Williams + Paddon Architects + Planners Inc.
DRAFT Master Plan Update – Wet Utility Infrastructure

PLACER COUNTY GOVERNMENT CENTER MASTER DRAINAGE REPORT

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DRAFT

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CARTWRIGHT

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Executive Summary

A master drainage report was prepared to support the wet utility infrastructure component of the Placer County Government Center (PCGC) Master Plan Update by providing a comprehensive analysis of the existing and anticipated future storm drain systems. No previous master drainage report has been prepared for PCGC, and thus far stormwater planning has occurred in a piecemeal fashion without overarching guidance on integrating individual projects into a cohesive stormwater system. A coupled hydrologic-hydraulic model was developed with the XP-STORM platform for the entire 200-acre PCGC to (1) gain an understanding of shortcomings in the existing stormwater infrastructure, and (2) guide stormwater planning according to state and local requirements as the campus evolves toward the ultimate buildout condition.

A baseline (existing conditions) model was developed based on site-specific survey data, engineering plans, and drainage reports, and later supplemented by a focused field investigation to resolve data gaps. The model was validated against stormwater studies from previous projects; in most instances the studies agreed with the baseline model results, and discrepancies that occurred could be explained by land use changes or the higher-resolution data used for the master drainage report. Five outfall locations (where stormwater leaves PCGC) were identified to establish limits for stormwater flow rates under the buildout condition. The baseline model highlighted several shortcomings in the existing stormwater system, including many undersized pipes in the southeast portion of the campus, roadway overtopping at Atwood Drive, shallow flooding near the Finance Administration Building, and insufficient freeboard in detention basins.

The findings from the baseline model were used to inform the alternatives analysis, and eventually identify a land plan for the Final Option. The stormwater management approach for the Final Option is to meet flood control and hydromodification management requirements with several detention basins strategically placed throughout the campus to provide flow controls for the entire PCGC, and leave meeting water-quality treatment requirements to individual projects as they come online.

A total of seven new stormwater basins and reconfiguration of three existing basins are needed to meet County requirements for flood control and hydromodification management. Several new pipes and upsizing of existing pipes in select locations are needed to convey the anticipated future flow rates. The model showed that the stormwater basins are effective in limiting peak flow rates at each of the five outfalls from PCGC to be no greater than the existing peak flow rates. Individual projects will incorporate water-quality features according to the West Placer Storm Water Quality Design Manual, and will include site design, source control, and storm water treatment features. Stormwater treatment features such as bioretention basins typically provide some level of flow control, and will likely reduce flow rates beyond the post-construction flow rates estimated by the XPSTORM model.

1 INTRODUCTION

This report supports the wet utility infrastructure component of the Placer County Government Center (PCGC) Master Plan Update by providing a comprehensive analysis of the existing and anticipated future storm drain systems. The Master Plan Update will build on the Comprehensive Facilities Master Plan for Placer County (1993) by providing a long-term vision and ongoing facilities planning with a 20-year horizon. As part of this effort, all utilities are being evaluated to ensure they are sufficient to support the proposed improvements. In the case of the storm drain system, the primary analysis tool for the 200-acre PCGC site is a coupled hydrologic-hydraulic model. This sub-regional scale model was developed to provide an integral understanding of the current storm drain system, and to facilitate comprehensive stormwater planning for the entire project site.

1.1 OBJECTIVES AND PURPOSE

The purpose of this report is to supplement the overall wet utility infrastructure study by Cartwright Engineers (in progress). The objectives of the Cartwright study are to support the Master Plan Update by providing a comprehensive overview of the wet utilities, encompassing the water, sewer and stormwater systems, through (1) collection of existing wet utilities information, and (2) assessment of the existing utility systems and the future utility system improvements needed for the planned PCGC buildout condition. The former objective has been addressed by Cartwright and is not discussed in detail herein. Nonetheless, to assist in preparing this report, a thorough understanding of the existing storm drain system was gained through a review of background information along with field investigations. The main purpose of this report is to address the second study objective through the following work flow:

1. Develop a coupled hydrologic-hydraulic model of baseline (i.e. existing) conditions for PCGC and its storm drain system;
2. Review and validate the model to be consistent with previously completed stormwater studies;
3. Present the baseline model results with particular emphasis on undersized portions of the storm drain system that have the potential to cause flooding;
4. Model multiple alternatives to inform the planning team of storm drain system considerations for each alternative, and to guide the selection of a preferred alternative;
5. Present the necessary storm drain system improvements for the preferred alternative that addresses site-specific constraints and meet state and local regulatory requirements; and
6. Develop an overall strategy and recommendations for a Storm Water and Drainage Management Plan for the Placer County Government Center Campus Master Plan that addresses the drainage planning elements necessary to mitigate storm water impacts including conveyance, storage, general recommendations for project specific water-quality treatment, and hydromodification controls.

1.2 DESIGN STANDARDS AND CRITERIA

Design Standards applied to the formulation of this study include:

- Placer County Flood Control and Water Conservation District Stormwater Management Manual

(September 1994);

- Placer County Land Development Manual (October 1996);
- Waste Discharge Requirements (WDRs) for Storm Water Discharges from Small Municipal Separate Storm Sewer Systems (MS4s; State Water Resources Control Board Water Quality Order No. 2013-001-DWQ); and
- West Placer Storm Water Quality Design Manual (April 2016).

2 BASELINE MODEL

The baseline model was generated using the XPSTORM software package (XP Solutions, 2016) to evaluate the existing conditions of the PCGC. A preliminary model was built based on background information on the existing infrastructure, and then was refined with additional information collected during a site visit. The final baseline model was validated against flowrates estimated in previously completed drainage reports and other hydraulic analyses throughout PCGC.

2.1 BACKGROUND INFORMATION

Cartwright Engineers compiled a large amount of background information on the existing stormwater infrastructure within and around the PCGC. The background information consisted of engineering plans, drainage reports, survey information, and additional reports that provided insight into how the stormwater infrastructure for the site functions. The following documents were reviewed to prepare the baseline model:

- City of Auburn Stormwater Management Plan by City of Auburn Department of Public Works (2008);
- Placer County Government Center Storm Drain Plan by West Yost Associates (2010);
- Auburn/Bowman Community Plan Hydrology Study by Placer County Department of Public Works (1992);
- Engineering Plans for Placer County Finance Administration Building: Drainage and Sewer Plan by Morton and Pitalo, Inc (1995);
- Engineering Plans for Placer County Dewitt Center Auburn Justice Center: Utilities by Placer County (2004);
- Draft - Placer County Government Center Master Plan Update - Wet Utilities Infrastructure by Cartwright Engineers (2016, in progress);
- NRCS Hydrologic Soil Report downloaded from the USDA Web Soil Survey (data accessed October 2016);
- Drainage Report, Home Depot Shopping Center, Auburn, Placer County, CA by Blair, Church and Flynn Consulting Engineers (2007);
- Photos and notes from site visit by Cartwright Engineers (August 2016);
- Preliminary Drainage Study for Placer County Auburn Animal Shelter by Wood Rodgers (2013);
- DeWitt Center Stormwater Detention Storage Study for Watersheds A1-A3 by A.R. Associates (2003a);
- DeWitt Center Land Development Building Stormwater Detention Storage Study for Bell Road/Rock Creek Watershed (Shed B1) by A.R. Associates (2003b);
- Placer County Main Jail Housing Unit 4 Stormwater Detention Pond Final Report by A.R. Associates (2000);
- Grading Study, Placer Animal Shelter by Wood Rodgers (2013);

- Draft - Stormwater and Surface Water Quality Best Management Practices (BMP) Plan for Placer County Auburn Animal Shelter by Wood Rodgers (2013);
- Improvement Plans for Olympus Village, Placer County, CA, Engineering Plans by GW Consulting Engineers (2002);
- Improvement Plans Atwood Ranch Unit III, Placer County, CA, Engineering As-Built Plans by A.R. Associates (2008);
- DeWitt Center Site Directory for Utilities, Engineering Plans by County of Placer (1994).
- Administrative Draft Storm Drainage and Water Quality Existing Conditions Report by URS (2002);
- 2012 Monitoring Report for the DeWitt Government Center by Dudek (2012);
- Drainage Study for Timberline at Auburn Phase 1 by Wood Rodgers (2014);
- Willow Creek Shopping Center On-Site Improvement Plans by TSD Engineering, Inc (2015);
- Drainage Report for Sunset Terrace Estates by Western Planning and Engineering (1990);
- Willow Creek Retail Placer County, CA, Preliminary Hydrology and Hydraulics, by TSD Engineering (2014);
- Placer County, CA Flood Insurance Rate Map Number 0601C0275 F by FEMA (Effective Date June 8, 1998);
- Placer County, CA Flood Insurance Rate Map Number 0601C0288F by FEMA (Effective Date June 8, 1998);
- Improvement Plans Land Development Building Placer County, CA by A.R. Associates (2004);
- Home Depot Utility Engineering As-built Plans by Lars Andersen and Associates (2005);
- Atwood Ranch 1 drainage calculations by A.R. Associates (1988);
- County of Placer DeWitt Center Auburn Justice Center technical drawings by Beverly Prior Architects (2004); and
- Aerial mapping of DeWitt County Center by Andregg Geomatics (May 1, 2015).

2.2 HYDROLOGIC SETTING

The PCGC spans a topographic high point, with the western portion of the site (roughly 80% of the total area) draining south into the North Auburn Ravine watershed (Catchments 1, 2, 3, and 6) and the eastern portion draining north into the Rock Creek Watershed (Catchments 4 and 5). The project site was separated into six main catchments based on the site topography and the locations of outfalls where stormwater leaves the site. Each catchment was then divided into sub-catchments based on the locations of storm drain inlets and storage basins. The six catchments were divided into a total of 37 sub-catchments for a total tributary area of 231 acres (Plate 2).

Mean annual precipitation at the PCGC is on the order of 36 inches. Precipitation is almost entirely rainfall, with the winter months typically being the wettest time of year, although intense rainfall during summer thunderstorms is also common.

The following paragraphs provide a description of the individual catchments and the general layout of the storm drain network within each catchment. See Plate 1 for a general map of PCGC and the names of nodes and links used in the XPSTORM baseline model. Catchments and subcatchments are shown in Plate 2.

- Catchment 1. Catchment 1 (C1) is located on the western edge of the PCGC and is bounded by Bell Road and the Combie Canal to the north and Atwood Drive to the south. C1 has an area of 88.5 acres and drains to the North Auburn Ravine watershed. Subcatchment 1J is the highest portion of C1 and includes the northern portion of the Community Development Resources Center. Flows from 1J are detained in a 0.3-acre-foot detention basin (1J/Storage1) at the southeast corner of the intersection of Bell Road and Richardson Drive. Flows leaving the detention basin flow west along Bell Road in a combination of pipes and open channels to the intersection of Bell Road and Olympic Way. Flow crosses Olympic Way through a 24-inch culvert and drains southwest in a natural channel. The natural channel continues until it enters a 0.05-acre-foot on-line detention basin (Node 1I/Storage1) located to the west of the Olympic Residential Development. The outflow from the detention basin is controlled by an 18-inch pipe and overflow spillway. The flow continues down the natural channel toward B Avenue. On the upstream side of B Avenue a small amount of flow ponds (Node 1E/Storage2) before entering three parallel box culverts (each 5.2-feet wide by 3.7-feet high) under B Avenue. Runoff from Sub-catchments 1F and 1G also flows into Node E1/Storage upstream of B Avenue. Flow passing under B Avenue then enters the large southwest pond (Node 1B/Storage1, capacity 13.4 acre-feet). Runoff from sub-catchment 1D is collected by a series of inlets along B Avenue and piped through an 18-inch pipe into the northeastern end of this pond. Flows from the Animal Services Center (Subcatchment 1C) also flow into Node 1B/Storage1 on the southern end, after being collected and detained by a 0.33-acre-foot detention basin (Node 1C/Storage1). Flows out of Node 1B/Storage are controlled by a weir box at the southern end and flow through a 48-inch diameter pipe to a natural channel. Flow is constricted at an old 6-foot wide concrete structure represented by Link 122. The southernmost storage (Node 1A/Storage1) is a natural depression, created from the natural topography and the Atwood Drive road embankment. The outfall from C1 is a 48-inch culvert under Atwood Drive with open channel downstream.
- Catchment 2. Catchment 2 (C2) has an area of 41.4 acres. C2 spans the central portion of the PCGC and drains to the south towards the North Auburn Ravine watershed. The upstream extent of C2 is the southern portion of the Community Development Resources Center, Finance Administration Building, Auburn Justice Center, and associated parking lots. A series of inlets collects runoff from Subcatchments 2B through 2F and conveys flow under Catchment 3 (the Jail complex) and to a 3-acre-foot detention basin (Node 2A/Storage1) located to the west of Jail House #4. Outflow from the detention basin is metered by an outlet control structure (Node 2A Control Structure: a combination orifice, v-notch weir and overflow weir); this feature was modeled in XPSTORM by a stage-discharge table. Flow leaves through a 42-inch pipe to a natural channel and storage pond (Node 2A/Storage2) located just north of Atwood Road. Flow from Catchment 3, and overflow from Catchment 6 combine at the Node 2A/Storage2 pond. Two culverts (one 30- and one 15-inch pipe) convey flows under Atwood Drive to an existing pond south of Atwood Road (not modeled), with the inflow to the pond designated as Outfall C2/C3.
- Catchment 3. Catchment 3 (C3) has an area of 12.8 acres located in the south-central portion of the PCGC and drains to the North Auburn Ravine watershed. Catchment 3 drains the Jail and

Juvenile Detention Center. Runoff is collected by the parking lots and drains through a 42-inch diameter pipe to the southern end of the Jail Complex and into the natural pond (Node 2A/Storage2).

- Catchment 4. Catchment 4 (C4) has an area of 12.7 acres located in the northeast corner of the PCGC and drains to the Rock Creek watershed. The Ophir Canal traverses C4, but is not part of the storm drain system and only receives direct precipitation. C4 collects runoff from the Health and Human Services buildings and parts of 1st Street, with flow directed beneath the Ophir Canal in a pipe. Flow travels north to an inlet at the eastern boundary of the PCGC, then is conveyed offsite via a 24-inch pipe directed east toward Professional Drive.
- Catchment 5. Catchment 5 (C5) has an area of 29.9 acres located on the eastern boundary of the PCGC and drains to the Rock Creek Watershed. C5 includes the Home Depot development and the 1st Street and Professional Drive stormwater basins. Runoff from the southern end of C5 (Subcatchment 5C) drains to the 1.03-acre-foot 1st Street detention basin (Node 5C/Storage1) where the outflow is controlled by an orifice outlet. Outflow from the 1st Street detention basin combines with runoff from Subcatchment 5E and is piped along Willow Creek Drive and under the Home Depot parking lot. Runoff from the west end of C5 (Subcatchment 5D) drains to the 2.00-acre-feet Professional Drive detention basin (Node 5D/Storage1) where the outflow is controlled by an orifice outlet. Flow is then piped along the northern edge of Home Depot and routed to the 30-inch outlet pipe (Outlet C5) located at the northeast corner of the C5. On-site runoff from Home Depot is collected and detained in an underground storage facility (Node 5A/Storage1) beneath the parking lot. Flows are controlled by multiple orifices before entering the 30-inch outfall pipe.
- Catchment 6. Catchment 6 (C6) drains the southeastern 45.8-acre portion of the PCGC and drains to the North Auburn Ravine watershed. C6 includes the County Government offices, the Corporation Yard, and the Atwood Ranch 1 development (Subcatchement 6I). The C6 storm drain system appeared to contain some of the oldest storm drain infrastructure of the PCGC. All runoff in C6 drains toward an open channel along the north side of Atwood Drive. Many of the collector storm pipes were not incorporated into the XPSTORM baseline model as they are less than 10 inches in diameter. The model simplified the feeder drain layout by selecting a main point of concentration for each of the subcatchments. Runoff from the northern Subcatchments 6E, 6F, 6G and 6H are piped to a common junction at Richardson Drive. Runoff from Subcatchments 6B, 6C and 6D are piped towards Atwood Drive where they daylight into the open channel along Atwood Drive. Flows then converge at Richardson Drive and flow west under the road through a 36-inch culvert. After the culvert a natural channel routes flow to the west to a junction and culvert along Atwood Drive. A 22-inch culvert goes under Atwood Drive as Outfall C6. At that junction (Node 6A/6) an overflow weir allows flows in excess of the capacity of the 22-inch culvert to overflow to the east along an open channel to the Node 3A/Storage1 pond, and leave the site through Outfall C2/C3

Two irrigation canals traverse the PCGC: the Combie Canal runs along the western boundary and the Ophir Canal runs parallel to 1st Street though the eastern portion of the site. Neither canal is known to be managed as part of the storm drain system, although it is possible they receive small amounts of runoff during extreme storm events.

2.3 BASELINE MODEL DEVELOPMENT

The hydrology and storm drain hydraulics of PCGC were modeled using the XPSTORM software package (XP Solutions, 2016). XPSTORM integrates hydrologic and hydraulic computations into a single model thereby streamlining the modeling process. The model includes multiple hydrologic parameterization methods which allow the rainfall-to-runoff calculations to conform to the prevailing engineering standards. Lastly, XPSTORM is well-suited for modeling complex urban watersheds because it is capable of simulating a variety of features including pipes, manholes, ponds, weirs, and overland flow. The model development described herein is consistent with the methods described in Section V.3 of the Placer County Flood Control and Water Conservation District Stormwater Management Manual (SWMM) for HEC-1 models, which are required for master planning models. XPSTORM has the ability to parameterize a hydrologic model in an identical manner as HEC-1.

Table 1 – Baseline model catchment parameters

Catchment / Subcatchment	Area (acres)	Percent Impervious (%)
Catchment 1		
1A	12.4	3.0
1B	13.4	8.6
1C	5.0	59.5
1D	3.7	37.8
1E	14.8	16.4
1F	10.6	51.2
1G	11.8	67.9
1H	2.9	41.9
1I	7.7	43.2
1J	6.3	46.0
C1 Total	88.5	33.0
Catchment 2		
2A	1.7	8.0
2B	15.9	43.2
2C	3.2	68.1
2D	3.9	55.6
2E	5.7	42.6
2F	7.4	74.2
2G	3.6	52.6
C2 Total	41.4	51.1
Catchment 3		
3A	3.3	36.7
3B	4.7	75.5
3C	4.7	69.2
C3 Total	12.8	63.0

Catchment / Subcatchment	Area (acres)	Percent Impervious (%)
Catchment 4		
4A	2.4	12.0
4B	9.0	39.9
4C	1.3	75.3
C4 Total	12.7	38.3
Catchment 5		
5A	11.2	81.2
5B	0.8	99.2
5C	3.6	57.9
5D	11.6	41.9
5E	2.6	0.1
C5 Total	29.9	56.6
Catchment 6		
6A	11.5	55.7
6B	7.7	76.3
6C	1.4	99.4
6D	5.3	64.7
6E	4.9	67.7
6F	2.4	42.4
6G	2.3	69.9
6H	3.8	90.4
6I	6.5	59.4
C6 Total	45.8	66.2
Total	231.0	47.8

Rainfall depths for the 2-, 10-, and 100-year, 24-hour storms were estimated from the Design Storm Procedures presented in Appendix V-B of the SWMM as 2.78, 4.53, and 6.73 inches, respectively. The depths were adjusted for the average elevation of PCGC, approximately 1,400 feet (NAVD88). Design storm hyetographs were generated for each storm using the depth-duration-frequency coefficients in Appendix V-A of the SWMM.

The runoff routing of the catchments was modeled using the Kinematic wave method for overland flow. The required data for this method includes area (acres), percent impervious, subcatchment width (feet), and slope (feet/feet). Rainfall abstractions were represented as an initial loss of zero and a constant infiltration rate of 0.16 inches/hour.

The geometry of the storm drain system was assimilated into the model based on the background information for pipe sizes, lengths, alignments, materials, and elevations. Manning's roughness values were applied based on the pipe material or assumed to be 0.014 if no material was known. The storm system was simplified for the modeling, and smaller (less than 10-inch) lateral pipes were generally excluded from the model. The sections of the storm drain system with open channel flow were modeled as either an irregular or a trapezoidal channel shape, as appropriate. The roughness for the channels was approximated during the site visits and averaged over the channel length.

In instances where the amount of flow was greater than the capacity of the storm drain system, the baseline model was configured to show temporary surface flooding at the model nodes. When flooding occurred at a node, water was stored above ground at the respective node until there is sufficient hydraulic capacity within the system for it to reenter the network. This method allows areas to be identified where the storm drain system is inadequately sized to convey flood flows and where shallow surface flooding would be expected.

The baseline PCGC model included 13 storage basins which represented the existing stormwater detention basins or natural depressions that attenuate stormwater runoff. Storage in the basins was modeled using stage-storage tables generated from the background information or from survey data provided by Cartwright Engineers.

As discussed previously, PCGC has five outfall locations, with three along Atwood Drive (C1, C2/C3, and C6), one on Professional Drive (C4), and one between Highway 49 and Heritage Oaks Circle (C5). All outfalls were set with a free outfall boundary condition, with the depth set equal to the minimum of the normal or critical depth for the flow in the outfall conduit. The three outfalls along Atwood Road are all free outfalls because the 100-year flood level in the pond in the Atwood Ranch development (A.R. Associates, 2008) is lower than the invert of the outfall pipe. For the other two outfalls the 10- and 100-year events are completely contained within the pipes. No previous studies were found that suggested there is a tailwater condition that would violate the assumption of a free outfall boundary condition, and this study assumed that the downstream pipe system was designed to convey the 10- and 100-year events.

The detention ponds along Atwood Drive were modeled by A.R. Associates (2000) for the addition of Unit 4 to the Placer County Main Jail House. A wetland area located south of the jail was slated to be used as a detention pond (approximate location of Node 3A/Storage1), however, the wetland

designation prompted relocating the detention pond to the east (location of Node 2A/Storage2). Based on an October 2016 site visit it is clear the wetland area receives stormwater runoff from Atwood Drive. The culvert along Atwood Drive (Link 136) was partially blocked with sediment at the time of the site visit (the Mannings roughness was set to 0.1 to account for the loss of capacity), which causes water to overflow into the wetland. For this reason, the wetland was modeled as a shallow storage facility in the baseline model.

The precise alignment of a storm drain pipe along Atwood Drive between 1st Street and F Avenue (Link 16 in Subcatchment 6B) could not be confirmed through the background information or field verification. This pipe connects the northern Corporation Yard to an open channel along Atwood Drive. The pipe size for this link was assumed to be the same as the upstream pipe sized leaving the Corporation Yard, and its length was estimated from an assumed alignment.

Sunset Terrace Estates was assumed to drain to the north and away from the PCGC stormwater catchments. The Sunset Terrace Estates are located to the northwest of the Bell Road and Richardson Drive intersection. The drainage report for Sunset Terrace Estates (Western Planning and Engineering, 1990) showed all on-site stormwater draining to an 18-inch pipe located near a low point on the north side of Bell Road. The 18-inch pipe is not believed to turn south and drain through Catchment 1 because (1) it would require crossing the Combie Canal, and (2) no storm drain outlet was found in this location during field investigation. For these reasons, runoff from Sunset Terrace Estates was assumed to drain north.

2.4 BASELINE MODEL VALIDATION

The baseline model was validated by comparing the modeled output flow rates to other modeled/calculated rates obtained from previous studies in the background research. No recorded streamflow or flood data is available for calibration with actual storm events, so the validation effort focused on comparing flow rates at major outfalls from the project site to previously completed work.

Table 2 – Baseline model validation: comparison of modeled flow rates to previously-completed modeled flow rates by others

Point Description	Peak Flow Rate - Q (cfs)				Source #
	Modeled 10-yr	Sourced 10-yr	Modeled 100-yr	Sourced 100-yr	
Outfall C5	20.1	9.7	34.2	-	1
5D/Storage - Professional Drive Basin	2.9	3.1	4.1	3.9	2
5C/Storage - First Street Basin	2.9	2.7	3.4	3.7	2
5A/Storage - Home Depot Onsite	8.4	7.3	16.9	12.9	2
1J/Storage - Bell and Richardson Drive	3.4	4.0	4.6	6.0	3
2A/Storage 1- West of Jail House #4	29.6	46.0	54.4	92.0	4

Source key:
1 Willow Creek Retail Placer County, CA, Preliminary Hydrology and Hydraulics by TSD Engineering (2014).
2 Drainage Report Home Depot Shopping Center, by Blair, Church and Flynn Consulting Engineers (2007).
3 DeWitt Center Land Development Building Stormwater Detention Storage Study for Bell Road/Rock Creek Watershed (Shed B1) by A.R. Associates (2003).
4 Placer County Main Jail Housing Unit 4 Stormwater Detention Pond Final Report by A.R. Associates (2000).

Slight variations in the output between any two models are expected. However, any significant differences were investigated to determine if there is a reasonable explanation for the difference. Table 2 is a list of the locations near outfalls where there was available information that could be used in the model validation process. Explanations for significant differences in flow rates are discussed in the following paragraphs.

The Willow Creek Retail Preliminary Hydrology and Hydraulic report (TSD Engineering, 2014) described a peak flow of 7.3 cfs during the 10-year event flowing from the Home Depot storm drain system to the receiving 30-inch pipe. Review of the drainage report for Home Depot (Blair, Church and Flynn Consulting Engineers, 2007) showed that 7.3 cfs is just the effluent from the on-site underground detention system (XPSTORM Node 5A/Storage1) and that additional flow enters the 30-inch pipe from off-site facilities, namely the 1st Street and Professional Drive detention basins (XPSTORM Nodes 5C/Storage1 and 5D/Storage1, respectively). The simulated flows in the XPSTORM model closely agreed with the modeling from the Home Depot Report for the outflows of the respective detention basins (Blair, Church and Flynn Consulting Engineers, 2007). Moreover, the XPSTORM baseline model considered flow rates from the entire watershed (Catchment 5). For these reasons, the XPSTORM results are a more complete representation of flow rates in the 30-inch pipe (i.e. Outfall C5), and are adopted as the baseline flow rates.

The model estimated outflow from Node 2A/Storage1 (the detention basin west of Jail House) to be much lower than described by A.R. Associates (2000). It appears three factors are related to the difference in flows between the modeling described by A.R. Associates (2000) and the XPSTORM results:

1. A.R. Associates (2000) estimated the contributing watershed area using 20-foot interval contours. The XPSTORM model was developed with a one-foot contour photogrammetric survey (i.e. Andregg Geomatics, 2015), which resulted in a contributing watershed area that is roughly 20% smaller than the A.R. Associates (2000) area. The difference is attributed to the ability to detect small-scale topographic features, and changes to the land use and site grading since 2000.

2. Development in the vicinity of the Jail has occurred in the years since the study by A.R. Associates (2000), which has changed local drainage patterns. Specifically, A.R. Associates (2000) routed runoff from an 8-acre area north of the Jail through the permanent detention pond. Field investigation and the storm drain mapping by West Yost Associates (2010)—showed that this area (Subcatchments 3B and 3C) bypasses the detention basin via a 48-inch main directed beneath the Jail building complex.

3. The watershed maps in the A.R. Associates (2000) study and review of historical aerial images on Google Earth show three complexes of original DeWitt Center buildings in 2000 that no longer exist. The absence of these buildings in the XPSTORM baseline model manifested as a decrease in the impervious coverage for a nearly 10-acre area spread over Subcatchments 2B, 2D, and 2E. The expected effect from this change in land use would be more infiltration of rainfall and lower peak flow rates.

For the reasons described above, the XPSTORM results can be considered more representative of

present day conditions, and are adopted as baseline flow rates at Node 2A/Storage1. The A.R. Associates (2000) study described the permanent detention pond as being sized to accommodate a complete buildout condition of the contributing watershed and is discussed further in the alternatives analysis portion of this report.

2.5 BASELINE MODEL RESULTS

The baseline model was run for the 2-year, 10-year, and the 100-year, 24-hour storms. The unabridged baseline model output tables are included in Appendix A of this report. The outfall flowrates are provided in Table 3. Locations where the existing storm drain does not have the capacity to convey the full 10- and 100-year flow rates are shown in Plate 2 and Plate 3.

Table 3 – Baseline model results for peak flow rates at outfalls from PCGC

Outfall Location by Catchment	Peak Flow Rate (cfs)		
	2-yr	10-yr	100-yr
C1	17.8	43.8	84.8
C2/3	23.5	41.4	67.0
C4	9.0	17.7	29.4
C5	9.2	20.1	34.2
C6	22.8	32.1	37.7

The model showed the area subject to the most significant flooding (for both the 10- and 100-year event) as being along Atwood Road just west of the Richardson Drive intersection. The model suggested that during a 100-year event 1.5 acre-feet of runoff is not contained within the storm drain system and overtops Atwood Road. Sediment accumulation was observed in the Atwood Road culvert during the October 2016 field investigations; the existing 18-inch pipe was at least 80% blocked and likely exacerbates the overtopping.

The model also showed spot areas of flooding at various locations throughout Catchment 6, with the collection point for Subcatchment 6H having an overflow volume of 0.55 acre-feet during the 100-year storm event; this excess volume of water is presumed to sheet flow southwesterly via E Avenue and parking areas. Many of the Catchment 6 buildings appear to be original to the World War II era DeWitt Center, and the storm drains are likely of the same vintage. Many of the Catchment 6 pipes are 8 and 10 inches in diameter and do not meet the current storm drain sizing requirements (i.e. 12-inch minimum diameter). The flooding throughout Catchment 6 is likely the reason why flowrates in Table 3 for the 10- and 100-year events are more similar compared to other catchments; ponded water would have the effect of lowering peak flow rates in Catchment 6.

The model suggested that flooding occurs during a 100-year event in the upper portions of Catchment 2, near the Finance Administrative Building. The total volume of overtopping in this location is relatively small, no more than 0.22 acre-feet at any location. Given that the flooding will be minor and only occurs during an extreme event, this finding is treated as acceptable and will not be treated as a major consideration in the alternatives analysis.

The stormwater storage basins were evaluated based on the available freeboard between the maximum water surface elevation for a storm event and the existing ground elevation of the basin rim.

The Placer County SWMM requires a minimum of 1 feet of freeboard above the 100-year water level. For the 100-year storm event, five of the 13 basins do not meet this requirement (Table 4). The basins that do not meet the freeboard requirement would not have excess capacity (as currently designed) for additional stormwater storage.

Table 4 – Freeboard availability in stormwater detention basins

Model Node	Description	Basin Rim Ground Elev. (ft)	Max Water Surface Elev. (ft)		Remaining Freeboard (ft)	
			10-yr	100-yr	10-yr	100-yr[1]
1A/Storage1	Natural Storage Basin on Atwood Drive	1,367.5	1,358.1	1,358.5	9.4	9.0
1B/Storage1	Southwest Pond	1,384.3	1,381.2	1,381.6	3.1	2.7
1C/Storage1	Animal Control Center SW Pond/Basin	1,392.3	1,390.9	1,391.6	1.3	***0.7***
1E/Storage2	Natural Upstream Storage from Southwest SW Pond	1,387.7	1,382.7	1,383.1	5.0	4.6
1I/Storage1	Olympic Way SW Basin	1,393.0	1,390.5	1,391.7	2.5	1.3
1J/Storage1	Bell/Richardson SW Basin	1,429.4	1,427.7	1,428.6	1.7	***0.8***
2A/Storage1	Jail House #4 SW Basin	1,399.0	1,397.2	1,398.2	1.8	***0.8***
2A/Storage2	Natural Pond Along Atwood Drive	1,391.1	1,388.4	1,389.3	2.7	1.8
3A/Storage1	Old SW Basin, now Wetland	1,390.0	1,388.7	1,389.3	1.3	***0.7***
5A/Storage1	Home Depot On-site Underground SW Detention	1,406.0	1,390.4	1,390.7	NA[2]	NA[2]
5C/Storage1	1st Street SW Basin	1,435.0	1,430.3	1,431.2	4.7	3.8
5D/Storage1	Professional Drive SW Basin	1,412.7	1,409.9	1,412.0	2.8	***0.7***

1. Italicized cells do not meet the 1-foot minimum freeboard requirement set by Placer County SWMM.
2. Storage is provided by underground pipes and not subject to freeboard requirements.

2.6 CONSIDERATIONS FOR ALTERNATIVES ANALYSIS

The following list is a summary of findings from the baseline model and the review of background information. This information is highlighted as being pertinent to the alternatives analysis.

- The 1st Street detention basin (Node 5C/Storage1) appears to have additional volume available for stormwater attenuation that could be utilized while still meeting the County requirement for one feet of freeboard during the 100-year event. The amount of extra storage is relatively small, and supplemental detention facilities in the vicinity of Subcatchment 5C should be anticipated. Nonetheless, using the extra volume in the 1st Street detention basin in conjunction with other facilities has the potential to decrease the required size of the supplemental detention facilities.

- The "permanent detention pond" (Node 2A/Storage1) located west of the jail complex was originally described as being sized to accommodate a complete buildout condition of the contributing watershed. However, the current modeling suggested that the basin has insufficient freeboard under existing conditions. The basin could still be utilized, but its volume will need to be increased.

- The baseline model showed the storm drain system as having inadequate capacity to contain storm flows throughout Catchment 6. Development within this portion of PCGC will likely require an overhaul of much of the storm drain system.

3 ALTERNATIVES ANALYSIS

Three land use alternatives developed by Williams + Paddon Architects + Planners, Inc. (Appendix B) were reviewed in order to provide input and recommendations on how each land plan would affect the PCGC storm drain system. The illustrative nature of the land plans did not lend to reliably simulating each alternative with the XPSTORM model, however, the results for the baseline conditions model were applied to highlight where proposed development coincides with existing deficiencies of the storm drain system. This section documents the input and recommendations provided to the design team in order to guide the alternatives analysis process.

3.1 SITE-SPECIFIC ISSUES AND RECOMMENDATIONS

Alternative land use appears to impact on existing storm drain infrastructure in the following locations:

- The 1st Street Stormwater Detention Basin is located southwest of the roundabout between 1st Street and F Avenue. All three alternatives propose development that would impact this detention basin. The detention basin is part of the off-site improvements for Home Depot. Any alteration of the 1st Street Stormwater Detention Basin will require that the stormwater management system for Home Depot and the surrounding area be evaluated in detail.

- The Professional Drive Stormwater Detention Basin is located at the corner of Professional Drive and 1st Street. Alternatives 1 and 3 propose development that has the potential to impact this detention basin. Although Alternative 2 shows this area as green space, it will still need to account for this basin being part of the future landscape. The detention basin is part of off-site improvements for Home Depot. Any alteration of the Professional Drive Detention Basin will require that the stormwater management system for Home Depot and the surrounding area be evaluated in detail.

Alternatives appear to coincide with areas that are prone to flooding and/or the storm drain system is undersized in the following locations:

- The baseline model suggested that Subcatchments 6E, 6G, and 6H flood in the 10-year storm event. Any development that increases the impervious area within this area would exacerbate existing flooding problems. The storm drain system in this area appears to have been constructed as part of the original DeWitt Center, and would require upgrades extending down to Atwood Drive at the jail maintenance access driveway (approximately 1,600 linear feet) to comply with current Placer County design standards.

- Field observations and topographic mapping suggest that Atwood Drive may overtop during extreme events. In part, the overtopping is likely related to inadequacies in the storm drain system described in the item above.

- Field efforts thus far have not been conclusive in mapping stormwater infrastructure in the vicinity of Subcatchment 6B; it is likely that key manholes and/or drain inlets have been buried or overgrown with vegetation. Any development in this area will either require (1) subsurface investigations to conclusively locate existing stormwater infrastructure or (2) treating the area as new development requiring an entirely new stormwater system.

- The baseline model suggests that the area of the proposed Buildings D & L (Clerk Recorder Facility and CDRC Growth and Consolidation) floods in the 100-year storm event. Development of this area may require stormwater detention facilities to meet current Placer County design standards and should focus on minimizing additional impervious area.

Additional considerations are summarized as follows:

- Based on the available data, Building H (Fire Station Expansion) does not appear to be hydrologically connected to one of the five outfalls analyzed herein. Since its drainage does not impact campus-scale storm water planning issues, it was not included in the model. Improvements to this portion of PCGC should be accompanied by an on-site drainage report.

- The Ophir Canal is located along the eastern boundary between Willow Creek Drive and 1st Street. Any development in this area would need to accommodate the canal through the project footprint.

3.2 CAMPUS-WIDE ISSUES AND RECOMMENDATIONS

Beyond addressing the specific issues described in the previous section, additional considerations apply for stormwater planning of the entire PCGC campus. Given there are multiple stormwater basins already in place, controlling flow rates through regional basins may be an effective stormwater management strategy. Several regional basins could be designed to control the 10- and 100-year events, as well as the 2-year, 24-hour event to meet hydromodification requirements. It would be difficult, however, to design the regional basins to meet water-quality treatment criteria. Moreover, centralized water-quality treatment is not consistent with guidance of the West Placer Storm Water Quality Design Manual (2016). As such, water-quality treatment should be the responsibility of individual projects, and would achieve the favored dispersed treatment approach. Each project should plan to set aside roughly 10 percent of the total acreage for water-quality treatment features, although advanced planning and calculations may demonstrate less space is required.

The success of this approach hinges on the assumption that the land use of individual developments will closely resemble the land use assumed for sizing of regional basins. If an individual project violates this assumption, the project may need to provide additional on-site flow control and storage volume. The approach will also require coordination with County staff for approval, and establishing a fee structure to share the cost of the regional basins among individual developments.

4 STRATEGY FOR OVERALL STORMWATER MANAGEMENT PLAN

This section describes the model results for the final PCGC Master Plan option ('Final Option'), and the full suite of stormwater infrastructure needed to meet regulatory requirements for flow and water-quality controls. The Final Option represents the evolution of the alternatives presented in the previous section into a vision for the PCGC that incorporates feedback from a variety of stakeholders and community members. This section begins by presenting the development of the XPSTORM model for the Final Option, goes on to present the model results and required improvements to the storm drain system, and concludes by providing general recommendations for how individual projects should approach water-quality treatment.

4.1 FINAL OPTION MODEL DEVELOPMENT

The baseline model was adapted to simulate the ultimate buildout condition for the Final Option. Land use in the model was revised based on a land plan provided by Williams + Paddon (dated April 5, 2017; see Appendix C). Land use was able to be revised in greater detail for two projects where preliminary grading plans were available: the Health and Human Services building in Subcatchements 2H and 2I and the Affordable Housing project in Subcatchment 4A. Subcatchment boundaries were revised based on the land plan and by assuming minimal adjustments to the existing topography (Plate 5). Subcatchment parameters were updated to reflect land use under the buildout condition (Table 5). The total watershed area for the Final Option is 9.2 acres larger than the baseline condition; the increase is from development in Subcatchments 1K, 4A, 5E, and 5F which were conservatively assumed to all drain to one of the five outfalls. Stormwater basins were added to the model, and in some cases, their shape and size adjusted. Similarly, outlet control structures for stormwater basins were redesigned to provide the appropriate flow controls. In all cases, preliminary grading plans for stormwater basins were developed to ensure they fit within the spatial confines of the land plan. Where new development coincided with pipes that the baseline model suggested to be undersized, the pipes were upsized to convey the 100-year event. The Final Option model maintained the same five outfalls from the PCGC campus as the baseline model, and utilized the baseline flow rates from the outfalls to establish thresholds for future outflow rates.

Preliminary designs for stormwater basins were developed according to Section VII of the SWMM to meter outflow rates from the 2-year, 24-hour event (to meet hydromodification requirements) up to the 100-year, 24-hour event. The PCGC stormwater basins will not be incorporated into the Flood Control District's drainage system, so only one foot of freeboard for the 100-year event is required. Outlet structures for all basins were designed to have a low-flow orifice at the basin floor, and a high-flow weir near the basin rim elevation (with the exception of Basin 2A where the outlet structure will remain unchanged). All side slopes are no steeper than 3:1 (horizontal to vertical). None of the basins are so big as to fall under the jurisdiction of the California Division of Safety of Dams.

All pipes that the model indicated to be undersized are assumed to be upsized according to Section VI of the SWMM, which requires new development to convey the 10-year event and to prevent property damage and loss of life during the 100-year event. Without a detailed grading and site plan it is difficult to predict how the 100-year event might pose a threat to property or loss of life. For this reason, and to provide a conservative estimate for pipe sizes, all upsized pipes were designed to convey the entire 100-year event. However, parts of the PCGC not slated for redevelopment (for

instance, the parking area for the Community Development Resource Center where the baseline model showed to pipes to surcharge) were not upsized.

Table 5 – Final Option model catchment parameters

Catchment / Subcatchment	Area (acres)	Percent Impervious (%)
Catchment 1		
1A	10.3	3.4
1B	13.3	8.7
1C	5.0	58.6
1D	3.7	38.7
1E	12.5	16.6
1F	10.6	51.2
1G	11.8	72.6
1H	2.9	31.8
1I	7.7	43.2
1J	6.1	46.2
1K	3.2	37.5
1L	2.3	60.0
C1 Total	89.4	35.3
Catchment 2		
2A	1.7	8.0
2B	4.4	68.5
2C	3.2	68.8
2D	4.2	62.2
2E	6.9	43.0
2F	7.0	69.5
2G	3.6	52.6
2H	5.5	76.7
2I	6.2	77.5
C2 Total	42.5	62.6

Catchment / Subcatchment	Area (acres)	Percent Impervious (%)
Catchment 3		
3A	3.3	36.7
3B	4.7	75.5
3C	4.7	69.2
C3 Total	12.8	63.0
Catchment 4		
4A	2.9	54.9
4B	9.9	63.3
C4 Total	12.8	61.4
Catchment 5		
5A	11.2	81.6
5C	5.8	83.2
5D	6.6	54.4
5E	9.0	68.6
5F	2.0	65.6
C5 Total	34.6	72.4
Catchment 6		
6A	11.6	61.8
6B	7.2	75.2
6C	1.4	99.4
6D	2.0	99.8
6E	1.6	70.0
6F	8.6	57.8
6G	8.9	61.7
6I	6.5	59.4
C6 Total	47.8	65.7
Total	239.7	54.4

4.2 FINAL OPTION MODEL RESULTS

A total of seven new stormwater basins and reconfiguration of three existing basins are needed to meet County requirements for flood control and hydromodification management. The locations of the stormwater basins are shown in Plate 6, and their preliminary dimensions are summarized in Table 6. Other pertinent details for each basin are summarized as follows:

- Basin 1K: A new small basin to control runoff from the proposed residential development in the southwest corner of the planning area.

- Basin 1L: A new small basin to control runoff from Subcatchments 1G and 1L.

- Basin 2A: A large existing basin to control runoff from Catchment 2. A study by A.R. Associates (2000) indicated this basin is sized to accommodate a complete buildout condition, however, their study assumed different ultimate watershed conditions than the current land plan. The model indicated that the volume of Basin 2A will need to be increased to accommodate the additional runoff. We assumed the basin could be expanded into the open space south of the solar farm. The footprint of the basin would need to be increased by roughly 8,700 square feet (34 percent).

- Basin 4A: A new small basin located in the northeast corner of the project site to control runoff from the north portion of the Affordable Housing project (Subcatchment 4A). The basin is proposed for dual use for flood control and water-quality treatment for the north portion of the Affordable Housing project.

- Basin 4B: A new small basin to control runoff from Subcatchment 4B.

- Basin 5C (1st Street Basin): An existing basin, the volume of which is not fully utilized under existing conditions. The land plan impinges on the footprint of the existing basin, and it will need to be regraded to be compatible with the Final Option. The outlet structure will need to be reconfigured to control flow rates in a way that enhances utilization of the storage volume.

- Basin 5D (Professional Drive Basin): An existing basin that controls runoff from Subcatchment 5D. The model suggested that the increase in impervious area in 5D will be offset by the decrease in subcatchment area. The land plan shows a parking area in Subcatchment 4A encroaching on the Basin 5D footprint; the modeling assumed the total volume will not be affected by grading in 4A. The only change to Basin 5D is minor regrading of the emergency overflow outlet to raise its elevation by 0.4 feet so the basin will comply with the County freeboard criteria.

- Basin 5E: A new medium-sized basin to control runoff from Subcatchment 5E. While the footprint of this basin fits within the current land plan, it may be possible to reduce its size by grading a portion of Subcatchment 5E to drain to Basin 5C, thereby using the extra volume in Basin 5C to control flows from a portion of Subcatchment 5E.

- Basin 6A: A new large basin to control runoff from Catchment 6. If the size of Basin 6A cannot fit within the final land plan, there appear to be other areas in Catchment 6 (for instance, the landscaped area along Atwood Drive) that could be utilized to meet the total storage volume requirement.

- Basin 6F: A new medium-sized basin to control runoff from Subcatchments 6G and 6F. The footprint of Basin 6A was maximized within the open space south of the jail building, and Basin 6F is needed to provide supplemental storage to alleviate flooding that the baseline model showed to occur along Atwood Drive.

Table 6 - Summary new and retrofitted stormwater basins

Basin	Basin Geometry				Outlet Structure Geometry		
	Bottom Area (ft^2)	Top Area (ft^2)	Depth (ft)	Total Volume (ac-ft)	Orifice Diameter (in)	Weir Width (ft)	Weir Stage (ft)
1K	1,665	5,588	6	0.48	6	4	4.25
1L	2,600	8,244	6	0.72	15	5	3.75
2A	14,939	35,720	6.5	3.81	same as existing		
4A	3,320	5,760	3	0.31	1.0' x 0.5'	3	1.5
4B	572	2,783	4	0.15	12	4	2.5
5C	6,975	16,207	7	1.83	8	5	7
5D	923	20,015	7	2.00	same as existing		
5E	9,705	21,156	8	2.78	4	5	6
6A	32,015	49,549	6	5.59	2.5' x 2.5'	4	4.75
6F	8,303	16,678	6	1.70	10	4	4.85

1. Basin geometry is inclusive of the required 1 foot of freeboard for storage basins.
2. All orifices were designed at stage = 0.0 ft (at basin floor).
3. For basins proposed to be regraded (e.g. 2A and 5C) the figures are for the total new size.

Pipe segments that the model indicated do not meet County standard for conveyance are shown in Plate 6. Many new minor collector pipes will need to be installed as each project comes online, but are not shown since the model focused on major arterial pipes only. Drainage studies for individual projects should consider how the capacity of collector pipes is affected by the hydraulic grade lines in the receiving arterial pipes.

The land plan shows development occurring on top of the existing Ophir Canal alignment. The canal is not known to be part of the current storm drain system, as such, enclosing the canal should simply consider the anticipated flow rates and other Nevada Irrigation District design requirements (beyond the scope of this report).

The model suggests that the stormwater basins are effective in limiting peak flow rates at each of the five outfalls from PCGC to be no greater than the existing peak flow rates. Table 7 compares peak flow rates under existing conditions to the buildout condition. The buildout condition will be implemented in four phases. Each phase will be completed within five years for a total time of 20 years to achieve the buildout condition. Interim site conditions—where the campus would be somewhere between the existing and buildout condition—will be evaluated in detail in a forthcoming update to this document wherein the stormwater system will be modeled at the completion of each phase. The interim conditions analysis will also include recommendations for an appropriate phasing strategy for the various improvements to the PCGC stormwater system.

Table 7 – Comparison of peak flow rates at outfalls under existing and buildout conditions

Outfall	2-yr Event			10-yr Event			100-yr Event		
	Existing	Buildout	Change	Existing	Buildout	Change	Existing	Buildout	Change
C1	17.8	17.1	-0.7	43.8	41.8	-2.0	84.8	82.1	-2.7
C2/3	23.5	20.2	-3.3	41.4	33.8	-7.6	67.0	63.6	-3.4
C4	9.0	8.7	-0.3	17.7	17.1	-0.6	29.4	29.2	-0.2
C5	9.2	9.0	-0.2	20.1	19.1	-1.0	34.2	32.3	-1.9
C6	22.8	12.5	-10.3	32.1	22.7	-9.4	37.7	31.6	-6.1

4.3 WATER-QUALITY TREATMENT

The design team along with the County has agreed on a strategy of meeting flood control and hydromodification requirements with several campus-scale stormwater basins while leaving water-quality treatment requirements to individual projects as they come online. The West Placer Storm Water Quality Design Manual (WPSWQDM; 2016) provides detailed guidance for integrating low-impact development (LID) strategies into the site design for projects so that they will comply with Clear Water Act regulations, specifically, the National Pollutant Discharge Eliminations System (NPDES) Municipal Separate Storm Sewer System (MS4) Permit.

The requirements for a project will vary depending on the amount of impervious area to be created or replaced, but in general, the work flow to meet water-quality requirements will be as follows:

1. Complete a site assessment to evaluate local conditions and identify constraints and opportunities for LID features;

2. Develop a site layout that includes site design measures, source control, and stormwater treatment features;

3. Implement site design measures to reduce surface runoff to the maximum extent practicable by infiltration, evapotranspiration, and/or harvesting;

4. Include source control measures to reduce the potential for stormwater and pollutants from coming in contact with one another (e.g. trash enclosures, covered storage areas, and developing "good housekeeping" operational practices);

5. Treat the remaining portion of the post-construction 85th percentile, 24-hour storm with stormwater treatment features (infiltration-based features such as bioretention basins are preferred, but flow-through systems may be permitted in special cases);

6. Develop a Post-Construction Storm Water Quality Plan (SWQP) using the templates provided in the WPSWQDM and submit to the County for review and approval; and

7. Maintain the LID features for the life of the project through a well-developed operations and maintenance plan.

The WPSWQDM presents hydromodification management as a component of water-quality treatment design process, however, the campus-scale stormwater basins for PCGC will be designed to control the 2-year, 24-hour event to meet the County's hydromodification criteria. Bioretention basins and other LID features typically provide some level of flow control, and will likely reduce flow rates beyond the post-construction flow rates estimated by the XPSTORM model.

5 CONCLUSIONS

The XPSTORM model showed that regulatory requirements for flood control and hydromodification can be met by installing a series of campus-scale stormwater basins. Water-quality treatment will be addressed as each individual project comes online, the design for which will follow the guidance of the WPSWQDM.

Several arterial storm drain pipes will need to be upsized to accommodate the flow rates anticipated for the Final Option, particularly in Catchment 6 where much of the storm drain infrastructure has not been improved since the original DeWitt Center was constructed.

All of the improvements to the storm drain system are expected to alleviate the surface flooding problems that predicted under existing conditions during the 10-year and larger events. The only exception is the shallow flooding the model predicted for the parking lot of the Community Development Resources Center, which is outside of the development envelope for the Final Option.

Minor collector storm drain pipes were not modeled as part of this study; this subject will be addressed in subsequent drainage reports for individual projects, and should consider how the hydraulics of the arterial storm drain pipes affects the performance of smaller, feeder pipes.

6 LIMITATIONS

This report was prepared in general accordance with the accepted standard of practice in surface-water hydrology existing in Northern California for projects of similar scale at the time the investigations were performed. No other warranties, expressed or implied, are made.

As is customary, we note that readers should recognize that interpretation and evaluation of subsurface conditions and physical factors affecting the hydrologic context of any site is a difficult and inexact art. Judgments leading to conclusions and recommendations are generally made with an incomplete knowledge of the conditions present. More extensive or extended studies, including additional hydrologic baseline monitoring, can reduce the inherent uncertainties associated with such studies. We note, in particular, that many factors affect local and regional ground-water levels. If the client wishes to further reduce the uncertainty beyond the level associated with this study, the authors should be notified for additional consultation.

We have used standard environmental information such as precipitation, topographic mapping, and soil mapping, in our analyses and approaches without verification or modification, in conformance with local custom. New information or changes in regulatory guidance could influence the plans or recommendations, perhaps fundamentally. As updated information becomes available, the interpretations and recommendations contained in this report may warrant change. To aid in revisions, we ask that readers or reviewers advise us of new plans, conditions, or data of which they are aware.

Concepts, findings and interpretations contained in this report are intended for the exclusive use of the Cartwright Engineering under the conditions presently prevailing except where noted otherwise. Their use beyond the boundaries of the site could lead to environmental or structural damage, and/or to noncompliance with water-quality policies, regulations or permits. Data developed or used in this report were collected and interpreted solely for developing an understanding of the hydrologic context at the site as an aid to stormwater master planning. They should not be used for other purposes without great care, updating, review of sampling and analytical methods used, and consultation with the authors.

Finally, we ask once again that readers who have additional pertinent information, who observed changed conditions, or who may note material errors should contact us with their findings at the earliest possible date, so that timely changes may be made.

Plates

PLATE 1 – BASELINE MODEL CONFIGURATION SCHEMATIC
PLATE 2 – BASELINE MODEL RESULTS FOR THE 10-YEAR EVENT
PLATE 3 – BASELINE MODEL RESULTS FOR THE 100-YEAR EVENT
PLATE 4 – FINAL OPTION MODEL CONFIGURATION SCHEMATIC
PLATE 5 – FINAL OPTION CATCHMENTS AND SUBCATCHMENTS
PLATE 6 – FINAL OPTION MODEL RESULTS

DRAFT Master Plan Update - Wet Utility Infrastructure Master Drainage Report

Plate 1. Baseline model configuration schematic, Placer County Government Center, Placer County, California

Basemap source: ESRI ArcGIS Online and data partners

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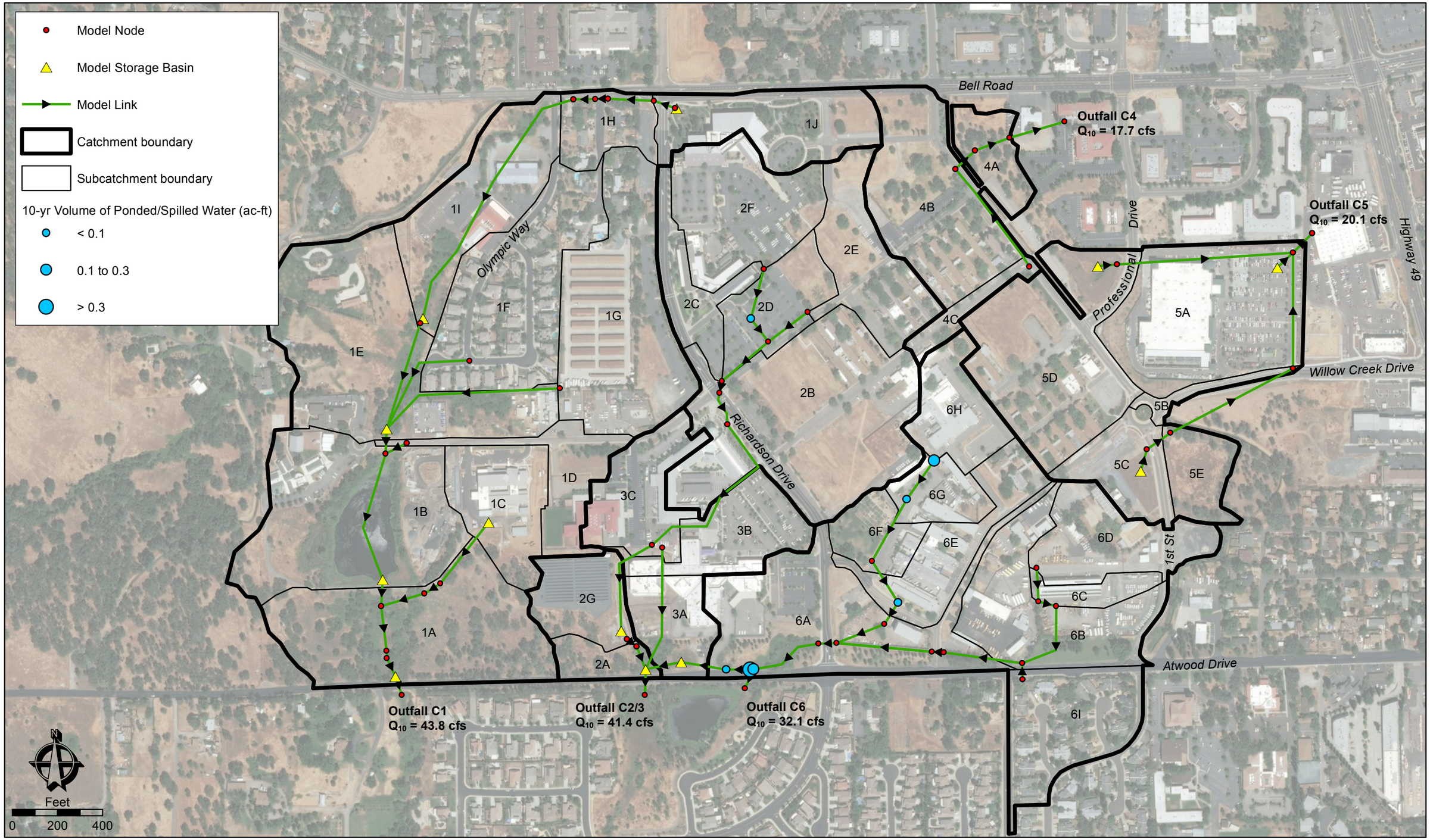

DRAFT Master Plan Update - Wet Utility Infrastructure
Master Drainage Report

Plate 2. Baseline model results for the 10-year event
Placer County Government Center, Placer County, California

Basemap source: ESRI ArcGIS Online and data partners

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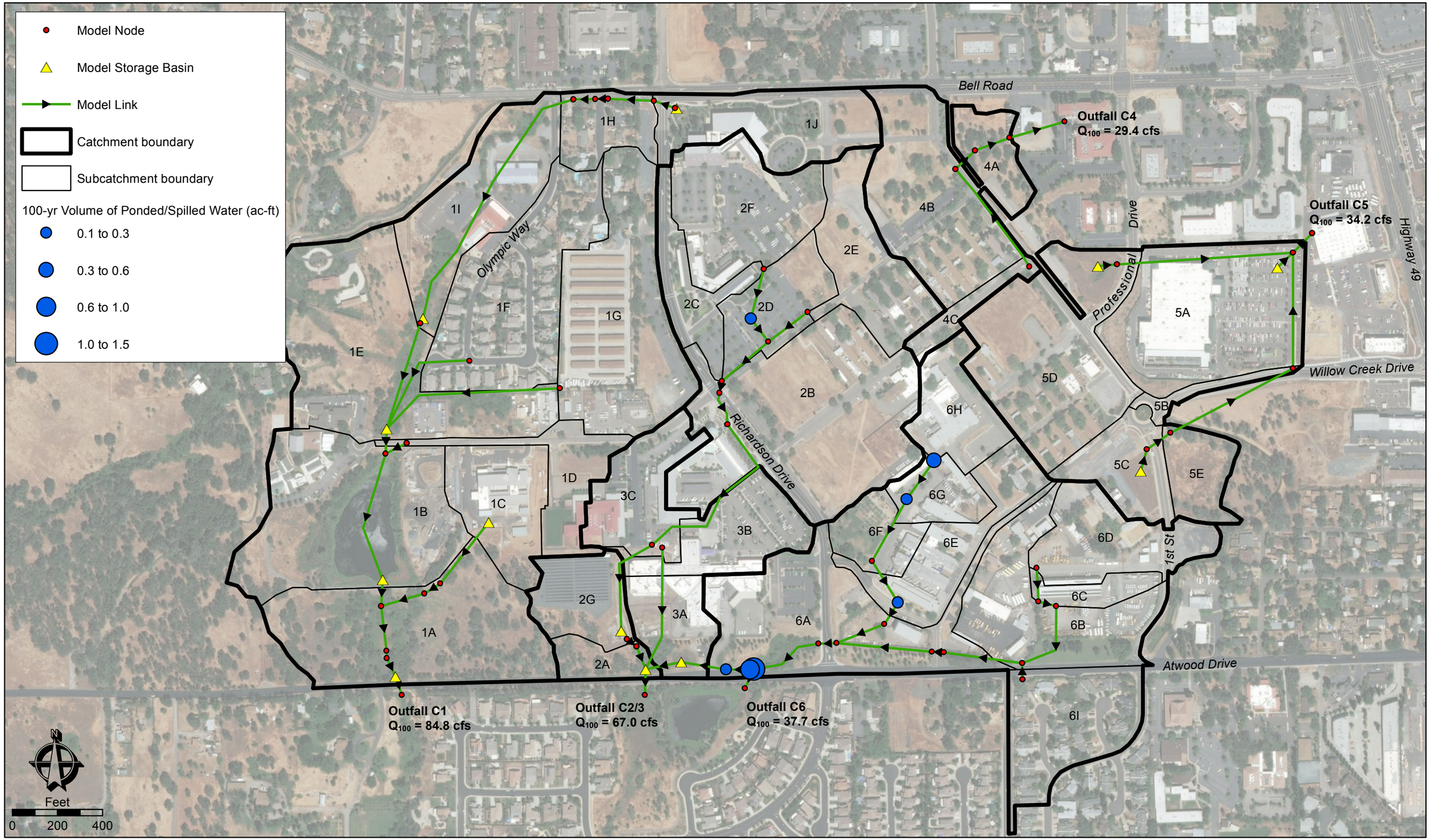

DRAFT Master Plan Update - Wet Utility Infrastructure
Master Drainage Report

Plate 3. Baseline model results for the 100-year event
Placer County Government Center, Placer County, California

Basemap source: ESRI ArcGIS Online and data partners

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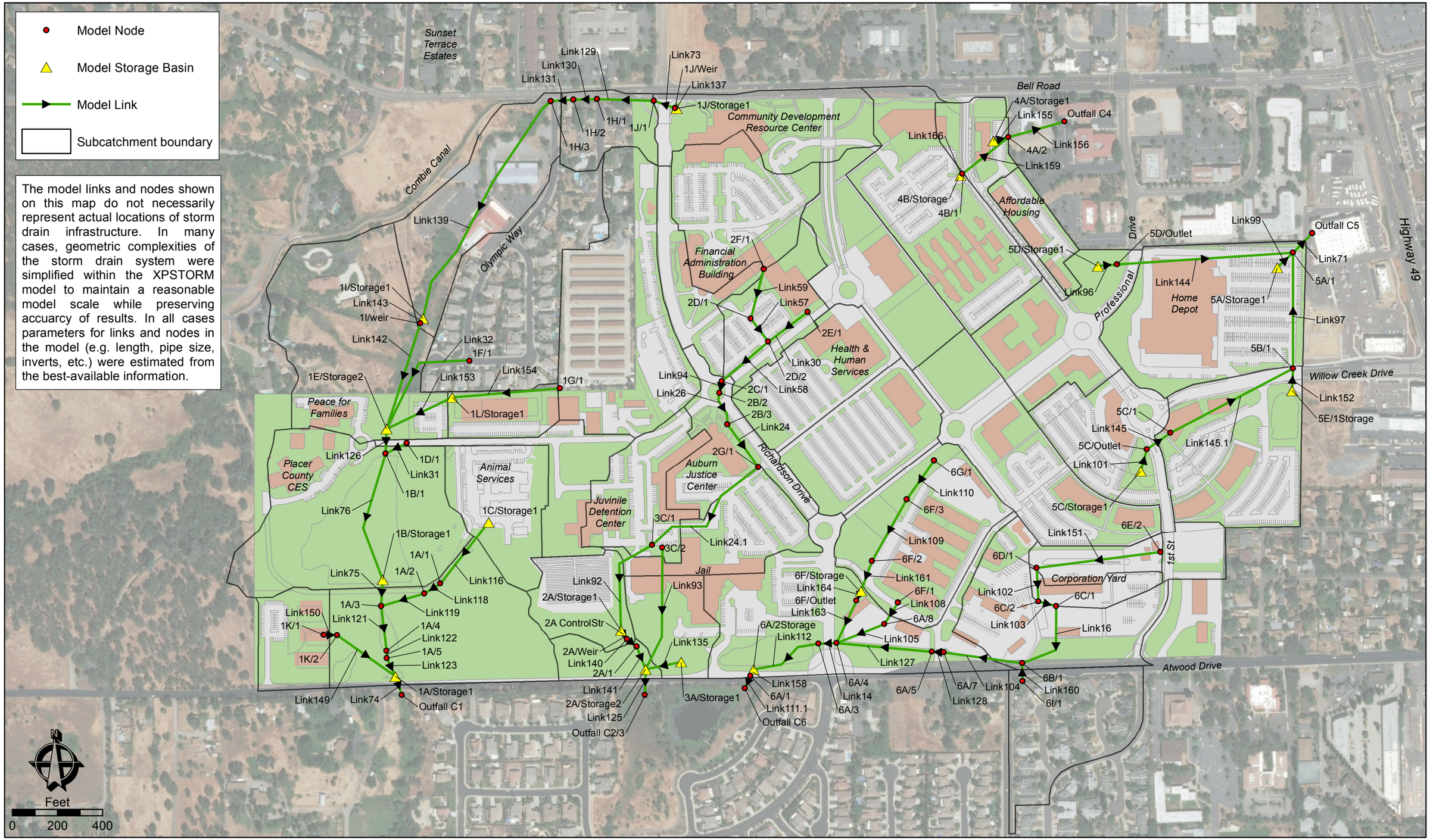

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Master Drainage Report

Plate 4. Final Option model configuration schematic, Placer County Government Center, Placer County, California

Basemap source: Williams + Paddon Architects + Planners, Inc.

...ojects\216145 Placer Government Center Stormwater\figures\216145 Plate 4.mxd

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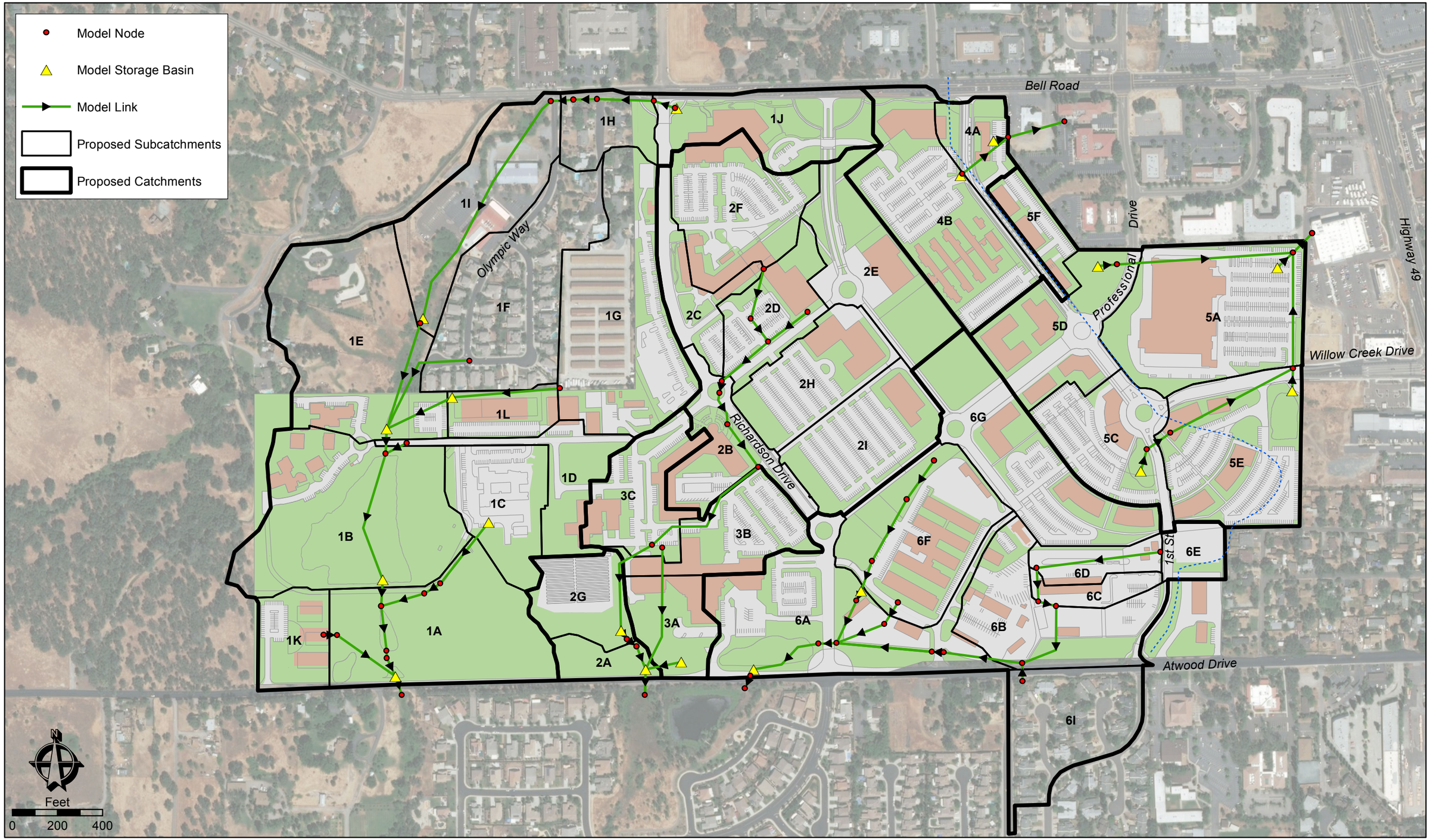

DRAFT Master Plan Update - Wet Utility Infrastructure
Master Drainage Report

Plate 5. Final Option catchments and subcatchments, Placer County Government Center, Placer County, California

Basemap source: ESRI ArcGIS Online and data partners

...ojects\216145 Placer Government Center Stormwater\figures\216145 Plate 5.mxd

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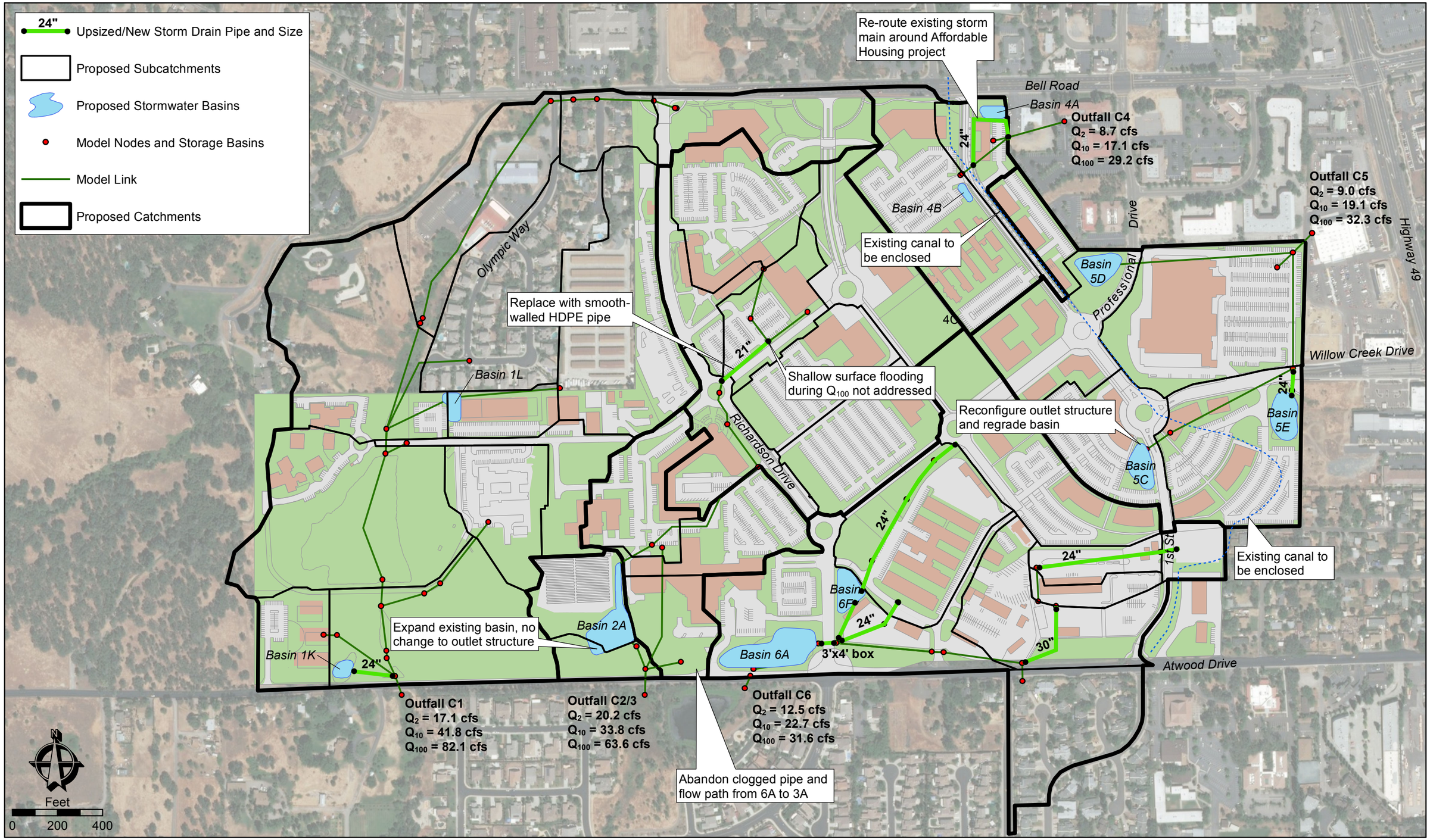

DRAFT Master Plan Update - Wet Utility Infrastructure
Master Drainage Report

Plate 6. Final Option model results,
Placer County Government Center, Placer County, California

Basemap source: ESRI ArcGIS Online and data partners

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Appendix A: XP STORM Model Results

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Type
1A/1	1387.50	1385.50	Node
1A/2	1388.00	1383.00	Node
1A/3	1370.00	1365.00	Node
1A/4	1366.00	1360.00	Node
1A/5	1365.00	1358.00	Node
1A/Storage1	1367.50	1357.00	Storage Node
1B/1	1387.70	1380.80	Node
1B/Storage1	1384.30	1380.00	Storage Node
1C/Storage1	1396.50	1390.00	Storage Node
1D/1	1387.26	1382.85	Node
1E/Storage2	1387.70	1381.90	Storage Node
1F/1	1395.00	1386.00	Node
1G/1	1398.78	1395.00	Node
1H/1	1426.00	1423.50	Node
1H/2	1425.00	1423.00	Node
1H/3	1425.10	1421.50	Node
1I/Storage1	1393.00	1388.50	Storage Node
1I/weir	1393.00	1388.50	Node
1J/1	1428.00	1425.00	Node
1J/Storage1	1430.00	1426.00	Storage Node
1J/Weir	1430.00	1425.50	Node
2A/1	1399.00	1392.50	Node
2A/Storage1	1402.00	1393.00	Storage Node
2A/Storage2	1390.50	1387.00	Storage Node
2A/Weir	1398.00	1393.00	Node
2B/2	1411.17	1407.20	Node
2B/3	1411.46	1406.50	Node
2C/1	1411.61	1408.58	Node
2D/1	1417.18	1412.81	Node
2D/2	1417.19	1410.38	Node
2E/1	1422.00	1417.59	Node
2F/1	1421.76	1416.64	Node
3A/Storage1	1390.50	1386.00	Storage Node
3C/1	1405.92	1400.00	Node
3C/2	1404.34	1395.14	Node
4A	1415.50	1412.20	Node
4A/1	1425.84	1420.00	Node
4B/1	1429.00	1424.00	Node
4C/1	1429.30	1427.00	Node
5A/1	1405.36	1385.30	Node
5A/Storage1	1406.00	1388.76	Storage Node
5B/1	1404.76	1395.62	Node
5C/1	1433.00	1423.00	Node
5C/Outlet	1435.00	1428.00	Node
5C/Storage1	1435.00	1428.00	Storage Node

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Type
5D/Outlet	1413.00	1388.32	Node
5D/Storage1	1413.00	1406.00	Storage Node
6A/1	1408.00	1405.50	Node
6A/2	1407.70	1403.50	Node
6A/3	1405.00	1402.49	Node
6A/4	1397.86	1394.86	Node
6A/5	1395.89	1392.89	Node
6A/6	1393.00	1388.20	Node
6A/7	1391.00	1388.00	Node
6A/Weir	1390.00	1388.20	Node
6B/1	1416.50	1413.62	Node
6C/1	1427.57	1420.44	Node
6C/2	1426.04	1419.60	Node
6D/1	1426.99	1421.76	Node
6E/1	1406.89	1403.61	Node
6F/1	1414.95	1412.25	Node
6G/1	1424.34	1422.29	Node
6H/1	1429.78	1427.00	Node
6I/1	1417.00	1413.89	Node
Outfall C1	1358.00	1354.00	Node
Outfall C2/3	1390.00	1385.00	Node
Outfall C4	1407.00	1401.30	Node
Outfall C5	1391.57	1385.14	Node
Outfall C6	1391.00	1388.00	Node

Node Name	Ground Elevation (ft)	Maximum HGL (ft)		
		2-yr	10-yr	100-yr
1A/1	1387.50	1386.08	1386.34	1386.53
1A/2	1388.00	1385.24	1385.32	1385.36
1A/3	1370.00	1365.15	1365.25	1365.38
1A/4	1366.00	1360.21	1360.38	1360.59
1A/5	1365.00	1358.36	1358.63	1358.97
1A/Storage1	1367.50	1357.69	1358.08	1358.53
1B/1	1387.70	1381.07	1381.25	1381.66
1B/Storage1	1384.30	1380.74	1381.17	1381.64
1C/Storage1	1396.50	1390.61	1390.90	1391.57
1D/1	1387.26	1383.31	1383.48	1383.71
1E/Storage2	1387.70	1382.46	1382.76	1383.11
1F/1	1395.00	1386.84	1387.24	1388.08
1G/1	1398.78	1395.41	1395.62	1395.89
1H/1	1426.00	1424.00	1424.08	1424.19
1H/2	1425.00	1423.47	1423.54	1423.64
1H/3	1425.10	1421.76	1421.83	1421.89
1I/Storage1	1393.00	1389.88	1390.57	1391.75
1I/weir	1393.00	1388.91	1389.05	1389.20
1J/1	1428.00	1425.53	1425.62	1425.75
1J/Storage1	1430.00	1426.97	1427.72	1428.60
1J/Weir	1430.00	1426.17	1426.28	1426.44
2A/1	1399.00	1392.89	1393.03	1393.20
2A/Storage1	1402.00	1395.67	1397.24	1398.20
2A/Storage2	1390.50	1387.93	1388.40	1389.36
2A/Weir	1398.00	1393.89	1394.21	1394.61
2B/2	1411.17	1408.48	1408.99	1409.30
2B/3	1411.46	1407.80	1408.31	1408.73
2C/1	1411.61	1409.32	1409.72	1409.98
2D/1	1417.18	1413.61	1417.31	1418.20
2D/2	1417.19	1412.68	1416.81	1417.60
2E/1	1422.00	1418.09	1418.65	1421.83
2F/1	1421.76	1417.39	1420.28	1422.19
3A/Storage1	1390.50	1388.70	1388.76	1389.35
3C/1	1405.92	1401.18	1401.67	1402.07
3C/2	1404.34	1395.73	1395.92	1396.14
4A	1415.50	1412.83	1413.10	1413.40
4A/1	1425.84	1420.39	1420.55	1420.69
4B/1	1429.00	1424.74	1425.10	1427.04
4C/1	1429.30	1427.47	1428.79	1429.47
5A/1	1405.36	1386.38	1386.90	1387.43
5A/Storage1	1406.00	1390.16	1390.45	1390.70
5B/1	1404.76	1396.09	1396.25	1396.44
5C/1	1433.00	1423.39	1423.53	1423.68
5C/Outlet	1435.00	1428.38	1428.47	1428.52

Node Name	Ground Elevation (ft)	Maximum HGL (ft)		
		2-yr	10-yr	100-yr
5C/Storage1	1435.00	1429.58	1430.28	1431.24
5D/Outlet	1413.00	1406.39	1406.50	1406.60
5D/Storage1	1413.00	1408.25	1409.95	1412.01
6A/1	1408.00	1406.03	1406.24	1406.46
6A/2	1407.70	1404.44	1404.62	1404.77
6A/3	1405.00	1403.11	1403.19	1403.26
6A/4	1397.86	1396.43	1396.93	1397.57
6A/5	1395.89	1393.55	1393.72	1395.02
6A/6	1393.00	1391.66	1393.40	1394.74
6A/7	1391.00	1390.74	1391.51	1392.14
6A/Weir	1390.00	1390.74	1391.51	1392.14
6B/1	1416.50	1414.94	1415.28	1415.64
6C/1	1427.57	1421.06	1421.36	1426.27
6C/2	1426.04	1420.70	1421.29	1426.19
6D/1	1426.99	1422.42	1422.65	1426.85
6E/1	1406.89	1405.97	1407.19	1407.89
6F/1	1414.95	1412.76	1412.91	1413.13
6G/1	1424.34	1424.42	1424.71	1425.10
6H/1	1429.78	1430.29	1430.83	1431.40
6I/1	1417.00	1415.18	1416.11	1417.27
Outfall C1	1358.00	1354.69	1355.08	1355.53
Outfall C2/3	1390.00	1385.93	1386.40	1387.30
Outfall C4	1407.00	1401.92	1402.20	1402.50
Outfall C5	1391.57	1386.10	1386.58	1387.04
Outfall C6	1391.00	1389.83	1389.83	1389.83

Name	Shape	Diameter/ Height (ft)	Bottom Width (ft)	Length (ft)	Downstream Invert Elevation (ft)	Upstream Invert Elevation (ft)	Slope (ft/ft)
Link14	Circular	3	--	80	1392.89	1394.86	0.025
Link16	Circular	2	--	486	1414.00	1419.60	0.012
Link24	Circular	4	--	864	1400.00	1406.50	0.008
Link26	Circular	3	--	154	1406.50	1407.20	0.005
Link30	Circular	1.5	--	218	1411.24	1417.59	0.029
Link31	Circular	1.5	--	80	1381.00	1382.85	0.023
Link32	Circular	2	--	70	1385.25	1386.00	0.011
Link33	Circular	0.833	--	542	1424.00	1427.00	0.006
Link34	Circular	1.67	--	120	1420.00	1424.00	0.033
Link35	Circular	2	--	250	1401.30	1412.20	0.044
Link57	Circular	1.75	--	129	1410.78	1412.81	0.016
Link58	Circular	1.75	--	270	1408.58	1410.38	0.007
Link59	Circular	1.5	--	227	1412.81	1416.64	0.017
Link71	Circular	3	--	70	1385.14	1385.30	0.002
Link73	Circular	1.5	--	115	1425.00	1425.50	0.004
Link74	Circular	4	3	70	1354.00	1357.00	0.043
Link75	Circular	4	3	125	1365.00	1380.00	0.120
Link76	Trapezoidal	4.3	100	110	1380.00	1380.80	0.007
Link87	Trapezoidal	3	6	600	1383.00	1395.00	0.020
Link92	Circular	3.5	--	167	1398.00	1400.00	0.012
Link93	Circular	3.5	--	407	1387.00	1395.14	0.020
Link95	Trapezoidal	0.5	1.2	154	1412.20	1420.00	0.051
Link97	Circular	2	--	516	1385.30	1395.62	0.020
Link102	Circular	2	--	150	1420.44	1421.76	0.009
Link103	Circular	2.5	--	84	1419.60	1420.44	0.010
Link104	Circular	2.5	--	350	1405.50	1414.00	0.024
Link105	Circular	1.5	--	227	1394.86	1402.49	0.034
Link107	Circular	1.25	--	217	1403.61	1412.25	0.040
Link108	Circular	1.25	--	112	1402.49	1403.61	0.010
Link109	Circular	0.667	--	313	1412.25	1422.29	0.032
Link110	Circular	0.5	--	211	1422.29	1427.00	0.022
Link111	Special	2.83	--	93	1388.00	1388.20	0.002
Link112	Natural	0	--	325	1388.20	1392.89	0.014
Link116	Circular	1.25	--	256	1385.50	1390.00	0.018
Link118	Circular	1.25	--	54	1383.00	1385.50	0.046
Link119	Trapezoidal	3	3	165	1365.00	1385.00	0.121
Link121	Trapezoidal	3	25	33	1360.00	1365.00	0.152
Link122	Trapezoidal	6	6	33	1358.00	1360.00	0.061
Link123	Trapezoidal	2	15	50	1357.00	1358.00	0.020
Link129	Circular	1.5	--	250	1423.50	1425.00	0.006
Link130	Trapezoidal	2	2	105	1423.00	1423.50	0.005
Link131	Circular	2	1.5	100	1421.50	1423.00	0.015
Link134	Circular	1.5	--	200	1386.00	1388.00	0.010

Name	Shape	Diameter/ Height (ft)	Bottom Width (ft)	Length (ft)	Downstream Invert Elevation (ft)	Upstream Invert Elevation (ft)	Slope (ft/ft)
Link135	Trapezoidal	2	7	60	1387.00	1388.50	0.025
Link136	Trapezoidal	1	1	120	1388.00	1388.20	0.002
Link139	Natural	0	--	1200	1390.00	1421.50	0.026
Link140	Circular	3.5	--	40	1392.50	1393.00	0.013
Link141	Natural	0	--	100	1387.00	1392.50	0.055
Link142	Natural	0	--	500	1381.90	1388.50	0.013
Link144	Circular	1.25	--	822	1388.32	1406.00	0.022
Link145	Circular	1.5	--	268	1423.00	1428.00	0.019
Link145.1	Circular	1.5	--	630	1395.62	1423.00	0.043
Link147	Circular	1.5	--	83	1413.62	1413.89	0.003

Name	Maximum Flow Rate (cfs)		
	2-yr	10-yr	100-yr
Link14	25.8	41.5	58.5
Link16	5.7	10.1	14.7
Link24	25.1	46.9	67.1
Link26	15.7	27.5	32.9
Link30	4.0	7.9	13.9
Link31	3.0	5.5	9.2
Link32	7.9	15.2	26.2
Link33	1.1	2.0	2.2
Link34	7.6	14.8	23.7
Link35	9.3	18.2	29.5
Link57	6.3	11.7	14.9
Link58	13.1	22.8	25.2
Link59	6.4	11.3	13.9
Link71	9.2	20.1	34.2
Link73	2.5	3.4	4.6
Link74	17.8	44.2	85.3
Link75	13.5	33.7	64.5
Link76	34.7	69.7	120.3
Link87	8.7	16.8	29.1
Link92	25.0	46.9	67.3
Link93	8.0	14.3	23.7
Link95	7.6	14.8	23.7
Link97	3.5	6.4	10.5
Link102	4.5	8.0	13.3
Link103	4.5	8.0	13.3
Link104	17.8	31.7	47.7
Link105	8.1	10.1	11.2
Link107	4.2	5.8	8.2
Link108	8.1	10.1	11.2
Link109	2.2	2.2	2.3
Link110	1.0	1.1	1.1
Link111	22.8	32.1	37.7
Link112	25.6	41.5	101.9
Link116	3.7	6.8	8.7
Link118	3.7	6.8	8.7
Link119	3.7	6.8	8.7
Link121	14.6	36.3	72.4
Link122	14.6	36.3	72.4
Link123	14.6	36.3	72.4
Link129	2.5	3.4	4.6
Link130	2.5	3.4	4.6
Link131	2.5	3.4	4.6
Link134	1.4	1.6	1.8
Link135	3.8	6.2	9.7
Link136	3.4	6.3	6.1

Name	Maximum Flow Rate (cfs)		
	2-yr	10-yr	100-yr
Link139	3.6	6.5	9.7
Link140	14.9	29.6	54.9
Link141	14.9	29.6	54.9
Link142	8.0	16.2	26.9
Link144	1.9	2.9	4.1
Link145	1.8	2.9	3.4
Link145.1	3.1	5.5	8.5
Link147	5.5	9.8	14.1

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Type
1A/1	1387.50	1385.50	Node
1A/2	1388.00	1383.00	Node
1A/3	1370.00	1365.00	Node
1A/4	1366.00	1360.00	Node
1A/5	1365.00	1358.00	Node
1A/Storage1	1367.50	1357.00	Storage Node
1B/1	1387.70	1380.80	Node
1B/Storage1	1384.30	1380.00	Storage Node
1C/Storage1	1396.50	1390.00	Storage Node
1D/1	1387.26	1382.85	Node
1E/Storage2	1387.70	1381.90	Storage Node
1F/1	1395.00	1386.00	Node
1G/1	1400.00	1397.00	Node
1H/1	1426.00	1423.50	Node
1H/2	1425.00	1423.00	Node
1H/3	1425.10	1421.50	Node
1I/Storage1	1393.00	1388.50	Storage Node
1I/weir	1393.00	1388.50	Node
1J/1	1428.00	1425.00	Node
1J/Storage1	1430.00	1426.00	Storage Node
1J/Weir	1430.00	1425.50	Node
1K/1	1386.00	1380.00	Node
1K/2	1385.00	1360.00	Storage Node
1L/Storage1	1395.00	1389.00	Storage Node
2A/1	1399.00	1392.50	Node
2A/Storage1	1401.50	1393.00	Storage Node
2A/Storage2	1390.50	1387.00	Storage Node
2A/Weir	1398.00	1393.00	Node
2B/2	1411.17	1407.20	Node
2B/3	1411.46	1406.50	Node
2C/1	1411.61	1408.58	Node
2D/1	1417.18	1412.81	Node
2D/2	1417.19	1410.38	Node
2E/1	1422.00	1417.59	Node
2F/1	1421.76	1416.64	Node
2G/1	1411.95	1404.74	Node
3A/Storage1	1390.50	1386.00	Storage Node
3C/1	1405.92	1400.00	Node
3C/2	1404.34	1395.14	Node
4A/2	1420.00	1412.00	Node
4A/Storage1	1415.00	1412.00	Storage Node
4B/1	1430.00	1423.32	Node
4B/Storage	1430.00	1426.00	Storage Node
5A/1	1405.36	1385.30	Node
5A/Storage1	1406.00	1388.76	Storage Node

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Type
5B/1	1404.76	1395.62	Node
5C/1	1433.00	1423.00	Node
5C/Outlet	1435.00	1428.00	Node
5C/Storage1	1435.00	1428.00	Storage Node
5D/Outlet	1413.00	1388.32	Node
5D/Storage1	1413.00	1406.00	Storage Node
5E/1Storage	1412.00	1405.00	Storage Node
6A/1	1393.00	1388.00	Node
6A/2Storage	1394.00	1388.00	Storage Node
6A/3	1395.89	1392.89	Node
6A/4	1397.86	1394.86	Node
6A/5	1407.70	1403.50	Node
6A/7	1408.00	1405.50	Node
6A/8	1405.00	1401.00	Node
6B/1	1416.50	1413.62	Node
6C/1	1426.04	1419.60	Node
6C/2	1427.57	1420.44	Node
6D/1	1426.99	1421.76	Node
6E/2	1436.00	1427.45	Node
6F/1	1406.89	1403.61	Node
6F/2	1414.95	1412.25	Node
6F/3	1424.34	1422.29	Node
6F/Outlet	1412.00	1405.00	Node
6F/Storage	1412.00	1406.00	Storage Node
6G/1	1429.78	1427.00	Node
6I/1	1417.00	1413.89	Node
Outfall C1	1358.00	1354.00	Node
Outfall C2/3	1390.00	1385.00	Node
Outfall C4	1407.00	1401.30	Node
Outfall C5	1391.57	1385.14	Node
Outfall C6	1391.00	1388.00	Node

Node Name	Ground Elevation (ft)	Maximum HGL (ft)		
		2-yr	10-yr	100-yr
1A/1	1387.50	1386.08	1386.34	1386.53
1A/2	1388.00	1385.24	1385.32	1385.36
1A/3	1370.00	1365.14	1365.24	1365.37
1A/4	1366.00	1360.21	1360.37	1360.58
1A/5	1365.00	1358.35	1358.62	1358.96
1A/Storage1	1367.50	1357.68	1358.05	1358.50
1B/1	1387.70	1381.05	1381.23	1381.64
1B/Storage1	1384.30	1380.74	1381.15	1381.62
1C/Storage1	1396.50	1390.61	1390.90	1391.57
1D/1	1387.26	1383.31	1383.48	1383.71
1E/Storage2	1387.70	1382.42	1382.68	1383.05
1F/1	1395.00	1386.84	1387.24	1388.07
1G/1	1400.00	1397.57	1397.86	1398.24
1H/1	1426.00	1424.00	1424.08	1424.19
1H/2	1425.00	1423.47	1423.54	1423.64
1H/3	1425.10	1421.76	1421.83	1421.89
1I/Storage1	1393.00	1389.88	1390.56	1391.75
1I/weir	1393.00	1388.91	1389.05	1389.20
1J/1	1428.00	1425.54	1425.62	1425.75
1J/Storage1	1430.00	1426.95	1427.68	1428.55
1J/Weir	1430.00	1426.17	1426.28	1426.44
1K/1	1386.00	1381.42	1382.68	1384.19
1K/2	1385.00	1360.29	1360.36	1360.41
1L/Storage1	1395.00	1392.15	1393.24	1393.89
2A/1	1399.00	1392.89	1393.00	1393.20
2A/Storage1	1401.50	1395.56	1397.02	1398.21
2A/Storage2	1390.50	1387.85	1388.19	1389.08
2A/Weir	1398.00	1393.88	1394.15	1394.61
2B/2	1411.17	1408.74	1409.36	1409.83
2B/3	1411.46	1407.80	1408.31	1408.68
2C/1	1411.61	1409.42	1410.00	1410.67
2D/1	1417.18	1413.50	1416.17	1418.00
2D/2	1417.19	1411.88	1415.51	1417.42
2E/1	1422.00	1418.19	1418.65	1422.20
2F/1	1421.76	1417.38	1418.84	1422.09
2G/1	1411.95	1406.19	1406.74	1407.18
3A/Storage1	1390.50	1386.00	1386.00	1389.08
3C/1	1405.92	1401.33	1401.85	1402.30
3C/2	1404.34	1395.73	1395.92	1396.15
4A/2	1420.00	1412.61	1412.89	1413.22
4A/Storage1	1415.00	1413.59	1413.84	1414.14
4B/1	1430.00	1423.90	1424.14	1424.44
4B/Storage	1430.00	1427.95	1428.37	1428.87
5A/1	1405.36	1386.36	1386.85	1387.36

Node Name	Ground Elevation (ft)	Maximum HGL (ft)		
		2-yr	10-yr	100-yr
5A/Storage1	1406.00	1390.16	1390.45	1390.70
5B/1	1404.76	1396.07	1396.21	1396.36
5C/1	1433.00	1423.37	1423.50	1423.64
5C/Outlet	1435.00	1428.36	1428.44	1428.50
5C/Storage1	1435.00	1429.88	1430.79	1432.01
5D/Outlet	1413.00	1406.37	1406.46	1406.55
5D/Storage1	1413.00	1407.92	1409.25	1410.91
5E/1Storage	1412.00	1407.80	1409.93	1411.35
6A/1	1393.00	1390.18	1391.00	1392.07
6A/2Storage	1394.00	1390.34	1391.53	1393.00
6A/3	1395.89	1393.55	1393.74	1393.94
6A/4	1397.86	1395.45	1395.73	1396.06
6A/5	1407.70	1404.42	1404.58	1404.74
6A/7	1408.00	1406.00	1406.19	1406.41
6A/8	1405.00	1401.55	1401.74	1401.99
6B/1	1416.50	1414.90	1415.21	1415.55
6C/1	1426.04	1420.42	1420.71	1421.09
6C/2	1427.57	1420.95	1421.13	1421.37
6D/1	1426.99	1422.32	1422.50	1422.72
6E/2	1436.00	1427.80	1427.91	1428.04
6F/1	1406.89	1404.29	1404.54	1404.86
6F/2	1414.95	1412.77	1412.95	1413.17
6F/3	1424.34	1422.94	1423.18	1423.49
6F/Outlet	1412.00	1405.34	1405.41	1405.47
6F/Storage	1412.00	1407.54	1408.65	1410.11
6G/1	1429.78	1427.68	1427.93	1428.27
6I/1	1417	1415.15	1416.04	1417.00
Outfall C1	1358	1354.67	1355.05	1355.49
Outfall C2/3	1390	1385.85	1386.19	1387.08
Outfall C4	1407	1401.91	1402.19	1402.52
Outfall C5	1391.574	1386.09	1386.54	1386.99
Outfall C6	1391	1389.83	1389.83	1389.83

Name	Shape	Diameter/ Height (ft)	Bottom Width (ft)	Length (ft)	Downstream Invert Elevation (ft)	Upstream Invert Elevation (ft)	Slope (ft/ft)
Link14	Rectangular	3	4	80	1392.89	1394.86	0.025
Link16	Circular	2.5	--	486	1414.00	1419.60	0.012
Link24	Circular	4	--	234	1404.74	1406.50	0.008
Link24.1	Circular	4	--	630	1400.00	1404.74	0.008
Link26	Circular	3	--	154	1406.50	1407.20	0.005
Link30	Circular	1.5	--	218	1411.24	1417.59	0.029
Link31	Circular	1.5	--	80	1381.00	1382.85	0.023
Link32	Circular	2	--	70	1385.25	1386.00	0.011
Link57	Circular	1.75	--	129	1410.78	1412.81	0.016
Link58	Circular	1.75	--	270	1408.58	1410.38	0.007
Link59	Circular	1.5	--	227	1412.81	1416.64	0.017
Link71	Circular	3	--	70	1385.14	1385.30	0.002
Link73	Circular	1.5	--	115	1425.00	1425.50	0.004
Link74	Circular	4	3	70	1354.00	1357.00	0.043
Link75	Circular	4	3	125	1365.00	1380.00	0.120
Link76	Trapezoidal	4.3	100	110	1380.00	1380.80	0.007
Link92	Circular	3.5	--	167	1398.00	1400.00	0.012
Link93	Circular	3.5	--	407	1387.00	1395.14	0.020
Link97	Circular	2	--	516	1385.30	1395.62	0.020
Link102	Circular	2	--	150	1420.44	1421.76	0.009
Link103	Circular	2.5	--	84	1419.60	1420.44	0.010
Link104	Circular	2.5	--	350	1405.50	1414.00	0.024
Link105	Circular	2	--	227	1394.86	1401.00	0.027
Link108	Circular	2	--	112	1401.00	1403.61	0.023
Link109	Circular	2	--	313	1412.25	1422.29	0.032
Link110	Circular	2	--	211	1422.29	1427.00	0.022
Link111.1	Special	2.83	--	60	1388.00	1388.10	0.002
Link112	Natural	0	--	325	1388.20	1392.89	0.014
Link116	Circular	1.25	--	256	1385.50	1390.00	0.018
Link118	Circular	1.25	--	54	1383.00	1385.50	0.046
Link119	Trapezoidal	3	3	165	1365.00	1385.00	0.121
Link121	Trapezoidal	3	25	33	1360.00	1365.00	0.152
Link122	Trapezoidal	6	6	33	1358.00	1360.00	0.061
Link123	Trapezoidal	2	15	50	1357.00	1358.00	0.020
Link129	Circular	1.5	--	250	1423.50	1425.00	0.006
Link130	Trapezoidal	2	2	105	1423.00	1423.50	0.005
Link131	Circular	2	1.5	100	1421.50	1423.00	0.015
Link135	Trapezoidal	2	7	60	1387.00	1388.50	0.025
Link139	Natural	0	--	1200	1390.00	1421.50	0.026
Link140	Circular	3.5	--	40	1392.50	1393.00	0.013
Link141	Natural	0	--	100	1387.00	1392.50	0.055
Link142	Natural	0	--	500	1381.90	1388.50	0.013
Link144	Circular	1.25	--	822	1388.32	1406.00	0.022

Name	Shape	Diameter/ Height (ft)	Bottom Width (ft)	Length (ft)	Downstream Invert Elevation (ft)	Upstream Invert Elevation (ft)	Slope (ft/ft)
Link145	Circular	1.5	--	268	1423.00	1428.00	0.019
Link145.1	Circular	1.5	--	630	1395.62	1423.00	0.043
Link149	Circular	2	--	200	1358.00	1360.00	0.010
Link151	Circular	2	--	569	1421.76	1427.45	0.010
Link154	Trapezoidal	3	6	960	1390.00	1397.00	0.007
Link156	Circular	2	--	250	1401.30	1412.00	0.043
Link159	Circular	2	--	350	1412.00	1423.32	0.032
Link160	Circular	1.5	--	83	1413.62	1413.89	0.003
Link161	Circular	2	--	70	1408.00	1412.25	0.061
Link163	Circular	2	--	235	1394.86	1405.00	0.043

Name	Maximum Flow Rate (cfs)		
	2-yr	10-yr	100-yr
Link14	25.2	44.4	69.3
Link16	4.4	7.8	12.7
Link24	25.7	46.6	62.6
Link24.1	31.1	56.1	78.3
Link26	22.4	40.4	52.0
Link30	5.6	10.2	15.0
Link31	3.0	5.5	9.2
Link32	7.9	15.2	26.2
Link57	6.0	10.7	18.4
Link58	15.0	27.1	31.4
Link59	6.0	10.6	13.9
Link71	9.0	19.1	32.4
Link73	2.5	3.4	4.6
Link74	17.1	41.9	82.2
Link75	13.5	32.5	63.0
Link76	31.5	60.6	111.6
Link92	31.0	56.2	78.2
Link93	8.1	14.4	23.8
Link97	3.3	5.6	8.7
Link102	3.2	5.6	9.2
Link103	3.2	5.6	9.2
Link104	16.2	28.6	43.8
Link105	7.2	12.9	21.5
Link108	7.2	12.9	21.5
Link109	7.6	13.5	22.4
Link110	7.6	13.5	22.4
Link111.1	12.5	22.7	31.6
Link112	25.2	44.4	69.3
Link116	3.7	6.7	8.7
Link118	3.7	6.8	8.7
Link119	3.7	6.8	8.7
Link121	14.4	34.9	70.4
Link122	14.4	34.9	70.4
Link123	14.4	34.9	70.4
Link129	2.5	3.4	4.6
Link130	2.5	3.4	4.6
Link131	2.5	3.4	4.6
Link135	0.0	0.0	0.0
Link139	3.6	6.5	9.7
Link140	14.5	26.4	55.1
Link141	14.5	26.4	55.1
Link142	7.9	16.1	26.8
Link144	1.7	2.5	3.5
Link145	1.6	2.3	3.0
Link145.1	2.7	4.8	7.7

Name	Maximum Flow Rate (cfs)		
	2-yr	10-yr	100-yr
Link149	1.0	1.5	1.9
Link151	1.4	2.4	4.0
Link154	8.8	16.8	29.0
Link156	8.7	17.7	30.0
Link159	7.5	14.4	24.5
Link160	5.5	9.8	13.5
Link161	7.6	13.5	22.4
Link163	2.4	3.7	4.8

Appendix B: Alternative Land Plans

LEGEND

A	HEALTH AND HUMAN SERVICES
B	COUNTY ADMINISTRATIVE CENTER
C	AGRICULTURAL COMMISSIONER AND FARM ADVISOR FACILITY
D	CLERK RECORDER ELECTIONS TRAINING/ WAREHOUSE FACILITY
E	MUSEUM WAREHOUSE FACILITY
F	CORPORATION YARD ADMINISTRATION AND TRAINING CENTER
G	SHERIFF AND PROBATION SUPPORT FACILITY
H	FIRE STATION 180 EXPANSION
I	ADMINISTRATION SERVICES IT/TELECOM/ WAREHOUSE
J	FINANCE ADMINISTRATION BUILDING ANNEX
K	COMMUNITY BUILDING
L	CDRC GROWTH AND CONSOLIDATION

NEW COUNTY BUILDINGS

RESIDENTIAL

HOTEL

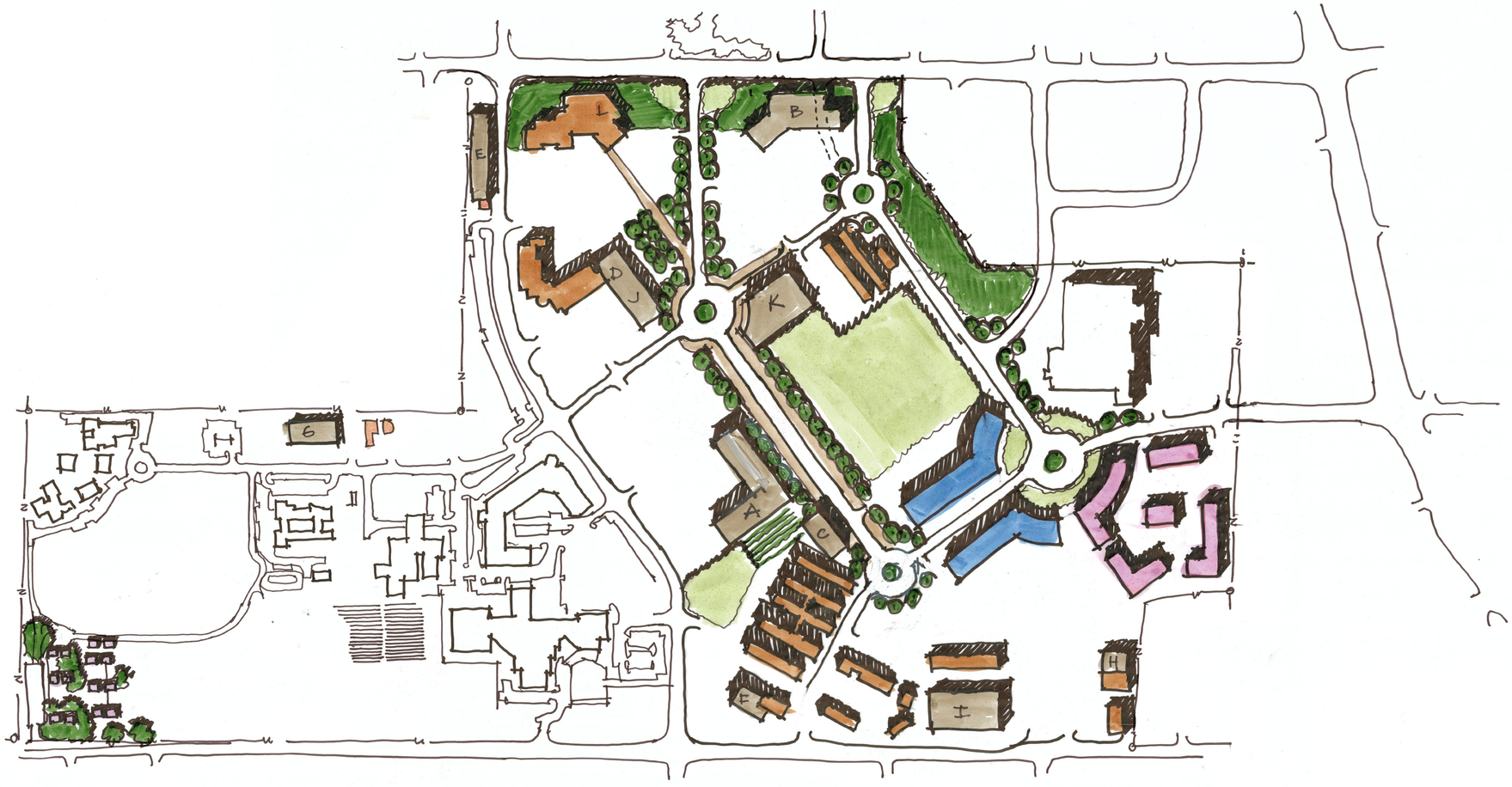
FLEX SPACE/ MIXED-USE

RETAIL

OPTION 1

PLACER COUNTY GOVERNMENT CENTER
AUBURN, CALIFORNIA

0 200 800
100 400

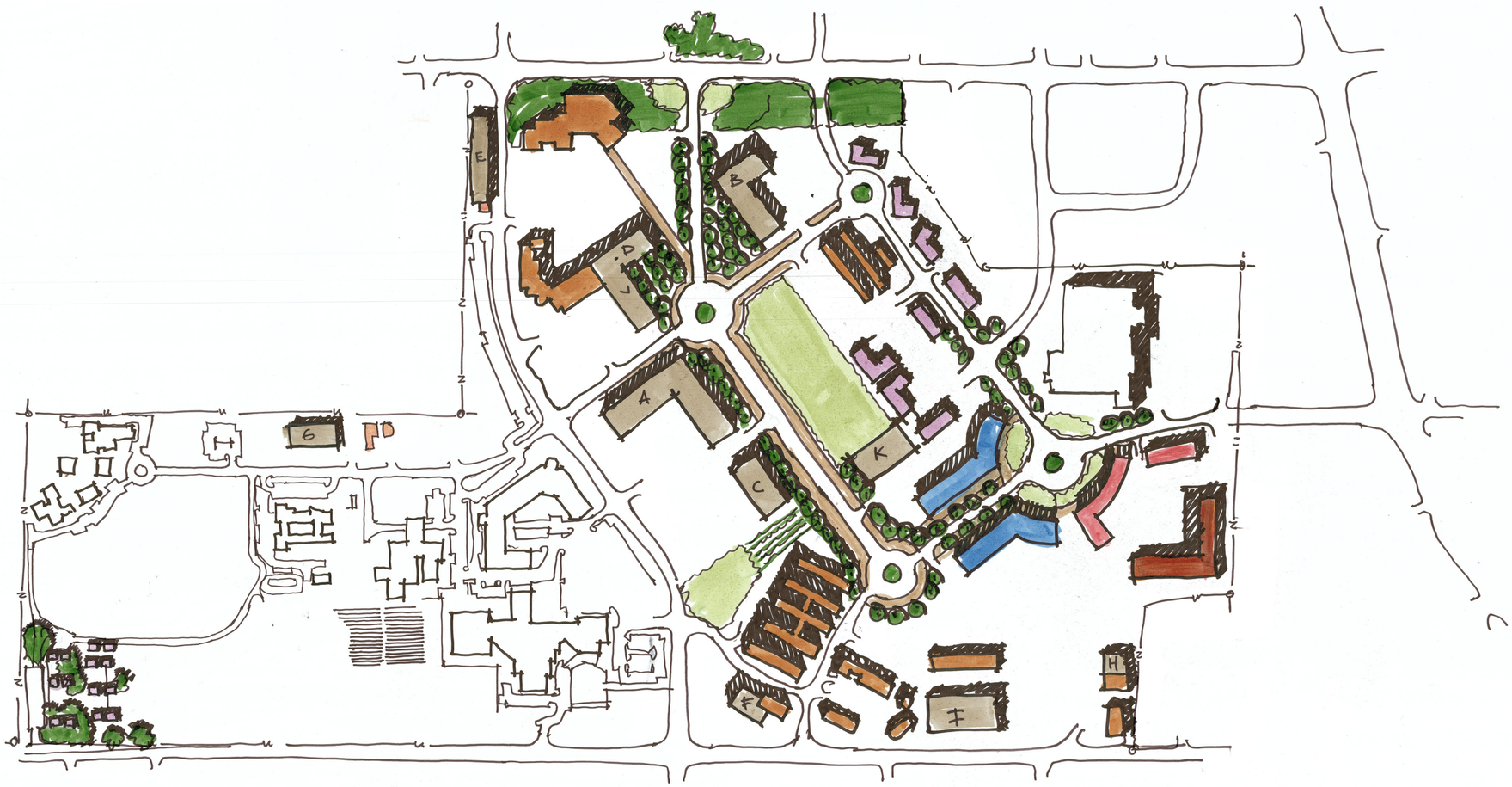


LEGEND	
A	HEALTH AND HUMAN SERVICES
B	COUNTY ADMINISTRATIVE CENTER
C	AGRICULTURAL COMMISSIONER AND FARM ADVISOR FACILITY
D	CLERK RECORDER ELECTIONS TRAINING/ WAREHOUSE FACILITY
E	MUSEUM WAREHOUSE FACILITY
F	CORPORATION YARD ADMINISTRATION AND TRAINING CENTER
G	SHERIFF AND PROBATION SUPPORT FACILITY
H	FIRE STATION 180 EXPANSION
I	ADMINISTRATION SERVICES IT/TELECOM/ WAREHOUSE
J	FINANCE ADMINISTRATION BUILDING ANNEX
K	COMMUNITY BUILDING
L	CDRC GROWTH AND CONSOLIDATION
	NEW COUNTY BUILDINGS
	RESIDENTIAL
	HOTEL
	FLEX SPACE/ MIXED-USE
	RETAIL

OPTION 2

PLACER COUNTY GOVERNMENT CENTER
AUBURN, CALIFORNIA

0 200 800
100 400



LEGEND

A	HEALTH AND HUMAN SERVICES
B	COUNTY ADMINISTRATIVE CENTER
C	AGRICULTURAL COMMISSIONER AND FARM ADVISOR FACILITY
D	CLERK RECORDER ELECTIONS TRAINING/ WAREHOUSE FACILITY
E	MUSEUM WAREHOUSE FACILITY
F	CORPORATION YARD ADMINISTRATION AND TRAINING CENTER
G	SHERIFF AND PROBATION SUPPORT FACILITY
H	FIRE STATION 180 EXPANSION
I	ADMINISTRATION SERVICES IT/TELECOM/ WAREHOUSE
J	FINANCE ADMINISTRATION BUILDING ANNEX
K	COMMUNITY BUILDING
L	CDRC GROWTH AND CONSOLIDATION

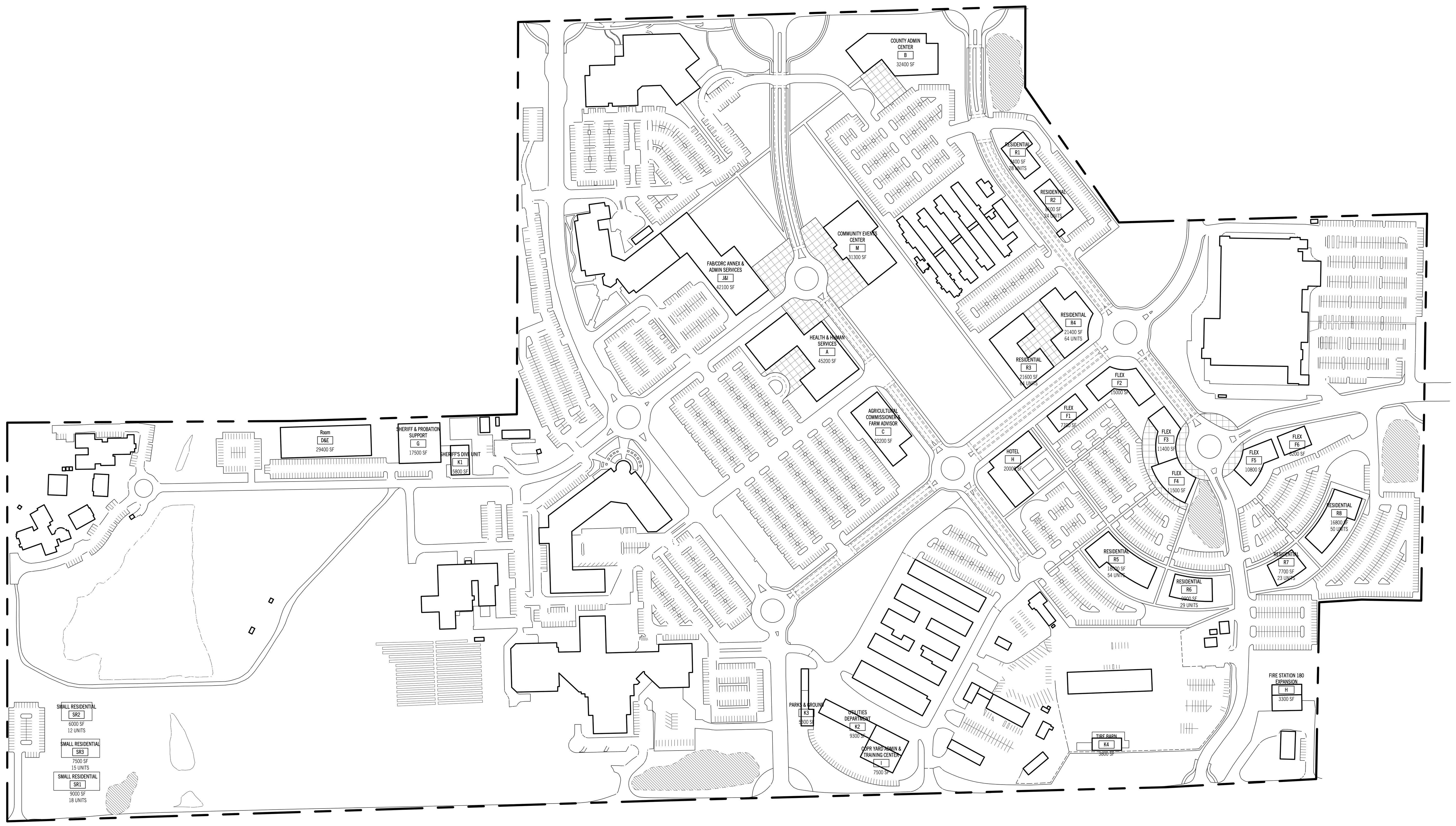
- NEW COUNTY BUILDINGS
- RESIDENTIAL
- HOTEL
- FLEX SPACE/ MIXED-USE
- RETAIL

OPTION 3

PLACER COUNTY GOVERNMENT CENTER
AUBURN, CALIFORNIA

0 200 800
100 400

Appendix C: Final Option Land Plan


COUNTY ADMIN CENTER
B
32400 SF
RESIDENTIAL
R1
9400 SF
28 UNITS
RESIDENTIAL
R2
8100 SF
24 UNITS
COMMUNITY EVENTS CENTER
M
31300 SF
FAB/CDRC ANNEX & ADMIN SERVICES
J&I
42100 SF
RESIDENTIAL
R4
21400 SF
64 UNITS
RESIDENTIAL
R3
21600 SF
64 UNITS
HEALTH & HUMAN SERVICES
A
45200 SF
FLEX
F2
15000 SF
FLEX
F1
7700 SF
AGRICULTURAL COMMISSIONER & FARM ADVISOR
C
22200 SF
HOTEL
H
20000 SF
FLEX
F3
11400 SF
FLEX
F4
11500 SF
FLEX
F5
10800 SF
FLEX
F6
6200 SF
RESIDENTIAL
R8
16800 SF
50 UNITS
RESIDENTIAL
R7
7700 SF
23 UNITS
RESIDENTIAL
R5
18000 SF
54 UNITS
RESIDENTIAL
R6
9900 SF
29 UNITS
Room
D&E
29400 SF
SHERIFF & PROBATION SUPPORT
G
17500 SF
SHERIFF'S DIVE UNIT
K1
5800 SF
PARKS & GROUND
K3
5300 SF
UTILITIES DEPARTMENT
K2
9300 SF
CDPR YARD ADMIN & TRAINING CENTER
I
7500 SF
TIRE BARN
K4
3800 SF
FIRE STATION 180 EXPANSION
H
3300 SF
SMALL RESIDENTIAL
SR2
6000 SF
12 UNITS
SMALL RESIDENTIAL
SR3
7500 SF
15 UNITS
SMALL RESIDENTIAL
SR1
9000 SF
18 UNITS

Williams + Paddon Architects + Planners Inc.

PRELIMINARY STORMWATER ANALYSIS FOR THE PLACER COUNTY GOVERNMENT CENTER – HEALTH AND HUMAN SERVICES SITE

February 7, 2018

A report prepared by Peter Kulchawik, P.E. on behalf of:

CARTWRIGHT

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FIGURE 4. POST-PROJECT UNMITIGATED HGLS IN THE RECEIVING STORM DRAIN SYSTEM
FIGURE 5. POST-PROJECT MITIGATED HGLS IN THE RECEIVING STORM DRAIN SYSTEM
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FORM 3-6 VOLUME-BASED INFILTRATING BIORETENTION MEASURES

1 INTRODUCTION

Placer County would like to move forward with a project-level Environmental Impact Report (EIR) for the Health and Human Services (HHS) Site (the 'Project'). The purpose of this report is to provide technical documentation for the analysis of the stormwater infrastructure for the Project, with particular emphasis on the hydrology and water quality checklist items in Appendix G of the California Environmental Quality Act (CEQA) Guidelines. This analysis builds directly on work completed for the PCGC Master Drainage Report (Appendix D of the Master Plan Update – Wet Utility Infrastructure, 2017); it is assumed that readers of this analysis have the Master Drainage Report available for cross reference.

2 PROJECT DESCRIPTION

The Project proposes to develop a site located in the central portion of the PCGC bounded by B Avenue to the north, D Avenue to the south, future County Center Drive to the east, and Richardson Drive to the west (Figure 1). The entire area bounded by these four streets will be developed in two phases: the HHS project will develop roughly three quarters (8 acres) of the site during the first phase, and the remaining one quarter will be developed during a future phase. Under existing conditions, runoff from the Project site generally drains to the southwest toward Richardson Drive (existing Subcatchment 2B[1]). Storm drain inlets along Richardson Drive collect the runoff and direct it to a 48-inch pipe that runs beneath the Auburn Justice Center and Jail, and ultimately leaves PCGC to the south via a culvert under Atwood Drive (Outfall C2/C3).

Under proposed conditions, drainage at the Project site will be managed in terms of three drainage management areas (DMAs; see Figure 2). The DMAs have been designed to follow the existing topography, and drainage patterns will be similar to pre-project conditions. The site will be graded to drain to two landscaped areas along Richardson Drive. The landscaped areas will include bioretention areas that will serve as the primary water-quality treatment features for the Project. The bioretention areas will have outlet control structures that collect treated runoff and convey it to the storm main in Richardson Drive. The bioretention basins are sized to treat stormwater for the HHS project as well as the future phase (DMA 3). A small portion of the site is self-treating and will drain toward B Avenue.

Stormwater in excess of the water-quality treatment volume will be conveyed to an existing stormwater basin located to the southwest of the jail building (Basin 2A). The campus-wide strategy for stormwater management described in the Master Drainage Report is to meet flood control and hydromodification management requirements[2] with several regional basins, and leave water-quality treatment requirements to individual projects as they come online. Basin 2A will provide the flow controls for the Project to meet the County criteria for flood control and hydromodification.

[1] Labeling of subcatchments, outfalls, and stormwater basins refers to nomenclature of the Master Drainage Report (Appendix D of the Master Plan Update – Wet Utility Infrastructure, 2017).
[2] The Placer County criteria for hydromodification management is to control post-construction runoff for the 2-year, 24-hour event to be less than or equal to the pre-project runoff rate.

3 HYDROLOGIC AND HYDRAULIC MODELING

3.1 ANALYSES COMPLETED

The campus-scale XPSTORM model developed for the PCGC Master Drainage Report (Appendix D of the Master Plan Update – Wet Utility Infrastructure, 2017) was adapted to analyze hydrology of the Project site and the hydraulics of the receiving storm drain system. An updated site plan was provided by Williams + Paddon Architects + Planners (dated December 13, 2017), and was used to revise the land use parameters and subcatchment boundaries of the XPSTORM model. The revised subcatchments were further divided into DMAs. The XPSTORM model was used to assess the following three issues:

1. Will the Project increase peak flow rates for the 2-, 10-, and 100-year events at the outfalls from PCGC beyond what was predicted for the Final Option in the Master Drainage Report?

2. Will the Project cause the hydraulics of the receiving storm drain system to be out of compliance with County criteria?

3. Will the planned regional detention facilities provide flood and hydromodification controls for the Project as well as the Final Option buildout condition?

Water quality features for the Project were sized using the stormwater quality templates described in the West Placer Storm Water Quality Design Manual (2016). DMAs were estimated based on a preliminary grading plan prepared by the Cartwright team.

3.2 RESULTS

Changes to the hydrology and hydraulics caused by the Project were analyzed in the context of the entire PCGC stormwater system. While this approach allows for comprehensive planning of the stormwater system, studying the impact from an individual project can be challenging since flow in any given pipe of the XPSTORM model can include runoff from the Project as well as from other parts of the campus. To overcome this challenge, results from the XPSTORM model are presented at analysis points along the storm drain system. Two analysis points were selected to quantify changes to the PCGC hydrology: (1) at the 48-inch storm main that runs beneath the jail building (Link 24.1) and before passing through Basin 2A for flood and hydromodification controls, and (2) Outfall C2/3 to quantify the change in runoff rates leaving the PCGC campus after passing through Basin 2A.

Table 1 – XPSTORM model results for peak flow rates downstream of the Project

Analysis Point	Pre-Project Peak Flow Rate (cfs)			Post-Project Peak Flow Rate (cfs)		
	2-Year	10-Year	100-Year	2-Year	10-Year	100-Year
1 (Link 24.1)	24.1	46.1	66.4	35.9	64.6	91.7
2 (Outfall C2/3)	23.4	41.4	67.0	21.2	39.3	67.0

The model results in Table 1 show that the Project will cause peak flow rates to increase by as much as 48 percent immediately downstream of the Project, but the flow and hydromodification controls at Basin 2A will limit post-project flow rates to no greater than pre-project flow rates. Since Basin 2A

provides all of the required detention for flood control and hydromodification requirements, supplemental detention is not needed within the Project site. The bioretention basins will provide additional detention, however, the total storage volume is small and the bioretention basins were not included in the XPSTORM model.

The Master Drainage Report indicated that Basin 2A does not have sufficient freeboard under existing conditions to meet the 1-foot minimum freeboard requirement during the 100-year event, and that the volume of the basin would need to be increased to accommodate the additional runoff from the Final Option and meet the freeboard requirement. The model showed that the peak water surface elevation during the 100-year event in Basin 2A would increase by 0.5 feet once the Project is implemented (without altering the basin to increase its volume), which would leave roughly 0.3 feet of freeboard. Basin 2A should be regraded to increase its volume by 1.2 acre-feet as described in the Master Drainage Report. Regrading should include increasing the basin footprint by roughly 8,700 square feet to increase the storage volume in lower elevation bands of the basin. Doing so will allow Basin 2A to control flows to no greater than pre-project conditions with no alterations to the current outlet structure. It is important to note that according to the Williams + Paddon land plan (dated April 5, 2017), the HHS project represents the largest increase in impervious area within Catchment 2. Any changes to the land plan involving additional new impervious area in Catchment 2 should be carefully evaluated against the flow controls in Basin 2A, particularly for 100-year for which all available storage in Basin 2A will be fully utilized once the HHS project is implemented.

The Placer County SWMM requires that the hydraulic grade line (HGL) for the 10-year event be no higher than 6 inches below manhole covers and other inlets. The storm main that begins along B Avenue, turns down Richardson Drive, flows beneath the jail building, and terminates in Basin 2A, conveys nearly all runoff from the Project site under pre- and post-project conditions. The 10-year event output from the XPSTORM model for this storm main is shown in Figure 3 for pre-project conditions and Figure 4 for unmitigated post-project conditions. Under pre-project conditions, the storm main meets the County criteria for the 10-year event with the exception of Node 2D/2 which has only 4.5 inches of freeboard to the inlet grate. The existing non-compliant portion of the storm drain system was noted in the Master Drainage Report to cause shallow surface flooding during the 100-year event. Under post-project conditions, the model showed the HGL in Node 2D/2 (without mitigation) will rise leaving only 2 inches of freeboard to the inlet grate. The cause of the increase is likely due to more backwater in downstream links. The B Avenue storm drain system should be reconfigured as part of the Project to meet the County criteria for the 10-year event. The XPSTORM model showed that by replacing Link58 with a smooth-walled, high-density polyethylene (HDPE) pipe[3] the capacity of the system would be increased, and the 10-year HGL at Node 2D/2 would meet County standards (see Figure 5 for model results). Aside from Node 2D/2, the model showed that the Project would cause HGLs in the rest of the receiving storm drain system to increase, but not to the point of putting the storm main out of compliance with the 6-inch freeboard criteria.

The Project will meet requirements for water quality-treatment with two bioretention basins (BIO 1 and BIO 2, see Figure 2) along Richardson Drive. The bioretention basins will provide water-quality treatment by capturing the water-quality volume (the post-construction 85th percentile, 24-hour

[3] A Mannings roughness value of 0.011 was assumed for the HDPE pipe in the XPSTORM model.

storm event) and allowing it to infiltrate through a bio-soil layer. Once runoff passes through the bio-soil layer, it will flow through a gravel layer where it is collected in an underdrain and routed to the storm main. Additional water-quality features may be included in the final design (for instance, site design and source control measures), however, this analysis focused on the calculations to support the sizing of stormwater treatment features only. Detailed sizing calculations are enclosed in Forms 3-5 and 3-6 from the stormwater quality templates from the West Placer Storm Water Quality Design Manual (2016). The bioretention basin in DMA 2 (BIO 2) is sized to treat runoff from DMA 2 and DMA 3. DMA 3 is not part of the Project and will be constructed during a later phase, but BIO 2 is sized to treat runoff from future development in DMA 3.

4 CEQA CHECKLIST ITEMS

From the results of the stormwater analysis described above, responses to the most pertinent hydrology and water quality checklist items in Appendix G of the CEQA guidelines are summarized as follows:

- *Does the project violate any water quality standards or waste discharge requirements?* Water-quality features for the Project were designed according to the guidance of the West Placer County Storm Water Design Manual (2016). The purpose of the Manual is to provide regulatory guidance to comply with the Clean Water Act and the Phase II MS4 Permit (SWRCB Water Quality Order No. 2013-001-DWQ), so by following the design guidance of the Manual the Project will not violate any water-quality standards.

- *Does the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, in a manner which would result in substantial erosion or siltation on- or off-site?* The Project will disturb more than one acre and will require a Storm Water Pollution Prevention Plan (SWPPP). Although beyond the scope of this document, the SWPPP will provide guidance for stabilizing the Project site to prevent on-site erosion immediately following construction. Runoff from the Project does not enter a natural drainageway until it leaves the PCGC campus at the Atwood Drive culvert. Flows will have passed through Basin 2A for flood and hydromodification controls prior to leaving the campus and will not exceed pre-project levels. As such, the erosive energy in off-site natural drainageways will be no greater than under pre-project conditions.

- *Does the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, or substantially increase the rate or amount of surface runoff in a manner which would result in flooding on- or off-site?* The grading plan for the Project generally follows the existing topography, and will not substantially alter drainage patterns. The on-site storm drain system along B Avenue will need to be reconfigured to comply with County criteria for 10-year HGLs; recommendations are included in this report. Off-site flooding will be mitigated through flood control at Basin 2A.

- *Does the project create or contribute runoff water which would exceed the capacity of existing or planned stormwater drainage systems or provide substantial additional sources of polluted runoff?* The storm drain modeling indicates that, without mitigation, the Project will cause the HGL in Node

2D/2 during the 10-year event to increase. The increase will result in less freeboard to the grate elevation, but is not anticipated to cause surface flooding. The B Avenue storm drain will need to be reconfigured (replace Link58 with an HDPE pipe) to comply with County criteria for 10-year HGLs. The model showed the remainder of the receiving storm drain system to have sufficient capacity to convey the post-project flow rates while meeting the County's hydraulic criteria for design of drainage conduits. All runoff from the Project will be routed through water-quality features prior to entering the storm drain system.

- *Does the project otherwise substantially degrade water quality?* Water-quality features for the Project were designed according to the guidance of the West Placer County Storm Water Design Manual (2016). Additional water-quality features may be incorporated into the site design as the Project advances. A Post-Construction Storm Water Quality Plan must be submitted to the County as part of the approval process, and will include a comprehensive description of all on-site water-quality features.

- *Does the project place housing within a 100-year flood hazard area as mapped on a federal Flood Hazard Boundary or Flood Insurance Rate Map or other flood hazard delineation map?* The Project site appears on FEMA Flood Insurance Rate Map (FIRM) Number 06061C0275F (effective June 8, 1998), and is not mapped as a special flood hazard area inundated by the 100-year flood.

- *Does the project expose people or structures to a significant risk of loss, injury or death involving flooding, including flooding as a result of the failure of a levee or dam?* The drainage system for the Project will be designed according to County standards to avoid property damage and loss of life. The Project site is not protected by levees, and is not within a mapped dam failure inundation zone.

5 CONCLUSIONS

The XPSTORM model developed for the PCGC Master Drainage Report was adapted to analyze changes to the hydrology and hydraulics of the PCGC campus from the HHS Project. The model showed that the regional stormwater basin (Basin 2A) will provide adequate flood and hydromodification controls, but will need to be reconfigured to comply with County criteria for freeboard during the 100-year event by expanding the basin to increase the volume by 1.2 acre-feet. The receiving storm drain system will have sufficient capacity to convey the intended flow rates with minor modifications, as recommended in this report, to a conduit along B Avenue. The on-site water quality features have been sized to provide treatment for the Project, as well as for future development in DMA 3. The discussion of the CEQA checklist items showed that with proper implementation of the controls for stormwater quantity and quality presented herein, the Project would not adversely affect water quality, property damage, or risk of loss of life.

This hydrologic and hydraulic modeling was prepared in general accordance with the accepted standard of practice in surface-water hydraulics existing in Northern California for projects of similar scale at the time the investigations were performed. No other warranties, expressed or implied, are made. Hydrologic and hydraulic modeling are difficult and inexact arts, and a variety of physical factors can affect the results from what has been presented herein; in particular, inaccuracies in storm drain mapping, geomorphic change, redevelopment, and groundwater levels. Concepts and findings contained in this report are

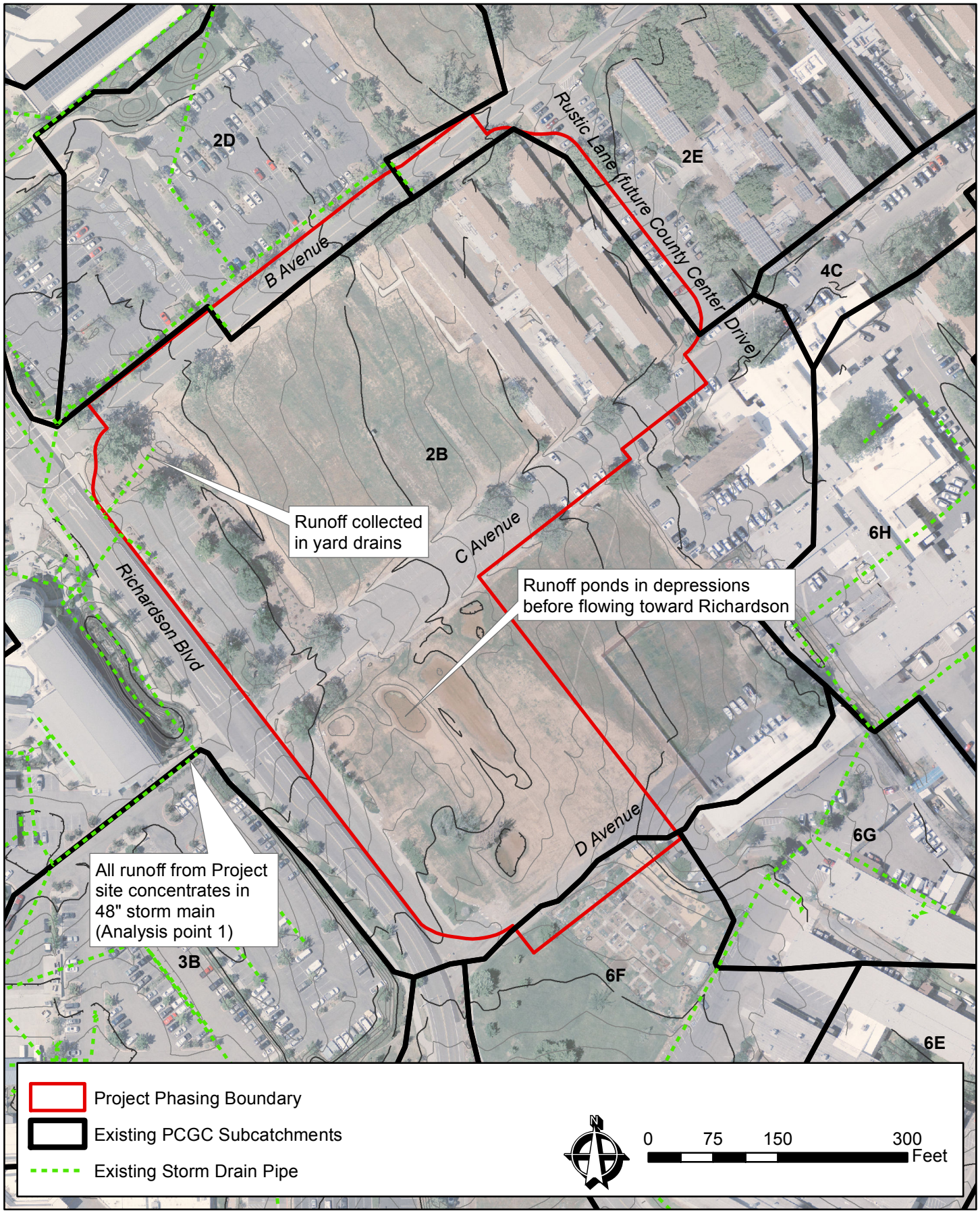
intended for characterizing the impacts to the PCGC storm drain system from the HHS Project only, and should not be used for other purposes without great care, updating, review of analytical methods used, and consultation with the authors. Finally, readers who have additional pertinent information, who observed changed conditions, or who may note material errors should contact the author with their findings at the earliest possible date, so that timely changes may be made.

REFERENCES:

cbec eco engineering, inc. and CDM Smith, 2016, West Placer Storm Water Quality Design Manual, 68 p + appendices.

Kulchawik, P. and Garrison, T., 2017, Appendix D of the Master Plan Update – Wet Utility Infrastructure: draft report submitted to Cartwright Engineers, June 12, 2017, 22 p + attachments.

Placer County Flood Control and Water Conservation District, 1994, Stormwater Management Manual, 135 p + appendices.



Preliminary Stormwater Analysis
Health and Human Services Site
Placer County Government Center

Figure 1. Pre-project drainage,

Basemap source: Andregg Geomatics

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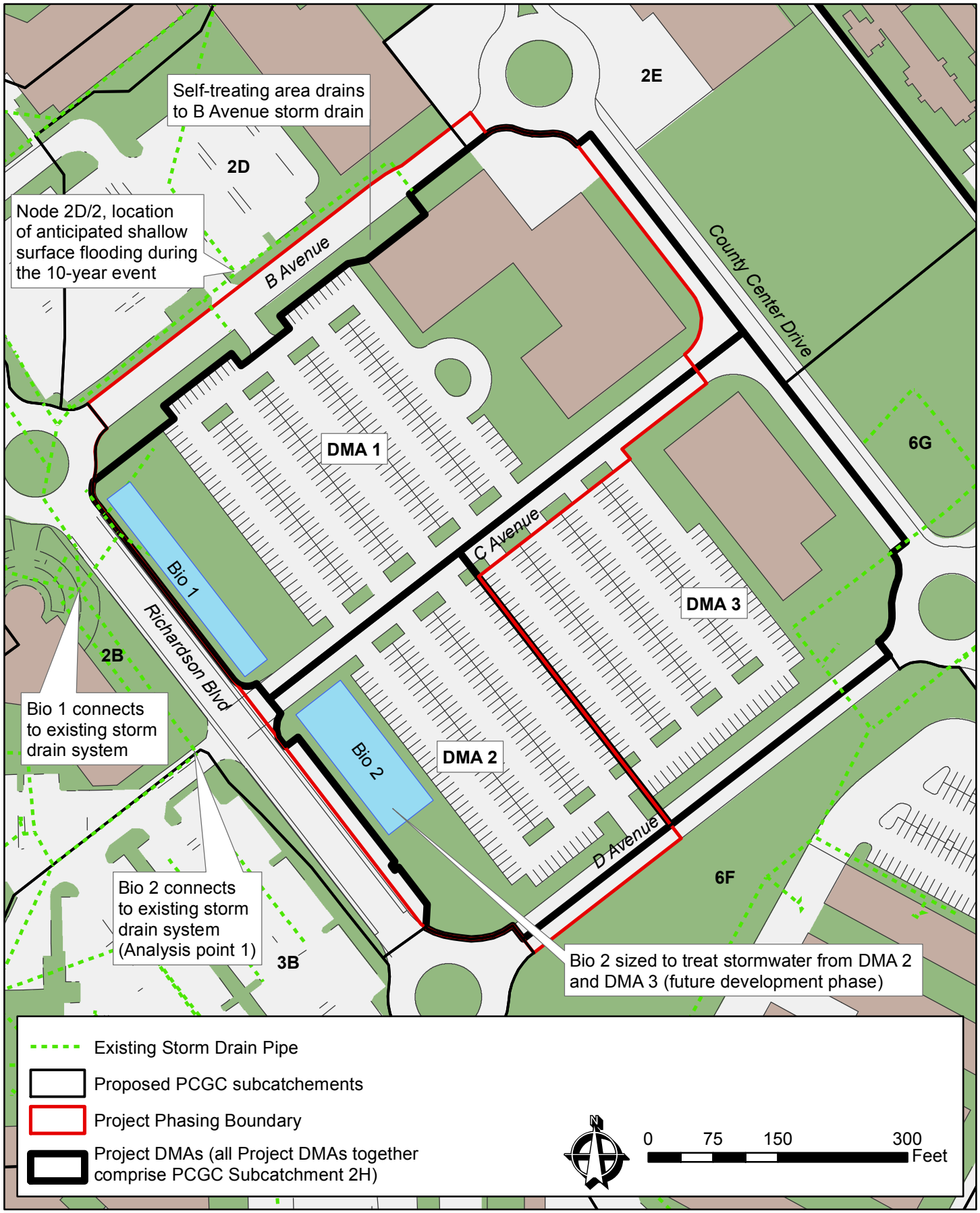

Preliminary Stormwater Analysis
Health and Human Services Site
Placer County Government Center

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Figure 2. Post-project drainage

Basemap adapted from Williams + Paddon

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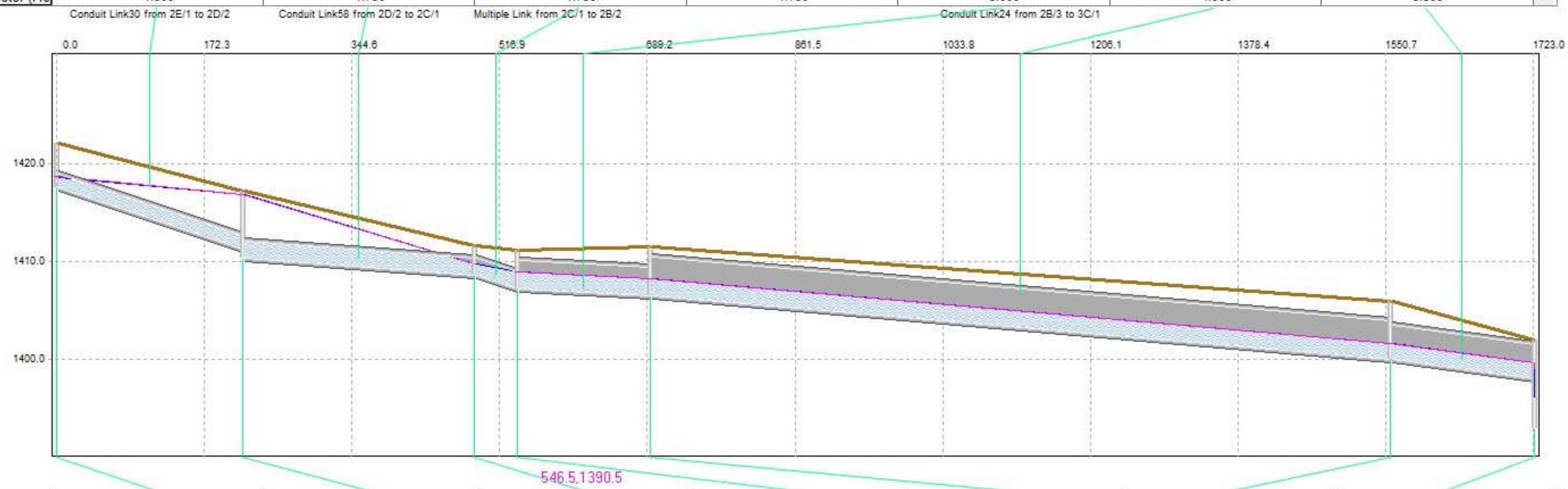
links

	Link30	Link58	21a	21b	Link26	Link24	Link92
Storm	10yr	10yr	10yr	10yr	10yr	10yr	10yr
Max Flow	7.948	22.755	0.000	0.000	27.513	46.861	46.920
Conduit Slop	2.913	0.668	2.706	2.706	0.455	0.753	1.198
Shape	Circular	Circular	Circular	Circular	Circular	Circular	Circular
Length	217.990	269.500	51.000	51.000	153.800	863.670	167.000
Diameter (He	1.500	1.750	1.750	1.750	3.000	4.000	3.500



	2E/1	2D/2	2C/1	2B/2	2B/3	3C/1	2A/Storage1
Storm	10yr	10yr	10yr	10yr	10yr	10yr	10yr
Ground Eleva	1422.002	1417.188	1411.605	1411.169	1411.460	1405.920	1402.000
Max Water El	1418.653	1416.814	1409.717	1408.990	1408.314	1401.666	1397.236

Figure 3. Pre-project HGLs in the receiving storm drain system

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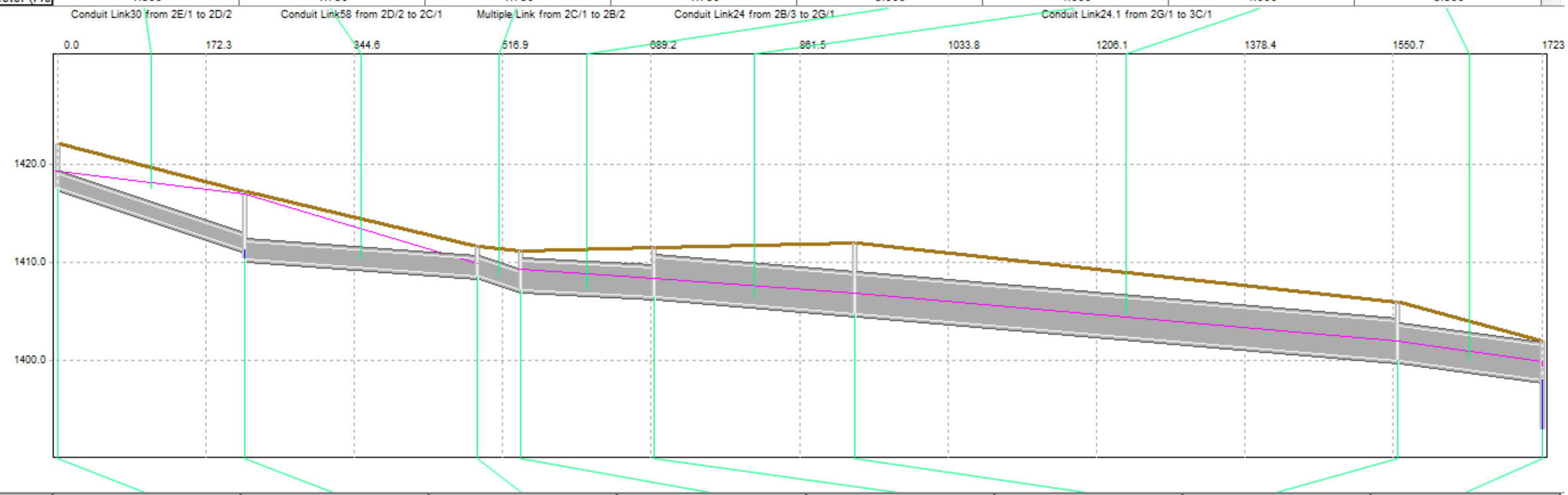
links

	Link30	Link58	21a	21b	Link26	Link24	Link24.1	Link92
Storm	10yr	10yr	10yr	10yr	10yr	10yr	10yr	10yr
Max Flow	10.160	24.146	0.000	0.000	37.429	52.079	61.571	61.536
Conduit Slop	2.913	0.668	2.706	2.706	0.455	0.753	0.753	1.198
Shape	Circular	Circular	Circular	Circular	Circular	Circular	Circular	Circular
Length	217.990	269.500	51.000	51.000	153.800	233.670	630.000	167.000
Diameter (He	1.500	1.750	1.750	1.750	3.000	4.000	4.000	3.500



	2E/1	2D/2	2C/1	2B/2	2B/3	2G/1	3C/1	2A/Storage1
Storm	10yr	10yr	10yr	10yr	10yr	10yr	10yr	10yr
Ground Eleva	1422.002	1417.188	1411.605	1411.169	1411.460	1411.950	1405.920	1402.000
Max Water El	1419.240	1417.009	1409.901	1409.327	1408.426	1406.851	1401.958	1397.791

Figure 4. Post-project unmitigated HGLs in the receiving storm drain system

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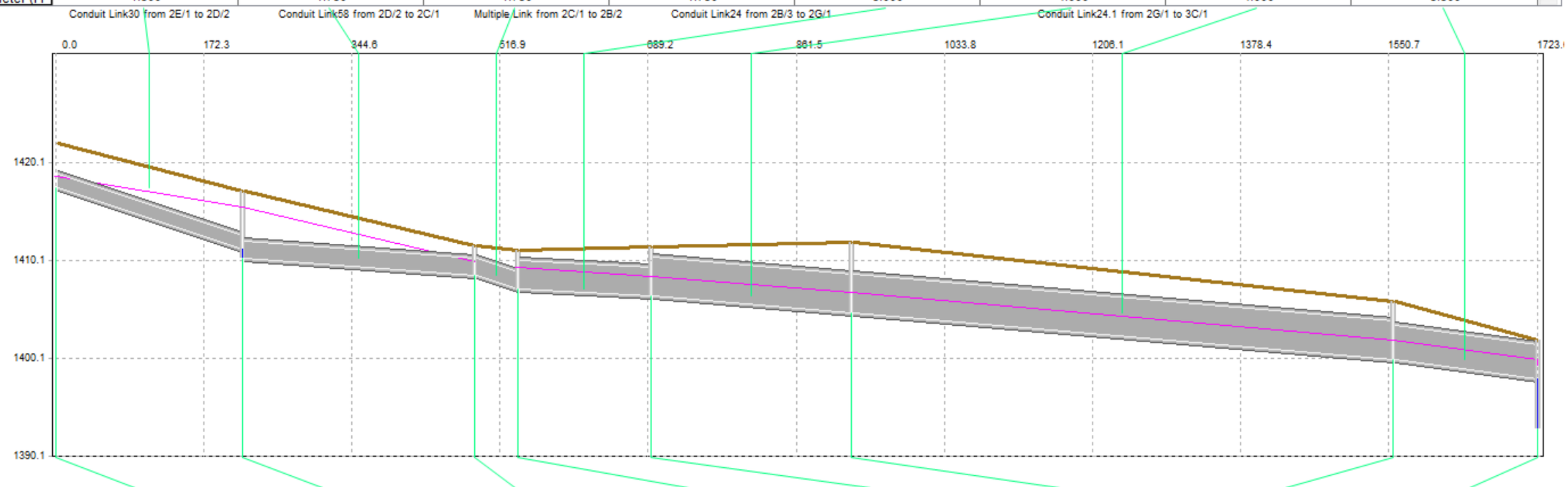
links

	Link30	Link58	21a	21b	Link26	Link24	Link24.1	Link92
Storm	10yr	10yr	10yr	10yr	10yr	10yr	10yr	10yr
Max Flow	10.158	27.084	0.000	0.000	40.376	55.031	64.561	64.568
Conduit Slop	2.913	0.668	2.706	2.706	0.455	0.753	0.753	1.198
Shape	Circular	Circular	Circular	Circular	Circular	Circular	Circular	Circular
Length	217.990	269.500	51.000	51.000	153.800	233.670	630.000	167.000
Diameter (H	1.500	1.750	1.750	1.750	3.000	4.000	4.000	3.500



	2E/1	2D/2	2C/1	2B/2	2B/3	2G/1	3C/1	2A/Storage1
Storm	10yr	10yr	10yr	10yr	10yr	10yr	10yr	10yr
Ground Eleva	1422.002	1417.188	1411.605	1411.169	1411.460	1411.950	1405.920	1402.000
Max Water El	1418.653	1415.520	1410.045	1409.431	1408.489	1406.911	1402.019	1397.373

Figure 5. Post-project mitigated HGLs in the receiving storm drain system

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Form 3-5 Computation of Water Quality Design Criteria for Stormwater Treatment and Baseline Hydromodification Measures

DMA ID No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
[1]Total impervious area requiring treatment	184009	70540	139473																			
[2] Impervious area untreated by Site Design Measures (ft^2) *Item 1 – Form 3-4 Item 11*	184009	70540	139473	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[3]Additional pervious area draining to BMP (ft^2)	70540	39888	21495																			
[4] Composite DMA Runoff Coefficient (Rc) *Enter area weighted composite runoff coefficient representing entire DMA*	0.77	0.70	0.83																			
[5] Water Quality Volume (WQV) (ft^3) *WQV = 1/12 * [Item 2 + Item 3) *Item 4] * Unit WQV*	10617	4187	7237	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[6] Water Quality Flow (WQF) (cfs) *WQF = 1/43,200 * [0.2* (Item 2 + Item 3) * Item4]*	0.907	0.358	0.619	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

DMA ID No.	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
[1]Total impervious area requiring treatment																						
[2] Impervious area untreated by Site Design Measures (ft^2) *Item 1 – Form 3-4 Item 11*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[3]Additional pervious area draining to BMP (ft^2)																						
[4] Composite DMA Runoff Coefficient (Rc) *Enter area weighted composite runoff coefficient representing entire DMA*																						
[5] Water Quality Volume (WQV) (ft^3) *WQV = 1/12 * [Item 2 + Item 3) *Item 4] * Unit WQV*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[6] Water Quality Flow (WQF) (cfs) *WQF = 1/43,200 * [0.2* (Item 2 + Item 3) * Item4]*	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Form 3-6 Volume-Based Infiltrating Bioretention Measures				
[1]**DMA ID No.** *If combining multiple DMAs from Form 3-5, enter a new unique DMA ID No.*	**1**	**2**	**3**	**4**
[2]WQV (ft^3) *Item 5 in Form 3-5* *If combining multiple DMAs from Form 3-5, enter the sum of their respective WQVs.*	10617	4187	7237	
[3]Surface Loading Rate *Maximum 5.0 in/hr*	5	5	5	
[4]BMP Surface Area (ft^2) *Top of BMP*	6500	3500	6000	
[5]Infiltration rate of underlying soils (in/hr)	0.16	0.16	0.16	
[6]Maximum ponding depth (ft) *BMP specific, see BMP design details*	0.5	0.5	0.5	
[7]Ponding Depth (ft) d_{BMP} *= Minimum of (1/12 * Item 5 * 48 hrs) or Item 6*	0.5	0.5	0.5	-
[8]Infiltrating surface area, SA_{BMP} (ft^2) *Bottom of BMP*	8500	3500	6000	
[9]Planting media depth, d_{media} (ft)	1.5	1.5	1.5	
[10]Planting media porosity	0.30	0.30	0.30	
[11]Gravel depth, d_{media} (ft) *Only included in certain BMP types*	1.0	1.0	1.0	
[12]Gravel porosity	0.30	0.30	0.30	
[13]Retention Volume (ft^3) $V_{retention}$ *= Item 8 * [Item7 + (Item 9 * Item 10) + (Item 11 * Item 12) + (1.5* (Item 5 / 12))]*	10,625.0	4,375.0	7,500.0	-
[14]Untreated Volume (ft^3) $V_{untreated}$ *= Item 2 – Item 13* *If greater than zero, adjust BMP sizing variables and re-compute retention volume*	0	0	0	0
[15] Treated Flow Rate (ft^3/s) $Q_{treated}$ *= 1/43,200*(Item 3 * Item 4)*	0.7523	0.4051	0.6944	0.0000
[16]Total Treated Flow Rate for Project (ft^3/s) Q_{total} *= Sum of Item 15 for all DMAs*	2.0254			
[17]**Is WQV for each DMA treated on-site?** *Check Yes if Item 14 equals 0 for all DMAs*	**Yes**	**X**	**No**	

Williams + Paddon Architects + Planners Inc.

PRELIMINARY STORMWATER ANALYSIS FOR THE PLACER COUNTY GOVERNMENT CENTER – AFFORDABLE HOUSING SITE

April 12, 2018

A report prepared by Peter Kulchawik, P.E. and Teresa Garrison, P.E. on behalf of:

CARTWRIGHT

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1 INTRODUCTION

Placer County would like to move forward with a project-level Environmental Impact Report (EIR) for the Affordable Housing Site (AHS; referred to herein as the 'Project'). The purpose of this report is to provide technical documentation for the analysis of the stormwater infrastructure for the Project, with particular emphasis on the hydrology and water quality checklist items in Appendix G of the California Environmental Quality Act (CEQA) Guidelines. This analysis builds directly on work completed for the PCGC Master Drainage Report (Appendix D of the Master Plan Update – Wet Utility Infrastructure, 2017); it is assumed that readers of this analysis have the Master Drainage Report available for cross reference.

2 PROJECT DESCRIPTION

The Project proposes to develop an approximately 3-acre (net acreage) site located along 1st Street in the northeast portion of the PCGC, east of the intersection of B Avenue and 1st Street. The improvements will also include realignment of a portion of the northern end of 1st Street. Under existing conditions runoff from the Project site drains in three directions (**Figure 1**). The north portion of the site (existing Subcatchment 4A[1]) drains to a 24-inch pipe (Outfall C4) that connects to the storm drain system for the Bell Professional Plaza. The central portion of the site sheet flows to the east toward the Oak Knoll Professional Center. The south portion of the site (existing Subcatchment 5D) drains to the south toward an existing stormwater basin at the corner of 1st Street and Professional Drive. The Ophir Canal runs along the west margin of the Project site parallel to 1st Street; the Canal receives direct precipitation, but is not part of the PCGC storm drain system. Rainfall that falls directly onto the canal was not taken into account in the pre-project modeling.

Under proposed conditions the northern third of the site will drain to the 24-inch pipe (Outfall C4) along the boundary of the PCGC campus, and the southern two-thirds of the site will drain to Basin 5D. A stormwater basin (Basin 4A) will be constructed on the northern drainage to control flow rates to meet Placer County flood control and hydromodification requirements. The proposed Project layout is presented in **Figure 2**. The Project site will collect runoff for the existing uncontrolled central portion of the site and, as such, the Project will not adversely affect erosion, siltation, or flooding to those areas. Erosion, siltation, and flooding to the storm drain system downstream of Outfall C4 will be mitigated by flow controls in the proposed Basin 4A and water-quality features on the Project site. The Project will enclose approximately 850 linear feet of the Ophir Canal in an underground conduit. The conduit will be designed to convey the anticipated canal flow rates, and the enclosed portion will no longer receive direct precipitation. Neither of these factors are anticipated to adversely affect the performance of the canal.

Though the site presently contains some impervious cover in the form of parking lots, overall impervious cover will increase, and Basin 5D and the new Basin 4A will fully mitigate any impacts related to the land use changes.

[1] Labeling of subcatchments, outfalls, and stormwater basins refers to nomenclature of the Master Drainage Report (Kulchawik and Garrison, 2017).

3 HYDROLOGIC AND HYDRAULIC MODELING

3.1 ANALYSES COMPLETED

The campus-scale XPSTORM model developed for the PCGC Master Drainage Report (Appendix D of the Master Plan Update – Wet Utility Infrastructure, 2017) was adapted to analyze hydrology of the Project site and the hydraulics of the receiving storm drain system. An updated site plan was provided by Williams + Paddon Architects + Planners (dated March 21, 2018), and was used to revise the land use parameters and subcatchment boundaries of the XPSTORM model. The revised subcatchments were further divided into drainage management areas (DMAs). The XPSTORM model was used to assess the following three issues:

1. Will the Project increase peak flow rates for the 2-, 10-, and 100-year events at the outfalls from PCGC beyond what was predicted for the Final Option in the Master Drainage Report?

2. Will the Project cause the hydraulics of the receiving storm drain system to be out of compliance with County criteria?

3. Will the planned regional detention facilities provide flood and hydromodification controls for the Project as well as the Final Option buildout condition?

Water-quality features for the Project were sized using the stormwater quality templates described in the West Placer Storm Water Quality Design Manual (2016). DMAs were estimated based on a preliminary grading plan prepared by the Cartwright team.

3.2 RESULTS

Changes to the hydrology and hydraulics caused by the Project were analyzed in the context of the entire PCGC stormwater system. While this approach allows for comprehensive planning of the stormwater system, studying the impact from an individual project can be challenging since flow in any given pipe of the XPSTORM model can include runoff from the Project as well as from other parts of the campus. To overcome this challenge, results from the XPSTORM model are presented at analysis points along the storm drain system. Two analysis points were selected to quantify changes to the PCGC hydrology: (1) Outfall C4 to quantify the change in runoff rates for the northern portion of the site, and (2) Outfall C5 to quantify the change in runoff rates for the southern portion of the site.

Table 1 – XPSTORM model results for peak flow rates downstream of the Project

Analysis Point	Pre-Project Peak Flow Rate (cfs)			Post-Project Peak Flow Rate (cfs)		
	2-Year	10-Year	100-Year	2-Year	10-Year	100-Year
1 (Outfall C4)	9.0	17.7	29.4	8.1	16.5	29.3
2 (Outfall C5)	9.2	20.1	34.2	9.1	20.0	34.2

The model results in Table 1 show that the Project will not cause an increase in peak flows at Outfalls C4 or C5. The bioretention basins and vegetated swales located throughout the Project will provide additional detention, however, the total storage volume of these features is small so they were not included in the XPSTORM model, with the exception of Basin 4A which functions as both a detention

basin and bioretention basin.

The Master Drainage Report indicated that Basin 5D does not have sufficient freeboard under existing conditions to meet the 1-foot minimum freeboard requirement during the 100-year event, and that the volume of the basin would need to be increased to accommodate the additional runoff from the Final Option and meet the freeboard requirement. The model showed that the peak water surface elevation during the 100-year event in Basin 5D would increase by 0.1 feet once the Project is implemented (without altering the basin to increase its volume), which would leave roughly 0.6 feet of freeboard. The far eastern corner of Basin 5D is proposed to be regraded to increase its freeboard to meet the County requirement, with the grading shown on the Cartwright Preliminary Grading and Drainage Plan (enclosed). An addition of as little as 0.4 feet of grading would allow the basin to detain the proposed development and meet the County freeboard requirement. Re-grading the eastern corner will increase the capacity of Basin 5D allowing it to control flows to no greater than pre-project conditions with no alterations to the current outlet structure. It is important to note that the area graded to drain to Basin 5D was maximized in the buildout design of the AHS Project. Any changes to the Williams + Paddon land plan (dated April 5, 2017) involving additional new impervious area in Catchment 5 should be carefully evaluated against the flow controls in Basin 5D, particularly for the 100-year design storm for which all available storage in Basin 5D will be fully utilized once the AHS project is implemented.

Flows directly leaving Basin 5D will increase slightly for the 100-year event from 4.06 cfs to 4.13 cfs, for a 0.07 cfs increase. However, the peak release at Basin 5D is offset from the peak flow leaving Outfall C5 by roughly 1.5 hours. As a result, the increase in flow leaving Basin 5D has no impact on the 100-year peak flow at Outfall C5.

The southern portion of the Project will meet requirements for water-quality treatment with a bio-swale (SWALE 1) and two bioretention basins (BIO 1 and BIO 2,) within the site. See **Figure 3** for the bioretention basin locations and the drainage management areas (DMAs). The bioretention basins will provide water-quality treatment by capturing the water-quality volume (the post-construction 85th percentile, 24-hour storm event) and allowing it to infiltrate through a bio-soil layer. Once runoff passes through the bio-soil layer, it will flow through a gravel layer where it is collected in an underdrain and routed to Basin 5D. The vegetated swales are site design measures sized per the requirements of the West Placer County Stormwater Quality Design Manual and flows to Basin 5D after treatment (see attached Calculations for sizing parameters). Additional water-quality features may be included in the final design (for instance, site design and source control measures), however, this analysis focused on the calculations to support the sizing of stormwater treatment features only. Detailed sizing calculations are enclosed in Forms 3-4, 3-5 and 3-6 from the stormwater quality templates from the West Placer Storm Water Quality Design Manual (2016).

DMA 1 will be treated by Basin 4A which has a bioretention basin as the bottom portion of the detention basin. Basin 4A was designed for the northern portion of the Project site and has a capacity of 13,000 cubic feet. Basin 4A is proposed to be three feet deep including 1-foot of freeboard to meet the 100-year County requirement. Basin 4A will also serve as a bioretention basin to provide water-quality treatment for the proposed AHS developments in Catchment 4. DMA 2 is treated through a vegetated swale (SWALE 2) and sized to meet the design requirements of Fact Sheet SDM-6 from the

West Placer County Stormwater Design Manual. DMA 3 is a self-treating area with no impervious surfaces, thus is not accounted for in the LID facilities. DMA 4 splits and is treated by BIO 1 and BIO 2. DMAs 5, 6, and 7 flow south and are treated in SWALE 1. DMA 8 is a self-treating area with no impervious surfaces, thus is not accounted for in the LID facilities.

Under interim conditions, existing flows from the west side of existing 1st Street will continue to be collected in the existing storm drain network and will be routed through the Project site in a new 24-inch diameter pipe that bypasses Basin 4A. This interim scenario with the new Basin 4A and the bypass pipe were modeled in XPSTROM and showed no increase in peak flow rates at Outfall C4 (Table 1). Under ultimate build out conditions of the Master Plan and based on the full buildout of Catchment 4A (Master Drainage Report), the existing storm system will require additional modifications and additional detention storage (other than Basin 4A) will be required.

The sizing of the water-quality features was based on the proposed impervious surface as part of the planning phases of this project. The current design for DMAs 1, 4, 5, 6 and 7 do not consider any Site Design Measures[2] (soil quality improvements, tree planter, etc.) which would reduce the needed water-quality treatment volume and potentially the footprint of the proposed bioretention facilities. During the planning phase of the Project sufficient space and volume have been set aside to meet the County water-quality requirements, however it is recommended that the applicability and use of Site Design Measures be investigated for final development.

4 CEQA CHECKLIST ITEMS

From the results of the stormwater analysis described above, responses to the most pertinent hydrology and water quality checklist items in Appendix G of the CEQA guidelines are summarized as follows:

- *Does the project violate any water quality standards or waste discharge requirements?* Water-quality features for the Project were designed according to the guidance of the West Placer County Storm Water Design Manual (2016). The purpose of the Manual is to provide regulatory guidance to comply with the Clean Water Act and the Phase II MS4 Permit (SWRCB Water Quality Order No. 2013-001-DWQ). By following the design guidance of the Manual, the Project will not violate any water-quality standards.

- *Does the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, in a manner which would result in substantial erosion or siltation on- or off-site?* The Project will disturb more than one acre and will require a Storm Water Pollution Prevention Plan (SWPPP). Although beyond the scope of this document, the SWPPP will provide guidance for stabilizing the Project site to prevent on-site erosion during and immediately following construction. Runoff from the Project does not enter a natural drainageway until it leaves the PCGC campus. Flows will have passed through water-quality features and Basin 5D or Basin 4A for flood and hydromodification controls prior to leaving the

[2] Site Design Measures as defined and used in the West Placer Storm Water Quality Design Manual, 2016.

campus and will not exceed pre-project levels. As such, the erosive energy in off-site natural drainageways will be no greater than under pre-project conditions.

- *Does the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, or substantially increase the rate or amount of surface runoff in a manner which would result in flooding on- or off-site?* The grading plan for the Project generally follows the existing topography, and will not substantially alter drainage patterns. A portion of the southern drainage area will increase from the pre-project drainage areas presented in the Drainage Master Plan. However, this additional drainage area has been accounted for and, with modifications, Basin 5D can accommodate the additional runoff volume without impacting downstream peak flows. Off-site flooding in the northern drainage will be mitigated through flood control at Basin 4A.

- *Does the project create or contribute runoff water which would exceed the capacity of existing or planned stormwater drainage systems or provide substantial additional sources of polluted runoff?* The storm drain modeling indicates that, without mitigation, the Project will increase the water surface elevation in Basin 5D for the 100-year design storm event. However, Basin 5D is not currently meeting County design requirements for freeboard. Mitigation as part of this Project will regrade Basin 5D to accommodate the additional runoff volume from the proposed project and modify the basin to meet County standards for freeboard. Peak flow rates at the C4 and C5 outfalls analysis points are modeled to be less than pre-project flows.

- *Does the project otherwise substantially degrade water quality?* Water-quality features for the Project were designed according to the guidance of the West Placer County Storm Water Design Manual (2016). Additional water-quality features may be incorporated into the site design as the Project advances. A Post-Construction Storm Water Quality Plan must be submitted to the County as part of the approval process, and will include a comprehensive description of all on-site water-quality features.

- *Does the project place housing within a 100-year flood hazard area as mapped on a federal Flood Hazard Boundary or Flood Insurance Rate Map or other flood hazard delineation map?* The Project site appears on FEMA Flood Insurance Rate Map (FIRM) Number 06061C0275F (effective June 8, 1998), and is not mapped as a special flood hazard area inundated by the 100-year flood.

- *Does the project expose people or structures to a significant risk of loss, injury or death involving flooding, including flooding as a result of the failure of a levee or dam?* The drainage system for the Project will be designed according to County standards to avoid property damage and loss of life. The Project site is not protected by levees, and is not within a mapped dam failure inundation zone.

5 CONCLUSIONS

The XPSTORM model developed for the PCGC Master Drainage Report was adapted to analyze changes to the hydrology and hydraulics of the PCGC campus from the AHS Project. The model showed that the regional stormwater basin (Basin 5D) along with the proposed local stormwater basin (Basin 4A) will provide adequate flood and hydromodification controls. Basin 5D will need to be reconfigured to comply

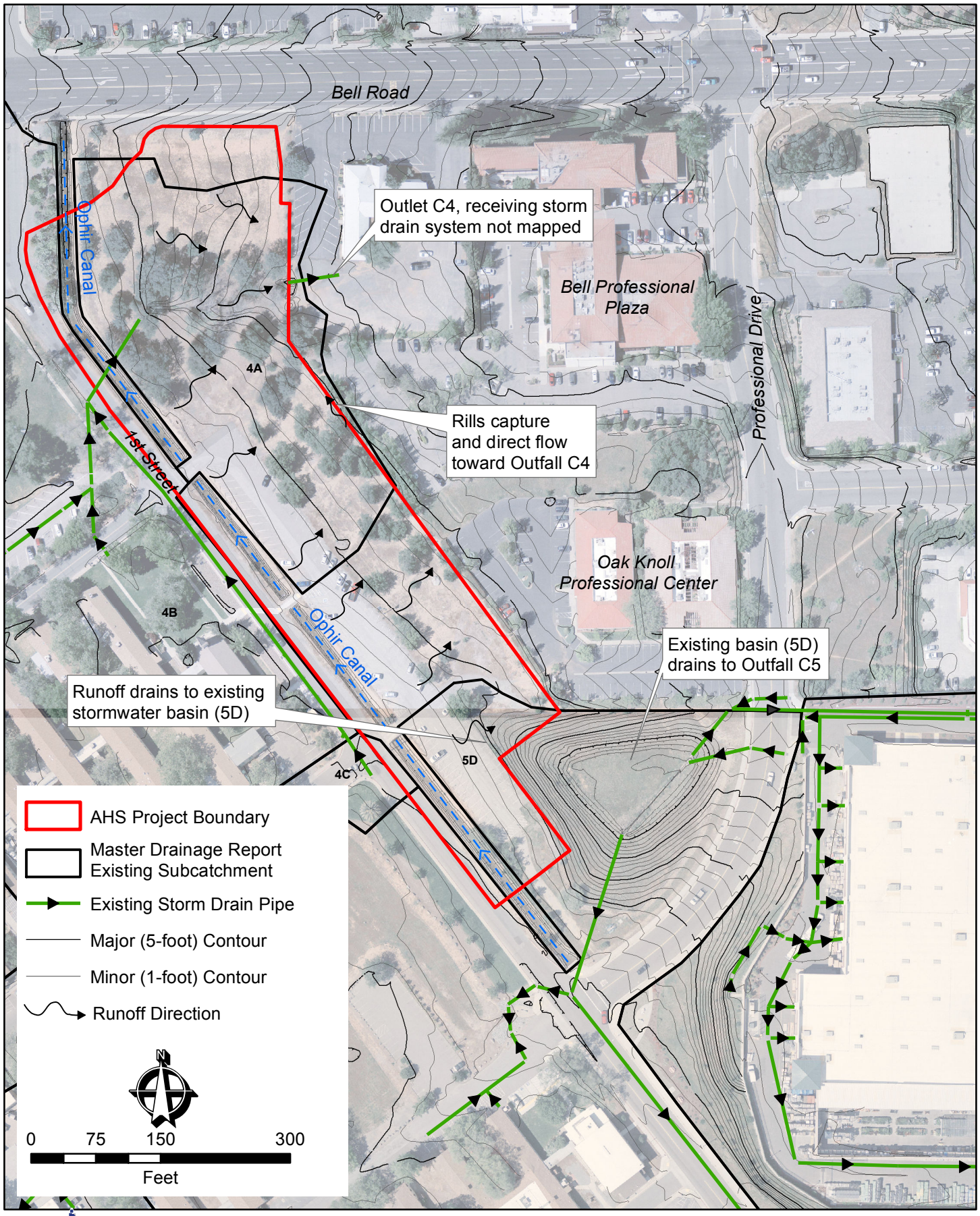
with County criteria for freeboard during the 100-year event by increasing the top of berm level by a minimum of 0.4 feet. The additional grading of Basin 5D has been shown to be a feasible mitigation measure and included as part of the proposed project. The receiving storm drain system will have sufficient capacity to convey the intended flow rates at both Outfalls C4 and C5 since overall peak flow rates will be reduced. The on-site water-quality features have been sized to provide treatment for the Project. The discussion of the CEQA checklist items showed that with proper implementation of the controls for stormwater quantity and quality presented herein, the Project would not adversely affect water quality, property damage, or risk of loss of life.

REFERENCES:

cbec eco engineering, inc. and CDM Smith, 2016, West Placer Storm Water Quality Design Manual, 68 p + appendices.

Kulchawik, P. and Garrison, T., 2017, Appendix D of the Master Plan Update – Wet Utility Infrastructure: draft report submitted to Cartwright Engineers, June 12, 2017, 22 p + attachments.

Placer County Flood Control and Water Conservation District, 1994, Stormwater Management Manual, 135 p + appendices.



Balance Hydrologics, Inc.

Figure 1. Existing Surface Water Drainage, Placer County Government Center

Basemap source: Andregg Geomatics

Y:\GIS\Projects\217245 PCGC Drainage Reports\AHS Proj Figures\217245 Fig1.mxd

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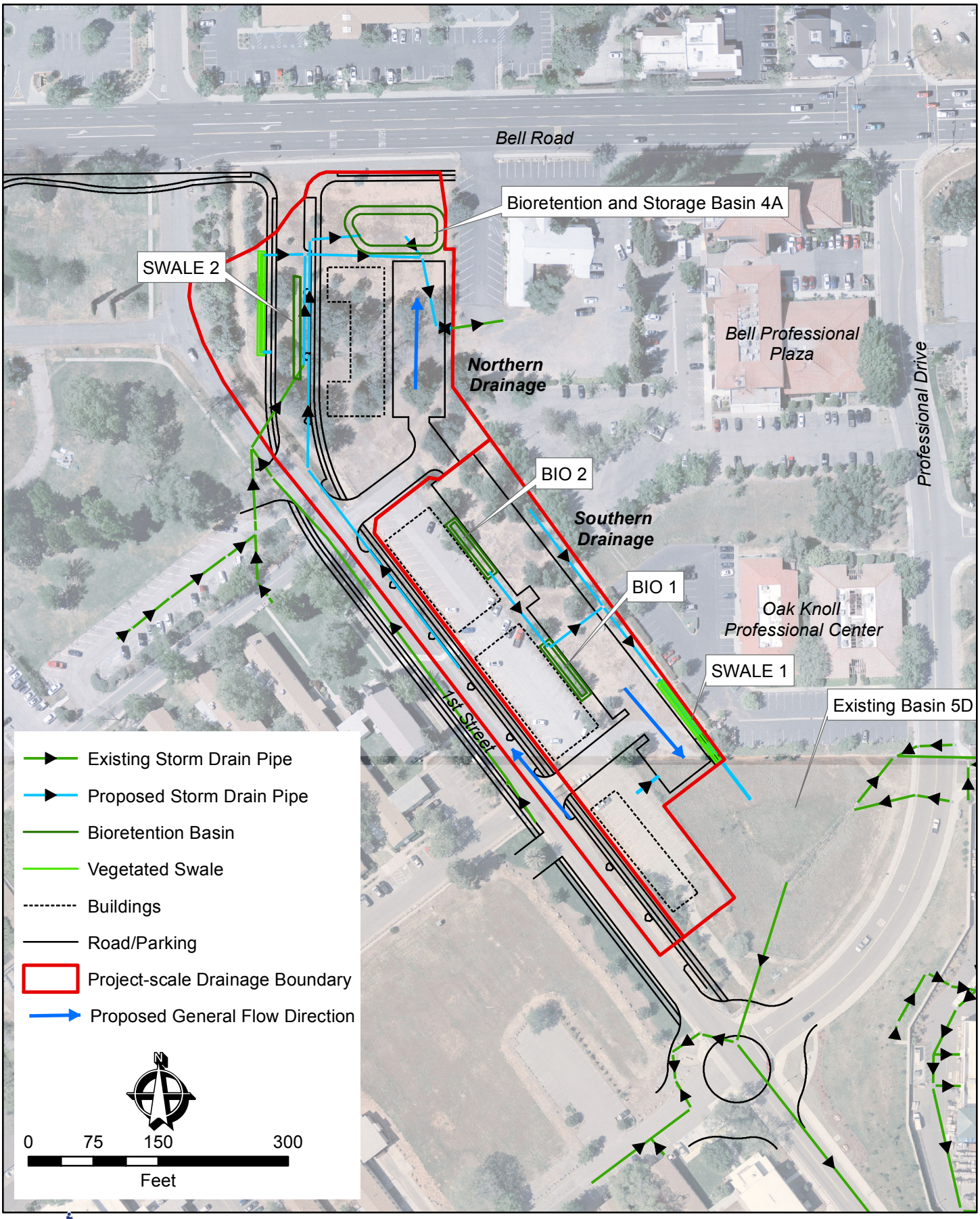

Balance Hydrologics, Inc.

Figure 2. Proposed Drainage Boundaries, Placer County Government Center

Basemap source: Andregg Geomatics

Y:\GIS\Projects\217245 PCGC Drainage Reports\AHS Proj Figures\217245 Fig2.mxd

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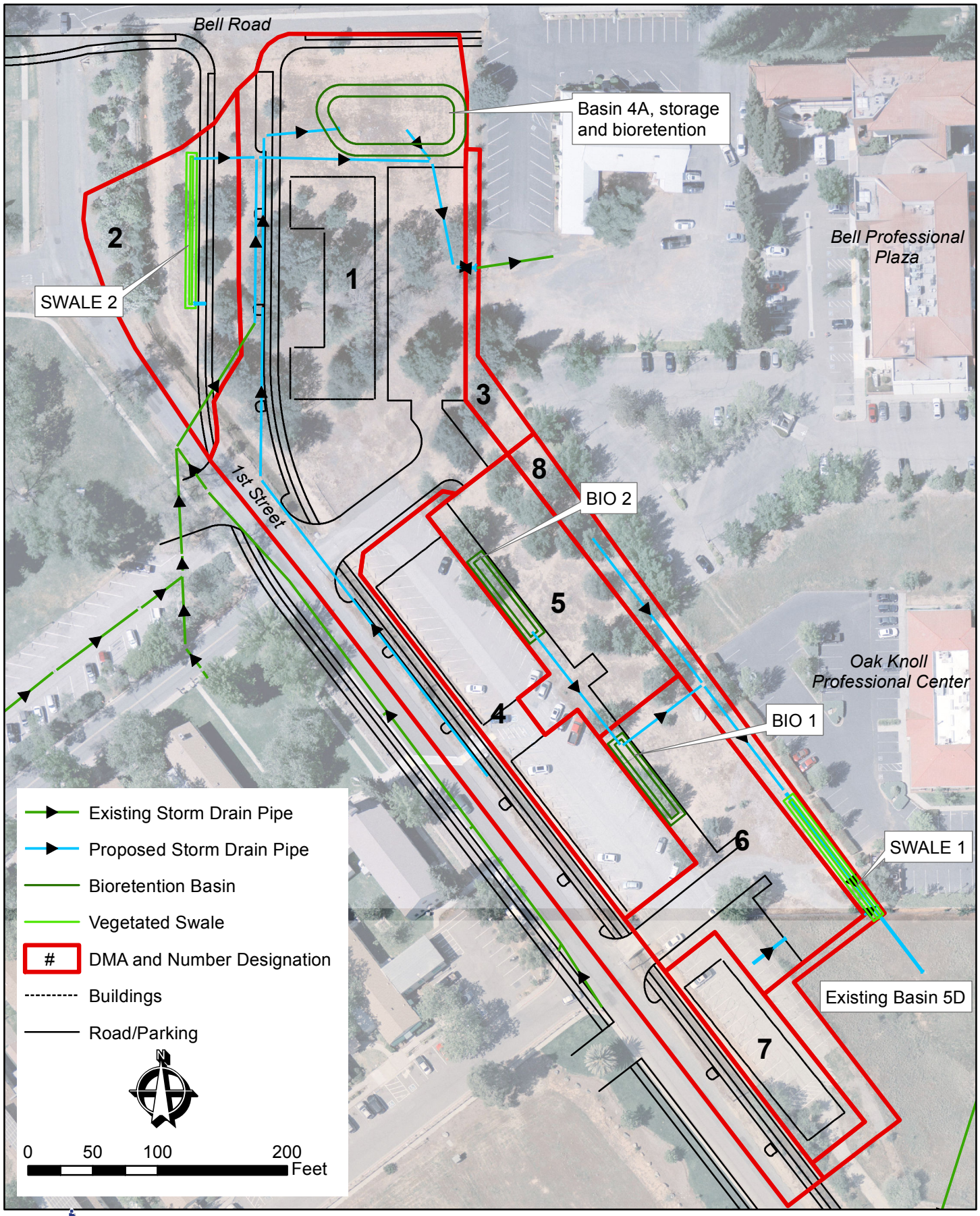

Balance Hydrologics, Inc.

Figure 3. Proposed Affordable Housing Site Drainage Management Areas and Water Quality Features, Placer County Government Center

Basemap source: Andregg Geomatics

Y:\GIS\Projects\217245 PCGC Drainage Reports\AHS Proj Figures\217245 Fig3.mxd

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Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Thursday, Apr 12 2018

AHS SITE SWALE 1 (DMA 5, 6 & 7) - Max Flow (1.16cfs)

Trapezoidal

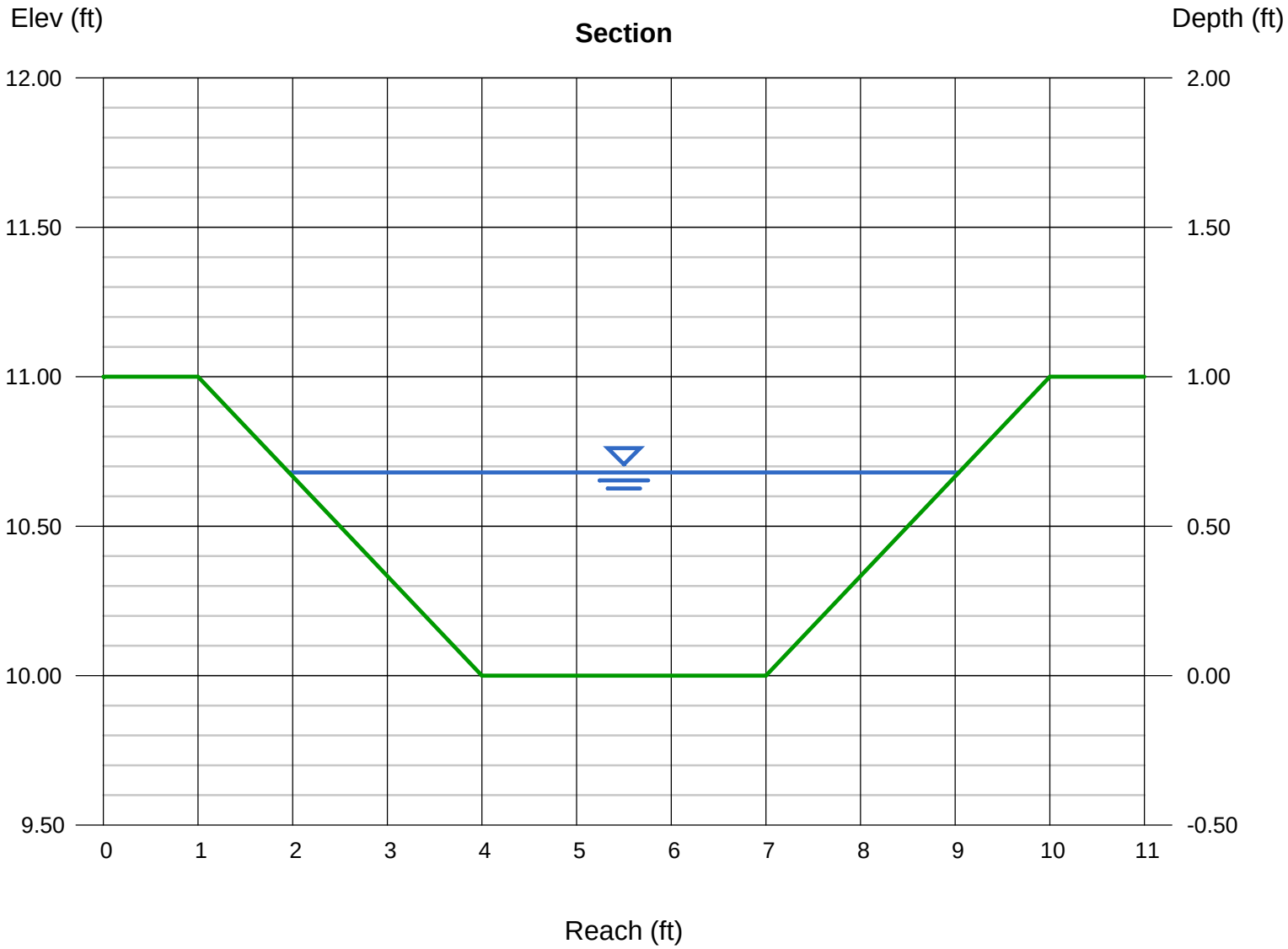
Bottom Width (ft)	= 3.00
Side Slopes (z:1)	= 3.00, 3.00
Total Depth (ft)	= 1.00
Invert Elev (ft)	= 10.00
Slope (%)	= 0.90
N-Value	= 0.250

Calculations

Compute by:	Known Q
Known Q (cfs)	= 1.16

Highlighted

Depth (ft)	= 0.68
Q (cfs)	= 1.160
Area (sqft)	= 3.43
Velocity (ft/s)	= 0.34
Wetted Perim (ft)	= 7.30
Crit Depth, Yc (ft)	= 0.16
Top Width (ft)	= 7.08
EGL (ft)	= 0.68



Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Thursday, Apr 12 2018

AHS SITE SWALE 1 (DMA 5, 6 & 7) - WQ Flow (0.17cfs)

Trapezoidal

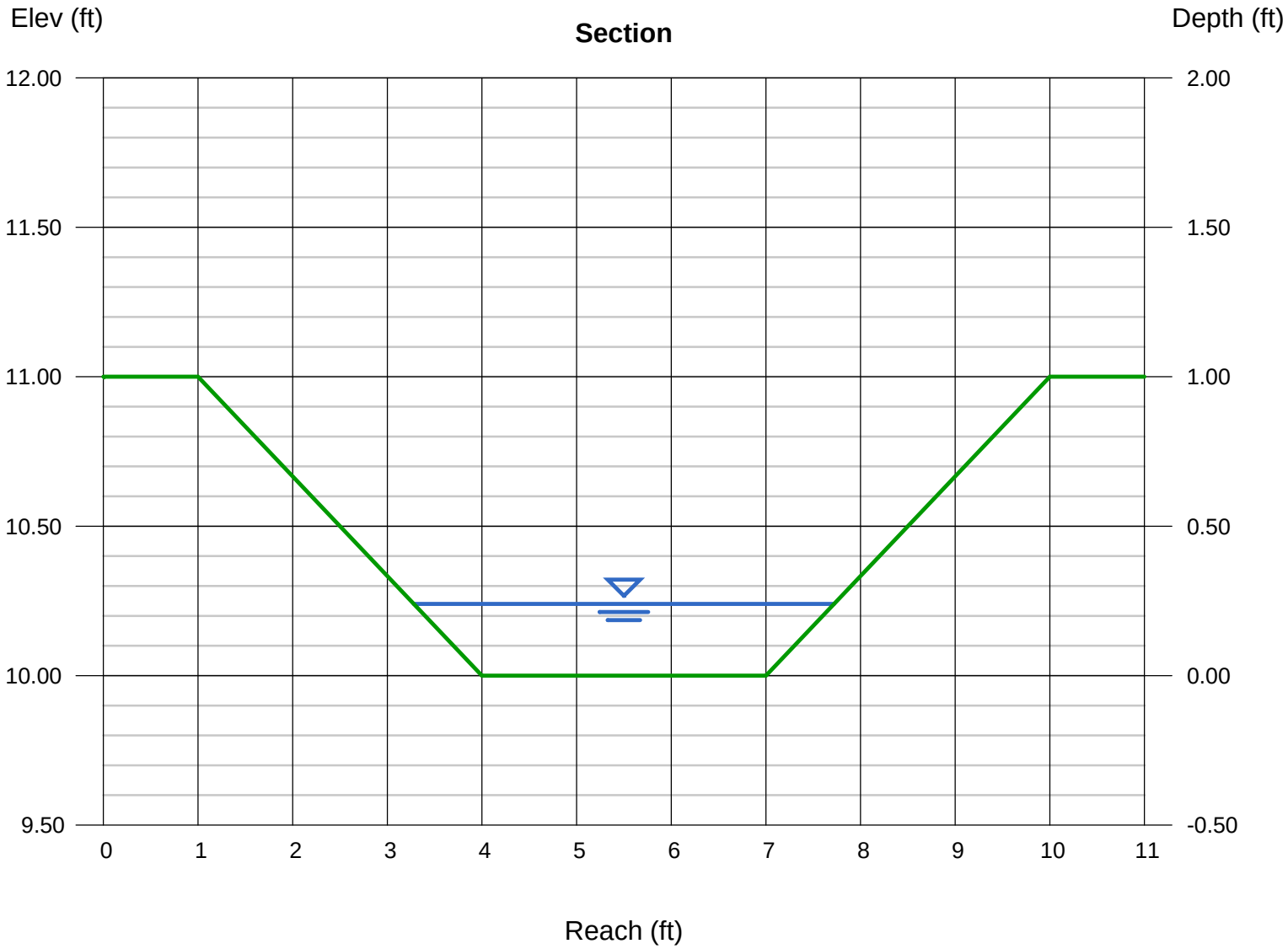
Bottom Width (ft)	= 3.00
Side Slopes (z:1)	= 3.00, 3.00
Total Depth (ft)	= 1.00
Invert Elev (ft)	= 10.00
Slope (%)	= 0.90
N-Value	= 0.250

Calculations

Compute by:	Known Q
Known Q (cfs)	= 0.17

Highlighted

Depth (ft)	= 0.24
Q (cfs)	= 0.170
Area (sqft)	= 0.89
Velocity (ft/s)	= 0.19
Wetted Perim (ft)	= 4.52
Crit Depth, Yc (ft)	= 0.05
Top Width (ft)	= 4.44
EGL (ft)	= 0.24



Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Thursday, Apr 12 2018

AHS SITE SWALE 2 (DMA 2) - Max Flow (0.32cfs)

Trapezoidal

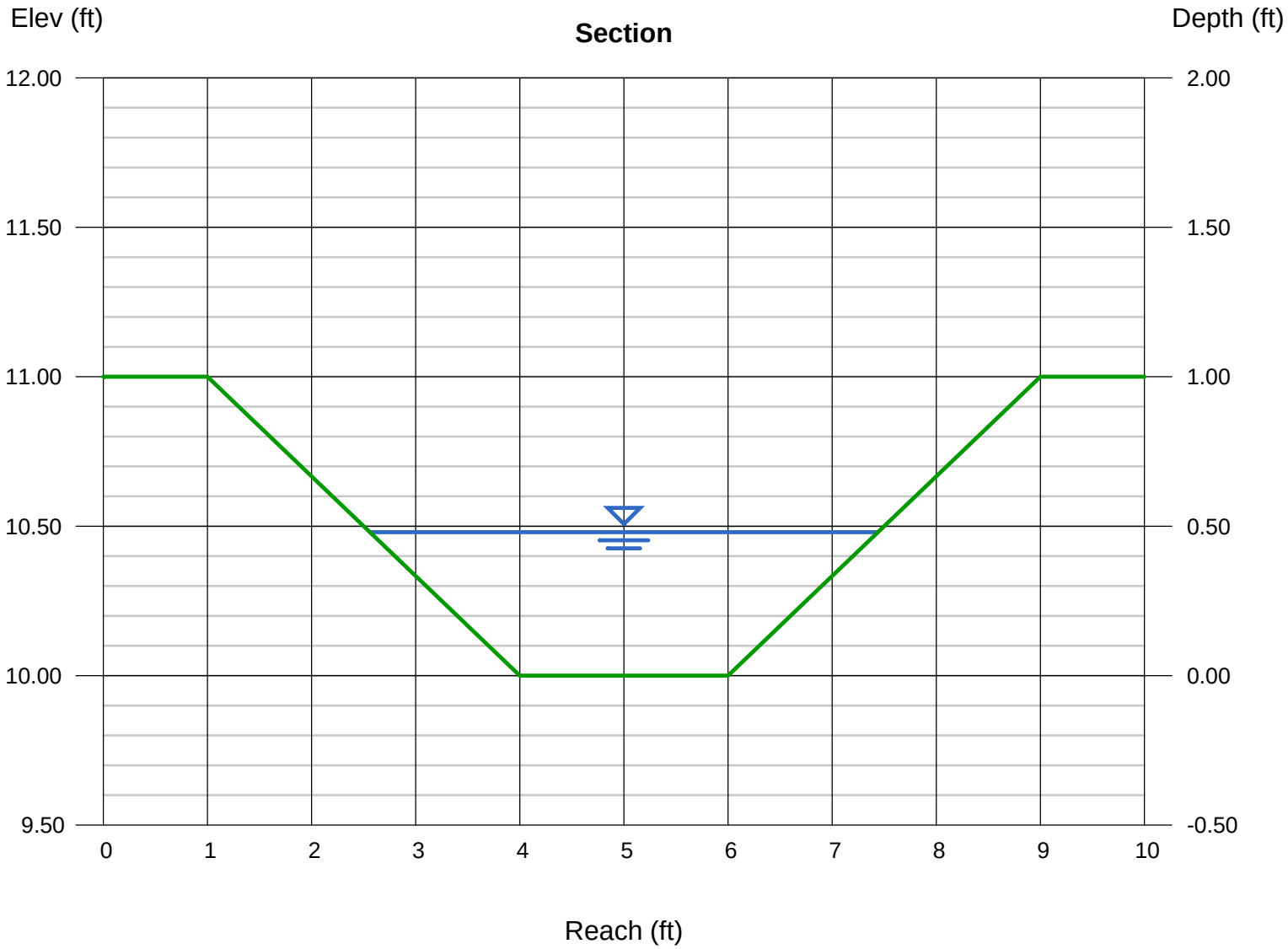
Bottom Width (ft)	=	2.00
Side Slopes (z:1)	=	3.00, 3.00
Total Depth (ft)	=	1.00
Invert Elev (ft)	=	10.00
Slope (%)	=	0.50
N-Value	=	0.250

Calculations

Compute by:		Known Q
Known Q (cfs)	=	0.32

Highlighted

Depth (ft)	=	0.48
Q (cfs)	=	0.320
Area (sqft)	=	1.65
Velocity (ft/s)	=	0.19
Wetted Perim (ft)	=	5.04
Crit Depth, Yc (ft)	=	0.09
Top Width (ft)	=	4.88
EGL (ft)	=	0.48



Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Thursday, Apr 12 2018

AHS SITE SWALE 2 (DMA 2) - WQ Flow (0.05cfs)

Trapezoidal

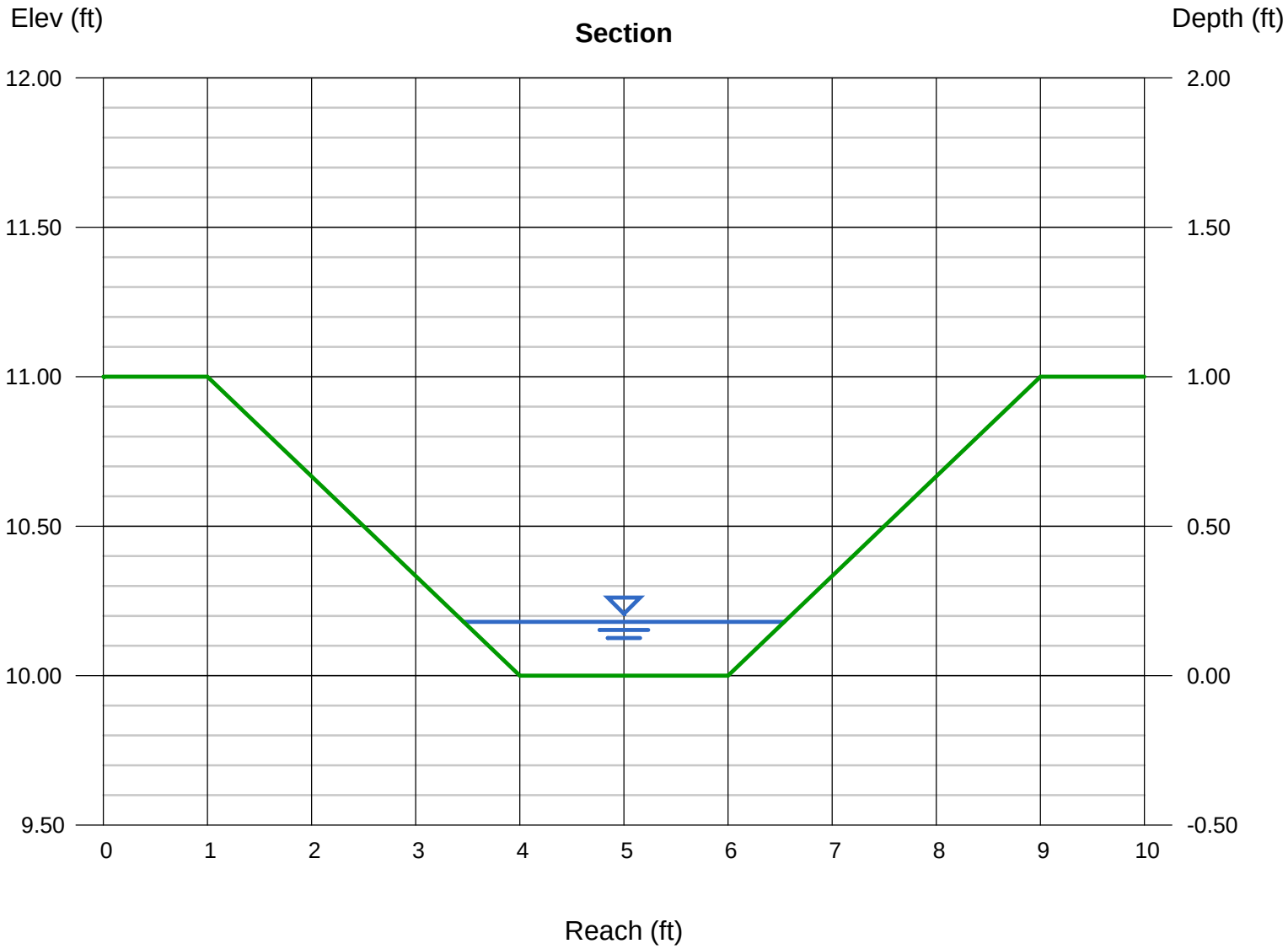
Bottom Width (ft)	=	2.00
Side Slopes (z:1)	=	3.00, 3.00
Total Depth (ft)	=	1.00
Invert Elev (ft)	=	10.00
Slope (%)	=	0.50
N-Value	=	0.250

Calculations

Compute by:		Known Q
Known Q (cfs)	=	0.05

Highlighted

Depth (ft)	=	0.18
Q (cfs)	=	0.050
Area (sqft)	=	0.46
Velocity (ft/s)	=	0.11
Wetted Perim (ft)	=	3.14
Crit Depth, Yc (ft)	=	0.03
Top Width (ft)	=	3.08
EGL (ft)	=	0.18



Form 3-4 Runoff Reduction Calculator for Site Design Measures on Regulated Projects

		[1]DMA ID No.	1		2		3		4	
Site Design Measure	**Runoff Reduction Parameters**			**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**
[2] Adjacent/On-Site Stream Setbacks and Buffers	A_{imp} (ft^2)	*impervious drainage area*		0		0		0		0
	V_{85} (in)	*runoff volume from 85th percentile, 24-hour storm*	1.0		1.0		1.0		1.0	
[3] Soil Quality Improvement and Maintenance	A_{pond} (ft^2)	*ponding area*		0		0		0		0
	D_{pond} (ft)	*ponding depth*								
	A_{sa} (ft^2)	*soil amendment area*								
	D_{sa} (ft)	*depth of amended soil*								
	n	*porosity of amended soil*								
[4] Tree Planting and Preservation	n_e	*number of new evergreen trees*		0		0		0		0
	n_d	*number of new deciduous trees*								
	A_{tc} (ft^2)	*canopy area of existing trees to remain on the property*	0							
	V_{85} (in)	*runoff volume from 85th percentile, 24-hour storm*	1.0		1.0		1.0		1.0	
[5] Rooftop and Impervious Area Disconnection	A_{imp} (ft^2)	*impervious drainage area*		0		0		0		0
	V_{85} (in)	*runoff volume from 85th percentile, 24-hour storm*	1.0		1.0		1.0		1.0	
[6] Porous Pavement	A_{res} (ft^2)	*area of gravel storage layer*		0		0		0		0
	D_{res} (ft)	*depth of gravel storage layer*								
	n_{agg}	*porosity of aggregate*								
	C	*efficiency factor*								
[7] Vegetated Swales	A_{imp} (ft^2)	*impervious drainage area*		0	5709	471		0		0
	V_{85} (in)	*runoff volume from 85th percentile, 24-hour storm*	1.0		1.0		1.0		1.0	
[8] Rain Barrels and Cisterns	N	*number of rain barrels and/or cisterns*		0		0		0		0
	V_a (ft^3)	*volume of each rain barrel and/or cistern*								
[9] Do all Site Design Measures meet the design requirements outlined in the Fact Sheets?					**Yes**	**X**	**No**			
[10] Total Volume Reduction (ft^3)			**0**		**471**		**0**		**0**	
[11] Effective Treated Impervious Area (ft^2)			**0**		**5709**		**0**		**0**	

Form 3-4 Runoff Reduction Calculator for Site Design Measures on Regulated Projects

DMA ID No.	5		6		7		8		9		10		11		12	
Runoff Reduction Parameters		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**		**Runoff Reduction (ft^3)**
impervious drainage area		0		0		0		0		0		0		0		0
runoff volume from 85th percentile, 24-hour storm	1.0		1.0		1.0		1.0		1.0		1.0		1.0		1.0	
ponding area		0		0		0		0		0		0		0		0
ponding depth																
soil amendment area																
depth of amended soil																
porosity of amended soil																
number of new evergreen trees		0		0		0		0		0		0		0		0
number of new deciduous trees																
canopy area of existing trees to remain on the property																
runoff volume from 85th percentile, 24-hour storm	1.0		1.0		1.0		1.0		1.0		1.0		1.0		1.0	
impervious drainage area		0		0		0		0		0		0		0		0
runoff volume from 85th percentile, 24-hour storm	1.0		1.0		1.0		1.0		1.0		1.0		1.0		1.0	
area of gravel storage layer		0		0		0		0		0		0		0		0
depth of gravel storage layer																
porosity of aggregate																
efficiency factor																
impervious drainage area	####	1042	####	1354	5469	451		0		0		0		0		0
runoff volume from 85th percentile, 24-hour storm	1.0		1.0		1.0		1.0		1.0		1.0		1.0		1.0	
number of rain barrels and/or cisterns		0		0		0		0		0		0		0		0
volume of each rain barrel and/or cistern																
Total Volume Reduction	**1042**		**1354**		**451**		**0**		**0**		**0**		**0**		**0**	
Effective Treated Impervious Area	**12636**		**16411**		**5469**		**0**		**0**		**0**		**0**		**0**	

Form 3-5 Computation of Water Quality Design Criteria for Stormwater Treatment and Baseline Hydromodification Measures

DMA ID No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
[1]Total impervious area requiring treatment	54124	5709	0	18531	12636	12653	5469	0														
[2] Impervious area untreated by Site Design Measures (ft^2) *Item 1 – Form 3-4 Item 11*	54124	0	0	18531	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[3]Additional pervious area draining to BMP (ft^2)	33232			4570																		
[4] Composite DMA Runoff Coefficient (Rc) *Enter area weighted composite runoff coefficient representing entire DMA*	0.60			0.80																		
[5] Water Quality Volume (WQV) (ft^3) *WQV = 1/12 * [Item 2 + Item 3) *Item 4] * Unit WQV*	3931	0	0	1386	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[6] Water Quality Flow (WQF) (cfs) *WQF = 1/43,200 * [0.2* (Item 2 + Item 3) * Item4]*	0.243	0.000	0.000	0.086	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

DMA ID No.	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
[1]Total impervious area requiring treatment																						
[2] Impervious area untreated by Site Design Measures (ft^2) *Item 1 – Form 3-4 Item 11*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[3]Additional pervious area draining to BMP (ft^2)																						
[4] Composite DMA Runoff Coefficient (Rc) *Enter area weighted composite runoff coefficient representing entire DMA*																						
[5] Water Quality Volume (WQV) (ft^3) *WQV = 1/12 * [Item 2 + Item 3) *Item 4] * Unit WQV*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
[6] Water Quality Flow (WQF) (cfs) *WQF = 1/43,200 * [0.2* (Item 2 + Item 3) * Item4]*	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Form 3-6 Volume-Based Infiltrating Bioretention Measures

[1]**DMA ID No.** *If combining multiple DMAs from Form 3-5, enter a new unique DMA ID No.*	**BIO 1 (1/2 DMA 4)**	**BIO 2 (1/2 DMA 4)**	**BIO 4A (DMA 1)**
[2]WQV (ft^3) *Item 5 in Form 3-5* *If combining multiple DMAs from Form 3-5, enter the sum of their respective WQVs.*	693	693	3931
[3]Surface Loading Rate *Maximum 5.0 in/hr*	5	5	5
[4]BMP Surface Area (ft^2) *Top of BMP*	504	504	4076
[5]Infiltration rate of underlying soils (in/hr)	0.16	0.16	0.16
[6]Maximum ponding depth (ft) *BMP specific, see BMP design details*	1.0	1.0	1.0
[7]Ponding Depth (ft) d_{BMP} *= Minimum of (1/12 * Item 5 * 48 hrs) or Item 6*	0.6	0.6	0.6
[8]Infiltrating surface area, SA_{BMP} (ft^2) *Bottom of BMP*	504	504	3320
[9]Planting media depth, d_{media} (ft)	1.5	1.5	1.5
[10]Planting media porosity	0.30	0.30	0.30
[11]Gravel depth, d_{media} (ft) *Only included in certain BMP types*	1.0	1.0	1.0
[12]Gravel porosity	0.30	0.30	0.30
[13]Retention Volume (ft^3) $V_{retention}$ *= Item 8 * [Item7 + (Item 9 * Item 10) + (Item 11 * Item 12) + (1.5* (Item 5 / 12))]*	700.6	700.6	4,614.8
[14]Untreated Volume (ft^3) $V_{untreated}$ *= Item 2 – Item 13* *If greater than zero, adjust BMP sizing variables and re-compute retention volume*	0	0	0
[15] Treated Flow Rate (ft^3/s) $Q_{treated}$ *= 1/43,200*(Item 3 * Item 4)*	0.0583	0.0583	0.4718
[16]Total Treated Flow Rate for Project (ft^3/s) Q_{total} *= Sum of Item 15 for all DMAs*			
[17]Is WQV for each DMA treated on-site? *Check Yes if Item 14 equals 0 for all DMAs*	**Yes**	**X**	**No**

BIOSWALE DESIGN CALCULATIONS

Calc'd by: *T. Garrison* Date: *4/9/2018*

DESIGN CRITERIA *West Placer County Storm Water Quality Design Manual, May 2017*

Design Storm	85th 24 hour		
Max velocity	1	ft/sec	
WSL < 2/3 height or 4 inches			
Slopes Range	0.5	% to	2.50%
Contact time	10	min	
Manning n	0.25	-	
Min length	100	ft	
Elevation	6040	ft	
85th 24-hr Rainfall Depth	1.0	inches	*West Placer County Storm Water Quality Design Manual, May 2017*
85th 24-hr Rainfall Depth	1.0	inches	*West Placer County Post-Construction Stormwater Quality Plan Calculator*

SWALE SIZING CALCULATIONS

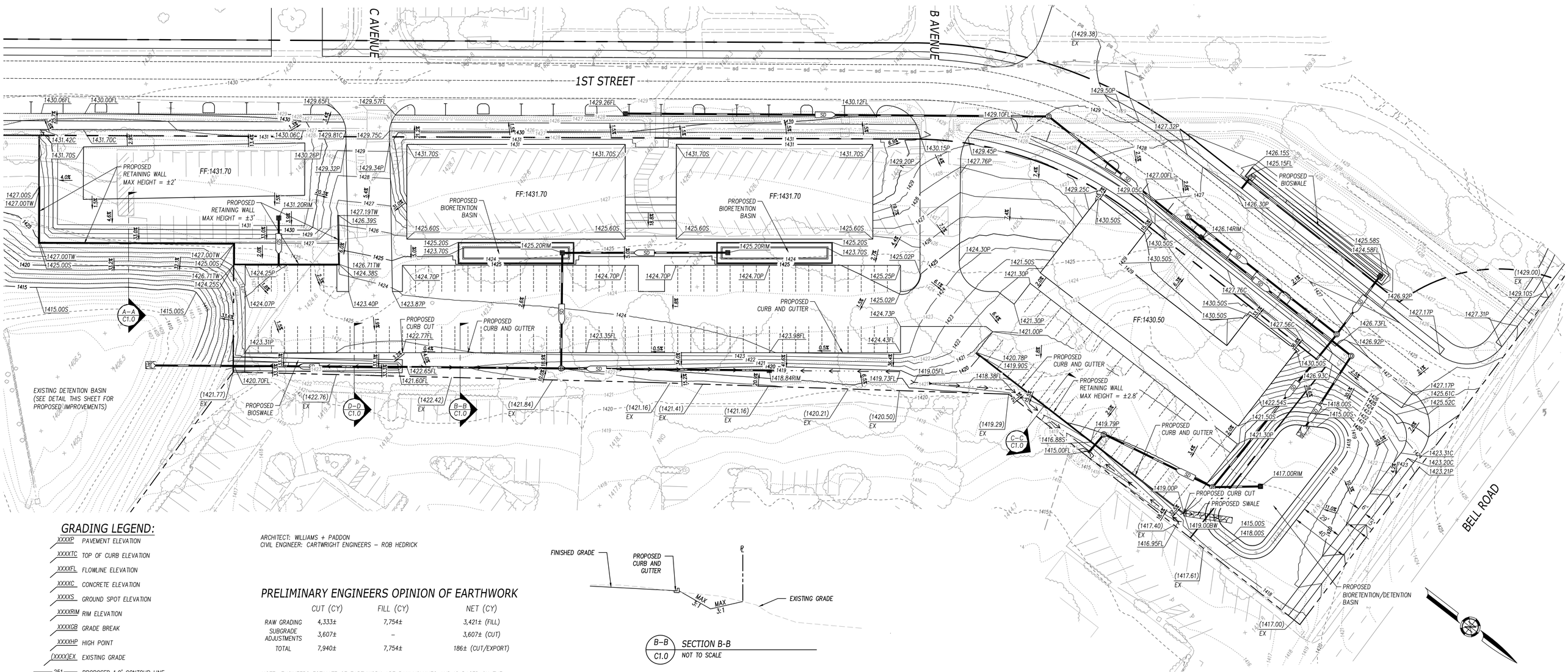
Drainage Area	Area (ft2)	Area (ac)	Impervious (ft2)	Runoff Coefficient (-)	Rainfall Depth (inches)	Runoff Volume (ft3)	WQ Volume (ft3)	Uniform Rainfall Intensity (in/hr)	10 YR Flow Rate (cfs)	Volume from HRT (ft3)	Rainfall Intensity 100 year (in/hr)	100 YR Flow Rate (cfs)
DMA 2	19650	0.45	5709	0.51	1	835	428	0.2	**0.05**	28	1.38	**0.32**
DMA 5,6,7	50420	1.16	34517	0.73	1	3,053	2,589	0.2	**0.17**	101	1.38	**1.16**

SWALE DESIGN PARAMETERS

Name	Side Slope (H:1)	Bottom Width (ft)	Slope (%)	Length (ft)	Estimate Depth (ft)	Total Swale Volume (cu. ft)	Design Storm Depth* (ft)	Design Storm Velocity* (cfs)	Design Storm Volume (cu. ft)	HRT (min)	Max Depth** (ft)
SWALE 2 (DMA2)	3.0	2.0	0.50%	100	1.0	500	0.18	0.11	90	**13.6**	0.48
SWALE 1 (DMA 5,6,7)	3.0	3.0	0.09%	100	1.0	600	0.24	0.19	144	**12.6**	0.68

*From AutoCAD Hydroflow Manning's calculations

** Max depth refers to the depth of flow in swale from the 100-year flow rate



GRADING LEGEND:

- XXXXP — PAVEMENT ELEVATION
- XXXXTC — TOP OF CURB ELEVATION
- XXXXFL — FLOWLINE ELEVATION
- XXXXC — CONCRETE ELEVATION
- XXXXS — GROUND SPOT ELEVATION
- XXXXRIM — RIM ELEVATION
- XXXXGB — GRADE BREAK
- XXXXHP — HIGH POINT
- (XXXX)EX — EXISTING GRADE
- 251 — PROPOSED 1.0' CONTOUR LINE
- 250 — PROPOSED 5.0' CONTOUR LINE
- 251 — EXISTING 1.0' CONTOUR LINE
- 250 — EXISTING 5.0' CONTOUR LINE
- SD — PROPOSED STORM DRAIN LINE
- PROPOSED STORM DRAIN INLET
- * TOP OF CURB (TC) ELEVATION = 0.5' + PAVEMENT (P) AND/OR CONCRETE (C) ELEVATIONS, UNLESS OTHERWISE NOTED.

ARCHITECT: WILLIAMS + PADDON
CIVIL ENGINEER: CARTWRIGHT ENGINEERS – ROB HEDRICK

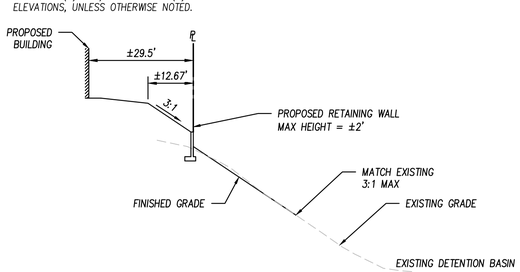
PRELIMINARY ENGINEERS OPINION OF EARTHWORK

	CUT (CY)	FILL (CY)	NET (CY)
RAW GRADING	4,333±	7,754±	3,421± (FILL)
SUBGRADE ADJUSTMENTS	3,607±	–	3,607± (CUT)
TOTAL	7,940±	7,754±	186± (CUT/EXPORT)

NOTE: ENGINEERS ESTIMATE OF EARTHWORK ARE RAW VOLUMES AND IS BASED ON THE SURFACE AT THE TIME OF THE SURVEY. IT INCLUDES ESTIMATED EXCAVATION FOR NEW PAVEMENT FOR THE PARKING LOT (ASSUMED 11" SECTION), BUILDING PAD (ASSUMED 8" SECTION), AND BIORETENTION AREA (30" SECTION). THE ESTIMATE DOES NOT INCLUDE STREET SECTIONS, TRENCH SPOILS OR SHRINK OR SWELL FACTORS. IT SHALL BE THE RESPONSIBILITY OF THE EARTHWORK CONTRACTOR TO VERIFY ALL EARTHWORK QUANTITIES PRIOR TO BID.

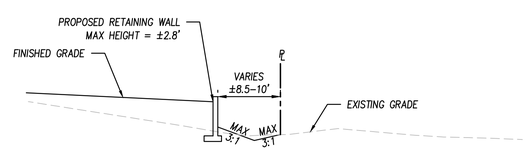
RETAINING WALL NOTES:

1. TOTAL PROPOSED RETAINING WALL LENGTH = 475 LF
2. TOTAL PROPOSED RETAINING WALL HEIGHT VARIES FROM ±1' – 3'

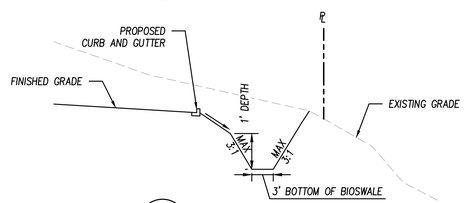


A–A C1.0 SECTION A-A
NOT TO SCALE

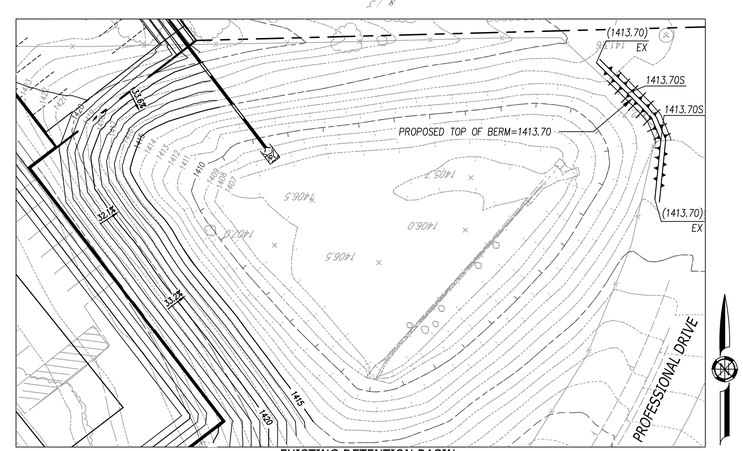
B–B C1.0 SECTION B-B
NOT TO SCALE



C–C C1.0 SECTION C-C
NOT TO SCALE



D–D C1.0 SECTION D-D
NOT TO SCALE



EXISTING DETENTION BASIN
BASIN 5D (PER MASTER DRAINAGE REPORT)
INTERIM IMPROVEMENTS
SCALE: 1"=30'

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PRELIMINARY GRADING AND DRAINAGE PLAN

WORKFORCE/AFFORDABLE HOUSING
PLACER COUNTY, CA

C1.0

4/04/2018
SCALE: 1"=30'
DRAWN BY: WY