Traffic Impact Study for the West College Avenue Shopping Center

Prepared for the City of Santa Rosa

Submitted by
W-Trans

January 3, 2017

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Executive Summary

The proposed West College Avenue shopping center would result in 6,000 square feet of commercial retail space and a 4,000 square foot drive-through bank or coffee shop to be located on the southwest corner of West College Avenue/Cleveland Avenue in the City of Santa Rosa. The project's anticipated trip generation includes up to 1,903 daily trips on average, with 210 trips during the weekday a.m. peak hour and 111 trips during p.m. peak hour.

The study area includes the intersection of West College Avenue/Cleveland Avenue and the segments of West College Avenue and Cleveland Avenue bordering the project site. Analysis indicates that the study intersection is expected to operate at an acceptable level of service upon the addition of project-generated traffic to both existing and future volumes, resulting in a less-than-significant traffic impact. Similarly, pedestrian, bicycle and transit access are all expected to be adequate upon completion of currently planned facilities as well as those proposed as part of the project.

Vehicles will access the project site via a new driveway on West College Avenue. Two exit-only driveways are also provided on West College Avenue and Cleveland Avenue. A left-turn lane is warranted for inbound movement from West College Avenue. Sight distance is expected to be adequate in both directions at the driveways on West College Avenue, but any signs or landscaping installed near the driveway should be low-lying or set-back so that they do not impact the availability of clear sight lines. Sight lines at the Cleveland Avenue driveway are impeded by an existing building, so this outbound driveway should be restricted to right-turns only.

Introduction

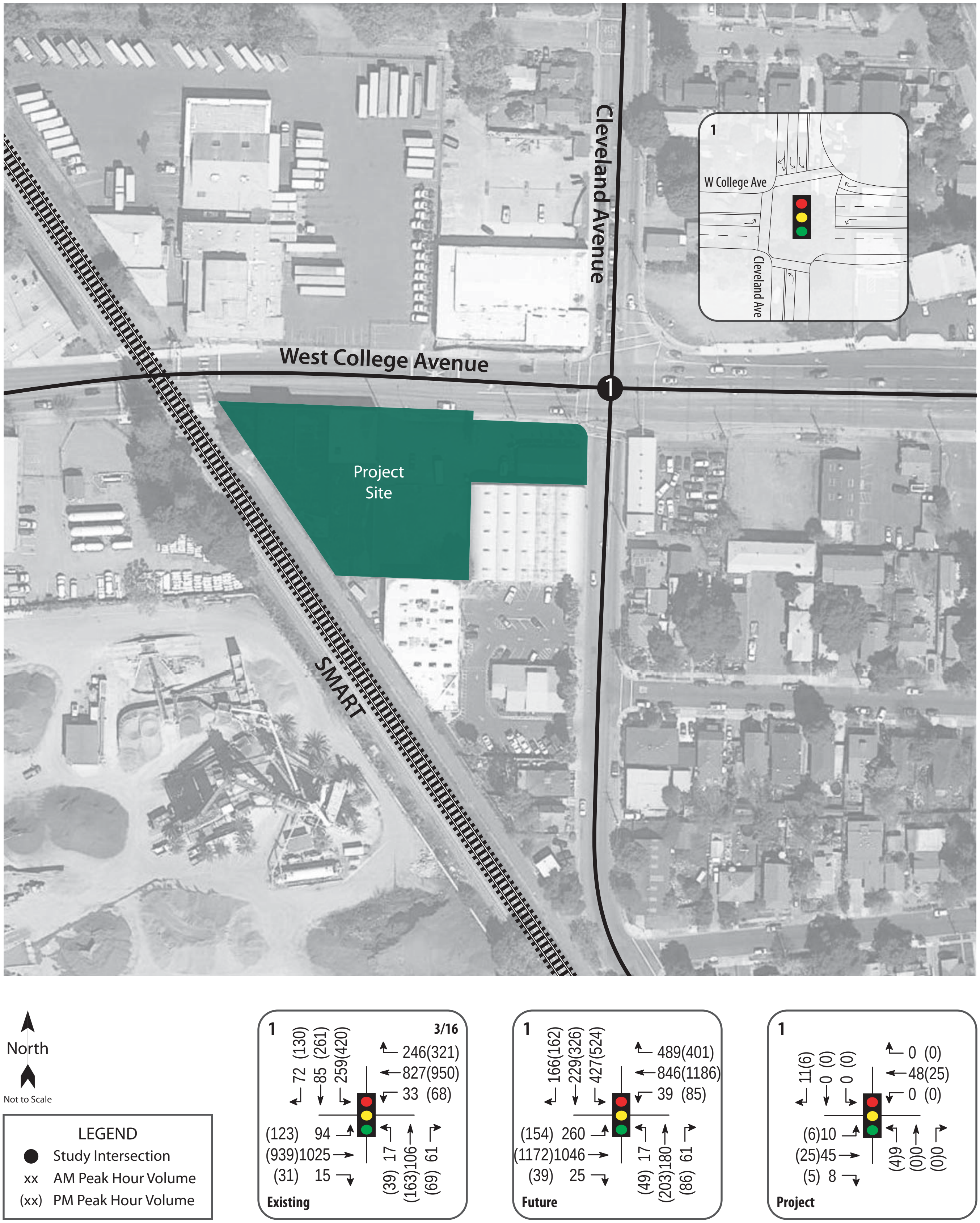
This report presents an analysis of the potential traffic impacts that would be associated with development of a proposed shopping center to be located at 6 West College Avenue in the City of Santa Rosa. The traffic study was completed in accordance with the criteria established by the City of Santa Rosa, reflects the scope of work approved by City staff, and is consistent with standard traffic engineering techniques.

Prelude

The purpose of a traffic impact study is to provide City staff and policy makers with data that they can use to make an informed decision regarding the potential traffic impacts of a proposed project, and any associated improvements that would be required in order to mitigate these impacts to a level of insignificance as defined by the City's General Plan or other policies. Vehicular traffic impacts are typically evaluated by determining the number of new trips that the proposed use would be expected to generate, distributing these trips to the surrounding street system based on existing travel patterns or anticipated travel patterns specific to the proposed project, then analyzing the impact the new traffic would be expected to have on critical intersections or roadway segments. Impacts relative to access for pedestrians, bicyclists, and to transit are also addressed.

Project Profile

The proposed project includes two new commercial buildings with 5,000 square feet of retail space and a 4,000 square foot drive-through bank or coffee shop. The approximately one-acre site is located on the southwest corner of West College Avenue/Cleveland Avenue and has two existing buildings that are currently vacant and would be demolished. The site will be accessed by new driveways on West College Avenue and Cleveland Avenue. The project site is located at 6 West College Avenue, as shown in Figure 1.



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Traffic Impact Study for the West College Avenue Shopping Center

Figure 1 – Study Area, Lane Configuration, and Traffic Volumes

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Transportation Setting

Operational Analysis

Study Area and Periods

The study area consists of the intersection of West College Avenue/Cleveland Avenue, the sections of West College Avenue and Cleveland Avenue fronting the project site and the project access points.

Operating conditions during the a.m. and p.m. peak periods were evaluated to capture the highest potential impacts for the proposed project as well as the highest volumes on the local transportation network. The morning peak hour occurs between 7:00 and 9:00 a.m. and reflects conditions during the home to work or school commute, while the p.m. peak hour occurs between 4:00 and 6:00 p.m. and typically reflects the highest level of congestion during the homeward bound commute.

Study Intersection

West College Avenue/Cleveland Avenue is a four-way signalized intersection with protected left-turn phasing at each approach. There are marked crosswalks on the north, south, and west legs of the intersection with pedestrian crossing signals.

Collision History

The collision history for the study area was reviewed to determine any trends or patterns that may indicate a safety issue. Collision rates were calculated based on records available from the California Highway Patrol as published in their Statewide Integrated Traffic Records System (SWITRS) reports. The most current five-year period available is March 1, 2010 through February 28, 2015.

The calculated collision rate for the study intersection was compared to average collision rates for similar facilities statewide, as indicated in *2012 Collision Data on California State Highways*, California Department of Transportation. There were 26 reported collisions at the intersection of West College Avenue/Cleveland Avenue. This equates to a collision rate of 0.41 collisions per million vehicles entering the intersection (c/mve) which is higher than the statewide average of 0.27 c/mve.

The most common type of collisions at West College Avenue/Cleveland Avenue were rear-end and broadside crashes, and a good number of them occurred on the southbound Cleveland Avenue approach. Observations indicate that collisions were primarily caused by unsafe speeding and violating traffic signals at the intersection. If increased enforcement at this intersection is possible, that could help to reduce the number of incidents. The collision rate calculations are provided in Appendix A.

Alternative Modes

Pedestrian Facilities

Pedestrian facilities include sidewalks, crosswalks, pedestrian signal phases, curb ramps, curb extensions, and various streetscape amenities such as lighting, benches, etc. In general, a network of sidewalks, crosswalks, pedestrian signals, and curb ramps provide access for pedestrians in the vicinity of the proposed project site; however, sidewalk gaps, obstacles, and barriers can be found along some or all of the roadways connecting to the project site. Existing gaps and obstacles along the connecting roadways impact convenient and continuous access for pedestrians and present safety concerns in those locations where appropriate pedestrian infrastructure would address potential conflict points.

- **West College Avenue** – Intermittent sidewalk coverage is provided on West College Avenue. The existing vacant building bordering the northwestern portion of the project currently sits closer to the street than the remainder of the project site, resulting in a narrow sidewalk on this segment of the project's frontage. This portion of sidewalk is approximately four feet wide, while the remaining sidewalk is approximately seven feet wide.

- **Cleveland Avenue** – Continuous sidewalks are provided along the east side of Cleveland Avenue between West College Avenue and Lincoln Street. Sidewalks are intermittent on the west side of the street, bordering the project. The existing building southeast of the project site sits at the edge of the curb, bordering the west side of Cleveland Avenue, which eliminates any available space for sidewalks on this segment. This building is *not* a part of the project, so this condition would remain upon completion of the project.

Curb ramps and crosswalks are available on the north, south, and west legs of the intersection of West College Avenue/Cleveland Drive. Lighting is provided by overhead street lights.

Bicycle Facilities

The *Highway Design Manual*, California Department of Transportation (Caltrans), 2012, classifies bikeways into three categories:

- **Class I Multi-Use Path** – a completely separated right-of-way for the exclusive use of bicycles and pedestrians with cross flows of motorized traffic minimized.

- **Class II Bike Lane** – a striped and signed lane for one-way bike travel on a street or highway.

- **Class III Bike Route** – signing only for shared use with motor vehicles within the same travel lane on a street or highway.

Guidance for Class IV Bikeways is provided in *Design Information Bulletin Number 89: Class IV Bikeway Guidance (Separated Bikeways/Cycle Tracks)*, Caltrans, 2015.

- **Class IV Separated Bikeway/Cycle Track** – a bikeway for the exclusive use of bicycles and requires physical separation such as grade separations, flexible posts, inflexible physical barriers, or on-street parking between the bikeway and through vehicular traffic.

In the project area, there are currently no bicycle facilities on West College Avenue or Cleveland Avenue within the project area, though there is an existing Class I path immediately west of the project site adjacent to the SMART tracks. A signalized crossing is provided where the path intersects West College Avenue. The 2010 *Santa Rosa Bicycle and Pedestrian Master Plan* outlines proposed bike lanes and bike routes within the project area. Table 1 summarizes the existing and planned bicycle facilities in the project vicinity, as contained in the Bicycle and Pedestrian Master Plan.

Table 1 – Bicycle Facility Summary				
Status **Facility**	**Class**	**Length (miles)**	**Begin Point**	**End Point**
Existing				
SMART Multi-Use Path	I	0.50	Eighth Street	College Avenue
Planned				
West College Avenue	II	0.57	SMART Multi-Use Path	Mendocino Avenue
Cleveland Avenue	II	0.62	Frances Street	Ninth Street
SMART Multi-Use Path	I	54	Sonoma County	Marin County

Source: *Santa Rosa Bicycle and Pedestrian Master Plan*, City of Santa Rosa, 2010

Transit Facilities

The Santa Rosa CityBus provides fixed route bus service in Santa Rosa. CityBus Route 3 provides loop service to destinations throughout the City and stops 200 feet east of the project site on West College Avenue at Cleveland Avenue and 500 feet west of the project site at /Maxwell Drive. Route 3 operates Monday through Friday with approximately one-half hour headways between 6:00 a.m. and 8:00 p.m. Saturday service operates with approximately one hour headways between 8:00 a.m. and 7:30 p.m. Sunday service operates with approximately on hour headways between 10:30 a.m. and 5:10 p.m.

Two bicycles can be carried on most Santa Rosa CityBus buses. Bike rack space is on a first come, first served basis. Additional bicycles are allowed on buses at the discretion of the driver.

Dial-a-ride, also known as paratransit, or door-to-door service, is available for those who are unable to independently use the transit system due to a physical or mental disability. Santa Rosa Paratransit is designed to serve the needs of individuals with disabilities within Santa Rosa and the greater Santa Rosa area.

Capacity Analysis

Intersection Level of Service Methodologies

Level of Service (LOS) is used to rank traffic operation on various types of facilities based on traffic volumes and roadway capacity using a series of letter designations ranging from A to F. Generally, Level of Service A represents free flow conditions and Level of Service F represents forced flow or breakdown conditions. A unit of measure that indicates a level of delay generally accompanies the LOS designation.

The study intersection was analyzed using the signalized methodology published in the *Highway Capacity Manual* (HCM), Transportation Research Board, 2010. This source contains methodologies for various types of intersection control, all of which are related to a measurement of delay in average number of seconds per vehicle. The signalized methodology is based on factors including traffic volumes, green time for each movement, phasing, whether or not the signals are coordinated, truck traffic, and pedestrian activity. Average stopped delay per vehicle in seconds is used as the basis for evaluation in this LOS methodology. For purposes of this study, delays were calculated using optimized signal timing.

The ranges of delay associated with the various levels of service are indicated in Table 2.

Table 2 – Signalized Intersection Level of Service Criteria	
LOS A	Delay of 0 to 10 seconds. Most vehicles arrive during the green phase, so do not stop at all.
LOS B	Delay of 10 to 20 seconds. More vehicles stop than with LOS A, but many drivers still do not have to stop.
LOS C	Delay of 20 to 35 seconds. The number of vehicles stopping is significant, although many still pass through without stopping.
LOS D	Delay of 35 to 55 seconds. The influence of congestion is noticeable, and most vehicles have to stop.
LOS E	Delay of 55 to 80 seconds. Most, if not all, vehicles must stop and drivers consider the delay excessive.
LOS F	Delay of more than 80 seconds. Vehicles may wait through more than one cycle to clear the intersection.

Reference: *Highway Capacity Manual*, Transportation Research Board, 2010

City of Santa Rosa

The City of Santa Rosa's adopted Level of Service (LOS) Standard is contained in *Santa Rosa General Plan 2035*. Standard TD-1 states that the City will try to maintain a Level of Service (LOS) D or better along all major corridors. Exceptions to meeting this standard are allowed where attainment would result in significant environmental degradation; where topography or environmental impacts make the improvement impossible; or where attainment would ensure loss of an area's unique character.

While a corridor level of service is applied by the City in its analysis of the entire City as part of the environmental documentation supporting the General Plan, this type of analysis only provides relevant data when performed on a much longer segment than the one included as the study area for the project. Therefore, although the City's standard does not specify criteria for intersections, for the purposes of this study a minimum operation of LOS D for the overall operation of signalized intersections was applied.

Existing Conditions

The Existing Conditions scenario provides an evaluation of current operation based on existing traffic volumes during the a.m. and p.m. peak periods. This condition does not include project-generated traffic volumes. Volume data was collected on March 1, 2016.

Under these existing volumes, the study intersection is operating acceptably at LOS C during both peak periods with an overall intersection delay of 20.8 seconds the a.m. peak hour and 30.2 seconds during the p.m. peak hour. The existing traffic volumes are shown in Figure 1 and copies of the Level of Service calculations for all scenarios are provided in Appendix B.

Future Conditions

Segment volumes for the horizon year of 2040 were obtained from the County's gravity demand model and translated to turning movement volumes at the study intersection using a the "Furness" method. The Furness method is an iterative process that employs existing turn movement data, existing link volumes and future link volumes to project likely turning future movement volumes at intersections.

Under the anticipated Future volumes the study intersection is expected to operate acceptably at LOS C during the a.m. peak hour with an overall intersection delay of 34.7. During the p.m. peak hour, the intersection is expected to have an average delay of 42.6 seconds and operate acceptably at LOS D. Future volumes are shown in Figure 1.

Project Description

The project consists of 5,000 square feet of commercial retail space and a 4,000 square foot space that could be used for either a drive-through bank or coffee shop. The site currently hosts two vacant buildings that will be demolished to make way for the project. As part of the proposed project a new left-turn lane would be striped on College Avenue providing access into the project's westerly driveway for westbound vehicles. The project will be accessed via new driveways on West College Avenue and Cleveland Avenue. The proposed project site plan is shown in Figure 2.

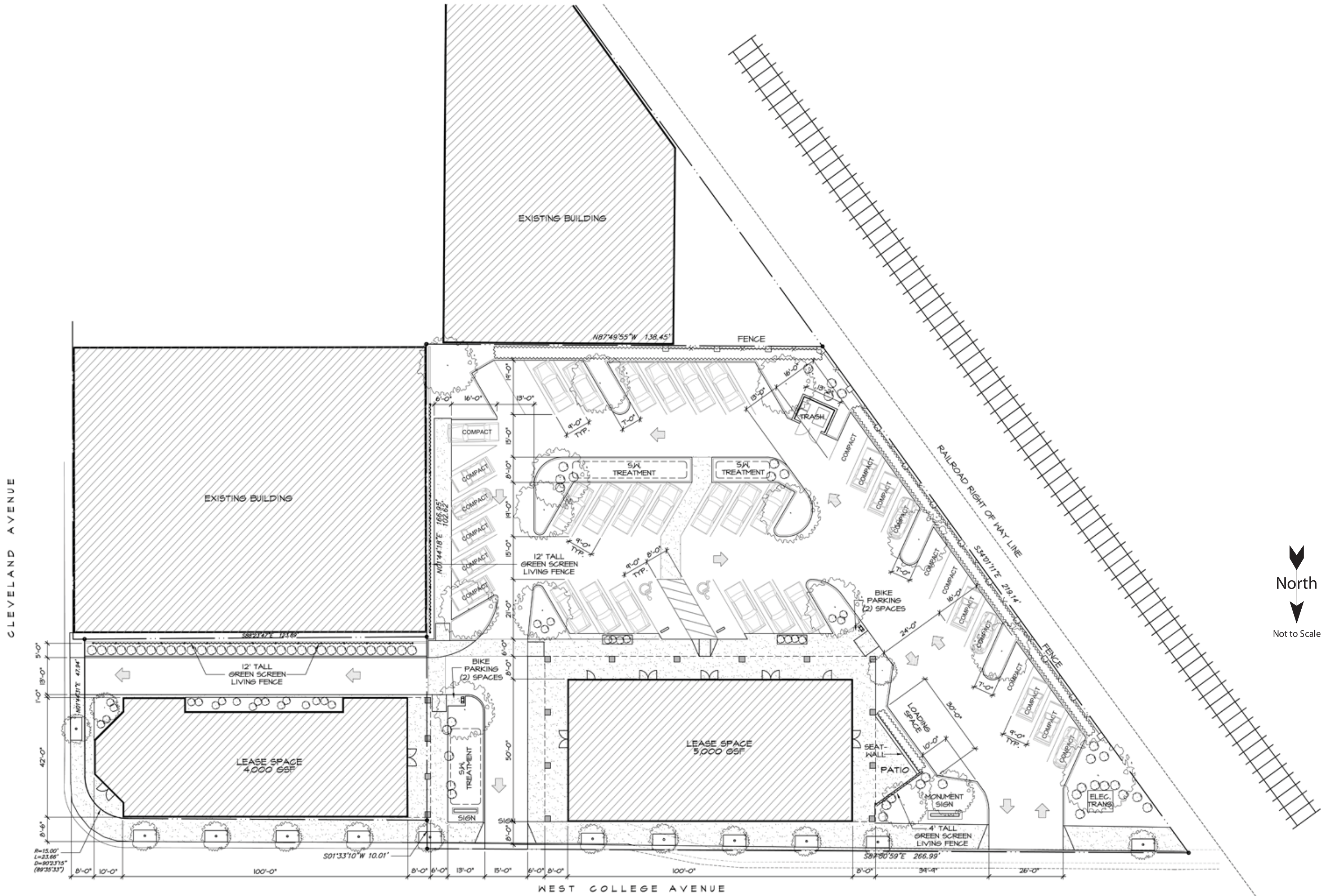
Trip Generation

Trip Generation

The anticipated trip generation for the proposed project was estimated using standard rates published by the Institute of Transportation Engineers (ITE) in *Trip Generation Manual*, 9th Edition, 2012 for "Specialty Retail Center" (ITE LU 826) in combination with "Shopping Center" (ITE LU 820) for the 5,000 square feet of retail space, and "Drive-In Bank" (ITE LU 912) or "Coffee/Donut Shop with Drive-Through" (LU 937) for the proposed 4,000 square foot drive-through bank or coffee shop.

Pass-by Trips

Some portion of traffic associated with a drive-through coffee shop would be drawn from existing traffic on nearby streets. These vehicle trips are not considered "new," but are instead comprised of drivers who are already driving on West College Avenue or Cleveland Avenue and choose to make an interim stop, and are referred to as "pass-by." The percentage of these pass-by trips was developed based on rates provided in the *Trip Generation Manual* for "Fast Food Restaurant with Drive-Through Window" (LU 934). This reference includes pass-by data collected at numerous locations for fast food restaurant uses. These rates were applied as a deduction to the overall trips generated by the project.



Source: Glass Architects 4/16

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Traffic Impact Study for the West College Avenue Shopping Center

Figure 2 – Site Plan

Total Project Trip Generation

The expected trip generation potential for the proposed project is indicated in Table 3. With a bank use, the proposed project would be expected to generate an average of 859 trips per day, including 54 trips during the a.m. peak hour and 113 during the p.m. peak hour. If a coffee shop use is proposed instead, the trip generation would include 1,903 net new daily trips, with 211 during the morning peak hour and 101 during the evening peak hour after the pass-by trips are deducted.

Since it has not been determined if the proposed drive-through will function as a bank or coffee shop, the a.m. peak hour trip generation for a drive-through coffee shop and the p.m. peak hour for a drive-through bank were used for project-generated traffic as these provide the most conservative analysis and would allow development of either to be covered by the analysis. Therefore, the proposed project would be expected to generate 1903 new daily trips with 211 during the a.m. peak hour and 113 during the p.m. peak hour.

Table 3 – Trip Generation Summary with a Bank

Land Use	Units	Daily		AM Peak Hour				PM Peak Hour			
		Rate	Trips	Rate	Trips	In	Out	Rate	Trips	In	Out
Proposed											
Retail	5 ksf	44.32	266	0.96	5	3	2	2.71	14	6	8
Drive-thru bank	4 ksf	148.15	593	12.08	48	28	20	**24.30**	**97**	**49**	**48**
Drive-thru Coffee Shop	4 ksf	818.58	**3274**	**100.58**	**402**	**205**	**197**	42.80	171	86	85
Pass-by		***50%***	***-1621***	***-49%***	***-197***	***-105***	***-100***	*-50%*	*-86*	*-43*	*-43*
Total			**1875**		**210**	**108**	**102**		**111**	**55**	**56**

Note: ksf = 1,000 square feet; **Bold** = values used for the traffic analysis

Trip Distribution

The pattern used to allocate new project trips to the street network was based on data from traffic counts collected on March 1, 2016. The applied distribution assumptions and resulting trips are shown in Table 4.

Table 4 – Trip Distribution Assumptions

Route	Percent	Daily Trips	AM Trips	PM Trips
College Ave (West)	38%	723	80	43
College Ave (East)	44%	838	93	50
Cleveland Ave (North)	10%	190	21	11
Cleveland Ave (South)	8%	152	17	9
TOTAL	**100%**	**1903**	**211**	**113**

The proposed project includes three access points. Two are located on College Avenue and one on Cleveland Avenue. The westerly driveway on College Avenue will accessible to inbound and outbound vehicles and is the only access point for vehicles entering the site. The easterly driveway on College Avenue and driveway on Cleveland will serve outbound traffic only. Based on the trip distribution assumptions presented in Table 4, trips were distributed amongst the three access points depending on their destination. Table 5 summarizes the resulting trips distributed at each access point.

Table 5 – Driveway Trip Distribution				
Route	**Percent of AM Trips**	**AM Trips**	**Percent of PM Trips**	**PM Trips**
Westerly College Ave. Driveway				
Inbound	51%	107	50%	56
Outbound	18%	38	19%	21
Easterly College Ave. Driveway (Outbound only)	26%	55	27%	30
Cleveland Ave. Driveway (Outbound only)	5%	10	4%	4
TOTAL		**210**	**100%**	**111**

Intersection Operation

Existing plus Project Conditions

Upon the addition of project-related traffic to the Existing volumes, the study intersection is expected to continue operating acceptably at LOS C during both peak periods. These results are summarized in Table 6. Project traffic volumes are shown in Figure 1.

Table 6 – Existing and Existing plus Project Peak Hour Intersection Levels of Service								
Study Intersection	**Existing Conditions**				**Existing plus Project**			
	AM Peak		**PM Peak**		**AM Peak**		**PM Peak**	
	Delay	**LOS**	**Delay**	**LOS**	**Delay**	**LOS**	**Delay**	**LOS**
1. W College Ave/Cleveland Ave	20.8	C	30.2	C	21.7	C	30.9	C

Notes: Delay is measured in average seconds per vehicle; LOS = Level of Service

Finding – The study intersections will continue operating acceptably with project traffic added, at the same Levels of Service as without it.

Future plus Project Conditions

Upon the addition of project-generated traffic to the anticipated Future volumes, the study intersection is expected to continue to operate acceptably. The Future plus Project operating conditions are summarized in Table 7.

Table 7 – Future (2040) and Future (2040) plus Project Peak Hour Levels of Service								
Study Intersection	**Future Conditions**				**Future plus Project**			
	AM Peak		**PM Peak**		**AM Peak**		**PM Peak**	
	Delay	**LOS**	**Delay**	**LOS**	**Delay**	**LOS**	**Delay**	**LOS**
1. W College Ave/Cleveland Ave	34.7	C	42.6	D	36.4	D	44.0	D

Notes: Delay is measured in average seconds per vehicle; LOS = Level of Service

Finding – The study intersections will continue operating acceptably with project traffic added at LOS D during both peak periods.

Alternative Modes

Pedestrian Facilities

Given the proximity of residences, transit stops, and the Smart Multi-Use path to the site, it is reasonable to assume that some project patrons and employees will want to walk, bicycle, and/or use transit to reach the project site.

Project Site – Sidewalks exist along the project frontage on West College Avenue and along the eastern border on Cleveland Avenue. The project will also realign the western half of the project frontage on West College Avenue with the eastern half which will widen the existing narrow sidewalk and provide a continuous sidewalk for pedestrians accessing the site. Additional pedestrian paths are provided through the project site, bordering the new buildings.

Finding – Pedestrian facilities serving the project site are expected to be adequate.

Bicycle Facilities

Per the City's *Bicycle and Pedestrian Master Plan*, Class II bike lanes are planned on West College Avenue and Cleveland Avenue. The proposed project includes plans to provide the planned Class II bike lane on West College Avenue. As designed, the location of the building at the easterly property line would preclude widening for a bike lane on Cleveland Avenue. It is noted that a bike lane along this segment would likely receive minimal use due to the proximity to the bike path adjacent to the SMART tracks, and there are other buildings so close to the sidewalk on both sides of the street that widening to provide bike lanes may be infeasible unless numerous properties are either redeveloped or purchased by the City. The City may wish to consider designating this section as a Class III facility to reflect the infeasibility of gaining width for bike lanes as well as the availability of a Class I path nearby.

Bicycle Storage

The project site plan identifies the provision of four bicycle parking spaces on-site. Per the City of Santa Rosa's Zoning Code (Standard 20-36.040), at least one bicycle parking space is required for every 5,000 square feet of retail space.

Finding – Bicycle facilities serving the project site will be adequate upon the implementation of the City's *Bicycle and Pedestrian Master Plan,* though it is noted that the City may wish to consider modifying the plan to designate Cleveland Avenue as a Class III bike route rather than planning for Class II bike lanes. The existing site plan includes plans to provide bicycle parking on-site.

Transit

Existing transit routes are adequate to accommodate project-generated transit trips. Existing stops are within acceptable walking distance of the site.

Finding – Transit facilities serving the project site are adequate.

Access and Circulation

Site Access

The project as proposed would have two driveways on West College Avenue, one two-way and one exit only, and one driveway on Cleveland Avenue for outbound vehicles exiting the drive-through window.

Sight Distance

Sight distances from the proposed project driveways on West College Avenue are expected to be adequate as there are no anticipated barriers to the lines of sight east and west of the planned driveways upon construction of the project. Any plans for new landscaping should ensure that plantings at the project driveways maintain clearance between three and seven feet in height to maximize sight lines.

Sight distance at the project driveway on Cleveland Avenue would not be adequate for left-turning vehicles exiting the project site as the existing building south of the site is located at the edge of the curb and restricts visibility of northbound traffic. This driveway should be restricted right turns outbound only.

Finding – Sight distance for both exiting and entering movements is adequate at the driveways on West College Avenue, but sight lines to the south on Cleveland Avenue are inadequate to allow left turns.

Recommendation – Due to sight distance restrictions at the Cleveland Avenue driveway, this outbound driveway should be restricted to right-turns only. Additionally, because landscaping and signs can impede clear sight lines, any new plantings or signs should be designed to ensure that adequate sight lines will be maintained at each driveway.

Access Analysis

Commercial vehicle circulation was analyzed to determine the largest vehicle that would be able to navigate the project site. It was determined that an SU-30 truck, which is the largest truck that would be expected to service the site, could adequately navigate the site using the planned entrance and exit on College Avenue. Turning movement diagrams are provided in Figure 3.

Left-Turn Lane Warrants

The need for left-turn lanes on West College Avenue at the project driveway was evaluated based on criteria contained in the *Intersection Channelization Design Guide*, National Cooperative Highway Research Program (NCHRP) Report No. 279, Transportation Research Board, 1985, as well as a more recent update of the methodology developed by the Washington State Department of Transportation. The NCHRP report references a methodology developed by M. D. Harmelink that includes equations that can be applied to expected or actual traffic volumes in order to determine the need for a left-turn pocket based on safety issues. With project generated traffic added to existing volumes a left-turn lane is warranted on West College Avenue at the proposed project driveway during both peak periods. As proposed, the project includes plans for a left-turn lane into the westerly driveway. A suggested left-turn lane design is provided in Figure 3.

Queuing

Project Driveway

Queuing at the project driveway was analyzed using a methodology contained in "Estimating Maximum Queue Length at Unsignalized Intersections," John T. Gard, *ITE Journal*, November 2001. Based on average a.m. peak hour

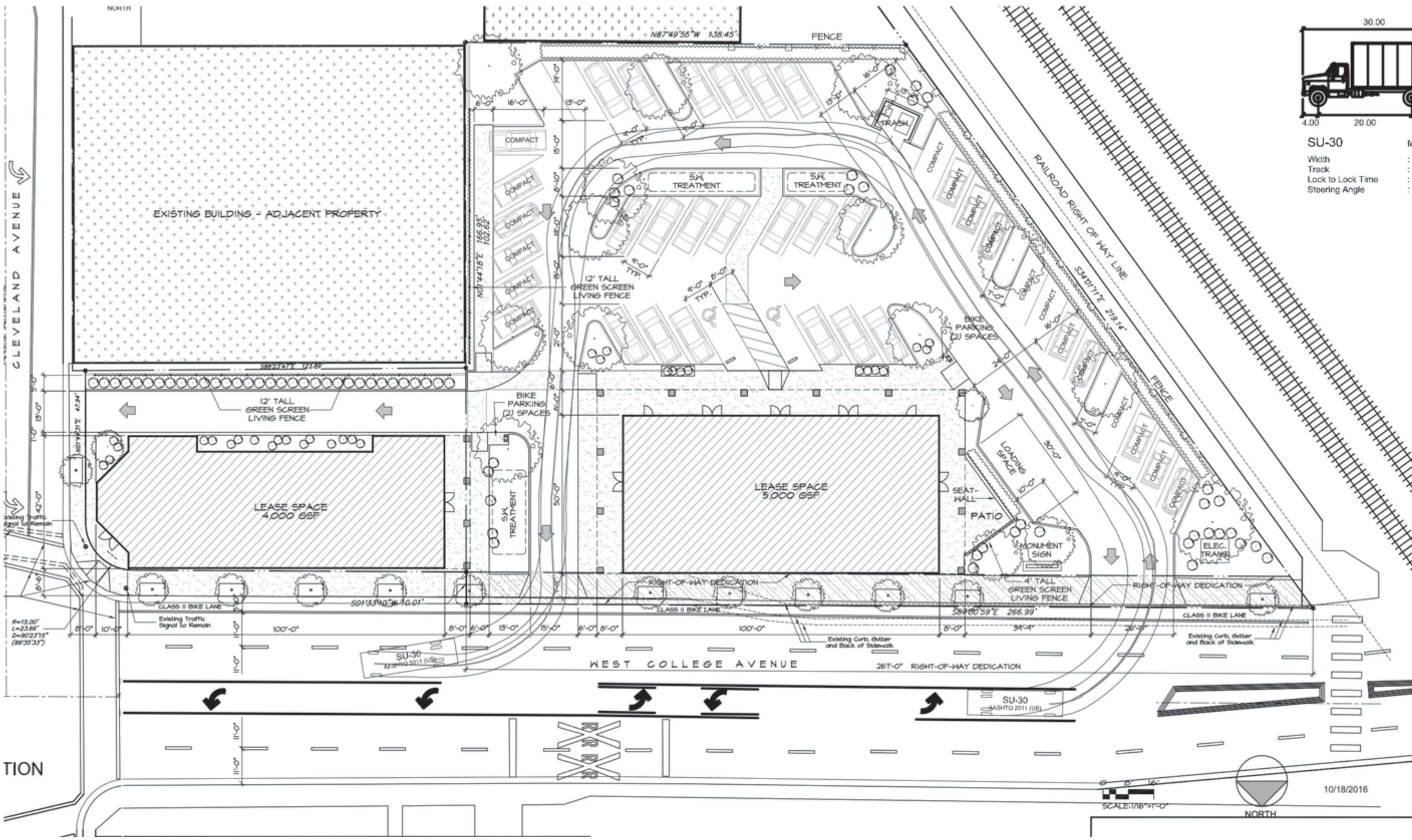

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Figure 3 – Recommended Left-Turn Channelization Layout

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volumes, the maximum queue was determined to be three vehicles. During the p.m. peak hour, the maximum peak was determined to be two vehicles. The planned left-turn lane into the project driveway would include a 50-foot left-turn pocket, which could accommodate approximately two vehicles, depending on their size. However, there is an additional 100 feet of storage capacity provided within the transition zone and center left-turn lane which would provide sufficient storage for the maximum queue at the project driveway.

Signalized Intersection

Under the existing plus project scenarios, the projected 95th percentile queue in the eastbound left-turn pocket on West College Avenue at the study intersection was determined using the SIMTRAFFIC application of Synchro, and averaging the maximum projected queue for each of ten runs. The project 95th percentile queue length is expected to be 153 feet during the a.m. peak hour and 156 feet during the p.m. peak hour. The eastbound left-turn pocket is currently 100 feet long and would be extended with the planned restriping through the installation of a center turn lane. The anticipated 95th percentile queue would exceed the existing storage length; however, the transition zone provided by the project would result in increased storage sufficient to adequately accommodate the expected queue length.

The projected queue lengths at both the project driveway and the intersection of West College Avenue/ Cleveland Avenue, although back-to-back, would not overlap during either peak period.

Finding – The project does not cause any queues to exceed available and planned storage. The anticipated queues at the project driveway and study intersection could be accommodated without would not be expected to overlap.

SMART Tracks

Consideration was given to the potential for project-generated traffic to stack onto, and block, the SMART tracks located 95 feet west of the site's westerly driveway. Drivers entering this driveway would be making a right turn, a movement that could be made with no delay unless there is a pedestrian on the sidewalk or another vehicle stopped in the drive aisle on-site. Given the limited potential for either of these to occur together with the short timer period during which they would exist, a queue is not expected to develop that would extend onto the railroad tracks as a result of eastbound inbound vehicles entering the site.

Drive-Through Operation

Based on the proposed site plan, the drive-through would have capacity for five vehicles to queue without stacking into the parking lot and seven without restricting circulation within the project site. If the queue extends beyond seven vehicles, however, on-site circulation would be impeded. Should stacking exceed the available capacity, the wait time would be such that most drivers would park and walk in rather than getting into a queue from which they cannot escape.

Standard queuing theory was used together with estimated service rates to determine the maximum number of vehicles that could be served without exceeding the storage space. A peak arrival rate of 103 vehicles per hour (one-half of customers) and a service rate of one minute were used for the coffee shop; this service rate was based on a previous study conducted for a drive-through coffee shop in Santa Rosa. However, these assumptions resulted in over-saturation of the drive-through. In order for the queue to be adequately accommodated within the drive-through storage, only 29 percent of inbound trips (59 vehicles) would be able to use the drive-through. At this point, with a full queue, additional vehicles would begin to stack up within the parking lot, blocking egress. As is typical of coffee shops, it is reasonable to anticipate that most customers will be repeat customers, so if experience has shown that excessive wait times are encountered when the queue exceeds seven vehicles, arriving customers will likely choose to walk in to order instead, or leave and purchase coffee elsewhere. However, if the service rate is better than assumed for this analysis, the wait times will be shorter and more customers may be acceptably served by the drive-through.

For the drive-through bank, it was assumed that 25 percent of the 49 inbound p.m. peak hour trips would use the drive-through. A service rate of four minutes was also assumed. Based on these assumptions, it was determined that the 95th percentile queue is expected to be seven vehicles. With capacity to store seven vehicles, the stacking space is sufficient to accommodate the queue associated with 14 vehicles using the drive-through.

The queuing analysis calculations are included in Appendix C.

Finding – The on-site stacking space for five vehicles is adequate to serve 59 vehicles per hour for a coffee shop and 14 vehicles per hour for a bank. Demand in excess of this would result in queuing that extends into the drive aisles and blocks access through the site. However, routine operation would result in either drivers avoiding the site or parking and walking in to avoid long waits at the drive-through, so while some delays and congestion could be encountered on-site, the queue could be contained on-site so would not impact operation of the adjacent street system.

Conclusions and Recommendations

Conclusions

- At buildout of the project, the site would generate an average of as many as 1,903 trips on a typical weekday, with 210 trips during the a.m. peak hour and up to 111 trips during the p.m. peak hour. The number of new trips would vary depending on whether the proposed drive-through is a bank or a coffee shop.
- The intersection of West College Avenue/Cleveland Avenue experienced collisions at an above-average rate for the five-year period evaluated. The bulk of these appear to be related to congestion.
- Under Existing conditions the intersection of West College Avenue/Cleveland Avenue operates at LOS C during both peak periods. The study intersection is expected to continue operating at the same level of service with the addition of project-generated trips.
- Under Future conditions the intersection of West College Avenue/Cleveland Avenue is expected to operate acceptably at LOS C during the weekday a.m. peak hour and LOS D for the p.m. peak hour. With the addition of project generated trips, the intersection is expected to operate acceptably at LOS D during both peak periods.
- Pedestrian, bicycle, and transit facilities are expected to adequately serve the project site upon completion of proposed pedestrian improvements and the addition of planned bike facilities contained in the City's bike plan.
- Sight distance at the project driveways on West College Avenue is adequate. Sight distance at the Cleveland Avenue driveway is not adequate for outbound vehicles to make a left turn because an existing building located on the southeast corner of the project site is located at the edge of the curb and restricts visibility to oncoming traffic.
- A left-turn lane is warranted on West College Avenue at the project driveway.
- The project does not cause any queues to exceed available and planned storage. The anticipated queues in the back-to-back turn lanes for the project driveway and intersection at West College Avenue/Cleveland Avenue would not be expected to overlap.
- The space available at the drive-through is sufficient for up to 59 vehicles an hour to use the drive through for a coffee shop or 14 if the space is developed as a bank. Demand in excess of these volumes would result in queuing that blocks the on-site drive aisles, though off-site traffic operation would not be impacted.

Recommendations

- A left-turn lane into the site from West College Avenue should be provided at the westerly driveway, as proposed.
- The easterly driveway on West College Avenue and the one on Cleveland Avenue should be restricted to right-turns only.
- Any new landscaping at the project driveways should be planted and maintained such that it is less than three feet or more than seven feet in height to maximize clear sight lines.

Study Participants and References

Study Participants

Principal in Charge	Dalene J. Whitlock, PE, PTOE
Assistant Planner	Shannon Baker
Graphics/Editing/Formatting	Angela McCoy

References

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SRO390

Appendix A

Collision Rate Calculations

Intersection Collision Rate Calculations

West College Avenue Shopping Center

Intersection # 1: West College Avenue & Cleveland Avenue
Date of Count: Tuesday, March 01, 2016

Number of Collisions: 26
Number of Injuries: 11
Number of Fatalities: 0
ADT: 35100
Start Date: March 1, 2010
End Date: February 28, 2015
Number of Years: 5

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{26 \times 1{,}000{,}000}{35{,}100 \times 365 \times 5}$$

	Collision Rate		Fatality Rate	Injury Rate
Study Intersection	**0.41**	**c/mve**	**0.0%**	**42.3%**
Statewide Average*	**0.27**	**c/mve**	**0.4%**	**41.9%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2012 Collision Data on California State Highways, Caltrans

Appendix B

Intersection Level of Service Calculations

HCM 2010 Signalized Intersection Summary
1: Cleveland Avenue & College Avenue

3/25/2016

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	94	1025	15	33	827	246	17	106	61	259	85	72
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	106	1152	17	37	929	276	19	119	69	291	96	81
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	137	1843	27	49	1652	898	30	163	94	392	227	192
Arrive On Green	0.08	0.52	0.52	0.03	0.47	0.47	0.02	0.15	0.15	0.11	0.25	0.25
Sat Flow, veh/h	1774	3569	53	1774	3539	1537	1774	1094	634	3442	924	780
Grp Volume(v), veh/h	106	571	598	37	929	276	19	0	188	291	0	177
Grp Sat Flow(s),veh/h/ln	1774	1770	1852	1774	1770	1537	1774	0	1728	1721	0	1704
Q Serve(g_s), s	4.9	19.1	19.1	1.7	15.7	7.6	0.9	0.0	8.6	6.8	0.0	7.2
Cycle Q Clear(g_c), s	4.9	19.1	19.1	1.7	15.7	7.6	0.9	0.0	8.6	6.8	0.0	7.2
Prop In Lane	1.00		0.03	1.00		1.00	1.00		0.37	1.00		0.46
Lane Grp Cap(c), veh/h	137	914	956	49	1652	898	30	0	257	392	0	419
V/C Ratio(X)	0.77	0.63	0.63	0.75	0.56	0.31	0.63	0.00	0.73	0.74	0.00	0.42
Avail Cap(c_a), veh/h	343	1196	1251	128	1965	1034	107	0	501	748	0	761
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	37.5	14.3	14.3	40.0	16.0	8.9	40.5	0.0	33.7	35.5	0.0	26.3
Incr Delay (d2), s/veh	9.0	0.7	0.7	20.5	0.3	0.2	19.3	0.0	4.0	2.8	0.0	0.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.7	9.4	9.8	1.1	7.7	3.2	0.6	0.0	4.4	3.4	0.0	3.5
LnGrp Delay(d),s/veh	46.5	15.0	15.0	60.5	16.3	9.1	59.7	0.0	37.6	38.3	0.0	27.0
LnGrp LOS	D	B	B	E	B	A	E		D	D		C
Approach Vol, veh/h		1275			1242			207			468	
Approach Delay, s/veh		17.6			16.0			39.7			34.0	
Approach LOS		B			B			D			C	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	13.4	16.3	6.3	46.8	5.4	24.4	10.4	42.7
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	8.8	10.6	3.7	21.1	2.9	9.2	6.9	17.7
Green Ext Time (p_c), s	0.7	1.7	0.0	21.7	0.0	2.2	0.1	18.9

Intersection Summary			
HCM 2010 Ctrl Delay			20.8
HCM 2010 LOS			C

HCM 2010 Signalized Intersection Summary
1: Cleveland Avenue & College Avenue

3/25/2016

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	123	939	31	68	950	321	39	163	69	420	261	130
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	129	988	33	72	1000	338	41	172	73	442	275	137
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	161	1568	52	92	1453	871	52	224	95	523	355	177
Arrive On Green	0.09	0.45	0.45	0.05	0.41	0.41	0.03	0.18	0.18	0.15	0.30	0.30
Sat Flow, veh/h	1774	3491	117	1774	3539	1535	1774	1231	523	3442	1165	581
Grp Volume(v), veh/h	129	501	520	72	1000	338	41	0	245	442	0	412
Grp Sat Flow(s),veh/h/ln	1774	1770	1838	1774	1770	1535	1774	0	1754	1721	0	1746
Q Serve(g_s), s	6.9	21.1	21.1	3.9	22.5	12.0	2.2	0.0	12.9	12.1	0.0	20.8
Cycle Q Clear(g_c), s	6.9	21.1	21.1	3.9	22.5	12.0	2.2	0.0	12.9	12.1	0.0	20.8
Prop In Lane	1.00		0.06	1.00		1.00	1.00		0.30	1.00		0.33
Lane Grp Cap(c), veh/h	161	795	826	92	1453	871	52	0	319	523	0	532
V/C Ratio(X)	0.80	0.63	0.63	0.78	0.69	0.39	0.79	0.00	0.77	0.84	0.00	0.77
Avail Cap(c_a), veh/h	293	1022	1061	110	1678	969	91	0	434	639	0	666
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	43.2	20.5	20.5	45.4	23.5	11.9	46.8	0.0	37.8	40.0	0.0	30.7
Incr Delay (d2), s/veh	8.9	0.8	0.8	25.4	1.0	0.3	23.0	0.0	5.6	8.6	0.0	4.5
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.8	10.5	10.9	2.6	11.2	5.1	1.4	0.0	6.8	6.4	0.0	10.7
LnGrp Delay(d),s/veh	52.1	21.3	21.3	70.9	24.5	12.2	69.8	0.0	43.4	48.6	0.0	35.2
LnGrp LOS	D	C	C	E	C	B	E		D	D		D
Approach Vol, veh/h		1150			1410			286			854	
Approach Delay, s/veh		24.8			23.9			47.2			42.1	
Approach LOS		C			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	18.8	21.6	9.0	47.6	6.8	33.6	12.8	43.8
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	14.1	14.9	5.9	23.1	4.2	22.8	8.9	24.5
Green Ext Time (p_c), s	0.6	2.7	0.0	20.5	0.0	3.5	0.2	15.3

Intersection Summary			
HCM 2010 Ctrl Delay			30.2
HCM 2010 LOS			C

HCM 2010 Signalized Intersection Summary
1: Cleveland Avenue & College Avenue

3/25/2016

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	260	1046	25	39	846	489	17	180	61	427	229	166
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	260	1046	25	39	846	489	17	180	61	427	229	166
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	268	1777	42	49	1345	812	26	228	77	499	301	219
Arrive On Green	0.15	0.50	0.50	0.03	0.38	0.38	0.01	0.17	0.17	0.14	0.30	0.30
Sat Flow, veh/h	1774	3530	84	1774	3539	1534	1774	1321	448	3442	996	722
Grp Volume(v), veh/h	260	524	547	39	846	489	17	0	241	427	0	395
Grp Sat Flow(s),veh/h/ln	1774	1770	1845	1774	1770	1534	1774	0	1769	1721	0	1718
Q Serve(g_s), s	15.4	22.1	22.1	2.3	20.6	23.5	1.0	0.0	13.8	12.8	0.0	22.0
Cycle Q Clear(g_c), s	15.4	22.1	22.1	2.3	20.6	23.5	1.0	0.0	13.8	12.8	0.0	22.0
Prop In Lane	1.00		0.05	1.00		1.00	1.00		0.25	1.00		0.42
Lane Grp Cap(c), veh/h	268	891	929	49	1345	812	26	0	306	499	0	520
V/C Ratio(X)	0.97	0.59	0.59	0.79	0.63	0.60	0.64	0.00	0.79	0.86	0.00	0.76
Avail Cap(c_a), veh/h	268	936	976	101	1538	896	84	0	401	585	0	600
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	44.7	18.6	18.6	51.2	26.7	17.5	51.9	0.0	41.9	44.2	0.0	33.4
Incr Delay (d2), s/veh	46.5	0.9	0.9	23.6	0.7	1.0	23.3	0.0	7.6	10.6	0.0	4.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	11.0	11.0	11.5	1.5	10.2	10.2	0.7	0.0	7.4	6.8	0.0	11.1
LnGrp Delay(d),s/veh	91.2	19.4	19.4	74.8	27.4	18.5	75.2	0.0	49.6	54.8	0.0	38.3
LnGrp LOS	F	B	B	E	C	B	E		D	D		D
Approach Vol, veh/h		1331			1374			258			822	
Approach Delay, s/veh		33.4			25.6			51.3			46.9	
Approach LOS		C			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	19.3	22.3	6.9	57.3	5.6	36.1	20.0	44.2
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	14.8	15.8	4.3	24.1	3.0	24.0	17.4	25.5
Green Ext Time (p_c), s	0.5	2.5	0.0	20.0	0.0	3.2	0.0	14.7

Intersection Summary			
HCM 2010 Ctrl Delay			34.7
HCM 2010 LOS			C

HCM 2010 Signalized Intersection Summary
1: Cleveland Avenue & College Avenue

3/25/2016

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	154	1172	39	85	1186	401	49	203	86	524	326	162
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	154	1172	39	85	1186	401	49	203	86	524	326	162
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	183	1592	53	94	1434	893	63	238	101	547	370	184
Arrive On Green	0.10	0.46	0.46	0.05	0.41	0.41	0.04	0.19	0.19	0.16	0.31	0.31
Sat Flow, veh/h	1774	3496	116	1774	3539	1583	1774	1243	527	3442	1176	584
Grp Volume(v), veh/h	154	593	618	85	1186	401	49	0	289	524	0	488
Grp Sat Flow(s),veh/h/ln	1774	1770	1842	1774	1770	1583	1774	0	1770	1721	0	1760
Q Serve(g_s), s	9.7	31.1	31.1	5.4	34.0	16.8	3.1	0.0	17.9	17.1	0.0	29.8
Cycle Q Clear(g_c), s	9.7	31.1	31.1	5.4	34.0	16.8	3.1	0.0	17.9	17.1	0.0	29.8
Prop In Lane	1.00		0.06	1.00		1.00	1.00		0.30	1.00		0.33
Lane Grp Cap(c), veh/h	183	806	839	94	1434	893	63	0	339	547	0	554
V/C Ratio(X)	0.84	0.74	0.74	0.90	0.83	0.45	0.78	0.00	0.85	0.96	0.00	0.88
Avail Cap(c_a), veh/h	251	875	911	94	1437	894	78	0	375	547	0	575
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	49.9	25.3	25.3	53.4	30.2	14.4	54.2	0.0	44.3	47.3	0.0	36.8
Incr Delay (d2), s/veh	16.6	3.0	2.9	62.8	4.1	0.4	31.8	0.0	15.9	28.2	0.0	14.4
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.6	15.8	16.5	4.3	17.5	7.4	2.1	0.0	10.2	10.3	0.0	16.7
LnGrp Delay(d),s/veh	66.5	28.3	28.2	116.2	34.3	14.8	86.0	0.0	60.1	75.5	0.0	51.2
LnGrp LOS	E	C	C	F	C	B	F		E	E		D
Approach Vol, veh/h		1365			1672			338			1012	
Approach Delay, s/veh		32.5			33.8			63.9			63.8	
Approach LOS		C			C			E			E	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	22.0	25.7	10.0	55.6	8.0	39.7	15.7	49.9
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	19.1	19.9	7.4	33.1	5.1	31.8	11.7	36.0
Green Ext Time (p_c), s	0.0	1.8	0.0	18.5	0.0	2.2	0.1	9.0

Intersection Summary			
HCM 2010 Ctrl Delay			42.6
HCM 2010 LOS			D

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	104	1070	23	33	875	246	26	106	61	259	85	83
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	117	1202	26	37	983	276	29	119	69	291	96	93
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	150	1861	40	48	1659	899	41	162	94	388	204	198
Arrive On Green	0.08	0.53	0.53	0.03	0.47	0.47	0.02	0.15	0.15	0.11	0.24	0.24
Sat Flow, veh/h	1774	3540	77	1774	3539	1537	1774	1094	634	3442	860	833
Grp Volume(v), veh/h	117	601	627	37	983	276	29	0	188	291	0	189
Grp Sat Flow(s),veh/h/ln	1774	1770	1847	1774	1770	1537	1774	0	1728	1721	0	1693
Q Serve(g_s), s	5.6	20.9	21.0	1.8	17.6	7.9	1.4	0.0	8.9	7.0	0.0	8.2
Cycle Q Clear(g_c), s	5.6	20.9	21.0	1.8	17.6	7.9	1.4	0.0	8.9	7.0	0.0	8.2
Prop In Lane	1.00		0.04	1.00		1.00	1.00		0.37	1.00		0.49
Lane Grp Cap(c), veh/h	150	930	971	48	1659	899	41	0	256	388	0	402
V/C Ratio(X)	0.78	0.65	0.65	0.76	0.59	0.31	0.70	0.00	0.73	0.75	0.00	0.47
Avail Cap(c_a), veh/h	330	1153	1204	124	1895	1002	103	0	483	721	0	729
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	38.6	14.6	14.6	41.5	16.8	9.2	41.7	0.0	35.0	36.9	0.0	28.1
Incr Delay (d2), s/veh	8.6	0.9	0.8	21.6	0.4	0.2	19.5	0.0	4.1	2.9	0.0	0.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.1	10.4	10.9	1.2	8.6	3.3	0.9	0.0	4.6	3.5	0.0	3.9
LnGrp Delay(d),s/veh	47.2	15.5	15.5	63.1	17.2	9.4	61.1	0.0	39.1	39.9	0.0	29.0
LnGrp LOS	D	B	B	E	B	A	E		D	D		C
Approach Vol, veh/h		1345			1296			217			480	
Approach Delay, s/veh		18.2			16.8			42.0			35.6	
Approach LOS		B			B			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	13.7	16.7	6.3	49.2	6.0	24.4	11.2	44.3
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	9.0	10.9	3.8	23.0	3.4	10.2	7.6	19.6
Green Ext Time (p_c), s	0.7	1.8	0.0	22.2	0.0	2.3	0.2	19.0

Intersection Summary	
HCM 2010 Ctrl Delay	21.7
HCM 2010 LOS	C

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	129	964	36	68	975	321	43	163	69	420	261	136
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	136	1015	38	72	1026	338	45	172	73	442	275	143
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	168	1582	59	92	1458	892	57	223	95	521	345	180
Arrive On Green	0.09	0.45	0.45	0.05	0.41	0.41	0.03	0.18	0.18	0.15	0.30	0.30
Sat Flow, veh/h	1774	3479	130	1774	3539	1583	1774	1242	527	3442	1156	601
Grp Volume(v), veh/h	136	516	537	72	1026	338	45	0	245	442	0	418
Grp Sat Flow(s),veh/h/ln	1774	1770	1840	1774	1770	1583	1774	0	1770	1721	0	1757
Q Serve(g_s), s	7.4	22.2	22.2	4.0	23.7	11.7	2.5	0.0	13.0	12.3	0.0	21.6
Cycle Q Clear(g_c), s	7.4	22.2	22.2	4.0	23.7	11.7	2.5	0.0	13.0	12.3	0.0	21.6
Prop In Lane	1.00		0.07	1.00		1.00	1.00		0.30	1.00		0.34
Lane Grp Cap(c), veh/h	168	805	837	92	1458	892	57	0	318	521	0	525
V/C Ratio(X)	0.81	0.64	0.64	0.78	0.70	0.38	0.79	0.00	0.77	0.85	0.00	0.80
Avail Cap(c_a), veh/h	288	1004	1044	108	1649	978	90	0	430	628	0	658
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	43.8	20.7	20.7	46.2	24.0	12.0	47.4	0.0	38.5	40.8	0.0	31.8
Incr Delay (d2), s/veh	8.9	0.9	0.9	26.3	1.2	0.3	20.8	0.0	5.8	9.1	0.0	5.4
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.0	11.0	11.4	2.6	11.8	5.1	1.5	0.0	6.8	6.6	0.0	11.2
LnGrp Delay(d),s/veh	52.7	21.7	21.6	72.5	25.2	12.2	68.2	0.0	44.4	49.9	0.0	37.2
LnGrp LOS	D	C	C	E	C	B	E		D	D		D
Approach Vol, veh/h		1189			1436			290			860	
Approach Delay, s/veh		25.2			24.5			48.1			43.7	
Approach LOS		C			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	18.9	21.7	9.1	48.9	7.2	33.5	13.4	44.7
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	14.3	15.0	6.0	24.2	4.5	23.6	9.4	25.7
Green Ext Time (p_c), s	0.6	2.7	0.0	20.6	0.0	3.4	0.2	15.0

Intersection Summary	
HCM 2010 Ctrl Delay	30.9
HCM 2010 LOS	C

HCM 2010 Signalized Intersection Summary
1: Cleveland Avenue & College Avenue

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	270	1091	33	39	894	489	26	180	61	427	229	177
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	270	1091	33	39	894	489	26	180	61	427	229	177
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	266	1769	54	49	1355	816	36	228	77	498	287	222
Arrive On Green	0.15	0.50	0.50	0.03	0.38	0.38	0.02	0.17	0.17	0.14	0.30	0.30
Sat Flow, veh/h	1774	3504	106	1774	3539	1534	1774	1321	448	3442	966	747
Grp Volume(v), veh/h	270	551	573	39	894	489	26	0	241	427	0	406
Grp Sat Flow(s),veh/h/ln	1774	1770	1840	1774	1770	1534	1774	0	1769	1721	0	1712
Q Serve(g_s), s	16.0	23.9	23.9	2.3	22.3	23.6	1.6	0.0	13.9	12.9	0.0	23.3
Cycle Q Clear(g_c), s	16.0	23.9	23.9	2.3	22.3	23.6	1.6	0.0	13.9	12.9	0.0	23.3
Prop In Lane	1.00		0.06	1.00		1.00	1.00		0.25	1.00		0.44
Lane Grp Cap(c), veh/h	266	893	929	49	1355	816	36	0	306	498	0	509
V/C Ratio(X)	1.02	0.62	0.62	0.79	0.66	0.60	0.73	0.00	0.79	0.86	0.00	0.80
Avail Cap(c_a), veh/h	266	929	966	100	1526	890	83	0	398	581	0	594
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	45.4	19.0	19.0	51.6	27.2	17.5	52.0	0.0	42.3	44.6	0.0	34.5
Incr Delay (d2), s/veh	59.1	1.2	1.1	23.6	0.9	1.0	24.3	0.0	7.8	10.9	0.0	6.6
Initial Q Delay(d3),s/veh	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	12.1	11.9	12.4	1.5	11.0	10.2	1.0	0.0	7.5	6.9	0.0	11.9
LnGrp Delay(d),s/veh	104.5	20.2	20.1	75.1	28.1	18.5	76.3	0.0	50.0	55.5	0.0	41.1
LnGrp LOS	F	C	C	E	C	B	E		D	E		D
Approach Vol, veh/h		1394			1422			267			833	
Approach Delay, s/veh		36.5			26.1			52.6			48.5	
Approach LOS		D			C			D			D	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	19.4	22.4	7.0	57.9	6.1	35.7	20.0	44.8
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	14.9	15.9	4.3	25.9	3.6	25.3	18.0	25.6
Green Ext Time (p_c), s	0.5	2.5	0.0	20.2	0.0	3.1	0.0	15.3

Intersection Summary		
HCM 2010 Ctrl Delay		36.4
HCM 2010 LOS		D

HCM 2010 Signalized Intersection Summary
1: Cleveland Avenue & College Avenue

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	160	1197	44	85	1211	401	53	203	86	524	326	168
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1900	1863	1863	1900
Adj Flow Rate, veh/h	160	1197	44	85	1211	401	53	203	86	524	326	168
Adj No. of Lanes	1	2	0	1	2	1	1	1	0	2	1	0
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	189	1595	59	93	1430	890	68	238	101	543	360	186
Arrive On Green	0.11	0.46	0.46	0.05	0.40	0.40	0.04	0.19	0.19	0.16	0.31	0.31
Sat Flow, veh/h	1774	3482	128	1774	3539	1583	1774	1243	527	3442	1160	598
Grp Volume(v), veh/h	160	608	633	85	1211	401	53	0	289	524	0	494
Grp Sat Flow(s),veh/h/ln	1774	1770	1840	1774	1770	1583	1774	0	1770	1721	0	1757
Q Serve(g_s), s	10.1	32.3	32.4	5.4	35.3	16.9	3.4	0.0	18.0	17.2	0.0	30.7
Cycle Q Clear(g_c), s	10.1	32.3	32.4	5.4	35.3	16.9	3.4	0.0	18.0	17.2	0.0	30.7
Prop In Lane	1.00		0.07	1.00		1.00	1.00		0.30	1.00		0.34
Lane Grp Cap(c), veh/h	189	810	843	93	1430	890	68	0	338	543	0	546
V/C Ratio(X)	0.85	0.75	0.75	0.91	0.85	0.45	0.78	0.00	0.85	0.96	0.00	0.90
Avail Cap(c_a), veh/h	249	869	904	93	1430	890	78	0	373	543	0	570
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	50.0	25.5	25.5	53.7	30.8	14.7	54.3	0.0	44.6	47.7	0.0	37.7
Incr Delay (d2), s/veh	18.2	3.4	3.3	64.6	4.9	0.4	34.6	0.0	16.2	29.6	0.0	17.6
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.9	16.5	17.1	4.3	18.1	7.4	2.3	0.0	10.4	10.5	0.0	17.5
LnGrp Delay(d),s/veh	68.2	28.9	28.8	118.4	35.7	15.0	88.9	0.0	60.8	77.3	0.0	55.2
LnGrp LOS	E	C	C	F	D	B	F		E	E		E
Approach Vol, veh/h		1401			1697			342			1018	
Approach Delay, s/veh		33.4			35.0			65.1			66.6	
Approach LOS		C			C			E			E	

Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	22.0	25.8	10.0	56.2	8.4	39.4	16.2	50.1
Change Period (Y+Rc), s	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Max Green Setting (Gmax), s	18.0	24.0	6.0	56.0	5.0	37.0	16.0	46.0
Max Q Clear Time (g_c+I1), s	19.2	20.0	7.4	34.4	5.4	32.7	12.1	37.3
Green Ext Time (p_c), s	0.0	1.8	0.0	17.8	0.0	1.9	0.1	7.9

Intersection Summary		
HCM 2010 Ctrl Delay		44.0
HCM 2010 LOS		D

Appendix C

Drive-Through Queuing Analysis Calculations

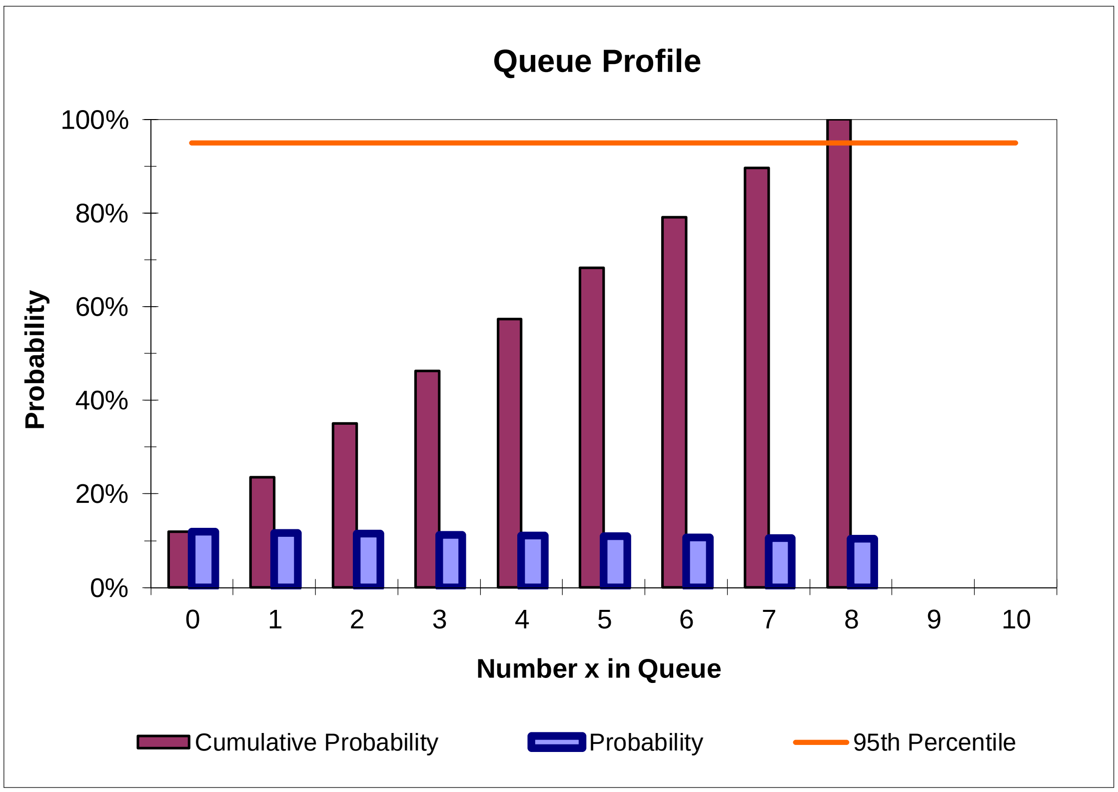
Drive Through Queueing Evaluation Worksheet

Project: Coffee shop with drive-through
Project No: SRO390

By: Shannon Baker
Date: 12/23/2016

Arrival Rate (veh/hr):	59	No. of Service Points:	1
Service Rate (veh/hr):	60	Queuing Capacity (veh):	7

Probability the System is Empty	12%
Probability the System is Full	10%
Probability That Customer Waits	88%
Average Time Customer Waits	4.4 minutes
Average Time Customer Waits To Get To Service Point	3.4 minutes
Probability That a Customer Elects Not to Enter the Queue	10%
Average In System	3.9 vehicles
Average Total Length of Vehicles in System	97 feet
95th Percentile in System	7 vehicles
95th Percentile Total Length of Vehicles in System	175 feet



Drive Through Queueing Evaluation Worksheet

Project: Bank with drive-through
Project No: SRO390

By: Shannon Baker
Date: 12/23/2016

Arrival Rate (veh/hr):	14	No. of Service Points:	1
Service Rate (veh/hr):	15	Queuing Capacity (veh):	7

Probability the System is Empty	14%	
Probability the System is Full	8%	
Probability That Customer Waits	86%	
Average Time Customer Waits	16.6	minutes
Average Time Customer Waits To Get To Service Point	12.6	minutes
Probability That a Customer Elects Not to Enter the Queue	8%	
Average In System	3.5	vehicles
Average Total Length of Vehicles in System	89	feet
95th Percentile in System	7	vehicles
95th Percentile Total Length of Vehicles in System	175	feet