Micro Storm Water Drainage Study

Sequoia, Lee's Summit

Southwest Corner of NW Olive St and NW Orchard Dr
City of Lee's Summit, Jackson County, Missouri

Created On:

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Prepared by:

12·11·19

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GENERAL INFORMATION

A. Project Location

The proposed Sequoia development is in the City of Lee's Summit, Jackson County, MO. The project is located on the southwest corner of NW Olive St and NW Orchard Dr and is 3.78 acres in size. The proposed location is currently 4 lots zoned for single family residential or vacant residential land. The entire site is located within the Cedar Creek Watershed. Table 1 lists the parcel information for each of the 4 proposed lots and all adjacent properties.

Table 1: Existing Lot Information

Parcel Description	Address	Parcel ID	Land Use Type
Proposed Parcel Information			
NE Corner of Proposed Lot	502 NW Olive St	61-310-05-12-00-0-00-000	1110 – Single Family Residence
NW Corner of Proposed Lot	500 NW Olive St	61-320-01-06-00-0-00-000	1101 – Vacant Residential Land
SE Corner of Proposed Lot	408 NW Olive St	61-310-06-01-00-0-00-000	1110 – Single Family Residence
SW Corner of Proposed Lot	No Address Assigned by City Lee's Summit, MO	61-320-07-01-00-0-00-000	1101-Vacant Residential Land
Adjacent Parcel Information			
N of Proposed Lot	221 NW Chipman Rd	61-320-01-02-00-0-00-000	3216 – Wholesale Trade
NE of Proposed Lot	504 NW Olive St	61-310-05-11-00-0-00-000	1110 – Single Family Residence
NE of Proposed Lot	502 NW Olive St	61-310-05-12-00-0-00-000	1110 – Single Family Residence
SE of Proposed Lot	406 NW Olive St	61-310-06-02-00-0-00-000	1110 – Single Family Residence
S of Proposed Lot	404 NW Olive St	61-310-06-03-00-0-00-000	1110 – Single Family Residence

Activities include the construction of a proposed duplex development and associated infrastructure. The proposed site will not impact downstream infrastructure because none exists. See Exhibit A for a site location map.

B. Federal Emergency Management Agency (FEMA) Classification

According to the Flood Insurance Rate Map (FIRM) panel number 29095C0417G, dated January 20, 2017, the property lies within Zone "X" (future base flood) as defined as areas having a one percent annual chance flood based on future conditions hydrology. See Exhibit B for a site location FEMA FIRM map.

C. Soil Classification

Soil classifications published by the United States Department of Agriculture/Natural Resources Conservation Service (USDA/NRCS) website for Jackson County, MO on October 16, 2018 indicate the existing site is made up of three soil types:

10082 Arisburg-Urban Land Complex, 1 to 5 percent slopes
Hydraulic Soil Group (HSG) Type C

10128 Sharpsburg-Urban Land Complex, 2 to 5 percent slopes
Hydraulic Soils Group (HSG) Type D

10181 Udarents -Urban Land - Sampsel, 5 to 9 percent slopes
Hydraulic Soils Group (HSG) Type C

See Exhibit C for a detailed soil report.

D. Drainage Patterns

Two existing sub basins were identified at the project location. ExNW was identified as the northern drainage area with a discharge point at the northwest corner of the sub basin. The second existing sub basin was identified as ExSE with a discharge point at the southeast corner of the sub basin. One offsite drainage area was identified at the project location. ExOffsite was identified at the southwest corner of the proposed lot contributing to the ExNW sub basin. See Exhibit D for an existing drainage map.

METHODOLOGY

This study was prepared in accordance with the provisions of "Section 5600 – Storm Drainage Systems and Facilities" (February 15, 2006) of the Kansas City Metropolitan Chapter of the American Public Works Association as adopted and modified (City of Lee's Summit Section 5600, August 8, 2011) for use in storm facilities design by the City of Lee's Summit, MO. Pre and post development runoff were determined using the curve number method described in SCS (now NRCS) Technical Release No. 55 "Urban Hydrology for Small Watersheds" (2nd Edition, June 1986) as provided for in APWA Sub-section 5602.2. Pre and post un-detained fringe lot drainage area runoff was determined using the Rational Method described in APWA 5602.2.A. Storm water management controls included in the post development TR55 analyses were designed to reduce peak discharges to or below pre-development values as stipulated in Sub-section 5601.5. The analyses were performed using the Type II 24-hour storm distribution for 2-year, 10-year and 100-year storm events. The rainfall depths used in the analyses corresponding to those events are shown in Table 2.

Table 2: Storm Analysis Table

Storm	Percent	Rainfall Depth (in)
2-Year	50%	3.50
10-Year	10%	5.30
100-Year	1%	7.70

EXISTING CONDITIONS ANALYSIS

Existing site drainage patterns are shown in Exhibit D – Existing Drainage Map. Exhibit D shows two on-site and one off-site drainage areas that were analyzed for existing conditions. The total drainage area of the existing site is 3.78 acres and includes 0.02 acres of offsite drainages.

The curve numbers used in the TR55 existing condition analysis are 74.0 (ExNW, >75% grass cover, good) and 83.0 (ExSE, ¼ acre lots, 38% impervious).

The existing drainage map (Exhibit D) identifies each sub basin discharge point and related area shown in Table 3 below. The existing conditions model results have been provided in Exhibit E. The time of concentration determined for each sub basin is shown in Table 4. The sub basin discharge for the three storm events investigated are shown in Table 5 and summarized in Table 6.

Comprehensive control was used in accordance with APWA 5608.4 to determine maximum release rates for each post development sub basin. This allows for a maximum discharge (cfs/acre) for 2-yr, 10-yr, and 100-yr storm events. The single off-site drainage contributor was documented with the existing conditions analysis. The sub basin allowable release rates for the three storm events investigated are shown in Table 7.

Table 3. Existing Discharge Points

Outfall	Direction
ExNW	Flow travels across the lot from east (NW Olive St) to west (Railroad ROW). Runoff that is discharged across the western property line is conveyed to the NW corner parallel to the railroad.
ExSW	Flow travels across the lot from north to south parallel to NW Olive St. Runoff is discharged in the SE corner of the sub basin.
ExOffsite	Flat portion of SW corner along the railroad ROW draining into ExNW. Discharge conveyed to ExNW Discharge Point A.

Table 4. Existing Time of Concentration Calculations

Sub Basin	Overland Flow	Shallow Concentrated Flow	Channel Flow	Tc (Min.)
ExNW	Length=100 ft Slope=2.8% N Value=0.30	Length= 380 ft Slope= 3.0% Short Grass Pasture	Length= n/a Slope= n/a Cross Section Area= n/a Wetted Perimeter= n/a	19.49
ExSE	Length=100 ft Slope=3.0% N Value=0.30	Length=150 ft Slope=3.70% Short Grass Pasture	Length= n/a Slope= n/a Cross Section Area= n/a Wetted Perimeter= n/a	15.72
ExOffsite	Length= 10 ft Slope= 0.1% N Value= 0.30	Length = n/a Slope = n/a	Length= n/a Slope= n/a Cross Section Area= n/a Wetted Perimeter= n/a	8.57

Table 5: Existing Site Hydrology and Flows

Sub Basin	Discharge Point	Outfall	Outfall Type	Area (Ac.)	T_c (min)	CN Value	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ExNW	A	NW	Low Point	2.74	19.49	74.00	3.67	7.95	14.21
ExSE	B	SE	Low Point	1.04	15.72	83.00	2.37	4.34	7.04
ExOffsite	A	NW	Low Point	0.02	8.57	74.00	0.03	0.07	0.13

Table 6: Total Outflow Summary

Sub Basin	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ExNW	3.67	7.95	14.21
ExSE	2.37	4.34	7.04
ExOffsite	0.03	0.07	0.13

Table 7: Allowable Release Rates per Existing Discharge Point

Sub Basin	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ExNW	1.37	5.48	8.22
ExSE	0.52	2.08	3.12

PROPOSED CONDITIONS ANALYSIS

The overall drainage pattern for the proposed condition has been updated to four sub basins with three discharge points. See Exhibit F for a proposed drainage map. The development will not add any area to the existing 3.78 acres, but the area of each sub basin has changed.

The curve number used for the proposed site was 90.0 (1/8 acre lots, 65% impervious). HSG C was assumed for the curve number calculations.

The proposed drainage map (Exhibit F) identifies the sub basin discharge points and related area shown in Table 8 below. The proposed conditions model results have been provided in Exhibit G. The time of concentration assumptions for each sub basin are shown in Table 9. The sub basin discharge for the three storm events investigated are shown in Table 10 and summarized in Table 11. The sub basin allowable release rates for the three storm events investigated are shown in Table 12.

Table 8. Proposed Discharge Points

Outfall	Direction
Northwest (ProNW)	Runoff is conveyed NW across the ProNW sub basin to a discharge point in the NW corner of the proposed lot.
South (ProS)	Runoff is conveyed SW across the ProS sub basin to a discharge point along the W property line.
Southeast (ProSE)	Runoff is conveyed SE across the ProSE sub basin to an existing roadway ditch and discharge point in the SE corner of the proposed lot.
West (ProW)	Runoff is conveyed across the ProW sub basin to a discharge point along the W property line of the proposed lot.

Table 9. Proposed Time of Concentration Calculations

Sub Basin	Overland Flow	Shallow Concentrated Flow	Channel Flow	Tc (Min.)
ProNW	Length= 100 ft Slope= 2.0% N Value= 0.30	Length= 111 ft Slope= 2.5% Short Grass	Length= 192 ft Slope= 2.4% Cross Section Area= 1.91 ft^2 Wetted Perimeter= 7.73 ft	18.61
ProS	Length= 35 ft Slope= 2.0% N Value= 0.30	Length= 169 ft Slope= 2.0% Paved	Length= 42 ft Slope= 2.0% Cross Section Area= 1.33 ft^2 Wetted Perimeter= 3.0 ft	8.09
ProSE	Length= 70 ft Slope= 2.5% N Value= 0.30	Length= 110 ft Slope= 3.5% Short Grass	Length= n/a Slope= n/a Cross Section Area= n/a Wetted Perimeter= n/a	12.61
ProW	Length= 29 ft Slope= 4.0% N Value= 0.30	Length= 105 ft Slope= 7.0% Short Grass	Length= n/a Slope= n/a Cross Section Area= n/a Wetted Perimeter= n/a	5.54

Table 10: Proposed Site Hydrology and Flows

Sub Basin	Discharge Point	Outfall Type	Area (Ac.)	T_c (min)	CN	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ProNW	A	Railroad ROW	1.79	18.61	90.00	4.91	8.16	12.45
ProS	C	Railroad ROW	0.90	8.09	90.00	3.15	5.22	7.96
ProSE	B	Un-Detained Discharge	0.73	12.61	90.00	2.29	3.80	5.79
ProW	C	Un-Detained Discharge	0.36	5.54	90.00	1.34	2.23	3.39

Table 11: Total Outflow Summary

Sub Basin	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ProNW	4.91	8.16	12.45
ProS	3.15	5.22	7.96
ProSE	2.29	3.80	5.79
ProW	1.34	2.23	3.39

Table 12: Comprehensive Control Allowable Release Rates

Sub Basin	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ProNW	0.90	3.85	5.37
ProS	0.45	1.80	2.70

DETENTION ANALYSIS

Detention analysis was completed according to APWA Section 5608: Stormwater Detention and Retention. The proposed detention analysis was completed per APWA 5608.4.C.1.a (pg 92) which allows a maximum peak discharge rate of 0.5 (2-yr), 2.0 (10-yr), and 3.0 (100-yr) cfs/acre for any development under runoff control strategies. Criteria from APWA 5608.4.C.1.b (pg 92) was also applied to ensure 40-hour extended detention of runoff for local 90% mean annual event. (1.37"/24-hour rainfall)

All outflow conditions assume free flow. All downstream pipes of the detention basin will be sized using manning's equation to carry the 100-year flow condition to site development. To mitigate this, we are proposing two detention basins on-site.

The proposed on-site detention consists of two above ground extended dry detention basins (EDDB) which accommodate wet detention for a 40-hour extended period. A 4" outfall pipe was assumed for the water quality outfall in each detention pond based on the minimum allowable cross-sectional area outlet.

The proposed northwest basin (ProNW) will have an invert elevation of 1008.00', a top of dam of 1012.80', and a 100-year HGL of 1011.20'. The total volume of the storage basin at the 100-year HGL is 0.44 acre-feet. Runoff is to be conveyed through 1-Perforated Riser (Invert = 1008.00', 40-hour extended dry detention outfall) and 1-12" HDPE Pipe (invert = 1009.45'). The 46-linear foot 12" HDPE pipe will be built at a 3.15% slope. Runoff from both outfall pipes will daylight on the existing property (Invert = 1007.00') and flow towards railroad right-of-way.

The emergency overflow structure consists of a 103' wide naturally graded trapezoidal weir at an elevation of 1011.7'. A minimum of 0.50' of freeboard is required between the emergency spillway crest and the maximum 100-year. For the 100-year maximum water surface elevation of 1011.20' the total provided freeboard is 0.50'.

The proposed south basin (ProS) will have an invert elevation of 1014.25', a top of dam of 1018.20', and a 100-year HGL of 1016.45'. The total volume of the storage basin at the 100-year HGL is 0.26 acre-feet. Runoff is to be conveyed through 1 – Perforated Riser (Invert = 1014.25', 40-hour extended dry detention outfall) and 1-12" HDPE Pipe (invert = 1015.50'). The 38-linear foot 12" pipe will be built at a 3.95% slope. Runoff from both outfall pipes will daylight on the existing property (Invert = 1014.00') and flow towards railroad right-of-way.

The emergency overflow structure consists of a 36' wide naturally graded trapezoidal weir at an elevation of 1017.10'. A minimum of 0.50' of freeboard is required between the emergency spillway crest and the maximum 100-year WSE. For the 100-year maximum water surface elevation of 1016.45' the total provided freeboard is 0.65'.

Please see Table 13 below for a summary of pipe velocities during 2, 10, and 100-year storms, Table 14 for a detention basin inflow/outflow summary, Table 15 for a detention basin summary, and Table 16 for an APWA 5608 peak discharge requirement summary.

Table 13: Summary of Pipe Velocities

Pipe	V_2 (fps)	V_{10} (fps)	V_{100} (fps)
Proposed NW Detention Basin			
12" HDPE	1.16	3.63	5.50
Proposed S Detention Basin			
12" HDPE	0.11	1.00	3.24

Table 14: Detention Basin Inflow/Outflow Summary

Storm Event	Q_{in} (cfs)	Ponding Elevation (ft)	Max Depth Attained (ft)	Q_{out} (cfs)
Proposed NW Pond				
100- Year	12.39	1011.20	3.20	4.32
10-Year	8.12	1010.49	2.49	2.85
2-Year	4.90	1009.93	1.93	0.91
Proposed S Pond				
100- Year	7.52	1016.45	2.20	2.54
10-Year	4.89	1015.93	1.68	0.78
2-Year	2.96	1015.60	1.35	0.09

Table 15: Summary of Detention Basin Design

Proposed NW Detention Basin	
Drainage Area	1.79 AC
Curve Number	90.00
Basin Flow Line Outfall	1008.00'
Pond Base Elevation	1008.00'
Outlet Structure	1 – 12" HDPE Pipes @ 1009.45' 1 – Perforated Pipe @ 1008.00'
Max 100-year HGL	1011.20
100-Year Emergency Weir Elevation	1011.70'
Top of Dam	1012.80'
Proposed S Detention Basin	
Drainage Area	0.90 AC
Curve Number	90.00
Basin Flow Line Outfall	1014.25'
Pond Base Elevation	1014.25'
Outlet Structure	1 – 12" HDPE Pipe @ 1015.50' 1 – Perforated Pipe @ 1014.25"
Max 100-year HGL	1016.45'
100-Year Emergency Weir Elevation	1017.10'
Top of Dam	1018.20'

Table 16. Summary of APWA 5608 Peak Discharge Requirements

Outfall Desc.	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ProNW Allowable	0.90	3.58	5.37
ProNW Actual	0.91	2.85	4.32
Difference	+0.01	-0.73	-1.05
ProS Allowable	0.45	1.80	2.70
ProS Actual	0.09	0.78	2.54
Difference	-0.36	-1.02	-0.16

APWA Section 5608.4.F.2 requires that the detention basin emergency spillway performance provides a minimum of 1.0 ft of freeboard from the design stage to the top of dam, assuming zero available storage in the basin and zero flow through the primary outlet. (100% clogged condition) FHWA HEC-22, Table 8-1, pg. 8-27 was used to determine a broad-crested weir coefficient of 2.7. Total 100-yr runoff flowrates were used to calculate the maximum energy grade line (EGL) for each pond assuming zero storage in the pond. Table 17 shows a summary of emergency spillway performance for the 100-yr storm event assuming zero flow through the primary outlet. Reference Exhibit H for 100-yr spillway flowrate and EGL performance calculations.

Table 17. Summary of Emergency Spillway Performance (100-Yr Event)

Outfall Desc.	Max Inflow (cfs)	Weirhh Elev (ft)	Length (ft)	Top of Dam Elev (ft)	Max WSE (ft)	Max EGL (ft)	Freeboard (ft)
ProNW	4.99	1011.70	103.00	1012.80	1011.77	1011.78	1.03
ProS	0.35	1017.10	36.00	1018.20	1017.13	1017.13	1.07

UN-DETAINED DRAINAGE AREA ANALYSIS

The proposed southeast sub basin (ProSE) is an un-detained drainage area. The existing discharge point (Discharge points B and C on Exhibits D & F) will remain the same for the ProSE sub basin but the drainage area has decreased. The decreased area will be un-detained and discharge to the existing roadside ditch at discharge point B. Updated curve number and drainage area data for the SE basin shows an overall reduction in runoff conveyed to discharge point B. See Table 18 below for a summary of existing and proposed conditions at discharge point B.

Table 18. Summary of Discharge Point B Conditions

Outfall Desc.	Area (AC)	CN	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ExSE	1.03	83.00	3.24	4.29	6.95
ProSE	0.73	90.00	2.29	3.80	5.79

The proposed west sub basin (ProW) is a 0.36 ac un-detained drainage area. This sub basin is a fringe lot that drains the back half of 4 proposed units towards the railroad right-of-way. A native vegetation swale and native vegetation are located along the western edge of this proposed sub basin. Runoff from the proposed development will be conveyed through this swale or sheet flow through native vegetation. See Exhibit F for the ProW sub basin location.

Existing and proposed conditions analyses were completed using the Rational Method per APWA 5602.2.A. A non-standard land use Rational C value was calculated using APWA 5602.2.C based on a total impervious area of 20% for the proposed conditions. The existing conditions Rational C = 0.30 and the proposed condition Rational C = 0.42. Time of concertation was assumed to be 5 minutes for existing and

proposed conditions to determine a conservative runoff estimate. See Table 19 for a summary of existing and proposed runoff for the ProW sub basin.

Table 19: ProW Existing & Proposed Conditions Summary

Storm Event	Intensity (in/hr)	Existing Flowrate (cfs)	Proposed Flowrate (cfs)	Increase (cfs)
2-Yr	5.41	0.58	0.80	0.22
10-Yr	7.35	0.80	1.08	0.28
100-Yr	10.32	1.39	1.90	0.51

A runoff increase for all storm events has been shown for the ProW sub basin under proposed conditions. Stormwater BMPs in the form of a native vegetation swale and native grass plantings have been proposed to treat the increased runoff. All excess runoff will be discharged into the railroad right-of-way.

WATER QUALITY ANALYSIS

MARC BMP Manual Section 4.0 was used to determine BMP requirements for the proposed site. Worksheet 1A (Required level of Service – Developed Site) was used to determine the existing site value rating based on the current single-family residential land use. An existing value rating of 17.65 was calculated based on the existing and proposed impervious area for the site. See Exhibit I for Worksheet 1A calculations.

MARC BMP Manual Section 4.0, Worksheet 2 was used to analyze the proposed site BMP mitigation package. Native vegetation swales, an inlet filter, extended-dry detention basins, and native vegetation plantings were used to treat 2.69 ac of the proposed site. See Exhibit I for MARC BMP Manual - Worksheet calculations and BMP descriptions. See Exhibit J for a proposed BMP location map.

APWA 5608.4 and Chapter 6 of the MARC/APWA BMP Manual require 40-hour extended detention to treat the Water Quality Storm. MARC BMP Manual Chapter 6 section 6.2 Short-Cut Method (pg 6-1) was used to determine the water quality volume for a proposed drainage area of less than 10 acres. Table 20 lists rainfall event, percent impervious area, and volumetric runoff coefficient assumptions made for the ProNW and ProS detention basin design. Table 21 lists the water quality volume calculations for each sub basin. EDDB calculations have been provided in Exhibit I.

Table 20. APWA/MARC Water Quality Volume

Rainfall Event (P, in/24-hrs)	1.37
Percent Site Imperviousness (I, %)	35
Volumetric Runoff Coefficient (Rv)	0.635

Table 21. APWA/MARC Water Quality Volume

Detention Basin	Area (AC)	Water Quality Volume (ac-ft)	Provided Water Quality Volume (ac-ft)
ProNW	1.79	0.13	0.13
ProS	0.90	0.06	0.06

SUMMARY

The proposed site will require stormwater detention because the development will increase runoff from the existing conditions. Table 22 summarizes the existing and proposed peak flows from the entire site with no stormwater detention.

Table 22. Summary of Existing and Proposed Peak Flows

Outfall Desc.	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
Total Existing Site	6.04	12.29	21.25
Total Proposed Site w/ out Detention	11.69	19.41	29.59

Two above ground extended dry detention basins, two vegetated swales, a curb inlet filter, and native vegetation have been added to the proposed site (ProNW, ProS, and ProW) to reduce the proposed site peak runoff, improve water quality, and control release rates for all required design storms. Table 23 summarizes the existing and proposed peak flowrate decrease with the included stormwater detention. The proposed detention meets all APWA 5608 peak discharge requirements. Table 24 summarizes allowable and actual proposed site peak discharge requirements.

Table 23. Summary of Total Existing and Proposed Peak Discharges

Outfall Desc.	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
Total Existing Site	6.00	12.22	21.13
Total Proposed Site w/ Detention	4.63	9.16	16.04

Table 24. Summary of Proposed Peak Discharge Requirements

Outfall Desc.	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ProNW Allowable	0.90	3.58	5.37
ProNW Actual	0.91	2.85	4.32
Difference	+0.01	-0.73	-1.05
ProS Allowable	0.45	1.80	2.70
ProS Actual	0.09	0.78	2.54
Difference	-0.36	-1.02	-0.16

The proposed site will have two un-detained sub basins (ProSE & ProW). A request for waiver from the City of Lee's Summit Design and Construction Manual requirement has been proposed based on an overall decrease in peak flowrate discharging to outlet point B (ProSE) and a fringe lot (ProW). Table 25 summarizes the existing and proposed peak flowrates for ProSE. Table 26 summarizes the existing and proposed peak flowrates for ProW.

Table 25. Summary of ProSE Existing & Proposed Conditions

Outfall Desc.	Area (AC)	CN	Q_2 (cfs)	Q_{10} (cfs)	Q_{100} (cfs)
ExSE	1.03	83.00	2.34	4.26	6.95
ProSE	0.73	90.00	2.29	3.80	5.79

Table 26: Summary of ProW Existing & Proposed Conditions

Storm Event	Intensity (in/hr)	Existing Flowrate (cfs)	Proposed Flowrate (cfs)	Increase (cfs)
2-Yr	5.41	0.58	0.80	0.22
10-Yr	7.35	0.80	1.08	0.28
100-Yr	10.32	1.39	1.90	0.51

CONCLUSION

The proposed Sequoia development is a 3.78 acre site in Lee's Summit, MO that will include the construction of 13 duplex units and associated infrastructure. Two above ground extended-dry detention basins have been proposed to control the increase runoff produced by the development.

The proposed development meets all stormwater criteria set forth by the City of Lee's Summit, Missouri and APWA 5600 design criteria. These requirements include an overall decrease in post development peak flowrates, 40-hour water quality extended detention, and a maximum allowable sub basin discharge rate.

A request for waiver from the City of Lee's Summit Design and Construction Manual requirement has been proposed for two un-detained sub basins (ProSE & ProW) based on a peak flowrate discharge decrease under proposed conditions and fringe lot conditions.

Based on this information, Renaissance Infrastructure Consulting recommends approval of this storm study. If you have any questions or need additional information, please contact me.

Sincerely,

Mick Slutter, PE

Jonathan Daldalian, EI

RENAISSANCE INFRASTRUCTURE CONSULTING