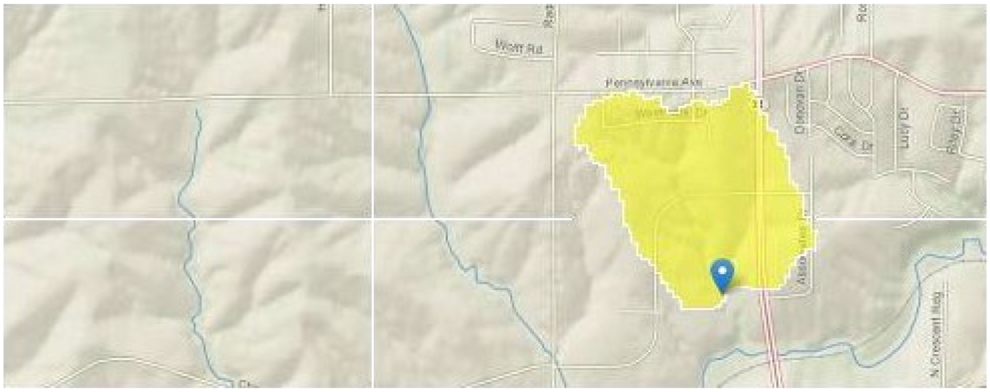


Figure 1.1: StreamStats report illustrating the watershed for the DIC2 basin and the drainage area

Parameter Code	Parameter Description	Value	Unit
SSURGOA	Percentage of area of Hydrologic Soil Type A from SSURGO	0	percent
SSURGOB	Percentage of area of Hydrologic Soil Type B from SSURGO	90	percent
SSURGOC	Percentage of area of Hydrologic Soil Type C from SSURGO	10	percent
SSURGOD	Percentage of area of Hydrologic Soil Type D from SSURGO	0	percent

Figure 2: StreamStats report describing the basin hydrologic characteristics

NRCS Water Lag Method Using USGS StreamStats for Iowa										
StreamStats Basin Characteristics										
DRNAREA	BSLDEM10M		LC11CRPHAY	LC11DEV	LC11IMP		SSURGOA	SSURGOB	SSURGOC	SSURGOD
(mi^2)	(%)		(%)	(%)	(%)		(%)	(%)	(%)	(%)
0.18	10.61		0	100	28		0	90	10	0
Approximate NRCS Weighted Curve Number										
		0	90	10	0	100				
Landuse	*Area (%)*	*CN-A*	*CN-B*	*CN-C*	*CN-D*	*CN*	*A*CN*	*Land Use/Land Cover*		
Crops	0	71	80	87	90	80.7	0.0	Row crops: SR+CR: Poor		
Impervious	28	98	98	98	98	98.0	2744.0	Impervious Areas		
Grassland	72	39	61	74	80	62.3	4485.6	Open Space: Good Condition		
Sum	100					*CN*	72.3			

Figure 1.2: determining the NRCS curve number using the NRCS Water Lag Method within Microsoft Excel

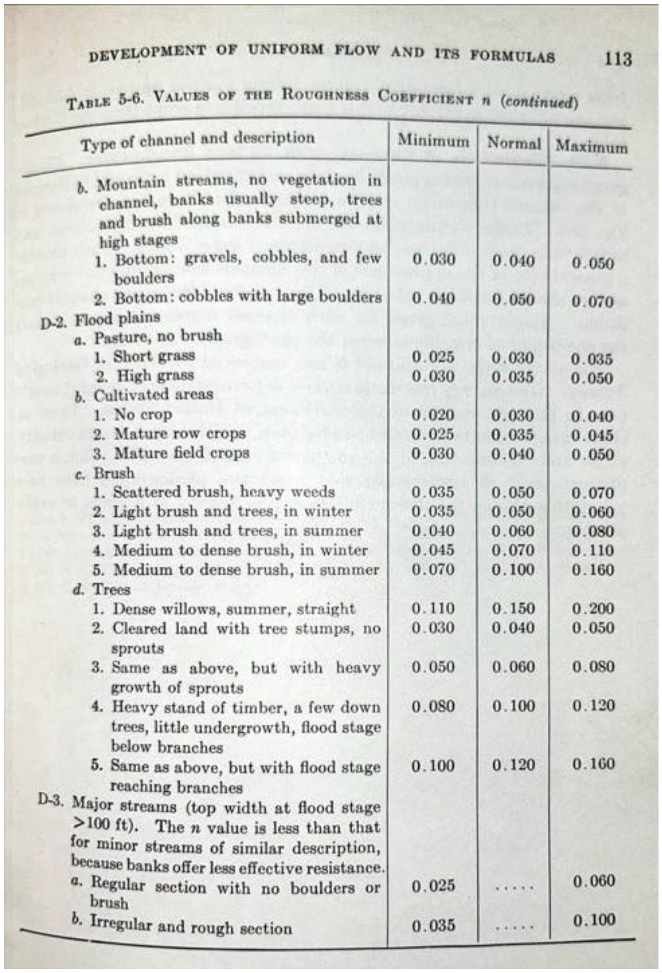
NRCS Watershed Lag Time of Concentration (Approximate Flow Length)					
Area	*L*	*S*	*Y*	t_c	t_c
(ac)	(ft)	(in)	(%)	(h)	(min)
115.2	3606	3.832	10.61	0.34	34.1

Figure 3: calculating the time of concentration [t_c] by using parameters determined by StreamStats and calculated using the NRCS storage equation

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n (*continued*)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	**0.022**	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	**0.030**	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

Figure 4: determining Manning's n for main channel from page 112 in *Open-Channel Hydraulics* by Chow



DEVELOPMENT OF UNIFORM FLOW AND ITS FORMULAS 113

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
D-3. Major streams (top width at flood stage >100 ft). The *n* value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

Figure 5: determining Manning's n for left and right bank from page 113 in *Open-Channel Hydraulics* by Chow

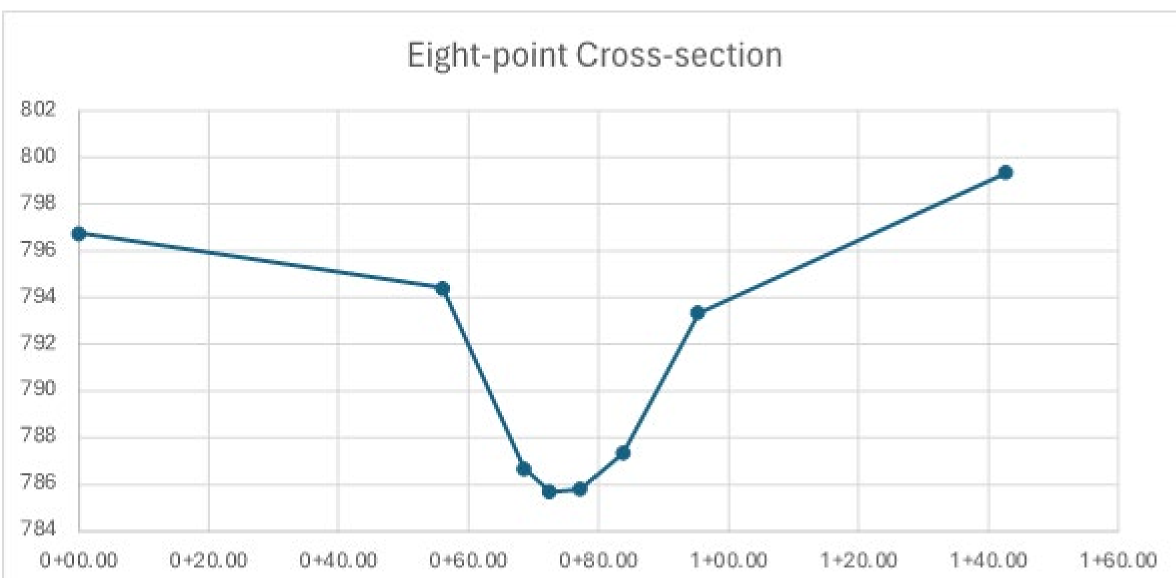


Figure 6: eight-point cross-section collected during field survey

Runoff Coefficient (Weighted for 90% Soil Group B and 10% Soil Group C with an average slope between 2-6% in Commercial)	0.89
Area [ac]	115.2
Intensity, I [in/hr]	0.99
Runoff, Q [CFS]	101.5
Index Flow [CFS]	50.8

Figure 7: rational method used for the index flow calculation

Urban Land Use	Lower Bounds	Upper Bounds
Parking	0.85	0.96
Commercial	0.71	0.89
Streets	0.70	0.91
Industrial	0.67	0.86
Residential Lots, High Density	0.25	0.54
Residential Lots, Medium Density	0.19	0.50
Residential Lots, Low Density	0.14	0.46
Railway Yards	0.20	0.35
Playgrounds	0.20	0.30
Sports Fields	0.20	0.35
Parks	0.10	0.25
Cemetaries	0.10	0.25

Figure 8: look up tables of runoff coefficient, C, for rational method from CivilWeb Spreadsheets – Engineering Calculations & Spreadsheets

Topographic Data		Frustrum of a Pyramid Method		Frustrum of a Pyramid Method	
Elev	Area	DV	V	DV	V
ft	ft2	ft3	ft3	Acre-ft	Acre-ft
797	5494.3		0		
		34745		0.80	
800	33130.9		34745		0.80
		91492		2.10	
802	59651.2		126237		2.90
		134753		3.09	
804	75409.3		260990		5.99
		162915		3.74	
806	87659.7		423905		9.73

Figure 9: elevation-Storage Data calculated by the Frustrum of a Pyramid method using 2-foot contours

Type of Structure and Design of Entrance	Coefficient, k_{en}
Concrete Pipe Projecting from Fill (no headwall):	
Socket end of pipe	0.2
Square cut end of pipe	0.5
Concrete Pipe with Headwall or Headwall and Wingwalls:	
Socket end of pipe (grooved end)	0.2
Square cut end of pipe	0.5
Rounded entrance, with rounding radius = 1/12 of diameter	0.2
Concrete Pipe:	
Mitered to conform to fill slope	0.7
End section conformed to fill slope	0.5
Beveled edges, 33.7 or 45 degree bevels	0.2
Side slope tapered inlet	0.2
Corrugated Metal Pipe or Pipe-Arch:	
Projected from fill (no headwall)	0.9
Headwall or headwall and wingwalls square edge	0.5
Mitered to conform to fill slope	0.7
End section conformed to fill slope	0.5
Beveled edges, 33.7 or 45 degree bevels	0.2
Side slope tapered inlet	0.2

Figure 10: entrance coefficient Look up tables from HEC-RAS Hydraulic Reference Manual

DEVELOPMENT OF UNIFORM FLOW AND ITS FORMULAS 111

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	**0.012**	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	**0.013**	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	**0.013**	0.015
2. In cement mortar	0.012	**0.015**	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	.....	0.500

Figure 11: material roughness table from page 111 in *Open-Channel Hydraulics* by Chow

Table 1: 6-hr temporal distribution acquired from NOAA Atlas 14

Percent (%)	Percent (%)
0	0
8.3333	1.47
16.667	3.77
25	8.59
33.333	14.06
41.667	21.24
50	30.7
58.333	41.37
66.667	53.27
75	66.47
83.333	78.82
91.667	89.47
100	100

Table 2: field survey data collected by the team

Description	Point Number	Northing	Easting	Elevation [ft] (NAVD 88)
VOID	10	5664688.536	3655990.532	843.191
VOID	11	5664688.536	3655990.532	843.191
pipe primary inlet, 3 foot size	100	5664731.726	3655428.73	797.221
pipe secondary inlet, 3 foot by 64 in square inlet	101	5664733.113	3655422.199	802.292
pipe primary outlet, 3.5 feet size	102	5664758.768	3655388.228	794.824
pipe primary outlet, knickpoint	103	5664761.276	3655383.913	794.391
pipe primary outlet, knickpoint	104	5664760.982	3655383.43	790.567
knickpoint	105	5664774.629	3655361.135	790.682
knickpoint	106	5664776.002	3655360.307	789.095
main channel flowline	107	5664801.832	3655318.958	786.44
Extra Point	108	5664768.78	3655296.026	794.976
DS Channel Cross-Section - Embankment R	109	5664732.679	3655262.869	802.02
DS Channel Cross-Section Mid-Embankment R	110	5664742.514	3655268.138	799.39
DS Channel Cross-Section - Bank R	111	5664781.21	3655298.367	793.373
DS Channel Cross-Section - Toe of Bank R	112	5664792.363	3655301.656	787.347
DS Channel Cross-Section - Main Channel Width R	113	5664798.091	3655305.123	785.803
DS Channel Cross-Section - Main Channel Width L	114	5664802.416	3655306.852	785.689
DS Channel Cross-Section - Toe of Bank L	115	5664805.897	3655309.034	786.681
DS Channel Cross-Section - Bank L	116	5664818.131	3655312.064	794.409
DS Channel Cross-Section Mid-Embankment L	117	5664871.339	3655329.679	796.776
top of emergency spillway	118	5664745.83	3655413.117	803.528

Appendix H DICW Basin Analysis

DRAINAGE AREAS
Seippel : 1.35 mi^2
Bergfeld : 4.15 mi^2

Reasoning: Drainage basin area, used Streamstats to find area for both, subtracted Bergfeld DA (4.09), then subtracted Seippel (1.35) and added back .06 to Bergfeld for the neglected drainage area. Also reflects the similar values from city plans.

STAGE-STORAGE RELATIONSHIP
Located in "Industrial Center West (Bergfeld Pond) Calcs, 1998"
Bergfeld Pond Relationship: Page 61
Seippel Pond Relationship: Page 127

TIME OF CONCENTRATION / CURVE NUMBER
NRSC Lag-Method

Needed info for all reaches
BASEFLOW
We are currently going with 0 baseflow but will change it later.

SEIPPEL OULET
2 8ft x 8ft culvert barrels
N = .012 concrete box culverts flowing full

Appendix I Inundation Maps

TEAM DESIGN
836.3 MSL

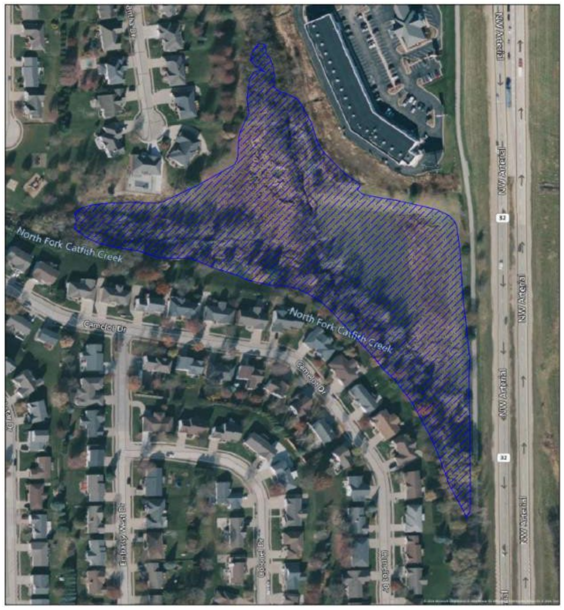


Figure 1: NW Arterial team design storm inundation map

SUDAS DESIGN
836.5 MSL

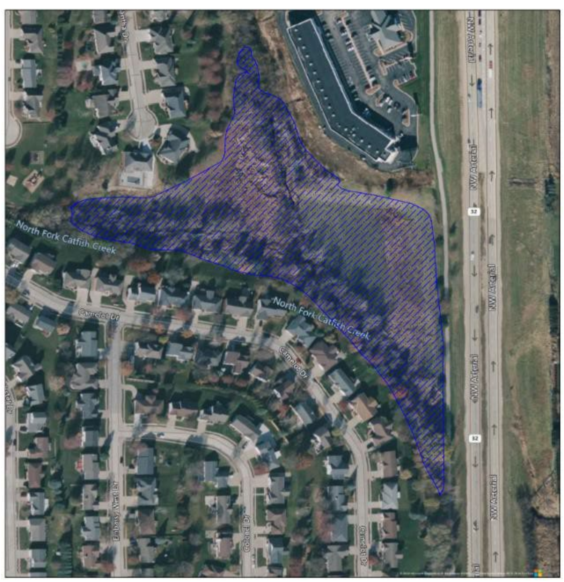


Figure 2: NW Arterial SUDAS design storm inundation map

TEAM DESIGN
802.5 MSL

Figure 3: DIC2 team design storm inundation map

SUDAS DESIGN
803.9 MSL

Figure 4: DIC2 SUDAS design storm inundation map

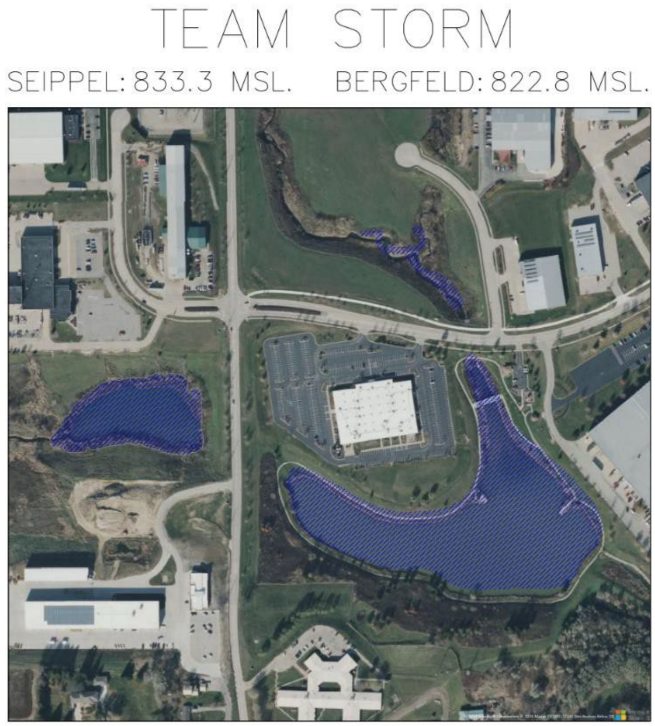


Figure 5: DIC2 team design storm inundation map

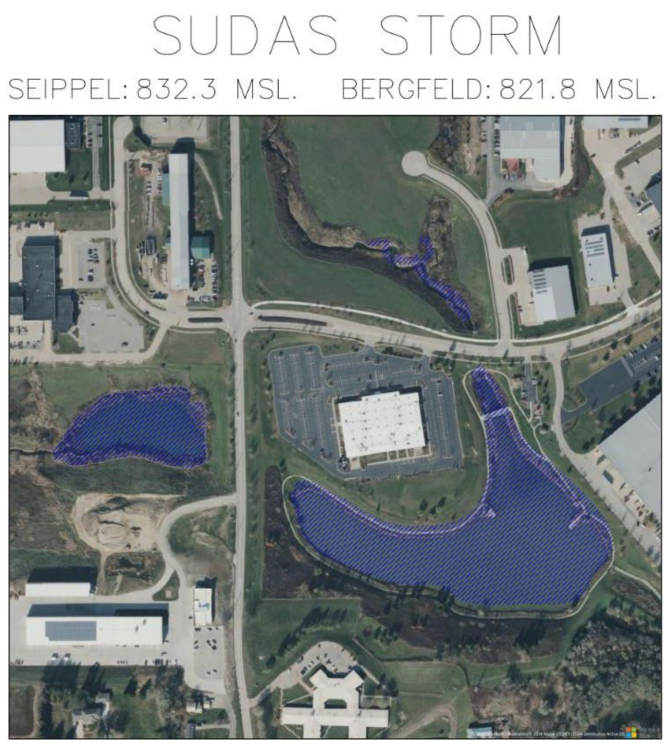


Figure 6: DIC2 SUDAS design storm inundation map

Appendix J Citations

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Design Drawings

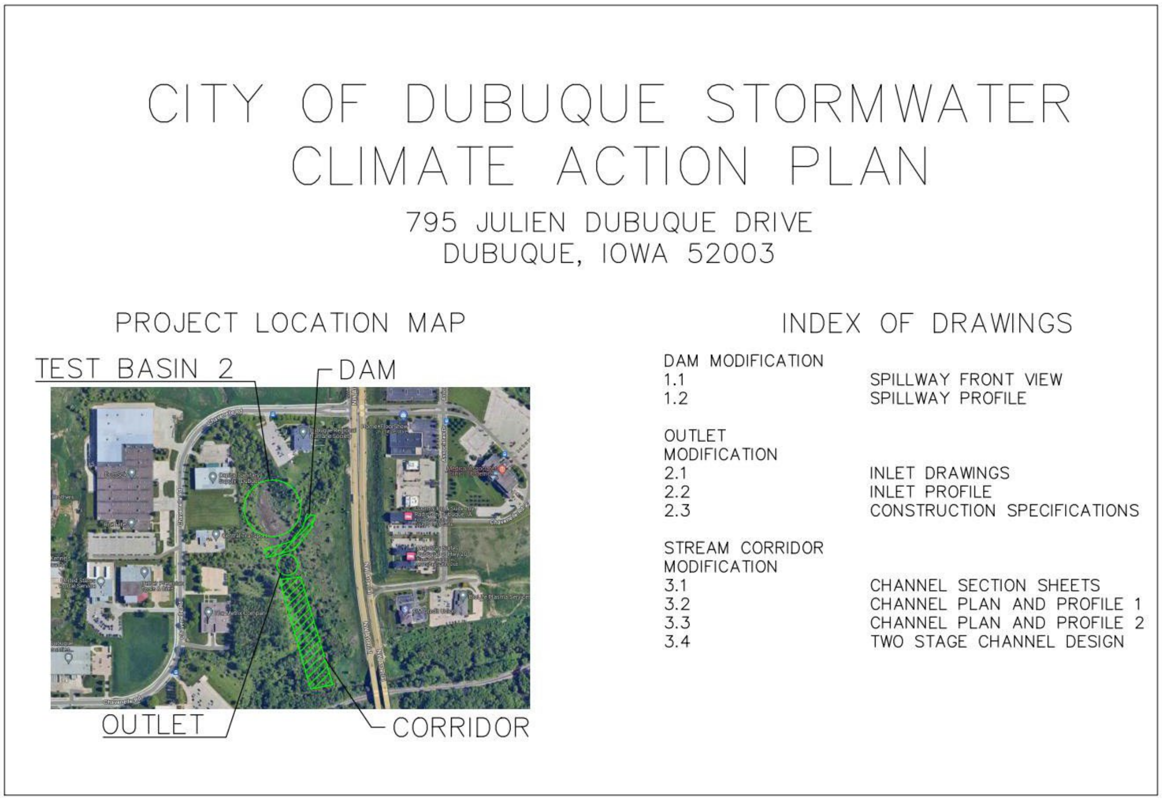

Figure 1: title sheet

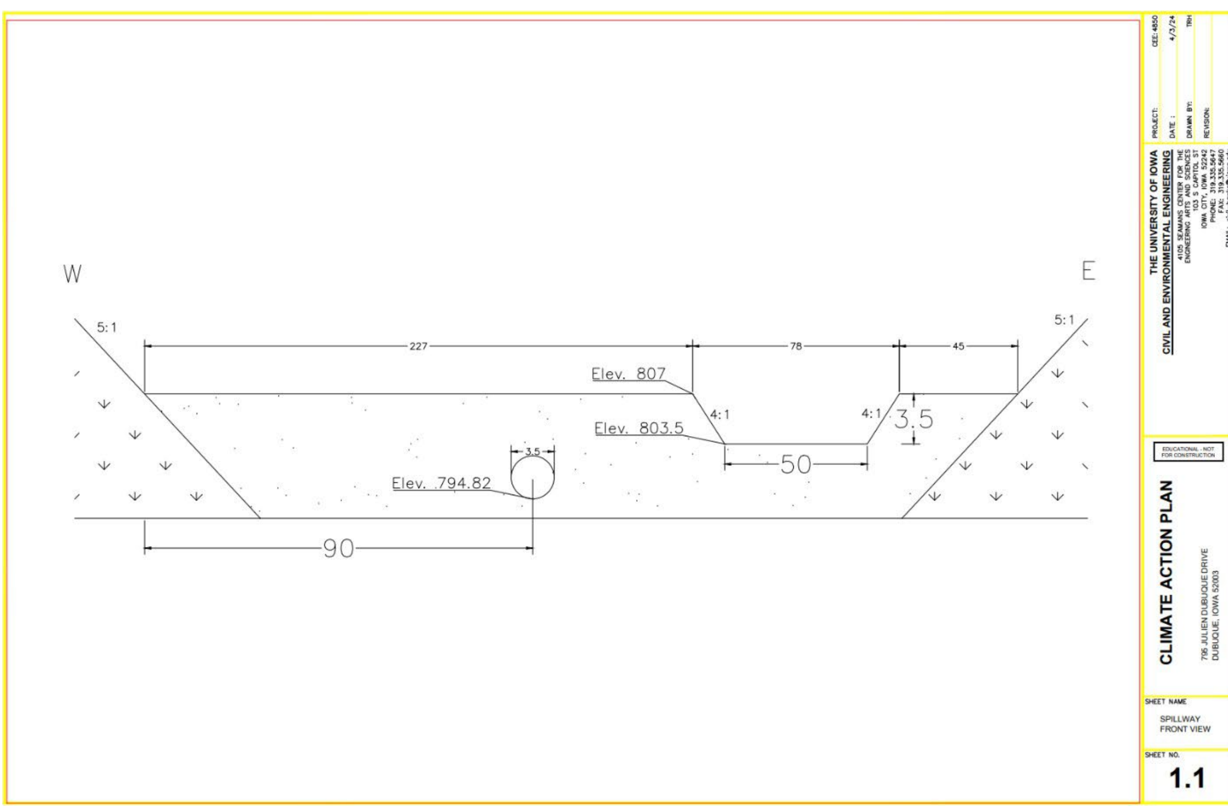

Figure 1.1: spillway redesign front view

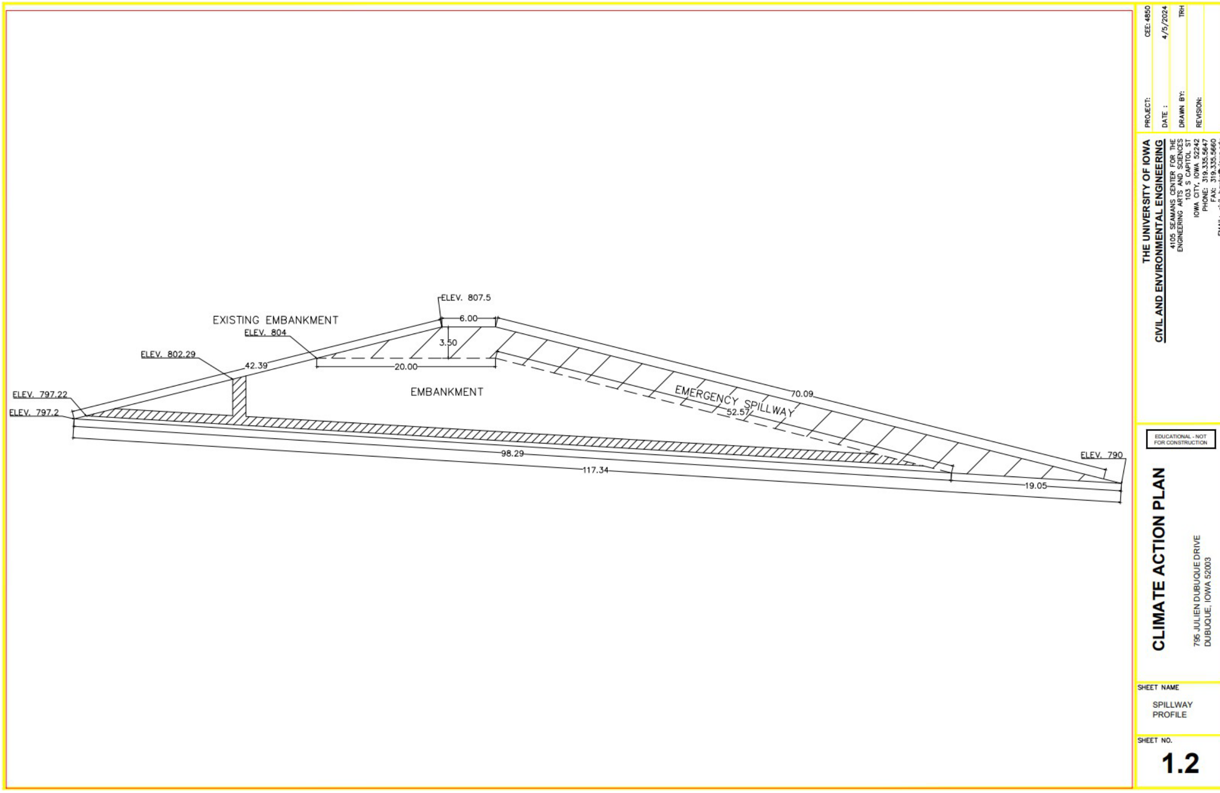

Figure 1.2: spillway redesign profile

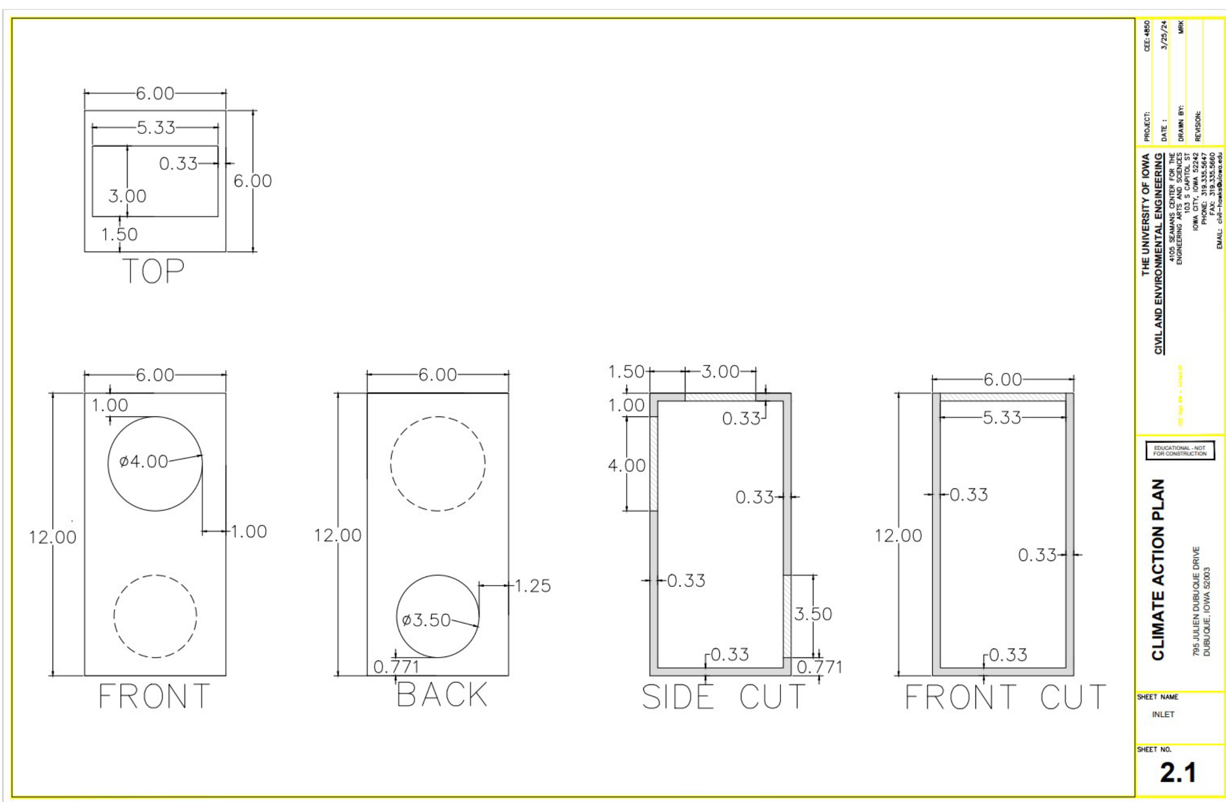

Figure 2.1: outlet structure drawing with dimensions

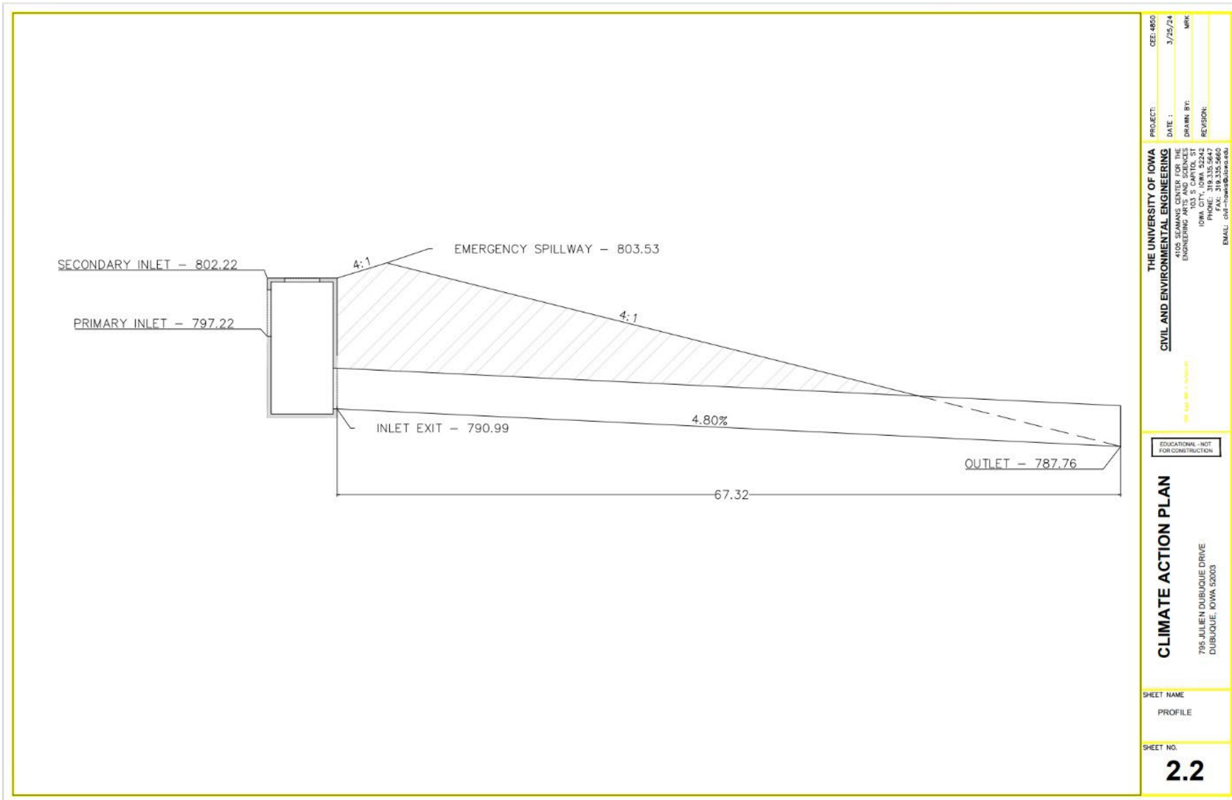

Figure 2.2: outlet structure profile with elevations

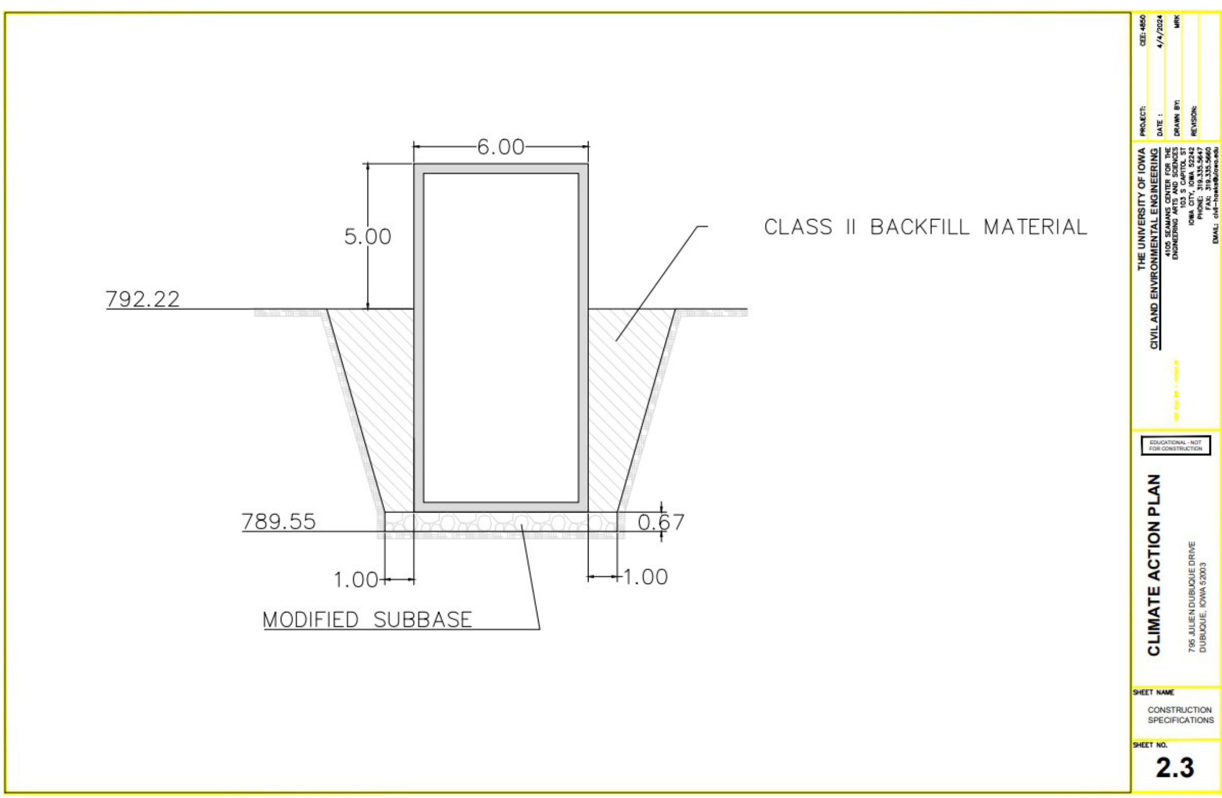

Figure 2.3: construction specifications for outlet structure

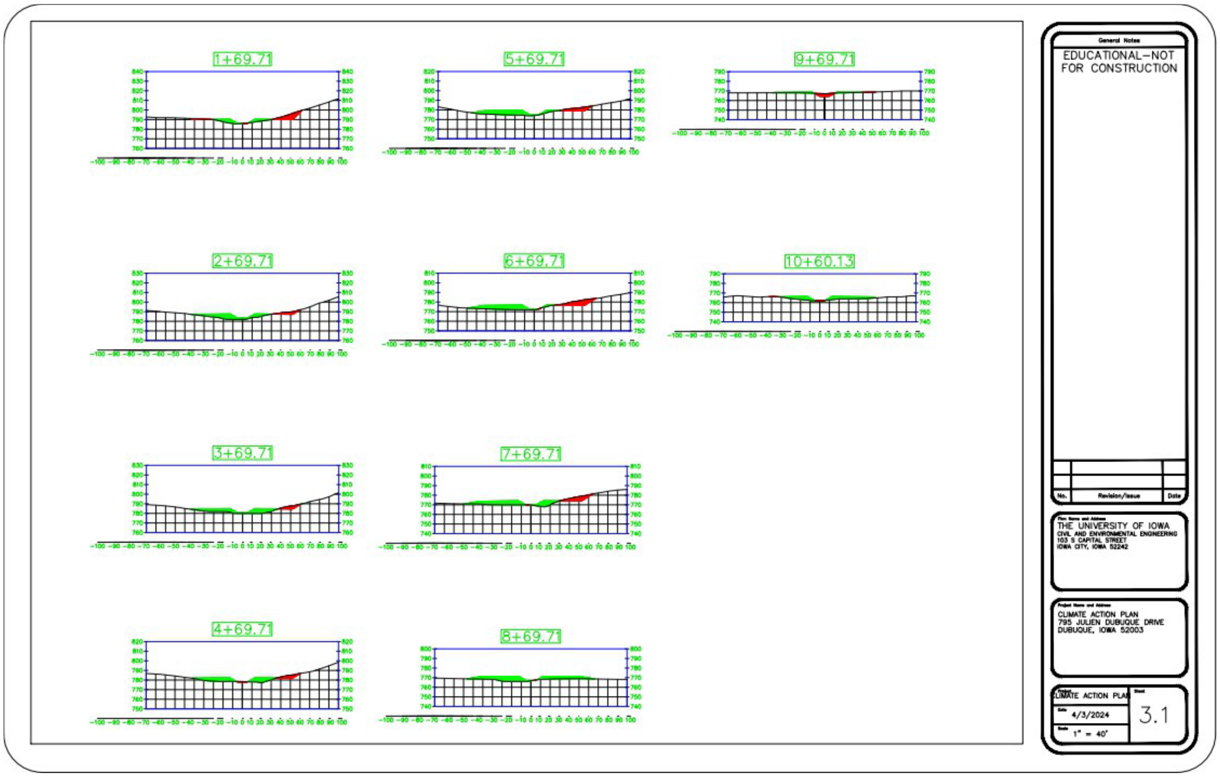

Figure 3.1: DIC2 downstream channel section views with cut and fill

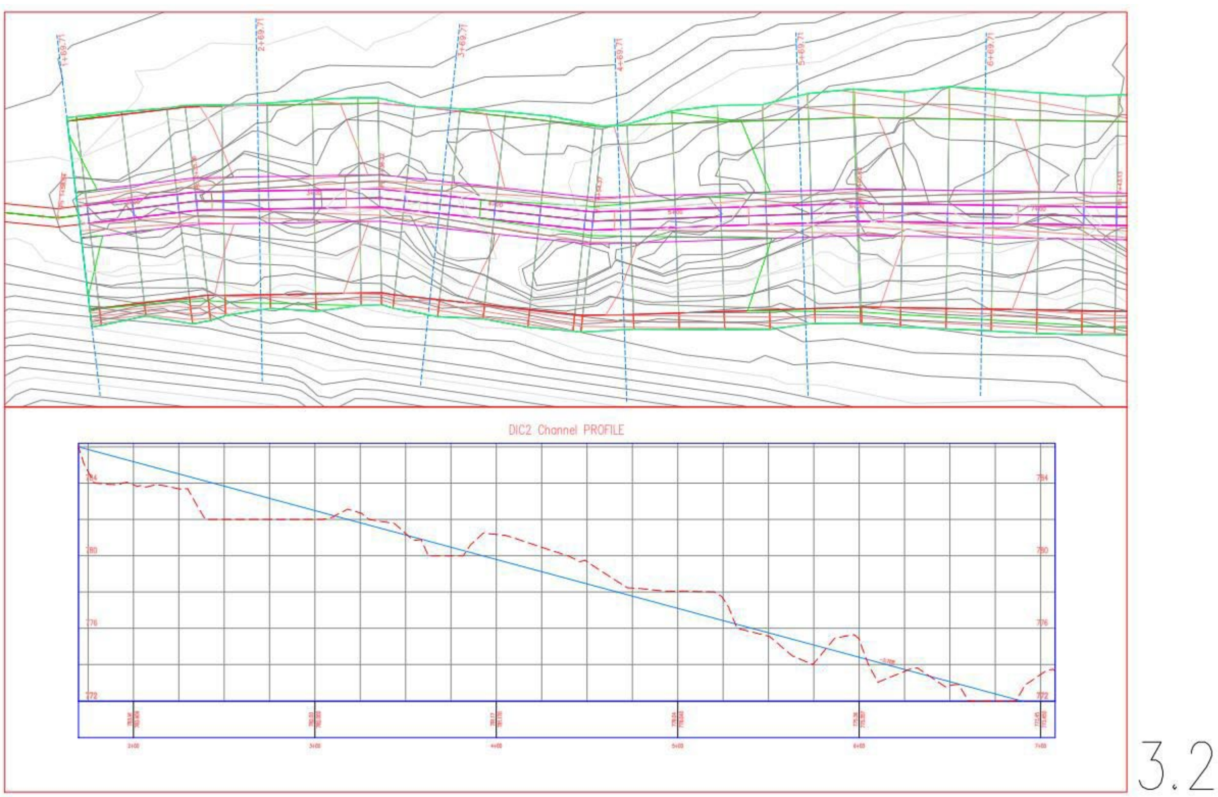

Figure 3.2: DIC2 downstream channel plan and profile

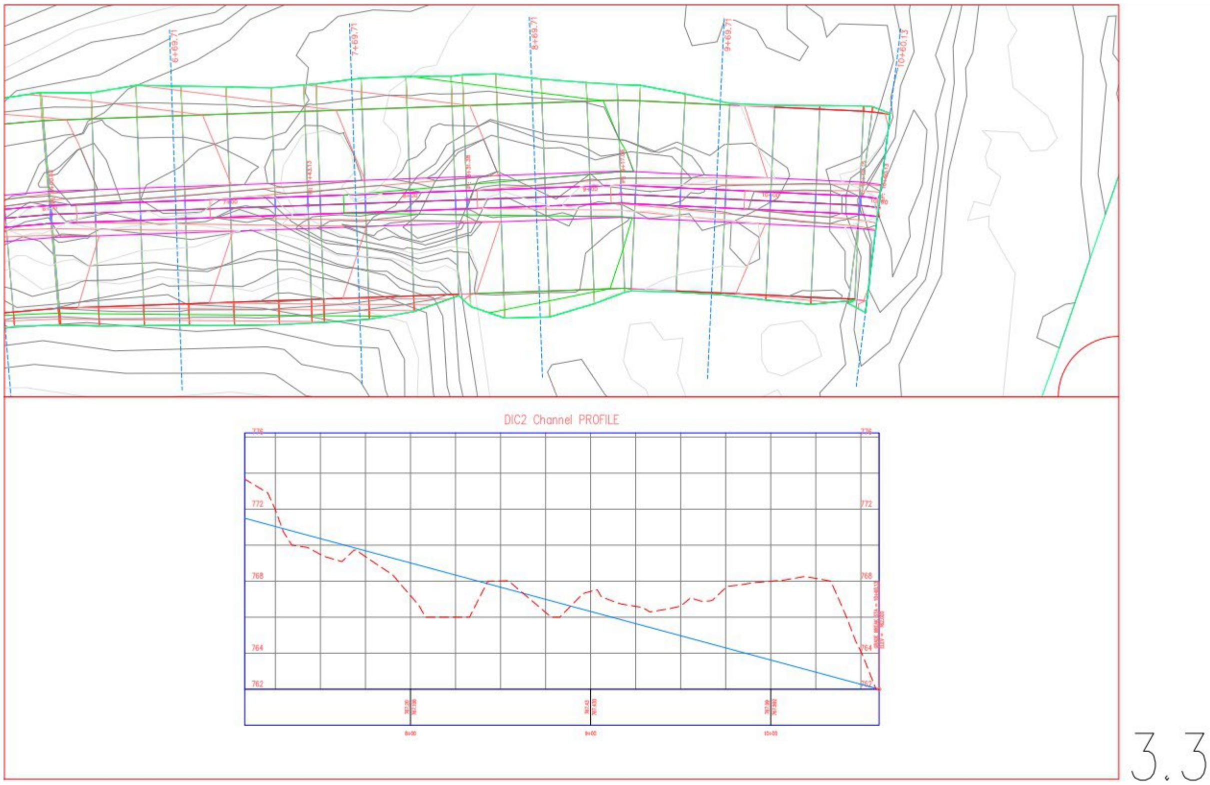

Figure 3.3: DIC2 downstream channel plan and profile

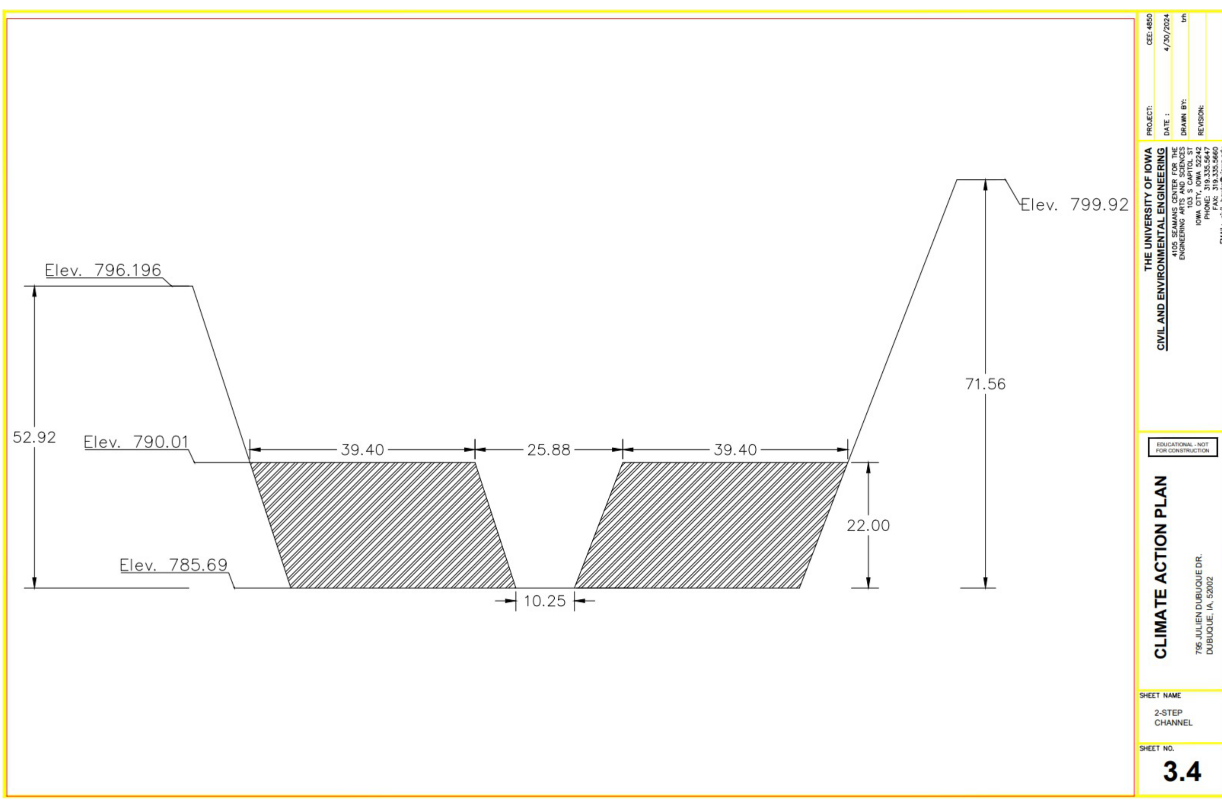

Figure 3.4: DIC2 2-Stage Channel Design