SCANNELL DEVELOPMENT
LEE'S SUMMIT LOGISTICS
NORTHWEST CORNER OF TUDOR ROAD & MAIN STREET

FINAL STORMWATER DRAINAGE STUDY FOR PHASE I

Prepared for:

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PE-2021026387
10/15/2021
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Olsson Project No. A21-04157
October 15, 2021

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1. GENERAL INFORMATION

This drainage study is an update to the previously approved preliminary stormwater drainage study (dated July 8, 2021) and presents the hydrologic impact generated by construction of Phase I of the project. This document builds on the preliminary study and new text that has been added or previous text that has been modified to describe the Phase I improvements is included in green text. Phase I includes the southernmost of the three proposed buildings from the preliminary stormwater drainage study, and its appurtenances. The attached Exhibits 1-6 in Appendix A show the limits of the Phase 1 improvements within the overall project boundary.

The Scannell Development project (the project) is approximately 83 acres of proposed industrial development including warehouses, loading docks, parking lots, stormwater detention basins, and open space. This project is located northwest of the intersection of NW Tudor Road and NW Sloan Street in Lee's Summit, Missouri. Stormwater from the project is conveyed into the Cedar Creek Watershed, primarily via Little Cedar Creek (which generally flows from east to west through the project boundary) and an unnamed tributary to Little Cedar Creek (which generally flows from south to north along the west side of the project boundary). Figure 1 shows the location and boundary of the project. It should be noted that the project boundary has been slightly modified from the property boundary for analysis purposes. The project boundary has been expanded to include NW Sloan Road, near the southeast corner of the project boundary, as portions of the road may need to be reconfigured or reconstructed as part of this project.

Figure 1. Location Map.

1.1 FEMA Floodplain Classifications

No changes have occurred to the FEMA floodplain since the approval of the preliminary stormwater drainage study.

The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) Panel Number 29095C0417G classifies portions of the project to be within the special flood hazard area (SFHA). SFHA's located within the project boundary include:

- Zone AE – Areas that are determined through detailed analyses to be subject to inundation from the 100-year (1-percent-annual-chance) flood and for which base flood elevations have been determined.
- Zone A – Areas that are determined through approximate analyses to be subject to inundation from the 100-year (1-percent-annual-chance) flood and for which base flood elevations have not been determined.
- Zone X – Areas that are determined to be moderate flood hazards areas and can be any of the following: areas of the 500-year (0.2-percent-annual-chance) flood; areas of average depths of less than one foot or with drainage areas less than one square mile; areas protected by levees from the 1% annual chance flood.

These SFHA's pertain to Little Cedar Creek and an unnamed tributary to Little Cedar Creek, which flow through the project site as described in Section 1. See Exhibit 1 in Appendix A for the location SFHA boundaries in relation to the project boundary.

1.2 Soil Classifications

No changes have occurred to the soil classifications since the approval of the preliminary stormwater drainage study.

Soil maps published on the Natural Resources Conservation Service's (NRCS) Web Soil Survey categorize soils within the project boundary as shown in Table 1. See Exhibit 2 in Appendix A for a map of soils on the property.

Table 1. Soil Classifications.

Symbol	Name	Slopes	Hydrologic Soil Group
10024	Greenton-Urban land complex	5-9 %	D
10082	Arisburg-Urban land complex	1-5 %	C
10120	Sharpsburg silt loam	2-5 %	C
10128	Sharpsburg-Urban land complex	2-5 %	D
10129	Sharpsburg-Urban land complex	5-9 %	D
10142	Snead-Rock outcrop complex	5-14 %	D
30080	Greenton silt clay loam	5-9 %	C/D

2. METHODOLOGY

No changes have occurred to the overall methodology from the preliminary stormwater drainage study.

This drainage study has been prepared to evaluate the hydrologic impact generated by the project. The base data for the models prepared for this report has been obtained from available online maps and aerial imagery. Stormwater management is based upon methods and objectives defined in the Kansas City Metropolitan Chapter of the American Public Works Association's (KC-APWA) 2011 design guidance document called "Section 5600 Storm Drainage Systems & Facilities".

The following software and methods were used in this study to model existing and proposed conditions for stormwater runoff:

United States Army Corps of Engineers Hydrologic Engineering Center Hydrology Modeling System (HEC-HMS) Version 4.7.1

- Soil Conservation Survey (SCS) Unit Hydrograph Method
  - 2-year, 10-year, and 100-year Return Frequency Storms
  - Antecedent Moisture Conditions II Soil Moisture Conditions
  - 24-Hour SCS Type II Rainfall Distribution
  - SCS Runoff Curve Numbers per SCS TR-55 (Tables 2-2a – 2-2c)

United States Department of Agriculture WinTR-55 Small Watershed Hydrology

  - SCS TR-55 methods for determination of time of concentration and travel time. Where specific data pertaining to channel geometry is not available, length and velocity estimates for channel flow travel time is used per Section 5600, KC-APWA Standard Specifications and Design Criteria.

Stormwater runoff models were created for the 2-, 10-, and 100-year design storm events. The precipitation depths used in the analysis have been interpolated from the "Technical Paper No. 40 Rainfall Frequency Atlas of the United States" (TP-40) isopluvial maps (May 1961). Table 2 below summarizes the rainfall depths used in this analysis:

Table 2. Precipitation Depths.

Return Period	24-Hour Precipitation Depth (inches)
2-Year (50% Storm)	3.60
10-year (10% Storm)	5.34
50-year (2% Storm)	6.96
100-Year (1% Storm)	7.90

Although not specifically analyzed in this study, the 50-year storm depth is also listed in Table 2 as the peak flow rate was needed for analysis with the corresponding flood study for this project.

3. EXISTING CONDITIONS

Points of interest and drainage boundaries in existing conditions remain the same as in the preliminary stormwater drainage study.

The following areas and points of interest have been used for existing and proposed conditions analysis to quantify the effects of developing this project. See Exhibit 3 in Appendix A.

Point 1 is located just downstream of the crossing of Little Cedar Creek at the Union Pacific Railroad and is the primary point-of-interest for this study. Little Cedar Creek and the unnamed tributary to Little Cedar Creek both drain to this common point-of-interest. All of the stormwater runoff from the developed portion of the site eventually drains to Point 1. Therefore, Point 1 was used as the comparison point for calculating allowable peak discharges and comparison to proposed peak discharges. The location of Point 1 was chosen strategically and placed at the upstream limit of FEMA's mapped floodway for Little Cedar Creek. The downstream limit of the hydraulic model that was created for this project is located at Point 1, which is discussed in further detail in the flood study.

Point 2 is located at the confluence of Little Cedar Creek and the unnamed tributary to Little Cedar Creek. All stormwater runoff from Drainage Area B and Drainage Area C eventually drains to this point in existing conditions. This point is used as an intermediate point for calculation purposes.

Drainage Area A discharges to Little Cedar Creek and is located upstream of Point 1. The total area modeled within this drainage area is approximately 20.3 acres in existing conditions, which includes portions of on-site and off-site drainage area. A small amount of on-site area is located at the northwest corner of the project boundary, just outside of Drainage Area A in existing conditions. This area will be further discussed in the proposed conditions analysis (Section 4.1).

Drainage Area B discharges to Little Cedar Creek and encompasses most of the northern portion of the site and off-site area upstream of the site. The total area modeled within this drainage area is approximately 150.9 acres in existing conditions.

Drainage Area C discharges to the unnamed tributary to Little Cedar Creek and encompasses most of the southern portion of the site and off-site area upstream of the site. The total area modeled within this drainage area is approximately 269.3 acres in existing conditions.

3.1 Hydrologic Analysis (Existing Conditions)

No changes have been made to the existing conditions hydrologic analysis from the preliminary stormwater drainage study.

To provide a direct comparison between the existing and proposed conditions models, the points of interest have been kept consistent throughout the analysis. Tables 3, 4, and 5 summarize the results of the existing conditions analysis. The proposed conditions data will be compared to these results in Section 4 of this study. Refer to Appendix B for existing conditions curve number and time of concentration calculations. Refer to Appendix C for the existing conditions HEC-HMS inputs and outputs.

Curve numbers were determined based on the soil classifications outlined in Section 1.2 and existing land use. Land use was determined from recent aerial imagery. Curve numbers were assumed as shown in Table 3.

Table 3. Curve Numbers.

Land Use	HSG	CN
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	C	74
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	D	80
Impervious Areas Paved parking lots, roofs, driveways	C/D	98
Residential Districts (1/4 acre)	C	83
Residential Districts (1/4 acre)	D	87
Urban Districts Commercial & Business	C	94
Urban Districts Commercial & Business	D	95

*HSG = hydrologic soil group, *CN = curve number

Table 4. Existing Conditions Drainage Area Data.

Drainage Area	On-site Area (acres)	Off-site Area (acres)	Total Area (acres)	T_C (hour)	Weighted CN
A	8.6	11.7	20.3	0.197	86
B	43.3	107.6	150.9	0.355	86
C	27.9	241.4	269.3	0.404	87

*Tc = time of concentration, *CN = curve number

Table 5. Existing Conditions Point of Interest Peak Flow Rates.

Point of Interest	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
Point 1	1,031	1,747	2,802

*Q = flow rate, *cfs = cubic feet per second

3.2 Detention Requirements

Per APWA Section 5608.4 and the City of Lee's Summit criteria, the performance criteria for comprehensive control is to provide detention to limit peak flow rates at downstream points of interest to maximum release rates:

- 50 percent storm peak rate less than or equal to 0.5 cfs per site acre
- 10 percent storm peak rate less than or equal to 2.0 cfs per site acre
- 1 percent storm peak rate less than or equal to 3.0 cfs per site acre
- Extended detention of the 90-percent mean annual event

Allowable release rates were calculated for the points of interest, allowing that discharges from off-site area and undeveloped portions of on-site area would be permitted to bypass the detention. Bypass peak flow rates were calculated as the percentage of the existing conditions, relating to the percentage of off-site/undeveloped on-site area flowing to each point. The development release rates for the project were calculated based on City of Lee's Summit detention criteria. The development release rates were added to the bypass peak flow rates to calculate an allowable peak flow rate for each point of interest. Refer to the equation below:

*Allowable Release Rate = (percent off-site area * existing peak flow) + (on-site area * allowable cfs per site acre)*

Tables 6 and 7 below summarize the amount of area on-site and the allowable discharges for each storm event. Methodology for determining detention requirements remains the same as in the preliminary stormwater drainage study.

There are minor changes to the on-site area and allowable release rates outlined in Tables 6 and 7 due to refinements to the grading boundary. For the purposed of this analysis, the proposed grading boundary for the project (all phases) has been considered as on-site area. There are portions of land within the project boundary that will remain undisturbed and are not included in the project boundary. The proposed re-routing of NW Main Street has not been considered as on-site area as this work eventually become new city-maintained right-of-way. The allowable release rates from this private site do not include the roadway improvements, however, curve number calculations for the off-site drainage areas have been updated to account for the increase in impervious area caused by the roadway improvements. Refer to Exhibit 4 in Appendix A for the on-site areas.

Table 6. Point of Interest On-site Area.

Point of Interest	Total Area[1] (acres)	On-site Area[1] (acres)	Percent On-site
Point 1	443.3	57.4	13.0%

*Q = flow rate, *cfs = cubic feet per second

[1]Total area draining to basins A-1, B-1, B-2, B-3, B-4, B-5, and C-1 in proposed conditions

Table 7. Allowable Peak Flow Rates.

Point of Interest	Allowable 2-Year (cfs)	Allowable 10-Year Q (cfs)	Allowable 100-Year Q (cfs)
Point 1	926	1,635	2,611

*Q = flow rate, *cfs = cubic feet per second

3.3 Stream Buffer

No changes to the existing stream buffers have occurred since the approval of the preliminary stormwater drainage study.

Little Cedar Creek and the unnamed tributary to Little Cedar Creek fall within the requirements of KC-APWA Section 5605.3 Stream Preservation and Buffers Zones. This approach to designating the stream buffer width includes defining the Ordinary High-Water Mark (OHM) and defining a width of preservation zone from the OHM on either side of the channel. The OHM for each channel was roughly defined using surveyed contours and aerial data.

Little Cedar Creek flows through the site and is located within Drainage Area B prior to its confluence with the unnamed tributary to Little Cedar Creek, at Point 2. Little Cedar Creek flows into the site on the eastern property boundary with approximately 150 acres of contributing drainage area at Point 2. Per KC-APWA Table 5605-1, the stream buffer width for this channel is defined as 60 feet measured outwards from the OHM in each direction.

The unnamed tributary to Little Cedar Creek flows through the site and is located within Drainage Area C. The tributary flows into the site along the western property boundary with approximately 270 acres of contributing drainage area at Point 2. Per KC-APWA Table 5605-1, the stream buffer width for this channel is defined as 100 feet measured outwards from the OHM in each direction. This same buffer width applies to Little Cedar Creek downstream of Point 2, which has approximately 440 acres of contributing drainage area at Point 1.

3.4 Required Level of Service and Stormwater BMP's

There is a minor change in post development curve numbers from the preliminary stormwater drainage study due to refinement of the impervious areas during Phase I.

The required level of service (LS) for the project was calculated based off the criteria outlined in the Mid-America Regional Council's (MARC) Manual of Best Management Practices for Stormwater Quality (BMP's). Worksheet 1 of the Marc BMP manual was used to calculate the required LS of 6 for the project by calculating the pre-development (82) and post-development (87) curve numbers. This value was used to design the proposed stormwater BMP's for the project, which are discussed in further detail in Section 4.6. See Appendix D for a copy of Worksheet 1.

4. PROPOSED CONDITIONS

The methodology for the proposed conditions section remains the same as in the preliminary stormwater drainage study.

The proposed conditions sections of this analysis assume completion the project. The difference between the existing conditions model and the proposed conditions model is a direct result of the project. Refer to Exhibit 5 in Appendix A for the proposed conditions drainage area map.

4.1 Effects of Development

The modeled drainage areas and points of interest are similar to the existing conditions model. However, throughout the site, some shifting of ridgelines will occur, accommodating proposed detention facilities and anticipated grading activities, which will change the relative areas draining to each point of interest. Proposed conditions drainage patterns remain the same as in the preliminary stormwater drainage study, with minor changes to acreage for individual drainage areas. Due to proposed grading activities, not all of the on-site area is able to be captured by proposed drainage areas. This also causes area that is not being developed (considered off-site for the purposes of this study) to drain to the proposed detention basins. For these reasons, the on-site area listed in Table 6 may not match up exactly with the total of the proposed conditions drainage areas (A-1, B-1, B-2, B-3, B-4, B-5, and C-1). The following is a summary of the proposed conditions drainage areas.

Drainage Area A in proposed conditions is approximately 22.3 acres overall. Proposed grading activities and construction of buildings on-site will alter ridgelines from existing conditions, shifting area between drainage areas A and B. A portion of the on-site area within this drainage area will be developed and re-graded and has been separated as **Drainage Area A-1**. Runoff from Drainage Area A-1 will be routed to an on-site detention basin and then discharged into Little Cedar Creek, upstream of Point 1. Proposed conditions for Drainage Area A-1 also includes area previously located outside of Drainage Area A, at the northwest corner of the site, which will be re-graded to provide room for the aforementioned detention basin. A small sliver of the on-site area along the western boundary still remains outside of the extents of Drainage Area A-1; this on-site area will not be disturbed as part of this project. The remaining area in Drainage Area A is off-site or will not be developed as part of the project.

Drainage Area B in proposed conditions is approximately 162.4 acres overall. Proposed grading activities and construction of buildings on-site will alter ridgelines from existing conditions, shifting area between drainage areas A, B, and C. The on-site portion of Drainage Area B has been split up into **drainage areas B-1, B-2, B-3, B-4,** and **B-5** based off the location of proposed low points. A detention basin will be constructed at the proposed low points for each of the aforementioned on-site drainage areas. Runoff from these drainage areas will be routed to the on-site detention basins and then discharged into Little Cedar Creek, upstream of Point 2. The remaining area in Drainage Area B is off-site or will not be developed as part of the project. Changes to the proposed grading since the preliminary stormwater drainage study have removed drainage area B-6 and the associated detention basin.

Drainage Area C in proposed conditions is approximately 258.7 acres overall. Proposed grading activities and construction of buildings on-site will alter ridgelines from existing conditions, shifting area between drainage areas B and C. The on-site portion of Drainage Area C has been split out into **Drainage Area C-1**. Runoff from Drainage Area C-1 will be routed to an on-site detention basin and then discharged into the unnamed tributary to Little Cedar Creek, upstream of Point 2. The remaining area in Drainage Area C is off-site or will not be developed as part of the project.

4.2 Hydrologic Analysis (Proposed Conditions)

The analysis provided in Section 3 established existing conditions of the development's drainage areas. The analysis in Section 4 will provide guidance for configuring the detention basin to meet the objectives established in Section 3. Proposed curve numbers for the on-site drainage areas were calculated based off impervious areas for the developed site. Proposed curve numbers for the off-site drainage areas (A, B, and C) were also adjusted accordingly to account for the on-site areas being split out into separate drainage areas. Times of concentration for on-site drainage areas within Phase I of the project (B-4, B-5, and C-1) were calculated using SCS TR-55 methodology, which were then used to estimate lag times. Lag times of 5-7 minutes were assumed for the remaining on-site drainage areas, which is consistent with the preliminary stormwater drainage study. Lag time can be estimated as 60-percent of the time of concentration for a watershed; for example, a lag time of 5 minutes corresponds to a time of concentration of 8.33 minutes (0.139 hours). Detailed calculations of lag times for the remaining on-site drainage areas will be completed and provided with subsequent submittals.

The following tables summarize the results of the proposed conditions analysis. Table 8 summarizes the proposed conditions drainage area data. Tables 9 and 10 assume no detention is provided, to demonstrate the effects of development for each drainage area. Refer to Appendix B for proposed conditions curve number calculations. Refer to Appendix C for the proposed conditions HEC-HMS inputs and outputs.

Table 8. Proposed Conditions Drainage Area Data.

Drainage Area	On-site Area (acres)	Off-site Area (acres)	Total Area (acres)	T_C[1] (hour)	Weighted CN
A	0.0	13.5	13.5	0.197	89
B	0.0	123.4	123.4	0.355	88
C	0.0	247.1	247.1	0.404	88
A-1	8.8	0.0	8.8	0.194	91
B-1	1.7	0.0	1.7	0.139	87
B-2	9.6	0.0	9.6	0.194	92
B-3	8.3	0.0	8.3	0.194	92
B-4	13.2	0.0	13.2	0.100	89
B-5	3.9	0.0	3.9	0.100	90
C-1	13.5	0.0	13.5	0.197	91

*Tc = time of concentration, *CN = curve number
[1]Hydrologic model elements are referenced by lag time, minimum time of concentration of 6 minutes (0.100 hours) per SCS TR-55

Table 9 shows post-development peak discharge values points of interest assuming no detention is provided. Table 10 compares these to the existing conditions from Section 3 at the points of interest. Negative values indicate a reduction in peak flow rate, while positive values indicate an increase. Without detention, flow rates will increase from existing conditions at Point 1 for the 2-, 10-, and 100-year storms. Proposed conditions peak flow rates without detention are higher than allowable release rates for the 2-, 10-, and 100-year storms. Section 4.4 will analyze the effects of detention on proposed conditions peak flow rates and provide a comparison to peak flow rates without detention to determine if detention is beneficial for this project.

Table 9. Proposed (No Detention) Conditions Point of Interest Peak Flow Rates.

Point of Interest	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
Point 1	1,094	1,813	2,865

*Q = flow rate, *cfs = cubic feet per second

Table 10. Proposed (No Detention) Conditions Point of Interest Peak Flows Comparison.

Point 1	ΔQ_2 (cfs)	ΔQ_{10} (cfs)	ΔQ_{100} (cfs)
Existing Conditions	+63	+66	+63
Allowable Release	+168	+178	+254

*Q = flow rate, *cfs = cubic feet per second, *Δ = difference in value

4.3 Proposed Detention Facilities

To mitigate the increases in peak flows (shown in the previous table) and, where possible, to decrease further to the allowable release rates established in Section 3, detention will be provided for each of the on-site drainage areas. These detention facilities will be constructed as part of the project. The detention facilities are designed to capture most of the site runoff and to mitigate increases in peak discharge from the site. The detention facilities will be located at various locations throughout the site, as shown on Exhibit 5 in Appendix A, and will meet the requirements outlined in Section 3.

Each detention facility will contain an outlet structure with a perforated riser set at the bottom of each structure. These risers will be sized to comply with the KC-AWPA requirement for 40-hour release of the 90-percent mean annual event for proposed conditions. Table 11 summarizes the outlet structure configurations and Table 12 summarizes the perforated riser configurations for each of the proposed detention facilities for Phase I of the project. Remaining outlet structures for proposed detention basins will be designed with future phases of the project. The top of each outlet structure will be open to allow water to overflow into the structure during the heavier intensity rainfall event. The structure height will be set so that the top of the structure is elevated above the depth of the water quality event WSE at the outlet. Each outlet structure will be a minimum height of 3 feet to allow room for the outlet pipe. Trash racks will be installed on top of each outlet structure to prevent debris from clogging the primary outlet pipes. Refer to Appendix D for perforated riser configuration calculations.

Each detention facility will also be equipped with an independent broad-crested weir graded into the berm of the basin to function as the emergency spillway. Proposed emergency spillways have been configured to meet the requirements as outlined in KC-APWA Section 5608.4 F. Table 13 summarizes minimum bottom lengths of the emergency spillways for each of the proposed detention facilities for Phase I of the project. Remaining emergency spillway configurations for proposed detention basins will be designed with future phases of the project.

Table 11. Outlet Structures Summary.

Detention Facility	Basin Top Elevation (feet)	Basin Bottom Elevation (feet)	Primary Outlet Pipe Diameter (inches)	Structure Length (feet)	Structure Width (feet)	Minimum Structure Height (feet)
B-4	947	960	15	4	4	3.3
B-5	985	979	15	4	4	3.0
C-1	978	966	15	4	4	5.6

Table 12. Perforated Riser Summary.

Detention Facility	Perforation Diameter (inches)	Number of Columns	Number of Rows[1]
B-4	1.1	1	10
B-5	1.6	1	3
C-1	0.8	1	17

[1]4-inch vertical spacing between perforations, center to center

Table 13. Emergency Spillway Summary.

Detention Facility[1]	100-Year Peak Inflow (cfs)	Spillway Bottom Elevation (feet)	Spillway Depth (feet)	Spillway Length (feet)	100-Year Depth through Spillway (feet)	Freeboard A[2] (feet)	Freeboard B[3] (feet)
B-4	130	957.0	3.0	25	1.4	2.0	1.6
B-5	39	983.0	2.0	25	0.7	1.4	1.3
C-1	117	976.4	1.6	100	0.6	0.5	1.0

*cfs = cubic feet per second, *WSE = water surface elevation
[1]Each emergency spillway is trapezoidal in shape with 4:1 horizontal to vertical side slopes
[2]Distance from peak 100-year WSE in basin to spillway bottom
[3]Distance from peak 100-year WSE through spillway to top of basin

Tables 14-16 include hydrologic summaries of the proposed detention facilities for the 2-, 10- and 100-year storm events, respectively.

Table 14. Proposed Conditions (2-Year) Detention Flow and Volume Data.

Detention Facility	Peak Q In (cfs)	TP In (hour)	Peak Q Out (cfs)	TP Out (hour)	Peak WSE (feet)	Stored Volume (acre-feet)
A-1	45	12.03	8	12.83	951.3	1.7

Detention Facility	Peak Q In (cfs)	TP In (hour)	Peak Q Out (cfs)	TP Out (hour)	Peak WSE (feet)	Stored Volume (acre-feet)
B-1	6	11.97	3	12.10	954.8	0.1
B-2	34	12.00	18	12.13	960.3	0.7
B-3	31	12.00	11	12.18	975.8	0.7
B-4	51	11.95	12	12.12	950.8	1.1
B-5	16	11.95	4	12.12	980.2	0.4
C-1	46	12.00	15	12.20	971.2	0.8

*Q = flow rate, *cfs = cubic feet per second, *TP = time of peak, *WSE = water surface elevation

Table 15. Proposed Conditions (10-Year) Detention Flow and Volume Data.

Detention Facility	Peak Q In (cfs)	TP In (hour)	Peak Q Out (cfs)	TP Out (hour)	Peak WSE (feet)	Stored Volume (acre-feet)
A-1	75	12.03	11	12.88	953.7	3.0
B-1	10	11.97	6	12.07	955.2	0.2
B-2	54	12.00	31	12.13	961.2	1.0
B-3	49	12.00	15	12.20	977.5	1.1
B-4	83	11.95	14	12.13	952.6	1.8
B-5	25	11.93	6	12.12	980.8	0.6
C-1	75	12.00	16	12.27	973.4	1.6

*Q = flow rate, *cfs = cubic feet per second, *TP = time of peak, *WSE = water surface elevation

Table 16. Proposed Conditions (100-Year) Detention Flow and Volume Data.

Detention Facility	Peak Q In (cfs)	TP In (hour)	Peak Q Out (cfs)	TP Out (hour)	Peak WSE (feet)	Stored Volume (acre-feet)
A-1	115	12.02	15	12.98	957.3	4.9
B-1	16	11.97	12	12.05	955.7	0.2
B-2	82	12.00	43	12.13	962.7	1.5
B-3	74	12.00	19	12.22	980.0	1.7
B-4	130	12.93	15	12.32	955.0	3.1
B-5	39	11.93	9	12.12	981.6	0.9
C-1	117	12.00	17	12.23	975.9	2.8

*Q = flow rate, *cfs = cubic feet per second, *TP = time of peak, *WSE = water surface elevation

4.4 Effects of Proposed Detention

The following tables compare the results of the proposed conditions analysis with the detention described above to the existing conditions from Section 3 at the points of interest. Table 17 shows peak discharge values at the point of interest. Table 18 compares these discharge values to existing and allowable discharge values. In Table 18, negative values indicate a reduction in peak flows, while positive values indicate an increase.

Table 17. Proposed (with Detention) Point of Interest Peak Flow Rates.

Point of Interest	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
Point 1	993	1,635	2,570

*Q = flow rate, *cfs = cubic feet per second

Table 18. Proposed (with Detention) Conditions Point of Interest Peak Flows Comparison.

Point 1	Δ Q_2 (cfs)	Δ Q_{10} (cfs)	Δ Q_{100} (cfs)
Existing Conditions	-38	-112	-232
Allowable Release	+67	0	-41
Proposed Conditions (No Detention)	-101	-178	-295

*Q = flow rate, *cfs = cubic feet per second, *Δ = difference in value

As shown in Table 18, with the addition of detention facilities, peak discharges at Point 1 will be at or below the allowable release rates for the 10-year and 100-year storm; however, the proposed conditions (with detention) peak flow rate for the 2-year storm is above the allowable release rate. The proposed conditions (with detention) peak flow rate for the 2-year storm is lower than existing conditions by approximately 3.7% and higher than allowable by approximately 7.2%. Proposed conditions peak flow rates (with detention) are lower than the proposed conditions peak flow rates (without detention) and lower than the existing conditions peak flow rates for the 2-, 10- and 100-year storms.

A waiver is requested for detention of the 2-year event at the point of interest. In order to detain to the 2-year allowable release rate, the current proposed conditions (with detention) release rate at Point 1 must be lowered by 67 cfs. The sum of the outflows for the proposed conditions detention basins (seen in Tables 14-16) is approximately 53 cfs; note that A1 and B2 are interconnected so only the outflow at A1 is considered. Therefore, it is not possible to detain the 2-year event for the given point of interest. This is due to the stringent restriction of allowing only 0.5 cfs per acre of on-site area for the 2-year event. For reference, the existing conditions 2-year peak flow rate at Point 1 is approximately 2.3 cfs per acre. The outflow from the detention basins could be lowered by placing a restrictor plate on the outflow pipes to the detention basins. The outlet pipes for the detention basins in Phase I of the project (B-4, B-5, and C-1) are all 15-inches in diameter. Decreasing the size of these pipes is not recommended as this increases the chance for clogging and maintenance needs. Further consideration will be given to the remaining detention basin outlets during subsequent phases of the project to lower the proposed 2-year release rate as reasonably able.

4.5 Impacts to Stream Buffer

Much of the defined stream buffer is not impacted by development; however, a few encroachments have been made to accommodate the proposed layout, which are summarized below:

Little Cedar Creek

Impacts to the stream buffer along Little Cedar Creek will occur at several locations along the site due to proximity of proposed roadway alignments and parking lots. These areas are summarized below and can be seen on Exhibit 6 of Appendix A:

- Toward the middle of the site, just upstream of NW Main St. The proposed alignment for NW Main St encroaches slightly on the 60-foot stream buffer for the south side of the stream. To account for this loss in stream buffer on the south side of the stream, additional width has been provided in this area on the north side. A waiver is requested for this area and has been included with this submittal in Appendix E.
- Just upstream of Point 2, on the north side of the stream. The 60-foot stream buffer in this area will be encroached upon with construction of the detention facility for Drainage Area B-1. To account for this loss in stream buffer on the north side of the stream, additional width has been provided in this area on the south side. A waiver will be requested in subsequent phases of the project when this area is developed.
- Just upstream of the stream’s crossing with the Union Pacific Railroad. The proposed loading dock in encroaches on the 100-foot stream buffer in this area on the west side of the stream. The stream in this area has been previously impacted and straightened by the nearby railroad crossing. The existing stream is confined and has little potential for migration due to the proximity of the railroad culvert. A waiver will be requested in subsequent phases of the project when this area is developed.

Additional temporary encroachments on the stream buffer may also take place with proposed grading and construction activities. These areas will be replanted with native grasses to restore the vegetation as much as possible.

Unnamed Tributary to Little Cedar Creek

Impacts to the stream buffer along the unnamed tributary to Little Cedar Creek will occur at several locations along the site due to proximity of proposed roadway alignments and parking lots. These areas are summarized below and can be seen on Exhibit 6 of Appendix A:

- The proposed roadway on the west side of the building encroaches on the 100-foot stream buffer on the stream’s east side, near the northwest corner of drainage area C-1. A waiver for this area is requested. The stream in this area has been previously impacted and straightened by the railroad to the west. The existing stream has little potential for significant migration to the east towards the loading docks due to its orientation, running parallel to the railroad. A waiver is requested for this area and has been included with this submittal in Appendix E.

Additional temporary encroachments on the stream buffer may also take place with proposed grading and construction activities. These areas will be replanted with native grasses to restore the vegetation as much as possible.

4.6 Provided Level of Service and Stormwater BMP's

As discussed in Section 3.4, the required LS for the project is 6 based on the pre-development and post-development curve numbers. Stormwater BMP calculations were performed using Worksheet 2 of the MARC BMP manual to design and select appropriate BMP's for the project and meet the required LS. The selected BMP's for the project are summarized below:

- Establishing and preserving native vegetation – This BMP includes the establishment or preservation of native plant types historically present on the existing site. These plant species are well adapted to the climate and natural disturbances in the region.
- Snout system to extended dry detention (treatment train) – Extended dry detention basins are detention facilities designed to detain the water quality volume for 40 hours, which are also vegetated with native plants. A snout system will be placed upstream of each detention basin, which will provide additional treatment and benefits to water quality prior to entering the basin and eventually discharging into the nearby streams. Per the manufacturer's website, "A snout is a vented fiberglass water quality hood that is installed over the outlet pipe in a storm water structure with a sump that skims oils, floatables, and trash off of the surface water while letting settleable solids sink to the bottom. The cleaner water exits from beneath the SNOUT, which is lower than the bottom of the pipe, but above the bottom of the structure allowing both floatable material and solids that sink to stay in the structure."

Portions of the site will be untreated due to grading restrictions preventing some areas from being able to drain to proposed detention basins or other treatment facilities. The provided LS for the project was calculated to be approximately 6.05, which is above the required LS of 6. See Appendix D for a copy of Worksheet 2.

5. FUTURE CONDITIONS

Future conditions represent the built-out state of Phase I of the project with the addition of a future parking to be installed in the undeveloped portion of drainage area C-1, generally located at the southeast corner of the site. Drainage areas B-4 and B-5 will be fully developed with Phase I of the project and therefore the proposed conditions are the same as the future conditions for these areas.

The additional parking for drainage area C-1 would cause an increase in impervious area, therefore increasing the curve number, peak flows, and peak volumes in the drainage area and to the proposed detention basin. Table 19 summarizes the effects of the future parking lot.

Table 19. Effects of Future Conditions on Drainage Area C-1.

Attribute	Proposed Conditions	Future Conditions
Impervious Area (Acres)	6.7	7.9
Total Area (Acres)	13.5	13.5
Curve Number	89	91
Emergency Spillway Freeboard[1]	1.04	0.84

[1]Freeboard from 100-year peak water surface elevation through spillway to top of detention basin, 1 foot minimum, dimension in feet

The increase in impervious area will require modifications to the proposed emergency spillway configuration in order to meet the city's design criteria. Alternatively, additional detention area could be provided by modifying the proposed detention basin or adding additional storage elsewhere in the drainage area. Prior to construction of the future parking lot, the time of concentration for area C-1 will need to be recalculated using the future grades in order to determine what modifications will be needed.

6. SUMMARY

This stormwater drainage study was prepared to evaluate the hydrologic impact generated by the Scannell Development project and to provide recommendations for a comprehensive stormwater management plan. The project is a proposed industrial development on approximately 83 acres, including warehouses, stormwater detention basins, and open space and vegetation along the existing streams that flow through the site

Increases in peak flow rates for the 2-, 10- and 100-year storms caused by the development will be mitigated for all points of interest through the site through a combination of detention facilities and drainage area changes. A waiver is requested for meeting the allowable release rates for the 2-year storm at the point of interest. The detention facilities will also serve as water quality basins and provide detention of the 90-percent mean annual event.

Stream buffers were designated based on watershed size, per KC-APWA standards. A waiver is requested for encroachments on the stream buffers as noted in Section 4.5. Where encroachments are necessary, the impacts will be mitigated with preservation of adjacent native vegetation and establishment of new native vegetation elsewhere on-site as able.

7. CONCLUSIONS AND RECOMMENDATIONS

This proposed stormwater management plan was designed to achieve compliance with current design criteria in effect for the City of Lee's Summit, Missouri; however, a waiver is requested for encroachments to stream buffers at several locations and for meeting the allowable release rates for the 2-year storm. Subsequent final drainage studies will be required with the submittal of future phases of this project.

The results of the analysis demonstrate that the future stormwater management plan for the project will achieve compliance with design criteria or the requested waiver. We therefore request approval of this Scannell Development Final Stormwater Drainage Study for Phase I of the project. This approval is conditional and should be substantiated with each phase of the project.

8. REFERENCES

Best Management Products, Inc. (2021). “Frequently Asked Questions” <https://www.bmpinc.com/faq/>

City of Lee’s Summit. (2020). “Section 5600 – Storm Drainage Systems & Facilities, City of Lee’s Summit, Missouri, Design Criteria”

FEMA (Federal Emergency Management Agency). (2021). “FEMA Flood Map Service Center”. <https://msc.fema.gov/portal/home> (Jun. 23, 2021).

KC-APWA (American Public Works Association, Kansas City Metropolitan Chapter). (2011). “Division V Section 5600 Storm Drainage Systems & Facilities”.

NRCS (Natural Resources Conservation Service). (2021). “Web Soil Survey” <https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx> (Jun. 23, 2021).

United States Weather Bureau. “Technical Paper No. 40 Rainfall Frequency Atlas of the United States”. (1961). Department of Commerce, Washington, D.C.

APPENDIX A

Site Maps

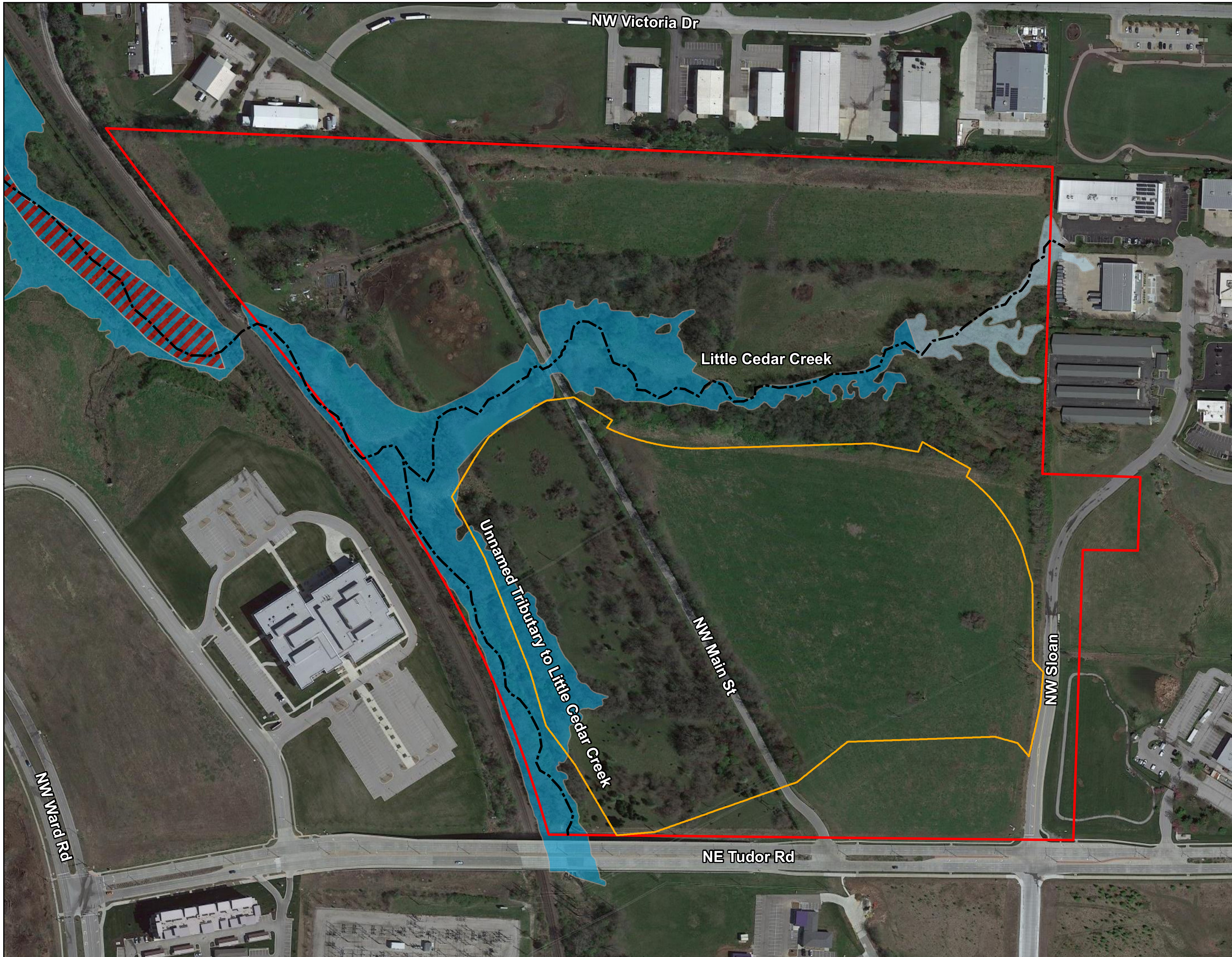

Exhibit 1.
Floodplain Map

Scannell Development Project
Phase I
Lee's Summit, MO

Project Boundary
Phase I Grading Boundary
Stream Centerline
FEMA Floodway
FEMA Zone AE
FEMA Zone A

0 250 500 Feet

1 Inch = 250 Feet

N

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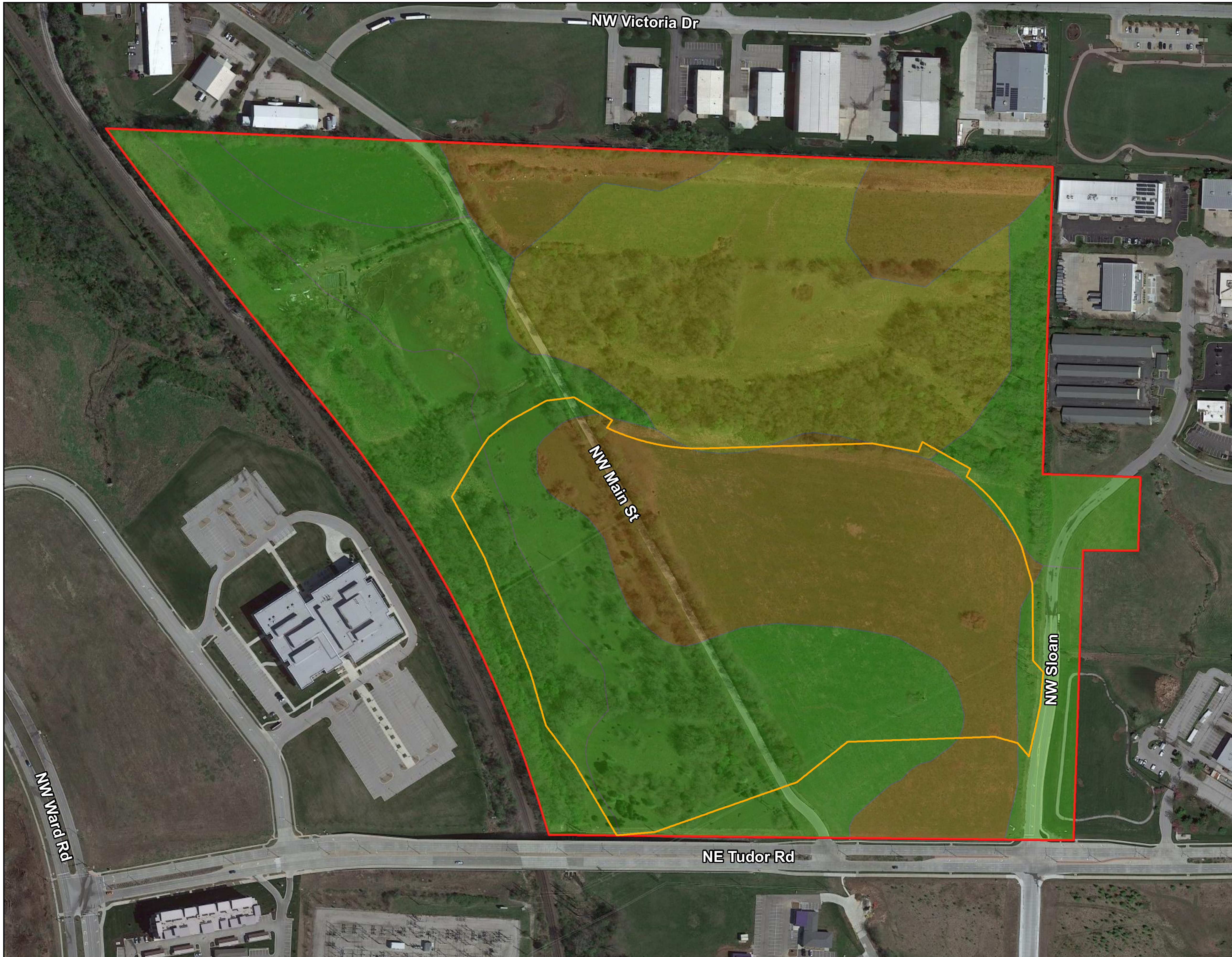

Exhibit 2.
Soils Map

Scannell Development Project
Lee's Summit, MO

Project Boundary
Phase I Grading Boundary
Group C
Groups C/D
Group D

0 250 500
Feet

1 Inch = 250 Feet

N

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Exhibit 3. Existing Conditions Drainage Areas

Scannell Development
Phase I
Lee's Summit, MO

Existing Contours
Phase I Grading Boundary
Drainage Area A
Drainage Area B
Drainage Area C
Project Boundary

0 600 1,200 Feet

1 Inch = 600 Feet

N

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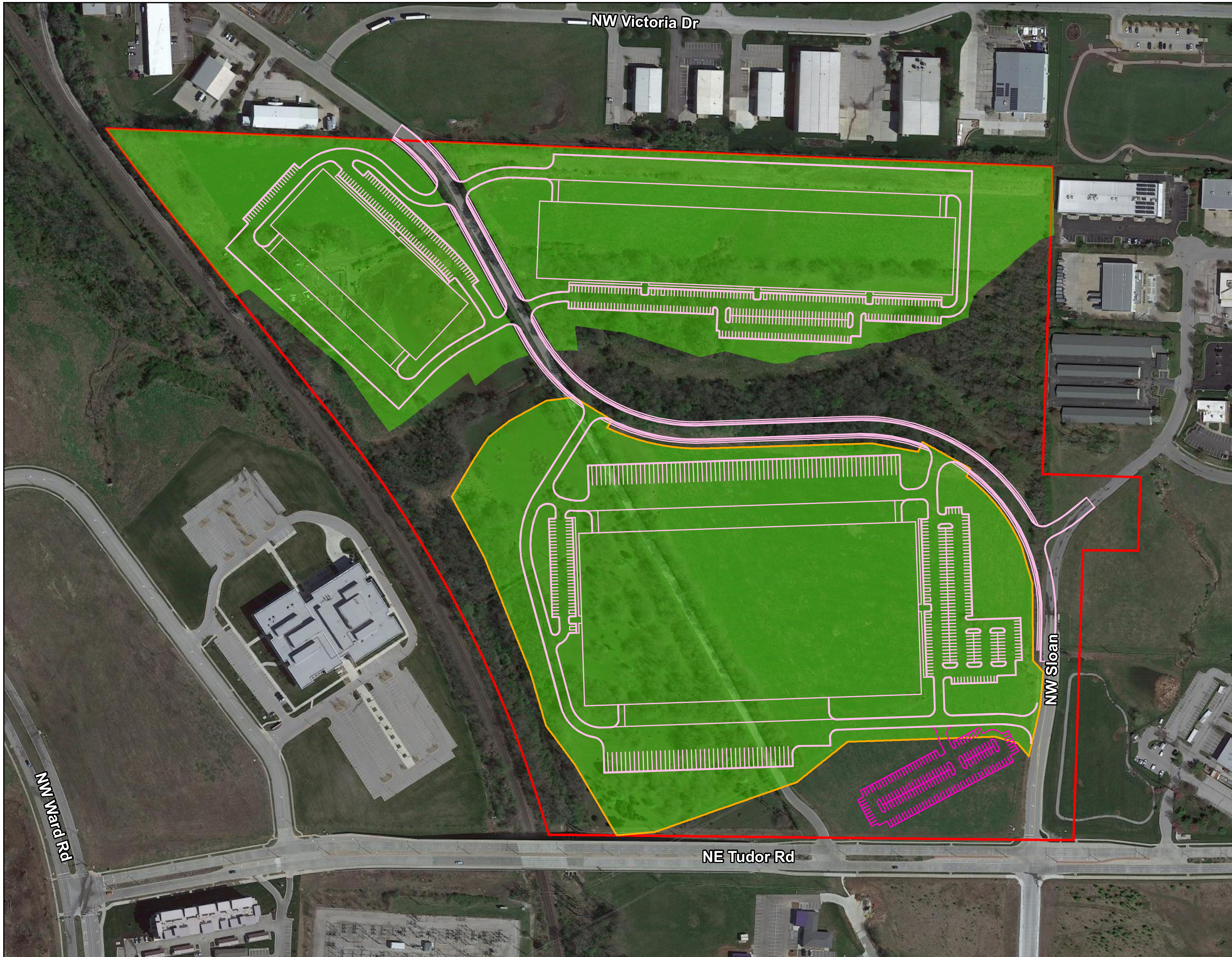

Exhibit 4.
On-Site Areas

Scannell Development Project
Phase I
Lee's Summit, MO

Site Layout (Future)
Site Layout (Proposed)
Phase I Grading Boundary
On-Site Area
Project Boundary

0 250 500 Feet

1 Inch = 250 Feet

N

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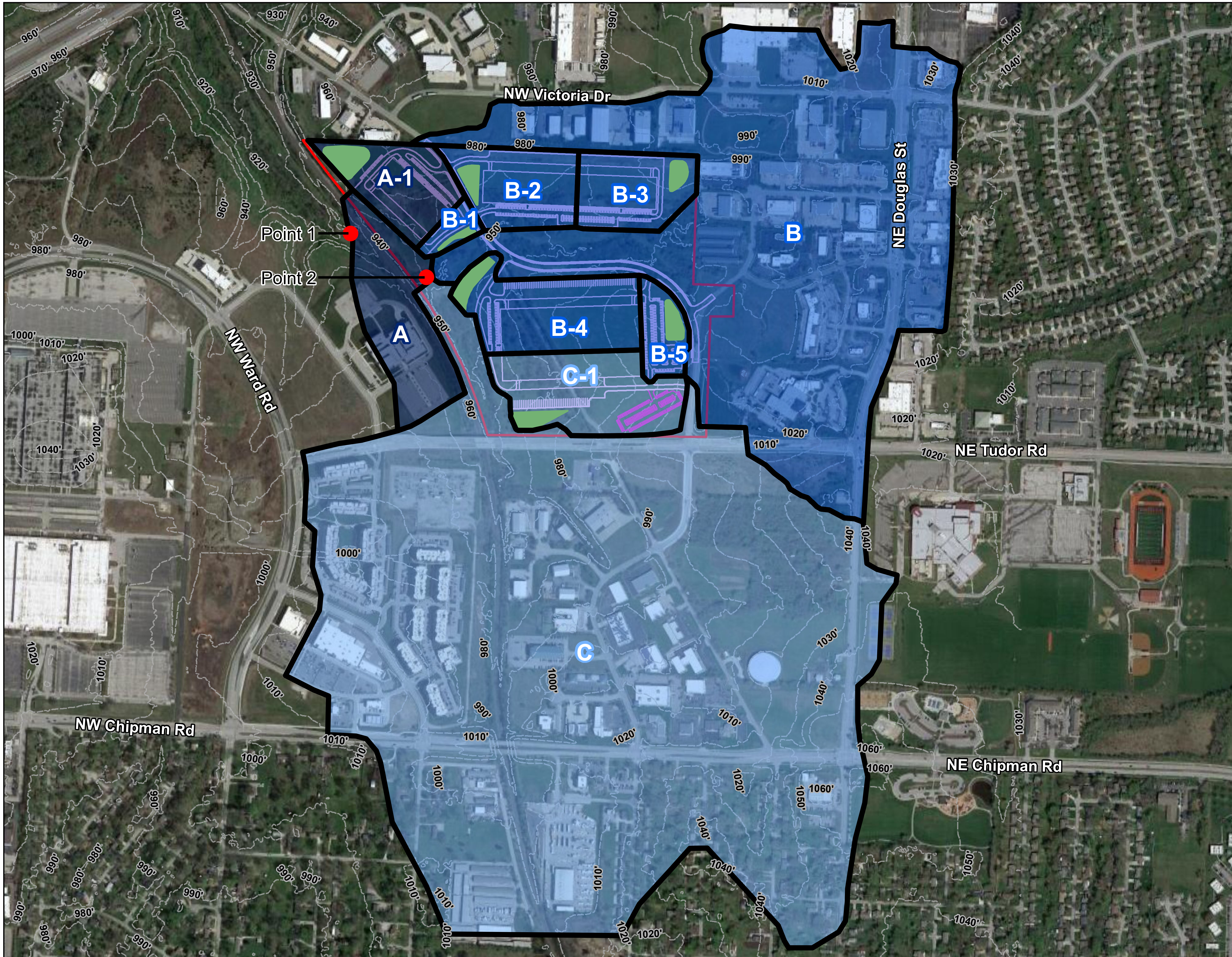

Exhibit 5.
Proposed Conditions
Drainage Areas

Scannell Development
Phase I
Lee's Summit, MO

Existing Contours
Site Layout (Future)
Site Layout (Proposed)
Proposed Drainage Area
Proposed Detention Basin
Project Boundary

0 600 1,200 Feet

1 Inch = 600 Feet

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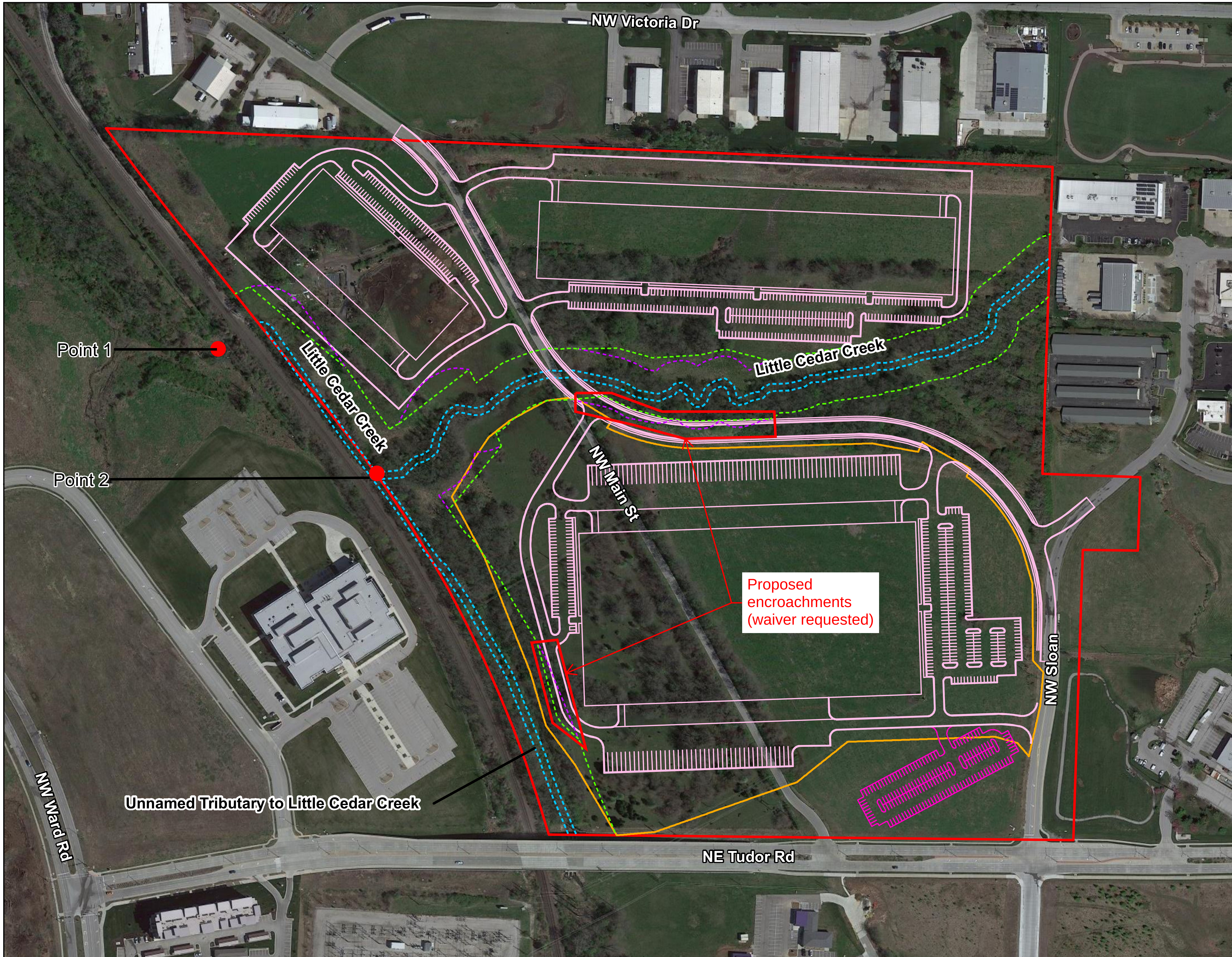
Exhibit 6.
Stream Buffer Map
Scannell Development Project
Phase I
Lee's Summit, MO



Ordinary High-water Mark
Required Stream Buffer
Proposed Stream Buffer
Site Layout (Future)
Site Layout (Proposed)
Phase I Grading Boundary
Project Boundary

0 250 500
Feet

1 Inch = 250 Feet

N

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APPENDIX B

Curve Number & Time of Concentration Calcs

Existing Curve Number – Drainage Area A, Curve Number = 86

Land Use	HSG	CN	Area (Acres)
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	C	74	0.7
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	D	80	10.6
Impervious Areas Paved parking lots, roofs, driveways	C/D	98	9.1
Residential Districts (1/4 acre)	C	83	0.0
Residential Districts (1/4 acre)	D	87	0.0
Urban Districts Commercial & Business	C	94	0.0
Urban Districts Commercial & Business	D	95	0.0

*HSG = Hydrologic Soil Group, *CN = Curve Number

Existing Curve Number – Drainage Area B, Curve Number = 86

Land Use	HSG	CN	Area (Acres)
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	C	74	46.7
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	D	80	24.2
Impervious Areas Paved parking lots, roofs, driveways	C/D	98	8.2
Residential Districts (1/4 acre)	C	83	0.0
Residential Districts (1/4 acre)	D	87	0.0
Urban Districts Commercial & Business	C	94	47.6
Urban Districts Commercial & Business	D	95	21.6

*HSG = Hydrologic Soil Group, *CN = Curve Number

Existing Curve Number – Drainage Area C, Curve Number = 87

Land Use	HSG	CN	Area (Acres)
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	C	74	54.2
Fully Developed Urban Areas Good Condition; Grass Cover > 75%	D	80	39.1
Impervious Areas Paved parking lots, roofs, driveways	C/D	98	34.7
Residential Districts (1/4 acre)	C	83	20.8
Residential Districts (1/4 acre)	D	87	17.7

Land Use	HSG	CN	Area (Acres)
Urban Districts Commercial & Business	C	94	67.0
Urban Districts Commercial & Business	D	95	35.8

*HSG = Hydrologic Soil Group, *CN = Curve Number

Existing Time of Concentration – Area A

Flow Type	Length (feet)	Slope (feet/feet)	Surface (Manning's n)	Velocity (feet per second)	Time (hour)
Sheet	100	0.027	Grass-Range, Short (0.15)		0.134
Shallow Concentrated	219	0.024	Unpaved		0.024
Channel	1,415			10	0.039
Total	1,734				0.197

Existing Time of Concentration – Area B

Flow Type	Length (feet)	Slope (feet/feet)	Surface (Manning's n)	Velocity (feet per second)	Time (hour)
Sheet	100	0.020	Grass-Range, Short (0.15)		0.152
Shallow Concentrated	520	0.050	Unpaved		0.040
Channel	4,118			7	0.163
Total	4,738				0.355

Existing Time of Concentration – Area C

Flow Type	Length (feet)	Slope (feet/feet)	Surface (Manning's n)	Velocity (feet per second)	Time (hour)
Sheet	100	0.020	Grass-Range, Short (0.15)		0.152
Shallow Concentrated	297	0.021	Unpaved		0.035
Channel	5,471			7	0.217
Total	5,898				0.404

Proposed Curve Numbers - Subareas

Drainage Area[1]	Total Area	Pervious Area	Pervious CN	Impervious Area	Impervious CN	Weighted CN
A-1	8.8	3.3	80	5.5	98	91
B-1	1.8	1.0	80	0.7	98	88
B-2	9.2	3.2	80	6.0	98	92
B-3	8.3	2.9	80	5.4	98	92
B-4	13.2	7.0	80	6.2	98	89
B-5	3.9	1.8	80	2.1	98	90
C-1	13.5	6.8	80	6.7	98	89

*CN = Curve Number, [1]All areas shown in this table are in acres

Existing Curve Numbers to Proposed Curve Number Adjustments Part 1

Drainage Area[1]	Existing Area	Existing CN	Proposed Area	Area Change	Area Change CN	Weighted CN
A	20.3	86	13.5	6.8	80	89
B	150.9	86	123.4	27.5	80	87
C	269.3	87	247.1	22.2	80	88

*CN = Curve Number, [1]All areas shown in this table are in acres
Note: These calculations account for shifting in drainage boundaries due to proposed grading activities

Existing Curve Numbers to Proposed Curve Number Adjustments Part 2

Drainage Area[1]	Proposed Area	Area Change	Area Change CN	Weighted CN 2
A	13.5	0.0	95	89
B	123.4	7.9	95	88
C	247.1	0.3	95	88

*CN = Curve Number, [1]All areas shown in this table are in acres
Note: These calculations account for construction of the NW Main Street Improvements

Proposed Time of Concentration – Area B4

Flow Type	Length (feet)	Slope (feet/feet)	Surface (Manning's n)	Velocity (feet per second)	Time (hour)[1]
Sheet	60	0.0639	Smooth Surface (0.011)		0.008
Shallow Concentrated	50	0.0190	Paved		0.005
Channel	1,265			10	0.032
Total	1,375				0.045

[1]Minimum time of concentration of 6 minutes used per SCS TR-55 methodology

Proposed Time of Concentration – Area B5

Flow Type	Length (feet)	Slope (feet/feet)	Surface (Manning's n)	Velocity (feet per second)	Time (hour)[1]
Sheet	60	0.020	Grass-Range, Short (0.15)		0.152
Shallow Concentrated					
Channel	515			10	0.014
Total	575				0.087

[1]Minimum time of concentration of 6 minutes used per SCS TR-55 methodology

Proposed Time of Concentration – Area C1

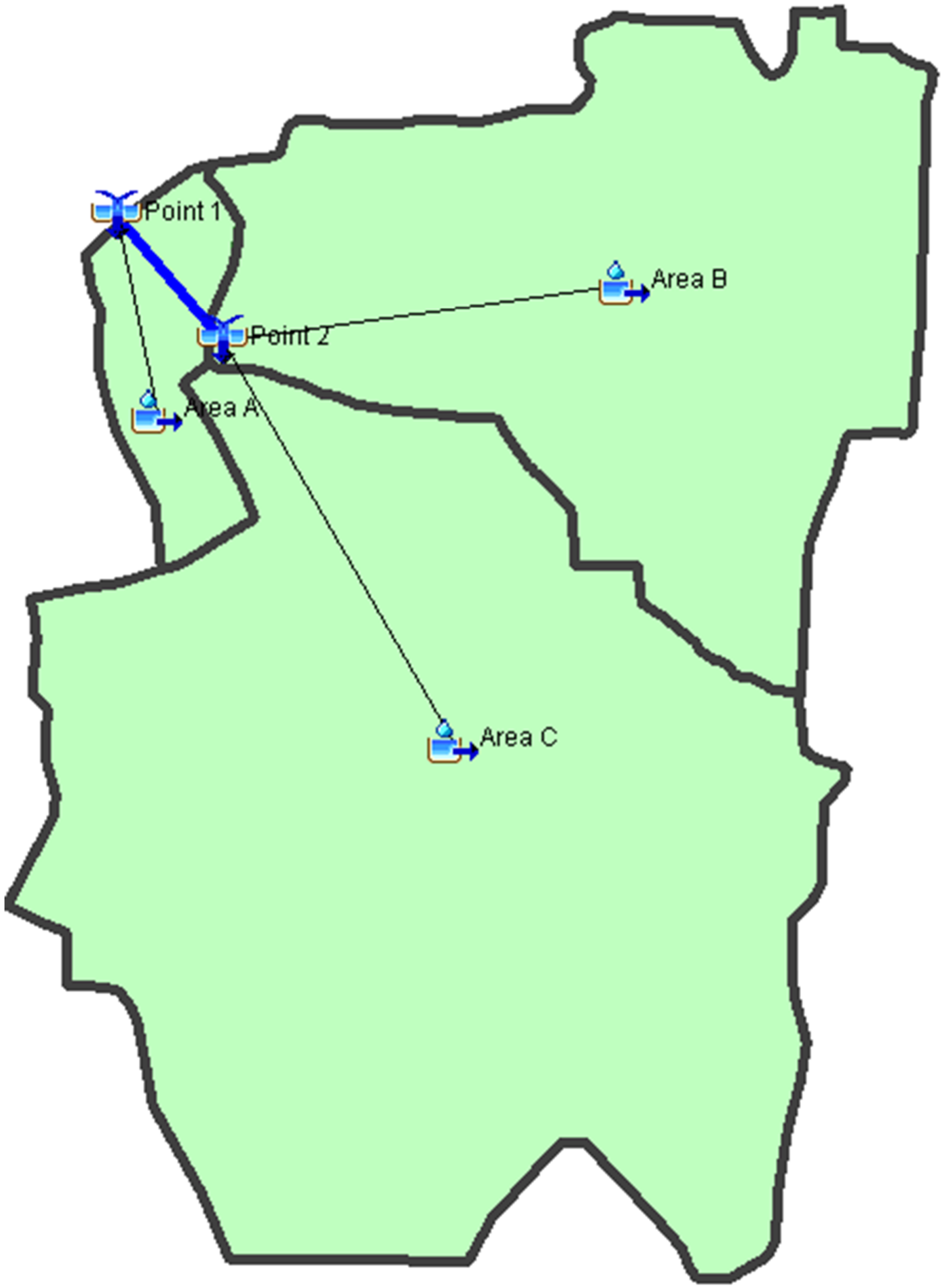
Flow Type	Length (feet)	Slope (feet/feet)	Surface (Manning's n)	Velocity (feet per second)	Time (hour)
Sheet	100	0.0504	Grass-Range, Dense (0.24)		0.157
Shallow Concentrated	277	0.0740	Unpaved		0.018
Channel	807			10	0.022
Total	1,184				0.197

APPENDIX C

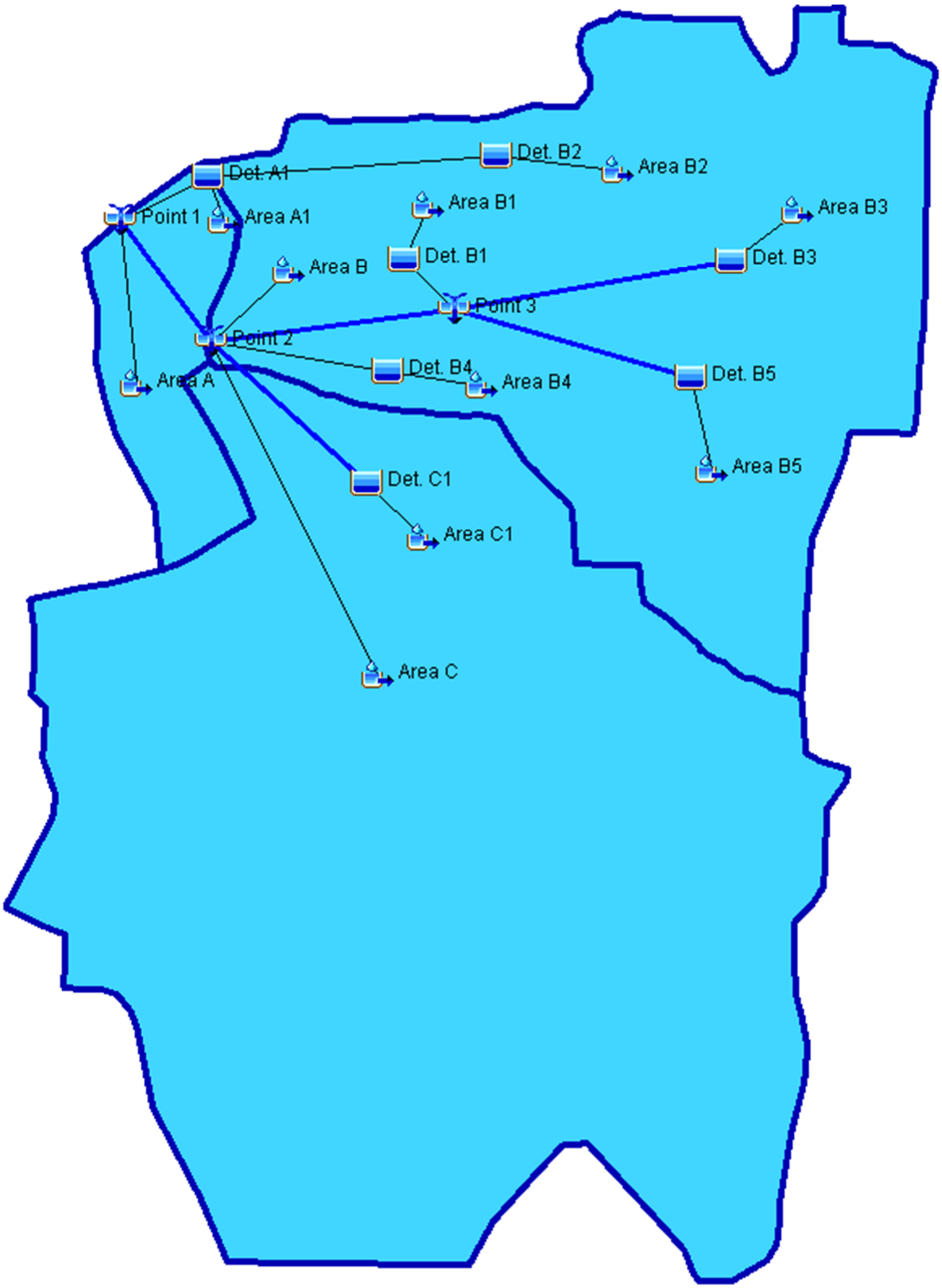
Hydrologic Inputs and Outputs

Note: Contains model schematics, detention basin storage information, detention basin outlet information, and emergency spillway information. An electronic copy of the HEC-HMS model has been included with this submittal. Please refer to the model for inflow hydrographs, rating curves, etc.

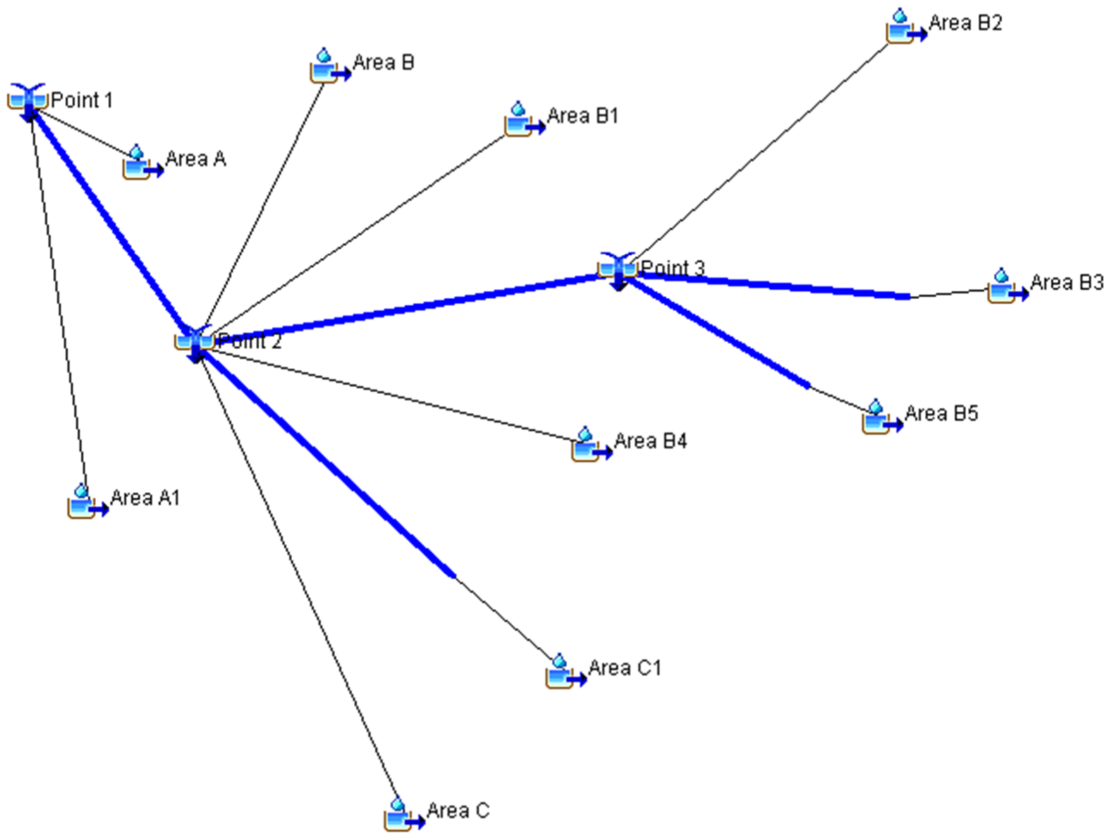
EXISTING CONDITIONS SCHEMATIC



PROPOSED CONDITIONS (WITH DETENTION) / FUTURE CONDITIONS SCHEMATIC



PROPOSED CONDITIONS (WITHOUT DETENTION) SCHEMATIC



Detention Basin Elevation Area Information

A1

Elevation (FT)	Area (FT2)
948.0	6763
960.0	45022

B1

Elevation (FT)	Area (FT2)
954.0	1539
958.0	11690

B2

Elevation (FT)	Area (FT2)
958.0	5000
965.0	24039

B3

Elevation (FT)	Area (FT2)
973.0	2897
982.0	21617

B4

Elevation (FT)	Area (FT2)
947.0	8140
948.0	10127
949.0	12194
950.0	14343
951.0	16572
952.0	18883
953.0	21276
954.0	23753
955.0	26314
956.0	28960
957.0	31693
958.0	34512
959.0	37420
960.0	40418

B5

Elevation (FT)	Area (FT2)
979.0	12269
980.0	14507
981.0	16845
982.0	19284
983.0	21824
984.0	24464
985.0	27204

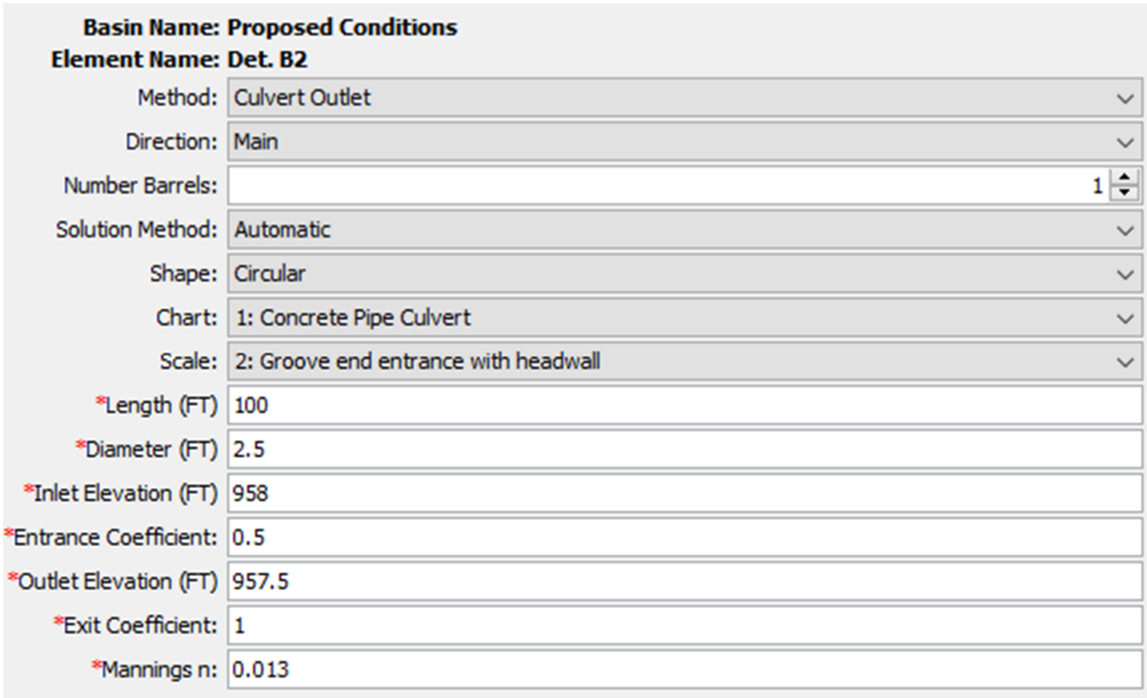
C1

Elevation (FT)	Area (FT2)
966.0	2824
967.0	4106
968.0	5600
969.0	7307
970.0	9225
971.0	11355
972.0	13696
973.0	16237
974.0	18971
975.0	21900
976.0	25022
977.0	28336
978.0	31776

Detention Basin Outlet Information

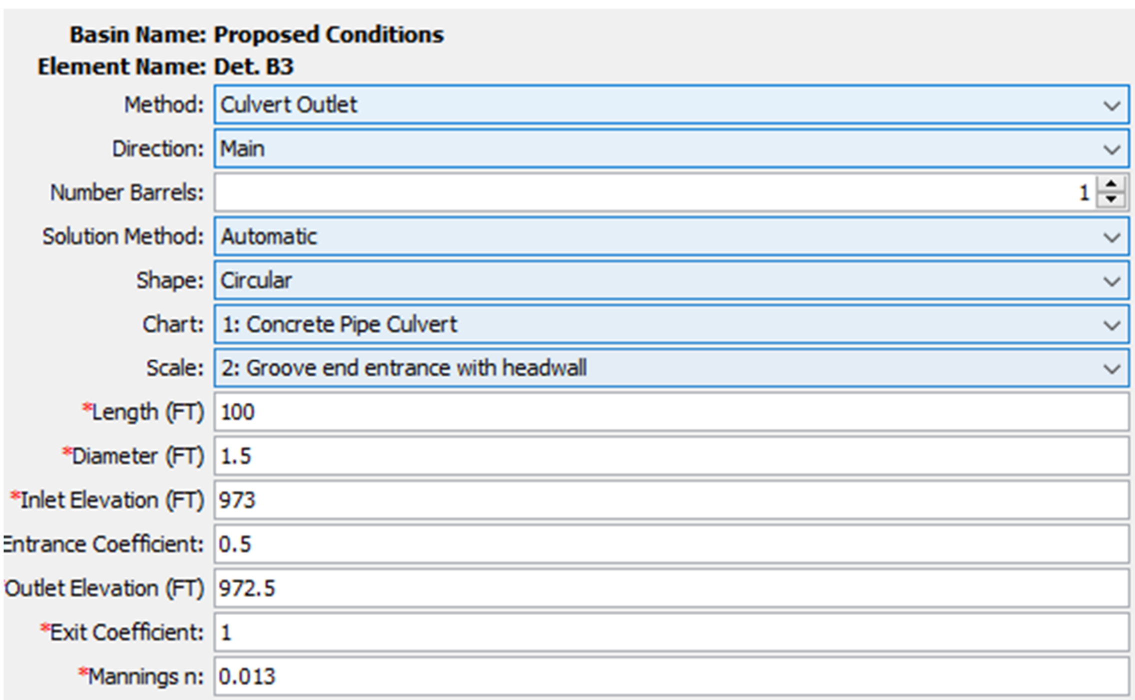
Basin Name:	**Proposed Conditions**
Element Name:	**Det. A1**
Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	1: Square edge entrance with headwall
*Length (FT)	100
*Diameter (FT)	1.25
*Inlet Elevation (FT)	948
*Entrance Coefficient:	0.5
*Outlet Elevation (FT)	947.5
*Exit Coefficient:	1
*Mannings n:	0.013

Basin Name:	**Proposed Conditions**
Element Name:	**Det. B1**
Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	1: Square edge entrance with headwall
*Length (FT)	100
*Diameter (FT)	3
*Inlet Elevation (FT)	954
*Entrance Coefficient:	0.5
*Outlet Elevation (FT)	953.5
*Exit Coefficient:	1
*Mannings n:	0.013



Basin Name: Proposed Conditions
Element Name: Det. B2

Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	2: Groove end entrance with headwall
*Length (FT)	100
*Diameter (FT)	2.5
*Inlet Elevation (FT)	958
*Entrance Coefficient:	0.5
*Outlet Elevation (FT)	957.5
*Exit Coefficient:	1
*Mannings n:	0.013



Basin Name: Proposed Conditions
Element Name: Det. B3

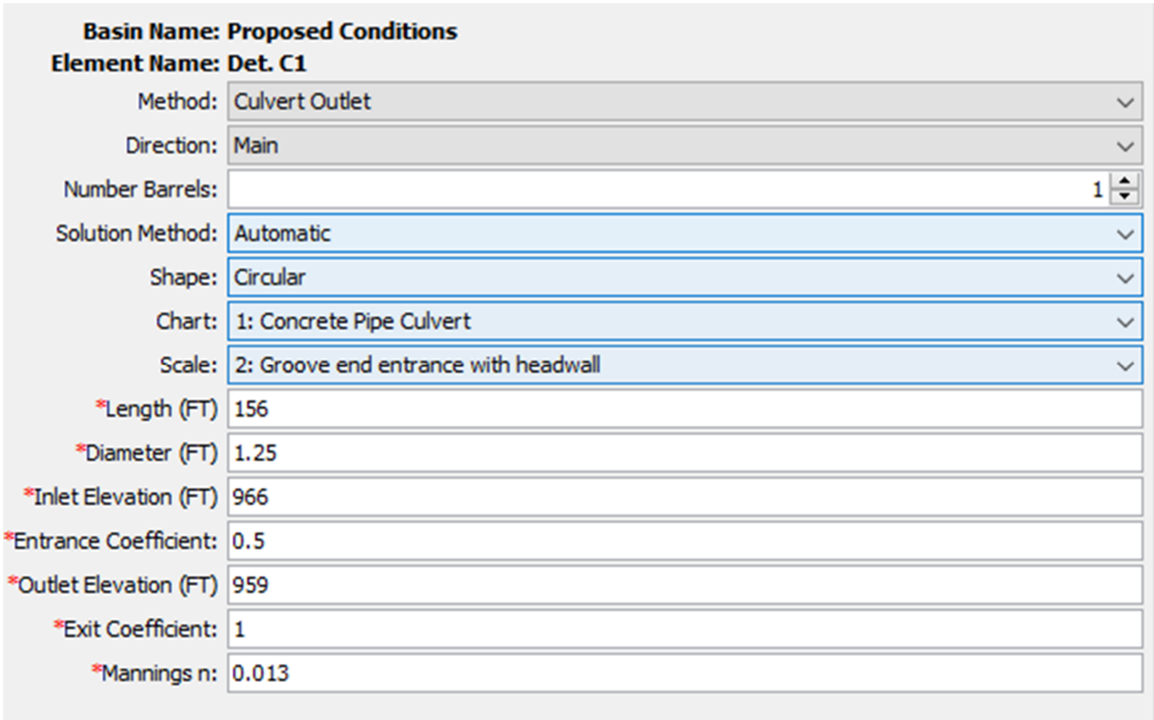
Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	2: Groove end entrance with headwall
*Length (FT)	100
*Diameter (FT)	1.5
*Inlet Elevation (FT)	973
Entrance Coefficient:	0.5
Outlet Elevation (FT)	972.5
*Exit Coefficient:	1
*Mannings n:	0.013

Basin Name: Proposed Conditions
Element Name: Det. B4

Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	2: Groove end entrance with headwall
*Length (FT)	156
*Diameter (FT)	1.25
*Inlet Elevation (FT)	947
*Entrance Coefficient:	0.5
*Outlet Elevation (FT)	941.3
*Exit Coefficient:	1
*Mannings n:	0.013

Basin Name: Proposed Conditions
Element Name: Det. B5

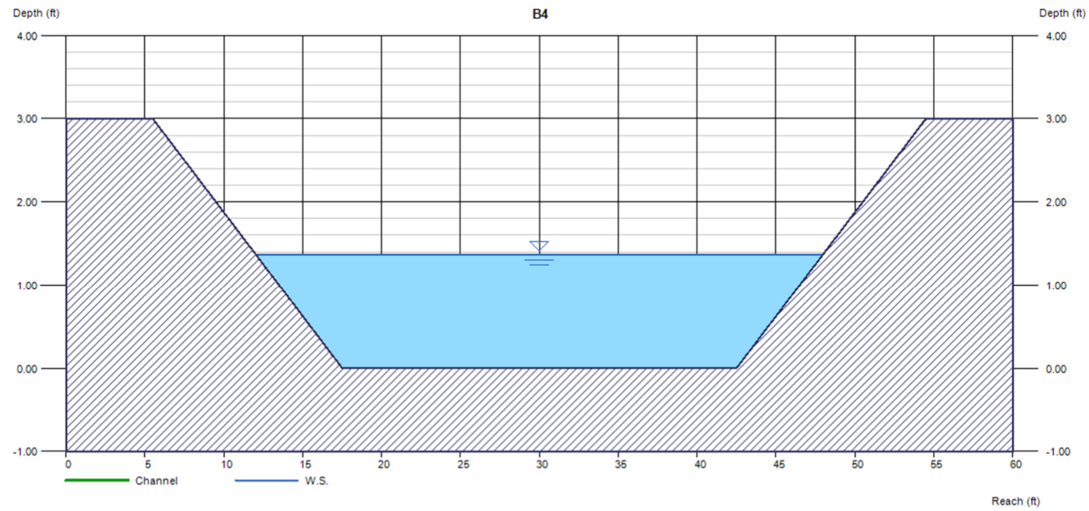
Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	2: Groove end entrance with headwall
*Length (FT)	173
*Diameter (FT)	1.25
*Inlet Elevation (FT)	979
*Entrance Coefficient:	0.5
*Outlet Elevation (FT)	975.7
*Exit Coefficient:	1
*Mannings n:	0.013



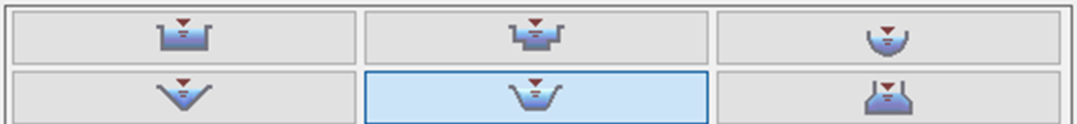
Basin Name: Proposed Conditions
Element Name: Det. C1

Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	2: Groove end entrance with headwall
*Length (FT)	156
*Diameter (FT)	1.25
*Inlet Elevation (FT)	966
*Entrance Coefficient:	0.5
*Outlet Elevation (FT)	959
*Exit Coefficient:	1
*Mannings n:	0.013

Detention Basin Emergency Spillway Information



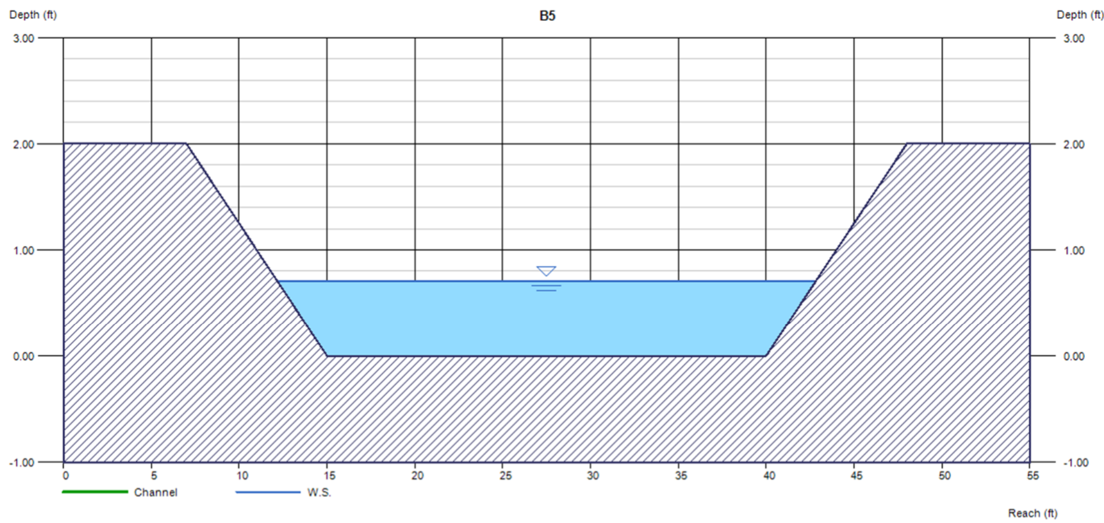
Depth	Q	Area	Veloc	TopWidth	Energy
(ft)	(cfs)	(sqft)	(ft/s)	(ft)	(ft)
1.36	130.00	41.40	3.14	35.88	1.51



Section	Item	Input
Weir	Weir Type =	Trapezoidal
	Crest =	Sharp
	Bottom Len (ft) =	25.00
	Total Depth (ft) =	3.00
	Side Slope, z:1 =	4.00
Calcs	Weir Coeff =	2.80
	Compute by =	Known Q
	Q (cfs) =	130.00

Clear

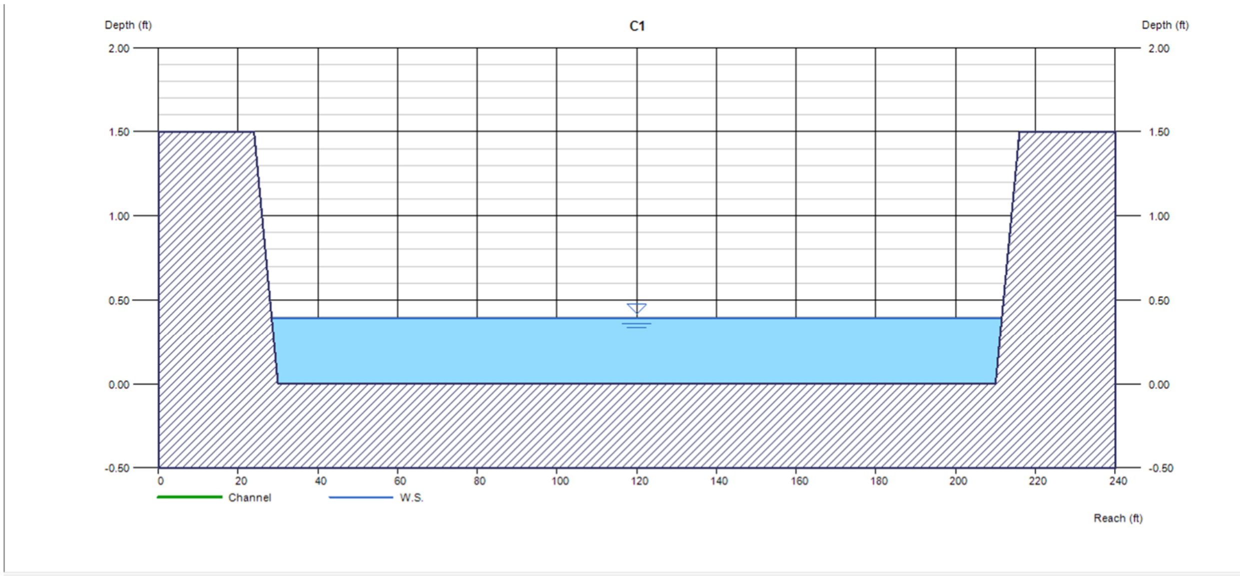
Run



Depth	Q	Area	Veloc	TopWidth	Energy
(ft)	(cfs)	(sqft)	(ft/s)	(ft)	(ft)
0.71	45.00	19.77	2.28	30.68	0.79

Section	Item	Input
Weir	Weir Type =	Trapezoidal
	Crest =	Sharp
	Bottom Len (ft) =	25.00
	Total Depth (ft) =	2.00
	Side Slope, z:1 =	4.00
Calcs	Weir Coeff =	2.80
	Compute by =	Known Q
	Q (cfs) =	45.00

Clear Run



Depth	Q	Area	Veloc	TopWidth	Energy
(ft)	(cfs)	(sqft)	(ft/s)	(ft)	(ft)
0.39	119.00	70.81	1.68	183.12	0.43

Section	Item	Input
Weir	Weir Type =	Trapezoidal
	Crest =	Sharp
	Bottom Len (ft) =	180.00
	Total Depth (ft) =	1.50
	Side Slope, z:1 =	4.00
Calcs	Weir Coeff =	2.80
	Compute by =	Known Q
	Q (cfs) =	119.00

Clear

Run

APPENDIX D

Level of Service and Stormwater BMP Calculations

WORKSHEET 1: REQUIRED LEVEL OF SERVICE-UNDEVELOPED SITE

Project:	Scannell Development - NW Corner of Tudor Rd & Main St	By:	JDA
Location:	Lee's Summit, MO	Checked:	BPS
		Date:	10/13/2021

1. Runoff Curve Number

A. Predevelopment CN

Cover Description	Soil HSG	CN	Area (ac)	Product of CN x Area
Impervious Area (Streets)	C/D	98	2.3	225.4
Pasture-Continuous (Fair)	C	79	20.0	1580
Pasture-Continuous (Fair)	D	84	32.9	2763.6
Woods-Grass (Fair)	C	76	2.1	159.6
Woods-Grass (Fair)	D	82	23.4	1918.8
Open Space - Turf (Good)	D	80	1.9	152
		Totals:	82.6	6799.4

Area-Weighted CN=total product/total area= 82 (Round to integer)

B. Postdevelopment CN

Cover Description	Soil HSG[1]	CN	Area (ac)	Product of CN x Area	(CHECK)
Impervious Areas (Buildings, Parking Lots, Roadways, etc.)	D	98	35.9	3518.2	
Open Space - Turf (Good)	D	80	37.1	2968.0	
Native Vegetation - Brush (Good)	D	73	9.6	700.8	
		Totals:	82.6	7187.0	

[1] Postdevelopment CN is one HSG higher for all cover types except perserved vegetation, absent documentation showing how postdevelopment soil structure will be preserved.

Area-Weighted CN=total product/total area= 87 (Round to integer)

C. Level of Service (LS) Calculation

Predevelopment CN:	82
Postdevelopment CN:	87
Difference:	5
LS Required (see scale at right):	6

Change in CN	LS
17+	8
7 to 16	7
4 to 6	6
1 to 3	5
0	4
-7 to -1	3
-8 to -17	2
-18 to -21	1
-22 -	0

Source:

U.S. Department of Agriculture, Natural Resource Conservation Service. Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55). 1986.

Mid American Regional Council. Stormwater Best Management Practices. 2012

WORKSHEET 2: DEVELOP MITIGATION PACKAGE(S) THAT MEET THE REQUIRED LS

		Date:	**10/14/2021**
Project:	**Scannell Development - NW Corner of Tudor Rd & Main St**	By:	**JDA**
Location:	**Lee's Summit, MO**	Checked:	**BPS**

1. Required LS (from Table 1 or 1A or Worksheet 1 or 1A, as appropriate): **6**

Note: Various BMPs may alter CN of proposed development and LS; recalculate both if applicable.

2. Proposed BMP Option Package No.: **1**

Cover/BMP Description	Treatment Area (acres)	VR from Table 4.4 or Table 4.5[1]	Product of VR x Area
Snout System to Extended Dry Detention[3]	58.7	7.00	410.90
Establish/Preserve Native Vegetation	9.6	9.25	88.80
Untreated	14.3	0.00	
Total[2]:	82.60	**Total:**	499.70
		Weighted VR:	**6.05**

[1] VR calculated for final BMP only in treatment train

[2] Total treatment area cannot exceed 100 percent of the actual site area.

[3] Treatment Train

Meets required LS (Yes/No)? **Yes** (If No, or if additional options are being tested, proceed below.)

2. Proposed BMP Option Package No.: **2**

Cover/BMP Description	Treatment Area (acres)	VR from Table 5 or Table 6[1]	Product of VR x Area
Untreated	0.00	0.00	
Total[2]:	0.00	**Total:**	0.00
		Weighted VR:	

[1] VR calculated for final BMP only in treatment train

[2] Total treatment area cannot exceed 100 percent of the actual site area.

Design Procedure Form: Extended Dry Detention Basin (EDDB)
Main Worksheet - B4

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

I. Basin Water Quality Volume

Step 1: Tributary area to EDDB, A_T (ac.) — A_T (ac) = 13.21

Step 2: Calculate WQ_V using methodology in Section 6 — WQ_V (ac-ft) = 0.71

Step 3: Add 20 percent to account for silt and sediment depositation in the basin — V_{DESIGN} (ac-ft) = **0.86**

IIa. Water Quality Outlet Type

Step 1: Set water quality outlet type: — Outlet Type = 1

Type 1 = Single Orifice
Type 2 = Perforated Riser or Plate
Type 3 = V-Notch Weir

Step 2: Proceed to part IIb, IIc, or IId based on water quality outlet type selected

IIb. Water Quality Pool Outlet, Single Orifice

Step 1: Depth of water quality volume at outlet, Z_{WQ} (ft) — Z_{WQ} (ft) = 3.25

Step 2: Average head of water quality volume over invert of orifice, H_{WQ} (ft) — H_{WQ} (ft) = 1.63

$H_{WQ} = 0.5 * Z_{WQ}$

Step 3: Average water quality outflow rate, Q_{WQ} (cfs) — Q_{WQ} (cfs) = 0.22

$Q_{WQ} = (WQ_V * 43{,}560) / (40*3{,}600)$

Step 4: Set value of orifice discharge coefficient, C_O — C_O = 0.66

C_O = 0.66 when thickness of riser/weir plate is = or < orifice diameter
C_O = 0.80 when thickness of riser/weir plate is > orifice diameter

Step 5: Water quality outlet orifice diameter (minimum of 1/2 inch), D_O (in) — D_O (in) = 2.42

$D_O = 12 * 2 (Q_{WQ} / (C_O * p * (2 * g * H_{WQ})^{0.5}))^{0.5}$

(if orifice diameter < 4 inches use outlet type 2 or 3)

Step 6: To size outlet orifice for EDDB with an irregular stage-volume relationship use the Single Orifice Worksheet

Design Procedure Form: Extended Dry Detention Basin (EDDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

IIc. Water Quality Outlet, Peforated Riser (Continued)

Step 1:	Depth of water quality volume at outlet, Z_{WQ} (ft)	Z_{WQ} (ft) =	3.25
Step 2:	Recommended maximum outlet area per row, A_O (in^2) $A_O = WQ_V / (0.013 * Z_{WQ}^2 + 0.22 * Z_{WQ} - 0.10)$	A_O (in^2) =	0.95
Step 3:	Circular perforation diameter per row assuming a single column, D_I (in)	D_I (in) =	1.10
Step 4:	Numbers of columns, n_c	n_c =	1
Step 5:	Design circular perforation diameter (from 1 to 2 inches), D_{Perf} (in)	D_{Perf} (in) =	1.10
Step 6:	Horizontal peforation column spacing when $n_c > 1$, center to center, S_c If D_{Perf} is not > or = 1, $S_c = 4$	S_c =	NA
Step 7:	Number of rows, 4" vertical spacing between perforations, center to center,	n_r =	10

IIc. Water Quality Outlet, V-Notch Weir

Step 1:	Depth of water quality volume above permanent pool, Z_{WQ} (ft)	Z_{WQ} (ft) =	
Step 2:	Average head of water quality pool volume over invert of v-notch H_{WQ} (ft) $H_{WQ} = 0.5 * Z_{WQ}$	H_{WQ} (ft) =	
Step 3:	Average water quality pool outflow rate, Q_{WQ} (cfs) $Q_{WQ} = (WQ_V * 43{,}560) / (40*3{,}600)$	Q_{WQ} (cfs) =	
Step 4	V-notch weir coefficient, C_v	C_v =	
Step 5:	V-notch weir angle, q (deg) $\theta = 2 *(180/\pi)*\arctan(Q_{WQ} / (C_v * H_{WQ}^{5.2}))$ V-notch angle should be at least 20 degrees. Set to 20 degrees if calculated angle is smaller.	q (deg) =	
Step 6:	V-notch weir top width, W_v (ft) $W_v = 2 * Z_{WQ} * \mathrm{TAN}(\theta/2)$	W_v (ft) =	
Step 7:	To calculate v-notch angle for EDW with an irregular stage-volume relationship, use th V-notch Weir Worksheet		

Design Procedure Form: Extended Dry Detention Basin (EDDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

III. Flood Control

Refer to APWA Specifications Section 5608

IV. Trash Racks

Step 1: Total outlet area, A_{ot} (in^2) — A_{ot} (in^2) = 7.85

Step 2: Required trash rack open area, A_t (in^2) — A_t (in^2) = 267.11

$A_t = A_{ot} * 77 * e^{(-0.124 *D)}$ for single orifice outlet

$A_t = (A_{ot} / 2) * 77 * e^{(-0.124*D)}$ for orifice plate or perforated riser outlet

$At = 4 * A_{ot}$ for v-notch weir outlet

V. Basin Shape

Step 1: Length to width ratio should be at least 3:1 (L:W) wherever practicable — (L:W) =

Step 2: Low flow channel side lining — Concrete: / Soil/Riprap: / No low flow channel:

Step 3: Top stage floor drainage slope (toward low flow channel), S_{TS} (%) — S_{TS} (%) =

Top stage depth, D_{TS} (ft) — D_{TS} (ft) =

Step 4: Bottom stage volume, V_{BS} (ac-ft) — V_{BS} (% of WQ_V) = / V_{BS} (ac-ft) =

VI. Forebay (Optional)

Step 1: Volume should be greater than 10% of WQ_V — Min Vol_{FB} (ac-ft) =

Step 2: Forebay depth, Z_{FB} (ft) — Z_{FB} (ft) =

Step 3: Forebay surface area, A_{FB} (ac) — A_{FB} (ac) =

Step 2: Paved/hard bottom and sides?

Design Procedure Form: Extended Dry Detention Basin (EDDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

VII. Basin side slopes

Basin side slopes should be at least 4:1 (H:V) — Side Slope (H:V) = ______

VIII. Dam Embankment side slopes

Dam Embankment side slops should be at least 3:1 (H:V) — Dam Embankment (H:V) = ______

IX. Vegetation

Check the method of vegetation planted in the EWDB or describe "Other"

☐ Native Grass

☐ Irrigated Turf Grass

☐ Other: ______

X. Inlet protection

Indicate method of inlet protection/energy dissipation at EDDB inlet ______

XI. Access

Indicate that access has been provided for maintenance vehicles ______

Design Procedure Form: Extended Dry Detention Basin (EDDB)
Main Worksheet - B5

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

I. Basin Water Quality Volume

Step 1:	Tributary area to EDDB, A_T (ac.)	A_T (ac) =	3.90
Step 2:	Calculate WQ_V using methodology in Section 6	WQ_V (ac-ft) =	0.24
Step 3:	Add 20 percent to account for silt and sediment depositation in the basin	V_{DESIGN} (ac-ft) =	**0.29**

IIa. Water Quality Outlet Type

Step 1: Set water quality outlet type: Outlet Type = 1

- Type 1 = Single Orifice
- Type 2 = Perforated Riser or Plate
- Type 3 = V-Notch Weir

Step 2: Proceed to part IIb, IIc, or IId based on water quality outlet type selected

IIb. Water Quality Pool Outlet, Single Orifice

Step 1: Depth of water quality volume at outlet, Z_{WQ} (ft) — Z_{WQ} (ft) = 0.94

Step 2: Average head of water quality volume over invert of orifice, H_{WQ} (ft) — H_{WQ} (ft) = 0.47

$H_{WQ} = 0.5 * Z_{WQ}$

Step 3: Average water quality outflow rate, Q_{WQ} (cfs) — Q_{WQ} (cfs) = 0.07

$Q_{WQ} = (WQ_V * 43{,}560) / (40*3{,}600)$

Step 4: Set value of orifice discharge coefficient, C_O — C_O = 0.66

$C_O = 0.66$ when thickness of riser/weir plate is = or < orifice diameter

$C_O = 0.80$ when thickness of riser/weir plate is > orifice diameter

Step 5: Water quality outlet orifice diameter (minimum of 1/2 inch), D_O (in) — D_O (in) = 1.91

$D_O = 12 * 2 (Q_{WQ} / (C_O * p * (2 * g * H_{WQ})^{0.5}))^{0.5}$

(if orifice diameter < 4 inches use outlet type 2 or 3)

Step 6: To size outlet orifice for EDDB with an irregular stage-volume relationship use the Single Orifice Worksheet

Design Procedure Form: Extended Wet Detention Basin (EWDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

IIc. Water Quality Outlet, Peforated Riser (Continued)

Step 1:	Depth of water quality volume at outlet, Z_{WQ} (ft)	Z_{WQ} (ft) =	0.94
Step 2:	Recommended maximum outlet area per row, A_O (in^2) $A_O = WQ_V / (0.013 * Z_{WQ}^2 + 0.22 *Z_{WQ} - 0.10)$	A_O (in^2) =	2.02
Step 3:	Circular perforation diameter per row assuming a single column, D_I (in)	D_I (in) =	1.60
Step 4:	Numbers of columns, n_c	n_c =	1
Step 5:	Design circular perforation diameter (from 1 to 2 inches), D_{Perf} (in)	D_{Perf} (in) =	1.60
Step 6:	Horizontal peforation column spacing when $n_c > 1$, center to center, S_c If D_{Perf} is not > or = 1, $S_c = 4$	S_c =	NA
Step 7:	Number of rows, 4" vertical spacing between perforations, center to center,	n_r =	3

IIc. Water Quality Outlet, V-Notch Weir

Step 1:	Depth of water quality volume above permanent pool, Z_{WQ} (ft)	Z_{WQ} (ft) =	
Step 2:	Average head of water quality pool volume over invert of v-notch H_{WQ} (ft) $H_{WQ} = 0.5 * Z_{WQ}$	H_{WQ} (ft) =	
Step 3:	Average water quality pool outflow rate, Q_{WQ} (cfs) $Q_{WQ} = (WQ_V * 43{,}560) / (40*3{,}600)$	Q_{WQ} (cfs) =	
Step 4	V-notch weir coefficient, C_v	C_v =	
Step 5:	V-notch weir angle, q (deg) $\theta = 2 *(180/ \pi)*\arctan(Q_{WQ} / (C_v * H_{WQ}^{5.2}))$ V-notch angle should be at least 20 degrees. Set to 20 degrees if calculated angle is smaller.	q (deg) =	
Step 6:	V-notch weir top width, W_v (ft) $W_v = 2* Z_{WQ} * TAN(\theta/2)$	W_v (ft) =	

Step 7: To calculate v-notch angle for EDW with an irregular stage-volume relationship, use th V-notch Weir Worksheet

Design Procedure Form: Extended Wet Detention Basin (EWDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

III. Flood Control

Refer to APWA Specifications Section 5608

IV. Trash Racks

Step 1: Total outlet area, A_{ot} (in^2) — A_{ot} (in^2) = 2.36

Step 2: Required trash rack open area, A_t (in^2) — A_t (in^2) = 80.13

$A_t = A_{ot} * 77 * e^{(-0.124 *D)}$ for single orifice outlet

$A_t = (A_{ot} / 2) * 77 * e^{(-0.124*D)}$ for orifice plate or perforated riser outlet

At = 4 * A_{ot} for v-notch weir outlet

V. Basin Shape

Step 1: Length to width ratio should be at least 3:1 (L:W) wherever practicable — (L:W) =

Step 2: Low flow channel side lining — Concrete: / Soil/Riprap: / No low flow channel:

Step 3: Top stage floor drainage slope (toward low flow channel), S_{TS} (%) — S_{TS} (%) =

Top stage depth, D_{TS} (ft) — D_{TS} (ft) =

Step 4: Bottom stage volume, V_{BS} (ac-ft) — V_{BS} (% of WQ_V) = / V_{BS} (ac-ft) =

VI. Forebay (Optional)

Step 1: Volume should be greater than 10% of WQ_V — Min Vol_{FB} (ac-ft) =

Step 2: Forebay depth, Z_{FB} (ft) — Z_{FB} (ft) =

Step 3: Forebay surface area, A_{FB} (ac) — A_{FB} (ac) =

Step 2: Paved/hard bottom and sides?

Design Procedure Form: Extended Wet Detention Basin (EWDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

VII. Basin side slopes

Basin side slopes should be at least 4:1 (H:V) Side Slope (H:V) = ________

VIII. Dam Embankment side slopes

Dam Embankment side slops should be at least 3:1 (H:V) Dam Embankment (H:V) = ________

IX. Vegetation

Check the method of vegetation planted in the EWDB or describe "Other"

☐ Native Grass

☐ Irrigated Turf Grass

☐ Other: ________

X. Inlet protection

Indicate method of inlet protection/energy dissipation at EDDB inlet ________

XI. Access

Indicate that access has been provided for maintenance vehicles ________

Design Procedure Form: Extended Dry Detention Basin (EDDB)
Main Worksheet - C1

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

I. Basin Water Quality Volume

Step 1: Tributary area to EDDB, A_T (ac.) — A_T (ac) = 13.50

Step 2: Calculate WQ_V using methodology in Section 6 — WQ_V (ac-ft) = 0.77

Step 3: Add 20 percent to account for silt and sediment deposition in the basin — V_{DESIGN} (ac-ft) = **0.92**

IIa. Water Quality Outlet Type

Step 1: Set water quality outlet type: — Outlet Type = 1

Type 1 = Single Orifice
Type 2 = Perforated Riser or Plate
Type 3 = V-Notch Weir

Step 2: Proceed to part IIb, IIc, or IId based on water quality outlet type selected

---

IIb. Water Quality Pool Outlet, Single Orifice

Step 1: Depth of water quality volume at outlet, Z_{WQ} (ft) — Z_{WQ} (ft) = 5.54

Step 2: Average head of water quality volume over invert of orifice, H_{WQ} (ft) — H_{WQ} (ft) = 2.77

$H_{WQ} = 0.5 * Z_{WQ}$

Step 3: Average water quality outflow rate, Q_{WQ} (cfs) — Q_{WQ} (cfs) = 0.23

$Q_{WQ} = (WQ_V * 43{,}560) / (40*3{,}600)$

Step 4: Set value of orifice discharge coefficient, C_O — C_O = 0.66

C_O = 0.66 when thickness of riser/weir plate is = or < orifice diameter
C_O = 0.80 when thickness of riser/weir plate is > orifice diameter

Step 5: Water quality outlet orifice diameter (minimum of 1/2 inch), D_O (in) — D_O (in) = 2.20

$D_O = 12 * 2 (Q_{WQ} / (C_O * p * (2 * g * H_{WQ})^{0.5}))^{0.5}$
(if orifice diameter < 4 inches use outlet type 2 or 3)

Step 6: To size outlet orifice for EDDB with an irregular stage-volume relationship use the Single Orifice Worksheet

Design Procedure Form: Extended Wet Detention Basin (EWDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

IIc. Water Quality Outlet, Peforated Riser (Continued)

Step 1:	Depth of water quality volume at outlet, Z_{WQ} (ft)	Z_{WQ} (ft) =	5.54
Step 2:	Recommended maximum outlet area per row, A_O (in^2) $A_O = WQ_V / (0.013 * Z_{WQ}^2 + 0.22 * Z_{WQ} - 0.10)$	A_O (in^2) =	0.51
Step 3:	Circular perforation diameter per row assuming a single column, D_l (in)	D_l (in) =	0.80
Step 4:	Numbers of columns, n_c	n_c =	1
Step 5:	Design circular perforation diameter (from 1 to 2 inches), D_{Perf} (in)	D_{Perf} (in) =	0.80
Step 6:	Horizontal peforation column spacing when $n_c > 1$, center to center, S_c If D_{Perf} is not > or = 1, $S_c = 4$	S_c =	NA
Step 7:	Number of rows, 4" vertical spacing between perforations, center to center,	n_r =	17

IIc. Water Quality Outlet, V-Notch Weir

Step 1:	Depth of water quality volume above permanent pool, Z_{WQ} (ft)	Z_{WQ} (ft) =	
Step 2:	Average head of water quality pool volume over invert of v-notch H_{WQ} (ft) $H_{WQ} = 0.5 * Z_{WQ}$	H_{WQ} (ft) =	
Step 3:	Average water quality pool outflow rate, Q_{WQ} (cfs) $Q_{WQ} = (WQ_V * 43{,}560) / (40*3{,}600)$	Q_{WQ} (cfs) =	
Step 4	V-notch weir coefficient, C_v	C_v =	
Step 5:	V-notch weir angle, q (deg) $\theta = 2 * (180/\pi) * \arctan(Q_{WQ} / (C_v * H_{WQ}^{5.2}))$ V-notch angle should be at least 20 degrees. Set to 20 degrees if calculated angle is smaller.	q (deg) =	
Step 6:	V-notch weir top width, W_v (ft) $W_v = 2 * Z_{WQ} * \tan(\theta/2)$	W_v (ft) =	
Step 7:	To calculate v-notch angle for EDW with an irregular stage-volume relationship, use th V-notch Weir Worksheet		

Design Procedure Form: Extended Wet Detention Basin (EWDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

III. Flood Control

Refer to APWA Specifications Section 5608

IV. Trash Racks

Step 1: Total outlet area, A_{ot} (in^2) — A_{ot} (in^2) = 13.35

Step 2: Required trash rack open area, A_t (in^2) — A_t (in^2) = 454.10

$A_t = A_{ot} * 77 * e^{(-0.124 *D)}$ for single orifice outlet

$A_t = (A_{ot} / 2) * 77 * e^{(-0.124*D)}$ for orifice plate or perforated riser outlet

At = 4 * A_{ot} for v-notch weir outlet

V. Basin Shape

Step 1: Length to width ratio should be at least 3:1 (L:W) wherever practicable — (L:W) =

Step 2: Low flow channel side lining — Concrete:
Soil/Riprap:
No low flow channel:

Step 3: Top stage floor drainage slope (toward low flow channel), S_{TS} (%) — S_{TS} (%) =
Top stage depth, D_{TS} (ft) — D_{TS} (ft) =

Step 4: Bottom stage volume, V_{BS} (ac-ft) — V_{BS} (% of WQ_V) =
V_{BS} (ac-ft) =

VI. Forebay (Optional)

Step 1: Volume should be greater than 10% of WQ_V — Min Vol_{FB} (ac-ft) =

Step 2: Forebay depth, Z_{FB} (ft) — Z_{FB} (ft) =

Step 3: Forebay surface area, A_{FB} (ac) — A_{FB} (ac) =

Step 2: Paved/hard bottom and sides?

Design Procedure Form: Extended Wet Detention Basin (EWDB)
Main Worksheet

Project:	Scannell Development	Date:	10/13/2021
Location:	Lee's Summit, MO	Company:	Olsson
Designer:	JDA	Checked:	BMJ

VII. Basin side slopes

Basin side slopes should be at least 4:1 (H:V) — Side Slope (H:V) = ______

VIII. Dam Embankment side slopes

Dam Embankment side slops should be at least 3:1 (H:V) — Dam Embankment (H:V) = ______

IX. Vegetation

Check the method of vegetation planted in the EWDB or describe "Other"

- ___ Native Grass
- ___ Irrigated Turf Grass
- ___ Other: ______

X. Inlet protection

Indicate method of inlet protection/energy dissipation at EDDB inlet ______

XI. Access

Indicate that access has been provided for maintenance vehicles ______

APPENDIX E

Waiver Requests

LEE'S SUMMIT
MISSOURI

DESIGN AND CONSTRUCTION MANUAL DESIGN MODIFICATION REQUEST

PROJECT NAME: Scannell Development - Phase I

PREMISE ADDRESS: NW Corner of Tudor Road & Main Street

PERMIT NUMBER: ______________________________

OWNER'S NAME: Scannell Properties, LLC

TO: The City Engineer

In accordance with the Lee's Summit Design and Construction Manual (DCM) Section 1002.A, I wish to apply for a modification to one or more specification (s). The following articulates my request for your review and action. (NOTE: Cite specific code sections and engineering justification and drawings.)
A waiver is requested for detention of the 2-year event at the site (outlined in Section 5608 of KC-APWA 5600). The allowable release rate at the point of interest for the 2-year event cannot be met with detention. If the proposed release rates for all detention basins were reduced to 0, the proposed peak flow rate at the point of interest would still be greater than the allowable release rate.

SUBMITTED BY:
NAME: Jacob Asgian ( ) OWNER (x) OWNER'S AGENT
ADDRESS: 7301 West 133rd St, Suite 200 Tel.# (913) 381-1170
CITY, STATE, ZIP: Overland Park, KS 66213
Email: jasgian@olsson.com SIGNATURE: ______________________

FORWARDING MANAGER: ____________________ RECOMMENDATION ( ) APPROVAL ( ) DENIAL

SIGNATURE: ______________________ DATE: ______________________

GEORGE BINGER III, P.E. – CITY ENGINEER: ( ) APPROVED ( ) DENIED

SIGNATURE: ______________________ DATE: ______________________

COMMENTS ____

A COPY MUST BE ATTACHED TO THE APPROVED PLANS

LEE'S SUMMIT
MISSOURI

DESIGN AND CONSTRUCTION MANUAL DESIGN MODIFICATION REQUEST

PROJECT NAME: Scannell Development - Phase I

PREMISE ADDRESS: NW Corner of Tudor Road & Main Street

PERMIT NUMBER: ____________________

OWNER'S NAME: Scannell Properties, LLC

TO: The City Engineer

In accordance with the Lee's Summit Design and Construction Manual (DCM) Section 1002.A, I wish to apply for a modification to one or more specification (s). The following articulates my request for your review and action. (NOTE: Cite specific code sections and engineering justification and drawings.)
A waiver is requested for stream setback requirements for Little Cedar Creek and the Unnamed Tributary to Little Cedar Creek, as outlined in Section 4.5 of the final stormwater drainage study. The proposed roadway realignment encroaches on the south stream buffer for Little Cedar Creek. Additional buffer has been provided on the north side to account for this loss. The proposed roadway on the west side of the building encroaches on the Unnamed Tributary to Little Cedar Creek. The stream in this area has been previously impacted and straightened by the railroad to the west and has little potential for significant migration to the east (towards the road/building).

SUBMITTED BY:
NAME: Jacob Asgian ( ) OWNER (x) OWNER'S AGENT
ADDRESS: 7301 West 133rd St, Suite 200 Tel.# (913) 381-1170
CITY, STATE, ZIP: Overland Park, KS 66213
Email: jasgian@olsson.com SIGNATURE: ____________________

FORWARDING MANAGER: ____________________ RECOMMENDATION ( ) APPROVAL ( ) DENIAL

SIGNATURE: ____________________ DATE: ____________________

GEORGE BINGER III, P.E. – CITY ENGINEER: ( ) APPROVED ( ) DENIED

SIGNATURE: ____________________ DATE: ____________________

COMMENTS____

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A COPY MUST BE ATTACHED TO THE APPROVED PLANS