DELTA CONSULTING & ENGINEERING
OF ST. HELENA

HYDROLOGY & DRAINAGE REPORT

FOR THE

BRIAN ARDEN WINERY

LOCATED AT

331 SILVERADO TRAIL
CALISTOGA, CA 94515

COUNTY: NAPA
APN: 011-050-030

INITIAL SUMBITTAL: JANUARY 18, 2012
SUBMITTAL # 2: FEBRUARY 01, 2012

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I. REPORT DESCRIPTION & BACKGROUND

This report analyzes the site hydrology for a proposed winery development at 331 Silverado Trail in Calistoga, Napa County. The subject property is approximately 2.25 acres, with the majority of the parcel being flat undeveloped land and located on the downstream side of a 2.14 acre watershed from Mount Washington. The property currently is undeveloped, with dense grass covering the majority of the property. The property is outside of the FEMA floodplain per FIRMette 06097C0625E.

The proposed winery will have a footprint of approximately 10,560 square feet. Other improvements to the parcel include landscaping, a paved main driveway, and the addition of approximately 29,700 square feet of vineyard.

Delta Consulting & Engineering was hired to perform a hydrologic analysis of the proposed set of Use Permit Plans for the property prepared by James Cassayre and dated January 18, 2012. See Appendix E to view an 11"x17" 'not-to-scale' copy of these plans. The items Delta Consulting & Engineering was requested to analyze include an analysis of the pre-project and post-project runoff flows, the detention basin sizing, and the site flow patterns.

The purpose of this report is to investigate the storm water runoff hydrologic flows for the pre-construction and post-construction conditions of the project. The design storm events evaluated in this report are the 10-year and the 100-year design storms using the SCS Method and the Rational Method, both industry standards for determining hydrologic conditions.

II. METHODOLOGY

One common practice in rainfall-runoff analysis is to develop a synthetic rainfall distribution to use in lieu of actual storm events. The intensity of rainfall varies considerably during storm events, as well as with geographic location. Therefore, the synthetic rainfall distribution model must account for both of these variables to appropriately recreate a storm event.

The National Resource Conservation Service (NRCS, formerly the SCS) developed four synthetic 24-Hour rainfall distributions types (I, IA, II, and III) from National Weather Service duration-frequency data. Each rainfall distribution type represents various regions of the United States; each modeling rainfall distributions typical to each geographic area. The design rainfall for this site was derived using the SCS (NRCS) Type IA 24-Hr Storm Distributions as this project site is located within the geographic boundaries of the specified Type IA rainfall distribution. The SCS 24-hour rainfall distributions are shown in Figure 1, with the corresponding geographic boundaries shown in Figure 2. The design rainfall for a storm with a 10-year return period in Calistoga, Napa County per NOAA (National Oceanic and Atmospheric Administration) isopluvial precipitation maps is approximately 6.8 inches in a 24 hour period, and the design rainfall for a 100-year return period is approximately 10 inches of rainfall. This information, combined with the site time of concentration and the surface curve numbers, the peak flows and the total storm runoff volumes are able to be determined. To assist with the creation of the storm water hydrographs and peak flow calculations, the software program StormNET by Boss International will be utilized in this analysis.

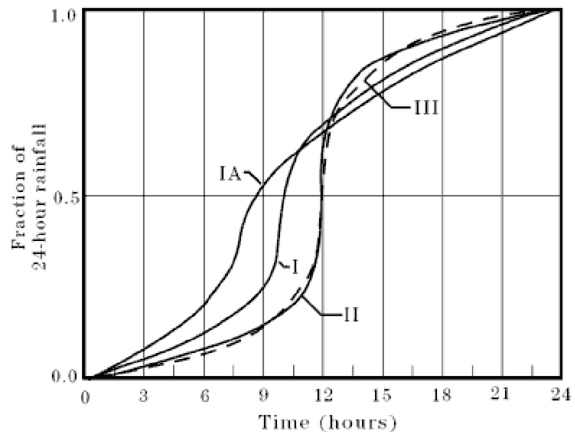

Figure 1: SCS 24-Hour Storm Distributions

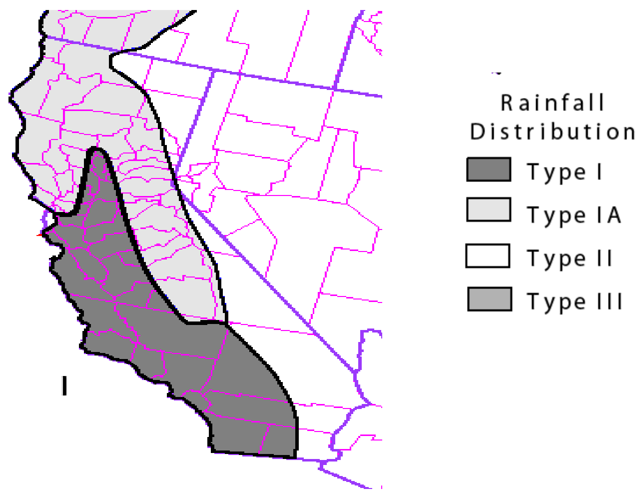

Figure 2: SCS Rainfall Distribution Geographic Boundaries in California

The time of concentration is the overland travel time it takes for storm water to travel from the most remote point in the watershed to the point of interest (also known as the concentration point). Storm water runoff travels through the watershed as sheet flow, shallow concentrated flow, open channel flow, or any combination of these depending on the site specific topography. The site parameters for each basin are defined, and the means of surface travel by the storm water runoff is determined. The time of concentration is determined by summing together all of the individual runoff travel times within each sub basin.

The SCS TR-55 curve number, another requirement in determining the hydrologic quantities of watersheds, is a simple, widely used, and efficient method for determining the fraction of precipitation depth that will translate into basin runoff. The curve number is based on the drainage area's hydrologic soil group, land use, and hydrologic conditions. A high curve number (100 being the highest) is used for impervious surfaces, and causes nearly all of the precipitation to translate into runoff. On the other side of the spectrum, a low curve number (1 being the lowest) value, such as for sandy soils, causes the majority of the precipitation to be captured as infiltration and not translate into runoff.

III. TOPOGRAPHICAL DATA

A field survey was performed on the property by Albion Surveys of St. Helena. However, the majority of the upstream watershed was not surveyed and no topographical information was received on this area. As such, topographical data for this project was obtained from Napa County 2002 Digital Terrain Modeling and Topographic Mapping.

IV. STORM WATER DETENTION ANALYSIS

To establish a base condition for the site and the contiguous areas, a watershed runoff model was developed using the pre-construction site conditions (current conditions) for the project watershed. The pre-construction project watershed is undeveloped, with a majority of the area being relatively flat with a range of 2 to 5 percent slopes. The storm water derived within the subject property currently sheet flows across the property prior to coming in contact with an existing berm located along the east property line. The existing berm redirects the storm water runoff to the south in a shallow concentrated flow next to the current property driveway. The water travels along this berm until crossing over the southern property line onto the neighbor's property. See Appendix B for a photographic analysis of the existing watershed flow patterns.

A. SCS TR-55 Method

Based on the pre-construction condition of the property, the composite SCS curve number for this area is 63.87. See Table 1 for a detailed area analysis of the pre-construction conditions. The time of concentration for these conditions is 37.19 minutes. The theoretical peak storm water runoff flow with the pre-construction conditions is 0.30 cubic feet per second for the 2-year design storm, 0.93 cubic feet per second for the 10-year design storm, and 3.37 cubic feet per second for the 100-year design storm. See Table 2 for a succinct summary of storm water flow based on the pre-construction conditions. See Figure 3 for the pre-construction hydrograph of the project watershed for the 100-year design storm.

Composite curve number

	Area (ac)	Area (%)	Curve Number	Soil Group	Description
1	1.76000	92.24	61	B	> 75% grass cover, Good
2	0.14800	7.76	98	B	Paved parking & roofs
3					
4					
5					
6					

Total area: 1.908 ac Total area: 100.00 % Weighted CN: 63.87

Table 1: Pre-Construction Watershed Parameters, 10-Year and 100-Year Design Storms

Pre-Construction: SCS Method

2-Year Design Storm	Total Precipitation (in)	4.5
	Peak Runoff (cfs)	0.3
10-Year Design Storm	Total Precipitation (in)	6.8
	Peak Runoff (cfs)	0.93
100-Year Design Storm	Total Precipitation (in)	10
	Peak Runoff (cfs)	2.02

Table 2: Pre-Construction Watershed Analysis, 2-Year, 10-Year and 100-Year Design Storms

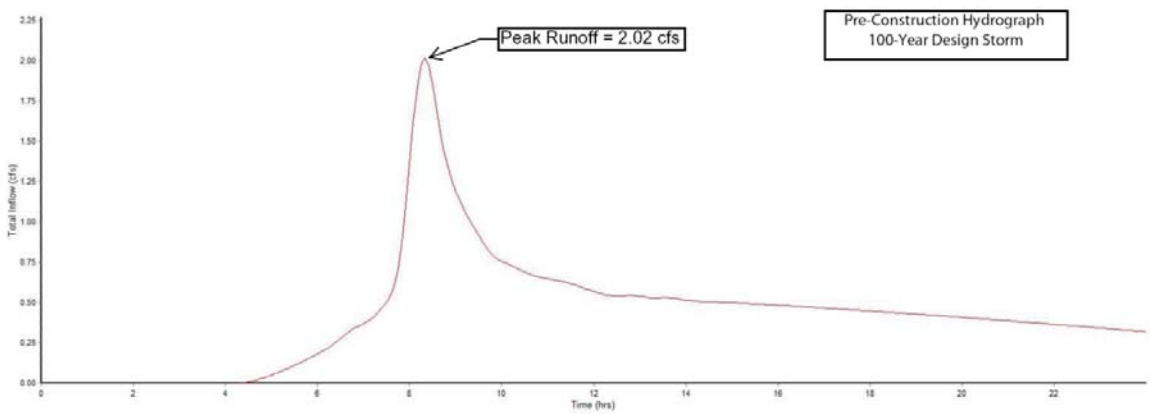

Figure 3: Pre-Construction Hydrograph, 100-Year Design Storm

The post-construction conditions add a winery building of approximately 10,560 square feet, a paved driveway, landscaping, and approximately 29,700 square feet of vineyard. The composite curve number for the post-construction conditions increases to 78.38, and the time of concentration lowers to 22.42 minutes. The peak storm water runoff rate increases to 0.96 cubic feet per second for the 2-year design storm, 1.94 cubic feet per second for the 10-year design storm and to 3.37 cubic feet per second for the 100-year design storm. The runoff patterns for the storm water flow are altered into a combination of sheet flow, shallow concentrated flow, and conveyance through a proposed subsurface drainage system. In order to reduce the post-construction flows, a proposed detention basin is to be installed to detain and release the water at a lower flowrate. The storm water runoff entering the subsurface drainage system will enter the proposed detention basin prior to being released into energy dissipaters and water spreaders in numerous locations. After being released, the storm water runoff will exit the property in the same manner as the pre-construction conditions. See Appendix A for pre-construction watershed maps.

See Table 3 for a detailed area analysis of the post-construction conditions. See Table 4 for a succinct summary of storm water flow based on the post-construction conditions, and see Figure 4 for the post-construction hydrograph for the 100-year design storm. See Appendix D for an overview of typical curve number values.

Composite curve number

	Area (ac)	Area (%)	Curve Number	Soil Group	Description
1	0.41800	21.91	98	B	Paved Driveway
2	0.17200	9.01	90	B	Landscape/Paver
3	0.10600	5.56	84	B	Covered Courtyard
4	0.16400	8.60	98	B	Buildings
5	0.35794	18.76	65	B	Landscaping, Good
6	0.69000	36.16	65	B	Untilled Vineyard, Good

Total area: 1.908 ac   Total area: 100.00 %   Weighted CN: 78.38

Table 3: Post-Construction Watershed Parameters

Post-Construction: SCS Method

2-Year Design Storm	Total Precipitation (in)	4.5
	Peak Runoff (cfs)	0.96
10-Year Design Storm	Total Precipitation (in)	6.8
	Peak Runoff (cfs)	1.94
100-Year Design Storm	Total Precipitation (in)	10
	Peak Runoff (cfs)	3.37

Table 4: Post-Construction Watershed Analysis

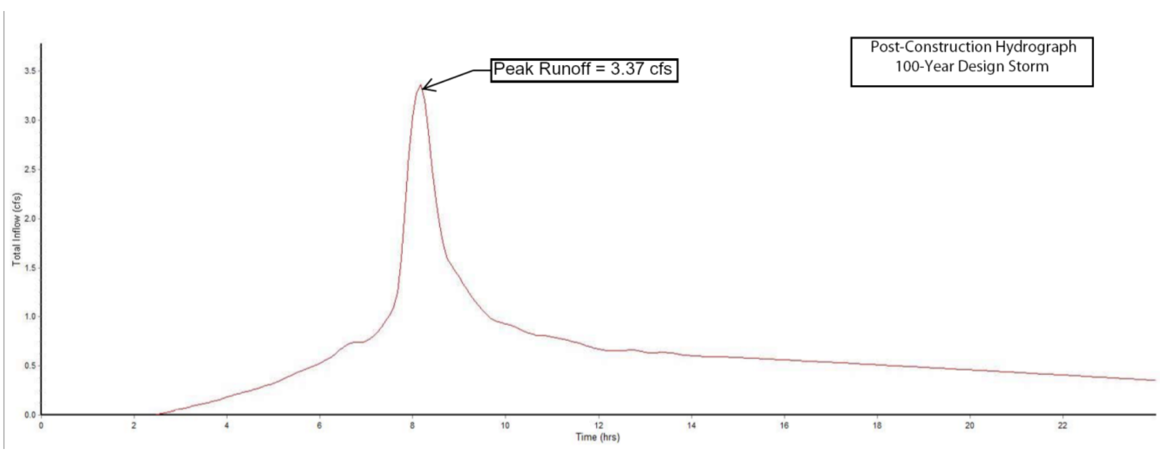

Figure 4: Post Construction Hydrograph

The pre-construction watershed has a peak storm water runoff of 2.02 cubic feet per second, while the post-construction watershed derives a peak storm water runoff of 3.37 for the 100-year design storm. By law, the owner is required to keep the post-construction storm water runoff flows equal or less than the pre-construction runoff flows. In order to meet this standard, the property would be required to infiltrate, or detain on-site the difference between the peak pre-construction flows (2.02 cubic feet per second) and the peak post-construction flows (3.37 cubic feet per second). The required on-site detention capacity can be determined by analyzing the difference between the pre-construction hydrograph and the post-construction hydrograph. Figure 5 shows how the storage requirement of 2,417 cubic feet for the 100-year design storm was determined. However, see Appendix A for the Post-Construction Watershed Maps.

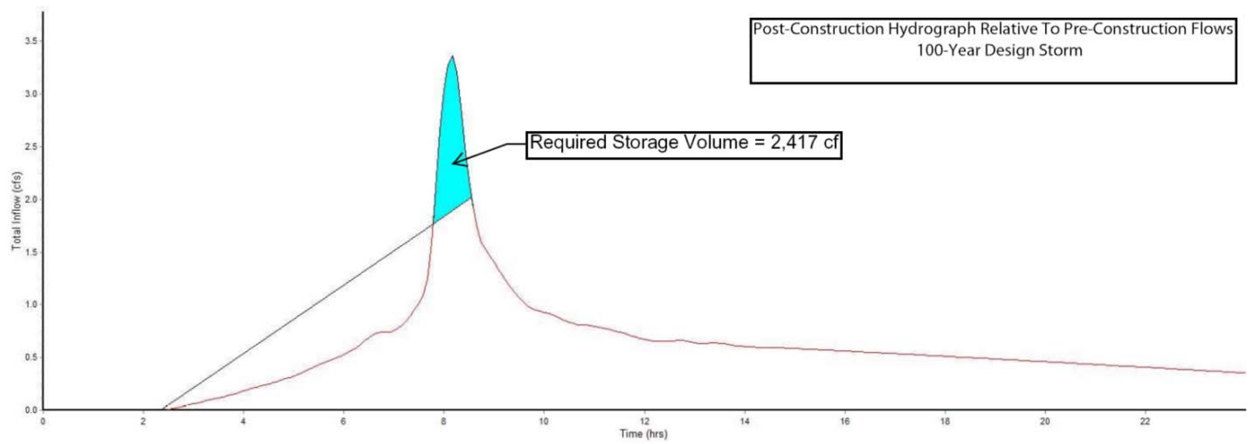

Figure 5: Post-Construciton Hydrograph Relative to Pre-Construction Peak Flow (2.02 cfs). Storage Determination for the 100-Year Design Storm

B. Rational Method

The City of Calistoga requested the 10 year and 100 year peak runoff be calculated using the Rational Method in addition to the standard SCS method. The Rational Method uses the Rational Equation 'Q=CiA', which is based on site specific intensity-duration-frequency curves (IDF curves), surface retardance roughness factors, and watershed areas. Per request from the City of Calistoga Public Works, two individual analyses will be presented for the Rational Method based on the following:

1.) City of Santa Rosa Public Storm Drain Standards
2.) Site Specific IDF curves generated from the National Oceanic and Atmospheric Administration (NOAA) Isopluvial Maps

1. City of Santa Rosa Public Storm Drain Standards

All values and references used with this method are based on the City of Santa Rosa Public Storm Drain Standards (PSDS). See Appendix K for excerpts from the PSDS. As the PSDS utilizes a single IDF curve for the entire county, an additional correction factor 'k' based on the mean seasonal precipitation of the specific location within Sonoma County is used. The calculations for the Pre- and Post-Construction Storm Water Runoff Flows are as follows:

Pre-Construction

Rational Method: Q=Cv*i*A*K

C= .35 (Agriculture and Open Space, >2-7% Slope) [per PSDS Table I-1]
A = 1.908 acres
T_c = 15 minutes [per PSDS T_c for Open Space]
$i_{10\ year}$ = 1.72 inches/hour, $i_{100\ year}$ = 2.40 inches/hour [per PSDS Figure I-1]
K = 35"/30 = 1.167 [per PSDS Figure I-2]

$Q_{10\text{-}year}$ = 0.35*(1.908 acres)*(1.72 inches / hour)*(1.167)
$Q_{10\text{-}year}$ = 1.34 cfs

$Q_{100\text{-}year}$ = 0.35*(1.908 acres)*(2.40 inches / hour)*(1.167)
$Q_{100\text{-}year}$ = 1.87 cfs

Post-Construction

Rational Method: Q=Cv*i*A*K

Determine C [per PSDS Table I-1]

C_1= 0.90	A_1 = 0.418 acre	Impervious Driveway
C_2= 0.49	A_2 = 0.172 acres	Landscape Area
C_3= 0.9	A_3 =0.106 acres	Covered Courtyard
C_4= 0.9	A_4=0.164 acres	Impervious Building
C_5= 0.35	A_5=0.69 acres	Vineyard
C_6= 0.35	A_6=0.358 acres	Undeveloped Area
	A_{Total} = 1.908 acres	

$Cv = C_1*(A_1 / A_{Total}) + C_2*(A_2 / A_{Total}) + C_3*(A_3/A_{Total}) + C_4*(A_4/A_{Total}) + C_5*(A_5/A_{Total}) + C_6*(A_6/A_{Total})$
= 0.9*(0.418/1.908) + 0.49*(0.172/1.908) + 0.9*(0.106/1.908) + 0.90*(0.164/1.908) +
0.35*(0.69/1.908) + 0.35*(.358/1.908)
Cv =0.56

T_c = 7 minutes [per PSDS T_c for Commercial]
$i_{10\ year}$ = 2.6 inches/hour, $i_{100\ year}$ = 4.1 inches/hour [per PSDS Figure I-1]
K = 35"/30 = 1.167 [per PSDS Figure I-2]

$Q_{10\text{-}year}$ = 0.56*(1.908 acres)*(2.6 inches / hour)*(1.167)
$Q_{10\text{-}year}$ = 3.24 cfs

$Q_{100\text{-}year}$ = 0.56*(1.908 acres)*(4.1 inches / hour)*(1.167)
$Q_{100\text{-}year}$ = 5.11 cfs

The Rational Method first introduced in 1889 for the purpose of estimating theoretical peak flows from drainage areas less than 200 acres. It is successful for determining the peak demand from watersheds to assist with the design of drainage features such as pipes or culverts. However, the Rational Method is not known for its accuracy in determining storage requirements from pre- and post-construction storms. However, we are including this in the report per request from the City of Calistoga. See Figure 6 below for the graphical determination of the storage requirement between the pre- and post-construction conditions.

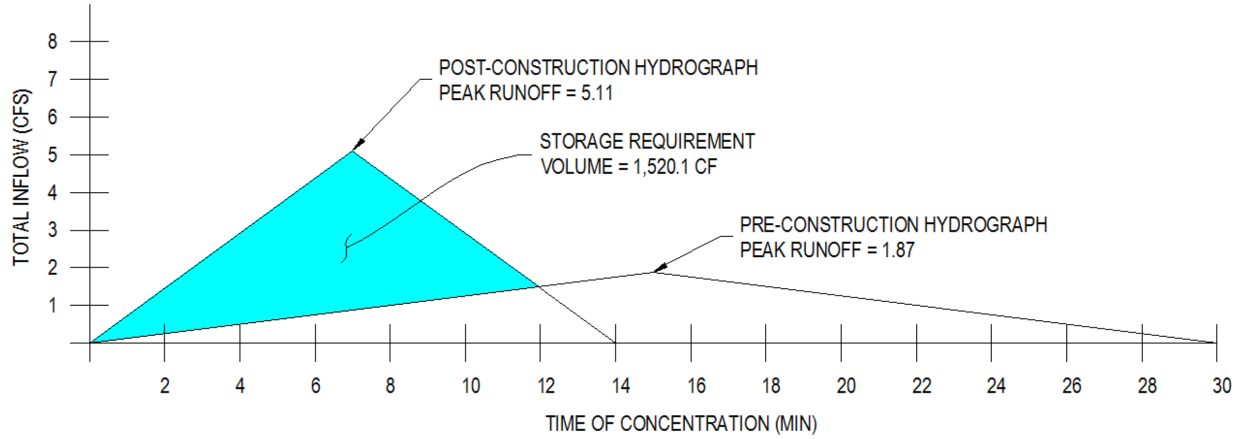

Figure 6: Pre- and Post-Construction Hydrographs. Storage Determination for the 100-Year Design Storm

Per the Rational Method using the City of Santa Rosa Standards, the required storage was found to be 920 cubic feet for the 10-year design storm and 1,520 cubic feet for the 100-year design storm.

2. Site Specific IDF curves generated from the NOAA Isopluvial Maps

The NOAA developed precipitation isopluvial maps of the Western States in Atlas 2, developed in 1973. These maps show typical precipitation contours using a 6-hour and 24-hour design storm over numerous design intervals. Using the precipitation information at the property site, IDF curves are able to be generated for use in the Rational Method. See Figure 7 for the IDF curves generated for this analysis:

General
ID: IDF Curve for APN: 011-050-030
Description:

Rainfall equation
- IDF table
- i = B/(Tc+D)^E
- i = A + B(LnX) + C(LnX)² + D(LnX)³
- Intensity direct entry

X-axis
- Linear
- Log

Y-axis
- Linear
- Log

Return period
Period: 100 years

Enter rainfall intensity in in/hr

Return Period (years)	Storm Duration (minutes)			
	5	15	30	60
1	0	0	0	0
2	3.39	2.22	1.54	0.98
3	0	0	0	0
5	4.69	3.08	2.1	1.35
10	5.6	3.8	2.5	1.6
25	6.8	4.5	3.09	1.96
50	7.76	5.2	3.5	2.3
100	8.6	5.7	3.91	2.48

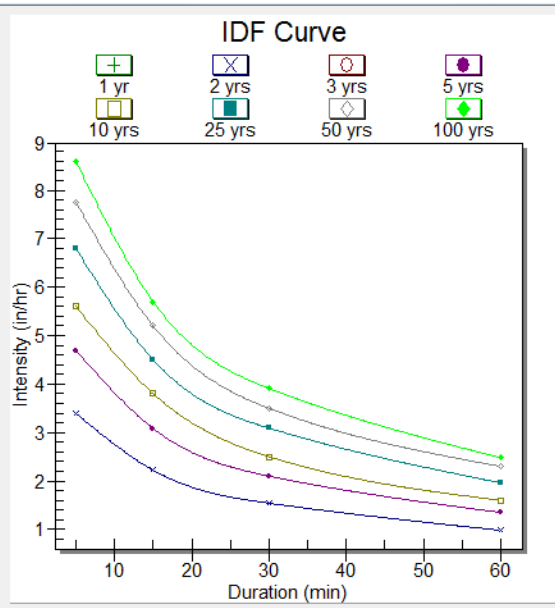

Figure 7: Site Specific IDF Curve for the proposed Brian Arden Winery

Based on the pre-construction condition of the property, the composite Runoff Coefficient for the property is 0.3. The time of concentration for these conditions is 41.61 minutes (per the SCS method for determining the time of concentration). The theoretical peak storm water runoff flow with the pre-construction conditions is 1.13 cubic feet per second for the 10-year design storm, and 4.19 cubic feet per second for the 100-year design storm. See Table 5 for a detailed area analysis of the pre-construction conditions. See Figure 8 for the pre-construction hydrograph of the project watershed for the 100-year design storm.

Composite runoff coefficient

	Area (ac)	Area (%)	Runoff Coefficient	Soil Group	Description
1	1.76000	92.24	0.25	B (2-6%)	Open Space, less than 25 years
2	0.14800	7.76	0.9	B (2-6%)	Streets, less than 25 years
3					
4					
5					
6					

Total area: 1.908 ac Total area: 100.00 % Weighted coeff: 0.30

Table 5: Pre-Construction Input Information for the Rational Method

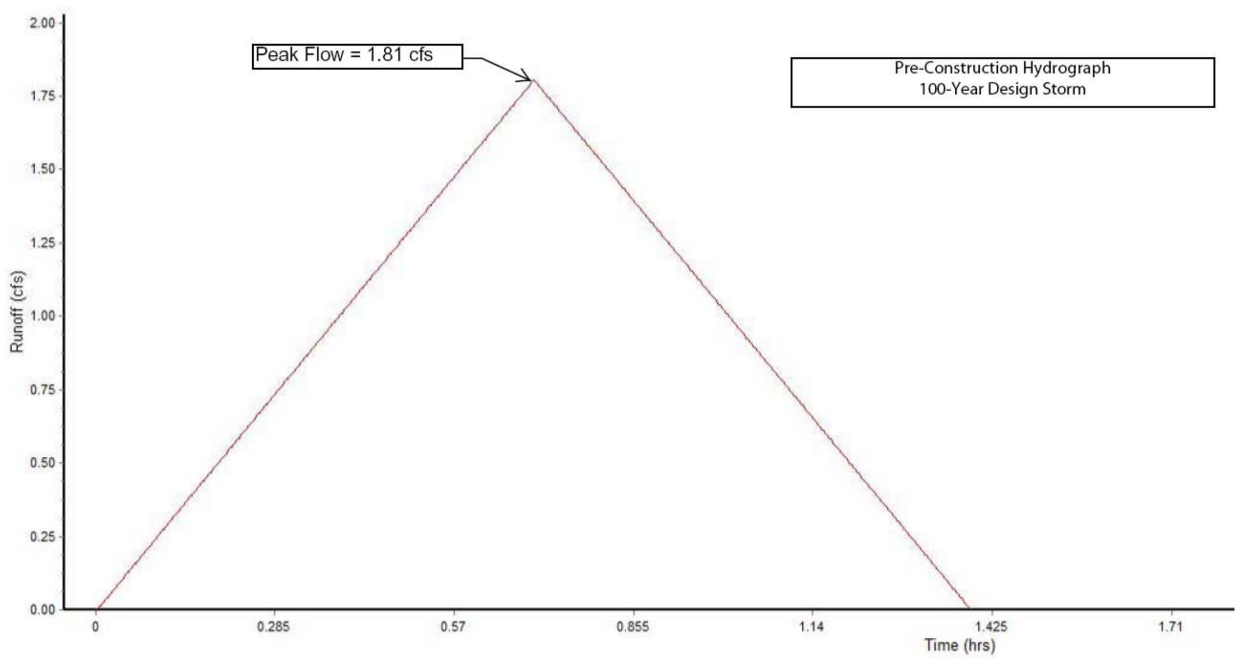

Figure 8: Pre-Construction Hydrograph, Rational Method

As noted above when discussing the SCS Method, the post-construction conditions add a winery building of approximately 10,560 square feet, a paved driveway, landscaping, and approximately 29,700 square feet of vineyard. The composite runoff coefficient for the post-construction conditions increases to 0.54, and the time of concentration lowers to 12.39 minutes. The peak storm water runoff rate increases to 4.19 cubic feet per second for the 10-year design storm and to 6.31 cubic feet per second for the 100-year design storm. While the runoff patterns for the storm water flow are altered into a combination of sheet flow, shallow concentrated flow, and conveyance through a proposed subsurface drainage system, the storm water runoff exiting the property will be similar to that of the pre-construction conditions.

See Table 6 for a detailed area analysis of the post-construction conditions, and see Figure 9 for the post-construction hydrograph for the 100-year design storm showing the required on-site storage.

Composite runoff coefficient

	Area (ac)	Area (%)	Runoff Coefficient	Soil Group	Description
1	0.41800	21.91	.9	B (2-6%)	Paved Driveway
2	0.17200	9.01	0.72	B (2-6%)	Landscape/Pavers
3	0.10600	5.56	.9	B (2-6%)	Covered Courtyard
4	0.16400	8.60	.9	B (2-6%)	Buildings
5	0.35794	18.76	.25	B (2-6%)	Open Space
6	0.69000	36.16	.3	B (2-6%)	Cultivated Land

Total area: 1.908 ac Total area: 100.00 % Weighted coeff: 0.54

Table 6: Post-Construction Input Variables, Rational Method

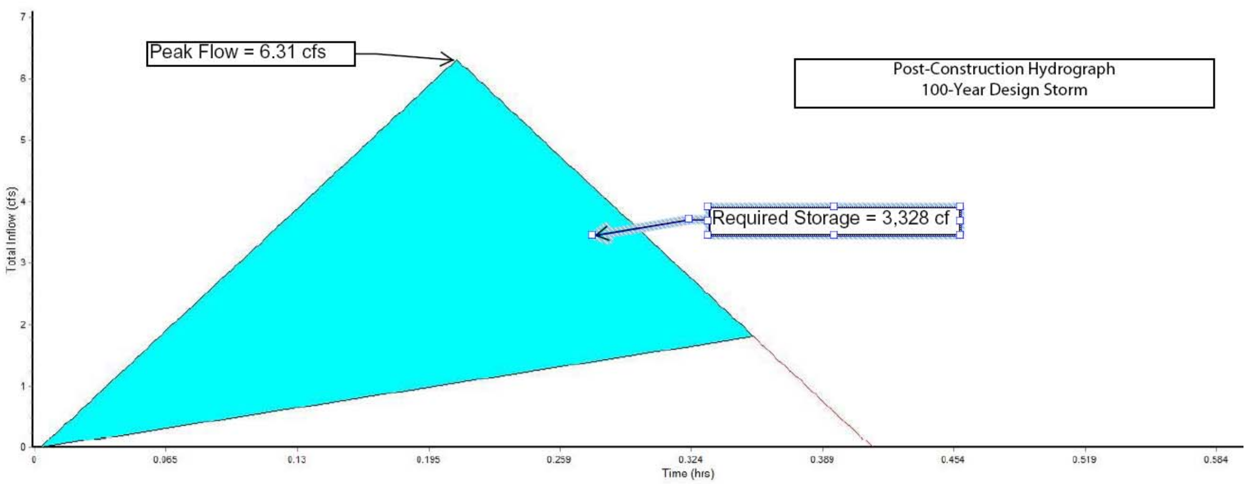

Figure 9: Post-Construction Hydrograph, Storage Determination for the 100-Year Design Storm

Based on this iteration of the Rational Method, the required on-site storm water detention is 3,328 cubic feet.

C. Input and Output Variables for the Pre- and Post-Construction Storm Water Detention Analysis

Each storm water detention method produced different results as a function of the base assumptions for each method and the inherent differences in the calculations. See Tables 7, 8, and 9 below for an overall analysis of each method.

Storm Water Runoff Values: 2-Year Design Storm									
	Tc_{PRE} (min)	Tc_{POST} (min)	CN_{PRE}	CN_{POST}	C_{PRE}	C_{POST}	Q_{PEAK_PRE} (cfs)	Q_{PEAK_POST} (cfs)	$V_{STORAGE\ REQ'D}$ (cf)
SCS Method	37.19	22.42	63.87	78.38	-	-	0.3	0.96	2,795

Table 7: Input and Output Variables for the 2-Year Design Storm Storm Detention Analysis

Storm Water Runoff Values: 10-Year Design Storm									
	Tc_{PRE} (min)	Tc_{POST} (min)	CN_{PRE}	CN_{POST}	C_{PRE}	C_{POST}	Q_{PEAK_PRE} (cfs)	Q_{PEAK_POST} (cfs)	$V_{STORAGE\ REQ'D}$ (cf)
SCS Method	37.19	22.42	63.87	78.38	-	-	0.93	1.94	2,253
Rational Method Using Santa Rosa Standards	15	7	-	-	0.4	0.55	1.34	3.24	920
Rational Method Using Site Specific Generated IDF Curve	41.61	12.39	-	-	0.3	0.54	1.13	4.19	2,245

Table 8: Input and Output Variables for the 10- Year Design Storm Detention Analysis

Storm Water Runoff Values: 100-Year Design Storm									
	Tc_{PRE} (min)	Tc_{POST} (min)	CN_{PRE}	CN_{POST}	C_{PRE}	C_{POST}	Q_{PEAK_PRE} (cfs)	Q_{PEAK_POST} (cfs)	$V_{STORAGE\ REQ'D}$ (cf)
SCS Method	37.19	22.42	63.87	78.38	-	-	2.02	3.37	2,417
Rational Method Using Santa Rosa Standards	15	7	-	-	0.4	0.55	1.87	5.11	1,520
Rational Method Using Site Specific Generated IDF Curve	41.61	12.39	-	-	0.3	0.54	1.81	6.31	3,328

Table 9: Input and Output Variables for the 100- Year Design Storm Detention Analysis

The post-construction watershed has a greater storm water runoff flow than the pre-construction watershed. The greatest difference was found using the Rational Method using site specific generated IDF Curves at 3,328 cubic feet. To mitigate for this increase in flow, the project site is required to store on-site 3,328 cubic feet of storm water runoff prior to releasing to reduce the post-construction flows to pre-construction conditions.

V. DRAINAGE BASIN ANALYSIS

The proposed Use Permit Plans dated December 28, 2011 proposes a 6,500 square foot subsurface detention basin, along with (2) 8,500 gallon above ground rain water tanks, to detain the required storm water runoff during a storm event. Rainwater derived on the building roofs will be conveyed into the proposed rain water tanks, which will be allowed to fill up to capacity prior to overflowing into a rocky swale. The majority of storm water runoff derived on the winery site and from the upstream watersheds will be conveyed to the proposed subsurface detention basin via a series of above grade swales and subsurface drainage pipes.

The property's surrounding area is known for having a high groundwater table during the winter. In order to determine the approximate groundwater table and determine the soil conditions on the property, the owners ordered bore holes to be drilled in May of 2011. The bore holes located the groundwater table at 6.5 feet below the ground surface. In addition, a well has been located approximately 1,500 feet away from the property with groundwater readings starting in the 1950's and continuing forward to the present day. Since 1950, the highest groundwater level recorded at the well has been at an elevation of 341 feet. The lowest point of the proposed drainage basin is at an elevation of 348.60, 7.6 feet above this reading. See Appendix F for the well location and the graphical interpretation of the groundwater elevations since the 1950s.

However, there still seems to be a concern that the groundwater at the property may be as high as the existing ground at the property after large rain events. As the owner is aware of this concern, he has directed the majority of the subsurface drainage basin to be located above the existing ground (but below the finish ground). In essence, the project proposes to bring in a large quantity of fill material to raise the buildings and surrounding area in order to keep the majority of the drainage basin above the existing ground.

In addition, water infiltration into the ground has not been taken into account in the analysis of the detention basin per the request of the City of Calistoga. Per the geotechnical report prepared by RGH Consultants dated June 8, 2011, the soil found within the property was brown sandy clay. This soil within the vicinity of the proposed detention basin was described as "soft to medium stiff, with abundant gravel, very porous, compressible, with abundant small gravel (topsoil). From this description, the gravel and sand content within the soil would allow for ample infiltration into the soil. This would reduce the off-site peak storm water runoff and the required size of the detention basin. Therefore, without the use of storm water infiltration within the detention basin, our analysis of the detention requirements and the post-construction flow are known to be conservative.

The drainage basin is to be located under parking areas and landscape areas on the property, and is to consist of 3 inch ballast rock. The geotechnical engineering firm RGH has performed analysis on this rock, and has determined the void ratio of the rock as 0.494.

As the entire drainage basin is not located above the existing grade, this report analyzes the storage capacity of the basin above the existing grade, along with the storage capacity below the existing grade. In the analysis, depths were determined for the finished grade, the base of proposed paving, the existing grade, and the rough grade (at the base of the proposed detention basin) around the extent of the detention basin. In addition, the rim elevation for an overflow drop inlet was taken into consideration as no water will be able to be stored above this elevation. For the points along the detention basin where the base of paving is above the overflow rim elevation, the overflow rim elevation was used in the calculations instead of the base of paving. From the outline of the detention basin, along with these grades, three-dimensional models were created using AutoCAD Civil 3D to determine the volume differences between the following:

1.) The base of proposed paving or overflow rim elevation and the existing ground (to determine storage above the existing ground)
2.) The existing ground and the rough grade (to determine storage below the existing ground).

Please see Appendix B for the analysis of the detention basin. The results from this analysis show the storage capacity of the detention basin above the existing grade is approximately 3,520 cubic feet. The storage capacity of the basin below the existing grade is approximately 4,700 cubic feet. The total storage capacity proposed in the detention basin is approximately 8,220 cubic feet. With the addition of 2,200 cubic feet of storage provided with the (2) 8,500 gallon tanks, the total storage capacity on the property is 10,420 cubic feet. The total storage capacity above the existing grade is approximately 5,720 cubic feet. This exceeds the required detention of 3,328 cubic feet 171%.

One necessity with all detention basins is the ability to maintain the system for the duration of its use. The proposed basin is proposed to be subsurface composed of ballast rock, limiting the maintenance opportunities of the system in the future. However, the designer did include maintenance features with the system to prevent future clogging of the rock content. Prior to entering the detention basin, storm water runoff is conveyed through proposed filter-fabric-wrapped Atlantis boxes surrounded by sand and rock media. Atlantis boxes are similar to old milk crates, which is to say they are permeable plastic boxes with void space in the center, and is to be installed to act as conveyance for the storm water once it passes through the sand and rock media. Manholes are proposed to be located above these boxes for visual inspection of the system. As the filter sand and rock around the Atlantis boxes becomes full of sediment and clogged, the owner will be able to remove and replace the filter media. At this time, the owner can inspect the filter fabric around the Atlantis box to determine and replace as necessary. This system will drastically reduce the solid pollutants entering the detention basin, and the upstream filter feature will be able to easily be inspected and cleaned.

VI. PEAK OFF-SITE STORM WATER RUNOFF FLOWS

As noted above, the post-construction storm water runoff flows are required to be less than the pre-construction storm water runoff flows when the water leaves the property. As the entirety of the proposed improvements will not be captured by the proposed detention basin, a secondary analysis was performed in order to determine the pre-construction and post-construction flows at the southerly corner of the property (the outlet location for the storm water runoff prior to and after construction of the proposed winery).

The pre-construction conditions were partitioned into two watersheds: the watershed above the property coming off of Mount Washington, named the 'Upstream Watershed', and the site watershed. As the post-construction conditions divert sections of the Upstream Watershed, the Upstream Watershed is partitioned into three sections in the post-construction analysis: Upstream Watershed 1, Upstream Watershed 2, and Upstream Watershed 3. In addition, the site is partitioned into two watersheds, or flow-producing areas: the detention basin outflow (which takes into account all the area on the property where the storm water runoff will be conveyed into the detention basin), and the Site Watershed Minus the Detention Basin Watershed. See Appendix G for a post-construction runoff map locating the areas on the site which will be directed into the proposed detention basin.

The original set of plans prepared by James Cassayre reviewed by Delta Consulting & Engineering was dated December 28, 2011. In performing the hydrologic analysis of these plans, it was determined the 10-year and 100-year post-construction flows were lower than the pre-construction flows at the southern corner of the property. However, the 2-year post-construction flows exceeded the pre-construction flows. This was due to two main factors: 1.) the outflow from the detention basin was constant, and 2.) the storm water runoff derived from impervious areas not conveyed into the proposed detention basin was causing a severe decrease in the time of concentration for the site watershed.

To alleviate these factors and reduce the post-construction flows from the property, Delta Consulting & Engineering has been in contact with and prepared recommendations for the owner regarding these issues. The owner was understanding of the situation, and agreed to revise the use permit plans to make the situation right. The recommendations included the following:

1.) Install a hydrologic metering box at the outlet of the detention basin to allow unique flows from the detention basin for the 2-year, 10-year, and 100-year design storms
2.) Regrade the proposed driveway towards Silverado Trail, and install an at-grade swale for the runoff to enter into and sheet flow from.

The metering box will allow variation in the flow rate of the water exiting the detention basin. When the 2-year design storm occurs, the water will be released at a slower flow rate than when the 10- or 100-year design storms occur. See Appendix H for the detail and calculations of the proposed metering box. By regrading the proposed driveway and adding the at-grade swale, the owner has greatly increased the time of concentration for the site watershed. This reduces the peak flow from the site watershed by spacing out the flow over a longer period of time.

These improvements have been included into the Use Permit Plans prepared by James Cassayre dated January 18, 2012. See Appendix E for an 11"x17" not-to-scale copy of these plans.

To determine the pre- and post-construction peak storm water runoff flows at the southern corner of the property, an analysis was performed using hydrographs derived from the above noted pre-construction and post-construction watersheds. The peak flow was determined based on a composite time of concentration where each of these watersheds combined to create the highest flow rate from the property. Typically, the peak flows for the post-construction watersheds occurred at an earlier time that the peak flows derived from the pre-construction watersheds. The analysis and results for the pre- and post-construction peak flows at the southern corner of the property can be viewed in Appendix I. A succinct overview of the results is located in Table 10 below:

Storm Water Runoff Flows at Southern Point of Property

	Peak Flowrate (cfs)	Reduction From Pre-Construction Flows (cfs)
$Q_{PRE,\ 100\ YEAR}$ =	4.70	0.17
$Q_{POST,\ 100\ YEAR}$ =	4.53	
$Q_{PRE,\ 10\ YEAR}$ =	2.46	0.24
$Q_{POST,\ 10\ YEAR}$ =	2.22	
$Q_{PRE,\ 2\ YEAR}$ =	1.04	0.04
$Q_{POST,\ 2\ YEAR}$ =	1.00	

Table 10: Peak Pre-Construction and Post-Construction Flow Rates

Based on the Use Permit Plans dated January 18, 2012, along with the revisions shown in Appendix E to be addressed in the construction documents, the proposed winery will not cause an increase in peak storm water flows at the southern corner of the property.

VII. STORM WATER RUNOFF FLOW ALONG SILVERADO TRAIL

The Use Permit Plans for the winery proposed to capture runoff generated from Upstream Watershed 2 and outlet the flow through a bubble-up near Silverado Trail. The City of Calistoga requested this analysis look into the affect on the existing swale along Silverado Trail and determine the effects, if any, to the existing flow patterns.

While there is a very slight 'swale' along the frontage of this property, the capacity of this swale is minimal. Silverado trail sits approximately 2.5 feet above the property, with an approximate 8 foot swale running adjacent and parallel to the road. However the lip of the swale on the property side is approximately 1 inch to 2 inches above the flow line of the swale. The swale comes to a conclusion at the existing private drive extending perpendicular to Silverado Trail. The apparent original design was for this water to continue through an existing culvert installed below the existing drive which would continue the water down Silverado Trail. However, the culvert is at a higher elevation than the swale, and the water naturally emerges over the lip of the swale and crosses the private drive without ever entering the swale. This runoff is blocked by an existing berm located along the private drive, and conveyed towards the existing winery beyond the southern point of the property. See Appendix B for photographs of the existing swale along Silverado Trail and the natural flow patterns of storm water runoff.

Currently, a portion of Watershed 1 enters into the swale along Silverado Trail. With the improvements noted in the Use Permit Plans, Watershed 2 will also be conveyed into the swale. The increase in flow will not increase any runoff that may enter the existing culvert, nor will it overflow onto Silverado Trail. The increase in flow will cause the storm water to overflow the property-side of the swale at a location between the midpoint of the frontage and the private drive. Once outside of the swale, the storm water runoff will sheet flow through the proposed vineyards towards the private drive.

VIII. CONCLUSION

Based on Delta Consulting & Engineering's analysis of the proposed site improvements per the Use Permit Plans prepared by James Cassayre dated January 18, 2012, the flow patterns of the storm water runoff will be maintained to pre-construction conditions. The runoff will exit the property at the southern corner of the property similar to the pre-construction conditions. In addition, the detention basin and other hydrologic energy reducing features proposed in the plans will reduce the post-construction flows to be less than the pre-construction conditions.