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TO:	Planning Commission	Agenda of:	July 24, 2025
FROM:	Evan Mattes, Senior Planner	Item No.:	2
RE:	CUP22-0013/Black Oak Mountain Winery	Legistar No.:	25-1239

The purpose of this memorandum is to provide the project Traffic Impact Study as additional documentation pursuant to public request.

Attachment: Transportation Impact Study

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MEMORANDUM

DATE: October 30, 2024

TO: Zach Oates | El Dorado County

FROM: Josh Pilachowski | DKS Associates

SUBJECT: Third Peer Review of the Black Oak Mountain Vineyards TIS, Cool, CA - DRAFT   Project #21197-020

DKS did an initial review of the Transportation Impact Study (TIS) for Black Oak Mountain Vineyards prepared by Prism Engineering in May 2024. This memorandum summarizes the third round of review of the revised study. Peer reviews are used to ensure that traffic impact studies are prepared in accordance with the standards of care, best practices, and established conventions and procedures typically used in the traffic engineering profession.

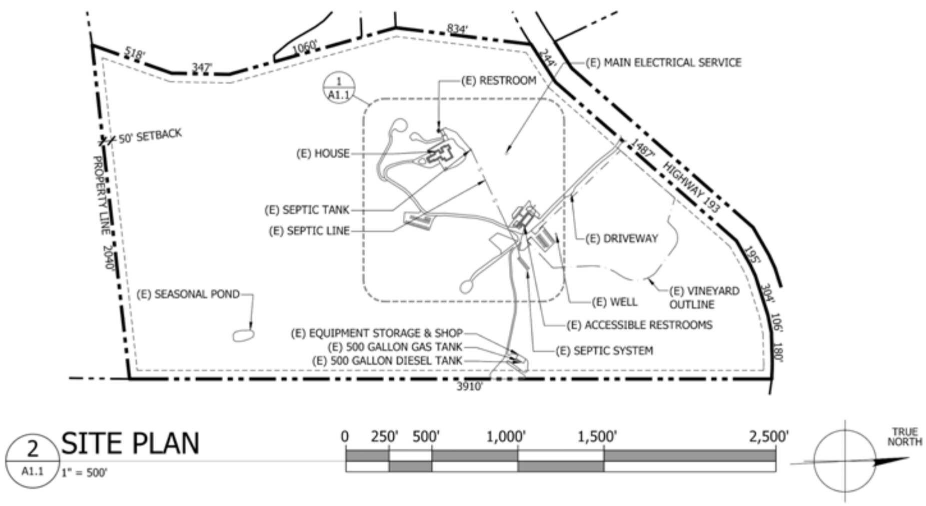
SCOPE OF REVIEW

DKS Associates has conducted a second round of peer review of the PDF document titled *Final Draft Transportation Impact Study for Black Oak Mountain Vineyards, Cool, CA*, prepared by PRISM Engineering and dated October 15, 2024. This is the second review of this document. Previous versions were dated May 17, 2024 and August 30, 2024.

REVIEW SCOPE

The document was reviewed for content and compliance. All previous comments were sufficiently addressed, and it is recommended that this study be approved.

FINAL TRANSPORTATION IMPACT STUDY
For APN: 074042002, in the County of El Dorado
2480 State Hwy 193, Cool, CA



Prepared for Black Oak Mountain Vineyards
2480 State Hwy 193, Cool, CA
APN: 074042002, in the County of El Dorado

October 15, 2024

This FINAL Report Prepared by:	*Prepared in accordance with study guidelines set forth by: El Dorado County DOT*
Traffic Engineering & Transportation Planning	***This report has been prepared and certified by Grant P. Johnson, TE, Principal. CA License #1453***

I. EXECUTIVE SUMMARY

This report analyzed existing and long-term cumulative scenarios for the proposed *Special Use Permit CUP, 2480 State Hwy 193, Cool, CA*. This study was prepared according to the scope of work approved by County DOT[1], and details prepared in accordance with the County's "Transportation Impact Study Guidelines[2]" document. This study is slightly different than most in that each of the study intersections is on a Caltrans Highway facility. We chose to supplement the existing and future long-term cumulative scenarios instructed by DOT as follows by adding more scenarios to cover the time periods that include the project's wedding venue traffic:

1. PROPOSED SCOPE
    a. Year 2024 Existing Conditions for both the AM and PM Peak Hour
    b. Year 2024 Existing plus Project Conditions, AM, PM Peak Hour
    c. Year 2044 Cumulative Conditions, AM, PM Peak Hour
    d. Year 2044 Cumulative Conditions plus Project, AM, PM Peak Hour

2. ADDITIONAL ANALYSIS
    a. Year 2024 Estimated Afternoon 3:00-4:00 pm Conditions
    b. Year 2024 Estimated Afternoon 3:00-4:00 pm Plus Project Conditions
    c. Year 2024 Estimated Late Evening 10:00-11:00 pm Conditions
    d. Year 2024 Estimated Late Evening 10:00-11:00 Plus Project Conditions
    e. Year 2044 Estimated Afternoon 3:00-4:00 pm Conditions
    f. Year 2044 Estimated Afternoon 3:00-4:00 pm Plus Project Conditions
    g. Year 2044 Estimated Late Evening 10:00-11:00 pm Conditions
    h. Year 2044 Estimated Late Evening 10:00-11:00 Plus Project Conditions

The conclusions developed in this study indicate that no mitigations are required of the project for any of the twelve analysis scenarios.

The project does not have any VMT impacts, and in fact offers VMT reducing benefits for wedding venue traffic already on the road for numerous wineries and wedding venues, being centrally located between two east west freeway corridors (I-80 and US 50).

The details of all analysis results are outlined in the body of this report.

[1] Communication by email from Zach Oates, Senior Civil Engineer, El Dorado County DOT April 5, 2024.
[2] *Transportation Impact Study Guidelines,* El Dorado County Community Development Agency, November 2014.

INTRODUCTION

This report documents the results of a transportation impact study (includes existing and cumulative analysis scenarios) completed for the *Special Use Permit CUP, 2480 State Hwy 193, Cool, CA*. The project is to take place on an existing business site, the Black Oak Mountain Vineyards site located at 2480 State Highway 193 in Cool, CA. The owner of this existing business wants to use the existing site and facilities to accommodate larger groups of people for various activities such as weddings, etc. The location of this existing business site has direct driveway access to the south side of SR 193 at a point 3.5 miles east of the intersection of SR 49 and SR 193. The project site has a gated entrance with a 60' throat length from the gate to the painted south edge line of SR 193. Internal to the project site there are many paved roads and parking areas which will allow for the easy turning around of a large emergency vehicle such as a fire truck.

This study was prepared according to the scope of work approved by County DOT[3], and details prepared in accordance with the County's "Transportation Impact Study Guidelines[4]" document.

This report evaluates the trip generation and resulting traffic impacts from the project site within the context of existing and future long-term cumulative scenarios as instructed by DOT as follows:

3. Year 2024 Existing Conditions for both the AM and PM Peak Hour
4. Year 2024 Existing plus Project Conditions, AM, PM Peak Hour
5. Year 2044 Cumulative Conditions, AM, PM Peak Hour
6. Year 2044 Cumulative Conditions plus Project, AM, PM Peak Hour

PROJECT DESCRIPTION

A special use permit CUP request to allow the use of the project area, an existing winery facility (permit number 0334381) and two remote ceremony sites for up to 150 special events per year, including, but not limited to live music, charitable events, weddings, etc. The zoning code maximum number of attendees for special events is 250 persons. The special events as proposed would include events which ordinarily include 150 persons maximum. No increase in the allowance of attendees is being proposed. All outdoor operations will cease by 10 PM. Events would primarily take place on Thursday through Sunday, with occasional midweek events. Events will primarily take place during the months of March through November, with a majority of events taking place in April through October.

The Black Oak Mountain Vineyards winery includes one parcel totaling 146.52 acres. The 146.52-acre parcel contains one single-family residence, 2400 square-foot winery building and production facilities (winery, tasting room, storage building and event center), approximately

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[3] Communication by email from Zach Oates, Senior Civil Engineer, El Dorado County DOT April 5, 2024.

[4] *Transportation Impact Study Guidelines,* El Dorado County Community Development Agency, November 2014.

4200 square-foot outdoor assembly area, two remote ceremony sites or outdoor use areas, and an outdoor restroom facility) permit number 037-0194). This parcel also contains over 5 acres of planted vineyards properly maintained for commercial crop production.

The existing uses and structures are allowed by winery ordinance and the AE/ PA zone for the 146.52-acre parcel and include charitable events, corporate events, art shows, live music, meetings, and mixers. The special use permit is required because the applicant seeks to hold more special events and facility rentals than are allowed by right of the winery ordinance for parcels within the zone. Specifically, wedding events are the focus of obtaining the Special use Permit. Figure 1 is a site plan in the context of the local vicinity including SR 193. It shows the location of the project site existing driveway and how it connects to SR 193, as well as all other internal existing paved roads and parking areas. Table 1 shows the project parcel and APN information. Figure 2 is a more detailed site plan.

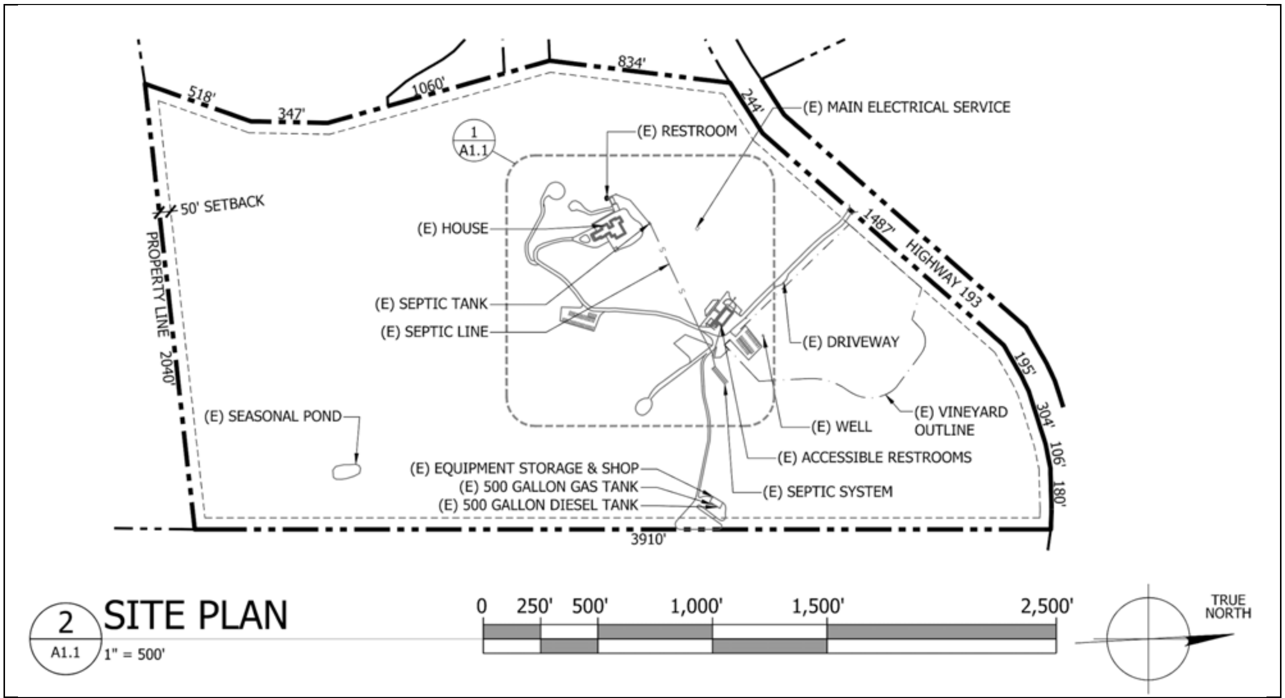


Figure 1. Site Plan and Local Vicinity SR 193 Highway

Table 1. Project Parcels and APN Information

Assessor's Parcel Number	Location
APN 074042002, 146.52 acres. Contains one SFR, 2400 square-foot winery building, 4200 square-foot outdoor assembly area, two remote ceremony sites, outdoor restroom facility, 5 acres of planted vineyards	This property is located 3.5 miles to the east of SR 49 and the intersection of SR 193. The project driveway is located just over ¼ mile east of Brush and Rocks Lane on SR 193.

Figure 2. Detailed and Enlarged Site Plan

Figure 3 shows an aerial photo of the venue site with various elements of the site plan labeled and corresponding to Table 1 above.

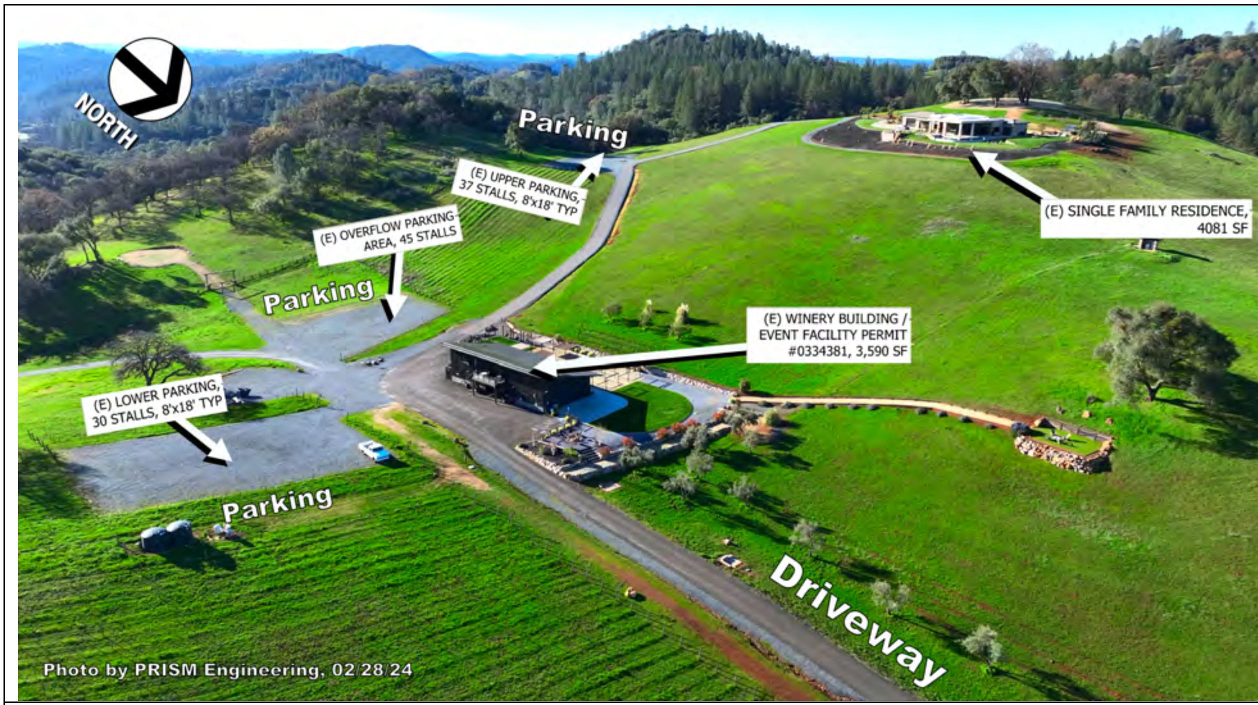


Figure 3. Venue Site Aerial Photo and Plan Details

TRIP GENERATION

The project venue has been described as being able to accommodate special events such as weddings, corporate events, charitable events, live music, etc. Since the enclosed physical building facilities on the project site are finite in size (2400 square foot winery building and a 4200 square foot outdoor assembly area) and parking is limited, the venue's capacity is generally limited to the size of the wedding or other event (the number of people possible). A reasonable assumption is to estimate that if a wedding is taking place at the Black Oak Mountain venue, then the lower parking lot nearest to the winery building and adjacent outdoor assembly area will be used (up to 30 stalls used), with the potential to use the overflow area nearby (up to 45 more stalls used), for a total of 75 spaces. The venue's parking capacity will be the typical size used for trip generation. The total number of parking spaces available on the site (many are located for convenience and not for a simultaneous use) is 112 available spaces:

- 30 vehicles in the first parking lot area with gravel surface measuring 125'x85'
- 45 vehicles in the second parking lot area with gravel surface measuring 125'x100'
- 37 vehicles in the third parking lot area with gravel surface measuring 170'x80'

For instance, If the wedding is to take place up on the hill near the upper parking lot which has only room for 37 stalls, then wedding participants will park nearest to the remote wedding location (out of convenience) in the outdoor nature setting in the upper parking lot. It is reasonable that a maximum number of vehicles present for any of the larger wedding events would be 75 cars (the sum of lower parking lot cars parked plus overflow parking lot cars parked). These 75 cars would be transporting the maximum 150 people scenario, having an average vehicle occupancy of 2 people per car.

Since the ITE Trip Generation Manual 11th Edition does not have a trip generation rate for special events let alone wedding events (because they are all different and specific to venue), PRISM Engineering utilized the proposed Black Oak Mountain project's Special Use Permit Operations plan and description to come up with this set of traffic assumptions that closely correlate with proposed use of the facilities, the amount of parking available, as well as the size of the venue facilities. One of the most important factors that we used to develop a trip generation rate was relying on the number of attendees for special events which the project proponent said would ordinarily be a maximum of 150 persons. They have stated that there will be no increase in the allowance of 150 persons in attendance. In addition, the operations statement said that all outdoor operations would cease by 10:00 p.m. (due to laws and ordinances relating to noise), and that the vast majority of events would be taking place between the months of March and November. The majority of the events would take place in April through October. The site already has an existing Winery facility permitted for this existing use (# 0334381). The site has options for two areas on the site which could serve as remote ceremony sites or other types of events (up to 150 special events per year) including live music, charitable events, etc. The County's Zoning Code for special events allows for a maximum number of 250 attendees, but as stated previously only a maximum of 150 persons at an event is being proposed for this project.

Since the project operations statement says that all outdoor operations would cease by 10:00 pm, an arrival time for the venue such as a wedding event can be determined from working backwards from that time using assumptions from a typical wedding program as follows:

- 4:00 pm: Wedding Ceremony begins
- 5:00 pm: Cocktail hour begins
- 6:00 pm: Dinner begins (two to three courses plus speeches)
- 8:00 pm: Dancing begins as does the Sunset Hour
- 8:30 pm: Sunset, Photos, etc.
- 10:00 pm: End of all outdoor activities (however, some indoor activities may continue, but generally this is the cue for the event to be concluded and participants will begin to drive off the site to enter SR 193 and go home)

Because a wedding is a single event that takes place at a pre-established time (assumed to be starting around 4:00 p.m. given the 10:00 p.m. ending time constraint). Because the program or ceremony starts at 4:00 p.m. it is assumed that all of the wedding party and wedding guests will be in attendance at the ceremony and seated, ready to go at 4:00 p.m. Therefore, the trip generation rate will be based on the number of people expected to be in attendance based on the project operation statement of 150 people. This is the capacity, the maximum number of people that would be present at an event, one of the potential of 150 events expected to take place over the course of the year (between March and November). Although there will be many events that will be much smaller than having 150 people, for the sake of being conservative in this traffic study we are assuming the need to plan for traffic that would involve the arrival of 150 people. The assumptions are laid out clearly in Table 2.

Table 2. Vehicle Trip Generation Summary of Project
(Based on Peak Times of Generator *taking place over a 4-hour time period*)

Trip Generation Calculation for a Special Occasion Venue such as an Outdoor Wedding, Corporate Event, Live Music, etc	HOURLY PEAK VEHICLE TRIP RATE per Person in Attendance	NUMBER of VEHICLE TRIPS in PEAK HOUR of VENUE, 100% Inbound	AFTERNOON and EVENING PEAK TRAFFIC				TOTAL TRIP ENDS
			TRIPS IN 3-4 PM	TRIPS OUT 9-10 PM	TRIPS OUT 10-11 PM	TRIPS OUT after 11	
Peak Hour of Generator IN (peak time assumed to be 3-4 PM for a Wedding)	0.5	75	75	0	0	0	75 in
Peak Traffic Going OUT (peak time assumed to be 10-11 PM for a Wedding)	0.25	37	0	20	37	18	75 out
							150 total

LEGEND
Inbound Project Traffic Volumes
Outbound Project Traffic Volumes
Not Applicable For hours Specified in Column Heading

Source: PRISM Engineering and Black Oak Mountain

Since the project description stipulates that the time for ending all outdoor activities is 10:00 p.m., It can be determined by reviewing the typical schedule above that there would need to be an arrival time sometime before 4:00 p.m., since the typical wedding ceremony would be beginning around that time. For the sake of this traffic study, we assumed based on well-established wedding schedule goals to choose a time that is an hour before sunset for photography purposes but could be sooner if the venue has a time constraint (as is the case in this project having a stated 10:00 pm cut off time for all outdoor activities). It was assumed that the guests would arrive sometime between 3:00 p.m. and 3:30 p.m. . The wedding guests usually travel together as couples, families or groups of friends. We assumed that there will be about 2 guests per vehicle (conservative), even though it will probably be higher. This means that 150 guests would arrive in about 75 vehicles for a large event with about 10% (wedding party and vendors) arriving much earlier than 3:00 pm to prepare all of the arrangements, etc. Ultimately what this means is that the venue's traffic will not be arriving during the typical pm peak hour such as 5:30 p.m. to 6:30 p.m. Also, since the event would typically be ending at 10:00 p.m. or no later than 10:00 p.m., the exiting traffic will not be entering the SR 193 highway during the typical busy peak hour (5:30 p.m. to 6:30 p.m.). The result of this is that the project will have very little direct critical impact to the adjacent road system and intersections except at times of the day when traffic volumes are much lower. Table 2 is the trip generation summary table developed with these various assumptions of the larger wedding venues in mind.

Using the car occupancy factor of two people per car gives a peak hour of generator trip rate (taking place around 3:00 pm to 4:00 pm) of 0.5 inbound trips per person in attendance (assuming a worst case of 100% entering during that hour and depicted in the red shaded cells of Table 2). Also, since the venue includes various reception areas during the course of the evening, the exiting traffic will be dispersed over a few hours and will not have as large of an impact in any one hour. This is shown in the blue shaded cells of Table 2.

It is important to note that although the vast majority of wedding venue traffic would miss the critical peak hours of traffic on the adjacent roadway system, there are other smaller events which could have traffic coinciding with the peak hours of adjacent street traffic, such as a corporate events, art shows, meetings and mixers which would be held at the existing winery facility and would not be subject to the traffic impact study requirement (since it is an already approved and permitted use). The purpose of this traffic study was to examine the newer and proposed larger events such as weddings which draw a much larger crowd of up to 150 people.

Since the project's larger events will be taking place during off peak time periods, PRISM Engineering utilized an El Dorado County traffic count for a nearby roadway in the study area, Black Oak Mine Road which has hourly variances in traffic volume that will give some idea of the magnitude of traffic in the late evening such as when an event will conclude.

TRIP DISTRIBUTION

The primary purpose of this traffic study is to examine the effect of wedding venue traffic which is the type of "special event" that is being considered above and beyond what other events or activities are already taking place at the site and already have approvals with existing zoning and need no traffic study. This trip generation and trip distribution analysis is for the Special Use Permit CUP only and specifically refers to small wedding events that are envisioned to take place in the future (up to 150 people maximum which includes guests). It is literally unknowable which direction these attendees will travel from to come to an event at this project location. For this reason, PRISM Engineering assumed that guests to a wedding would not all be local but actually super-regional in nature as many family and relatives will come to such events as well and could travel by plane or car and so consideration was given to areas far from the project site (such as would be accessed to and from the I-80 freeway, or the US 50 freeway, etc.). In order to provide some basis for this otherwise unknowable quantity of where the wedding guests will actually come from, since all families are different and live in close proximity, or very far away locations, PRISM Engineering utilized the existing traffic patterns or magnitudes of traffic volume on the local SR 193 and SR 49 highways to assume a basis whereby the project traffic could be assigned based on the majority travel patterns. It is also not possible for the County's travel demand model to make such a determination because this specific "wedding land use" is not in the model, nor are the formulas for productions and attractions in the model useful for such a special event land use relevant in any scientific way. For this reason, the trip distribution was developed manually in this study based on engineering judgment, and by examining existing volumes on the highways and adjoining roadways and intersections that may serve the project traffic to and from the west and to and from the east on SR 193 as shown in Table 3 and Table 4 below (trip distribution for 3-4 PM and for 10-11 PM, respectively).

The trip distribution assumptions for the immediate vicinity of the project and defined in Tables 3 and 4 were also developed with the aid of the Google Map Directions tool which gives estimated travel times. The details of our findings using this Google Map Directions Tool are summarized in Table 5 which shows the specific estimated travel times and distances for a trip to and from the project site driveway and the connection to the US 50 freeway at Placerville, the only location that continues to provide access to the regions round about this corridor via SR 49 to the south of Placerville (such as Jackson, Angels Camp, etc.). For example, for guests who would be coming from the US 50 corridor and all of the associated regions in that area, there are multiple pathways that they could travel. One pathway would be to take SR 193 from Placerville to and from the project site as one connecting path, and another would be by taking SR 49. As shown in Table 5, both pathways have very similar travel times, and the distance on the SR 193 path is actually 3 miles shorter which would give some advantage on mileage. Whether a driver would choose on or the other is actually a matter of personal preference and does not have a significant advantage either way. For this reason, PRISM Engineering generally split the difference onto both pathways since they are equivalent, and this is reflected in Tables 3 and 4, as one can see that there is 22% of the project traffic taking the SR 49 path (which is 3 miles longer to Placerville and more out of the way) and 25% taking the SR 193 path to and from Placerville (which is shorter and has similar travel times).

The overall trip distribution pattern for project traffic on SR 193 was to assign 25% to the east and 75% to the west towards SR 49, and the majority of that westbound traffic was assigned to the north on SR 49 (53% of the total project traffic) since the population centers are greatest to the north and west compared to the local project area. Also, this SR 193 split is more conservative in analyzing the project traffic impacts since any inbound project traffic at 3pm coming the project site from SR 193 east would have to make a left turn into the project site and since there is no left turn pocket there at the entrance gate, there is the potential to block SR 193 traffic if eastbound traffic does not have a gap in traffic. This is an existing condition and the project winery site already has approvals and does not require a left turn pocket. Our analysis of this turn move at Intersection #3 in all of the scenario analyses found it not to be a problem even for special event traffic levels studied in this report.

Table 3. Trip Distribution of Peak Project Arrival Traffic From 3 PM to 4 PM

		INBOUND TRAFFIC		
		GENERATOR VPH @ 3PM		
		TOTAL	IN	OUT
Assigned Location	**%**	**75**	**75**	**0**
SR 193 Westerly	**75%**	56 vph	56 vph	0 vph
SR 49 Northerly	***53%***	40 vph	40 vph	0 vph
SR 49 Southerly	***22%***	17 vph	17 vph	0 vph
SR 193 Easterly	**25%**	19 vph	19 vph	0 vph

Source: Caltrans and PRISM Engineering Traffic Counts and Freeway Destinations

Table 4. Trip Distribution of Peak Project Departure Traffic From 10 PM to 11 PM

		OUTBOUND TRAFFIC		
		GENERATOR VPH @ 10PM		
		TOTAL	IN	OUT
Assigned Location	**%**	**37**	**0**	**37**
SR 193 Westerly	**75%**	28 vph	0 vph	28 vph
SR 49 Northerly	***53%***	20 vph	0 vph	20 vph
SR 49 Southerly	***22%***	8 vph	0 vph	8 vph
SR 193 Easterly	**25%**	9 vph	0 vph	9 vph

Source: Caltrans and PRISM Engineering Traffic Counts and Freeway Destinations

As seen in Table 3 the project arrival traffic volumes assigned are a total of 75 inbound vehicles between the hours of 3 PM and 4 PM. These are off-peak volumes and will have a lesser overall impact to the surrounding road system due to the lower volumes that will exist on adjacent streets and highways outside of the 5:15 PM to 6:15 PM peak hour (such as during the 3 to 4 PM peak hour of the generator's inbound traffic, or during the 10 to 11 PM peak hour of the generator's outbound traffic). The outbound project traffic trip generation and assignment is shown in Table 4. The trip distribution for both the arrival and departure scenarios is also shown graphically in Figure 4. Part of the basis for these assumptions were informed by the Google travel times and choices summarized in Table 5. Figure 5 shows the Study Area with its four (4) study intersections with the existing lane configurations and traffic control shown. Intersection #3 is the existing Black Oak Mountain Winery property driveway.

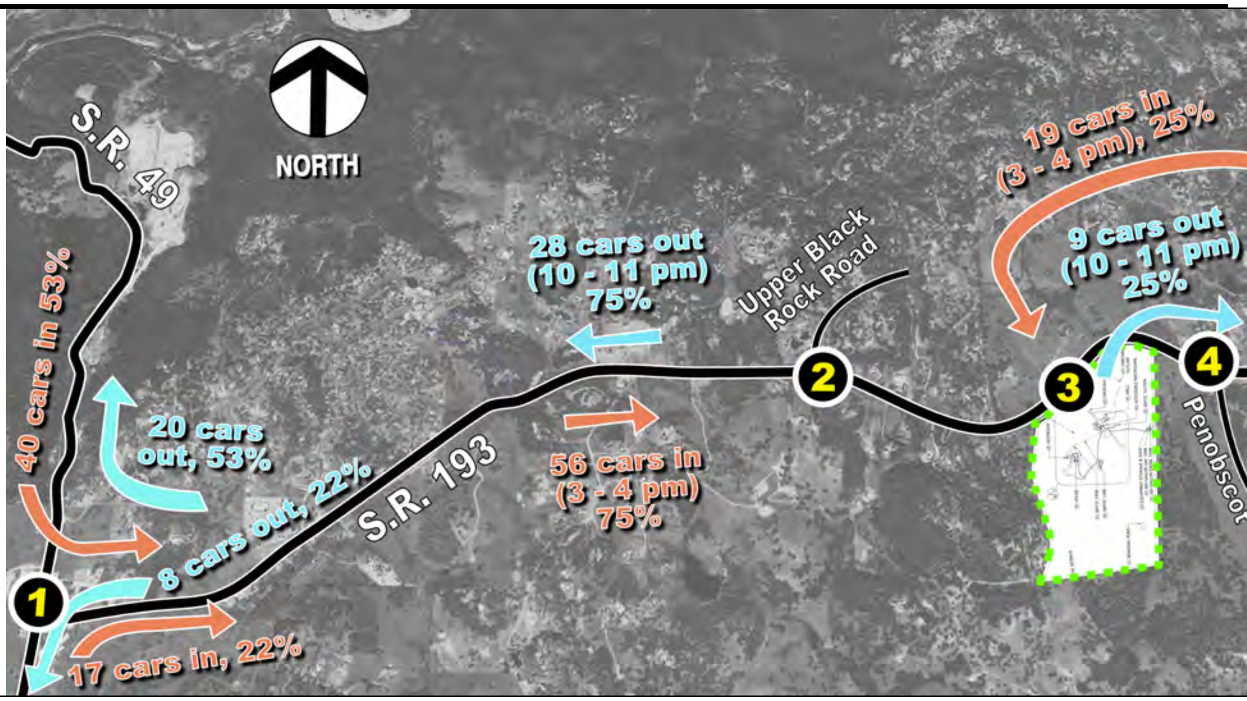


Figure 4. Project Trip Distribution of Project: IN at 3-4PM and OUT at 10-11PM

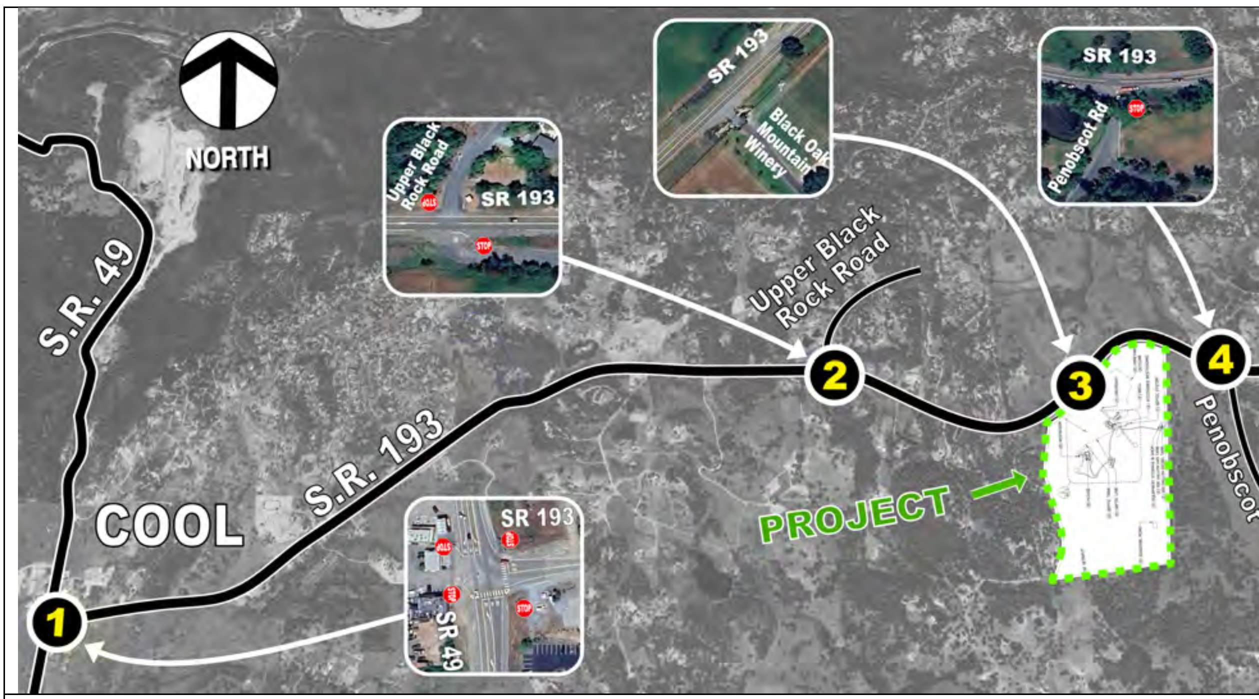
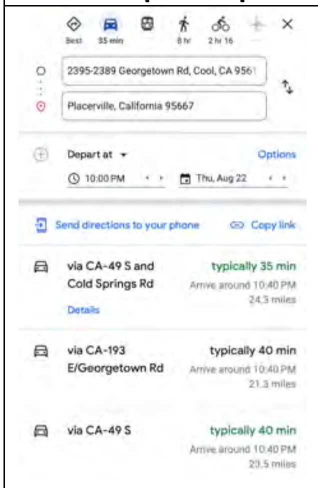


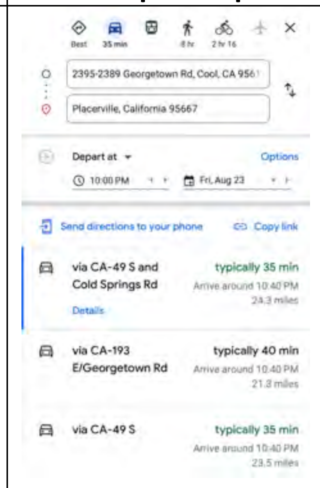
Figure 5. Study Intersections: Traffic Control and Lane Configurations

Table 5. Google Map Assist on Travel Times by Departure and Arrival Times on Thursday through Sunday To and From Placerville and Project Location

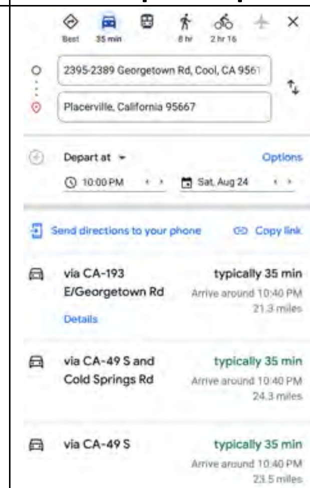
THU Depart 10pm



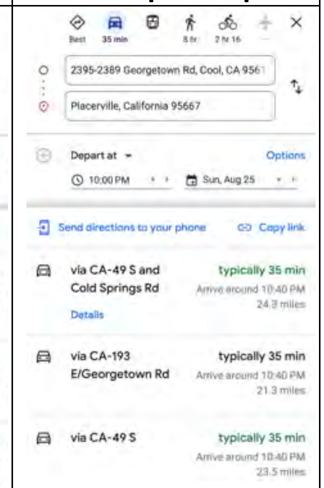
FRI Depart 10pm



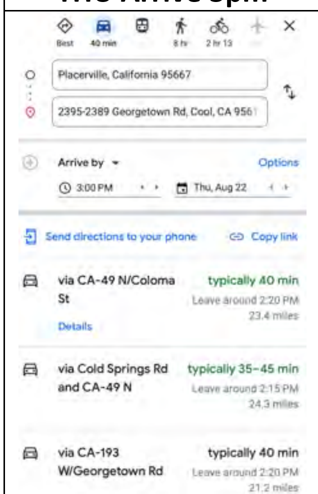
SAT Depart 10pm



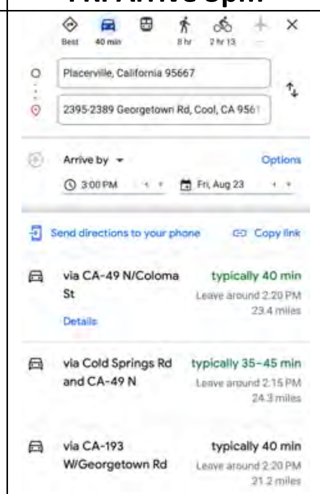
SUN Depart 10pm



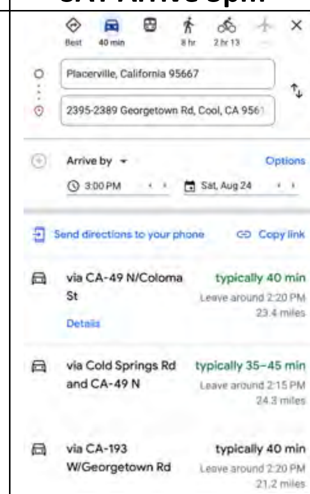
THU Arrive 3pm



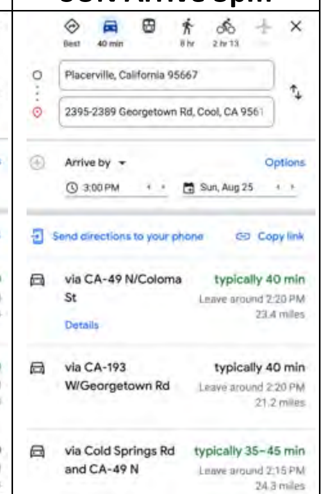
FRI Arrive 3pm



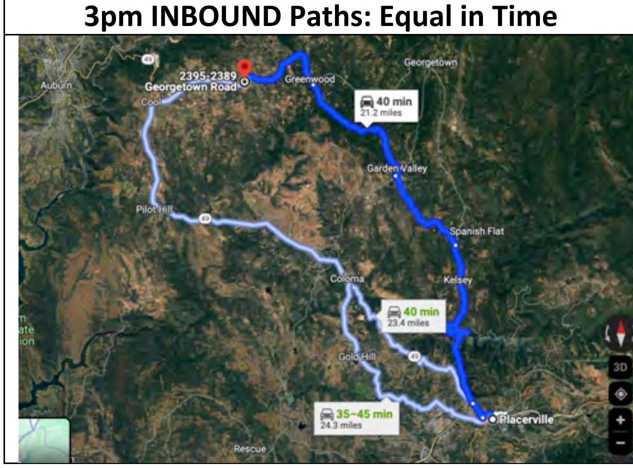
SAT Arrive 3pm



SUN Arrive 3pm



3pm INBOUND Paths: Equal in Time



10pm OUTBOUND Paths: Equal in Time

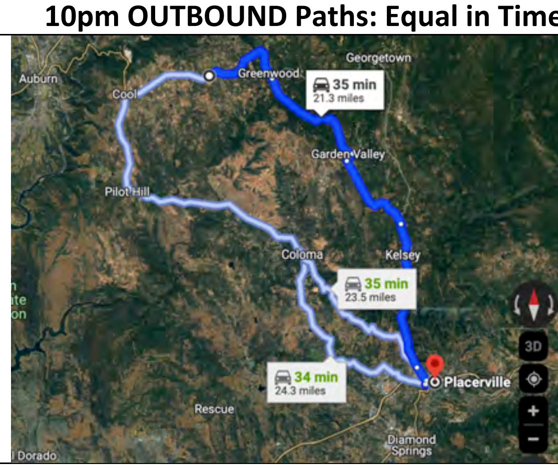


Figure 5 shows the location of each of the 4 study intersections, as well as the aerial view of the lane configurations and specific type of traffic control being used at each intersection (all are some form of stop sign control as shown). PRISM Engineering verified all intersection configurations shown, including all signing and striping with a detailed video drive-thru of each intersection and roadway in the study area. We also utilized extensive drone video to not only count traffic but verify lane striping and intersection configurations as installed on February 28, 2044. The 4 study intersections are as follows:

1. SR 193 and SR 49
2. SR 193 and Black Rock Lane / Upper Black Rock Road
3. SR 193 and Project Site Driveway (Black Oak Mountain Winery, existing)
4. SR 193 and Penobscot Road

The project has an existing gated connection to the SR 193 Highway which currently serves existing site uses (residence, permitted winery, and 5-acre grape orchard). This is identified at Intersection No. 3 in this study. SR 193 connects to SR 49 on the west and traverses to the east past the project site through Georgetown and beyond until it terminates with its intersection with SR 49 in Placerville, CA.

Project Volumes for Peak Hour of Generator

The following two figures have been prepared to visually clarify the Project Only Traffic that will travel through the study intersections at two different times: Inbound project traffic from 3:00 pm to 4:00 pm, and Outbound project traffic from 10:00 pm to 11:00 pm. These hours do not coincide with the peak hours of the normal street traffic typically called the "adjacent street traffic peak hour." Figures 6 and 7 show the proposed project traffic only that will travel through the study intersections during these two time periods, respectively.

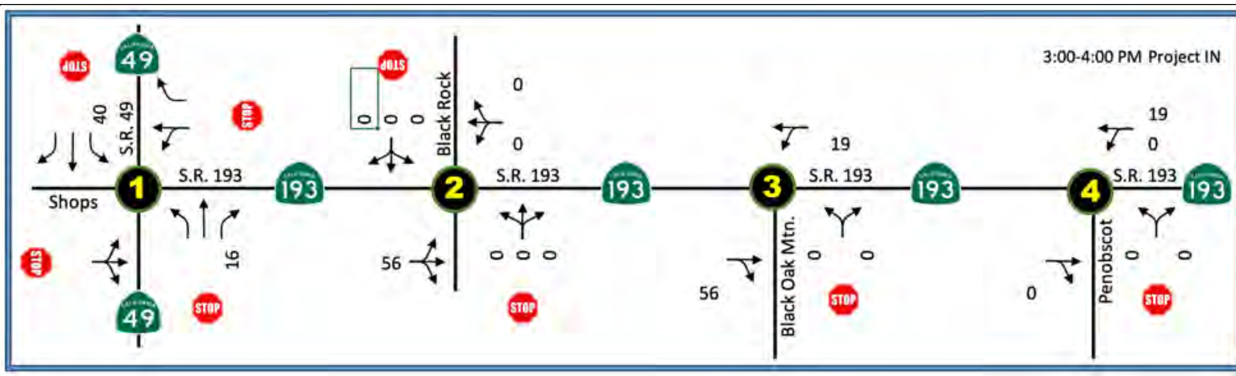


Figure 6. Project Inbound Traffic from 3:00 pm to 4:00 pm, Weekday

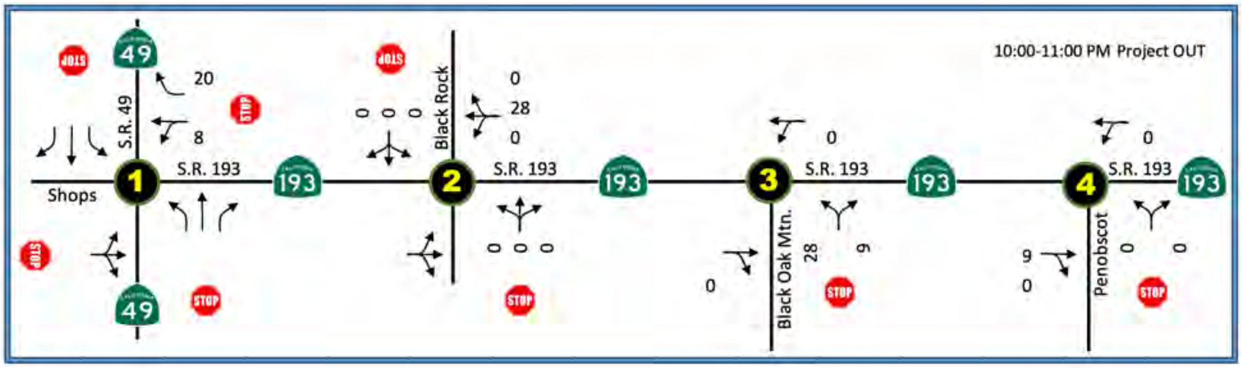


Figure 7. Project Outbound Traffic from 10:00 pm to 11:00 pm, Weekday

PROJECT AREA ROADWAYS and DESCRIPTIONS of EXISTING CONDITIONS

A description of all study area roadways analyzed in this study is contained in the pages that follow, including locations of bike facilities. There are no transit facilities in Cool, CA. The nearest transit bus stops are located in downtown Auburn, CA about 8 miles away from Cool, CA.

INTERSECTION #1: S.R. 49 at S.R. 193

This intersection is stop sign controlled and has four approaches. Figure 6 shows a detailed graphic prepared by PRISM Engineering that illustrates the specific lane striping and other traffic control markings and pavement legends shown and located to scale. The north and south approaches have a three-lane width, with the southbound approach being striped for three separate lanes, and the northbound approach has a left turn pocket striped. The westbound approach has a wide two-lane width, but there is no separate lane striping installed. The right-most portion of the wide lane is functioning as a right turn only area, and our field observations of the pavement show that this is the heavily used “lane.”

Crosswalks are installed at two approaches only: the westbound approach and the northbound approach (see Figure 8). Stop sign limit lines are installed on all four approaches.

Although there are edge lines installed for all approaches, there are no bike lanes installed Directly on the roadway itself, and the edge lines taper back to just a foot or two away from the edge of pavement beyond the intersection.

Figure 8 Intersection of SR 49 and SR 193 Existing Traffic Control and Striping

However, there is a dedicated and separate Class I paved Bike Lane installed on the west side of SR 49 south of SR 193 as shown in the photo below, and which continues on the north side of SR 193 to the east of SR 49 beginning at the northeast corner of the intersection.

This dedicated Class I Bike Lane continues for 1 mile from SR 193 to the south and terminates at Cave Valley Road which immediately serves the Northside Elementary School campus (southwest quadrant of this intersection). There are no bike lane facilities of any kind on SR 49 north of SR 193.

The speed limit of SR 49 north of SR 193 is set at 45 mph (shown in figures that follow) but is 55 mph to the south of SR 193 where a flashing yellow beacon sign is installed warning of a STOP AHEAD condition. In the southbound direction there is an "END OF 45 MPH" regulatory sign installed to once again establish the 55-mph prima facie speed.

As can be seen in Figures 5 and 8, there is a dedicated Class 1 bike lane facility on the north side of SR 193 which extends from the SR 49 Intersection for a distance of about a mile until it's terminus at American River Trail (a bike-friendly circuitous two-lane collector loop-road that serves residential development along the way and somewhat parallels SR 193, eventually connects to Sweetwater Trail which then connects directly to SR 193 at a point approximately 1.3 miles west of the Black Oak Mountain Vineyards driveway).

In the field photos taken by PRISM engineering and shown here, additional roadway and visual details are show how the intersection is seen from the driver's perspective.

For a four-way stop intersection, this unsignalized intersection configuration is very large and has more channelization than is necessary or effective, which can lead to poor performance in heavy traffic as driver's try to decide between each other who's turn it is to enter the intersection.

When there are nine lanes where each could be occupied by a driver that makes an independent decision to enter, delays can occur when there is confusion or indecision. This situation can be compounded when lane striping is missing or is unclear. The "car on the right" rule of who has the right-of-way at a four-way stop is not straight-forward when you have a three-lane approach with some drivers turning left, another going straight, and another turning right, each within their own lane (meaning they might simultaneously move).

This old intersection could benefit from some reconfiguration (suggested later in this report) to better channelize heavy flows of traffic which have grown over the decades, and free up capacity and take some of the vehicle conflicts out of the equation.

INTERSECTION #2: SR 193 and Upper Black Rock Road

This intersection is a minor intersection configured as follows. SR 193 is a two-lane highway with 55 mph through traffic flows and there are no left turn pockets in either direction. All intersection approaches are one lane, meaning, the left turn, the right turn, and the through movement all would share the same lane.

If a car were to make a left turn, or even a right turn from SR 193 to go north or south onto Upper Black Rock Road, it is possible that through traffic might have to slow down under busy traffic conditions while the vehicle exits SR 193. This is a typical condition that is not a problem at the current time.

INTERSECTION #3: SR 193 and Black Oak Mountain Winery Driveway

This intersection is at three-way T-intersection but is actually a gated driveway having access to SR 193 from the project parcel. That portion of the driveway outside of the gate and directly interfacing with SR 193 has a 60' throat for storage when and if the gate is closed. The gate has an electronic access code entry feature that opens an electric gate.

SR 193 at this location is a two-lane highway with 55 mph through traffic flows, and there are no left turn pockets in either direction. All intersection approaches are one lane. If a car were to make a left turn or right turn from SR 193 to go into the project site, it is possible that through traffic might have to slow down under busy traffic conditions while the vehicle exits SR 193.

However, in our observations of the project site during a weekday am peak hour as well as the pm peak hour, we did not observe any traffic going in or out of the site, indicating that the project site has nearly no impact during normal adjacent street traffic peak hours. Vehicles that exit the property do not have a stop sign as it is not needed with this location being a private property driveway.

INTERSECTION #4: SR 193 and Penobscot Road

This intersection is a three-way T-intersection with Penobscot Road being the south leg. Penobscot Road is stop controlled but SR 193 is not controlled by a stop sign. The westbound intersection approach shown to the left here is only one lane, and there is no left turn pocket on SR 193. The Penobscot approach is also striped as being only one lane.

However, the eastbound approach shown below has additional pavement width that allows for a separation of traffic turning right onto Penobscot Road, and the length of this effective unstriped "lane" is approximately 130 feet (where the solid SR 193 edge line breaks into a dashed line as seen in photo to the left).

Traffic volumes on Penobscot Road are low, being less than 30 vehicles per hour in both directions during peak hours. SR 193 in contrast has about 400 vehicles per hour at this location.

TRANSPORTATION STUDY METHODOLOGY

ANALYSIS METHODOLOGY

This report evaluates the trip generation and resulting traffic impacts from the project site within the context of existing and future scenarios as follows:

- Existing (2024) conditions unmitigated (identify any existing deficiencies) –
  - Based on current traffic counts taken by PRISM Engineering on February 28, 2024, and existing roadway geometry and traffic control.
- Existing (2024) plus Project conditions unmitigated (identify any existing deficiencies) –
  - Based on current traffic counts taken by PRISM Engineering on February 28, 2024, and existing roadway geometry and traffic control.
- Year 2044 Cumulative Traffic Condition (20 years out), unmitigated –
  - Based on anticipated growth in baseline traffic volumes determined by straight-line interpolation between Year 2024 existing counts and Year 2044 traffic projections.
- Year 2044 Cumulative plus Project Traffic Condition (20 years out), unmitigated –
  - Based on anticipated growth in baseline traffic volumes determined by straight-line interpolation between Year 2024 existing counts and Year 2044 traffic projections.

The purpose of the analysis of traffic in this report is to identify intersections and road segments that are expected to have congestion and unsatisfactory traffic conditions in the future[5], determine if the project has made a significant impact towards these problems, and to develop and discuss improvement measures that would solve the potential future traffic problems.

This traffic study scope and methodology follows all guidelines set forth by the El Dorado County Community Development Agency (via DOT), and follows the methodology and procedures outlined in the *Transportation Impact Study Guidelines* document prepared by the Community Development Agency, Long Range Planning division. It also follows the Caltrans threshold of LOS D being the maximum congestion level allowed for state highways.

HCM Operating Conditions and LOS Criteria for Intersections

Analysis of significant environmental impacts at intersections is based on the concept of Level of Service (LOS). The LOS of an intersection is a qualitative measure used to describe operational conditions, and ranges from LOS A (best, minimal delay) to LOS F (worst, heavy delays) where the intersection is operating at or near its functional capacity. Levels of Service for this study were determined using the *Highway Capacity Manual* (HCM) 6th edition methodologies as recommended by the County, which are implemented in the *Synchro version 12* traffic analysis

[5] Impacts are based upon the significance criteria set forth in the El Dorado County Traffic Study Guidelines.

software[6] including SimTraffic micro-simulation if needed. In this study, microsimulation is not needed because the study intersections are all unsignalized and this level of sophistication in analysis is beyond the scope of the available data.

Table 6 relates the operational characteristics associated with each LOS category for signalized and unsignalized intersections. The HCM 6th edition includes procedures for analyzing side-street two-way stop controlled (TWSC), all-way stop-controlled (AWSC), and signalized intersections. The TWSC procedure defines LOS as a function of average control delay for each minor street approach movement. Conversely, the AWSC and signalized intersection procedures define LOS as a function of average control delay for the intersection as a whole, or for individual approaches or even separate turning movements. The calculation of LOS for an intersection utilized PHF from traffic counts averaged for each intersection as a whole [4* (Max 15 min)/(Total Hour)] as per County procedure, and also used a general 2% Heavy Vehicle factor as per County guidelines to be conservative. Pedestrians were noticeably absent in the count data due to the nature of the high-speed highway conditions in the study area.

For the purposes of this study, only intersection LOS averages were summarized for each study intersection. For TWSC intersections, level of service is additionally reported for the worst approach.

Table 6. Intersection Level of Service Definitions

Level of Service	Description	Avg. delay per vehicle, sec/veh	
		Signalized	Un-Signalized
A	Free flow with no delays. Users are virtually unaffected by others in the traffic stream	≤ 10	≤ 10
B	Stable traffic. Traffic flows smoothly with few delays.	> 10 – 20	> 10 – 15
C	Stable flow but the operation of individual users becomes affected by other vehicles. Modest delays.	> 20 – 35	> 15 – 25
D	Approaching unstable flow. Significant effects by other vehicles. Delays may be more than one cycle during peak hours.	> 35 – 55	> 25 – 35
E	Unstable flow with operating conditions at or near the capacity level. Long delays and vehicle queuing.	> 55 – 80	> 35 – 50
F	Forced or breakdown flow; causes reduced capacity. Stop and go traffic conditions. Excessive long delays and vehicle queuing.	> 80	> 50

Sources: Transportation Research Board, Highway Capacity Manual 2010, National Research Council, 2010.

[6] The HCM 6th edition methodology was used in this study for existing unsignalized intersections (TWSC and AWSC). However, the signalized HCM 6th edition method was used to analyze any future scenarios with a traffic signal.

Thresholds of Significance for Intersections

A project's traffic impact at an intersection is considered to be significant under El Dorado County guidelines if the project causes an intersection to change from LOS E to LOS F. In other words, LOS E is considered acceptable by El Dorado County for roadways and state highways within the unincorporated areas of the County in the Community Regions and LOS D in the Rural Center and Rural Regions except as specified in the General Plan. The Black oak Mountain Vineyards project is located within a Rural Region; therefore, LOS D is the threshold for acceptable traffic conditions.

The County DOT Guidelines for traffic studies stipulates "*Projects that have impacts to Caltrans facilities shall use Caltrans LOS standards and significance thresholds in conjunction with the requirements of El Dorado County General Plan Circulation Policy TC-Xd*." PRISM Engineering used this guiding principle as the basis by which to evaluate and analyze the traffic impacts on the state highway system, which includes all four study intersections.

The effects of vehicle queuing were also analyzed for any future Year 2044 scenario where a traffic signal was considered as a mitigation for congested traffic levels. The existing conditions traffic has no need for a signal and does not have traffic congestion beyond LOS C conditions at SR 49 and SR 193. Where queues were analyzed, the 95th percentile queue is reported. The 95th percentile queue length represents a condition where 95 percent of the time during the peak period, traffic volumes and related queuing will be at, or less, than the queue length determined by the analysis. This is referred to as the "95th percentile queue." Average queuing is generally less.

Queuing is considered a potentially significant impact since queues that exceed turn pocket length can create potentially hazardous conditions by blocking or disrupting through traffic in adjacent travel lanes. However, these potentially hazardous queues are typically associated with left-turn movements. Locations where the right turn pocket storage is exceeded is not considered potentially hazardous because the right turn movement will go at the same time as the through movement and the additional vehicles that spill out over the turn pocket will not be hindering or disrupting the adjacent through traffic as would be the case in most left turn pockets.[7] Thus, for purposes of this analysis, a queuing impact is considered to occur under conditions where project traffic causes the queue in a left turn pocket to extend beyond the turn pocket by 25 feet or more (i.e., the length of one vehicle) into adjacent traffic lanes that operate (i.e., move) separately from the left turn lane. Where the vehicle queue already exceeds that turn pocket length under pre-project conditions, a project impact would occur if project traffic lengthened the queue by 25 feet or more.

[7] If a left turn movement operates (i.e. moves) at the same time as the through movement such as with split signal phasing, then a left turn queue that exceeds the turn storage is not considered an impact.

YEAR 2024 EXISTING CONDITIONS ANALYSES

In order to establish the existing conditions of traffic in the study area, PRISM Engineering conducted peak hour turning movement counts from video[8] cameras set up at the intersection and supplemented with drone video to see and record high level traffic turn and distribution patterns (from 390' up from ground level). All of the details of the two hour counts at the four study intersections are contained in Appendix A of this report, and include 15-minute interval counts, with calculated Peak Hour Factors (PHF) calculated from the weighted average of total peak hour volume using the following formula: 4* (Max 15 min)/(Total Hour). As per the County's policy, the PHF intersection average was used in calculations at each intersection location in the Synchro version 12 software, using the HCM 6th edition calculation option.

The counts at most of the locations were also documented with high resolution photos and video, which we use to facilitate more detailed analysis in the office (such as studying any overflows, traffic congestion operations, aerial video to study traffic patterns, as well as any other traffic characteristics relating to analysis parameters or specific driver behaviors that inform potential improvement).

The various Highway Capacity Manual (HCM 6th ed.) based analyses that are summarized in this report consists of the following (as required by El Dorado County guidelines):

- AM and PM Peak Hour ***Intersection Level of Service*** Calculations based on HCM 6th ed.
- AM and PM Peak Hour ***Signal Warrant Analysis*** for Unsignalized Intersections, CAMUTCD

The tables of capacity analysis summaries that follow are used to determine if there are any transportation deficiencies that need improvement. The intersection turning movement data used in all of the above analyses were collected on February 28, 2024, for all intersection turning movements. Other count data was obtained from other sources to aid in determining growth rates for different highways for developing the Year 2044 traffic projections from existing count data. Other sources of traffic growth related data collection included the County's Hourly Traffic Count Reports[9] webpage, a review of the County's traffic model projections for basic roadways, as well as using the Caltrans Traffic Census Program webpage[10] to download several spreadsheets of ADT counts for all state highways for a variety of years from 2013 to 2022.

The following figures document the various Year 2024 scenarios:

- Figure 9. AM Peak Hour Existing Year 2024 Intersection Turn Volumes
- Figure 10. PM Peak Hour Existing Year 2024 Intersection Turn Volumes
- Figure 11A. Afternoon 3:00-4:00 PM Peak Hour Intersection Turn Volumes to Coincide with Project Peak Inbound Traffic.

[8] Used a tripod mounted 360-degree video camera to facilitate viewing of the entire intersection and all movements.
[9] Downloaded various roadway hourly counts from The County's website: https://edcroads.edcgov.us/Traffic
[10] Downloaded traffic count spreadsheets (2013 to 2022): https://dot.ca.gov/programs/traffic-operations/census

- Figure 11B. Afternoon 3:00-4:00 PM Peak Hour Plus Project Turn Volumes.
- Figure 12A. Late Evening 10:00-11:00 PM Peak Hour Intersection Turn Volumes to Coincide with Project Peak Outbound Traffic.
- Figure 12B. Late Evening 10:00-11:00 PM Peak Hour Plus Project Turn Volumes.

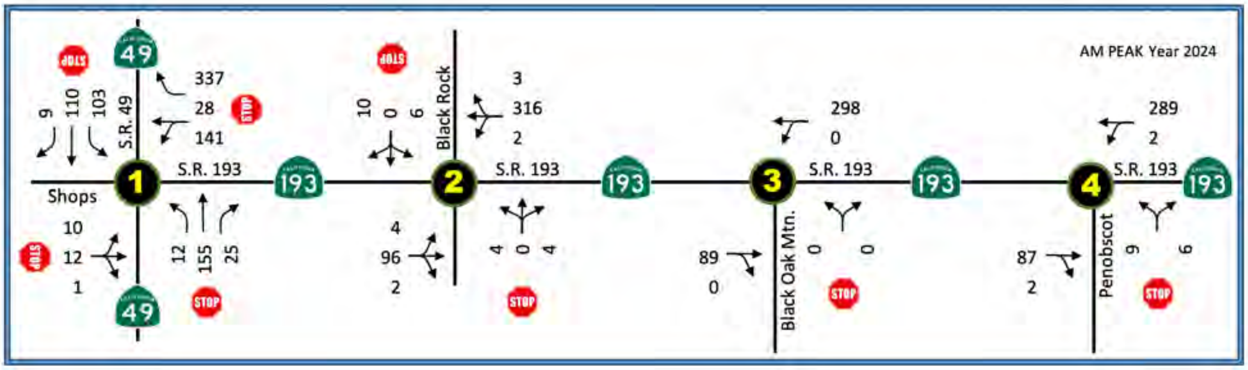


Figure 9. AM Peak Hour Existing Year 2024 Intersection Turn Volumes

Source: PRISM Engineering *(note: the Project does NOT have any activity during the normal 7:15-8:15 am peak hour)*

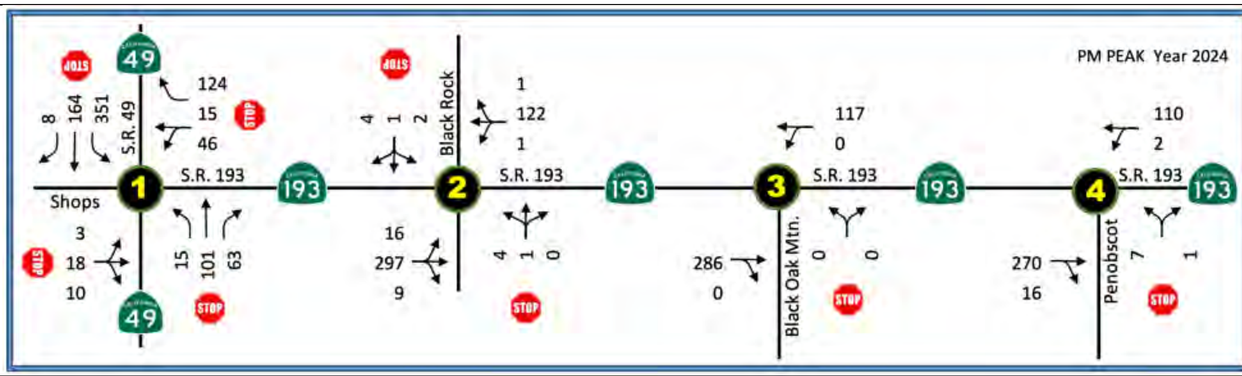


Figure 10. PM Peak Hour Existing Year 2024 Intersection Turn Volumes

Source: PRISM Engineering *(note: the Project does NOT have any activity during the normal 5:00-6:00 pm peak hour)*

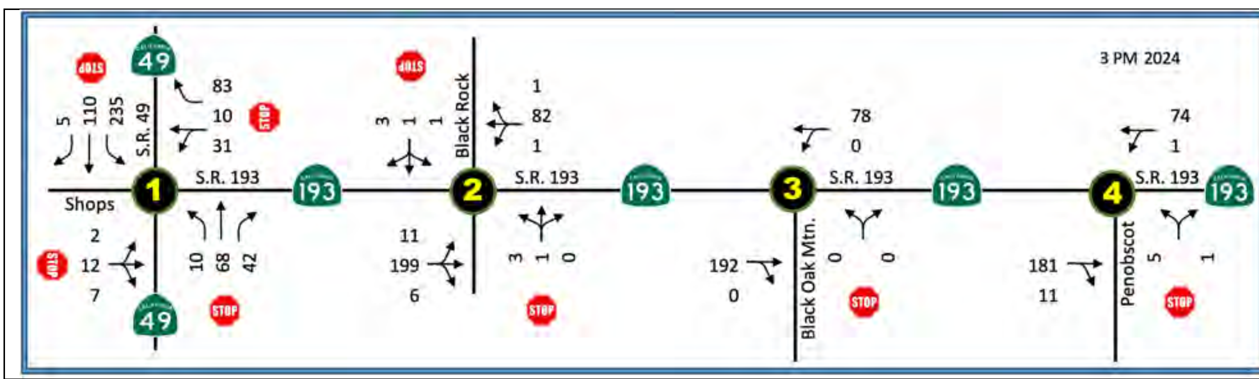


Figure 11A. 3:00-4:00 Afternoon Year 2024 Intersection Turn Volumes

Source: PRISM Engineering

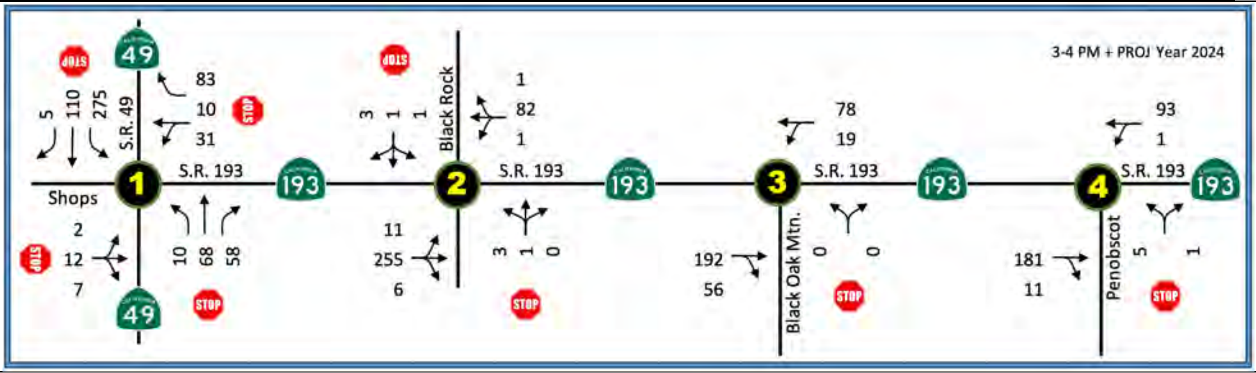


Figure 11B. 3:00-4:00 Afternoon Year 2024 Plus Project Volumes

Source: PRISM Engineering

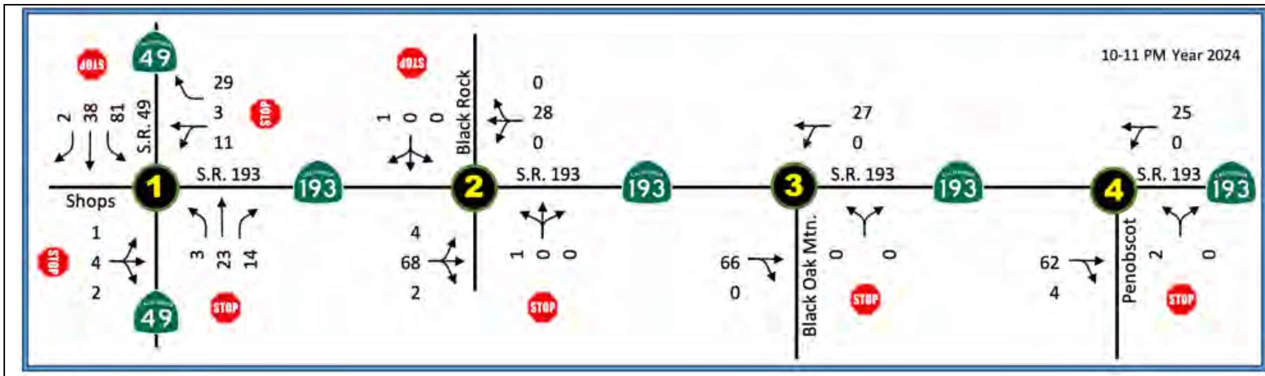


Figure 12A. 10:00-11:00 PM Year 2024 Intersection Turn Volumes

Source: PRISM Engineering

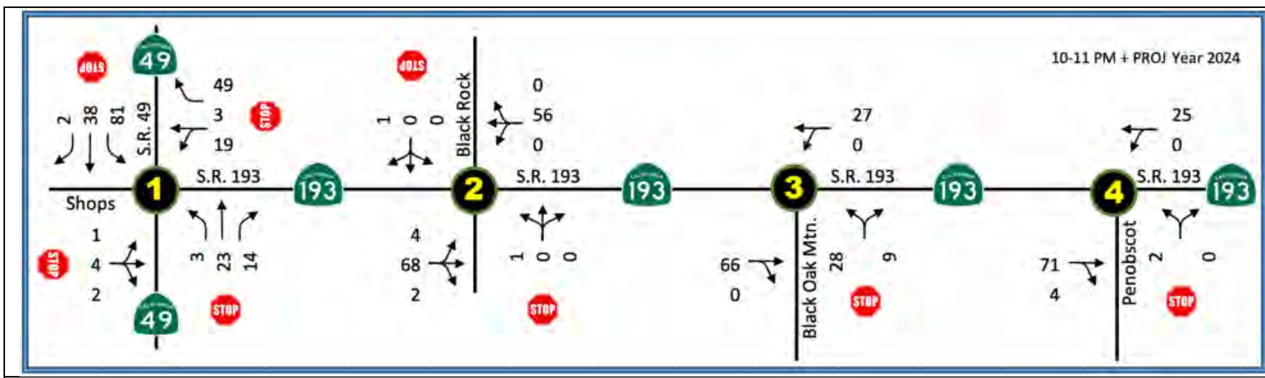


Figure 12B. 10:00-11:00 PM Year 2024 Plus Project Volumes

Source: PRISM Engineering

Queue Analyses, Year 2024 AM and PM Peak Scenarios for AWSC (All-Way Stop Control)

Table 7 summarizes the Existing Year 2024 AM and PM Peak Hour conditions for the SR 49 and SR 193 AWSC intersection, with and without the project. In the case of this scenario, the project volumes are zero as they do not appear during the am or pm peak hour of adjacent street traffic (typically 6:30 am to 8:30 am and 4:30 pm to 6:30 pm).

Table 7. Existing 2024 Scenarios for Peak Hour of Adjacent Street Traffic

95th percentile Queue Results ***(HCM 6th ed. 95th-tile Q values shown in FEET on last row of scenario)***

	SCENARIO	Control	Lane	NBLn1	NBLn2	NBLn3	EBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
1	Year 2024 AM Peak Hour of Adjacent Street Traffic	AWSC	HCM Lane V/C Ratio	0.033	0.39	0.057	0.062	0.397	0.658	0.274	0.274	0.02
			HCM Control Delay, s/veh	10.9	14.7	9.8	11.3	14	18.9	13.4	12.7	9.4
			HCM Lane LOS	B	B	A	B	B	C	B	B	A
			HCM 95th-tile Q	0.1	1.8	0.2	0.2	1.9	4.9	1.1	1.1	0.1
			Queue Length (in vehicles)	0.1	1.8	0.2	0.2	1.9	4.9	1.1	1.1	0.1
			Queue Length (in feet)	2.5	45	5	5	47.5	122.5	27.5	27.5	2.5
			Storage Length	150	650*	40		500*	50	200	390*	200
2	Year 2024 AM Peak Hour of Adjacent Street Traffic plus Project	AWSC	HCM Lane V/C Ratio	0.033	0.39	0.057	0.062	0.397	0.658	0.274	0.274	0.02
			HCM Control Delay, s/veh	10.9	14.7	9.8	11.3	14	18.9	13.4	12.7	9.4
			HCM Lane LOS	B	B	A	B	B	C	B	B	A
			HCM 95th-tile Q	0.1	1.8	0.2	0.2	1.9	4.9	1.1	1.1	0.1
			Queue Length (in vehicles)	0.1	1.8	0.2	0.2	1.9	4.9	1.1	1.1	0.1
			Queue Length (in feet)	2.5	45	5	5	47.5	122.5	27.5	27.5	2.5
			Storage Length	150	650*	40		500*	50	200	390*	200
3	Year 2024 PM Peak Hour of Adjacent Street Traffic	AWSC	HCM Lane V/C Ratio	0.032	0.208	0.116	0.068	0.135	0.234	0.686	0.297	0.013
			HCM Control Delay, s/veh	10.2	11.2	9.6	10.4	11.1	10.8	22.1	11.1	8
			HCM Lane LOS	B	B	A	B	B	B	C	B	A
			HCM 95th-tile Q	0.1	0.8	0.4	0.2	0.5	0.9	5.3	1.2	0
			Queue Length (in vehicles)	0.1	0.8	0.4	0.2	0.5	0.9	5.3	1.2	0
			Queue Length (in feet)	2.5	20	10	5	12.5	22.5	132.5	30	0
			Storage Length	150	650*	40		500*	50	200	390*	200
4	Year 2024 PM Peak Hour of Adjacent Street Traffic plus Project	AWSC	HCM Lane V/C Ratio	0.032	0.208	0.116	0.068	0.135	0.234	0.686	0.297	0.013
			HCM Control Delay, s/veh	10.2	11.2	9.6	10.4	11.1	10.8	22.1	11.1	8
			HCM Lane LOS	B	B	A	B	B	B	C	B	A
			HCM 95th-tile Q	0.1	0.8	0.4	0.2	0.5	0.9	5.3	1.2	0
			Queue Length (in vehicles)	0.1	0.8	0.4	0.2	0.5	0.9	5.3	1.2	0
			Queue Length (in feet)	2.5	20	10	5	12.5	22.5	132.5	30	0
			Storage Length	150	650*	40		500*	50	200	390*	200

**distance (feet) of THRU lane from intersection stop bar to nearest cross street or driveway in back of queue.*

Source: Synchro HCM 6th ed. and PRISM Engineering

It can be seen from Table 7 that the 95th percentile queue lengths do not exceed the maximum queue result of 5.3 vehicles for the SBL turn lane on SR 49. This calculates to 132.5 feet (assuming a stopped car length and space occupancy of 25 feet long front bumper to front bumper of next vehicle, typical). In the case where the WBR lane only has a 50 foot right turn pocket area but this movement is very heavy in the am peak hour with a 95th queue of 122.5 feet, it is important to note that the WBL lane which is very long only needs 47.5 feet in this worst-case 95th percentile calculation, and so there is enough room for all right turning vehicles to enter the right turn pocket lane without delay. It operates at LOS C. All other movements are LOS B or better conditions in terms of delay, and do not have any 95th percentile queue length overflows for these scenarios.

Intersection Levels of Service, Year 2024 AM and PM Peak Hour of Adjacent Street Traffic

Table 8 summarizes the future Year 2024 conditions scenario intersection level of service results for the adjacent street peak traffic for the am peak hour and pm peak hour, with and without the project traffic (a total of four scenarios are summarized in the table).

Table 8. Year 2024 *Peak Hour of Adjacent Street Traffic* Scenarios, Intersection Level of Service Analysis Results

			YEAR 2024 AM Peak 7:30-8:30 AM				YEAR 2024 PM Peak 5:15-6:15 PM			
			No PROJ		w/PROJ		No PROJ		w/PROJ	
	INTERSECTION LOCATION	Control	LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)
1	SR 49 at SR 193	AW	C	15.4	C	15.4	C	15.1	C	15.1
2	SR 193 at Upper Black Road	TW	A	0.6	A	0.6	A	0.6	A	0.6
		SB/NB	B	10.9	B	10.9	B	12.6	B	12.6
3	SR 193 at Black Oak Mountain Winery	TW	A	0.0	A	0.0	A	0.0	A	0.0
		NB	A	0.0	A	0.0	A	0.0	A	0.0
4	SR 193 at Penobscot Road	TW	A	0.5	A	0.5	A	0.2	A	0.2
		NB	B	10.2	B	10.2	B	10.9	B	10.9

Control: AW=All-Way Stop Control, TW=Tw-Way Stop Control with Stop Sign on Side Street, NB=NB approach Stop

NOTE: Calculations based on HCM 2010 6th ed. methdology for intersection level of service (AWSC and all-way stop and TWSC two-way)

Source: PRISM Engineering and HCM 6th ed. Analysis results

Queue Analyses, Year 2024 AM and PM Peak Hour of Generator (3-4 pm and 10-11 pm)

Table 9 summarizes the queue analysis results for the Year 2024 3-4 pm inbound project, and 10-11 PM outbound project traffic conditions for the Peak Hour of the Generator (the Project's Peak Hour) at the SR 49 and SR 193 AWSC intersection (with and without the project).

Table 9. Existing 2024 Scenarios for Peak Hour of Adjacent Street Traffic

95th percentile Queue Results *(HCM 6th ed. 95th-tile Q values shown in FEET on last row of scenario)*

	SCENARIO	Control	Lane	NBLn1	NBLn2	NBLn3	EBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
1	Year 2024 3-4 PM Peak Hour of Generator Traffic	AWSC	HCM Lane V/C Ratio	0.022	0.143	0.078	0.046	0.094	0.157	0.481	0.208	0.008
			HCM Control Delay, s/veh	9.5	9.9	8.6	9.4	10	9.3	14.3	9.7	7.5
			HCM Lane LOS	A	A	A	A	A	A	B	A	A
			HCM 95th-tile Q	0.1	0.5	0.3	0.1	0.3	0.6	2.6	0.8	0
			Queue Length (in vehicles)	0.1	0.5	0.3	0.1	0.3	0.6	2.6	0.8	0
			Queue Length (in feet)	2.5	12.5	7.5	2.5	7.5	15	65	20	0
			Storage Length	150	650*	40		500*	50	200	390*	200
2	Year 2024 3-4 PM Peak Hour of Generator plus Project Inbound Traffic	AWSC	HCM Lane V/C Ratio	0.023	0.146	0.111	0.048	0.097	0.163	0.569	0.211	0.008
			HCM Control Delay, s/veh	9.6	10	9	9.6	10.3	9.6	16.6	9.8	7.6
			HCM Lane LOS	A	A	A	A	B	A	C	A	A
			HCM 95th-tile Q	0.1	0.5	0.4	0.1	0.3	0.6	3.6	0.8	0
			Queue Length (in vehicles)	0.1	0.5	0.4	0.1	0.3	0.6	3.6	0.8	0
			Queue Length (in feet)	2.5	12.5	10	2.5	7.5	15	90	20	0
			Storage Length	150	650*	40		500*	50	200	390*	200
3	Year 2024 10-11 PM Peak Hour of Generator Traffic	AWSC	HCM Lane V/C Ratio	0.011	0.066	0.027	0.017	0.033	0.056	0.191	0.102	0.005
			HCM Control Delay, s/veh	8.5	8.3	7.3	8	8.6	7.6	9.4	8.2	6.9
			HCM Lane LOS	A	A	A	A	A	A	A	A	A
			HCM 95th-tile Q	0	0.2	0.1	0.1	0.1	0.2	0.7	0.3	0
			Queue Length (in vehicles)	0	0.2	0.1	0.1	0.1	0.2	0.7	0.3	0
			Queue Length (in feet)	0	5	2.5	2.5	2.5	5	17.5	7.5	0
			Storage Length	150	650*	40		500*	50	200	390*	200
4	Year 2024 10-11 PM Peak Hour of Generator Outbound Traffic	AWSC	HCM Lane V/C Ratio	0.006	0.04	0.021	0.013	0.041	0.073	0.15	0.064	0.002
			HCM Control Delay, s/veh	8.4	8	7.2	7.9	8.5	7.4	9.1	7.9	6.9
			HCM Lane LOS	A	A	A	A	A	A	A	A	A
			HCM 95th-tile Q	0	0.1	0.1	0	0.1	0.2	0.5	0.2	0
			Queue Length (in vehicles)	0	0.1	0.1	0	0.1	0.2	0.5	0.2	0
			Queue Length (in feet)	0	2.5	2.5	0	2.5	5	12.5	5	0
			Storage Length	150	650*	40		500*	50	200	390*	200

**distance (feet) of THRU lane from intersection stop bar to nearest cross street or driveway in back of queue.*

Source: Synchro HCM 6th ed. and PRISM Engineering

It can be seen from Table 9 that the 95th percentile queue lengths do not exceed maximum queue result of 3.6 vehicles* in any case, having a calculated 90 feet queue (assuming a stopped car length and space occupancy of 25 feet long front bumper to front bumper of next vehicle, typical).

*for SBL turn lane on SR 49

In Table 10 the traffic volumes used for all study intersections coincides with traffic levels that are estimated to exist between 3-4 PM for project arrival times, and 10-11 PM for project exiting times. These estimated volumes were based on factors documented in Appendix B, Hourly Traffic Counts on Study Area Vicinity Roadways (*Assume SR 193 traffic at 10 pm is 23% of the regular PM Peak Volumes taking place at 5-6 PM. Assume traffic at 3-4 PM is 67% of 5-6 PM volumes on SR 193*).

Table 10. Year 2024 *Peak Hour of Generator (Project)* Traffic Scenarios, Intersection Level of Service Analysis Results

			YEAR 2024 INBOUND PEAK 3:00-4:00 PM				YEAR 2024 OUTBOUND PEAK 10:00-11:00 PM			
			No PROJ		w/PROJ		No PROJ		w/PROJ	
	INTERSECTION LOCATION	Control	LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)
1	SR 49 at SR 193	AW	B	11.4	B	12.5	A	8.1	A	8.2
2	SR 193 at Upper Black Road	TW	A	0.6	A	0.5	A	0.5	A	0.4
		SB/NB	B	10.9	B	11.4	A	9.2	A	9.3
3	SR 193 at Black Oak Mountain Winery	TW	A	0.0	A	0.4	A	0.0	A	2.6
		NB	A	0.0	A	0.0	A	0.0	A	9.1
4	SR 193 at Penobscot Road	TW	A	0.2	A	0.2	A	0.2	A	0.2
		NB	A	9.9	B	10.0	A	9.0	A	9.0

Control: AW=All-Way Stop Control, TW=Tw-Way Stop Control with Stop Sign on Side Street, NB=NB approach Stop

NOTE: Calculations based on HCM 2010 6th ed. methdology for intersection level of service (AWSC and all-way stop and TWSC two-way)

Source: PRISM Engineering and HCM 6th ed. Analysis results

Signal Warrant Analysis, Year 2024 and Year 2044 Scenarios

The following Figure 13 shows the CAMUTCD method / chart for determining if a traffic signal is warranted from traffic volumes at any of the four unsignalized intersections in this report. This chart was adapted and expanded by PRISM Engineering to accommodate lower volumes for the MAJOR STREET.

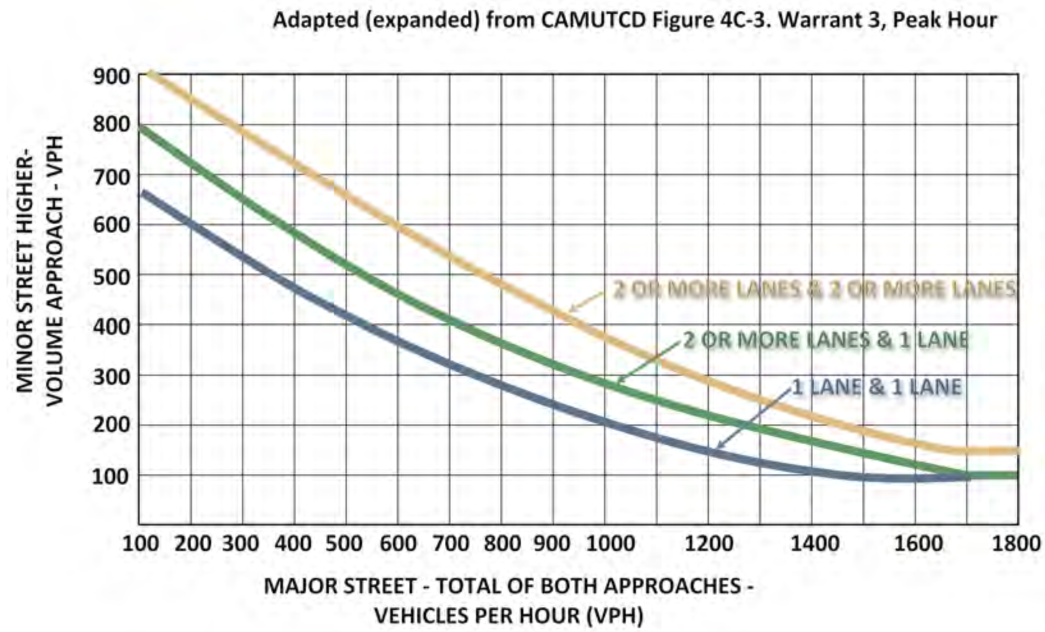


Figure 13. CAMUTCD Chart: Traffic Signal Warrant 3 Values for Peak Hour
Source: CAMUTCD and PRISM Engineering

Table 11 summarizes the signal warrant analyses for all years analyzed at the most critical intersection with the highest volume of traffic for the peak hour warrant: SR 49 at SR 193.

Table 11. Signal Warrant Analysis Results

INTERSECTION *(green indicates warranted)*	AM 2024 TOTAL BOTH APPR MAJOR	PM 2024 TOTAL BOTH APPR MAJOR	AM 2044 TOTAL BOTH APPR MAJOR	PM 2044 TOTAL BOTH APPR MAJOR	MINOR STREET HIGHER VOLUME AM 2024	PM 2024	AM 2044	PM 2044
S.R. 49 at S.R. 193	529	702	676	990	222	185	322	234
Minor Street Volume Needed	500	400	430	300	-278	-215	-108	-66
Signal Warranted??	NO	NO	NO	NO	NO	NO	NO	NO

Source: PRISM Engineering

It is important to note that the peak hour signal warrant is based on the peak traffic levels for the adjacent street traffic conditions (7:30-8:30 am and 5:15-6:15 pm) and not when the project traffic is expected to take place (3-4 pm and 10-11 pm).All other intersections have very small side street traffic during the peak hour time periods and were eliminated from the analysis by inspection due to the fact that the busiest intersection by far (SR 49 at SR 192) did not meet the signal warrant. The project while it adds some traffic during the non-peak hours of adjacent street traffic (3-4 pm and 10-11 pm) it does not add any traffic to the normal am or pm peak hour time periods and is an insignificant impact on meeting any signal warrants.

DEFICIENCIES for YEAR 2024 SCENARIOS

There were no deficiencies for the Year 2024 scenarios as shown in Tables 8 and 10 which indicate that the level of service at intersections will be at LOS C or better conditions, even with the project.

In addition, traffic signals are NOT warranted at any of the other unsignalized intersections in this study for existing or existing plus project conditions (see Table 11 for details).

Summary: No improvements are recommended for The Year 2024 scenarios.

Cumulative Year 2044 Analyses (20 Years in Future)

This section of the report documents how future volumes were considered and where growth rates were developed to factor the existing traffic counts into a future year 2044 condition.

Calculation of Growth Rates based on Future Background Traffic Growth. PRISM Engineering obtained a copy of the El Dorado County Department of Transportation Travel Demand Model files for use in this traffic study[11] to help develop future volumes and growth factors. However, due to the lack of detail in the model in Cool and lack of various local roadways, we determined that using a growth rate developed directly from Caltrans historical traffic counts was the best solution. Using the Caltrans traffic counts for highways taken at various milepost locations on SR 49 and SR 193, PRISM Engineering was able to develop reasonable growth rates that could be applied to existing traffic counts in order to calculate a cumulative Year 2044 traffic volume. These growth rate calculation results from the Caltrans counts obtained from their website and downloaded into a spreadsheet compare favorably to growth rate results developed by PRISM Engineering using the model in several other traffic studies in El Dorado County prepared by PRISM Engineering[12].

[11] Model files received from DOT via Civil Engineer Zach Oates via email.

[12] PRISM has calculated many other growth rates in El Dorado in previous traffic studies within El Dorado County which relied on the County's traffic model existing and future forecasts. The growth rate results have been between a low 0.7% per year in Diamond Springs to a high of 2.4% per year in Cameron Park.

Table 12, Growth Rate Calculations for Determining Year 2044 Traffic, shows the traffic counts taken by Caltrans as well as PRISM Engineering at the same locations, so a comparison of Year 2013 counts (Caltrans) can be made with 2022 counts (Caltrans) and 2024 counts (PRISM). It can be seen that the Caltrans Year 2024 traffic count projection is almost identical (within 1.5%) to the actual PRISM Engineering field counts for year 2024 (taken on Feb. 28, 2024). Yearly growth rates (compounded) for roadways within the project study area were calculated from the data shown in Table 12 below. The final result is the last column which is a 20-year growth factor based on the compounded yearly growth rate, which is used to multiply against the existing traffic counts taken by PRISM Engineering (Feb 28, 2024) to bring them up to a Cumulative Year 2044 level. These factors are 1.61 for SR 49 traffic volumes, and 1.27 for SR 193 traffic volumes.

Table 12. Growth Rate Calculations for Determining Year 2044 Traffic

	TRAFFIC COUNTS						
	CALTRANS COUNTS YEAR 2013	CALTRANS COUNTS YEAR 2022	YEAR 2024 PROJECTION w/Caltrans Growth Rate	PRISM COUNTS YEAR 2024	CALTRANS 10 YEAR GROWTH	10 YEAR CALTRANS GROWTH RATE	20 YR GROWTH FACTOR
CALTRANS COUNTS: on SR 49 at SR 193	540	680	715	725	140	2.4%	1.61
CALTRANS COUNTS: on SR 193 at SR 49	560	630	645	645	70	1.2%	1.27

Source: Caltrans Traffic Census Program[13], El Dorado County Traffic Model, and PRISM Engineering

The factors shown in the last column of Table 12 were used to prepare the Year 2044 projected traffic volumes from the PRISM 2024 counts. These calculated future volumes and turning movements are shown in Figures 14 and 15 for the AM and PM Cumulative scenarios and were subsequently used in the HCM 6th edition capacity analysis in the Synchro version 12 software for the Cumulative Year 2044 analysis scenarios.

[13] Traffic counts for Year 2013 to 2022 for all Caltrans Highways found at: https://dot.ca.gov/programs/traffic-operations/census and which were relied upon by PRISM Engineering to determine growth trends on State Highways.

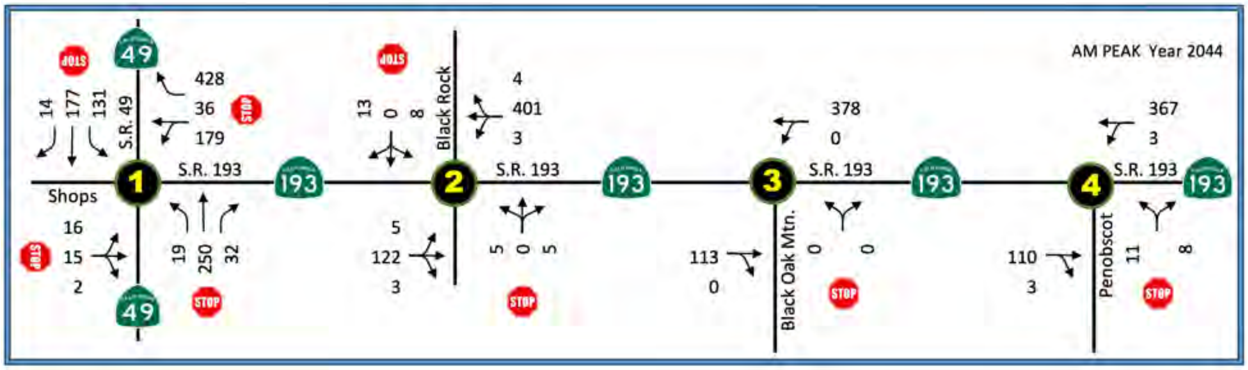


Figure 14. AM Peak Hour Cumulative Year 2044 Intersection Turn Volumes

Source: PRISM Engineering *(note: the Project does NOT have any activity during the normal 7:15-8:15 am peak hour)*

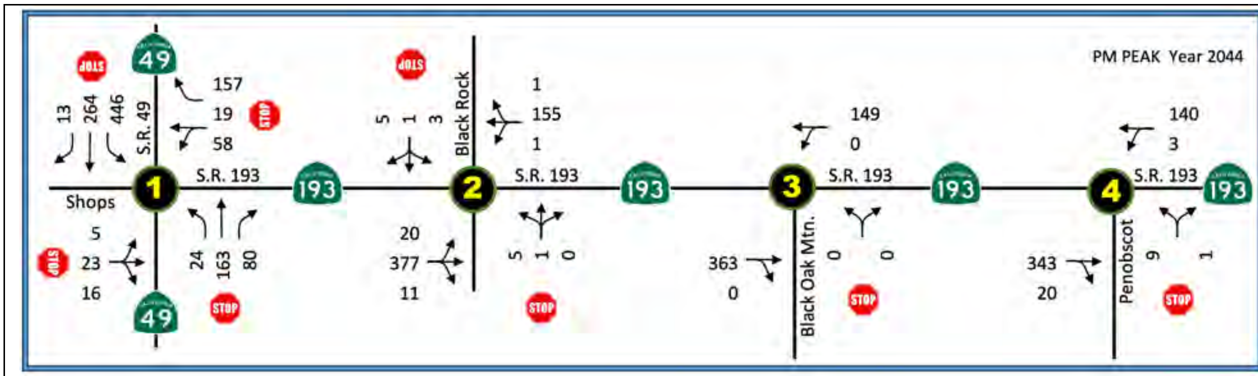


Figure 15. PM Peak Hour Cumulative Year 2044 Intersection Turn Volumes

Source: PRISM Engineering *(note: the Project does NOT have any activity during the normal 5:00-6:00 pm peak hour)*

Table 13 summarizes the Future Year 2044 AM and PM Peak Hour conditions for the SR 49 and SR 193 AWSC intersection, with and without the project. In the case of this scenario, the project volumes are zero as they do not appear during the am or pm peak hour of adjacent street traffic (typically 6:30 am to 8:30 am and 4:30 pm to 6:30 pm).

Table 13. Existing 2044 Scenarios for Peak Hour of Adjacent Street Traffic

SCENARIO		Control	95th percentile Queue Results *(HCM 6th ed. 95th-tile Q values shown in FEET on last row of scenario)*									
			Lane	NBLn1	NBLn2	NBLn3	EBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
1	Year 2044 AM Peak Hour of Adjacent Street Traffic	AWSC	HCM Lane V/C Ratio	0.059	0.736	0.088	0.114	0.594	1.01	0.41	0.525	0.038
			HCM Control Delay, s/veh	12.5	31.3	11.4	13.9	22.2	72.7	17.9	20.2	11
			HCM Lane LOS	B	D	B	B	C	F	C	C	B
			HCM 95th-tile Q	0.2	5.7	0.3	0.4	3.8	14.9	1.9	2.8	0.1
			Queue Length (in vehicles)	0.2	5.7	0.3	0.4	3.8	14.9	1.9	2.8	0.1
			Queue Length (in feet)	5	142.5	7.5	10	95	372.5	47.5	70	2.5
			Storage Length	150	650*	40		500*	50	200	390*	200
2	Year 2044 AM Peak Hour of Adjacent Street Traffic plus Project	AWSC	HCM Lane V/C Ratio	0.059	0.736	0.088	0.114	0.594	1.01	0.41	0.525	0.038
			HCM Control Delay, s/veh	12.5	31.3	11.4	13.9	22.2	72.7	17.9	20.2	11
			HCM Lane LOS	B	D	B	B	C	F	C	C	B
			HCM 95th-tile Q	0.2	5.7	0.3	0.4	3.8	14.9	1.9	2.8	0.1
			Queue Length (in vehicles)	0.2	5.7	0.3	0.4	3.8	14.9	1.9	2.8	0.1
			Queue Length (in feet)	5	142.5	7.5	10	95	372.5	47.5	70	2.5
			Storage Length	150	650*	40		500*	50	200	390*	200
3	Year 2044 PM Peak Hour of Adjacent Street Traffic	AWSC	HCM Lane V/C Ratio	0.073	0.456	0.205	0.135	0.231	0.412	1.136	0.629	0.028
			HCM Control Delay, s/veh	12.1	17.2	12	13.1	14	15.4	110.4	20.8	9.2
			HCM Lane LOS	B	C	B	B	B	C	F	C	A
			HCM 95th-tile Q	0.2	2.2	0.7	0.5	0.8	1.9	18.9	4.3	0.1
			Queue Length (in vehicles)	0.2	2.2	0.7	0.5	0.8	1.9	18.9	4.3	0.1
			Queue Length (in feet)	5	55	17.5	12.5	20	47.5	472.5	107.5	2.5
			Storage Length	150	650*	40		500*	50	200	390*	200
4	Year 2044 PM Peak Hour of Adjacent Street Traffic plus Project	AWSC	HCM Lane V/C Ratio	0.073	0.456	0.205	0.135	0.231	0.412	1.136	0.629	0.028
			HCM Control Delay, s/veh	12.1	17.2	12	13.1	14	15.4	110.4	20.8	9.2
			HCM Lane LOS	B	C	B	B	B	C	F	C	A
			HCM 95th-tile Q	0.2	2.2	0.7	0.5	0.8	1.9	18.9	4.3	0.1
			Queue Length (in vehicles)	0.2	2.2	0.7	0.5	0.8	1.9	18.9	4.3	0.1
			Queue Length (in feet)	5	55	17.5	12.5	20	47.5	472.5	107.5	2.5
			Storage Length	150	650*	40		500*	50	200	390*	200

***distance (feet) of THRU lane from intersection stop bar to nearest cross street or driveway in back of queue.**

Source: Synchro HCM 6th ed. and PRISM Engineering

Intersection Levels of Service, Year 2044 Scenarios

Table 14 summarizes the future Year 2044 conditions scenario intersection level of service results for the normal adjacent street peak traffic for the am peak hour and pm peak hour, with and without the project traffic (a total of four scenarios are summarized in the table). During this time the Black Oak Mountain Winery property has minimal to no traffic volumes and this condition is reflected in Table 14 where the levels of service remain the same for plus project conditions as they are for the no project condition.

Table 14. Year 2044 *Peak Hour of Adjacent Street Traffic* Scenarios, Intersection Level of Service Analysis Results

	INTERSECTION LOCATION	Control	YEAR 2044 AM Peak 7:30-8:30 AM				YEAR 2044 PM Peak 5:15-6:15 PM			
			No PROJ		w/PROJ		No PROJ		w/PROJ	
			LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)
1	SR 49 at SR 193	AW	E	39.1	E	39.1	E	49.7	E	49.7
2	SR 193 at Upper Black Road	TW	A	0.8	A	0.8	A	0.6	A	0.6
		SB/NB	B	12.0	B	12.0	B	14.1	B	14.1
3	SR 193 at Black Oak Mountain Winery	TW	A	0.0	A	0.0	A	0.0	A	0.0
		NB	A	0.0	A	0.0	A	0.0	A	0.0
4	SR 193 at Penobscot Road	TW	A	0.5	A	0.5	A	0.3	A	0.3
		NB	B	10.8	B	10.8	B	11.9	B	11.9

Control: AW=All-Way Stop Control, TW=Tw-Way Stop Control with Stop Sign on Side Street, NB=NB approach Stop

NOTE: Calculations based on HCM 2010 6th ed. methdology for intersection level of service (AWSC and all-way stop and TWSC two-way)

Source: PRISM Engineering and HCM 6th ed. Analysis results

Figures 16A shows the afternoon 3:00 pm to 4:00 pm hourly traffic on the adjacent streets when the project arrival traffic would be at its peak. These volumes, however, do not include the project for this scenario. Figure 16B shows the total traffic volumes for the same time period but with the project volumes added in. Figure 17A shows the late evening peak traffic volumes that will take place when the project traffic exits to SR 193 between 10:00 pm and 11:00 pm. Figure 17B shows the plus project volumes for the same time period (10:00 pm top 11:00 pm).

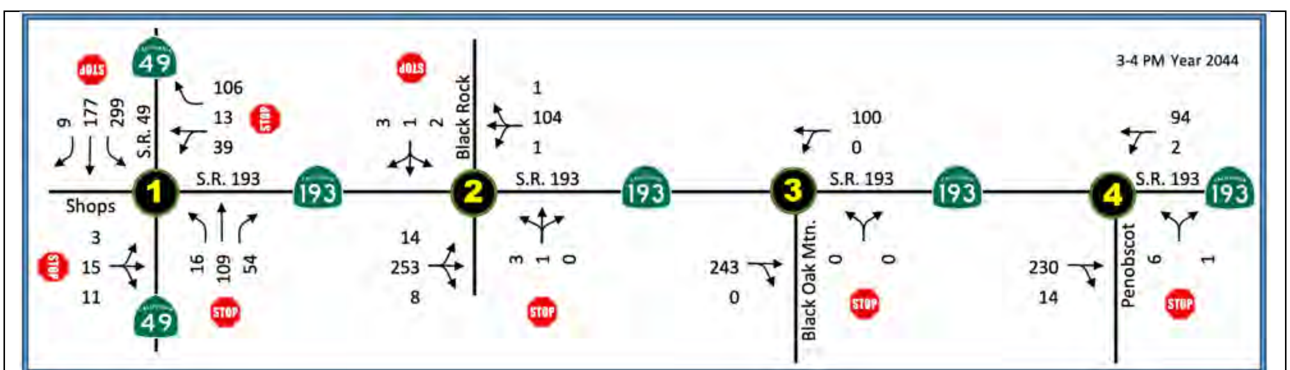


Figure 16A. 3:00-4:00 Afternoon Year 2044 Intersection Turn Volumes

Source: PRISM Engineering

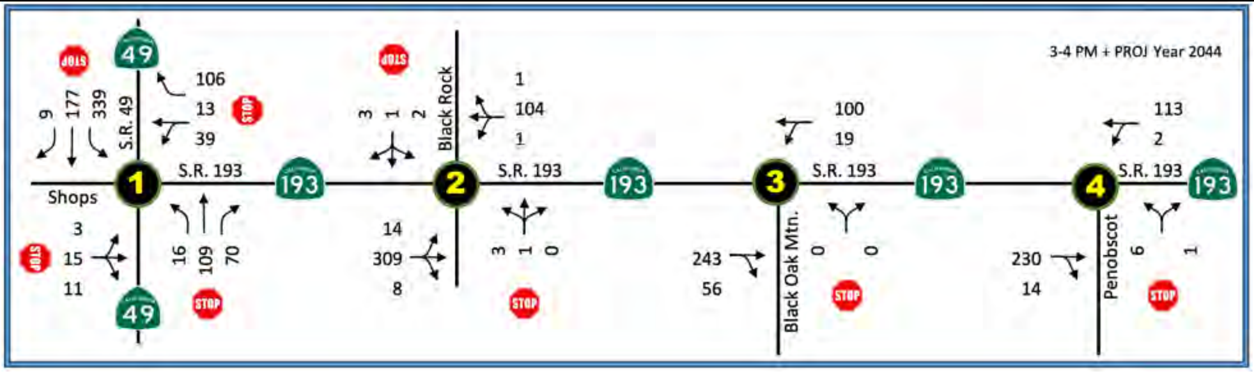


Figure 16B. 3:00-4:00 Afternoon Year 2044 Plus Project Volumes

Source: PRISM Engineering

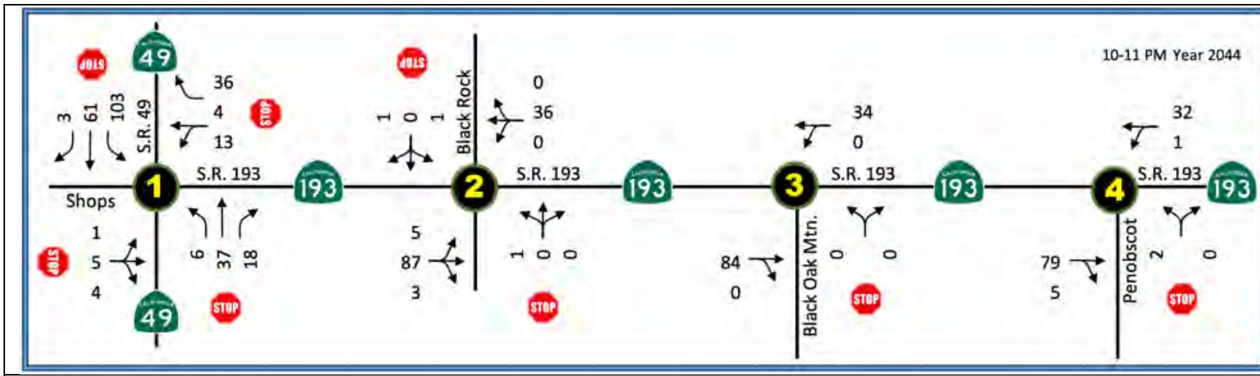


Figure 17A. 10:00-11:00 PM Year 2044 Intersection Turn Volumes

Source: PRISM Engineering

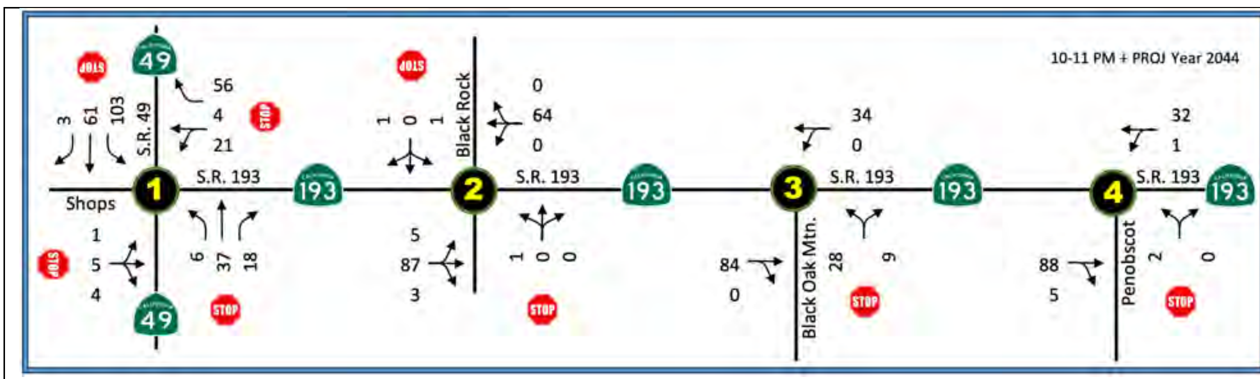


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Figure 17B. 10:00-11:00 PM Year 2044 Plus Project Volumes

Source: PRISM Engineering

These volumes were subsequently used in the Year 2044 No Project HCM 6th ed. capacity analysis scenarios using the Synchro version 12 software.

Table 15 summarizes the capacity analysis results of the future Cumulative "peak hour of generator" traffic Plus the Project. It should be clarified that these particular cumulative numbers represent growth that takes place during the afternoon (3:00 pm to 4:00 pm) and during the evening (10:00 pm to 11:00 pm). The traffic volumes used for all study intersections coincides with traffic levels that are estimated to exist between 3-4 PM for project arrival times, and between 10-11 PM for project departure times. These estimated volumes were based on factors documented in Appendix B, Hourly Traffic Counts on Study Area Vicinity Roadways.

Table 15. Year 2044 *Peak Hour of Generator* Traffic Scenarios, Intersection Level of Service Analysis Results

			YEAR 2044 INBOUND PEAK 3:00-4:00 PM				YEAR 2044 OUTBOUND PEAK 10:00-11:00 PM			
			No PROJ		w/PROJ		No PROJ		w/PROJ	
	INTERSECTION LOCATION	Control	LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)	LOS	Delay (secs)
1	SR 49 at SR 193	AW	B	14.7	C	17.4	A	8.5	A	8.6
2	SR 193 at Upper Black Road	TW	A	0.6	A	0.5	A	0.5	A	0.4
		SB/NB	B	11.7	B	12.3	A	9.4	A	9.6
3	SR 193 at Black Oak Mountain Winery	TW	A	0.0	A	0.4	A	0.0	A	2.2
		NB	A	0.0	A	0.0	A	0.0	A	9.2
4	SR 193 at Penobscot Road	TW	A	0.3	A	0.2	A	0.2	A	0.2
		NB	B	10.6	B	10.8	A	9.1	A	9.2

Control: AW=All-Way Stop Control, TW=Tw-Way Stop Control with Stop Sign on Side Street, NB=NB approach Stop

NOTE: Calculations based on HCM 2010 6th ed. methdology for intersection level of service (AWSC and all-way stop and TWSC two-way)

Source: PRISM Engineering and HCM 6th ed. Analysis results

Queue Analyses, Year 2044 AM and PM Peak Hour of Generator (3-4 pm and 10-11 pm)

Table 16 summarizes the queue analysis results for the Year 2044 3-4 pm inbound project, and 10-11 PM outbound project traffic conditions for the Peak Hour of the Generator (the Project's Peak Hour) at the SR 49 and SR 193 AWSC intersection (with and without the project).

It can be seen from Table 16 that none of the 95th percentile queue lengths shown for each scenario with or without the project exceed the available storage length at this late hour of 10-11 pm (the peak hour of the generator outbound traffic).

Table 16. Existing 2044 Scenarios for Peak Hour of Adjacent Street Traffic

95th percentile Queue Results *(HCM 6th ed. 95th-tile Q values shown in FEET on last row of scenario)*

	SCENARIO	Control	Lane	NBLn1	NBLn2	NBLn3	EBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
1	Year 2044 3-4 PM Peak Hour of Generator Traffic	AWSC	HCM Lane V/C Ratio	0.041	0.255	0.113	0.072	0.131	0.229	0.662	0.363	0.016
			HCM Control Delay, s/veh	10.3	11.8	9.6	10.5	11.2	10.9	21.1	12.1	8.1
			HCM Lane LOS	B	B	A	B	B	B	C	B	A
			HCM 95th-tile Q	0.1	1	0.4	0.2	0.4	0.9	4.9	1.6	0
			Queue Length (in vehicles)	0.1	1	0.4	0.2	0.4	0.9	4.9	1.6	0
			Queue Length (in feet)	2.5	25	10	5	10	22.5	122.5	40	0
			Storage Length	150	650*	40		500*	50	200	390*	200
2	Year 2044 3-4 PM Peak Hour of Generator plus Project Inbound Traffic	AWSC	HCM Lane V/C Ratio	0.041	0.26	0.149	0.075	0.135	0.237	0.759	0.369	0.016
			HCM Control Delay, s/veh	10.5	12.1	10	10.8	11.5	11.2	27.2	12.3	8.2
			HCM Lane LOS	B	B	A	B	B	B	D	B	A
			HCM 95th-tile Q	0.1	1	0.5	0.2	0.5	0.9	6.7	1.7	0
			Queue Length (in vehicles)	0.1	1	0.5	0.2	0.5	0.9	6.7	1.7	0
			Queue Length (in feet)	2.5	25	12.5	5	12.5	22.5	167.5	42.5	0
			Storage Length	150	650*	40		500*	50	200	390*	200
3	Year 2044 10-11 PM Peak Hour of Generator Traffic	AWSC	HCM Lane V/C Ratio	0.011	0.066	0.027	0.017	0.033	0.056	0.191	0.102	0.005
			HCM Control Delay, s/veh	8.5	8.3	7.3	8	8.6	7.6	9.4	8.2	6.9
			HCM Lane LOS	A	A	A	A	A	A	A	A	A
			HCM 95th-tile Q	0	0.2	0.1	0.1	0.1	0.2	0.7	0.3	0
			Queue Length (in vehicles)	0	0.2	0.1	0.1	0.1	0.2	0.7	0.3	0
			Queue Length (in feet)	0	5	2.5	2.5	2.5	5	17.5	7.5	0
			Storage Length	150	650*	40		500*	50	200	390*	200
4	Year 2044 10-11 PM Peak Hour of Generator Outbound Traffic	AWSC	HCM Lane V/C Ratio	0.011	0.068	0.028	0.018	0.05	0.089	0.194	0.104	0.005
			HCM Control Delay, s/veh	8.6	8.4	7.4	8.1	8.7	7.8	9.6	8.3	7
			HCM Lane LOS	A	A	A	A	A	A	A	A	A
			HCM 95th-tile Q	0	0.2	0.1	0.1	0.2	0.3	0.7	0.4	0
			Queue Length (in vehicles)	0	0.2	0.1	0.1	0.2	0.3	0.7	0.4	0
			Queue Length (in feet)	0	5	2.5	2.5	5	7.5	17.5	10	0
			Storage Length	150	650*	40		500*	50	200	390*	200

**distance (feet) of THRU lane from intersection stop bar to nearest cross street or driveway in back of queue.*

Source: Synchro HCM 6th ed. and PRISM Engineering

DEFICIENCIES for YEAR 2044 SCENARIOS

The intersection of SR 49 and SR 193 falls from LOS C to LOS E conditions with an average delay of 39.1 in the year 2044 AM Peak Hour condition, and 49.7 seconds per driver in the PM Peak Hour conditions without the project (see Table 11). For the “plus project” scenario during normal peak hours (7:15 am to 8:15 am) there is no project traffic to add in because it would have already arrived between 3:00 and 4:00 pm. The LOS results of this scenario do not change over the no project condition.

When the project traffic is added in to the estimated 3:00 to 4:00 afternoon peak cumulative traffic (see Table 12), the level of service for all locations is LOS B or better conditions, and no improvements are required.

The results shown in Table 11 have some failing levels of service, but the LOS E result at SR 49/SR 193 is 20 years out. During that time, traffic volumes may increase over existing projections (depending on development) and the traffic signal warrants could change, but at this time with the information we have a traffic signal is not warranted under any scenario for existing 2024 or year 2044 conditions. PRISM did develop, for information purposes only, several SR 49 at SR 193 intersection modification alternatives / concepts we used in making our analysis considerations complete, and to give guidance for any future considerations in future studies.

The intersection of SR 49 at SR 193 has many lanes for the various approaches, and this helps to offset some of the capacity issues that would take place with an otherwise small intersection size (such as long lines of traffic waiting to get through the intersection if it had one lane approaches). The modified lane striping was developed in CAD to determine what might be possible for intersection modifications by changing the lane striping only, as well as what would happen if a traffic signal were installed. Figures 18 and 18B illustrate this concept. We found that a traffic signal would mitigate an LOS F condition at this location (after Year 2044 beyond the scope of this study) to an LOS C condition and would be an adequate final mitigation for the intersection in the very long term. In other words, the intersection already has the pavement width and right-of way needed to install a traffic signal in the buildout scenario past the year 2044.

Unfunded West Slope Capital Improvement Projects. There were no programmed capital improvement projects for roadways in the study area. However, the County's CIP did show some public interest in trying to reduce the impact of truck and RV traffic, but there are no solutions proposed, nor is there any funding source for these projects. Hence, they are not approved or programmed. The table below is an extract from the County's CIP document.

OVERALL UNFUNDED CIP PROJECT LIST

	Project	Source	Type	Estimated Cost	Priority
26	GP177 - State Route 49 Passing Lanes from SR193 (in Cool) to the northern County Line	2015 CIP	Operations / Capacity	$3,482,000	Medium
	Suggested Projects not within County's purview				
162	Improvements to Reduce Impact of Truck and Recreational Vehicles on SR-49 - From Auburn to Cool	Public Comment	Trucks		unknown
166	SR-49 between Coloma and Cool - Add bike lanes to be integrated into SR-49 between Coloma and Cool	Public Comment	Bike/Ped		unknown

Figure 18A Intersection of SR 49 and SR 193, Signal Possibility After Year 2044

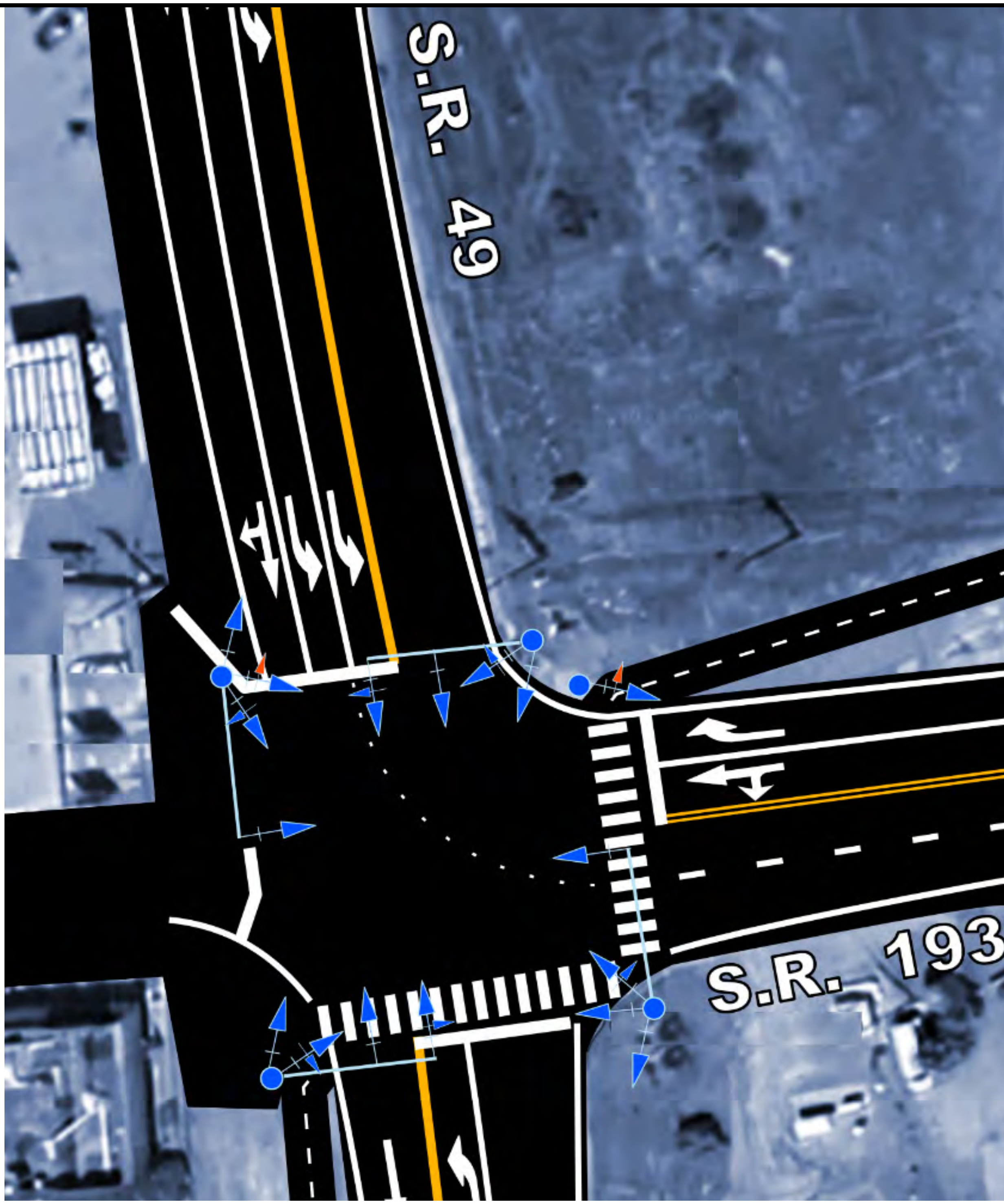

Figure 18B Close-up of SR 49 and SR 193 Mitigation: Signal Traffic Control

VEHICLE MILES TRAVELED (VMT) ANALYSIS

Due to the nature of the project being a wedding venue which typically have a wide reach for attendees not relating to a typical retail development that can be based on a theoretical gravity model of attractions and productions, but having a trip distribution that is random and is based solely on the random metric of where families and friends actually live, some even possibly being out of state. There is no traffic model that can replicate this with any scientific confidence and for this reason a qualitative VMT analysis was the appropriate method to choose for assessing and evaluating the VMT qualities relating to this project. The project is located in the California Sierra Foothills in the community of Cool, CA, and is situated regionally between two major east-west freeway corridors (I-80 to the north, and US 50 to the south). The project location offers viable alternatives to the many other existing wedding venues in the region (described below) and would most likely result in shortening many of the travel trip lengths, or a VMT reduction.

Regulatory Background, SENATE BILL 743.
Senate Bill 743 (SB 743), found in Public Resources Code section 21099, provided a new way that transportation impacts are assessed or analyzed. The legislation directed the Governor's Office of Planning and Research (OPR) to look at different metrics for identifying transportation as a California Environmental Quality Act (CEQA) impact. A key element of SB 743 is the elimination of vehicle delay in CEQA (level of service (LOS)), and other similar measures of vehicular capacity and traffic congestion as a basis for determining significant impacts in an EIR, and traffic congestion is no longer considered an environmental impact in CEQA. This new law switched the focus from LOS to VMT for transportation impact analysis which helps to balance the needs of congestion management with CA statewide goals such as promoting infill development, improving public health through active transportation, and reducing as much as is possible, greenhouse gas emissions.

CEQA Guidelines on Analyzing VMT
Section 15064.3 of the CEQA Guidelines were adopted in December 2018 to implement SB 743, and became effective July 1, 2020. This section of the law describes how to evaluate a project's transportation impacts and provides options/methods for government agencies to assess transportation impacts in terms of a project's VMT. The government agency can choose the most appropriate methodology, use professional judgment based on substantial evidence and/or reasons, to adjust its VMT analysis accordingly. This is especially true in terms of a traffic model where a *quantitative* models or method is unavailable to properly estimate the VMT for a particular project. According to the law/code government agencies are allowed to assess VMT *qualitatively*:

> *If existing models or methods are not available to estimate the vehicle miles traveled for the particular project being considered, a lead agency may analyze the project's*

> *vehicle miles traveled qualitatively. Such a qualitative analysis would evaluate factors such as the availability of transit, proximity to other destinations, etc.*[14]

The Technical Advisory (TA) on Evaluating Transportation Impacts in CEQA issued by the Governor's Office of Planning and Research (OPR) in December of 2018 provides guidance on assessing VMT using different methods, significance thresholds, and mitigation measures. The TA indicates several ways to assess VMT. These include trip-based assessment, tour-based assessment, and assessing change in total VMT. The TA states that government agencies should analyze the effects of retail projects by assessing the change in total VMT because retail-type projects typically re-route travel from other retail destinations. For example, a retail project may result in changes to VMT (increase or decrease) depending on the existing travel patterns for a particular area. If a local shopping center for instance is constructed near to residential development that previously had to go 30 miles into the nearby city to shop at larger stores, this development would most likely attract most customers from the nearby community, and not from the distant city. This will result in a net decrease to VMT that was taking place before the shopping center went in. Using an estimated total change in VMT allows for analysis of whether a project is likely to divert existing trips, and what the net effect of those diversions will be on the total VMT.

Although the project in this report is not a retail use, but a special event venue, it will have a similar result in reducing VMT and due to its centralized location between many other similar venues, is very likely to competitively re-route travel from other similar destinations that are farther away (such as other wineries in the region, of which there are many, and which also typically have wedding venue options of one size or another). In this report the Project Special Use Permit CUP will be analyzed via the total change to VMT for similar uses using a *qualitative* method. The County's travel demand model, which PRISM Engineering has and has also reviewed and evaluated for use in determining VMT for this project, cannot properly be used to evaluate a unique project type such as a wedding venue. The model is built on the gravity model which works for most land uses where distance and time play a primary factor in where people go to from their homes (they typically go to closer schools, closer stores, closer social activities, etc.), and since a wedding venue is NOT based on the factor of being "closer" to family and friends who for all typical and practical purposes do not necessarily live near to each other, and sometimes from other states, etc., the gravity model is actually an inappropriate tool to use to evaluate VMT changes with and without this particular project. For this reason a qualitative analysis is used in this study.

Project Location Plays a Factor

The location of the Project Site, the Black Oak Mountain Winery, is *centrally* located between numerous other similar winery locations in the area and it's location is shown with a yellow dot in Figure 19 below. In the context of Vehicle Miles Traveled, it is important to determine if the project ***creates*** or ***induces*** additional travel. Since the project site is centrally located between numerous other wineries in the region most of which also offer wedding venues of differing sizes

[14] Section 15064.3 (b)(3)

and facilities, it is important to note and realize that there is already an abundance of similar wedding venues that creates a competition of destination. Since the Black Oak Mountain Winery is a destination location and will soon be an alternative to the numerous other similar venues, it will not create a significant impact to VMT since there is an abundance of similar trips to wineries and wedding events *already* on the road. The project may capture traffic from a wedding event that may have otherwise traveled much further to an alternative venue site since the Black Oak Mountain Winery venue is centrally located among so many others. For example, if the Black Oak Mountain Winery were located say, in Colfax, there is little to no possibility that this location would offer a *closer* alternative to the wineries that now exist in the denser population areas near Auburn and Lincoln, or in Placerville or the numerous wineries to the south and east towards Plymouth and beyond. Such a hypothetical location being a destination farther away from all others would result in an increase to overall VMT totals in the area. However, there are numerous wineries and similar event wedding venues in the Sierra Mountain range from Oakhurst on the south along the SR 49 corridor and traveling all the way up past Placerville and Auburn to Nevada City on the north on SR 49 and much further beyond.

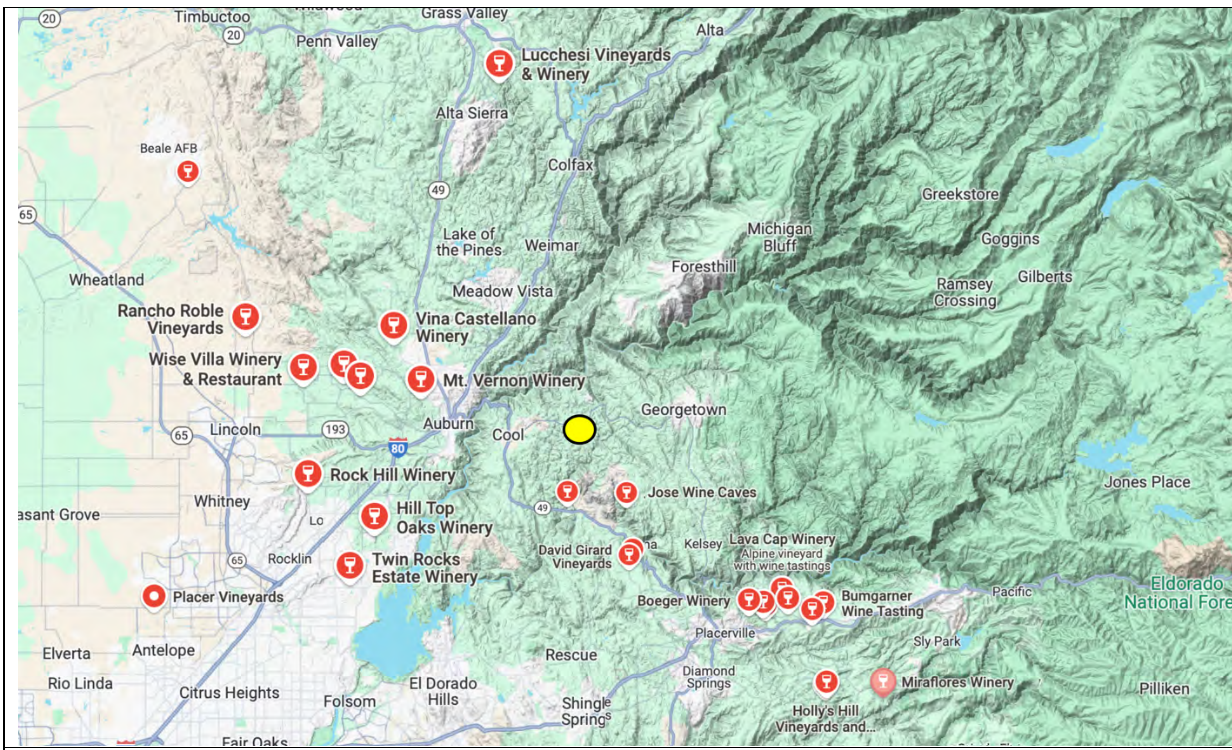

Figure 19. Various Winery Locations in Region

Having established that there are numerous vineyards with patrons that come from remote areas, and also visit many different vineyards for variety and activity, often traveling from one to the other, this type of traffic is already on the road. If some come to the Black Oak Mountain Winery instead it may actually lower VMT by providing an alternative closer venue to some of the larger population centers that would otherwise possibly gone to a winery past the Cool location. The Cool location actually offers an alternative for shorter wedding venue trips.

Methodology.

El Dorado County has discretion under CEQA to analyze, interpret, and apply its own thresholds of significance. Where existing models or methods are not available to estimate the vehicle miles traveled for a particular project, a lead agency may analyze the project's VMT qualitatively. In the case of the Black Oak Mountain Winery project special use permit for a wedding venue, the travel demand model cannot accurately estimate VMT for the Project because:

- The Travel Model cannot accurately capture the nature of a wedding venue because of its being super regional in nature (area of influence being much larger than El Dorado County).
- The ITE Trip Generation Manual does not have a trip generation rate for a wedding venue, and correspondingly the County's model also does not have an appropriate trip generating component based on a production or attraction (the crux of a travel demand model). The nature of a super-regional wedding venue that extends far beyond the cordon stations of El Dorado County cannot be replicated using a gravity-based model method.
- The inputs to the County's model are based on factors that have nothing to do with a wedding event. There is no employee count, no square footage, and most important of all, no ITE Trip Generation Rate that can be compared since one does not exist.
- Many comparative wineries and wedding venues within 20 or 30 mile radius of the Black Oak Mountain Winery are located outside of the model's region as shown in part within Figure 19.

VMT Measures Network Use and Efficiency.

It is important to note that VMT does not directly measure traffic operations such as LOS and turning movements, etc., but instead measures network use and efficiency. Because of this the County's travel demand model cannot account for the nuances of a wedding event's trips that are captured from all of the other numerous existing wedding venue trips throughout the region and which are in areas that are beyond the County's traffic model but are nonetheless relevant to the equation. There are highly subjective and numerous criteria that a bride and groom and their families use in their selection of a venue (availability, distance, amenities, size, beauty, etc.), and the location selection is the most subjective and unpredictable. In order to *qualitatively* account for VMT impacts, the total expected *change* in project VMT with and without the project must be assessed by answering the question "what is the net effect of the project on the ***area's*** VMT?" The threshold of significance for the proposed Project would be if there were to be a net increase in total area VMT due to the project. PRISM Engineering has qualitatively evaluated the Proposed Project's VMT impacts based on its location and proximity to other similar venues in the region in and beyond the boundaries of El Dorado County (as shown in Figure 19), and has determined the project will not create an increase to VMT in the region, explained in the sections that follow.

Analysis of VMT.

VMT is a measure of travel demand. There are many factors that can influence travel demand. PRISM Engineering carefully evaluated the specific trip generation for the project based on a wedding event that would have up to 150 guests and staff as the total attendance. Using qualitative methods we evaluated the super-regional area within the County and in the counties to the north (Placer) and even to the south (Amador) where it might be possible that driving patterns could change with the choice to select a closer location. It is important to understand the nature of wedding venue trips that would be generated by the Project, and by other similar and competing wedding venue locations (other similar and nearby attractions), and other project features that would further reduce VMT such as if the project is likely to divert existing trips that are already in play on the roadway, and what the effect of those diversions will be on VMT.

PRISM Engineering researched the locations of other wineries which also have wedding venue services of varying sizes using Google Map Search tool. We also researched the nature and patterns of traffic and times of day that wedding venues experience, and more especially the outdoor type that take place in part, at wineries and their beautiful outdoor settings around and near to the vineyard grounds. We calculated previously in this report the Trip Generation for this project based on a specific operations description and the number of guests (150 guests max). We assigned traffic to and from the site during hours when attendees would typically arrive (3 pm to 4 pm) and leave (10 pm to 11 pm) due to fixed outdoor noise constraints on the venue site's hours of operation where all noise must cease at 10 pm.

Vehicle trips to future wedding events at the Black Oak Mountain Winery location would most likely consist of attracting existing demand for wedding venue trips that would be redistributed and captured by the proposed Black Oak Mountain Winery Project. The project site being centrally located between numerous other wineries in the region to the north and south of Cool, CA, offers alternatives and will most likely effectively lower the total VMT for the region in El Dorado County by capturing some of this traffic that would have had to travel farther (such as into Placer County) and decreasing the distances that wedding venue attendees would otherwise have to travel (to venues on the further most regions such as to Placer County and beyond).

The proposed Special Use Permit CUP Project at Black Oak Mountain Winery for wedding events is not expected to substantially increase VMT since the area already includes numerous wineries with wedding event venues, and the project would capture a small fraction of the wedding venue trips that are already on the road generated by the existing nearby wedding venue attractions (as illustrated in Figure 19). Allowing for the approval of the Special Use Permit for the Black Oak Mountain Winery wedding venue location would also have the effect of diverting trips away from traveling to destinations *outside* the County, such as to Placer County to the north of Cool (Auburn, Grass Valley, etc.) and provide an alternative to El Dorado County residents that may have chosen a wedding venue in those regions instead, but changed their mind to opt for the Cool, CA location within El Dorado County. This means that the trip generating characteristics of the Black Oak Mountain Winery Special Use Permit for a wedding venue would not impact VMT but would attract and capture trips that are generated by existing wineries and their respective wedding venues throughout the region within the California Sierra Mountain foothills where vineyards are in abundance.

Finally, wedding venues are often a *group activity*, whereby carpooling is common and desirable, and therefore naturally achieves the State's goals of reducing VMT by inducing higher occupancy in vehicles coming to the site. We assumed a more conservative lower occupancy of only two people per car in our analyses, when it could easily be as high as three or four (or more) given the family nature of these events. This higher expected occupancy is a mitigation for VMT and in the case of wedding venues is a benefit to air quality compared to other activities such as shopping or work-related trips where vehicle occupancy is often one person.

Conclusions.

This VMT qualitative analysis assessed the potential change in VMT by looking at the projects expected impact on local VMT patterns and the projects impact on diverting trips from farther away destinations. The findings outlined in this section show that the proposed Black Oak Mountain Winery Special Use Permit CUP Project for a wedding venue would have a less than significant impact on VMT. The proposed Project wedding venue cannot be evaluated with the County's quantitative methods (using the travel demand model) because there is no corresponding land use that matches a wedding venue travel pattern, or a trip rate or trip generation. The ITE Trip Generation Manual also does not have a trip rate for a wedding venue due to the random and subjective criteria related to the same, and so this kind of land use is not represented in the current travel demand model. The model fails to capture the nature of wedding events and for this reason the Project required a qualitative evaluation. The proposed Special Event wedding venue trip generation analysis prepared earlier in this report were examined by reviewing the nature of wedding events and their corresponding trip generation characteristics. Overall, it was found that, the proposed Project is not expected to impact VMT because trips generated through the region for similar and competing wedding venue locations at wineries create a competition for existing such traffic that is already on the roadways for existing nearby wedding venue attraction destinations. Trips to the Project site are most likely to come from an alternative wedding venue location, resulting in a shorter overall average trip length and producing a less than significant VMT impact under this CEQA method. Most significant of all is the competitive and subjective nature of how a wedding venue is selected (Bride's and family's preference? Cost? Scheduling, etc.), and the Black Oak Mountain Winery location is centrally located among so many competing choices that it will not create or induce additional mileage that venue attendees must travel to get there, because the wedding venue traffic is already in play at so many nearby locations that are in competition with each other.

Appendix A. New Traffic Counts Taken by PRISM Engineering

SR 49 at SR 193, AM and PM PEAK HOUR VOLUMES

TRAFFIC COUNT SUMMARY, 15 MINUTE INTERVALS

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193			S.R. 49			Shopping Center			S.R. 49				Last
Interval	Time	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL	SBR	SBT	SBL	TOT	Hour
1	7:00 to 7:15	61	2	14	4	21	2	1	0	0	0	18	9	132	
2	7:15 to 7:30	107	6	27	6	47	2	0	1	1	1	13	27	238	
3	**7:30 to 7:45**	**91**	**6**	**22**	**5**	**42**	**0**	**0**	**1**	**0**	**0**	**25**	**28**	**220**	
4	**7:45 to 8:00**	**80**	**5**	**27**	**10**	**35**	**3**	**0**	**1**	**1**	**2**	**21**	**26**	**211**	**801**
5	**8:00 to 8:15**	**90**	**11**	**24**	**4**	**33**	**6**	**0**	**3**	**5**	**1**	**28**	**17**	**222**	**891**
6	**8:15 to 8:30**	**76**	**6**	**68**	**6**	**45**	**3**	**1**	**7**	**4**	**6**	**36**	**32**	**290**	**943**
7	8:30 to 8:45	13	2	17	11	15	0	0	2	1	0	7	10	78	801
8	8:45 to 9:00	10	3	15	12	13	1	1	1	0	2	8	11	77	667
	AM PEAK HOUR	**337**	**28**	**141**	**25**	**155**	**12**	**1**	**12**	**10**	**9**	**110**	**103**	**943**	
	7:30 - 8:30 AM														
											PHF for intersection			***0.813***	

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193			S.R. 49			Shopping Center			S.R. 49				Last
Interval	Time	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL	SBR	SBT	SBL	TOT	Hour
1	4:30 to 4:45	18	2	8	5	19	2	2	1	1	0	17	33	108	
2	4:45 to 5:00	21	3	20	22	20	1	0	2	0	2	25	60	176	
3	5:00 to 5:15	25	1	10	7	25	3	2	1	1	0	24	51	150	
4	**5:15 to 5:30**	**33**	**2**	**8**	**17**	**18**	**8**	**3**	**5**	**2**	**3**	**45**	**89**	**233**	**667**
5	**5:30 to 5:45**	**32**	**5**	**5**	**9**	**34**	**2**	**3**	**8**	**0**	**2**	**53**	**99**	**252**	**811**
6	**5:45 to 6:00**	**28**	**4**	**18**	**24**	**21**	**2**	**0**	**3**	**0**	**2**	**30**	**81**	**213**	**848**
7	**6:00 to 6:15**	**31**	**4**	**15**	**13**	**28**	**3**	**4**	**2**	**1**	**1**	**36**	**82**	**220**	**918**
8	6:15 to 6:30	26	1	20	19	22	1	2	8	2	3	31	49	184	869
	PM PEAK HOUR	**124**	**15**	**46**	**63**	**101**	**15**	**10**	**18**	**3**	**8**	**164**	**351**	**918**	
	5:15 - 6:15 PM														
											PHF for intersection			***0.911***	

SR 193 at Upper Black Rd, AM and PM PEAK HOUR VOLUMES

TRAFFIC COUNT SUMMARY, 15 MINUTE INTERVALS

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193			Black Rock Ln			S.R. 193			Upper Black Rock Rd				Last
Interval	Time	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL	SBR	SBT	SBL	TOT	Hour
1	7:00 to 7:15	0	58	0	1	0	1	0	20	1	2	0	2	85	
2	**7:15 to 7:30**	**1**	**70**	**0**	**2**	**0**	**1**	**1**	**28**	**2**	**3**	**0**	**3**	**111**	
3	**7:30 to 7:45**	**0**	**72**	**1**	**0**	**0**	**2**	**0**	**29**	**1**	**3**	**0**	**0**	**108**	
4	**7:45 to 8:00**	**1**	**79**	**1**	**1**	**0**	**0**	**0**	**25**	**0**	**2**	**0**	**2**	**111**	**415**
5	**8:00 to 8:15**	**1**	**95**	**0**	**1**	**0**	**1**	**1**	**14**	**1**	**2**	**0**	**1**	**117**	**447**
6	8:15 to 8:30	0	62	1	0	0	1	1	26	0	2	0	1	94	430
7	8:30 to 8:45	2	48	0	0	0	1	2	26	1	2	0	0	82	404
8	8:45 to 9:00	0	36	1	1	0	1	0	23	0	1	0	1	64	357
	AM PEAK HOUR	**3**	**316**	**2**	**4**	**0**	**4**	**2**	**96**	**4**	**10**	**0**	**6**	**447**	
	7:15 - 8:15 AM														

PHF for intersection ***0.955***

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193			Black Rock Ln			S.R. 193			Upper Black Rock Rd				Last
Interval	Time	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL	SBR	SBT	SBL	TOT	Hour
1	4:30 to 4:45	0	29	0	1	0	1	1	60	2	1	0	1	96	
2	4:45 to 5:00	1	24	0	1	0	1	2	66	4	1	0	0	100	
3	**5:00 to 5:15**	**0**	**28**	**0**	**0**	**0**	**2**	**2**	**75**	**3**	**2**	**0**	**1**	**113**	
4	**5:15 to 5:30**	**1**	**31**	**1**	**0**	**1**	**2**	**2**	**77**	**5**	**2**	**0**	**0**	**122**	431
5	**5:30 to 5:45**	**0**	**32**	**0**	**0**	**0**	**0**	**2**	**79**	**4**	**0**	**1**	**1**	**119**	454
6	**5:45 to 6:00**	**0**	**31**	**0**	**0**	**0**	**0**	**3**	**66**	**4**	**0**	**0**	**0**	**104**	**458**
7	6:00 to 6:15	0	36	0	0	0	0	0	66	0	0	0	0	102	447
8	6:15 to 6:30	0	33	0	1	0	1	1	60	2	1	0	0	99	424
	PM PEAK HOUR	**1**	**122**	**1**	**0**	**1**	**4**	**9**	**297**	**16**	**4**	**1**	**2**	**458**	
	5:00 - 6:00 PM														

PHF for intersection ***0.939***

SR 193 at BLACK OAK MOUNTAIN DRIVEWAY, AM and PM PEAK HOUR VOLUMES

TRAFFIC COUNT SUMMARY, 15 MINUTE INTERVALS

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193		PROJECT Driveway		S.R. 193			Last
Interval	Time	WBT	WBL	NBR	NBL	EBR	EBT	TOT	Hour
1	7:00 to 7:15	58	0	0	0	0	18	76	
2	**7:15 to 7:30**	**67**	**0**	**0**	**0**	**0**	**27**	**94**	
3	**7:30 to 7:45**	**69**	**0**	**0**	**0**	**0**	**26**	**95**	
4	**7:45 to 8:00**	**73**	**0**	**0**	**0**	**0**	**23**	**96**	**361**
5	**8:00 to 8:15**	**89**	**0**	**0**	**0**	**0**	**13**	**102**	**387**
6	8:15 to 8:30	63	0	0	0	0	26	89	382
7	8:30 to 8:45	51	0	0	0	0	24	75	362
8	8:45 to 9:00	38	0	0	0	0	22	60	326
	AM PEAK HOUR	**298**	**0**	**0**	**0**	**0**	**89**	**387**	

7:15 - 8:15 AM

PHF for intersection ***0.949***

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193		PROJECT Driveway		S.R. 193			Last
Interval	Time	WBT	WBL	NBR	NBL	EBR	EBT	TOT	Hour
1	4:30 to 4:45	27	0	0	0	0	57	84	
2	4:45 to 5:00	23	0	0	0	0	64	87	
3	**5:00 to 5:15**	**27**	**0**	**0**	**0**	**0**	**71**	**98**	
4	**5:15 to 5:30**	**30**	**0**	**0**	**0**	**0**	**74**	**104**	373
5	**5:30 to 5:45**	**32**	**0**	**0**	**0**	**0**	**76**	**108**	397
6	**5:45 to 6:00**	**28**	**0**	**0**	**0**	**0**	**65**	**93**	**403**
7	6:00 to 6:15	33	0	0	0	0	61	94	399
8	6:15 to 6:30	31	0	0	0	0	55	86	381
	PM PEAK HOUR	**117**	**0**	**0**	**0**	**0**	**286**	**403**	

5:00 - 6:00 PM

PHF for intersection ***0.933***

SR 193 at Penobscot Rd, AM and PM PEAK HOUR VOLUMES

TRAFFIC COUNT SUMMARY, 15 MINUTE INTERVALS

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193		Penobscot Rd		S.R. 193			Last
Interval	Time	WBT	WBL	NBR	NBL	EBR	EBT	TOT	Hour
1	7:00 to 7:15	60	1	2	2	1	21	87	
2	**7:15 to 7:30**	**64**	**0**	**3**	**3**	**2**	**25**	**97**	
3	**7:30 to 7:45**	**66**	**1**	**0**	**3**	**0**	**26**	**96**	
4	**7:45 to 8:00**	**72**	**0**	**2**	**1**	**0**	**23**	**98**	**378**
5	**8:00 to 8:15**	**87**	**1**	**1**	**2**	**0**	**13**	**104**	**395**
6	8:15 to 8:30	62	1	0	1	2	24	90	388
7	8:30 to 8:45	48	0	0	3	0	24	75	367
8	8:45 to 9:00	36	0	1	2	1	21	61	330
	AM PEAK HOUR	**289**	**2**	**6**	**9**	**2**	**87**	**395**	
	7:15 - 8:15 AM								

PHF for intersection **0.95**

PRISM ENGINEERING, Count taken: FEB 28, 2024 (WED)

		S.R. 193		Penobscot Rd		S.R. 193			Last
Interval	Time	WBT	WBL	NBR	NBL	EBR	EBT	TOT	Hour
1	4:30 to 4:45	26	0	1	1	2	55	85	
2	4:45 to 5:00	22	1	1	1	4	60	89	
3	**5:00 to 5:15**	**25**	**0**	**0**	**2**	**3**	**68**	**98**	
4	**5:15 to 5:30**	**28**	**1**	**0**	**2**	**4**	**70**	**105**	377
5	**5:30 to 5:45**	**29**	**0**	**0**	**3**	**4**	**72**	**108**	400
6	**5:45 to 6:00**	**28**	**1**	**1**	**0**	**5**	**60**	**95**	**406**
7	6:00 to 6:15	33	0	1	0	1	60	95	403
8	6:15 to 6:30	30	0	1	1	0	55	87	385
	PM PEAK HOUR	**110**	**2**	**1**	**7**	**16**	**270**	**406**	
	5:00 - 6:00 PM								

PHF for intersection **0.94**

Appendix B. Hourly Traffic Counts on Study Area Vicinity Roadways, Variances from Typical Peak Hours

Greenwood Road volumes during 5 pm to 6 pm are higher than the 2 pm to 3 pm hour on a Thursday (Peak of 93 at 6 pm, but 62 at 3 pm). At 10 pm the volume is 21 or about **23%** of the typical 6 pm peak hour traffic. This count is next to SR 193 (500' south) and is relevant to the SR 193 volume variances hourly *due to the residential nature of the road* and that SR 193 has *other numerous residential collectors or feeder roads into the highway*. Greenwood Road represents the travel characteristics of all of these other residential roadways connecting to SR 193. These are roadways providing a path to and from homes and SR 193, residential collector feeder roads such as Tegra Road, Upper Black Road, Sweetwater Trail, and numerous others.

Nearly all collector roads feeding into SR 193 between SR 49 and Placerville are residential in nature, and SR 193 primarily represents the only way or fastest way in and out of these collector roads which serve residential homes.

Greenwood Road is an excellent choice for representing the kinds of traffic volumes that come and go from SR 193 because it is one of the largest volume roads feeding into the state highway SR 193. With an hourly volume approaching and exceeding 100 vehicles per hour for many hours in the day, it is a representative sample of the volume variances that occur daily by time of day for homes along SR 193 in the region of the project site between Cool and Placerville.

Garden Valley Road is another residential collector road to the south of Greenwood that also directly accesses SR 193 on its south side towards Placerville, and is comparative to Greenwood Road being a residential collector. It had 50 vph at 6 pm on THU and 16 vph at 10 pm and dropping very low to 7 vph at 11 pm. This factor calculates to 10 pm to 11 pm traffic levels being very low compared to 6 pm as well.

*Conclusion: For this traffic study we assumed SR 193 at 10 pm is **23%** of the regular PM Peak Volumes. At 3 pm it was assumed that traffic levels are at **67%** of the regular PM Peak Volumes.*

EL DORADO COUNTY
DEPARTMENT OF TRANSPORTATION

Count Summary Beginning: **July 6, 2023**

Count Station:	1200056	Counter ID:	74
City/Town:	Greenwood	Mile Post:	**4.67**
Road Name:	**Greenwood Road**	Location:	**500 Ft. S. of S.R. 193**
Lanes:	2	Direction:	Combined

Date	9	10	11	12	6	7	8	Weekly	Wk Day
Day / Time	Sun	Mon	Tue	Wed	Thu	Fri	Sat	Average	Avg.
100	11	2	5	6	2	6	6	5	4
200	[illegible]	[illegible]	[illegible]	[illegible]	3	3	7	4	2
300	6	3	5	2	5	2	2	4	3
400	0	3	2	3	4	1	3	2	3
500	1	16	10	12	12	12	6	10	12
600	3	28	26	28	26	30	4	21	28
700	18	50	58	58	72	59	18	48	59
800	24	69	69	85	72	66	40	61	72
900	53	58	63	58	44	64	38	54	57
1000	80	47	66	60	57	50	68	61	56
1100	54	55	53	77	84	60	68	64	66
1200	73	72	57	71	72	60	48	65	66
1300	73	62	68	54	65	54	76	65	61
1400	69	73	76	66	66	65	82	71	69
1500	72	72	62	87	62	75	82	73	72
1600	46	82	79	90	85	122	52	79	92
1700	62	75	88	85	96	86	78	81	86
1800	52	88	96	102	93	82	73	84	92
1900	32	64	66	68	58	74	60	60	66
2000	36	36	40	38	51	52	40	42	43
2100	22	35	42	41	24	45	48	37	37
2200	23	21	32	40	21	36	32	29	30
2300	7	14	13	7	16	22	20	14	14
2400	10	7	11	9	3	12	12	9	8
Totals	833	1033	1088	1151	1093	1138	963	1043	1101
AM Peak Hr	10:00	12:00	8:00	8:00	11:00	8:00	10:00	12:00	8:00
AM Count	80	72	69	85	84	66	68	65	72
PM Peak Hr	1:00	6:00	6:00	6:00	5:00	4:00	2:00	6:00	6:00
PM Count	73	88	96	102	96	122	82	84	92

TOTAL ADT: **1,101**

EL DORADO COUNTY
DEPARTMENT OF TRANSPORTATION

Count Summary Beginning: **April 18, 2023**

Count Station:	1100053	Counter ID:	67
City/Town:	Garden Valley	Mile Post:	**0.05**
Road Name:	**Garden Valley Road**	Location:	**300 Ft. N. of S.R. 193**
Lanes:	2	Direction:	Combined

Date	23	24	18	19	20	21	22	Weekly	Wk Day
Day Time	Sun	Mon	Tue	Wed	Thu	Fri	Sat	Average	Avg.
100	3	1	1	1	1	2	6	2	1
200	3	3	0	0	3	0	0	1	1
300	1	5	0	0	1	1	2	1	1
400	1	1	2	5	1	0	2	2	2
500	0	3	6	5	3	4	0	3	4
600	3	12	1	5	9	8	1	6	7
700	9	28	25	28	21	24	6	20	25
800	9	62	53	49	57	49	18	42	54
900	15	29	21	28	27	26	31	25	26
1000	27	33	15	30	35	22	34	28	27
1100	20	23	28	31	24	28	36	27	27
1200	26	22	34	40	33	38	44	34	33
1300	44	38	28	35	32	43	39	37	35
1400	34	30	30	24	31	43	38	33	32
1500	25	51	41	46	56	40	38	42	47
1600	40	40	31	42	40	44	33	39	39
1700	33	46	43	43	57	42	28	42	46
1800	30	39	39	28	50	37	29	36	39
1900	26	28	24	29	24	30	18	26	27
2000	24	20	19	23	16	24	22	21	20
2100	17	15	17	12	26	16	12	16	17
2200	17	10	7	10	16	20	12	13	13
2300	8	4	7	6	7	6	13	7	6
2400	7	5	3	6	4	4	7	5	4
Totals	422	548	475	526	574	551	469	509	535
AM Peak Hr	10:00	8:00	8:00	8:00	8:00	8:00	12:00	8:00	8:00
AM Count	27	62	53	49	57	49	44	42	54
PM Peak Hr	1:00	3:00	5:00	3:00	5:00	4:00	1:00	3:00	3:00
PM Count	44	51	43	46	57	44	39	42	47

TOTAL ADT: 535

Garden Valley Road volumes during 2 pm to 3 pm are actually higher than the 5 pm to 6 pm hour on a Thursday (highest peak of 56 at 3 pm, but 50 at 6 pm). On other weekdays the 3 pm time is also similar to the 5 pm, probably due to school traffic and it being a remote and rural area where many kids are driven to and from school by a family member. *At 10 pm the volume is 16 or about **32%** of the typical 6 pm peak hour traffic, and at 11 pm the volume is only 7 vph or about **14%** of the 6 pm traffic levels.*

EL DORADO COUNTY
DEPARTMENT OF TRANSPORTATION

Count Summary Beginning: **April 18, 2023**

Count Station:	1150059	Counter ID:	75
City/Town:	Garden Valley	Mile Post:	**0.68**
Road Name:	**Black Oak Mine Road**	Location:	**3,600 Ft. E. of Marshall Rd.**
Lanes:	2	Direction:	Combined

Date	23	24	18	19	20	21	22	Weekly	Wk Day
Day / Time	Sun	Mon	Tue	Wed	Thu	Fri	Sat	Average	Avg.
100	10	9	7	6	3	8	13	8	7
200	4	3	1	5	2	5	8	4	3
300	2	7	0	3	4	2	6	3	3
400	4	5	6	6	12	6	6	6	7
500	1	6	6	17	10	6	7	8	9
600	11	55	46	54	46	43	7	37	49
700	12	103	124	127	129	108	25	90	118
800	27	259	261	265	274	242	51	197	260
900	102	167	189	194	173	161	107	156	177
1000	144	153	120	126	175	157	149	146	146
1100	152	125	116	120	150	131	147	134	128
1200	170	122	130	106	151	156	173	144	133
1300	185	126	122	144	164	157	167	152	143
1400	134	168	140	164	141	208	162	160	164
1500	143	229	202	222	239	221	150	201	223
1600	152	204	188	221	232	229	152	197	215
1700	120	205	188	247	224	193	146	189	211
1800	107	194	204	229	216	220	136	187	213
1900	80	123	131	130	168	134	90	122	137
2000	64	69	85	86	75	88	47	73	81
2100	44	71	79	87	51	70	53	65	72
2200	20	25	41	54	48	47	59	42	43
2300	26	19	24	30	24	37	32	27	27
2400	7	6	7	14	14	19	22	13	12
Totals	1721	2453	2417	2657	2725	2648	1915	2362	2580
AM Peak Hr	12:00	8:00	8:00	8:00	8:00	8:00	12:00	8:00	8:00
AM Count	170	259	261	265	274	242	173	197	260
PM Peak Hr	1:00	3:00	6:00	5:00	3:00	4:00	1:00	3:00	3:00
PM Count	185	229	204	247	239	229	167	201	223

TOTAL ADT: **2,580**

Black Oak Mine Road volumes during 2 pm to 3 pm are actually higher than the 5 pm to 6 pm hour on a Thursday (highest peak of 239 at 3 pm, but 216 at 6 pm). This is probably due to school traffic and it being a remote and rural area where many kids are driven to and from school by a family member. The 10 pm count on THU is on 48 cars. *This is of the typical 6 pm peak hour traffic*, and at 11 pm this drops to **11%** (24/216=11%).

Appendix C. Mitigations Considered but Not Used Due To Alignment Constraint Issues on SR 49 Southbound Through Movement

Appendix D. Synchro Version 12 HCM 6th Ed. Analysis Output Sheets

Included Under separate cover as Scenario Specific PDF files printed from Synchro Version 12 HCM 6th ed. reports output.