NAU NORTHERN ARIZONA UNIVERSITY

The College of Engineering, Informatics, and Applied Sciences

City of Flagstaff
Drywell Feasibility Study

Aggasiz

Municipal Consulting

Flagstaff, AZ

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CENE486 Capstone, Fall 2020

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List of Abbreviations

ADEQ - Arizona Department of Environmental Quality
cfs - cubic feet per second
COF - City of Flagstaff
d/s - downstream
EPA - Environmental Protection Agency
ft - foot
hr - hour
in - inch
LID - Low impact development
s - seconds
u/s - upstream

Acknowledgements

The team would like to acknowledge Mr. Ed Schenk with the City of Flagstaff's Stormwater Management Division and Mr. Tom Loomis for technical recommendations, Dr. Wilbert Odem for technical consulting, and Dr. Bridget Bero, Dr. Jeff Heiderscheidt, and Mr. Mark Lamer for valuable feedback and suggestions throughout this study.

1.0 Introduction

1.1 Project Background

Currently, the City of Flagstaff (COF) sets design requirements on all new construction projects to address stormwater runoff quantity and quality. The city requires the use of detention and retention basins which utilize low impact development (LID) techniques in order to control peak discharge rates and runoff water quality within the city. Low impact development techniques aim to preserve pre-development watershed characteristics after the construction of new infrastructure.

The objective of this study is to address the feasibility of drywells for LID stormwater management in Flagstaff. The Fourth Street area (*figure 1.1* below) was designed decades ago with little consideration for stormwater management given a growing urban area. As such, the CAL Ranch site, located on the southeast corner of Fourth Street and 7th Avenue, is evaluated in this feasibility study, as requested by the client, Mr. Douglas Slover, City of Flagstaff Stormwater Engineer. Currently, the CAL Ranch site has no LID stormwater management features. The site is being evaluated as a theoretical new development, with the assumption that the site is undeveloped and in a natural state. The developed state assumes the current condition of the site, where stormwater management facilities (drywell, retention basin, stormwater routing structures) will be incorporated for best management and LID practices in accordance with the COF stormwater design manual.

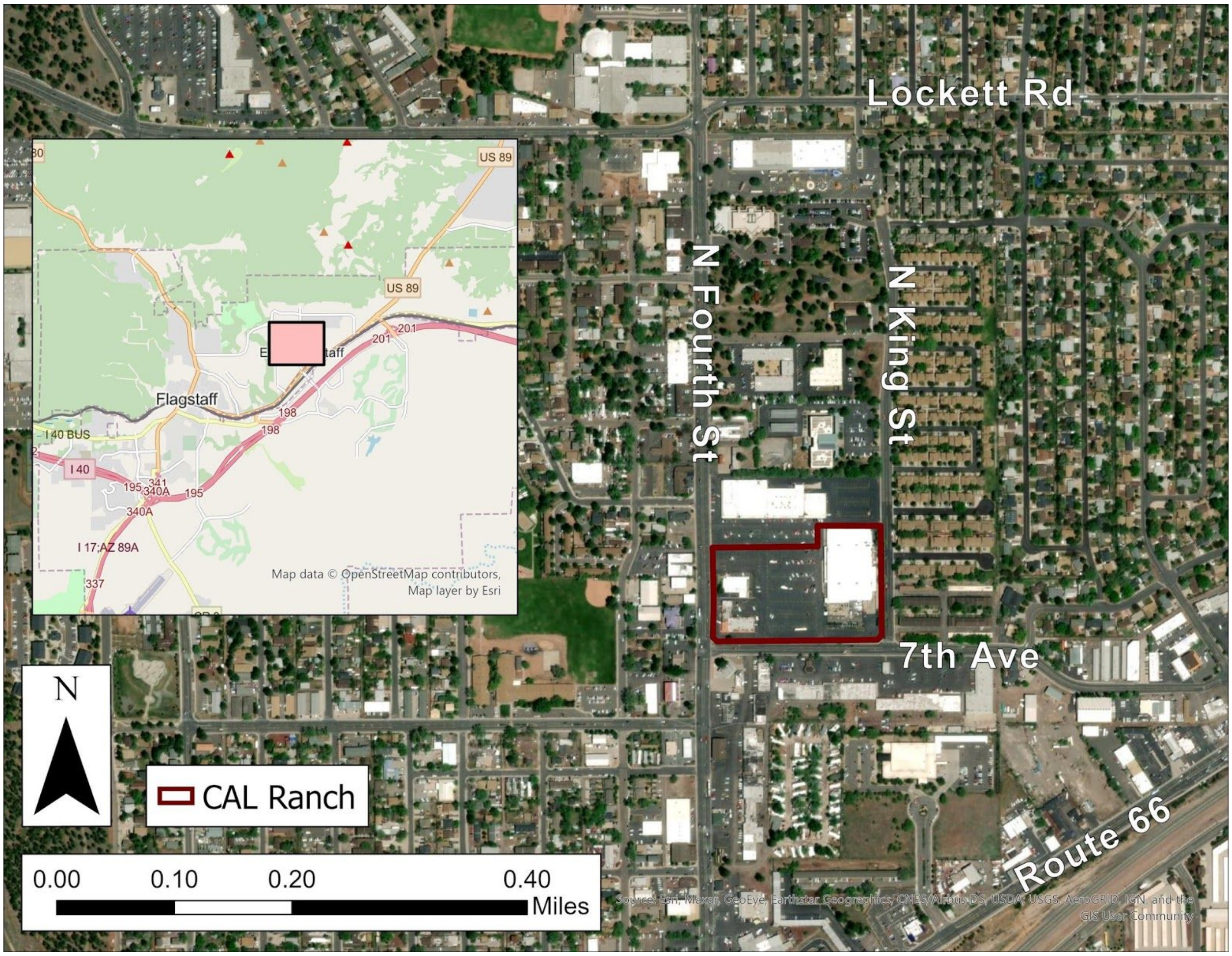

Figure 1.1: Fourth Street Area ad CAL Ranch Site

A drywell is a vertical retention system which allows excess storm runoff to infiltrate into the ground. Drywells allow runoff to collect in an underground chamber filled and lined with highly-permeable materials. Drywells commonly implement pretreatment systems to improve water quality. This pretreatment typically includes a sedimentation chamber to prevent sediment buildup within the drywell, which prolongs the lifespan of the drywell by decreasing the suspended solids of water entering the drywell, thus decreasing maintenance costs. Figure 1.2 shows a typical drywell design. Key considerations for the feasibility of drywells include soil characteristics, peak runoff volumes, storage capacity, water quality, constructability, and cost-effectiveness. Stormwater quality constraints are outlined by the Arizona Department of Environmental Quality (ADEQ) [1]. According to ADEQ it is not recommended that drywells be installed in areas where hazardous or toxic materials are present although it is not strictly prohibited. If such materials exist on site then an aquifer protection permit is required and special engineering precautions must be taken [1]. Typical drywell dimensions range from 2-8 feet in width and 4-14 feet in depth.

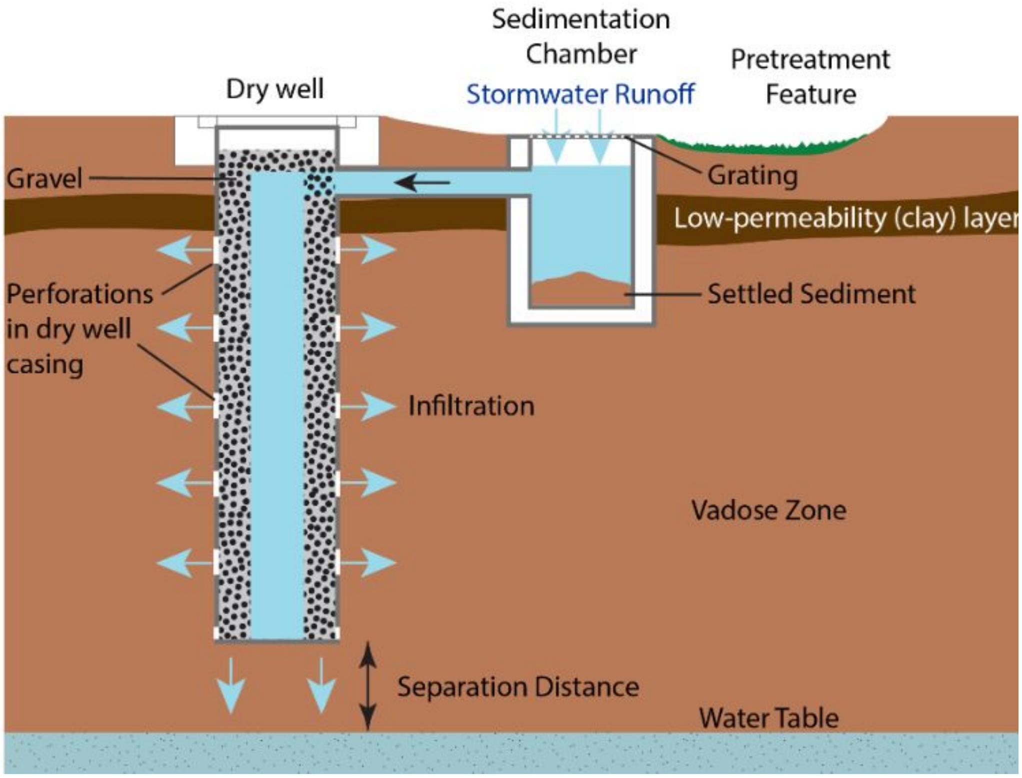

Figure 1.2: Typical Drywell Design [2]

1.2 Project Location

Figure 1.3 below shows the CAL Ranch site. This shows that the topography of the site (2ft contours) causes the site to drain from the northwest corner to the southeast corner. The site is located north of 7th Avenue between Fourth Street and King Street. Half of the adjacent easements were considered as part of the site for on-site stormwater calculations . The area of the CAL Ranch site and half of the adjacent easements is 9.39 acres. North of the CAL Ranch is a UPS store and various other small businesses with lots that are 100% impermeable. The site sits at a mean elevation of 6883 feet above sea level. Figure 1.3 also shows topographic contours (2'), showing the site is drained from the northwest to the southeast.

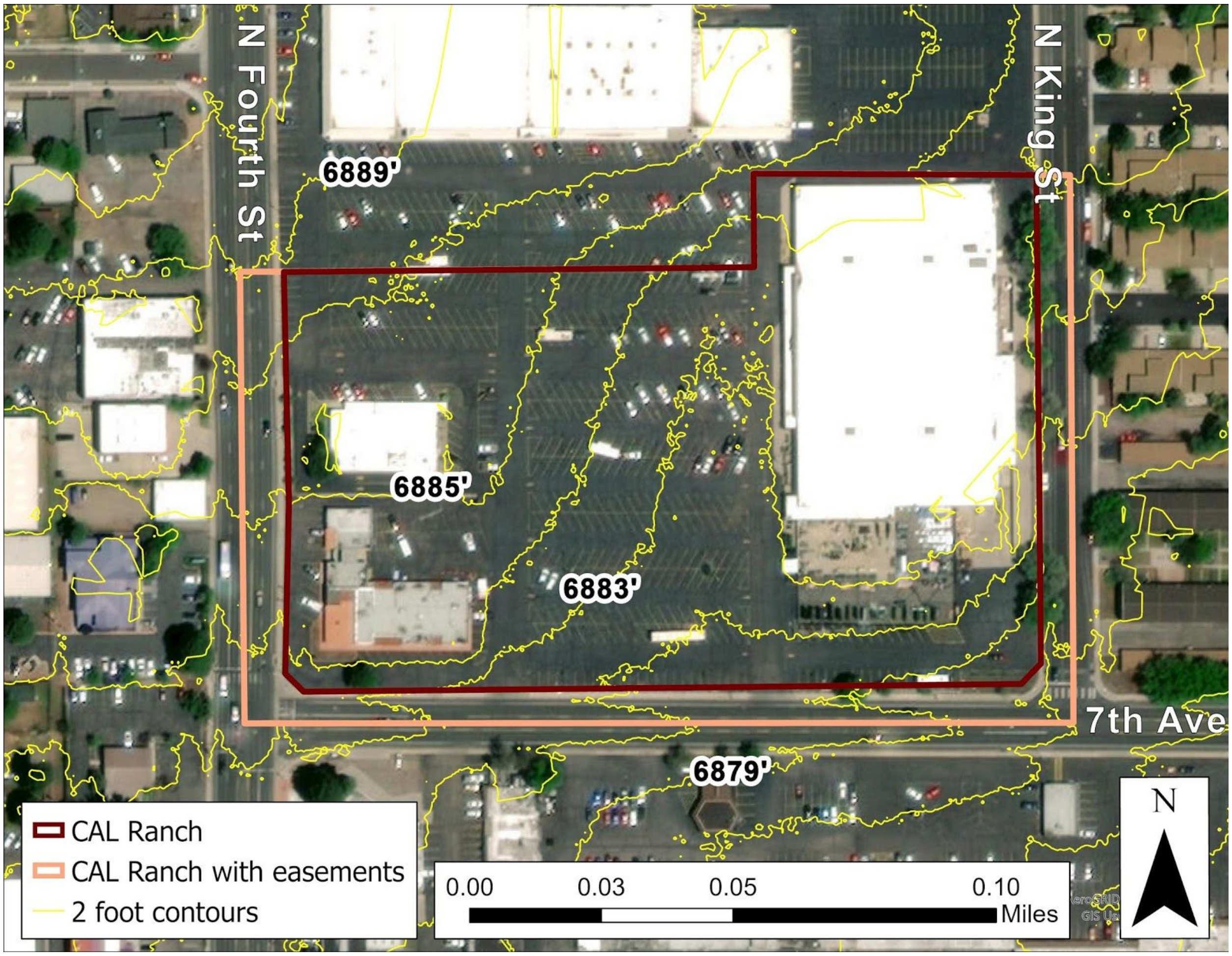

Figure 1.3: CAL Ranch Site Map with 1' Elevation Contours

Figure 1.4 below shows the two watersheds adjacent to the CAL Ranch Site, Spruce Ave Wash and King Street. Per recommendations made by Ed Schenk with the City of Flagstaff Stormwater Management division, it is assumed that flows from the Spruce Ave Wash watershed are conveyed past the site via existing subsurface stormwater infrastructure. Flows from the King Street watershed have been considered when designing off-site stormwater routing through the site. Flows from the surrounding areas follow Fourth Street and King Street south, and then continue east on 7th Avenue.

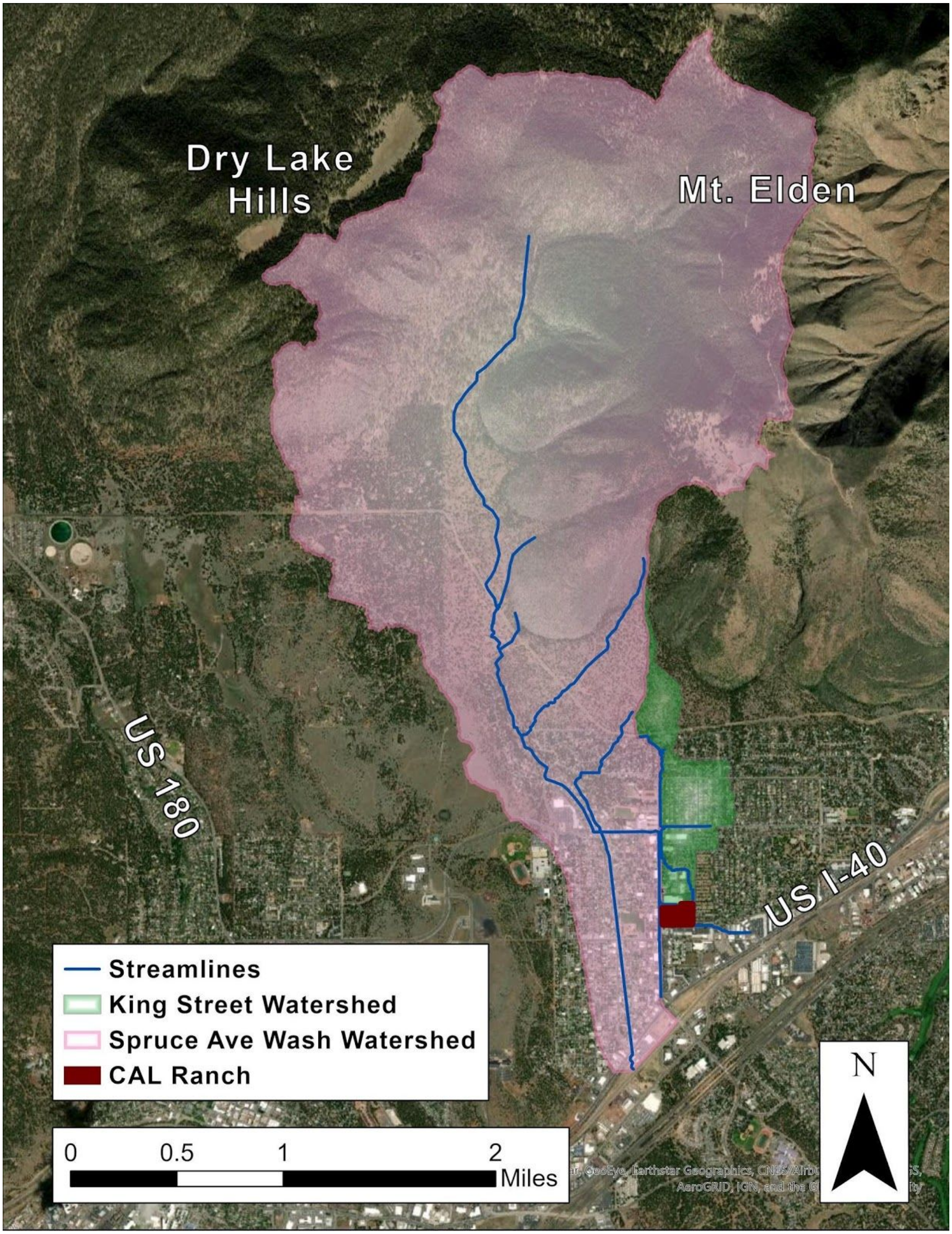

Figure 1.4: CAL Ranch and Adjacent Watersheds

1.3 Code Research

Coconino County requires all new construction to follow the codes and regulations. Codes are set in place to maintain and regulate public health, safety, and general structural and ecological welfare of a construction and its surroundings. Construction codes are set as laws in each jurisdiction regulated and enforced by a local governing body. In this case, pertinent codes and ordinances for drywells are compiled from local and neighboring governmental bodies in order to set guidelines for the drywell construction.

According to the ADEQ, it is a requirement that a drywell be drilled in alluvial sediments, through any overriding cemented and fine-grained zones, into a permeable layer of clay that is free of sand, cobbles, and gravel. The permeable layer acts as the inoculation zone for the storm water. Additionally, it is a requirement that there be at least 10 feet of separation between the water table and the saturated soil [1] . Since the groundwater in common instances occurs at a great depth in alluvial basins found in Arizona, constructors normally have substantial margin to find a remarkably permeable zone above the water table. The major objective of this is to maximize the performance of the drywell while preserving a much bigger separation distance than the minimum of 10-foot.

The inlet section of the drywell includes a manhole for ease of maintenance. Therefore, the initial 16 inches of the drywell serve as a manhole. For the manhole, a minimum drop of 0.10 foot is needed in all and through all storm drain manholes. As well as a drop of 0.3 feet is required in all drywells. In addition, the crown of the outlet pipe of the drywell must be set below the crown of the inlet pipe of the manhole section of the dry well. [1]

With respect to Maricopa County's Drainage Policies and Standards [3], under standard 6.10.13, it is a requirement that a drywell be constructed in a permeable area and the most appropriate location is 10 feet above saturated soils. Also, the drywell must be located 100 feet away from an area of water supply. The dimensions that are selected to fit the drywell must allow a percolation rate of at least 0.1 CFS. However, the storm water is restricted in instances whereby the bottom of the basin is characterized by an impermeable layer, within 4 feet of this depth. In addition, the upper boundary for the rate of percolation should not go beyond 0.5 CFS.

According to Buckeye Public Works Department (BPWD) Storm Water Drainage Design [4] under standard 6.4 drywells are required to penetrate approximately 10 feet into a stratum that is permeable. Also, a percolation test must be conducted on the drywell before it can be permitted. The drywell will adhere to the BPWD Storm Water Drainage Design code 6.4-14 which requires that drywells be located a minimum of 100 feet far from septic systems and water wells. The BPWD Storm Water Drainage Design code 6.4-15 states that drywells shall be located a minimum of 25 feet from stormwater underground storage structures. Property owners are responsible for maintenance of local on-site LID facilities as per 9.1 section J of the Buckeye Stormwater Design Manual [4].

Drywells are to comply with the Coconino County Code and Ordinances as per section 7.11 [C] subsection: Guidance for Design Installation, Maintenance, Operation, and Inspection of Drywells. Drywells are required to be registered and or approved by Coconino County and should be constructed by a contractor or group who is licensed by the County [5]

According to the Arizona Department of Environmental Quality Code: R18-9-C301. 2.01 [6] allows for a drywell that drains a place where hazardous substances are used, stored, loaded, or treated. In the permit, a permittee shall retain a drywell drilling contractor, licensed under 4 A.A.C.9, of Arizona department of Environmental Quality to close the drywell. Also, the permittee has to implement a Best Management Practices Plan for operation of the drywell Including control pollutants and saturation in the drywell drainage area [6].

2.0 Hydrologic Analysis

2.1 On-Site Post-development Runoff Using the Rational Method

Per the client's request, the CAL Ranch site is being evaluated as a "new" development with >90% impermeable coverage. Additionally, the retention/drywell system shall be designed to retain the 100-year 6-hour rainfall.

Due to the size of the site (9.39 acres with ½ of adjacent easements), on-site post-development runoff was computed using the Rational Method (equations 2.1 and 2.2 below) per the *City of Flagstaff Stormwater Management Design Manual* (2009) [5]. Rainfall intensities in inches per hour were obtained from the *NOAA Atlas 14 Point Precipitation Frequency Estimates* [7]. 0.503 inches per hour was taken as the 100-year 6-hour rainfall intensity at the CAL Ranch site [7]. Table 2.1 shows antecedent precipitation factors for the rational method. The antecedent precipitation factor is applied to account for less frequent higher intensity storms where losses such as infiltration and interception have a smaller effect on runoff. Table 2.2 below shows the runoff coefficients used for various cover types.

(eq. 2.1) $$Qp = Cf\ C\ I\ A$$ [5]

where Qp = *peak runoff (cfs)*
Cf = *antecedent precipitation factor (unitless)*
C = *runoff coefficient (unitless)*

I = rainfall intensity (in/hour)
A = drainage area contributing to the design location (acres)

(eq. 2.2) $$Cw = (C_1A_1 + C_2A_2 + \ldots + C_nA_n) / A_{total}$$ [5]

where *Cw - weighted runoff coefficient*
C_n *= runoff coefficient for a particular section*
A_n *= area for corresponding section (acres)*
A_{total} *= total area (acres)*

Table 2.1 - Antecedent Precipitation Factors [5]

Storm Frequency (years)	Factor, *Cf*
1, 2, 5, 10	1
25	1.1
50	1.2
100	1.25

Table 2.2: Rational Method Runoff Coefficients [5]

Surface Description	Coefficient, *C*
Streets	0.95
Asphalitc Concrete	0.95
Concrete	0.95
Woods (Slope <2%)	
Sandy Soils	0.05
Gravelly Soils	0.07
Clay Soils	0.1

On-site peak runoff for the Cal Ranch site for the 6-hour 100-year storm is 5.61 cfs. Table 2.3 below shows the input parameters used in the rational method peak runoff computation.

Table 2.3: Rational Method Peak Runoff Input Parameters

Weighted Runoff Coefficient		
Cover Type	Area (acres)	C
Concrete	0.39	0.95
Roofs	3.00	0.95
Asphalt/Pavement	6.00	0.95
Weighted Runoff Coefficient, C		0.95
Antecedent Precipitation Factor, Cf		1.25
6-hour 100-year Rainfall Intensity, I	0.503	in/hr
Area, A (w easement)	9.39	acres
Peak Runoff, Qp	**5.61**	**cfs**

2.2 LID Runoff Requirement

Per the *City of Flagstaff Stormwater Management Design Manual* [5], low impact development (LID) is required for all new subdivisions, commercial and industrial developments, and redevelopments of non-conforming sites. Primary LID requirements state that the first inch of runoff for a new or redeveloped site shall be retained and that the 2-year "first flush" runoff shall be routed through a stormwater quality system, in this case, a retention basin/drywell system. The minimum required LID volume is taken as the site area multiplied by the first inch of rainfall, or 9.39 acre-inches (34,086 ft^3).

2.3 Off-Site King Street Watershed Runoff Using the TR-55 Method

Per the *City of Flagstaff Stormwater Management Design Manual* (2009), runoff volumes for watersheds >20 acres and <2000 acres shall be calculated using the NRCS TR-55 method, also known as the NRCS Curve Number Method. The peak runoff from the 170 acre (0.27mi^2) King Street watershed was computed using the TR-55 method using an SCS Type II 24-hour 100-year storm. The King Street watershed is shown below in Figure 2.1. In order to compute peak discharge using the graphical peak discharge method, runoff depth, and time of concentration are needed (equations 2.3-2.11 below). The TR-55 method calculates runoff volumes based on the storage capacity of the land cover (a function of the curve number) and the initial abstraction, which accounts for losses such as interception, infiltration, and storage capacity. The runoff curve numbers for various land covers can be found in Appendix A Table A.1. For more information on the TR-55 method, readers are referred to the *United States Department of Agriculture Technical Release 55: Urban Hydrology for Small Watersheds* [8].

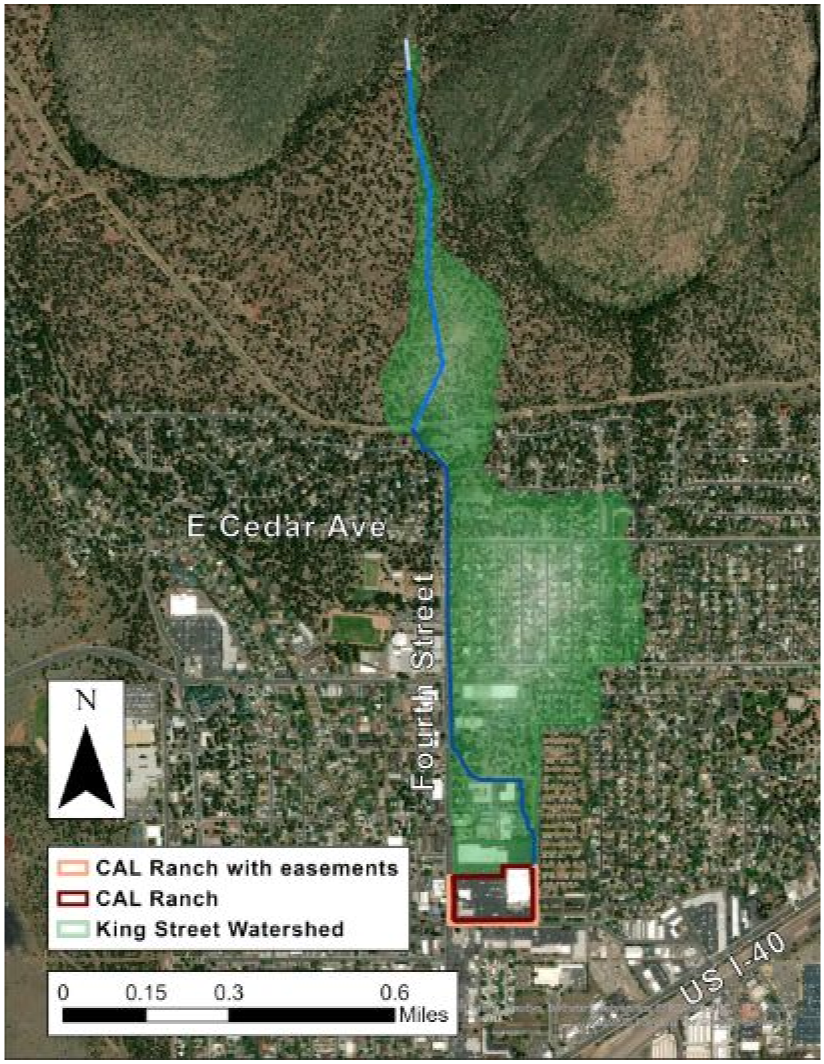

Figure 2.1: King Street Watershed and Flow Path

(eq. 2.3) $$Q = (P - Ia)^2 / ((P - Ia) + S)$$

(eq. 2.4) $$Ia = 0.2S$$

(eq. 2.5) $$S = (1000/CN) - 10$$

where Q = runoff (in)
P = 100-year 24-hour rainfall depth (in)
Ia = initial abstraction (in)
S = storage capacity (in)
CN = composite curve number (see Appendix A Table A.1

(eq. 2.6) $$Tc = T_1 + T_2 + T_3$$

where Tc = *time of concentration (hr)*
T_1 = *time of sheet flow (hr)*
T_2 = *time of shallow concentrated flow (hr)*
T_3 = *time of open channel flow (hr)*

(eq. 2.7) $$T_1 = (0.007(nL)^{0.8})/((P_2{}^{0.5})(s^{0.4}))$$

where T_1 = *time of sheet flow (hr)*
n = *Manning's n for sheet flow (see table 2.4)*
L = *length of sheet flow (ft)*
P_2 = *2-year 24-hour rainfall depth (in)*
S = *land slope (in/in)*

(eq. 2.8) $$T_2 = L/(3600V)$$

where L = *length of shallow concentrated flow (ft)*
V = *average flow velocity for shallow concentrated flow (see figure 2.1)*

(eq. 2.9) $$T_3 = L/(3600V_{open-channel})$$

where L = *length of open channel flow (ft)*
V = *open channel flow velocity (ft/s) (see equation 2.8)*

(eq. 2.10) $$V = (1.49\,(r^{2/3})(s^{1/2}))\,/\,n$$

where r = *hydraulic radius (flow area / wetted perimeter) (ft)*
S = *channel slope (ft/ft)*
n = *Manning's roughness coefficient for open channels (see table 2.4)*

(eq. 2.11) $$q_p = q_u A_m Q F_p$$

where q_p = *peak discharge (cfs)*
q_u = *unit peak discharge for SCS Type II rainfall distribution (cfs per* mi^2 *of drainage area per inch of runoff (see figure 2.2)*
A_m = *drainage area* (mi^2)
Q = *runoff (in) (from equation 3)*
F_p = *pond and swamp adjustment factor (see table 2.6)* [8]

Table 2.4: Manning's n for Sheet Flow [9]

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: [3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).

2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

3 When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

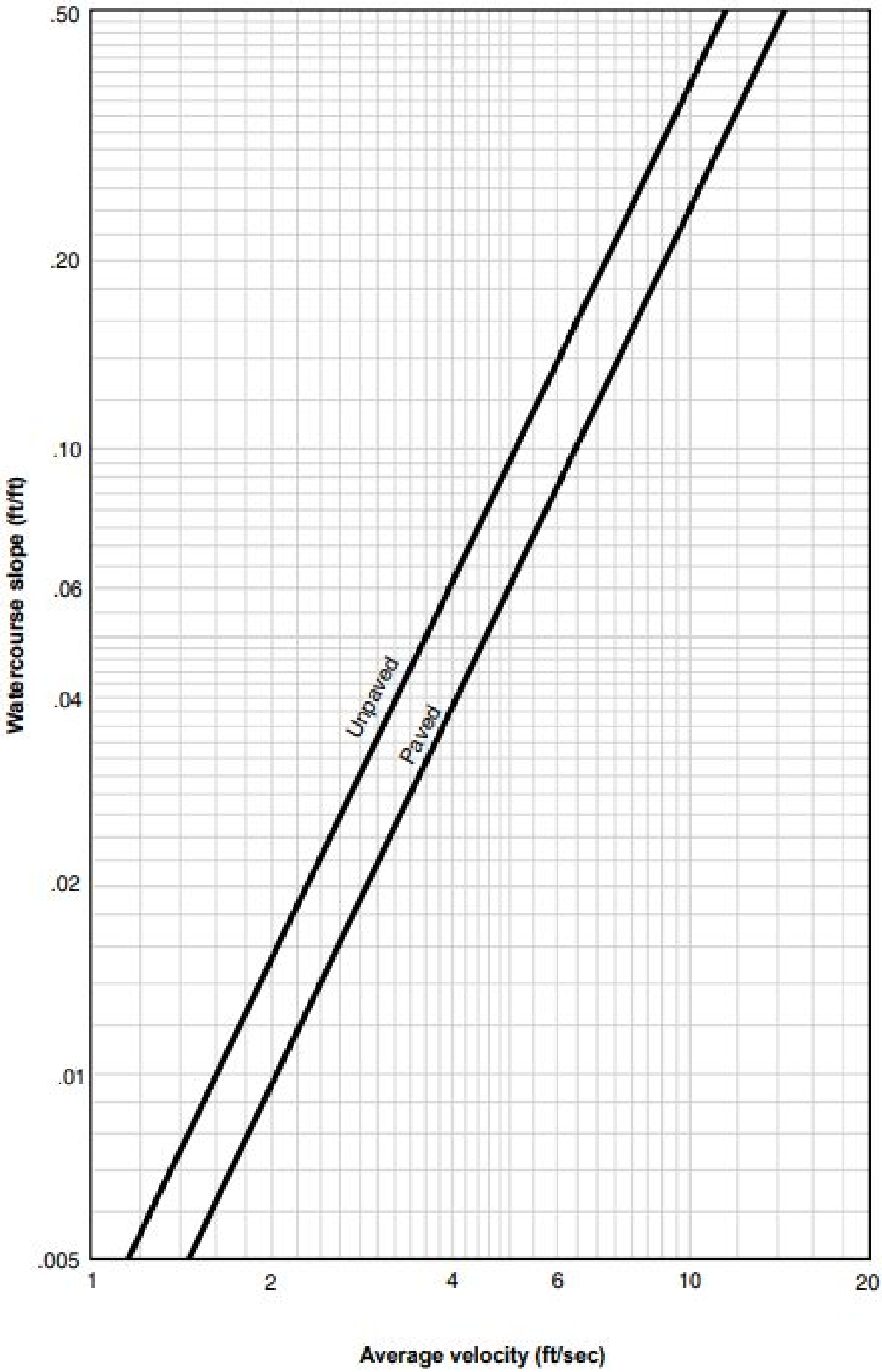

Figure 2.2: Average Flow Velocity for Shallow Concentrated Flow [5]

Table 2.5: Manning's n for Open Channels [5]

Type of Channel and Description	Minimum	Normal	Maximum
Natural streams - minor streams (top width at floodstage < 100 ft)			
1. Main Channels			
a. clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
b. same as above, but more stones and weeds	0.030	0.035	0.040
c. clean, winding, some pools and shoals	0.033	0.040	0.045
d. same as above, but some weeds and stones	0.035	0.045	0.050
e. same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. same as "d" with more stones	0.045	0.050	0.060
g. sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

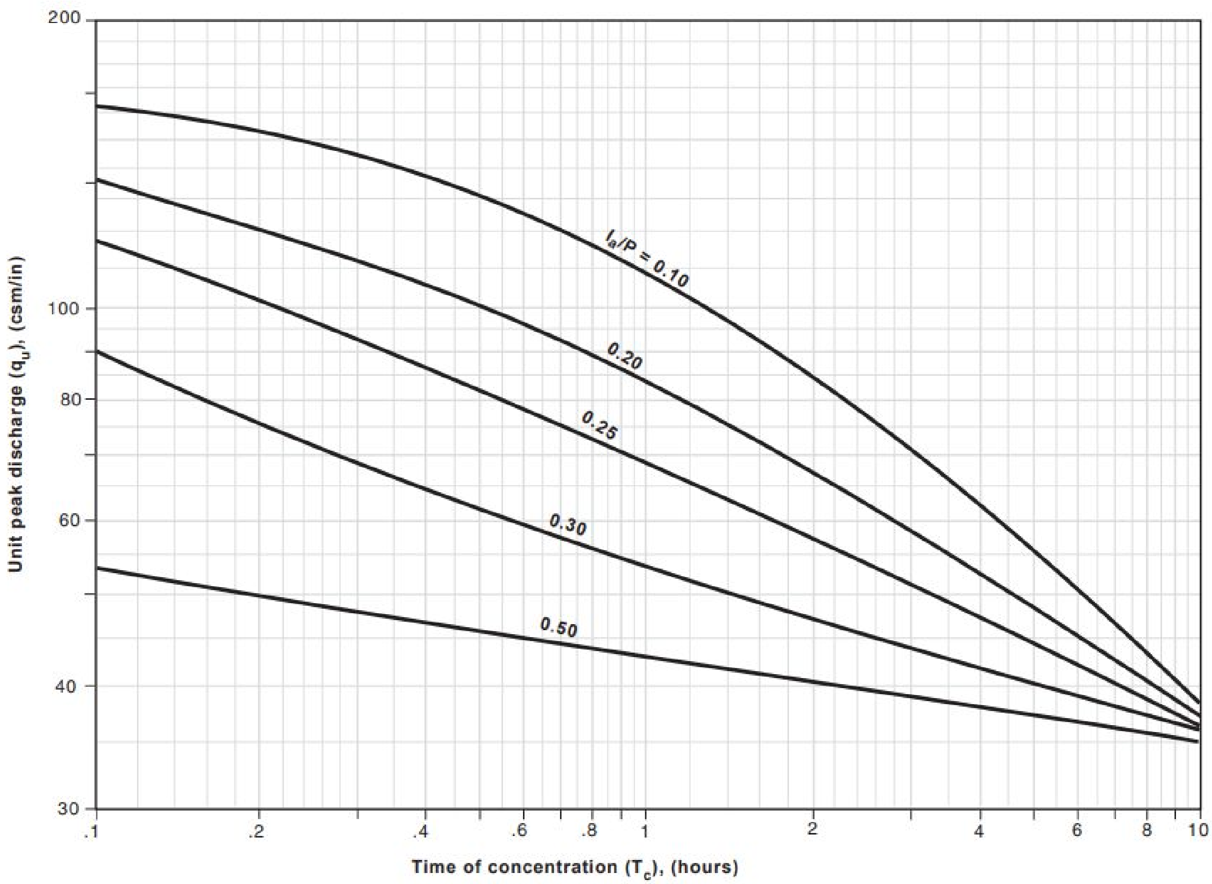

Figure 2.3: Unit Peak Discharge for SCS Type II Rainfall Distribution [5]

Table 2.6: Pond and Swamp Adjustment Factor [5]

Percentage of pond and swamp areas	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

Peak Runoff for the King Street watershed computed using the TR-55 method with a composite curve number of 84 is 72.2 cfs. The time of concentration for the watershed is 6.50 hours. The input parameters used in the time of concentration calculation, composite curve number calculator, and peak runoff calculation, are shown in Tables 2.7, 2.8, and 2.9 below.

Table 2.7: King Street Watershed Time of Concentration

Sheet Flow		
Manning's n	0.4	(Table 2.5)
Length, L	300	ft
P(2-year 24-hour)	2.67	in
u/s elevation	7674.9	ft
d/s elevation	7568.4	ft
Slope, S	0.355	ft/ft
Tsheet	**0.30**	**hours**
Shallow Concentrated Flow		
Length, L	3513.5	ft
u/s elevation	7568.4	ft
d/s elevation	7086.6	ft
Slope, S	0.1371282197	ft/ft
Velocity, V	10	ft/s (Figure 2.1)
Tshallow_concentrated	**0.10**	**hours**

Open Channel Flow		
Flow area, A	10.00	ft^2
Wetted perimeter, P	30.00	ft
Hydraulic Radius, r	0.33	ft
u/s elevation	7086.60	ft
d/s elevation	6875.80	ft
Length, L	5351.60	ft
Slope, S	0.04	ft/ft
Manning's n	0.05	(Table 2.5)
Velocity, V	0.24	ft/s
Topen_channel	**6.11**	**hours**
Time of Concentration, Tc	**6.50**	**hours**

Table 2.8: King Street Watershed Composite Curve Number

Terrain Type	Area (acres)	HSG	CN	CN * Area
Developed Open Space (fair condition)	9	C	79	711
Developed Low Intensity (1/2 acre residential lots)	21	C	80	1680
Developed Medium Intensity (1/8 acre residential lots)	72	C	90	6480
Developed High Intensity (urban commercial & buisiness)	23	C	94	2162
Evergreen Forest (woods fair condition)	45	C	73	3285
Shrub/Scrub (woods-grass combination)	0.22	C	81	18
Total Area	170.22	acres		
Composite CN		**84.2**		

Table 2.9: King Street Watershed Peak Runoff Calculation

Storm Frequency	100	yr
Rainfall, P (24 hour)	4.45	in
Storage Capacity, S	1.874	in
Initial Abstraction, Ia	0.375	in
Runoff, Q	**2.79**	**in**
Rainfall depth, P (100-year 24-hour)	4.45	in
Composite curve number, CN	84.22	
Initial abstraction, Ia	0.37	in
Ia/P	0.08	
Unit peak discharge, qu	100.00	csm/in
Drainage area, Am	0.27	mi^2
Runoff, Q	2.79	in
Ponding factor, Fp	1.00	
Peak discharge, qp	**74.2**	**cfs**

2.5 Street Flow Capacity

For the purpose of determining if the off-site runoff generated upstream of CAL-Ranch needs to be routed through the site, street flow capacity on King Street adjacent to CAL Ranch is computed using empirical street flow capacity equation described in the *City of Flagstaff Stormwater Drainage Design Manua*l, equation 2.12 below.

(eq. 2.12) $Q = (0.56/n)\, Sx^{1.67}\, S^{0.5}\, T^{2.67}$ [5]

where Q = *peak street flow discharge capacity (cfs)*
n = *Manning's roughness coefficient for open channels (see table 2.4)*
Sx = *pavement cross slope (ft/ft)*
S = *longitudinal cross slope (ft/ft)*
T = *width of flow, CL to curb (ft)*

The street flow capacity for King Street adjacent to CAL Ranch is 284.5 cfs which is greater than the peak runoff of the King Street watershed of 74.2 cfs, computed in section 2.4 above. Therefore, off-site flows need not be routed through the site, so long as the site elevation is raised above the King Street curb elevation. The input parameters and dimensions used in the street flow capacity computation are shown in Table 2.10 below.

Table 2.10: King Street Street Flow Capacity

u/s elev	6882.23	ft
d/s elev	6875.20	ft
Longitudinal length	507.3	ft
CL elev	6882.36	ft
Gutter elev	6882.13	ft
Low point elev	6881.65	ft
width from curb to low point	6	ft
Manning's n	0.015	
Pavement cross slope, Sx	0.0800	ft/ft
Longitudinal cross slope, S	0.0139	ft/ft
CL to curb flow width, T	22	ft
Street Flow Capacity, Q	**248.5**	**cfs**

Although the street flow capacity of King Street is greater than the off site runoff, the topography of the area on the northeast corner of the lot has the potential to divert the off site flow onto the CAL Ranch parcel. Raising the elevation of the northeast corner of the lot slightly will ensure that the flow does not flow over the curb onto the site.

3.0 Geotechnical Analysis

3.1 Surface Soils

Surface soil parameters used in hydrologic analysis (sections 2.1 - 2.4) was acquired from the USDA Web Soil Survey [10]. Figure 3.1 below shows a map of the surface soil types. CAL Ranch sits atop Paymaster Fine Sandy Loam with 0-3 percent slopes. North of the CAL Ranch is Bald Sandy loam with 2% slopes. Both soils are in hydrologic soil group (HSG) B, which means Group B have reasonably low runoff potential when thoroughly wet.

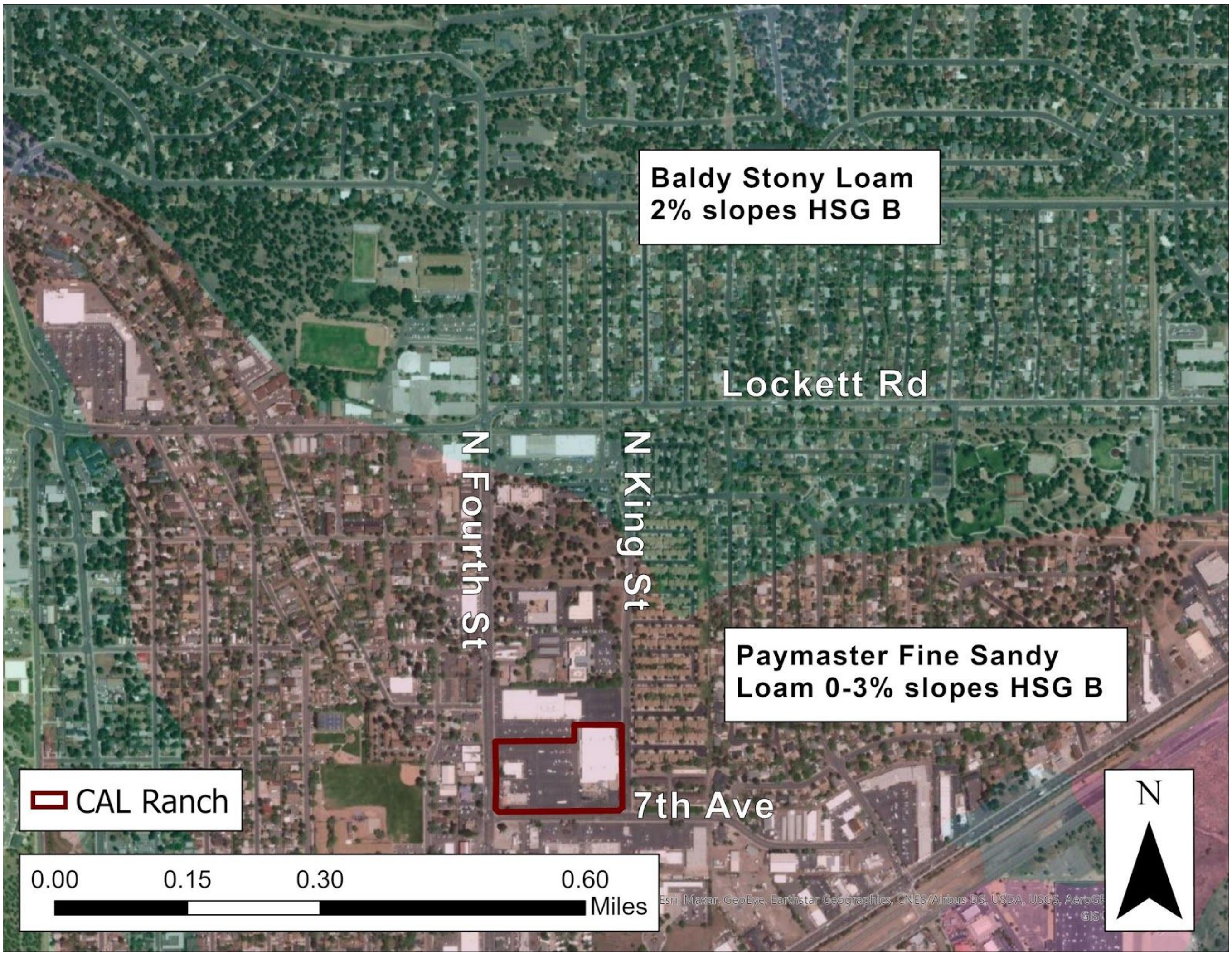

Figure 3.1: Soil Map showing Soil Name and Hydrologic Soil Group (HSG) [8]

3.2: Falling Head Percolation Test used to Determine Hydraulic Conductivity

Percolation tests were performed on September 30th, 2020, in accordance with Maricopa County standards for falling head percolation test procedure [9]. At least two tests must be performed in the bottom area of the proposed stormwater basin. The tests for this project were not performed on site since the entire lot is covered in asphalt. Instead the tests were performed in an existing basin just north of the CAL Ranch parcel. The basin was located on Coconino County Health Department property as shown in the Figure 3.2

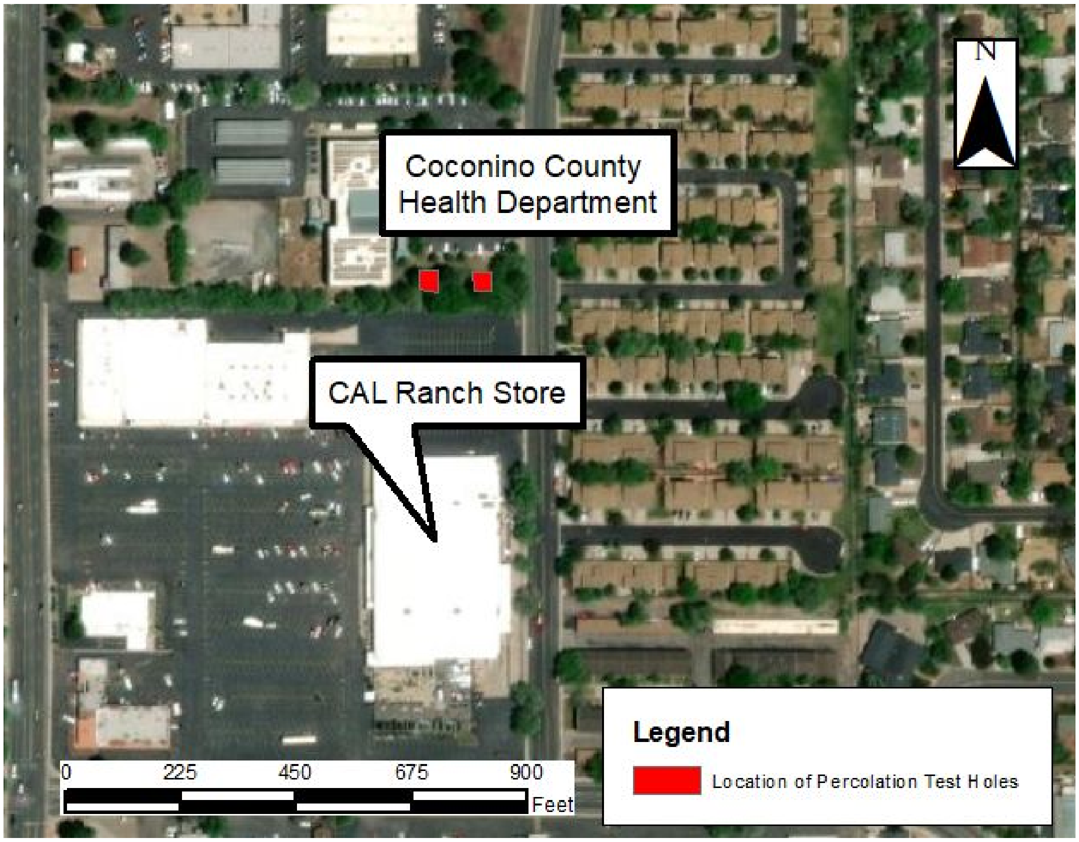

Figure 3.2: Location of Percolation Test

Proper precautions were taken prior to the conduction of the percolation test including receiving permission from the Coconino County Health Department and contacting AZ Blue Stake in order to ensure no utility lines would be disturbed. The NAU Field Safety Checklist was also completed prior to going to the field. The procedure used for testing the soil and calculating the percolation rate is as follows:

- Two uniform 12 inch diameter holes (identified as “east” and “west”) were dug 14 inches deep using a shovel. Since tools for digging deep were limited, it was assumed that the limiting soil horizon was captured within the minimum required 14 inches.
- All loose soil on the sides and bottom of the holes were removed using the shovel and gloved hands.
- Two inches of small diameter gravel was placed in the bottom of the hole in order to reduce likelihood of scour. This leaves 12 inches of depth that the water can occupy.
- The holes were filled with water to a depth of 12 inches above the gravel, which was also the top of the hole. The holes were allowed to soak overnight and which allowed the surrounding soil to saturate.
- Percolation rates were measured 15 hours after the soaking period began which is the minimum required time to wait after the initiation of soaking. The maximum time is 30 hours.

- Percolation data was collected by taking an initial measurement, at time 0; water was added prior to the test start so that the water level was 6 inches above the gravel layer.
- Every 30 minutes a measurement was taken over a 90 minute period.
- The percolation rate was calculated by dividing the final measured depth decrease by the 30 minute interval. [3]

This test was repeated in each hole for a total of two tests in each hole. An image of a test hole is shown in Figure 3.3.

Figure 3.3: Percolation Test Hole

The results obtained from this test are shown in Table 3.1 below. As seen in the table the results for each hole and each test were fairly consistent. The highest value was 3 in/hr and the lowest value was 2.625 in/hr. The median value for percolation was 2.875 in/hr. In order to be conservative for design the lowest percolation rate was used (2.625 in/hr).

Table 3.1: Percolation Test Results

Test 1	East	West
Time (hours)	Depth (in)	
0	6.0000	6.0000
0.5	4.5000	4.4375
1	3.0000	3.0000
1.5	1.5000	1.5625
	Hydraulic Conductivity, K (in/hr)	
	3.000	2.875

Test 1	East	West
Time (hours)	Depth (in)	
0	6.0000	6.0000
0.5	4.5000	4.5000
1	3.0625	3.1875
1.5	1.6250	1.8750
	Hydraulic Conductivity, K (in/hr)	
	2.875	2.625

4.0 Retention Basin and Drywell Design

4.1 Determination of Retention Basin and Drywell Dimensions and Details

The rate of discharge through percolation for an individual drywell was computed using the Hvorslev deep flow drywell flow rate solution (equation 4.1) [10]. The deep flow Hvorslev Solution was developed for computing flow through cylindrical wells in saturated systems with a high depth to the groundwater table (>30 ft). [10]

(eq. 4.1) $Q = (2\pi KLH) / \ln[(2L/r) + \sqrt{(1+(2L/r)2)}]$ [11]

where Q = *drywell percolation rate (cfs)*
K = *hydraulic conductivity (ft/s)*
L = *length of screened portion of the drywell (ft)*
H = *height of the drywell (ft)*
r = *radius of the drywell (ft)*

In the case of these drywells, the screened portion of the drywell (L) is assumed the same as the height (H) of the drywell. Using Equation 4.1, the chosen dimensions of the drywell, and the percolation rate (in this equation hydraulic conductivity) the discharge in cfs per drywell was calculated. The results of this calculation are shown in Table 4.1.

Table 4.1: Hvorlsev Deep Flow Drywell Flowrate

Hvorlsev Deep Flow Drywell Flowrate		
Hydraulic Conductivity, K	2.625	in/hr
Hydraulic Conductivity, K	6.076E-05	ft/sec
Length of Screen of Well, L	10	ft
Height of water in borehole, H	10	ft
Radius of Borehole, R	3	ft
Drywell Discharge Rate, Q	0.01471	cfs

All highlighted cells in this table are values that were input from typical drywell details or from results of percolation tests. The result shows that the discharge (through infiltration) rate of a drywell with these dimensions is about 0.015 cfs per drywell. The discharge (through infiltration) rate of the retention basin bottom, not including the manhole covers of the drywells, is 0.0005 cfs per square foot of retention basin bottom.

In the determination of the retention volume of the basin the dimensions of the basin had to be considered as well as the number of drywells that could fit within the basin given the 50 foot required on center spacing for sites with a depth to groundwater of greater than 30 feet [11]. Determining the design retention basin dimensions was an iterative process. The iterated dimensions of the retention basin and the number of drywells determined what the total flow into the soil would be in cfs and as determined in the hydrology section, the onsite inflow to the retention basin is 5.61 cfs. The total flow into the soil was determined by multiplying the discharge per drywell by the number of drywells in the basin and adding that to the flow rate per square foot of the retention basin (found using the results of the percolation test) multiplied by the surface area of the retention basin. Given the available dimensions of the site, multiple combinations of retention basin size and number of drywells were looked at. For example, a larger basin with no drywells was looked at, as well as a smaller basin with more drywells. The resulting optimal combination of retention basin and drywells was found to be 170X170 ft, with 10 wells with a depth of 10 feet and radius of 3 feet, spaced 50 feet center to center.

Both retention, and detention volumes are calculated using the Triangular Hydrograph per the COF SWMDM, shown in equation 4.2 below.

(eq. 4.2) $V = (1/2)(Qi - Qo)(Ti)$

where V = *Required retention/detention volume (ft^3)*
Qi = *peak inflow (cfs)*
Qo = *peak outflow (cfs)*
Ti = *Inflow duration (s)*

The difference between the outflow into the soil (Qout) and the peak inflow to the basin (Qin) is plotted on triangular hydrograph shown in Figure 4.1. This figure shows the hydrograph of the results of the basin dimensions and number of drywells that went to design. The dimensions that resulted in this hydrograph can be seen in Table 4.2. Details for the other iterations, including the most extreme cases of no drywells and a large retention basin, and many drywells and a small retention basin, can be seen in Appendix B.

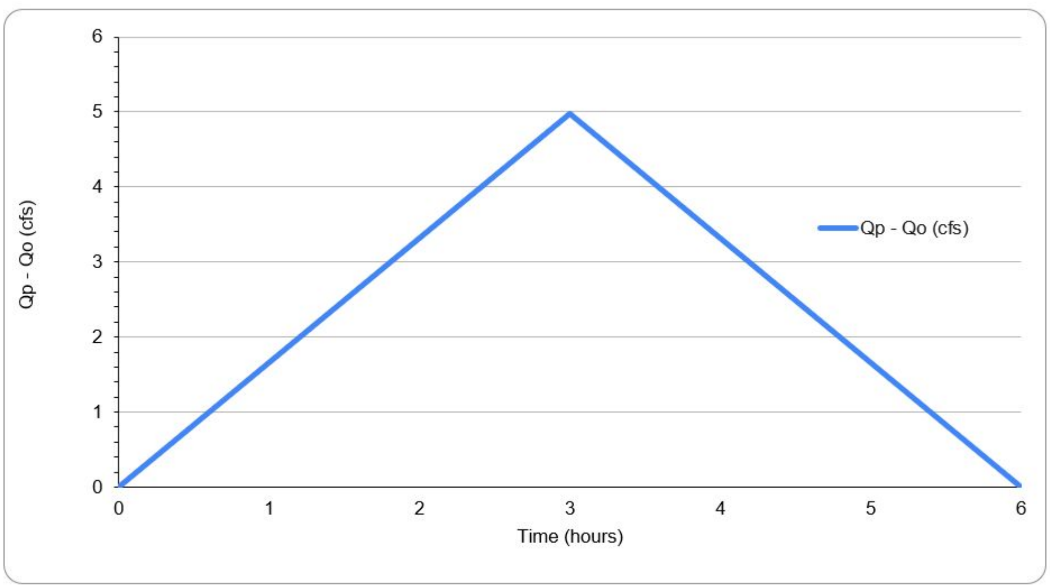

Figure 4.1: Hydrograph of Qin-Qout

Table 4.2: Design Retention Basin Dimensions and System Discharge

Retention Basin Dimensions and System Discharge		
Single Drywell Discharge Rate	0.01471	cfs/drywell
Design Retention Basin Infiltraton Rate	0.00001	ft/s/unit area
Retention Basin Disharge Rate	0.290	cfs
Single Drywell Discharge Volume / 36 hours	1,906.13	ft^3
Number of Drywells	10	
Retention Basin Length	170	ft
Retention Basin Width	170	ft
Retention Basin Area	28617.26	ft^2
Time to drain	36	hours

As shown, a 170X170 ft basin, with the maximum number of drywells, 10, will produce a difference in peak discharge of around 5 cfs. In order to obtain a required retention volume from this, the area under the triangular curve is calculated per City of Flagstaff Stormwater Design Manual [5]. This calculation is simply the peak discharge multiplied by the time of 6 hours divided by two. The time placement of the peak flow of the hydrograph is not relevant in relation to the City of Flagstaff Standards so the peak flow was located at the chronological midpoint which results in a symmetric graph. The calculations for this design volume is shown in Table 4.3.

Table 4.3: Triangular Hydrograph Retention Volume

COF Triangular Hydrograph Retention Volume		
Number of Drywells	10	
Storm Duration for Qpeak (rational method) = Qi	6	hours
On-Site Time of Concentration, Tc	3.19	minutes
Inflow Time, Ti	6.053	hours
Inflow Time, Ti	21791.4	secs
Inflow Rate, Qi	5.61	cfs
Drywell and Retention Outflow Rate, Qo	0.436894	cfs
Retention Volume, V	56321	ft^3
Mimimum Retention Volume per LID Requirement	34086	ft^3

As shown the retention volume is 56,321 cubic feet. This volume must drain within the 36 hours. If the retention volume had been less than the LID required retention volume then the conservative LID required retention volume would have been used. Alternative iterations required use of the LID required volume. As seen the time of concentration is just over 3 minutes which is negligible during the 6 hour storm period so the inflow time is effectively 6 hours. The drain time of the retention volume must be within 36 hours. The drain time can be calculated using Equation 4.2 from the Maricopa Stormwater Design Manual.

(eq. 4.3) $$T_d = \frac{V}{A_p \frac{P_d}{12}}$$

where Td= Calculated Retention Basin Drain Time (hr)
Ap = Percolation Area (Bottom Basin) (acres)
Pd = Design Percolation Rate from Percolation Test (in/hr)
V = Retention Basin Design Storage Volume (acre-feet)

The results of this check is shown in Table 4.4.

Table 4.4: Drain Time Check

System Drain Volume and Time		
Drywell Drain Volume in 36 hours	19061	ft^3
Retention Drain Volume in 36 hours	37,560	ft^3
Total Design Drain Volume in 36 hours	56621	ft^3
Drain Volume ≥ required retention volume?	TRUE	
System Drain Time	**35.81**	**hr**

Final Retention Basin dimensions are determined including basin water depth for design volume and required basin vertical depth including one foot of freeboard. This is shown in Table 4.5.

Table 4.5: Design Retention Basin Dimensions

Final Retention Basin Dimensions		
Freeboard	1.00	ft
Retention Basin Length	170	ft
Retention Basin Width	170	ft
Retention Basin Height + Freeboard	3.00	ft
Full Retention Basin Water Depth	2.00	ft
Retention Basin Volume	56321	ft^3

Other considerations for this design are the maximum water depth of the 100 year storm which is three feet and a side slope no steeper than 4:1[9]. The depth of the retention basin is the water depth plus freeboard which is to 3 feet with a side slope of 4:1. A standard settling basin was added to the drywell in order to capture sediment before it enters the drywell. This will improve the performance and longevity of the drywell. The details of the design drywell and retention basin is shown in Figures 4.2, 4.3 and 4.4.

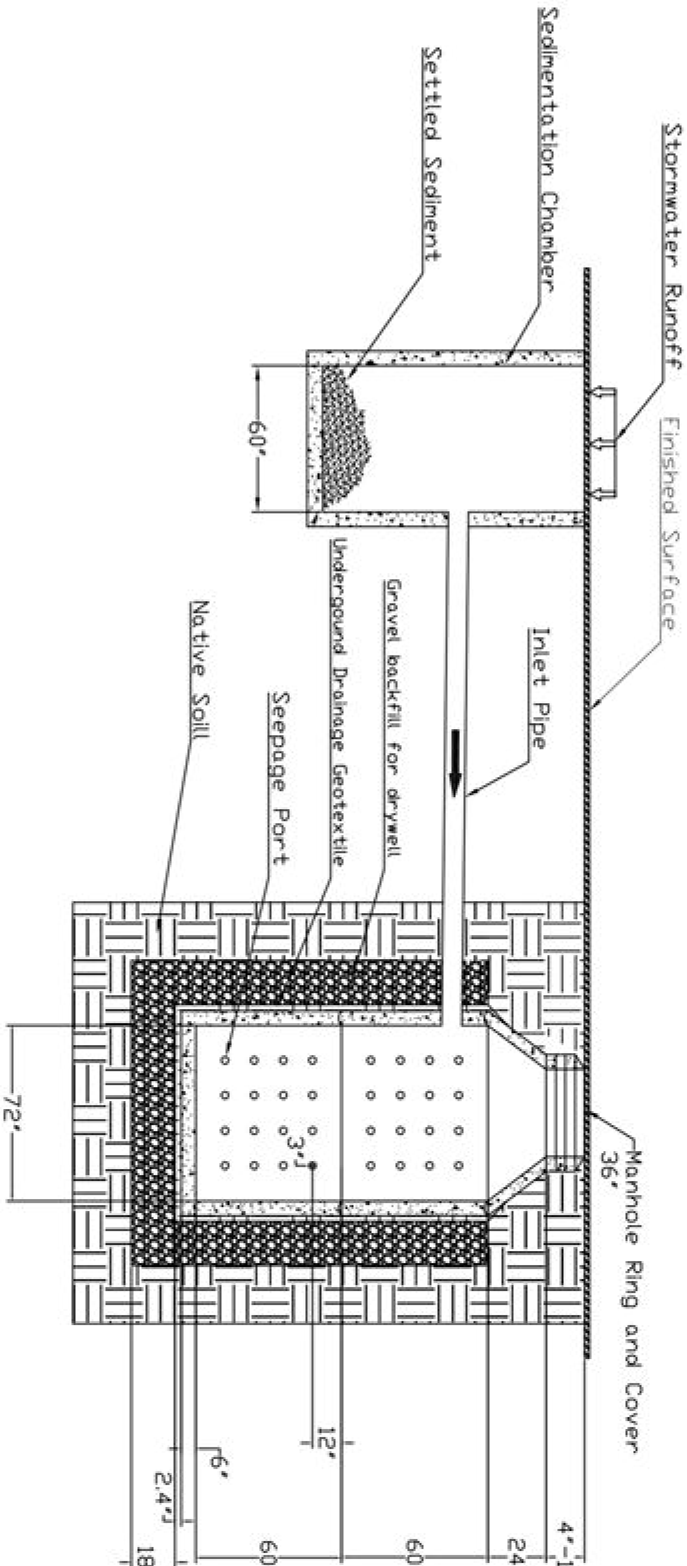

Figure 4.2: Drywell Cross Sectional Detail

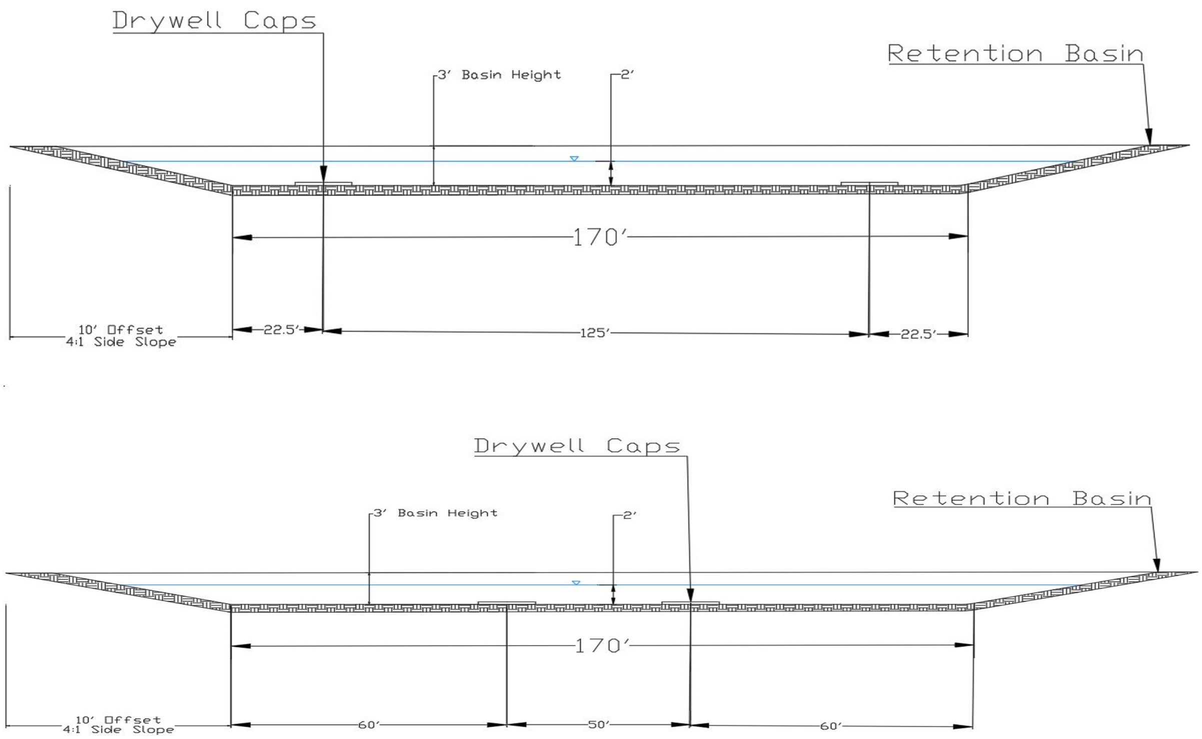

Figure 4.3: Retention Basin Cross Section Detail

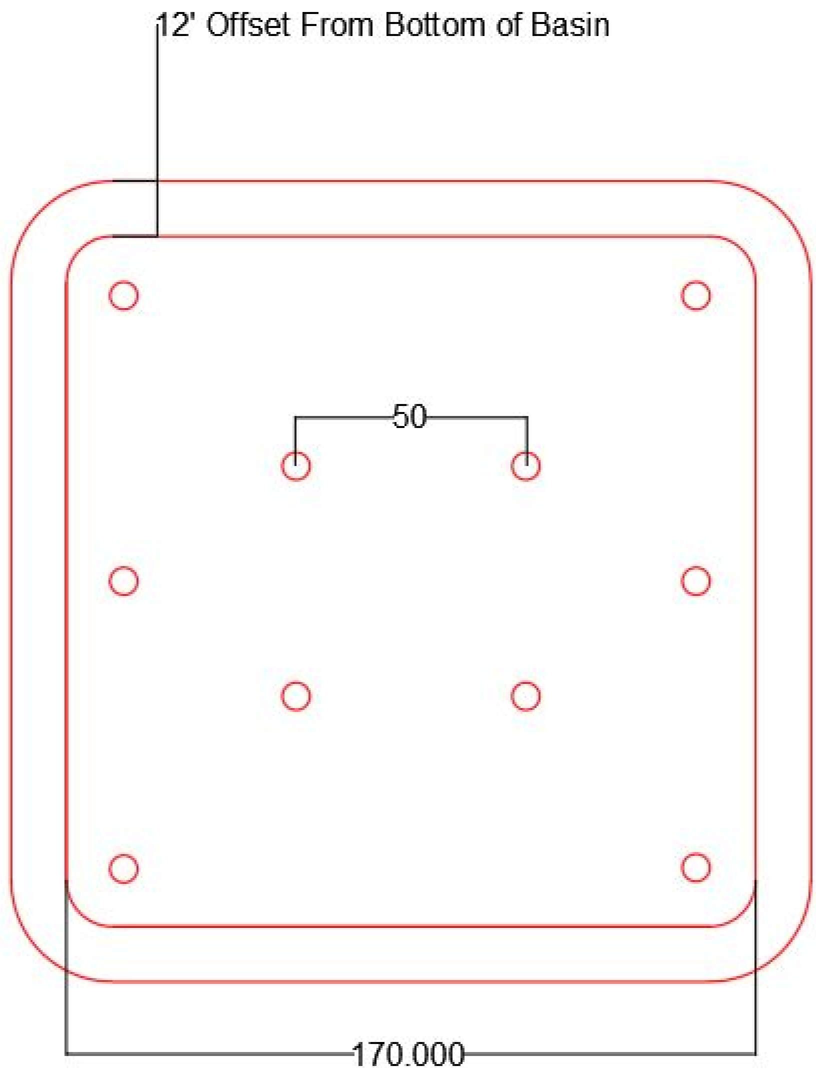

Figure 4.4: Retention Basin Plan View

The bottom of the retention basin will be 170X170 ft with a vertical depth to bottom of 3 feet, side slopes of 4:1. Two typical cross sections will be used as shown in Figure 4.3 above which will result in the 10 drywells spaced at least 50 feet center to center.

4.2 Design Off Site Flow Routing

Since the street capacity of King Street is greater than the offsite flow, there should not be any offsite flow onto King Street, but since the north east corner of the lot is low enough to convey the flow onto the site, an offsite flow routing system must be considered. Concerning the offsite flow from King Street to the CAL Ranch lot, two options were considered for routing. The first option considered was to raise the elevation of the north east section of the lot by about one foot in order to raise the required head needed for the offsite flow to encroach on the CAL Ranch site with an offsite flow of 75 cfs. The second option considered is a trapezoidal swale with a 20 foot top width, 3:1 side slopes, 3 foot max depth and a 2 foot bottom width with slope matching the existing grade, an inlet at the northeast corner of the lot and an outlet at the southeast corner of the lot. These dimensions will be able to contain the entire 75 cfs of the 100 year 6 hour storm for the offsite flow. The outlet would drain into the existing drainage. The more economical option will be used but the preferred off site flow routing system would be the northeast corner elevation adjustment in order to use less developable space.

To raise the northeast corner of the lot by one foot, 218 cubic yards of fill will be required. The typical price to move 218 cubic yards of soil is $50 per cubic yard. 218 cubic yards of fill at $50 per cubic yard yields a cost of $10,900.

To construct a swale with dimensions previously outlined, it will cost $20 per linear foot. Figure 4.5 shows that the necessary length of the swale will be just over 500 ft. at $20 per linear foot and a length of 500 ft the total cost of the swale will be $11,000.

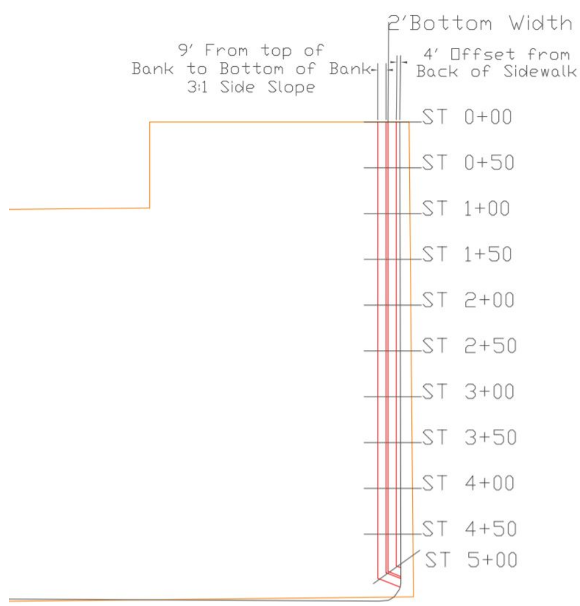

Figure 4.5: Plan View of Potential Swale

The raw prices between the two possible off site water routing methods are only about $1,000 different with the slightly less expensive option being to raise the northeast corner of the lot. Considering the overall redevelopment of the lot, the 218 cubic yards of fill will be negated by excavation on the rest of the lot in order to create a smoother slope directly into the retention basin. The swale will have an increased overall price since it is taking up more of the usable redevelopment space. Since the price of the preferred option is less expensive than the alternative, the preferred option will be used. This will conserve space as well as money. It is concluded that raising the northeast corner of the lot is the most economical and practical way to route off site stormwater. Figure 4.6 shows the redeveloped topography of the site with the raising of the northeast corner. This will also be the final plan view design of the redevelopment of the site.

Figure 4.6: Plan of Topography, Retention Basin, and Drywells in Relation to Existing Parcel

The proposed grade will match the existing grade around the borders of the lot, but the topography in the lot will be more uniform in draining into the retention basin instead of pooling in certain areas of the parking lot. The earthwork of this grade will have a net cut of 21,744 cubic yards. This translates to about $1 million in redevelopment earthwork but may vary depending on the placement of buildings, other structures, or financial capability.

4.3 Maintenance

Maintenance of the drainage systems follow ADEQ guidelines for drywell maintenance; Inspection of the swale and basin needs to be performed yearly by the property owner [2]. The inspection of the drainage system is accompanied by the elimination of deposited silt and sediments found in the drywell pretreatment sedimentation chambers upon inspection. It is also important that the maintenance process includes the cleaning of the screen which is the inlet that connects the sedimentation chamber with the drywell, removal and replacement of sediments, and hazardous waste containment systems such as grease and oil from the cars. The eliminated materials should be disposed of properly in accordance with landfill or hazardous waste standards. Most importantly, the records regarding sediment disposal, basin, and drywell maintenance need to be recorded and up to date by the property owner or local government maintenance division.

4.4 Determination of Alternative Detention Basin Dimensions

To compare the cost of the proposed drywell/retention basin to a traditional detention basin, the required detention volume was computed. Per the COF SWMDM, detention basins must discharge no more than the pre-development peak runoff with incremental stage-storage outlets for the 2, 10, and 100 year storms. The required detention volume was determined using the pre-development runoff for the 100 year 6 hour storm as the allowable outflow rate and post-development runoff for the 100 year 6 hour storm as the inflow rate, computed using the rational method (equation 2.1) and the triangular hydrograph method (equation 4.2). The required detention volume was 51,435. Table 4.6 below shows the input parameters and calculation result.

Table 4.6: Detention Volume

Rational Method Pre/Post Peak Runoff and Required Detention Volume		
Antecedent Precipitation Factor, Cf	1.25	
Pre-Development Weighted Runoff Cofficeint, C	0.15	woods, clay soils
Post-Development Weighted Runoff Cofficeint, C	0.95	asphalt, concrete
6-hour 100-year Rainfall Intensity, I	0.503	in/hr
Area, A (w easement)	9.39	acres
Pre-Development Peak Runoff, Qo	0.89	cfs
Post-Development Peak Runoff, Qi	5.61	cfs
Inflow Time (Tc + 6 hours)	6.053	hours
Detention Volume	**51,435**	**ft^3**

5.0 Cost Analysis

5.1 Comparison of Costs of Drywell/Retention System and Other Stormwater Management Systems

The average cost of excavation including labor and supplies is $0.75 per cubic foot or $20.25 per cubic yard [12]. To prevent erosion and reduce future maintenance costs, it is recommended that the retention basin be lined with grass. Grass materials and labor costs $4.69 per square yard. Due to the lack of local drywell manufacturers, the cost of material and labor per drywell is estimated at $4951.00 per well, including the sedimentation chamber [12] [13] [14]. The total cost of construction of the proposed design for 10 drywells with a 170’ x 170’ basin is estimated at $114.209. Table 5.1 below outlines the cost estimate for the optimal drywell/retention design alternative.

Table 5.1: Retention Basin and Drywell Construction Cost Estimate

Excavation Volume	56321	cubic feet
Excavation Cost	$0.75	per cubic foot
Total excavation cost	*$42,240.75*	
Grass + Instillation Cost	$4.69	per square yard
Total grass cost	*$22,457.80*	
Total Retention Basin Constuction Cost		*$64,698.55*
# of Drywells	10	drywells
Cost per concrete drywell + Instillation	$4,951	per drywell
Total Drywell Construction Cost		*$49,510*
Total Retention Basin and Drywell Constuction Cost		***$114,209***

Intensive maintenance of the retention basin and drywells is estimated to be $1000.00 per year, and includes weed control, fertilization, and mowing [15]. Because the entire site will be covered in concrete and asphalt, it is not expected that large amounts of sediment will enter the basin, although it is recommended that the property manager or other designated personnel inspect the retention basin, drywells, and sediment settling basins for excess sediment and debris accumulation. With 0.989 acres of grass area encompassing the basin bottom and side slopes, the estimated annual maintenance cost for a 40,000 square foot bottom retention basin with 4:1 side slopes is $989.00 per year. Table 5.2 below outlines the annual cost of maintenance for the retention basin.

Table 5.2: Retention Basin Annual Maintenance Cost Estimate

Basin Grass Area	0.989	acres
Intensive Annual Maintenance	$1,000.00	per acre
Annual Maintenance Cost		***$989.00***

5.2 Value of Land Saved Through Implementation of Drywells

In order to achieve acceptable drainage rates through the retention basin without any drywells, an additional 15,200 square feet would be required to meet storage volume and discharge rate requirements. Three undeveloped plots of land were discovered within one mile of the CAL Ranch site. The average cost per square foot for these three example sites was $666,043 per acre. [16]. In order to achieve enough infiltration to drain in 36 hours, a retention basin with no drywells would need to be 210'x210'. The value of land saved (0.349 acres) through drywell implementation was estimated to be $76,441,12. Table 5.3 below outlines the cost of land for each example site and the determination of the value of land saved through drywell implementation.

Table 5.3: Value of Land Saved Through Drywells

	Area (sq. ft)	Area (acres)	Price ($)	Price per acre
Example Site 1	9147	0.2100	135,000	$642,899.31
Example Site 2	46173.6	1.0600	650,000	$613,207.55
Example Site 3	39639.6	0.9100	675,000	$741,758.24
Average cost per acre of undeveloped land				$665,955.03
Land saved through drywell implementation (acres)				0.349
Cost of land saved through drywell implementation				***$232,381.00***

5.3 Comparison of Costs of Various Off-Site Stormwater Management Facilities

While the average construction cost of a retention facility is $0.75 per cubic foot, the average construction cost of a detention facility is about $0.30 per cubic foot [12]. The estimated cost of a reinforced concrete outlet structure including labor is $20,000.00 [13]. It is recommended that the detention outlet be lined with 1 ft high by 4 square yards of rip-rap at both the entrance and exit, which costs an average of $22.22 per cubic yard [13]. The average cost of a detention facility is estimated at $35,489, resulting in a savings of $232.831 over the retention basin/drywell system. Table 5.4 below outlines the estimated cost of a detention facility.

Table 5.4: Detention Facility Construction Cost Estimate

Excavation Volume	51,435	cubic feet
Excavation Cost	$0.30	per cubic foot
Reinforeced Concrete Outlet	$20,000.00	(including labor)
Riprap Cost	$22.22	per cu. yard
Riprap Area	2.67	cu. yards
Total Cost of Detention Basin		***$35,489.83***
Savings over retention/drywell system		***$78,718.73***

6.0 Feasibility Determination

Based upon the estimated cost of construction, it is not economically feasible to construct a drywell and retention basin stormwater management facility at the CAL Ranch site. The primary reason for this is that the low infiltration rates require a large storage volume and retention area/number of drywells to achieve a drainage time less than 36 hours, as required in the City of Flagstaff Drainage Design Manual. Because this part of the City does not rely on groundwater as a water source, groundwater recharge through drywells is not a priority. Although drywell use at this specific site is deemed infeasible, one might consider the use of drywells in an area with higher infiltration rates, such as the Cinder Hills area east of town. It is recommended that more studies be done in this area and others to determine if drywell implementation is feasible at these locations. It may be more feasible for detention basins to be used in tandem with drywells rather than retention basins. The detention basin would have smaller dimensions than the retention basin because it allows flow to leave the basin, and the dimensions would be reduced by a drywell. The significance of the dimension reduction caused by drywells in a detention basin is not known at this time but it is recommended that studies be done to determine the significance.

7.0 Impact Assessment

One of the most effective tools in stormwater drainage aquifer recharge, drywells have been in use for more than a century now. Starting as simple holes drilled in the ground and filled with rocks, drywells have evolved over the last century with many improvements being made to the design to allow faster and effective stormwater drainage. While the majority of drywells in Arizona have been constructed in Phoenix and its surrounding areas, legislation has encouraged people in the state to adopt drywells for real estate development. In this context, this discussion will explore the economical, social, and environmental impacts of the drywells located in Flagstaff.

7.1 Economic Impact Assessment

The site developer is responsible for the construction and maintenance cost of a stormwater management system that is in compliance with all City of Flagstaff standards and Arizona Department of Environmental Quality standards. Section 5.0 above outlines the costs of a drywell/retention system and a traditional detention system.

Proper design, installation, and maintenance of the drywell and retention basin is imperative to avoid flooding which can lead to motor vehicle crashes and property damages. Car crashes with no fatalities can cost from $7,000 to $60,000 so any crashes that are a result of the retention basin and drywells will add to the cost of construction, installation, and maintenance. Fatal accidents can cost over $1 million. If there was a failure, an additional cost for redesigning and reconstruction another stormwater management facility would be added in order to maintain compliance with local stormwater standards. Based on the general topography, the area that has the highest risk of flooding would be the North side of 7th Ave and the intersection of 7th Ave and King St. 7th Ave typically has the higher volume of traffic so it would have a greater chance of causing an accident if flooded. It is important to install, construct, and maintain the drywells and retention basin correctly in order to avoid thousands of extra dollars from auto crashes due to flooding of a failed retention basin.

7.2 Social Impact Assessment

Unless otherwise improved, typical retention and detention basins do not increase the social/aesthetic value of a site. In areas where groundwater recharge is a priority, drywells would be encouraged by the general public. Concerns regarding the pollution of the groundwater may be a concern of the general public in areas that produce hazardous waste. This can be accounted for with proper design and needs to be portrayed to the public. Many people throughout Arizona get their freshwater from groundwater so there may be concern with water quality so proper installation and design of drywell is necessary to maintain a positive public perception. In the case of the CAL Ranch site, groundwater recharge is not a primary concern, and so a drywell/retention system for stormwater management is expected to have a neutral impact on the social acceptance of this type of stormwater management system.

In some areas of town perched aquifers provide freshwater to residents through wells. In areas where perched aquifers are located and residents use that water, precautions must be taken in order to avoid contamination of residents groundwater. Even when proper precautions are taken, there may still be opposition to the use of drywells in those areas and with the implementation of the drywells there may be unhappy residents.

If proper maintenance of the drywells and retention basin is not performed, there is potential for flooding. This could cause complaints from the public and could lead to accidents which may lead to injury. It is imperative that the drywell and retention basin be kept in proper condition to properly manage the stormwater and avoid flooding, motor vehicle crashes and property damages .

7.3 Environmental Impact Assessment

Due to the large depth to the groundwater table, and the CAL Ranch site being zoned for commercial development, rather than chemical/industrial, there is a very low chance for negative environmental impact of drywell implementation at the CAL Ranch site. Due to the depth to the aquifer at the site and the commercial development an Aquifer Protection Permit is not required at this site. Throughout Flagstaff, however, there are perched aquifers at various locations which reduces the distance between the surface and the aquifer. Aquifer Protection Permits would likely be required at these locations. These perched aquifers are at a greater risk of becoming contaminated due to the proximity of the aquifer to the surface where potential hazards are located. In the locations of these aquifers, residents use wells to pump tap water from the aquifer. Just south of Little America along Herold Ranch Rd is one such area. If drywells are utilized on or upstream of these perched aquifers greater caution or exclusion of drywells will need to be considered. As long as maintenance is performed and design infiltration rates are achieved, there will be little chance of a water quality hazard in the aquifer as a result of long-term surface water retention.

8.0 Summary of Engineering Work

Table 8.1 below shows the proposed hours for each task and the actual hours spent working on each task. As shown, the team estimated that the project would take 1107 hours, although it only ended up taking 310 hours. Of those 310 hours, roughly half were allocated to the project manager, and a third to the project engineer.

Table 8.1: Summary of Engineering Work

Tasks	Proposed Hours	Actual Hours
Task 1: Hydrologic/Hydraulic Analysis	194	74
Task 2: Geotechnical Analysis	97	34
Task 3: Retention Basin and Drywell Design	102	32
Task 4: Cost Analysis	114	18
Task 5: Feasibility Determination	176	15
Task 6: Impact Assessment	48	23.5
Task 7: Project Deliverables	160	66
Task 8: Project Management	216	47.5
Total	*1107*	*310*
Team Roles	**Proposed Hours**	**Actual Hours**
Senior Engineer	183	50
Project Manager	320	153
Project Engineer	604	107
Total	*1107*	*310*

9.0 Summary of Engineering Costs

Table 8.2 below outlines the total engineering costs based on the proposed and actual hours worked. The proposed cost was $112,567, and the actual cost for all hours worked was $33,469.

Table 8.2: Summary of Engineering Costs

		Proposed Hours & Cost		Actual Hours & Cost	
Team Role	**Rate, $/hr**	**Hours**	**Cost**	**Hours**	**Cost**
Senior Engineer	168	183	$30,824	50	$8,400
Project Manager	110	320	$35,235	153	$16,830
Project Engineer	77	604	$46,508	107	$8,239
		Proposed Total	**$112,567**	**Actual Total**	**$33,469**

10.0 Conclusions

From the hydrologic analysis, it was found that the peak on-site runoff inflow was 5.61 cfs. The discharge through infiltration from the retention basin and drywells given a maximum drain time of 36 hours was 0.437 cfs. Due to the low infiltration discharge relative to the runoff inflow, the optimal retention/drywell system yielded a retention volume of 1.3 acre-feet (56,321 cubic feet) and 10 drywells with a radius of 3 feet and depth of 10 feet. The estimated cost for this retention/drywell system was $114,209 The estimated cost for a detention-only system was estimated to be $35,490, resulting in a savings of $78,719. Due to the high cost of the drywell/retention system, and lack of need for groundwater recharge in the area, it is concluded that a drywell stormwater management system is not feasible at the CAL Ranch site.

11.0 References

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Appendices

Appendix A: TR-55 Curve Numbers

Table A.1: TR-55 Curve Numbers [5]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Pasture, grassland, or range—continuous forage for grazing. 2/	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. 3/	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 4/	48	65	73
Woods—grass combination (orchard or tree farm). 5/	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. 6/	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 4/	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

Appendix B: Design Iterations

Table B.1: Design Iteration 1 Depth of 3ft

Retention Basin Dimensions and System Discharge		
Single Drywell Discharge Rate	0.01471	cfs/drywell
Design Retention Basin Infiltraton Rate	0.00001	ft/s/unit area
Retention Basin Disharge Rate	0.20026	cfs
Single Drywell Discharge Volume / 36 hours	1,906.13	ft^3
Number of Drywells	8	
Retention Basin Length	200	ft
Retention Basin Width	100	ft
Retention Basin Area	19773.80533	ft^2
Time to drain	36	hours

System Drain Volume and Time		
Drywell Drain Volume in 36 hours	15249	ft^3
Retention Drain Volume in 36 hours	25,953	ft^3
Total Design Drain Volume in 36 hours	41202	ft^3
Drain Volume ≥ required retention volume?	FALSE	
System Drain Time	**50.34**	**hr**

Final Retention Basin Dimensions		
Freeboard	1.00	ft
Retention Basin Length	200	ft
Retention Basin Width	100	ft
Retention Basin Height + Freeboard	3.881	ft
Full Retention Basin Water Depth	2.881	ft
Retention Basin Volume	57617	ft^3

Result: Dimensions could retain volume but the basin and drywells could not drain within 36 hours.

Table B.2:Design Iteration 2 Elongated Basin

Retention Basin Dimensions and System Discharge		
Single Drywell Discharge Rate	0.01471	cfs/drywell
Design Retention Basin Infiltraton Rate	0.00001	ft/s/unit area
Retention Basin Disharge Rate	0.40051	cfs
Single Drywell Discharge Volume / 36 hours	1,906.13	ft^3
Number of Drywells	16	
Retention Basin Length	400	ft
Retention Basin Width	100	ft
Retention Basin Area	39547.61066	ft^2
Time to drain	36	hours

System Drain Volume and Time		
Drywell Drain Volume in 36 hours	30498	ft^3
Retention Drain Volume in 36 hours	51,906	ft^3
Total Design Drain Volume in 36 hours	82404	ft^3
Drain Volume ≥ required retention volume?	TRUE	
System Drain Time	**23.66**	**hr**

Final Retention Basin Dimensions		
Freeboard	1.00	ft
Retention Basin Length	400	ft
Retention Basin Width	100	ft
Retention Basin Height + Freeboard	2.354	ft
Full Retention Basin Water Depth	1.354	ft
Retention Basin Volume	54153	ft^3

Results: Basin and drywell combination could retain the required volume and drain in the required time while maintaining the 3 foot depth maximum. The problem with this iteration is that it is so long and crosses multiple contour lines. The cut for this iteration is less than optimal and it would be more efficient in this case to have a more square basin that did not cross as many contour lines.

Table B.3: Design Iteration 3, No Drywells, Large Retention Basin

Retention Basin Dimensions and System Discharge		
Single Drywell Discharge Rate	0.01471	cfs/drywell
Design Retention Basin Infiltraton Rate	0.00001	ft/s/unit area
Retention Basin Disharge Rate	0.45573	cfs
Single Drywell Discharge Volume / 36 hours	1,906.13	ft^3
Number of Drywells	0	
Retention Basin Length	225	ft
Retention Basin Width	200	ft
Retention Basin Area	45000	ft^2
Time to drain	36	hours

System Drain Volume and Time		
Drywell Drain Volume in 36 hours	0	ft^3
Retention Drain Volume in 36 hours	59,063	ft^3
Total Design Drain Volume in 36 hours	59063	ft^3
Drain Volume ≥ required retention volume?	TRUE	
System Drain Time	**34.20**	**hr**

Final Retention Basin Dimensions		
Freeboard	1.00	ft
Retention Basin Length	225	ft
Retention Basin Width	200	ft
Retention Basin Height + Freeboard	2.247	ft
Full Retention Basin Water Depth	1.247	ft
Retention Basin Volume	56116	ft^3

Results: Larger Retention Basin meets drainage volume and storage time requirements, although cost of land lost through larger basin offsets any advantage of larger retention basin.

Table B.4: Design Iteration 4, 20 Drywells, Small Retention Basin

Retention Basin Dimensions and System Discharge		
Single Drywell Discharge Rate	0.01471	cfs/drywell
Design Retention Basin Infiltraton Rate	0.00001	ft/s/unit area
Retention Basin Disharge Rate	0.22214	cfs
Single Drywell Discharge Volume / 36 hours	1,906.13	ft^3
Number of Drywells	20	
Retention Basin Length	150	ft
Retention Basin Width	150	ft
Retention Basin Area	21934.51332	ft^2
Time to drain	36	hours

System Drain Volume and Time		
Drywell Drain Volume in 36 hours	38123	ft^3
Retention Drain Volume in 36 hours	28,789	ft^3
Total Design Drain Volume in 36 hours	66912	ft^3
Drain Volume ≥ required retention volume?	TRUE	
System Drain Time	**29.84**	**hr**

Final Retention Basin Dimensions		
Freeboard	1.00	ft
Retention Basin Length	150	ft
Retention Basin Width	150	ft
Retention Basin Height + Freeboard	3.465	ft
Full Retention Basin Water Depth	2.465	ft
Retention Basin Volume	55456	ft^3

Result: Increasing the number of drywells and decreasing the size of the retention basin meets drainage volume and storage time requirements, although minimum spacing requirements for drywells are exceeded unless the retention basin is largely oversized, resulting in inefficient use of land and loss land value, as well as an increased cost for construction of a greater number of drywells.