TRAFFIC IMPACT ANALYSIS

Choctaw Nation Hochatown Resort
US-259 & SH-259A
Broken Bow, Oklahoma

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INTRODUCTION

This traffic study was conducted to analyze the potential traffic impacts of a proposed Choctaw Nation Hochatown Resort. The development is to be located on undeveloped property at the southeast corner of the northern intersection of US-259 and SH-259A (North), in Broken Bow, Oklahoma, McCurtain County. A vicinity map of the subject area is provided in **Figure 1**. The proposed development is planned to include a hotel, casino, office, restaurant, retail, outdoor entertainment space, and a convenience store with fuel pumps.

Three (3) access driveways are proposed for the development. All proposed access is from US-259 and SH-259A (North). One (1) full-access driveway will be provided on US-259 south of SH-259A (North). Two (2) driveways will be provided on SH-259A, east of US-259. The eastern driveway will provide full-access and the western driveway will be enter only. The specific focus of this study is to analyze the potential impact that the proposed development will have on the adjacent roadways and intersections by comparing the projected operating conditions at the study area intersections with and without the proposed development.

It is assumed that the proposed development will be completed within two (2) years, with a target date for occupancy/site build-out in 2023. The site plan for the proposed development is provided in **Figure 2**. The following elements were compiled and/or are addressed in the study:

Data Collection

- Obtained 24-hour traffic volumes and intersection turning movement counts at relevant locations at or near the existing site collected in July of 2021.
- Obtained the proposed site plan.

Traffic Analysis

- Estimated the number of trips to be generated by the proposed development.
- Estimated directional distribution of traffic approaching/departing the proposed development.
- Assigned the estimated traffic to the study area intersections and roadways.
- Projected area traffic growth.
- Performed capacity analyses for the critical intersections/roadways within the study area.
- Performed signal warrant analysis for the proposed driveway intersection with US-259.
- Compared capacity analysis results to assess the projected impacts of the proposed development along the adjacent roadways and the study area intersections.

Recommendations

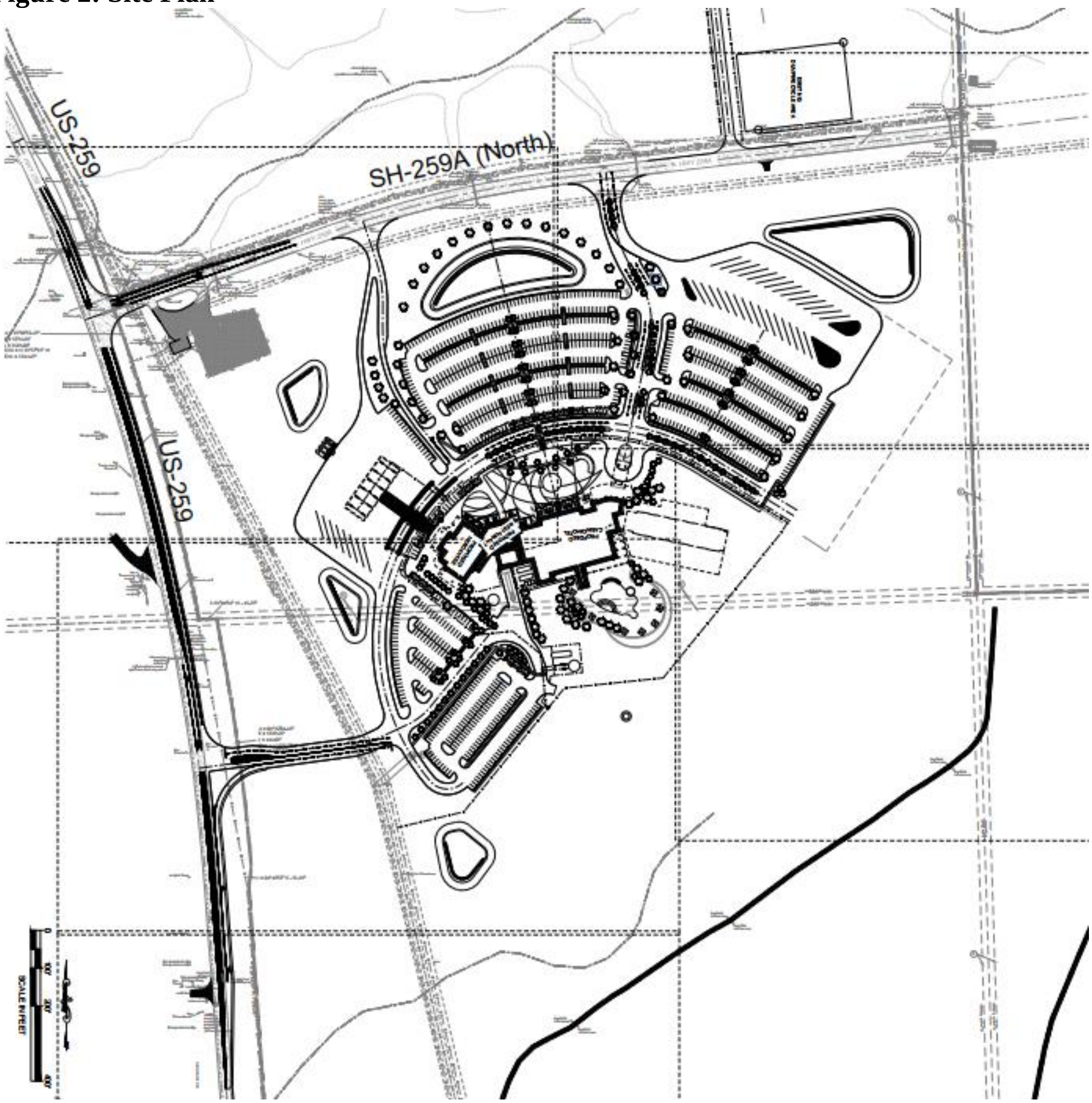
- Determined if roadway improvements are needed to accommodate projected traffic generated by the proposed development and recommended appropriate roadway designs, intersection lane configurations, and traffic control to accommodate the proposed development.

Documentation

- Prepared this report documenting the study procedures and results.

Figure 1: Vicinity Map

Figure 2: Site Plan



EXISTING CONDITIONS

The following is a brief description of the existing conditions near the proposed development pertaining to land use, roadway features, and traffic characteristics.

Land Use

Within and adjacent to the study area, land use is generally rural and undeveloped. At the intersection between US-259 and SH-259A (North), the McCurtain County National Bank was recently constructed on the west side of US-259 with a driveway approximately 100 feet south of SH-259A (North). East of the proposed development, there are scattered residences and cabins, but no significant development. Further east and north of the proposed development lie Beavers Bend and Hochatown State Parks, which are protected recreational areas.

Roadway Characteristics

<u>US-259</u> – US-259 near Broken Bow Lake is a two-lane highway west of the proposed development site that runs north-south. For most of the study area, the roadway typical section is composed of two 13’ driving lanes with 6’ shoulders and a posted speed limit of 55 miles per hour (MPH). US-259 is classified by the *Oklahoma Department of Transportation* (ODOT) as a Principal Arterial. One (1) new full access site driveway is proposed along US-259. This driveway will be south of the SH-259A (North) intersection. The pavement along US-259 was noted to be in good condition.

<u>SH-259A</u> – SH-259A is a ten-mile loop that connects US-259 to Broken Bow Lake and Beavers Bend State Park. The northern junction, SH-259A (North) is within the study area and is a two-lane highway with 12’ driving lanes and 4’ shoulders. This roadway is classified as a Major Collector by ODOT and has a posted speed limit of 55 MPH. Two (2) access driveways are proposed along SH-259A; one (1) driveway proposed as full-access and one (1) driveway proposed as enter only. SH-259A (North) is stop-controlled at its terminus, the intersection with US-259. The pavement along SH-259A was noted to be in good condition.

The existing lane configurations of the study area roadways and intersections are shown in **Figure 3**. The proposed build-out lane configurations of the study area roadways, site driveways, and intersections are shown in **Figure 4**.

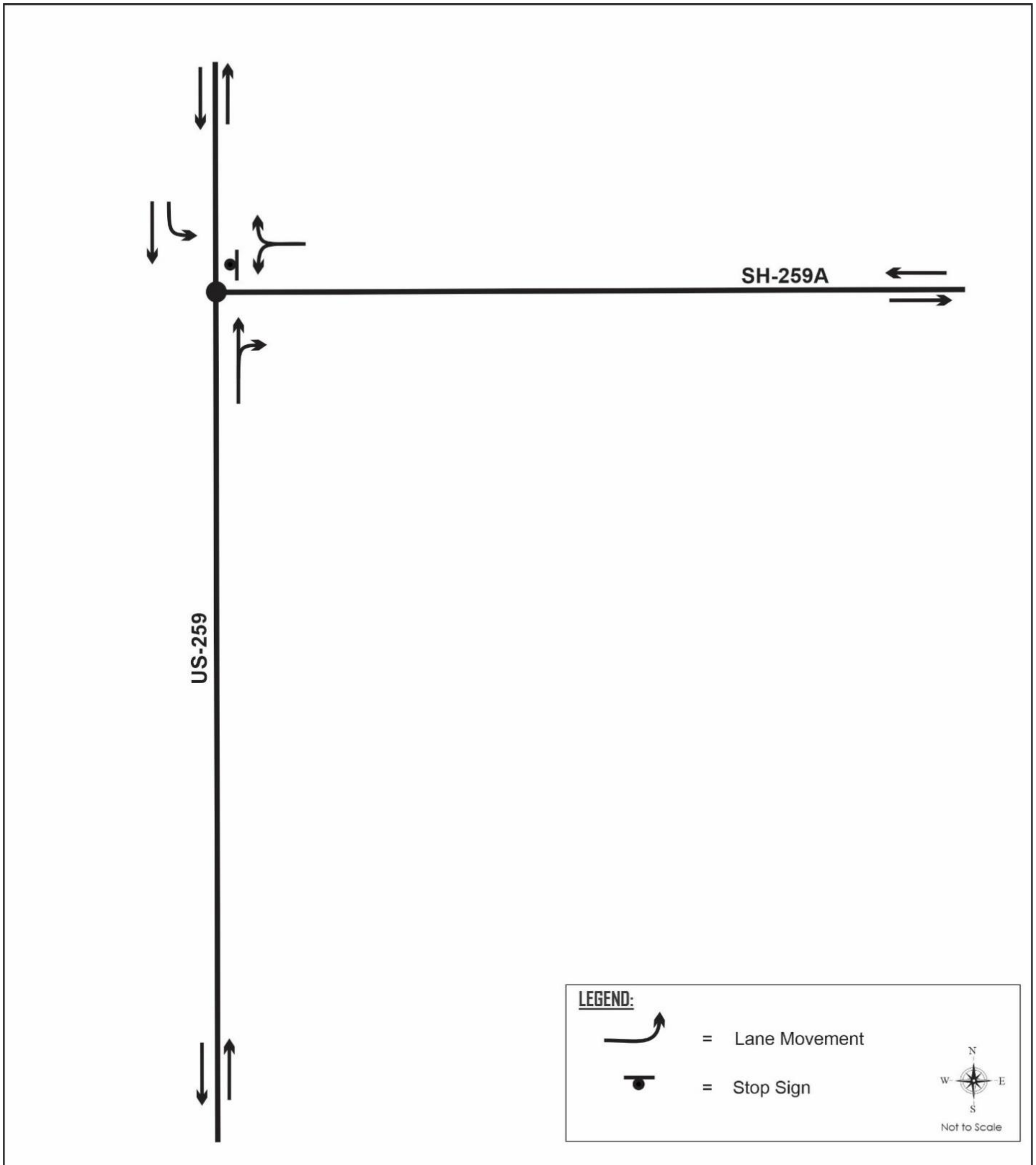


Figure 3

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EXISTING LANE CONFIGURATIONS

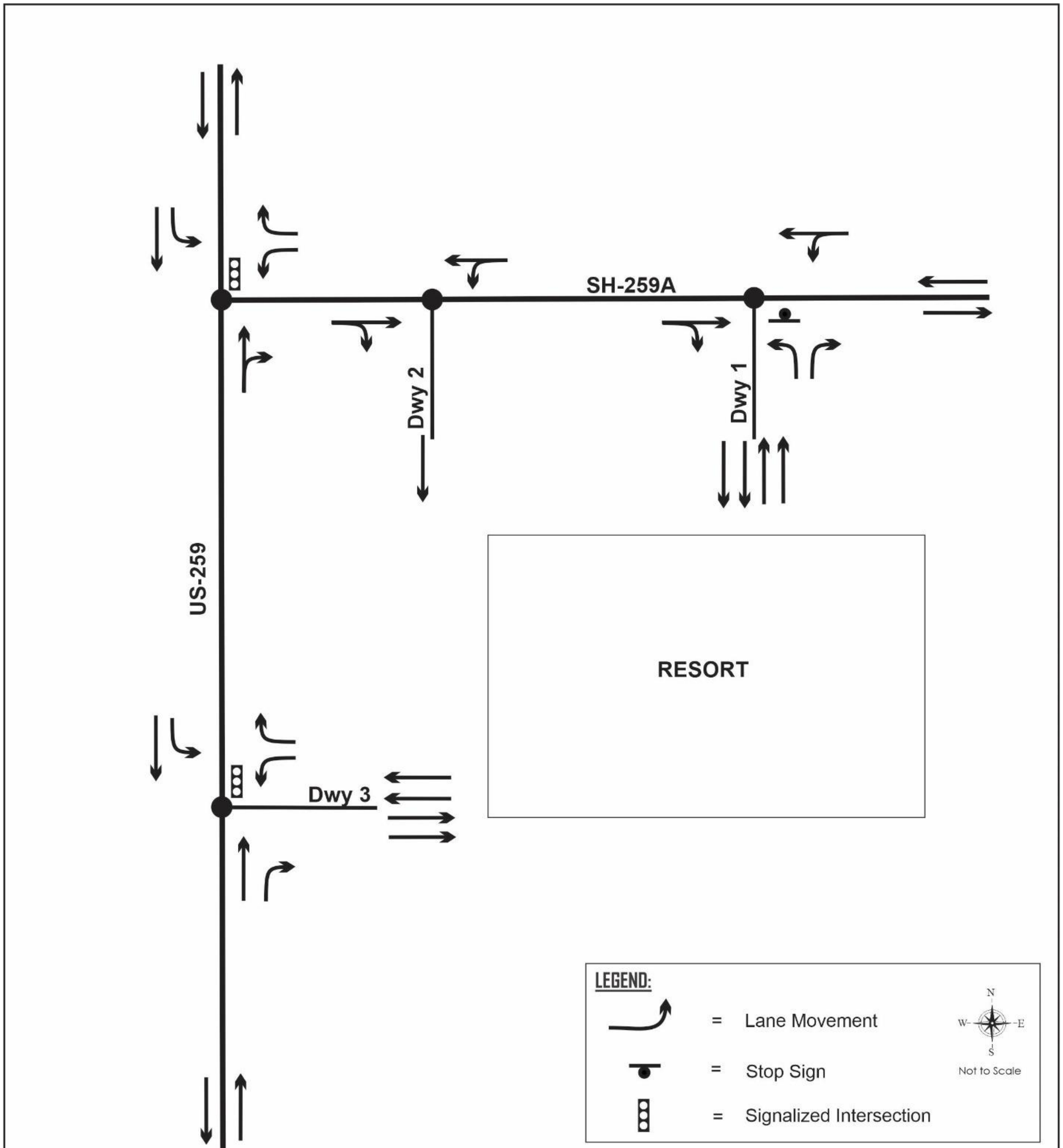

Figure 4

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PROPOSED LANE CONFIGURATIONS

Traffic Characteristics

Existing Traffic Volumes

Existing 24-hour bi-directional traffic volumes were collected on Friday, July 30, 2021, and Saturday, July 31, 2021, on US-259 and SH-259A (North). Observed truck percentages were collected for the same 24-hour periods from which the hourly traffic volume data was obtained. Intersection turning movement counts were also collected during the same 24-hour periods at the unsignalized intersection of US-259 and SH-259A (North).

Friday and Saturday were considered to be when the proposed development would have the most significant impact on the surrounding roadway network due to high site traffic demands and corresponding peak travel along US-259. The data was captured in 15-minute intervals in order to determine the peak one-hour volumes to analyze. Results indicate peak hour traffic conditions near the subject site beginning at 10:45 AM for the morning peak hour and 3:30 PM for the afternoon peak hour on a typical Friday. On a typical Saturday, the afternoon peak hour near the subject site begins at 12:45 PM.

Impacts of COVID-19 Pandemic

Due to the COVID-19 pandemic, existing traffic counts are generally suppressed due to travel demand changes. The existing traffic counts were analyzed and compared to historic growth within the area to determine if adjustments were needed to reflect existing demand that would likely be present without the pandemic's effects. However, upon comparing historical data with current growth and collected turning movement counts, a COVID-19 adjustment factor was determined not necessary for the purpose of this analysis.

Figure 5 summarizes the existing (2021) volume data. The raw intersection turning movement count data is included in the Appendix.

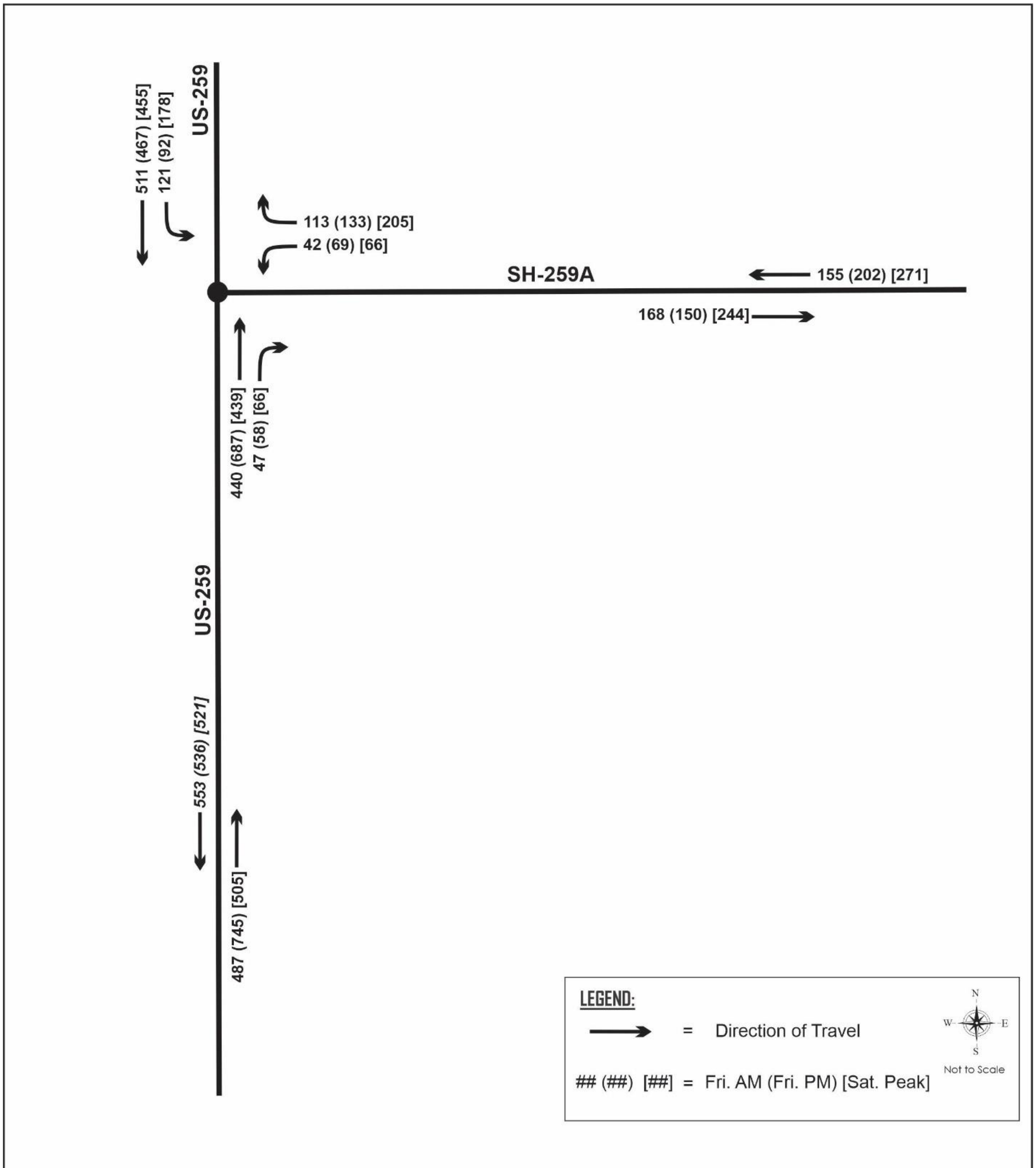

Figure 5

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EXISTING (2021) PEAK HOUR VOLUMES

PROPOSED SITE

Proposed Site Layout

The layout of the proposed development was provided previously in Figure 2. All of the development will take place south of SH-259A (North). All land uses can be accessed from any of the proposed three (3) driveways. Driveway 2 provides the most direct access to the proposed convenience store with 12 fueling positions, although access to the other land uses is also available. Parking is proposed around the north and west sides of the development. No other future developments were included in the analysis.

Site Accessibility

Site accessibility describes the ease with which vehicles can get to and from a development. A site's accessibility is affected by the geographical location of the development with respect to other activity areas, the roadway system, turning movement restrictions, and physical constraints such as rivers or lakes.

There are two (2) driveways proposed on SH-259A (North). All driveways provide through access to all proposed land uses; however, one (1) driveway along SH-259A is proposed to provide direct access to the convenience store and fueling stations. The eastern full-access driveway along SH-259A will be referred to as DWY 1, and the western driveway providing enter-only access will be referred to as DWY 2. DWY 1 is anticipated to be a four-lane driveway with two lanes for entering and exiting traffic. DWY 2 is anticipated to be a two-lane one-way driveway for entering vehicles only (southbound).

There is one (1) full access driveway proposed on US-259 located south of SH-259A (North). This driveway will be referred to as DWY 3. The driveway is anticipated to be a four-lane driveway with two-lanes in both directions and a channelized right-turn lane for entering northbound traffic. This driveway is proposed approximately 1,200 feet south of the existing US-259 and SH-259A (North) intersection.

TRIP GENERATION

The number of vehicle trips generated by the proposed development was estimated based on the information published by the *Institute of Transportation Engineers* (ITE), as contained in the *Trip Generation Manual, Latest Edition*, which includes trip generation estimates for different types of land use sites. The data cited in the information sources were collected for the average daily Weekday, Friday AM and PM peak hour of generator, average daily Saturday, and Saturday peak hour of generator conditions.

The proposed resort offers an Outdoor Entertainment Space of 25,000 square feet. For the purpose of this analysis, the space is assumed to act similar to a concert or event venue and will likely only be utilized during planned event days. Therefore, two scenarios were analyzed for the proposed development:

1. The proposed resort on a typical Friday during the AM and PM peak, and on a typical Saturday peak ***without*** an event.
2. The proposed resort on a typical Friday during the PM peak, and on a typical Saturday peak ***with*** a planned sold-out event.

The trip generation rates/equations used for this development and the directional splits for the proposed land uses are shown in **Table 1.**

The *ITE Trip Generation Manual, Latest Edition*, does not provide data for the proposed casino and outdoor entertainment land uses. Therefore, trip generation rates and directional splits were estimated using research data published in *The Final Environmental Impact Statement, Cowlitz Indian Tribe Trust Acquisition and Casino Project* report and supporting data from *Allentown Arena and City Center Development* traffic analysis.

For the proposed outdoor entertainment space of 25,000 square feet, a maximum capacity event was estimated using ten (10) square feet per attendee resulting in 2,500 people at maximum capacity. A trip generation rate used to determine entering trips was determined by using an assumed 2.75 persons per car occupancy factor and by assuming 61 percent of vehicle trips arrive during the one (1) hour peak evaluated. Upon using the calculated trip generation rate to obtain the Friday PM peak and Saturday peak hour with event total entering trips, exiting trips were estimated by assuming ten (10) percent of the total entering trips exited within the same peak hour. This would account for ride-sharing (such as Uber and Lyft) and other drop-off vehicles.

Additional information regarding the research utilized is provided in the Appendix.

Table 1: Trip Generation Rates for Proposed Development

LAND USE	**Land Use**	**Hotel**	**Casino[1]**	**Outdoor Entertainment Center[1]**	**Gasoline Station w/ Convenience Market**
	ITE Land Use Code	**310**	***N/A***	***N/A***	**945**
	Independent Variable	**Rooms**	**1,000 SF**	**Attendees**	**Fueling Positions**
RATES[2]	Average Weekday	T = 8.36(X)	T = 74.63(X)	-	T = 205.36(X)
	Friday AM	Ln(T)=0.84Ln(X)+ 0.25	T = 2.95(X)	-	T = 13.66(X)
	Friday PM	Ln(T)=0.93Ln(X)- 0.14	T = 9.18(X)	T = 0.24(X)	T = 15.87(X)
	Average Saturday	T = 8.19(X)	T = 93.24(X)	-	T = 154.02(X)[3]
	Saturday Peak Hour	T = 0.72(X)	T = 15.50(X)	T = 0.24(X)	T = 19.28(X)
DIRECTIONAL SPLIT (% in / % out)	Average Weekday	50 / 50	50 / 50	-	50 / 50
	Friday AM	54 / 46	70 / 30	-	51 / 49
	Friday PM	58 / 42	53 / 47	90 / 10	50 / 50
	Average Saturday	50 / 50	50 / 50	-	50 / 50
	Saturday Peak Hour	56 / 44	62 / 38	90 / 10	50 / 50

T = TRIP ENDS, X = UNIT VARIABLE

[1]The ITE manual does not provide specific data for a casino and outdoor entertainment land use. Therefore, trip generation rates and directional splits from *The Final Environmental Impact Statement, Cowlitz Indian Tribe Trust Acquisition and Casino Project* report and *the Allentown Arena and City Center Development* traffic analysis were used for estimating trip generation values for the casino and outdoor entertainment land uses.

[2]Weekday rates are used for ITE Trip Generation land uses that do not specify rates for Fridays specifically.

[3]Daily Saturday rate is not available for this land use. Assumed average Saturday would equal 75% of Average Weekday rate.

Using the trip generation rates/equations from Table 1, the resulting estimated trips generated by the proposed development for a typical Friday ***without*** a planned event are provided inError! Reference source not found.. Estimated trips generated by the proposed development for a typical Saturday ***without*** a planned event are provided in **Table 3.**

Table 2: Estimated Trip Generation for Proposed Development – Friday (No Event)

Trip Generator	ITE Land Use Code	Average Weekday			Friday AM Peak Hour			Friday PM Peak Hour		
		Total	In	Out	Total	In	Out	Total	In	Out
Hotel (200 Rooms)	**310**	1,672	836	836	110	59	51	120	70	50
Casino (36,000 GFA)	**N/A[1]**	2,688	1,344	1,344	106	74	32	330	175	155
Gasoline Station w/ Convenience Store (12 Fueling Positions)	**945**	2,464	1,232	1,232	164	84	80	190	95	95
Total Trips Generated		6,824	3,412	3,412	380	217	163	640	340	300
Internal Capture		-	-	-	(70)	(35)	(35)	(178)	(89)	(89)
TOTAL PEAK HOUR EXTERNAL TRIPS		**6,824**	**3,412**	**3,412**	**310**	**182**	**128**	**462**	**251**	**211**

[1]The ITE manual does not provide specific data for a casino and outdoor entertainment land use. Therefore, trip generation rates and directional splits from *The Final Environmental Impact Statement, Cowlitz Indian Tribe Trust Acquisition and Casino Project* report and *the Allentown Arena and City Center Development* traffic analysis were used for estimating trip generation values for the casino and outdoor entertainment land uses.

Table 3: Estimated Trip Generation for Proposed Development – Saturday (No Event)

Trip Generator	ITE Land Use Code	Average Saturday			Saturday Peak Hour Generator		
		Total	In	Out	Total	In	Out
Hotel (200 Rooms)	**310**	1,638	819	819	144	81	63
Casino (36,000 SF GFA)	**N/A[1]**	3,358	1,679	1,679	558	346	212
Gasoline Station w/ Convenience Store (12 Fueling Positions)	**945**	1,848	924	924	231	115	116
Total Trips Generated		6,844	3,422	3,422	933	542	391
Internal Capture		-	-	-	(216)	(108)	(108)
TOTAL PEAK HOUR EXTERNAL TRIPS		**6,844**	**3,422**	**3,422**	**717**	**434**	**283**

[1]The ITE manual does not provide specific data for a casino and outdoor entertainment land use. Therefore, trip generation rates and directional splits from *The Final Environmental Impact Statement, Cowlitz Indian Tribe Trust Acquisition and Casino Project* report and *the Allentown Arena and City Center Development* traffic analysis were used for estimating trip generation values for the casino and outdoor entertainment land uses.

Pass-by trips are essentially site-generated trips that materialize out of mere convenience. In other words, a trip may be generated simply because a vehicle regularly travels the adjacent street and decides to “stop in” because of the favorable location. Thus, the land use generating the trip would not be responsible for that vehicle’s presence on the adjacent street. Due to the low existing traffic volumes on the adjacent street, the rural site location, and in order to provide a more conservative estimate of generated trips, a reduction in total site generated trips due to pass-by trips was not factored. However, it is likely the proposed convenience store and fueling station will experience pass-by trips.

Internally captured trips can be a significant component in the travel patterns at multi-use developments. An internal capture rate can generally be defined as a percentage reduction that can be applied to the trip generation estimated for individual land uses to account for trips internal to the overall site. Chapter 7 of *ITE’s Trip Generation Handbook, Latest Edition* outlines the procedure for estimating trip generation within a multi-use development. For the Choctaw Nation Hochatown Resort development, this procedure was applied to the Casino, Outdoor Entertainment Space, and Hotel land uses. The Casino and Hotel are located in the same building and will share many internal trips within the overall site. Similarly, attendees of the Outdoor Entertainment Space may travel internally from the Casino or Hotel land use. Internal capture rates between these three land uses were estimated and applied to each peak hour and each scenario analyzed. Worksheets detailing the process and calculations utilized for the internal capture totals are included in the Appendix.

Using the trip generation rates/equations from Table 1, the resulting estimated trips generated by the proposed development for a typical Friday and Saturday ***with*** a planned event are provided in **Table 4**.

The proposed Outdoor Entertainment Space is anticipated to provide 25,000 square feet of event space. For the purpose of this analysis, one (1) attendee was assumed for every 10 square feet of space, resulting in a maximum capacity of 2,500 attendees for a sold-out event.

Table 4: Estimated Trip Generation for Proposed Development – Friday and Saturday (<u>Sold Out Event</u>)

Trip Generator	ITE Land Use Code	Friday PM Peak Hour			Saturday Peak Hour		
		Total	In	Out	Total	In	Out
Hotel (200 Rooms)	**310**	120	70	50	144	81	63
Casino (36,000 SF GFA)	**N/A[1]**	330	175	155	558	346	212
Outdoor Entertainment Center[1] (2,500 Attendees)	**N/A**	611	555	56	611	555	56
Gasoline Station w/ Convenience Store (12 Fueling Positions)	**945**	190	95	95	231	115	116
Total Trips Generated		1,251	895	356	1,544	1,097	447
Internal Capture		(366)	(183)	(183)	(442)	(221)	(221)
TOTAL PEAK HOUR EXTERNAL TRIPS		**885**	**712**	**173**	**1,102**	**876**	**226**

[1]The ITE manual does not provide specific data for a casino and outdoor entertainment land use. Therefore, trip generation rates and directional splits from *The Final Environmental Impact Statement, Cowlitz Indian Tribe Trust Acquisition and Casino Project* report and *the Allentown Arena and City Center Development* traffic analysis were used for estimating trip generation values for the casino and outdoor entertainment land uses.

TRIP DISTRIBUTION & TRAFFIC ASSIGNMENT

The distribution of the site-generated traffic estimated to be entering and exiting the adjacent roadway network was determined based on the existing distribution of traffic established from the collected data and the estimated daily traffic volumes resulting from the proposed development. All site access is provided via US-259 and SH-259A (North). **Figure 6** shows the assumed distribution percentages for the Friday AM and PM peak hours and the Saturday peak hour at the proposed site access driveways. Most of the generated traffic is anticipated to enter and exit the study area via US-259 as it is a major north/south highway for the area.

Applying the assumed distribution to the trip generation totals from Table 2 and Table 3 allows us to create **Figure 7,** which shows the resulting site generated traffic volumes for the typical Friday AM and PM peak hours and the Saturday peak hour at the proposed site access driveways and on the adjacent roadway network under Scenario 1 – No Event conditions.

Applying the assumed distribution to the trip generation totals from Table 4 allows us to create **Figure 8,** which shows the resulting site generated traffic volumes for the typical Friday PM peak hour and the Saturday peak hour at the proposed site access driveways and on the adjacent roadway network under Scenario 2 – Sold Out Event conditions.

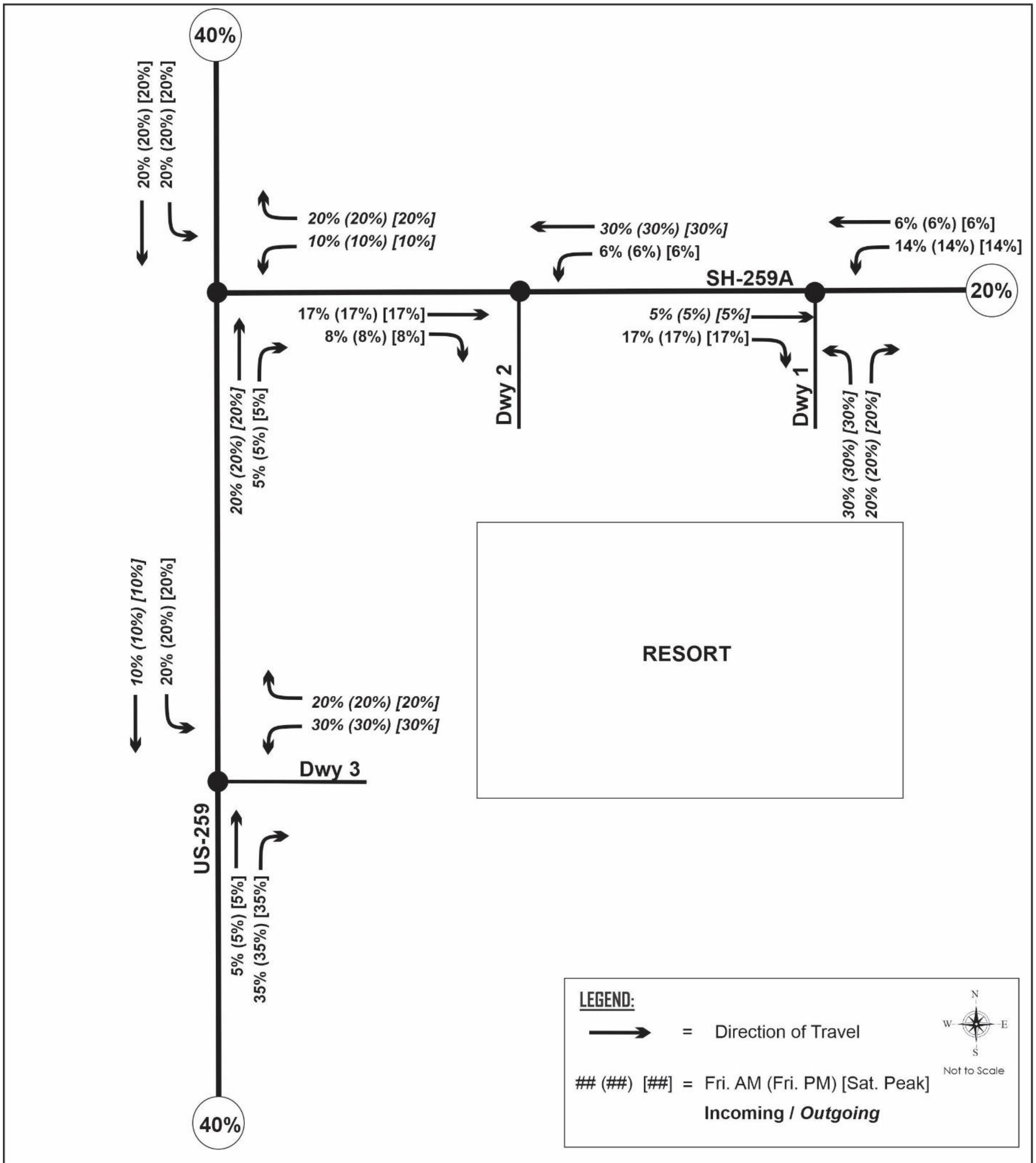

Figure 6

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ASSUMED DIRECTIONAL DISTRIBUTION

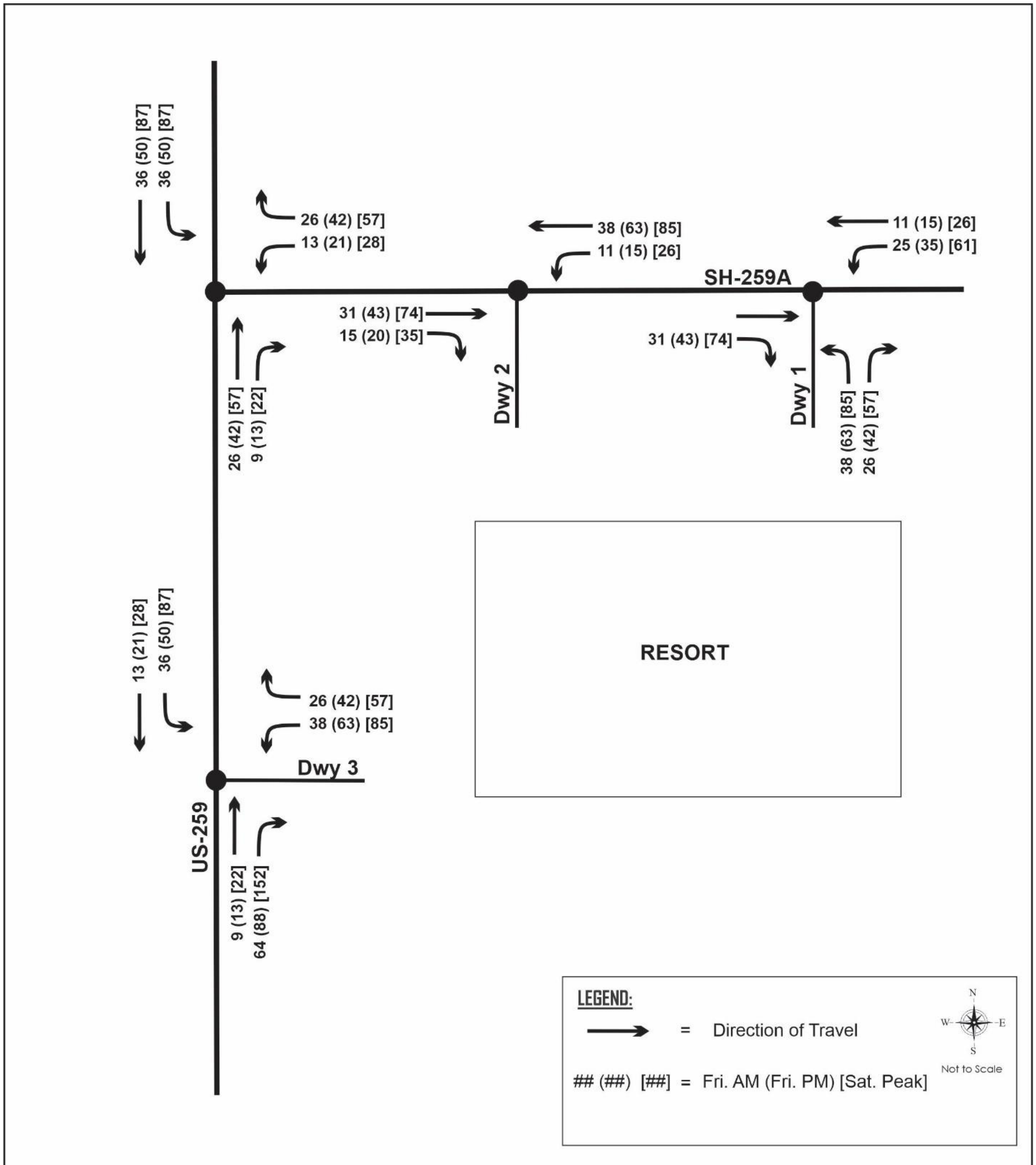

Figure 7

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PROPOSED SITE GENERATED VOLUMES
(SCENARIO 1 - NO EVENT)

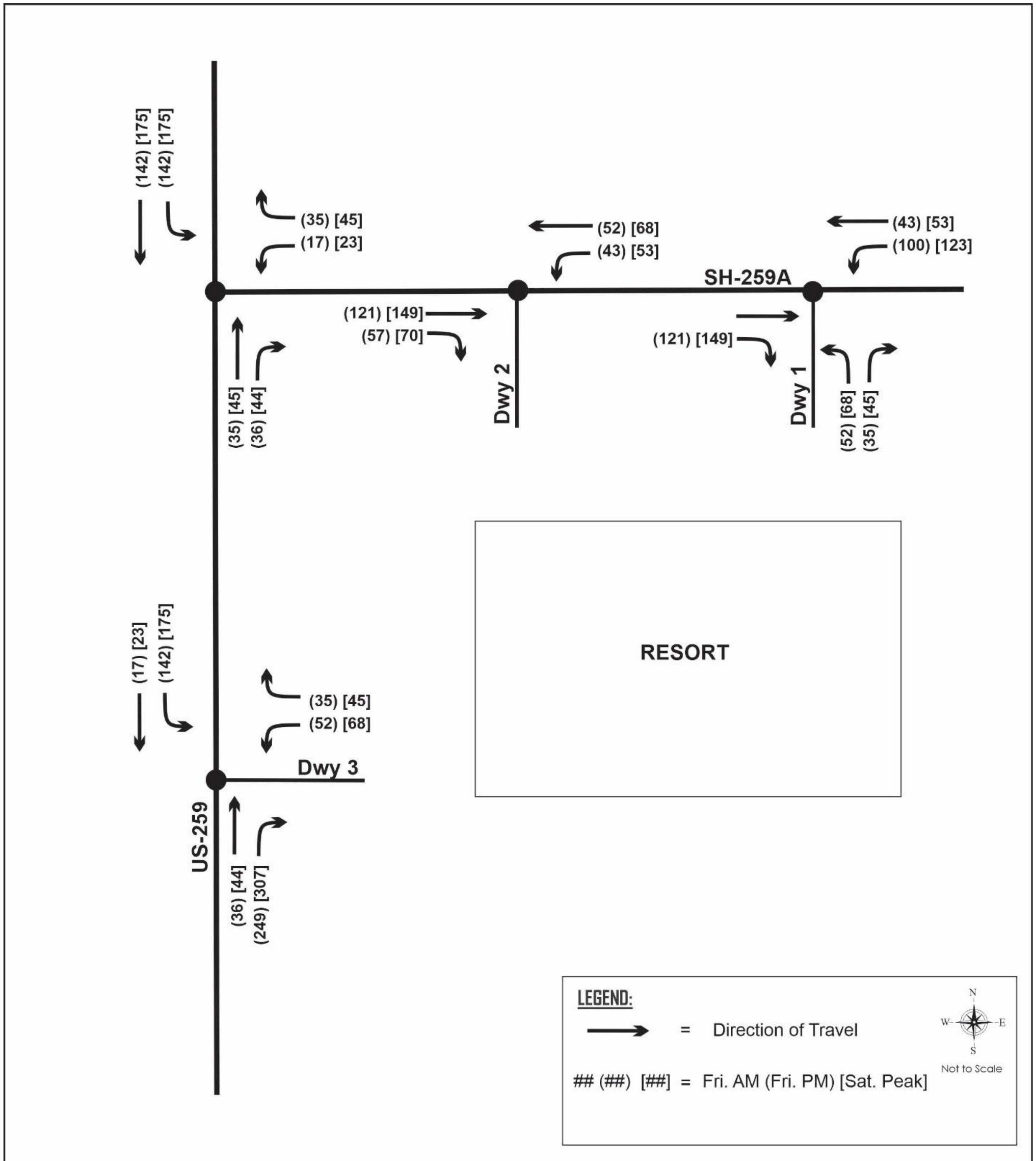

Figure 8

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PROPOSED SITE GENERATED VOLUMES
(SCENARIO 2 - SOLD-OUT EVENT)

BACKGROUND AND TOTAL TRAFFIC CONDITIONS

Historical 24-hour traffic volumes near the study area were obtained from the ODOT Planning & Research Division and are presented in **Table 5**.

Table 5: Historical Traffic Counts

Year	Traffic Count Location				
	US-259 N. of SH-259A (North)	**US-259 S. of SH-259A (North)**	**US-259 between SH-259A (North) & (South)**	**SH-259A (North), E. of US-259**	**SH-259A (South), E. of US-259**
2015	3,500	4,000	3,600	600	460
2016	3,900	4,200	3,900	610	540
2017	4,000	4,300	4,000	630	560
2018	4,100	4,400	4,100	640	570
2019	5,200	4,800	4,800	790	790
Average Growth	**12.1%**	**5.0%**	**8.3%**	**7.9%**	**17.9%**

The traffic volumes in Table 5 show that traffic within the study area has increased approximately 5.0% - 17.9% annually during the most recent 5-year period that data is available. Therefore, to represent a conservative analysis, an annual growth rate of ten percent (10%) was used for determining background traffic conditions for the site build-out year (2023).

The Build-Out (2023) Background Peak Hour Volumes at the study intersections are provided in **Figure 9**. For background traffic volumes, only the existing roadway network, without any additional site development, was considered for the year analyzed. Existing peak hour traffic volumes from Figure 5 were grown by ten percent (10%) annually for two (2) years to develop the 2023 background condition. Based on information from *ODOT's Eight-Year Construction Work Plan (2021 to 2028),* the portion of US-259 from 6.25 miles north of SH-3, extending six (6) miles north, is planned for safety improvements. It is assumed the intersection of US-259 and SH-259A (North) will be converted from a two-way stop-controlled intersection to signalized control. Therefore, the existing intersection has been analyzed as signalized in the Build-Out (2023) Background and Build-Out (2023) Total Traffic condition for both scenarios.

The Build-Out (2023) Total Traffic Volumes for Scenario 1 are shown in **Figure 10** and are comprised of the projected background conditions for the build-out year (from Figure 9) combined with the added subject site-generated traffic (from Figure 7). Similarly, the Build-Out (2023) Total Traffic Volumes for Scenario 2 are shown in Error! Reference source not found. and a re comprised of the projected background conditions for the build-out year (from Figure 9) combined with the added subject site-generated traffic (from Figure 8).

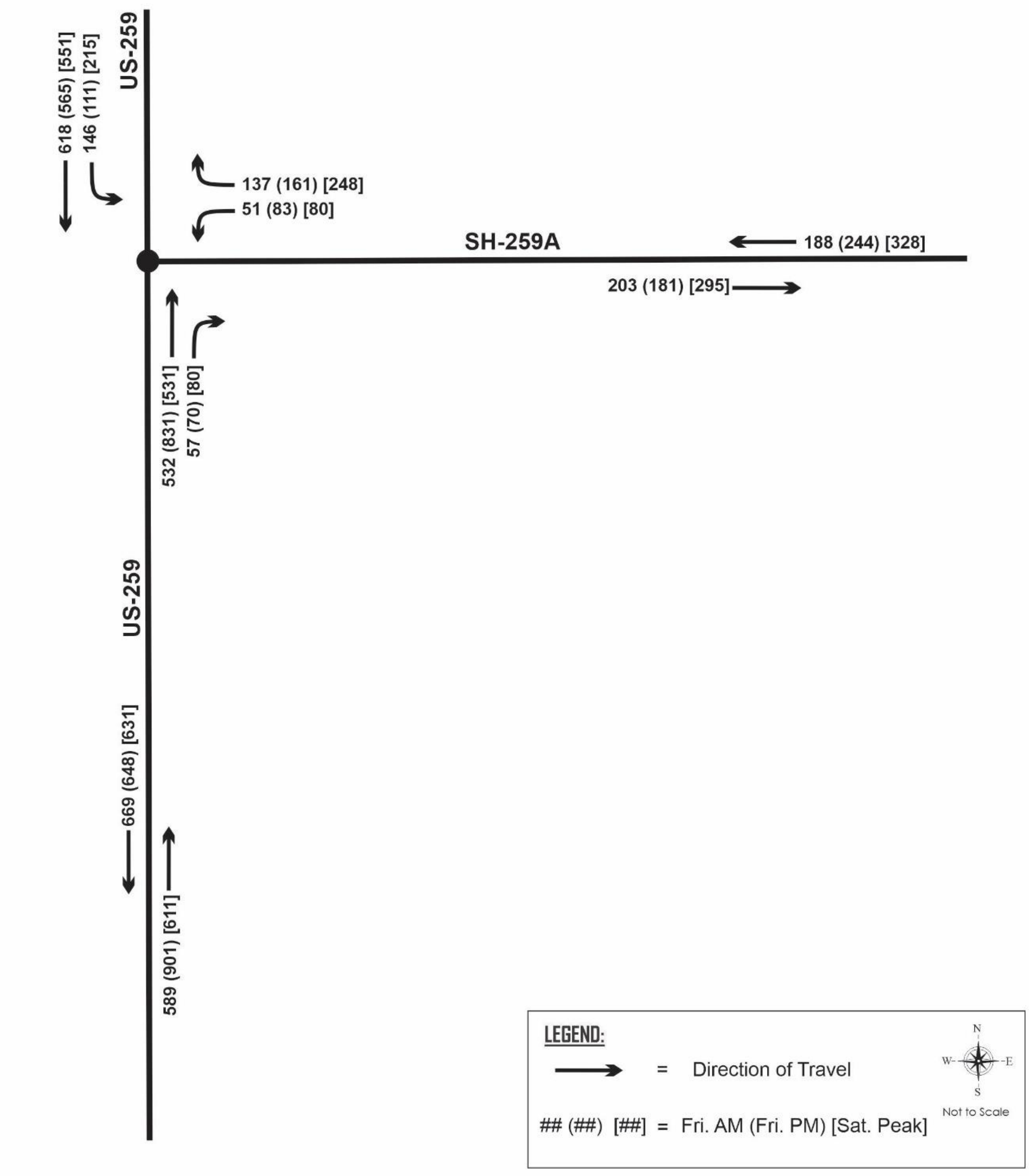

Figure 9

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BUILD-OUT (2023) BACKGROUND TRAFFIC VOLUMES

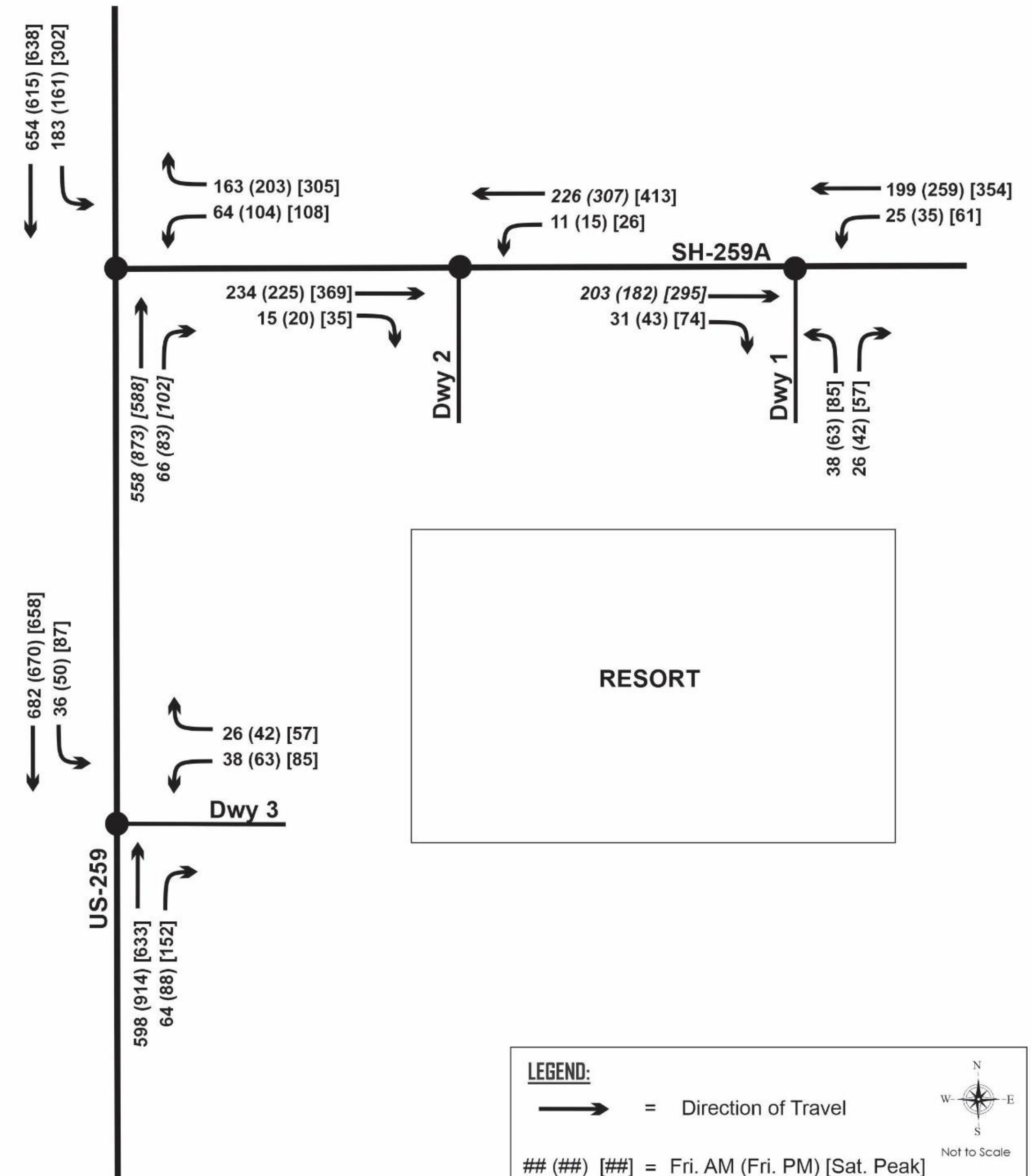

Figure 10

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BUILD-OUT (2023) TOTAL PEAK HOUR VOLUMES
(SCENARIO 1)

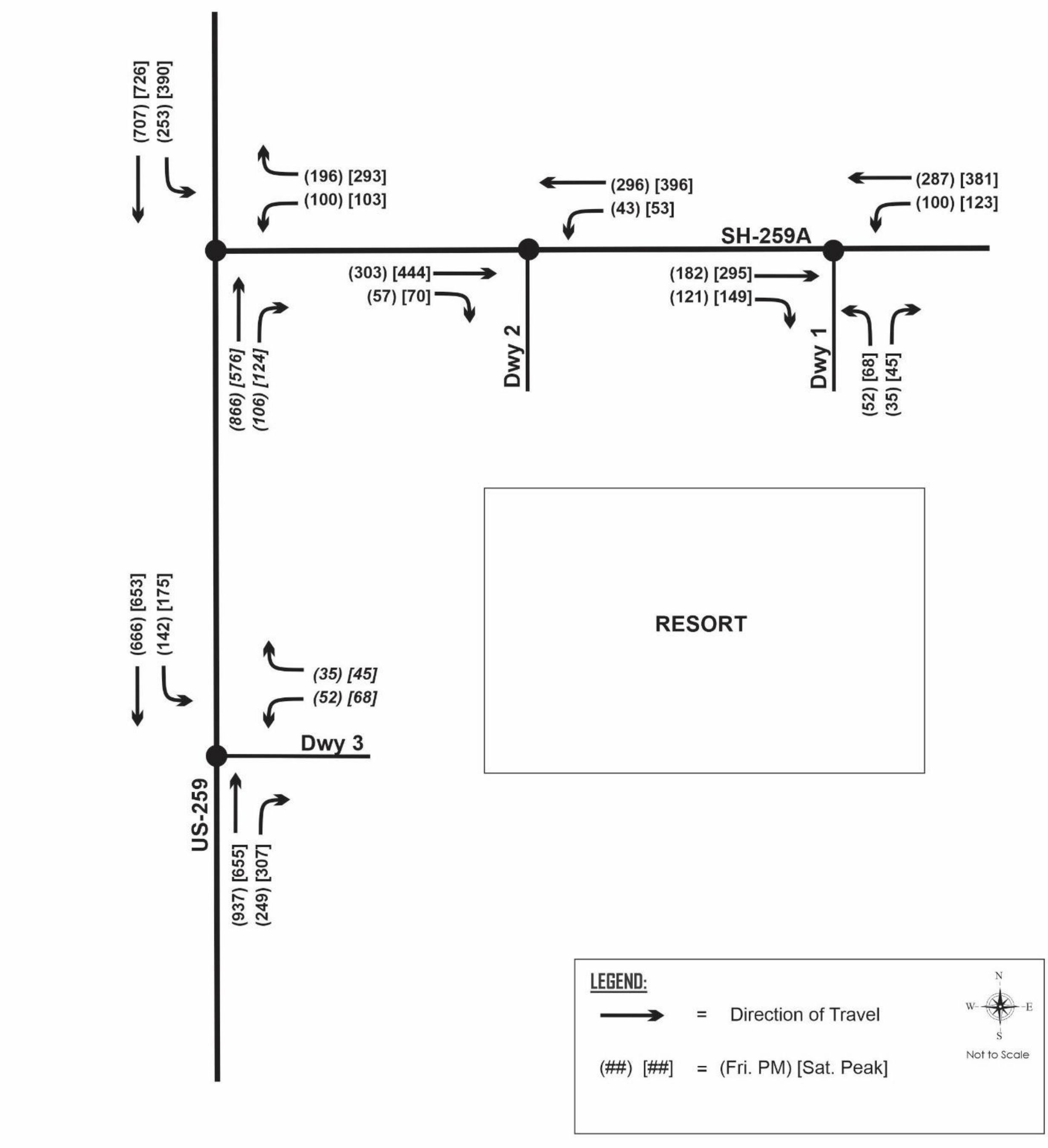

Figure 11

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BUILD-OUT (2023) TOTAL PEAK HOUR VOLUMES
(SCENARIO 2)

TRAFFIC SIGNAL WARRANT ANALYSIS

Introduction

A future traffic signal warrant analysis has been conducted for the intersection of US-259 and proposed Driveway 3 to determine if signalization will be warranted at this location upon the completion of the Choctaw Nation Hochatown Resort. This report summarizes the results of the traffic signal warrant analysis conducted for the intersection.

The analysis was performed using predicted Build-Out (2023) Total traffic volumes for a typical weekday at the intersection under Scenario 1 (No Event).

The traffic signal warrant analysis presented in this report is based on the traffic signal warrants contained in Chapter 4C, “Traffic Control Signal Needs Studies,” of the *Manual on Uniform Traffic Control Devices* (MUTCD), *latest edition*. Nine warrants are included in the manual for warranting a traffic signal installation. These warrants are:

Warrant 1 – Eight-Hour Vehicular Volume
Warrant 2 – Four-Hour Vehicular Volume
Warrant 3 – Peak Hour
Warrant 4 – Pedestrian Volume
Warrant 5 – School Crossing
Warrant 6 – Coordinated Signal System
Warrant 7 – Crash Experience
Warrant 8 – Roadway Network
Warrant 9 – Intersection Near a Railroad Grade Crossing

The most current population estimate for the nearby City of Broken Bow is 4,104 (*US Census Bureau, 2019 US Census*).

US-259 & Driveway 3 Intersection

US-259 is a two-lane undivided highway with a posted speed limit of 55 MPH near the study intersection. US-259 is classified as a Principal Arterial by ODOT. Driveway 3 is proposed south of Pinyon Road and would provide access east of US-259. The proposed site plan depicts Driveway 3 with separate westbound right and left-turn lanes for vehicles exiting the resort. For purposes of this analysis, Driveway 3 was considered a one-lane approach, and the right-turn volumes were not removed from consideration as conflict with right-turning vehicles entering the major roadway is anticipated. A dedicated southbound left-turn lane and dedicated northbound channelized right-turn lane along US-259 are also shown on the site plan.

A full description of Warrants 1 through 9 for the US-259 and Driveway 3 intersection is included in the Appendix.

Warrant Summary

A summary of the traffic signal warrants for the intersection of US-259 and Driveway 3 under future conditions is provided in **Table 6**.

Based on the projected traffic volumes and analysis, traffic signal warrants are satisfied for the intersection of US-259 and Driveway 3 under predicted Build-Out (2023) Total traffic conditions. For purposes of this analysis, Driveway 3 was considered a one-lane approach, and the right-turn volumes were not removed from consideration as conflict with right-turning vehicles entering the major roadway is anticipated. A summary of the traffic signal warrants is provided in Table 6**.**

Table 6: Warrant Summary (US-259 and Driveway 3)

Warrant	Warrant Met?	Notes
1 – Eight-Hour Vehicular Volume	YES	11 hours met (8 required)
2 – Four-Hour Vehicular Volume	YES	9 hours met (4 required)
3 – Peak Hour	N/A	Not considered a special generator
4 – Pedestrian Volume	NOT EVALUATED	Pedestrian data not collected
5 – School Crossing	N/A	Not an established school crossing
6 – Coordinated Signal System	N/A	Not part of a progressive signal system
7 – Crash Experience	NO	Collision history does not meet warrants
8 – Roadway Network	N/A	Not an intersection of two major routes
9 – Near a Grade Crossing	N/A	Not adjacent to a railroad grade crossing

Based on the results of this traffic signal warrant analysis, the installation of a traffic signal at the intersection of US-259 and Driveway 3 is predicted to be warranted with build-out of the proposed development. It is recommended that traffic demands be monitored alongside new development and a traffic signal be installed at this location as development traffic is realized.

OPERATIONAL ANALYSIS

Roadway Link Capacity Analysis

Roadway capacity is defined as the volume of traffic that a roadway can accommodate based on the road's width, traffic control, parking conditions, and several other factors. Service volume for collector roadways (such as SH-259A North) are generally considered at 8,750 vehicles per day per lane for a LOS E. Service volume for 2-lane arterials (such as US-259) are generally considered at 17,100 vehicles per day per lane for LOS E. Roadway link capacity can be found by comparing the daily volumes to the LOS E criteria volumes.

- if Volume/Service Volume Ratio is <= 0.45, then LOS = A or B
- if Volume/Service Volume Ratio is > 0.45 and <= 0.65, then LOS = C
- if Volume/Service Volume Ratio is > 0.65 and <= 0.80, then LOS = D
- if Volume/Service Volume Ratio is > 0.80 and <= 1.00, then LOS = E
- if Volume/Service Volume Ratio is > 1.00, then LOS = F

Table 7 provides the roadway link capacity of US-259 and SH-259A (North) using ADT volumes obtained in July 2021. The following tables present ADT on a typical weekday using Friday data collection and estimates. Existing (2021) ADT was grown at ten (10) percent annually for two (2) years to obtain Background ADT for year 2023. For Build-Out ADT, the total predicted ADT for the proposed development on a typical weekday (6,824 total trips) was added to the Background ADT estimate assuming forty (40) percent of total trips are using US-259 from the north and south, and twenty (20) percent of total trips are using SH-259A.

Table 7: Roadway Link Capacity Analysis – 2-Lane Facility

Roadway	Facility Type	LOS E Capacity (vpd)	Analysis Period	ADT (Existing)	v/c Ratio	LOS
US-259 north of SH-259A (North)	2-lane	17,100	Existing	15,828	0.93	**E**
			Background	19,151	1.13	**F**
			Build-out	21,881	1.28	**F**
US-259 south of SH-259A (North)	2-lane	17,100	Existing	14,558	0.85	**E**
			Background	17,615	1.03	**F**
			Build-out	20,345	1.19	**F**
SH-259A east of US-259	2-lane	17,100	Existing	8,750	0.51	C
			Background	10,588	0.62	C
			Build-out	11,953	0.70	D

vpd = vehicles per day; ADT = average daily traffic; v/c = volume to capacity ratio; LOS = Level of Service

As shown in Table 7, US-259 north and south of SH-259 A (North) currently performs at LOS E during Existing conditions and LOS F during Build-Out Background (2023) and Build-Out (2023) conditions. **Table 8** presents predicted roadway link capacity level of service for US-259 assuming it is widened to three (3) or four (4) lane cross-sections.

Table 8: Roadway Link Capacity Analysis – 3-Lane and 4-Lane Facility

Roadway	Facility Type	LOS E Capacity (vpd)	Analysis Period	ADT	v/c Ratio	LOS
US-259 north of SH-259A (North)	3-lane	17,955	Background	19,151	1.06	**F**
	3-lane		Build	21,881	1.22	**F**
	4-lane	34,200	Background	19,151	0.46	A
	4-lane		Build	21,881	0.64	C
US-259 south of SH-259A (North)	3-lane	17,955	Background	17,615	0.98	**F**
	3-lane		Build	20,345	1.13	**F**
	4-lane	34,200	Background	17,615	0.52	C
	4-lane		Build	20,345	0.59	C

vpd = vehicles per day; ADT = average daily traffic; v/c = volume to capacity ratio; LOS = Level of Service

As shown in Table 8, the estimated ADT under Build-Out Background (2023) and Build-Out Total (2023) conditions is predicted to operate at LOS F with a three (3) lane section. US-259 is predicted to perform at LOS C or better with a four (4) lane cross-section. It is recommended that US-259 be widened to four (4) travel lanes in the vicinity of SH-259A (North) to accommodate existing and future background traffic volumes.

Intersection Capacity and Level of Service

The Level of Service (LOS) of an intersection is a qualitative measure of capacity and operating conditions that is directly related to vehicle delay. For unsignalized intersections, the levels of service, as shown in **Table 9,** are defined by average control delay in seconds per vehicle. Additional performance measures such as volume to capacity (v/c) ratios and queue lengths also provide an indication of operations. For example, at two-way stop-controlled intersections, main street traffic volumes may impose longer average delays for a small number of side-street vehicles, thus creating vehicle delays which correspond to a poor level of service. Motorists and agencies will typically accept longer delays (LOS E or F) if gaps in the traffic stream are anticipated within a reasonable timeframe and the side street traffic volumes do not warrant a traffic signal. As a general guide, gap acceptance thresholds for the longer delay values can be defined when the v/c ratios are under 0.80, which corresponds to 80 percent capacity usage for that movement.

Table 9: Level of Service Criteria for Unsignalized Intersections

Level-of-Service (LOS)	**Average Control Delay (seconds/vehicle)**	**Description**
A	≤ 10.0	No delays at intersections with continuous flow of traffic. Uncongested operations: high frequency of long gaps available for all left and right turning traffic. No observable queues.
B	10.1 to 15.0	No delays at intersections with continuous flow of traffic. Uncongested operations: high frequency of long gaps available for all left and right turning traffic. No observable queues.
C	15.1 to 25.0	Moderate delays at intersections with satisfactory to good traffic flow. Light congestion; infrequent backups on critical approaches.
D	25.1 to 35.0	Increased probability of delays along every approach. Significant congestion on critical approaches, but intersection functional. No standing long lines formed.
E	35.1 to 50.0	Heavy traffic flow condition. Heavy delays probable. No available gaps for cross-street traffic or main street turning traffic. Limit of stable flow.
F	> 50.0	Unstable traffic flow. Heavy congestion. Traffic moves in forced flow condition. Average delays greater than one minute highly probable. Total breakdown.

SOURCE: *Highway Capacity Manual, Latest Edition*, Transportation Research Board

The LOS criteria for a signalized intersection are shown in **Table 10.** LOS is given a letter designation from A to F, with LOS A representing very short delays (less than 10 seconds of average control delay per vehicle) and LOS F representing very long delays (more than 80 seconds of average control delay per vehicle).

Table 10: Level of Service Criteria for Signalized Intersections

Level-of-Service (LOS)	Average Control Delay (seconds/vehicle)	Description
A	≤ 10.0	Very low vehicle delays, free flow, signal progression extremely favorable, most vehicles arrive during given signal phase.
B	10.1 to 20.0	Good signal progression, more vehicles stop and experience higher delays than for LOS A.
C	20.1 to 35.0	Stable flow, fair signal progression, significant number of vehicles stop at signals.
D	35.1 to 55.0	Congestion noticeable, longer delays and unfavorable signal progression, many vehicles stop at signals.
E	55.1 to 80.0	Limit of acceptable delay, unstable flow, poor signal progression, traffic near roadway capacity, frequent cycle failures.
F	> 80.0	Unacceptable delays, extremely unstable flow and congestion, traffic exceeds roadway capacity, stop-and-go conditions.

SOURCE: *Highway Capacity Manual, Latest Edition*, Transportation Research Board

The intersection capacity analyses were conducted using *Highway Capacity Manual* (HCM) methodologies in *Synchro 11*, a traffic analysis software package. For Existing traffic conditions, no improvements to the study area roadways and intersections were assumed, and the existing lane configurations and traffic control were used. For Build-Out (2023) Background traffic conditions, the intersection of US-259 and SH-259A (North) was analyzed with signalized control. For Build-Out (2023) Total traffic conditions, a traffic signal was also assumed at the proposed driveway along US-259, which is predicted to meet signal warrants.

Capacity analyses of the Friday AM and PM peak hour conditions, as well as the Saturday peak hour condition were conducted for the Existing (2021), Build-Out (2023) Background, and Build-Out (2023) Total traffic analysis scenarios using *Synchro 11.*

Analysis of Existing (2021) Traffic Conditions

The Friday AM and PM peak hours and the Saturday peak hour were analyzed for the existing study area intersections utilizing the Existing (2021) traffic volumes previously presented in Figure 5. **Table 11** presents the capacity analysis results for the Existing (2021) traffic conditions. The table details the LOS for each approach as well as the control delay, in seconds, for each approach.

Table 11: Capacity Analysis Summary – Existing (2021) Traffic Conditions

***Intersection*/Approach**	**Friday AM Peak Hour**		**Friday PM Peak Hour**		**Saturday Peak Hour**	
	LOS	**Delay (sec)**	**LOS**	**Delay (sec)**	**LOS**	**Delay (sec)**
(OWSC)						
US-259 & SH-259A (N)	*	3.7	*	11.4	*	13.2
SH-259A (N) WB	C	23.7	**F**	**80.7**	**F**	**62.7**
US-259 NB	*	0	*	0	*	0
US-259 SB Left	A	8.8	A	9.9	A	9.2

*LOS results are not calculated for OWSC intersections or free movements within a OWSC intersection.

The intersection of US-259 and SH-259A (North) is presently one-way stop-controlled (OWSC) with a stop sign on the westbound approach. The results indicate that the westbound approach is currently operating at **LOS F** during the Friday PM and Saturday peak hours. Therefore, the intersection is recommended for signalized control, consistent with ODOT's planned improvements for the area.

Analysis of Build-Out (2023) Background Conditions

The Friday AM and PM peak hours and the Saturday peak hour were analyzed for the existing study area intersections utilizing the Build-Out (2023) Background traffic volumes previously presented in Figure 9. **Table 12** presents the capacity analysis results for the Build-Out (2023) Background traffic conditions. The table details the LOS for each approach within the study area intersection as well as the control delay, in seconds, for each approach.

Table 12: Capacity Analysis Summary – Build-Out (2023) Background Conditions

***Intersection*/Approach**	**Friday AM Peak Hour**		**Friday PM Peak Hour**		**Saturday Peak Hour**	
	LOS	**Delay (sec)**	**LOS**	**Delay (sec)**	**LOS**	**Delay (sec)**
(Signalized)						
US-259 & SH-259A (N)	C	20.7	C	33.2	C	28.9
SH-259A (N) WB	C	30.6	D	52.2	D	47.4
US-259 NB	C	26.2	D	45.0	D	35.8
US-259 SB	B	14.1	B	10.5	B	15.3

The intersection of US-259 and SH-259A (North) is planned for signalized control and was analyzed as such. No additional roadway improvements were considered for the Build-Out (2023) Background analysis. The results indicate that the study intersection and individual turning movements are generally expected to operate at LOS D or better during all peak hours.

Analysis of Build-Out (2023) Total Traffic Conditions

The Friday AM and PM peak hours and the Saturday peak hour were analyzed under total build-out conditions, with the addition of the three (3) proposed site access driveways and site-generated traffic for the 2023 analysis year for Scenario 1 and Scenario 2 as described below:

1. *Scenario 1* - The proposed resort on a typical Friday during the AM and PM peak, and on a typical Saturday peak ***without*** an event taking place.
2. *Scenario 2* - The proposed resort on a typical Friday during the PM peak, and on a typical Saturday peak ***with*** a planned sold-out event.

The Build-Out (2023) Total traffic conditions analysis was conducted utilizing the volumes previously presented in Figure 10 for Scenario 1 and Figure 11 for Scenario 2. **Table 13** presents the capacity analysis results for the Build-Out (2023) Total traffic conditions for *Scenario 1 (No Event)*. The table details the LOS for each approach within the study area intersections as well as the control delay, in seconds, for each approach.

The intersection of US-259 and SH-259A (North) was analyzed as signalized, consistent with ODOT's planned improvements for the area. The proposed Driveway 3 on US-259 is predicted to meet signal warrants as described in the Appendix of this report. Therefore, the proposed intersection of US-259 and Driveway 3 was analyzed as a signalized intersection and was coordinated with the planned signal at US-259 and SH-259A (North).

Table 13: Capacity Analysis Summary – Build-Out (2023) Total Conditions – Scenario 1 (No Event)

***Intersection*/Approach**	**Friday AM Peak Hour**		**Friday PM Peak Hour**		**Saturday Peak Hour**	
	LOS	**Delay (sec)**	**LOS**	**Delay (sec)**	**LOS**	**Delay (sec)**
(Signalized)						
US-259 & SH-259A (N)	C	22.8	C	33.0	**E**	**58.5**
SH-259A (N) WB	**F**	**127.4**	**F**	**166.9**	**F**	**202.3**
US-259 NB	A	5.1	A	10.0	C	27.3
US-259 SB	A	7.5	A	8.2	B	18.0
(OWSC Driveway)						
DWY 1 & SH-259A (N)	*	1.8	*	2.5	*	3.3
SH-259A (N) EB	*	0	*	0	*	0
SH-259A (N) WB Left	A	7.8	A	7.8	A	8.3
DWY 1 NB	B	11.4	B	12.4	C	17.9
(OWSC Driveway)						
DWY 2 & SH-259A (N)	*	*	*	*	*	*
SH-259A (N) EB	*	*	*	*	*	*
SH-259A (N) WB	*	*	*	*	*	*
(Signalized)						
DWY 3 & US-259	B	17.0	C	24.3	C	20.3
DWY 3 WB	C	34.9	**E**	**59.1**	D	41.2
US-259 NB	C	33.5	D	37.4	D	36.8
US-259 SB	A	1.7	A	2.5	A	2.4

*LOS results are not calculated for OWSC intersections or free movements within a OWSC intersection.

The results indicate that most study intersections and individual turning movements are predicted to operate at LOS D or better during most peak hours under Scenario 1. However, the intersection of US-259 and SH-259A (North) is predicted to operate at LOS E during the Saturday peak hour, and the westbound approach is predicted to operate at **LOS F** during all peak hours. The westbound approach at the intersection of US-259 and Driveway 3 is predicted to operate at LOS E during the Friday PM peak hour.

Table 14 presents the capacity analysis results for the Build-Out (2023) Total traffic conditions for *Scenario 2 (Sold-Out Event)*. The results shown in Table 13 incorporate signal timing adjustments for the peak hour sold-out event. An event plan should be incorporated into the signal controllers for the specific event peak periods upon Build-Out.

Table 14: Capacity Analysis Summary – Build-Out (2023) Total Conditions – Scenario 2 (Sold-Out Event)

Intersection/Approach	Friday PM Peak Hour		Saturday Peak Hour	
	LOS	Delay (sec)	LOS	Delay (sec)
(Signalized)				
US-259 & SH-259A (N)	D	36.6	D	50.2
SH-259A (N) WB	**F**	**210.1**	**F**	**215.6**
US-259 NB	B	11.9	B	14.0
US-259 SB	A	8.1	B	14.2
(OWSC Driveway)				
DWY 1 & SH-259A (N)	*	2.7	*	3.6
SH-259A (N) EB	*	0	*	0
SH-259A (N) WB Left	A	8.2	A	8.8
DWY 1 NB	C	15.2	C	24.2
(OWSC Driveway)				
DWY 2 & SH-259A (N)	*	*	*	*
SH-259A (N) EB	*	*	*	*
SH-259A (N) WB	*	*	*	*
(Signalized)				
DWY 3 & US-259	C	24.2	C	23.2
DWY 3 WB	**E**	**59.1**	D	52.5
US-259 NB	D	37.1	D	43.3
US-259 SB	A	5.5	A	3.2

*LOS results are not calculated for OWSC intersections or free movements within a OWSC intersection.

The results indicate that most intersections and individual turning movements are predicted to operate at LOS D or better during both peak hours. However, the westbound approach at the intersection of US-259 and SH-259A (North) is predicted to operate at LOS F during both peak hours with an event. The westbound approach at the intersection of US-259 and Driveway 3 is predicted to operate at LOS E during the Friday PM peak hour with an event.

To improve levels of service for the Build-Out (2023) Total traffic conditions, the intersections along US-259 were further analyzed with recommended mitigation measures. It is recommended that US-259 be widened to four (4) travel lanes in the vicinity of SH-259A (North). Another westbound approach lane is recommended on SH-259A (North) to provide a separate left-turn lane and channelized right-turn lane with a 'YIELD' sign. It is also recommended that the westbound right-turn lane along Driveway #3 be constructed as a channelized right with a 'YIELD' sign. **Table 15** and **Table 16** summarize the level of service outputs under Scenario 1 and Scenario 2.

Table 15: Capacity Analysis Summary – Build-Out (2023) Total Conditions – Scenario 1 (No Event) with Mitigation

Intersection/Approach	Friday AM Peak Hour		Friday PM Peak Hour		Saturday Peak Hour	
	LOS	Delay (sec)	LOS	Delay (sec)	LOS	Delay (sec)
(Signalized)						
US-259 & SH-259A (N)	C	31.1	C	31.2	C	26.9
SH-259A (N) WB	**E**	**69.7**	**E**	**64.8**	D	54.2
US-259 NB	**E**	**63.0**	D	47.8	D	49.7
US-259 SB	A	4.3	A	6.3	A	7.0
(Signalized)						
DWY 3 & US-259	C	25.6	C	26.4	C	22.0
DWY 3 WB	**E**	**75.2**	**E**	**69.6**	**E**	**56.1**
US-259 NB	D	53.0	D	44.0	D	42.9
US-259 SB	A	0.2	A	0.4	A	0.3

Table 16: Capacity Analysis Summary – Build-Out (2023) Total Conditions – Scenario 2 (Sold-Out Event) with Mitigation

Intersection/Approach	Friday PM Peak Hour		Saturday Peak Hour	
	LOS	Delay (sec)	LOS	Delay (sec)
(Signalized)				
US-259 & SH-259A (N)	C	26.3	C	27.3
SH-259A (N) WB	**E**	**65.3**	**E**	**64.9**
US-259 NB	D	41.0	D	53.3
US-259 SB	A	7.3	A	7.5
(Signalized)				
DWY 3 & US-259	C	25.2	C	24.9
DWY 3 WB	**E**	**72.3**	**E**	**68.6**
US-259 NB	D	43.6	D	51.6
US-259 SB	A	0.7	A	0.2

The results in Table 15 and Table 16 indicate that both signalized intersections are predicted to operate at LOS C during all scenarios and peak hours with the addition of the recommended mitigation measures. The westbound approaches are predicted to operate at LOS E during most scenarios, which is the result of the coordinated signal timing set up with preference to the northbound and southbound phases. These westbound approaches are not anticipated to experience significant queuing, as discussed later in this report.

The intersections were initially analyzed without widening US-259 to four (4) lanes; however, significant queuing was predicted, especially along US-259 in the northbound and southbound directions between the traffic signals. Therefore, the widening of US-259 is needed to reduce queue lengths between the traffic signals.

Right-Turn Deceleration Lane Analysis

Right-turn deceleration lanes do not presently exist within the study area. Area drivers have an expectation of right-turn movements occurring from the through lanes.

For highways, ODOT design guidelines indicate that an auxiliary right-turn deceleration lane should be considered for any driveway with a right-turn volume greater than 40 vehicles per hour (vph). **Table 17** summarizes the predicted right-turn volumes under Build-Out (2023) Total traffic conditions.

Table 17: Right-Turn Deceleration Lane Analysis

Intersection	**Approach**	**Speed Limit (mph)**	**Peak Hour & Scenario**	**Right-Turn Volume**	**Threshold (vph)**	**Exceed Threshold?**
SH-259A (N) & DWY 1	EB	55	Friday AM	31	40	NO
			Friday PM	43		**YES**
			Saturday Peak	74		**YES**
			Friday PM (Scenario 2 – Event)	121		**YES**
			Saturday Peak (Scenario 2 – Event)	149		**YES**
SH-259A (N) & DWY 2	EB	55	Friday AM	15	40	NO
			Friday PM	20		NO
			Saturday Peak	35		NO
			Friday PM (Scenario 2 – Event)	57		**YES**
			Saturday Peak (Scenario 2 – Event)	70		**YES**
US-259 & DWY 3	NB	55	Friday AM	64	40	**YES**
			Friday PM	88		**YES**
			Saturday Peak	152		**YES**
			Friday PM (Scenario 2 – Event)	249		**YES**
			Saturday Peak (Scenario 2 – Event)	307		**YES**

Based on the results in Table 17, all of the proposed driveways are anticipated to meet the guidelines for consideration of a right-turn deceleration lane.

At a signalized intersection, design guidelines warrant an exclusive right-turn lane when right-turning volumes exceed 300 vph. For the signalized intersection of US-259 and SH-259A (N), right-turning vehicles on the northbound US-259 approach are expected to increase, yet not exceed this threshold during the peak hour periods analyzed under the Build-Out (2023) Total traffic conditions. However, the westbound right-turn movement exceeds 300 vph during a typical Saturday peak hour without a planned sold-out event. A westbound right-turn lane is recommended based on the intersection operational analysis. Consideration of a northbound right-turn lane is recommended as a safety and capacity mitigation measure.

The proposed intersection of US-259 and Driveway 3 exceeds 300 vph on the northbound right-turn movement during a typical Saturday peak hour with a sold-out event. The site plan currently denotes a northbound channelized right-turn lane which was included in the capacity analysis. A northbound right-turn lane is recommended as a safety and capacity mitigation measure.

Left-Turn Deceleration Lane Analysis

Left-turn lanes should be considered for unsignalized intersections that meet advancing and opposing vehicle volumes as defined in the ODOT design guidelines. This process was utilized to analyze all proposed site driveways during the expected peak hours for Build-Out (2023) Total traffic conditions. The westbound left-turn movements at the proposed intersections of SH-259A (North) at Driveway 1 and Driveway 2 are predicted to warrant left-turn lanes during the Friday PM peak hour with an event and Saturday peak hour operations with and without an event. Therefore, exclusive westbound left-turn lanes with adequate storage are recommended for consideration along the westbound approach of SH-259A (North) at the proposed intersections with Driveway 1 and Driveway 2.

At a signalized intersection, design guidelines warrant an exclusive left-turn lane when left-turning vehicle volumes exceed 100 vph. The proposed signalized intersection of US-259 and Driveway 3 exceeds left-turn lane criteria on the southbound approach during Friday PM and typical Saturday peak hour operations with a sold-out event. An exclusive southbound left-turn lane is recommended at Driveway 3 as proposed on the current site plan. The westbound left-turn movement at the intersection does not exceed 100 vph; however, a westbound left-turn lane is proposed on the current site plan.

Site Considerations & Driveway Spacing

From review of the site layout plan provided in Figure 2, no configuration issues have been noted. Queues for the site driveways are anticipated to be easily managed within the site. The ODOT driveway spacing guidelines identifies a minimum driveway spacing of 105-feet for commercial driveways. The spacing between all site driveways are proposed to exceed ODOT's minimum driveway spacing guidelines.

Sight Distance

Based on field measured sight distances, adequate intersection sight distance appears to be available for motorists at the proposed site driveways along US-259 and SH-259A (North). For the proposed site driveways along SH-259A (North), the available sight distance is limited due to the horizontal curvature of the roadway and the presence of trees along the south side of SH-259A (North). The results of the sight distance analysis are provided in **Table 18**. It is recommended that the trees along the south side of SH-259A (North) be trimmed or removed during construction of the proposed development to ensure adequate sight distance at the site driveways.

Table 18: Sight Distance Analysis

Major Roadway	US-259		SH-259A (North)	
Posted Speed Limit	55 MPH		55 MPH	
Minor Roadway	DWY 3		DWY 1	DWY 2
Design Vehicle	Single-Unit Truck	Passenger Car	Passenger Car	Passenger Car
Required Intersection Sight Distance				
Exiting (to the Left)	770 ft	610 ft	610 ft	610 ft
Exiting (to the Right)	690 ft	530 ft	530 ft	530 ft
Available Sight Distance				
Available Sight Distance - Left	>1,800 ft	>1,800 ft	1,400 ft *	650 ft
Available Sight Distance - Right	>1,300 ft	>1,300 ft	>1,000 ft	650 ft
Sight Distance Available > Required				
To the Left	**YES**	**YES**	**YES**	**YES**
To the Right	**YES**	**YES**	**YES**	**YES**

* Distance from DWY to the intersection of US-259 at SH-259A (North).

Queuing Analysis

In order to ensure that installation of traffic signals and the recommended mitigation measures do not adversely impact traffic flow along US-259, *SimTraffic* was used to evaluate the 95th percentile queue lengths under Build-Out (2023) Total traffic conditions. The 95th percentile queue represents a queue length that has only a 5-percent probability of being exceeded during the analysis hours. A summary of the 95th percentile queue lengths reported is provided in **Table 19** and **Table 20** for Scenario 1 and Scenario 2.

Table 19: Queuing Analysis - Scenario 1 (No Event) with Mitigation

Intersection/Approach	Available Storage	Friday AM Peak Hour	Friday PM Peak Hour	Saturday Peak Hour
	Feet	Queue (feet)	Queue (feet)	Queue (feet)
(Signalized)				
US-259 & SH-259A (N)				
SH-259A (N) WB Left	650	77	126	138
SH-259A (N) WB Right	150*	16	51	62
US-259 NB Through	>1,000	118	159	133
US-259 NB Through/Right	>1,000	91	138	119
US-259 SB Left	175	103	116	138
US-259 SB Through	>1,000	95	120	188
(Signalized)				
DWY 3 & US-259				
DWY 3 WB Left	450	58	77	93
DWY 3 WB Right	450	0	0	0
US-259 NB Through	>1,000	76	119	100
US-259 SB Left	250	41	53	68
US-259 SB Through	>1,000	67	82	88

*A minimum storage length of 150 feet is recommended.

Table 20: Queuing Analysis - Scenario 2 (Sold Out Event) with Mitigation

Intersection/Approach	Available Storage	Friday PM Peak Hour	Saturday Peak Hour
	Feet	Queue (feet)	Queue (feet)
(Signalized)			
US-259 & SH-259A (N)			
SH-259A (N) WB Left	650	122	136
SH-259A (N) WB Right	150*	61	67
US-259 NB Through	>1,000	189	155
US-259 NB Through/Right	>1,000	179	142
US-259 SB Left	175	134	143
US-259 SB Through	>1,000	203	323
(Signalized)			
DWY 3 & US-259			
DWY 3 WB Left	450	68	80
DWY 3 WB Right	450	0	0
US-259 NB Through	>1,000	139	110
US-259 SB Left	250	103	102
US-259 SB Through	>1,000	81	86

*A minimum storage length of 150 feet is recommended.

The queuing analysis indicates that significant queuing is not predicted on any approach of the signalized intersections along US-259 after installation of coordinated traffic signals at US-259 / US-259A (North) and US-259 / Driveway 3 and the additional recommended mitigation measures including widening of US-259 to a four lane roadway in the vicinity of SH-259A (North), a designated westbound right-turn lane and consideration of a northbound right-turn lane at the intersection of US-259 and US-259A (North), and dedicated left and right-turn lanes in all directions at the intersection of US-259 and Driveway 3. It should be noted that queues predicted along US-259 between the traffic signals are less than 200 feet, which is the equivalent of 8 vehicles per lane.

MITIGATION MEASURES

Based on the analysis conducted as part of this study, it is recommended that US-259 be widened to provide four (4) travel lanes in the vicinity of SH-259A (North).

A traffic signal is predicted to be warranted at the intersection of US-259 and Driveway 3 with build-out of the proposed development. Traffic demands should be monitored, and installation of a traffic signal is recommended at this location as development traffic is realized. Additionally, advanced intersection warning signs should be installed prior to the beginning of signal operations to alert motorists of the new signal if it is installed prior to the planned traffic signals at US-259 and SH-259A (North). Exclusive right-turn and left-turn lanes are recommended for all approaches and are recommended to coincide with signal installation. Intersection lighting is also recommended.

Exclusive right-turn and left-turn lanes are also recommended for the eastbound and westbound approaches on SH-259A (North) at Driveway 1 and Driveway 2.

ODOT plans to install a traffic signal at the intersection of US-259 and SH-259A (North). A westbound right-turn lane is recommended based on the intersection operational analysis. Consideration of a northbound right-turn lane is recommended as a safety and capacity mitigation measure.

Based on field measured sight distances, adequate intersection sight distance appears to be available for motorists at the proposed site driveways along US-259 and SH-259A (North). It is recommended that the trees along the south side of SH-259A (North) be trimmed or removed during construction of the proposed development to ensure adequate sight distance at the site driveways.

Figure 12 summarizes the recommended mitigation measures.

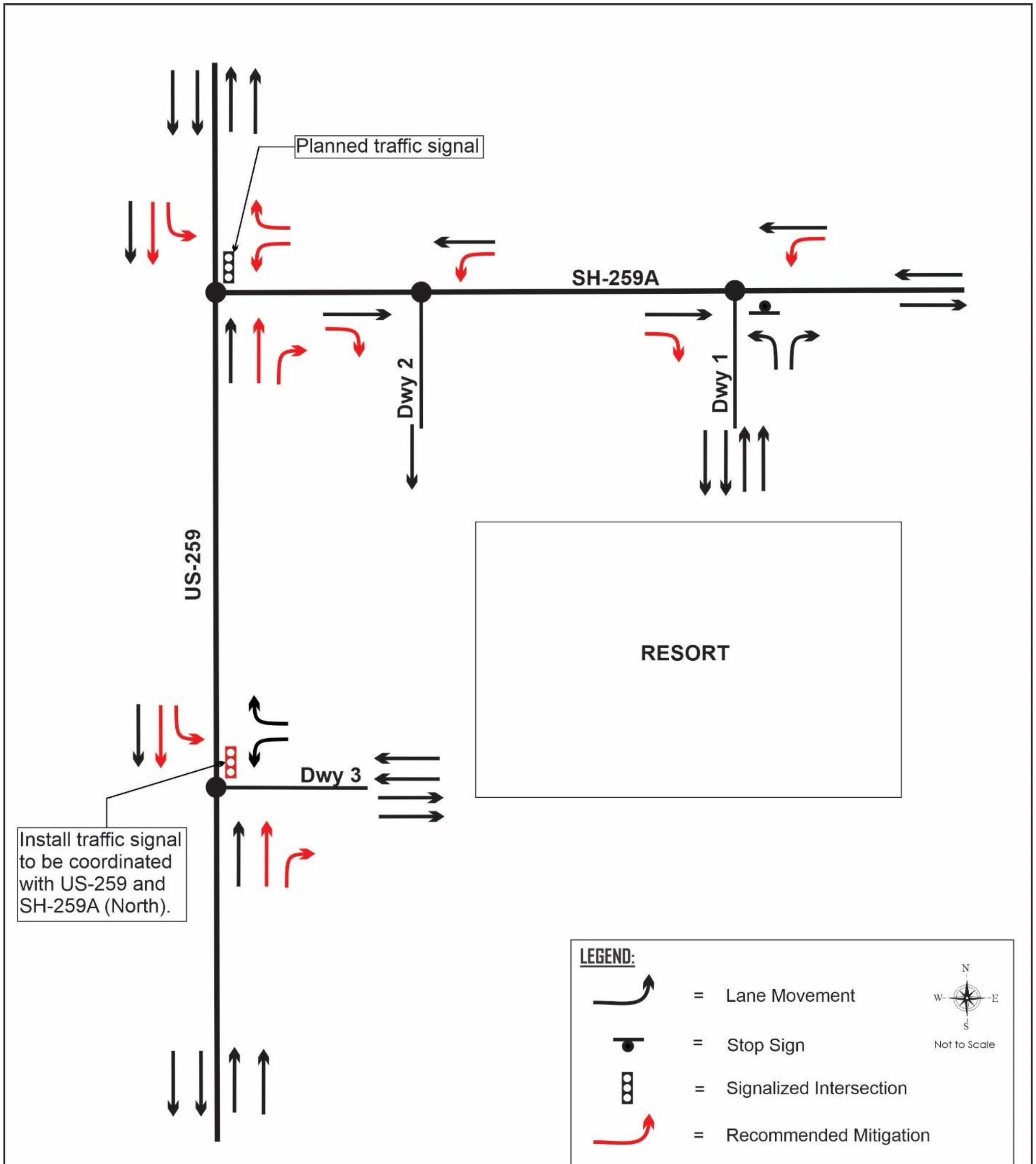


Figure 12

525 Central Park Drive,
Suite 107
Oklahoma City, OK 73105
(405) 384-5127

LEE ENGINEERING

RECOMMENDED MITIGATION MEASURES

CONCLUSIONS

The following conclusions are provided per the analyses conducted as part of this study and based on the information and assumptions presented:

- Based on the development's land use, the proposed development is expected to generate 6,824 total new trips within the study area on a given Friday. For an average Saturday, 6,844 total new trips are anticipated. These new trips will be distributed among the existing roadway network. Conservatively, trip generation estimates in this study were not reduced to account for pass-by trips, though pass-by trips will likely be experienced by the proposed convenience store and fueling positions.

- Based on the roadway link capacity analysis, US-259 north and south of the SH-259A (North) intersection operates at LOS E currently with a 2-lane cross-section. US-259 would operate at LOS C or better under Build-Out (2023) Total Traffic Conditions with a 4-lane cross-section. SH-259A is predicted to operate at LOS D or better under Build-Out (2023) Total Traffic Conditions with the existing 2-lane cross-section.

- Capacity analysis results indicate that the westbound approach of US-259 and SH-259A (North) currently operates at LOS F during the Friday PM and Saturday peak hours.

- Assuming a 10% increase in traffic volumes (background growth estimate), the intersection of US-259 and SH-259A (North), with its existing configuration and updated signalized control, is predicted to operate at LOS C during peak hour conditions in 2023 **without** the proposed site-generated traffic. The westbound approach is predicted to operate at LOS E during the Friday PM peak hour.

- Under full site build-out conditions, capacity analysis results indicate that the traffic impacts associated with the proposed development will create the need for traffic mitigation measures.

- Based on expected build-out year traffic volumes, a traffic signal is predicted to be warranted at the intersection of US-259 and Driveway 3.

- Exclusive left and right-turn lanes are predicted to be warranted at the intersection of US-259 and Driveway 3.

- Exclusive left and right-turn lanes are predicted to be warranted on the eastbound and westbound approaches along SH-259A (North) at Driveway 1 and Driveway 2.

- An exclusive westbound right-turn lane is warranted at the US-259 and SH-259A (North) intersection.

- Based on the conditions present during the field visit, intersection sight distance availability at the proposed site driveway locations on US-259 and SH-259A (North) is adequate

APPENDIX

TRAFFIC DATA

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/30/2021
Page No: 1

Turning Movement Data

Start Time	US 259 Southbound					SH 259 A Westbound					US 259 Northbound					
	Thru	Left	U-Turn	Peds	App. Total	Right	Left	U-Turn	Peds	App. Total	Right	Thru	U-Turn	Peds	App. Total	Int. Total
12:00 AM	12	1	0	0	13	1	1	0	0	2	1	8	0	0	9	24
12:15 AM	7	2	0	0	9	2	0	0	0	2	1	9	0	0	10	21
12:30 AM	6	1	0	0	7	0	0	0	0	0	2	9	0	0	11	18
12:45 AM	3	0	0	0	3	1	0	0	0	1	0	4	0	0	4	8
Hourly Total	28	4	0	0	32	4	1	0	0	5	4	30	0	0	34	71
1:00 AM	4	2	0	0	6	1	1	0	0	2	0	5	0	0	5	13
1:15 AM	3	1	0	0	4	0	0	0	0	0	0	4	0	0	4	8
1:30 AM	1	0	0	0	1	0	1	0	0	1	0	8	0	0	8	10
1:45 AM	5	0	0	0	5	0	2	0	0	2	0	3	0	0	3	10
Hourly Total	13	3	0	0	16	1	4	0	0	5	0	20	0	0	20	41
2:00 AM	2	1	0	0	3	0	0	0	0	0	2	2	0	0	4	7
2:15 AM	5	0	0	0	5	0	0	0	0	0	1	3	0	0	4	9
2:30 AM	5	0	0	0	5	0	0	0	0	0	0	5	0	0	5	10
2:45 AM	4	0	0	0	4	0	0	0	0	0	0	7	0	0	7	11
Hourly Total	16	1	0	0	17	0	0	0	0	0	3	17	0	0	20	37
3:00 AM	2	1	0	0	3	0	1	0	0	1	0	1	0	0	1	5
3:15 AM	1	0	0	0	1	0	0	0	0	0	0	3	0	0	3	4
3:30 AM	6	1	0	0	7	0	0	0	0	0	0	2	0	0	2	9
3:45 AM	7	0	0	0	7	0	1	0	0	1	0	7	0	0	7	15
Hourly Total	16	2	0	0	18	0	2	0	0	2	0	13	0	0	13	33
4:00 AM	2	0	0	0	2	0	0	0	0	0	0	4	0	0	4	6
4:15 AM	6	0	0	0	6	0	0	0	0	0	0	7	0	0	7	13
4:30 AM	9	0	0	0	9	0	0	0	0	0	0	11	0	0	11	20
4:45 AM	6	0	0	0	6	0	0	0	0	0	0	8	0	0	8	14
Hourly Total	23	0	0	0	23	0	0	0	0	0	0	30	0	0	30	53
5:00 AM	6	1	0	0	7	0	0	0	0	0	0	10	0	0	10	17
5:15 AM	10	0	0	0	10	0	0	0	0	0	1	13	0	0	14	24
5:30 AM	19	4	0	0	23	1	0	0	0	1	2	12	0	0	14	38
5:45 AM	15	2	0	0	17	1	1	0	0	2	1	40	0	0	41	60
Hourly Total	50	7	0	0	57	2	1	0	0	3	4	75	0	0	79	139
6:00 AM	15	3	0	0	18	2	1	0	0	3	11	31	0	0	42	63
6:15 AM	15	4	0	0	19	2	2	0	0	4	3	34	0	0	37	60
6:30 AM	28	6	0	0	34	0	0	0	0	0	1	50	0	0	51	85
6:45 AM	19	4	0	0	23	0	1	0	0	1	4	59	0	0	63	87
Hourly Total	77	17	0	0	94	4	4	0	0	8	19	174	0	0	193	295
7:00 AM	29	2	0	0	31	0	1	0	0	1	3	60	0	0	63	95

7:15 AM	42	3	0	0	45	3	1	0	0	4	5	68	0	0	73	122
7:30 AM	29	3	0	0	32	1	6	0	0	7	4	49	0	0	53	92
7:45 AM	44	2	0	0	46	8	4	0	0	12	14	59	0	0	73	131
Hourly Total	144	10	0	0	154	12	12	0	0	24	26	236	0	0	262	440
8:00 AM	51	7	0	0	58	6	2	0	0	8	7	69	0	0	76	142
8:15 AM	47	14	0	0	61	9	5	0	0	14	7	84	0	0	91	166
8:30 AM	48	6	0	0	54	6	9	0	0	15	10	74	0	0	84	153
8:45 AM	73	17	0	0	90	9	6	0	0	15	8	79	0	0	87	192
Hourly Total	219	44	0	0	263	30	22	0	0	52	32	306	0	0	338	653
9:00 AM	79	17	0	0	96	9	14	0	0	23	9	71	0	0	80	199
9:15 AM	103	13	0	0	116	17	8	0	0	25	5	78	0	0	83	224
9:30 AM	82	27	0	0	109	11	6	0	0	17	5	92	0	0	97	223
9:45 AM	99	19	2	0	120	14	8	0	0	22	16	99	0	0	115	257
Hourly Total	363	76	2	0	441	51	36	0	0	87	35	340	0	0	375	903
10:00 AM	94	30	0	0	124	27	15	0	0	42	10	127	0	0	137	303
10:15 AM	130	31	0	0	161	24	8	0	0	32	10	121	0	0	131	324
10:30 AM	109	28	0	0	137	28	9	0	0	37	15	107	0	0	122	296
10:45 AM	119	28	0	0	147	27	14	0	0	41	15	124	0	0	139	327
Hourly Total	452	117	0	0	569	106	46	0	0	152	50	479	0	0	529	1250
11:00 AM	135	27	0	0	162	29	10	0	0	39	11	101	0	0	112	313
11:15 AM	129	27	0	0	156	31	10	0	0	41	9	104	0	0	113	310
11:30 AM	128	39	0	0	167	26	8	0	0	34	12	111	0	0	123	324
11:45 AM	97	19	0	0	116	21	6	0	0	27	14	97	0	0	111	254
Hourly Total	489	112	0	0	601	107	34	0	0	141	46	413	0	0	459	1201
12:00 PM	91	25	0	0	116	28	6	0	0	34	10	116	0	0	126	276
12:15 PM	124	30	0	0	154	53	12	0	0	65	11	98	0	0	109	328
12:30 PM	111	33	1	0	145	20	8	0	0	28	18	130	0	0	148	321
12:45 PM	84	21	1	0	106	51	9	0	0	60	11	105	0	0	116	282
Hourly Total	410	109	2	0	521	152	35	0	0	187	50	449	0	0	499	1207
1:00 PM	100	26	1	0	127	33	11	0	0	44	14	104	0	0	118	289
1:15 PM	123	25	0	0	148	27	16	0	0	43	16	105	0	0	121	312
1:30 PM	87	31	0	0	118	29	14	0	0	43	8	108	0	0	116	277
1:45 PM	115	28	1	0	144	15	12	0	0	27	6	140	0	0	146	317
Hourly Total	425	110	2	0	537	104	53	0	0	157	44	457	0	0	501	1195
2:00 PM	111	28	0	0	139	26	12	0	0	38	15	98	0	0	113	290
2:15 PM	111	24	0	0	135	39	13	0	0	52	10	120	0	0	130	317
2:30 PM	138	31	0	0	169	40	17	0	0	57	9	131	0	0	140	366
2:45 PM	140	34	0	0	174	36	12	0	0	48	17	152	0	0	169	391
Hourly Total	500	117	0	0	617	141	54	0	0	195	51	501	0	0	552	1364
3:00 PM	122	25	0	0	147	32	8	0	0	40	16	148	0	0	164	351
3:15 PM	117	34	0	0	151	29	19	0	0	48	23	136	0	0	159	358
3:30 PM	103	22	0	0	125	37	17	0	0	54	15	171	0	0	186	365
3:45 PM	109	21	0	0	130	26	14	0	0	40	18	174	0	0	192	362
Hourly Total	451	102	0	0	553	124	58	0	0	182	72	629	0	0	701	1436
4:00 PM	135	31	0	0	166	33	14	0	0	47	8	176	0	0	184	397
4:15 PM	120	18	0	0	138	37	24	0	0	61	17	166	0	0	183	382
4:30 PM	102	18	0	0	120	30	10	0	0	40	20	162	0	0	182	342
4:45 PM	108	24	0	0	132	33	7	1	0	41	12	164	0	0	176	349
Hourly Total	465	91	0	0	556	133	55	1	0	189	57	668	0	0	725	1470
5:00 PM	106	18	0	0	124	37	7	0	0	44	10	138	0	0	148	316
5:15 PM	96	22	0	0	118	26	14	0	0	40	18	161	0	0	179	337
5:30 PM	95	18	1	0	114	24	7	0	0	31	23	150	0	0	173	318

5:45 PM	90	29	1	0	120	15	16	0	0	31	5	156	0	0	161	312
Hourly Total	387	87	2	0	476	102	44	0	0	146	56	605	0	0	661	1283
6:00 PM	98	15	0	0	113	19	9	0	0	28	15	144	0	0	159	300
6:15 PM	94	22	0	0	116	20	7	0	0	27	17	147	0	0	164	307
6:30 PM	99	10	0	0	109	21	7	0	0	28	9	118	0	0	127	264
6:45 PM	88	18	0	0	106	26	15	0	0	41	11	155	0	0	166	313
Hourly Total	379	65	0	0	444	86	38	0	0	124	52	564	0	0	616	1184
7:00 PM	97	14	0	0	111	24	2	0	0	26	12	105	0	0	117	254
7:15 PM	96	20	1	0	117	24	5	0	0	29	9	104	0	0	113	259
7:30 PM	87	13	0	0	100	14	13	0	0	27	9	111	0	0	120	247
7:45 PM	83	7	0	0	90	18	5	0	0	23	5	91	0	0	96	209
Hourly Total	363	54	1	0	418	80	25	0	0	105	35	411	0	0	446	969
8:00 PM	85	11	1	0	97	13	11	0	0	24	12	99	0	0	111	232
8:15 PM	85	13	0	0	98	15	12	0	0	27	10	85	0	0	95	220
8:30 PM	54	12	0	0	66	8	10	0	0	18	10	74	0	0	84	168
8:45 PM	45	13	0	0	58	19	7	0	0	26	7	73	0	0	80	164
Hourly Total	269	49	1	0	319	55	40	0	0	95	39	331	0	0	370	784
9:00 PM	67	9	0	0	76	10	6	0	0	16	5	69	0	0	74	166
9:15 PM	60	3	0	0	63	14	3	0	0	17	7	72	0	0	79	159
9:30 PM	36	3	0	0	39	2	2	0	0	4	7	50	0	0	57	100
9:45 PM	48	7	0	0	55	2	1	0	0	3	7	57	0	0	64	122
Hourly Total	211	22	0	0	233	28	12	0	0	40	26	248	0	0	274	547
10:00 PM	44	12	0	0	56	8	4	0	0	12	4	50	0	0	54	122
10:15 PM	40	3	0	0	43	3	1	0	0	4	4	36	0	0	40	87
10:30 PM	34	4	0	0	38	3	0	0	0	3	5	47	0	0	52	93
10:45 PM	37	3	0	0	40	2	3	0	0	5	7	40	0	0	47	92
Hourly Total	155	22	0	0	177	16	8	0	0	24	20	173	0	0	193	394
11:00 PM	23	1	0	0	24	5	0	0	0	5	1	28	0	0	29	58
11:15 PM	23	3	0	0	26	2	0	0	0	2	3	32	0	0	35	63
11:30 PM	22	5	0	0	27	2	0	0	0	2	0	16	0	0	16	45
11:45 PM	9	0	0	0	9	2	0	0	0	2	0	22	0	0	22	33
Hourly Total	77	9	0	0	86	11	0	0	0	11	4	98	0	0	102	199
Grand Total	5982	1230	10	0	7222	1349	584	1	0	1934	725	7267	0	0	7992	17148
Approach %	82.8	17.0	0.1	-	-	69.8	30.2	0.1	-	-	9.1	90.9	0.0	-	-	-
Total %	34.9	7.2	0.1	-	42.1	7.9	3.4	0.0	-	11.3	4.2	42.4	0.0	-	46.6	-
Lights	5679	1221	10	-	6910	1342	578	1	-	1921	715	6925	0	-	7640	16471
% Lights	94.9	99.3	100.0	-	95.7	99.5	99.0	100.0	-	99.3	98.6	95.3	-	-	95.6	96.1
Buses	2	0	0	-	2	1	0	0	-	1	1	3	0	-	4	7
% Buses	0.0	0.0	0.0	-	0.0	0.1	0.0	0.0	-	0.1	0.1	0.0	-	-	0.1	0.0
Single-Unit Trucks	113	7	0	-	120	5	5	0	-	10	7	149	0	-	156	286
% Single-Unit Trucks	1.9	0.6	0.0	-	1.7	0.4	0.9	0.0	-	0.5	1.0	2.1	-	-	2.0	1.7
Articulated Trucks	188	2	0	-	190	1	1	0	-	2	2	190	0	-	192	384
% Articulated Trucks	3.1	0.2	0.0	-	2.6	0.1	0.2	0.0	-	0.1	0.3	2.6	-	-	2.4	2.2
Pedestrians	-	-	-	0	-	-	-	-	0	-	-	-	-	0	-	-
% Pedestrians	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/30/2021

US 259 [N]

Out	In	Total
8277	6910	15187
4	2	6
154	120	274
191	190	381
0	0	0
8626	**7222**	**15848**

T	L	U	P
5679	1221	10	0
2	0	0	0
113	7	0	0
188	2	0	0
0	0	0	0
5982	**1230**	**10**	**0**

SH 259 A [E]

Out	In	Total
1937	1921	3858
1	1	2
14	10	24
4	2	6
0	0	0
1956	**1934**	**3890**

R	L	U	P
1342	578	1	0
1	0	0	0
5	5	0	0
1	1	0	0
0	0	0	0
1349	**584**	**1**	**0**

07/30/2021 12:00 AM
Ending At
07/31/2021 12:00 AM

Lights
Buses
Single-Unit Trucks
Articulated Trucks
Pedestrians

U	T	R	P
0	6925	715	0
0	3	1	0
0	149	7	0
0	190	2	0
0	0	0	0
0	**7267**	**725**	**0**

Out	In	Total
6257	7640	13897
2	4	6
118	156	274
189	192	381
0	0	0
6566	**7992**	**14558**

US 259 [S]

Turning Movement Data Plot

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/30/2021

Turning Movement Peak Hour Data (10:45 AM)

	US 259 Southbound					SH 259 A Westbound					US 259 Northbound					
Start Time	Thru	Left	U-Turn	Peds	App. Total	Right	Left	U-Turn	Peds	App. Total	Right	Thru	U-Turn	Peds	App. Total	Int. Total
10:45 AM	119	28	0	0	147	27	14	0	0	41	15	124	0	0	139	327
11:00 AM	135	27	0	0	162	29	10	0	0	39	11	101	0	0	112	313
11:15 AM	129	27	0	0	156	31	10	0	0	41	9	104	0	0	113	310
11:30 AM	128	39	0	0	167	26	8	0	0	34	12	111	0	0	123	324
Total	511	121	0	0	632	113	42	0	0	155	47	440	0	0	487	1274
Approach %	80.9	19.1	0.0	-	-	72.9	27.1	0.0	-	-	9.7	90.3	0.0	-	-	-
Total %	40.1	9.5	0.0	-	49.6	8.9	3.3	0.0	-	12.2	3.7	34.5	0.0	-	38.2	-
PHF	0.946	0.776	0.000	-	0.946	0.911	0.750	0.000	-	0.945	0.783	0.887	0.000	-	0.876	0.974
Lights	488	119	0	-	607	113	41	0	-	154	47	405	0	-	452	1213
% Lights	95.5	98.3	-	-	96.0	100.0	97.6	-	-	99.4	100.0	92.0	-	-	92.8	95.2
Buses	0	0	0	-	0	0	0	0	-	0	0	0	0	-	0	0
% Buses	0.0	0.0	-	-	0.0	0.0	0.0	-	-	0.0	0.0	0.0	-	-	0.0	0.0
Single-Unit Trucks	8	2	0	-	10	0	1	0	-	1	0	21	0	-	21	32
% Single-Unit Trucks	1.6	1.7	-	-	1.6	0.0	2.4	-	-	0.6	0.0	4.8	-	-	4.3	2.5
Articulated Trucks	15	0	0	-	15	0	0	0	-	0	0	14	0	-	14	29
% Articulated Trucks	2.9	0.0	-	-	2.4	0.0	0.0	-	-	0.0	0.0	3.2	-	-	2.9	2.3
Pedestrians	-	-	-	0	-	-	-	-	0	-	-	-	-	0	-	-
% Pedestrians	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/30/2021
Page No: 6

US 259 [N]

Out	In	Total
518	607	1125
0	0	0
21	10	31
14	15	29
0	0	0
553	**632**	**1185**

T	L	U	P
488	119	0	0
0	0	0	0
8	2	0	0
15	0	0	0
0	0	0	0
511	**121**	**0**	**0**

Peak Hour Data

07/30/2021 10:45 AM
Ending At
07/30/2021 11:45 AM

Lights
Buses
Single-Unit Trucks
Articulated Trucks
Pedestrians

SH 259 A [E]

Out	In	Total
166	154	320
0	0	0
2	1	3
0	0	0
0	0	0
168	**155**	**323**

R	L	U	P
113	41	0	0
0	0	0	0
0	1	0	0
0	0	0	0
0	0	0	0
113	**42**	**0**	**0**

U	T	R	P
0	405	47	0
0	0	0	0
0	21	0	0
0	14	0	0
0	0	0	0
0	**440**	**47**	**0**

Out	In	Total
529	452	981
0	0	0
9	21	30
15	14	29
0	0	0
553	**487**	**1040**

US 259 [S]

Turning Movement Peak Hour Data Plot (10:45 AM)

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/30/2021

Turning Movement Peak Hour Data (3:30 PM)

Start Time	US 259 Southbound					SH 259 A Westbound					US 259 Northbound					
	Thru	Left	U-Turn	Peds	App. Total	Right	Left	U-Turn	Peds	App. Total	Right	Thru	U-Turn	Peds	App. Total	Int. Total
3:30 PM	103	22	0	0	125	37	17	0	0	54	15	171	0	0	186	365
3:45 PM	109	21	0	0	130	26	14	0	0	40	18	174	0	0	192	362
4:00 PM	135	31	0	0	166	33	14	0	0	47	8	176	0	0	184	397
4:15 PM	120	18	0	0	138	37	24	0	0	61	17	166	0	0	183	382
Total	467	92	0	0	559	133	69	0	0	202	58	687	0	0	745	1506
Approach %	83.5	16.5	0.0	-	-	65.8	34.2	0.0	-	-	7.8	92.2	0.0	-	-	-
Total %	31.0	6.1	0.0	-	37.1	8.8	4.6	0.0	-	13.4	3.9	45.6	0.0	-	49.5	-
PHF	0.865	0.742	0.000	-	0.842	0.899	0.719	0.000	-	0.828	0.806	0.976	0.000	-	0.970	0.948
Lights	457	92	0	-	549	132	69	0	-	201	56	672	0	-	728	1478
% Lights	97.9	100.0	-	-	98.2	99.2	100.0	-	-	99.5	96.6	97.8	-	-	97.7	98.1
Buses	0	0	0	-	0	0	0	0	-	0	1	0	0	-	1	1
% Buses	0.0	0.0	-	-	0.0	0.0	0.0	-	-	0.0	1.7	0.0	-	-	0.1	0.1
Single-Unit Trucks	5	0	0	-	5	1	0	0	-	1	1	6	0	-	7	13
% Single-Unit Trucks	1.1	0.0	-	-	0.9	0.8	0.0	-	-	0.5	1.7	0.9	-	-	0.9	0.9
Articulated Trucks	5	0	0	-	5	0	0	0	-	0	0	9	0	-	9	14
% Articulated Trucks	1.1	0.0	-	-	0.9	0.0	0.0	-	-	0.0	0.0	1.3	-	-	1.2	0.9
Pedestrians	-	-	-	0	-	-	-	-	0	-	-	-	-	0	-	-
% Pedestrians	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/30/2021
Page No: 8

US 259 [N]

Out	In	Total
804	549	1353
0	0	0
7	5	12
9	5	14
0	0	0
820	**559**	**1379**

T	L	U	P
457	92	0	0
0	0	0	0
5	0	0	0
5	0	0	0
0	0	0	0
467	**92**	**0**	**0**

Peak Hour Data

07/30/2021 3:30 PM
Ending At
07/30/2021 4:30 PM

Lights
Buses
Single-Unit Trucks
Articulated Trucks
Pedestrians

SH 259 A [E]

Out	In	Total
148	201	349
1	0	1
1	1	2
0	0	0
0	0	0
150	**202**	**352**

R	L	U	P
132	69	0	0
0	0	0	0
1	0	0	0
0	0	0	0
0	0	0	0
133	**69**	**0**	**0**

U	T	R	P
0	672	56	0
0	0	1	0
0	6	1	0
0	9	0	0
0	0	0	0
0	**687**	**58**	**0**

Out	In	Total
526	728	1254
0	1	1
5	7	12
5	9	14
0	0	0
536	**745**	**1281**

US 259 [S]

Turning Movement Peak Hour Data Plot (3:30 PM)

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/31/2021

Turning Movement Data

Start Time	US 259 Southbound					SH 259 A Westbound					US 259 Northbound					
	Thru	Left	U-Turn	Peds	App. Total	Right	Left	U-Turn	Peds	App. Total	Right	Thru	U-Turn	Peds	App. Total	Int. Total
12:00 AM	12	0	0	0	12	1	1	0	0	2	1	12	0	0	13	27
12:15 AM	13	2	0	0	15	2	0	0	0	2	3	16	1	0	20	37
12:30 AM	14	1	0	0	15	1	0	0	0	1	2	13	0	0	15	31
12:45 AM	16	1	0	0	17	0	0	0	0	0	1	14	0	0	15	32
Hourly Total	55	4	0	0	59	4	1	0	0	5	7	55	1	0	63	127
1:00 AM	11	1	0	0	12	0	1	0	0	1	0	9	0	0	9	22
1:15 AM	5	1	0	0	6	0	1	0	0	1	1	12	0	0	13	20
1:30 AM	5	0	0	0	5	0	1	0	0	1	2	5	0	0	7	13
1:45 AM	27	0	0	0	27	1	0	0	0	1	0	4	0	0	4	32
Hourly Total	48	2	0	0	50	1	3	0	0	4	3	30	0	0	33	87
2:00 AM	29	2	0	0	31	0	0	0	0	0	3	4	0	0	7	38
2:15 AM	13	1	0	0	14	0	2	0	0	2	0	6	0	0	6	22
2:30 AM	14	1	0	0	15	0	0	0	0	0	0	5	0	0	5	20
2:45 AM	9	0	0	0	9	1	1	0	0	2	0	3	0	0	3	14
Hourly Total	65	4	0	0	69	1	3	0	0	4	3	18	0	0	21	94
3:00 AM	5	0	0	0	5	0	1	0	0	1	0	3	0	0	3	9
3:15 AM	9	0	0	0	9	0	1	0	0	1	0	8	0	0	8	18
3:30 AM	7	0	0	0	7	0	0	0	0	0	0	3	0	0	3	10
3:45 AM	2	0	0	0	2	1	0	0	0	1	0	7	0	0	7	10
Hourly Total	23	0	0	0	23	1	2	0	0	3	0	21	0	0	21	47
4:00 AM	5	1	0	0	6	0	0	0	0	0	0	1	0	0	1	7
4:15 AM	2	0	0	0	2	0	0	0	0	0	0	3	0	0	3	5
4:30 AM	5	0	0	0	5	0	0	0	0	0	0	5	0	0	5	10
4:45 AM	2	0	0	0	2	0	1	0	0	1	1	12	0	0	13	16
Hourly Total	14	1	0	0	15	0	1	0	0	1	1	21	0	0	22	38
5:00 AM	2	0	0	0	2	0	1	0	0	1	0	12	0	0	12	15
5:15 AM	2	0	0	0	2	0	0	0	0	0	0	13	0	0	13	15
5:30 AM	2	7	0	0	9	0	0	0	0	0	1	17	0	0	18	27
5:45 AM	5	1	0	0	6	2	1	0	0	3	1	9	0	0	10	19
Hourly Total	11	8	0	0	19	2	2	0	0	4	2	51	0	0	53	76
6:00 AM	11	4	0	0	15	1	1	0	0	2	0	13	0	0	13	30
6:15 AM	9	2	0	1	11	1	1	0	0	2	4	8	0	0	12	25
6:30 AM	14	5	0	0	19	0	1	0	0	1	1	15	0	0	16	36
6:45 AM	8	3	0	0	11	2	3	0	0	5	3	15	0	0	18	34
Hourly Total	42	14	0	1	56	4	6	0	0	10	8	51	0	0	59	125
7:00 AM	22	4	0	0	26	3	2	0	0	5	4	26	0	0	30	61

7:15 AM	28	2	0	0	30	3	4	0	0	7	5	39	0	0	44	81
7:30 AM	23	3	0	0	26	7	3	0	0	10	6	36	0	0	42	78
7:45 AM	46	13	0	0	59	1	3	0	0	4	6	38	0	0	44	107
Hourly Total	119	22	0	0	141	14	12	0	0	26	21	139	0	0	160	327
8:00 AM	30	14	0	0	44	0	2	0	0	2	4	37	0	0	41	87
8:15 AM	30	13	0	0	43	14	6	0	0	20	11	67	0	0	78	141
8:30 AM	53	9	0	0	62	11	17	0	0	28	16	52	0	0	68	158
8:45 AM	46	20	0	0	66	9	13	0	0	22	14	52	0	0	66	154
Hourly Total	159	56	0	0	215	34	38	0	0	72	45	208	0	0	253	540
9:00 AM	52	16	0	0	68	11	5	0	0	16	8	49	0	0	57	141
9:15 AM	63	25	0	0	88	9	11	0	0	20	9	78	0	0	87	195
9:30 AM	71	28	0	0	99	15	10	0	0	25	10	93	0	0	103	227
9:45 AM	91	25	0	0	116	11	5	0	0	16	12	96	0	0	108	240
Hourly Total	277	94	0	0	371	46	31	0	0	77	39	316	0	0	355	803
10:00 AM	80	33	0	0	113	23	7	0	0	30	11	106	0	0	117	260
10:15 AM	110	39	0	0	149	35	11	1	0	47	12	129	0	0	141	337
10:30 AM	86	39	1	0	126	27	10	0	0	37	14	109	2	0	125	288
10:45 AM	86	38	0	0	124	36	7	0	0	43	10	121	0	0	131	298
Hourly Total	362	149	1	0	512	121	35	1	0	157	47	465	2	0	514	1183
11:00 AM	115	28	1	0	144	35	12	0	0	47	21	126	0	0	147	338
11:15 AM	92	52	0	0	144	42	8	0	0	50	22	100	0	0	122	316
11:30 AM	85	45	1	0	131	37	9	0	0	46	20	101	0	0	121	298
11:45 AM	105	37	0	0	142	25	19	0	0	44	13	95	0	0	108	294
Hourly Total	397	162	2	0	561	139	48	0	0	187	76	422	0	0	498	1246
12:00 PM	130	38	0	0	168	48	17	0	0	65	19	102	0	0	121	354
12:15 PM	132	63	0	0	195	45	14	0	0	59	18	99	0	0	117	371
12:30 PM	109	27	1	0	137	50	19	0	0	69	15	103	0	0	118	324
12:45 PM	104	44	3	0	151	48	15	0	0	63	16	128	0	0	144	358
Hourly Total	475	172	4	0	651	191	65	0	0	256	68	432	0	0	500	1407
1:00 PM	110	44	0	0	154	62	18	0	0	80	17	109	0	0	126	360
1:15 PM	115	29	0	0	144	47	11	0	0	58	11	103	0	0	114	316
1:30 PM	88	41	0	0	129	34	16	0	0	50	13	110	0	0	123	302
1:45 PM	98	56	0	0	154	29	18	0	0	47	14	115	0	0	129	330
Hourly Total	411	170	0	0	581	172	63	0	0	235	55	437	0	0	492	1308
2:00 PM	118	41	0	0	159	42	8	0	0	50	13	112	0	0	125	334
2:15 PM	99	37	0	0	136	47	16	0	0	63	11	115	0	0	126	325
2:30 PM	103	40	0	0	143	54	8	0	0	62	19	119	0	0	138	343
2:45 PM	116	40	0	0	156	57	20	0	0	77	4	108	0	0	112	345
Hourly Total	436	158	0	0	594	200	52	0	0	252	47	454	0	0	501	1347
3:00 PM	91	47	0	0	138	58	11	0	0	69	15	109	0	0	124	331
3:15 PM	118	32	0	0	150	43	14	0	0	57	21	130	0	0	151	358
3:30 PM	84	17	0	0	101	65	12	0	0	77	13	143	0	0	156	334
3:45 PM	80	27	0	0	107	40	22	0	0	62	13	102	0	0	115	284
Hourly Total	373	123	0	0	496	206	59	0	0	265	62	484	0	0	546	1307
4:00 PM	96	29	0	0	125	54	18	0	0	72	17	116	0	0	133	330
4:15 PM	125	17	0	0	142	51	17	0	0	68	18	111	0	0	129	339
4:30 PM	101	19	0	0	120	28	9	0	0	37	13	111	0	0	124	281
4:45 PM	98	39	0	0	137	37	10	0	0	47	14	124	0	0	138	322
Hourly Total	420	104	0	0	524	170	54	0	0	224	62	462	0	0	524	1272
5:00 PM	106	17	0	0	123	32	12	0	0	44	7	106	0	0	113	280
5:15 PM	96	25	0	0	121	35	24	0	0	59	12	110	0	0	122	302
5:30 PM	97	18	0	0	115	31	14	0	0	45	15	86	0	0	101	261

5:45 PM	99	12	0	0	111	38	13	0	0	51	10	92	0	0	102	264
Hourly Total	398	72	0	0	470	136	63	0	0	199	44	394	0	0	438	1107
6:00 PM	103	19	1	0	123	30	16	0	0	46	9	92	0	0	101	270
6:15 PM	111	31	0	0	142	29	11	0	0	40	6	92	0	0	98	280
6:30 PM	100	18	0	0	118	49	18	0	0	67	7	82	0	0	89	274
6:45 PM	109	13	0	0	122	18	6	0	0	24	15	71	0	0	86	232
Hourly Total	423	81	1	0	505	126	51	0	0	177	37	337	0	0	374	1056
7:00 PM	97	12	1	0	110	17	8	0	0	25	9	93	0	0	102	237
7:15 PM	93	12	0	0	105	27	14	0	0	41	5	70	0	0	75	221
7:30 PM	89	22	0	0	111	14	4	0	0	18	11	79	0	0	90	219
7:45 PM	86	11	0	0	97	13	7	0	0	20	4	76	0	0	80	197
Hourly Total	365	57	1	0	423	71	33	0	0	104	29	318	0	0	347	874
8:00 PM	102	19	0	0	121	22	14	0	0	36	5	78	0	0	83	240
8:15 PM	108	11	0	0	119	29	11	0	0	40	4	58	0	0	62	221
8:30 PM	81	18	0	0	99	14	6	0	0	20	4	50	0	0	54	173
8:45 PM	76	6	0	0	82	9	6	0	0	15	3	65	0	0	68	165
Hourly Total	367	54	0	0	421	74	37	0	0	111	16	251	0	0	267	799
9:00 PM	89	12	0	0	101	13	3	0	0	16	1	55	0	0	56	173
9:15 PM	50	7	0	0	57	4	5	0	0	9	3	50	0	0	53	119
9:30 PM	54	9	0	0	63	3	3	0	0	6	8	44	0	0	52	121
9:45 PM	46	7	0	0	53	9	2	0	0	11	3	37	0	0	40	104
Hourly Total	239	35	0	0	274	29	13	0	0	42	15	186	0	0	201	517
10:00 PM	47	6	0	0	53	4	2	0	0	6	4	55	0	0	59	118
10:15 PM	40	3	0	0	43	3	3	0	0	6	0	32	1	0	33	82
10:30 PM	31	1	0	0	32	4	1	0	0	5	2	28	0	0	30	67
10:45 PM	31	2	0	0	33	3	1	0	0	4	3	36	0	0	39	76
Hourly Total	149	12	0	0	161	14	7	0	0	21	9	151	1	0	161	343
11:00 PM	36	3	0	0	39	2	1	0	0	3	1	13	0	0	14	56
11:15 PM	37	3	0	0	40	1	3	0	0	4	1	10	0	0	11	55
11:30 PM	15	1	0	0	16	2	4	0	0	6	2	21	0	0	23	45
11:45 PM	23	3	0	0	26	0	3	0	0	3	1	18	0	0	19	48
Hourly Total	111	10	0	0	121	5	11	0	0	16	5	62	0	0	67	204
Grand Total	5739	1564	9	1	7312	1761	690	1	0	2452	701	5765	4	0	6470	16234
Approach %	78.5	21.4	0.1	-	-	71.8	28.1	0.0	-	-	10.8	89.1	0.1	-	-	-
Total %	35.4	9.6	0.1	-	45.0	10.8	4.3	0.0	-	15.1	4.3	35.5	0.0	-	39.9	-
Lights	5681	1560	9	-	7250	1759	687	1	-	2447	699	5695	4	-	6398	16095
% Lights	99.0	99.7	100.0	-	99.2	99.9	99.6	100.0	-	99.8	99.7	98.8	100.0	-	98.9	99.1
Buses	1	0	0	-	1	2	1	0	-	3	0	3	0	-	3	7
% Buses	0.0	0.0	0.0	-	0.0	0.1	0.1	0.0	-	0.1	0.0	0.1	0.0	-	0.0	0.0
Single-Unit Trucks	19	3	0	-	22	0	2	0	-	2	2	14	0	-	16	40
% Single-Unit Trucks	0.3	0.2	0.0	-	0.3	0.0	0.3	0.0	-	0.1	0.3	0.2	0.0	-	0.2	0.2
Articulated Trucks	38	1	0	-	39	0	0	0	-	0	0	53	0	-	53	92
% Articulated Trucks	0.7	0.1	0.0	-	0.5	0.0	0.0	0.0	-	0.0	0.0	0.9	0.0	-	0.8	0.6
Pedestrians	-	-	-	1	-	-	-	-	0	-	-	-	-	0	-	-
% Pedestrians	-	-	-	100.0	-	-	-	-	-	-	-	-	-	-	-	-

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/31/2021

US 259 [N]

Out	In	Total
7463	7250	14713
5	1	6
14	22	36
53	39	92
0	0	0
7535	**7312**	**14847**

T	L	U	P
5681	1560	9	0
1	0	0	0
19	3	0	0
38	1	0	0
0	0	0	1
5739	**1564**	**9**	**1**

SH 259 A [E]

Out	In	Total
2260	2447	4707
0	3	3
5	2	7
1	0	1
0	0	0
2266	**2452**	**4718**

R	L	U	P
1759	687	1	0
2	1	0	0
0	2	0	0
0	0	0	0
0	0	0	0
1761	**690**	**1**	**0**

07/31/2021 12:00 AM
Ending At
08/01/2021 12:00 AM

Lights
Buses
Single-Unit Trucks
Articulated Trucks
Pedestrians

U	T	R	P
4	5695	699	0
0	3	0	0
0	14	2	0
0	53	0	0
0	0	0	0
4	**5765**	**701**	**0**

Out	In	Total
6372	6398	12770
2	3	5
21	16	37
38	53	91
0	0	0
6433	**6470**	**12903**

US 259 [S]

Turning Movement Data Plot

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Turning Movement Peak Hour Data (10:15 AM)

Start Time	US 259 Southbound					SH 259 A Westbound					US 259 Northbound					
	Thru	Left	U-Turn	Peds	App. Total	Right	Left	U-Turn	Peds	App. Total	Right	Thru	U-Turn	Peds	App. Total	Int. Total
10:15 AM	110	39	0	0	149	35	11	1	0	47	12	129	0	0	141	337
10:30 AM	86	39	1	0	126	27	10	0	0	37	14	109	2	0	125	288
10:45 AM	86	38	0	0	124	36	7	0	0	43	10	121	0	0	131	298
11:00 AM	115	28	1	0	144	35	12	0	0	47	21	126	0	0	147	338
Total	397	144	2	0	543	133	40	1	0	174	57	485	2	0	544	1261
Approach %	73.1	26.5	0.4	-	-	76.4	23.0	0.6	-	-	10.5	89.2	0.4	-	-	-
Total %	31.5	11.4	0.2	-	43.1	10.5	3.2	0.1	-	13.8	4.5	38.5	0.2	-	43.1	-
PHF	0.863	0.923	0.500	-	0.911	0.924	0.833	0.250	-	0.926	0.679	0.940	0.250	-	0.925	0.933
Lights	395	143	2	-	540	133	40	1	-	174	57	479	2	-	538	1252
% Lights	99.5	99.3	100.0	-	99.4	100.0	100.0	100.0	-	100.0	100.0	98.8	100.0	-	98.9	99.3
Buses	0	0	0	-	0	0	0	0	-	0	0	0	0	-	0	0
% Buses	0.0	0.0	0.0	-	0.0	0.0	0.0	0.0	-	0.0	0.0	0.0	0.0	-	0.0	0.0
Single-Unit Trucks	2	0	0	-	2	0	0	0	-	0	0	1	0	-	1	3
% Single-Unit Trucks	0.5	0.0	0.0	-	0.4	0.0	0.0	0.0	-	0.0	0.0	0.2	0.0	-	0.2	0.2
Articulated Trucks	0	1	0	-	1	0	0	0	-	0	0	5	0	-	5	6
% Articulated Trucks	0.0	0.7	0.0	-	0.2	0.0	0.0	0.0	-	0.0	0.0	1.0	0.0	-	0.9	0.5
Pedestrians	-	-	-	0	-	-	-	-	0	-	-	-	-	0	-	-
% Pedestrians	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/31/2021

US 259 [N]

Out	In	Total
614	540	1154
0	0	0
1	2	3
5	1	6
0	0	0
620	**543**	**1163**

T	L	U	P
395	143	2	0
0	0	0	0
2	0	0	0
0	1	0	0
0	0	0	0
397	**144**	**2**	**0**

Peak Hour Data

07/31/2021 10:15 AM
Ending At
07/31/2021 11:15 AM

Lights
Buses
Single-Unit Trucks
Articulated Trucks
Pedestrians

SH 259 A [E]

Out	In	Total
201	174	375
0	0	0
0	0	0
1	0	1
0	0	0
202	**174**	**376**

R	L	U	P
133	40	1	0
0	0	0	0
0	0	0	0
0	0	0	0
0	0	0	0
133	**40**	**1**	**0**

U	T	R	P
2	479	57	0
0	0	0	0
0	1	0	0
0	5	0	0
0	0	0	0
2	**485**	**57**	**0**

US 259 [S]

Out	In	Total
437	538	975
0	0	0
2	1	3
0	5	5
0	0	0
439	**544**	**983**

Turning Movement Peak Hour Data Plot (10:15 AM)

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/31/2021
Page No: 7

Turning Movement Peak Hour Data (12:15 PM)

Start Time	US 259 Southbound					SH 259 A Westbound					US 259 Northbound					
	Thru	Left	U-Turn	Peds	App. Total	Right	Left	U-Turn	Peds	App. Total	Right	Thru	U-Turn	Peds	App. Total	Int. Total
12:15 PM	132	63	0	0	195	45	14	0	0	59	18	99	0	0	117	371
12:30 PM	109	27	1	0	137	50	19	0	0	69	15	103	0	0	118	324
12:45 PM	104	44	3	0	151	48	15	0	0	63	16	128	0	0	144	358
1:00 PM	110	44	0	0	154	62	18	0	0	80	17	109	0	0	126	360
Total	455	178	4	0	637	205	66	0	0	271	66	439	0	0	505	1413
Approach %	71.4	27.9	0.6	-	-	75.6	24.4	0.0	-	-	13.1	86.9	0.0	-	-	-
Total %	32.2	12.6	0.3	-	45.1	14.5	4.7	0.0	-	19.2	4.7	31.1	0.0	-	35.7	-
PHF	0.862	0.706	0.333	-	0.817	0.827	0.868	0.000	-	0.847	0.917	0.857	0.000	-	0.877	0.952
Lights	450	177	4	-	631	205	66	0	-	271	66	436	0	-	502	1404
% Lights	98.9	99.4	100.0	-	99.1	100.0	100.0	-	-	100.0	100.0	99.3	-	-	99.4	99.4
Buses	0	0	0	-	0	0	0	0	-	0	0	0	0	-	0	0
% Buses	0.0	0.0	0.0	-	0.0	0.0	0.0	-	-	0.0	0.0	0.0	-	-	0.0	0.0
Single-Unit Trucks	1	1	0	-	2	0	0	0	-	0	0	0	0	-	0	2
% Single-Unit Trucks	0.2	0.6	0.0	-	0.3	0.0	0.0	-	-	0.0	0.0	0.0	-	-	0.0	0.1
Articulated Trucks	4	0	0	-	4	0	0	0	-	0	0	3	0	-	3	7
% Articulated Trucks	0.9	0.0	0.0	-	0.6	0.0	0.0	-	-	0.0	0.0	0.7	-	-	0.6	0.5
Pedestrians	-	-	-	0	-	-	-	-	0	-	-	-	-	0	-	-
% Pedestrians	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Lee Engineering, LLC
Phoenix, Arizona - Dallas, Texas
Oklahoma City, Oklahoma - San Antonio, Texas
Albuquerque, New Mexico, United States
eshaw@lee-eng.com

Count Name: US 259 & 259 A
Site Code:
Start Date: 07/31/2021
Page No: 8

Peak Hour Data

07/31/2021 12:15 PM
Ending At
07/31/2021 1:15 PM

Lights
Buses
Single-Unit Trucks
Articulated Trucks
Pedestrians

US 259 [N]

Out	In	Total
645	631	1276
0	0	0
0	2	2
3	4	7
0	0	0
648	**637**	**1285**

T	L	U	P
450	177	4	0
0	0	0	0
1	1	0	0
4	0	0	0
0	0	0	0
455	**178**	**4**	**0**

SH 259 A [E]

Out	In	Total
243	271	514
0	0	0
1	0	1
0	0	0
0	0	0
244	**271**	**515**

R	L	U	P
205	66	0	0
0	0	0	0
0	0	0	0
0	0	0	0
0	0	0	0
205	**66**	**0**	**0**

U	T	R	P
0	436	66	0
0	0	0	0
0	0	0	0
0	3	0	0
0	0	0	0
0	**439**	**66**	**0**

Out	In	Total
516	502	1018
0	0	0
1	0	1
4	3	7
0	0	0
521	**505**	**1026**

US 259 [S]

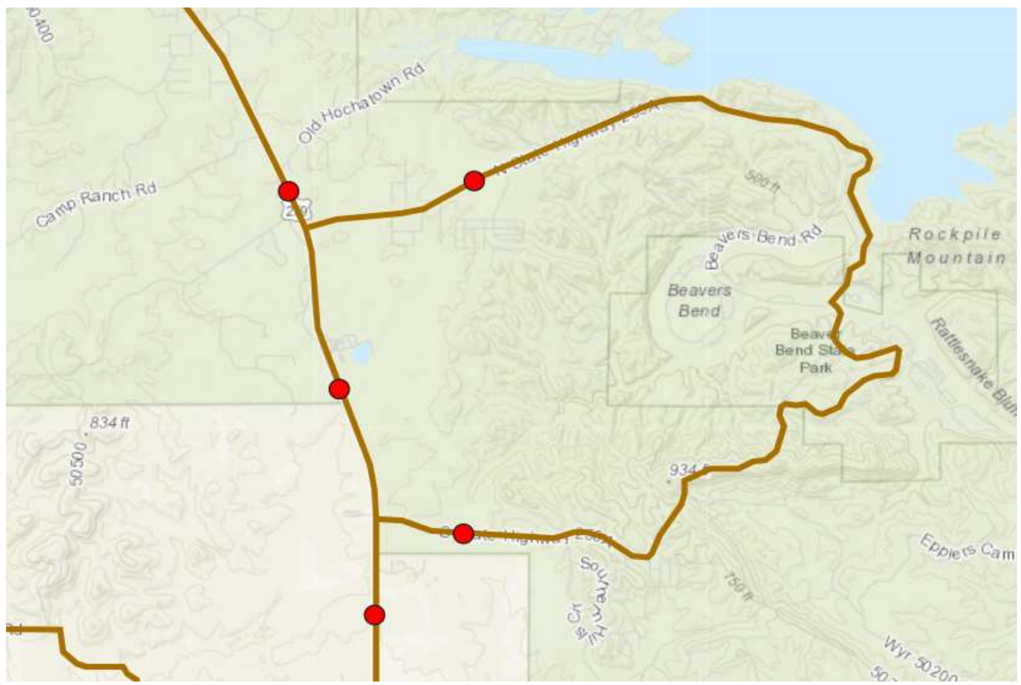
Turning Movement Peak Hour Data Plot (12:15 PM)

GROWTH CALCULATIONS

EVALUATION OF BACKGROUND TRAFFIC GROWTH

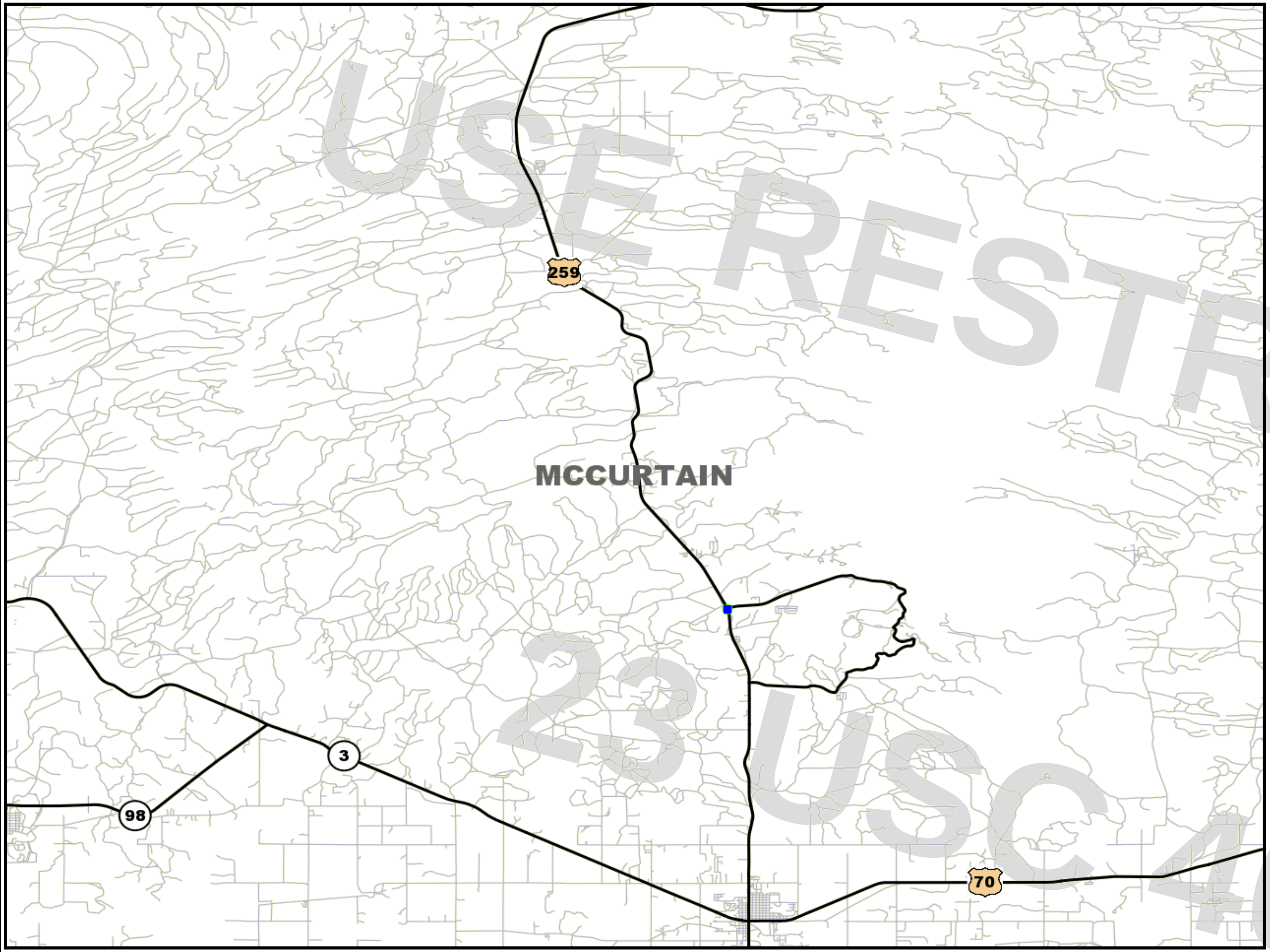
SH-259 N. of SH-259A			SH-259 S. of SH-259A			SH-259 b/t SH-259A North & South			SH-259A North			SH-259A South		
Site ID: 00450013			Site ID: 00450017			Site ID: 00450015			Site ID: 00450014			Site ID: 00450016		
Year	AADT	Growth Percent	Year	AADT	Growth Percent	Year	AADT	Growth Percent	Year	AADT	Growth Percent	Year	AADT	Growth Percent
2015	3,500	---	2015	4,000	---	2015	3,600	---	2015	600	---	2015	460	---
2016	3,900	11.4%	2016	4,200	5.0%	2016	3,900	8.3%	2016	610	1.7%	2016	540	##
2017	4,000	2.6%	2017	4,300	2.4%	2017	4,000	2.6%	2017	630	3.3%	2017	560	3.7%
2018	4,100	2.5%	2018	4,400	2.3%	2018	4,100	2.5%	2018	640	1.6%	2018	570	1.8%
2019	5,200	26.8%	2019	4,800	9.1%	2019	4,800	17.1%	2019	790	23.4%	2019	790	##
Average (Individual):		10.8%	Average (Individual):		4.7%	Average (Individual):		7.6%	Average (Individual):		7.5%	Average (Individual):		15.4%
Average (Overall):		12.1%	Average (Overall):		5.0%	Average (Overall):		8.3%	Average (Overall):		7.9%	Average (Overall):		17.9%

Average (Individual):	**9.2%**
Average (Overall):	**10.3%**
Use:	**10.0%**
Traffic Count Year	**2021**
Build-Out Year	**2023**



COLLISION DATA

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021
by Srinivas Minnekanti



Study Map & Totals

Legend

- ▲ Fatality
- ■ Injury
- ● Property Damage

Remarks:

PREPARED FOR LEE ENGINEERING, LLC.

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 thru 12-31-2019

	2010						2011						2012					
	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot
Collisions						0						0						0
Persons						0						0						0

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

	2013						2014						2015					
	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot
Collisions						0						0					2	2
Persons						0						0						0

	2016						2017						2018*					
	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot
Collisions						0				1	1	2			1		1	2
Persons						0				1		1			2			2

* DENOTES A YEAR FOR WHICH DATA MAY BE INCOMPLETE.

	2019*					
	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot
Collisions				2	2	4
Persons				2		2

* DENOTES A YEAR FOR WHICH DATA MAY BE INCOMPLETE.

	Study Total					
	Fatality	Suspected Serious Injury	Non-Incapacitating Injury	Possible Injury	Property Damage	Total
Collisions			1	3	6	10
Persons			2	3		5

STUDY TOTALS - BY CITY AND HWY CLASS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

STUDY TOTALS

	HIGHWAY COLLISIONS				CITY STREET COLLISIONS				COUNTY ROAD COLLISIONS				TOTAL COLLISIONS			
Year	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
2015			2	2											2	2
2017		1	1	2										1	1	2
2018 *		1	1	2										1	1	2
2019 *		2	2	4										2	2	4
Total:		4	6	10				0				0		4	6	10

* DENOTES A YEAR FOR WHICH DATA MAY BE INCOMPLETE.

County: (45) MCCURTAIN

	HIGHWAY COLLISIONS				CITY STREET COLLISIONS				COUNTY ROAD COLLISIONS				TOTAL COLLISIONS			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
(00) - RURAL -			1	1											1	1
(05) BROKEN BOW		4	5	9										4	5	9
Total:		4	6	10				0				0		4	6	10

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

TABULATION OF COLLISIONS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Collisions By Type Of Collision

Type Of Collision	2010				2011				2012				2013				2014			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Rear-End (front-to-rear)																				
Head-On (front-to-front)																				
Right Angle (front-to-side)																				
Angle Turning																				
Other Angle																				
Sideswipe Same Direction																				
Sideswipe Opposite Direction																				
Fixed Object																				
Pedestrian																				
Pedal Cycle																				
Animal																				
Overturn/Rollover																				
Vehicle-Train																				
Other Single Vehicle Crash																				
Other																				
Total																				
Percent																				

Collisions By Type Of Collision

Type Of Collision	2015				2016				2017				2018*				2019*			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Rear-End (front-to-rear)			1	1						1	1	2		1		1		2	1	3
Head-On (front-to-front)																				
Right Angle (front-to-side)																				
Angle Turning															1	1				
Other Angle																				
Sideswipe Same Direction																				
Sideswipe Opposite Direction																				
Fixed Object			1	1															1	1
Pedestrian																				
Pedal Cycle																				
Animal																				
Overturn/Rollover																				
Vehicle-Train																				
Other Single Vehicle Crash																				
Other																				
Total			2	2						1	1	2		1	1	2		2	2	4
Percent			20.0	20.0						10.0	10.0	20.0		10.0	10.0	20.0		20.0	20.0	40.0

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

TABULATION OF COLLISIONS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Collisions By Type Of Collision

Type Of Collision	Total				
	Fat	Inj *	PD	Tot	Pct
Rear-End (front-to-rear)		4	3	7	70.0
Head-On (front-to-front)					
Right Angle (front-to-side)					
Angle Turning			1	1	10.0
Other Angle					
Sideswipe Same Direction					
Sideswipe Opposite Direction					
Fixed Object			2	2	20.0
Pedestrian					
Pedal Cycle					
Animal					
Overturn/Rollover					
Vehicle-Train					
Other Single Vehicle Crash					
Other					
Total		4	6	10	100
Percent		40.0	60.0	100	

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Units By Unit Type

Unit Type	2010				2011				2012				2013				2014			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Train																				
Pedestrian																				
Animal																				
Pedal Cycle																				
Parked Vehicle																				
CMV																				
Other Single Vehicle																				
Other Multi-Vehicle																				
Total																				
Percent																				

Units By Unit Type

Unit Type	2015				2016				2017				2018*				2019*			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Train																				
Pedestrian																				
Animal																				
Pedal Cycle																				
Parked Vehicle																				
CMV															1	1		1		1
Other Single Vehicle			1	1															1	1
Other Multi-Vehicle			2	2						2	2	4		3	1	4		3	2	5
Total			3	3						2	2	4		3	2	5		4	3	7
Percent			15.8	15.8						10.5	10.5	21.1		15.8	10.5	26.3		21.1	15.8	36.8

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

TABULATION OF COLLISIONS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Units By Unit Type

Unit Type	Total				
	Fat	Inj *	PD	Tot	Pct
Train					
Pedestrian					
Animal					
Pedal Cycle					
Parked Vehicle					
CMV		1	1	2	10.5
Other Single Vehicle			2	2	10.5
Other Multi-Vehicle		8	7	15	78.9
Total		9	10	19	100
Percent		47.4	52.6	100	

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Vehicles By Vehicle Type

Vehice Type	2010				2011				2012				2013				2014			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Passenger Vehicle-2 Door																				
Passenger Vehicle-4 Door																				
Passenger Vehicle-Convertible																				
Pickup Truck																				
Single-Unit Truck (2 axles)																				
Single-Unit Truck (3 or more axles)																				
School Bus																				
Truck/Trailer																				
Truck-Tractor (bobtail)																				
Truck-Tractor/Semi-Trailer																				
Truck-Tractor/Double																				
Truck-Tractor/Triple																				
Bus/Large Van (9-15 seats)																				
Bus (16+ seats)																				
Motorcycle																				
Motor Scooter/Moped																				
Motor Home																				
Farm Machinery																				
ATV																				
Sport Utility Vehicle (SUV)																				
Passenger Van																				
Truck More Than 10,000 lbs.																				
Van (10,000 lbs. or less)																				
Other																				
Total																				
Percent																				

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
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TABULATION OF COLLISIONS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Vehicles By Vehicle Type

Vehice Type	2015				2016				2017				2018*				2019*			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Passenger Vehicle-2 Door																				
Passenger Vehicle-4 Door															1	1			2	2
Passenger Vehicle-Convertible																				
Pickup Truck			2	2							3	3		1		1			1	1
Single-Unit Truck (2 axles)																				
Single-Unit Truck (3 or more axles)																				
School Bus																				
Truck/Trailer																				
Truck-Tractor (bobtail)																				
Truck-Tractor/Semi-Trailer															1	1			1	1
Truck-Tractor/Double																				
Truck-Tractor/Triple																				
Bus/Large Van (9-15 seats)																				
Bus (16+ seats)																				
Motorcycle																				
Motor Scooter/Moped																				
Motor Home																				
Farm Machinery																				
ATV																				
Sport Utility Vehicle (SUV)			1	1						1		1		1	1	2		2	1	3
Passenger Van																				
Truck More Than 10,000 lbs.																				
Van (10,000 lbs. or less)																				
Other																				
Total			3	3						1	3	4		2	3	5		2	5	7
Percent			15.8	15.8						5.3	15.8	21.1		10.5	15.8	26.3		10.5	26.3	36.8

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

TABULATION OF COLLISIONS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Vehicles By Vehicle Type

Vehice Type	Total				
	Fat	Inj *	PD	Tot	Pct
Passenger Vehicle-2 Door					
Passenger Vehicle-4 Door			3	3	15.8
Passenger Vehicle-Convertible					
Pickup Truck		1	6	7	36.8
Single-Unit Truck (2 axles)					
Single-Unit Truck (3 or more axles)					
School Bus					
Truck/Trailer					
Truck-Tractor (bobtail)					
Truck-Tractor/Semi-Trailer			2	2	10.5
Truck-Tractor/Double					
Truck-Tractor/Triple					
Bus/Large Van (9-15 seats)					
Bus (16+ seats)					
Motorcycle					
Motor Scooter/Moped					
Motor Home					
Farm Machinery					
ATV					
Sport Utility Vehicle (SUV)		4	3	7	36.8
Passenger Van					
Truck More Than 10,000 lbs.					
Van (10,000 lbs. or less)					
Other					
Total		5	14	19	100
Percent		26.3	73.7	100	

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

TABULATION OF COLLISIONS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Day And Time Of Occurrence Of Collisions

Day	Hour Of The Day																								Tot	Pcnt
	AM												PM													
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	Tot	Pcnt
Sunday					1										1										2	20.0
Monday															1				1						2	20.0
Tuesday													1												1	10.0
Wednesday																				1					1	10.0
Thursday																										
Friday								1								1	1								3	30.0
Saturday																	1								1	10.0
	Early Morning - Sunrise						Morning Peak			Mid Morning/Afternoon						PM Peak			Evening - Late Night						Tot	100
Total	1						1			3						3			2						10	
Percent	10.0						10.0			30.0						30.0			20.0						100	

Roadway/Lighting

Roadway Conditions	Lighting Conditions					Total	Percent
	Daylight	Darkness	Twilight	Lighted	Unknown		
Dry	6	2		1		9	90.0
Wet (Water)	1					1	10.0
Ice, Snow, or Slush							
Mud, Dirt, Gravel, or Sand							
Other							
Total	7	2		1		10	100
Percent	70.0	20.0		10.0		100	

Weather Conditions

Weather Conditions	Total	Percent
Clear	4	40.0
Clouds Present	6	60.0
Raining/Fog		
Snowing/Sleet/Hail		
Other		
Total	10	100

TABULATION OF COLLISIONS

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Drivers By Driver Conditions

Unsafe/Unlawful	Apparently Normal			Alcohol Involved						Sleep Suspected			Drug Use Indicated			Unknown Condition			Total				
				Ability Impaired			Odor Detected																
	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Total	Pcnt
Failed to Yield																							
Failed to Stop			1																		1	1	5.3
Failed to Signal																							
Improper Turn																							
Improper Start																							
Improper Stop																							
Improper Backing																							
Improper Parking																							
Improper Passing																							
Improper Lane Change																							
Left of Center			1																		1	1	5.3
Following Too Close		2	1														1			3	1	4	21.1
Unsafe Speed		1	1																	1	1	2	10.5
DWI																							
Inattention			2																		2	2	10.5
Negligent Driving																							
Defective Vehicle																							
Wrong Way																							
No Improper Action		5	4																	5	4	9	47.4
Other																							
Total		8	10														1			9	10	19	100
Percent		42.1	52.6														5.3			47.4	52.6	100	

Severities Indicate Highest Severity in Collision

Collisions By Special Feature

Special Feature	Total			
	Fat	Inj *	PD	Tot
Bridge				
Work Zone				
Cross Median				
Train Collision				

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

COLLISION CONCENTRATION LISTING

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

								------INTERSECTING------					
COUNTY	CITY	HWY CL	INT ID	CS/ ST.1	HWY	INT-REL/ TERM-LOC	CITY STREET NAME	CITY STREET NAME	HWY	MILE/ ST.2	SEV INDEX	NUM COLLS	RANK
(45)MCCURTAIN	(05)BROKEN BOW	7	19	16	US-259	INTER	PARK		US259A	08.23	14	9	1
(45)MCCURTAIN	(05)BROKEN BOW	7		16	US-259	INTER	PARK		US259A	08.23	1	1	2

SEVERITY INDEX = (1 * NUMBER OF PROPERTY DAMAGE ONLY COLLISIONS) + (2 * NUMBER OF POSSIBLE INJURY COLLISIONS) + (3 NUMBER OF NON INCAPACITATING INJURY COLLISIONS) + (4 * NUMBER OF SUSPECTED SERIOUS COLLISIONS) + (5 * NUMBER OF FATALITY COLLISIONS)

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Collision Rate Point Analysis

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT
Time Period: 01-01-2010 to 12-31-2019 (3652 days)

RATE = No. of Collisions per 100 Million Vehicles

Rate Type	Location Rates
Queried Collisions:	68.21
Fatal Collisions:	0.00
Vis. Injury Collisions *:	6.82

Collision History Summary
(Number of Years = 10)

# Collisions		# People	
Involving Fatality:	0	Killed:	0
Vis. Injury *:	1	Vis. Injured *:	2
Poss. Injury:	3	Poss. Injured:	3
Property Damage Only:	6		
TOTAL:	10		

* Includes Suspected Serious and Non-Incapacitating Injuries.

Road Characteristics

Roadway Length (miles):	00.00
Roadway Width (feet):	24
Number of Lanes :	TWO-LANES
Access Control :	NONE
Urban Area Type :	RURAL
Rural or Municipal :	RURAL
Median Type :	UNDIVIDED
Median Width (feet):	0

List of Intersection LEG ADTs.

US-259 : 3770 (Main CS, Mile pt. 08.23)
US-259 : 3620 (Main CS, Mile pt. 08.23)
US259A : 639 (Joining CS, at Mile pt. 10.178)

$$\text{RATE} = \frac{100{,}000{,}000 \times \text{NO. OF COLLISIONS}}{\text{ENTERING VEHICLES} \times \text{NO. OF DAYS IN REPORT}}$$

$$\text{ENTERING VEHICLES} = \frac{\text{SUM INTERSECTION LEG ADTs}}{2}$$

HIGHWAY SYSTEM COLLISION LISTING

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Cnty	City	CS #	Int. #	Mile Post	Location	Features	Int. Related	On Map	Dir. 1	Dir. 2	# Veh.	# Inj.*	# Fat.	Type of Collision	Unsafe Unlawful	Lighting Cond.	Roadway Cond.	Severity	Date
(45) MCCURTAIN				(00)	HWY: US-259, PARK			AT: US259A											
45		16	19	08.23	US259A		YES	Y	N	N	2			REAR-END	INATT	DYLGT	DRY	PDO	02-10-2015
45	05	16	19	08.23	US259A		YES	Y	S	S	2	1		REAR-END	FOL-CLOSE	DYLGT	DRY	P INJ	07-23-2017
45	05	16	19	08.23	US259A		YES	Y	S	S	3	2		REAR-END	FOL-CLOSE	DYLGT	DRY	N-I INJ	04-27-2018
45	05	16	19	08.23	US259A		YES	Y	S	S	2			ANGLE-TURNING	L-CENTER	DYLGT	DRY	PDO	05-21-2018
45	05	16	19	08.23	US259A		YES	Y	S	S	2	1		REAR-END	UNSAF-SPD	DYLGT	DRY	P INJ	02-08-2019
45	05	16	19	08.23	US259A		YES	Y	S	S	2	1		REAR-END	FOL-CLOSE	DYLGT	WET	P INJ	03-29-2019
45	05	16	19	08.23	US259A		YES	Y	S	S	2			REAR-END	INATT	DARK	DRY	PDO	11-27-2019
(45) MCCURTAIN				(05) BROKEN BOW	HWY: , US-259A			AT: US-259											
45	05	20	19	10.18	US-259		YES	Y	W	-	1			F-O EMBANKMENT	F-STOP	DARK	DRY	PDO	09-27-2015
45	05	20	19	10.18	US-259		YES	Y	W	W	2			REAR-END	FOL-CLOSE	DYLGT	DRY	PDO	03-18-2017
(45) MCCURTAIN				(05) BROKEN BOW	HWY: US-259, PARK			AT: US259A,						00.19 after BEG 55 MPH					
45	05	16		08.23			YES	Y	S	-	1			F-O GROUND	UNSAF-SPD	DARK	DRY	PDO	12-09-2019

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

SH-259 AT SH-259A BROKEN BOW COLLISION REPORT

Date Range: 01-01-2010 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

ROADWAY / REGION

QUERY OVER	SELECTIONS
Control Section	County: 45, Control Section: 16, CS Query On: intersection, Mile: 08.23

DATE

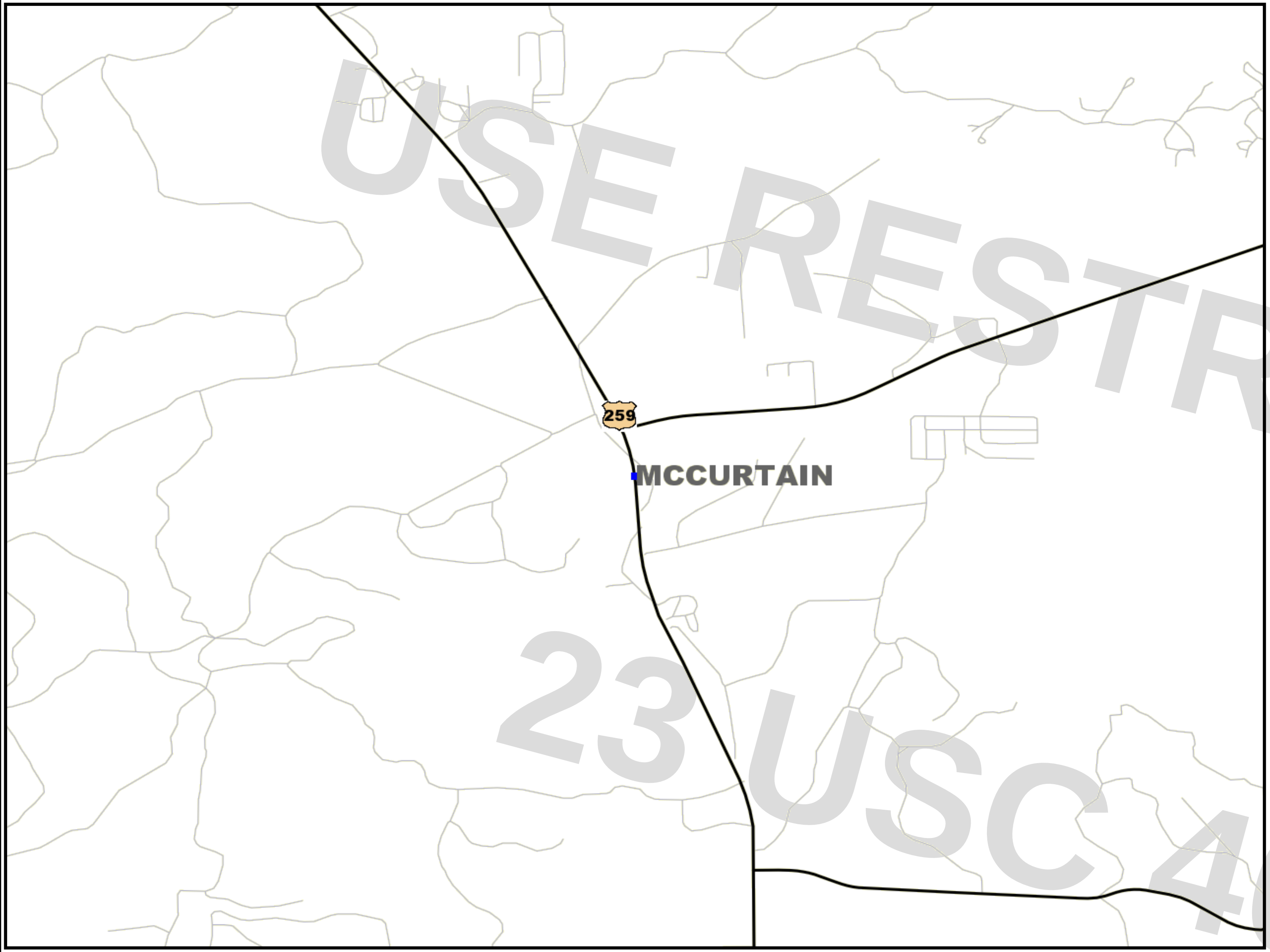
Date Range	01-01-2010 to 12-31-2019

FILTER COLLISIONS

Roadway Type	All Collision Data
Incl. Crashes Assoc. w/ Every Int.	Checked
Environment Fields	

REPORT SECTIONS

Collision Map & Study Totals	(Included)
Collision Analysis Tables	(Included)
- Totals By City, Hwy Class	Checked
- Other Analysis Tables	Checked
Concentration Listing	(Included)
- Sort Concentration List By	Number of Collisions
Rate Analysis	(Included)
Collision Listing	(Included)
- Highway Collision Listing	Checked, By Control Section
- City Street Collision Listing	Checked
- County Road Collision Listing	Checked
Query Criteria	(Included)



Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021
by Srinivas Minnekanti

Study Map & Totals

Legend

- ▲ Fatality
- ■ Injury
- ● Property Damage

Remarks:

PREPARED FOR LEE ENGINEERING, LLC.

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 thru 12-31-2019

	2015						2016						2017					
	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot
Collisions				1		1						0						0
Persons				1		1						0						0

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

	2018*						2019*					
	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot	Fat	SRS Inj	Non-Incap Inj	Poss Inj	PD	Tot
Collisions						0						0
Persons						0						0

* DENOTES A YEAR FOR WHICH DATA MAY BE INCOMPLETE.

	Study Total					
	Fatality	Suspected Serious Injury	Non-Incapacitating Injury	Possible Injury	Property Damage	Total
Collisions				1		1
Persons				1		1

STUDY TOTALS - BY CITY AND HWY CLASS

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

County: (45) MCCURTAIN

	HIGHWAY COLLISIONS				CITY STREET COLLISIONS				COUNTY ROAD COLLISIONS				TOTAL COLLISIONS			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
(05) BROKEN BOW		1		1										1		1

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Collisions By Type Of Collision

Type Of Collision	2015				2016				2017				2018*				2019*			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Rear-End (front-to-rear)		1		1																
Head-On (front-to-front)																				
Right Angle (front-to-side)																				
Angle Turning																				
Other Angle																				
Sideswipe Same Direction																				
Sideswipe Opposite Direction																				
Fixed Object																				
Pedestrian																				
Pedal Cycle																				
Animal																				
Overturn/Rollover																				
Vehicle-Train																				
Other Single Vehicle Crash																				
Other																				
Total		1		1																
Percent		100.0		100.0																

Collisions By Type Of Collision

Type Of Collision	Total				
	Fat	Inj *	PD	Tot	Pct
Rear-End (front-to-rear)		1		1	100.0
Head-On (front-to-front)					
Right Angle (front-to-side)					
Angle Turning					
Other Angle					
Sideswipe Same Direction					
Sideswipe Opposite Direction					
Fixed Object					
Pedestrian					
Pedal Cycle					
Animal					
Overturn/Rollover					
Vehicle-Train					
Other Single Vehicle Crash					
Other					
Total		1		1	100
Percent		100.0		100	

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
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SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Units By Unit Type

Unit Type	2015				2016				2017				2018*				2019*			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Train																				
Pedestrian																				
Animal																				
Pedal Cycle																				
Parked Vehicle																				
CMV																				
Other Single Vehicle																				
Other Multi-Vehicle		2		2																
Total		2		2																
Percent		100.0		100.0																

Units By Unit Type

Unit Type	Total				
	Fat	Inj *	PD	Tot	Pct
Train					
Pedestrian					
Animal					
Pedal Cycle					
Parked Vehicle					
CMV					
Other Single Vehicle					
Other Multi-Vehicle		2		2	100.0
Total		2		2	100
Percent		100.0		100	

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Vehicles By Vehicle Type

Vehice Type	2015				2016				2017				2018*				2019*			
	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot	Fat	Inj *	PD	Tot
Passenger Vehicle-2 Door																				
Passenger Vehicle-4 Door																				
Passenger Vehicle-Convertible																				
Pickup Truck		1	1	2																
Single-Unit Truck (2 axles)																				
Single-Unit Truck (3 or more axles)																				
School Bus																				
Truck/Trailer																				
Truck-Tractor (bobtail)																				
Truck-Tractor/Semi-Trailer																				
Truck-Tractor/Double																				
Truck-Tractor/Triple																				
Bus/Large Van (9-15 seats)																				
Bus (16+ seats)																				
Motorcycle																				
Motor Scooter/Moped																				
Motor Home																				
Farm Machinery																				
ATV																				
Sport Utility Vehicle (SUV)																				
Passenger Van																				
Truck More Than 10,000 lbs.																				
Van (10,000 lbs. or less)																				
Other																				
Total		1	1	2																
Percent		50.0	50.0	100.0																

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

TABULATION OF COLLISIONS

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Vehicles By Vehicle Type

Vehice Type	Total				
	Fat	Inj *	PD	Tot	Pct
Passenger Vehicle-2 Door					
Passenger Vehicle-4 Door					
Passenger Vehicle-Convertible					
Pickup Truck		1	1	2	100.0
Single-Unit Truck (2 axles)					
Single-Unit Truck (3 or more axles)					
School Bus					
Truck/Trailer					
Truck-Tractor (bobtail)					
Truck-Tractor/Semi-Trailer					
Truck-Tractor/Double					
Truck-Tractor/Triple					
Bus/Large Van (9-15 seats)					
Bus (16+ seats)					
Motorcycle					
Motor Scooter/Moped					
Motor Home					
Farm Machinery					
ATV					
Sport Utility Vehicle (SUV)					
Passenger Van					
Truck More Than 10,000 lbs.					
Van (10,000 lbs. or less)					
Other					
Total		1	1	2	100
Percent		50.0	50.0	100	

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
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Collision Analysis and Safety Branch
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Day And Time Of Occurrence Of Collisions

Day	Hour Of The Day																									
	AM												PM													
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	Tot	Pcnt
Sunday																										
Monday																										
Tuesday																										
Wednesday																										
Thursday																										
Friday																					1				1	100.0
Saturday																										
	Early Morning - Sunrise						Morning Peak			Mid Morning/Afternoon						PM Peak			Evening - Late Night						Tot	100
Total																			1						1	
Percent																			100.0						100	

Roadway/Lighting

Roadway Conditions	Lighting Conditions					Total	Percent
	Daylight	Darkness	Twilight	Lighted	Unknown		
Dry		1				1	100.0
Wet (Water)							
Ice, Snow, or Slush							
Mud, Dirt, Gravel, or Sand							
Other							
Total		1				1	100
Percent		100.0				100	

Weather Conditions

Weather Conditions	Total	Percent
Clear		
Clouds Present	1	100.0
Raining/Fog		
Snowing/Sleet/Hail		
Other		
Total	1	100

TABULATION OF COLLISIONS

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Drivers By Driver Conditions

Unsafe/Unlawful	Apparently Normal			Alcohol Involved						Sleep Suspected			Drug Use Indicated			Unknown Condition			Total				
				Ability Impaired			Odor Detected																
	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Fat	Inj *	PD	Total	Pcnt
Failed to Yield																							
Failed to Stop																							
Failed to Signal																							
Improper Turn																							
Improper Start																							
Improper Stop																							
Improper Backing																							
Improper Parking																							
Improper Passing																							
Improper Lane Change																							
Left of Center																							
Following Too Close																							
Unsafe Speed																							
DWI																							
Inattention		1																		1		1	50.0
Negligent Driving																							
Defective Vehicle																							
Wrong Way																							
No Improper Action		1																		1		1	50.0
Other																							
Total		2																		2		2	100
Percent		100.0																		100.0		100	

Severities Indicate Highest Severity in Collision

Collisions By Special Feature

Special Feature	Total			
	Fat	Inj *	PD	Tot
Bridge				
Work Zone				
Cross Median				
Train Collision				

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

								------INTERSECTING------					
COUNTY	CITY	HWY CL	INT ID	CS/ ST.1	HWY	INT-REL/ TERM-LOC	CITY STREET NAME	CITY STREET NAME	HWY	MILE/ ST.2	SEV INDEX	NUM COLLS	RANK
(45)MCCURTAIN	(05)BROKEN BOW	7		16	US-259		PARK			08.03	2	1	1

SEVERITY INDEX = (1 * NUMBER OF PROPERTY DAMAGE ONLY COLLISIONS) + (2 * NUMBER OF POSSIBLE INJURY COLLISIONS) + (3 NUMBER OF NON INCAPACITATING INJURY COLLISIONS) + (4 * NUMBER OF SUSPECTED SERIOUS COLLISIONS) + (5 * NUMBER OF FATALITY COLLISIONS

Oklahoma Department of Transportation

HIGHWAY SYSTEM COLLISION LISTING

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

Cnty	City	CS #	Int. #	Mile Post	Location	Features	Int. Related	On Map	Dir. 1	Dir. 2	# Veh.	# Inj.*	# Fat.	Type of Collision	Unsafe Unlawful	Lighting Cond.	Roadway Cond.	Severity	Date
(45) MCCURTAIN				(05) BROKEN BOW	HWY: US-259, PARK								AT: 00.01 before BEG 55 MPH						
45	05	16		08.03		DRIVEWAY	NO	Y	S	S	2	1		REAR-END	INATT	DARK	DRY	P INJ	08-21-2015

* INCLUDES SUSPECTED SERIOUS, NON-INCAPACITATING, AND POSSIBLE INJURIES.

STUDY CRITERIA

SH-259 AT DRIVEWAY 3 COLLISION REPORT

Date Range: 01-01-2015 Thru 12-31-2019

Program Provided by:
Traffic Engineering Division
Collision Analysis and Safety Branch
(405) 522-0985
Created: 08/13/2021 by Srinivas Minnekanti

ROADWAY / REGION

QUERY OVER	SELECTIONS
Draw Area on Map	User Selection on Map

DATE

Date Range	01-01-2015 to 12-31-2019

FILTER COLLISIONS

Roadway Type	All Collision Data
Incl. Crashes Assoc. w/ Every Int.	Checked
Environment Fields	

REPORT SECTIONS

Collision Map & Study Totals	(Included)
Collision Analysis Tables	(Included)
- Totals By City, Hwy Class	Checked
- Other Analysis Tables	Checked
Concentration Listing	(Included)
- Sort Concentration List By	Severity Index
Rate Analysis	(Included)
Collision Listing	(Included)
- Highway Collision Listing	Checked, By Control Section
- City Street Collision Listing	Checked
- County Road Collision Listing	Checked
Query Criteria	(Included)

SUPPORTING TRIP GENERATION RESEARCH

TRAFFIC PLANNING AND DESIGN, INC.

Est. 1989

Allentown Arena and City Center Development

Traffic Analysis

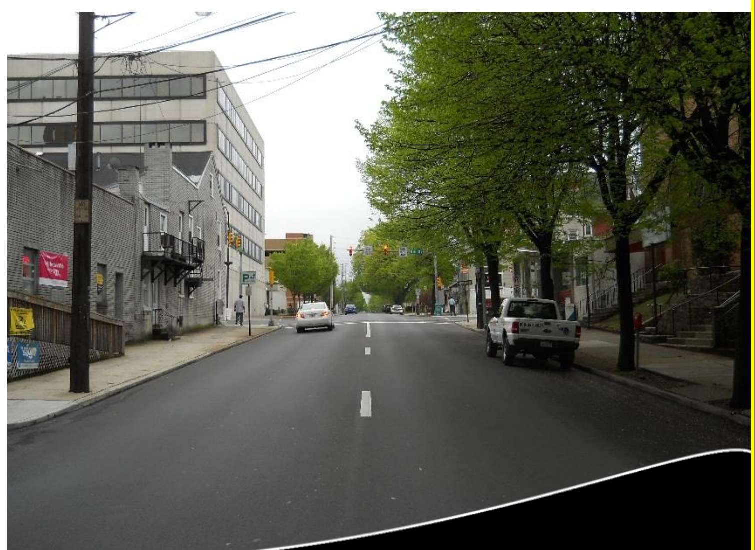
City of Allentown, Lehigh County, PA

For Submission To:

City of Allentown and PennDOT District 5-0

Last Revised: May 5, 2014

TPD# HCSD.A.00001



812 W. Hamilton Street | Suite 402 | Allentown, PA | 18101 | 610.625.4242 | TPD@TrafficPD.com

TRIP GENERATION

The trip generation calculations for the traffic analysis are based upon data published by the Institute of Transportation Engineers (ITE) and the Urban Land Institute (ULI).

ITE Trip Generation Manual

The ITE *Trip Generation Manual*, Ninth Edition, 2012, is the primary source utilized by traffic engineers to determine the trip generation characteristics of a given land use. The statistics in *Trip Generation* are empirical data based on more than 4,800 trip generation studies. The data are categorized by Land Use Codes, with total vehicular trips for a given land use estimated using an independent variable and statistically generated rates or equations. For each land use, TPD calculated the number of vehicular trips the development will generate during the following time periods: (1) weekday A.M. peak hour; (2) weekday P.M. peak hour; and (3) Saturday midday peak hour. **Table 5** shows the rates/equations and directional percentages for the analyzed time periods.

TABLE 5
ITE TRIP GENERATION DATA

Land Use	ITE #	Time Period	Equations/Rates	Independent Variable	Entering %	Pass-By %
Apartments	220	Weekday A.M. Peak Hour	T = 0.49*(X) + 3.73	dwelling units	20%	--
		Weekday P.M. Peak Hour	T = 0.55*(X) + 17.65	dwelling units	65%	--
		Saturday Midday Peak Hour	T = 0.41*(X) + 19.23	dwelling units	50%	--
Hotel	310	Weekday A.M. Peak Hour	T = 0.53*(X)	rooms	59%	--
		Weekday P.M. Peak Hour	T = 0.60*(X)	rooms	53%	--
		Saturday Midday Peak Hour	T = 0.72*(X)	rooms	56%	--
Health/Fitness Club	492	Weekday A.M. Peak Hour	T = 1.41*(X)	ksf	50%	--
		Weekday P.M. Peak Hour	T = 3.53*(X)	ksf	57%	--
		Saturday Midday Peak Hour	T = 2.78*(X)	ksf	45%	--
General Office Building	710	Weekday A.M. Peak Hour	Ln(T) = 0.80*Ln(X) + 1.57	ksf	88%	--
		Weekday P.M. Peak Hour	T = 1.12*(X) + 78.45	ksf	17%	--
		Saturday Midday Peak Hour	T = 0.43*(X)	ksf	54%	--
Shopping Center	820	Weekday A.M. Peak Hour	T = 0.96*(X)	ksf	62%	24%
		Weekday P.M. Peak Hour	T = 3.71*(X)	ksf	48%	34%
		Saturday Midday Peak Hour	T = 4.82*(X)	ksf	52%	26%
Quality Restaurant	931	Weekday A.M. Peak Hour	T = 0.81*(X)	ksf	50%	0%
		Weekday P.M. Peak Hour	T = 7.49*(X)	ksf	67%	44%
		Saturday Midday Peak Hour	T = 10.82*(X)	ksf	59%	34%
High-Turnover (Sit-Down) Restaurant	932	Weekday A.M. Peak Hour	T = 10.81*(X)	ksf	55%	33%
		Weekday P.M. Peak Hour	T = 9.85*(X)	ksf	60%	43%
		Saturday Midday Peak Hour	T = 14.07*(X)	ksf	53%	33%

T = number of site-generated vehicular trips
X = independent variable

Characteristics of Retail Uses

The proposed retail uses consist of complementary street-level retail on the first floor of the proposed office and apartment buildings. The proposed retail uses are intended to serve people who are already walking through the neighborhood or working downtown, similar to the existing retail uses along Hamilton Street. TPD evaluated the trip generation characteristics of these uses based on the average rates and regression equations contained in the *Trip Generation* manual for Land Use Code 820. It is TPD's opinion that the regression equation substantially overestimates the trip generation of the proposed retail. TPD believes that the average rates represent a conservative trip generation estimate, and therefore these results were utilized for the capacity analysis.

Arena Trip Generation

The published data in the *Trip Generation* manual for arenas (ITE Land Use Code 460) is based upon a single site surveyed in California in 1970. The data is limited to the number of trips generated over the course of an average weekday. No peak hour trip generation data is included in the manual. Because the data is based upon a single study completed more than four decades ago, TPD determined this information was not a reliable indicator of the traffic that will be generated by the proposed arena.

Therefore, TPD researched applicable data for arenas, stadiums, and concerts. **Table 6** below summarizes the occupancy rates (persons per car) for vehicles arriving at arenas or similar facilities:

TABLE 6
VEHICLE OCCUPANCY DATA

Data Source	Vehicle Occupancy Rate
1994 ITE Report on Stadia and Arenas	3.0 to 3.5
ULI Shared Parking Manual (p. 62) - Arena (Concerts)	2.0
ULI Shared Parking Manual (p. 63) - Arena Public Parking	3.0
ULI Shared Parking Manual (p. 69) – Stadium (Football Game)	3.3
ITE Discussion Group – 2003 San Antonio Amphitheatre Study (9,000 attendees)	2.60 -2.75
Average	**2.88**

The proposed arena will be designed to seat 8,500 attendees for hockey games, and up to 10,500 attendees for concerts and other events. In order to provide a conservative analysis, TPD evaluated the trip generation for a sold-out event with 10,500 attendees, and estimated that the vehicle occupancy rate will be 2.75 persons per car. This would result in a total of 3,818 vehicles.

In 2002, The Traffic Group conducted a post-development study for a 6,000 seat minor league baseball stadium in Aberdeen, Maryland. The results of this study indicated that 61% of attendees arrived at the stadium during the hour prior to the event beginning. Additionally, a study published by the Transportation Research Board in 2001 found that for large special events approximately 60% of attendees will arrive within one hour of the start of the event.

Based upon these studies, it was assumed that 61% of the 3,818 vehicles generated by a sold-out event would arrive in the hour prior to the start of the event. This results in a total of 2,329 peak hour trips. This is equivalent to a peak hour trip generation rate of 0.222 trips per attendee. This rate was applied for the weekday PM peak hour and Saturday midday peak hour.

Comparison to Minor League Baseball Stadiums

In order to verify these results, TPD reviewed trip generation data for two minor league stadiums. The above-referenced study in Aberdeen, MD concluded that a typical sold-out minor league baseball game (6,000 attendees) generated a total of 1,607 additional trips when compared to a non-game day. As noted above, 61% of traffic (980 trips) arrived in the hour prior to the event. This results in a peak hour trip generation rate of 0.164 trips per attendee.

As a final comparison, TPD conducted trip generation counts at Coca-Cola Park in Allentown, Pennsylvania. Coca-Cola Park was constructed in 2008 and is home to the Lehigh Valley Iron Pigs, AAA-affiliate of the Philadelphia Phillies. The capacity of the stadium is 10,000 fans. The counts were conducted from 5:00-7:30 PM on Friday, May 13, 2011. The first pitch of the game was at 7:05, and the attendance for the game was 9,660. A total of 2,709 trips were generated by the stadium between 5:00-7:30. 1,651 trips (61% of traffic) arrived from 6:00-7:00, which is consistent with the studies outlined above. This results in a peak hour trip generation rate of 0.170 trips per attendee. A summary of data collected at Coca-Cola Park is included in **Appendix E**.

Table 7 below compares the trip generation calculations conducted by TPD to the trip generation rates observed at the minor league baseball stadiums. Based upon this data, TPD is confident that the trip generation calculations for the proposed arena represent a conservative estimate.

TABLE 7
TRIP GENERATION RATE COMPARISON

Data Source	Trip Generation Rate
Ripken Stadium (Aberdeen, MD)	T = 0.164*(X)
Coca-Cola Park (Allentown, PA)	T = 0.170*(X)
Vehicle Occupancy Data (Utilized for this Analysis)	T = 0.222*(X)

T = number of site-generated vehicular trips
X = independent variable (attendees)

Non-Event Peak Hours

Given that there are typically no events associated with the arena during the weekday AM peak hour, TPD utilized data published in the ITE *Trip Generation* manual for an office, as shown in **Table 8**. The office use accounts for arena employees and office staff. It is anticipated that the arena will have approximately 100 daytime employees. TPD also utilized data for this land use to evaluate weekday P.M. and Saturday midday peak hours on non-event days.

TABLE 8
ITE TRIP GENERATION DATA

Land Use	ITE #	Time Period	Equations/Rates	Entering %
General Office	710	Weekday A.M. Peak Hour	Ln(T) = 0.86*Ln(X) + 0.24	88%
		Weekday P.M. Peak Hour	T = 0.37*(X) + 60.08	17%
		Saturday Midday Peak Hour	T = 0.09*(X)	54%

T = number of site-generated vehicular trips
X = independent variable (employees)

Transit/Pedestrian/Bicycle Trips

The trip generation rates published in the ITE *Trip Generation* manual are based upon data collected at isolated suburban locations that are predominantly dependent on automobiles. For developments that accommodate alternative modes of transportation (i.e. rail, bus, bicycle and pedestrians), it is expected that a percentage of site-generated traffic will be non-automobile trips. As noted above, the proposed development will be located in an urban core area with high-quality pedestrian accommodations and access to mass transit provided by LANTA. Therefore, a 10 percent reduction factor was applied to the trip generation calculations.

The total trip generation of the proposed development is summarized in **Tables 9 through 12**.

TABLE 9A
TRIP GENERATION SUMMARY - ARENA COMPLEX
EVENT DAYS

Land Use	Total Trips	Non-Auto Trips	Total Auto Trips			Pass-By Trips			New Trips		
			Total	Enter	Exit	Total	Enter	Exit	Total	Enter	Exit
Weekday A.M. Peak Hour											
Arena (100 employees)	67	-7	60	53	7	0	0	0	60	53	7
8,820 s.f. Casual Restaurant	--	--	--	--	--	--	--	--	--	--	--
13,060 s.f. Casual Restaurant	--	--	--	--	--	--	--	--	--	--	--
164,657 s.f. General Office	285	-29	256	225	31	0	0	0	256	225	31
65,863 s.f. Fitness Center	93	-9	84	42	42	0	0	0	84	42	42
180-Room Hotel	95	-10	85	50	35	0	0	0	85	50	35
Total	***540***	***-55***	***485***	***370***	***115***	***0***	***0***	***0***	***485***	***370***	***115***
Weekday P.M. Peak Hour											
10,500 seat Arena	2329	-233	2096	2096	0	0	0	0	2096	2096	0
8,820 s.f. Casual Restaurant	87	-9	78	47	31	32	16	16	46	31	15
13,060 s.f. Casual Restaurant	129	-13	116	70	46	48	24	24	68	46	22
164,657 s.f. General Office	263	-26	237	40	197	0	0	0	237	40	197
65,863 s.f. Fitness Center	232	-23	209	119	90	0	0	0	209	119	90
180-Room Hotel	108	-11	97	49	48	0	0	0	97	49	48
Total	***3148***	***-315***	***2833***	***2421***	***412***	***80***	***40***	***40***	***2753***	***2381***	***372***
Saturday Midday Peak Hour											
10,500 seat Arena	2329	-233	2096	2096	0	0	0	0	2096	2096	0
8,820 s.f. Casual Restaurant	124	-12	112	59	53	36	18	18	76	41	35
13,060 s.f. Casual Restaurant	184	-18	166	88	78	54	27	27	112	61	51
164,657 s.f. General Office	71	-7	64	35	29	0	0	0	64	35	29
65,863 s.f. Fitness Center	183	-18	165	74	91	0	0	0	165	74	91
180-Room Hotel	130	-13	117	66	51	0	0	0	117	66	51
Total	***3021***	***-301***	***2720***	***2418***	***302***	***90***	***45***	***45***	***2630***	***2373***	***257***

TABLE 9B
TRIP GENERATION SUMMARY - ARENA COMPLEX
NON-EVENT DAYS

Land Use	Total Trips	Non-Auto Trips	Total Auto Trips			Pass-By Trips			New Trips		
			Total	Enter	Exit	Total	Enter	Exit	Total	Enter	Exit
Weekday A.M. Peak Hour											
Arena (100 employees)	67	-7	60	53	7	0	0	0	60	53	7
8,820 s.f. Casual Restaurant	--	--	--	--	--	--	--	--	--	--	--
13,060 s.f. Casual Restaurant	--	--	--	--	--	--	--	--	--	--	--
164,657 s.f. General Office	285	-29	256	225	31	0	0	0	256	225	31
65,863 s.f. Fitness Center	93	-9	84	42	42	0	0	0	84	42	42
180-Room Hotel	95	-10	85	50	35	0	0	0	85	50	35
Total	***540***	***-55***	***485***	***370***	***115***	***0***	***0***	***0***	***485***	***370***	***115***
Weekday P.M. Peak Hour											
Arena (100 employees)	97	-10	87	15	72	0	0	0	87	15	72
8,820 s.f. Casual Restaurant	87	-9	78	47	31	32	16	16	46	31	15
13,060 s.f. Casual Restaurant	129	-13	116	70	46	48	24	24	68	46	22
164,657 s.f. General Office	263	-29	256	225	31	0	0	0	256	225	31
65,863 s.f. Fitness Center	232	-9	84	42	42	0	0	0	84	42	42
180-Room Hotel	108	-10	85	50	35	0	0	0	85	50	35
Total	***916***	***-92***	***824***	***340***	***484***	***80***	***40***	***40***	***744***	***300***	***444***
Saturday Midday Peak Hour											
Arena (100 employees)	9	-1	8	4	4	0	0	0	8	4	4
8,820 s.f. Casual Restaurant	124	-12	112	59	53	36	18	18	76	41	35
13,060 s.f. Casual Restaurant	184	-18	166	88	78	54	27	27	112	61	51
164,657 s.f. General Office	71	-7	64	35	29	0	0	0	64	35	29
65,863 s.f. Fitness Center	183	-18	165	74	91	0	0	0	165	74	91
180-Room Hotel	130	-13	117	66	51	0	0	0	117	66	51
Total	***701***	***-69***	***632***	***326***	***306***	***90***	***45***	***45***	***542***	***281***	***261***

TABLE 10
TRIP GENERATION SUMMARY - TWO CITY CENTER

Land Use	Total Trips	Non-Auto Trips	Total Auto Trips			Pass-By Trips			New Trips		
			Total	Enter	Exit	Total	Enter	Exit	Total	Enter	Exit
Weekday A.M. Peak Hour											
8,000 s.f. Quality Restaurant	6	1	5	3	2	0	0	0	5	3	2
20,000 s.f. Retail	19	2	17	11	6	4	2	2	13	9	4
272,000 s.f. General Office	426	43	383	337	46	0	0	0	383	337	46
Total	***451***	***46***	***405***	***351***	***54***	***4***	***2***	***2***	***401***	***349***	***52***
Weekday P.M. Peak Hour											
8,000 s.f. Quality Restaurant	60	6	54	36	18	22	11	11	32	25	7
20,000 s.f. Retail	74	7	67	32	35	22	11	11	45	21	24
272,000 s.f. General Office	383	38	345	59	286	0	0	0	345	59	286
Total	***517***	***51***	***466***	***127***	***339***	***44***	***22***	***22***	***422***	***105***	***317***
Saturday Midday Peak Hour											
8,000 s.f. Quality Restaurant	87	9	78	46	32	26	13	13	52	33	19
20,000 s.f. Retail	96	10	86	45	41	22	11	11	64	34	30
272,000 s.f. General Office	117	12	105	57	48	0	0	0	105	57	48
Total	***300***	***31***	***269***	***148***	***121***	***48***	***24***	***24***	***221***	***124***	***97***

TABLE 11
TRIP GENERATION SUMMARY - THREE CITY CENTER

Land Use	Total Trips	Non-Auto Trips	Total Auto Trips			Pass-By Trips			New Trips		
			Total	Enter	Exit	Total	Enter	Exit	Total	Enter	Exit
Weekday A.M. Peak Hour											
175,000 s.f. General Office	299	30	269	237	32	0	0	0	269	237	32
Total	***299***	***30***	***269***	***237***	***32***	***0***	***0***	***0***	***269***	***237***	***32***
Weekday P.M. Peak Hour											
175,000 s.f. General Office	274	27	247	42	205	0	0	0	247	42	205
Total	***274***	***27***	***247***	***42***	***205***	***0***	***0***	***0***	***247***	***42***	***205***
Saturday Midday Peak Hour											
175,000 s.f. General Office	75	8	67	36	31	0	0	0	67	36	31
Total	***75***	***8***	***67***	***36***	***31***	***0***	***0***	***0***	***67***	***36***	***31***

TABLE 12
TRIP GENERATION SUMMARY - FOUR CITY CENTER

Land Use	Total Trips	Non-Auto Trips	Total Auto Trips			Pass-By Trips			New Trips		
			Total	Enter	Exit	Total	Enter	Exit	Total	Enter	Exit
Weekday A.M. Peak Hour											
168 Apartments	86	9	77	15	62	0	0	0	77	15	62
37,500 s.f. Retail	36	4	32	20	12	6	3	3	26	17	9
Total	***122***	***13***	***109***	***35***	***74***	***6***	***3***	***3***	***103***	***32***	***71***
Weekday P.M. Peak Hour											
168 Apartments	110	11	99	64	35	0	0	0	99	64	35
37,500 s.f. Retail	139	14	125	60	65	42	21	21	83	39	44
Total	***249***	***25***	***224***	***124***	***100***	***42***	***21***	***21***	***182***	***103***	***79***
Saturday Midday Peak Hour											
168 Apartments	88	9	79	40	39	0	0	0	79	40	39
37,500 s.f. Retail	181	18	163	85	78	42	21	21	121	64	57
Total	***269***	***27***	***242***	***125***	***117***	***42***	***21***	***21***	***200***	***104***	***96***

TRIP DISTRIBUTION

The distribution and assignment of new trips generated by the proposed development was based upon a gravity model analysis utilizing 2010 U.S. Census population data. TPD assumed that the majority of traffic would originate within a 45 minute drive of Allentown. Therefore, data for the following counties was included in the analysis: Lehigh, Northampton, Carbon, Monroe, Schuylkill, Berks, Montgomery, Bucks, Warren (NJ), and Hunterdon (NJ). Although Philadelphia is located outside of the 45-minute driving radius, TPD assumed a small number of trips would originate in Philadelphia due to the Lehigh Valley Phantoms association with the Philadelphia Flyers. For each county, the estimated population within a 45-minute drive of Allentown was multiplied by the inverse of the travel distance to determine the weighted population for use in the analysis. After determining the total traffic originating from each county, TPD assigned the traffic to nine primary routes to enter/exit downtown Allentown.

FINAL
ENVIRONMENTAL IMPACT STATEMENT

COWLITZ INDIAN TRIBE
TRUST ACQUISITION AND CASINO PROJECT

MAY 2008

Lead Agency:

U.S. Department of the Interior
Bureau of Indian Affairs
Northwest Regional Office
911 N.E. 11th Avenue
Portland, Oregon 97232

4.8 TRANSPORTATION/CIRCULATION

4.8.1 INTRODUCTION

This section identifies and discusses impacts to the transportation network anticipated under each alternative. A detailed traffic study entitled *Final Cowlitz Indian Tribe Casino Project Traffic Impact Study* was developed for the Proposed Project by Parsons Brinckerhoff Quade & Douglas, Inc. (Parsons Brinckerhoff, 2006a). This study and its associated appendices are presented as DEIS Vol. II, **Appendix T**. Further, Parsons Brinckerhoff prepared the *Cowlitz Indian Tribe Casino Project Traffic Impact Study – Supplemental Report* for the FEIS (Parsons Brinckerhoff, 2006e) (**Appendix O** of the FEIS).

METHODOLOGY

Traffic Volumes

Projected 2010 traffic volumes were derived by applying a growth factor to the historical traffic counts in the project area as described in **Section 3.8**, Transportation/Circulation. Based on historical count data obtained from the Regional Transportation Council (RTC) regional traffic counts program, growth factors of 2.45% per year for arterials and collectors and 2.0% per year for Interstate 5 (I-5) were used. The growth factors were applied to the 2005 traffic volumes collected by Parsons Brinckerhoff (2006a) to provide the *Build-out Without Project* condition. This baseline condition also assumes that the projects currently funded in the Washington Department of Transportation (WsDOT), City of Ridgefield, and Clark County transportation improvement programs (identified below) will be completed.

Trip Generation

Typically, project trip generation is derived from trip rates provided in the Institute of Transportation Engineers (ITE) Trip Generation Manual. However, because the Proposed Project and Alternatives are regional trip generators and are unique compared to other land uses in the County, a more customized approach has been developed. Relevant casino trip generation case studies in environments similar to that of the alternative project sites (i.e. rural or suburban fringe, lack of a well-established traffic circulation system, little or no fixed-route transit service, and no competing casino-resorts within 50 miles of the site) were reviewed to estimate the project trips. These case studies are limited, as the trip generation characteristics of the more common scenario, i.e. large clusters of casinos like those found in Las Vegas, are not directly transferable to the alternatives discussed in this report.

Certain characteristics, such as size, location and type of casino complex contribute to the trip generation of a proposed project. Other relevant characteristics include the number of on-site hotel rooms, the total square footage of the casino gaming-floor area, and/or the total number of employees. Additional characteristics include whether the casino has convention space, a conference or entertainment venue, retail uses such as restaurants, or lounges and convenience stores, the recreational vehicle (RV) Park, and event trips. For this analysis, the square footage of the casino gaming-floor area is used as the primary trip generation variable because of the perceived limitation that using a gaming position rate would place on the development proposals. The proposed 5,000 seat multi-purpose room and on-site hotel are also calculated into the primary trip generation rate (Parsons Brinckerhoff, 2006a) (DEIS Vol. II**, Appendix T**).

Case Studies

Empirical data collected at Tulalip Tribal Casino, Muckleshoot Indian Tribe Casino, Chinook Winds Casino, Spirit Mountain Casino and Emerald Queen Casino, coupled with seven other studies of similar casino/resorts, provided comparisons and a reasonableness check to the final trip generation calculations for the Cowlitz casino alternatives (A, B, C and E) (Parsons Brinckerhoff, 2006a, 2006c). In the following citations, the weekday PM peak-hour trip rate is included for comparison.

1. *Tulalip Tribal Casino* – Marysville, Washington (empirical trip data collected) – This site was counted on a summer peak Friday evening as well as on summer peak Saturday evenings both without and with event traffic. This casino is located within one hour of much of the Seattle/Everett metropolitan area. It has a 2,300-seat amphitheatre and restaurants/retail shops within the casino area. This site was selected due to similarities with the Cowlitz site. The resultant trip rates were 18.0 and 15.5 trips per 1,000 gross square feet for PM peak weekday and Saturday peak hour, respectively, or 0.62 weekday PM peak trips and 0.54 Saturday peak trips per gaming position.

2. *Muckleshoot Indian Tribe Casino* – Auburn, Washington (empirical trip data collected) – This site was counted on a peak Friday summer evening. While it does not have a concert/event venue nor does it have on-site lodging, it was selected for counting due to its being located within 20 miles of the Seattle and Tacoma metropolitan areas, similar to the location of the proposed Cowlitz Casino within 20 miles of the Portland/Vancouver metropolitan area. The resultant weekday PM peak rates were 10.40 trips per 1,000 gross square feet of gaming area and 0.31 trips per gaming position.

3. *Shingle Springs Rancheria Hotel-Casino Traffic Study* – Trip generation within the Shingle Springs traffic study was based on surveys of inbound/outbound traffic at five northern California Indian gaming casinos ranging in size from 17,300 square feet to 70,000 square feet during PM peak hours – 4:00-6:00 - on weekdays in October, 1988 and May, 1999. Sites included: Alturas Casino; Elk Valley; Lucky 7; Rolling Hills and Twin Pines casinos. The trip rate for the weekday PM peak hour in this study is 4.95/1,000 square feet of casino gaming floor.

4. *Gaming Casino Traffic* – Paul Box and William Bunte, ITE Journal, March 1998. Examined casino trips at two casinos located near St. Louis, MO: Casino St. Charles (2,500 gaming positions) and Casino Queen. The Casino St. Charles observed weekday PM trip rates were 0.54 trips per gaming position during the site peak (6-7 p.m.) and 0.43 trips per gaming position for the surrounding roadway system peak (4:30 to 5:30 p.m.); the Saturday peak rate was 0.64 trips per gaming position. Thus, the trip generation rate for the system peak is 80% of the trip rate for the site peak during the PM peak period. The report also concluded that between 7 and 8% of the daily total trip generation occurred during the PM peak weekday hour. The Casino Queen (East St. Louis, IL) has 1,200 gaming positions and exhibited rates of 0.57 trips per gaming position for the weekday PM peak hour.

5. *San Diego County Casino Study* – The San Diego County Department of Public Works prepared a study of casino trip generation entitled "Report on the Potential Impacts of Tribal Gaming on Northern and Eastern San Diego County." Based on surveys of numerous southern California Indian gaming casinos, the San Diego reports established that traffic for gaming casinos should assume a trip generation rate of 100 trips per 1,000 square feet of gaming floor on an average weekday (all day). The trip rate for the weekday PM peak hour is 3.93/1,000 square feet of casino gaming floor area.

6. *Jamul Indian Village Final Environmental Impact Study (FEIS)* – The "Jamul Indian Village FEIS" was referenced as it is an EIS that examined four casino alternatives for placing 101 acres into Federal trust for the Tribal Government. The preferred alternative included the development of a hotel and casino complex, events center, tribal offices and other ancillary uses on-site. For comparison to the Cowlitz proposal, Alternative D (of the Jamul project) was chosen as the most suitable, with 74,376 square feet of gaming floor and a 300 room hotel, among other similarities. The trip rate for the weekday PM peak hour is 4.94/1,000 square feet of casino gaming floor area.

7. *Gun Lake Casino Traffic Study* – This study was used because of its similarities to the Cowlitz proposal: it is located on a state highway; the character of the surrounding area is predominately tourism in a rural setting; and the casino has two restaurants (though not a hotel). The casino itself is comprised of 98,879 square feet of gaming space and includes 2,500 slot machines and 92 gaming tables. The restaurants include casual dining, buffet style, fast food and bars/lounges, plus an on-site retail component. The trip rate cited in this study is 6.81/1,000 square feet of casino gaming floor area.

8. *Enterprise Rancheria Casino-Hotel Traffic Impact Study* – This study was used because of its similarities and extensive research. The Enterprise trip generation rates were established by plotting rates for seven casinos ranging in size from 17,000 square feet to 447,600 square feet

with a best-fit curve. The resulting weekday PM peak hour trip rate cited is 3.93/1,000 square feet of casino gaming floor area.

9. *Chinook Winds Casino* – Lincoln City, Oregon (empirical trip data collected). This casino is similar in size to what is proposed under Alternatives A, B, and E (of the Cowlitz project) and includes restaurants, an adjacent hotel/motel, and an entertainment center. During the weekday PM peak-hour the two entrances were observed from 4:00-5:00 p.m. – the resulting trip rate for these observations was 4.8/1,000 square feet of casino gaming floor area.

10. *Spirit Mountain Casino* – Grand Ronde, Oregon (empirical trip data collected). During the weekday PM peak-hour the two entrances were observed from 4:00-5:00 p.m. on a peak Friday – the resulting trip rate for these observations was 6.4/1,000 square feet of casino gaming floor area for the weekday PM peak hour or 0.30 trips per gaming position.

11. *Emerald Queen Casino* – Tacoma, Washington (empirical trip data collected). During the weekday PM peak-hour the two entrances were observed from 4:00-5:00 p.m. – the resulting trip rate for these observations was 3.7/1,000 square feet of casino gaming floor area.

12. *Mohegan Sun Casino* – Traffic counts from an independent traffic audit were compiled and reviewed for comparisons to trip rates from the west coast casinos, the relationship between peak hour and daily traffic volumes, and traffic arrival characteristics on days of events at the events center. This study indicates that the weekday and Saturday peak hour trip generation rates are less than those observed for the west coast sites, but the daily trip generation rate is higher.

Analysis of the empirical data at Chinook Winds, Spirit Mountain and Emerald Queen led to the conclusion that the presence of an adjoining hotel and restaurants reduces the overall PM peak hour trip rate compared to adding the trip generation for each separate use (Parsons Brinckerhoff, 2006a). In other words, guests at the on-site hotel would patronize the casino and simply walk between the two. Guests of the casino would also tend to use the on-site restaurant and other amenities, thus generating far fewer vehicle trips.

A review of the independently-collected traffic counts provided by Mohegan Sun indicates that the Friday peak hour trip generation rate at that casino-resort may be lower than the empirical data collected for the West Coast casinos. The data also appears to indicate that the Mohegan Sun casino-resort has significantly higher daily trip generation rates than what was observed for the West Coast sites (Parsons Brinckerhoff, 2006a). To be conservative, the higher casino-only peak hour trip generation rates calculated from the West Coast casinos are used for peak hour traffic impact analysis, while the higher daily trip rate from the Mohegan Sun casino complex are used to estimate daily traffic and air quality impacts. Therefore, the following casino trip generation rates are used for this study.

- Weekday AM peak hour: 2.95 trips per 1,000 gross square feet of gaming floor area (GFA).
- Weekday system PM peak hour: 9.18 trips per 1,000 gross square feet or 0.31 trips per gaming position.
- Weekday site PM peak hour: 10.94 trips per 1,000 GFA.
- Weekday daily trips: 74.63 trips per 1,000 gross square feet or 2.54 daily trips per gaming position.
- Saturday peak hour: 15.50 trips per 1,000 gross square feet or 0.53 trips per gaming position.
- Saturday daily trips: 93.24 trips per 1,000 gross square feet or 3.24 daily trips per gaming position.

Hotel Trips

The Shingle Springs Draft Environmental Impact Report (DEIR) found that when a hotel is part of a casino-hotel complex, the hotel portion of the project would generate 2.06 trips per room on an average weekday. The ITE Trip Generation Manual shows that a standard hotel (land-use #310) will generate 8.23 trips per room on an average weekday. Thus, the Shingle Springs casino study found that a hotel at a casino (in a semi-rural environment) will generate 25% of the trips a stand-alone hotel would generate on an average weekday. The reduced number accounts for those who stay at the hotel and walk, rather than drive, to the associated casino and other amenities. Observations at the other sites for which empirical data were collected corroborate this. Therefore, a 75% reduction in trip generation for the hotel portion of the Cowlitz casino project could be assumed. However, further investigation indicated that there is potential for the hotel to attract pass-by (transient lodging) trips off of I-5 that are not casino-destination trips, due to lack of other hotels in the area and growth in the La Center area. Thus, a 50% trip reduction for trip internalization is assumed instead of a 75% reduction (Parsons Brinckerhoff, 2006e).

Multi-Purpose Event Center

A Multi-purpose room with seating for 5,000 people is a component of all the gaming alternatives (A, B, C and E) for the Proposed Project. Approximately 20 to 30 events would occur on an annual basis (approximately one large event every three weeks) in the event center that will have the potential of filling most of the seats.

In accordance with the study methodology approved by Clark County, the City of Ridgefield and WsDOT, the PM peak weekday, and Saturday peak hour trip generation rates include an “85th percentile event” at the Multi-purpose room, which is consistent with the assumptions used for The Amphitheatre at Clark County. An 85th percentile event has a higher attendance than 85% of the events and a lower attendance than 15% of the events. Using The Amphitheatre at Clark County as an example, their 85th percentile event in 2005 drew 8,400 people, or close to 85% of the highest attended event. Thus for Alternatives A, B, C, and E, an 85th percentile event would be an event that fills 4,250 seats. It is assumed that for each of the 20 to 30 events per year, 15% will have a higher attendance and 85% will have a lower attendance.

Using the report *Mode Split at Large Special Events* prepared by Charles Green for the Transportation Research Board in 1991, a weekday PM peak event would experience average auto occupancy of 2.62. Based on traffic observations for the Mohegan Sun events center, auto occupancies range from 1.8 to 2.2 persons per vehicle. Therefore, to be conservative for this analysis, a low-end average auto occupancy of 1.8 persons per vehicle was used (Parsons Brinckerhoff, 2006a). Thus, during an 85th percentile event, 4,250 event-goers will arrive in approximately 2,400 vehicles.

Traffic counts were collected by an independent auditor at the Mohegan Sun casino-resort on event and non-event days for weekdays, Fridays, and weekend days. The result of this analysis indicates that the presence of the casino/hotel, restaurant, and entertainment facilities affects arrivals and departures on event days, and is also measurably different than arrival/departure characteristics for a stand-alone facility such as an amphitheatre or an arena (Parsons Brinckerhoff, 2006a). Thus, instead of almost 50% of vehicles arriving in the 1-2 hour period prior to an event (during the transportation system's peak hour), such as what has been observed at The Amphitheatre at Clark County, the Mohegan Sun experiences significantly less event-related traffic impacts during the weekday PM peak hour.

For events at the La Center Interchange or Ridgefield Interchange sites, weekday and Saturday evening events will likely have 8:00 p.m. starting times, compared with 7:00 p.m. or 7:30 p.m. starting times for events at other entertainment venues in the Portland/Vancouver area. The later starting time is due to the desire to encourage attendees to take advantage of other offerings at the casino-resort, including the casino, restaurant, and hotel. The later starting time has a secondary implication: the number of vehicles arriving to an event during the 4:45-5:45 p.m. weekday transportation system peak hour is less than what would occur for an earlier-starting event. Based on an 8:00 p.m. event start time (consistent with the Mohegan Sun events center), approximately 8% of those traveling to an event at the project site would arrive during the local transportation system's peak hour (roughly 4:45 to 5:45 p.m.). A peak of approximately 19% of arrivals would occur during the 6 p.m. to 7 p.m. hour, which is after the system's weekday peak. To be conservative for this analysis, a 19% peak hour factor was used for the traffic impact analysis.

Further detail regarding the Mohegan Sun counts and the calculations that derived the traffic numbers shown in this report are found in Appendix A to the Final Traffic Impact Study (Parsons Brinckerhoff, 2006a) (DEIS Vol. II, **Appendix T**).

Data collected at the Tulalip Casino site indicates that approximately 42% of the event-goers arrive in the one-hour period prior to the start of the event, or 6:30 to 7:30 p.m. For the purposes of this analysis they are assumed to arrive at the site between 6:00 and 7:00 p.m., although many will arrive much later for an event that starts at 8:00 p.m. Using event-day counts taken by the Mohegan Sun Casino as well as the Mode Split at Large Special Events paper, approximately one-third or 33% of the attendees will arrive at the transportation system PM peak hour of 5:00 to 6:00 p.m.

Other

Another conservative assumption was that no trip reduction would be taken for "pass-by" trips, which are those people already traveling on the roadway system that decide to deviate from their travel path to the casino site. Checking 24-hour traffic counts by hour in the area of the I-5/La Center interchange (ramp counts as well as La Center Road counts and also in Ridgefield), the 6:00 to 7:00 p.m. time period on weekdays carries approximately 75% of the 5:00 to 6:00 p.m. peak hour traffic volumes. For a sensitivity analysis, two Year 2010 PM peak scenarios were analyzed for the I-5/La Center interchange area to determine the "worst case" scenario to be analyzed in this report:

- System PM Peak Hour: The 5:00 to 6:00 p.m. period, using peak hour traffic projections for the system plus the 5:00 to 6:00 p.m. trip generation estimates for Alternative A/B.
- Site Peak Hour: The 6:00 to 7:00 p.m. time period, using the site's peak trip generation estimates plus 75% of the road system peak hour volumes.

Trip Distribution and Assignment

The RTC travel demand model does not provide adequate trip distribution data due to the uniqueness of the proposed use. Thus, for the casino alternatives, a special trip distribution methodology was used. Based on investigating studies conducted elsewhere, casino and event–related trip distribution is related to:

- The amount of competing gaming: The Lucky Eagle Casino in Rochester, Washington (approximately 90 miles from the Cowlitz site), and the Spirit Mountain Casino in Grande Ronde, Oregon (approximately 60 miles from the Cowlitz site) would likely compete for the gaming customers as well as concert-goers, since both sites offer entertainment (the Spirit Mountain Casino concert hall hosts concerts similar to the Cowlitz site). While there are as many as two other casinos being discussed or studied in the Portland metropolitan area, for this study they were not considered as being open; otherwise, they would compete with the Cowlitz site and the number of casino trips would be less than under our assumption.
- Time and distance: The Cowlitz site is a regional "one of a kind" generator, and as such, with the lack of accessible, competing uses, will attract trips from many locations in northwestern Oregon and southwestern Washington. Our investigation indicates that time and distance affect the time of the trip (Portland residents may leave for the casino at a later time to avoid peak hour traffic congestion, but they will still make the trip) more than they affect the

decision to make the trip. Thus, the trip distribution model has a peak weighting factor for travel time ($20 per hour) and distance (40¢ per mile), similar to a gravity model, but all geographic areas assumed to be in the Cowlitz trip draw basin are factored into the model. Travel speeds for Portland were taken from the Portland State University congestion study.

- Population: As a one-of-a-kind generator, the Cowlitz casino will draw from a large population base, and the number of trips from a geographic area will be directly related to the number of people living in that area. Work by EcoNorthwest for casinos in Oregon confirm the large, geographically-dispersed draw of the Oregon casinos.

Vehicle distribution will likely be more reflective of the general population densities of Cowlitz, Clark and Skamania counties in Washington and the greater Portland metropolitan area in Oregon. Given that competing casino uses exist in Rochester, Washington and near Grand Ronde, Oregon, few trips are expected to be attracted from outside of the southwest Washington and Portland metropolitan areas. This is consistent with the findings of the Gaming Market Assessment, which predicts that 91% of visitors will come from within the Portland-Vancouver metro area (The Innovation Group, 2006 in Parsons Brinckerhoff, 2006a).

Trip distribution for the gaming alternatives is based on the population of the surrounding areas. Trips to and from the north of the alternative project sites will travel from the City of Woodland and Cowlitz County, as well as some trips from Columbia County, Oregon. Trips to the Pekin Ferry area cannot exit to points north, east, or west because of the Columbia River; thus, there would likely be only a small percentage traveling in that direction. Trips to the east would travel to La Center, Amboy, Yacolt, and northeast Clark County.

A separate distribution percentage is proposed for trips that travel from Ridgefield, Duluth (NE 10th Avenue at NE 219th Street), Battle Ground, and other rural areas within three miles of the alternative project sites. The remainder of the trips will travel from south of the (State Route) SR-501/Pioneer Interchange, from southern Clark County, Skamania County, and the tri-county Portland area.

These adjustments slightly increased the distribution percentages of the project traffic, or concentration of population in the northern part of the study area, as well as the percentage in the Ridgefield/Central County area; they slightly reduced the percentage in the southern Clark County/Portland area.

Access points as shown on the alternative site plans were also considered in assigning project trips. Additionally, trips were assigned to each project driveway based on the number of parking spaces (structured, surface or valet) and project component that could be accessed via each driveway.

INTERNAL CAPTURE WORKSHEETS

*

Analyst ANH
Date 8/5/2021

MULTI-USE DEVELOPMENT TRIP GENERATION AND INTERNAL CAPTURE SUMMARY

Name of Dvlpt BROKEN BOW
Time Period FRI AM

W/o Event

LAND USE A CASINO

ITE LU Code –
Size 36,000 SF

	Total	Internal	External
Enter	74	24	50
Exit	32	11	21
Total	106	35	71
%	100%	33%	67%

Exit to External
Enter from External

Land Use A to/from Land Use B:

33% 11 Demand	33% 24 Demand	
11 Balanced	24 Balanced	
75% 44 Demand	75% 38 Demand	

Land Use A to/from Land Use C:

% Demand	% Demand
Balanced	Balanced
% Demand	% Demand

LAND USE B Hotel

ITE LU Code 310
Size 200 Rooms

	Total	Internal	External
Enter	59	11	48
Exit	51	24	27
Total	110	35	75
%	100%	32%	68%

Exit to External
Enter from External

Land Use B to/from Land Use C:

Demand %	Balanced	Demand %
% Demand	Balanced	% Demand

LAND USE C

ITE LU Code
Size

	Total	Internal	External
Enter			
Exit			
Total			
%			

Enter from External
Exit to External

Net External Trips for Multi-Use Development

	LAND USE A	LAND USE B	LAND USE C	TOTAL
Enter	50	48		98
Exit	21	27		48
Total	71	75		146
Single-Use Trip Gen. Est.	106	110		216

Source: Kaku Associates, Inc.

INTERNAL CAPTURE 32.4%

$(35+35)/216 = 32.4\%$

1

Analyst ANH
Date 8/[illegible]/2021

MULTI-USE DEVELOPMENT TRIP GENERATION AND INTERNAL CAPTURE SUMMARY

Name of Dvlpt Broken Bow
Time Period FRI PM

LAND USE A Casino

ITE LU Code —
Size 36,000 SF

	Total	Internal	External
Enter	175	38	137
Exit	155	51	104
Total	330	89	241
%	100%	27%	73%

Exit to External
Enter from External

73% 51 Demand
51 Balanced
75% 53 Demand

53% 58 Demand
38 Balanced
75% 38 Demand

% Demand
Balanced
% Demand

% Demand
Balanced
% Demand

LAND USE B Hotel

ITE LU Code 310
Size 200 Rooms

	Total	Internal	External
Enter	70	51	19
Exit	50	38	12
Total	120	89	31
%	100%	74%	26%

Exit to External
Enter from External

Demand % | Balanced | Demand %

% Demand | Balanced | % Demand

LAND USE C

ITE LU Code
Size

	Total	Internal	External
Enter			
Exit			
Total			
%			

Enter from External
Exit to External

Net External Trips for Multi-Use Development

	LAND USE A	LAND USE B	LAND USE C	TOTAL
Enter	137	19		156
Exit	104	12		116
Total	241	31		272
Single-Use Trip Gen. Est.	330	120		450

Source: Kaku Associates, Inc.

INTERNAL CAPTURE
39.6%

$(84+89)/450 = 39.6\%$

Analyst ANH
Date 8/11/2021

MULTI-USE DEVELOPMENT TRIP GENERATION AND INTERNAL CAPTURE SUMMARY

Name of Dvlpt Broken Bow
Time Period SAT PEAK

W/o Event

LAND USE A Casino

ITE LU Code —
Size 36,000 SF

	Total	Internal	External
Enter	346	47	299
Exit	212	61	151
Total	558	108	450
%	100%	19%	81%

Exit to External
Enter from External

A ↔ B:

Demand	Balanced	Demand
33% 70	61	75% 61
33% 114	47	75% 47

A ↔ C:

Demand	Balanced	Demand
%		%
%		%

LAND USE B Hotel

ITE LU Code 310
Size 200 Rooms

	Total	Internal	External
Enter	81	61	20
Exit	63	47	16
Total	144	108	36
%	100%	75%	25%

Exit to External
Enter from External

B ↔ C:

Demand	Balanced	Demand
%		%
%		%

LAND USE C

ITE LU Code
Size

	Total	Internal	External
Enter			
Exit			
Total			
%			

Enter from External
Exit to External

Net External Trips for Multi-Use Development

	LAND USE A	LAND USE B	LAND USE C	TOTAL
Enter	299	20		319
Exit	151	16		167
Total	450	36		486
Single-Use Trip Gen. Est.	558	144		702

Source: Kaku Associates, Inc.

INTERNAL CAPTURE 30.8%

$(108+108)/702 = 30.8\%$

3

Analyst ANH
Date 8/10/2021

MULTI-USE DEVELOPMENT TRIP GENERATION AND INTERNAL CAPTURE SUMMARY

Name of Dvlpt Broken Bow
Time Period FRi PM

W/ Event

LAND USE A Casino

ITE LU Code —
Size 36,000 SF

	Total	Internal	External
Enter	175	80	95
Exit	155	102	53
Total	330	182	149
%	100%	55%	45%

Exit to External
Enter from External

Between Land Use A and Land Use B:
- 33% 51 Demand; 51 Balanced; 75% 53 Demand
- 33% 58 Demand; 38 Balanced; 75% 38 Demand

Between Land Use A and Land Use C:
- 33% 51 Demand; 51 Balanced; 75% 46 Demand
- 33% 58 Demand; 42 Balanced; 75% 42 Demand

LAND USE B Hotel

ITE LU Code 310
Size 200 Rooms

	Total	Internal	External
Enter	70	52	18
Exit	50	38	12
Total	120	90	30
%	100%	75%	25%

Exit to External
Enter from External

Between Land Use B and Land Use C:
- Demand 1% 1; Balanced 1; Demand 2% 1
- 0% 0 Demand; 0 Balanced; 0% 0 Demand

LAND USE C Outdoor Entertainment/venue

ITE LU Code —
Size 2,500 max cap.

	Total	Internal	External
Enter	555	51	504
Exit	56	43	13
Total	~~450~~	94	517
%	100%	15%	84%

Enter from External
Exit to External

Net External Trips for Multi-Use Development

	LAND USE A	LAND USE B	LAND USE C	TOTAL
Enter	95	18	504	617
Exit	53	12	13	78
Total	148	30	517	695
Single-Use Trip Gen. Est.	330	120	611	1061

Source: Kaku Associates, Inc.

INTERNAL CAPTURE
34.5%

$(90+182+94)/1061$

4

Analyst ANH
Date 8/11/2021

MULTI-USE DEVELOPMENT TRIP GENERATION AND INTERNAL CAPTURE SUMMARY

Name of Dvlpt Broken Bow
Time Period ~~FRI~~ SAT PEAK

W/ Event

LAND USE A Casino

ITE LU Code —
Size 361,000 SF

	Total	Internal	External
Enter	346	~~109~~ 89	257
Exit	212	131	81
Total	558	220	338
%	100%	39%	61%

Exit to External
Enter from External

Between Land Use A and Land Use B:
33% 70 Demand
61 Balanced
75% 61 Demand
33% 114 Demand
47 Balanced
75% ~~X~~ 47 Demand

Between Land Use A and Land Use C:
33% 70 Demand
70 Balanced
75% 416 Demand
33% 114 Demand
42 Balanced
75% 42 Demand

LAND USE B Hotel

ITE LU Code 310
Size 200 Rooms

	Total	Internal	External
Enter	81	62	19
Exit	63	47	16
Total	144	109	35
%	100%	76%	24%

Exit to External
Enter from External

Between Land Use B and Land Use C:
Demand 1% 1
Balanced 1
two% Demand 2% 1
0% — Demand
— Balanced
0% — Demand

LAND USE C Outdoor Entertainment

ITE LU Code —
Size 2,500 max-Cap

	Total	Internal	External
Enter	555	70	485
Exit	56	42	98
Total	611	112	499
%	100%	18%	82%

Enter from External
Exit to External

Net External Trips for Multi-Use Development

	LAND USE A	LAND USE B	LAND USE C	TOTAL
Enter	257	62	485	804
Exit	81	47	98	226
Total	338	109	499	946
Single-Use Trip Gen. Est.	558	144	611	1313

Source: Kaku Associates, Inc.

INTERNAL CAPTURE 33.6%

$(109+220+112)/1313 = 33.6\%$

5

SYNCHRO WORKSHEETS

Intersection	
Int Delay, s/veh	3.7

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	42	113	440	47	121	511
Future Vol, veh/h	42	113	440	47	121	511
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	100	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	0	-	-	0
Peak Hour Factor	97	97	97	97	97	97
Heavy Vehicles, %	2	2	8	2	2	4
Mvmt Flow	43	116	454	48	125	527

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	1255	478	0	0	502	0
Stage 1	478	-	-	-	-	-
Stage 2	777	-	-	-	-	-
Critical Hdwy	6.42	6.22	-	-	4.12	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	-	-	2.218	-
Pot Cap-1 Maneuver	189	587	-	-	1062	-
Stage 1	624	-	-	-	-	-
Stage 2	453	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	167	587	-	-	1062	-
Mov Cap-2 Maneuver	167	-	-	-	-	-
Stage 1	624	-	-	-	-	-
Stage 2	400	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	23.7	0	1.7
HCM LOS	C		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	349	1062	-
HCM Lane V/C Ratio	-	-	0.458	0.117	-
HCM Control Delay (s)	-	-	23.7	8.8	-
HCM Lane LOS	-	-	C	A	-
HCM 95th %tile Q(veh)	-	-	2.3	0.4	-

Intersection	
Int Delay, s/veh	11.4

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	69	133	687	58	92	467
Future Vol, veh/h	69	133	687	58	92	467
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	100	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	0	-	-	0
Peak Hour Factor	95	95	95	95	95	95
Heavy Vehicles, %	2	2	2	3	2	2
Mvmt Flow	73	140	723	61	97	492

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	1440	754	0	0	784	0
Stage 1	754	-	-	-	-	-
Stage 2	686	-	-	-	-	-
Critical Hdwy	6.42	6.22	-	-	4.12	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	-	-	2.218	-
Pot Cap-1 Maneuver	146	409	-	-	834	-
Stage 1	465	-	-	-	-	-
Stage 2	500	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	129	409	-	-	834	-
Mov Cap-2 Maneuver	129	-	-	-	-	-
Stage 1	465	-	-	-	-	-
Stage 2	442	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	80.7	0	1.6
HCM LOS	F		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	235	834	-
HCM Lane V/C Ratio	-	-	0.905	0.116	-
HCM Control Delay (s)	-	-	80.7	9.9	-
HCM Lane LOS	-	-	F	A	-
HCM 95th %tile Q(veh)	-	-	7.6	0.4	-

Intersection	
Int Delay, s/veh	13.2

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	66	205	439	66	178	455
Future Vol, veh/h	66	205	439	66	178	455
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	100	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	0	-	-	0
Peak Hour Factor	95	95	95	95	95	95
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	69	216	462	69	187	479

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	1350	497	0	0	531	0
Stage 1	497	-	-	-	-	-
Stage 2	853	-	-	-	-	-
Critical Hdwy	6.42	6.22	-	-	4.12	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	-	-	2.218	-
Pot Cap-1 Maneuver	166	573	-	-	1036	-
Stage 1	611	-	-	-	-	-
Stage 2	418	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	136	573	-	-	1036	-
Mov Cap-2 Maneuver	136	-	-	-	-	-
Stage 1	611	-	-	-	-	-
Stage 2	342	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	62.7	0	2.6
HCM LOS	F		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	321	1036	-
HCM Lane V/C Ratio	-	-	0.889	0.181	-
HCM Control Delay (s)	-	-	62.7	9.2	-
HCM Lane LOS	-	-	F	A	-
HCM 95th %tile Q(veh)	-	-	8.3	0.7	-

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	51	137	532	57	146	618
Future Volume (veh/h)	51	137	532	57	146	618
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1781	1870	1870	1841
Adj Flow Rate, veh/h	53	141	548	59	151	637
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	2	8	2	2	4
Cap, veh/h	66	176	620	67	190	1134
Arrive On Green	0.15	0.15	0.39	0.39	0.11	0.62
Sat Flow, veh/h	444	1182	1581	170	1781	1841
Grp Volume(v), veh/h	195	0	0	607	151	637
Grp Sat Flow(s),veh/h/ln	1635	0	0	1751	1781	1841
Q Serve(g_s), s	6.9	0.0	0.0	19.2	4.9	12.1
Cycle Q Clear(g_c), s	6.9	0.0	0.0	19.2	4.9	12.1
Prop In Lane	0.27	0.72		0.10	1.00	
Lane Grp Cap(c), veh/h	243	0	0	686	190	1134
V/C Ratio(X)	0.80	0.00	0.00	0.88	0.79	0.56
Avail Cap(c_a), veh/h	494	0	0	853	209	1329
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	24.5	0.0	0.0	16.8	26.0	6.7
Incr Delay (d2), s/veh	6.1	0.0	0.0	9.3	17.3	0.4
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.6	0.0	0.0	7.4	2.7	2.4
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	30.6	0.0	0.0	26.2	43.2	7.1
LnGrp LOS	C	A	A	C	D	A
Approach Vol, veh/h	195		607			788
Approach Delay, s/veh	30.6		26.2			14.1
Approach LOS	C		C			B

Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	13.4	30.3				43.7		15.8
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	7.0	29.0				43.0		18.0
Max Q Clear Time (g_c+I1), s	6.9	21.2				14.1		8.9
Green Ext Time (p_c), s	0.0	2.1				3.8		0.4

Intersection Summary	
HCM 6th Ctrl Delay	20.7
HCM 6th LOS	C

Notes

User approved volume balancing among the lanes for turning movement.

Movement	WBL	WBR	NBT	NBR	SBL	SBT	
Lane Configurations							
Traffic Volume (veh/h)	83	161	831	70	111	565	
Future Volume (veh/h)	83	161	831	70	111	565	
Initial Q (Qb), veh	0	0	0	0	0	0	
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00		
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	
Work Zone On Approach	No		No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1856	1870	1870	
Adj Flow Rate, veh/h	87	169	875	74	117	595	
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	
Percent Heavy Veh, %	2	2	2	3	2	2	
Cap, veh/h	99	193	890	75	192	1233	
Arrive On Green	0.18	0.18	0.52	0.52	0.05	0.66	
Sat Flow, veh/h	558	1083	1701	144	1781	1870	
Grp Volume(v), veh/h	257	0	0	949	117	595	
Grp Sat Flow(s),veh/h/ln	1647	0	0	1844	1781	1870	
Q Serve(g_s), s	13.1	0.0	0.0	43.5	2.4	13.7	
Cycle Q Clear(g_c), s	13.1	0.0	0.0	43.5	2.4	13.7	
Prop In Lane	0.34	0.66		0.08	1.00		
Lane Grp Cap(c), veh/h	293	0	0	965	192	1233	
V/C Ratio(X)	0.88	0.00	0.00	0.98	0.61	0.48	
Avail Cap(c_a), veh/h	364	0	0	965	198	1240	
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	
Upstream Filter(I)	1.00	0.00	0.00	1.00	1.00	1.00	
Uniform Delay (d), s/veh	34.4	0.0	0.0	20.1	20.1	7.3	
Incr Delay (d2), s/veh	17.8	0.0	0.0	24.9	5.1	0.3	
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	
%ile BackOfQ(50%),veh/ln	6.2	0.0	0.0	21.1	1.3	3.7	
Unsig. Movement Delay, s/veh							
LnGrp Delay(d),s/veh	52.2	0.0	0.0	45.0	25.2	7.6	
LnGrp LOS	D	A	A	D	C	A	
Approach Vol, veh/h	257		949			712	
Approach Delay, s/veh	52.2		45.0			10.5	
Approach LOS	D		D			B	
Timer - Assigned Phs	1	2				6	8
Phs Duration (G+Y+Rc), s	11.7	52.0				63.7	22.3
Change Period (Y+Rc), s	7.0	7.0				7.0	7.0
Max Green Setting (Gmax), s	5.0	45.0				57.0	19.0
Max Q Clear Time (g_c+I1), s	4.4	45.5				15.7	15.1
Green Ext Time (p_c), s	0.0	0.0				3.6	0.3
Intersection Summary							
HCM 6th Ctrl Delay			33.2				
HCM 6th LOS			C				
Notes							

User approved volume balancing among the lanes for turning movement.

Movement	WBL	WBR	NBT	NBR	SBL	SBT	
Lane Configurations							
Traffic Volume (veh/h)	80	248	531	80	215	551	
Future Volume (veh/h)	80	248	531	80	215	551	
Initial Q (Qb), veh	0	0	0	0	0	0	
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00		
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	
Work Zone On Approach	No		No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	
Adj Flow Rate, veh/h	84	261	559	84	226	580	
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	
Percent Heavy Veh, %	2	2	2	2	2	2	
Cap, veh/h	93	290	609	92	290	1060	
Arrive On Green	0.24	0.24	0.38	0.38	0.08	0.57	
Sat Flow, veh/h	396	1229	1589	239	1781	1870	
Grp Volume(v), veh/h	346	0	0	643	226	580	
Grp Sat Flow(s),veh/h/ln	1629	0	0	1827	1781	1870	
Q Serve(g_s), s	14.6	0.0	0.0	23.8	5.2	13.8	
Cycle Q Clear(g_c), s	14.6	0.0	0.0	23.8	5.2	13.8	
Prop In Lane	0.24	0.75		0.13	1.00		
Lane Grp Cap(c), veh/h	385	0	0	701	290	1060	
V/C Ratio(X)	0.90	0.00	0.00	0.92	0.78	0.55	
Avail Cap(c_a), veh/h	413	0	0	772	290	1133	
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	
Upstream Filter(I)	1.00	0.00	0.00	1.00	1.00	1.00	
Uniform Delay (d), s/veh	26.3	0.0	0.0	20.8	16.0	9.7	
Incr Delay (d2), s/veh	21.1	0.0	0.0	15.0	12.6	0.5	
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	
%ile BackOfQ(50%),veh/ln	7.1	0.0	0.0	11.0	2.5	4.0	
Unsig. Movement Delay, s/veh							
LnGrp Delay(d),s/veh	47.4	0.0	0.0	35.8	28.7	10.1	
LnGrp LOS	D	A	A	D	C	B	
Approach Vol, veh/h	346		643			806	
Approach Delay, s/veh	47.4		35.8			15.3	
Approach LOS	D		D			B	
Timer - Assigned Phs	1	2				6	8
Phs Duration (G+Y+Rc), s	13.0	34.2				47.2	23.8
Change Period (Y+Rc), s	7.0	7.0				7.0	7.0
Max Green Setting (Gmax), s	6.0	30.0				43.0	18.0
Max Q Clear Time (g_c+I1), s	7.2	25.8				15.8	16.6
Green Ext Time (p_c), s	0.0	1.5				3.3	0.2

Intersection Summary		
HCM 6th Ctrl Delay		28.9
HCM 6th LOS		C

Notes

User approved volume balancing among the lanes for turning movement.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	64	163	558	66	183	654
Future Volume (veh/h)	64	163	558	66	183	654
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1781	1870	1870	1841
Adj Flow Rate, veh/h	66	168	575	68	189	674
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	2	8	2	2	4
Cap, veh/h	59	150	777	92	620	1236
Arrive On Green	0.13	0.13	0.99	0.99	0.07	0.67
Sat Flow, veh/h	460	1170	1563	185	1781	1841
Grp Volume(v), veh/h	235	0	0	643	189	674
Grp Sat Flow(s),veh/h/ln	1637	0	0	1748	1781	1841
Q Serve(g_s), s	9.0	0.0	0.0	0.6	3.3	13.3
Cycle Q Clear(g_c), s	9.0	0.0	0.0	0.6	3.3	13.3
Prop In Lane	0.28	0.71		0.11	1.00	
Lane Grp Cap(c), veh/h	210	0	0	869	620	1236
V/C Ratio(X)	1.12	0.00	0.00	0.74	0.30	0.55
Avail Cap(c_a), veh/h	210	0	0	869	640	1236
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.88	1.00	1.00
Uniform Delay (d), s/veh	30.5	0.0	0.0	0.1	6.3	6.0
Incr Delay (d2), s/veh	96.9	0.0	0.0	5.0	0.3	1.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	8.8	0.0	0.0	1.2	0.8	3.1
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	127.4	0.0	0.0	5.1	6.6	7.7
LnGrp LOS	F	A	A	A	A	A
Approach Vol, veh/h	235		643			863
Approach Delay, s/veh	127.4		5.1			7.5
Approach LOS	F		A			A

Timer - Assigned Phs	1	2	6	8
Phs Duration (G+Y+Rc), s	12.2	41.8	54.0	16.0
Change Period (Y+Rc), s	7.0	7.0	7.0	7.0
Max Green Setting (Gmax), s	6.0	34.0	47.0	9.0
Max Q Clear Time (g_c+I1), s	5.3	2.6	15.3	11.0
Green Ext Time (p_c), s	0.0	4.0	4.2	0.0

Intersection Summary	
HCM 6th Ctrl Delay	22.8
HCM 6th LOS	C

Notes

User approved volume balancing among the lanes for turning movement.

Intersection	
Int Delay, s/veh	1.8

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	203	31	25	199	38	26
Future Vol, veh/h	203	31	25	199	38	26
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	221	34	27	216	41	28

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	255	0	508	238
Stage 1	-	-	-	-	238	-
Stage 2	-	-	-	-	270	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1310	-	525	801
Stage 1	-	-	-	-	802	-
Stage 2	-	-	-	-	775	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1310	-	513	801
Mov Cap-2 Maneuver	-	-	-	-	513	-
Stage 1	-	-	-	-	802	-
Stage 2	-	-	-	-	757	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.9	11.4
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	513	801	-	-	1310	-
HCM Lane V/C Ratio	0.081	0.035	-	-	0.021	-
HCM Control Delay (s)	12.6	9.7	-	-	7.8	0
HCM Lane LOS	B	A	-	-	A	A
HCM 95th %tile Q(veh)	0.3	0.1	-	-	0.1	-

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	38	26	598	64	36	682
Future Volume (veh/h)	38	26	598	64	36	682
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	41	28	650	0	39	741
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	94	84	741		586	1398
Arrive On Green	0.05	0.05	0.40	0.00	0.50	1.00
Sat Flow, veh/h	1781	1585	1870	1585	1781	1870
Grp Volume(v), veh/h	41	28	650	0	39	741
Grp Sat Flow(s),veh/h/ln	1781	1585	1870	1585	1781	1870
Q Serve(g_s), s	1.6	1.2	22.5	0.0	0.0	0.0
Cycle Q Clear(g_c), s	1.6	1.2	22.5	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	94	84	741		586	1398
V/C Ratio(X)	0.44	0.33	0.88		0.07	0.53
Avail Cap(c_a), veh/h	127	113	1042		586	1398
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	1.00	1.00	0.00	0.78	0.78
Uniform Delay (d), s/veh	32.1	32.0	19.6	0.0	11.9	0.0
Incr Delay (d2), s/veh	3.2	2.3	13.9	0.0	0.0	1.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.7	0.5	10.6	0.0	0.3	0.4
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	35.3	34.3	33.5	0.0	11.9	1.1
LnGrp LOS	D	C	C		B	A
Approach Vol, veh/h	69		650	A		780
Approach Delay, s/veh	34.9		33.5			1.7
Approach LOS	C		C			A

Timer - Assigned Phs	1	2	6	8
Phs Duration (G+Y+Rc), s	24.6	34.7	59.3	10.7
Change Period (Y+Rc), s	7.0	7.0	7.0	7.0
Max Green Setting (Gmax), s	5.0	39.0	51.0	5.0
Max Q Clear Time (g_c+I1), s	2.0	24.5	2.0	3.6
Green Ext Time (p_c), s	0.0	3.2	6.5	0.0

Intersection Summary	
HCM 6th Ctrl Delay	17.0
HCM 6th LOS	B

Notes

Unsignalized Delay for [NBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	108	305	588	102	302	638
Future Volume (veh/h)	108	305	588	102	302	638
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	114	321	619	107	318	672
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	85	240	646	112	369	1169
Arrive On Green	0.20	0.20	0.83	0.83	0.12	0.63
Sat Flow, veh/h	427	1202	1553	269	1781	1870
Grp Volume(v), veh/h	436	0	0	726	318	672
Grp Sat Flow(s),veh/h/ln	1633	0	0	1822	1781	1870
Q Serve(g_s), s	16.0	0.0	0.0	26.5	7.6	16.8
Cycle Q Clear(g_c), s	16.0	0.0	0.0	26.5	7.6	16.8
Prop In Lane	0.26	0.74		0.15	1.00	
Lane Grp Cap(c), veh/h	327	0	0	757	369	1169
V/C Ratio(X)	1.34	0.00	0.00	0.96	0.86	0.57
Avail Cap(c_a), veh/h	327	0	0	757	397	1169
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.82	1.00	1.00
Uniform Delay (d), s/veh	32.0	0.0	0.0	6.2	16.4	8.8
Incr Delay (d2), s/veh	170.3	0.0	0.0	21.1	16.6	2.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	21.1	0.0	0.0	6.6	3.9	5.2
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	202.3	0.0	0.0	27.3	33.0	10.8
LnGrp LOS	F	A	A	C	C	B
Approach Vol, veh/h	436		726			990
Approach Delay, s/veh	202.3		27.3			18.0
Approach LOS	F		C			B

Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	16.7	40.3				57.0		23.0
Change Period (Y+Rc), s	7.0	* 7				7.0		7.0
Max Green Setting (Gmax), s	11.0	* 33				50.0		16.0
Max Q Clear Time (g_c+I1), s	9.6	28.5				18.8		18.0
Green Ext Time (p_c), s	0.1	1.8				4.2		0.0

Intersection Summary			
HCM 6th Ctrl Delay			58.5
HCM 6th LOS			E

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection						
Int Delay, s/veh	3.3					

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	295	74	61	354	85	57
Future Vol, veh/h	295	74	61	354	85	57
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	321	80	66	385	92	62

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	401	0	878	361
Stage 1	-	-	-	-	361	-
Stage 2	-	-	-	-	517	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1158	-	318	684
Stage 1	-	-	-	-	705	-
Stage 2	-	-	-	-	598	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1158	-	295	684
Mov Cap-2 Maneuver	-	-	-	-	295	-
Stage 1	-	-	-	-	705	-
Stage 2	-	-	-	-	555	-

Approach	EB	WB	NB
HCM Control Delay, s	0	1.2	17.9
HCM LOS			C

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	295	684	-	-	1158	-
HCM Lane V/C Ratio	0.313	0.091	-	-	0.057	-
HCM Control Delay (s)	22.7	10.8	-	-	8.3	0
HCM Lane LOS	C	B	-	-	A	A
HCM 95th %tile Q(veh)	1.3	0.3	-	-	0.2	-

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	85	57	633	152	87	658
Future Volume (veh/h)	85	57	633	152	87	658
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	92	62	688	0	95	715
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	132	118	770		571	1404
Arrive On Green	0.07	0.07	0.41	0.00	0.50	1.00
Sat Flow, veh/h	1781	1585	1870	1585	1781	1870
Grp Volume(v), veh/h	92	62	688	0	95	715
Grp Sat Flow(s),veh/h/ln	1781	1585	1870	1585	1781	1870
Q Serve(g_s), s	4.0	3.0	27.4	0.0	0.0	0.0
Cycle Q Clear(g_c), s	4.0	3.0	27.4	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	132	118	770		571	1404
V/C Ratio(X)	0.70	0.53	0.89		0.17	0.51
Avail Cap(c_a), veh/h	200	178	1052		571	1404
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	1.00	1.00	0.00	0.61	0.61
Uniform Delay (d), s/veh	36.1	35.7	21.9	0.0	14.3	0.0
Incr Delay (d2), s/veh	6.4	3.6	14.9	0.0	0.1	0.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.0	1.3	13.0	0.0	0.9	0.3
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	42.6	39.3	36.8	0.0	14.3	0.8
LnGrp LOS	D	D	D		B	A
Approach Vol, veh/h	154		688	A		810
Approach Delay, s/veh	41.2		36.8			2.4
Approach LOS	D		D			A

Timer - Assigned Phs	1	2	6	8
Phs Duration (G+Y+Rc), s	27.1	40.0	67.1	12.9
Change Period (Y+Rc), s	7.0	7.0	7.0	7.0
Max Green Setting (Gmax), s	5.0	45.0	57.0	9.0
Max Q Clear Time (g_c+I1), s	2.0	29.4	2.0	6.0
Green Ext Time (p_c), s	0.0	3.6	6.2	0.1

Intersection Summary	
HCM 6th Ctrl Delay	20.3
HCM 6th LOS	C

Notes

Unsignalized Delay for [NBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	108	305	588	102	302	638
Future Volume (veh/h)	108	305	588	102	302	638
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	114	321	619	107	318	672
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	85	240	646	112	369	1169
Arrive On Green	0.20	0.20	0.83	0.83	0.12	0.63
Sat Flow, veh/h	427	1202	1553	269	1781	1870
Grp Volume(v), veh/h	436	0	0	726	318	672
Grp Sat Flow(s),veh/h/ln	1633	0	0	1822	1781	1870
Q Serve(g_s), s	16.0	0.0	0.0	26.5	7.6	16.8
Cycle Q Clear(g_c), s	16.0	0.0	0.0	26.5	7.6	16.8
Prop In Lane	0.26	0.74		0.15	1.00	
Lane Grp Cap(c), veh/h	327	0	0	757	369	1169
V/C Ratio(X)	1.34	0.00	0.00	0.96	0.86	0.57
Avail Cap(c_a), veh/h	327	0	0	757	397	1169
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.82	1.00	1.00
Uniform Delay (d), s/veh	32.0	0.0	0.0	6.2	16.4	8.8
Incr Delay (d2), s/veh	170.3	0.0	0.0	21.1	16.6	2.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	21.1	0.0	0.0	6.6	3.9	5.2
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	202.3	0.0	0.0	27.3	33.0	10.8
LnGrp LOS	F	A	A	C	C	B
Approach Vol, veh/h	436		726			990
Approach Delay, s/veh	202.3		27.3			18.0
Approach LOS	F		C			B

Timer - Assigned Phs	1	2	6	8
Phs Duration (G+Y+Rc), s	16.7	40.3	57.0	23.0
Change Period (Y+Rc), s	7.0	* 7	7.0	7.0
Max Green Setting (Gmax), s	11.0	* 33	50.0	16.0
Max Q Clear Time (g_c+I1), s	9.6	28.5	18.8	18.0
Green Ext Time (p_c), s	0.1	1.8	4.2	0.0

Intersection Summary	
HCM 6th Ctrl Delay	58.5
HCM 6th LOS	E

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Int Delay, s/veh	3.3

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	295	74	61	354	85	57
Future Vol, veh/h	295	74	61	354	85	57
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	321	80	66	385	92	62

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	401	0	878	361
Stage 1	-	-	-	-	361	-
Stage 2	-	-	-	-	517	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1158	-	318	684
Stage 1	-	-	-	-	705	-
Stage 2	-	-	-	-	598	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1158	-	295	684
Mov Cap-2 Maneuver	-	-	-	-	295	-
Stage 1	-	-	-	-	705	-
Stage 2	-	-	-	-	555	-

Approach	EB	WB	NB
HCM Control Delay, s	0	1.2	17.9
HCM LOS			C

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	295	684	-	-	1158	-
HCM Lane V/C Ratio	0.313	0.091	-	-	0.057	-
HCM Control Delay (s)	22.7	10.8	-	-	8.3	0
HCM Lane LOS	C	B	-	-	A	A
HCM 95th %tile Q(veh)	1.3	0.3	-	-	0.2	-

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	85	57	633	152	87	658
Future Volume (veh/h)	85	57	633	152	87	658
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	92	62	688	0	95	715
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	132	118	770		571	1404
Arrive On Green	0.07	0.07	0.41	0.00	0.50	1.00
Sat Flow, veh/h	1781	1585	1870	1585	1781	1870
Grp Volume(v), veh/h	92	62	688	0	95	715
Grp Sat Flow(s),veh/h/ln	1781	1585	1870	1585	1781	1870
Q Serve(g_s), s	4.0	3.0	27.4	0.0	0.0	0.0
Cycle Q Clear(g_c), s	4.0	3.0	27.4	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	132	118	770		571	1404
V/C Ratio(X)	0.70	0.53	0.89		0.17	0.51
Avail Cap(c_a), veh/h	200	178	1052		571	1404
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	1.00	1.00	0.00	0.61	0.61
Uniform Delay (d), s/veh	36.1	35.7	21.9	0.0	14.3	0.0
Incr Delay (d2), s/veh	6.4	3.6	14.9	0.0	0.1	0.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.0	1.3	13.0	0.0	0.9	0.3
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	42.6	39.3	36.8	0.0	14.3	0.8
LnGrp LOS	D	D	D		B	A
Approach Vol, veh/h	154		688	A		810
Approach Delay, s/veh	41.2		36.8			2.4
Approach LOS	D		D			A

Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	27.1	40.0				67.1		12.9
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	5.0	45.0				57.0		9.0
Max Q Clear Time (g_c+I1), s	2.0	29.4				2.0		6.0
Green Ext Time (p_c), s	0.0	3.6				6.2		0.1

Intersection Summary	
HCM 6th Ctrl Delay	20.3
HCM 6th LOS	C

Notes

Unsignalized Delay for [NBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	100	196	866	106	253	707
Future Volume (veh/h)	100	196	866	106	253	707
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1856	1870	1870
Adj Flow Rate, veh/h	105	206	912	112	266	744
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	3	2	2
Cap, veh/h	81	158	959	118	525	1360
Arrive On Green	0.15	0.15	1.00	1.00	0.08	0.73
Sat Flow, veh/h	554	1087	1634	201	1781	1870
Grp Volume(v), veh/h	312	0	0	1024	266	744
Grp Sat Flow(s),veh/h/ln	1647	0	0	1834	1781	1870
Q Serve(g_s), s	16.0	0.0	0.0	0.0	6.1	19.8
Cycle Q Clear(g_c), s	16.0	0.0	0.0	0.0	6.1	19.8
Prop In Lane	0.34	0.66		0.11	1.00	
Lane Grp Cap(c), veh/h	240	0	0	1077	525	1360
V/C Ratio(X)	1.30	0.00	0.00	0.95	0.51	0.55
Avail Cap(c_a), veh/h	240	0	0	1077	583	1360
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.57	1.00	1.00
Uniform Delay (d), s/veh	47.0	0.0	0.0	0.0	6.6	6.8
Incr Delay (d2), s/veh	163.1	0.0	0.0	11.9	0.8	1.6
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	17.0	0.0	0.0	3.6	1.9	5.9
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	210.1	0.0	0.0	11.9	7.3	8.4
LnGrp LOS	F	A	A	B	A	A
Approach Vol, veh/h	312		1024			1010
Approach Delay, s/veh	210.1		11.9			8.1
Approach LOS	F		B			A

Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	15.4	71.6				87.0		23.0
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	12.0	61.0				80.0		16.0
Max Q Clear Time (g_c+I1), s	8.1	2.0				21.8		18.0
Green Ext Time (p_c), s	0.3	9.5				5.1		0.0

Intersection Summary	
HCM 6th Ctrl Delay	36.6
HCM 6th LOS	D

Notes

User approved volume balancing among the lanes for turning movement.

Intersection						
Int Delay, s/veh	2.7					

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	182	121	100	287	52	35
Future Vol, veh/h	182	121	100	287	52	35
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	198	132	109	312	57	38

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	330	0	794	264
Stage 1	-	-	-	-	264	-
Stage 2	-	-	-	-	530	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1229	-	357	775
Stage 1	-	-	-	-	780	-
Stage 2	-	-	-	-	590	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1229	-	319	775
Mov Cap-2 Maneuver	-	-	-	-	319	-
Stage 1	-	-	-	-	780	-
Stage 2	-	-	-	-	527	-

Approach	EB	WB	NB
HCM Control Delay, s	0	2.1	15.2
HCM LOS			C

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	319	775	-	-	1229	-
HCM Lane V/C Ratio	0.177	0.049	-	-	0.088	-
HCM Control Delay (s)	18.7	9.9	-	-	8.2	0
HCM Lane LOS	C	A	-	-	A	A
HCM 95th %tile Q(veh)	0.6	0.2	-	-	0.3	-

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	52	35	937	249	142	666
Future Volume (veh/h)	52	35	937	249	142	666
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	57	38	1018	0	154	724
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	84	75	1086		422	1544
Arrive On Green	0.05	0.05	0.58	0.00	0.36	1.00
Sat Flow, veh/h	1781	1585	1870	1585	1781	1870
Grp Volume(v), veh/h	57	38	1018	0	154	724
Grp Sat Flow(s),veh/h/ln	1781	1585	1870	1585	1781	1870
Q Serve(g_s), s	3.5	2.6	55.1	0.0	0.0	0.0
Cycle Q Clear(g_c), s	3.5	2.6	55.1	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	84	75	1086		422	1544
V/C Ratio(X)	0.68	0.51	0.94		0.36	0.47
Avail Cap(c_a), veh/h	113	101	1275		422	1544
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	1.00	1.00	0.00	0.67	0.67
Uniform Delay (d), s/veh	51.6	51.2	21.2	0.0	28.0	0.0
Incr Delay (d2), s/veh	9.3	5.2	15.9	0.0	0.4	0.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.8	1.1	24.5	0.0	2.7	0.3
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	60.9	56.4	37.1	0.0	28.3	0.7
LnGrp LOS	E	E	D		C	A
Approach Vol, veh/h	95		1018	A		878
Approach Delay, s/veh	59.1		37.1			5.5
Approach LOS	E		D			A

Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	26.9	70.9				97.8		12.2
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	7.0	75.0				89.0		7.0
Max Q Clear Time (g_c+I1), s	2.0	57.1				2.0		5.5
Green Ext Time (p_c), s	0.2	6.8				6.4		0.0

Intersection Summary	
HCM 6th Ctrl Delay	24.2
HCM 6th LOS	C

Notes

Unsignalized Delay for [NBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	103	293	576	124	390	726
Future Volume (veh/h)	103	293	576	124	390	726
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	108	308	606	131	411	764
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	80	229	685	148	527	1253
Arrive On Green	0.19	0.19	0.92	0.92	0.14	0.67
Sat Flow, veh/h	423	1206	1490	322	1781	1870
Grp Volume(v), veh/h	417	0	0	737	411	764
Grp Sat Flow(s),veh/h/ln	1632	0	0	1812	1781	1870
Q Serve(g_s), s	19.0	0.0	0.0	17.6	11.4	22.8
Cycle Q Clear(g_c), s	19.0	0.0	0.0	17.6	11.4	22.8
Prop In Lane	0.26	0.74		0.18	1.00	
Lane Grp Cap(c), veh/h	310	0	0	833	527	1253
V/C Ratio(X)	1.34	0.00	0.00	0.88	0.78	0.61
Avail Cap(c_a), veh/h	310	0	0	833	615	1253
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.81	1.00	1.00
Uniform Delay (d), s/veh	40.5	0.0	0.0	2.9	13.8	9.2
Incr Delay (d2), s/veh	175.1	0.0	0.0	11.1	5.5	2.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	22.2	0.0	0.0	3.8	4.3	7.4
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	215.6	0.0	0.0	14.0	19.4	11.4
LnGrp LOS	F	A	A	B	B	B
Approach Vol, veh/h	417		737			1175
Approach Delay, s/veh	215.6		14.0			14.2
Approach LOS	F		B			B

Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	21.0	53.0				74.0		26.0
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	19.0	41.0				67.0		19.0
Max Q Clear Time (g_c+I1), s	13.4	19.6				24.8		21.0
Green Ext Time (p_c), s	0.6	4.5				5.3		0.0

Intersection Summary	
HCM 6th Ctrl Delay	50.2
HCM 6th LOS	D

Intersection						
Int Delay, s/veh	3.6					

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	295	149	123	381	68	45
Future Vol, veh/h	295	149	123	381	68	45
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	321	162	134	414	74	49

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	483	0	1084	402
Stage 1	-	-	-	-	402	-
Stage 2	-	-	-	-	682	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1080	-	240	648
Stage 1	-	-	-	-	676	-
Stage 2	-	-	-	-	502	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1080	-	201	648
Mov Cap-2 Maneuver	-	-	-	-	201	-
Stage 1	-	-	-	-	676	-
Stage 2	-	-	-	-	421	-

Approach	EB	WB	NB
HCM Control Delay, s	0	2.1	24.2
HCM LOS			C

Minor Lane/Major Mvmt	NBLn1	NBLn2	EBT	EBR	WBL	WBT
Capacity (veh/h)	201	648	-	-	1080	-
HCM Lane V/C Ratio	0.368	0.075	-	-	0.124	-
HCM Control Delay (s)	33	11	-	-	8.8	0
HCM Lane LOS	D	B	-	-	A	A
HCM 95th %tile Q(veh)	1.6	0.2	-	-	0.4	-

Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Traffic Volume (veh/h)	68	45	655	307	175	653		
Future Volume (veh/h)	68	45	655	307	175	653		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Work Zone On Approach	No		No			No		
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870		
Adj Flow Rate, veh/h	74	49	712	0	190	710		
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	106	94	785		657	1497		
Arrive On Green	0.06	0.06	0.42	0.00	0.62	1.00		
Sat Flow, veh/h	1781	1585	1870	1585	1781	1870		
Grp Volume(v), veh/h	74	49	712	0	190	710		
Grp Sat Flow(s),veh/h/ln	1781	1585	1870	1585	1781	1870		
Q Serve(g_s), s	4.1	3.0	35.7	0.0	0.0	0.0		
Cycle Q Clear(g_c), s	4.1	3.0	35.7	0.0	0.0	0.0		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	106	94	785		657	1497		
V/C Ratio(X)	0.70	0.52	0.91		0.29	0.47		
Avail Cap(c_a), veh/h	160	143	1141		657	1497		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00		
Upstream Filter(I)	1.00	1.00	1.00	0.00	0.58	0.58		
Uniform Delay (d), s/veh	46.1	45.6	27.2	0.0	12.7	0.0		
Incr Delay (d2), s/veh	8.0	4.4	16.1	0.0	0.1	0.6		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	2.0	1.3	17.5	0.0	1.7	0.3		
Unsig. Movement Delay, s/veh								
LnGrp Delay(d),s/veh	54.2	50.0	43.3	0.0	12.8	0.6		
LnGrp LOS	D	D	D		B	A		
Approach Vol, veh/h	123		712	A		900		
Approach Delay, s/veh	52.5		43.3			3.2		
Approach LOS	D		D			A		
Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	38.1	49.0				87.0		13.0
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	9.0	61.0				77.0		9.0
Max Q Clear Time (g_c+I1), s	2.0	37.7				2.0		6.1
Green Ext Time (p_c), s	0.3	4.3				6.2		0.1

Intersection Summary		
HCM 6th Ctrl Delay		23.2
HCM 6th LOS		C

Notes

Unsignalized Delay for [NBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	64	163	558	66	183	654
Future Volume (veh/h)	64	163	558	66	183	654
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1781	1870	1870	1841
Adj Flow Rate, veh/h	66	0	575	68	189	674
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	2	8	2	2	4
Cap, veh/h	86		688	81	1064	2921
Arrive On Green	0.05	0.00	0.23	0.23	0.55	0.84
Sat Flow, veh/h	1781	1585	3138	360	1781	3589
Grp Volume(v), veh/h	66	0	319	324	189	674
Grp Sat Flow(s),veh/h/ln	1781	1585	1692	1717	1781	1749
Q Serve(g_s), s	4.4	0.0	21.5	21.7	0.2	4.7
Cycle Q Clear(g_c), s	4.4	0.0	21.5	21.7	0.2	4.7
Prop In Lane	1.00	1.00		0.21	1.00	
Lane Grp Cap(c), veh/h	86		382	387	1064	2921
V/C Ratio(X)	0.77		0.83	0.84	0.18	0.23
Avail Cap(c_a), veh/h	401		705	715	1064	2921
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.98	0.98	1.00	1.00
Uniform Delay (d), s/veh	56.4	0.0	44.3	44.4	11.9	2.0
Incr Delay (d2), s/veh	13.3	0.0	18.6	18.7	0.1	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.2	0.0	10.5	10.7	2.1	0.7
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	69.7	0.0	62.9	63.0	12.0	2.2
LnGrp LOS	E		E	E	B	A
Approach Vol, veh/h	66	A	643			863
Approach Delay, s/veh	69.7		63.0			4.3
Approach LOS	E		E			A

Timer - Assigned Phs	1	2	6	8
Phs Duration (G+Y+Rc), s	73.1	34.1	107.2	12.8
Change Period (Y+Rc), s	7.0	7.0	7.0	7.0
Max Green Setting (Gmax), s	22.0	50.0	79.0	27.0
Max Q Clear Time (g_c+I1), s	2.2	23.7	6.7	6.4
Green Ext Time (p_c), s	0.4	3.4	4.4	0.1

Intersection Summary	
HCM 6th Ctrl Delay	31.1
HCM 6th LOS	C

Notes

Unsignalized Delay for [WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	38	26	598	64	36	682
Future Volume (veh/h)	38	26	598	64	36	682
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	41	0	650	0	39	741
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	55		799		1101	3029
Arrive On Green	0.03	0.00	0.22	0.00	1.00	1.00
Sat Flow, veh/h	1781	1585	3647	1585	1781	3647
Grp Volume(v), veh/h	41	0	650	0	39	741
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1585	1781	1777
Q Serve(g_s), s	2.7	0.0	20.8	0.0	0.0	0.0
Cycle Q Clear(g_c), s	2.7	0.0	20.8	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	55		799		1101	3029
V/C Ratio(X)	0.74		0.81		0.04	0.24
Avail Cap(c_a), veh/h	282		1984		1101	3029
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	0.98	0.98
Uniform Delay (d), s/veh	57.7	0.0	44.1	0.0	0.0	0.0
Incr Delay (d2), s/veh	17.5	0.0	8.9	0.0	0.0	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.5	0.0	9.7	0.0	0.0	0.1
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	75.2	0.0	53.0	0.0	0.0	0.2
LnGrp LOS	E		D		A	A
Approach Vol, veh/h	41	A	650	A		780
Approach Delay, s/veh	75.2		53.0			0.2
Approach LOS	E		D			A

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	75.3	34.0		10.7		109.3
Change Period (Y+Rc), s	7.0	7.0		7.0		7.0
Max Green Setting (Gmax), s	13.0	67.0		19.0		87.0
Max Q Clear Time (g_c+I1), s	2.0	22.8		4.7		2.0
Green Ext Time (p_c), s	0.0	4.2		0.0		5.0

Intersection Summary		
HCM 6th Ctrl Delay		25.6
HCM 6th LOS		C

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	104	203	873	83	161	615
Future Volume (veh/h)	104	203	873	83	161	615
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1856	1870	1870
Adj Flow Rate, veh/h	109	0	919	87	169	647
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	3	2	2
Cap, veh/h	136		1054	100	849	2868
Arrive On Green	0.08	0.00	0.43	0.43	0.43	0.81
Sat Flow, veh/h	1781	1585	3374	311	1781	3647
Grp Volume(v), veh/h	109	0	498	508	169	647
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1814	1781	1777
Q Serve(g_s), s	7.2	0.0	30.7	30.7	0.7	5.2
Cycle Q Clear(g_c), s	7.2	0.0	30.7	30.7	0.7	5.2
Prop In Lane	1.00	1.00		0.17	1.00	
Lane Grp Cap(c), veh/h	136		571	583	849	2868
V/C Ratio(X)	0.80		0.87	0.87	0.20	0.23
Avail Cap(c_a), veh/h	356		844	862	849	2868
HCM Platoon Ratio	1.00	1.00	1.33	1.33	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.94	0.94	1.00	1.00
Uniform Delay (d), s/veh	54.5	0.0	32.1	32.1	19.2	2.7
Incr Delay (d2), s/veh	10.3	0.0	15.8	15.6	0.1	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.5	0.0	13.4	13.7	2.6	1.0
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	64.8	0.0	47.9	47.7	19.3	2.9
LnGrp LOS	E		D	D	B	A
Approach Vol, veh/h	109	A	1006			816
Approach Delay, s/veh	64.8		47.8			6.3
Approach LOS	E		D			A

Timer - Assigned Phs	1	2	6	8
Phs Duration (G+Y+Rc), s	58.3	45.6	103.8	16.2
Change Period (Y+Rc), s	7.0	7.0	7.0	7.0
Max Green Setting (Gmax), s	18.0	57.0	82.0	24.0
Max Q Clear Time (g_c+I1), s	2.7	32.7	7.2	9.2
Green Ext Time (p_c), s	0.3	5.9	4.2	0.2

Intersection Summary	
HCM 6th Ctrl Delay	31.2
HCM 6th LOS	C

Notes

Unsignalized Delay for [WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	63	42	914	88	50	670
Future Volume (veh/h)	63	42	914	88	50	670
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	68	0	993	0	54	728
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	88		1187		880	2963
Arrive On Green	0.05	0.00	0.33	0.00	0.88	1.00
Sat Flow, veh/h	1781	1585	3647	1585	1781	3647
Grp Volume(v), veh/h	68	0	993	0	54	728
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1585	1781	1777
Q Serve(g_s), s	4.5	0.0	31.0	0.0	0.0	0.0
Cycle Q Clear(g_c), s	4.5	0.0	31.0	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	88		1187		880	2963
V/C Ratio(X)	0.77		0.84		0.06	0.25
Avail Cap(c_a), veh/h	267		2073		880	2963
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	0.97	0.97
Uniform Delay (d), s/veh	56.4	0.0	36.9	0.0	3.2	0.0
Incr Delay (d2), s/veh	13.3	0.0	7.1	0.0	0.0	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.4	0.0	13.7	0.0	0.2	0.1
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	69.6	0.0	44.0	0.0	3.2	0.2
LnGrp LOS	E		D		A	A
Approach Vol, veh/h	68	A	993	A		782
Approach Delay, s/veh	69.6		44.0			0.4
Approach LOS	E		D			A
Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	60.0	47.1		12.9		107.1
Change Period (Y+Rc), s	7.0	7.0		7.0		7.0
Max Green Setting (Gmax), s	11.0	70.0		18.0		88.0
Max Q Clear Time (g_c+I1), s	2.0	33.0		6.5		2.0
Green Ext Time (p_c), s	0.0	7.1		0.1		4.9
Intersection Summary						
HCM 6th Ctrl Delay			26.4			
HCM 6th LOS			C			

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	100	196	866	106	253	707
Future Volume (veh/h)	100	196	866	106	253	707
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1856	1870	1870
Adj Flow Rate, veh/h	105	0	912	112	266	744
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	3	2	2
Cap, veh/h	131		1001	123	865	2877
Arrive On Green	0.07	0.00	0.63	0.63	0.44	0.81
Sat Flow, veh/h	1781	1585	3279	391	1781	3647
Grp Volume(v), veh/h	105	0	509	515	266	744
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1800	1781	1777
Q Serve(g_s), s	7.0	0.0	29.8	29.9	5.0	6.0
Cycle Q Clear(g_c), s	7.0	0.0	29.8	29.9	5.0	6.0
Prop In Lane	1.00	1.00		0.22	1.00	
Lane Grp Cap(c), veh/h	131		559	566	865	2877
V/C Ratio(X)	0.80		0.91	0.91	0.31	0.26
Avail Cap(c_a), veh/h	297		800	810	865	2877
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.93	0.93	1.00	1.00
Uniform Delay (d), s/veh	54.7	0.0	20.8	20.8	19.3	2.7
Incr Delay (d2), s/veh	10.6	0.0	20.3	20.1	0.2	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.4	0.0	9.3	9.4	4.2	1.2
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	65.3	0.0	41.1	41.0	19.5	3.0
LnGrp LOS	E		D	D	B	A
Approach Vol, veh/h	105	A	1024			1010
Approach Delay, s/veh	65.3		41.0			7.3
Approach LOS	E		D			A

Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	59.4	44.7				104.2		15.8
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	25.0	54.0				86.0		20.0
Max Q Clear Time (g_c+I1), s	7.0	31.9				8.0		9.0
Green Ext Time (p_c), s	0.6	5.9				5.0		0.2

Intersection Summary		
HCM 6th Ctrl Delay		26.3
HCM 6th LOS		C

Notes

Unsignalized Delay for [WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	52	35	937	249	142	666
Future Volume (veh/h)	52	35	937	249	142	666
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	57	0	1018	0	154	724
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	74		1212		881	2991
Arrive On Green	0.04	0.00	0.34	0.00	0.88	1.00
Sat Flow, veh/h	1781	1585	3647	1585	1781	3647
Grp Volume(v), veh/h	57	0	1018	0	154	724
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1585	1781	1777
Q Serve(g_s), s	3.8	0.0	31.7	0.0	0.0	0.0
Cycle Q Clear(g_c), s	3.8	0.0	31.7	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	74		1212		881	2991
V/C Ratio(X)	0.77		0.84		0.17	0.24
Avail Cap(c_a), veh/h	223		1955		881	2991
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	0.96	0.96
Uniform Delay (d), s/veh	56.9	0.0	36.5	0.0	3.3	0.0
Incr Delay (d2), s/veh	15.4	0.0	7.1	0.0	0.1	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.0	0.0	14.0	0.0	0.4	0.1
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	72.3	0.0	43.6	0.0	3.4	0.2
LnGrp LOS	E		D		A	A
Approach Vol, veh/h	57	A	1018	A		878
Approach Delay, s/veh	72.3		43.6			0.7
Approach LOS	E		D			A
Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	60.1	47.9		12.0		108.0
Change Period (Y+Rc), s	7.0	7.0		7.0		7.0
Max Green Setting (Gmax), s	18.0	66.0		15.0		91.0
Max Q Clear Time (g_c+I1), s	2.0	33.7		5.8		2.0
Green Ext Time (p_c), s	0.3	7.2		0.1		4.8
Intersection Summary						
HCM 6th Ctrl Delay			25.2			
HCM 6th LOS			C			

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Traffic Volume (veh/h)	108	305	588	102	302	638		
Future Volume (veh/h)	108	305	588	102	302	638		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Work Zone On Approach	No		No			No		
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870		
Adj Flow Rate, veh/h	114	0	619	107	318	672		
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	145		734	127	926	2767		
Arrive On Green	0.08	0.00	0.32	0.32	0.47	0.78		
Sat Flow, veh/h	1781	1585	3124	523	1781	3647		
Grp Volume(v), veh/h	114	0	362	364	318	672		
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1776	1781	1777		
Q Serve(g_s), s	6.3	0.0	19.0	19.1	5.1	5.2		
Cycle Q Clear(g_c), s	6.3	0.0	19.0	19.1	5.1	5.2		
Prop In Lane	1.00	1.00		0.29	1.00			
Lane Grp Cap(c), veh/h	145		430	430	926	2767		
V/C Ratio(X)	0.79		0.84	0.85	0.34	0.24		
Avail Cap(c_a), veh/h	410		604	604	926	2767		
HCM Platoon Ratio	1.00	1.00	1.33	1.33	1.00	1.00		
Upstream Filter(I)	1.00	0.00	0.97	0.97	1.00	1.00		
Uniform Delay (d), s/veh	45.1	0.0	32.1	32.1	14.7	3.0		
Incr Delay (d2), s/veh	9.1	0.0	17.4	17.7	0.2	0.2		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	3.0	0.0	8.8	8.9	3.7	1.0		
Unsig. Movement Delay, s/veh								
LnGrp Delay(d),s/veh	54.2	0.0	49.5	49.8	14.9	3.2		
LnGrp LOS	D		D	D	B	A		
Approach Vol, veh/h	114	A	726			990		
Approach Delay, s/veh	54.2		49.7			7.0		
Approach LOS	D		D			A		
Timer - Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	53.7	31.2				84.9		15.1
Change Period (Y+Rc), s	7.0	7.0				7.0		7.0
Max Green Setting (Gmax), s	22.0	34.0				63.0		23.0
Max Q Clear Time (g_c+I1), s	7.1	21.1				7.2		8.3
Green Ext Time (p_c), s	0.8	3.2				4.4		0.2

Intersection Summary			
HCM 6th Ctrl Delay			26.9
HCM 6th LOS			C

Notes
Unsignalized Delay for [WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	85	57	633	152	87	658
Future Volume (veh/h)	85	57	633	152	87	658
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	92	0	688	0	95	715
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	119		866		958	2819
Arrive On Green	0.07	0.00	0.24	0.00	0.96	1.00
Sat Flow, veh/h	1781	1585	3647	1585	1781	3647
Grp Volume(v), veh/h	92	0	688	0	95	715
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1585	1781	1777
Q Serve(g_s), s	5.1	0.0	18.2	0.0	0.0	0.0
Cycle Q Clear(g_c), s	5.1	0.0	18.2	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	119		866		958	2819
V/C Ratio(X)	0.77		0.79		0.10	0.25
Avail Cap(c_a), veh/h	356		1670		958	2819
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	0.96	0.96
Uniform Delay (d), s/veh	45.9	0.0	35.5	0.0	0.9	0.0
Incr Delay (d2), s/veh	10.2	0.0	7.4	0.0	0.0	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.6	0.0	8.1	0.0	0.1	0.1
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	56.1	0.0	42.9	0.0	1.0	0.2
LnGrp LOS	E		D		A	A
Approach Vol, veh/h	92	A	688	A		810
Approach Delay, s/veh	56.1		42.9			0.3
Approach LOS	E		D			A

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	55.0	31.4		13.7		86.3
Change Period (Y+Rc), s	7.0	7.0		7.0		7.0
Max Green Setting (Gmax), s	12.0	47.0		20.0		66.0
Max Q Clear Time (g_c+I1), s	2.0	20.2		7.1		2.0
Green Ext Time (p_c), s	0.1	4.2		0.2		4.7

Intersection Summary						
HCM 6th Ctrl Delay			22.0			
HCM 6th LOS			C			

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	103	293	576	124	390	726
Future Volume (veh/h)	103	293	576	124	390	726
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	108	0	606	131	411	764
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	135		683	147	996	2870
Arrive On Green	0.08	0.00	0.47	0.47	0.51	0.81
Sat Flow, veh/h	1781	1585	3001	627	1781	3647
Grp Volume(v), veh/h	108	0	370	367	411	764
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1757	1781	1777
Q Serve(g_s), s	7.2	0.0	22.7	22.8	10.5	6.3
Cycle Q Clear(g_c), s	7.2	0.0	22.7	22.8	10.5	6.3
Prop In Lane	1.00	1.00		0.36	1.00	
Lane Grp Cap(c), veh/h	135		417	413	996	2870
V/C Ratio(X)	0.80		0.89	0.89	0.41	0.27
Avail Cap(c_a), veh/h	371		577	571	996	2870
HCM Platoon Ratio	1.00	1.00	2.00	2.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.97	0.97	1.00	1.00
Uniform Delay (d), s/veh	54.6	0.0	30.4	30.4	15.6	2.8
Incr Delay (d2), s/veh	10.3	0.0	22.6	23.2	0.3	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.5	0.0	9.1	9.1	5.8	1.3
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	64.9	0.0	53.0	53.6	15.9	3.1
LnGrp LOS	E		D	D	B	A
Approach Vol, veh/h	108	A	737			1175
Approach Delay, s/veh	64.9		53.3			7.5
Approach LOS	E		D			A

Timer - Assigned Phs	1	2	6	8
Phs Duration (G+Y+Rc), s	68.7	35.2	103.9	16.1
Change Period (Y+Rc), s	7.0	7.0	7.0	7.0
Max Green Setting (Gmax), s	35.0	39.0	81.0	25.0
Max Q Clear Time (g_c+I1), s	12.5	24.8	8.3	9.2
Green Ext Time (p_c), s	1.2	3.4	5.2	0.2

Intersection Summary	
HCM 6th Ctrl Delay	27.3
HCM 6th LOS	C

Notes

Unsignalized Delay for [WBR] is excluded from calculations of the approach delay and intersection delay.

HCM 6th Signalized Intersection Summary
4: US-259 & DWY 3

Build (2023 - EVENT) - SAT Mitigation
Choctaw Broken Bow Resort

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	68	45	655	307	175	653
Future Volume (veh/h)	68	45	655	307	175	653
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	74	0	712	0	190	710
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	95		866		1028	2949
Arrive On Green	0.05	0.00	0.24	0.00	1.00	1.00
Sat Flow, veh/h	1781	1585	3647	1585	1781	3647
Grp Volume(v), veh/h	74	0	712	0	190	710
Grp Sat Flow(s),veh/h/ln	1781	1585	1777	1585	1781	1777
Q Serve(g_s), s	4.9	0.0	22.7	0.0	0.0	0.0
Cycle Q Clear(g_c), s	4.9	0.0	22.7	0.0	0.0	0.0
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	95		866		1028	2949
V/C Ratio(X)	0.78		0.82		0.18	0.24
Avail Cap(c_a), veh/h	282		1658		1028	2949
HCM Platoon Ratio	1.00	1.00	1.00	1.00	2.00	2.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	0.96	0.96
Uniform Delay (d), s/veh	56.1	0.0	42.9	0.0	0.0	0.0
Incr Delay (d2), s/veh	12.5	0.0	8.7	0.0	0.1	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.5	0.0	10.5	0.0	0.0	0.1
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	68.6	0.0	51.6	0.0	0.1	0.2
LnGrp LOS	E		D		A	A
Approach Vol, veh/h	74	A	712	A		900
Approach Delay, s/veh	68.6		51.6			0.2
Approach LOS	E		D			A

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	70.3	36.3		13.4		106.6
Change Period (Y+Rc), s	7.0	7.0		7.0		7.0
Max Green Setting (Gmax), s	24.0	56.0		19.0		87.0
Max Q Clear Time (g_c+I1), s	2.0	24.7		6.9		2.0
Green Ext Time (p_c), s	0.5	4.5		0.1		4.7

Intersection Summary	
HCM 6th Ctrl Delay	24.9
HCM 6th LOS	C

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

SIGNAL WARRANT ANLAYSIS

EXISTING TRAFFIC SIGNAL WARRANT ANALYSIS

COUNT DATE:	7/30/2021	85th-percentile speed on major street exceeds 40 mph? (Y or N)	Y
ANALYST:	Lee Engineering, LLC	Isolated community with population less than 10,000? (Y or N)	Y
		Apply 56% warrant to Warrant 1, Combination Warrant? (Y or N)	N
MAJOR STREET:	**SH-259**	Approach Lanes - Major?	1
MINOR STREET:	**SH-259A**	Approach Lanes - Minor?	1

VOLUME SUMMARY	MAJOR STREET	MINOR STREET	WARRANT 1 - EIGHT-HOUR VEHICULAR VOLUME								WARRANT 2 - FOUR-HOUR VEHICULAR VOLUME	WARRANT 4 - PEDESTRIAN VOLUME		
			CONDITION A		CONDITION B		COMBINATION 56% of 1A		COMBINATION 56% of 1B					
	Total of both approaches	Higher volume approach (adjusted)[1]	Major Street	Minor Street	Major Street	Minor Street	Major Street	Minor Street	Major Street	Minor Street		Pedestrian Volume - Across Major Street	Four-Hour	Peak Hour
12 - 1 AM	80.58922323	4	23%	4%	15%	8%	29%	5%	19%	10%	1%	0	0%	0%
1 - 2 AM	46.53743877	3	13%	3%	9%	6%	17%	4%	11%	7%	1%	0	0%	0%
2 - 3 AM	41.99720084	2	12%	2%	8%	4%	15%	2%	10%	5%	1%	0	0%	0%
3 - 4 AM	37.45696291	2	11%	2%	7%	4%	13%	2%	9%	5%	1%	0	0%	0%
4 - 5 AM	60.15815255	3	17%	3%	11%	6%	21%	4%	14%	7%	1%	0	0%	0%
5 - 6 AM	157.773268	9	45%	9%	30%	17%	56%	11%	38%	21%	3%	0	0%	0%
6 - 7 AM	334.8425472	18	96%	17%	64%	34%	120%	21%	80%	43%	9%	0	0%	0%
7 - 8 AM	499.4261721	27	143%	26%	95%	51%	178%	32%	119%	64%	21%	0	0%	0%
8 - 9 AM	741.1938418	40	212%	38%	141%	75%	265%	48%	176%	95%	63%	0	0%	0%
9 - 10 AM	1024.958712	55	293%	52%	195%	104%	366%	65%	244%	131%	92%	0	0%	0%
10 - 11 AM	1380	64	394%	61%	263%	121%	493%	76%	329%	152%	107%	0	0%	0%
11 A - 12 P	1363.206438	73	389%	70%	260%	138%	487%	87%	325%	174%	122%	0	0%	0%
12 - 1 PM	1370.016795	74	391%	70%	261%	140%	489%	88%	326%	176%	123%	0	0%	0%
1 - 2 PM	1356.396081	73	388%	70%	258%	138%	484%	87%	323%	174%	122%	0	0%	0%
2 - 3 PM	1548.221134	83	442%	79%	295%	157%	553%	99%	369%	198%	138%	0	0%	0%
3 - 4 PM	1629.945416	88	466%	84%	310%	166%	582%	105%	388%	210%	147%	0	0%	0%
4 - 5 PM	1722	105	492%	100%	328%	198%	615%	125%	410%	250%	175%	0	0%	0%
5 - 6 PM	1456.281316	78	416%	74%	277%	147%	520%	93%	347%	186%	130%	0	0%	0%
6 - 7 PM	1343.910427	72	384%	69%	256%	136%	480%	86%	320%	171%	120%	0	0%	0%
7 - 8 PM	1099.872638	59	314%	56%	209%	111%	393%	70%	262%	140%	98%	0	0%	0%
8 - 9 PM	889.886634	48	254%	46%	170%	91%	318%	57%	212%	114%	80%	0	0%	0%
9 - 10 PM	620.8775367	33	177%	31%	118%	62%	222%	39%	148%	79%	37%	0	0%	0%
10 - 11 PM	447.213436	24	128%	23%	85%	45%	160%	29%	106%	57%	17%	0	0%	0%
11PM - 12AM	225.8768369	12	65%	11%	43%	23%	81%	14%	54%	29%	5%	0	0%	0%
			Threshold		Threshold		Threshold		Threshold		MUTCD Figure 4C-1 or 4C-2	Threshold	MUTCD Figure 4C-5 or 4C-6	MUTCD Figure 4C-7 or 4C-8
			350	105	525	53	280	84	420	42		107 & 133		
			Summary		Summary		Summary		Summary		Summary	Summary	Summary	Summary
			Total	1	Total	11	Total	2	Total	12	Total 9	Met 4-hr? NO	Total 0	Total 0
			Met?	NO	Met?	YES	Total	2	Met?	NO	Met? YES	Met PH? NO	Met? NO	Met? NO

WARRANT 7 - CRASH EXPERIENCE	
Number of collisions potentially correctable by a signal:	0
Requirement:	
5 total potentially correctable within 12 month period	Not Met
Volume - Either Warrant 1 Condition A 80% or Condition B 80% met	Met
WARRANT 7:	Not Met

TRAFFIC SIGNAL WARRANT ANALYSIS

Introduction

A future traffic signal warrant analysis has been conducted for the intersection of US-259 and proposed Driveway 3 to determine if signalization will be warranted at this location upon the completion of the Choctaw Nation Hochatown Resort. This report summarizes the results of the traffic signal warrant analysis conducted for the intersection.

The analysis was performed using predicted Build-Out (2023) Total traffic volumes for a typical weekday at the intersection under Scenario 1 (No Event).

The traffic signal warrant analysis presented in this report is based on the traffic signal warrants contained in Chapter 4C, “Traffic Control Signal Needs Studies,” of the *Manual on Uniform Traffic Control Devices* (MUTCD), *latest edition*. Nine warrants are included in the manual for warranting a traffic signal installation. These warrants are:

Warrant 1 – Eight-Hour Vehicular Volume
Warrant 2 – Four-Hour Vehicular Volume
Warrant 3 – Peak Hour
Warrant 4 – Pedestrian Volume
Warrant 5 – School Crossing
Warrant 6 – Coordinated Signal System
Warrant 7 – Crash Experience
Warrant 8 – Roadway Network
Warrant 9 – Intersection Near a Railroad Grade Crossing

The most current population estimate for the nearby City of Broken Bow is 4,104 (*US Census Bureau, 2019 US Census*).

<u>US-259 & Driveway 3 Intersection</u>

US-259 is a two-lane undivided highway with a posted speed limit of 55 MPH near the study intersection. US-259 is classified as a Principal Arterial by ODOT. Driveway 3 is proposed south of Pinyon Road and would provide access east of US-259. The proposed site plan depicts Driveway 3 with separate westbound right and left-turn lanes for vehicles exiting the resort. For purposes of this analysis, Driveway 3 was considered a one-lane approach, and the right-turn volumes were not removed from consideration as conflict with right-turning vehicles entering the major roadway is anticipated. A dedicated southbound left-turn lane and dedicated northbound channelized right-turn lane along US-259 are also shown on the site plan.

Warrant 1 – Eight-Hour Vehicular Volume

Warrant 1 is based on the combined volumes from both approaches on the major street and the higher approach volume on the minor street. It also uses the number of lanes for moving traffic on each approach. Either Condition A or Condition B of this warrant must be met for Warrant 1 to be satisfied.

The *MUTCD* allows for the use of a reduced warranting threshold (70%) for intersections where the posted or 85^{th}-percentile speed exceeds 40 MPH or if the intersection is located in a community with a population under 10,000. Since the posted speed on the major street (US-259) does exceed 40 MPH (55 MPH posted), the reduced threshold was used for this warrant.

Condition A of Warrant 1 is met when, for each of any eight hours of an average day, the warranting volumes exist on the major street and on the higher-volume minor street approach to the intersection during the same eight hours. The warranting threshold for a single lane approach on the major street and a single lane approach on the minor street is:

Major Street: 350 vph (total of both approaches)
Minor Street: 105 vph (higher volume approach; one direction only)

Warrant 1A threshold volumes are exceeded for one (1) hour of the day. Eight (8) hours are required for this warrant condition. Warrant 1A is not satisfied at this location.

Condition B of Warrant 1 applies to operating conditions where the major street traffic is so heavy that it creates excessive delay or hazardous conditions for minor street traffic when entering or crossing the major street. The warrant condition is met when, for each of any eight hours of an average day, the warranting volumes exist on the major street and on the higher-volume minor street approach to an intersection. The warranting threshold for a single lane approach on the major street and a single lane approach on the minor street is:

Major Street: 525 vph (total of both approaches)
Minor Street: 53 vph (higher volume approach; one direction only)

Warrant 1B threshold volumes are exceeded for eleven (11) hours of the day. Eight (8) hours are required for this warrant condition. Warrant 1B is satisfied at this location.

Table 1 shows the results of this analysis.

Warrant 1 is MET for this intersection.

Table 1: Warrant Analysis (US-259 & Driveway 3)

US-259 & Driveway 3
US-259: 55 mph
Driveway 3: 25
mph
1 Major / 1 Minor

13 Hour Approach Volumes - Build-Out (2023) Total Conditions

Hour Begin	MAJOR			MINOR			Meets Warrant Volume		
	NB	SB	TOTAL	EB	WB	TOTAL	1A	1B	2
1	43	38	81	0	4	4	-	-	-
2	25	22	47	0	3	3	-	-	-
3	22	20	42	0	2	2	-	-	-
4	20	18	37	0	2	2	-	-	-
5	32	28	60	0	3	3	-	-	-
6	84	74	158	0	9	9	-	-	-
7	178	157	335	0	18	18	-	-	-
8	266	234	499	0	27	27	-	-	-
9	395	347	741	0	40	40	-	-	-
10	546	479	1025	0	55	55	-	Y	-
11	662	718	1380	0	64	64	-	Y	Y
12	726	638	1363	0	73	73	-	Y	Y
13	729	641	1370	0	74	74	-	Y	Y
14	722	634	1356	0	73	73	-	Y	Y
15	824	724	1548	0	83	83	-	Y	Y
16	868	762	1630	0	88	88	-	Y	Y
17	1002	720	1722	0	105	105	Y	Y	Y
18	775	681	1456	0	78	78	-	Y	Y
19	715	629	1344	0	72	72	-	Y	Y
20	585	514	1100	0	59	59	-	Y	-
21	474	416	890	0	48	48	-	-	-
22	330	290	621	0	33	33	-	-	-
23	238	209	447	0	24	24	-	-	-
24	120	106	226	0	12	12	-	-	-
TOTAL	20,570	9,098	19,479	0	1,052	1,052	1	11	9

Warrant 2 – Four-Hour Volumes

Warrant 2 is satisfied when the volumes for any four (4) hours of an average day, when plotted on Figure 4C-1 (or 4C-2 when applicable) of the *MUTCD*, fall above the curve for the appropriate number of lanes. Based on the posted speed limit on US-259 (55 MPH), the reduced warrant threshold was used for this warrant, and Figure 4C-2 was used for the analysis.

Based on the traffic volumes presented in Table 6 and plotted using Figure 4C-2, nine (9) hours of the day fall above the curve for the appropriate number of lanes. Four (4) hours are required for this warrant condition.

Warrant 2 is MET for this intersection.

Warrant 3 – Peak Hour Volume

Warrant 3 is intended for application when traffic conditions are such that for at least one (1) hour of the day, the minor street traffic experiences undue delays entering or crossing the major street. Warrant 3 should only be applied in unusual cases where a "special generator" of traffic exists that will disperse a large number of vehicles over a short time period. Examples of those types of facilities include industrial plants and office complexes.

The Choctaw Nation Hochatown Resort is not anticipated to experience the significant peak hour demands required of a "special generator." Thus, this intersection cannot be considered as part of a "special generator" and cannot be analyzed for Warrant 3.

Warrant 3 is NOT APPLICABLE for this intersection.

Warrant 4 – Minimum Pedestrian Volume

Warrant 4 applies to conditions where the major street traffic is so heavy that pedestrians experience excessive delay in crossing the major street. It is intended for application at an intersection or midblock location and requires that one (1) of the following conditions be met:

1. For each of any 4 hours of an average day, the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding pedestrians per hour crossing the major street (total of all crossings) fall above the curve in Figure 4C-5 (or Figure 4C-6); or

2. For one (1) hour (any four consecutive 15-minute periods) of an average day, the plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding pedestrians per hour crossing the major street (total of all crossings) fall above the curve in Figure 4C-7 (or Figure 4C-8).

This warrant applies only to those locations where the nearest traffic signal along the major street is greater than 300-feet and where a new traffic signal at the study intersection would not unduly restrict platooned flow of traffic.

Pedestrian count data was not collected at this intersection due to lack of pedestrian facilities. Pedestrian volumes of the levels required to satisfy this warrant (93 pedestrians during the peak hour) are not expected to cross the roadways at this intersection.

Warrant 4 was NOT EVALUATED for this intersection.

Warrant 5 – School Crossing

This warrant applies at an established school crossing where a traffic engineering study of the frequency and adequacy of gaps in the vehicular traffic stream as related to the number and size of groups of school children at the school crossing shows that the number of adequate gaps in the traffic during the period when the children are using the crossing is less than the number of minutes in the same period.

This intersection is not an established school crossing.

Warrant 5 is NOT APPLICABLE at this intersection.

Warrant 6 – Coordinated Signal System

Progressive movement control sometimes requires traffic signal installations at intersections where they would not otherwise be warranted in order to maintain proper platooning of vehicles and effectively regulate group speed. This warrant is met when one (1) of the following requirements are met:

1. On a one-way street or a street which has predominantly unidirectional traffic, the adjacent signals are so far apart that they do not provide the required degree of platooning.
2. On a two-way street, adjacent signals do not provide the necessary degree of platooning and the proposed and adjacent signals could constitute a progressive signal system.

This warrant should not be applied where the ultimate signal spacing would be less than 1,000-feet. There is a traffic signal planned at the intersection of US-259 and SH-259A (North), approximately 1,000 feet to the north of Driveway 3. If a traffic signal is installed at Driveway 3, it is recommended these signals be coordinated. No other traffic signals are located near this location.

Warrant 6 is NOT APPLICABLE at this intersection.

Warrant 7 – Crash Experience

The warrant is satisfied when:

1. Adequate trial of less restrictive remedies with satisfactory observance and enforcement has failed to reduce the crash frequency; and
2. Five or more reported crashes of types susceptible to correction by traffic signal control, have occurred within a 12-month period, each crash involving personal injury or property damage apparently exceeding the applicable requirements for a reportable crash; and
3. For each of any 8 hours of an average day, the vph given in both of the 80 percent columns of Condition A in Table 4C-1, or the vph in both of the 80 percent columns of Condition B

in Table 4C-1 exists on the major-street and the higher-volume minor-street approach, respectively, to the intersection, or the volume of pedestrian traffic is not less than 80 percent of the requirements specified in the Pedestrian Volume warrant. These major-street and minor-street volumes shall be for the same 8 hours. On the minor street, the higher volume shall not be required to be on the same approach during each of the 8 hours. If the posted or statutory speed limit or the 85^{th}-percentile speed on the major street exceeds 40 MPH, or if the intersection lies within the built-up area of an isolated community having a population of less than 10,000, the traffic volumes in the 70 percent columns in Table 4C-1 may be used in place of the 80 percent columns.

This intersection does not currently exist, and no collisions have been reported.

Warrant 7 is NOT MET at this intersection.

Warrant 8 – Roadway Network

The systems warrant is intended to encourage concentration and organization of traffic flow networks. This warrant is applicable when the common intersection of <u>two</u> major routes:

1. Has a total existing, or immediately projected, entering volume of at least 1,000 vehicles during the peak hour of a typical weekday and has five-year projected traffic volumes, based on an engineering study, which meet one or more of Warrants 1, 2, and 3 during an average weekday; or
2. Has a total existing or immediately projected entering volume of at least 1,000 vehicles for each of any five hours of a Saturday and/or Sunday.

A major route as used in this signal warrant shall have one or more of the following characteristics:

1. It is part of the street or highway system that serves as the principal roadway network for through traffic flow; or
2. It includes rural or suburban highways outside, entering or traversing a City; or
3. It appears as a major route on an official plan, such as a major street plan in an urban area traffic and transportation study.

Driveway 3 is not considered a major route.

Warrant 8 is NOT APPLICABLE at this intersection.

Warrant 9 – Intersection Near a Railroad Grade Crossing

This signal warrant is intended for use at a location where none of the conditions described in the other eight traffic signal warrants are met, but the proximity to the intersection of a grade crossing on an intersection approach controlled by a 'STOP' or 'YIELD' sign is the principal reason to consider installing a traffic control signal.

The need for a traffic control signal shall be considered if an engineering study finds that both of the following criteria are met:

1. A grade crossing exists on an approach controlled by a 'STOP' or 'YIELD' sign and the center of the track nearest to the intersection is within 140-feet of the stop line or yield line on the approach; and
2. During the highest traffic volume hour during which rail traffic uses the crossing, the plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the minor-street approach that crosses the track (one direction only, approaching the intersection) falls above the applicable curve in Figure 4C-9 or 4C-10 for the existing combination of approach lanes over the track and the distance D, which is the clear storage distance as defined in Section 1A.13 of the MUTCD.

A railroad grade crossing is not located within 140-feet of this intersection.

Warrant 9 is NOT APPLICABLE for this intersection.

Warrant Conclusion

Based on the projected traffic volumes and analysis, traffic signal warrants are satisfied for the intersection of US-259 and Driveway 3 under predicted Build-Out (2023) Total traffic conditions. For purposes of this analysis, Driveway 3 was considered a one-lane approach, and the right-turn volumes were not removed from consideration as conflict with right-turning vehicles entering the major roadway is anticipated. A summary of the traffic signal warrants is provided in **Table 2.**

Table 2: Warrant Summary (US-259 and Driveway 3)

Warrant	Warrant Met?	Notes
1 – Eight-Hour Vehicular Volume	YES	11 hours met (8 required)
2 – Four-Hour Vehicular Volume	YES	9 hours met (4 required)
3 – Peak Hour	N/A	Not considered a special generator
4 – Pedestrian Volume	NOT EVALUATED	Pedestrian data not collected
5 – School Crossing	N/A	Not an established school crossing
6 – Coordinated Signal System	N/A	Not part of a progressive signal system
7 – Crash Experience	NO	Collision history does not meet warrants
8 – Roadway Network	N/A	Not an intersection of two major routes
9 – Near a Grade Crossing	N/A	Not adjacent to a railroad grade crossing

Based on the results of this traffic signal warrant analysis, the installation of a traffic signal at the intersection of US-259 and Driveway 3 is predicted to be warranted with build-out of the proposed development. It is recommended that traffic demands be monitored alongside new development and a traffic signal be installed at this location as development traffic is realized.

SIMTRAFFIC WORKSHEETS

Intersection: 1: US-259 & SH-259A

Movement	WB	WB	NB	NB	SB	SB	SB
Directions Served	L	R	T	TR	L	T	T
Maximum Queue (ft)	89	44	143	116	118	144	92
Average Queue (ft)	39	2	59	40	57	37	24
95th Queue (ft)	77	16	118	91	103	95	66
Link Distance (ft)	491		905	905		1150	1150
Upstream Blk Time (%)							
Queuing Penalty (veh)							
Storage Bay Dist (ft)		300			100		
Storage Blk Time (%)					2	0	
Queuing Penalty (veh)					6	0	

Intersection: 2: DWY 1 & SH-259A

Movement	WB	NB	NB
Directions Served	LT	L	R
Maximum Queue (ft)	52	48	48
Average Queue (ft)	7	21	17
95th Queue (ft)	29	46	43
Link Distance (ft)	1180	343	343
Upstream Blk Time (%)			
Queuing Penalty (veh)			
Storage Bay Dist (ft)			
Storage Blk Time (%)			
Queuing Penalty (veh)			

Intersection: 3: DWY 2 & SH-259A

Movement	WB
Directions Served	LT
Maximum Queue (ft)	35
Average Queue (ft)	3
95th Queue (ft)	17
Link Distance (ft)	551
Upstream Blk Time (%)	
Queuing Penalty (veh)	
Storage Bay Dist (ft)	
Storage Blk Time (%)	
Queuing Penalty (veh)	

Intersection: 4: US-259 & DWY 3

Movement	WB	NB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T
Maximum Queue (ft)	71	87	60	53	80	87
Average Queue (ft)	26	31	14	14	20	24
95th Queue (ft)	58	76	44	41	59	67
Link Distance (ft)	505	700	700		905	905
Upstream Blk Time (%)						
Queuing Penalty (veh)						
Storage Bay Dist (ft)				300		
Storage Blk Time (%)						
Queuing Penalty (veh)						

Network Summary

Network wide Queuing Penalty: 6

Intersection: 1: US-259 & SH-259A

Movement	WB	WB	NB	NB	SB	SB	SB
Directions Served	L	R	T	TR	L	T	T
Maximum Queue (ft)	160	87	174	164	121	171	103
Average Queue (ft)	68	11	79	66	68	51	26
95th Queue (ft)	126	51	159	138	116	120	70
Link Distance (ft)	491		905	905		1150	1150
Upstream Blk Time (%)							
Queuing Penalty (veh)							
Storage Bay Dist (ft)		300			100		
Storage Blk Time (%)					5	0	
Queuing Penalty (veh)					14	0	

Intersection: 2: DWY 1 & SH-259A

Movement	WB	NB	NB
Directions Served	LT	L	R
Maximum Queue (ft)	46	62	60
Average Queue (ft)	8	30	24
95th Queue (ft)	32	53	50
Link Distance (ft)	1180	343	343
Upstream Blk Time (%)			
Queuing Penalty (veh)			
Storage Bay Dist (ft)			
Storage Blk Time (%)			
Queuing Penalty (veh)			

Intersection: 3: DWY 2 & SH-259A

Movement	WB
Directions Served	LT
Maximum Queue (ft)	36
Average Queue (ft)	3
95th Queue (ft)	18
Link Distance (ft)	551
Upstream Blk Time (%)	
Queuing Penalty (veh)	
Storage Bay Dist (ft)	
Storage Blk Time (%)	
Queuing Penalty (veh)	

Intersection: 4: US-259 & DWY 3

Movement	WB	NB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T
Maximum Queue (ft)	94	142	124	67	87	101
Average Queue (ft)	40	59	35	24	29	34
95th Queue (ft)	77	119	86	53	71	82
Link Distance (ft)	505	700	700		905	905
Upstream Blk Time (%)						
Queuing Penalty (veh)						
Storage Bay Dist (ft)				300		
Storage Blk Time (%)						
Queuing Penalty (veh)						

Network Summary

Network wide Queuing Penalty: 15

Intersection: 1: US-259 & SH-259A

Movement	WB	WB	NB	NB	SB	SB	SB
Directions Served	L	R	T	TR	L	T	T
Maximum Queue (ft)	151	91	236	230	124	301	233
Average Queue (ft)	66	14	92	81	88	82	43
95th Queue (ft)	122	61	189	179	134	203	133
Link Distance (ft)	491		905	905		1150	1150
Upstream Blk Time (%)							
Queuing Penalty (veh)							
Storage Bay Dist (ft)		300			100		
Storage Blk Time (%)					12	1	
Queuing Penalty (veh)					42	2	

Intersection: 2: DWY 1 & SH-259A

Movement	EB	WB	NB	NB
Directions Served	TR	LT	L	R
Maximum Queue (ft)	2	94	64	58
Average Queue (ft)	0	21	28	22
95th Queue (ft)	2	63	56	47
Link Distance (ft)	551	1180	343	343
Upstream Blk Time (%)				
Queuing Penalty (veh)				
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

Intersection: 3: DWY 2 & SH-259A

Movement	WB
Directions Served	LT
Maximum Queue (ft)	80
Average Queue (ft)	11
95th Queue (ft)	46
Link Distance (ft)	551
Upstream Blk Time (%)	
Queuing Penalty (veh)	
Storage Bay Dist (ft)	
Storage Blk Time (%)	
Queuing Penalty (veh)	

Intersection: 4: US-259 & DWY 3

Movement	WB	NB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T
Maximum Queue (ft)	74	167	151	129	90	97
Average Queue (ft)	36	75	46	53	23	34
95th Queue (ft)	68	139	109	103	64	81
Link Distance (ft)	505	700	700		905	905
Upstream Blk Time (%)						
Queuing Penalty (veh)						
Storage Bay Dist (ft)				300		
Storage Blk Time (%)						
Queuing Penalty (veh)						

Network Summary

Network wide Queuing Penalty: 44

Intersection: 1: US-259 & SH-259A

Movement	WB	WB	NB	NB	SB	SB	SB
Directions Served	L	R	T	TR	L	T	T
Maximum Queue (ft)	164	91	155	157	125	263	191
Average Queue (ft)	77	16	64	54	92	76	42
95th Queue (ft)	138	62	133	119	138	188	119
Link Distance (ft)	491		905	905		1150	1150
Upstream Blk Time (%)							
Queuing Penalty (veh)							
Storage Bay Dist (ft)		300			100		
Storage Blk Time (%)					11	0	
Queuing Penalty (veh)					36	1	

Intersection: 2: DWY 1 & SH-259A

Movement	WB	NB	NB
Directions Served	LT	L	R
Maximum Queue (ft)	102	108	68
Average Queue (ft)	21	42	29
95th Queue (ft)	63	82	55
Link Distance (ft)	1180	343	343
Upstream Blk Time (%)			
Queuing Penalty (veh)			
Storage Bay Dist (ft)			
Storage Blk Time (%)			
Queuing Penalty (veh)			

Intersection: 3: DWY 2 & SH-259A

Movement	WB
Directions Served	LT
Maximum Queue (ft)	77
Average Queue (ft)	11
95th Queue (ft)	47
Link Distance (ft)	551
Upstream Blk Time (%)	
Queuing Penalty (veh)	
Storage Bay Dist (ft)	
Storage Blk Time (%)	
Queuing Penalty (veh)	

Intersection: 4: US-259 & DWY 3

Movement	WB	NB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T
Maximum Queue (ft)	114	130	102	79	90	106
Average Queue (ft)	51	52	28	33	30	41
95th Queue (ft)	93	100	71	68	72	88
Link Distance (ft)	505	700	700		905	905
Upstream Blk Time (%)						
Queuing Penalty (veh)						
Storage Bay Dist (ft)				300		
Storage Blk Time (%)						
Queuing Penalty (veh)						

Network Summary

Network wide Queuing Penalty: 37

Intersection: 1: US-259 & SH-259A

Movement	WB	WB	NB	NB	SB	SB	SB
Directions Served	L	R	T	TR	L	T	T
Maximum Queue (ft)	159	112	192	189	125	384	307
Average Queue (ft)	75	16	72	62	108	141	70
95th Queue (ft)	136	67	155	142	143	323	210
Link Distance (ft)	491		905	905		1150	1150
Upstream Blk Time (%)							
Queuing Penalty (veh)							
Storage Bay Dist (ft)		300			100		
Storage Blk Time (%)					21	1	
Queuing Penalty (veh)					76	5	

Intersection: 2: DWY 1 & SH-259A

Movement	EB	WB	NB	NB
Directions Served	TR	LT	L	R
Maximum Queue (ft)	6	171	117	54
Average Queue (ft)	0	45	43	24
95th Queue (ft)	5	119	92	49
Link Distance (ft)	551	1180	343	343
Upstream Blk Time (%)				
Queuing Penalty (veh)				
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

Intersection: 3: DWY 2 & SH-259A

Movement	WB
Directions Served	LT
Maximum Queue (ft)	94
Average Queue (ft)	20
95th Queue (ft)	64
Link Distance (ft)	551
Upstream Blk Time (%)	
Queuing Penalty (veh)	
Storage Bay Dist (ft)	
Storage Blk Time (%)	
Queuing Penalty (veh)	

Intersection: 4: US-259 & DWY 3

Movement	WB	NB	NB	SB	SB	SB
Directions Served	L	T	T	L	T	T
Maximum Queue (ft)	105	130	119	126	92	107
Average Queue (ft)	41	57	35	53	22	38
95th Queue (ft)	80	110	87	102	66	86
Link Distance (ft)	505	700	700		905	905
Upstream Blk Time (%)						
Queuing Penalty (veh)						
Storage Bay Dist (ft)				300		
Storage Blk Time (%)						
Queuing Penalty (veh)						

Network Summary

Network wide Queuing Penalty: 81