Appendix H

Hydrology, Drainage, and Recharge Report

“VALLEY’S EDGE” SPECIFIC PLAN

MIXED-USE DEVELOPMENT

CITY OF CHICO, BUTTE COUNTY, CALIFORNIA

“DRAFT” DRAINAGE REPORT

April 29, 2020

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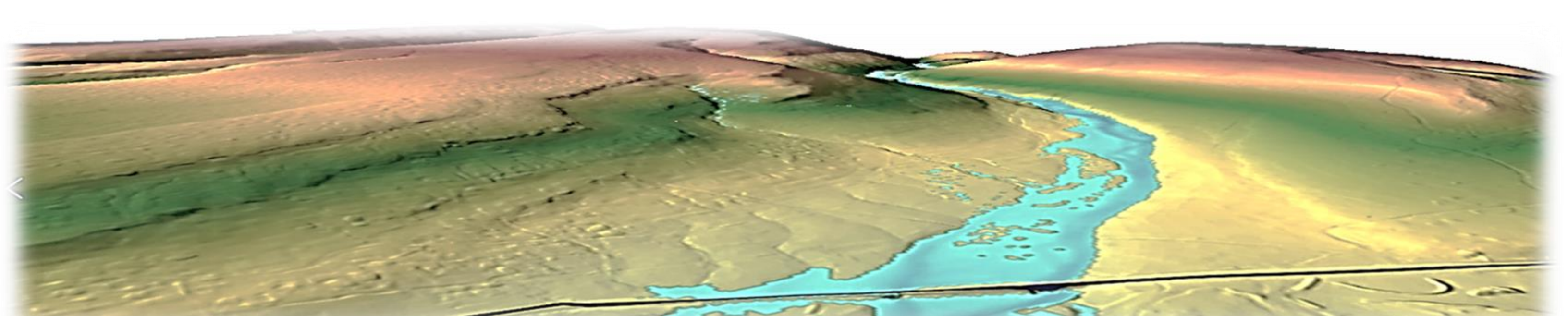

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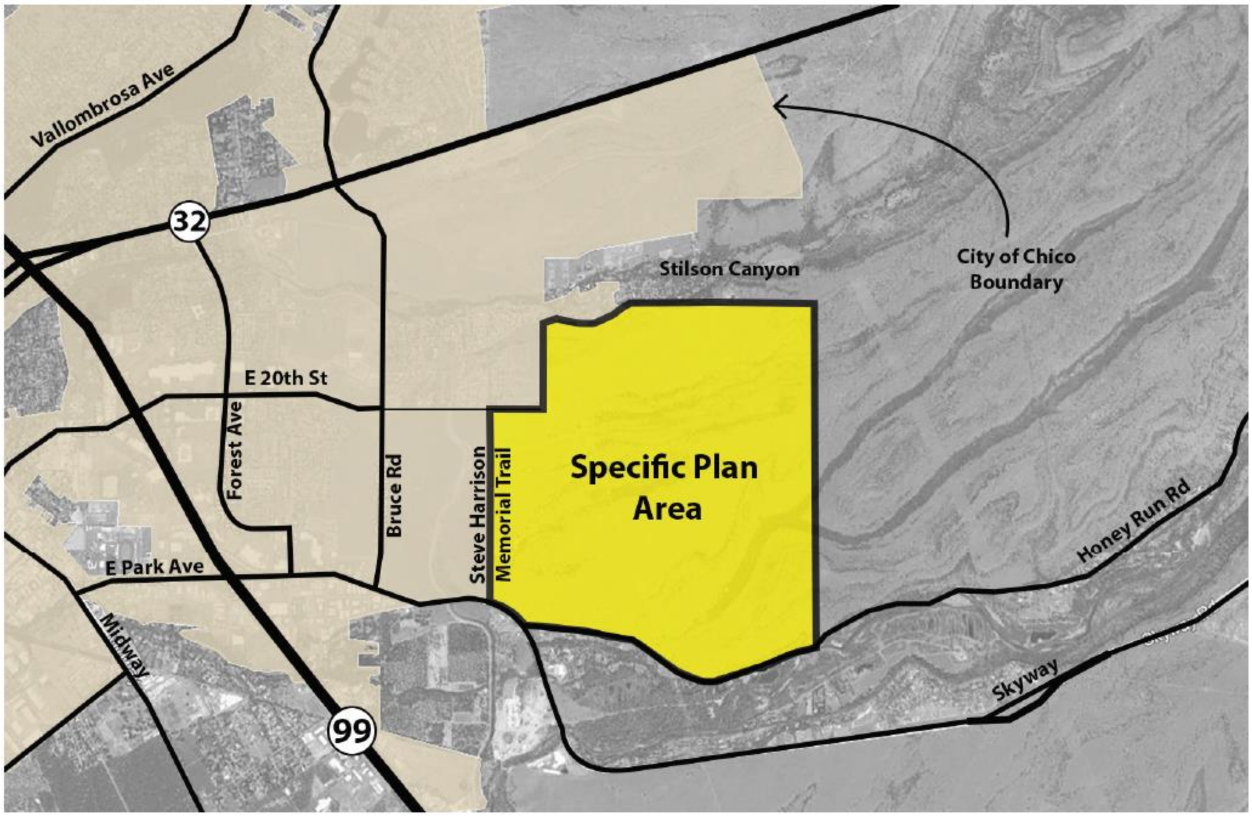


FIGURE 1 – VICINITY MAP

I. Introduction

1.01 Purpose

The objective of this preliminary drainage study is to analyze existing onsite drainage conditions that the planned unit development "Valley's Edge" would be obligated to upgrade & design, so as to not increase any downstream run-off or discharge volumes/quantities. "Valley's Edge" is a Specific Plan which proposes; Single Family, Multi-family & Light Commercial land uses and infrastructure required to accommodate these land uses. The finalized report will establish the required on-site drainage improvements and/or retention required to achieve compliance with no increase in run-off after post development (at complete build-out). Compliance standards and requirements will be set forth in accordance with "Butte County" and the "City of Chico," which are the governing jurisdictions and officials that would be reviewing the tentative maps and construction improvement plans accompanying the proposed development.

1.02 Scope of Investigation

(a) ***<u>Establish Existing Historic Flows</u>***

Determine the probable external and internal drainage areas responsible for runoff in the development.

(b) ***<u>Estimate Developed Flow Increases From Proposed Improvements</u>***

Determine the probable 10-year volume and flow rates (V-10/Q-10) and 100-year volume and flow rates (V-100/Q-100) for the external and internal drainage areas for the undeveloped and developed states of the project.

(c) ***<u>Mitigation of Increased Runoff</u>***

Determine the size, approximate location and outlet control characteristics of storm water ponding areas that will reduce the developed flow rates for the site to levels at or below the undeveloped flow rates for the site.

1.03 Site Description and Characteristics

The **<u>"Valley's Edge" Specific Plan</u>** encompasses a **<u>1,448 Acres</u>** site, located on the eastern boundary of the City of Chico, within Butte County, California. The City of Chico has agreed to the annexation of the property in whole upon acceptance of the prerequisites stated in the conditions of approval set forth by the City of Chico review staff. The entire site is situated within the Little Chico Creek/Butte Creek watershed. (See **<u>"Exhibit B"</u>** folder & files: included in the folder "***Exhibit-B-catchment/water sheds)\PDF*** *…specified* *files…" "***Exhibit B.1.1** ***Pre-Development – Off-site Basin Map"*** and ***"Exhibit B.1.2*** ***Pre-Development – On-site Basin Map"***)

(a) ***Surrounding Area***

The Doe Mill/Honey Run Special Planning Area (SPA), also referred to as "Valley's Edge" (APN: 017-210-006), is located just east of Chico. It resides in Butte County and is soon to be annexed into The City of Chico. The site is adjacent to undeveloped foothills; directly 1.7 miles east of E 20th St and Highway 99 intersection. Rural residential development currently exists along the northern and southern boundaries of the property, open grazing land to the east, and urban residential and protected wetland and species habitat to the west.

(b) ***Shed Areas and Channels***

The site experiences runoff from three major shed areas with 4 other minor shed areas (Exhibit B.1.1). The offsite drainage basins, just east of the site, slope from the east to the west with average 4-5% slopes along channel beds. The major creeks are formed by these basins upon entrance, along with a few rivulets. Minor onsite creeks form and meander through the site along with the major ones that enter the site from the east. Offsite and onsite runoff channelizes into these creeks and is discharged through multiple culverts and pipes on the south and west side of the Site. Exhibit D.1.0 depicts the onsite channel network along with hydraulic features. A more detailed onsite drainage network description can be found in the hydraulic analysis portion of the report.

(c) ***Vegetation/Other Features***

The undeveloped area is characterized by: grasslands and blue oaks in valley areas; grasslands with sparse vegetation across gradually sloping ridgelines; and corridors of mixed oak and mixed woodlands along major creeks and seasonal streams. Vernal swales exist along the western edge of the site. Dirt roads and trails are found across the site as well as lava rock boulders.

(d) ***Soil Type***

The site consists of approximately 1448 Acres of mostly type D soil (95%), having a highly impermeable overall surface. The majority of the site consists of flat plateau areas which support sporadic grass growth where thin soil "mounds" are present. The remaining plateau areas surrounding the soil mounds have been eroded over time revealing the lahar/cobblestone that does not support grass growth. Swale or channelized onsite areas support grass growth because of sufficient soil development. In addition, seepage from the adjacent rock masses contribute moisture. Specifics on the types of soils and the descriptions of the soil's classifications are discussed in further detail in section 3.03 of this report.

(For an onsite and offsite USGS Soil Survey map please refer to Exhibits C.3.2 and C.3.1 respectively. For a more accurate representation of soil types please see the Geotechnical Report (Exhibit C.0.1), which has been provided by Geo Plus Partners. A more detailed soils description can also be found in the hydrologic section of this report.)

1.04 Regional Watershed Description

The City of Chico is made up of the Big Chico Creek watershed and the Little Chico Creek/Butte Creek watershed and is located in the north-central portion of the Sacramento River Hydrologic Region. The City of Chico lies above the **Sacramento Valley Groundwater Basin and the West Butte and Vina sub basins**. The West Butte Sub basin is bounded on the west and south by the Sacramento River, on the north by Big Chico Creek, on the northeast by the Chico Monocline, and on the east by Butte Creek (DWR, 2004a). Big Chico and Butte creeks serve as sub basin boundaries in the near surface. The West Butte Sub basin is hydrologically contiguous with the Vina Sub basin at depth (DWR, 2004a). The Vina Sub basin is bounded on the west by the Sacramento River, on the north by Deer Creek, on the east by the Chico Monocline, and on the south by Big Chico Creek (DWR, 2004b). The aquifer system underlying Chico is comprised of continental deposits of Tertiary to late Quaternary age. The Quaternary deposits include the Holocene stream channel deposits and basin deposits and the Pleistocene Modesto Formation, Riverbank Formation, and Sutter Buttes alluvium. The Tertiary deposits consist of the Pliocene Tehama Formation and the Tuscan Formation (DWR, 2004a, 2004b). The aquifer system underlying Chico supplies the municipal and agricultural water demands of the city. Approximately 60 percent of the groundwater pumped for the city and most of the stormwater runoff from impervious development returns to either the groundwater system as recharge or the surface water system as discharge. Another 16 percent returns through septic systems (Butte LAFCO, 2006). The portion of water that does not return to the aquifer is consumed by landscape plants, lost through evapotranspiration, or discharged as treated wastewater to the Sacramento River. In addition, the groundwater system is largely sustained by recharge in the foothills located east of Chico, streamflow infiltration from Big Chico and Little Chico creeks and Lindo Channel, and to a lesser degree by direct infiltration of precipitation. The Lower Tuscan Formation is the primary groundwater-producing aquifer in the region. Most of the recharge areas of the Tuscan Formation are located along the base of the Sierra Nevada foothills in Butte County. Groundwater quality is generally acceptable, but there are some areas of concern (see below). The Chico region's geology plays a major role in the water resources, as some geological formations (aquifers) can transport and hold considerable amounts of water, while others do not. Also, some geological formations are permeable, allowing rapid infiltration of surface water, while other are relatively impermeable and greatly restrict recharge of groundwater. The Tuscan Formation extends from just west of the Sacramento River into the Sierra Nevada. It averages 1,700 feet in depth in the eastern portions of this swath to approximately 300 feet near the "Big Chico Creek" watershed.

(a) The "Little Chico Creek"

The "Little Chico Creek" originates in the foothills of the northern Sierra Nevada (Platte Mountain is located at the northern terminus of the watershed) and travels nearly 16 miles in a southwesterly direction through steep canyons before flattening out along the floor of the Sacramento Valley. The topography of Little Chico Creek is diverse, including the relatively flat valley floor, the low angle slope of the creek's alluvial fan, lower canyons, and steep-sloped headwaters. Before Little Chico Creek enters the City of Chico urban area, it passes a diversion structure constructed in the 1960s, which is intended to divert high flow from Little Chico Creek into Butte Creek. The creek flows another 9 miles, west through the City of Chico and then southwest, before intermingling with the numerous braided channels that make up the eastern floodplain of the Sacramento River.

(b) The "Butte Creek"

The "Butte Creek" originates in the Lassen National Forest at over 7,000 feet. Butte Creek travels through canyons through the northwestern region of Butte County and through the valley, entering the floor near Chico. The northern Sierra Nevada and southern Cascade mountain ranges generally divide the valley section from the mountainous section of the Butte Creek watershed in Butte County. Once Butte Creek enters the valley section of the watershed near Chico, it travels approximately 45 miles before it enters the Sacramento River (BCWC, 1998). Levees were constructed along Butte Creek in the 1950s by the USACE, extending over 14 miles along the Butte Creek channel.

(c) The "Comanche Creek"

The "Comanche Creek" parallels Little Chico Creek to the south and extends approximately 6 miles upstream into the Sierra Nevada foothills. The creek flows year-round due to the diversion of waters from Butte Creek (approximately 4 miles east of Skyway Rd) into the creek for conveyance to agricultural users to the west of the city. Comanche Creek, also known as Edgar Slough and Crough Ditch, flows along the southern fringe of the City of Chico before intersecting Little Chico Creek on the Sacramento River floodplain.

(d) The "Sacramento River Hydrologic Region"

The "Sacramento River Hydrologic Region" covers approximately 17.4 million acres (27,200 square miles) Geographically, the Sacramento River Hydrologic Region extends south from the Modoc Plateau near the Oregon border to the Sacramento-San Joaquin River Delta. The southernmost area, mainly high desert plateau, is characterized by hot, dry summers and cold, snowy winters with only moderate rainfall. The Sacramento Valley, which forms the core of the region, is bounded to the east by the crest of the Sierra Nevada and southern Cascades and to the west by the crest of the Coast Range and Klamath Mountains. Another significant feature is the Sacramento River, which is the longest river system in the State of California with major tributaries the Pit, Feather, Yuba, Bear, and American rivers. Overall, annual precipitation in the Sacramento River Hydrologic Region generally increases as one moves from south to north and west to east. The heavy snow and rain that falls in this region contributes to the overall water supply for the entire state. Annual runoff in the Sacramento River Hydrologic Region averages about 22.4 million acre-feet, which is nearly one-third of the state's total natural runoff. Major water supplies in the region are provided through surface storage reservoirs. Shasta Lake is one of the two largest surface water projects in the region. In total, the region has 43 reservoirs with a combined capacity of almost 16 million acre-feet (DWR, 2005). Major reservoirs in the region not only provide water supply but are also the source of recreation, power generation, and other environmental and flood control benefits. In addition, the region has a network of creeks and rivers that convey water for use throughout the region and provide nesting and rearing ground for major fish and wildlife species. Approximately eight million acre-feet of water go to municipal, industrial, and agricultural uses, while approximately 2.5 million acre-feet are stored as groundwater. Much of the remainder of the runoff goes to dedicated natural flows, which support various environmental requirements, including in-stream fishery flows and flushing flows in the Sacramento River Delta.

1.05 Terrain Characteristics

The "Valley's Edge" development is situated at the most southern base of the Sierra Nevada Foothills. The existing topography is comprised of relatively steep (+/- 4-5%) side slopes along minor drainage reaches that were carved out of the prehistoric Tuscan lava cap flows, accounting for the majority of the soils present onsite. Elevations range from 260' to 550' onsite and from 550' to 1400' at the top of the offsite drainage basins. Most offsite flows are channelized prior to entering the project boundary. Whereas Upon exiting the site, the runoff is diverted by means of a diversion channel (Butte Creek Diversion Channel) on the west side and a diversion channel (Crough Ditch) on the south, eventually finding its way into Butte Creek. The offsite and onsite delineation of existing watersheds can be seen on Exhibits B.1.1 and B.1.2 respectively.

1.06 FEMA Flood Mapping

A "Flood Emergency Management Association" (FEMA) digital flood map for Butte County and incorporated areas indicates all flood hazard areas around "Valley's Edge." The **"FIRM"** (*Flood Insurance Rate Map)* was last updated and was revised on "January 6, 2011". The "Valley's Edge" Edge's complete boundary of "On-Site" water sheds can be located on **FIRM Map # 06007C0510E**. Whereas, the remaining offsite shed areas can be located on **FIRM Map #'s 06007C0375E, and 06007C0530E**. The Site **is** oriented in a **"Zone X"** area which is outside the 100yr. 0.2% annual chance floodplain. Please refer to "Exhibit C.1.1, C.1.2 & C.1.3" for detailed "FEMA FIRM maps".

1.07 Surrounding Dam Failure Inundation Zone Findings

Research has been performed to confirm that the "Valley's Edge" development boundary is situated outside of potential dam failure inundation areas from ***Black Butte Reservoir***, ***Whiskeytown Reservoir*** or **Shasta Lake**. Localized minor flooding in the immediate adjacent 4 major drainage reaches and crossings may occur from time to time, however, with the proposed open space designations to be localized around these reaches, it is highly unlikely to occur or have any direct impact on any proposed commercial development and/or residential lotting scheduled for the final design.

1.08 Stormwater Quality

Onsite runoff is produced from undeveloped mountainous terrain upstream along with the onsite area. Exhibit B.1.2 portrays contributing basins which account for both sheet and channelized flow. Velocities onsite are generally moderate and some scouring of natural channels may occur during periods of higher runoff due to the nature of the topsoil deposits. The sediment carrying capacity of the natural water courses is generally mild-to-moderate in the undisturbed state. However, sediment depositions may be exhibited after periods of higher flows, especially in areas where channel velocities may change due to changes in vertical or horizontal alignment of the non-prismatic channel bed shape and meandering alignment. Peruviol fan soil deposits exist at the points of convergence flows exiting the creeks. The native vegetation helps to stabilize the soil against erosive forces generated in the runoff and provides some reinforcement of the natural slopes. Storm water quality of existing undeveloped conditions have documented sedimentary deposits at the discharged peruviol fans of the 6 established reaches allowing for the conveyance of upstream offsite drainage flows. More information can be found under the "Large Shed Area Water Quality" section of the report.

(a) ***Stormwater Management During Construction Activities***

The Doe Mill/Honey Run Special Planning Area (SPA), also referred to as "Valley's Edge" is located on the southeastern boundary of the city of Chico. Development of the Site will require development and implementation of a Storm Water Pollution Prevention Plan (SWPPP). The SWPP will include and institute the implementation of procedural phased LID's and BMP's throughout the duration of construction preemptively and consecutively.

(b) ***Post Construction Stormwater Quality Management***

The site will be required to implement post construction stormwater quality measures consistent with City of Chico standards with other applications specified in the "Environmental Impact Report" (EIR). Implementation of BMP's and LID'S will be required in compliance with the National Clean Water Act Section 404. All of this shall be provided with access to accomplish prescribed & routine scheduled maintenance of all Storm Water Pollution Best Management Practices (BMP's).

1.09 Project Analysis

(a) ***Hydrology***

The master watershed modeling for "Valley's Edge" is a single hydrology model (HEC-HMS). The rain gauges for the modeling analysis were gathered after seeing that the property overlays several rainfall regions/ gauges/ depths. The NOAA Atlas 14, Section 6 (California) rainfall intensity Isopluvials/ cartographs were utilized (see Article IV, Section 4.01 of this report and the attached Exhibit J packet). The rational method of basin run-off was conservatively applied to comply with the "City of Chico" and "Butte County" established standards for small Commercial/ and Land Development site runoffs. SCS method was utilized as well as the rational method because of the enormous size of the project. SCS method seems more appropriate for the scope of the project. The SCS TR-55 Method was analyzed and commutates the runoff of a significantly large water shed by instituting calculations including anti-seeded moister content, evapotranspiration and much more.

(b) ***Hydraulics***

GeoHEC-RAS software was utilized to build a 2D HEC-RAS model and perform an unsteady state pre-project/post project analysis for a range of storm events. After setting up the model, the hydrological "Q" values were input at specific reach and tributary reach locations (Exhibit F.1.0, in order the perform all unsteady state runs. The 2 year, 10 year and 100 year storm events were analyzed. Emphasis was placed on evaluating parameters such as velocity, depth, discharge and water surface elevation (HGL). The hydraulic analysis section of the report has a more in-depth hydraulic discussion.

The HEC-RAS model also includes the auxiliary system along Reach 1 which was designed by NorthStar Engineering for the purposes of avoiding over topping the Belvedere Subdivision adjacent to our site. The auxiliary system consists of a trapezoidal grass lined channel, a 54" pipe network (and associated manhole structures), along with inlet and outlet headwalls. More information can be found under the "Hydraulic Analysis" portion of the report or by referring to the study performed by North Star Engineering titled "Bancroft Drive Preliminary Flood Analysis – Design Memorandum."

Hydraulic Features such as proposed culverts were modeled for Post-Developed conditions. Exhibit D.1.1 depicts all existing as well as proposed hydraulic features onsite. Hydraulic features are subject to change depending on factors such as type and configuration of basins required for the specific detention needs.

(c) ***Large Shed Area Water Quality***

A Storm Water Pollution Prevention Plan (SWPPP) will be prepared to satisfy the construction site storm water runoff control requirements. The SWPPP will contain appropriate site-specific construction Best Management Practices (BMPs) that meet the requirement to control storm water pollution due to construction activities.

A Post Construction Storm Water Management Plan will be initiated to help reduce project site runoff. The developed portions of the project site will be divided into separate Drainage Managed Areas (DMAs), each implementing their own site design measures, source controls, storm water treatment and baseline hydromodification measures as defined in Section 15.50.080 in the City of Chico Code of Ordinances, in order to reduce project site runoff. Site design measures will utilize Low Impact Development (LID) standards to manage stormwater as close to its source as possible. The Regulated "Valley's Edge" Development will conform to the City of Chico LID and Hydromodification requirements.

II. Hydrologic Analysis

2.01 Methodology and Standards

(a) ***Methodology***

The **Methodology** for the hydrologic models included in the ""Valley's Edge" Drainage Report" were performed and provided utilizing ***Autodesk® Storm and Sanitary Analysis 2016 - Version 13.2.147 (Build 0)*** (which will be referred to as "Civil 3D-SSA / HEC-HMS" throughout the remainder of this report). The ASCII Output Report analysis summary was performed in multiple storm intensity events, which are the 2 yr., 10 yr., 25 yr., 50 yr. & 100 yr. return storm events by applying the NOAA Atlas 14, Volume 6, & The NOAA Website most recently updated Precipitation Frequency/&/Depth Gauge's for the 24 hour storm duration. (https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca) utilizing the latest updated *"ca2y, 5y, 10y, 25y, 50y & 100y24h-NOAA RAINFALL INTENSITY ISOPLUVALS/ CARTOGRAPH"* provided by NOAA and dated to be updated as of 2011. (see Exhibit J)
Incorporated Calculation Method are as follows;

- Rational Method
- TR-20 Method
- TR-55 Method
- HEC-1 Method

The overall area and size of the projects watersheds as a whole and individually exceed the maximum requirements for applying the "Rational Method" and the "TR-20 Method", however, they were requested to be reflected alongside of the TR-55 Method for comparison purposes. The TR-55 would ultimately be required. In addition, the HEC-1 Method was also included to accompany the report for anticipated request by reviewing agencies. All calculations and formulas used to achieve calculations are provided on the ASCII output reports provided by the software after running each scenario. Printed and accompanying this report in the printed versions are the ASCII output reports for the 100YR 24HR rain event in the TR-55 method and the Rational Method.

(b) ***Standards***

The **Standards** applied to the study for hydrology and flood potential analysis is based on a review of published information, reports, and plans regarding regional hydrology, climate, geology, water quality, and regulations. Relevant documents include the Chico Stormwater Master Plan, FEMA FIRM Maps, Big Chico Creek Watershed Alliance Existing Conditions Report (BCCWA, 2000), Butte County Flood Mitigation Plan (Butte County, 2006), the Butte Creek Watershed Project Existing Conditions Report (BCWC, 1998), and the California Water Plan Update (DWR, 2009)

(c) ***Hydrographs***

Hydrographs and hyetographs were developed utilizing the SCS TR-55 Method for each sub basin. the runoff data set is sampled for both a 1-hour interval and a 15-min interval, for the 2YR, 10YR and 100YR 24HR storm events. Both graphs were provided for clarity. The Hydrographs for the 15-min intervals were provided on 11x17 sheets (see Exhibit K.1.1).

2.02 Soil Types and Land Cover (Pre-Development)

A detailed and investigative Geotechnical Report, provided by "Geo Plus Partners" (February 27, 2019), was requested as an investigative report directly for the "Valley's Edge" development (See "Exhibit C.0.1"). The purpose of their investigation was to explore and evaluate the sub-surface conditions at various locations across the site in order to provide geotechnical design and construction recommendations for the project's infrastructure, roadway, utilities and structural design constraints or concerns. Their study results concluded that the site can be made suitable for a planned residential and light commercial development. The report includes the identification of near-surface hard bedrock, geotechnical issues that will impact some aspects of the infrastructure, roadway design and structural foundations. They have confirmed existing perched groundwater springs and possible seepage into trenches and under roadway seepage that will result in differential settling of soil fills and excavations. Existing Soil conditions identified in the mentioned geological report are a predominant hydrologic soil type D. "Doe mill-Jokerst" encompasses 94.4% of the total site and is the most dominant soil type present on the site. The geotechnical report referenced the review of the "National Resource & Conservation Services" "NRCS" WEB soils Survey website (https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx). Refer to NRCS soil survey for more specific definitions, characteristic and other properties of each soil type identified in the geotechnical report. (see Exhibits C.2.1, C.2.2, C.3.1, C.3.2 & C.4.0)

(a) ***Soil Identification #, Common Name and Description***

614--Doemill-Jokerst (0 to 3 % slopes)
This soil type is found on ridges between the elevation of 160 to 520 ft and has a very high runoff potential. The typical profile is layered and composed of very cobbly loam, followed by gravelly loam and finally bedrock.

615--Doemill-Jokerst (3 to 8 % slopes)
Having the same classification and runoff potential this soil is found between an elevation of 160 to 1000 ft. The typical profile is layered and composed of very cobbly loam, followed by gravelly loam and finally bedrock.

616 and 617--Jokerst-Doemill Typic Haploxeralfs (8 to 30 % slopes)
These soil types are found on ridges and have a very high runoff potential. 661 is found between elevations of 160 and 1120 ft while 617 is found between elevations of 260 and 800 ft. The typical profile is layered and composed of very cobbly loam, followed by gravelly loam, followed by gravelly clay loam and

bedrock. This soil type originated from loamy residuum weathered from volcanic breccia.

620 and 621--Doemill-Jokerst-Ultic Haploxeralfs, thermic complex
These soil types are found on ridges and have a very high runoff potential. 620 is found at 3 to 8 % slopes while 621 is found at 8 to 15 % slopes. They are both seen between elevations of 400 and 1700 ft. The typical profile is layered and composed of very cobbly loam, followed by gravelly loam and finally bedrock. This soil type originated from loamy residuum weathered from volcanic breccia.

622 and 623—Xerothents, shallow-Typic Haploxeralfs-Rock outcrop, cliffs complex
These soil types are generally found in canyons and have a very high runoff potential. 622 is found at 15 to 30 % slopes while 623 is found at 30 to 50 % slopes. They are both seen between elevations of 200 and 1500 ft. The typical profile is composed of gravelly clay loam, very gravelly clay loam, very cobbly clay loam and bedrock. This soil type originated from loamy residuum and/or colluvium derived from volcanic rock.

(b) ***<u>Land Cover/Land Use</u>***

Valley's Edge and the upslope drainage basin topography is comprised of relatively steep (+/- 4-5%) lava cap ridge top. Elevations range from 260' to 550' onsite and as high as 1400' at the high point of the offsite drainage basin. Both onsite and offsite basins lack vegetation including trees and bushes. The surface of the formation has been weathered into a relatively smooth surface with a network of small creeks and rivulets. Dirt roads and trails can be found throughout the site as well as sporadic lava rocks.

2.03 Watershed Delineation and Calculations (Pre-Development)

Please refer to Exhibits B.1.1 and B.1.2 for Pre-Developed offsite and onsite shed delineation maps. For a comparison of Pre-Developed and Post-Developed shed areas refer to Exhibit B.2.1 – "Pre-Developed vs. Post-Developed Shed Area."

(a) ***<u>Hydrologic Modeling Analytic Software</u>***

Storm and Sanitary Analysis Solution software was used for the hydrological analysis of "Valley's Edge". Utilizing the versatility of the program/software provided the opportunity to analyze the site in a multitude of hydrologic analysis methods, of which the Rational Method, SCS TR-55, SCS TR-20, HEC-1, were applied. The total site runoff was calculated for a range of different storm events with in each method listed above. Regional time series hydrographs of anticipated storm rainfall frequency estimates as provided by the "NOAA Atlas 14, Volume 6, Version. Data for the 24hr. storm duration with a 2yr, 5yr, 10yr, 25yr, 50yr and 100yr. event scenarios were applied. All calculations and formulas used to achieve calculations are provide on the ascii output reports provided by the software after running each scenario. Printed and accompanying this report in the printed versions are the ASCII output reports for the 100YR 24HR rain event in the TR-55 method and the Rational Method.

(b) Pre-Developed Model—Standard Delineation Techniques

Standard delineation techniques were used to divide the onsite and offsite areas into basins and sub-basins. Ridgelines were first drawn to isolate catchment areas (shed areas) and the boundaries were determined offsite as well as onsite. The "Valley's Edge", swales, ridges and hills were determined and labeled. The longest water course was drawn for each shed area and all creeks were located. The direction of drainage was then determined for each basin and sub-basin. The base model consists of seven (7) distinct Major Watershed Basins onsite and (6) distinct major Watershed Basins offsite. There is a total of forty-three (43) interior sub-basin areas in whole with thirty-seven (37) of these sub-basins lying within the proposed developments boundary, and the other six (6) are offsite (upstream). Exhibit B.1.1 depicts the offsite shed delineation while Exhibit B.1.2 depicts the onsite sheds.

(c) Pre-Developed Model—Applied Hydrologic modeling Calculations

The Pre-Developed Drainage Study for "Valley's Edge" consists of a total **3,896.61 Ac. (6.08845 Sq. Mi.)** catchment area of which **1,448 Ac.** is "ON-SITE" and the remaining **2,448.61 Ac.** "OFF-SITE" contributing inflow volumes from the eastern boundary of the project. The Pre-Developed project has a total of ***37 sub-basins*** onsite that directly or indirectly combine into the overall ***7 watershed areas***. Each of the 7 watershed areas discharge/outfall individually. Of which Approximately 50% of "Valley's Edge" direct runoff which contributes to the "Butte Creek" from the southernmost on site studied are from shed areas **"F & G" and the eastern portion of Shed Area "E"** and are conveyed through sub drainage culverts for reaches "R-5 & R-6" passing under honey run road. With 45% of direct run-off from shed areas **"B, C, D" & the western portion of Shed Area "E"** directly discharging to the "Little Chico Creek-Butte Creek Diversion Channel", conveyed through multiple sub drainage culverts reaches "R-1, R-2, R-3 (*R-2 & R3 are the eastern most upstream terminus of Comanche Creek*) & R-4. These flows outlet/outfall on the western boundary of this projects study limits. The remaining 5% of direct run-off distributed from **Shed Area "A"** is comprised of 6 individual sub-basins that discharge to the north by means of small overland sheet flows and/or small shallow concentrated flows through the adjacent developed neighborhood and contribute to the "Little Chico Creek".

(d) Pre-Developed Model – Storm Drainage Characteristics

Un-Developed – Offsite

The contributing offsite shed consists of 6 shed areas. Offsite **Shed Area's 1 & 4** are the major contributors to the main reaches that traverse the property from east to west. The offsite shed area is in its entirety the peak hydrologic shed for the Comanche creek water shed. Comanche creek traverses the "Valley's Edge" property boundary. The other offsite **Shed Area's "2, 3, 5 & 6"** contribute inflow to **Shed Area's F & G6**, which sheds south towards the Butte Creek diversion channel.

Pre-Developed - Onsite

The onsite area consists of 7 Main shed areas identified as **Shed Area's: A, B, C, D, E, F and G**. Each shed area has multiple interior sub basins: **A1-A6, B1-B3, C1-C4, D1-D7, E1-E2, F1-F8 and G1-G6**. This provides a total of 37 onsite basins or individual catchment areas. **Shed "A"** drains off to the north towards an existing developed area, with a discharge leaving the project boundary for each sub basin individually by means of sheet flow. **Shed "B"** has a small inflow from offsite **Shed Area 4**, with sheet flow and minor shallow concentrated flows proceeding through **Shed Areas B3 & B2**, accumulating at the Belvedere Subdivision. **Shed B** then continues via a bypass drain that was installed within the previous past years to accommodate the overflow that was flooding the belvedere subdivision. The bypass and existing infrastructure then discharge runoff into **Shed Area B1** which is then conveyed through a sub drainage pipe that combines the flow with **Shed Area C** immediately after crossing the western property boundary. **Shed C** has a large inflow from the eastern offsite **Shed Area 1** as channelized flow and continues through the remaining onsite sub basins for **Shed Area C** through this channelized flow. **Shed D** is completely onsite with no contributing offsite inflows. This shed area in whole contributes enough runoff to establish a channelized flow. **Shed F** has a large inflow from the eastern offsite **Shed Area 1** which is categorized as channelized flow and continues through the remaining onsite sub basins for **Shed Area F**. Please see Exhibit K.1.1 for Flow Hydrographs per basin for the Pre-Developed scenario.

2.04 Watershed Delineation and Calculations (Post-Development)

(a) ***<u>Post-Developed Model—Standard Delineation Techniques</u>***

Standard delineation techniques were used to divide the onsite and offsite areas into basins and sub-basins, such as those used in the Pre-Developed model. In addition, Valley's Edge Specific Plan (VESP) was implemented in order to delineate the Post-Developed proposed basins. Basins were developed by utilizing the details of the VESP, such as proposed roadways and areas to be developed, in conjunction with the Pre-Developed terrain characteristics, so as to not alter the sites natural drainage patterns. The base model consists of seven (7) distinct Major Watershed Basins onsite and six (6) distinct Major Watershed Basins offsite. There are thirty-four (34) interior sub-basin areas in whole for the Post-Developed model. Twenty-Eight (28) of these sub-basins lie within the proposed developments boundary, and the other six (6) are offsite (upstream). The Post-Developed shed areas can be seen on Exhibits D.1.1 – "Post Channels and Hydraulic Features" and F.1.1 – "Post HEC-RAS Geometry and Inflow." Exhibit B.2.1 compares the Post-Developed shed areas with the Pre-Developed sheds.

(b) Post-Developed Model—Applied Hydrologic modeling Calculations

The Post-Development Drainage Study for "Valley's Edge" consists of a total **3,896.61 Ac. (6.08845 Sq. Mi.)** catchment area, of which **1,448 Ac.** is "ON-SITE" and the remaining **2,448.61 Ac.** is OFF-SITE. Off-Site basins contribute to inflow volumes from the eastern boundary of the project. The Post-Developed project has a total of ***27 sub-basins*** onsite that directly or indirectly combine into the overall ***7 watershed areas*** as identified in the Un-Developed Model. Each of the 7 watershed areas discharge/outfall individually through dedicated culverts/sub drainage crossings. Approximately 50% of "Valley's Edge" direct runoff contributing to "Butte Creek" is conveyed from **Shed Areas "F & G and the eastern portion of shed area "E"** and is discharged through sub drainage culverts for reaches "R-5 & R-6" passing under Humbug Rd. 45% of direct run-off comes from **Shed Areas "B, C, D & the western portion of shed area "E"** and is directly discharged into the "Little Chico Creek-Butte Creek Diversion Channel," through multiple sub drainage culverts located in reaches "R-1, R-2, R-3 (*R-2 & R3 are the eastern most upstream terminus of Comanche Creek*) & R-4. These flows outlet/outfall on the western boundary of the project study limits. The remaining 5% of direct run-off is distributed from **Shed Area "A"** and is comprised of 6 individual sub-basins that discharge to the north by means of small overland sheet flows and/or small shallow concentrated flows through the adjacent neighborhood and contribute to "Little Chico Creek." Please see Exhibit K.1.2 for Flow Hydrographs per basin for the Post-Developed scenario.

(c) Post-Developed Model – Storm Drainage Characteristics

Post-Developed – Offsite

The contributing offsite shed is not projected to be developed and is situated on the foothills of the adjacent mountain range along the eastern boundary of the City of Chico. These foothills are protected for scenic appeal as well as grazing, falling under the protection of Williams act. The contributing offsite shed consists of 6 shed areas. Offsite **Shed Area's 1 & 4** are the major contributors to the main reaches that traverse the property from east to west.

Post-Developed – Onsite

The contributing onsite shed areas were determined by using the onsite existing terrain in conjunction with "Valley's Edge Specific Plan (VESP)." The VESP proposes Single Family, Multi-family & Light Commercial land uses and infrastructure for Post-Development. Post-Developed shed maps can be found on Exhibits D.1.1, F.1.1 and B.2.1. Exhibit B.2.1 compares the existing (Pre-Developed) shed areas with the proposed (Post-Developed) shed areas. The onsite area consists of 7 main shed areas identified as **Shed Area's: A, B, C, D, E, F and G.** Each shed area has multiple interior sub-basins: **A1-A6, B1-B2, C1-C4, D1-D5, E1-E2, F1-F2 and G1-G6.** This provides a total of 27 onsite basins or individual catchment areas. Shed **A** continues to drain off to the north towards an existing developed area. Shed **B** has a small inflow from offsite shed **4**, with sheet flow and minor shallow concentrated flows proceeding through shed areas **B1** and **B2**, accumulating at Belvedere Subdivision.

This runoff is then conveyed via a bypass drain and combines the flow with shed area **C**. Shed **C** has a large inflow from the eastern offsite shed area **1** as channelized flow and continues through the remaining onsite sub-basins for shed area **C** through this channelized flow. Shed **D** channelizes onsite and creates Reach 4 and Reach 4T. Shed **E**, which is categorized as overland flow, produces runoff that exits the site on the South-West side through multiple existing culverts. Shed F has a large inflow from the eastern offsite shed area **1** which is categorized as channelized flow and continues through the remaining onsite sub-basins for shed area **F**. Shed **G** will remain undisturbed. Please see Exhibit K.1.2 for Flow Hydrographs per basin obtained from SSA for the Post-Developed scenario.

2.05 CN Summary

(a) Pre-Developed Existing Site

A CN of 83 was derived from the chart (Composite Site see Hydrology Table in" Exhibit I.1.1" for specific shed area weighted/ composite CN values which were applied and utilized in the calculations)

Total Un-Developed Open Space area = 639.65 acres
(approximately 45% of "Valley's Edge" Overall Property boundary)

(b) Post-Developed Site

Exhibit B.3.1 – "Post-Development Impervious Calculations" depicts a table that demonstrates the calculation of impervious areas in the Post-Developed scenario.

ROADWAYS
Principle Arterials/Minor Arterials & Collector Right-Of-Way's – Impervious area width of the typical Major roadways have a 22' paved driving lane width on both sides of a 13' wide landscaped median, with a 5' wide pedestrian sidewalk/pathway on both sides of the driving lanes. The major roadways account for an overall approximate centerline length of 20,530 linear feet.
"Paved curbs & storm Sewers" has a ***CN = 98***

Minor local Right-Of-Way's – Impervious area width of the typical minor local roadways have a 36' paved driving lane width, with a 5' wide pedestrian sidewalk/pathway on a single side of the roadways. The minor local roadways account for an overall approximate centerline length of 89,663 linear feet.
"Paved curbs & storm Sewers" has a ***CN = 98***

<u>TOTAL ROAD AREA</u>
Calculations of the overall Impervious area of the driving lanes and parallel Pathways/Sidewalks.
<u>Total Roadway impervious area = 122.19 acres</u>
(approximately 9% of "Valley's Edge" Overall Property boundary)
<u>Total Landscape corridor pervious area = 274.06 acres</u>
(approximately 19% of "Valley's Edge" Overall Property boundary)

<u>LOT AREA(S) & CALCULATIONS</u>
Typical Lot "65' Wide x 106' Deep" with an overall average Area = 6,900 SF
<u>LOT AREA(S) & CALCULATIONS</u>
Typical Lot "65' Wide x 106' Deep"

<u>Total overall lotting area = 397.00 acres</u>
(approximately 28% of "Valley's Edge" Overall Property boundary)

Typical house pad = 3,000 S.F. ***CN – 98***
Typical drive pad 35' x 20' = 700 S.F. ***CN -98***
With the optional accessory building of a maximum = 300 S.F. ***CN – 98***
<u>Total Overall lotting Impervious Area = 214.43 acres</u>
(approximately 15% of "Valley's Edge" Overall Property boundary)

Front & Rear Yard (Considering the existing subsurface of minimal pervious lava flow deposits)
<u>Total Overall lotting Impervious Area = 183.00 acres</u>
(approximately 13% of "Valley's Edge" Overall Property boundary)

(c) <u>Post-Developed "CN" Values</u>

*<u>USE **CN 98** FOR DEVELOPED CN SITE AREA</u>*
A Hydrology Table was developed for the project which takes into account, the various soil types and areas of the undeveloped and developed areas affecting the project. A copy of the table can be seen below.

FULLY DEVELOPED URBAN AREAS	**Vegetation**					
Open space (lawns, parks, etc.)						
grass cover < 50%	Poor	68	79	86	89	< 50% grass cover
grass cover 50% to 75%	Fair	49	69	79	84	50 - 75% grass cover
grass cover > 75%	Good	39	61	74	80	> 75% grass cover
Impervious Areas						
Paved parking lots, roofs, driveways		98	98	98	98	Paved parking & roofs
Streets and roads						
Paved: curbs and storm sewers		98	98	98	98	Paved roads with curbs & sewers
Paved: open ditches (with right-of-way)		83	89	92	93	Paved roads with open ditches 50% imp
Gravel (with right-of-way)		76	85	89	91	Gravel roads
Dirt (with right-of-way)		72	82	87	89	Dirt roads
Urban Districts	**impervious**					
Commercial & business	85% imp	89	92	94	95	Urban commercial
Industrial	72% imp	81	88	91	93	Urban industrial
Residential Districts						
(by average lot size)	impervious					
1/8 acre (town houses)	65% impervious	77	85	90	92	1/8 acre lots
1/4 acre	38% impervious	61	75	83	87	1/4 acre lots
1/3 acre	30% impervious	57	72	81	86	1/3 acre lots
1/2 acre	25% impervious	54	70	80	85	1/2 acre lots
1 acre	20% impervious	51	68	79	84	1 acre lots
2 acre	12% impervious	46	65	77	82	2 acre lots
Western Desert Urban Areas						
Natural desert (pervious areas only)		63	77	85	88	Natural western desert
Artificial desert landscaping		96	96	96	96	Artificial desert landscape
DEVELOPING URBAN AREA	**(No Vegetation)**					
Newly graded area (pervious only)		77	86	91	94	Newly graded area
OTHER AGRICULTURAL LAND						
Pasture, grassland, or range	Poor	68	79	86	89	Pasture, grassland, or range
Fair		49	69	79	84	Pasture, grassland, or range
Good		39	61	74	80	Pasture, grassland, or range
Meadow, continuous grass, non-grazed		30	58	71	78	Meadow, non-grazed
Brush or brush/weed/grass mixture	Poor	48	67	77	83	Brush
Fair		35	56	70	77	Brush
Good		30	48	65	73	Brush
Woods & grass combination	Poor	57	73	82	86	Woods & grass combination
Fair		43	65	76	82	Woods & grass combination
Good		32	58	72	79	Woods & grass combination
Woods	Poor	45	66	77	83	Woods
Fair		36	60	73	79	Woods
Good		30	55	70	77	Woods
Farmsteads		59	74	82	86	Farmsteads

Reference: Autodesk® Storm and Sanitary Analysis, Version 2020

Table 1: Curve Number Table (per SSA)

2.06 HMS Model Results & Discussion of Proposed Improvements

(a) Flow Rate & Volumetric Comparison

A summary of the existing and developed site storm flows from the HMS Model output is listed below. The information presented below represents runoff peak flows and total volumes for the entire system using Tr55 method. System discharge values are smaller than cumulative Q values due to evaporation, infiltration and transpiration that SSA takes into account. Exhibit I.0.0 – "Basin Peak Runoff for Entire System (PRE)" and Exhibit I.0.1 – "Basin Peak Runoff for Entire System (POST)" demonstrate the Pre-Developed and Post-Developed peak runoff for the entire system in SSA.

PRE-DEVELOPED STATE				POST-DEVELOPED STATE			
Q10 (cfs)	V10 (A.F)	Q100 (cfs)	V100 (A.F)	Q10 (cfs)	V10 (A.F)	Q100 (cfs)	V100 (A.F)
4329	1093	6849	1697	5209	1128	10007	2156

AP-Whole Site (see Exhibit – H & I for individual Watershed/ Catchment area calculations and add point calculations of Quantities and flow Volumetrics. The tables found in Exhibits I.1.0 and I.1.1 represent only the Pre-Developed values. Tables for the Post-Developed scenario are found as excel files in the Exhibit I folder, as well as in the SSA model).

(Historic Complete) Comprised of:
-Offsite Up-Stream Areas = 2,450.07 Acres
-Un-developed On Site Areas = 639.65 Acres
-Developed On-Site Areas = 794 Acres

The development of Valley's Edge without mitigation will increase the total runoff in the system by **880 cfs** and **3158 cfs** for the 10 year and 100 year storm events respectively. The total volume of runoff without mitigation will increase by **35 acre-feet** and **459 acre-feet** for the 10 year and 100 year storm events. These values represent the minimum amount of runoff that will need to be detained in order to match the Pre-Developed conditions. Proposed roadways will require culvert crossings, with detention basins in some areas, thus minimizing the flow. The hydraulic analysis portion of the report demonstrates the Post-Developed discharge leaving the site to be lower than the Pre-Developed conditions.

All existing channels will remain undisturbed in the Post-Developed scenario, with the exception of proposed culvert crossings directly upstream of proposed roadways. Detention basins are to be proposed in areas where required. Type and configuration of basins required for the specific detention/retention needs will be provided during the final design phase.

2.07 Storm Water Retention Summary

(a) ***Crossing Up-Stream Retention Reductions***

In order to provide a net reduction of storm water runoff from the site, multiple on-site retention/detention staging areas will be utilized. The retention will be assigned a specific associated shed area and a design storage retention volume and Time of Retention drainage along with the actual computed storm water runoff values from the HMS Model. The City of Chico Design Standards indicates that the proposed ponding volumes shall be set at 100% of the "design value." Ponding in Post-Development will only occur at existing roadways as it does in the Pre-Developed scenario. Proposed HMS output volumes and the proposed retention/detention pond sizes with the percentage (%) excess capacity will be determined during the final design phase.

III. Hydraulic Analysis

3.01 Methodology & Standards

(a) ***Analysis Method***

The "**Methodology"** used for the hydraulic models included in the "Valley's Edge" Drainage Report consists of various software including GeoHEC-RAS, HEC-RAS and Civil 3D along with best engineering judgement. The Engineering Software GeoHEC-RAS was used for the majority of the hydraulic analysis. GeoHEC-RAS is an AutoCAD, MicroStation and ESRI ArcGIS compatible interactive 2D/3D graphical user interface data wrapper to HEC-RAS. It is used to construct HEC-RAS models from a variety of data sources. The final results were then exported into HEC-RAS for submittal. The software will be referred to as HEC-RAS for the remainder of the report. The hydraulic model was utilized to perform a full 2D unsteady state run for a range of storm events.

Running the analysis yielded discharge and velocity rates as well as depth and water surface elevations at locations of interest. Results from a range of storm events, including the 2-year, 10 year and 100 year can be seen in the results section of this article.

Culverts and roadways were analyzed by grouping them into SA-2D connections that contain the extents of the entire floodway in that region. SA-2D connections in HEC-RAS are used to define connections such as weirs and roads where overflow might occur. Exhibit D.1.0 – "Channels and Hydraulic Features" illustrates "Road Connections" and the culverts associated with them. Floodways and channels however, were analyzed by station numbers along the alignment of each reach (Exhibit E.1.0- "Floodway Section Analysis").

(b) ***Standards***

The **"Standards"** that were applied to the hydraulic study are based on review of published information, reports, and plans regarding regional hydraulic regulations.

3.02 Hydraulic Descriptions & Findings (PRE-PROJECT)

Please refer to Exhibit D.1.0 which depicts the major and minor reaches running through "Valley's Edge" Edge, along with the locations of culverts, pipes and other hydraulic features. Manning's "n" values were obtained from the USGS database and can be seen on Exhibit A.1.1- "USGS Land Cover Map." The file in TIF format can be seen under the "HEC-RAS (Model and Attachments)" folder.

(a) ***Reach 1***

Reach 1 channelizes onsite from runoff forming onsite and partial runoff entering in from the east. The channel meanders to the west towards Belvedere Subdivision and is intercepted by an improved channel. More than half of the flow goes through a pipe at the property line adjacent to Belvedere and outflows on the south side of Belvedere back unto our site (Exhibit E.1.0). High flows however, are diverted around Belvedere subdivision, in order to avoid over topping Dawncrest Dr during a high frequency storm event. A study on this channel has been performed by NorthStar Engineering before and after the diversion channel was in place for the purposes of avoiding this problem. The report is titled "Storm Water Analysis for Storm Drain Augmentation Belvedere Subdivision." The auxiliary system consists of a trapezoidal grass lined channel, a 54" pipe network (and associated manhole structures), along with inlet and outlet headwalls. Culvert C1A represents the auxiliary bypass pipe (Exhibit D.1.0). Runoff is discharged back unto the site after leaving the pipes and exits through a culvert on the West side (Potter Rd) of the property.

Pipe C1A

C1A is a 54" PVC bypass auxiliary pipe that captures the remainder of the runoff that does not get passed off into C1B. The inlet consists of a 72" square edged headwall with a headwall at the outlet as well. A trash guard protects the pipe from debris that could possibly enter inside. The pipe moves underground alongside Belvedere subdivision (Exhibit D.1.0) with an average slope of 2%. More detailed information can be found in the HEC-RAS model. Please refer to Figure 2A below for an illustration.

Pipe C1B

C1B is a 42" storm drain conveyance system that goes through Belvedere subdivision. It consists of a 42" headwall at the inlet and outlet. A trash guard protects the pipe from debris. The 42" PVC pipe is sloped at an average of 2% and makes four 90-degree bends before exiting back unto our site. Refer to Figure 2A and 2B for an illustration.

Figure 2A: Headwall and trash guard at inlet for C1A

Figure 2B: Headwall and trash guard at outlet for C1A

Figure 2C: Headwall and trash guard at inlet for C1B

Figure 2D: Headwall and trash guard at outlet for C1B

Culvert C1E

Culvert C1E represents the discharge point of the flow exiting our site. The CMP (Corrugated Metal Pipe) pipe arch shaped culvert with a 48" rise and a 66" span is sloped at 0.6% and is 51' in length. An illustration is depicted on Figure 2E.

Figure 2E: Culvert C1E Inlet

(b) ***Reach 2***

Runoff from the offsite shed 1 enters the site from the NE side as both sheet and channelized flow. The channelized flow enters from the east and meanders through the site. This channel is referred to as **Reach 2 (R2).** Onsite tributary areas channelize and find its way into this main channel until it exits the site through two underground culverts on the west side going through and across Potter Road. The culverts have a parallel orientation and are spaced about a foot away from each other. The culverts are sloping east to west at 4 % and are 72" wide with a 45" rise. The creek continues offsite and disperses into the Butte Creek Diversion Channel. A natural earthen levee runs along both sides of Potter Rd covering reaches R1, R2, R3 and R4.

Culverts C2A and C2B

These identical CMP culverts are sloped downward at 0.3% projecting from fill. They are pipe arched culverts with a 45" rise and a 72" span. Upon discharging, the creek continues offsite and disperses into the Butte Creek Diversion Channel.

Figure 3: Culverts C2A & C2B inlet

(c) ***Reach 3***

Reach 3 forms by onsite channelization and travels from east to west until it exits the site in an underground culvert going across Potter Road. The pipe is 12" in diameter and about 75' long. Channelization continues offsite until the runoff is discharged into the Butte Creek Diversion Channel.

Culvert C3A

Reach 3 has a 12" corrugated metal pipe culvert projecting from fill that disperses flow offsite. The pipe is about 75' in length and sloped at about 1%.

Figure 4A: Culvert C3A Inlet

Figure 4B: Culvert C3A Outlet

(d) ***Reach 4***

Onsite runoff channelizes mid site and forms **Reach 4 (R4).** It moves through the site in the general east-west direction until it converges with a tributary reach (R4T) and then curves towards the northern direction where it travels through two different culverts under Potter Rd and discharges off the site. A natural earthen levee runs along both sides of Potter Rd.

Culvert C4A and C4B

There are 2 culverts leaving the site, C4A and C4B. C4A is a reinforced concrete pipe (RCP) culvert with a groove end entrance. It is 48" in diameter and is projecting from fill. C4B however, is a CMP type culvert with a headwall at the inlet and is 36" in diameter. They both share an average slope of 1.7%. Tree branches are causing an obstruction at the upstream end. Refer to the HEC-RAS model for more details. An illustration can be seen in the figures below.

Figure 5A: Culverts C4A and C4B Inlet

Figure 5B: Culverts C4A and C4B Outlet

(e) ***Reach 5***

Onsite runoff channelizes mid site and forms **Reach 5 (R5).** Water in the channel moves through the site in the general NE-SW direction until it goes through two CMP culverts under Humbug Rd and then converges with Reach 6 (R6) upon exit. Under. The channel continues downstream and merges with the Butte Creek Diversion Channel.

Culvert C5A

Two identical CMP culverts pass flow offsite in reach 5. The culverts are pipe arch in shape with a 36" rise and a 58" span, projecting from fill. C5A is sloped at 0.3% while C5B is sloped at 0.6%. Upon exiting, the reach converges with reach 6 and the flow eventually finds its way into Butte Creek.

Figure 6A: Culverts C5A and C5B Inlet

Figure 6B: Culverts C5A and C5B Outlet

(f) ***Reach 6***

Comanche Creek (**Reach 6**) forms from the runoff of offsite sheds 2 (North of Doe Mill Ridge) and offsite shed 3 (South of Doe Mill Ridge). Exhibit B.1.2 clearly depicts the delineated shed areas as well as Comanche Creek. The creek traverses the Southern area of the Valley, entering in from the east in two reaches and converging into one shortly after it enters. Due to the high runoff potential, reach 6 (Comanche Creek) experiences the most flow. There is a total of 4 CMP culverts present at the discharge point in reach 6. It meanders through the site and exits the site through these culverts on the south side at Humbug Road. The creek continues moving downstream and eventually merges with the Butte Creek Diversion Channel.

Culverts C6 (C6A, C6B, C6C, C6D)

Three identical pipe arch culverts with a 40" rise and a 65" span run parallel to each other. These are C6A, C6B and C6C They are concrete lined at the bottom and share an average slope of 0.4%.

C6D is a circular CMP culvert projecting from fill with a diameter of 84". It is sloped downward at an average 0.8% slope and is about 48' in length. Please refer to Figure 7 below for an illustration.

Figure 7A: C6 Culverts inlet with obstructing branches

Figure 7B: C6 Culverts outlet

(g) ***Minor Rivulets and Culverts***

Besides the main channels and the culverts associated with them, there are other onsite rivulets and culverts along the property. This includes culverts CA THROUGH CK (EXHIBIT D.1.0). Most of these culverts are 12" CMP culverts and pass flow that is considered overland release. Some of them are RCP culverts with a diameter of up to 24." For detailed information please examine the HEC-RAS model or refer to Exhibit D.1.0. Most of these culverts discharge offsite. Culverts CF through CK discharge into Crough Ditch, which finds its way into Comanche Creek and eventually into Butte Creek. Pictures of these minor culverts are found under the "video's pics" folder which is located in the exhibits folder.

3.03 Model Development (PRE-PROJECT)

The development of the 2D model began by identifying the existing onsite conditions. GeoHEC-RAS was used for the development of the model. Discharge locations were determined based on the characteristics of the channels and basin boundaries. Exhibit F.1.0 shows the discharge locations and values used in the model, as well as the geometry used in HEC-RAS. Steps taken to develop the Pre-Project (Pre-Developed) model are outlined below.

(a) ***<u>Adding and Importing Layers</u>***

Geospatial Base Map

Firstly, map coordinates were assigned using a projected CRS region of NAD83(HARN)/California zone 2 (ft.us.). This is also known as a GIS coordinate system of CAHP-IIF. A geospatial base map layer was then added into GeoHECRAS with a Hybrid Google Map as the background imagery.

Land Cover Data

Next, a Land Cover Data (NLCD) layer was added into the model to depict manning's "n" values to be used in the 2D flow area. The projected NLCD Land Cover information is derived from the USGS land cover database. It contains 8 levels of classification and generalizes to the level of vegetative physiognomy. Exhibit A.1.0 demonstrates the manning's "n" values used for the onsite hydraulic analysis. The file associated with Land Cover is located under the "HEC-RAS (Model and Attachments)" folder.

Land XML Surface

A Land XML surface was then exported out of Civil 3D and imported into GeoHEC-RAS as a terrain layer. The projected CRS region was defined as CAHP-IIF to align the surface with the geospatial base map layer. An elevation color fill option was selected for the purposes of distinguishing between ridges, high points and low points on the terrain.

Civil 3D Layers

Three Civil 3D layers were also added into the model for the purposes of defining the study parameters. An Undeveloped Basin (Exhibit B.1.2), Property Boundary and a Channel Network (Exhibit D.1.0) DWG drawing were imported into the model.

Shapefiles

Lastly, linework in the form of shapefiles was exported out of the Topographic survey and into the HECRAS model for multipurpose use. This includes culvert centerline's, road networks and all onsite buildings. These shapefiles were later used to define and delineate all parameters in the 2D flow area.

(b) ***Defining Flow Areas and all Flow Elements***

Exhibit F.1.0 clearly illustrates the HEC-RAS geometric parameters used such as 2D Flow Area, BC lines (discharge locations), break lines and SA-2D connections (roads).

2D Flow Area

A 2D Flow Area was drawn to include all channels and the extents of the study. The Element spacing for the uniform 2D mesh was defined to be 30ft for the major flow area and between 15' and 20' for major break lines. The remaining areas use an element spacing value that varies across the site.

Boundary Condition Lines (Discharge Locations)

Boundary Condition (BC) Lines were drawn at areas of interest based on the Undeveloped Basin Civil 3D drawing that was projected. These BC lines are used in defining inflow and outflow at specific locations. BC lines were drawn along the river network at the shed area discharge locations where flow was introduced. BC lines were also drawn offsite at the downstream end as normal depth BC lines, representing the outflow locations.

Break Lines

Break lines were drawn along the flow lines of the river network as well as some overbank areas. The roads were enforced as break lines as well in order to properly align the mesh cells. Break lines were also drawn at locations where profile sections would need to be cut for analysis purposes. These break lines ensure that the faces of mesh cells align with locations of interest, including roads and section profiles, so as to improve the accuracy of the model.

SA-2D Connections (Roadways)

Shapefiles defining roadway centerlines (CL's) were imported from Civil 3D into the model. These CL's were assigned to be SA-2D Connections in HEC-RAS that define the roadways through which culverts and pipes pass water through. The remaining Connections were drawn in the model to represent culverts or pipes and any potential weir overflow that may occur. A SA-2D Connection that is not a roadway was defined as well and named "RDC1A" This was done in order to assign the 54" bypass auxiliary pipe that diverts the flow coming towards Belvedere subdivision.

Culverts

Shapefiles from Civil 3D were imported into HEC-RAS and assigned as culvert CL's (Centerlines). Invert elevations extracted from a topographic survey were assigned to all culverts. Parameters on the type and size of culverts were then defined. Please refer to the HEC-RAS model for additional information.

(c) ***Unsteady Flow Data and Computational Options***

Unsteady Flow Data

After the geometry has been properly defined, we proceeded to input hydrograph flow data. For all storm intensities modeled, a 24-hr. storm with 15-minute intervals was used. All BC lines, except for the most downstream ones, were defined with flow hydrograph data. Normal depth was assigned to downstream BC lines with respective slopes defined. Exhibit F.1.0 shows the discharge locations used to input hydrograph data for a range of storm events.

Computational Options

Diffusion wave was used as the computational equation for the analysis with a subcritical flow regime. A 1-day simulation time window was used with a 1 second computational time step interval. The output intervals were set to 10 minutes. For additional information please refer to the HEC-RAS model.

3.04 Results (PRE-PROJECT)

Time series results were produced after the analysis was performed. The peak values from the 2yr, 10yr and 100yr storm events were analyzed and used to determine existing onsite conditions. Emphasis was placed on analyzing the velocity, depth, HGL (Hydraulic Grade Line) and discharge values along culverts, roads and along entire channel lengths. Discharge values may deviate from original values when performing a new analysis due to HEC-RAS' computational method. A few culverts are partially blocked or completely buried, thus restricting their conveyance potential.

(a) ***Discharge at Existing Culverts and Roads (PRE)***

Culverts and roadways were analyzed by grouping them into SA-2D connections in HEC-RAS that contain the extents of the entire floodway in that region. They are referred to as "Road Connections" in the Exhibits and they contain culverts/pipes that were analyzed as a group. The name of each "Connection" specifies the roadway and the culverts associated with that roadway (Table 2: Pre-Developed Discharge at Existing Connections). Exhibit D.1.0 illustrates "Road Connections" and the culverts or pipes associated with them. These outfall locations receive inflow from designated channels, also referred to as "Reaches." Table 1 below shows the magnitude of discharge at all roads (SA-2D Connections) that was computed through HEC-RAS. A 3-Dimensional Video of the floodway is available for the 100-year scenario and can be found in the folder titled "Videos-Pics," which is located under Exhibits. For any additional information refer to the HEC-RAS model.

DISCHARGE AT EXISTING CONNECTIONS (HECRAS)							
2 Year Storm (cfs)							
R1		**R1+R2+R3**		**R4+R4T**		**R5+R6**	
RD(Dawncrest)C1A,C1B		**RD(PotterN)C1-C3**		**RD(PotterS)C4**		**RD(Humbug)C5,C6**	
Qtot =	89.4 cfs	Qtot =	593.3 cfs	Qtot =	276.6 cfs	Qtot =	1440.2 cfs
C1A =	49.5 cfs	C1E =	69.1 cfs	C4A =	96.4 cfs	C5A,B =	161.9 cfs
C1B =	40.0 cfs	C2A,B =	197.0 cfs	C4B =	68.2 cfs	C6A,B,C =	290.9 cfs
Weir Flow =	0	C3A =	0	Weir Flow =	111.9 cfs	C6D =	166.3 cfs
		Weir Flow =	323.4 cfs			Weir Flow =	821.2 cfs
10 Year Storm (cfs)							
R1		**R1+R2+R3**		**R4+R4T**		**R5+R6**	
RD(Dawncrest)C1A,C1B		**RD(PotterN)C1-C3**		**RD(PotterS)C4**		**RD(Humbug)C5,C6**	
Qtot =	153.1 cfs	Qtot =	1027.5 cfs	Qtot =	392.2 cfs	Qtot =	2360.5 cfs
C1A =	88.5 cfs	C1E =	94.6 cfs	C4A =	102.9 cfs	C5A,B =	165.5 cfs
C1B =	64.6 cfs	C2A,B =	221.1 cfs	C4B =	71.6 cfs	C6A,B,C =	324.0 cfs
Weir Flow =	0	C3A =	0	Weir Flow =	217.7 cfs	C6D =	202.7 cfs
		Weir Flow =	707.7 cfs			Weir Flow =	1668.4 cfs
100 Year Storm (cfs)							
R1		**R1+R2+R3**		**R4+R4T**		**R5+R6**	
RD(Dawncrest)C1A,C1B		**RD(PotterN)C1-C3**		**RD(PotterS)C4**		**RD(Humbug)C5,C6**	
Qtot =	306.1 cfs	Qtot =	2048.2 cfs	Qtot =	822.3 cfs	Qtot =	4941.2 cfs
C1A =	170.1 cfs	C1E =	139.3 cfs	C4A =	117.0 cfs	C5A,B =	174.5 cfs
C1B =	111.4 cfs	C2A,B =	260.5 cfs	C4B =	79.2 cfs	C6A,B,C =	375.2 cfs
Weir Flow =	24.6 cfs	C3A =	0	Weir Flow =	626.1 cfs	C6D =	275.4 cfs
		Weir Flow =	1644.1 cfs			Weir Flow =	4113.3 cfs

Table 2: Pre-Developed Discharge at Existing Connections (HEC-RAS)

CONNECTION: "RD(Dawncrest)C1A, C1B"

Flow from Reach 1 (R1) is passed through two PVC pipes (C1A and C1B) that are part of this connection. The 54" and 42" pipes were able to convey runoff from the 2- and 10-year storm events, but not the 100-year storm event. The rise in backwater during the 100-year storm event caused water to spill over into the adjacent subdivision (Belvedere). A computed flow rate of **25 cfs** out of a total of **306 cfs** was observed to go over Dawncrest Dr and into the adjacent subdivision. This contradicts the study on these pipelines, which was done by NorthStar Engineering, where they reported that the PVC pipes convey the full 100-year flow. Since we are using a higher frequency TR-55 rain gauge in our study, the recorded discharge (Q) values at that connection are higher, therefore exceeding the handling capacity of the pipes. Table 1 above depicts the discharge values going over and through this connection.

CONNECTION: "RD(PotterN)C1-C3"

This Connection includes the North side of Potter Rd with culvert crossings including C1E, C2A, C2B and C3A. Channels R1, R2 and R3 convey flow that outflows over this road and through the culverts. This connection experiences weir overflow during the 2yr, 10yr and 100yr storm events. During the 2-year storm event a flow of **323 cfs** is passed over the roadway. The 10yr and 100yr storm events experience a weir overflow of **708 cfs** and **1644 cfs** respectively. The ponding depth of the spillway can be seen on Exhibit E.1.0.

CONNECTION: "RD(PotterS)C4"

Reach 4 (R4) and Reach 4 Tributary (R4T) with their respective catchments pass flow to culverts C4A and C4B. Weir overflow occurs during all storm events at this connection and overtops the road. A field investigation indicates that the inlets at culverts C4A and C4B are partially blocked, therefore minimizing discharge conveyance. Observe Table 1 above for all conveyed discharge values on major road Connections.

CONNECTION: "RD(Humbug)C5,C6"

This connection experiences the highest runoff rate due to its large contributing basins. Channels R5 and R6 discharge water into all C5 and C6 culverts. The flow rates of the overflow going over and through this connection during the 2- and 10-year storm events are **1440** cfs and **2361** cfs respectively. A 100-year storm event delivers a total discharge of **4941** cfs with **4113** cfs going over the road.

Minor CONNECTIONS: Culverts CA-CK

Minor connection regions were observed as well in order to determine the handling capacity of the culverts. It was observed that during a 100-year storm event, culverts CA and CK couldn't handle the peak stormwater runoff.

(b) __DISCHARGE COMPARISON (HEC-RAS VS. SSA) (PRE)__

The HEC-RAS generated discharge values were compared with the cumulative basin outflow values obtained from Storm and Sanitary Analysis (SSA). Most of the discharge values obtained from HEC-RAS (Table 1) are relatively close to those obtained from SSA (Table2). Table 2 below depicts the cumulative basin outflow (Q) values at road Connections obtained from SSA. Refer to Exhibit L.1.1- "Flow Hydrographs at Existing Connections (HEC-RAS)" for hydrograph data at connections obtained from HEC-RAS.

DISCHARGE AT EXISTING CONNECTIONS (SSA)							
2 Year Storm (cfs)							
R1		**R1+R2+R3**		**R4+R4T**		**R5+R6**	
RD(Dawncrest)C1A,C1B		**RD(PotterN)C1-C3**		**RD(PotterS)C4**		**RD(Humbug)C5,C6**	
Qtot =	94.11	Qtot =	703.44	Qtot =	289.35	Qtot =	1602.83
10 Year Storm (cfs)							
R1		**R1+R2+R3**		**R4+R4T**		**R5+R6**	
RD(Dawncrest)C1A,C1B		**RD(PotterN)C1-C3**		**RD(PotterS)C4**		**RD(Humbug)C5,C6**	
Qtot =	159.18	Qtot =	1187.19	Qtot =	403.63	Qtot =	2721.79
100 Year Storm (cfs)							
R1		**R1+R2+R3**		**R4+R4T**		**R5+R6**	
RD(Dawncrest)C1A,C1B		**RD(PotterN)C1-C3**		**RD(PotterS)C4**		**RD(Humbug)C5,C6**	
Qtot =	312.5	Qtot =	2333.32	Qtot =	849.43	Qtot =	5406.92

Table 3: Pre-Developed Discharge at Existing Connections (SSA)

(c) ***<u>Floodway and Channel Analysis (PRE)</u>***

Floodways and channels were analyzed by station numbers along the alignment of each reach (Exhibit E.1.0) as well as at road Connections. Reach 3 has been disregarded because it was analyzed with the floodway of Reach 2. Profile line sections were cut every 500ft or 1000ft along channels and at other critical locations. They were assigned station values beginning at the most upstream ends of the channels. Exhibit E.1.0 shows the stations, profile cuts and detailed hydraulic information at each station, including Water Surface Elevation (HGL), Velocity and Depth. For a schematic of depth and velocity values along the floodplain see Exhibits G.1.1- "Floodway Depth (HEC-RAS)" and G.2.1- "Floodway Velocity (HEC-RAS)."

Water overtopped all major roads during the 100-year storm event due to the inability of the culverts to convey the flow. This backwater effect caused backwater flooding directly upstream of these roadways. Exhibit E.1.0 illustrates all Pre-Developed floodways and shows overtopping at each location. Hydrographs and Profile Plots generated by HEC-RAS can be seen on Exhibits L.1.1 and M.1.1-"Reach Profile Plots (HEC-RAS)" respectively. HEC-RAS Generated reports can be seen on Exhibits O.1.1, O.1.2 and O.1.3 for the 2yr, 10yr and 100yr scenarios respectively. Videos of the modeled floodway were recorded for the 100-year storm scenario and are located under the folder titled "Videos-Pics," which is located in the "Exhibits" folder.

Reach 1

Reach 1 forms by onsite channelization and moves in the general East to West direction. Maximum peak values such as the Hydraulic Grade Line (HGL), Velocity and Depth were obtained at various station locations. Exhibit E.1.0 clearly depicts these values for the 2, 10-

and 100-year storm events. Pipes C1A and C1B are unable to convey the channel flow during the 100-year storm event, therefore causing water to spill over unto Dawncrest Dr. The profile section at Reach 1 station 33+42 (R1-33) shows an HGL of **312.87** ft for the 100-year storm.

Reach 2

Channel R2 enters the site from the East and moves in the general East to West direction until exiting the site through multiple culverts on the West. The Maximum HGL, Depth and Velocity values at various section profiles can be seen on Exhibit E.1.0. Culverts C1E, C2A, C2B and C3A receive the flow conveyed by channels R1, R2 and R3. Weir overflow occurs during the 2, 10- and 100-year storm events and causes ponding on the roadway. Exhibit E.1.0 demonstrates the degree to which ponding occurs.

Reach 4/4T

Reach 4 forms by onsite channelization and moves in the general East to West direction until it is intercepted by a tributary reach (R4T). The culverts at this connection are not able to convey discharge during any of the storm events, thus causing water to back up and spill onto the roadway (Potter Rd).

Reach 5/6

Reach 6 enters the site from the East side and moves in the general Southwest direction. The high runoff that is experienced forms a floodway along R6 to which R5 contributes as well. Reach 6 station 56+62 (R6-56+62) shows a maximum HGL of **261.75** ft during the 100-year storm event.

3.05 Proposed Hydraulic Improvements (POST-PROJECT)

All Existing onsite Hydraulic Features will remain undisturbed with the exception of any plugged/buried culverts that will need to be cleaned before development can begin. These culverts are included in the Post Developed model. Additional culvert crossings are proposed at connections where roadways will be required. This includes collector roadways as well as local residential roadways. Please refer to "Valley's Edge Specific Plan (VESP)" for more details regarding types of roadways to be proposed. Exhibit D.1.1- "Channel and Hydraulic Features Post" depicts the modeled roadway as well as proposed hydraulic features. These culvert crossings ensure that water is detained directly upstream of roadways that span across existing channels. Exhibit E.1.1- "Post Floodway Section Analysis" demonstrates the extents of the water that will be detained.

All Culverts are proposed to be of Corrugated Metal Pipe (CMP) material with headwalls at the inlets. Multiple culvert sizes had to be used at each connection in order to account for all storm events. The culverts were configured in a way that minimizes discharge from high frequency storms while also allowing limited conveyance during low frequency storms without overtopping the road. Some culverts had to be placed above others in order to meet the Post-Developed discharge requirements (Exhibit N.1.0 – "Proposed SA-2D Connections"). Culvert types, sizes and configurations may be subject to change during the Final design phase. Factors such as, type and configuration of basins required for the specific detention/retention needs, will dictate the types of culverts to be used. Culvert inverts and their configurations may change depending on the earthwork cut volumes depicted from retention/detention design. Profiles of the proposed roadways are subject to change as well. Roadways will be designed to ensure that they do not overtop during a 100 year storm event. These changes will affect floodway factors such as depth, velocity and HGL as well as flood extents, especially directly upstream of the proposed connections. These design methods and proposed improvements are aimed to reduce Post-Developed flows. All final designs will meet City of Chico and Butte County standard specification requirements.

SA-2D connections were created representing the proposed collector roadway spanning across Reaches 2, 4 and 4T as well as the proposed local residential roadway crossing over Reach 5 and 6 (Exhibit D.1.1). With trial and error, these connections were raised high enough to detain runoff directly upstream without overtopping the roadways. These proposed connections were named based on the type of roadways proposed and the culverts going through them. The proposed culverts are named based on the reach that they are associated with. For example, the proposed connection "RD(Collector)CP2" represents a collector roadway with proposed culvert 2 going through it.

Please refer to Exhibit D.1.1- "Channel and Hydraulic Features Post" which depicts the major and minor reaches running through "Valley's Edge" along with any existing and proposed onsite hydraulic features.
Exhibit N.1.0 – "Proposed Connections" illustrates a section view of the proposed connections and associated culverts. For a quantitative analysis of discharge values please see the "Results (Post-Project)" section below.

(a) ***Reach 1 (Post)***

Reach 1 channelizes onsite from runoff forming onsite and partial runoff entering in from the east. In order for the existing pipe network to convey the runoff without overtopping Dawncrest Dr, some of this runoff is diverted towards Reach 2 (32.8 cfs for 100yr). The new proposed basins will minimize the runoff coming towards the Belvedere subdivision. Exhibit D.1.1 shows the existing along with the proposed onsite Hydraulic Features, as well as the proposed Post-Developed basins.

(b) ***Reach 2-CONNECTION: "RD(Collector)CP2" (Post)***

Before exiting the site, Reach 2 is intercepted by a collector roadway with proposed Culvert Crossings **CP2A**, **CP2B** and **CP2C** (Culvert Proposed 2C) about 3000 ft away from the Western property line. This ensures that the 2, 10 and 100 year runoff leaving the site after Post Development is lower than the Pre Development. The connection through which these culverts go through is called "**RD(Collector)CP2**." Exhibit D.1.1 depicts the existing along with the proposed onsite hydraulic features.

Culvert **CP2A** is **8**' in diameter while **CP2B** is **6**'. Culvert **CP2C**, which is **4'** in diameter, was added so that the road does not overtop during the 100 year storm event. All culverts modeled are made of CMP material with headwalls at the inlets.

(c) <u>Reach 3- CONNECTION: "RD(Collector)CP3" (Post)</u>

Besides having a new basin configuration contributing to the flow, Reach 3 is left undisturbed for the most part. Culverts **CP3A** and **CP3B,** crossing the proposed collector roadway at connection "**RD(Collector)CP3,"** contribute flow into channel R3. It was observed that 68.3 cfs during the 100 year storm event is conveyed through the culverts and into Reach 3.

Culvert **CP3A** is **2'** in diameter and placed above **CP3B,** which is **3'** in diameter. This geometrical configuration ensures that the 100 year runoff is conveyed without overtopping the proposed roadway, while minimizing flow from the lower frequency storm events. Exhibit N.1.0 – "Proposed Connections" illustrates the sizes and configuration of these culverts.

(d) <u>Reach 4- CONNECTION: "RD(Collector)CP4" (Post)</u>

Along the way this channel is intercepted by a proposed Collector Roadway with 2 CMP culverts that pass water under the road. This connection "**RD(Collector)CP4**" with proposed culverts **CP4A** and **CP4B** helps minimize the runoff leaving the site at Connection RD(PotterS)C4 while allowing water to pass under the proposed roadway. Exhibit D.1.1 shows the existing as well as the proposed hydraulic features onsite.

Culvert **CP4A** has a diameter of **4'** while **CP4B** is **4.5**'. All culverts modeled are of CMP material with headwalls at the inlets. Please see Exhibit N.1.0 for an illustration of this configuration.

(e) <u>Reach 4T- CONNECTION: "RD(Collector)CP4T" (Post)</u>

Before merging with Reach 4, Reach 4T is intercepted by a proposed collector roadway (Exhibit D.1.1). A connection named **"RD(Collector)CP4T"** was created and culverts **CP4TA** and **CP4TB** were added to the connection in HEC-RAS. This connection represents a potentially proposed roundabout through which water will be passed coming from Reach 4T.

Culvert **CP4TA** is **2'** in diameter while **CP4TB** has a diameter of **3.5'**. This geometrical configuration ensures that the 100 year runoff is conveyed without overtopping the proposed roadway, while minimizing flow from the lower frequency storm events. For a quantitative analysis of discharge values please see the "Results (Post-Project)" section below. Exhibit N.1.0 depicts a section view of the culvert configurations at this connection.

(f) <u>Reach 6- CONNECTION: "RD(Minor)CP6" (Post)</u>

Before exiting the site, Reach 6 is intercepted by a local residential roadway with proposed culvert crossings **CP6A, CP6B** and **CP5** about 600 ft away from the property line at Humbug Rd. This allows for conveyance under the roadway while ensuring that the 2,10 and 100 year runoff leaving the site after Post Development is lower than that during Pre Development. The connection through which these culverts go through was created in HEC-RAS and named "**RD(Minor)CP6."**

Two identical **12'x12'** CMP box culverts (**CP6A and CP6B)** are placed at the connection in Reach 6. Culvert **CP5**, which is a **circular 6'** culvert, is placed above Reach 5 in order to convey runoff from the 100 year storm event. This configuration can be seen on Exhibit N.1.0.

(g) ***Plugged Minor Culverts (POST)***

The Post-Developed model includes plugged culverts **CB3** and **CE**. It is assumed that these culverts will be cleaned prior to development. **CB3** and **CE** are both 1' (12") in diameter. More information can be obtained from Exhibit D.1.1 or from observing the HEC-RAS model.

3.06 Model Development (POST-PROJECT)

The Pre- Project model was utilized in creating the Post-Developed model. Discharge locations were determined based on the characteristics of the channels and developed basin boundaries. Exhibit F.1.1-"Post HEC-RAS Geometry and Inflow" shows the discharge locations and values used in the Proposed model, as well as the geometry used in HEC-RAS. A new surface was created in HEC-RAS consisting of the existing terrain and the surface of the proposed collector roadway. SA-2D connections were created along the roadway where culverts are to be proposed. Proposed culverts were placed at appropriate locations ensuring that the discharge leaving the existing site is not exceeded. Connections, with their corresponding culverts, were raised high enough to ensure that water doesn't spill over the road during the 100 year storm event.

Refer to the "Model Development (Pre-Project)" section for definitions and uses of model parameters.

(a) ***Adding and Importing Layers***

Land XML Surface

A Land XML surface was then exported out of Civil 3D representing the proposed "Collector Roadway" and imported into GeoHEC-RAS as a terrain layer. A new surface was created by merging the Existing (Pre) project surface with the Proposed (Post) project Roadway surface. The projected CRS region was defined as CAHP-IIF to align the surface with the geospatial base map layer.

Civil 3D Layers

Additional Civil 3D layers were added into the model for the purposes of defining the study parameters and implementing proposed improvements. AutoCAD drawings were added into the model that includes the developed basins as well as a proposed collector roadway.

Shapefiles

All existing shapefiles remained in the model and additional ones were added. The Topographic survey was used to export Culvert CL's for any plugged culverts, which were then imported into the model and used to model culverts. It is only reasonable to assume that any existing culverts that are plugged or buried will be cleaned prior to development.

(b) ***Defining Flow Areas and all Flow Elements (POST)***

Exhibit F.1.1 clearly illustrates the Proposed (Post) HEC-RAS geometric parameters used such as 2D Flow Area, BC lines (discharge locations), break lines and SA-2D connections (roads).

2D Flow Area

The Pre-Developed model 2D Flow Area was used and updated to include new Connections, Profile Lines and BC Inflow lines.

Boundary Condition Lines (Discharge Locations)

Boundary Condition (BC) Lines were drawn at areas of interest based on the Developed Basin Civil 3D drawing that was projected. BC lines were drawn along the river network at the shed area discharge locations where flow was introduced. BC lines were also drawn offsite at the downstream end as normal depth BC lines.

Break Lines

All break lines remained the same between the Pre-Developed and Post-Developed Models.

SA-2D Connections (Roadways)

Line work representing the Proposed Roadway Center Lines were drawn and assigned to be SA-2D Connections in HEC-RAS, defining the roadways through which culverts and pipes pass water through.

Culverts

Linework representing culvert CL's (Centerlines) were drawn at all roadway culvert crossing locations. Parameters on the type and size of culverts were then defined depending on how much water will need to be detained. Please refer to the HEC-RAS model for additional information.

(c) ***Unsteady Flow Data and Computational Options (POST)***

Unsteady Flow Data

After the geometry has been properly defined, we proceeded to input hydrograph flow data. For all storm intensities modeled, a 24-hr. storm with 15-minute intervals was used. All BC lines, except for the most downstream ones, were defined with flow hydrograph data. Normal depth was assigned to downstream BC lines with respective slopes defined. Exhibit F.1.1 shows the discharge locations used to input hydrograph data for a range of storm events.

Computational Options

Diffusion wave was used as the computational equation for the analysis with a subcritical flow regime. A 1-day simulation time window was used with a 1 second computational time step interval. The output intervals were set to 10 minutes. For additional information please refer to the HEC-RAS model.

3.07 Results (POST-PROJECT)

Time series results were produced after the analysis was performed. The peak values from the 2yr, 10yr, and 100yr storm events were analyzed and used to determine the Post Project floodway extents. Emphasis was placed on analyzing the magnitude of discharge leaving the site, ensuring that it doesn't exceed the Pre-Developed project values. The velocity, depth, HGL (Hydraulic Grade Line) and discharge values along culverts, roads and along entire channel lengths were analyzed as well.

Weir overflow still exists at existing connections but not at proposed connections. Discharge values may change during final design to more closely reflect the Pre-Developed conditions. The discharge leaving the site however, will always be lower in the Post-Developed conditions than in the Pre-Developed conditions, due to retention requirements. Discharge values may deviate from original values when performing a new analysis due to HEC-RAS' computational method.

Refer to Exhibit D.1.1 for an illustration of the proposed improvements. A section view of the proposed culverts is found on Exhibit N.1.0. Please see Exhibit E.1.1 – "Floodway Section Analysis (Post)" for a representation of floodway extents as well as the depth, velocity and water surface elevation (HGL) at cross-sections along the reaches.

(a) ***<u>Discharge at Proposed Culverts and Roads (POST)</u>***

Culvert crossings are proposed at connections where roadways will be required. All culverts were modeled to be made of CMP material with headwalls at the inlets. Culvert types, sizes and configurations may be subject to change during the final design phase. All final designs will meet City of Chico and Butte County standard requirements.

Culverts and roadways were analyzed by grouping them into SA-2D connections in HEC-RAS that contain the extents of the entire floodway in that region. Table 3 below depicts the discharge values going through all proposed roadways (Connections) for the 2, 10 and 100 year storm events.

All culverts are able to convey runoff for the 2yr, 10yr and 100yr storm events without overtopping the road. Table 3 below displays a value of zero across all columns for the weir flow, indicating that there is no occurrence of weir flow.

RD(Collector)CP2

It was observed that a total of **457 cfs, 729 cfs** and **1243 cfs** goes through this proposed connection during the 2, 10 and 100 year storm events respectively. Most of the runoff is conveyed exclusively through culvert **CP2A** during the 2 year storm event. With the rise of

Water Surface Elevation (HGL) during lower frequency storms, culverts **CP2B** and **CP2C** begin conveying a lot more flow, since they are placed above culvert **CP2A**.

RD(Collector)CP3

Connection **RD(Collector)CP3** experiences runoff from developed onsite basin C2 (D-OS-Basin-C2) which is conveyed through culverts **CP3A** and **CP3B.** During a 2 year storm the discharge is carried through culvert **CP3A** while runoff from lower frequency storms is carried mostly by culvert **CP3B**.

RD(Collector)CP4

A total of **151 cfs**, **227 cfs** and **362 cfs** is conveyed through connection **RD(Collector)CP4** during a 2yr, 10yr and 100yr storm events respectively. Culvert **CP4A** carries the majority of the inflow during a 2 year storm. During a 100 year storm however, culverts **CP4A** and **CP4B** share similar conveyance.

RD(Collector)CP4T

Connection **RD(Collector)CP4T** experiences a total flow of **44.4 cfs**, **75.7 cfs** and **123.2 cfs** for the 2yr, 10yr and 100 yr storm events respectively. During a 2 year storm event culverts **CP4TA** and **CP4TB** convey the same flow. For lower frequency storms however, culvert **CP4TB** carries most of the runoff.

RD(Minor)CP6

Out of all channels Reach 6 experiences the most flow. A flow of **4286.4 cfs** passes through connection **RD(Minor)CP6** during a 100 year storm event. Culverts **CP6A** and **CP6B** carry the majority of the runoff. As the storm intensity increases and the water surface elevation rises, culvert **CP5** begins conveying the excess flow.

DISCHARGE AT PROPOSED CONNECTIONS (HECRAS)									
2 Year Storm (cfs)									
R2		**R3**		**R4**		**R4T**		**R6**	
RD(Collector)CP2		**RD(Collector)CP3**		**RD(Collector)CP4**		**RD(Collector)CP4T**		**RD(Minor)CP6**	
Qtot	457.1	Qtot	26.3	Qtot	150.9	Qtot	44.4	Qtot	1380.5
CP2A	440	CP3A	26.2	CP4A	139	CP4TA	22	CP6A,B	1355.7
CP2B	17.2	CP3B	0.1	CP4B	11.9	CP4TB	22.4	CP5	24.7
CP2C	0	Weir Flow	0	Weir Flow	0	Weir Flow	0	Weir Flow	0
Weir Flow	0								
10 Year Storm (cfs)									
R2		**R3**		**R4**		**R4T**		**R6**	
RD(Collector)CP2		**RD(Collector)CP3**		**RD(Collector)CP4**		**RD(Collector)CP4T**		**RD(Minor)CP6**	
Qtot	728.8	Qtot	39.6	Qtot	227.4	Qtot	75.7	Qtot	2301.6
CP2A	619	CP3A	2.5	CP4A	161.1	CP4TA	23	CP6A,B	2169.9
CP2B	90.4	CP3B	37.1	CP4B	66.3	CP4TB	52.7	CP5	131.7
CP2C	19.5	Weir Flow	0	Weir Flow	0	Weir Flow	0	Weir Flow	0
Weir Flow	0								
100 Year Storm (cfs)									
R2		**R3**		**R4**		**R4T**		**R6**	
RD(Collector)CP2		**RD(Collector)CP3**		**RD(Collector)CP4**		**RD(Collector)CP4T**		**RD(Minor)CP6**	
Qtot	1243.2	Qtot	68.3	Qtot	362.4	Qtot	123.2	Qtot	4286.4
CP2A	820.6	CP3A	12.2	CP4A	190.1	CP4TA	24.7	CP6A,B	3915
CP2B	297.1	CP3B	56	CP4B	172.4	CP4TB	98.5	CP5	371.4
CP2C	125.5	Weir Flow	0	Weir Flow	0	Weir Flow	0	Weir Flow	0
Weir Flow	0								

Table 4: Post-Developed Discharge at Proposed Connections (HEC-RAS)

(b) ***Discharge at Existing Culverts and Roads (PRE VS. POST)***

Discharge values exiting the site were examined for both the Pre-Developed and Post-Developed scenarios by comparing the Pre and Post stage and flow Hydrographs for the SA-2D Connections. Culverts were designed such that upon development (Post-Developed), runoff exiting the site will be lower than the Pre-Developed scenario for the 2, 10 and 100 year storm events. Discharge values may change during final design to more closely reflect the Pre-Developed conditions.

Discharge values may deviate from original values when performing a new analysis due the HEC-RAS' computational method.

Tables 5.1-5.3 below show the magnitude of "Pre" and "Post" discharge at all existing roads (SA-2D Connections) that was computed through HEC-RAS. Connections are categorized by the Reaches and Culverts associated with them. Please observe the HEC-RAS model for discharge values along all Minor connections.

Exhibit L.1.1– "Flow Hydrographs at Existing Connections (HEC-RAS)" and Exhibit L.1.2- "Post Flow Hydrographs at Existing Connections (HEC-RAS)" depict the hydrographs representing discharge at connections for the Pre-Developed and Post-Developed scenarios respectively. For Post-Developed hydrographs at proposed connections refer to Exhibit L.2.1. A 3-Dimensional Video of the floodway is available for the 100-year scenario and can be found in the folder titled "Videos-Pics," which is located under Exhibits. For any additional information refer to the HEC-RAS model.

RD(Dawncrest)C1A,C1B (Belvedere Subdivision)

Dawncrest Dr is situated directly west from site in the adjacent Belvedere subdivision. It was observed that the road no longer overtops during a 100 year storm event, due to the diversion of excess flow coming towards the subdivision. This connection experiences a Post-Developed flow of **241.7 cfs** for the 100 year storm. This is a decrease of **64.4 cfs** when compared to the Pre-Developed flow. This decrease in flow allows for full conveyance through pipes **C1A** and **C1B** without overtopping the road.

RD(PotterN)C1-C3

Compared to Pre-Developed conditions, **RD(PotterN)C1-C3** experiences a decrease of **6.7 cfs**, **97 cfs** and **424 cfs** in the Post-Developed scenario during a 2yr, 10yr and 100yr storm events respectively. At Post-Development, a volumetric flow rate of **1624.2 cfs** goes through this connection during a 100 year storm event. The decrease of flows decreased the amount of weir flow overtopping the road.

RD(PotterS)C4

All Post-Developed flows entering this connection are lower than the Pre-Developed flows. A decrease of **7.4 cfs**, **4.1 cfs** and **170.0 cfs** has been observed during a 2yr, 10yr and 100yr respectively. A total discharge of **652.3 cfs** is passed through this connection during a 100 year storm event. The weir overflow that occurs has been lowered for all storm events.

RD(Humbug)C5,C6

RD(Humbug)C5,C6 experiences a Post-Developed flow of **4354.9 cfs** for the 100 year storm event. This is a decrease of **586.3 cfs** when compared to the Pre-Developed conditions. Weir overflow has been reduced by **557.9 cfs** during a 100 year storm event and by **18.7 cfs** during a 10 year event.

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
2 Year Storm (cfs)											
R1			**R1+R2+R3**			**R4+R4T**			**R5+R6**		
RD(Dawncrest)C1A,C1B			**RD(PotterN)C1-C3**			**RD(PotterS)C4**			**RD(Humbug)C5,C6**		
	PRE (cfs)	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**
Qtot =	89.4	89	Qtot =	593.3	586.6	Qtot =	276.6	269.2	Qtot =	1440.2	1411.4
C1A =	49.5	48.2	C1E =	69.1	67.7	C4A =	96.4	95.9	C5A,B =	161.9	160
C1B =	40	40.7	C2A,B =	197	196.2	C4B =	68.2	68	C6A,B,C =	290.9	288.7
Weir Flow =	0	0	C3A =	0	4	Weir Flow =	111.9	105.4	C6D =	166.3	166.1
			Weir Flow =	323.4	319.1				Weir Flow =	821.2	796.7

Table 5.1: Pre vs. Post Discharge at Existing Connections for 2 year (HEC-RAS)

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
10 Year Storm (cfs)											
R1			**R1+R2+R3**			**R4+R4T**			**R5+R6**		
RD(Dawncrest)C1A,C1B			**RD(PotterN)C1-C3**			**RD(PotterS)C4**			**RD(Humbug)C5,C6**		
	PRE (cfs)	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**
Qtot =	153.1	135.5	Qtot =	1027.5	930.5	Qtot =	392.2	388.1	Qtot =	2360.5	2338.6
C1A =	88.5	77.1	C1E =	94.6	86.2	C4A =	102.9	102.7	C5A,B =	165.5	164.7
C1B =	64.6	58.4	C2A,B =	221.1	215.9	C4B =	71.6	71.5	C6A,B,C =	324	321.5
Weir Flow =	0	0	C3A =	0	4.1	Weir Flow =	217.7	213.9	C6D =	202.7	203.3
			Weir Flow =	707.7	625.4				Weir Flow =	1668.4	1649.7

Table 5.2: Pre vs. Post Discharge at Existing Connections for 10 year (HEC-RAS)

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
100 Year Storm (cfs)											
R1			**R1+R2+R3**			**R4+R4T**			**R5+R6**		
RD(Dawncrest)C1A,C1B			**RD(PotterN)C1-C3**			**RD(PotterS)C4**			**RD(Humbug)C5,C6**		
	PRE (cfs)	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**
Qtot =	306.1	241.7	Qtot =	2048.2	1624.2	Qtot =	822.3	652.3	Qtot =	4941.2	4354.9
C1A =	170.1	144.3	C1E =	139.3	121.1	C4A =	117	112.4	C5A,B =	174.5	173.3
C1B =	111.4	97.4	C2A,B =	260.5	245.8	C4B =	79.2	76.7	C6A,B,C =	375.2	363.6
Weir Flow =	24.6	0	C3A =	0	4.2	Weir Flow =	626.1	463.2	C6D =	275.4	262.5
			Weir Flow =	1644.1	1253.1				Weir Flow =	4113.3	3555.4

Table 5.3: Pre vs. Post Discharge at Existing Connections for 100 year (HEC-RAS)

(c) ***Discharge Comparison (HEC-RAS VS. SSA) (POST)***

These discharge values were compared with the cumulative basin outflow values obtained from SSA (Storm and Sanitary Analysis). Most of the discharge values obtained from HEC-RAS are relatively close to those obtained from SSA, except for the post-developed flows. Tables 6 and 7 below depict the cumulative basin outflow (Q) values at road Connections obtained from SSA. Runoff values from Table 4: "Post-Developed Discharge at Proposed Connections (HEC-RAS)" can be compared to Table 6: "Post-Developed Discharge at Proposed Connections (SSA)." Similarly, Tables 5.1-5.3 titled "Pre vs. Post Discharge at Existing Connections" can be compared to "Table 7: Pre vs. Post Discharge at Existing Connections (SSA)."

DISCHARGE AT PROPOSED CONNECTIONS (SSA)									
2 Year Storm (cfs)									
R2		**R3**		**R4**		**R4T**		**R6**	
RD(Collector)CP2		**RD(Collector)CP3**		**RD(Collector)CP4**		**RD(Collector)CP4T**		**RD(Minor)CP6**	
Qtot	663.3	Qtot	30.5	Qtot	237.2	Qtot	107.1	Qtot	1687.2
10 Year Storm (cfs)									
R2		**R3**		**R4**		**R4T**		**R6**	
RD(Collector)CP2		**RD(Collector)CP3**		**RD(Collector)CP4**		**RD(Collector)CP4T**		**RD(Minor)CP6**	
Qtot	1075.9	Qtot	46.1	Qtot	365.8	Qtot	170.3	Qtot	2831.2
100 Year Storm (cfs)									
R2		**R3**		**R4**		**R4T**		**R6**	
RD(Collector)CP2		**RD(Collector)CP3**		**RD(Collector)CP4**		**RD(Collector)CP4T**		**RD(Minor)CP6**	
Qtot	2034.6	Qtot	81.2	Qtot	654.6	Qtot	313.7	Qtot	5565.3

Table 6: Post-Developed Discharge at Proposed Connections (SSA)

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (SSA)											
2 Year Storm (cfs)											
R1			**R1+R2+R3**			**R4+R4T**			**R5+R6**		
RD(Dawncrest)C1A,C1B			**RD(PotterN)C1-C3**			**RD(PotterS)C4**			**RD(Humbug)C5,C6**		
	PRE (cfs)	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**
Qtot =	94.1	101.5	Qtot =	703.4	791.4	Qtot =	289.4	448.4	Qtot =	1602.8	1756.6
10 Year Storm (cfs)											
R1			**R1+R2+R3**			**R4+R4T**			**R5+R6**		
RD(Dawncrest)C1A,C1B			**RD(PotterN)C1-C3**			**RD(PotterS)C4**			**RD(Humbug)C5,C6**		
	PRE (cfs)	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**
Qtot =	159.2	156.2	Qtot =	1187.2	1280	Qtot =	403.6	688.3	Qtot =	2721.8	2943.5
100 Year Storm (cfs)											
R1			**R1+R2+R3**			**R4+R4T**			**R5+R6**		
RD(Dawncrest)C1A,C1B			**RD(PotterN)C1-C3**			**RD(PotterS)C4**			**RD(Humbug)C5,C6**		
	PRE (cfs)	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**		**PRE (cfs)**	**POST (cfs)**
Qtot =	312.5	279.2	Qtot =	2333.3	2413.2	Qtot =	849.4	1228.6	Qtot =	5406.9	5776

Table 7: Pre vs. Post Discharge at Existing Connections (SSA)

(d) ***Floodway and Channel Analysis (POST)***

Floodways and channels were analyzed by station numbers along the alignment of each reach (Exhibit E.1.1) as well as at road Connections. The same Reaches and Profile lines were used in the Post-Developed model as were used in the Pre-Developed one. Reach 5 has been disregarded because the floodway along Reach 6 extends into Reach 5. Additional profile lines were added to the model just upstream of proposed Roadway Connections.

Floodway extents are subject to change, especially directly upstream of proposed connections. Detention basin sizes in the design phase will depict the extents of the floodway as well as the maximum depth of water upstream of all proposed connections. The flood extents will also be affected by the roadway profile as well as the culvert types and sizes used in final design. Flood extents shown in this report are based only on the proposed roadway and its associated culverts used in the Post-Developed HEC-RAS model.

Exhibit E.1.1 shows the stations, profile cuts and detailed hydraulic information at each station, including Water Surface Elevation (HGL), Velocity and Depth. For a schematic of depth and velocity values along the floodplain in the Post-Developed scenario see Exhibits G.1.2- "Post Floodway Depth (HEC-RAS)" and G.2.2- "Post Floodway Velocity (HEC-RAS)."

Reach 1

Reach 1 forms by onsite channelization and moves in the general East to West direction until going through connection "RD(Dawncrest)C1A,C1B." The new developed basins minimized the discharge at this connection (Belvedere Subdivision), thus allowing for full conveyance of water through PVC pipes C1A and C1B during all storm events.

There was a decrease of **0.4 cfs**, **17.6 cfs** and **64.4 cfs** between the Pre and Post scenarios for the 2 year, 10 year and 100 year storm events respectively.

Maximum peak values such as the Hydraulic Grade Line (HGL), Velocity and Depth were obtained at various station locations. Exhibit E.1.1 clearly depicts these values for the 2, 10- and 100-year storm events.

Reach 2

Channel R2 enters the site from the East and moves in the general East to West direction until it intercepts a proposed Collector roadway having multiple CMP culverts. This connection causes water to back up directly upstream of the roadway while also limiting the amount of outflow. The maximum depth of water directly upstream of the roadway (station 58+09) is **9.4 ft**, **11.8 ft** and **16.1 ft** for the 2yr, 10yr and 100yr storm events. These depth values are subject to change depending the size of the detention basin that will be designed.

The Post-Developed HGL values directly upstream of Potter Rd along R2 (station 87+83) are **271.1 ft** and **271.5 ft** for the 10 and 100 year storm events. When compared to the Pre-Developed scenario these values are lower by **0 ft** and **0.3 ft** for the 10 and 100 year storm events.

There is no weir overflow that occurs at the proposed connections, even during the 100 year storm event. The HGL directly upstream of the proposed roadway (station 58+09) is **324.4 ft** for the 100 year storm event. The existing connection at Potter Rd however, still experiences weir overflow. The maximum HGL, depth and velocity values at various section profiles can be seen on Exhibit E.1.1.

Reach 4

Reach 4 forms by onsite channelization and moves in the general East to West direction until it is intercepted by a proposed Collector roadway (RDCollectorCP4). Water backs up during all storm events and is conveyed through multiple CMP culverts (C4A, C4B) until exiting the site on the south side of Potter Rd. Although no overflow occurs at the proposed connection, the existing connection however still experiences weir overflow.

Maximum peak values such as the Hydraulic Grade Line (HGL), Velocity and Depth were obtained at various station locations. The Maximum HGL, Depth and Velocity values at various section profiles can be seen on Exhibit E.1.1. The maximum depth values at station 30+80, which is directly upstream of the proposed connection "RD(Collector)CP4," are **9.0 ft**, **11.3 ft** and **14.9 ft** for the range of storm events in increasing order.

Reach 4T

Reach 4 forms by onsite channelization and moves in the general East to West direction until it is intercepted by a proposed Collector roadway (RDCollectorCP4T). Upon conveyance of runoff through the proposed CMP culverts, the channel merges into Reach 4 and exits the site.

The depth directly upstream of the connection is 10.8 ft for the 100 year storm event while the HGL is 284.8 ft.

Reach 6

Only Reach 6 enters the site from the East side and moves in the general Southwest direction and is intercepted by a proposed local residential roadway. The water backs up directly upstream of this connection and is conveyed through the proposed CMP culverts (CP6A, CP6B, CP5). The high runoff that is experienced forms a floodway along R6 to which R5 contributes as well.

Maximum peak values such as the Hydraulic Grade Line (HGL), Velocity and Depth were obtained at various station locations. The Maximum HGL, Depth and Velocity values at various section profiles can be seen on Exhibit E.1.1.

The HGL values directly upstream of proposed connection "RD(Minor)CP6" are **269.5 ft**, **272.5 ft** and **278.0 ft** for the 2yr, 10yr and 100yr storm events respectively.

Floodway Analysis Conclusion

Although the post developed Q values have decreased, water still overtops most major roads during the 100-year storm event. This backwater effect causes backwater flooding directly upstream of these roadways. Exhibit E.1.1 illustrates all Post Developed floodways and shows overtopping at each location. Post Developed Hydrographs generated by HEC-RAS can be seen on Exhibits L.1.2 and L.2.1 for existing and proposed connections respectively. A HEC-RAS generated report for the Pre and Post developed scenarios can be seen on Exhibits O.1.1 through O.2.3 for a range of storm events. Videos of the modeled floodway were recorded for the 100-year storm scenario and are located under the folder titled "Videos-Pics."

IV. Large Shed Area Water Quality

4.01 Stormwater Management During Construction Activities

The proposed development is subject to construction site storm water runoff control requirements. A Storm Water Pollution Prevention Plan (SWPPP) will be prepared and substituted for the ESCP. The plan will include Erosion and Sediment Control drawings/plans or any additional exhibits that may be required. The SWPPP will contain appropriate site-specific construction Best Management Practices (BMPs) that meet the minimum requirements to control storm water pollution due to construction activities.

4.02 Post Construction Stormwater Management

A Post Construction Storm Water Management Plan will be initiated to help reduce project site runoff. The developed portions of the project site will be divided into separate Drainage Managed Areas (DMAs), each implementing their own site design measures, source controls, storm water treatment and baseline hydromodification measures as defined in Section 15.50.080 in the City of Chico Code of Ordinances, in order to reduce project site runoff. Site design measures will utilize Low Impact Development (LID) standards to manage stormwater as close to its source as possible. The regulated "Valley's Edge" development will conform to the City of Chico LID and Hydromodification requirements.

(a) ***Hydromodification***

The proposed LID measures will assist in the reduction of the potential for hydrograph modification, also known as "Hydromodification." Hydromodification is the alteration of the natural flow of water through a landscape. The conversion of open space to features such as roads, buildings and parking lots adds impervious surfaces and modifies runoff patterns, causing rainfall to run off into streams more quickly with a higher discharge. Post-Developed runoff for this hydromodification management project will not exceed the estimated Pre-Developed flow rate for the 2-year, 10-year and 100-year 24-hour storm events.

(b) ***Low Impact Development (LID)***

The project will employ the use of Low Impact Development (LID) measures, also referred to as Green Infrastructure (GI). LID refers to the design approach that uses natural onsite features to protect water quality and habitat. LID design work aims to maintain the natural drainage patterns used which may reduce or maintain existing peak flows upon mitigation while improving water quality. Both on-site LID along with "Best Management Practices" (BMPs) will be used to mitigate and treat the discharge into creeks. Some BMP features that will be used include the incorporation of significant open space in the development plan. This includes implementation of neighborhood parks, Creekside greenways and linear parks, resulting in roughly 900 acres of open space, which encompasses 63% of the total area. The following is a list of additional LID features that may be implemented into the project DMAs.

- Tree planting and preservation
- Porous Pavement
- Green Roofs
- Rain Barrels and Cisterns
- Impervious Area Disconnections
- Rain Gardens
- Infiltration Trenches
- Bioretention
- Retention or Detention Basins

4.03 Steps to Implement Proper LIDs

(a) ***Site Assessment***

Firstly, a site assessment per Section 15.50.080(D)(1) will be performed to evaluate the site and determine the placement of buildings such that it conforms to the sites' natural drainage patterns.

(b) ***Drainage Managed Areas (DMAs)***

Secondly, the developed portions of the project site will be divided into discrete Drainage Managed Areas (DMAs) in order to manage runoff from each DMA using site design measures, source controls and/or storm water treatment and baseline hydromodification measures.

(c) ***Site Design Measures***

Next, site design measures as defined in Section 15.50.080(A) will be implemented in order to achieve infiltration, evapotranspiration and/or harvesting/reuse of the 85th percentile 24-hour storm runoff event. State Water Board SMARTS post-construction calculator will be used to quantify and submit to the City of Chico the runoff reduction resulting from implementation of site design measures. Any remaining runoff from impervious DMAs will be directed to one or more bioretention facilities as specified in Section 15.50.080(D)(6). Table 7 below indicates site design and treatment control measures that may be incorporated into the design.

(d) ***Source Control Measures***

Source Control Measures will be implemented for DMAs with pollutant-generating activities per Section 15.50.080(C). The measures will be designed consistent with recommendations from the CASQA Storm Water BMP Handbook for new development.

(e) ***Storm Water Treatment and Hydromodification***

Finally, Storm Water Treatment and Baseline Hydromodification Management Measures will be implemented. Any remaining runoff from impervious DMAs will be directed to one or more facilities designed to evapotranspire, infiltrate and/or bioretain the amount of runoff specified in Section 15.50.080(D)(3). A Volumetric BMP Sizing Tool will be used in order to determine the volume or flow required for treatment such that we meet the required hydraulic sizing design criteria per Section 15.50.080(D)(3). Either Volumetric or Flow based criteria will be met depending on each individual DMA. Table 7 below indicates site design and treatment control measures that may be incorporated into the design.

Site Design or Treatment Control Measure	Description	CASQA Specification	Sizing Criteria
Stream setbacks and vegetated buffers *(Site Design Measure)*	*Preservation of a green strip or vegetated buffer between the development and the discharge point through which storm water runoff passes.*	TC-10	Flow
Soil quality improvement *(Site Design Measure)*	*Commonly used in conjunction with landscaping, bioretention, or storm water gardens. Also known as "engineered soils", through which storm water can infiltrate. This provides additional on-site storage and reduces peak flow rates.*	TC-40	Volume
Tree planting and preservation *(Site Design Measure)*	*Incorporated into the site's landscaping. Trees reduce the energy of falling rain and help to reduce peak flow rates.*	SD-10	SMARTS Calculator
Porous pavement *(Site Design Measure)*	*Porous asphalt, concrete, or pavers; cobbles or rock covered surfaces; typically with at least 18" of drainage rock below the porous surface covering to store and infiltrate storm water.*	SD-20	Volume
Green roofs *(Site Design Measure)*	*Plants and growing media permanently installed on a rooftop to allow a certain amount of storm water infiltration and storage.*	TC-40	Volume
Vegetated swales *(Site Design Measure)*	*Storm water conveyance swales that are vegetated to stabilize the swale and prevent erosion. Vegetated swales improve water quality by providing filtration and bio- uptake of pollutants and by promoting sedimentation of suspended particles. Often, vegetative swales are used in conjunction with "soil quality improvement" to provide greater infiltration and / or with retention or detention basins.*	TC-30	Flow
Rain harvesting and reuse *(Site Design Measure)*	*Large scale or small scale capture, collection and re-use of storm water runoff. Includes rain barrels used at downspouts and large cisterns and collection systems.*	TC-12	Volume
Bioretention and rain gardens *(Treatment Control Measure)*	*Depressed landscaped areas to which storm water runoff flows. These rain gardens are designed with engineered soils so that they facilitate infiltration and storage of storm water.*	TC-32	Volume
Infiltration trench, Flow-through Planter, or Tree Wells *(Treatment Control Measure)*	*Similar in concept to a French drain or a leach field, in which storm water runoff is able to drain to a trench or pit that has been filled with rock. It provides underground storage of the water until it can infiltrate into the soils.*	TC-10	Volume and Flow
Retention and detention basins *(Treatment Control Measure)*	*Aboveground storage of storm water runoff in a basin that allows it to infiltrate into soils and / or be stored and released at a slower flow rate. Impounded water must be infiltrated or discharged within 72 hours to avoid vector breeding problems.*	TC-11 TC-12 TC-22 TC-40	Volume

Table 8: Site Design and Treatment Control Measures

V. Conclusions

5.01 Hydrologic Analysis

The Hydrologic Analysis was performed by utilizing Autodesk-Storm and Sanitary Analysis (SSA) software for both the Pre-Developed and Post-Developed scenarios. The storm intensity events modeled include the 2 year, 10 year, 25 year, 50 year and 100 year events, although only the 2yr, 10yr and 100yr events were utilized in this report.

(a) ***Pre-Developed***

The development of the Pre-Developed SSA model began by identifying existing onsite conditions. The delineation of shed areas onsite as well as those contributing from offsite was performed. The base model consists of seven (7) distinct major watershed basins onsite and six (6) distinct major watershed basins offsite. There is a total of forty-three (43) interior sub-basin areas in whole with thirty-seven (37) of these sub-basins lying within the proposed development boundary.

Results were obtained after running the analysis for the Pre-Developed scenario. Hydrographs and Hyetographs were developed utilizing the SCS TR-55 Method for each sub basin. The runoff data was sampled for both a 1-hour and a 15-min interval for the 2yr, 10yr and 100yr storm events, with the latter being utilized in HEC-RAS. Exhibit K.1.1 depicts sub-basin hydrograph data for a range of storm events. For more details please see the Hydrological Analysis section or the SSA model.

(b) ***Post-Developed***

The development of the Post-Developed SSA model began by implementing the Valley's Edge Specific Plan (VESP) into the delineation of the new proposed basins. Basins were developed by utilizing the details of the VESP, such as proposed roadways and areas to be developed, in conjunction with the Pre-Developed terrain characteristics, so as to not alter the sites natural drainage patterns.

Results were obtained after running the analysis for the Post-Developed scenario. Hydrographs and Hyetographs were developed with 15-min data sample intervals for a 24-hr storm duration. Results were obtained for a 2yr, 10yr and 100yr storm events. Excel files containing sub-basin hydrograph data for a range of storm events are located in folder Exhibit K. For more details please see the Hydrological Analysis section or the SSA model.

5.02 Hydraulic Analysis

The Hydraulic Analysis was performed by creating a 2D-Unsteady State HEC-RAS model using GeoHEC-RAS software and performing an analysis run for both the Pre-Developed and Post-Developed conditions. The Pre-Developed scenario represents the hydraulic parameters such as floodways present onsite in the existing state. The Post-Developed scenario represents the hydraulic parameters after development. A range of storm events including the 2 year, 10 year and 100 year events were analyzed.

Emphasis was placed on evaluating parameters such as velocity, depth, discharge and water surface elevation also referred to as Hydraulic Grade Line (HGL).

Post-Developed conditions were based on proposed improvements required to satisfy the planned unit development.

(a) ***<u>Pre-Developed Model</u>***

The development of the 2D model began by identifying the existing onsite conditions. Emphasis was placed on analyzing 6 major onsite channels (Reach1-Reach6). All onsite culverts going through roadways were modeled using SA-2D connections in HEC-RAS. Hydrograph data was then input at specific inflow locations governed by the delineated basins.

Upon running the Pre-Developed HEC-RAS model the results were analyzed. It was concluded that the existing downstream roadways adjacent to our site overtop during all storm events, except for Dawncrest Dr located in the Belvedere Subdivision. Dawncrest Dr overtops only during a 100 year storm event. The Pre-Developed floodway was analyzed by the reaches associated with it, as well as SA-2D connections along those reaches. All results obtained can be found under the Hydraulic Analysis section of this report.

(b) ***<u>Post-Developed Model</u>***

The Post-Developed model began by utilizing the Pre-Developed model and making Hydraulic/Hydrological changes. A surface of a proposed collector roadway was merged with the existing surface. Exhibit D.1.1 depicts this roadway along with proposed and existing hydraulic features. SA-2D connections were defined along the roadway where it crosses over existing channels (Reaches). Culverts going through connections were defined and made to be of CMP material with headwalls. The surface of the collector roadway crossing over the channels was raised high enough so that it doesn't overtop during a 100 year storm event.

After running the Post-Developed model, results were analyzed to ensure that the Post-Developed flows leaving the site do not exceed the Pre-Developed flows. More information is found under the Hydraulic Analysis section of the report along with accompanying Exhibit E.1.1.

Please note that culvert types and sizes are bound to change during the final design phase. Factors such as, type and configuration of basins required for the specific detention needs, will dictate the types of culverts to be used. Culvert inverts and their configurations may change depending on the earthwork cut volumes needed for detention needs. Profiles of the proposed roadways are subject to change as well. Roadways will be design to ensure that they do not overtop during a 100 year storm event.

5.03 Stormwater Quality

(a) ***Stormwater Management During Construction Activities***

A Storm Water Pollution Prevention Plan (SWPPP) will be prepared to satisfy construction site storm water runoff control requirements. The plan will include Erosion and Sediment Control drawings/plans meeting the requirements to control storm water pollution due to construction activities.

(b) ***Post Construction Stormwater Management***

A post Construction Storm Water Management Plan will be initiated to help reduce project site runoff. The developed portions of the project site will be divided into separate DMA's, each implementing their own site design measures, source controls, storm water treatment and baseline hydromodification measures in order to reduce runoff. The regulated "Valley's Edge" development will conform to the City of Chico LID and Hydromodification requirements.

Hydromodification

Post-Developed runoff for this hydromodification management project will not exceed the estimated Pre-Developed flow rate for the 2-year, 10-year and 100-year 24-hour storm events.

Hydromodification

The project will employ the use of Low Impact Development (LID) measures where appropriate in order to mitigate and treat discharge into creeks while maintaining natural drainage patterns. The LID features that may be implemented include retention/detention basins, bioretention, infiltration trenches, porous pavement, tree preservation, impervious area disconnections and more.

(c) ***Steps to Implement Proper LIDs***

Firstly, a site assessment will be performed to determine placement of buildings such that the site's natural drainage patterns are maintained.

Secondly, the developed portions will be divided into discrete DMA's in order to manage runoff from each DMA. Site design measures, source controls and storm water treatment measures will be performed for each DMA.

Next, site design measures will be implemented in order to achieve infiltration, evapotranspiration and/or harvesting/reuse of the 85th percentile 24-hour storm runoff event. Any remaining runoff will be directed to one or more bioretention facilities.

Source Control Measures, consistent with recommendations from the CASQA Storm Water BMP Handbook for New Development, will be implement for DMAs with pollutant-generating activities.

Finally, Storm Water Treatment and Baseline Hydromodification Management Measures will be implemented. Remaining runoff from impervious DMAs will be determined using a Volumetric BMP Sizing Tool and directed towards facilities that evapotranspire, infiltrate and/or bioretain.

June 8, 2021

SUBJECT: Valley's Edge ~ Reach 1 Phasing Question

TO: Mike Sawley, AICP
mike.sawley@chicoca.gov
Principal Planner (Environmental Program Manager)
City of Chico Community Development Department

Dear Mike,

We are providing this correspondence to address the phasing question in Reach 1 and how the future drainage may be handled to avoid any potential impact on the existing Belvedere subdivision. As the Valley's Edge planning area is built out, construction will progress in phases and the objective is that no negative drainage impacts will occur downstream due to the phasing or at the time of completion of the planning area. Hopefully, the detailed explanation below provides some insight to this matter.

-Reach 1-Undeveloped

- As you are aware Reach 1 (as shown in our Drainage Report) is the unnamed drainage that drains through our site from east to west towards the Belvedere subdivision and Dawncrest Drive. In its current state the improvements completed with the Belvedere Subdivision are sufficient to convey the 2-year and 10-year storm events. However, as shown in our drainage report, the undeveloped condition of our planning area overtops Dawncrest Drive for the 100-year storm event, which is not an acceptable overland release path. The city recently installed drainage pipe that will handle most of the flow, but not the entire amount. The problem still exists in a clogged pipe condition, without the build out of the VESP plan area.

-Reach 1-During Construction and Construction Phasing

- Upon completion of our planning area, stormwater produced by the 100-year event will be diverted south to Reach 2 to relieve the existing storm drain system and prevent overtopping. This diversion also reduces runoff generated by the 100-year event by 64.4 cfs once the planning area is fully developed as mentioned in our Drainage Report dated 4-29-2020.

- Currently it is anticipated that the first phase of construction will occur at the eastern terminus of 20th and to South. This first phase will not impact the drainage system. However, as progress to the North and East of 20th street is built out, we will have to deal with that drainage shed and implement a phasing plan approach. This phase could potentially affect Reach 1, including the installation of a portion of the proposed storm drain system.

- It is unlikely that the initial phases will include permanent improvements necessary to divert the entire flow to prevent overtopping of Dawncrest Drive in case of a clogged pipe condition. Temporary measures will be implemented to divert and detain stormwater that is not collected by the subsequent phases of the storm drain improvements.

- On a planning area of this size temporary facilities will need to be installed to support construction. These facilities will include "semi-temporary" access roads, detention basins, sedimentation basins, construction entrances, etc. The planning area will be using these facilities to achieve the necessary diversion and detention until the planning area is fully constructed.

- It is anticipated that a substantial construction access road might be constructed close to the alignment of the main north-south collector as shown on Page 5-3 of the Specific Plan, as deemed necessary to facilitate construction as construction progresses to the south. During the initial phases of construction, it is the intention to essentially mimic the flow characteristics of the fully built out planning area. By building temporary swales, berms, and other methods we intend to divert stormwater to the south towards the main collector/access road where treatment will be provided to mitigate any increases of flow. Construction of the access road, in conjunction with detention and sedimentation basins, and the planning area SWPPP will provide the necessary measures to ensure that stormwater discharge will meet city and state requirements.

- A Storm Drainage study will be provided with the first submittal of the improvement plans that impact this shed and may contribute to additional runoff. The plan will detail the methods used to prevent overtopping of Dawncrest Drive for the 100-year event.

- During the final design of the improvements impacting this shed, we anticipate a full drainage design will be addressed in the Strom Drainage report. We will also detail the methods we intend to deploy to prevent erosion as well as ensure that Little Chico Creek is not impacted by other pollutants associated with construction.

-Reach 1-Developed Condition

- As mentioned above, the undeveloped condition overtops Dawncrest Drive. Upon completion of our planning area, stormwater produced by the 100-year event will be diverted southerly to Reach 2 to relieve the existing storm drain system and prevent overtopping. This diversion also reduces runoff generated by the 100-year event by about 64 cfs once the planning area is fully developed. In addition to improving the existing condition, our planning area will comply with all City, State, and Federal laws and acquire all permits relevant to stormwater. Stormwater will be cleaned, retained, and detained as necessary to provide a superior storm drainage system and watershed in general.

We understand that a planning area of this size will require detailed design even for temporary facilities to make sure that local watersheds are not adversely affected by construction. We anticipate that detailed phasing and the detained improvements necessary to deliver what is discussed above will be provided to the city for review during the design stage and prior to any construction. It is important to us that we protect the local environment from both a personal and professional standpoint and we will do everything possible to ensure that happens.

Please let me know if you have any questions or require further discussion.

Sincerely,

Tony Frayji, PE
FRAYJI DESIGN GROUP, INC.
CC: Bill Brouhard, Brian Spillman & file

June 8, 2021

SUBJECT: Valley's Edge ~ Reaches 5&6 EIR Comments

TO: Mike Sawley, AICP
mike.sawley@chicoca.gov
Principal Planner (Environmental Program Manager)
City of Chico Community Development Department

We are providing this memorandum to address the potential elimination of the connecting street to Honeyrun Road and the need for alternative ways to mitigate the planning area's increased flow that was proposed to be detained with the culvert downsizing under the roadway as discussed in the drainage report we prepared dated 4.29.2020.

Below please find our alternative discussion and the mitigation measure would be needed:

-Detention Requirement

- This planning area is required to mitigate any increase in runoff due to being built out. One of the challenges is that the existing storm drain facilities at Honeyrun Road were shown by our model results in the existing condition overtopping for the 2-, 10-, and 100-year storm events. Per the attached hydrograph one can see that the 100-year flow for the developed condition is higher than the undeveloped hydrograph for the majority of the 100-year event. Thus, detention of the 100-year event will be required. The amount of detention required has been determined to be about 7.5-acre feet. This was determined by calculating the area between the developed and undeveloped hydrographs for the 100-year event. This calculation is likely quite conservative. Once the planning area enters the improvement plan phase and a Storm Drainage Master Plan is submitted it is very likely that stormwater discharge rates will be quite lower due to routing through the storm drain system and overall increase in time of concentration.

  Please note that the information used to produce the hydrographs has also been included in a non-graphical format so that you can see the difference in flow and detention required per 15-minute interval. This data was produced using AutoCAD Storm and Sewer Sanitary Analysis software.

- The connecting road to Honeyrun Road shown in our Drainage Report was used to detain the increased flow. However, with this road potentially being eliminated, the detention needs to be mitigated.

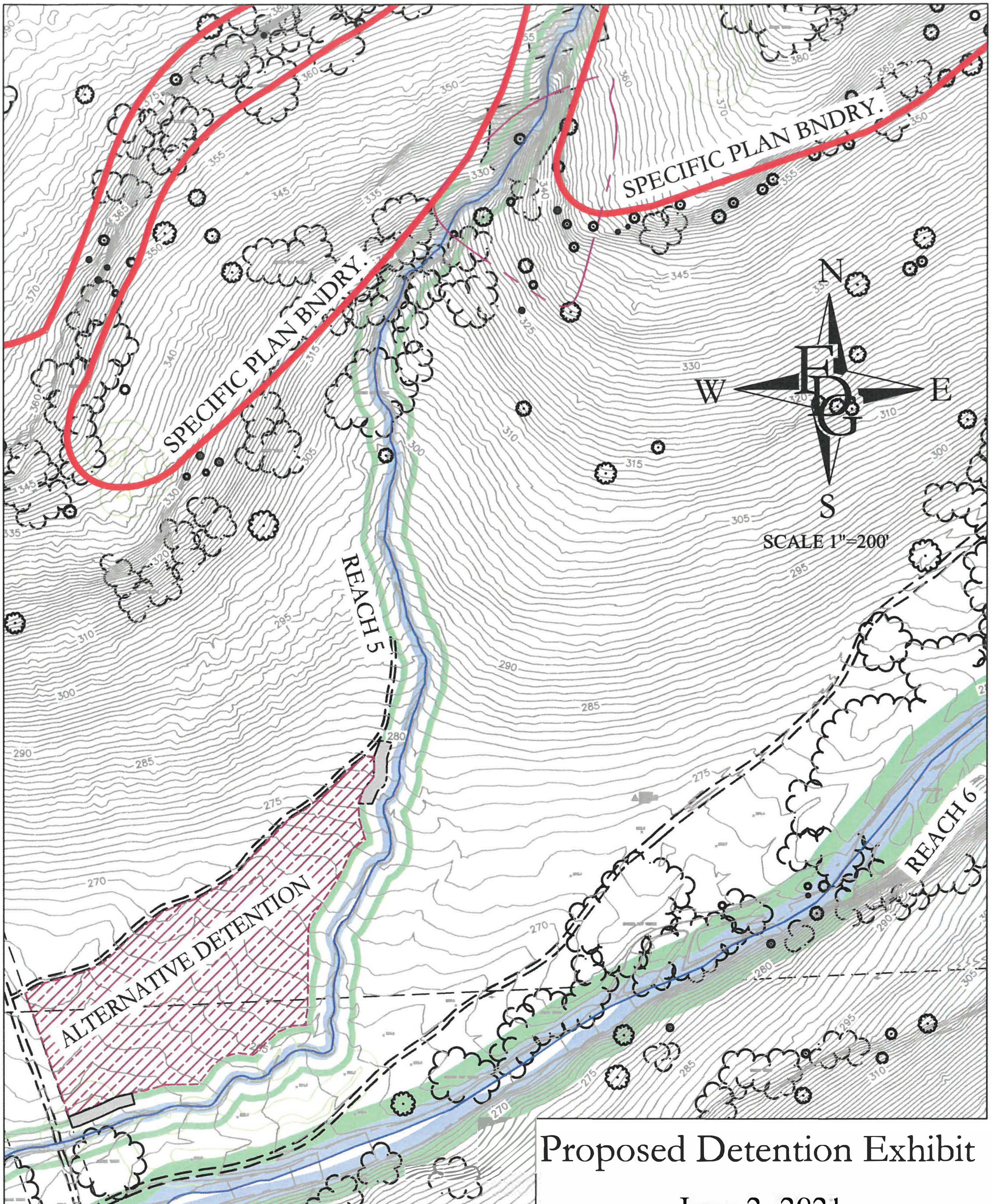
-Mitigation Measures

- In order to decrease the storm water flows at Honeyrun Road to match the undeveloped condition we are proposing the construction of a detention basin as shown on the attached exhibit. This location was selected as it is generally more desirable soil, and it is one of the flatter locations in the drainage. Note to create the basin, a more detailed inlet and outlet design will have to be provided and all the permitting will have to be obtained prior to any construction moving forward. It is also understood that these drainage basins will be constructed during the grading phase of construction of the relevant phase and thus mitigating any potential increases prior to any improvements being completed and or houses being built.
- Additional measures may include attention measuring within the roadway and/or within individual subdivisions or phases as may be determined during the design phase and approved by the city.

Please note that data presented herein is preliminary, and the location of the detention basin is approximate. Both the size and location of the basin are subject to change once the developed condition is modeled in more detail, and approved by the City of Chico.

Sincerely,

Tony Frayji, PE
FRAYJI DESIGN GROUP, INC.
CC: Bill Brouhard, Brian Spillman & file



Proposed Detention Exhibit

June 2, 2021

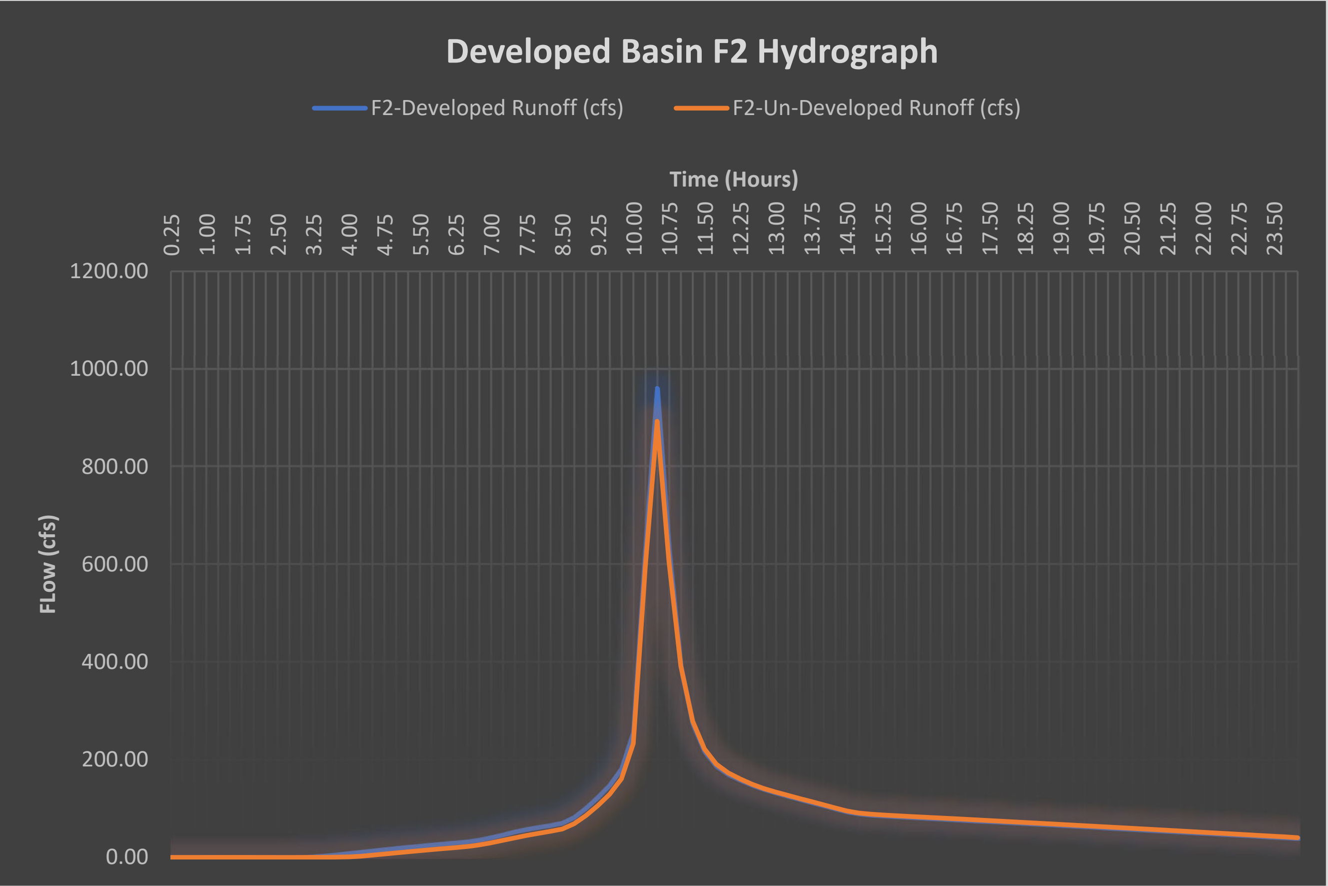
FRAYJi DESIGN GROUP

CIVIL ENGINEERING | PLANNING | SURVEYING

Time (hrs)	Reach 5 Prorated -Developed Runoff (cfs)	Reach 5 Undeveloped	Developed - Undeveloped (cfs)	Reach 5 Volume Requirement per 15 minute interval	
0.25	0.00	0.00	0.00	0.00	
0.50	0.00	0.00	0.00	0.00	
0.75	0.00	0.00	0.00	0.00	
1.00	0.00	0.00	0.00	0.00	
1.25	0.00	0.00	0.00	0.00	
1.50	0.00	0.00	0.00	0.00	
1.75	0.00	0.00	0.00	0.00	
2.00	0.00	0.00	0.00	0.00	
2.25	0.00	0.00	0.00	0.00	
2.50	0.00	0.00	0.00	0.00	
2.75	0.00	0.00	0.00	0.00	
3.00	0.03	0.00	0.03	24.91	
3.25	0.24	0.00	0.24	212.92	
3.50	0.62	0.00	0.62	560.00	
3.75	1.08	0.02	1.06	955.32	
4.00	1.58	0.12	1.45	1309.39	
4.25	2.09	0.33	1.76	1582.13	
4.50	2.62	0.62	1.99	1794.23	
4.75	3.13	0.95	2.18	1959.43	
5.00	3.61	1.29	2.32	2087.35	
5.25	4.08	1.64	2.44	2197.71	
5.50	4.53	1.98	2.55	2296.50	
5.75	4.97	2.31	2.66	2389.66	
6.00	5.39	2.64	2.75	2475.68	
6.25	5.80	2.96	2.84	2557.08	
6.50	6.24	3.29	2.96	2662.87	
6.75	6.93	3.67	3.25	2927.99	
7.00	7.89	4.20	3.69	3319.69	
7.25	8.99	4.85	4.14	3724.70	
7.50	10.16	5.60	4.56	4102.60	
7.75	11.17	6.36	4.80	4324.34	
8.00	11.99	7.05	4.93	4439.86	
8.25	12.73	7.68	5.05	4542.86	
8.50	13.66	8.32	5.34	4803.52	
8.75	15.93	9.37	6.56	5904.71	
9.00	19.60	11.21	8.38	7546.50	
9.25	23.89	13.76	10.13	9118.15	
9.50	28.70	16.81	11.89	10697.41	
9.75	35.21	20.65	14.56	13100.45	
10.00	49.69	27.39	22.31	20075.63	
10.25	122.15	53.04	69.11	62202.90	
10.50	186.29	99.34	86.95	78251.07	
10.75	122.19	105.96	16.24	14613.98	
11.00	75.66	79.35	-3.68	-3316.12	
11.25	53.00	56.35	-3.35	-3018.38	
11.50	42.02	42.42	-0.41	-366.79	
11.75	36.22	34.14	2.07	1865.45	
12.00	32.91	29.21	3.70	3334.46	
12.25	30.68	26.09	4.58	4126.18	
12.50	28.68	23.84	4.84	4352.44	
12.75	27.02	22.01	5.01	4508.37	
13.00	25.64	20.66	4.99	4486.70	
13.25	24.36	19.57	4.79	4314.81	
13.50	23.10	18.56	4.54	4087.59	
13.75	21.85	17.59	4.25	3827.69	
14.00	20.59	16.64	3.96	3560.73	
14.25	19.32	15.68	3.65	3280.64	
14.50	18.12	14.74	3.38	3041.65	
14.75	17.30	13.93	3.37	3037.38	
15.00	16.86	13.37	3.49	3139.36	
15.25	16.55	13.01	3.55	3191.06	
15.50	16.29	12.74	3.55	3194.06	
15.75	16.04	12.52	3.52	3169.00	
16.00	15.79	12.31	3.48	3128.33	
16.25	15.54	12.12	3.42	3077.51	
16.50	15.29	11.93	3.36	3024.04	
16.75	15.04	11.75	3.30	2969.40	
17.00	14.80	11.56	3.24	2913.78	
17.25	14.54	11.37	3.17	2855.97	
17.50	14.29	11.18	3.11	2801.04	
17.75	14.04	10.99	3.05	2745.09	
18.00	13.79	10.80	2.99	2690.36	
18.25	13.53	10.61	2.93	2633.55	
18.50	13.28	10.42	2.86	2577.62	
18.75	13.03	10.22	2.80	2522.83	
19.00	12.77	10.03	2.74	2467.50	
19.25	12.51	9.83	2.68	2412.29	
19.50	12.26	9.64	2.62	2355.27	
19.75	12.00	9.44	2.56	2301.70	
20.00	11.74	9.25	2.50	2247.17	
20.25	11.49	9.05	2.43	2191.11	
20.50	11.23	8.86	2.37	2136.26	
20.75	10.97	8.66	2.31	2081.96	
21.00	10.71	8.46	2.25	2027.09	
21.25	10.45	8.26	2.19	1973.08	
21.50	10.19	8.06	2.13	1917.64	
21.75	9.93	7.86	2.07	1863.14	
22.00	9.68	7.67	2.01	1809.03	
22.25	9.41	7.47	1.95	1753.72	
22.50	9.15	7.27	1.89	1699.77	
22.75	8.89	7.07	1.83	1645.61	
23.00	8.63	6.87	1.77	1590.52	
23.25	8.37	6.66	1.71	1536.74	
23.50	8.11	6.46	1.65	1482.47	
23.75	7.85	6.26	1.59	1428.44	
24.00	7.59	6.06	1.53	1374.63	
				Q100 DETENTION REQUIRED	
	1682.2827	1230.2689	452.0138	406812	FT^3
				9.339	ACRE FT
				3.500	AVERAGE BASIN DEPTH
				2.668	AREA REQUIRED

Time (hrs)	Reach 6 Prorated -Developed Runoff (cfs)	Reach 6 Undeveloped	Developed - Undeveloped (cfs)	Reach 6 Volume Requirement per 15 minute interval	
0.25	0.00	0.00	0.00	0.00	
0.50	0.00	0.00	0.00	0.00	
0.75	0.00	0.00	0.00	0.00	
1.00	0.00	0.00	0.00	0.00	
1.25	0.00	0.00	0.00	0.00	
1.50	0.00	0.00	0.00	0.00	
1.75	0.00	0.00	0.00	0.00	
2.00	0.00	0.00	0.00	0.00	
2.25	0.00	0.00	0.00	0.00	
2.50	0.00	0.00	0.00	0.00	
2.75	0.00	0.00	0.00	0.00	
3.00	0.12	0.00	0.12	103.51	
3.25	0.98	0.00	0.98	884.61	
3.50	2.59	0.00	2.59	2326.60	
3.75	4.49	0.03	4.46	4012.46	
4.00	6.54	0.43	6.11	5500.57	
4.25	8.68	1.73	6.96	6260.45	
4.50	10.87	3.54	7.33	6598.37	
4.75	13.00	5.51	7.49	6743.57	
5.00	15.01	7.49	7.52	6772.45	
5.25	16.95	9.44	7.51	6759.33	
5.50	18.82	11.36	7.45	6707.24	
5.75	20.63	13.25	7.38	6643.76	
6.00	22.39	15.10	7.29	6559.07	
6.25	24.11	16.92	7.18	6463.90	
6.50	25.94	18.85	7.09	6378.95	
6.75	28.78	21.64	7.15	6433.27	
7.00	32.76	25.35	7.41	6664.81	
7.25	37.36	29.67	7.68	6915.96	
7.50	42.21	34.28	7.93	7132.98	
7.75	46.39	38.42	7.97	7168.69	
8.00	49.81	41.99	7.82	7033.78	
8.25	52.87	45.34	7.54	6782.09	
8.50	56.75	49.55	7.20	6481.45	
8.75	66.17	59.14	7.03	6323.73	
9.00	81.43	74.19	7.24	6516.78	
9.25	99.25	91.81	7.44	6698.51	
9.50	119.23	112.11	7.12	6404.94	
9.75	146.28	140.22	6.07	5459.81	
10.00	206.45	205.63	0.83	743.44	
10.25	507.50	541.68	-34.19	-30767.62	
10.50	773.95	793.72	-19.77	-17793.32	
10.75	507.68	498.82	8.86	7971.41	
11.00	314.35	312.08	2.27	2045.30	
11.25	220.20	222.51	-2.31	-2077.91	
11.50	174.56	179.35	-4.79	-4310.11	
11.75	150.47	156.26	-5.80	-5216.87	
12.00	136.74	143.68	-6.95	-6251.56	
12.25	127.45	134.26	-6.81	-6130.08	
12.50	119.15	125.80	-6.65	-5982.96	
12.75	112.25	118.79	-6.54	-5887.25	
13.00	106.54	112.95	-6.40	-5763.72	
13.25	101.21	107.44	-6.23	-5605.95	
13.50	95.99	101.99	-6.00	-5396.62	
13.75	90.77	96.51	-5.75	-5170.67	
14.00	85.55	91.01	-5.46	-4914.26	
14.25	80.29	85.44	-5.16	-4640.93	
14.50	75.27	80.19	-4.92	-4427.77	
14.75	71.88	76.79	-4.91	-4414.74	
15.00	70.04	74.98	-4.94	-4449.65	
15.25	68.77	73.71	-4.95	-4453.32	
15.50	67.67	72.60	-4.93	-4437.92	
15.75	66.63	71.52	-4.89	-4402.39	
16.00	65.60	70.46	-4.86	-4369.85	
16.25	64.57	69.39	-4.83	-4343.90	
16.50	63.54	68.31	-4.77	-4296.68	
16.75	62.50	67.23	-4.72	-4252.44	
17.00	61.47	66.14	-4.68	-4208.58	
17.25	60.42	65.05	-4.63	-4169.52	
17.50	59.38	63.96	-4.58	-4119.86	
17.75	58.33	62.85	-4.52	-4068.57	
18.00	57.28	61.75	-4.46	-4016.13	
18.25	56.23	60.63	-4.41	-3964.82	
18.50	55.17	59.51	-4.34	-3907.68	
18.75	54.12	58.39	-4.27	-3843.94	
19.00	53.06	57.26	-4.21	-3787.74	
19.25	51.99	56.14	-4.15	-3731.66	
19.50	50.93	55.00	-4.08	-3669.26	
19.75	49.86	53.87	-4.00	-3601.80	
20.00	48.79	52.73	-3.93	-3540.42	
20.25	47.72	51.59	-3.87	-3482.20	
20.50	46.65	50.44	-3.79	-3411.91	
20.75	45.58	49.29	-3.71	-3341.31	
21.00	44.51	48.14	-3.63	-3270.05	
21.25	43.43	46.99	-3.56	-3201.34	
21.50	42.35	45.83	-3.48	-3128.27	
21.75	41.28	44.67	-3.39	-3053.33	
22.00	40.20	43.51	-3.31	-2981.56	
22.25	39.11	42.35	-3.24	-2911.58	
22.50	38.03	41.18	-3.15	-2838.06	
22.75	36.95	40.01	-3.06	-2755.61	
23.00	35.86	38.84	-2.98	-2679.26	
23.25	34.78	37.67	-2.89	-2603.22	
23.50	33.69	36.50	-2.81	-2527.54	
23.75	32.61	35.32	-2.71	-2443.17	
24.00	31.52	34.15	-2.63	-2367.67	
				Q100 DETENTION REQUIRED	
	6989.2777	7080.2697	-90.9920	-81893	FT^3
				-1.880	ACRE FT
				3.500	AVERAGE BASIN DEPTH
				-0.537	AREA REQUIRED

Time (hrs)	F2-Developed Runoff (cfs)	F2-Un-Developed Runoff (cfs)	Developed - Undeveloped (cfs)	Volume Requirement per 15 minute interval	
0.25	0.00	0.00	0.00	0.00	
0.50	0.00	0.00	0.00	0.00	
0.75	0.00	0.00	0.00	0.00	
1.00	0.00	0.00	0.00	0.00	
1.25	0.00	0.00	0.00	0.00	
1.50	0.00	0.00	0.00	0.00	
1.75	0.00	0.00	0.00	0.00	
2.00	0.00	0.00	0.00	0.00	
2.25	0.00	0.00	0.00	0.00	
2.50	0.00	0.00	0.00	0.00	
2.75	0.00	0.00	0.00	0.00	
3.00	0.14	0.00	0.14	128.42	
3.25	1.22	0.00	1.22	1097.52	
3.50	3.21	0.00	3.21	2886.60	
3.75	5.57	0.05	5.52	4967.78	
4.00	8.12	0.55	7.57	6809.96	
4.25	10.77	2.06	8.71	7842.58	
4.50	13.49	4.17	9.33	8392.60	
4.75	16.13	6.46	9.67	8703.00	
5.00	18.62	8.78	9.84	8859.79	
5.25	21.02	11.07	9.95	8957.05	
5.50	23.34	13.34	10.00	9003.74	
5.75	25.60	15.56	10.04	9033.42	
6.00	27.78	17.74	10.04	9034.75	
6.25	29.91	19.88	10.02	9020.98	
6.50	32.18	22.14	10.05	9041.82	
6.75	35.71	25.31	10.40	9361.27	
7.00	40.65	29.55	11.09	9984.49	
7.25	46.35	34.53	11.82	10640.66	
7.50	52.37	39.89	12.48	11235.58	
7.75	57.55	44.78	12.77	11493.03	
8.00	61.79	49.05	12.75	11473.64	
8.25	65.60	53.02	12.58	11324.95	
8.50	70.41	57.87	12.54	11284.97	
8.75	82.09	68.51	13.59	12228.44	
9.00	101.03	85.40	15.63	14063.27	
9.25	123.14	105.57	17.57	15816.66	
9.50	147.92	128.92	19.00	17102.35	
9.75	181.49	160.87	20.62	18560.26	
10.00	256.15	233.01	23.13	20819.07	
10.25	629.65	594.72	34.93	31435.28	
10.50	960.24	893.06	67.18	60457.75	
10.75	629.87	604.78	25.09	22585.38	
11.00	390.02	391.43	-1.41	-1270.82	
11.25	273.20	278.86	-5.66	-5096.29	
11.50	216.58	221.78	-5.20	-4676.91	
11.75	186.68	190.41	-3.72	-3351.42	
12.00	169.65	172.89	-3.24	-2917.10	
12.25	158.12	160.35	-2.23	-2003.90	
12.50	147.83	149.64	-1.81	-1630.52	
12.75	139.27	140.80	-1.53	-1378.88	
13.00	132.19	133.61	-1.42	-1277.02	
13.25	125.57	127.00	-1.43	-1291.14	
13.50	119.09	120.55	-1.45	-1309.03	
13.75	112.62	114.11	-1.49	-1342.98	
14.00	106.14	107.65	-1.50	-1353.52	
14.25	99.61	101.12	-1.51	-1360.29	
14.50	93.39	94.93	-1.54	-1386.12	
14.75	89.18	90.71	-1.53	-1377.36	
15.00	86.90	88.35	-1.46	-1310.29	
15.25	85.32	86.72	-1.40	-1262.27	
15.50	83.96	85.34	-1.38	-1243.86	
15.75	82.67	84.04	-1.37	-1233.39	
16.00	81.39	82.77	-1.38	-1241.53	
16.25	80.11	81.52	-1.41	-1266.39	
16.50	78.83	80.25	-1.41	-1272.64	
16.75	77.55	78.97	-1.43	-1283.04	
17.00	76.26	77.70	-1.44	-1294.80	
17.25	74.96	76.42	-1.46	-1313.55	
17.50	73.67	75.14	-1.47	-1318.82	
17.75	72.37	73.84	-1.47	-1323.48	
18.00	71.07	72.54	-1.47	-1325.77	
18.25	69.76	71.24	-1.48	-1331.27	
18.50	68.45	69.93	-1.48	-1330.06	
18.75	67.14	68.61	-1.47	-1321.11	
19.00	65.83	67.29	-1.47	-1320.23	
19.25	64.51	65.97	-1.47	-1319.37	
19.50	63.18	64.64	-1.46	-1313.99	
19.75	61.87	63.31	-1.44	-1300.10	
20.00	60.54	61.98	-1.44	-1293.25	
20.25	59.21	60.64	-1.43	-1291.09	
20.50	57.88	59.30	-1.42	-1275.65	
20.75	56.55	57.95	-1.40	-1259.35	
21.00	55.22	56.60	-1.38	-1242.96	
21.25	53.88	55.25	-1.36	-1228.27	
21.50	52.55	53.89	-1.35	-1210.63	
21.75	51.21	52.53	-1.32	-1190.19	
22.00	49.87	51.18	-1.30	-1172.52	
22.25	48.53	49.81	-1.29	-1157.86	
22.50	47.18	48.45	-1.26	-1138.29	
22.75	45.84	47.08	-1.23	-1110.00	
23.00	44.50	45.71	-1.21	-1088.74	
23.25	43.15	44.34	-1.18	-1066.48	
23.50	41.80	42.96	-1.16	-1045.07	
23.75	40.46	41.58	-1.13	-1014.73	
24.00	39.10	40.21	-1.10	-993.04	
				Q100 DETENTION REQUIRED	
	8671.5604	8310.5385	361.0218	324920	FT^3
				7.459	ACRE FT



Developed Basin F2 Hydrograph
F2-Developed Runoff (cfs)
F2-Un-Developed Runoff (cfs)
Time (Hours)
0.25
1.00
1.75
2.50
3.25
4.00
4.75
5.50
6.25
7.00
7.75
8.50
9.25
10.00
10.75
11.50
12.25
13.00
13.75
14.50
15.25
16.00
16.75
17.50
18.25
19.00
19.75
20.50
21.25
22.00
22.75
23.50
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
FLow (cfs)

"VALLEY'S EDGE" SPECIFIC PLAN

MIXED-USE DEVELOPMENT

CITY OF CHICO, BUTTE COUNTY, CALIFORNIA

Drainage Report Addendum #1

September 14, 2021

PREPARED BY:

Joseph Stebakov

UNDER THE DIRECT GUIDANCE AND SUPPERVISION OF:

Tony Frayji

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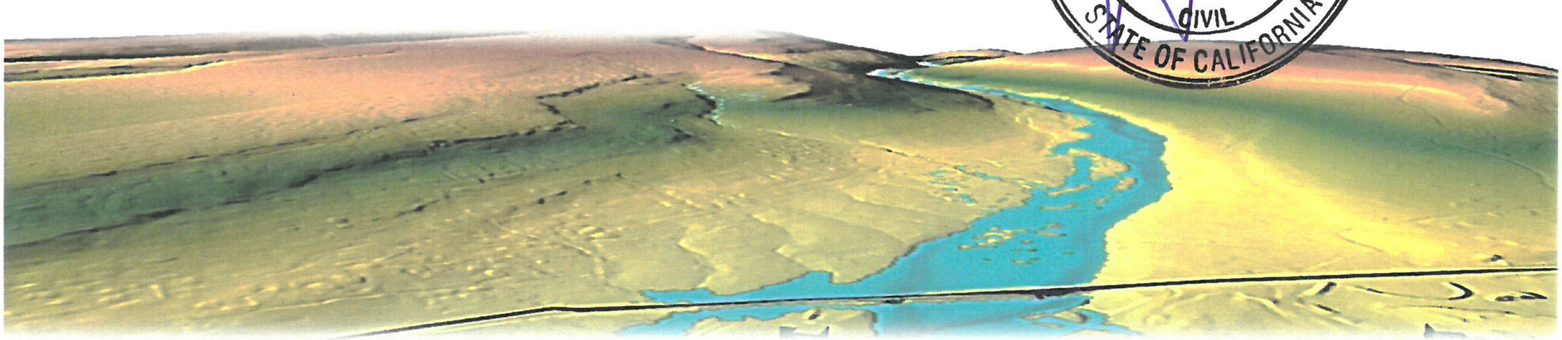
STATE OF CALIFORNIA



Purpose of Addendum

We are providing this report to address the potential elimination of the connecting street to Honeyrun Road, along with the proposed development around reach 6, and the need for alternative ways to mitigate the planning area's increased flow that was proposed to be detained with the culvert downsizing under the roadway as discussed in the drainage report dated 4/29/2020. The connecting road to Honeyrun Road shown in the Drainage report was used to detain the increased flow. However, with this road potentially being eliminated, the detention needs to be mitigated. Various software and tools were used to calculate the difference in flow and the amount of runoff that needs to be detained for the 100 year storm event to maintain existing condition flows.

Summary of Work Performed

The storm and Sanitary Analysis model (SSA) has been updated to exclude the development around reach 6, thus reducing the runoff produced by shed area F2, while also increasing it due to the absence of the initially proposed berm and culverts. The new discharge values produced by shed area F2 (F2A & F2B) were then input into HEC-RAS and the proposed culverts and roadway intersecting reaches 5 and 6 have been removed. The HEC-RAS model was then updated to reflect the detention inflow required in order to account for the increase in discharge. A spreadsheet was then created to represent the volume of storage required for the 100 year storm event due to the updated development. Please see sections below for more information.

Post-Dev Storm and Sanitary Analysis (SSA)

The storm and Sanitary Analysis (SSA) model has been updated to exclude the development around reach 6, thus reducing the total runoff going into Reach 6 (R6). She area F2 was then divided into subshed areas F2A and F2B. This was done in order to determine the exact runoff going into Reach 5 (R5). Shed area F1 was adjusted as well. The CN values and areas representing those values were then updated in the model. A CN value of 98 was used for roadways and paved parking/roofs. The open space areas maintained a CN value of 83. A CN value of 79 was added to the model for the woods/trees area to match the pre-developed model. A CN value of 80 was used for all landscaping. It was also assumed that 55% of lot areas consist of landscaping while 45% of it was considered impervious parking/roofs. Please see Figure 1 below for CN values used. The analysis was then performed and new time series plots were generated for shed areas F1, F2A and F2B for the 2yr, 10yr and 100yr storm events. All other time series plots remained the same as before. Please see **Exhibit 1 – Post-TimeSeriesPlotsR5-R6 (SSA)** for the new discharge values obtained for shed areas F1, F2A and F2B. The **Updated Storm and Sanitary Analysis (SSA)** model has also been provided for your review.

General
Subbasin ID: BASIN-F2A

Connectivity
Rain gage: Rain Gage-Butte-Cher
Outlet node: JUNCTION-15

Description:

Physical Properties | SCS TR-55 TOC | Curve Number

Composite curve number

	Area (ac)	Area (%)	Curve Number	Soil Group	Description
1	31.5500	24.94	80	D	> 75% grass cover, Good
2	25.8100	20.40	98	D	Paved parking & roofs
3	14.1100	11.15	98	D	Paved roads with curbs & sewers
4	55.0500	43.51	83	D	Brush, Poor
5					
6					

Total area: 126.520 ac Total area: 100.00 % Weighted CN: 86.98

	Subbasin ID	Area	Wt. CN	TOC	Rain Gage ID
1	BASIN-F2A	126.520	86.98	33.45	Rain Gage-Butte-
2	{Drainage-UnDEV}.D	9.518	86.98	33.45	Rain Gage-Butte-
3	{Drainage-UnDEV}.D	9.990	84.52	16.06	Rain Gage-Butte-
4	{Drainage-UnDEV}.D	10.740	84.41	17.49	Rain Gage-Butte-
5	{Drainage-UnDEV}.D	4.502	84.84	15.92	Rain Gage-Butte-
6	{Drainage-UnDEV}.D	7.555	84.43	16.01	Rain Gage-Butte-

Figure 1: Curve Numbers (CN) used for Post-Developed Shed F2A

Post-Dev (HEC-RAS)

The Post-Developed HEC-RAS model was then updated to include the new time series plots for shed Areas F1 and F2 (F2A+F2B). The berm at connection "RD (Minor) CP6" was removed as well as the initially proposed culverts. The first analysis was performed assuming no detention around Reach 5 (R5). The 2yr, 10yr and 100yr storm events were analyzed. Once the results were obtained the detention requirements were determined. The next set of runs implemented the detention inflow that would be required for mitigation. Please see Tables 1 through 6 below for a comparison of the 2yr, 10yr and 100yr discharge rates (Q's) at existing roadways (Connections) before and after detention is taken into account. As you can see, different flow values are only seen in connection "RD(Humbug)C5,C6" when comparing to the report. These are highlighted in blue within the tables. Tables 1, 2 and 3 show original discharge rates for the Pre-Developed state and new values for the Post-Developed state, assuming no mitigation. Tables 4, 5 and 6 show original discharge rates for the Pre-Developed state and new Q values for the Post-Developed state, with mitigation taken into account. Results are shown for the 2yr, 10yr and 100yr storm events. Please see the attached **Updated HEC-RAS** model for more information and the attached **Spreadsheet 1 – Detention Basin Calcs (R6 Dev. Removed)** for detention requirement calculations. Discharge values are subject to change for the Post-Developed conditions during the final phases of design due to multiple factors. These values however will not exceed the Pre-Developed flow values.

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
2 Year Storm (cfs)											
R1			R1+R2+R3			R4+R4T			R5+R6		
RD(Dawncrest)C1A,C1B			RD(PotterN)C1-C3			RD(PotterS)C4			RD(Humbug)C5,C6		
	PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)
Qtot =	89.4	89	Qtot =	593.3	586.6	Qtot =	276.6	269.2	Qtot =	1440.2	1535.3
C1A =	49.5	48.2	C1E =	69.1	67.7	C4A =	96.4	95.9	C5A,B =	161.9	161.8
C1B =	40	40.7	C2A,B =	197	196.2	C4B =	68.2	68	C6A,B,C =	290.9	291.9
Weir Flow =	0	0	C3A =	0	4	Weir Flow =	111.9	105.4	C6D =	166.3	170.0
			Weir Flow =	323.4	319.1				Weir Flow =	821.2	912.0

Table 1: 2yr Pre vs. Post Discharge at Existing Connections (No development around R6 and no detention)

1316 BLUE OAKS BLVD | ROSEVILLE, CA 95678 | (916) 782-3000 | FRAYJIDG.COM

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
10 Year Storm (cfs)											
R1			R1+R2+R3			R4+R4T			R5+R6		
RD(Dawncrest)C1A,C1B			RD(PotterN)C1-C3			RD(PotterS)C4			RD(Humbug)C5,C6		
	PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)
Qtot =	153.1	135.5	Qtot =	1027.5	930.5	Qtot =	392.2	388.1	Qtot =	2360.5	2617.5
C1A =	88.5	77.1	C1E =	94.6	86.2	C4A =	102.9	102.7	C5A,B =	165.5	167.0
C1B =	64.6	58.4	C2A,B =	221.1	215.9	C4B =	71.6	71.5	C6A,B,C =	324	329.5
Weir Flow =	0	0	C3A =	0	4.1	Weir Flow =	217.7	213.9	C6D =	202.7	211.3
			Weir Flow =	707.7	625.4				Weir Flow =	1668.4	1909.7

Table 2: 10yr Pre vs. Post Discharge at Existing Connections (No development around R6 and no detention)

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
100 Year Storm (cfs)											
R1			R1+R2+R3			R4+R4T			R5+R6		
RD(Dawncrest)C1A,C1B			RD(PotterN)C1-C3			RD(PotterS)C4			RD(Humbug)C5,C6		
	PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)
Qtot =	306.1	241.7	Qtot =	2048.2	1624.2	Qtot =	822.3	652.3	Qtot =	4941.2	5251.8
C1A =	170.1	144.3	C1E =	139.3	121.1	C4A =	117	112.4	C5A,B =	174.5	178.1
C1B =	111.4	97.4	C2A,B =	260.5	245.8	C4B =	79.2	76.7	C6A,B,C =	375.2	377.4
Weir Flow =	24.6	0	C3A =	0	4.2	Weir Flow =	626.1	463.2	C6D =	275.4	283.5
			Weir Flow =	1644.1	1253.1				Weir Flow =	4113.3	4412.8

Table 3: 100yr Pre vs. Post Discharge at Existing Connections (No development around R6 and no detention)

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
2 Year Storm (cfs)											
R1			R1+R2+R3			R4+R4T			R5+R6		
RD(Dawncrest)C1A,C1B			RD(PotterN)C1-C3			RD(PotterS)C4			RD(Humbug)C5,C6		
	PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)
Qtot =	89.4	89	Qtot =	593.3	586.6	Qtot =	276.6	269.2	Qtot =	1440.2	1415.4
C1A =	49.5	48.2	C1E =	69.1	67.7	C4A =	96.4	95.9	C5A,B =	161.9	161.3
C1B =	40	40.7	C2A,B =	197	196.2	C4B =	68.2	68	C6A,B,C =	290.9	286.3
Weir Flow =	0	0	C3A =	0	4	Weir Flow =	111.9	105.4	C6D =	166.3	164.2
			Weir Flow =	323.4	319.1				Weir Flow =	821.2	804.2

Table 4: 2yr Pre vs. Post Discharge at Existing Connections (No development around R6, with Detention)

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
10 Year Storm (cfs)											
R1			R1+R2+R3			R4+R4T			R5+R6		
RD(Dawncrest)C1A,C1B			RD(PotterN)C1-C3			RD(PotterS)C4			RD(Humbug)C5,C6		
	PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)
Qtot =	153.1	135.5	Qtot =	1027.5	930.5	Qtot =	392.2	388.1	Qtot =	2360.5	2356.2
C1A =	88.5	77.1	C1E =	94.6	86.2	C4A =	102.9	102.7	C5A,B =	165.5	165.7
C1B =	64.6	58.4	C2A,B =	221.1	215.9	C4B =	71.6	71.5	C6A,B,C =	324	322.0
Weir Flow =	0	0	C3A =	0	4.1	Weir Flow =	217.7	213.9	C6D =	202.7	202.1
			Weir Flow =	707.7	625.4				Weir Flow =	1668.4	1666.4

Table 5: 10yr Pre vs. Post Discharge at Existing Connections (No development around R6, with detention)

PRE VS. POST DISCHARGE AT EXISTING CONNECTIONS (HECRAS)											
100 Year Storm (cfs)											
R1			R1+R2+R3			R4+R4T			R5+R6		
RD(Dawncrest)C1A,C1B			RD(PotterN)C1-C3			RD(PotterS)C4			RD(Humbug)C5,C6		
	PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)		PRE (cfs)	POST (cfs)
Qtot =	306.1	241.7	Qtot =	2048.2	1624.2	Qtot =	822.3	652.3	Qtot =	4941.2	4892.0
C1A =	170.1	144.3	C1E =	139.3	121.1	C4A =	117	112.4	C5A,B =	174.5	176.7
C1B =	111.4	97.4	C2A,B =	260.5	245.8	C4B =	79.2	76.7	C6A,B,C =	375.2	372.6
Weir Flow =	24.6	0	C3A =	0	4.2	Weir Flow =	626.1	463.2	C6D =	275.4	274.9
			Weir Flow =	1644.1	1253.1				Weir Flow =	4113.3	4067.8

Table 6: 100yr Pre vs. Post Discharge at Existing Connections (No development around R6, with detention)

Detention Basin Calculations (Reaches 5 and 6)

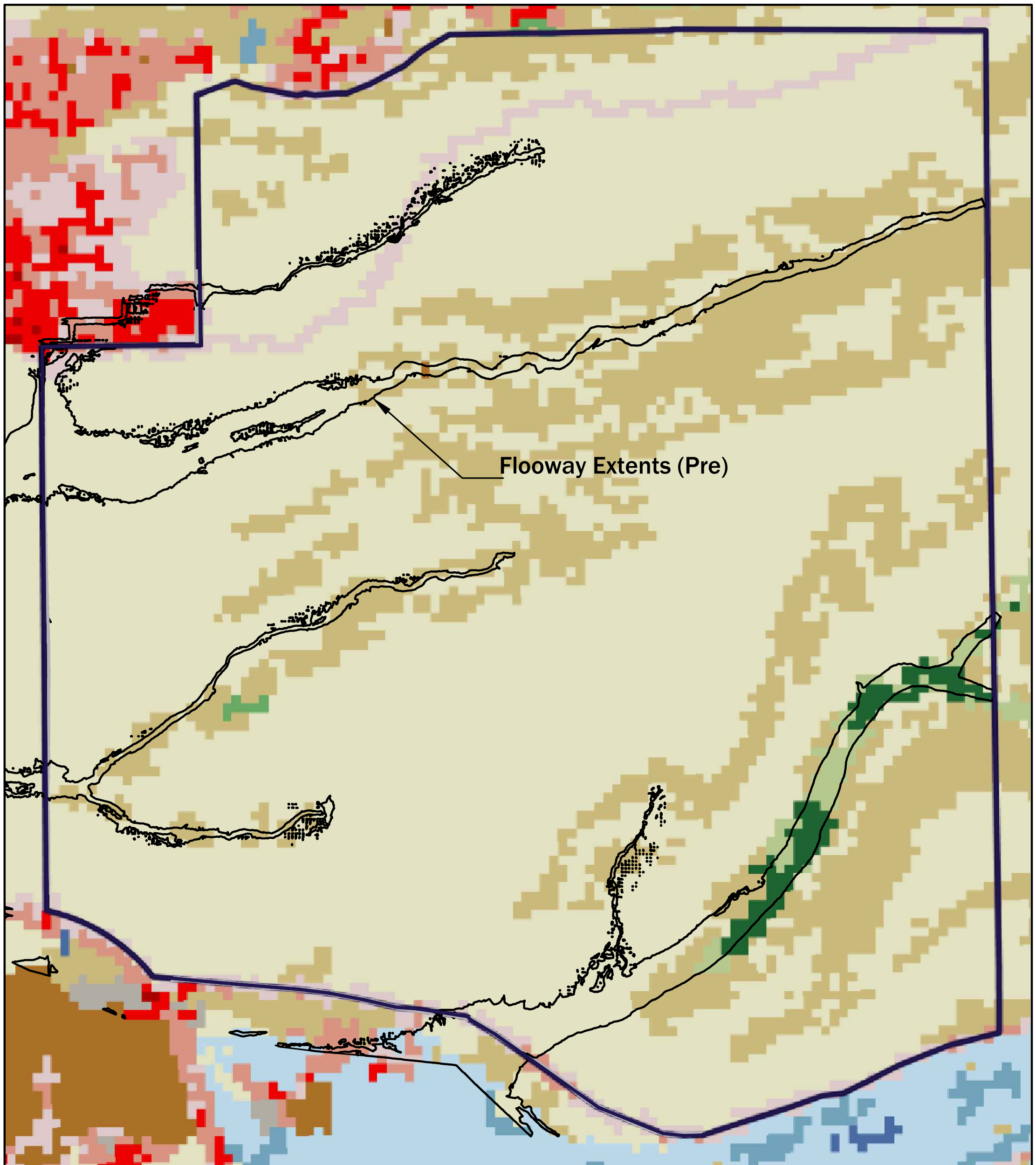
Time series plots produced by HEC-RAS at connection "RD(Humbug)C5,C6" were used to calculate the basin requirements for Reaches 5 and 6. An excel spreadsheet was used for calculating the volume of storage required for the 100 year event (see attached **Spreadsheet 1 – Detention Basin Calcs (R6 Dev. Removed)**). An equation was set up to take the difference between the developed (unmitigated) and undeveloped Q values obtained from HEC-RAS for each 10 min time interval. This flow was then multiplied by 60 (seconds) and then by 15 (minutes) to give a volume of 605448 ft^3. This means that the amount of detention required for a 24 hour storm event is about 14 AC-FT. An assumed basin depth of 4 ft was applied, giving a minimum required detention acreage of 3.5 AC. Please see **Exhibit 2 – Proposed Detention Exhibit (R6 Dev. Removed)**, which shows the location and acreage of the proposed detention basin area.

Proposed Mitigation Measures (Reaches 5 and 6)

In order to decrease the storm water flows at Honeyrun Road to match the undeveloped condition we are proposing the construction of a detention basin as shown on the attached **Exhibit 2 – Proposed Detention Exhibit (R6 Dev. Removed)**. Additional measures may include attention measuring within the roadway and/or within individual subdivisions or phases as may be determined during the design phase and once approved by the city. Please note that data presented herein is preliminary, and the location of the detention basin is approximate. Once the planning area enters the improvement plan phase and a Storm Drainage Master Plan is submitted, it is very likely that stormwater discharge rates will be quite lower due to routing through the storm drain system and overall increase in time of concentration. Therefore, both the size and location of the basin are subject to change.

It is understood that these drainage basins will be constructed during the grading phase of construction of the relevant phase and thus mitigating any potential increases prior to any improvements being completed and/or houses being built. A more detailed inlet and outlet design will have to be provided and all permitting will have to be obtained prior to any construction moving forward.

EXHIBIT A.1.1



Land Cover Name	Manning's	Color
Agricultural, Cultivated Crops	0.035	
Agricultural, Pasture/Hay	0.03	
Developed, High Density	0.15	
Developed, Low Density	0.10	
Developed, Medium Density	0.08	
Developed, Open Space	0.04	
Open Water	0.04	
Undeveloped, Barren Land	0.025	
Undeveloped, Deciduous Forest	0.16	
Undeveloped, Evergreen Forest	0.16	
Undeveloped, Grassland	0.035	
Undeveloped, Mixed Forest	0.16	
Undeveloped, Shrub/Scrub	0.10	
Wetlands, Forested	0.12	

NOTES:
THIS EXHIBIT IS INTENDED TO SERVE "ONLY" AS A VISUAL ILLUSTRATION OF THE ONSITE MANNING'S ROUGHNESS COEFFICIENTS ("n" VALUES) USED IN THE HEC-RAS MODEL.

ALL MANNING'S "n" VALUES WERE OBTAINED FROM THE USGS LAND COVER DATABASE AND APPLIED TO THE HEC-RAS MODEL. THE ACTUAL FILE IN TIF FORMAT CAN BE FOUND IN THE SUBMITTED HEC-RAS PROJECT FOLDER

N
W E
S

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USGS Land Cover Data (HEC-RAS)

Valley's Edge Drainage

CITY OF CHICO, BUTTE COUNTY, CALIFORNIA

DATE: 03-20-2020

SCALE: NTS

28100- Valley's Edge

PAGE 1 OF 1