ENGR 491

Senior Design

Spring 2022

Administration Building for the State Bank of Whittington

Drainage and Structural Design

JPM

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ABSTRACT

The purpose of this project was to redevelop the site and build an administrative building for the State Bank of Whittington. The preexisting site consisted of five single-residential homes, one small commercial building, and two small parking lots. These structures were demolished, and the parking lots were removed. The proposed building is an 8,305 square foot, single story building that was designed to equip offices and conference rooms. The drainage system for the site collects all storm water runoff from the roof, parking lot, and surrounding areas into two detention basins that drain into the preexisting city storm sewer. In addition to the site design, the administration building was structurally designed. The post construction site will consist of one, steel frame, administration building, one parking lot, and a sufficient draining system. Collectively, the site and structural aspects were designed to meet the project and community's needs.

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1. INTRODUCTION

The City of Benton proposed a new administration / operation building for the Whittington Bank near the existing State Bank of Whittington. The drainage and structural aspects of the project need to be designed. This project is located in Benton, Illinois which is approximately 68 miles west of the University of Southern Indiana (USI). The aerial view of the project is shown below in Figure 1.

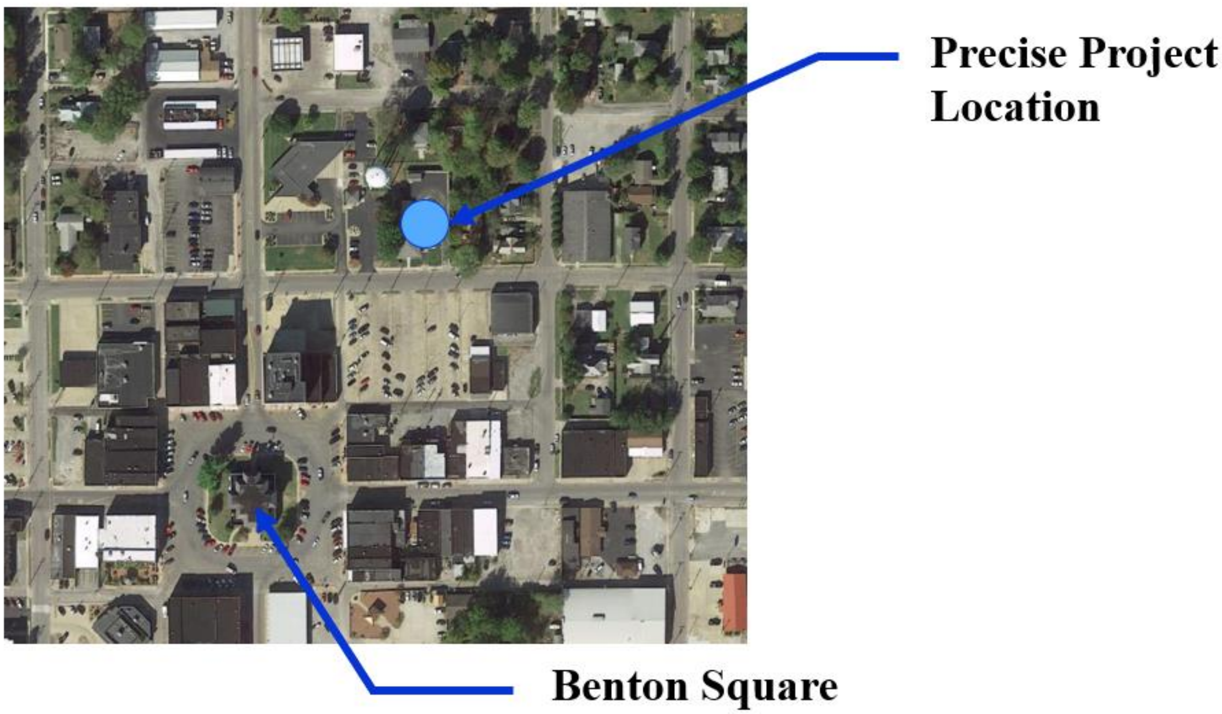


***Figure 1**: Ariel view of project site*

As engineers, public welfare is always held as a priority when designing, inspecting, estimating, etc. Therefore, the project is being designed and constructed with the safety and well-being of the public in mind. The design of the building will carefully follow the steel design code to ensure that the building will be safe to inhabit. Safety factors are always used in design calculations to ensure that the capacity of the structural frame exceeds the demand from the anticipated loads.

In addition, the project is changing the current landscape and the existing topography. This will create changes in the drainage of the project location. Careful design and calculations will be done to account for the proposed plan and make sure that the site will drain properly and be able to handle extreme rainfall events. The project location is near residential houses, and it needs to be guaranteed that the changes in the site's features do not create drainage issues that flood those houses.

The construction of the building will have beneficial economic impacts on the surrounding community. Utilization of local businesses to provide construction materials and supplies will be helpful to the community.

The State Bank of Whittington also has great values, so the community would be served well by a company that values all their customers. These banks have served the people of Southern Illinois since 1919 and plan to continue to do so by adding an administration building and their ultimate goal is to, "to provide our customers with the best customized service possible." Therefore, this building would continue to help the community socially, culturally, and economically. This project will be safely completed with these benefits kept in mind.

For Illinois, the state has adopted the 2015 International Building Code (IBC) which includes American Society of Civil Engineer (ASCE) 7-10 code and American Institute of Steel Construction (AISC) 14th edition. ASCE 7-10 is the code for minimum design loads for buildings and other structures. These codes will be used to be sure to protect the public by setting up the minimum acceptable level of safety for the building and the surrounding area. The software is used to compute design values and to model the design. The codes will be referenced periodically in this report.

Codes:

- IBC 2015
- AISC
- ASCE 7-10
- City of Decatur Drainage Policy
- USDA TR-55 Manual

Software:

- Civil 3D
- Autodesk Storm and Sanitary Analysis
- Rapid Interactive Structural Analysis (RISA)
- Revit
- Excel

This report outlines the analyses, design, and estimating which began in January of 2022.

2. PROJECT DELIVERY TEAM

Once the project deliverables were analyzed by all three team members, the scope of work was outlined for the project. The scope of work highlighted each individuals' strengths and interests. For that reason, Paul was the teams' drainage engineer and estimator. Madelyn and Justin were the teams' structural engineers. Figure 2 below shows how the team was organized and how communication worked. Meetings were arranged with the team when problems developed.

Figure 2: *Organizational chart of the project delivery team.*

Collectively, the team is referred to as JPM where each letter in JPM represents each team members first name. JPM brings the qualities needed to obtain success and effective designs in projects. JPM embodies engineering experience and skill which implements those characteristics in real world applications where the main goal is to design and improve the community's infrastructure, while keeping the public's welfare as the top priority. Therefore, the team has the necessary skills, education, and teamwork to successfully design this project.

3. Structural Design

The analysis and design of the administration building will follow the Load and Resistance Factor Design (LRFD) design standards, which is modern and more commonly used. LRFD uses ultimate level loads rather than service level loads which Allowable Strength Design (ASD) uses. LRFD takes the service level loads and applies certain load design factors (that are greater than 1) to acquire an ultimate limit. Nevertheless, the structure was analyzed using LRFD design standards.

3.1. Load Design Scenarios

Before designing members and analyzing the structure, load design scenarios had to be evaluated. Therefore, all the loads were calculated and used in the basic load combinations from ASCE 7-10 on page 7. Structures, components, and foundations should be designed so that demand loads are less than the capacity. The load combinations for LRFD include the following:

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.5W)
4. 1.2D + 1.0W + L + 0.5(L_r or S or R)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.0W
7. 0.9D + 1.0E

Where D represents the dead load, L represents the live load, L_r is the roof live load, S is the snow load, R serves as the rain load, W acts as the wind load, and E is the seismic, or earthquake load.

Each load discusses previously will be analyzed and tabulated in the following sections.

3.1.1. Dead Load

Dead load includes all self-weight of every member within the system as well as the weight of every system that is permanently installed in the building. This includes the HVAC system, water and sprinkler, electrical, roof membrane and metal decking on top. TABLE – C3-1 on page 399 from ASCE 7-10 was used to approximate the dead load for the building.

Table 1: Components considered for the deadload

Component	Load (psf)
Deck, Metal, 20 Gage	2.5
Single Ply Waterproofing Membrane	0.7
Mechanical Duct Allowance	4
Insulation - Gypsum Board	2.2
Misc. MEP	10
TOTAL	**19.4**

After evaluating all items that will be permanently installed in the building, the pounds per square foot (psf) for each item was totaled. The anticipated dead load is 19.6 psf, but to be conservative, the value was rounded to 20 psf. The self-weight of the structural members is also included but it is a separate calculation.

The two vaults located in the center of the bank have an additional dead weight associated with them. Each vault has a large concrete cap that encases the vault from above. This load must be accounted for as well. The average normal-weight concrete is 150 pounds per cubic foot (pcf). Since the cap is to be 1 foot (ft) thick, the cap will be 150 psf. Adding this to the 20 psf dead load, the total dead load for the tops of the vaults will be 170 psf.

3.1.2. Roof Live Load

The following equation ASCE 7-10: 4.8-1 is used to determine the roof live load.

$$L_r = L_o * R_1 * R_2 \qquad \text{Eq. 1}$$

Where L_o is the unreduced live load per square foot of horizontal projection supported by the member. R_1 and R_2 are reduction factors that can be added to the live roof load to require a lighter load. L_r is the reduced live load for a given member.

According to Table 4-1 of the ASCE 7-10, the unreduced live roof load for this project will be 20 psf.

Reduction case 1 depends on how large the entire area of the roof is. If the roof of the system is less than 200 square feet (ft^2), no reduction will be added. If the area is within the bounds 200 to 600 ft^2 then the area must be used into the equation below for reduction factor 1.

$$R_1 = 1.2 - 0.001 * A_t \quad \text{Eq. 2}$$

For the roof of the Administration Building for the State Bank of Whittington, both the low and high portions of the roof are both greater than 600 ft^2. When this case happens, a roof live load reduction can be used of 0.6.

Reduction Case 2, R_2, depends on the slope of the roof. Specifically, the variable F will be used to determine rise/run. For the case that a roof slope is less than 4, then live roof R_2 is 1.0. The Whittington Bank's rise over run came out to 0.25, which means that no reduction can be used for R_2.

Solving Equation 2 above for the reduced live load, it comes out to be 12 psf. However, according to the 2015 IBC, roof live loads cannot have reductions added to them in the state of Illinois, so the L_r that will be used for the Bank of Whittington will be 20 psf.

3.1.3. Live Load

Referring to Table 4-1 on page 18 of ASCE 7-10, the occupancy or use of this project is "Office Buildings – Lobbies & First Floor Corridor". Therefore, the live load, L, is assumed to be 100 psf.

3.1.4. Snow Load

The slope of the roof is ¼" per ft; therefore, since the roof has a sloop of $\leq 5^\circ$, the roof was treated as a flat roof for this load scenario. Equation 3 produces the snow load on a flat food which can also be found on page 29 in the ASCE 7-10 code.

$$P_f = 0.7(C_e * C_t * I_S * P_G) \quad \text{Eq. 3}$$

Where P_f is the snow load on a flat roof, C_e is the exposure factor, C_t represents the thermal factor, P_g is the ground snow loads, and I_s is the importance factor.

In addition, the minimum snow load for a low slope roof is also calculated in Equation 4 which is found on page 29 in ASCE 7-10. This is calculated to see which load controls.

$$P_m = I_S * P_G \qquad \text{Eq. 4}$$

Where P_m is the minimum snow load on a low slope roof, P_g is the ground snow loads, and I_s is the importance factor.

First, by using the ASCE 7-10, each variable and constant for the snow equations were found. Then they were inserted into the above equations and solved for.

First, the C_e or exposure factor was found to be 0.9. To find this, first a roughness coefficient had to be found. The table below shows roughness coefficients for different types of building surroundings.

***Table 2**: Exposure Factor, C_e*

	Exposure of Roof[a]		
Terrain Category	Fully Exposed	Partially Exposed	Sheltered
B (see Section 26.7)	0.9	1.0	1.2
C (see Section 26.7)	0.9	1.0	1.1
D (see Section 26.7)	0.8	0.9	1.0
Above the treeline in windswept mountainous areas.	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within a 2-mile (3-km) radius of the site.	0.7	0.8	N/A

The terrain category and roof exposure condition chosen shall be representative of the anticipated conditions during the life of the structure. An exposure factor shall be determined for each roof of a structure.

[a]Definitions: Partially Exposed: All roofs except as indicated in the following text. Fully Exposed: Roofs exposed on all sides with no shelter[b] afforded by terrain, higher structures, or trees. Roofs that contain several large pieces of mechanical equipment, parapets that extend above the height of the balanced snow load (h_b), or other obstructions are not in this category. Sheltered: Roofs located tight in among conifers that qualify as obstructions.

[b]Obstructions within a distance of $10h_o$ provide "shelter," where h_o is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used. Note that these are heights above the roof. Heights used to establish the Exposure Category in Section 26.7 are heights above the ground.

Once roughness B was chosen, it had to be determined how exposed the bank was. This is asking how many buildings butt up against the one being designed as well as how high the buildings are around it. The ASCE 7-10 describes a C_e=0.9 to be a roughness B area that is fully exposed or exposed on all four sides.

Next the thermal factor was found. The table below shows the thermal factor for different buildings uses. From this table C_t was found to be 1.0.

Table 3*:Thermal Factor, C_t*

Thermal Condition[a]	C_t
All structures except as indicated below	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds 25 °F × h × ft²/Btu (4.4 K × m²/W).	1.1
Unheated and open air structures	1.2
Structures intentionally kept below freezing	1.3
Continuously heated greenhouses[b] with a roof having a thermal resistance (R-value) less than 2.0 °F × h × ft²/Btu (0.4 K × m²/W)	0.85

[a]These conditions shall be representative of the anticipated conditions during winters for the life of the structure.
[b]Greenhouses with a constantly maintained interior temperature of 50 °F (10 °C) or more at any point 3 ft above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

Next is the importance factor. To find this, a risk category must be selected. Below is a table that describes how a building could fall in each risk category.

Table 4*: Importance Factors by Risk Category*

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it is dependent on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

The Administration Building for the State Bank of Whittington is a risk category 2. The importance factor for snow loads for a risk 2 building is 1.0. Inserting all the variables, the snow load for the building comes out to be 20 psf.

3.1.5. Snow Drift Load

Snow drift is a case separate from the other load combinations outlined in section 2.1 of this report. This load case will have no safety factor multiplied to it and it will be added to the load combinations.

Snow drift happens in two separate ways depending on which direction the wind is blowing and the geometry of the roof. For this section of the report, the first section will be the windward direction, and the other will be referred to as the leeward direction. For both types of snow drift, the height of the drift is needed, h_f, and then used to find the weight of the snow on the roof. Once the h_f is found for both the windward and leeward cases, the largest one is selected as the controlling load for this new load case.

The lengths of the upper roof (L_u) and lower roof (L_l) are 30 ft and 52 ft, respectively. Using Figure 7-9 in ASCE 7-10 shown in the Appendix A, h_d = 1.5 ft. Section 7.7.1 in ASCE 7-10 explains that Leeward drift height is equal to the drift height, h_d. Windward is taken at ¾ of h_d, which is 2 ft x ¾ = 1.5 ft. The largest of the windward or leeward will control which is h_d = 1.5 ft for this project.

The clear height from top of balanced snow load to closest point on adjacent upper roof, h_c, is found using the architectural plans. h_c = 7 ft.

The snow density, γ, is found by using Equation 7.7-1 from ASCE 7-10 which is shown below in Equation # of this report. γ = 16.6 pcf.

$$\gamma = 0.13P_g + 14; \leq 30\ pcf \qquad \text{Eq. 5}$$

Where P_g = 20 psf which is found from ground snow loads, Fig 7-1 ASCE 7-10.

The height of balanced snow load in feet, h_b, is found by using Equation 6. h_b = 1.06 ft.

$$h_b = \frac{P_f}{\gamma} \qquad \text{Eq. 6}$$

Where P_f = 17.6 psf which is uniform snow balance from previous snow load calculations

If $h_c / h_b > 0.2$, then snow drifts are required to be applied. Therefore, $h_c / h_b = 6.6$, so snow drift loads will be applied.

Since h_d is equal to or less than h_c, the drift width, w, is calculated using Equation 7. w = 6 ft.

$$w = 4h_d \qquad \text{Eq. 7}$$

The maximum intensity of the drift surcharge load, P_d, is solved for using Equation #. P_d =24.9 psf.

$$P_d = \gamma h_d \qquad \text{Eq. 8}$$

3.1.6. Ice Load

Because the project is located in Benton, Illinois; the region is not affected by ice in a way that would be a deciding factor in the load combinations. Therefore, for the rest of the project, it will be assumed that the ice load (I) is 0 psf.

3.1.7. Rain Load

Because the project is located in Benton, Illinois; the region is not affected by rain in a way that would be a deciding factor in the load combinations. Therefore, for the rest of the project, it will be assumed that the rain load (R) is zero 0 psf.

3.1.8. Inputting Loads into Load Combinations

Out of the nine ASCE load combinations, only a few need to be considered for this project. Only the ones that could be the highest value will be calculated. When modeling the structure, only the maximum of the load combos will be used. Since the wind load, the rain load, the ice load, and the seismic loads will not be considered, and live load is only on the foundation and not on any of the structural members, these variables will all be set to zero. Because some of the variables will be set to zero, many of the combos cannot control. Below are the equations for the load combos that could be controlled.

1. 1.4D
2. 1.2D + 1.6L_r
3. 1.2D + 1.6S

These edited load combinations were then implemented into the structural software.

3.2. Modeling and Design

3.2.1. Revit

Concurrently, the project was modeled in Revit, an Autodesk product. Revit is a building information modeling (BIM) software that a variety of engineering disciplines use to represent the project. All anticipated columns, beams, shear walls, cross-braces and stubs were sketched to represent the structure. The 3D view of the Revit drawing can be seen below in Figure 3.

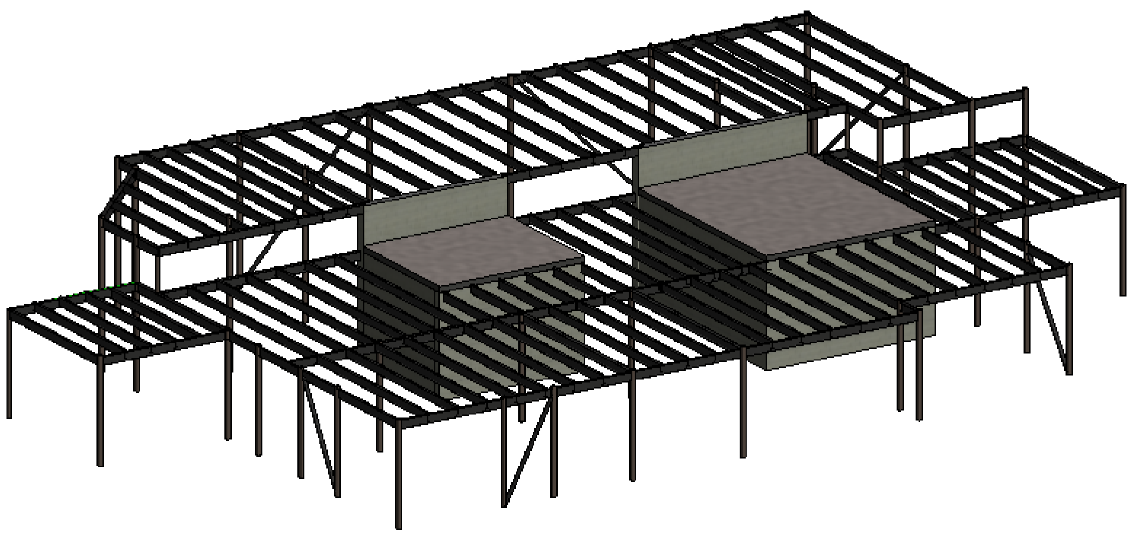


Figure 3: *Revit drawing of the structural skeleton.*

In the figure above, the steel skeleton of the bank being designed, along with the bank's vaults designed with a layer of CMU, or concrete masonry unit, blocks on either side with a thin layer of concrete in between. For aesthetic purposes, a mockup of what the building will look like post construction was also made in Revit software. Figure 4 below is a rendering of the Administration Building for the State Bank of Whittington including its architectural brick, windows, doors, roof, and landscape around it.

***Figure 4**: Revit rendering of the project post construction.*

These renderings of the building are not only extremely helpful to the client to envision what the final product will look like, but also helpful to the architectural designer and constructor of the project to find discrepancies or problems that need adjusted.

3.2.2. Risa

The Revit model was then downloaded as a Risa 3D model. Risa is a structural engineering software for analysis and design. Before loads were applied, the model had to be adjusted appropriately since there were multiple translation errors from Revit to Risa. Once the Risa file was sufficient constraints and loads were applied. Lateral and boundary condition assumptions then had to be made. First it was assumed that every joint in the structure was to be pinned and not fixed. Next, it was assumed that the diaphragm of the structure was the metal decking on the roof. The decking is used to hold the structure together, as well as transfer the loads to the lateral system.

Even though the lateral loads, such as the seismic and wind loads, will be ignored for this project, it is still important for the software to make sure the structure will not collapse on itself due to the gravity loads. This is done by first setting a diaphragm and second designing a lateral system. The metal roof decking acts as a diaphragm since it encloses the structure. For this project, there will be two different types of lateral systems. The first part of the lateral system is the concrete walls of the vaults in the center of the structure. These types of systems are called shear walls and they are sufficient at transferring lateral loads down into the foundation and into the earth. This was the main type of lateral system for this structure however, because they were centered in the building the outside steel members were weak in stability. Multiple cross bracing members were added to the exterior columns to add stability to the outside of the structure. Together the cross bracing and the shear walls help create a lateral system that is stable enough to support all the gravity loads.

Figure 5 is the Risa model of the Administration Building for the State Bank of Whittington, including the lateral system.

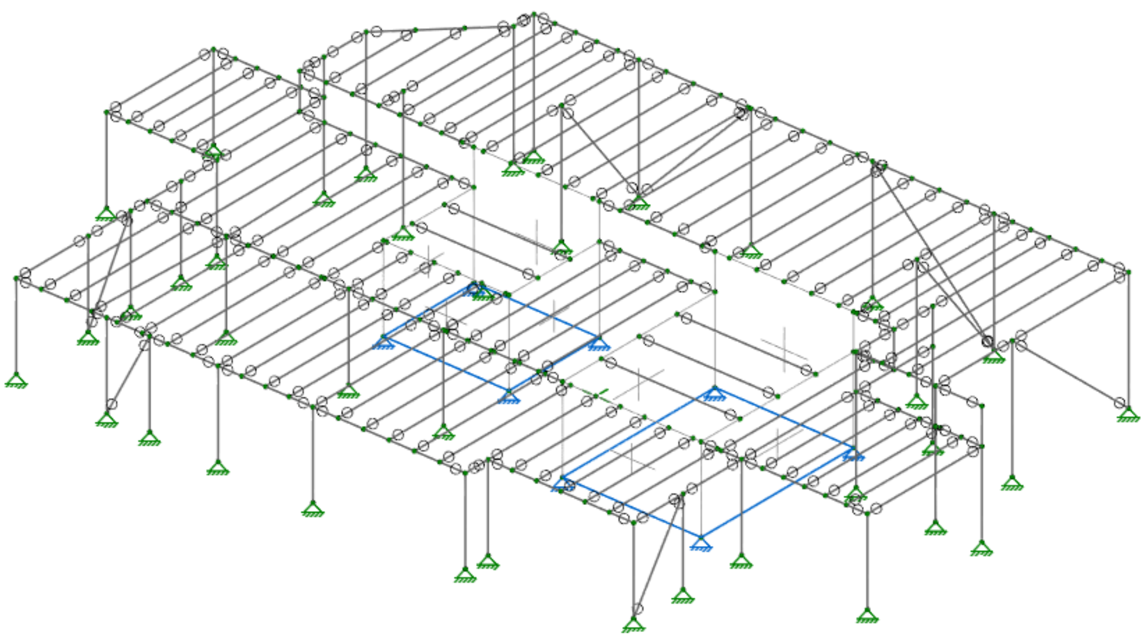


***Figure 5**: Risa model of the structural skeleton*

All the loads, previously established in this report, were inputted into the Risa model. In addition, Risa has an integrated tool to account for all the self-weight of all the structural members; therefore, the dead load will be increased.

Once every piece of the model has been entered, such as the structural members, lateral system, and loads, the software solves the loading scenario and gives feedback. Every failure mode is calculated simultaneously and can be displayed in each member of the structure. Risa also calculates which is the controlling failure mode for each member and decides if the member modeled is sufficient. To decide if a beam is sufficient, the model calculates the ultimate load on the member as well as the members capacity for each failure mode with a safety factor multiplied on. If the ultimate load divided by the factored capacity is less than 1.0, then the member passes, if the member has a ratio greater than 1.0 it fails. If a member is not sufficient or overdesigned Risa will show this by displaying members in certain colors. Red means that load to strength ratio is greater than 1.0, whereas blue indicates it is not close to any type of failure. Multiple colors in between such as green and yellow show that these members are closer to 1.0, for example 0.8 to 0.9 but not at failure. This is a good range for the members to be because it means it can handle the maximum loads, however it is not extremely over designed. Also, a value is associated with each member. This is the value that should be below 1.0 for a safe design. Below, in Figure 6, is the reference for which color means what range of values.

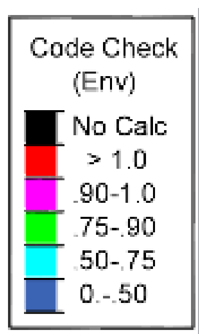


Figure 6*: Outlines members in colors based on if they passed the ultimate/capacity ratio check*

Below, Figure 7 of the first trial in Risa showing which members are overdesigned and which members need to be increased to handle the ultimate load safely.

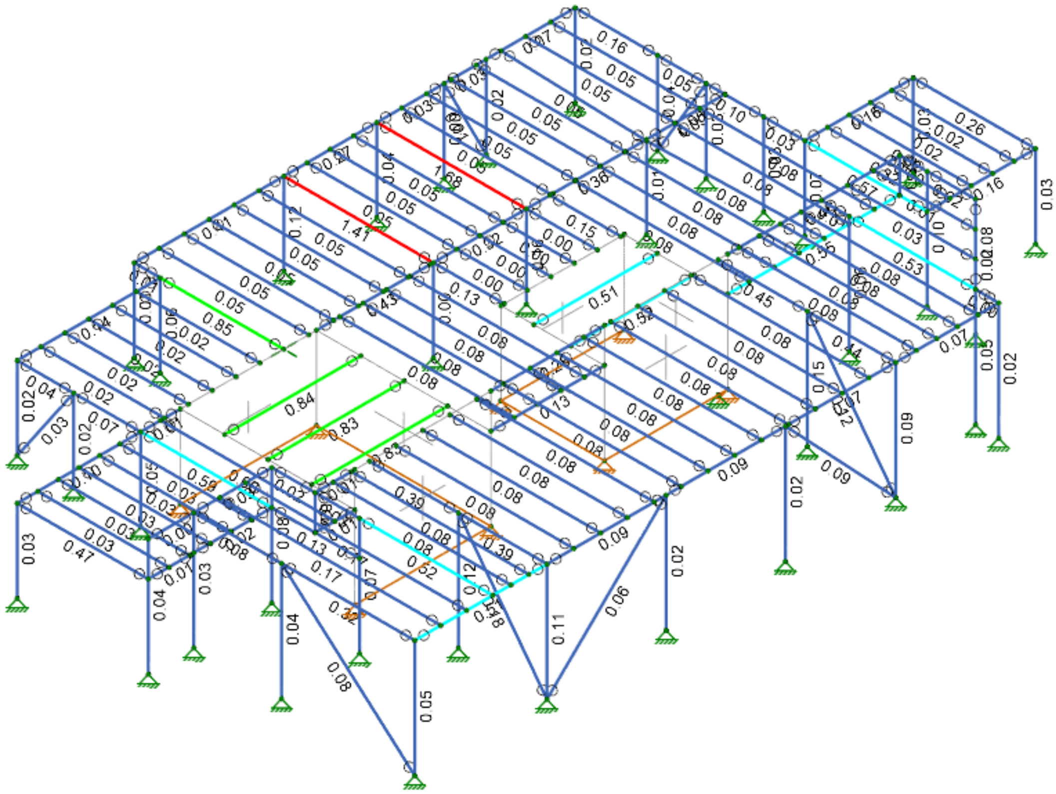


Figure 7: *RISA model of structure after analysis has been ran.*

It is the job of the designer to find appropriate sizes for each member. If it fails, a larger, heavier, or stronger member will have to be chosen. Many times, constraints will hold some of these attributes from changing. For example, larger than 12” nominal height beams will be too large for the architectural spacing, so increasing a member’s size will not be acceptable. It is also often not cost effective to go to a stronger grade steel, so this was an option that was cut as well. This leaves the weight to be the only variable that can change. As the weight of a structural member increases, the strength increases, however it gets more expensive. The goal is to shrink all overdesigned to the cheapest members that will still be safe and increase the failing members to a size and weight that will pass Risa’s check. Therefore, the design must be sufficient while also being efficient. Figure 8, below, is the finalized Risa check with the updated members.

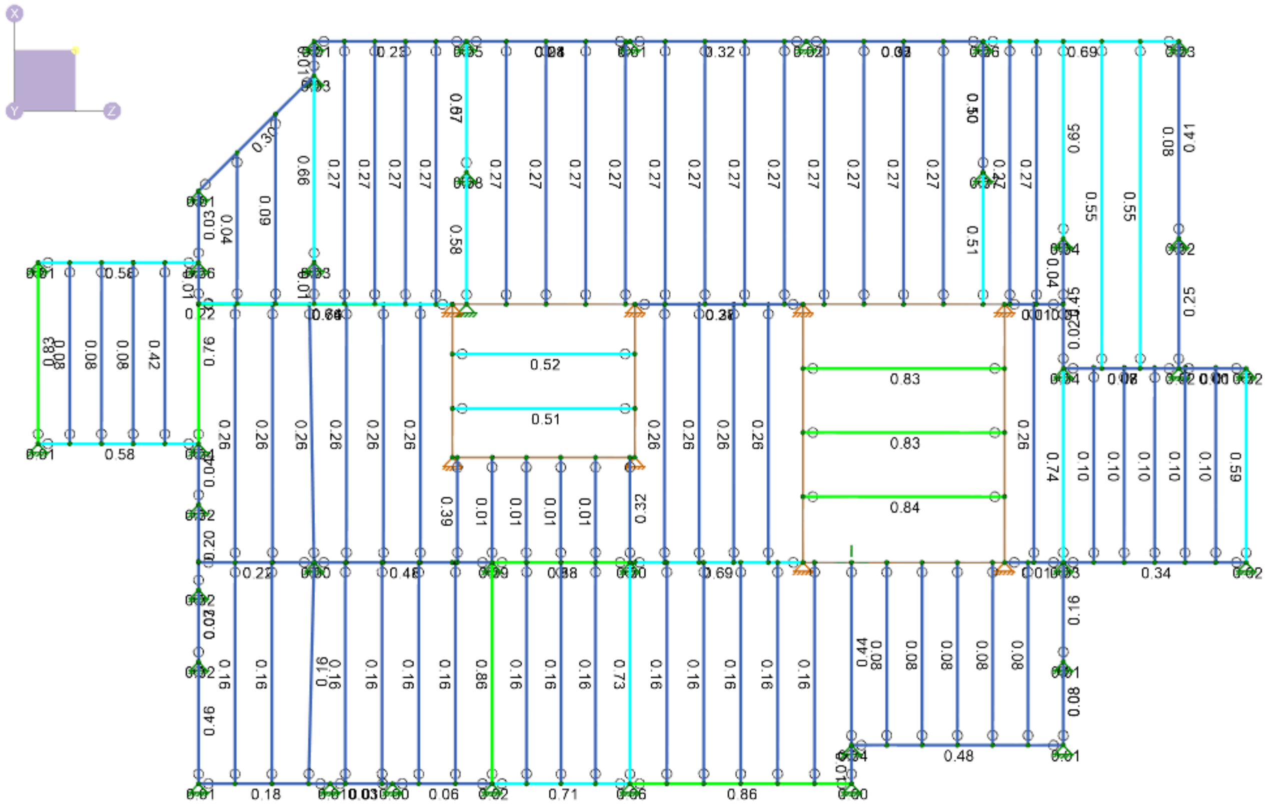


***Figure 8**: Final RISA design of the structure with accurately sized members.*

Another view, this one in 3D, of the final design in the Risa software can be seen below in Figure 9.

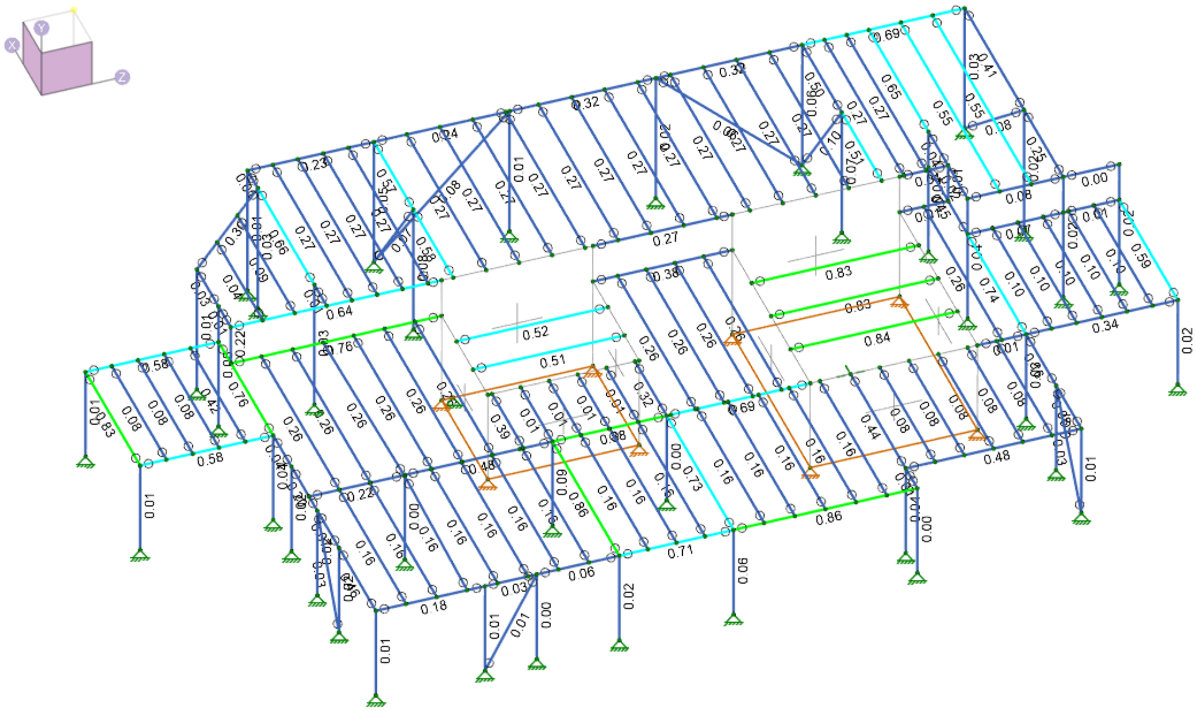


***Figure 9**: 3D view of the final RISA design with appropriately sized members.*

It is also important to check Risa's solutions to ensure everything was imputed into the model correctly. Often a model will run with no issues, even if there is something missing that can skew the results.

3.3. Final Structural Design

Now that the structural design has been completed, a list of all structural members has been created. Below is Table 5 describing how much material of each shape and size is needed to complete the structural component of this project.

***Table 5**: Material list for final structural design.*

Material	Size	Pieces	Length (ft)	Weight (k)
Hot Rolled Steel				
A500 Gr. B RECT	HSS 4 X 4 X 4	8	167.2	2.062
A500 Gr. B RECT	HSS 6 X 6 X 6	37	559.1	15.511
A500 Gr. B RND	HSS 6.000 X 0.375	1	5.3	0.121
A992	W10 X 33	5	100.7	3.328
A992	HSS 6.000 X 0.375	1	5.3	0.113
Steel ASTM A992	W12 X 14	109	2055.1	29.091
Steel ASTM A992	W12 X 22	15	284.7	6.277
Steel ASTM A992	W12 X 26	3	65	1.692
Total HR Steel		179	3242.5	58.194
Concrete Walls			Volume (yd^3)	
Concrete Masonry Units		8	102.9	417.657
Total Concrete		8	102.9	417.657

4. SITE DRAINAGE DESIGN

4.1. General Project Information

The project is a 2.2-acre site in the center of Benton, Illinois. The property currently has five single-family homes and a small administrative building. There is also gravel and concrete driveways for each house and two attached parking lots to the administrative building. All buildings and pavement will be demolished and removed from the project site before construction. The City of Benton's water tower is on the project site, but it will be left undisturbed for the entirety of the project. The project limits can be viewed in Figure 10.

Figure 10*:Aerial Photographic with defined project limits*

The city of Benton has no prepared stormwater drainage policy, so it was up to the discretion of the engineer to design the drainage design of the development. As with any project, the goal was to create a design that improved the existing conditions. Currently the project site has extraordinarily little stormwater runoff management plan; therefore, any development in this plan will be an improvement. It was decided to design a system that was in accordance with the City of Decatur Illinois' Stormwater runoff policy. This decision was made because Decatur, Illinois is close to the project that rainfall events will be similar, and the policy was well prepared and conservative with requirements.

The Natural Resources Conservation Service (NRCS) method will be used to estimate stormwater runoff from the existing project site. Precipitation data will be collected from NOAA's Precipitation Frequency Data Server for Benton, Illinois which can be referenced in the Appendix. According to the City of Decatur's Stormwater Policy, new developments must not exceed 1.0 cubic feet per second (cfs) of runoff per acre of development during a 100-year rainfall event. This project is a 2.2-acre site, so the maximum allowed runoff will be 2.2 cfs during a 100-year rainfall event.

4.2. Predevelopment Drainage Characteristics

For this project, the NRCS method was chosen to estimate the stormwater runoff from the project before and after development. This method was chosen over the other evaluation methods because it provides a more accurate estimate and will create a more conservative design.

4.2.1. Predevelopment Curve Number

Because the site had no current drainage design, it was decided to evaluate the site as it was before any development. Therefore, the site was defined as a 2.2-acre lot with 50%-70% grass cover. With these conditions, the below curve number was determined for the predevelopment site conditions.

$$\text{Curve Number}_e = 79$$

4.2.2. Predevelopment Time of Concentration

The time of concentration for the project site was determined using the TR-55 method. Time of concentration is defined by TR-55 as the "…the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest in the watershed." The point of interest for this project will be defined as the point where runoff reaches the storm sewer of the City of Benton on the northeast corner of the project site.

The time of concentration for the existing site was based on the provided topographic survey of the existing site. The contour map, Figure 11, was used to find the flow paths and slope of the project site. Using the built in Flow Path tool in Civil 3D, the longest flow path was able to be determined and calculated. This path that controlled the time of concentration is shown below. The first 100 feet was defined as sheet flow which then changed to shallow concentrated flow for the remainder of the path. The time of concentration that was calculated for the predevelopment site conditions was found to be as follows:

$$Tc_e = 15 \text{ minutes}$$

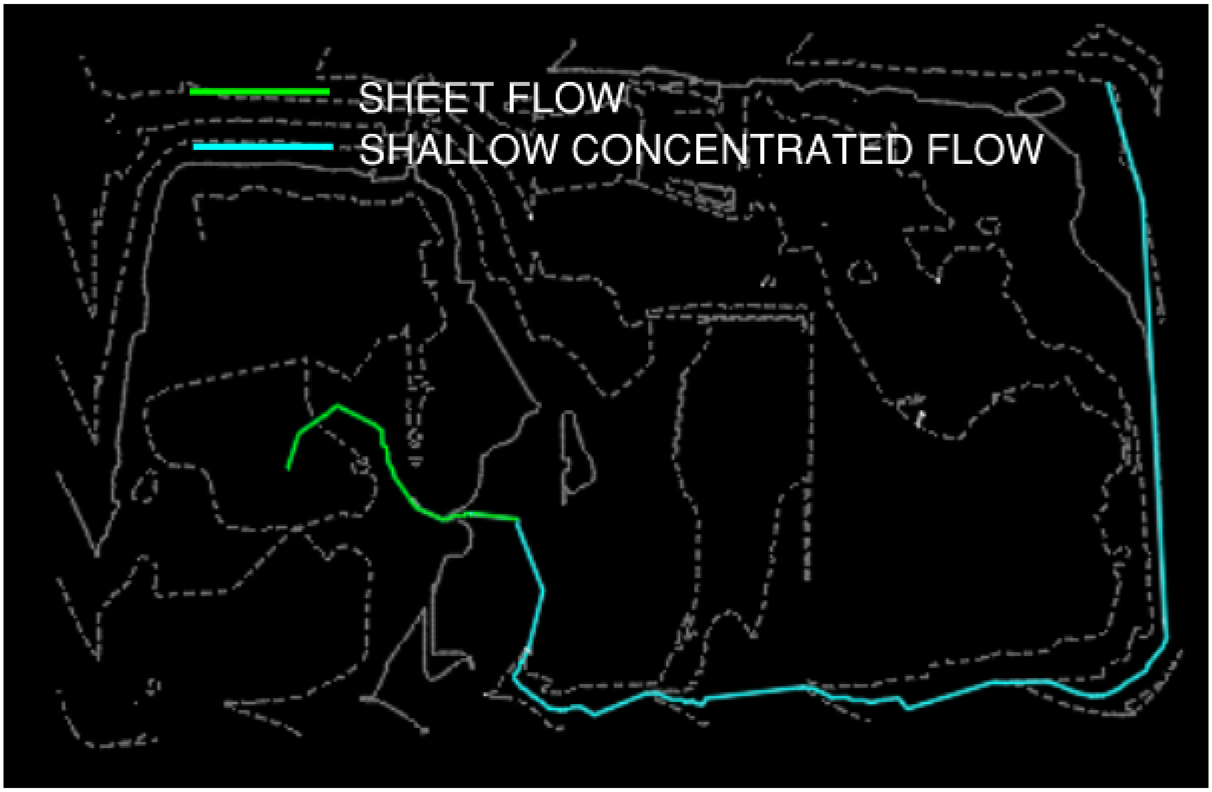


Figure 11: *Time of Concentration Flow Path*

4.2.3. Predevelopment Runoff Analysis

The approximate runoff from the undeveloped project area will be estimated by the NRCS TR-20 method. The runoff was calculated for six rainfall events. This was done to create a range of runoff approximations to see the amount of water that this site handles. Table 6 shows the peak discharge and runoff amount of each rainfall that was simulated in Autodesk Storm and Sanitary Analysis 2022.

__Table 6:__ Predevelopment Precipitation and Runoff Values

Predevelopment		
Rainfall Event	Preciptation (in)	Peak Runoff (cfs)
2-year	3.16	1.14
5-year	4.00	1.77
10-year	4.62	2.27
25-year	5.79	3.24
50-year	6.71	4.02
100-year	7.73	4.91

4.3. Proposed Site Plan

The Bank of Whittington had provided a site layout of where the building and parking lots were to be located. A grading plan was also provided with the layout. This plan defined the elevations of the building pad and parking lots. It was asked to design a stormwater drainage system that utilized detention storage because the grading plan need to fill more dirt than what was being cut. Digging a detention basin would provide fill for other parts of the site and dirt would not have to be imported from another location and increase cost. It was also decided that the site would be better if two detention basins were designed on the north and south side of the building to make sure that all runoff was caught on site and directed to the desired drainage outlet.

Below in Figure 12, is the proposed site layout that the Bank of Whittington envisioned for the new building development.

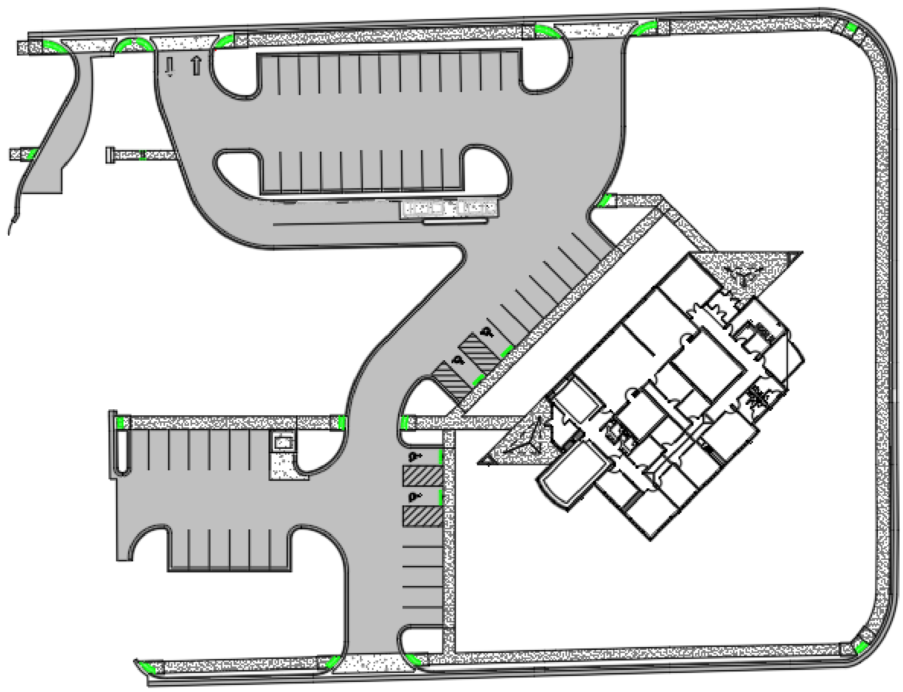


Figure 12: *Proposed Site Layout from the Bank of Whittington*

4.4. Post Development Drainage Characteristics

The plans that were provided by the Bank of Whittington for the new development had no drainage plan for the site. Because the site was being redeveloped and the grading and surfaces were being changed, an evaluation of the change in the drainage of the must be completed. A site after being developed cannot release stormwater runoff any faster than it did before redevelopment. For this site, the pre-existing conditions had little to no accommodation for stormwater management so any systems implemented would be an improvement.

The City of Decatur's Stormwater Policy requires that for a site of this size, post development runoff cannot exceed 1.0 cfs of runoff per acre of development during a 100-year rainfall event. This site is 2.2-acres and therefore is allowed to only release a maximum of 2.2 cfs. To meet this criteria, two dry detention basins are being proposed to be designed that will be capable of storing the stormwater and release at the acceptable rate into the city's storm sewer.

4.4.1. Post Development Curve Number

The proposed site is composed of 0.88 acres of impervious surfaces such as roofs, parking lots, and sidewalks. The remaining 1.32 acres will be grass surface. Using these areas, a weighted curve number that defined the proposed land use was created and is shown below:

$$\text{Curve Number}_P = 84$$

4.4.2. Post Development Time of Concentration

The time of concentration for the proposed site was based on the proposed grading plan and pipe network. The proposed contours were evaluated with the assistance of the Flow Path tool in Civil 3D. Flow across the parking lot was sheet flow that then became channel flow when it entered the proposed storm pipes. The longest flow path of the site was only 3.27 minutes (min). This is lower than the minimum allowable time of concentration defined by the TR-55 method. Therefore, the time of concentration was decided to be changed to meet the minimum requirements and is shown below:

$$Tc_P = 6 \text{ min}$$

4.4.3. Detention System Design

As previously mentioned, two detention basins were proposed to contain the stormwater before it was released into the city storm sewer. These detention basins are on opposite sides of the site and will be connected by a pipe. This pipe will drain the secondary basin into the main basin. It will also allow the main basin to backflow into the secondary if the water level in the main is increasing too rapidly. Because of this design, the two basins were analyzed as being combined and the project site was considered as one drainage area. Curb inlets will be installed in the parking lots to capture the runoff and transport it to the basins. Roof drains will also collect runoff and direct it to the basins. Figure 13 shows the layout of the basins with respect to the building.

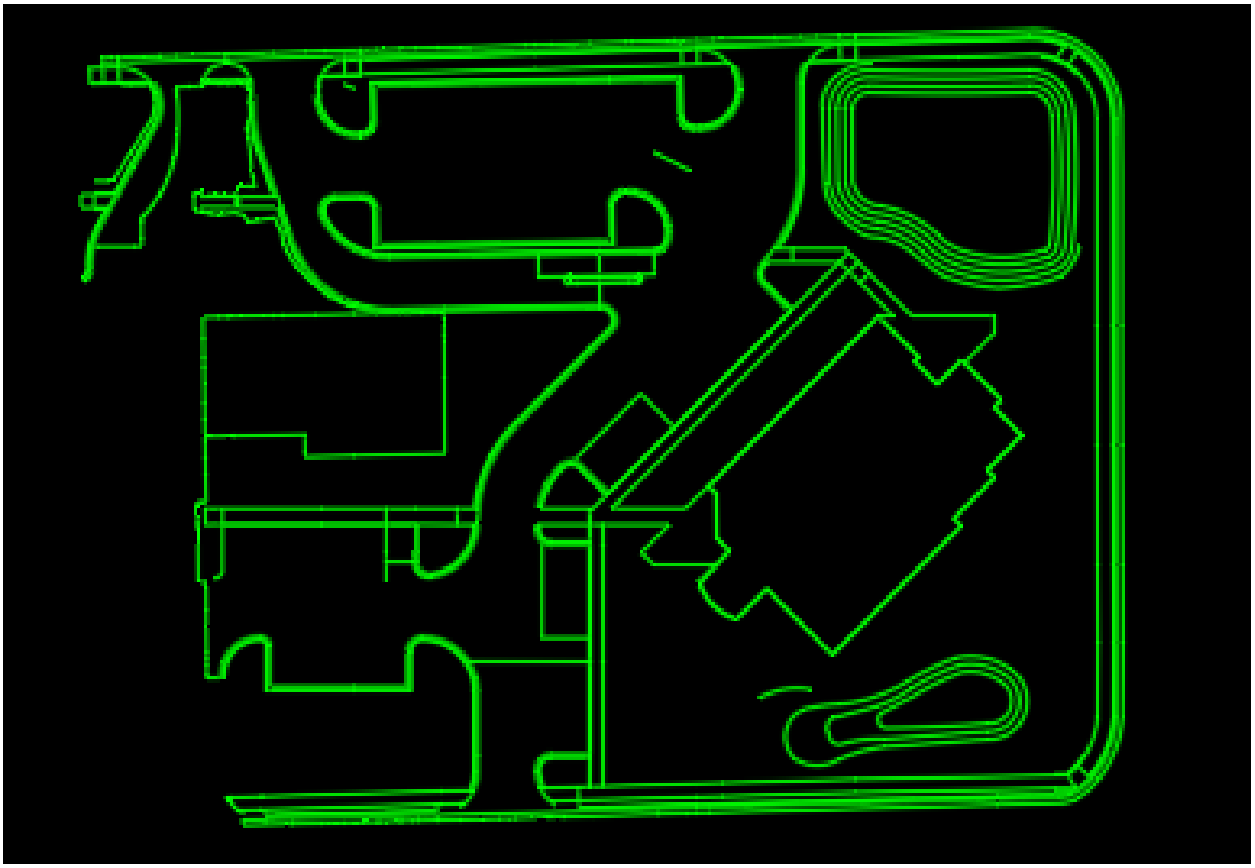


***Figure** 13: Proposed Site Layout with Detention Basins*

The primary detention basin has a maximum capacity of 16,585 cubic feet (ft^3). The surrounding area of the basin is graded so that if the basin exceeds the maximum elevation of 468, the water will be directed away from the building and into the adjacent street. This acts as the emergency spillway for the basin.

The secondary detention basin has a maximum capacity of 2,879 ft^3. As with the primary basin, the secondary basin is graded so that if the maximum elevation of 472 is exceeded, the water will run away from the building and into the adjacent street.

The outlet structure of the secondary basin is a flared concrete end piece attached to a 12 inch (") concrete culvert that slopes down to the primary detention basin. The primary detention basin's outlet structure will be a flared concrete end piece that will have a 7" orifice plate installed between it and the 12" concrete pipe that connects to the city's storm sewer. This orifice plate will act as a flow restrictor and will make the basins meet the release criteria laid out by drainage policy being used.

These conditions were modeled in Storm and Sanitary Analysis and ran with several different rainfall events to see who the detention system handled each event. Table 7 shows the runoff from the site for each storm event. The detention system successfully met the criteria of the stormwater policy guidelines by only allowing 2.14 cfs of runoff to be released during the 100-year rainfall.

***Table** 7: Post Development Precipitation and Runoff Values*

Postdevelopment		
Rainfall Event	Preciptation (in)	Peak Runoff (cfs)
2-year	3.16	1.49
5-year	4.00	1.64
10-year	4.62	1.73
25-year	5.79	1.89
50-year	6.71	2.02
100-year	7.73	2.14

4.4.4. Storm Pipe Network Design

The next step of the drainage design was designing a storm pipe network system that will successfully channel all runoff from the site to the detention basins and finally into the city of Benton’s existing storm sewer. The tie-in to the storm sewer will be done at the northeast corner of the site.

The drainage policy used in this design dictates that a system must be designed to successfully handle a 100-year rainfall event. The storm pipes designed for this site are called out to be SDR-35 PVC that range from 8” in size to 15”. The inlets are called out to be standard 24x24 curb inlets with associated 24x24 structures whose depth will depend on the location of the site. All pipes were designed to have a minimum slope of .5% as defined by the drainage policy used in this design. The roof drains were defined to be 4” downspouts that will be tied into the main storm network. Below in Figure 14 shows the proposed storm network for the site.

The parking lots and surrounding area is proposed to be graded so that if an inlet is clogged or overflows, the water will not pond and flood the parking lot but instead flow away from the site and into the adjacent streets. Once in the streets, it will be able to make its way to the city storm sewer.

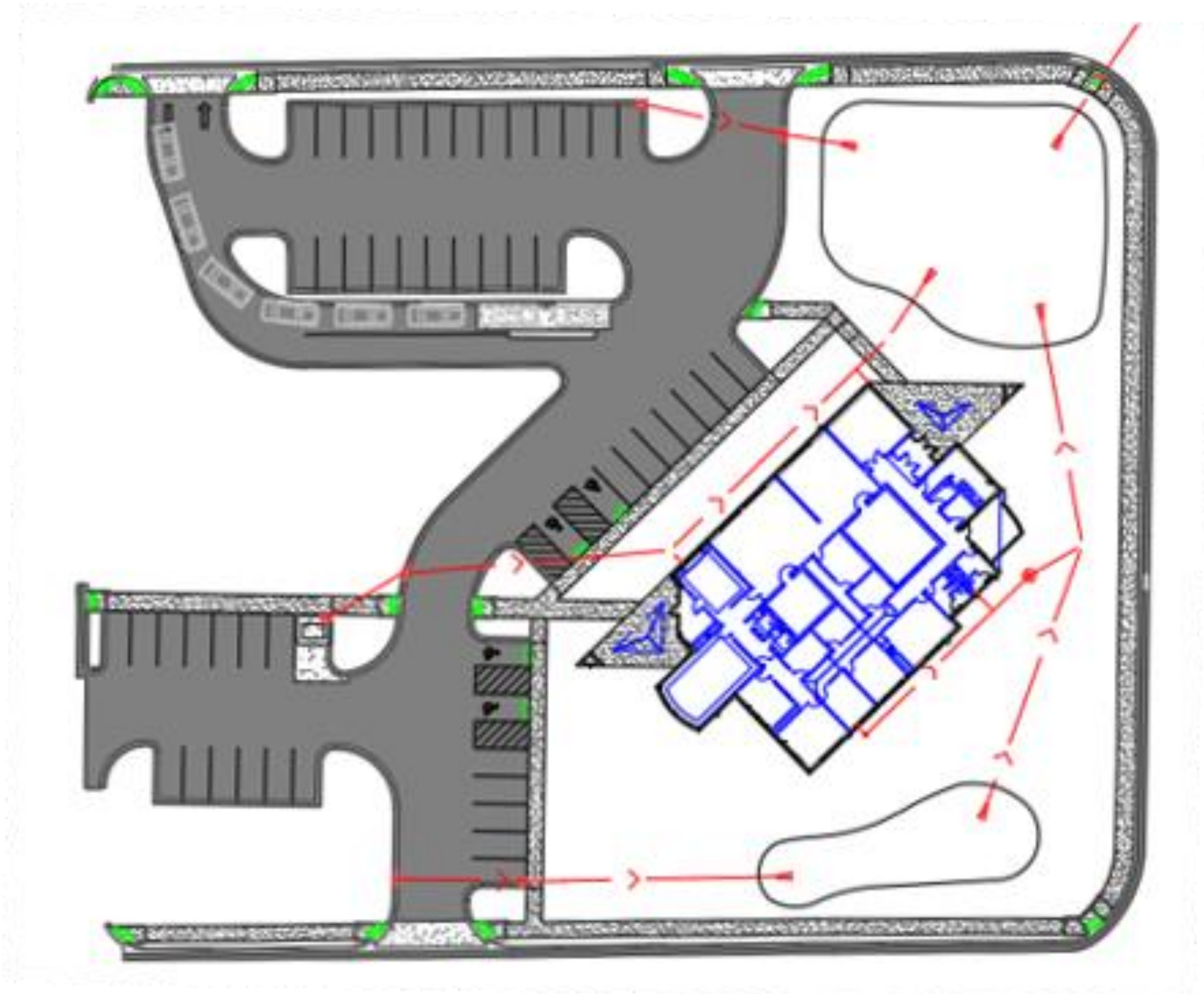


***Figure 14**: Site Layout with Proposed Storm Pipe Network*

5. Cost Estimate

An estimate of the scope of work was created to assist in the financial planning and the bidding process of the project. The estimate covered the scope of work that was outlined in this report which includes the structural skeleton and vaults of the proposed building, site excavation and grading, and pipe network. Only material will be included in the estimate and no profit or overhead will be added.

For the structural design, RISA 3D provided a take-off summary of the amount of steel and CMUs were needed to be used in the building. This summary is shown below in Table 8 with the associated cost of this material.

***Table 8**: Quantity Takeoff of Structural Design*

Steel			
Material	Size	Pieces	Length (ft)
A500 Gr.B RECT	HSS4X4X4	8	167.2
A500 Gr.B RECT	HSS6X6X6	37	559.1
A500 Gr.B RND	HSS6.000X0.375	1	5.3
A992	W10X33	5	100.7
A992	HSS6.000X0.375	1	5.3
Steel ASTM A992	W12X14	109	2055.1
Steel ASTM A992	W12X22	15	284.7
Steel ASTM A992	W12X26	3	65
Concrete			
			Units (ea)
CMU	16x8x8		4688
		Total	$119,981.80

For the drainage design, the cut and fill report for the proposed grading plan was created in Civil 3D and is provided in the appendix. The estimate was determined “at cost” of labor to grade out the proposed site and haul off cut material. The cost for this work was found to be $40,000 for all labor and equipment. The pipe quantities and sizes, along with inlets, are shown below in Table 9.

Table 9: *Quantity Takeoff of Pipes and Inlets*

Pipes and Inlets		
Description	Quantity	Unit
4" Roof Drains	4	EA
8" SDR 35	340	FT
12" SDR 35	372	FT
12" RCP	65	FT
24"x24" Curb Inlets	6	EA
Total	$19,250.00	

In total, the estimated cost for this scope of work for the project is $179,230.00. This estimate is subject to change due to volatility in the steel and pipe industry.

6. REFERENCES

American Institute of Steel Construction, Manual of Steel Construction, 14th Edition

"ASCE Standard ASCE/SEI 7-10." ISBN 978-0-7844-1115-5. 2011.

City of Decatur Drainage Plan

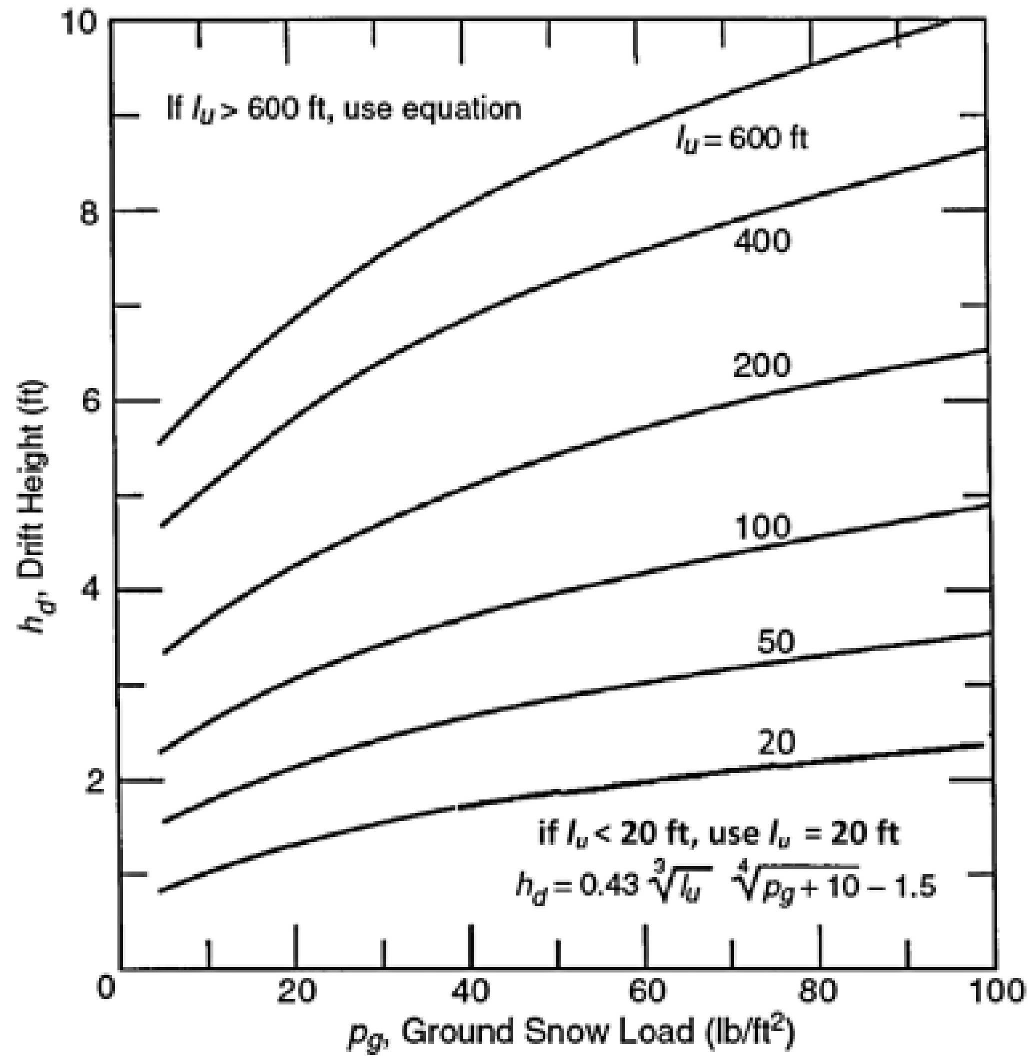
International Building Code, 2015

TR-55 manual

7. APPENDIX

A. Determining Drift Height

The figure below is Figure 7-9 from ASCE 7-10 to determine the Drift Height to develop the Snow Drift Loads.



To convert lb/ft² to kN/m², multiply by 0.0479.
To convert ft to m, multiply by 0.3048.

FIGURE 7-9 Graph and Equation for Determining Drift Height, h_d.

B. Cut/Fill Report

Below is the Cut/Fill Report for the project site which was considered for the cost estimate.

Cut/Fill Report

Generated:	2022-03-22 21:38:04
By user:	User
Drawing:	C:\Users\User\OneDrive - University of Southern Indiana\Documents\4 - Senior Year\CE 491 - Senior Design\Drawings\7839_SBW GRD BASE - Standard\C:\Users\User\OneDrive - University of Southern Indiana\Documents\4 - Senior Year\CE 491 - Senior Design\Drawings\7839_SBW GRD BASE - Standard\Senior Design - Paul Bohlen - Real Thing.dwg

Volume Summary

Name	**Type**	**Cut Factor**	**Fill Factor**	**2d Area** (Sq. Ft.)	**Cut** (Cu. Yd.)	**Fill** (Cu. Yd.)	**Net** (Cu. Yd.)
Surface3	full	1.0000	1.0000	96116.86	2126.34	1850.08	276.26<Cut>

Totals

	2d Area (Sq. Ft.)	**Cut** (Cu. Yd.)	**Fill** (Cu. Yd.)	**Net** (Cu. Yd.)
Total	96116.86	2126.34	1850.08	276.26<Cut>

* Value adjusted by cut or fill factor other than 1.0

C. Precipitation Data for Project Site

Precipitation data for the project site which was found using the NOAA (National Oceanic and Atmospheric Administration) data.

NOAA Atlas 14, Volume 2, Version 3
Location name: Benton, Illinois, USA*
Latitude: 38.0069°, Longitude: -88.9104°
Elevation: 455.71 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF 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	0.413 (0.373-0.457)	0.491 (0.443-0.542)	0.582 (0.524-0.643)	0.655 (0.588-0.722)	0.748 (0.669-0.822)	0.821 (0.732-0.903)	0.892 (0.792-0.981)	0.967 (0.854-1.06)	1.07 (0.936-1.17)	1.14 (0.996-1.26)
10-min	0.642 (0.579-0.710)	0.767 (0.692-0.847)	0.905 (0.814-0.999)	1.01 (0.908-1.11)	1.14 (1.02-1.26)	1.25 (1.11-1.37)	1.34 (1.19-1.48)	1.44 (1.27-1.59)	1.57 (1.38-1.73)	1.67 (1.45-1.83)
15-min	0.787 (0.710-0.870)	0.938 (0.846-1.03)	1.11 (1.00-1.23)	1.24 (1.12-1.37)	1.41 (1.26-1.55)	1.54 (1.37-1.69)	1.67 (1.48-1.83)	1.80 (1.59-1.97)	1.96 (1.72-2.15)	2.08 (1.81-2.29)
30-min	1.04 (0.939-1.15)	1.25 (1.13-1.39)	1.52 (1.37-1.68)	1.73 (1.55-1.90)	2.00 (1.79-2.19)	2.20 (1.96-2.42)	2.41 (2.14-2.65)	2.62 (2.31-2.88)	2.91 (2.55-3.19)	3.12 (2.72-3.43)
60-min	1.27 (1.15-1.41)	1.54 (1.39-1.70)	1.91 (1.72-2.11)	2.20 (1.97-2.42)	2.59 (2.32-2.85)	2.90 (2.59-3.19)	3.22 (2.86-3.54)	3.56 (3.14-3.90)	4.02 (3.52-4.41)	4.38 (3.82-4.81)
2-hr	1.55 (1.38-1.74)	1.87 (1.68-2.10)	2.33 (2.08-2.61)	2.69 (2.40-3.02)	3.19 (2.84-3.57)	3.60 (3.19-4.01)	4.02 (3.55-4.47)	4.45 (3.92-4.95)	5.06 (4.42-5.62)	5.55 (4.82-6.18)
3-hr	1.67 (1.49-1.88)	2.01 (1.80-2.27)	2.51 (2.24-2.83)	2.91 (2.59-3.28)	3.47 (3.08-3.89)	3.93 (3.48-4.40)	4.42 (3.89-4.94)	4.93 (4.31-5.50)	5.65 (4.91-6.30)	6.24 (5.38-6.96)
6-hr	2.01 (1.80-2.28)	2.42 (2.17-2.75)	3.00 (2.69-3.41)	3.48 (3.11-3.95)	4.15 (3.69-4.68)	4.70 (4.17-5.30)	5.28 (4.66-5.94)	5.89 (5.18-6.62)	6.76 (5.90-7.60)	7.47 (6.47-8.39)
12-hr	2.38 (2.13-2.72)	2.87 (2.57-3.26)	3.55 (3.17-4.02)	4.09 (3.66-4.64)	4.85 (4.32-5.49)	5.48 (4.85-6.18)	6.12 (5.40-6.90)	6.81 (5.97-7.67)	7.77 (6.76-8.75)	8.55 (7.39-9.63)
24-hr	2.81 (2.57-3.11)	3.38 (3.10-3.74)	4.20 (3.84-4.64)	4.86 (4.43-5.37)	5.78 (5.25-6.39)	6.53 (5.92-7.21)	7.32 (6.61-8.08)	8.15 (7.31-9.00)	9.30 (8.29-10.3)	10.2 (9.05-11.3)
2-day	3.27 (3.00-3.58)	3.93 (3.61-4.31)	4.89 (4.48-5.35)	5.68 (5.20-6.21)	6.79 (6.19-7.42)	7.71 (7.00-8.43)	8.69 (7.85-9.51)	9.74 (8.74-10.7)	11.2 (9.99-12.3)	12.5 (11.0-13.7)
3-day	3.49 (3.23-3.79)	4.18 (3.87-4.54)	5.18 (4.80-5.63)	6.02 (5.56-6.53)	7.22 (6.64-7.83)	8.22 (7.52-8.91)	9.29 (8.45-10.1)	10.4 (9.44-11.3)	12.1 (10.8-13.2)	13.5 (11.9-14.7)
4-day	3.70 (3.45-3.99)	4.43 (4.13-4.77)	5.48 (5.11-5.91)	6.36 (5.92-6.86)	7.65 (7.08-8.23)	8.73 (8.04-9.38)	9.89 (9.05-10.6)	11.1 (10.1-12.0)	13.0 (11.7-14.0)	14.5 (12.9-15.7)
7-day	4.25 (3.98-4.55)	5.09 (4.77-5.45)	6.28 (5.88-6.73)	7.27 (6.79-7.79)	8.70 (8.11-9.32)	9.90 (9.18-10.6)	11.2 (10.3-12.0)	12.5 (11.5-13.5)	14.5 (13.2-15.6)	16.1 (14.5-17.4)
10-day	4.80 (4.51-5.12)	5.74 (5.39-6.13)	7.09 (6.65-7.56)	8.20 (7.68-8.74)	9.80 (9.13-10.4)	11.1 (10.3-11.9)	12.5 (11.6-13.4)	14.0 (12.9-15.0)	16.2 (14.7-17.4)	17.9 (16.1-19.3)
20-day	6.61 (6.26-7.00)	7.85 (7.43-8.31)	9.41 (8.90-9.95)	10.7 (10.1-11.3)	12.4 (11.7-13.1)	13.7 (12.9-14.5)	15.1 (14.2-16.0)	16.6 (15.4-17.6)	18.5 (17.1-19.7)	20.1 (18.4-21.4)
30-day	8.13 (7.73-8.57)	9.61 (9.14-10.1)	11.3 (10.8-11.9)	12.7 (12.1-13.4)	14.6 (13.8-15.4)	16.1 (15.2-17.0)	17.6 (16.6-18.5)	19.1 (17.9-20.2)	21.2 (19.7-22.4)	22.7 (21.0-24.2)
45-day	10.2 (9.72-10.7)	12.0 (11.4-12.6)	14.0 (13.3-14.6)	15.5 (14.8-16.3)	17.7 (16.8-18.6)	19.4 (18.4-20.3)	21.0 (19.9-22.1)	22.7 (21.4-23.9)	24.9 (23.4-26.3)	26.6 (24.8-28.2)
60-day	12.0 (11.5-12.6)	14.2 (13.5-14.9)	16.4 (15.7-17.3)	18.2 (17.4-19.1)	20.6 (19.6-21.6)	22.4 (21.2-23.5)	24.1 (22.8-25.3)	25.8 (24.4-27.2)	28.1 (26.3-29.6)	29.7 (27.7-31.4)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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