Appendix G:
Traffic Analysis Report

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Hexagon Transportation Consultants, Inc.

Quick Quack Car Wash

Draft Transportation Analysis

Prepared for:

First Carbon Solutions

November 6, 2024

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Executive Summary

This report presents the results of the transportation analysis (TA) conducted for the proposed 3,588 square feet car wash at 2100 North Park Boulevard in Pittsburg, California. The project site is located about 800 feet south of the intersection of Loveridge Road and North Park Boulevard/California Avenue The proposed car wash would occupy an existing vacant lot. The project proposes a single tunnel car wash and vacuuming spaces. An existing driveway would provide access to the site.

The potential impacts of the project were evaluated in accordance with the standards and methodologies set forth by the City of Pittsburg, California Environmental Quality Act (CEQA), and Contra Costa Transportation Authority (CCTA). All new development projects in Pittsburg must evaluate their impact on the transportation system using the vehicle miles traveled (VMT) metric per CEQA requirements. The City of Pittsburg and the CCTA also require a local transportation analysis (LTA) that evaluates the potential transportation effects of the project on traffic operations in accordance with the standards and methodologies set forth by the City and CCTA.

VMT Analysis

Since the City of Pittsburg has not yet formally adopted VMT criteria, standards, or thresholds at the time this report was prepared, this assessment follows the Office of Planning and Research (OPR) and CCTA's current guidance related to VMT. The procedures for determining project impacts on VMT are based on the project description, characteristics, and location. If a project meets the screening criteria, it is then presumed that the project would result in a less-than-significant VMT impact, and a VMT analysis is not required. For a project that does not screen out, the project's VMT is compared to the appropriate thresholds of significance based on the project location and type of development.

Car washes typically serve the local surrounding community. There are approximately ten existing car washes in the Pittsburg and Antioch area, and patrons usually go to the car wash nearest their homes/places of work, along their commute route, or closest to them. Therefore, the average trip length of those land uses is short and generates low VMTs. Thus, the proposed project is considered a local serving use. In addition, the project qualifies as a small project since it is less than 10,000 square feet. Small projects and local serving projects are screened out, and therefore, the car wash is expected to have a less than significant VMT impact.

Project Trip Estimates

Based on the trip generation rates published in the Institute of Transportation Engineers' (ITE) *Trip Generation Manual*, 11th Edition the proposed project would generate 78 new trips (39 inbound and 39 outbound) during the PM peak hour

Intersection Traffic Operations

The results of the analysis show that all study intersections would operate at acceptable levels of service under all scenarios.

Table ES1
Intersection Level of Service Summary

#	Intersection	Intersection Control	Peak Hour	Existing Conditions		Existing Plus Project			Background Conditions		Background Plus Project		
				Delay [1] (sec)	LOS	Delay [1] (sec)	LOS	Incr. in Delay	Delay [1] (sec)	LOS	Delay [1] (sec)	LOS	Incr. in Delay
1	Project Driveway & North Park Boulevard	Side-Street Stop	PM	11.6	B	12.0	B	0.4	11.6	B	12.6	B	1.0
2	Loveridge Road & North Park Boulevard/California	Signal	PM	38.2	D	39.3	D	1.1	39.8	D	40.9	D	1.1

Notes:
[1] The delay reported for the signalized intersections is the average stopped delay for all vehicles entering the intersection. The delay reported for the side street stop controlled intersection is the delay experienced by vehicles on the stop controlled approach.

Other Transportation Issues

Hexagon conducted a site plan review, queuing analysis, pedestrian, bicycle, and transit facility analysis and parking analysis for the proposed project. The project would not have an adverse effect on the existing transit or on pedestrian and bicycle facilities in the study area. The proposed site plan shows adequate site access, and no adverse traffic operational issues are expected to occur at the project driveway as a result of the project.

The on-site circulation review shows that cars exiting the wash tunnel and turning right towards the vacuum stations won't be able to make the turn in one try. Cars would need to conduct a three-point movement to avoid the structure that separates the vacuum stations, which is undesirable and can result in collisions.

> **Recommendation**: It is recommended to remove two vacuum stations to allow sufficient space for vehicles to exit the tunnel and turn right towards the vacuum stations. In addition, a STOP sign should be placed at the end of the tunnel for exiting vehicles to give the right of way to cars leaving the site.

1.
Introduction

This report presents the results of the transportation analysis (TA) conducted for the proposed 3,588 square feet car wash at 2100 North Park Boulevard in Pittsburg, California. The project site is located about 800 feet south of the intersection of Loveridge Road and North Park Boulevard/California Avenue (see Figure 1). The proposed car wash would occupy an existing vacant lot. The project proposes a single tunnel car wash and vacuuming spaces. Access to the site would be provided by an existing driveway. The project's site plan is shown on Figure 2.

Scope of Study

This study was conducted to identify the potential transportation impacts of the project with respect to both the California Environmental Quality Act (CEQA), Contra Costa Transportation Authority (CCTA), and City of Pittsburg policies. All new development projects within Pittsburg must evaluate their impact on the transportation system using the vehicle miles traveled (VMT) metric per CEQA requirements. The City of Pittsburg and the CCTA also require a local transportation analysis (LTA) that evaluates the potential transportation effects of the project in accordance with the standards and methodologies set forth by the City and the CCTA.

Vehicle Miles Traveled

On July 15, 2020, the Contra Costa Transportation Authority (CCTA) adopted criteria, standards, and thresholds for the assessment of VMT (CCTA, Approval of the Vehicle Miles Traveled Analysis Methodology for Land Use Projects in the Growth Management Program, July 15, 2020). The methods and thresholds adopted by CCTA follow the guidance and recommendations of the Office of Planning and Research (OPR) pertaining to the implementation of SB 743.

As the City of Pittsburg has not yet formally adopted VMT criteria, standards, or thresholds at the time this report was prepared, this assessment follows the current OPR and CCTA guidance related to VMT. The procedures for determining project impacts on VMT are based on project description, characteristics, and location. VMT is the total miles of travel by personal motorized vehicles a project is expected to generate in a day. VMT measures the entire distance of personal motorized vehicle trips with one end within the project.

If a project meets the screening criteria, it is then presumed that the project would result in a less-than-significant VMT impact, and a VMT analysis is not required. For a project that does not screen out, the project's VMT is compared to the appropriate thresholds of significance based on the project location and type of development.

Figure 1
Site Location

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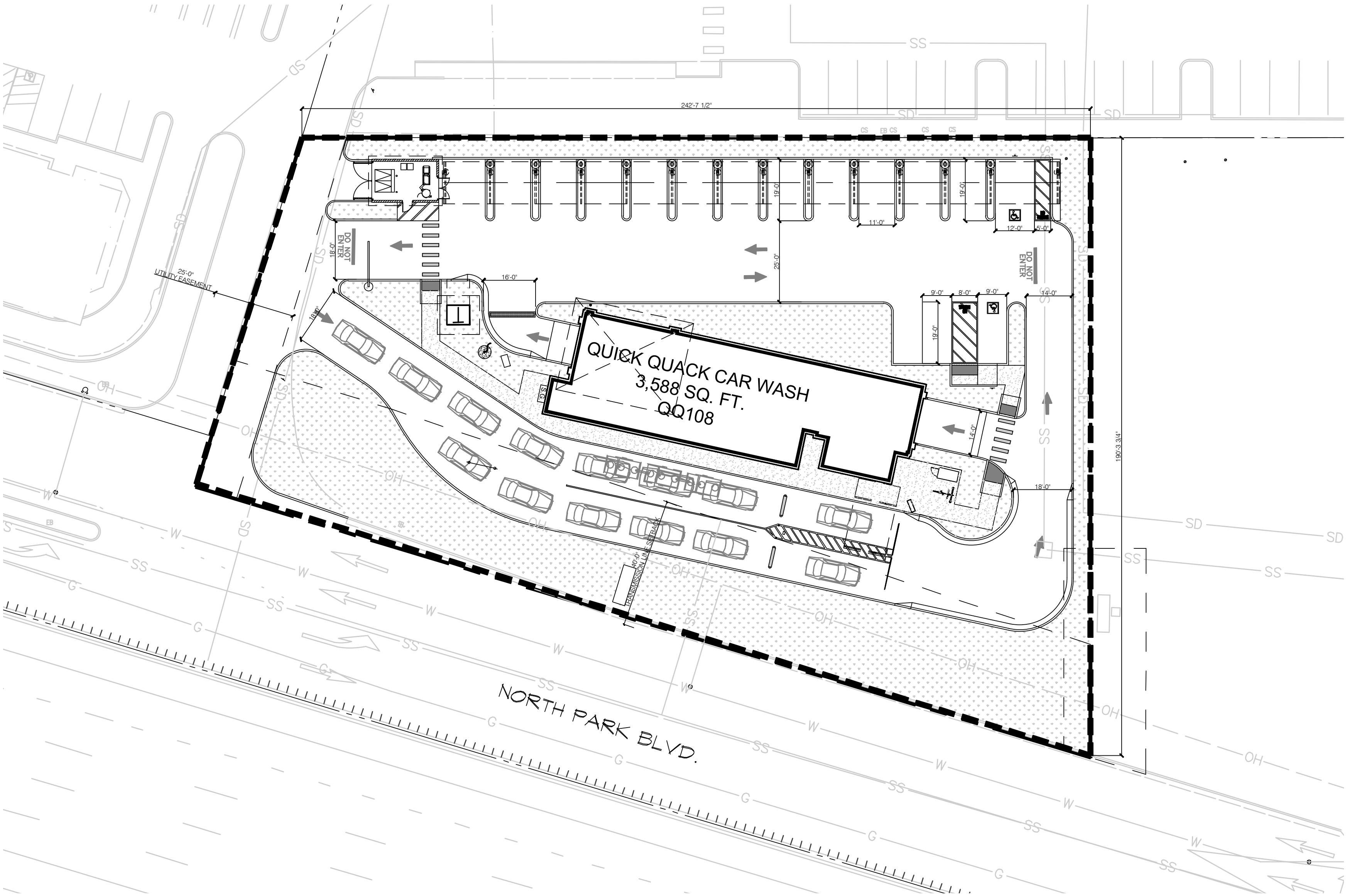

Figure 2
Site Plan

The policy's screening criteria state that the following types of projects may be presumed to have a less than significant VMT impact:

- **Small projects –** Small projects can be presumed to cause a less-than-significant VMT impact. Small projects are defined as having 10,000 square feet or less of non-residential space or 20 residential units or less, or otherwise generating less than 836 VMT per day.
- **Local-Serving Uses –** Projects that consist of Local-Serving Uses can generally be presumed to have a less-than-significant impact absent substantial evidence to the contrary since these types of projects will primarily draw users and customers from a relatively small geographic area that will lead to short-distance trips and trips that are linked to other destinations.
- **Projects Located in Transit Priority Areas (TPAs) –** Projects located within a TPA can be presumed to have a less-than-significant impact absent substantial evidence to the contrary.
- **Projects located in Low VMT Areas –** residential and employment-generating projects located within a low VMT-generating area can be presumed to have a less-than-significant impact absent substantial evidence to the contrary

Car washes typically serve the local surrounding community. There are approximately ten existing car washes in the Pittsburg and Antioch area, and patrons usually go to the car wash nearest their homes/places of work, along their commute route, or closest to them. Therefore, the average trip length of those land uses is short and generates low VMTs. Thus, the proposed project is considered a local serving use. In addition, the project qualifies as a small project since it is less than 10,000 square feet. Small projects and local serving projects are screened out, and therefore, the car wash is expected to have a less than significant VMT impact.

Local Transportation Analysis

The LTA component of the TA includes an intersection operational analysis to evaluate the traffic operational effects of the project on key intersections in the vicinity of the site. Two study intersections, as listed below, were selected to satisfy the requirements of the City of Pittsburg and the CCTA, which serves as the Congestion Management Agency (CMA) for Contra Costa County (see Figure 1).

1. Project Driveway and North Park Boulevard (unsignalized)
2. Loveridge Road and North Park Boulevard

Traffic conditions at the study intersections were analyzed for the weekday PM peak hour. The weekday PM peak hour is typically between 4:00 and 6:00 PM. It is during this period that the most congested traffic conditions occur on a typical weekday.

Intersection traffic conditions were evaluated for the following scenarios:

- ***Existing Conditions.*** Existing PM peak-hour traffic volumes were obtained from new turning-movement counts conducted on September 19, 2024 (see Appendix A).
- ***Existing Plus Project Conditions.*** Existing plus project traffic volumes were estimated by adding to the existing traffic volumes the additional traffic generated by the project. Existing plus project conditions were evaluated relative to existing conditions to determine potential adverse project effects.
- ***Background Conditions*****.** Background traffic volumes were estimated by adding to existing peak-hour volumes the projected volumes from future developments in the vicinity of the project.
- ***Background Plus Project Conditions.*** Background plus project traffic volumes were estimated by adding to background traffic volumes the additional traffic generated by the project.

Background plus project conditions were evaluated relative to background conditions to determine potential adverse project effects.

The LTA also includes a review of site access and on-site circulation, an evaluation of potential effects on transit, bicycle, and pedestrian facilities, queuing, and parking.

Intersection Operations Analysis Methodology

This section presents the methods used to determine the traffic conditions at the study intersections. It includes descriptions of the data requirements, the analysis methodologies, and the applicable intersection level of service standards.

Data Requirements

The data required for the analysis were obtained from new traffic counts and field observations. The following data were collected from these sources:

- Existing traffic volumes
- Lane configurations
- Signal timing and phasing
- List of future developments

Level of Service Analysis Methodology and Standards

Traffic conditions at the study intersections were evaluated using level of service (LOS). *Level of Service* is a qualitative description of operating conditions ranging from LOS A, or free-flow conditions with little or no delay, to LOS F, or jammed conditions with excessive delays. The analysis methods are described below.

Both study intersections were evaluated according to the requirements set forth by the CCTA using the methodology described in the *Technical Procedures* updated November 2022.

Signalized Intersections

The level of service at signalized intersections was based on the latest *Highway Capacity Manual (HCM)* level of service methodology using Synchro or HCS-signal software. The HCM method evaluates signalized intersection operations on the basis of average control delay for all vehicles at the intersection. The correlation between average delay and level of service is shown in Table 1. This study utilizes the Synchro software to determine intersection levels of service based on the *HCM* 7th Edition methodology.

Unsignalized Intersections

Level of service analysis at unsignalized intersections is generally used to determine the need for modification in the type of intersection control (i.e., all-way stop or signalization).

For side street stop-controlled intersections (two-way or T-intersections), operations are defined by the average control delay experienced by vehicles entering the intersection from the stop-controlled approaches on minor streets or from left-turn approaches on major streets. The level of service is reported based on the average delay for the worst approach. The level of service definitions for unsignalized intersections is shown in Table 2. This study utilizes the Synchro software to determine intersection levels of service based on the *HCM* 7th Edition methodology for unsignalized intersections.

Table 1
Signalized Intersection Level of Service Definitions Based on Control Delay

Level of Service	Description	Average Control Delay Per Vehicle (sec.)
A	Signal progression is extremely favorable. Most vehicles arrive during the green phase and do not stop at all. Short cycle lengths may also contribute to the very low vehicle delay.	10.0 or less
B	Operations characterized by good signal progression and/or short cycle lengths. More vehicles stop that with LOS A, causing higher levels of average vehicle delay.	10.1 to 20.0
C	Higher delays may result from fair signal progression and/or longer cycle lengths. Individual cycle failures may begin to appear at this level. The number of vehicles stopping is significant, though some vehicles may still pass through the intersection without stopping.	20.1 to 35.0
D	The influence of congestion becomes more noticeable. Longer delays may result from some combination of unfavorable signal progression, long cycle lengths, or high volume-to-capacity (V/C) ratios. Many vehicles stop and individual cycle failures are noticeable.	35.1 to 55.0
E	This is considered to the be the limit of acceptable delay. These high delays values generally indicated poor signal progression, long cycle lengths, and high volume-to-capacity (V/C) ratios. Individual cycle failures occur frequently.	55.1 to 80.0
F	This level of delay is considered unacceptable by most drivers. This condition often occurs with oversaturation, that is, when arrival flow rates exceed the capacity of the intersection. Poor progression and long cycle lengths may also be major contributing causes of such delay levels.	Greater than 80.0

Source: Transportation Research Board, *Highway Capacity Manual 7th Edition,* (Washington, D.C., 2023).

Table 2
Unsignalized Intersection Level of Service Definitions Based on Average Delay

Level of Service	Description	Average Delay Per Vehicle (sec.)
A	Little or no traffic delay	10.0 or less
B	Short traffic delays	10.1 to 15.0
C	Average traffic delays	15.1 to 25.0
D	Long traffic delays	25.1 to 35.0
E	Very long traffic delays	35.1 to 50.0
F	Extreme traffic delays	greater than 50.0

Source: Transportation Research Board, Highway Capacity Manual 7th Edition, (Washington, D.C., 2023).

The goal of the City of Pittsburg is to maintain LOS D at intersections in all areas except downtown, at key schools, and at freeway ramps, as established in the East County Action Plan.

Report Organization

This report has a total of three chapters. Following the introduction in Chapter 1, Chapter 2 describes the existing conditions, including the existing roadway network, transit service, and bicycle and pedestrian facilities. Chapter 3 presents the vehicle operational analysis, including the method by which project traffic is estimated, the project's traffic effects on the intersection operations, and the analyses of other transportation-related issues, including queuing, site access and on-site circulation, potential effects on bicycle, pedestrian, and transit facilities, and parking.

2.
Existing Conditions

This chapter describes the existing conditions for transportation facilities in the project area including the roadway network, transit services, pedestrian and bicycle facilities, and traffic operations at the study intersections.

Existing Roadway Network

Regional roadway access to the project site is provided via State Route 4 (SR-4). Local access is provided by North Park Boulevard/California Avenue and Loveridge Road. These facilities are described below.

State Route 4 is defined as a Route of Regional Significance in CCTA's East County Action Plan for Routes of Regional Significance. It is an east-west freeway that extends from Hercules in the west to Stockton and beyond in the east. The facility is an eight-lane freeway within the study area, with an interchange at Loveridge Road.

Loveridge Road is a north-south local road with two lanes in each direction in the vicinity of the site. The posted speed limit is 35 to 40 mph. Loveridge Road serves primarily commercial and industrial businesses in the vicinity of the project. Access to the project site is provided from Loveridge Road via North Park Boulevard.

North Park Boulevard/ California Avenue is an east-west road with two lanes in each direction in the vicinity of the site. It transitions to California Avenue at the intersection with Loveridge Road. The posted speed limit is 20 mph. Access to the project site is provided directly from the project driveway on North Park Boulevard.

Existing Pedestrian Facilities

The overall network of sidewalks and crosswalks in the project vicinity provides limited connectivity. There are gaps in the pedestrian routes connecting the project to neighboring areas. Sidewalks within the project vicinity, north of the freeway, are missing along the following street sections (see Figure 3):

- North side of North Park Boulevard from Pace Boulevard to 325 feet east of Loveridge Road
- North side of North Park Boulevard, just west of Century Boulevard
- South side of North Park Boulevard
- North side and south side of California Avenue, east of Loveridge Road
- East and west side of Loveridge Road, around north of railroad tracks
- Several segments along the west side of Loveridge Road, north of California Ave
- South side of California Avenue, west of Loveridge Road

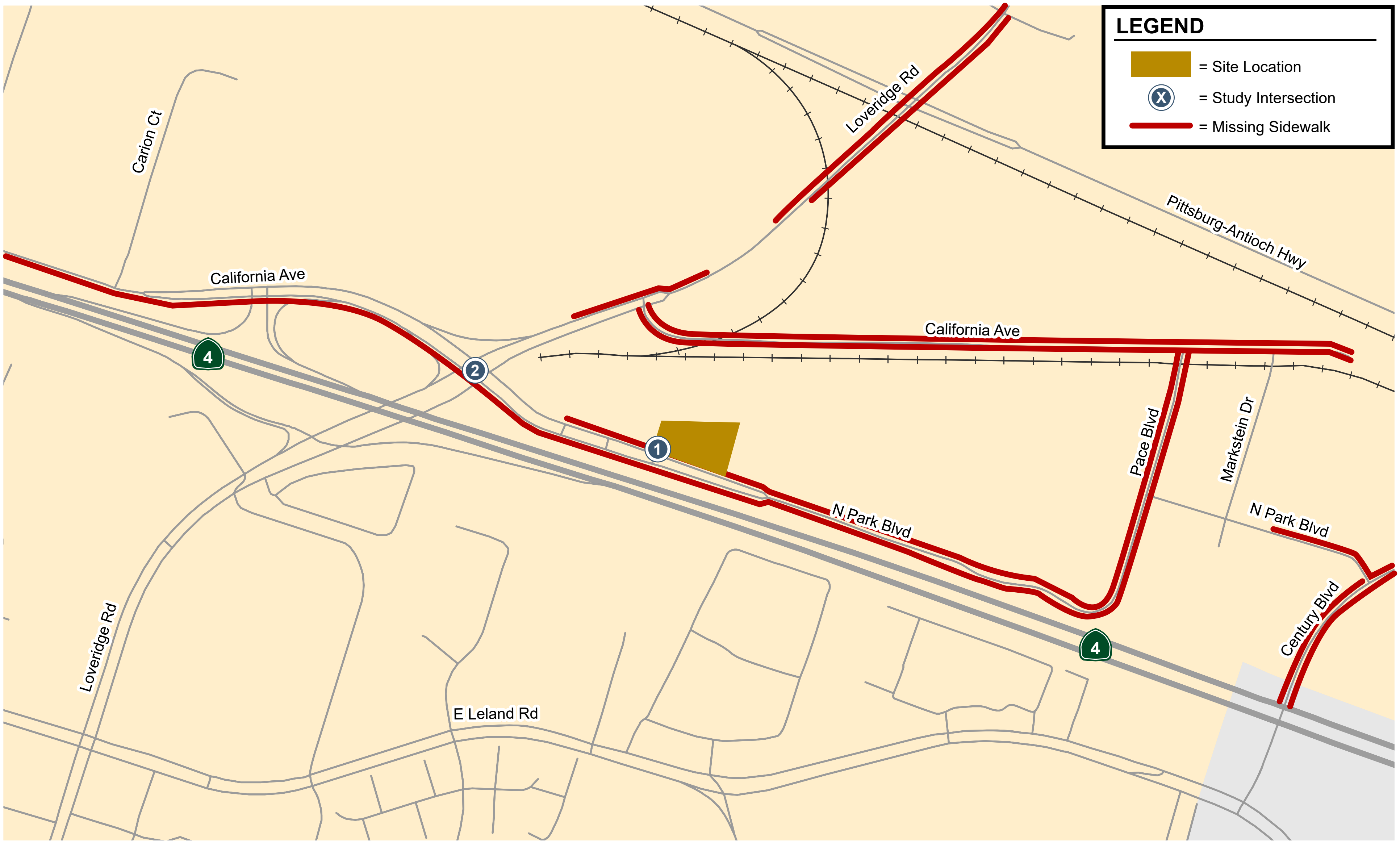

Figure 3
Missing Sidewalks

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Existing Bicycle Facilities

Existing and proposed bicycle facilities within the vicinity of the project site are shown on Figure 4 . Bike paths are shared between pedestrians and bicyclists and separated from motor vehicle traffic. Bike lanes are striped preferential lanes on the roadway for one-way bicycle travel. Some bicycle lanes include a striped buffer on one or both sides to increase separation from the traffic lane or from parked cars. Protected bike lanes are sheltered by physical barriers such as flexible bollards, raised curb, parking, or planter boxes.

- Loveridge Road
- East Leland Road
- California Avenue, east of Loveridge Road
- North Park Boulevard from Pace Boulevard to Century Boulevard
- Markstein Drive

Overall, the larger area around the project site is well-served by bicycle facilities. However, bicycle facilities are missing along North Park Boulevard, which provides direct access to the project.

The Pittsburg Moves Active Transportation Plan, adopted in December 2020, lists several proposed bicycle facilities in the project vicinity. The proposed facilities in the vicinity of the site are as follows:

Class I Bike Path

- Pittsburg-Antioch Highway
- Markstein Drive
- Century Boulevard

Class IV Protected Bike Lane

- Loveridge Road
- California Avenue, west of Loveridge Road

Existing Transit Services

Existing transit service to the study area is provided by Tri Delta Transit. Route 388 serves the project area and travels between the Pittsburg BART station and Kaiser Antioch Medical Center. The bus stop closest to the project site is located on Loveridge Road, which is approximately 0.3 miles walking distance from the site. Route 388 operates at varying headways ranging from about 20 to 60 minutes on weekdays. The transit services are summarized in Table 3 and shown on Figure 5.

Table 3
Transit Services

Route	Route Description	Weekday Hours of Operation	Headways[1] (minutes)	Nearby Bus Stops/Stations	Walking Distance to Project Site
Tri Delta Transit					
388	Pittsburg-Bay Point BART/ Kaiser Antioch Medical Center	4:49 AM to 10:17 PM	20 - 60	Loveridge Road and California Avenue	0.3 mile

Notes:
1 Headways on weekdays as of October 2024.

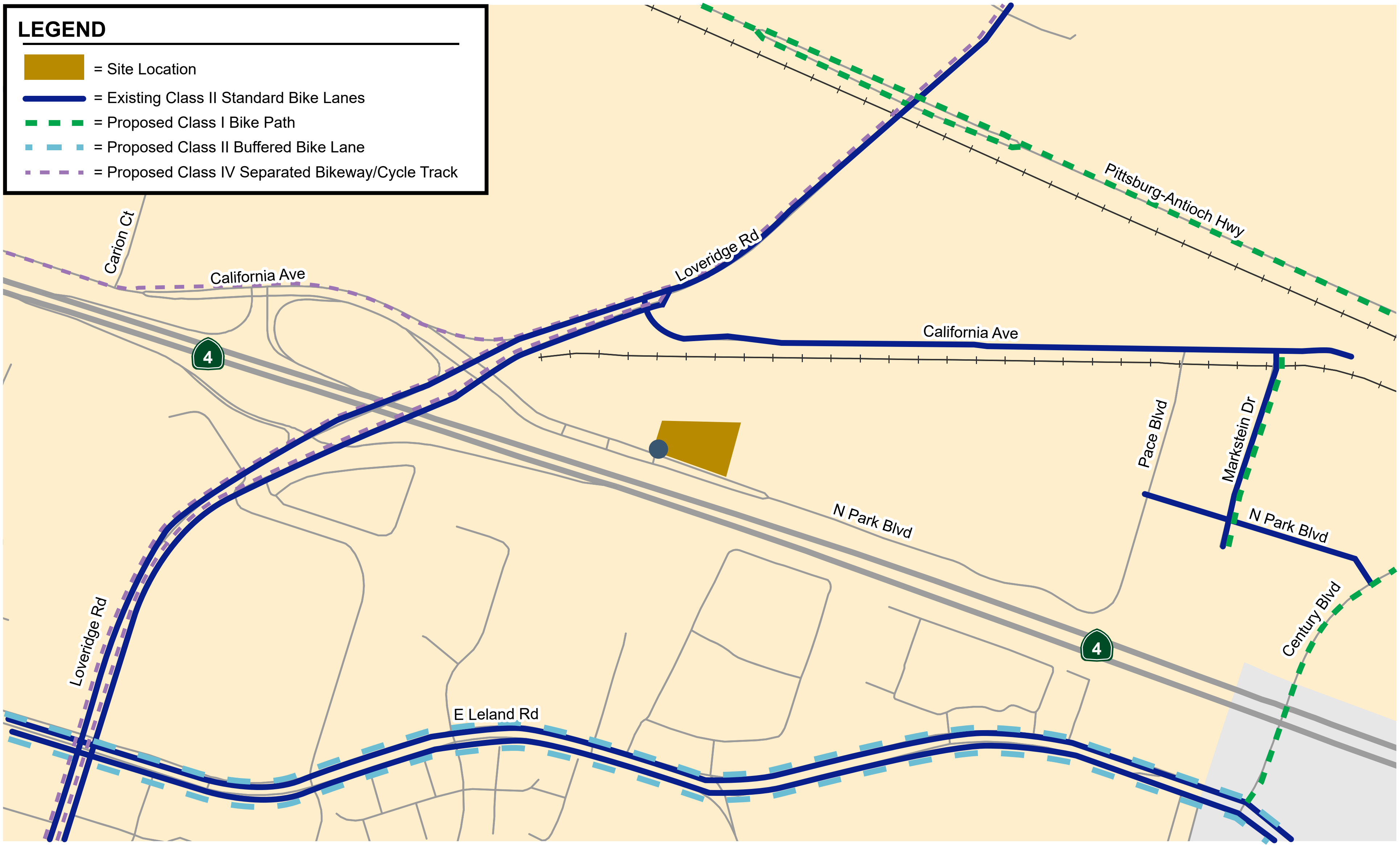

Figure 4
Existing and Proposed Bicycle Facilities

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Figure 5
Existing Transit Service

Existing Intersection Lane Configurations

The existing lane configurations at the study intersections were determined by observations in the field and are shown on Figure 6.

Existing Traffic Volumes

Existing PM peak-hour traffic volumes (see Figure 7) were obtained from new traffic counts collected on September 19, 2024. Traffic volumes for the study scenarios are tabulated in Appendix B.

Quick Quack Car Wash TA

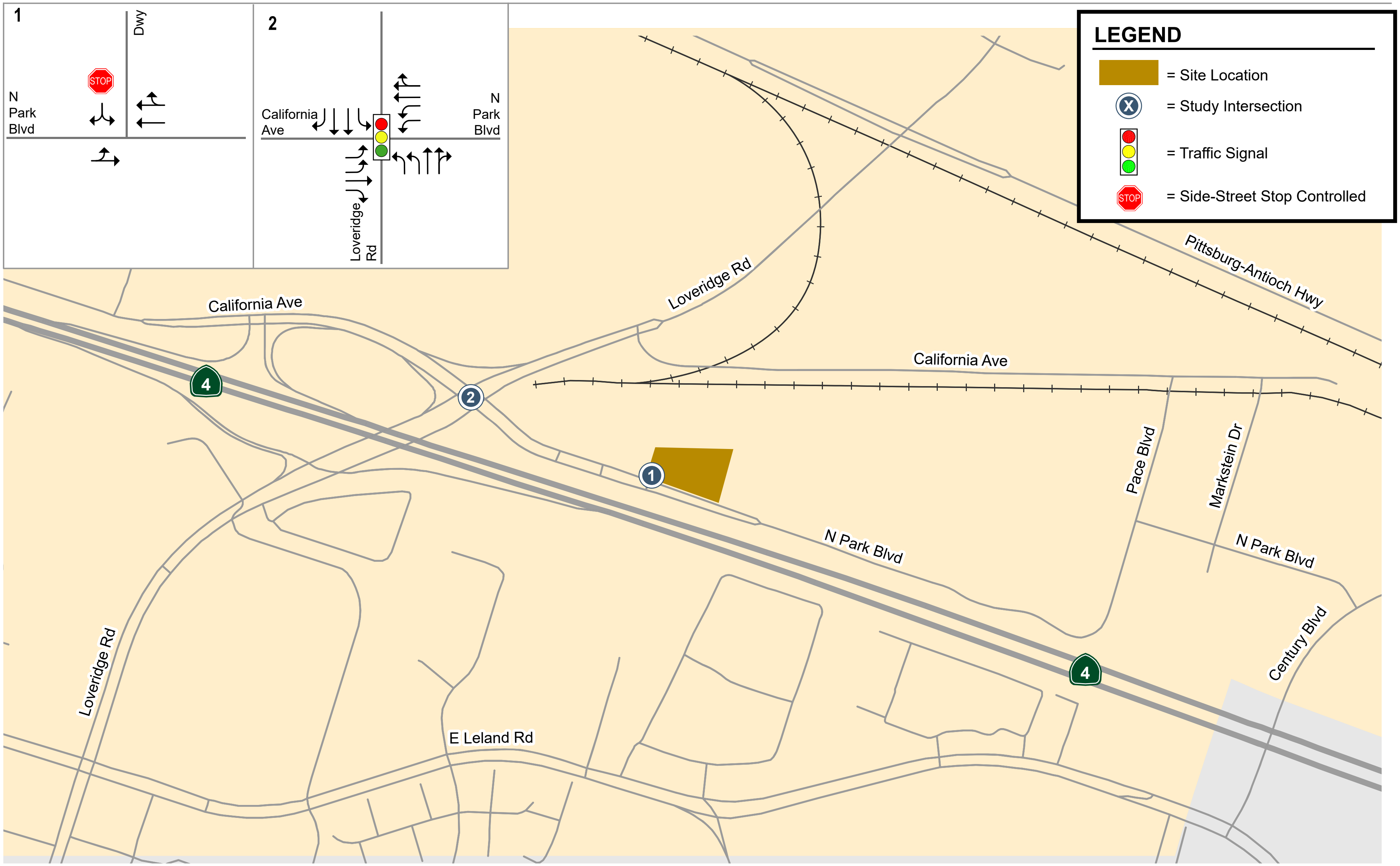

Figure 6
Existing Lane Configurations

Quick Quack Car Wash TA

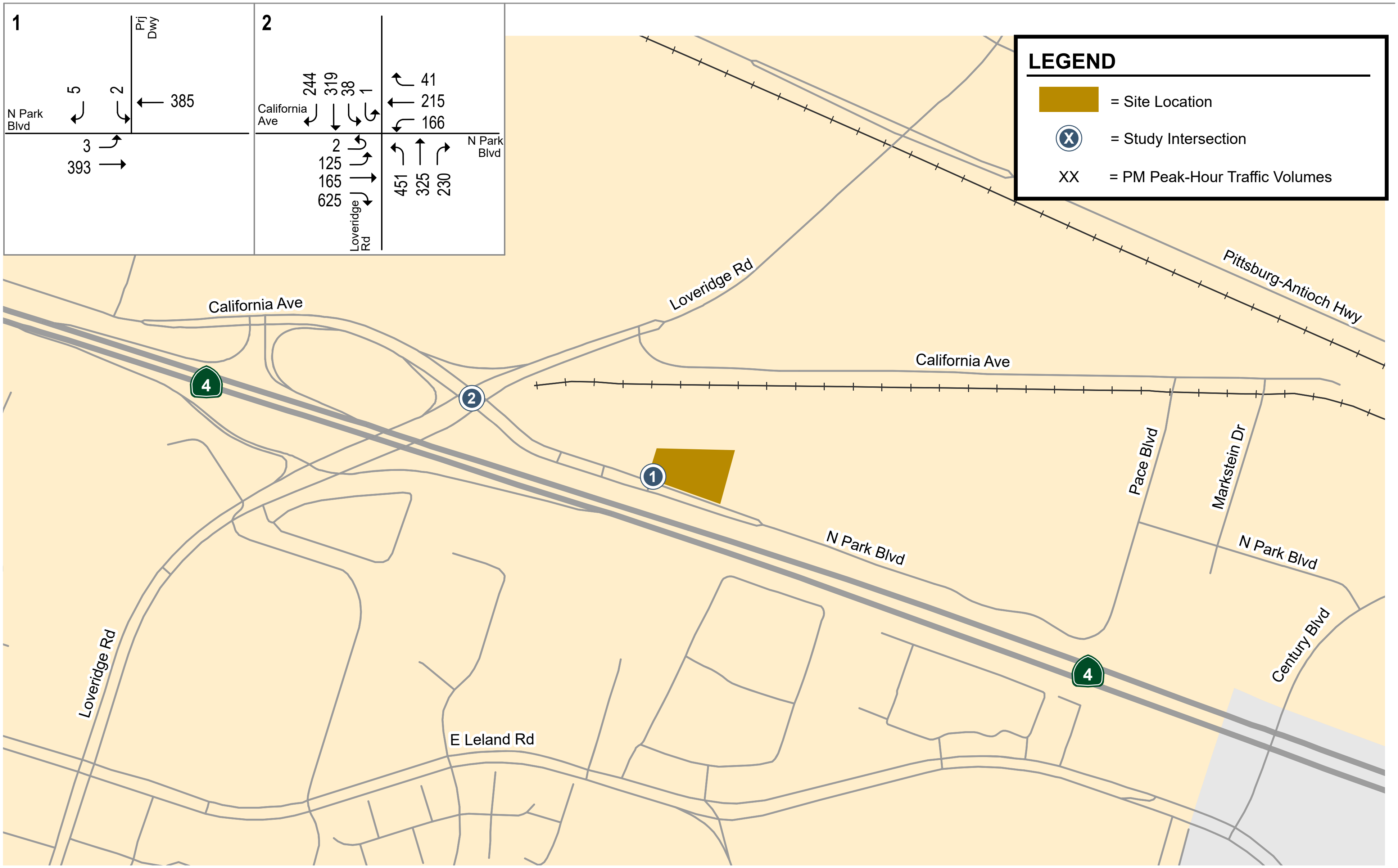

Figure 7
Existing Traffic Volumes

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3.
Future Traffic Conditions

This chapter describes the future traffic conditions, the method by which project traffic is estimated, intersection operations analysis for existing, existing plus project, background, and background plus project, any adverse effects on study intersections caused by the project, intersection vehicle queuing analysis, site access and on-site circulation review, and the effects on bicycle, pedestrian, and transit facilities.

Roadway Network

The roadway network under background and project conditions is assumed to be the same as under existing conditions.

Project Trip Estimates

The magnitude of traffic produced by a new development and the locations where that traffic would appear are estimated using a three-step process: (1) trip generation, (2) trip distribution, and (3) trip assignment. In determining project trip generation, the magnitude of traffic entering and exiting the site is estimated for the PM peak hour. As part of the project trip distribution, the directions to and from which the project trips would travel are estimated. In the project trip assignment, the project trips are assigned to specific streets and intersections. These procedures are described below.

Trip Generation

The magnitude of traffic produced by a new development is estimated by applying the size of the project to the applicable trip generation rates contained in the Institute of Transportation Engineers' (ITE) *Trip Generation Manual*, 11th Edition. Trips that would be generated by the proposed car wash were estimated using the ITE trip rates for "Automated Car Wash" (Land Use 948). Based on the trip generation rates, the proposed project would generate 78 trips (39 inbound and 39 outbound) during the PM peak hour (see Table 4). Pass-by trips for the project's car wash are not included in the ITE Handbook, and although some project trips would already be on the road, to be conservative, no pass-by trips were assumed in this analysis.

Table 4
Project Trip Generation Estimates

Land Use	Size	Units	PM Peak-Hour Trips Rate	In	Out	Total
Proposed						
Car Wash [1]	1	tunnel	77.5	39	39	78

Notes:
[1] Car wash trip generation is based on the rates published in the ITE Trip Generation Manual, 11th Edition for Automated Car Wash (Land Use Code 948).

Trip Distribution and Assignment

The trip distribution pattern for the project trips was estimated based on the surrounding roadway network, the locations of complementary land uses, and highway access points. The PM peak-hour vehicle trips generated by the project were assigned to the roadway network in accordance with the trip distribution patterns (see Figure 8).

Existing Plus Project Traffic Volumes

Project trips were added to existing traffic volumes to obtain existing plus project traffic volumes (see Figure 9).

Background Trip Estimates

Background PM peak-hour traffic volumes were estimated by adding to existing traffic volumes the trips generated by nearby future projects. Hexagon considered both the location and size of the future developments to eliminate those that were too far away or too small to affect traffic conditions at the selected study intersections.

The background trips of the following future developments trips were included:

- Pittsburg Renal Center – 11,225 s.f. of commercial shell building for future development of a dialysis clinic
- Home 2 Suites Hotel – 115 suite-style rooms

Trips that would be generated by the dialysis clinic and hotel were estimated using the ITE trip rates for “Clinic” (Land Use 630) and “All Suites Hotel” (Land Use 311), respectively. Based on the trip generation rates, the future developments would generate 82 trips (32 inbound and 50 outbound) during the PM peak hour (see Table 5)

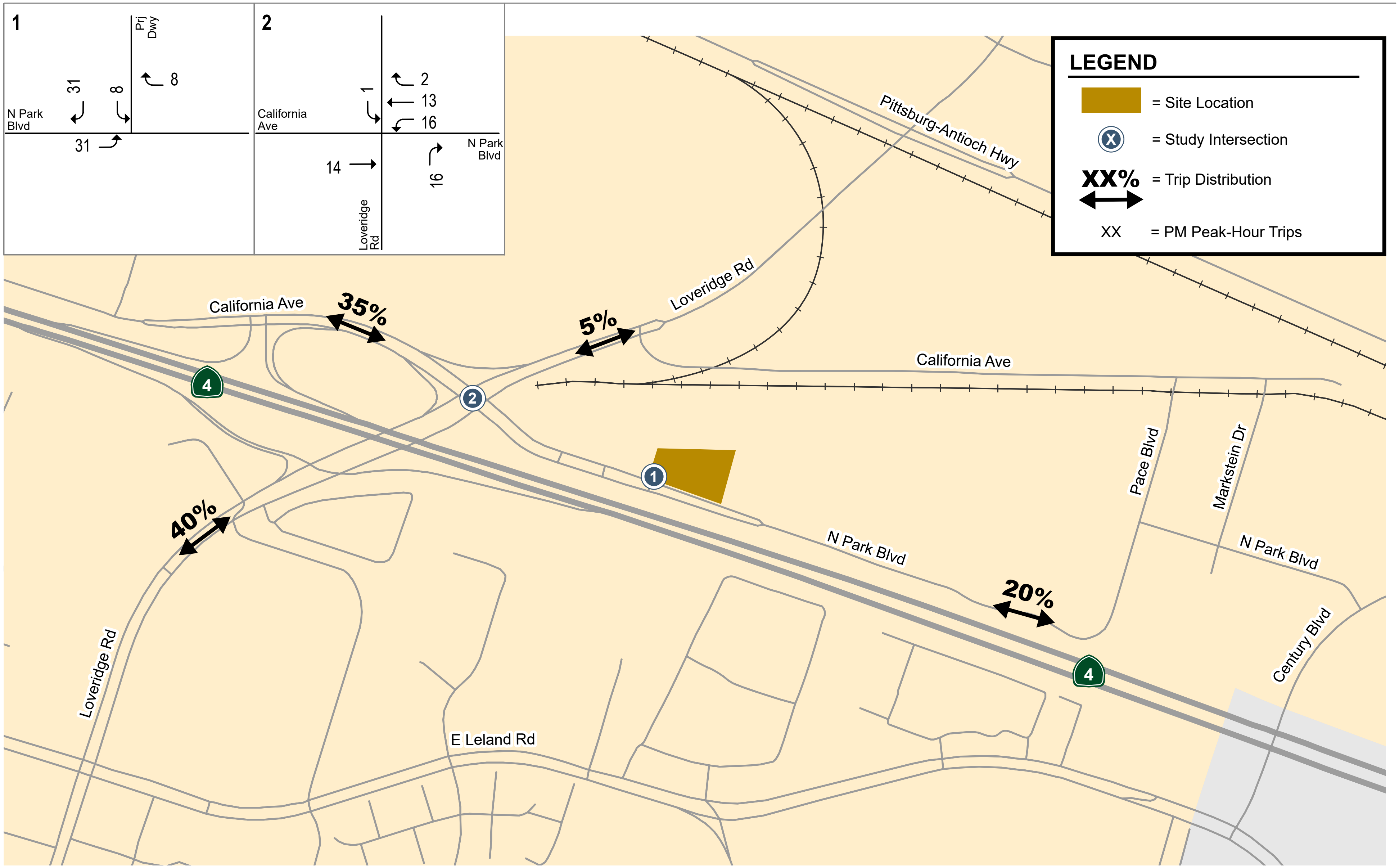

Figure 8
Project Trip Distribution and Assignment

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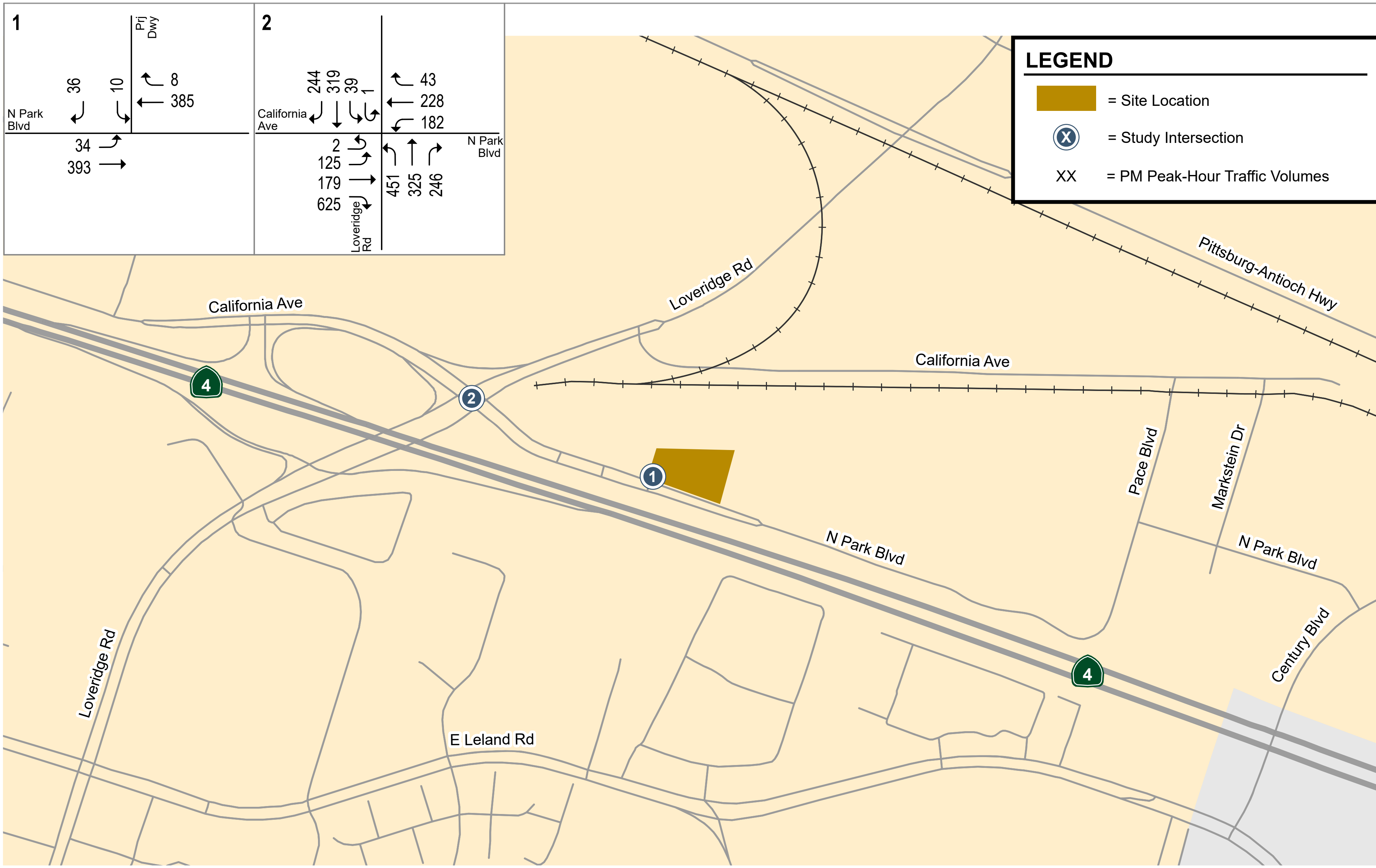

Figure 9
Existing Plus Project Traffic Volumes

Table 5
Background Project Trip Generation Estimates

			PM Peak-Hour Trips			
Land Use	**Size**	**Units**	**Rate**	**In**	**Out**	**Total**
Proposed						
Dialysis Clinic [1]	11,225	s.f.	3.69	12	29	41
Hotel [2]	115	rooms	0.36	20	21	41
Background Trips				**32**	**50**	**82**

Notes:
s.f. = square feet

[1] Dialysis clinic trip generation is based on the rates published in the *ITE Trip Generation Manual, 11th Edition* for Clinic (Land Use Code 630).

[2] Hotel trip generation is based on the rates published in the *ITE Trip Generation Manual, 11th Edition* for All Suites Hotel (Land Use Code 311).

Trip Distribution and Assignment

The trip distribution pattern for the dialysis clinic was assumed to be the same as the project. The trip distribution pattern for the future hotel was estimated based on the surrounding roadway network and highway access points. The trip distribution and assignment of these future projects are shown on Figure 10.

Background Traffic Volumes

The PM peak-hour vehicle trips generated by future projects were assigned to the roadway network in accordance with the trip distribution patterns (see Figure 10). The background PM peak-hour traffic volumes are shown on Figure 11.

Background Plus Project Traffic Volumes

Project trips were added to background traffic volumes to obtain background plus project traffic volumes (see Figure 12).

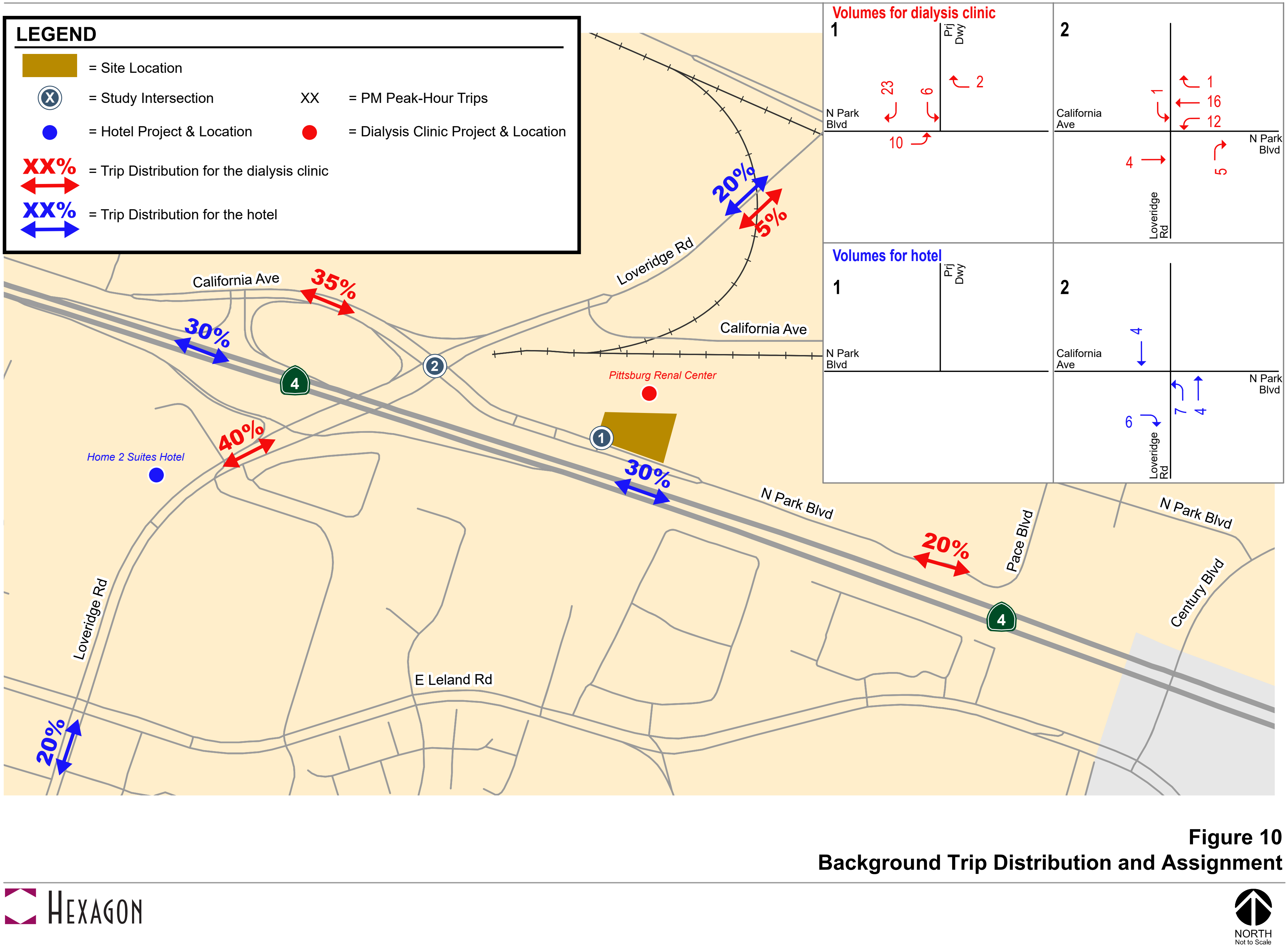

Figure 10
Background Trip Distribution and Assignment

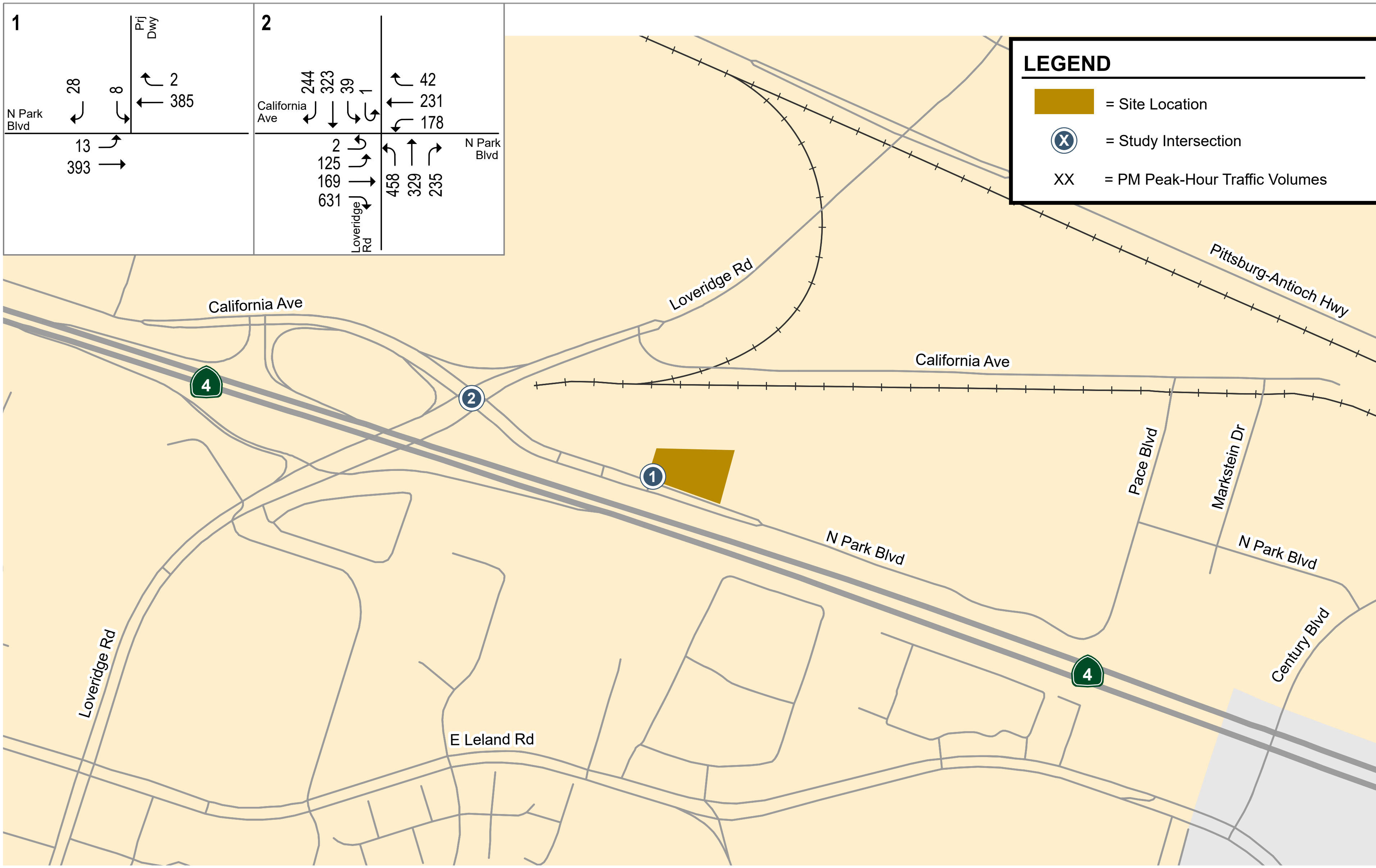

Figure 11
Background Traffic Volumes

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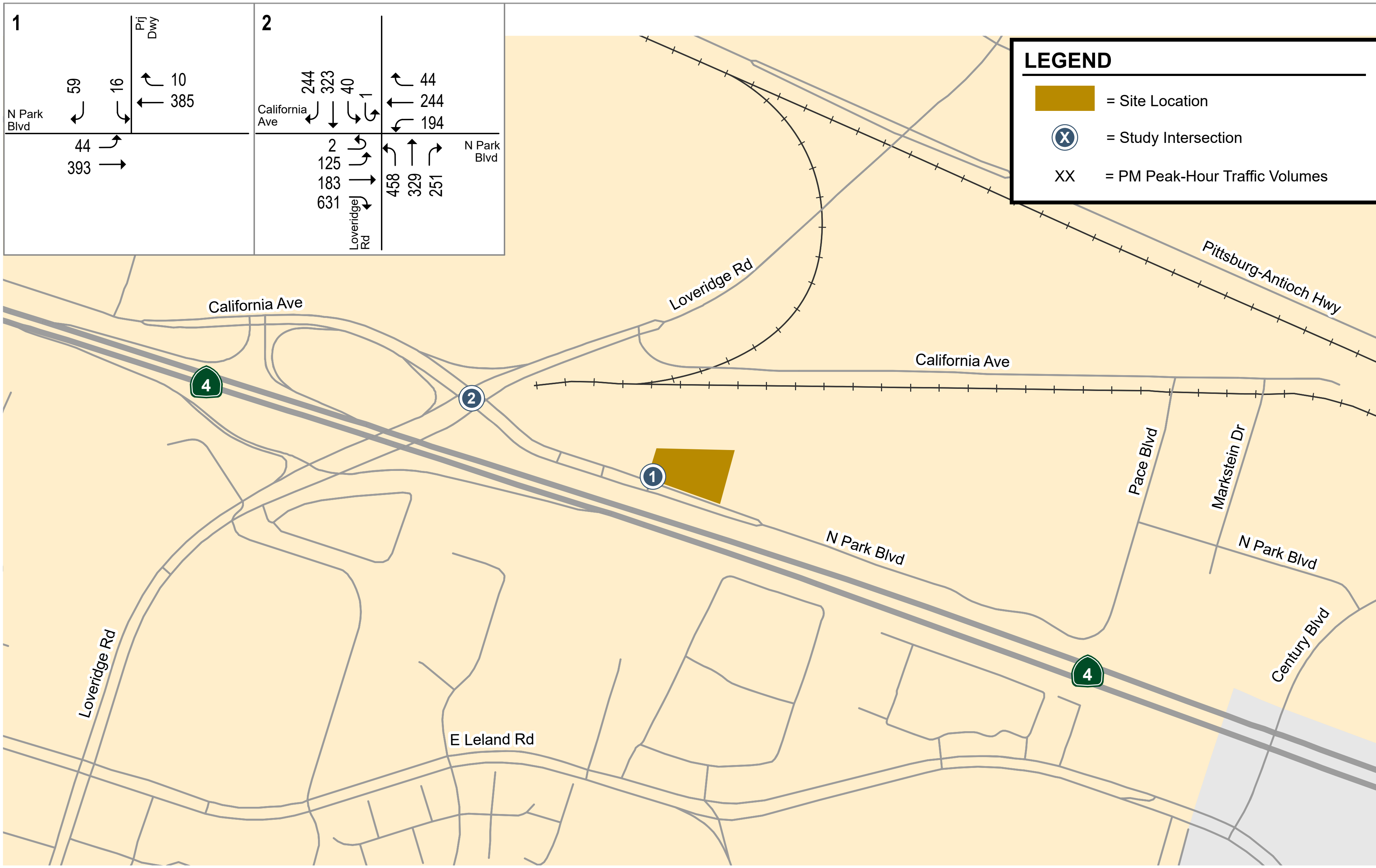


Figure 12
Background Plus Project Traffic Volumes

Intersection Levels of Service

The results of the intersection level of service analysis are shown in Table 6. The detailed intersection level of service calculation sheets for all study scenarios are included in Appendix C.

The results of the analysis show that all study intersections would operate at acceptable levels of service under all scenarios. Note that the delay at the Project Driveway and North Park Boulevard intersection is the same under existing and background conditions. While the background scenario adds traffic to the intersection, most of the increase in traffic are southbound right-turns which do not increase the average delay at the intersection.

Table 6
Intersection Level of Service

#	Intersection	Intersection Control	Peak Hour	Existing Conditions Delay [1] (sec)	Existing Conditions LOS	Existing Plus Project Delay [1] (sec)	Existing Plus Project LOS	Existing Plus Project Incr. in Delay	Background Conditions Delay [1] (sec)	Background Conditions LOS	Background Plus Project Delay [1] (sec)	Background Plus Project LOS	Background Plus Project Incr. in Delay
1	Project Driveway & North Park Boulevard	Side-Street Stop	PM	11.6	B	12.0	B	0.4	11.6	B	12.6	B	1.0
2	Loveridge Road & North Park Boulevard/California	Signal	PM	38.2	D	39.3	D	1.1	39.8	D	40.9	D	1.1

Notes:
[1] The delay reported for the signalized intersections is the average stopped delay for all vehicles entering the intersection. The delay reported for the side street stop controlled intersection is the delay experienced by vehicles on the stop controlled approach.

Intersection Queuing Analysis

The analysis of intersection operations was supplemented with a vehicle queuing analysis at study intersections where the project would add a noteworthy number of trips to the turning movements.

An evaluation of vehicle queuing was conducted using the Synchro software. The basis of the analysis is as follows: (1) the Synchro software is used to estimate the 95th percentile maximum number of queued vehicles; (2) the estimated maximum number of vehicles in the queue is translated into a queue length, assuming 25 feet per vehicle; and (3) the estimated maximum queue length is compared to the available storage capacity. The queuing analysis included a queuing storage analysis for the following movements:

Project Driveway and North Park Boulevard

- Southbound movement
- Eastbound left-turn

Loveridge Road and North Park Boulevard/California Avenue

- Westbound left-turn

The analysis showed that the project would not extend the queues beyond the available storage. The vehicle queue estimates, and a tabulated summary of the findings are provided in Table 7.

Table 7
Queuing Summary

	Project Driveway & North Park Boulevard		Loveridge Road & North Park Boulevard/ California Avenue
Movement	**SB**	**EBL**	**WBL**
Peak Hour Period	**PM**	**PM**	**PM**
Existing			
Volume (vphpl)	7	3	83
95th %. Queue (ft/ln)[1]	0	0	66
95th %. Queue (veh/ln)[1]	0	0	3
Storage (ft/ln)	80	150	215
Adequate (Y/N)	Y	Y	Y
Existing Plus Project			
Volume (vphpl)	46	34	91
95th %. Queue (ft/ln)[1]	25	0	72
95th %. Queue (veh/ln)[1]	1	0	3
Storage (ft./ln)	80	150	215
Adequate (Y/N)	Y	Y	Y
Background			
Volume (vphpl)	36	13	89
95th %. Queue (ft/ln)[1]	25	0	71
95th %. Queue (veh/ln)[1]	1	0	3
Storage (ft/ln)	80	150	215
Adequate (Y/N)	Y	Y	Y
Background Plus Project			
Volume (vphpl)	75	44	97
95th %. Queue (ft/ln)[1]	25	25	76
95th %. Queue (veh/ln)[1]	1	1	3
Storage (ft./ln)	80	150	215
Adequate (Y/N)	Y	Y	Y

Notes:

SB = southbound movement; EBL = eastbound left-turn; WBL = westbound left-turn

[1] Vehicle queues are from Synchro outputs and are rounded up to the next whole number. Assumes 1 vehicle equals 25 feet of queue.

Vehicular Site Access and On-Site Circulation

The site access and circulation evaluations are based on the site plan dated March 12, 2024 (see Figure 2 in Chapter 1). Site access was evaluated to determine the adequacy of the site's driveway with regard to the following: traffic volume, vehicle queues, geometric design, and stopping sight distance. On-site vehicular circulation and parking layout were reviewed in accordance with generally accepted traffic engineering standards and transportation planning principles.

Site Access

Site access is provided via an existing driveway that is shared with the dental office. The driveway would also provide access to the proposed medical clinic, just north of the site. The site plan does not show any changes to the existing driveway. The driveway is approximately 30 feet wide, which meets the City of Pittsburg Standard Detail R-3 for commercial developments.

Sight Distance at the Driveway

The existing driveway to the site was checked for adequate sight distance. Sight distance generally should be provided in accordance with Caltrans standards. The minimum acceptable sight distance is often considered the Caltrans stopping sight distance. Sight distance requirements vary depending on the roadway speeds. For North Park Boulevard, which has a speed limit of 20 mph, the Caltrans stopping sight distance is 150 feet (based on a design speed of 25 mph). This means that a driver must be able to see 150 feet down North Park Boulevard to locate a sufficient gap to turn out of the driveway. This also gives drivers traveling along North Park Boulevard adequate time to react to vehicles exiting the driveway. There are about 500 feet of sight distance looking both ways from the driveway which is more than adequate.

On-Site Queuing Analysis

A queuing analysis was conducted to identify the potential queuing of vehicles accessing the project site and to determine whether vehicles waiting to access the car wash would spill back onto the shared driveway, and possibly onto North Park Boulevard. The 95th-percentile queue is generally applied as the acceptable limit for on-site circulation impacts. To assess the potential queuing for the site, factors such as the storage capacity, arrival rate and service rate were considered. The arrival rate is defined as the number of vehicles arriving at the facility per hour. Similarly, the service rate is defined as the number of vehicles served within an hour. The applied service rate was based on data regarding the typical time needed to completely service each vehicle. Based on the information provided by the applicant, about 80% of the customers are members. Members don't stop their car to pay and can drive up to the gate, where their license plate is scanned, which opens the gate. The carwash can load a car onto the conveyor every 15 seconds, and the system can wash about 155 cars in one hour. It takes approximately 2 minutes and 45 seconds to go through the washing station from the time a vehicle is on the conveyor. To be conservative and to account for the 20% of the customers that have to pay which delays the process, a service rate of 120 vehicles per hour was assumed. Applying these rates produced a calculated 95th percentile queue length approaching the car wash entrance of three vehicles. The on-site storage capacity provides space for approximately 14 vehicles between the entry and the car wash entrance. The queuing calculation worksheet is included in Appendix D. Based on the assumed arrival and service rates, the proposed on-site vehicle storage capacity is expected to adequately accommodate the vehicle queue, and no spillover onto the driveway or onto North Park Boulevard is expected to occur.

On-Site Circulation

The site plan includes a car wash aisle at the south end of the site and a single parking drive aisle for the vacuuming station at the north end of the site. The project includes 13 vacuuming stations/parking spaces and three parking spaces for staff. The vacuuming spaces are for visitors to use. The project drive aisle to the vacuuming spaces is 25 feet wide, which is sufficient for maneuvering in and out of parking/vacuum spaces. For the car wash operations, vehicles enter through the project driveway, pass through the pay stations, and then enter the car wash tunnel at the southeast corner of the site. Vehicles would then either exit the project site or go to the vacuuming spaces before exiting. For visitors not going through the car wash tunnel, the entry lane provides direct access to the vacuuming spaces, as shown on the site plan. Figures 13 and 14 show the turning templates to determine if cars can easily turn into and out of the wash tunnel and vacuum spaces. As shown in Figure 14, cars exiting the tunnel and turning right towards the vacuum stations won't be able to make the turn in one try. Cars would need to conduct a three-point movement to avoid the structure that separates the vacuum stations, which is undesirable and can result in collisions.

> **Recommendation**: It is recommended to remove two vacuum stations to allow sufficient space for vehicles to exit the tunnel and turn right towards the vacuum stations. In addition, a STOP sign should be placed at the end of the tunnel for exiting vehicles to give the right of way to cars leaving the site.

Truck Access and Circulation

The project would include a trash enclosure at the northwest corner of the site. The trash enclosure opens out towards the driveway. The driveway would serve as the access point for garbage trucks. The garbage trucks can turn around through the adjacent parking lot and then turn back onto the driveway.

Parking

Vehicle Parking

According to the City of Pittsburg Municipal Code (18.78.040), automobile washing uses would need to provide four plus one parking space per 500 s.f. of building. The project proposes a 3,588 s.f. car wash tunnel. Therefore, the project would be required to provide 12 parking spaces. The project proposes 13 parking spaces/vacuuming spaces and three parking spaces (non-vacuuming) for staff. This meets the City’s parking requirements

Bicycle Parking

According to the City of Pittsburg Municipal Code (18.78.045), for uses that require 11 to 20 off-street parking spaces, four bicycle parking spaces must be provided. The bicycle parking spaces can be provided as a rack or locker. The project would be required to provide 12 parking spaces and would therefore need to provide four bicycle parking spaces. The project proposes one bicycle locker and one bicycle rack. This would not meet the City’s requirements. However, since all customers of the car wash would arrive by car and the fact the car wash would only employ a few workers, additional bicycle parking spaces beyond the two spaces would not be necessary.

Effects on Pedestrian Facilities

There are no continuous pedestrian paths to the project site. The sidewalk along North Park Boulevard provides inadequate access to nearby points of interest and transit. Even though there are no sidewalks along most of North Park Boulevard, it is anticipated that, given the location of the project relative to the residential areas, employees are expected to arrive by car or bicycle, and all customers would arrive by car.

Effects on Bicycle Facilities

Class II striped bike lanes are present on Loveridge Road, Pittsburg-Antioch Highway, and East Leland Road. Some employees may ride bikes to the site. The existing bike lanes along these roads provide separate travel lanes for bicyclists from vehicular traffic. While no bicycle facilities are provided along North Park Boulevard, bicyclists share the road with vehicular traffic. The project proposes no features that would be hazardous to bicycle travel.

Effects on Transit Services

The project site is served by Route 388 on Loveridge Road. The bus stop closest to the project site is located along Loveridge Road, near its intersection with California Avenue. There are no continuous sidewalks that connect the project site to the nearest bus stop. Due to the small size of the project and the number of employees that would work at the site, the project is not expected to generate new riders for transit. Any increase in new transit riders from the proposed project could be accommodated by the currently available capacity of the bus services in the study area, and improvement of the existing transit service would not be necessary with the project.

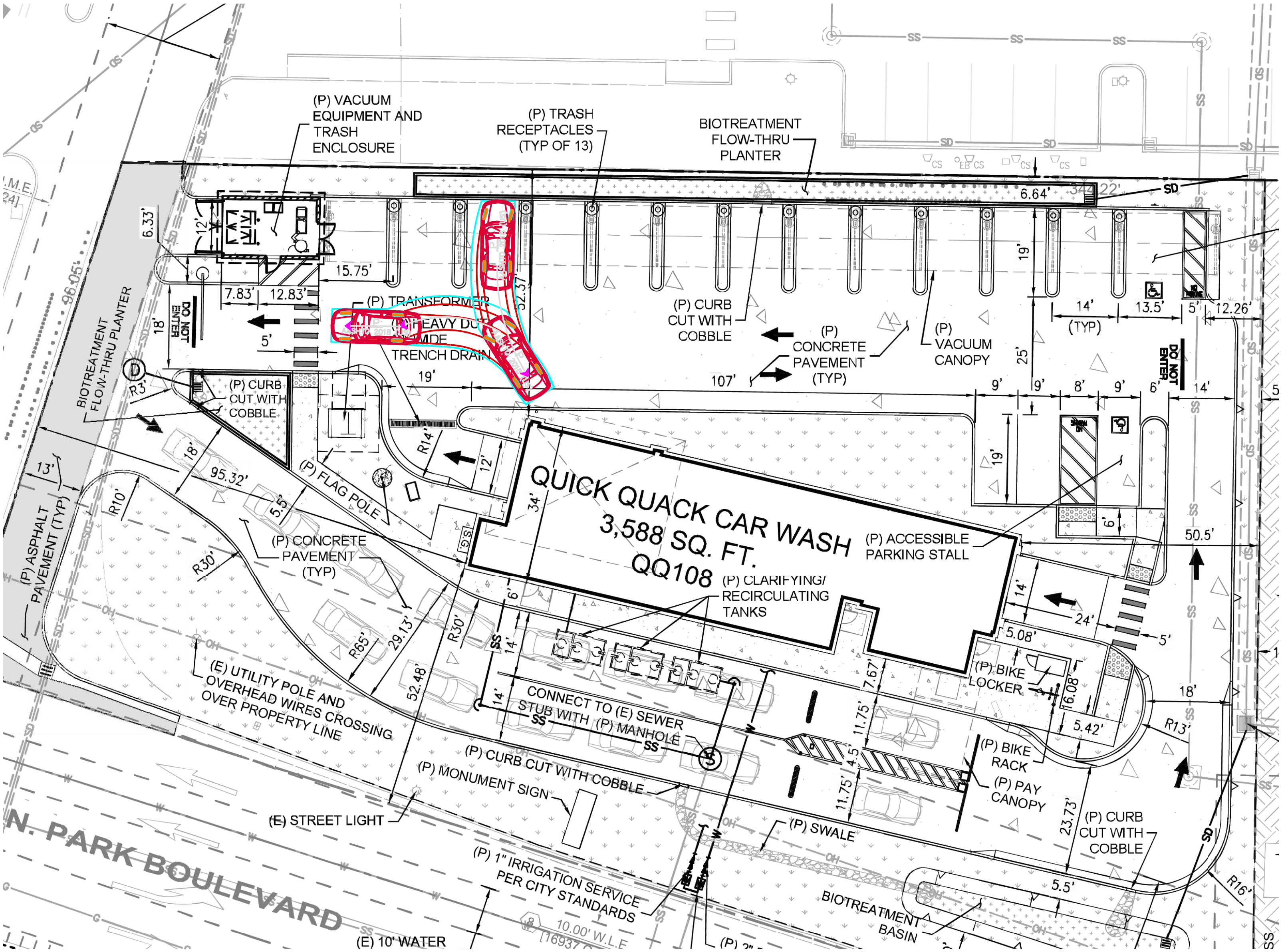
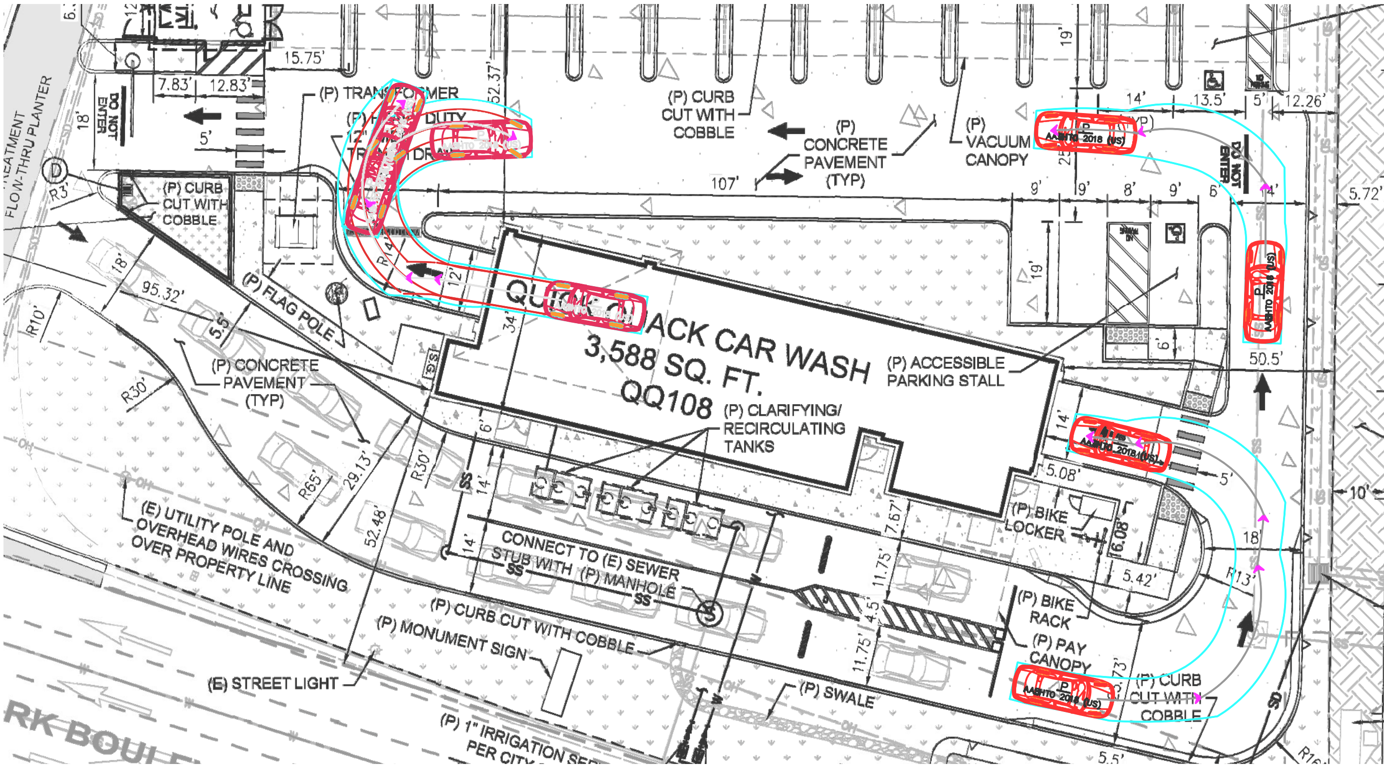


Figure 13
Vacuum Space Turning Movement

Right Turning Movement



Left Turning Movement

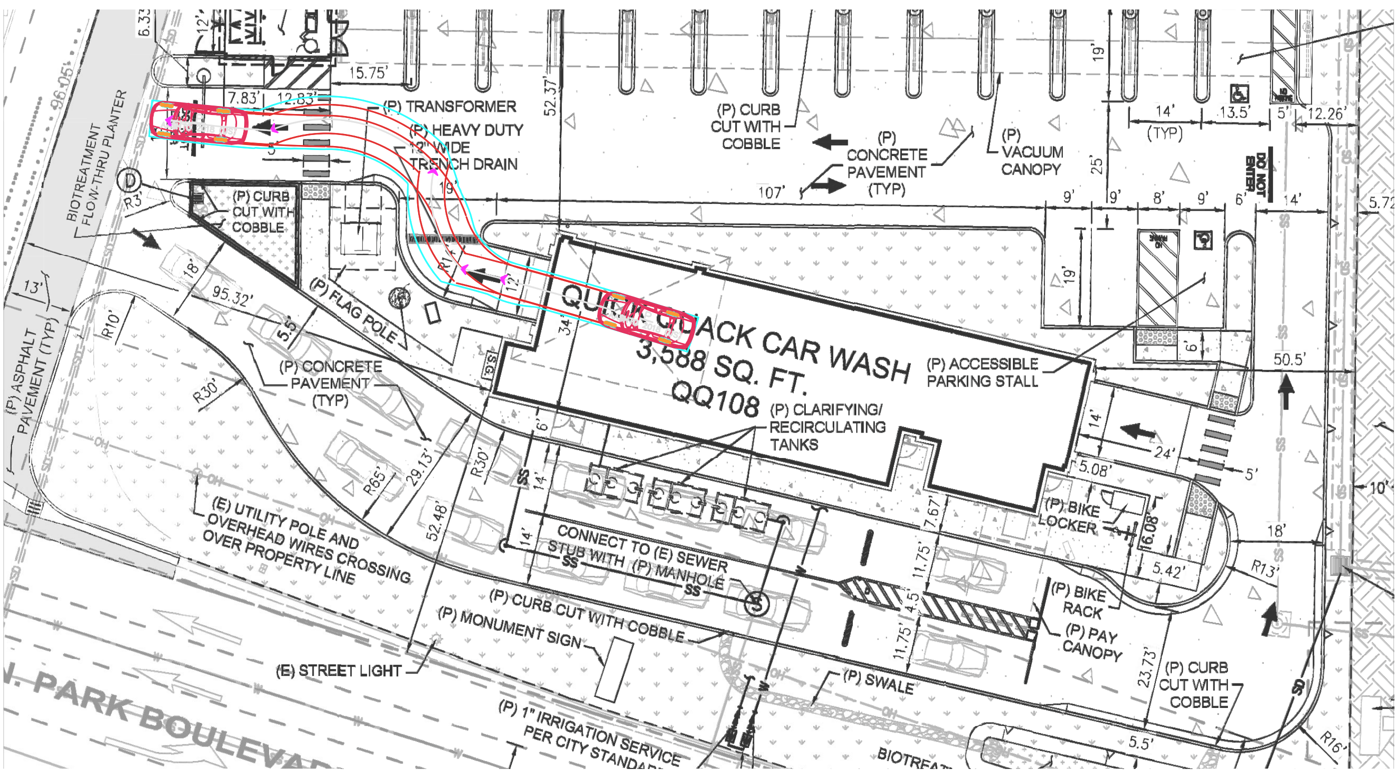


Figure 14
Car Wash Tunnel Turning Movement

Quick Quack Car Wash
Transportation Analysis

Technical Appendices

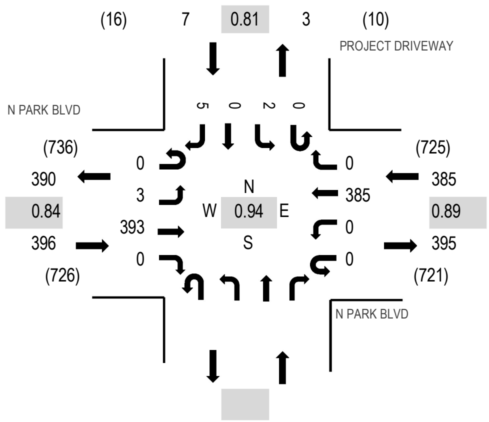
November 6, 2024

Appendix A
Traffic Counts

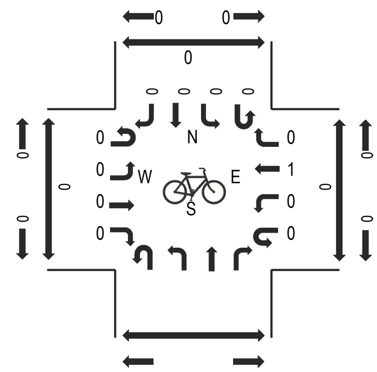
(303) 216-2439
www.alltrafficdata.net

Location: 1 PROJECT DRIVEWAY & N PARK BLVD PM
Date: Thursday, September 19, 2024
Peak Hour: 04:30 PM - 05:30 PM
Peak 15-Minutes: 04:45 PM - 05:00 PM

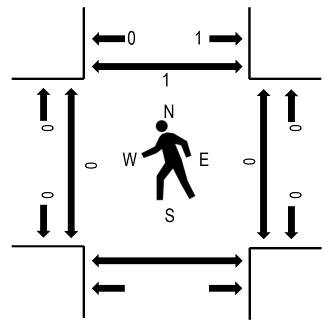
Peak Hour - Motorized Vehicles



Peak Hour - Bicycles



Peak Hour - Pedestrians



Note: Total study counts contained in parentheses.

Traffic Counts - Motorized Vehicles

Interval Start Time	N PARK BLVD Eastbound U-Turn	Left	Thru	Right	N PARK BLVD Westbound U-Turn	Left	Thru	Right	Northbound U-Turn	Left	Thru	Right	PROJECT DRIVEWAY Southbound U-Turn	Left	Thru	Right	Total	Rolling Hour	Pedestrian Crossings West	East	South	North
4:00 PM	0	0	80	0	0	0	82	0					0	0	0	3	165	752	0	0		7
4:15 PM	0	1	94	0	0	0	88	1					0	1	0	3	188	785	0	0		1
4:30 PM	0	1	90	0	0	0	97	0					0	1	0	1	190	788	0	0		0
4:45 PM	0	1	117	0	0	0	87	0					0	1	0	3	209	764	0	0		0
5:00 PM	0	0	90	0	0	0	108	0					0	0	0	0	198	715	0	0		0
5:15 PM	0	1	96	0	0	0	93	0					0	0	0	1	191		0	0		1
5:30 PM	0	3	79	0	0	0	84	0					0	0	0	0	166		0	0		1
5:45 PM	0	1	72	0	0	0	84	1					0	0	0	2	160		0	0		0

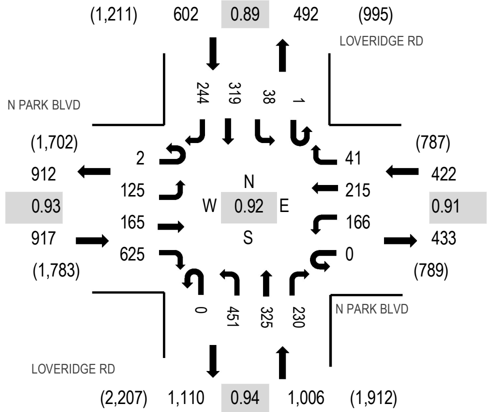
Peak Rolling Hour Flow Rates

Vehicle Type	Eastbound U-Turn	Left	Thru	Right	Westbound U-Turn	Left	Thru	Right	Northbound U-Turn	Left	Thru	Right	Southbound U-Turn	Left	Thru	Right	Total
Articulated Trucks	0	0	1	0	0	0	1	0					0	0	0	0	2
Lights	0	3	391	0	0	0	383	0					0	2	0	5	784
Mediums	0	0	1	0	0	0	1	0					0	0	0	0	2
Total	0	3	393	0	0	0	385	0					0	2	0	5	788

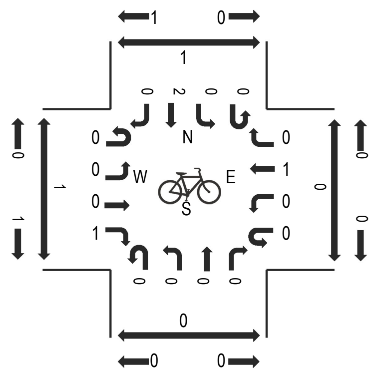
ALL TRAFFIC DATA SERVICES
(303) 216-2439
www.alltrafficdata.net

Location: 2 LOVERIDGE RD & N PARK BLVD PM
Date: Thursday, September 19, 2024
Peak Hour: 04:30 PM - 05:30 PM
Peak 15-Minutes: 05:00 PM - 05:15 PM

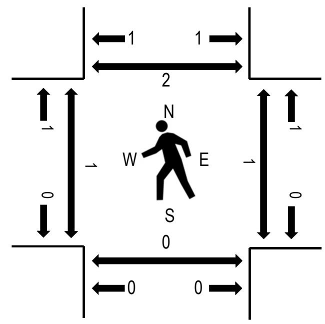
Peak Hour - Motorized Vehicles



Peak Hour - Bicycles



Peak Hour - Pedestrians



Note: Total study counts contained in parentheses.

Traffic Counts - Motorized Vehicles

Interval Start Time	N PARK BLVD Eastbound U-Turn	Left	Thru	Right	N PARK BLVD Westbound U-Turn	Left	Thru	Right	LOVERIDGE RD Northbound U-Turn	Left	Thru	Right	LOVERIDGE RD Southbound U-Turn	Left	Thru	Right	Total	Rolling Hour	Pedestrian Crossings West	East	South	North
4:00 PM	0	37	32	154	0	43	40	13	1	108	78	47	0	9	101	59	722	2,847	0	0	0	0
4:15 PM	1	26	38	132	0	38	45	9	0	95	94	53	0	8	99	61	699	2,922	0	2	0	7
4:30 PM	0	29	42	140	0	43	52	10	0	106	77	57	0	3	69	56	684	2,947	0	0	0	1
4:45 PM	1	38	44	140	0	41	47	7	0	111	90	69	1	13	80	60	742	2,946	0	0	0	0
5:00 PM	1	27	41	162	0	40	63	13	0	135	86	52	0	11	84	82	797	2,846	1	1	0	0
5:15 PM	0	31	38	183	0	42	53	11	0	99	72	52	0	11	86	46	724		0	0	0	1
5:30 PM	0	43	31	161	1	32	31	14	2	102	82	42	0	12	72	58	683		1	0	0	0
5:45 PM	1	32	35	143	0	44	50	5	0	97	70	35	0	13	75	42	642		0	0	0	0

Peak Rolling Hour Flow Rates

Vehicle Type	Eastbound U-Turn	Left	Thru	Right	Westbound U-Turn	Left	Thru	Right	Northbound U-Turn	Left	Thru	Right	Southbound U-Turn	Left	Thru	Right	Total
Articulated Trucks	0	3	1	1	0	0	0	1	0	2	6	0	0	0	1	4	19
Lights	2	118	163	617	0	166	214	40	0	447	310	230	1	38	315	238	2,899
Mediums	0	4	1	7	0	0	1	0	0	2	9	0	0	0	3	2	29
Total	2	125	165	625	0	166	215	41	0	451	325	230	1	38	319	244	2,947

Appendix B
Volume Summary Sheet

Quick Quack Car Wash
TA Volumes

Intersection Number: 1
Synchro Node Number: 1
Intersection Name: Project Driveway and N Park Boulevard
Peak Hour: PM
Count Date: 9/19/2024

	Movements												
	North Approach			East Approach			South Approach			West Approach			
Scenario:	RT	TH	LT	RT	TH	LT	RT	TH	LT	RT	TH	LT	Total
Existing Conditions	5	0	2	0	385	0	0	0	0	0	393	3	788
Project Trips	31	0	8	8	0	0	0	0	0	0	0	31	78
Existing Plus Project Conditions	36	0	10	8	385	0	0	0	0	0	393	34	866
Background Project Trips	23	0	6	2	0	0	0	0	0	0	0	10	41
Background Conditions	28	0	8	2	385	0	0	0	0	0	393	13	829
Background Plus Project Conditions	59	0	16	10	385	0	0	0	0	0	393	44	907

Intersection Number: 2
Synchro Node Number: 2
Intersection Name: Loveridge Road and N Park Boulevard
Peak Hour: PM
Count Date: 9/19/2024

	Movements												
	North Approach			East Approach			South Approach			West Approach			
Scenario:	RT	TH	LT	RT	TH	LT	RT	TH	LT	RT	TH	LT	Total
Existing Conditions	244	319	39	41	215	166	230	325	451	625	165	127	2947
Project Trips	0	0	1	2	13	16	16	0	0	0	14	0	62
Existing Plus Project Conditions	244	319	40	43	228	182	246	325	451	625	179	127	3009
Background Project Trips	0	4	1	1	16	12	5	4	7	6	4	0	60
Background Conditions	244	323	40	42	231	178	235	329	458	631	169	127	3007
Background Plus Project Conditions	244	323	41	44	244	194	251	329	458	631	183	127	3069

Appendix C
Level of Service Calculations

Intersection	
Int Delay, s/veh	0.1

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	3	393	385	0	2	5
Future Vol, veh/h	3	393	385	0	2	5
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	3	427	418	0	2	5

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	418	0	-	0	852	209
Stage 1	-	-	-	-	418	-
Stage 2	-	-	-	-	434	-
Critical Hdwy	4.13	-	-	-	6.63	6.93
Critical Hdwy Stg 1	-	-	-	-	5.83	-
Critical Hdwy Stg 2	-	-	-	-	5.43	-
Follow-up Hdwy	2.219	-	-	-	3.519	3.319
Pot Cap-1 Maneuver	1139	-	-	-	314	797
Stage 1	-	-	-	-	633	-
Stage 2	-	-	-	-	653	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1139	-	-	-	313	797
Mov Cap-2 Maneuver	-	-	-	-	313	-
Stage 1	-	-	-	-	631	-
Stage 2	-	-	-	-	653	-

Approach	EB	WB	SB
HCM Control Delay, s/v	0.06	0	11.6
HCM LOS			B

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1139	-	-	-	553
HCM Lane V/C Ratio	0.003	-	-	-	0.014
HCM Control Delay (s/veh)	8.2	-	-	-	11.6
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0	-	-	-	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	127	165	625	166	215	41	451	325	230	39	319	244
Future Volume (veh/h)	127	165	625	166	215	41	451	325	230	39	319	244
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width 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
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
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772
Adj Flow Rate, veh/h	138	179	679	180	234	45	490	353	250	42	347	265
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	198	709	860	244	1170	221	566	654	455	53	685	396
Arrive On Green	0.06	0.40	0.40	0.07	0.41	0.41	0.17	0.34	0.34	0.03	0.20	0.20
Sat Flow, veh/h	3274	1772	1502	3274	2825	534	3274	1897	1320	1688	3367	1502
Grp Volume(v), veh/h	138	179	679	180	138	141	490	313	290	42	347	265
Grp Sat Flow(s),veh/h/ln	1637	1772	1502	1637	1683	1676	1637	1683	1534	1688	1683	1502
Q Serve(g_s), s	5.0	8.1	42.5	6.5	6.3	6.5	17.6	18.0	18.4	3.0	11.0	19.0
Cycle Q Clear(g_c), s	5.0	8.1	42.5	6.5	6.3	6.5	17.6	18.0	18.4	3.0	11.0	19.0
Prop In Lane	1.00		1.00	1.00		0.32	1.00		0.86	1.00		1.00
Lane Grp Cap(c), veh/h	198	709	860	244	697	694	566	580	529	53	685	396
V/C Ratio(X)	0.70	0.25	0.79	0.74	0.20	0.20	0.87	0.54	0.55	0.79	0.51	0.67
Avail Cap(c_a), veh/h	665	919	1038	665	873	869	774	1110	1012	147	1718	857
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	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	55.5	24.1	20.1	54.6	22.5	22.6	48.5	31.8	31.9	58.0	42.6	39.7
Incr Delay (d2), s/veh	4.4	0.2	3.5	4.4	0.1	0.1	7.7	0.8	0.9	22.8	0.6	2.0
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	2.2	3.5	15.0	2.8	2.5	2.6	7.8	7.4	7.0	1.6	4.7	7.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	59.9	24.3	23.5	59.0	22.7	22.7	56.2	32.6	32.8	80.8	43.2	41.6
LnGrp LOS	E	C	C	E	C	C	E	C	C	F	D	D
Approach Vol, veh/h		996			459			1093			654	
Approach Delay, s/veh		28.7			36.9			43.2			45.0	
Approach LOS		C			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	25.3	29.0	11.8	54.4	8.3	46.1	13.5	52.7
Change Period (Y+Rc), s	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Max Green Setting (Gmax), s	28.5	61.5	24.5	62.5	10.5	79.5	24.5	62.5
Max Q Clear Time (g_c+I1), s	19.6	21.0	7.0	8.5	5.0	20.4	8.5	44.5
Green Ext Time (p_c), s	1.3	3.5	0.4	1.8	0.0	4.5	0.5	3.7

Intersection Summary	
HCM 7th Control Delay, s/veh	38.2
HCM 7th LOS	D

Notes

User approved pedestrian interval to be less than phase max green.

Intersection	
Int Delay, s/veh	1

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	34	393	385	8	10	36
Future Vol, veh/h	34	393	385	8	10	36
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	37	427	418	9	11	39

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	427	0	-	0	924	214
Stage 1	-	-	-	-	423	-
Stage 2	-	-	-	-	501	-
Critical Hdwy	4.13	-	-	-	6.63	6.93
Critical Hdwy Stg 1	-	-	-	-	5.83	-
Critical Hdwy Stg 2	-	-	-	-	5.43	-
Follow-up Hdwy	2.219	-	-	-	3.519	3.319
Pot Cap-1 Maneuver	1130	-	-	-	283	792
Stage 1	-	-	-	-	630	-
Stage 2	-	-	-	-	608	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1130	-	-	-	274	792
Mov Cap-2 Maneuver	-	-	-	-	274	-
Stage 1	-	-	-	-	609	-
Stage 2	-	-	-	-	608	-

Approach	EB	WB	SB
HCM Control Delay, s/v	0.66	0	12.04
HCM LOS			B

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1130	-	-	-	561
HCM Lane V/C Ratio	0.033	-	-	-	0.089
HCM Control Delay (s/veh)	8.3	-	-	-	12
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0.1	-	-	-	0.3

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	127	179	625	182	228	43	451	325	246	40	319	244
Future Volume (veh/h)	127	179	625	182	228	43	451	325	246	40	319	244
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width 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
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
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772
Adj Flow Rate, veh/h	138	195	679	198	248	47	490	353	267	43	347	265
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	197	709	859	261	1189	222	563	630	469	54	682	394
Arrive On Green	0.06	0.40	0.40	0.08	0.42	0.42	0.17	0.34	0.34	0.03	0.20	0.20
Sat Flow, veh/h	3274	1772	1502	3274	2832	528	3274	1840	1369	1688	3367	1502
Grp Volume(v), veh/h	138	195	679	198	146	149	490	322	298	43	347	265
Grp Sat Flow(s),veh/h/ln	1637	1772	1502	1637	1683	1677	1637	1683	1526	1688	1683	1502
Q Serve(g_s), s	5.1	9.2	43.6	7.3	6.8	7.0	18.0	19.2	19.7	3.1	11.3	19.5
Cycle Q Clear(g_c), s	5.1	9.2	43.6	7.3	6.8	7.0	18.0	19.2	19.7	3.1	11.3	19.5
Prop In Lane	1.00		1.00	1.00		0.32	1.00		0.90	1.00		1.00
Lane Grp Cap(c), veh/h	197	709	859	261	707	704	563	576	522	54	682	394
V/C Ratio(X)	0.70	0.28	0.79	0.76	0.21	0.21	0.87	0.56	0.57	0.79	0.51	0.67
Avail Cap(c_a), veh/h	650	897	1018	650	852	849	756	1084	982	143	1677	838
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	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	57.0	25.0	20.6	55.7	22.8	22.8	49.8	33.0	33.2	59.4	43.8	40.8
Incr Delay (d2), s/veh	4.5	0.2	3.6	4.5	0.1	0.1	8.4	0.9	1.0	22.1	0.6	2.0
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	2.2	3.9	15.5	3.2	2.7	2.8	8.0	8.0	7.4	1.7	4.8	7.4
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	61.5	25.2	24.3	60.1	22.9	23.0	58.2	33.9	34.2	81.5	44.4	42.8
LnGrp LOS	E	C	C	E	C	C	E	C	C	F	D	D
Approach Vol, veh/h		1012			493			1110			655	
Approach Delay, s/veh		29.5			37.9			44.7			46.2	
Approach LOS		C			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	25.7	29.5	11.9	56.3	8.5	46.8	14.4	53.9
Change Period (Y+Rc), s	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Max Green Setting (Gmax), s	28.5	61.5	24.5	62.5	10.5	79.5	24.5	62.5
Max Q Clear Time (g_c+I1), s	20.0	21.5	7.1	9.0	5.1	21.7	9.3	45.6
Green Ext Time (p_c), s	1.2	3.5	0.4	1.9	0.0	4.6	0.5	3.8

Intersection Summary	
HCM 7th Control Delay, s/veh	39.3
HCM 7th LOS	D

Notes

User approved pedestrian interval to be less than phase max green.

Intersection	
Int Delay, s/veh	0.6

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	13	393	385	2	8	28
Future Vol, veh/h	13	393	385	2	8	28
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	14	427	418	2	9	30

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	421	0	-	0	875	210
Stage 1	-	-	-	-	420	-
Stage 2	-	-	-	-	455	-
Critical Hdwy	4.13	-	-	-	6.63	6.93
Critical Hdwy Stg 1	-	-	-	-	5.83	-
Critical Hdwy Stg 2	-	-	-	-	5.43	-
Follow-up Hdwy	2.219	-	-	-	3.519	3.319
Pot Cap-1 Maneuver	1137	-	-	-	304	796
Stage 1	-	-	-	-	632	-
Stage 2	-	-	-	-	638	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1137	-	-	-	300	796
Mov Cap-2 Maneuver	-	-	-	-	300	-
Stage 1	-	-	-	-	624	-
Stage 2	-	-	-	-	638	-

Approach	EB	WB	SB
HCM Control Delay, s/v	0.26	0	11.63
HCM LOS			B

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1137	-	-	-	582
HCM Lane V/C Ratio	0.012	-	-	-	0.067
HCM Control Delay (s/veh)	8.2	-	-	-	11.6
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0	-	-	-	0.2

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	127	169	631	178	231	42	458	329	235	40	323	244
Future Volume (veh/h)	127	169	631	178	231	42	458	329	235	40	323	244
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width 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
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
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772
Adj Flow Rate, veh/h	138	184	686	193	251	46	498	358	255	43	351	265
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	196	712	865	255	1196	216	570	651	456	54	681	394
Arrive On Green	0.06	0.40	0.40	0.08	0.42	0.42	0.17	0.34	0.34	0.03	0.20	0.20
Sat Flow, veh/h	3274	1772	1502	3274	2848	514	3274	1891	1325	1688	3367	1502
Grp Volume(v), veh/h	138	184	686	193	147	150	498	318	295	43	351	265
Grp Sat Flow(s),veh/h/ln	1637	1772	1502	1637	1683	1679	1637	1683	1533	1688	1683	1502
Q Serve(g_s), s	5.2	8.7	44.6	7.2	6.9	7.1	18.5	19.1	19.5	3.2	11.6	19.8
Cycle Q Clear(g_c), s	5.2	8.7	44.6	7.2	6.9	7.1	18.5	19.1	19.5	3.2	11.6	19.8
Prop In Lane	1.00		1.00	1.00		0.31	1.00		0.86	1.00		1.00
Lane Grp Cap(c), veh/h	196	712	865	255	707	705	570	579	528	54	681	394
V/C Ratio(X)	0.70	0.26	0.79	0.76	0.21	0.21	0.87	0.55	0.56	0.79	0.52	0.67
Avail Cap(c_a), veh/h	641	885	1012	641	841	839	746	1070	975	142	1655	828
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	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	57.7	25.0	20.7	56.5	23.0	23.1	50.3	33.2	33.3	60.1	44.4	41.3
Incr Delay (d2), s/veh	4.6	0.2	3.8	4.6	0.1	0.1	9.1	0.8	0.9	22.1	0.6	2.0
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	2.3	3.7	15.9	3.2	2.8	2.9	8.3	7.9	7.4	1.7	4.9	7.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	62.3	25.1	24.5	61.1	23.2	23.2	59.4	34.0	34.2	82.2	45.0	43.4
LnGrp LOS	E	C	C	E	C	C	E	C	C	F	D	D
Approach Vol, veh/h		1008			490			1111			659	
Approach Delay, s/veh		29.8			38.1			45.4			46.8	
Approach LOS		C			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	26.3	29.8	12.0	57.0	8.5	47.5	14.2	54.8
Change Period (Y+Rc), s	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Max Green Setting (Gmax), s	28.5	61.5	24.5	62.5	10.5	79.5	24.5	62.5
Max Q Clear Time (g_c+I1), s	20.5	21.8	7.2	9.1	5.2	21.5	9.2	46.6
Green Ext Time (p_c), s	1.2	3.5	0.4	1.9	0.0	4.5	0.5	3.7

Intersection Summary	
HCM 7th Control Delay, s/veh	39.8
HCM 7th LOS	D

Notes

User approved pedestrian interval to be less than phase max green.

Intersection	
Int Delay, s/veh	1.4

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	44	393	385	10	16	59
Future Vol, veh/h	44	393	385	10	16	59
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	48	427	418	11	17	64

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	429	0	-	0	947	215
Stage 1	-	-	-	-	424	-
Stage 2	-	-	-	-	523	-
Critical Hdwy	4.13	-	-	-	6.63	6.93
Critical Hdwy Stg 1	-	-	-	-	5.83	-
Critical Hdwy Stg 2	-	-	-	-	5.43	-
Follow-up Hdwy	2.219	-	-	-	3.519	3.319
Pot Cap-1 Maneuver	1128	-	-	-	274	791
Stage 1	-	-	-	-	629	-
Stage 2	-	-	-	-	594	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1128	-	-	-	263	791
Mov Cap-2 Maneuver	-	-	-	-	263	-
Stage 1	-	-	-	-	602	-
Stage 2	-	-	-	-	594	-

Approach	EB	WB	SB
HCM Control Delay, s/v	0.84	0	12.63
HCM LOS			B

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1128	-	-	-	553
HCM Lane V/C Ratio	0.042	-	-	-	0.147
HCM Control Delay (s/veh)	8.3	-	-	-	12.6
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0.1	-	-	-	0.5

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	127	183	631	194	244	44	458	329	251	41	323	244
Future Volume (veh/h)	127	183	631	194	244	44	458	329	251	41	323	244
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width 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
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
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772	1772
Adj Flow Rate, veh/h	138	199	686	211	265	48	498	358	273	45	351	265
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	195	712	863	273	1214	217	567	625	469	57	678	392
Arrive On Green	0.06	0.40	0.40	0.08	0.43	0.43	0.17	0.34	0.34	0.03	0.20	0.20
Sat Flow, veh/h	3274	1772	1502	3274	2854	510	3274	1832	1375	1688	3367	1502
Grp Volume(v), veh/h	138	199	686	211	155	158	498	328	303	45	351	265
Grp Sat Flow(s),veh/h/ln	1637	1772	1502	1637	1683	1680	1637	1683	1524	1688	1683	1502
Q Serve(g_s), s	5.3	9.7	45.8	8.1	7.5	7.7	19.0	20.5	20.9	3.4	11.9	20.3
Cycle Q Clear(g_c), s	5.3	9.7	45.8	8.1	7.5	7.7	19.0	20.5	20.9	3.4	11.9	20.3
Prop In Lane	1.00		1.00	1.00		0.30	1.00		0.90	1.00		1.00
Lane Grp Cap(c), veh/h	195	712	863	273	716	715	567	574	520	57	678	392
V/C Ratio(X)	0.71	0.28	0.79	0.77	0.22	0.22	0.88	0.57	0.58	0.79	0.52	0.68
Avail Cap(c_a), veh/h	626	864	993	626	821	820	728	1045	946	138	1616	810
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	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	59.2	25.8	21.3	57.5	23.3	23.3	51.6	34.6	34.7	61.4	45.6	42.5
Incr Delay (d2), s/veh	4.7	0.2	4.0	4.7	0.1	0.2	9.8	0.9	1.0	21.0	0.6	2.0
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	2.3	4.2	16.4	3.5	3.0	3.1	8.6	8.5	7.9	1.8	5.1	7.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	63.9	26.0	25.3	62.2	23.4	23.5	61.5	35.5	35.8	82.5	46.2	44.5
LnGrp LOS	E	C	C	E	C	C	E	D	D	F	D	D
Approach Vol, veh/h		1023			524			1129			661	
Approach Delay, s/veh		30.7			39.1			47.0			48.0	
Approach LOS		C			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	26.7	30.3	12.1	59.0	8.8	48.2	15.2	55.9
Change Period (Y+Rc), s	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Max Green Setting (Gmax), s	28.5	61.5	24.5	62.5	10.5	79.5	24.5	62.5
Max Q Clear Time (g_c+I1), s	21.0	22.3	7.3	9.7	5.4	22.9	10.1	47.8
Green Ext Time (p_c), s	1.2	3.5	0.4	2.0	0.0	4.7	0.6	3.6

Intersection Summary	
HCM 7th Control Delay, s/veh	40.9
HCM 7th LOS	D

Notes

User approved pedestrian interval to be less than phase max green.

Appendix D
Queue Calculations

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	127	165	625	166	215	41	451	325	230	39	319	244
Future Volume (vph)	127	165	625	166	215	41	451	325	230	39	319	244
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Lane Width (ft)	12	12	12	12	12	12	12	12	12	12	12	12
Grade (%)		0%			0%			0%			0%	
Storage Length (ft)	330		200	200		110	0		0	140		345
Storage Lanes	1		1	1		1	2		0	1		1
Taper Length (ft)	25			25			25			25		
Right Turn on Red			Yes			Yes			Yes			Yes
Link Speed (mph)		30			30			30			30	
Link Distance (ft)		665			794			628			1045	
Travel Time (s)		15.1			18.0			14.3			23.8	
Confl. Peds. (#/hr)												
Confl. Bikes (#/hr)												
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Growth Factor	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%
Bus Blockages (#/hr)	0	0	0	0	0	0	0	0	0	0	0	0
Parking (#/hr)												
Mid-Block Traffic (%)		0%			0%			0%			0%	
Shared Lane Traffic (%)												
Lane Group Flow (vph)	138	179	679	180	279	0	490	603	0	42	347	265
v/c Ratio	0.45	0.34	0.68	0.52	0.27		0.58	0.48		0.34	0.62	0.41
Control Delay (s/veh)	58.4	32.4	14.1	57.6	27.9		44.7	27.1		66.0	51.4	6.2
Queue Delay	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	0.0
Total Delay (s/veh)	58.4	32.4	14.1	57.6	27.9		44.7	27.1		66.0	51.4	6.2
Queue Length 50th (ft)	47	98	195	61	73		153	140		28	119	0
Queue Length 95th (ft)	106	181	428	131	124		#318	291		85	226	68
Internal Link Dist (ft)		585			714			548			965	
Turn Bay Length (ft)	330		200	200						140		345
Base Capacity (vph)	731	1012	1046	731	1892		850	2325		161	1893	962
Starvation Cap Reductn	0	0	0	0	0		0	0		0	0	0
Spillback Cap Reductn	0	0	0	0	0		0	0		0	0	0
Storage Cap Reductn	0	0	0	0	0		0	0		0	0	0
Reduced v/c Ratio	0.19	0.18	0.65	0.25	0.15		0.58	0.26		0.26	0.18	0.28

Intersection Summary

Area Type: Other

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	127	179	625	182	228	43	451	325	246	40	319	244
Future Volume (vph)	127	179	625	182	228	43	451	325	246	40	319	244
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Lane Width (ft)	12	12	12	12	12	12	12	12	12	12	12	12
Grade (%)		0%			0%			0%			0%	
Storage Length (ft)	330		200	200		110	0		0	140		345
Storage Lanes	1		1	1		1	2		0	1		1
Taper Length (ft)	25			25			25			25		
Right Turn on Red			Yes			Yes			Yes			Yes
Link Speed (mph)		30			30			30			30	
Link Distance (ft)		665			794			628			1045	
Travel Time (s)		15.1			18.0			14.3			23.8	
Confl. Peds. (#/hr)												
Confl. Bikes (#/hr)												
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Growth Factor	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%
Bus Blockages (#/hr)	0	0	0	0	0	0	0	0	0	0	0	0
Parking (#/hr)												
Mid-Block Traffic (%)		0%			0%			0%			0%	
Shared Lane Traffic (%)												
Lane Group Flow (vph)	138	195	679	198	295	0	490	620	0	43	347	265
v/c Ratio	0.45	0.36	0.69	0.54	0.28		0.59	0.50		0.35	0.62	0.42
Control Delay (s/veh)	59.5	33.3	14.7	58.4	28.0		45.9	27.7		67.4	52.4	6.3
Queue Delay	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	0.0
Total Delay (s/veh)	59.5	33.3	14.7	58.4	28.0		45.9	27.7		67.4	52.4	6.3
Queue Length 50th (ft)	48	109	205	69	78		157	147		29	122	0
Queue Length 95th (ft)	107	199	448	143	131		#325	300		88	227	68
Internal Link Dist (ft)		585			714			548			965	
Turn Bay Length (ft)	330		200	200						140		345
Base Capacity (vph)	717	993	1033	717	1860		834	2278		158	1856	949
Starvation Cap Reductn	0	0	0	0	0		0	0		0	0	0
Spillback Cap Reductn	0	0	0	0	0		0	0		0	0	0
Storage Cap Reductn	0	0	0	0	0		0	0		0	0	0
Reduced v/c Ratio	0.19	0.20	0.66	0.28	0.16		0.59	0.27		0.27	0.19	0.28

Intersection Summary	
Area Type:	Other

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	127	169	631	178	231	42	458	329	235	40	323	244
Future Volume (vph)	127	169	631	178	231	42	458	329	235	40	323	244
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Lane Width (ft)	12	12	12	12	12	12	12	12	12	12	12	12
Grade (%)		0%			0%			0%			0%	
Storage Length (ft)	330		200	200		110	0		0	140		345
Storage Lanes	1		1	1		1	2		0	1		1
Taper Length (ft)	25			25			25			25		
Right Turn on Red			Yes			Yes			Yes			Yes
Link Speed (mph)		30			30			30			30	
Link Distance (ft)		665			794			628			1045	
Travel Time (s)		15.1			18.0			14.3			23.8	
Confl. Peds. (#/hr)												
Confl. Bikes (#/hr)												
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Growth Factor	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%
Bus Blockages (#/hr)	0	0	0	0	0	0	0	0	0	0	0	0
Parking (#/hr)												
Mid-Block Traffic (%)		0%			0%			0%			0%	
Shared Lane Traffic (%)												
Lane Group Flow (vph)	138	184	686	193	297	0	498	613	0	43	351	265
v/c Ratio	0.46	0.34	0.69	0.54	0.28		0.60	0.49		0.36	0.63	0.42
Control Delay (s/veh)	59.9	32.8	14.9	58.8	28.2		46.3	28.1		67.8	52.7	6.3
Queue Delay	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	0.0
Total Delay (s/veh)	59.9	32.8	14.9	58.8	28.2		46.3	28.1		67.8	52.7	6.3
Queue Length 50th (ft)	48	102	213	68	80		162	149		30	124	0
Queue Length 95th (ft)	107	188	457	141	133		#339	298		88	229	68
Internal Link Dist (ft)		585			714			548			965	
Turn Bay Length (ft)	330		200	200						140		345
Base Capacity (vph)	711	984	1030	711	1844		826	2263		156	1840	942
Starvation Cap Reductn	0	0	0	0	0		0	0		0	0	0
Spillback Cap Reductn	0	0	0	0	0		0	0		0	0	0
Storage Cap Reductn	0	0	0	0	0		0	0		0	0	0
Reduced v/c Ratio	0.19	0.19	0.67	0.27	0.16		0.60	0.27		0.28	0.19	0.28

Intersection Summary	
Area Type:	Other

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	127	183	631	194	244	44	458	329	251	41	323	244
Future Volume (vph)	127	183	631	194	244	44	458	329	251	41	323	244
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Lane Width (ft)	12	12	12	12	12	12	12	12	12	12	12	12
Grade (%)		0%			0%			0%			0%	
Storage Length (ft)	330		200	200		110	0		0	140		345
Storage Lanes	1		1	1		1	2		0	1		1
Taper Length (ft)	25			25			25			25		
Right Turn on Red			Yes			Yes			Yes			Yes
Link Speed (mph)		30			30			30			30	
Link Distance (ft)		665			794			628			1045	
Travel Time (s)		15.1			18.0			14.3			23.8	
Confl. Peds. (#/hr)												
Confl. Bikes (#/hr)												
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Growth Factor	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%
Bus Blockages (#/hr)	0	0	0	0	0	0	0	0	0	0	0	0
Parking (#/hr)												
Mid-Block Traffic (%)		0%			0%			0%			0%	
Shared Lane Traffic (%)												
Lane Group Flow (vph)	138	199	686	211	313	0	498	631	0	45	351	265
v/c Ratio	0.46	0.36	0.70	0.56	0.29		0.61	0.51		0.37	0.63	0.42
Control Delay (s/veh)	60.9	33.6	15.6	59.4	28.2		47.4	28.8		69.2	53.6	6.3
Queue Delay	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	0.0
Total Delay (s/veh)	60.9	33.6	15.6	59.4	28.2		47.4	28.8		69.2	53.6	6.3
Queue Length 50th (ft)	50	114	223	76	85		167	158		32	126	0
Queue Length 95th (ft)	108	205	473	152	140		#343	309		91	231	69
Internal Link Dist (ft)		585			714			548			965	
Turn Bay Length (ft)	330		200	200						140		345
Base Capacity (vph)	700	970	1021	700	1822		815	2228		154	1813	933
Starvation Cap Reductn	0	0	0	0	0		0	0		0	0	0
Spillback Cap Reductn	0	0	0	0	0		0	0		0	0	0
Storage Cap Reductn	0	0	0	0	0		0	0		0	0	0
Reduced v/c Ratio	0.20	0.21	0.67	0.30	0.17		0.61	0.28		0.29	0.19	0.28

Intersection Summary	
Area Type:	Other

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Queuing Analysis

Arrival Rate	λ	39
Service Rate	μ	120
Number of Car Wash Tunnels	s	1
Traffic Intensity	ρ	0.33
Server Utilization	α	0.33
Average Number of Vehicles	L	0.48
Average Time Spent in System	W	0.74
Average Time Spent in Queue	WQ	0.24
Average Vehicles in Queue	LQ	0.16
Probability of Zero Vehicles in Queue	p_0	68%
Probability of n Vehicles in Queue	$p_n = 1$	22%
Probability of n Vehicles in Queue	$p_n = 2$	7%
Probability of n Vehicles in Queue	$p_n = 3$	2%
Probability of n Vehicles in Queue	$p_n = 4$	1%
https://www.omnicalculator.com/math/queueing-theory#a-new-checkout-opens-the-mms-queues		

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