Section 3L:

Isolated Intersections

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

INTRODUCTION

This section documents the results of traffic operations evaluations for several isolated intersections in Auburn, Alabama. The intersections identified for analysis include:

- Cox Road at Longleaf Drive
- Martin Luther King Drive/AL-14 at Webster Road
- Martin Luther King Drive/AL-14 at Shug Jordan Parkway Ramps
- E. Thach Avenue at S. Ross Street
- Gay Street at Shelton Mill Road
- Martin Luther King Drive/AL-14 at Willow Creek Road

The locations of the isolated study intersections are illustrated in **Figure 1**.

To accomplish the traffic operations evaluations for the isolated intersections, the following tasks were undertaken:

- existing peak hour turning movement counts were conducted for the study intersections;
- capacity analyses were conducted for the study intersections;
- current traffic operational deficiencies were identified;
- projections for ten (10) year growth in traffic for the study intersections were developed;
- geometric and traffic control improvements were developed for the study intersections to address traffic operational and safety deficiencies for existing and projected ten (10) year conditions.

Sources of information used in this section include: The City of Auburn, Alabama; the Alabama Department of Transportation; the Institute of Transportation Engineers; American Association of State Highway and Transportation Officials; the Manual on Uniform Traffic Control Devices; the Transportation Research Board; and the files and field reconnaissance efforts of Skipper Consulting, Inc.

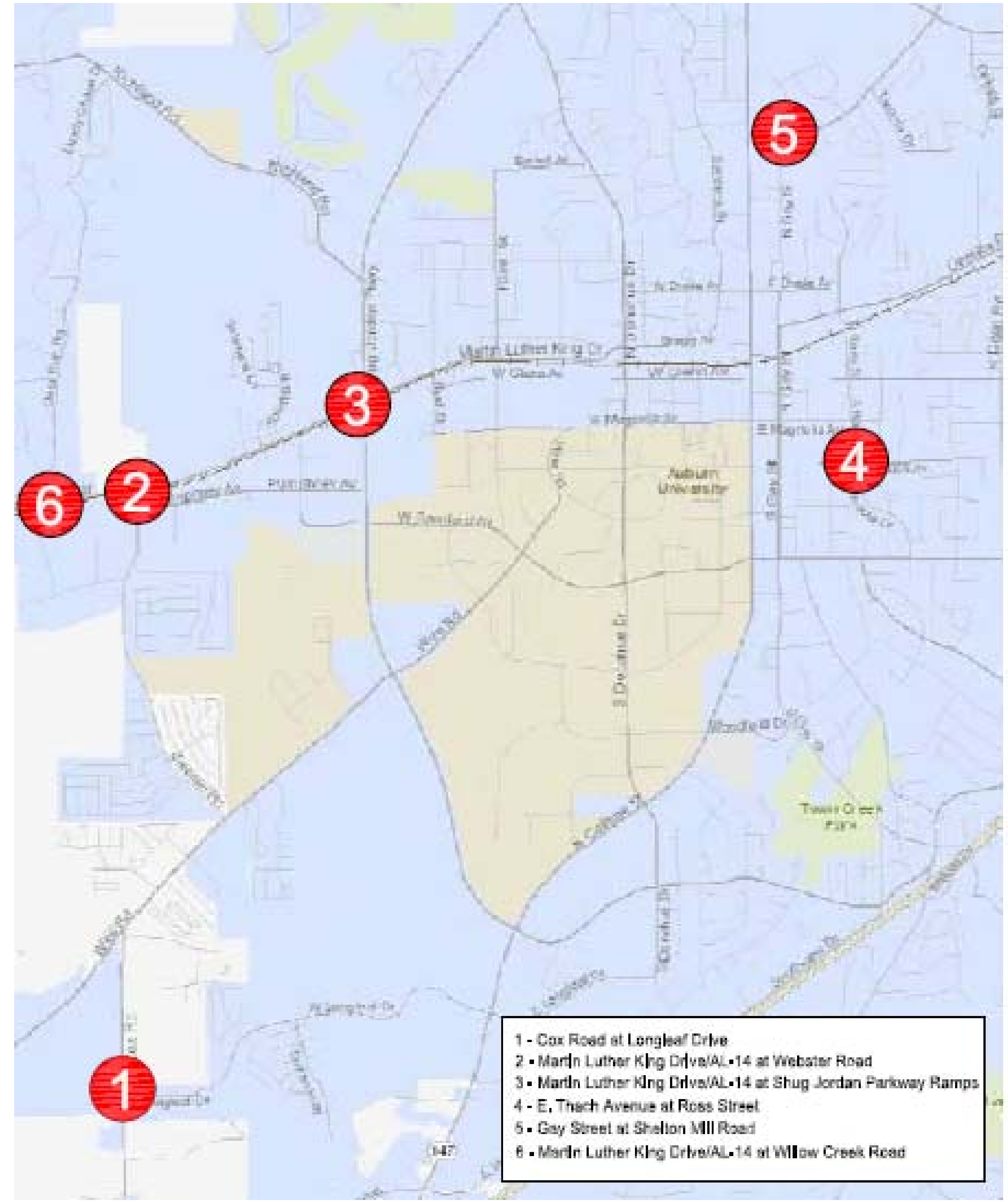


Figure 1 - Isolated Study Intersections

ISOLATED INTERSECTION #1: COX ROAD AT LONGLEAF DRIVE

BACKGROUND INFORMATION

Study Intersection and Roadways

Cox Road is a major collector roadway from Wire Road through the I-85 interchange and to the intersection with S. College Street (where the Cox Road designation ends; the road continues as County Road 10 to its intersection with County Road 54 and is a major collector the entire length). Cox Road is moderately developed near the I-85 interchange but is sparsely developed primarily with single family residences between I-85 and Wire Road. Near the study intersection, Cox Road is a three-lane roadway, including a center turn lane, with a posted speed limit of 40mph.

Longleaf Drive is a local road connecting Cox Road to S. College Street. Between Cox Road and S. College Street, Longleaf Drive serves a few large residential developments but otherwise is surrounded by undeveloped land. Longleaf Drive is a three-lane cross section with a posted speed limit of 35mph.

A traffic signal was installed at this intersection in early 2016. The southbound approach of Cox Road includes a left turn lane and operates under protective/permissive control via a five-section signal head. Both Cox Road northbound and Longleaf Drive operate permissively. Currently, there are no marked crosswalks on the roadways, no sidewalks in the vicinity, nor pedestrian signal heads or pushbuttons in place.

Characteristics of the roadways within the vicinity are summarized in **Table 1**.

Table 1 - Corridor Roadway Characteristics – Cox Rd at Longleaf Dr

Roadway	Parking	# of Lanes	Travel Direction	Posted Speeds (mph)	Classification
Cox Road (Either Side of Longleaf Dr)	None	2 (w/ left turn lanes)	North/South	40	Major Collector
Longleaf Drive (At Cox Road)	None	3 (w/ TWLTL)	East/West	35	Local Road

Peak Hour Traffic Counts

Morning (7:00-9:00 am) and afternoon (4:00-6:00 pm) peak hour turning movement counts were conducted at the study intersection on October 11, 2016. In addition, twenty-four hour approach counts were collected at the intersection on April 4, 2018. Traffic count data utilized for the analyses of these intersections is summarized in **Figure 2**.

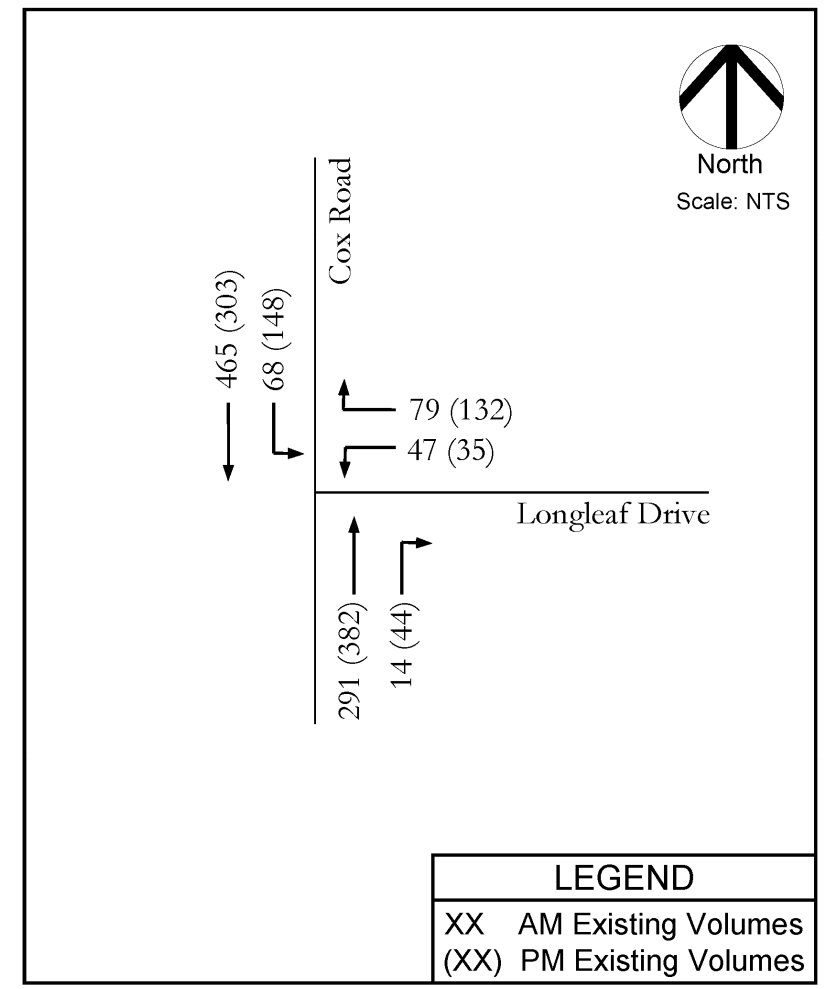


Figure 2 - Existing Peak Hour Traffic Volumes – Cox Rd at Longleaf Dr

EXISTING CONDITIONS ANALYSES

Existing Intersection Capacity Analysis

Capacity analyses for peak hour conditions at the intersection of Cox Road and Longleaf Drive were conducted for the morning and afternoon peak hour periods using methods outlined in the *Highway Capacity Manual, 2000*. According to methods of the *Highway Capacity Manual*, capacity is expressed as levels of service ranging from "A" (best) through "F" (worst). In general, a level of service "C" is considered desirable while a level of service "D" is considered acceptable during peak hour operations. Results of these capacity analyses for existing conditions are summarized in **Table 2**.

Table 2 - Existing Intersection Levels of Service – Cox Rd at Longleaf Dr

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
Cox Road at Longleaf Drive (signalized	WB Longleaf Dr	Left	C	C
		Right	B	B
		Overall	C	C
	NB Cox Rd	Through/Right	A	B
	SB Cox Rd	Left	A	A
		Through	A	A
		Overall	A	A
	Overall LOS		**A**	**B**

As shown in **Table 2**, each movement/lane group at the intersection currently operates at acceptable levels of service for both peak periods evaluated.

Right-Turn Lane Warrant Evaluations

Existing peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for the following approaches:

- Northbound Cox Road at Longleaf Drive

The results of these comparisons indicate:

- Northbound Cox Road at Longleaf Drive – During both the morning and afternoon peak hours, existing traffic volumes are not sufficient to meet the criteria for a right-turn lane.

Intersection Crash Evaluation

Skipper Consulting, Inc. performed a citywide crash study for intersections and roadway segments maintained by the City of Auburn. The results of this crash study have been documented in a separate bound report. The citywide crash study initially included the Cox Road at Longleaf Drive intersection. Screening procedures and crash analyses were conducted to determine any locations that are worthy of safety-based roadway improvements. The Cox Road at Longleaf Drive intersection was eliminated from further analysis during the initial screening process due to a low crash rate.

EXISTING CONDITIONS ANALYSES WITH IMPROVEMENTS

Recommended Improvements

Based on the results of the existing conditions analysis, there are no obvious deficiencies that need to be mitigated. No recommended improvements are suggested at this stage of analysis.

PROJECTED TRAFFIC GROWTH

Growth rates were calculated for the study roadways based on historical traffic volumes and growth trends. The historical growth rate calculated for roadways in the vicinity of the Cox Road at Longleaf Drive intersection was 0.9% per year. The annual growth rate was applied for a ten (10) year period to result in an overall growth rate of 9.0% percent for study intersection traffic volumes. Existing peak hour traffic volumes were increased 9.0% to reflect ten (10) year projected traffic volumes for the study intersection. The projected future peak hour traffic volumes are illustrated in **Figure 3.**

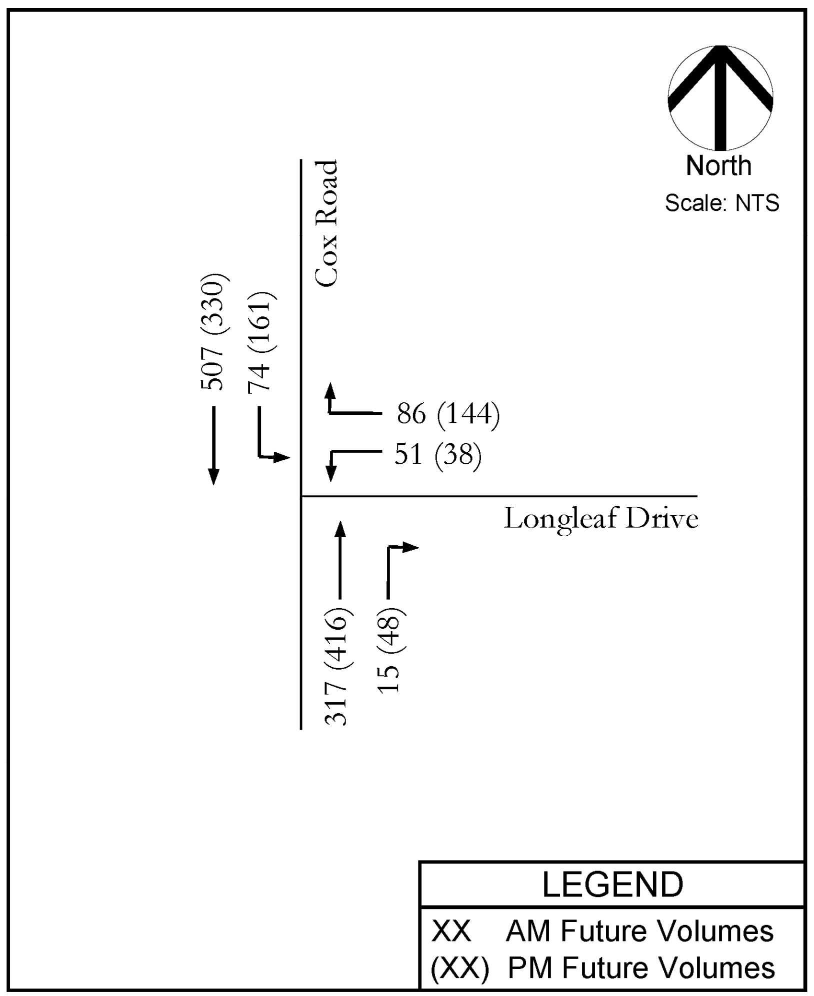


Figure 3 - Future Peak Hour Traffic Volumes – Cox Rd at Longleaf Dr

ANALYSES WITH PROJECTED TRAFFIC GROWTH

Analyses conducted for this scenario assumes projected traffic volumes for ten (10) years would be in place.

Intersection Capacity Analysis with Projected Traffic Growth

Capacity analyses for projected ten (10) year peak hour conditions were conducted for the study intersection using methods outlined in the *Highway Capacity Manual, 2000*. Results of these capacity analyses are summarized in **Table 3**.

Table 3 - Intersection Levels of Service w/Projected Traffic Growth – Cox Rd at Longleaf Dr

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
Cox Road at Longleaf Drive (signalized	WB Longleaf Dr	Left	B	C
		Right	B	B
		Overall	B	C
	NB Cox Rd	Through/Right	B	B
	SB Cox Rd	Left	A	A
		Through	A	A
		Overall	A	A
	Overall LOS		**A**	**B**

As shown in **Table 3**, all the study intersection operates with overall acceptable levels of service for both future peak periods evaluated.

Right-Turn Lane Warrant Evaluations with Projected Traffic Growth

Future peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for the following approaches:

- Northbound Cox Road at Longleaf Drive

The results of these comparisons indicate:

- Northbound Cox Road at Longleaf Drive – During both the morning and afternoon peak hours, future traffic volumes are not sufficient to meet the criteria for a right-turn lane.

RECOMMENDED IMPROVEMENTS WITH PROJECTED TRAFFIC GROWTH

Based upon the analyses and evaluations conducted for the study intersection for existing conditions and projected ten (10) year conditions, recommendations are made help improve traffic operations and to address any capacity or safety deficiencies identified. The recommended improvements are as follows:

- Modify signal split times based on the future volumes to reduce intersection delay

ANALYSES WITH RECOMMENDED IMPROVEMENTS & PROJECTED TRAFFIC GROWTH

Intersection Capacity Analysis with Improvements and Projected Traffic Growth

Capacity analyses were conducted for the study intersections assuming recommended improvements (outlined above) and projected ten (10) traffic volumes would be in place. Capacity analyses were conducted using methods of the *Highway Capacity Manual*, as previously introduced. **Table 4** provides a summary of the levels of service for the study intersection with recommended improvements and projected ten (10) traffic volumes in place.

Table 4 - Intersection Levels of Service w/Improvements and Projected Traffic Growth

Cox Rd at Longleaf Dr

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
Cox Road at Longleaf Drive (signalized)	WB Longleaf Dr	Left	*C*	C
		Right	*B*	C
		Overall	*B*	C
	NB Cox Rd	Through/Right	B	B
	SB Cox Rd	Left	A	A
		Through	A	A
		Overall	A	A
	Overall LOS		**A**	**B**

Table 4 indicates the intersection would operate with overall levels of service "D" or better with the recommended improvements and projected traffic volumes in place.

ISOLATED INTERSECTION #2: MARTIN LUTHER KING DRIVE/AL-14 AT WEBSTER ROAD

BACKGROUND INFORMATION

Study Intersection and Roadways

Martin Luther King Drive/AL-14 is a minor principal arterial roadway in the vicinity of the Webster Road intersection. MLK/AL-14, oriented east/west, is a two-lane cross section, with a westbound left turn lane with 215' storage/100' taper provided at Webster Road, and a posted speed limit of 45 mph. In the immediate vicinity of the intersection, there are industrial land uses on the south side of the roadway and a single restaurant on the north side.

Webster Road, oriented north/south, is a major collector roadway with a three-lane cross section; however, at the intersection with MLK/AL-14, Webster Road has only two lanes.

The north leg of the intersection is a driveway serving an individual restaurant. The driveway is paved; however, a Two-Direction Large Arrow sign (W1-7) is installed in the center of the driveway facing northbound Webster Road traffic.

The intersection operates as a two way stop control configuration with stop signs installed on the minor streets.

Characteristics of the roadways of the study intersection are summarized in **Table 5**.

Table 5 - Corridor Roadway Characteristics – MLK/AL-14 at Webster Rd

Roadway	Parking	# of Lanes	Travel Direction	Posted Speeds (mph)	Classification
MLK/AL-14 (Either Side of Webster Road)	None	2 (w/ WB left turn lanes)	East/West	45	Minor Principal Arterial
Webster Road (At MLK/AL-14)	None	3	North/South	35	Major Collector

Existing Webster Road Highway-Rail Grade Crossing

CSX Transportation operates an at-grade railroad crossing (Crossing #831203Y) located on Webster Road, measuring approximately 30' between the MLK/AL-14 edge of pavement and the nearest rail. The crossing is equipped with active warning devices including lights, bells, and gates. According to the current crossing inventory form (obtained from the Federal Railroad Administration website), approximately seven (7) trains pass through the crossing daily at a maximum speed of 45 mph.

Of note, the crossing appears to have used passive control measures until being converted to an active crossing with the installation of lights/bells/gates around 1997.

Railroad crossing advanced warning signs and markings are in place on Webster Road and both approaches of MLK/AL-14, although, they are not in conformance with current ALDOT standards.

Two issues were identified during review of the crossing geometric conditions. First, the crossing elevation is approximately 3'-4' feet above the MLK/AL-14 edge of pavement. Low Ground Clearance Grade Crossing signs (W10-5) are installed on both approaches of Webster Road to warn trucks; however, gouges in the asphalt serve as evidence of the low clearance issue. Secondly, 30' between the rail and MLK/AL-14 is an insufficient amount of storage for a tractor-trailer (WB-67) type vehicle. If a truck is stopped on Webster Road waiting to enter MLK/AL-14, the trailer extends across the railroad tracks. Truck/train collisions of this type are documented in the crash data reviewed as a part of this study. The crash data is detailed further in a subsequent section of this report.

Bicycle and Pedestrian Accommodations

The MLK/AL-14 at Webster Road intersection includes some bicycle and pedestrian accommodations. In the northwest quadrant, the Roberta Jackel Bike Path is a multi-use path, marked and signed, which continues to the west approximately 1 mile to Chadwick Lane.

On the west leg of the intersection, a crosswalk is in place which connects the Roberta Jackel Bike Path to the south side of MLK/AL-14. There is no path or sidewalk to connect with in the southwest quadrant.

Peak Hour Traffic Counts

Morning (7:00-9:00 am) and afternoon (4:00-6:00 pm) peak hour turning movement counts were conducted at the study intersection on April 26, 2018. In addition, twenty-four hour approach counts were collected at the intersection on April 4, 2018. Traffic count data utilized for the analyses of these intersections is summarized in **Figure 4**.

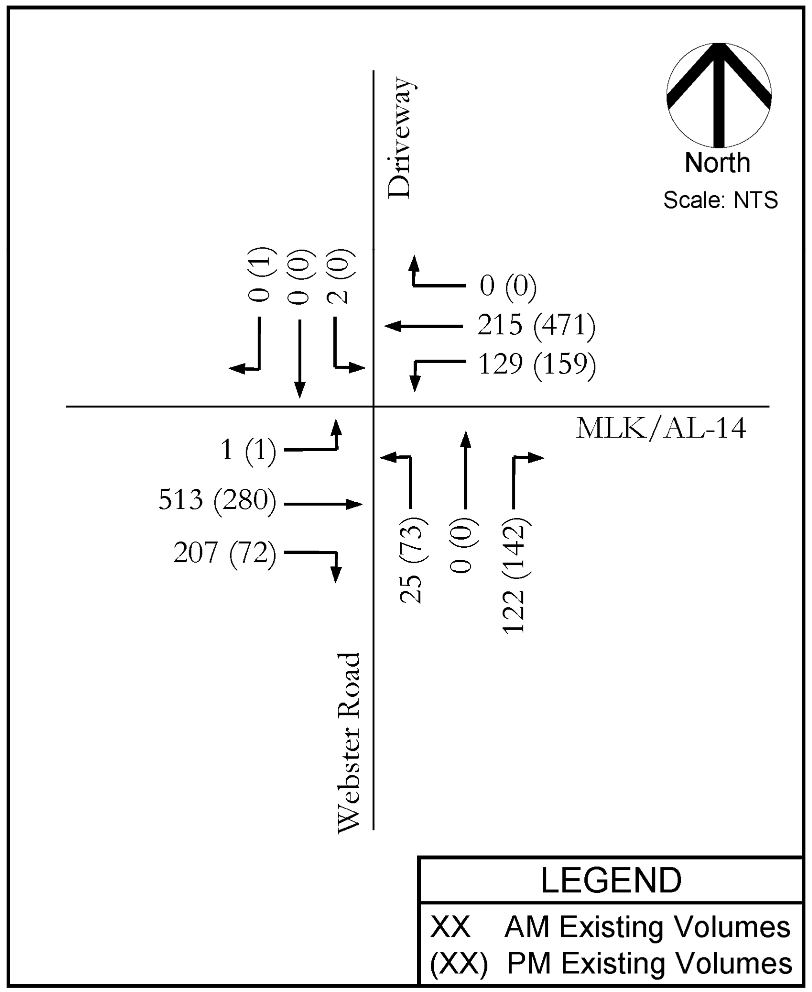


Figure 4 - Existing Peak Hour Traffic Volumes - MLK/AL-14 at Webster Rd

EXISTING CONDITIONS ANALYSES

Existing Intersection Capacity Analysis

Capacity analyses for peak hour conditions at the intersection of MLK/AL-14 and Webster Road were conducted for the morning and afternoon peak hour periods using methods outlined in the *Highway Capacity Manual, 2000*. According to methods of the *Highway Capacity Manual*, capacity is expressed as levels of service ranging from "A" (best) through "F" (worst). In general, a level of service "C" is considered desirable while a level of service "D" is considered acceptable during peak hour operations. Results of these capacity analyses for existing conditions are summarized in **Table 6**.

Table 6 - Existing Intersection Levels of Service - MLK/AL-14 at Webster Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Webster Road (unsignalized)	EB MLK/AL-14	Left/Through/Right	A	A
	WB MLK/AL-14	Left	B	A
		Through/Right	--	--
	NB Webster Rd	Left/Through/Right	D	F
	SB Driveway	Left/Through/Right	E	B

As shown in **Table 6**, the MLK/SR-14 approaches to the intersection operate at acceptable levels of service. The restaurant driveway operates at a level of service "E" during the a.m. peak period, and Webster Road operates at a level of service "F" during the p.m. peak period.

Right-Turn Lane Warrant Evaluations

Existing peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for the following approaches:

- Eastbound MLK/AL-14 at Webster Road

The results of these comparisons indicate:

- Eastbound MLK/AL-14 – During both the morning and afternoon peak hours, existing traffic volumes are sufficient to meet the criteria for a right-turn lane.

Although an eastbound right turn lane is warranted, geometric conditions appear to make construction impossible. It would require significant horizontal and vertical realignment of MLK/AL-14 and potential relocation of railroad equipment including the north gate arm assembly. Due to the constructability issue, the eastbound right turn lane is NOT included as a recommendation.

Intersection Crash Evaluation

Skipper Consulting, Inc. performed a citywide crash study for intersections and roadway segments maintained by the City of Auburn. The results of this crash study have been documented in a separate bound report. The citywide crash study initially included the MLK/AL-14 at Webster Road intersection. Screening procedures and crash analyses were conducted to determine any locations that are worthy of safety-based roadway improvements. This particular intersection was eliminated from further analysis during the initial screening process due to a low crash rate.

Railroad Crossing Related Crash Evaluation

Due to the proximity of the railroad crossing, Skipper Consulting, Inc. performed a more in-depth analysis of available crash data to identify crashes directly related to the railroad crossing. In particular, the FRA accident reports as well as the most recent crash reports from the City of Auburn were obtained and reviewed.

The FRA accident data primarily documents the occurrence of a collision between a train and a vehicle or other object at the crossing. This data included 11 accidents from 1982 to 2015. Of the 11 total accidents, 2 accidents involved tractor-trailers, and only one accident resulted in an injury.

The City of Auburn provided actual crash reports (2014 – 2017) for crashes that occurred near the intersection in order to review the details and narrative of the crashes. Between 2014 and 2017, there were 18 total crashes, of which, only one crash was directly related to a train on the railroad crossing.

Signal Warrant Evaluation and Results

Due to the existing poor levels of service at the intersection and its proximity to the railroad crossing, Skipper Consulting, Inc. performed a signal warrant analysis based upon the criteria for the installation of traffic signals presented in the latest edition of the *Manual on Uniform Traffic Control Devices (MUTCD)*, as published by the Federal Highway Administration. To warrant traffic signalization, an intersection must satisfy one or more of the nine warrants presented in the MUTCD. For the purpose of this study process, the traffic signal warrant evaluations were limited to applicable warrants for the study intersection, specifically:

- Warrant 1 – Eight Hour Vehicular Volume
- Warrant 2 – Four Hour Vehicular Volume
- Warrant 9 – Intersection Near a Grade Crossing

The existing hourly approach volumes are shown in **Table 7**.

It should be noted that the signal warrant analysis was undertaken primarily due to the grade crossing rather than the traffic volumes. However, for Warrant 1 and Warrant 2, no right turn reduction was applied to the Webster Road approach since there is no right turn lane provided and because the grade crossing impacts the right turn maneuver. The results of the signal warrant evaluations are summarized in **Table 8**.

Table 7 – Existing Hourly Approach Volumes - MLK/AL-14 at Webster Rd

Hour	WEBSTER RD	MLK/AL-14	
	NB	EB	WB
12:00 AM-1:00AM	36	15	42
1:00AM-2:00AM	19	9	22
2:00AM-3:00AM	18	9	15
3:00AM-4:00AM	19	22	19
4:00AM-5:00AM	32	60	41
5:00AM-6:00AM	63	163	118
6:00AM-7:00AM	145	434	241
7:00AM-8:00AM	221	749	357
8:00AM-9:00AM	150	437	271
9:00AM-10:00AM	120	295	228
10:00AM-11:00AM	123	293	239
11:00AM-NOON	201	306	355
NOON-1:00PM	190	321	404
1:00PM-2:00PM	171	286	371
2:00PM-3:00PM	189	327	413
3:00PM-4:00PM	342	352	473
4:00PM-5:00PM	393	342	650
5:00PM-6:00PM	298	311	659
6:00PM-7:00PM	236	293	428
7:00PM-8:00PM	129	168	354
8:00PM-9:00PM	93	115	319
9:00PM-10:00PM	69	86	211
10:00PM-11:00PM	61	50	122
11:00PM-12:00M	40	29	82
TOTAL	**3,358**	**5,472**	**6,434**

Table 8 - Traffic Signal Warrant Evaluation Results MLK/AL-14 at Webster Rd

Traffic Signal Warrant	*MLK/AL-14 At Webster Road*	
	Warrant Met	*Warrant Not Met*
Warrant 1 – Eight Hour Vehicular Volume	X	
Warrant 2 – Four Hour Vehicular volume	X	
Warrant 9 – Intersection Near a Grade Crossing	X	

EXISTING CONDITIONS ANALYSES WITH IMPROVEMENTS

Recommended Improvements

Roadway and traffic control improvements have been developed to help address capacity and safety deficiencies identified in the capacity analyses conducted for the study intersection. The following outlines the recommended improvements for existing conditions at the MLK/AL-14 at Webster Road intersection:

- Install a traffic signal which includes railroad pre-emption. Signalization will provide acceptable levels of service on all intersection approaches and the railroad pre-emption will help reduce crashes at the grade crossing. Blank out signs and other appurtenances will be necessary.
- If the intersection is signalized the crosswalk should be retained and pedestrian features should be added per ADA requirements. These features would include an all-weather landing in the SW quadrant, pedestrian heads, pedestrian pushbuttons, etc. The crosswalk should be restriped as a high-visibility crosswalk.
- On the Webster Road NB approach, the Stop sign located south of the railroad tracks should be removed. It may be replaced with the Stop Here When Flashing (R8-10a) or similar sign.
- The railroad crossing signing and marking should be modified and brought up to current standards.

A conceptual drawing of the recommended improvements is included in **Figure 5**.

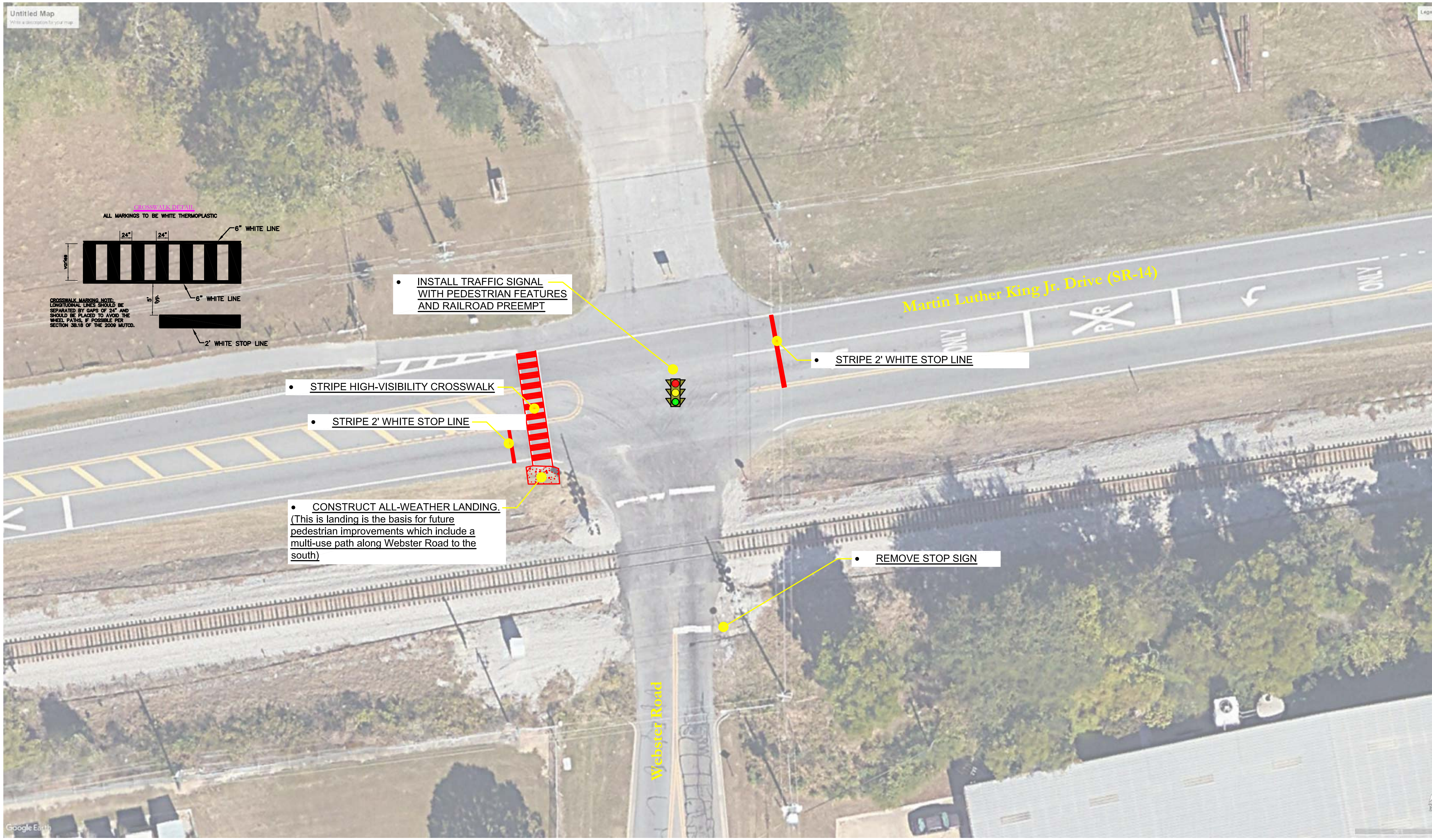
Existing Intersection Capacity Analysis with Improvements

Capacity analyses for peak hour conditions at the study intersection were conducted assuming improvements for existing conditions would be in place. Capacity analyses were conducted using methods outlined in the *Highway Capacity Manual, 2000.* Results of these capacity analyses are summarized in **Table 9**.

Table 9 - Existing Intersection Levels of Service with Improvements - MLK/AL-14 at Webster Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Webster Road (signalized)	EB MLK/AL-14	Left/Through/Right	B	B
	WB MLK/AL-14	Left	A	A
		Through/Right	A	A
		Overall	A	A
	NB Webster Rd	Left/Through/Right	C	B
	SB Driveway	Left/Through/Right	C	B
	Overall LOS		**B**	**B**

As shown in **Table 9**, each movement/lane group at the study intersection operates at acceptable levels of service for both peak periods evaluated with the recommended improvements in place.



SKIPPER CONSULTING INC

Legend

North

Graphic Scale: NTS

Figure 5
MLK Drive at Webster Road Recommended Improvements

Auburn, Alabama

March 2019

PROJECTED TRAFFIC GROWTH

Growth rates were calculated for the study roadways based on historical traffic volumes and growth trends. The historical growth rate calculated for roadways in the vicinity of the MLK/AL-14 at Webster Road intersection was 2.7% per year. The annual growth rate was applied for a ten (10) year period to result in an overall growth rate of 27.0% percent for study intersection traffic volumes. Existing peak hour traffic volumes were increased 27.0% to reflect ten (10) year projected traffic volumes for the study intersection. The projected future peak hour traffic volumes are illustrated in **Figure 6.**

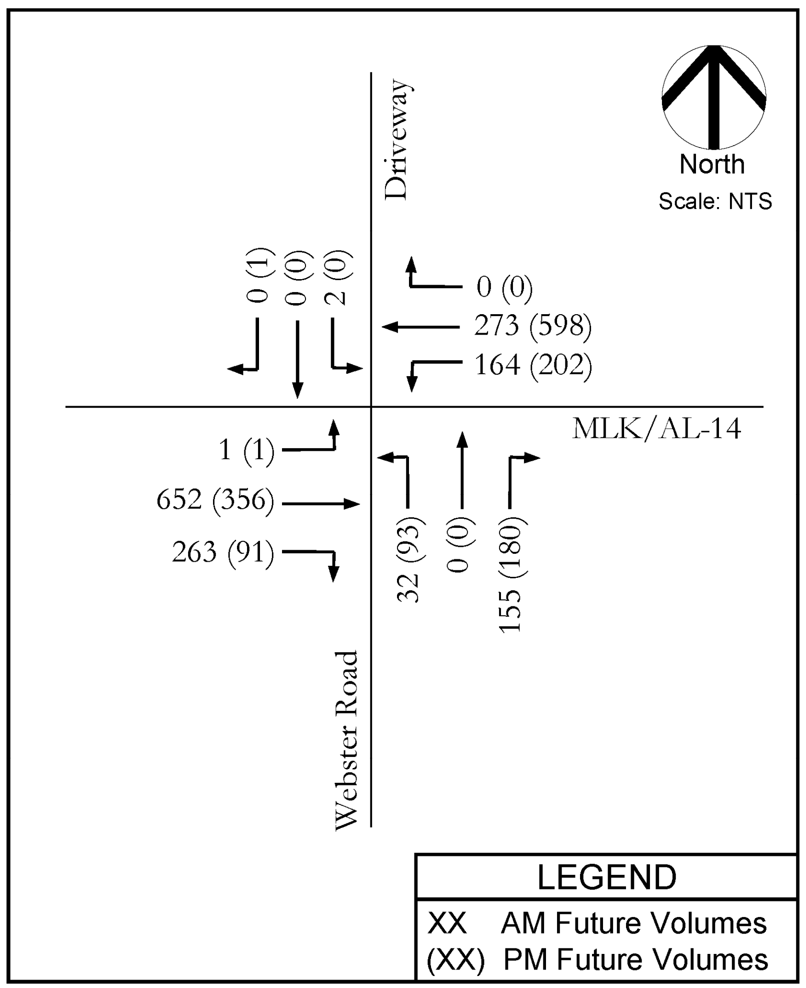


Figure 6 - Future Peak Hour Traffic Volumes - MLK/AL-14 at Webster Rd

ANALYSES WITH PROJECTED TRAFFIC GROWTH

Analyses conducted for this scenario assumes projected traffic volumes for ten (10) years would be in place and the improvements recommended for existing conditions would also be in place.

Intersection Capacity Analysis with Projected Traffic Growth

Capacity analyses for projected ten (10) year peak hour conditions were conducted for the study intersection using methods outlined in the *Highway Capacity Manual, 2000*. Results of these capacity analyses are summarized in **Table 10.**

Table 10 - Intersection Levels of Service w/ Projected Traffic Growth - MLK/AL-14 at Webster Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Webster Road (Signalized)	EB MLK/AL-14	Left/Through/Right	C	B
	WB MLK/AL-14	Left	A	A
		Through/Right	A	A
		Overall	A	A
	NB Webster Rd	Left/Through/Right	C	B
	SB Driveway	Left/Through/Right	C	B
	Overall LOS		**C**	**B**

As shown in **Table 10**, each movement/lane group at the study intersection operates at acceptable levels of service for both peak periods evaluated.

RECOMMENDED IMPROVEMENTS WITH PROJECTED TRAFFIC GROWTH

Based upon the analyses and evaluations conducted for the study intersection for existing conditions and projected ten (10) year conditions, recommendations are made help improve traffic operations and to address any capacity or safety deficiencies identified. The recommended improvements are as follows:

- Modify signal cycle and split times based on the future volumes to reduce intersection delay

ANALYSES WITH RECOMMENDED IMPROVEMENTS & PROJECTED TRAFFIC GROWTH

Intersection Capacity Analysis with Improvements and Projected Traffic Growth

Capacity analyses were conducted for the study intersections assuming recommended improvements (outlined above) and projected ten (10) traffic volumes would be in place. Capacity analyses were conducted using methods of the *Highway Capacity Manual, 2000* as previously introduced. **Table 11** provides a summary of the levels of service for the study intersection with recommended improvements and projected ten (10) year traffic volumes in place.

Table 11 - Intersection Levels of Service with Improvements and Projected Traffic Growth

MLK/AL-14 at Webster Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Webster Road (Signalized)	EB MLK/AL-14	Left/Through/Right	B	B
	WB MLK/AL-14	Left	A	A
		Through/Right	A	A
		Overall	A	A
	NB Webster Rd	Left/Through/Right	C	C
	SB Driveway	Left/Through/Right	C	B
	Overall LOS		**B**	**B**

As shown in **Table 11**, each movement/lane group at the study intersection operates at acceptable levels of service for both peak periods evaluated with the recommended improvements in place and the projected ten (10) year traffic volumes in place.

ISOLATED INTERSECTION #3: MLK/AL-14 AT SHUG JORDAN PARKWAY RAMPS

BACKGROUND INFORMATION

Study Intersection and Roadways

Martin Luther King Drive/AL-14 is a minor arterial roadway west of the Shug Jordan Parkway Ramps intersection and a major collector east of the intersection. MLK/AL-14, oriented east/west, is a three-lane cross section, including an eastbound left turn lane with 410' storage/100' taper and a posted speed limit of 45 mph. In the immediate vicinity of the intersection, there are primarily single family residential land uses on the north side of MLK/AL-14. The CSX Railroad runs parallel to MLK/AL-14 on the south side of the road. MLK/AL-14 westbound includes a right turn lane with approximately 150' storage/125' taper and is channelized via painted island to a yield sign and marking at the Shug Jordan Parkway Ramps northbound lane.

The Shug Jordan Parkway Ramps connect Shug Jordan Parkway to MLK/AL-14. The ramps are similar to a two-lane service road, approximately 1,300' in length. The Shug Jordan Parkway Ramps southbound approach includes a left turn lane and a flared right turn lane at the intersection with MLK/AL-14.

The intersection is side street stop controlled (Shug Jordan Parkway Ramps) with the southbound lefts having a stop condition and the southbound rights approaching a channelizing painted island and a yield sign prior to entering MLK/AL-14 westbound.

Characteristics of the roadways near the study intersection are summarized in **Table 12**.

Table 12 - Corridor Roadway Characteristics – MLK/AL-14 at SJP Ramps

Roadway	Parking	# of Lanes	Travel Direction	Posted Speeds (mph)	Classification
MLK/AL-14 (Either Side of Shug Jordan Parkway Ramps)	None	2 (plus EB left turn lane)	East/West	45	Minor Arterial – West of Intersection Major Collector – East of Intersection
Shug Jordan Parkway Ramps (At MLK/AL-14)	None	2	North/South	--	Local Road

Peak Hour Traffic Counts

Morning (7:00-9:00 am) and afternoon (4:00-6:00 pm) peak hour turning movement counts were conducted at the study intersection on April 26, 2018. In addition, twenty-four hour approach counts were collected at the intersection on April 4, 2018. Traffic count data utilized for the analyses of these intersections is summarized in **Figure 7**.

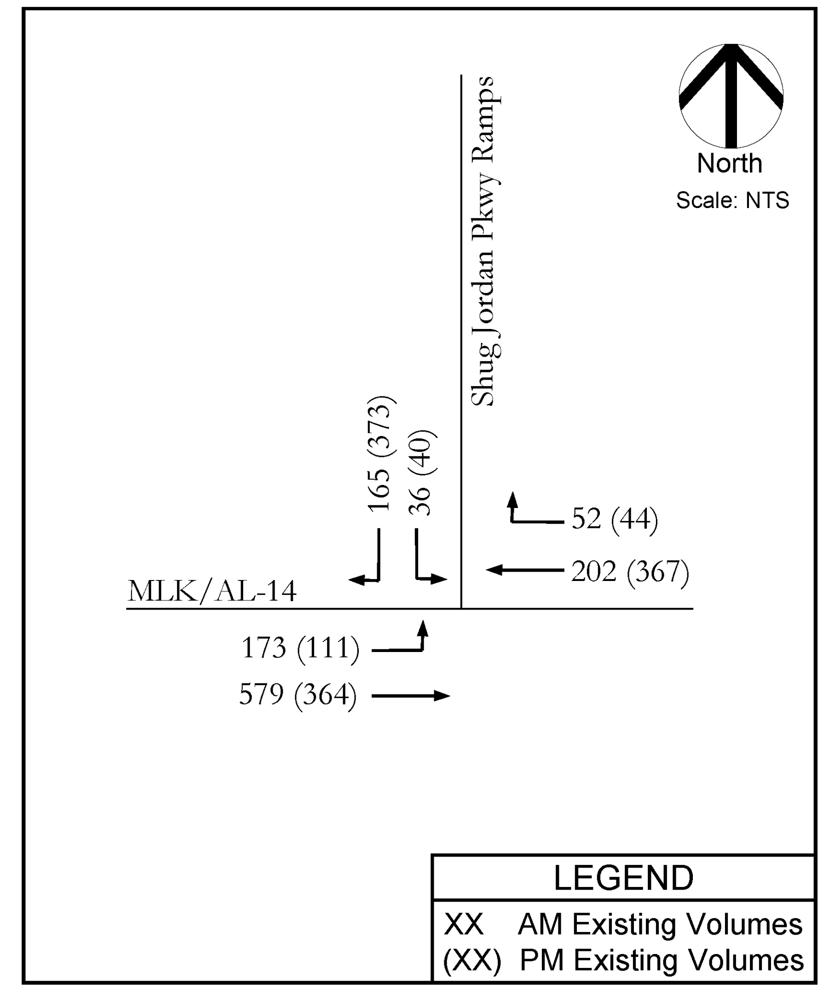


Figure 7 - Existing Peak Hour Traffic Volumes - MLK/AL-14 at SJP Ramps

EXISTING CONDITIONS ANALYSES

Existing Intersection Capacity Analysis

Capacity analyses for peak hour conditions at the intersection of MLK/AL-14 and Shug Jordan Parkway Ramps were conducted for the morning and afternoon peak hour periods using methods outlined in the *Highway Capacity Manual, 2000*. According to methods of the *Highway Capacity Manual*, capacity is expressed as levels of service ranging from “A” (best) through “F” (worst). In general, a level of service “C” is considered desirable while a level of service “D” is considered acceptable during peak hour operations. Results of these capacity analyses for existing conditions are summarized in **Table 13**.

Table 13 - Existing Intersection Levels of Service - MLK/AL-14 at SJP Ramps

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service	
			A.M. Peak Hour	P.M. Peak Hour
MLK/AL-14 at Shug Jordan Parkway Ramps (unsignalized)	EB MLK/AL-14	Left	A	A
		Through	-	-
	WB MLK/AL-14	Through	-	-
		Right	-	-
	SB Shug Jordan Pkwy Ramps	Left/Right	C (23’) [49’]	D (95’) [193’]

(X’) = (Synchro 95th %ile queue length)
[X’] = [SimTraffic 95th %ile queue length]

As shown in **Table 13**, each movement/lane group at the intersection currently operates at acceptable levels of service for both peak periods evaluated. However, in the pm peak hour, the Shug Jordan Parkway Ramps southbound left turn has an estimated 95th percentile queue of approximately 200’ based on the SimTraffic simulation model. The flared right turn lane only leaves 25’-35’ of storage in the southbound left turn lane before it potentially blocks the right turn lane. The primary conflict causing the southbound queue is the southbound right turning vehicles waiting for a gap in the westbound mainline traffic.

Turn Lane Warrant Evaluations

Existing peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for the following approaches:

- Southbound Shug Jordan Parkway Ramps at MLK/AL-14

The results of these comparisons indicate:

- Southbound Shug Jordan Parkway Ramps at MLK/AL-14 – During the afternoon peak hour, existing traffic volumes are sufficient to meet the criteria for a right-turn lane. Also, simulation models of the existing pm peak conditions indicate a maximum queue length of 200’, which is significantly longer than the existing storage.

Intersection Crash Evaluation

Skipper Consulting, Inc. performed a citywide crash study for intersections and roadway segments maintained by the City of Auburn. The results of this crash study have been documented in a separate bound report. The citywide crash study included the intersection of MLK/AL-14 at the Shug Jordan Parkway Ramps. Screening procedures and crash analyses were conducted to determine any locations that are worthy of safety-based roadway improvements. The crash analysis indicated the following:

- Moderate Priority Intersections - this indicates the crash experience should be monitored in the near future and could be worthy of a safety-based roadway improvement if crash experience trends upward. This does not warrant a safety-based improvement at this time, but a safety-based improvement should be incorporated in any roadway improvement at this location.
    - MLK/AL-14 at Shug Jordan Parkway Ramps

EXISTING CONDITIONS ANALYSES WITH IMPROVEMENTS

Recommended Improvements

Roadway and traffic control improvements have been developed to help address capacity and safety deficiencies identified in the capacity analyses conducted for the study intersection. The following outlines the recommended improvements for existing conditions at the MLK/AL-14 at Shug Jordan Parkway Ramps intersection:

Alternate 1

- Construct an additional westbound lane on MLK/AL-14 such that the Shug Jordan Parkway Ramps southbound right turns free flow into the auxiliary lane then merge the westbound lanes into a single lane prior to the Solamere Lane intersection.
- Extend the Shug Jordan Parkway Ramps southbound right turn lane to provide 250' of full width storage.

Alternate 2

- Convert the existing intersection into a single lane roundabout.

Existing Intersection Capacity Analysis with Improvements

Capacity analyses for peak hour conditions at the study intersection were conducted assuming improvements for existing conditions would be in place. Capacity analyses were conducted using methods outlined in the *Highway Capacity Manual, 2000.* Results of these capacity analyses are summarized in **Table 14**.

Table 14 - Existing Intersection Levels of Service with Improvements – Alternate 1 (Auxiliary Lane)

MLK/AL-14 at SJP Ramps

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Shug Jordan Parkway Ramps (unsignalized)	EB MLK/AL-14	Left	A	A
		Through	-	-
	WB MLK/AL-14	Through	-	-
		Right	-	-
	SB Shug Jordan Pkwy Ramps	Left	B	C
		Right	-	-

As shown in **Table 14**, each movement/lane group at the study intersection operates at acceptable levels of service for both peak periods evaluated with the recommended improvements in place. (It should be noted that the southbound approach LOS is the same or worse than in the existing scenario. This is due to the HCM methodology ignoring the free flow right vehicles (zero delay) and basing the delay calculations solely on the left turns (long delay).

Table 15 - Existing Intersection Levels of Service with Improvements – Alternate 2 (Roundabout)

MLK/AL-14 at SJP Ramps

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Shug Jordan Parkway Ramps (roundabout)	EB MLK/AL-14	Overall	C	B
	WB MLK/AL-14	Overall	A	B
	SB Shug Jordan Pkwy Ramps	Overall	A	D
	Overall LOS		**B**	**C**

As shown in **Table 15**, each approach of the study intersection operates at acceptable levels of service for both peak periods evaluated under Alternate 2 (Roundabout).

PROJECTED TRAFFIC GROWTH

Growth rates were calculated for the study roadways based on historical traffic volumes and growth trends. The historical growth rate calculated for roadways in the vicinity of the MLK/AL-14 and Shug Jordan Parkway Ramps intersection was 2.7% per year. The annual growth rate was applied for a ten (10) year period to result in an overall growth rate of 27.0% percent for study intersection traffic volumes. Existing peak hour traffic volumes were increased 27.0% to reflect ten (10) year projected traffic volumes for the study intersection. Future traffic volumes are summarized in **Figure 8**.

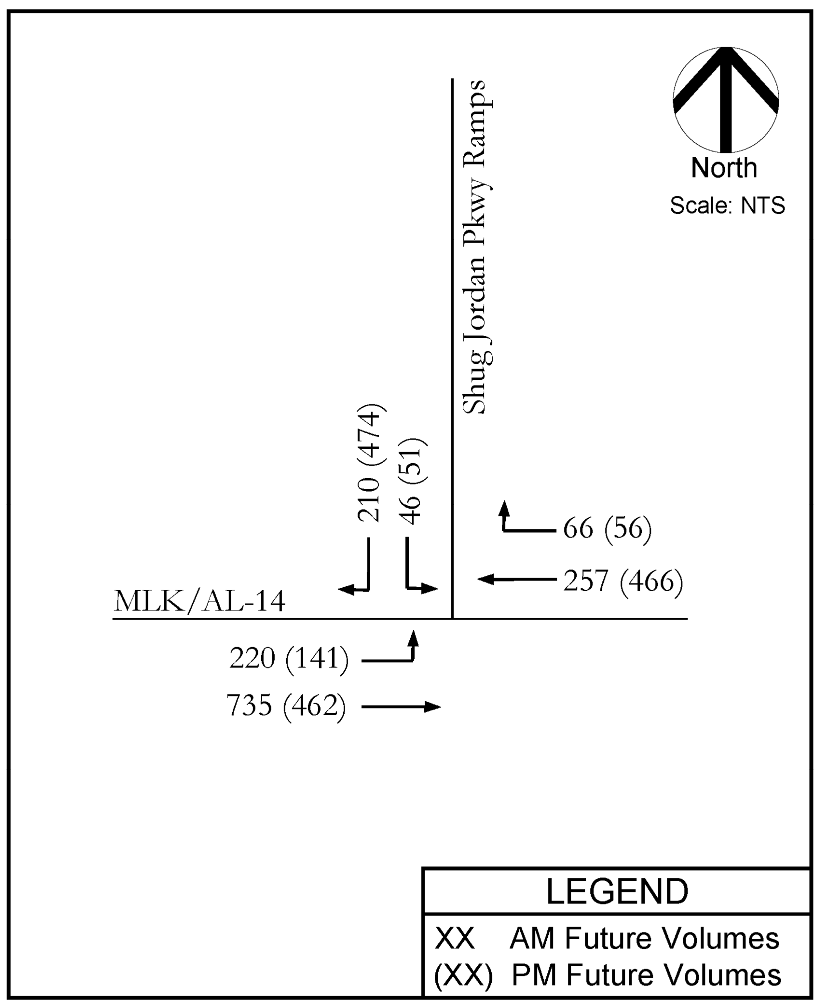


Figure 8 - Future Peak Hour Traffic Volumes - MLK/AL-14 at SJP Ramps

ANALYSES WITH PROJECTED TRAFFIC GROWTH

Analyses conducted for this scenario assumes projected traffic volumes for ten (10) years would be in place and the improvements recommended for existing conditions would also be in place.

Intersection Capacity Analysis with Projected Traffic Growth

Capacity analyses for projected ten (10) year peak hour conditions were conducted for the study intersection, assuming the existing recommendations were in place, using methods outlined in the *Highway Capacity Manual, 2000*. Results of these capacity analyses are summarized in **Table 16**.

Table 16 - Intersection Levels of Service w/ Projected Traffic Growth – Alternate 1 (Auxiliary Lane)
MLK/AL-14 at SJP Ramps

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Shug Jordan Parkway Ramps (unsignalized)	EB MLK/AL-14	Left	A	A
		Through	-	-
	WB MLK/AL-14	Through	-	-
		Right	-	-
	SB Shug Jordan Pkwy Ramps	Left	C	E
		Right	-	-

As shown in **Table 16**, the Shug Jordan Parkway Ramps southbound left operates at an inadequate level of service for the p.m. peak period evaluated with the future growth applied to the recommended existing conditions improvements.

Table 17 - Intersection Levels of Service w/ Projected Traffic Growth – Alternate 2 (Roundabout)
MLK/AL-14 at SJP Ramps

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Shug Jordan Parkway Ramps (roundabout)	EB MLK/AL-14	Overall	E	B
	WB MLK/AL-14	Overall	A	B
	SB Shug Jordan Pkwy Ramps	Overall	A	D
	Overall LOS		**D**	**C**

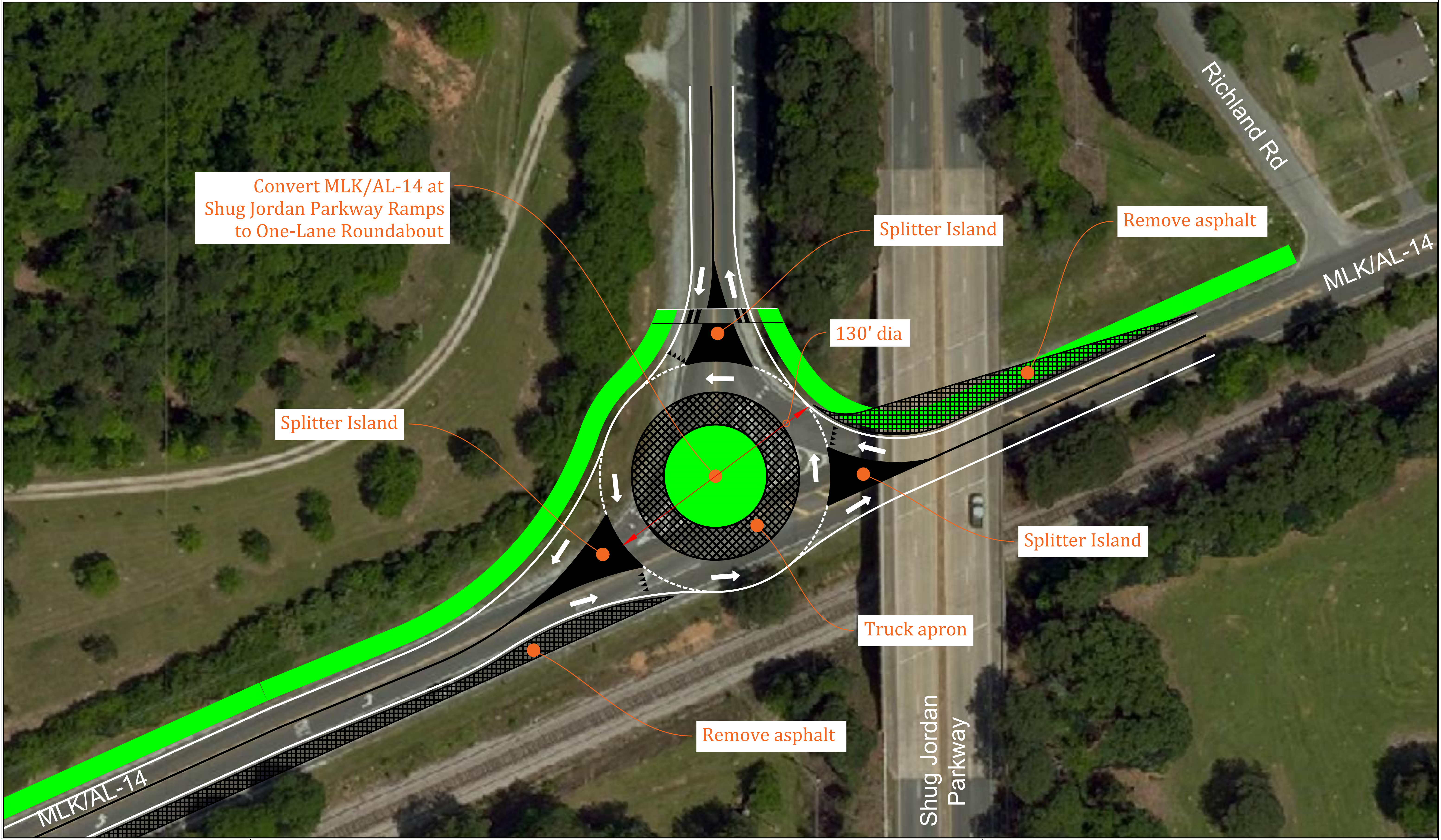
As shown in **Table 17**, each approach of the study intersection operates at acceptable levels of service for the p.m. peak period evaluated under Alternate 2 (Roundabout), but with an inadequate level of service on the eastbound approach during the a.m. peak period.

RECOMMENDED IMPROVEMENTS WITH PROJECTED TRAFFIC GROWTH

Based upon the analyses and evaluations conducted for the study intersection for existing conditions and projected ten (10) year conditions, the results summarized in **Table 16** and **Table 17** indicate that a single lane roundabout would provide better traffic operations initially and through the ten (10) year projected scenario. The projected traffic growth analysis seems to further support selection of the single lane roundabout. A preliminary layout of the roundabout is shown in **Figure 9**.

Pedestrian and Bicycle Improvements

The City of Auburn currently has plans to extend the multi-use trail along MLK/AL-14 from its current terminus at Webster Road to tie into the existing sidewalk at the intersection of MLK/Richland Road. The design of the proposed roundabout will need to accommodate a crossing of this multi-use trail crossing the northern leg of the roundabout.



SKIPPER CONSULTING INC

Figure 9 - Recommended Improvements
MLK/AL-14 at Shug Jordan Parkway Ramps
Auburn, Alabama

NOTES
- Conceptual RAB based on 130' dia inscribed circle
- Location of RAB constrained by railroad, bridge piers

Scale: Not to Scale
Date: Oct 2018

ISOLATED INTERSECTION #4: E. THACH AVENUE AT S. ROSS STREET

BACKGROUND INFORMATION

Study Intersection and Roadways

E. Thach Avenue is a two-lane major collector roadway stretching from Donahue Drive to Dean Road. In the vicinity of the intersection with S. Ross Street/Chewacla Drive, E. Thach Avenue has sidewalks and marked bike lanes on both sides of the roadway with a posted speed limit of 25mph.

S. Ross Street is a two-lane major collector roadway extending from Samford Avenue to north of Opelika Road. Near the E. Thach Avenue intersection, S. Ross Street has sidewalks on both sides of the roadway south of the intersection and only on the west side north of the intersection and has a posted speed limit of 25mph.

The intersection is signalized, has protected/permissive left turn phasing on all four approaches, marked crosswalks, pedestrian signal heads, and pedestrian pushbuttons on all four corners. All four approaches have left turn lanes ranging with approximately 50' to 70' of storage.

Characteristics of the intersection roadways are summarized in **Table 18**.

Table 18 - Corridor Roadway Characteristics – Thach Ave at Ross St

Roadway	Parking	# of Lanes	Travel Direction	Travel Speeds (mph)	Classification
E. Thach Avenue (Either Side of S. Ross Street)	None	2 (w/ left turn lanes)	East/West	25	Major Collector
S. Ross Street/Chewacla Drive (Either Side of E. Thach Street)	None	2 (w/ left turn lanes)	North/South	25	Major Collector

Peak Hour Traffic Counts

Morning (7:00-9:00 am) and afternoon (4:00-6:00 pm) peak hour turning movement counts were conducted at the study intersection on April 16, 2013. In addition, twenty-four hour approach counts were collected at the intersection on April 26, 2018. Traffic count data utilized for the analysis of this intersection is summarized in **Figure 10**.

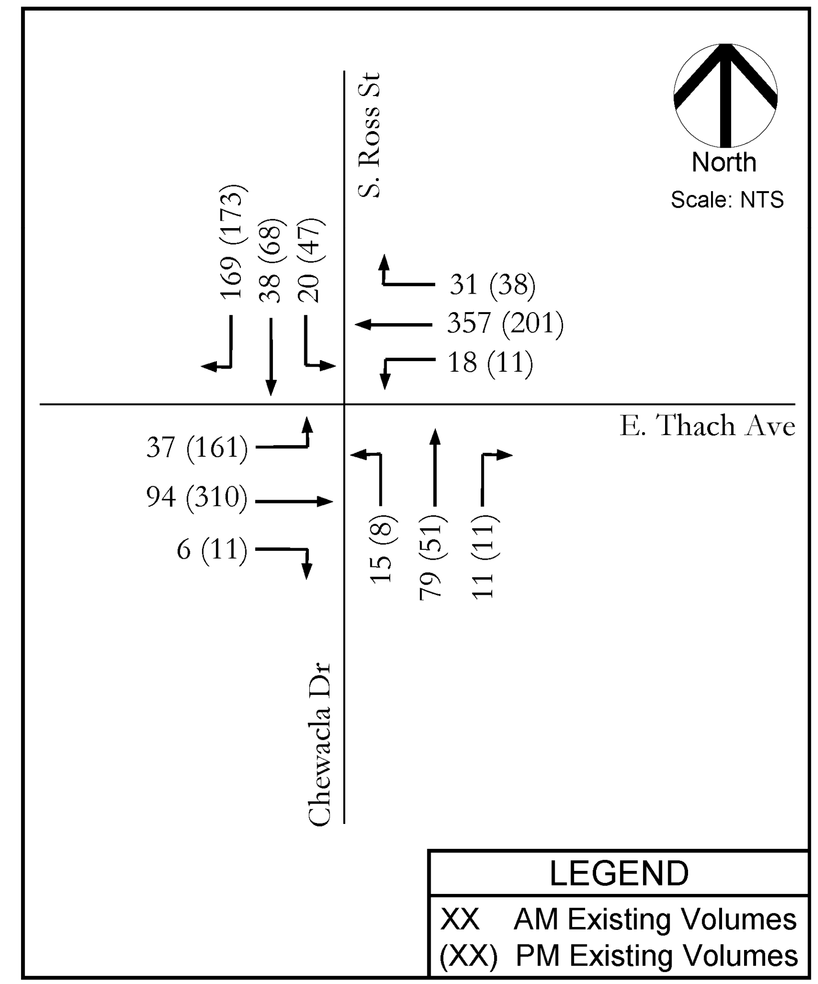


Figure 10 - Existing Peak Hour Traffic Volumes - Thach Ave at Ross St

EXISTING CONDITIONS ANALYSES

Existing Intersection Capacity Analysis

Capacity analyses for peak hour conditions at the intersection of E. Thach Avenue and S. Ross Street were conducted for the morning and afternoon peak hour periods using methods outlined in the *Highway Capacity Manual, 2000*. According to methods of the *Highway Capacity Manual*, capacity is expressed as levels of service ranging from "A" (best) through "F" (worst). In general, a level of service "C" is considered desirable while a level of service "D" is considered acceptable during peak hour operations. Results of these capacity analyses for existing conditions are summarized in **Table 19**.

Table 19 - Existing Intersection Levels of Service - Thach Ave at Ross St

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
E. Thach Avenue at S. Ross Street (Signalized)	EB E. Thach Ave	Left	B	B
		Through/Right	B	B
		Overall	B	B
	WB E. Thach Ave	Left	A	A
		Through/Right	B	B
		Overall	B	B
	NB Chewacla Dr	Left	A	A
		Through/Right	B	A
		Overall	B	A
	SB S. Ross St	Left	A	B
		Through/Right	B	B
		Overall	B	B
	Overall LOS		**B**	**B**

As shown in **Table 19**, each movement/lane group at the intersection currently operates at acceptable levels of service for both peak periods evaluated.

Right-Turn Lane Warrant Evaluations

Existing peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for all four approaches of the study intersection. The results of these comparisons indicate that none of the approaches warrant a right turn lane. This is primarily due to the low speed limit which has the greatest impact on determining the minimum threshold of right turn volumes.

Intersection Crash Evaluation

Skipper Consulting, Inc. performed a citywide crash study for intersections and roadway segments maintained by the City of Auburn. The results of this crash study have been documented in a separate bound report. The citywide crash study initially included the E. Thach Avenue and S. Ross Street intersection. Screening procedures and crash analyses were conducted to determine any locations that are worthy of safety-based roadway improvements. The study intersection was eliminated from further analysis during the initial screening process due to a low crash rate.

EXISTING CONDITIONS ANALYSES WITH IMPROVEMENTS

Recommended Improvements

Based on the results of the existing conditions analysis, there are no obvious deficiencies that need to be mitigated. No recommended improvements are suggested at this stage of analysis.

PROJECTED TRAFFIC GROWTH

Growth rates were calculated for the study roadways based on historical traffic volumes and growth trends. The historical growth rate calculated for roadways in the vicinity of the E. Thach Avenue and S. Ross Street intersection was 1.4% per year. The annual growth rate was applied for a ten (10) year period to result in an overall growth rate of 14.0% percent for study intersection traffic volumes. Existing peak hour traffic volumes were increased 14.0% to reflect 10 (10) year projected traffic volumes for the study intersection. Future traffic data utilized for the analysis of this intersection is summarized in **Figure 11**.

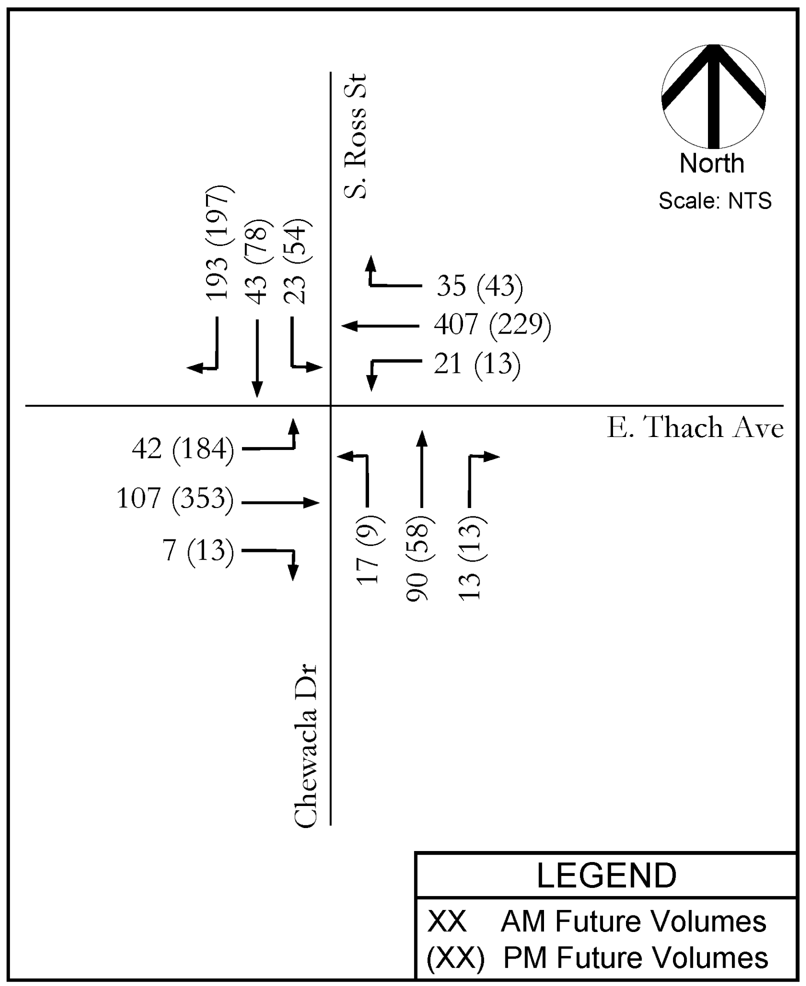


Figure 11 - Future Peak Hour Traffic Volumes - Thach Ave at Ross St

ANALYSES WITH PROJECTED TRAFFIC GROWTH

Analyses conducted for this scenario assumes projected traffic volumes for ten (10) years would be in place.

Intersection Capacity Analysis with Projected Traffic Growth

Capacity analyses for projected ten (10) year peak hour conditions were conducted for the study intersection using methods outlined in the *Highway Capacity Manual, 2000*. Results of these capacity analyses are summarized in **Table 20**.

Table 20 - Intersection Levels of Service with Projected Traffic Growth - Thach Ave at Ross St

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
E. Thach Avenue at S. Ross Street (Signalized)	EB E. Thach Ave	Left	B	B
		Through/Right	B	B
		Overall	B	B
	WB E. Thach Ave	Left	B	B
		Through/Right	D	C
		Overall	D	C
	NB Chewacla Dr	Left	B	C
		Through/Right	B	C
		Overall	B	C
	SB S. Ross St	Left	B	B
		Through/Right	B	C
		Overall	B	C
	Overall LOS		**C**	**B**

As shown in **Table 20**, all the study intersection operates with overall acceptable levels of service for both future peak periods evaluated.

Right-Turn Lane Warrant Evaluations with Projected Traffic Growth

Future peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for all four approaches of the study intersection. The results of these comparisons indicate that none of the approaches warrant a right turn lane. This is primarily due to the low speed limit which has the greatest impact on determining the minimum threshold of right turn volumes.

RECOMMENDED IMPROVEMENTS WITH PROJECTED TRAFFIC GROWTH

Based upon the analyses and evaluations conducted for the study intersection for existing conditions and projected ten (10) year conditions, recommendations are made help improve traffic operations and to address any capacity or safety deficiencies identified. The recommended improvements are as follows:

- Modify signal cycle length and split times based on the future volumes to reduce intersection delay

ANALYSES WITH RECOMMENDED IMPROVEMENTS & PROJECTED TRAFFIC GROWTH

Intersection Capacity Analysis with Improvements and Projected Traffic Growth

Capacity analyses were conducted for the study intersections assuming recommended improvements (outlined above) and projected ten (10) year traffic volumes would be in place. Capacity analyses were conducted using methods of the *Highway Capacity Manual*, as previously introduced. **Table 21** provides a summary of the levels of service for the study intersection with recommended improvements and projected ten (10) traffic volumes in place.

Table 21 - Intersection Levels of Service with Improvements and Projected Traffic Growth

Thach Ave at Ross St

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service	
			A.M. Peak Hour	P.M. Peak Hour
E. Thach Avenue at S. Ross Street (Signalized)	EB E. Thach Ave	Left	B	B
		Through/Right	B	B
		Overall	B	B
	WB E. Thach Ave	Left	B	B
		Through/Right	C	C
		Overall	C	C
	NB Chewacla Dr	Left	B	C
		Through/Right	C	C
		Overall	C	C
	SB S. Ross St	Left	B	B
		Through/Right	C	C
		Overall	C	C
	Overall LOS		**C**	**B**

As shown in **Table 21**, all the study intersection operates with overall acceptable levels of service for both future peak periods evaluated.

ISOLATED INTERSECTION #5: GAY STREET AT SHELTON MILL ROAD

BACKGROUND INFORMATION

Study Intersection and Roadways

Gay Street is a two-lane minor arterial roadway stretching from Shelton Mill Road to East University Drive. In the vicinity of the intersection with Shelton Mill Road, Gay Street has a sidewalk on the west side of the roadway. The posted speed limit on Gay Street is 35 mph.

Shelton Mill Road is a two-lane minor arterial roadway extending from College Street to U.S. Highway 280. Near the Gay Street intersection, Shelton Mill Road has a sidewalk on the south side of the roadway. The posted speed limit on Shelton Mill Road is 35mph.

The intersection is unsignalized and is controlled by a side street stop on Gay Street. There is a marked crosswalk crossing Gay Street crossing the south leg of the intersection. No turn lanes are provided on any approach to the intersection.

Characteristics of the intersection roadways are summarized in **Table 22**.

Table 22 - Corridor Roadway Characteristics – Gay St at Shelton Mill Rd

Roadway	Parking	# of Lanes	Travel Direction	Travel Speeds (mph)	Classification
Gay Street	None	2	North/South	35	Minor Arterial
Shelton Mill Road	None	2	East/West	35	Minor Arterial

Peak Hour Traffic Counts

Morning (7:00-9:00 am) and afternoon (4:00-6:00 pm) peak hour turning movement counts were conducted at the study intersection on March 4, 2015. Traffic count data utilized for the analysis of this intersection is summarized in **Figure 12**.

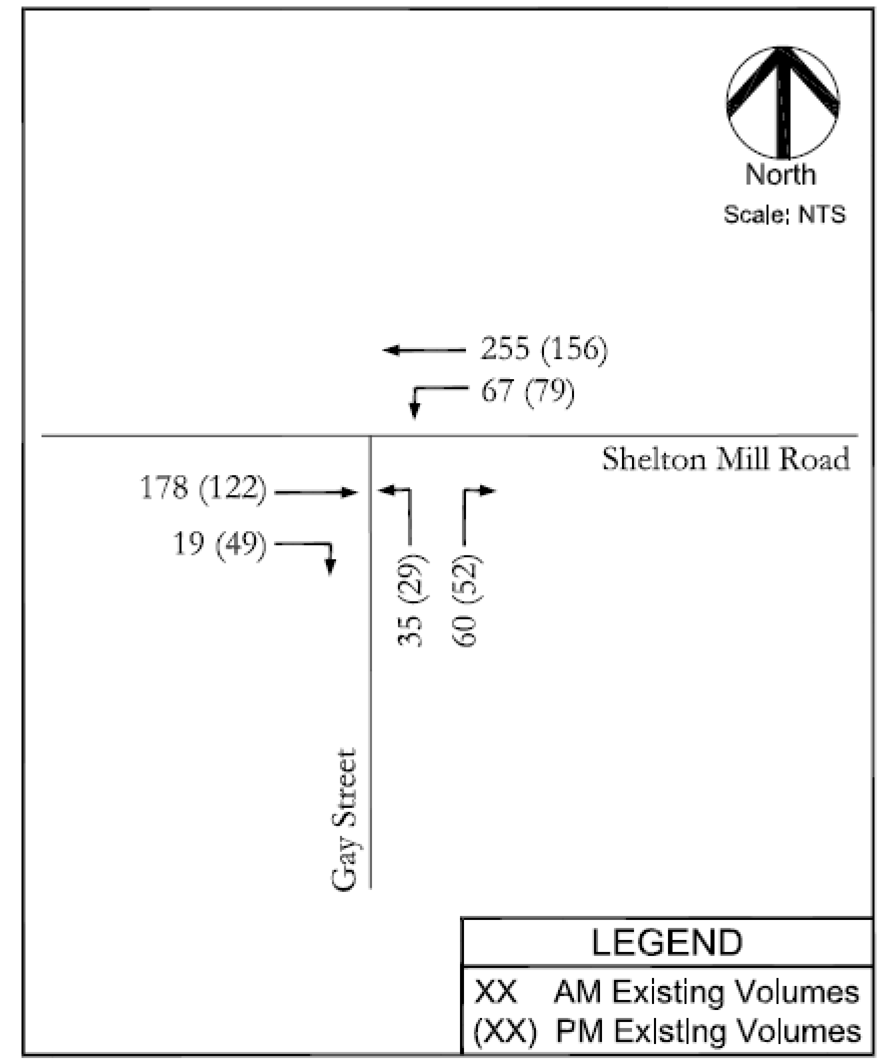


Figure 12 - Existing Peak Hour Traffic Volumes - Gay St at Shelton Mill Rd

EXISTING CONDITIONS ANALYSES

Existing Intersection Capacity Analysis

Capacity analyses for peak hour conditions at the intersection of Gay Street and Shelton Mill Road were conducted for the morning and afternoon peak hour periods using methods outlined in the *Highway Capacity Manual, 2000*. According to methods of the *Highway Capacity Manual*, capacity is expressed as levels of service ranging from "A" (best) through "F" (worst). In general, a level of service "C" is considered desirable while a level of service "D" is considered acceptable during peak hour operations. Results of these capacity analyses for existing conditions are summarized in **Table 23**.

Table 23 - Existing Intersection Levels of Service - Gay St at Shelton Mill Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
Gay Street at Shelton Mill Road (unsignalized)	EB Shelton Mill Road	Through/Right	-	-
	WB Shelton Mill Road	Left/Through	A	A
	NB Gay Street	Left/Right	B	B

As shown in **Table 23**, each movement/lane group at the intersection currently operates at acceptable levels of service for both peak periods evaluated.

Right-Turn Lane Warrant Evaluations

Existing peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for the eastbound Shelton Mill Road approach. The result of this comparison indicates that the eastbound approach does not warrant a right turn lane.

Intersection Crash Evaluation

Skipper Consulting, Inc. performed a citywide crash study for intersections and roadway segments maintained by the City of Auburn. The results of this crash study have been documented in a separate bound report. The citywide crash study initially included the Gay Street and Shelton Mill Road intersection. Screening procedures and crash analyses were conducted to determine any locations that are worthy of safety-based roadway improvements. The study intersection was eliminated from further analysis during the initial screening process due to a low crash rate.

EXISTING CONDITIONS ANALYSES WITH IMPROVEMENTS

Recommended Improvements

Based on the results of the existing conditions analysis, there are no obvious deficiencies that need to be mitigated. No recommended improvements are suggested at this stage of analysis.

PROJECTED TRAFFIC GROWTH

Growth rates were calculated for the study roadways based on historical traffic volumes and growth trends. The historical growth rate calculated for roadways in the vicinity of the Gay Street and Shelton Mill Road intersection was 1.4% per year. The annual growth rate was applied for a ten (10) year period to result in an overall growth rate of 14.0% percent for study intersection traffic volumes. Existing peak hour traffic volumes were increased 14.0% to reflect 10 (10) year projected traffic volumes for the study intersection. Future traffic data utilized for the analysis of this intersection is summarized in **Figure 13**.

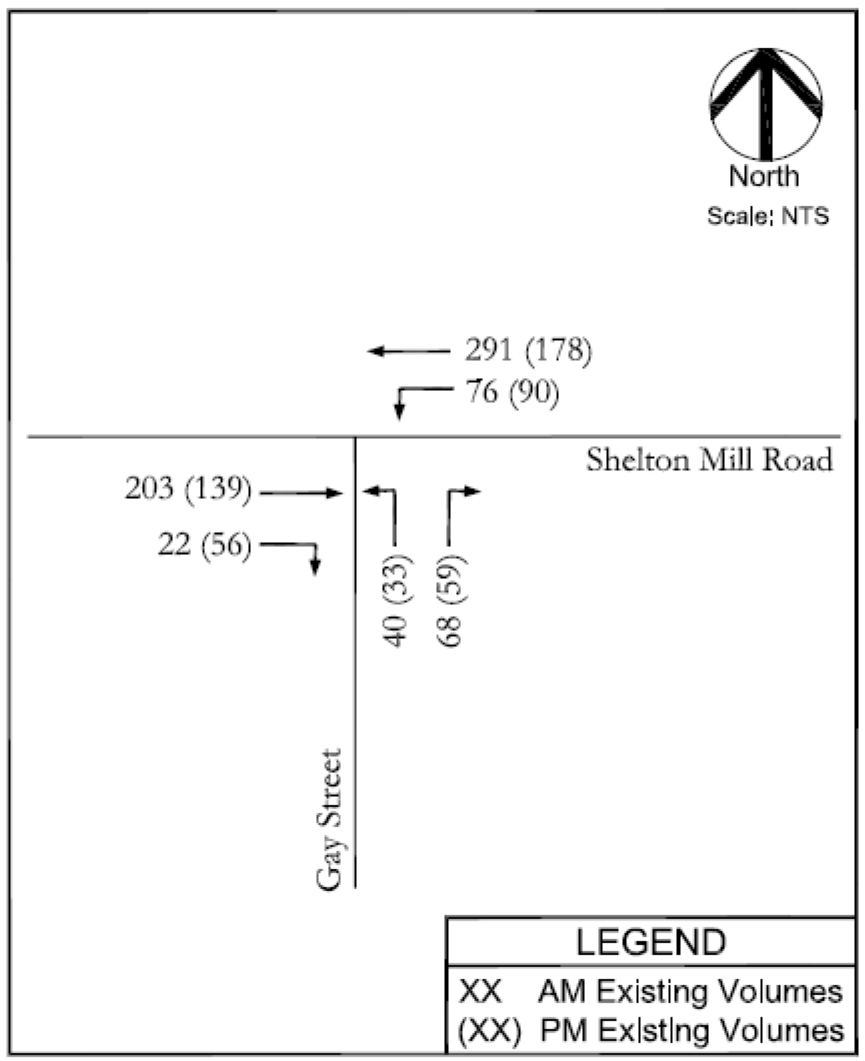


Figure 13 - Future Peak Hour Traffic Volumes - Gay St at Shelton Mill Rd

ANALYSES WITH PROJECTED TRAFFIC GROWTH

Analyses conducted for this scenario assumes projected traffic volumes for ten (10) years would be in place.

Intersection Capacity Analysis with Projected Traffic Growth

Capacity analyses for projected ten (10) year peak hour conditions were conducted for the study intersection using methods outlined in the *Highway Capacity Manual, 2000*. Results of these capacity analyses are summarized in **Table 24**.

Table 24 - Intersection Levels of Service w/Projected Traffic Growth- Gay St at Shelton Mill Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
Gay Street at Shelton Mill Road (unsignalized)	EB Shelton Mill Road	Through/Right	-	-
	WB Shelton Mill Road	Left/Through	A	A
	NB Gay Street	Left/Right	B	B

As shown in **Table 24**, all the study intersection operates with overall acceptable levels of service for both future peak periods evaluated.

Right-Turn Lane Warrant Evaluations with Projected Traffic Growth

Future peak hour traffic volumes were compared with the turn lane warrant criteria outlined in the National Cooperative Highway Research Program (NCHRP) Report 457 *Evaluating Intersection Improvements: An Engineering Study Guide*, published by the Transportation Research Board. For evaluation purposes, the posted speed limit was utilized for roadways. Evaluations were conducted for the eastbound Shelton Mill Road approach. The result of this comparison indicates that the eastbound approach does not warrant a right turn lane.

RECOMMENDED IMPROVEMENTS WITH PROJECTED TRAFFIC GROWTH

Based upon the analyses and evaluations conducted for the study intersection for existing conditions and projected ten (10) year conditions, no improvements are recommended for the intersection of Gay Street at Shelton Mill Road.

ISOLATED INTERSECTION #6: MARTIN LUTHER KING DRIVE/AL-14 AT WILLOW CREEK ROAD

BACKGROUND INFORMATION

Study Intersection and Roadways

Martin Luther King Drive/AL-14 is a two-lane minor arterial roadway in the vicinity of the intersection with Willow Creek Road, MLK/AL-14 has a multi-use path on the north side of the roadway. The posted speed limit on MLK/AL-14 is 50 mph.

Willow Creek Road is a two-lane local roadway Near the MLK/AL-14 intersection, Shelton Mill Road has no sidewalks. The posted speed limit on Willow Creek Road is 25 mph.

The intersection is unsignalized and is controlled by a side street stop on Willow Creek Road. There is a marked crosswalk crossing Willow Creek Road crossing the north leg of the intersection for the multi-use path. There is a right turn lane provided on MLK/AL-14 turning right onto Willow Creek Road. The right turn lane has 205 feet of full-width storage and 180 feet of taper.

Characteristics of the intersection roadways are summarized in **Table 25**.

Table 25 - Corridor Roadway Characteristics – MLK/AL-14 at Willow Creek Rd

Roadway	Parking	# of Lanes	Travel Direction	Travel Speeds (mph)	Classification
MLK/AL-14	None	2	East/West	50	Minor Arterial
Willow Creek Road	None	2	North/South	25	Local

Peak Hour Traffic Counts

Morning (7:00-9:00 am) and afternoon (4:00-6:00 pm) peak hour turning movement counts were conducted at the study intersection on March 17, 2015. Traffic count data utilized for the analysis of this intersection is summarized in **Figure 14**.

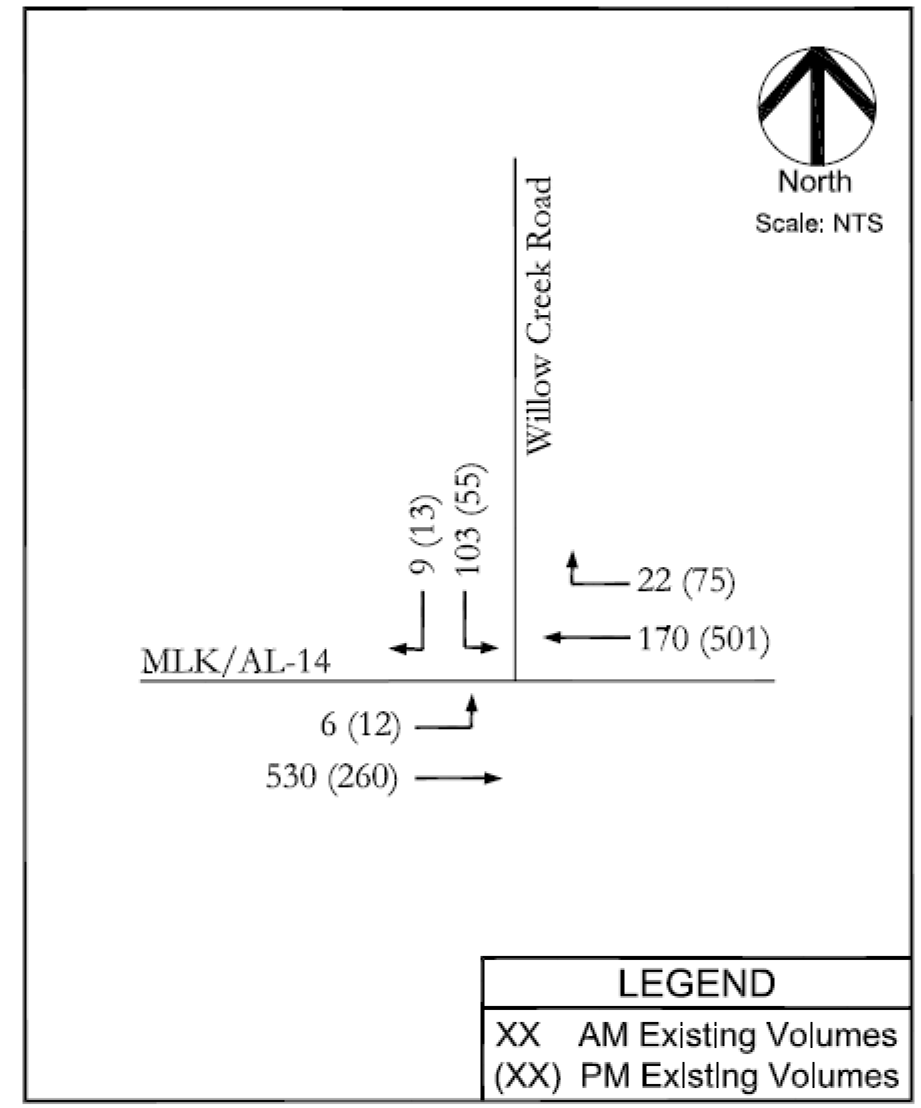


Figure 14 - Existing Peak Hour Traffic Volumes - MLK/AL-14 at Willow Creek Rd

EXISTING CONDITIONS ANALYSES

Existing Intersection Capacity Analysis

Capacity analyses for peak hour conditions at the intersection of MLK/AL-14 and Willow Creek Road were conducted for the morning and afternoon peak hour periods using methods outlined in the *Highway Capacity Manual, 2000*. According to methods of the *Highway Capacity Manual*, capacity is expressed as levels of service ranging from "A" (best) through "F" (worst). In general, a level of service "C" is considered desirable while a level of service "D" is considered acceptable during peak hour operations. Results of these capacity analyses for existing conditions are summarized in **Table 26**.

Table 26 - Existing Intersection Levels of Service - MLK/AL-14 at Willow Creek Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service	
			A.M. Peak Hour	P.M. Peak Hour
MLK/AL-14 at Willow Creek Road (unsignalized)	EB MLK/AL-14	Left/Through	A	A
	WB MLK/AL-14	Through	-	-
		Right	-	-
	SB Willow Creek Road	Left/Right	C	C

As shown in **Table 26**, each movement/lane group at the intersection currently operates at acceptable levels of service for both peak periods evaluated.

Right-Turn Lane Warrant Evaluations

Peak hour traffic right turn lane warrant evaluations were not performed for the intersection of MLK/AL-14 at Willow Creek Road since there is currently a right turn lane on MLK/AL-14 westbound.

Intersection Crash Evaluation

Skipper Consulting, Inc. performed a citywide crash study for intersections and roadway segments maintained by the City of Auburn. The results of this crash study have been documented in a separate bound report. The citywide crash study initially included the MLK/AL-14 and Willow Creek Road intersection. Screening procedures and crash analyses were conducted to determine any locations that are worthy of safety-based roadway improvements. The study intersection was eliminated from further analysis during the initial screening process due to a low crash rate.

EXISTING CONDITIONS ANALYSES WITH IMPROVEMENTS

Recommended Improvements

Based on the results of the existing conditions analysis, there are no obvious deficiencies that need to be mitigated. No recommended improvements are suggested at this stage of analysis.

PROJECTED TRAFFIC GROWTH

Growth rates were calculated for the study roadways based on historical traffic volumes and growth trends. The historical growth rate calculated for roadways in the vicinity of the MLK/AL-14 and Willow Creek Road intersection was 2.7% per year. The annual growth rate was applied for a ten (10) year period to result in an overall growth rate of 27.0% percent for study intersection traffic volumes. Existing peak hour traffic volumes were increased 27.0% to reflect 10 (10) year projected traffic volumes for the study intersection. Future traffic data utilized for the analysis of this intersection is summarized in **Figure 15**.

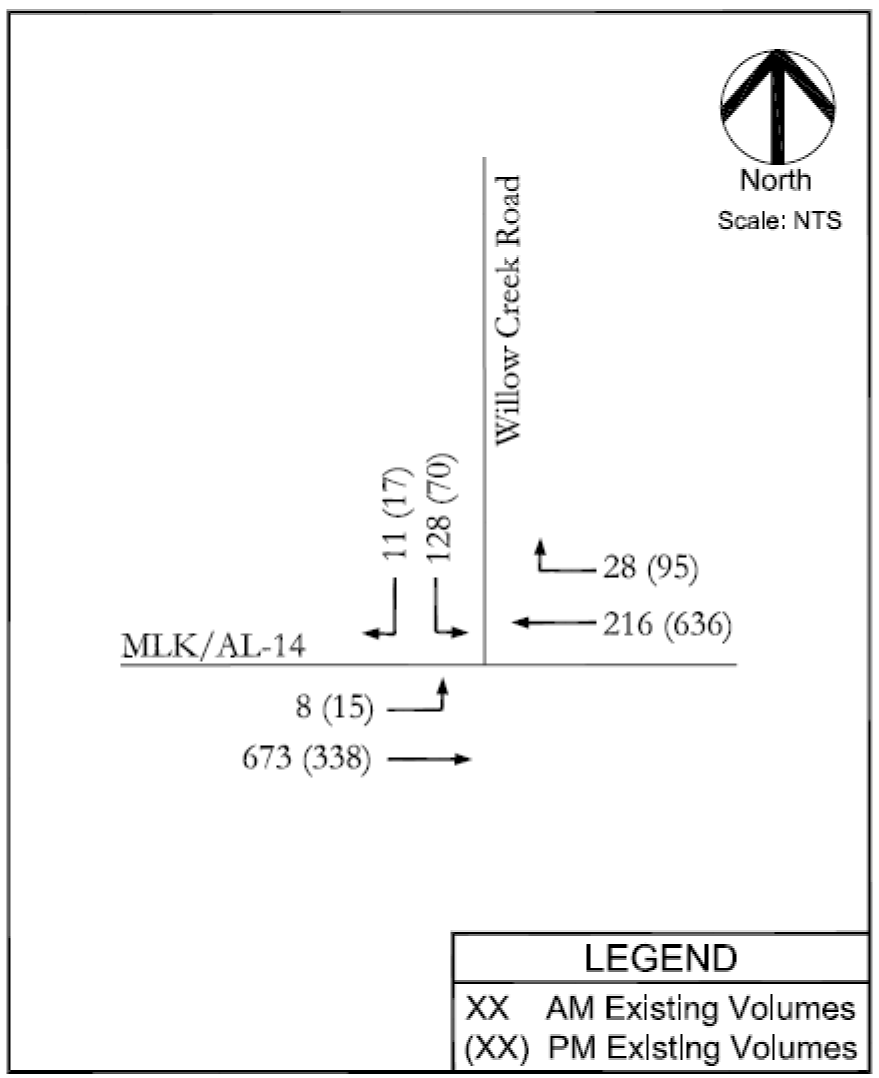


Figure 15 - Future Peak Hour Traffic Volumes - MLK/AL-14 at Willow Creek Rd

ANALYSES WITH PROJECTED TRAFFIC GROWTH

Analyses conducted for this scenario assumes projected traffic volumes for ten (10) years would be in place.

Intersection Capacity Analysis with Projected Traffic Growth

Capacity analyses for projected ten (10) year peak hour conditions were conducted for the study intersection using methods outlined in the *Highway Capacity Manual, 2000*. Results of these capacity analyses are summarized in **Table 27**.

Table 27 - Intersection Levels of Service w/Projected Traffic Growth - MLK/AL-14 at Willow Creek Rd

Intersection (traffic control)	Approach	Movement/Lane Group	Level of Service A.M. Peak Hour	Level of Service P.M. Peak Hour
MLK/AL-14 at Willow Creek Road (unsignalized)	EB MLK/AL-14	Left/Through	A	A
	WB MLK/AL-14	Through	-	-
		Right	-	-
	SB Willow Creek Road	Left/Right	D	D

As shown in **Table 27**, all the study intersection operates with overall acceptable levels of service for both future peak periods evaluated.

RECOMMENDED IMPROVEMENTS WITH PROJECTED TRAFFIC GROWTH

Based upon the analyses and evaluations conducted for the study intersection for existing conditions and projected ten (10) year conditions, no improvements are recommended for the intersection of MLK/AL-14 at Willow Creek Road.