TECHNICAL MEMORANDUM

US 231 (Campbell Lane) Corridor Traffic Study

Bowling Green, KY

BOWLING GREEN - WARREN COUNTY
METROPOLITAN PLANNING ORGANIZATION

In cooperation with

RFM Transportation Consultants, LLC

Project Description

The purposes of this Technical Memorandum are to summarize an analysis of the existing and future operational performances of the signalized intersections along US 231 (Campbell Lane) from Scottsville Road to Russellville Road and to determine cost effective solutions to provide the most efficient traffic flow possible along the study corridor. In addition, non-motorized level of service is presented in order to assess the quality of bicycle and pedestrian transportation accommodations within the study corridor.

In this study, a detailed inventory of the existing transportation infrastructure along the corridor will be compiled followed by detailed analyses of existing and future traffic operations and demand. The analyses were performed to determine the most feasible solutions to optimize the traffic operations in the study area. Short-term and long-term improvements will be identified.

In order to achieve the objectives described, a digital model of the corridor, utilizing Synchro/Simtraffic microsimulation software was developed. Three planning horizon scenarios have been developed and analyzed. Scenario 1, existing corridor with existing year (2015) traffic volumes, includes the setup, calibration and validation of the existing Synchro traffic model. Scenario 2 describes the existing corridor with future year (2020) projected volumes. The future traffic projections have been developed based on future land use, regional travel patterns, and currently planned improvements to facilities and services and have been coordinated with the City of Bowling Green, City-County Planning Commission and MPO. Scenario 3 includes recommendations for the projected future volumes to improve future traffic flow based on the results of analysis in Scenario 2.

As mentioned previously, the study area for this effort has been defined along Campbell Lane from Scottsville Road to Russellville Road. The study area for this project is illustrated in Figure 1. As shown in Figure 1, for the purpose of this study, the signalized intersections have been numbered 1 through 13 and are listed below.

1) U.S. 231 and Scottsville Rd
2) U.S. 231 and Gary Farms Blvd
3) U.S. 231 and Walton Ave
4) U.S. 231 and American Ave
5) U.S. 231 and Crossridge Blvd/Westpark Dr
6) U.S. 231 and McIntosh St
7) U.S. 231 and Weston St
8) U.S. 231 and Smallhouse Rd
9) U.S. 231 and Thoroughbred Dr
10) U.S. 231 and Harvard Dr
11) U.S. 231 and Nashville Rd
12) U.S. 231 and Industrial Dr
13) U.S. 231 and Russellville Rd

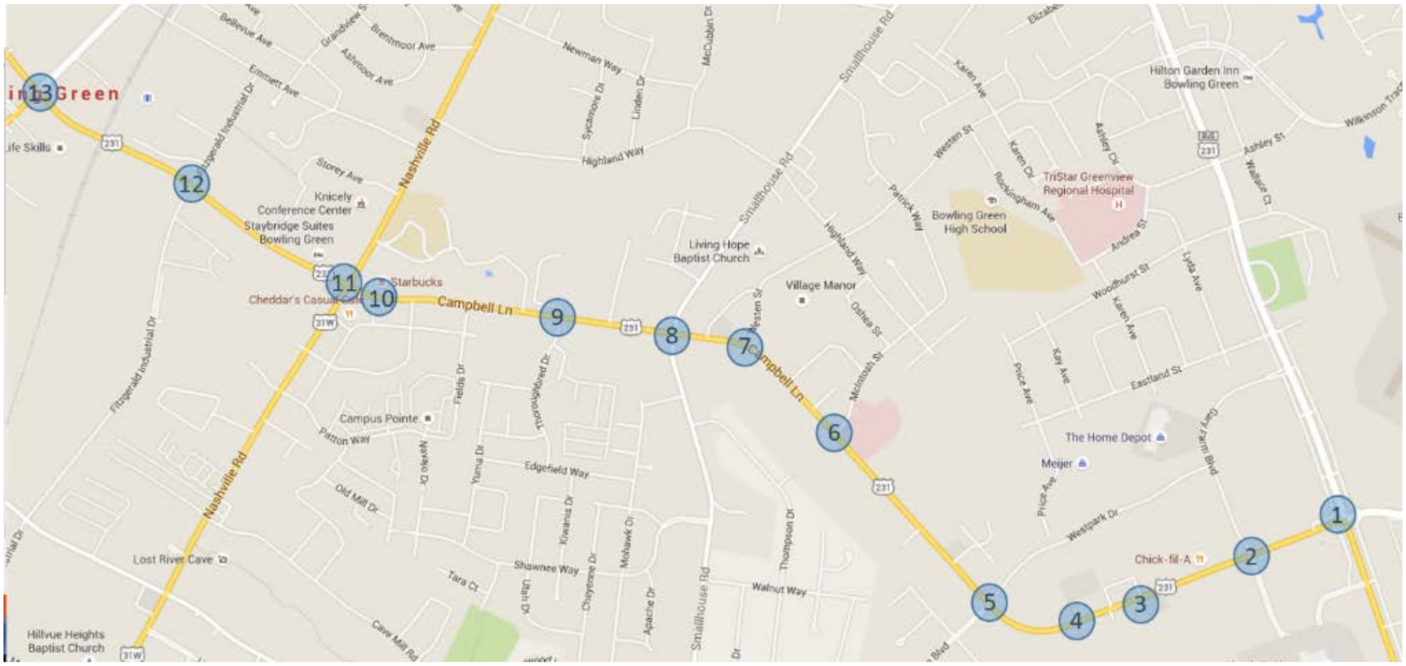

Figure 1. Project Study Area

Data Collection

In order to provide data for this study, turning movement volumes for AM and PM peak hours were conducted by RPM Transportation Consultants, LLC (RPM) using Miovision's video camera system. The traffic counts were conducted on October 21st and October 22nd of 2015 for three hours in the morning (6:00 AM – 9:00 AM) and three hours in the afternoon (3:00 PM – 6:00 PM). From these counts it was determined that the peak hours of traffic flow within the study area occur from 7:15-8:15 AM, and 3:45-4:45 PM. In addition to the vehicular turning movements, pedestrian movements were also counted at each of the intersections.

Existing signal timing data was obtained from Kentucky Transportation Cabinet (KYTC) and field observations including existing lane configuration and bike/pedestrian accommodations were performed by RPM.

In order to account for the traffic growth in the study area to develop future scenarios, the traffic history along the corridor (from KYTC website) was analyzed and coordinated with the Bowling Green-Warren County MPO. It was determined to use 1.5% annual growth along the corridor. In addition to the general background growth assumption, a specific trip generation process was used to quantify possible new trips along the corridor. This process was limited to four currently undeveloped parcels as shown in Figure 2.

In making trip generation assumptions, several parcel-related considerations were made, including: parcel site, potential access to Campbell Lane and the local street network, proximity to and type of adjacent land uses, current zoning, future land use plans, likely trip reductions (internal capture, pass-by, etc.). Specific factors for the trip generation were taken from ITE's Trip Generation, Ninth Edition. Table 1 presents the daily, AM and PM peak hour trip generations for the forecasted new trips for year 2020.

Figure 2. Locations of the assumed developments

Table 1. Assumed Developments Trip Generation – Year 2020

PARCEL	POSSIBLE LAND USES	SIZE (ACRES)	GENERATED TRAFFIC				
			DAILY TRAFFIC	AM PEAK		PM PEAK	
				Enter	Exit	Enter	Exit
A	Single Family Residential, Multifamily Residential, Commercial, Mixed Use	46	9,214	148	591	548	295
B	Single Family Residential, Multifamily Residential, Commercial, Mixed Use	13.8	3,871	55	34	164	178
C	Convenience Market with Gasoline Pumps, Fast Food Restaurant, Bank	1.2	815	33	33	41	41
D	Single Family Residential, Multifamily Residential, Commercial, Mixed Use	56.8	4,700	75	300	280	151
SUBTOTAL			18,599	311	958	1,033	664
TOTAL			**18,599**	**1,268**		**1,697**	
Note: Calculations above represent new traffic generated by the project site reduced to account for the pass-by trips, diversion to other corridors and alternative mode trips.							

Source: Trip Generation, Ninth Edition

Traffic Analysis

To determine the current and projected operation of the study intersections, queue analyses, travel times, and capacity analyses were conducted for the AM and PM peak hours. The traffic analyses were performed using Synchro/SimTraffic microsimulation software. In this study, the existing system with existing traffic volumes was first modeled. Then, a calibration process was conducted to confirm the accuracy of the model by replicating the existing conditions.

Travel time along the corridor was used as the main calibration factor. Based on the *FHWA Traffic Analysis Toolbox Volume III,* calibration criteria for travel time are listed in Table 2.

Table 2. Calibration Criteria for Travel Time

Criteria	Acceptance Targets
Modeled travel time within $\pm$ 1 minute for routes with observed travel time less than 7 minutes.	All routes identified in the Data Collection Plan
Modeled travel time within $\pm$ 15% for routes with observed travel times greater than 7 minutes.	All routes identified in the Data Collection Plan

Two routes were identified initially in the data collection plan to measure the travel time along the study corridor including eastbound and westbound travel between Scottsville Road and

Russellville Road. An average of multiple travel time runs was used as the observed travel time for each route. Table 3 represents the travel time outputs from SimTraffic and the related comparison to the observed travel times. As it is shown in Table 3, travel times for both routes under AM and PM peak scenarios meet the calibration criteria.

Table 3. Travel Time Calibration

Scenario	Route	Observed Travel Time	SimTraffic Output Travel Time	Difference (sec)	Meet Criteria
AM Peak Hour	Eastbound	00:08:49	00:08:16	-33	Within ± 15% (Yes)
	Westbound	00:08:19	00:07:48	-31	Within ± 15% (Yes)
PM Peak Hour	Eastbound	00:08:25	00:08:55	+30	Within ± 15% (Yes)
	Westbound	00:08:51	00:08:40	-11	Within ± 15% (Yes)

Calibration results indicate the Synchro model replicates existing conditions within an acceptable level which provides sufficient confidence to create and analyze future year models based on the existing model to achieve realistic results. The calibrated model was then modified to model the future scenarios.

Capacity analyses were performed in order to evaluate the exiting operational performance of the signalized intersections along the corridor and to determine the impacts of future traffic growth on the intersections in the study area. The capacity calculations were performed according to the methods outlined in the *Highway Capacity Manual,* TRB 2000. The capacity analyses result in the determination of a Level of Service (LOS) for an intersection. The LOS is a concept used to describe how well an intersection or roadway operates. LOS A is the best, while LOS F is the worst. LOS D is typically considered as the minimum acceptable LOS for an intersection in an urbanized area. Table 4 presents the descriptions of LOS for signalized intersections.

Table 4: Description of Level of Service for Signalized Intersections

LEVEL OF SERVICE	DESCRIPTION	CONTROL DELAY (sec/veh)
A	Operations with very low delay. This occurs when progression is extremely favorable. Most vehicles do not stop at all.	$\leq$ 10
B	Operations with stable flows. This generally occurs with good progression and/or short cycle lengths. More vehicles stop than for LOS A, causing higher levels of average delay.	>10 and $\leq$ 20
C	Operations with stable flow. Occurs with fair progression and/or longer cycle lengths. The number of vehicles stopping is significant, although many still pass through the intersection without stopping.	>20 and $\leq$ 35
D	Approaching unstable flow. The influence of congestion becomes more noticeable. Longer delays may result from some combination of unfavorable progression, long cycle lengths, or high V/C ratios. Many vehicles stop.	>35 and $\leq$ 55
E	Unstable flow. This is considered to be the limit for acceptable delay. These high delays generally indicate poor progression, long cycle lengths, and high V/C ratios.	>55 and $\leq$ 80
F	Unacceptable delay. This condition often occurs with over saturation or with high V/C ratios. Poor progression and long cycle lengths may also cause such delay levels.	>80.0

Source: *Highway Capacity Manual*, TRB 2000

The results of the capacity analyses for the existing and projected conditions for horizon year 2020 at the study area intersections are presented in Table 5 and Table 6. For the analyses, the intersection configurations were the same as the existing conditions. As shown in Table 5 and Table 6, almost half of the study intersections were found to operate at acceptable LOS D or better through the 2020 planning horizon. The rest of the intersections are expected to operate with LOS E or F by future year 2020 which are described below.

- The intersection of Campbell Lane and Scottsville Rd is expected to operate at LOS E during the weekday AM peak hour and LOS F during the weekday PM peak hour by year 2020. During the AM peak hour the northbound through movement is expected to experience the highest amount of delay while eastbound left and eastbound right turning movements are identified as critical movements during the PM peak hour.

- The intersection of Campbell Lane and Walton Ave is expected to operate at LOS E during the weekday PM peak hour by year 2020. Northbound and southbound approaches are found to experience the longest delays.
- The intersection of Campbell Lane and McIntosh St is expected to operate at LOS F during the weekday AM peak hour and LOS E during the weekday PM peak hour by year 2020. Additional phases (westbound left turn and northbound approach) for the signal timing were added based on an assumed northbound leg at this intersection.
- The intersection of Campbell Lane and Smallhouse Rd is expected to operate at LOS F during the weekday AM peak hour and LOS E during the weekday PM peak hour by year 2020. The majority of the movements are expected to experience long delays with LOS F.
- The intersection of Campbell Lane and Nashville Rd is expected to operate at LOS F during both weekday AM and PM peak hours by year 2020. During the AM peak hour the westbound approach is expected to experience the highest amount of delay while the eastbound approach is expected to experience the highest amount of delay during the PM peak hour.
- The intersection of Campbell Lane and Russellville Rd is expected to operate at LOS E during both weekday AM and PM peak hours by year 2020. During the AM peak hour the northbound approach is expected to experience the highest amount of delay while all approaches are expected to experience LOS E during the PM peak hour.

In addition to calculated delay and LOS, multi-runs of simulation were conducted for each scenario using SimTraffic and 95^{th} queue lengths were obtained. Table 7 and Table 8 indicates the queue results for each scenario for both AM and PM peak hours. These analyses were used to evaluate the need for roadway and traffic control improvements within the study area.

Table 5. 2015 and 2020 Existing System Intersection Operation Results – AM Peak Hour

US 231 Intersection	Scenario	Criteria	EBL	EBT	EBR	Approach	WBL	WBT	WBR	Approach	NBL	NBT	NBR	Approach	SBL	SBT	SBR	Approach	Overall
Scottsville Rd	Existing (2015)	Delay(sec)	41.7	60.3	48.7	54.5	62.8	61.9	50.0	58.3	61.2	65.5	28.7	60.9	28.7	57.0	34.5	38.9	55.2
		LOS	D	E	D	D	E	E	D	E	E	E	C	E	C	E	C	D	E
	Future (2020)	Delay(sec)	39.6	62.3	46.2	54.6	62.8	61.4	48.2	57.6	86.0	125.9	32.0	108.1	57.5	35.1	29.7	39.3	75.4
		LOS	D	E	D	D	E	E	D	E	F	F	C	F	E	D	C	D	E
Gary Farms Blvd	Existing (2015)	Delay(sec)	12.2	18.3	295.3	39.1	6.4	11.7		11.4		48.7	42.0	47.8		47.8	40.3	44.6	27.0
		LOS	B	B	F	D	A	B		B		D	D	D		D	D	D	C
	Future (2020)	Delay(sec)	13.2	20.8	67.0	23.9	6.7	12.4		12.1		53.3	41.9	51.8		50.5	40.3	46.0	22.2
		LOS	B	C	E	C	A	B		B		D	D	D		D	D	D	C
Walton Ave	Existing (2015)	Delay(sec)	4.4	4.7	1.1	4.3	1.6	2.7		2.6		48.8	40.8	44.4		41.4		41.4	7.7
		LOS	A	A	A	A	A	A		A		D	D	D		D		D	A
	Future (2020)	Delay(sec)	4.5	5.0	1.1	4.5	2.3	3.3		3.2		51.2	40.0	45.4		40.6		40.6	7.8
		LOS	A	A	A	A	A	A		A		D	D	D		D		D	A
American Ave	Existing (2015)	Delay(sec)	5.5	8.0	6.4	7.8	2.4	3.6		3.5		42.9	39.3	41.1		42.1		42.1	9.5
		LOS	A	A	A	A	A	A		A		D	D	D		D		D	A
	Future (2020)	Delay(sec)	5.7	9.2	6.8	8.9	2.8	3.8		3.7		43.4	38.9	41.3		41.8		41.8	10.0
		LOS	A	A	A	A	A	A		A		D	D	D		D		D	A
Crossridge Blvd	Existing (2015)	Delay(sec)	5.8	9.9	7.6	9.2	9.2	13.4		13.3		38.7	28.1	34.9	28.0	27.9	28.0	28.0	15.1
		LOS	A	A	A	A	A	B		B		D	C	C	C	C	C	C	B
	Future (2020)	Delay(sec)	10.6	11.3	7.9	11.0	11.4	17.5		17.3		46.9	29.6	41.0	31.2	30.7	29.9	30.3	18.4
		LOS	B	B	A	B	B	B		B		D	C	D	C	C	C	C	B
McIntosh St	Existing (2015)	Delay(sec)	3.0	4.0		3.9		2.8	2.5	2.7					65.7			65.7	7.6
		LOS	A	A		A		A	A	A					E			E	A
	Future (2020)	Delay(sec)	53.5	89.8		86.9	41.2	50.6	35.7	48.3		162.3		162.3		21.1		21.1	84.0
		LOS	D	F		F	D	D	D	D		F		F		C		C	F

US 231 Intersection	Scenario	Criteria	EBL	EBT	EBR	Approach	WBL	WBT	WBR	Approach	NBL	NBT	NBR	Approach	SBL	SBT	SBR	Approach	Overall
Westen St	Existing (2015)	Delay(sec)	15.3	4.9		7.9		8.0		8.0					63.7		59.3	61.0	12.1
		LOS	B	A		A		A		A					E		E	E	B
	Future (2020)	Delay(sec)	45.9	11.7		19.9		35.4		35.4					67.9		56.0	62.2	29.6
		LOS	D	B		B		D		D					E		E	E	C
Smallhouse Rd	Existing (2015)	Delay(sec)	31.9	49.2	166.2	51.3	29.8	31.6	48.7	33.5	27.1	56.6	38.3	45.7	33.6	39.9	38.0	38.0	44.7
		LOS	C	D	F	D	C	C	D	C	C	E	D	D	C	D	D	D	D
	Future (2020)	Delay(sec)	61.3	264.5	158.0	231.4	63.0	238.3	19.2	196.6	24.9	53.9	37.1	43.4	28.7	34.9	33.4	33.0	163.0
		LOS	E	F	F	F	E	F	B	F	C	D	D	D	C	C	C	C	F
Thoroughbred Dr	Existing (2015)	Delay(sec)	18.8	34.8		32.8	18.6	17.1		17.2		117.3	32.7	95.3		120.0	33.3	87.6	42.3
		LOS	B	C		C	B	B		B		F	C	F		F	C	F	D
	Future (2020)	Delay(sec)	67.0	87.9		85.6	55.2	65.4		65.1		219.9	32.8	169.1		229.9	33.4	160.9	93.5
		LOS	E	F		F	E	E		E		F	C	F		F	C	F	F
Harvard Dr	Existing (2015)	Delay(sec)	2.9	18.4		18.0	4.5	7.1		6.9	113.2	43.4		66.9	46.7	42.6		44.8	19.4
		LOS	A	B		B	A	A		A	F	D		E	D	D		D	B
	Future (2020)	Delay(sec)	4.0	23.1		22.6	8.6	10.3		10.2	139.1	53.8		79.9	48.8	42.7		46.3	22.4
		LOS	A	C		C	A	B		B	F	D		E	D	D		D	C
Nashville Rd	Existing (2015)	Delay(sec)	61.6	78.9	40.3	72.6	97.1	87.4	155.5	112.5	90.2	48.3	39.1	51.9	95.5	40.1	14.6	55.6	72.3
		LOS	E	E	D	E	F	F	F	F	F	D	D	D	F	D	B	E	E
	Future (2020)	Delay(sec)	72.6	83.4	36.6	77.2	174.2	76.9	96.0	106.2	92.8	64.3	52.1	65.0	100.5	46.3	24.0	66.6	81.2
		LOS	E	F	D	E	F	E	F	F	F	E	D	E	F	D	C	E	F
Industrial Dr	Existing (2015)	Delay(sec)	8.8	18.2	11.6	16.7	13.2	17.4	14.7	16.9		60.6	27.7	56.4	30.6	28.7		29.4	23.1
		LOS	A	B	B	B	B	B	B	B		E	C	E	C	C		C	C
	Future (2020)	Delay(sec)	11.1	21.6	12.4	19.9	15.2	20.9	15.9	20.0		67.2	27.5	61.8	31.9	28.5		29.9	25.8
		LOS	B	C	B	B	B	C	B	C		E	C	E	C	C		C	C
Russellville Rd	Existing (2015)	Delay(sec)	33.1	55.7	26.1	44.8	31.6	45.6		42.3	24.4	125.6	33.1	68.3	32.1	34.5		33.8	52.9
		LOS	C	E	C	D	C	D		D	C	F	C	E	C	C		C	D
	Future (2020)	Delay(sec)	40.1	79.2	28.1	61.1	37.1	55.7		51.3	30.0	192.6	44.1	99.9	40.5	37.4		38.4	71.6
		LOS	D	E	C	E	D	E		D	C	F	D	F	D	D		D	E

Table 6. 2015 and 2020 Existing System Intersection Operation Results – PM Peak Hour

US 231 Intersection	Scenario	Criteria	EBL	EBT	EBR	Approach	WBL	WBT	WBR	Approach	NBL	NBT	NBR	Approach	SBL	SBT	SBR	Approach	Overall
Scottsville Rd	Existing (2015)	Delay(sec)	61.6	88.6	338.8	198.4	68.6	74.9	56.8	68.0	75.2	35.0	28.3	47.8	64.9	51.5	30.1	51.2	92.7
		LOS	E	F	F	F	E	E	E	E	E	C	C	D	E	D	C	D	F
	Future (2020)	Delay(sec)	101.3	120.8	500.9	291.1	70.1	101.8	57.0	81.6	111.8	36.9	29.1	63.7	67.7	64.6	31.8	60.8	128.2
		LOS	F	F	F	F	E	F	E	F	F	D	C	E	E	E	C	E	F
Gary Farms Blvd	Existing (2015)	Delay(sec)	37.1	21.0	4.0	19.7	23.9	41.3		40.1		72.6	43.3	65.0		81.1	43.3	67.8	40.7
		LOS	D	C	A	B	C	D		D		E	D	E		F	D	E	D
	Future (2020)	Delay(sec)	55.4	26.3	16.5	27.7	25.6	60.7		58.5		84.5	41.6	74.0		118.2	44.8	90.8	54.5
		LOS	E	C	B	C	C	E		E		F	D	E		F	D	F	D
Walton Ave	Existing (2015)	Delay(sec)	5.2	12.5	15.7	12.8	25.7	16.7		18.5		73.3	38.2	58.8		137.2		137.2	34.1
		LOS	A	B	B	B	C	B		B		E	D	E		F		F	C
	Future (2020)	Delay(sec)	6.7	12.2	10.8	11.8	42.5	21.5		25.1		143.0	38.3	103.2		295.9		295.9	56.3
		LOS	A	B	B	B	D	C		C		F	D	F		F		F	E
American Ave	Existing (2015)	Delay(sec)	9.0	14.2	10.3	13.7	12.4	16.2		15.8		74.0	46.0	63.2		46.4		46.4	20.5
		LOS	A	B	B	B	B	B		B		E	D	E		D		D	C
	Future (2020)	Delay(sec)	11.2	18.7	12.3	17.8	19.5	20.0		19.9		76.8	43.4	65.1		43.9		43.9	24.1
		LOS	B	B	B	B	B	B		B		E	D	E		D		D	C
Crossridge Blvd	Existing (2015)	Delay(sec)	8.5	12.8	8.3	12.0	7.8	16.5		15.9		30.3	28.3	29.4	29.5	29.4	29.1	29.2	16.3
		LOS	A	B	A	B	A	B		B		C	C	C	C	C	C	C	B
	Future (2020)	Delay(sec)	42.5	15.7	9.1	20.7	11.9	42.4		40.7		38.4	30.3	36.1	32.1	31.4	37.1	36.0	31.6
		LOS	D	B	A	C	B	D		D		D	C	D	C	C	D	D	C
McIntosh St	Existing (2015)	Delay(sec)	5.8	6.7		6.7		7.1	4.8	6.8					66.6			66.6	12.2
		LOS	A	A		A		A	A	A					E			E	B
	Future (2020)	Delay(sec)	169.0	14.9		32.0	191.1	14.7	8.2	33.6		84.5		84.5		201.6		201.6	60.4
		LOS	F	B		C	F	B	A	C		F		F		F		F	E

US 231 Intersection	Scenario	Criteria	EBL	EBT	EBR	Approach	WBL	WBT	WBR	Approach	NBL	NBT	NBR	Approach	SBL	SBT	SBR	Approach	Overall
Westen St	Existing (2015)	Delay(sec)	67.4	3.2		14.8		13.3		13.3					109.8		87.9	95.4	23.9
		LOS	E	A		B		B		B					F		F	F	C
	Future (2020)	Delay(sec)	92.5	5.5		17.3		31.9		31.9					119.9		89.3	101.6	33.1
		LOS	F	A		B		C		C					F		F	F	C
Smallhouse Rd	Existing (2015)	Delay(sec)	27.8	27.3	13.1	26.2	64.9	24.1	8.3	29.1	64.7	85.5	70.0	76.8	61.4	109.7	73.4	91.7	42.7
		LOS	C	C	B	C	E	C	A	C	E	F	E	E	E	F	E	F	D
	Future (2020)	Delay(sec)	90.7	73.8	32.0	72.5	356.6	26.4	6.0	75.8	66.3	90.8	73.8	80.2	60.9	109.2	71.8	89.3	76.7
		LOS	F	E	C	E	F	C	A	E	E	F	E	F	E	F	E	F	E
Thoroughbred Dr	Existing (2015)	Delay(sec)	7.9	11.3		11.2	5.9	5.8		5.8		129.7	77.1	110.5		116.1	77.1	99.7	19.4
		LOS	A	B		B	A	A		A		F	E	F		F	E	F	B
	Future (2020)	Delay(sec)	14.6	16.3		16.2	81.0	6.1		9.3		136.3	74.3	111.8		126.9	74.2	106.2	22.7
		LOS	B	B		B	F	A		A		F	E	F		F	E	F	C
Harvard Dr	Existing (2015)	Delay(sec)	8.5	14.1		13.7	15.8	6.6		8.0	168.7	80.8		120.8	135.5	79.0		104.1	27.2
		LOS	A	B		B	B	A		A	F	F		F	F	E		F	C
	Future (2020)	Delay(sec)	12.6	22.0		21.4	97.6	5.7		19.5	209.0	83.6		133.0	587.1	79.5		315.7	46.8
		LOS	B	C		C	F	A		B	F	F		F	F	E		F	D
Nashville Rd	Existing (2015)	Delay(sec)	56.5	62.4	24.3	55.0	92.6	48.6	36.6	58.6	57.1	60.7	50.5	57.0	62.8	60.4	30.6	58.9	57.4
		LOS	E	E	C	D	F	D	D	E	E	E	D	E	E	E	C	E	E
	Future (2020)	Delay(sec)	60.2	175.1	25.6	143.6	185.3	76.1	41.4	100.1	57.4	63.0	59.6	60.8	67.5	63.7	31.8	62.8	95.7
		LOS	E	F	C	F	F	E	D	F	E	E	E	E	E	E	C	E	F
Industrial Dr	Existing (2015)	Delay(sec)	7.9	12.3	9.0	11.8	7.5	13.5	8.9	12.9		38.3	27.0	35.5	32.8	28.5		30.9	15.9
		LOS	A	B	A	B	A	B	A	B		D	C	D	C	C		C	B
	Future (2020)	Delay(sec)	9.5	14.6	9.6	14.0	8.7	16.1	9.5	15.3		38.5	26.8	35.4	37.6	28.3		33.7	17.8
		LOS	A	B	A	B	A	B	A	B		D	C	D	D	C		C	B
Russellville Rd	Existing (2015)	Delay(sec)	40.1	57.0	38.3	48.2	41.8	46.1		44.8	37.4	79.8	25.0	52.2	56.6	45.0		48.5	48.4
		LOS	D	E	D	D	D	D		D	D	E	C	D	E	D		D	D
	Future (2020)	Delay(sec)	43.5	76.9	43.9	62.3	105.0	58.8		72.5	39.7	96.5	28.0	59.3	110.6	45.7		66.7	65.4
		LOS	D	E	D	E	F	E		E	D	F	C	E	F	D		E	E

Table 7. 2015 and 2020 Existing System 95th Percentile Queue Length (AM Peak Hour)

		95th Queue Length (Feet)											
US 231 Intersection	Scenario	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Scottsville Rd	Existing (2015)	107	202	90	114	225	167	273*	466	193	143	186	41
	Future (2020)	137	231	111	120	256	181	485*	664	409*	156	198	53
Gary Farms Blvd	Existing (2015)	44	147	34	34	184	217	84		29	113		49
	Future (2020)	55	180	39	36	218	249	96		29	121		51
Walton Ave	Existing (2015)	9	81	24	52	45	41	70		47	38		
	Future (2020)	12	93	36	57	39	38	76		48	38		
American Ave	Existing (2015)	4	131	38	50	56	75	85		54	67		
	Future (2020)	12	156	57	46	60	81	76		55	70		
Crossridge Blvd	Existing (2015)	76	112	17	34	115	125	138		61	19	19	56
	Future (2020)	124	180	23	54	165	182	163		62	54	60	82
McIntosh St	Existing (2015)	47	88	NA	NA	79	36	NA	NA	NA	129		
	Future (2020)	210	428	438	142	357	131	445			243		
Westen St	Existing (2015)	213	133	NA	NA	127	144	NA	NA	NA	87	NA	86
	Future (2020)	340*	330	NA	NA	715	729	NA	NA	NA	111	NA	119
Smallhouse Rd	Existing (2015)	250	386	48	80	225	62	228*	592	256	88	132	75
	Future (2020)	291	1,557	390*	356*	497	325*	227*	596	286*	104	157	116
Thoroughbred Dr	Existing (2015)	109	312	273	26	106	113	223		69	145		63
	Future (2020)	251	807	807	42	215	231	244		108	168		77
Harvard Dr	Existing (2015)	59	187	179	79	118	148	148	97		47	46	
	Future (2020)	59	216	206	125	193	219	140	139		62	46	
Nashville Rd	Existing (2015)	307*	487	279*	152	270	153	222	380	203	141	154	15
	Future (2020)	330*	627	400*	276*	388	353	257	448	253	185	159	16
Industrial Dr	Existing (2015)	110	207	41	38	149	36	193		34	61	58	
	Future (2020)	123	253	41	93	286	47	226		36	78	69	
Russellville Rd	Existing (2015)	126	253	69	158	224	275	269	651	213	163	210	173
	Future (2020)	156	305	78	214	298	377	1,332	1,816	1,679	196	234	187

*95th queue length exceeds the turn lane storage length.

Table 8. 2015 and 2020 Existing System 95th Queue Length (PM Peak Hour)

		95th Queue Length (Feet)											
US 231 Intersection	Scenario	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Scottsville Rd	Existing (2015)	251	346	395*	135	177	54	151	261	54	272*	436	197
	Future (2020)	288	656	568*	222*	345	106	286*	281	53	362*	498	316*
Gary Farms Blvd	Existing (2015)	125	249	97	170	390	402	324		73	357		107
	Future (2020)	171	365	162	224	503	521	402		164	427		133
Walton Ave	Existing (2015)	96	276	82	168	229	223	208		96	240		
	Future (2020)	90	288	89	228	346	329	248		129	253		
American Ave	Existing (2015)	12	301	177*	95	181	174	244		79	55		
	Future (2020)	44	356	197*	110	210	214	271		103	51		
Crossridge Blvd	Existing (2015)	133	228	34	114	275	275	77		47	66	76	112
	Future (2020)	227	326	65	247	829	833	143		57	81	190	238
McIntosh St	Existing (2015)	55	209	NA	NA	209	50	NA	NA	NA	269		
	Future (2020)	237	388	389	348	961	149	556			603		
Westen St	Existing (2015)	210	159	NA	NA	375	369	NA	NA	NA	157	NA	247
	Future (2020)	275	297	NA	NA	767	764	NA	NA	NA	187	NA	554
Smallhouse Rd	Existing (2015)	243	434	179	298	451	204*	194	331	198	191	471	254
	Future (2020)	264	822	390*	459*	535	240*	221	527	279	227	536	269
Thoroughbred Dr	Existing (2015)	51	260	267	63	95	104	228		73	96		51
	Future (2020)	66	416	420	86	128	139	246		93	143		61
Harvard Dr	Existing (2015)	123	261	264	162	216	191	260	178		134	159	
	Future (2020)	143	285	286	245	555	516	416	295		140	175	
Nashville Rd	Existing (2015)	159	403	201*	281*	395	161	159	255	183	258	320	37
	Future (2020)	398*	1,284	509*	489*	534	371	183	279	322	376	393	43
Industrial Dr	Existing (2015)	42	211	45	65	258	43	135		43	97	79	
	Future (2020)	85	290	91	87	271	76	135		53	115	100	
Russellville Rd	Existing (2015)	83	251	135	370	326	359	284	390	121	235	406	347
	Future (2020)	249	538	177	467	416	440	304	503	200	217	637	555

*95th queue length exceeds the turn lane storage length.

Traffic Recommendations

Based on the capacity and queue analyses presented, various recommendations are made to improve traffic operations at signalized intersections along the corridor. These improvements include the following:

- Optimize the signal timings and coordination along the corridor to obtain more effective traffic flow. It is noteworthy that some limitations were considered in the corridor's optimization. According to the existing signal timing provided by KYTC, intersections of US 231/Scottsville Rd, US 231/Nashville Rd, and US 231/Russellville Rd are coordinated along the northbound and southbound approaches of the cross road. In the optimization, the cycle length and coordination orientation for those intersections remained as existing to avoid any negative impact on the traffic flow along Scottsville Rd, Nashville Rd and Russellville Rd. Further study will be required to optimize Scottsville Rd, Nashville Rd, and Russellville Rd corridors separately to improve LOS at those intersections.
- Provide an overlap signal timing phase for the eastbound right turning movement at the intersection of US 231 and Scottsville Rd.
- Widen US 231 to provide a second eastbound left turning lane at the intersection of US 231 and Scottsville Rd.
- Construction of a new leg at the intersection of US 231 and McIntosh Street for the northbound approach would be required to be built upon the completion of any future development at this location by the developer. In addition, a separate left turn lane and a separate right turn lane for the southbound approach is recommended along with corresponding left turn signal phasing. As a result, additional phasing to the existing signal timing needs to be provided. Kentucky Transportation Cabinet (KYTC) would adjust the signal timing once the developer constructed the other requirements.
- Extend right turn lane storage on the southbound approach of Smallhouse Rd at the intersection of US 231 and Smallhouse Rd and provide a center two-way left turn lane. These recommendations were obtained from a previous study provided by the City of Bowling Green at this intersection.
- Modify lane configurations at the intersection of US 231 and Thoroughbred Dr to provide one left turn lane and one thru/right turn lane for both northbound and southbound approaches on Thoroughbred Dr. This would provide a better LOS due to the larger number of northbound and southbound left turning volumes compared to the through and right turning movements. A signal modification would be required to provide left turning phases for southbound and northbound approaches as well.
- Widen US 231 at the intersection of US 231 and Russellville Road to provide a second westbound left turn lane.
- Widen southbound approach of Russellville Road to provide a second southbound left turn lane.

Capacity and queue analyses were also conducted for the study intersections after implementing the above recommendations. Table 8 and Table 9 present the operations of the study intersections. The capacity analyses presented in Table 8 indicate that overall LOS improvement can be achieved with the proposed recommendations at the study intersections. As shown by the results of the queue length analyses in Table 9, a few movements are expected to continue to experience queue lengths longer than the available turn lane storage.

However, these storage deficiencies are relatively small and the impacts of lengthening the turn lanes are not justified as additional recommendations.

Table 8. 2020 Proposed System Intersection Operation Results

US 231 Intersection	Scenario	Criteria	EBL	EBT	EBR	Approach	WBL	WBT	WBR	Approach	NBL	NBT	NBR	Approach	SBL	SBT	SBR	Approach	Overall
Scottsville Rd	AM Peak Hour	Delay(sec)	75.3	54.4	32.8	51.2	62.8	56.8	48.2	55.1	58.9	70.1	27.3	63.4	70.0	40.9	34.1	46.3	56.3
		LOS	E	D	C	D	E	E	D	E	E	E	C	E	E	D	C	D	E
	PM Peak Hour	Delay(sec)	73.8	100.6	96.6	94.1	127.5	110.4	44.1	96.9	62.9	33.8	26.9	43.9	67.7	94.8	34.7	82.3	79.2
		LOS	E	F	F	F	F	F	D	F	E	C	C	D	E	F	C	F	E
Gay Farms Blvd	AM Peak Hour	Delay(sec)	8.8	92.5	9.6	9.5	12.6	18.2		18.0		30.9	27.5	30.5		34.8	27.8	31.7	16.6
		LOS	A	A	A	A	B	B		B		C	C	C		C	C	C	B
	PM Peak Hour	Delay(sec)	26.1	20.0	3.7	17.8	35.2	68.8		66.7		94.2	35.1	79.7		92.5	35.7	71.3	51.3
		LOS	C	C	A	B	D	E		E		F	D	E		F	D	E	D
Walton Ave	AM Peak Hour	Delay(sec)	5.5	5.6	5.1	5.5	4.3	5.9		5.8		67.9	55.9	61.6		56.5		56.5	10.9
		LOS	A	A	A	A	A	A		A		E	E	E		E		E	B
	PM Peak Hour	Delay(sec)	11.5	17.1	5.1	15.0	22.1	19.2		19.7		60.6	25.8	47.4		74.2		74.2	26.8
		LOS	B	B	A	B	C	B		B		E	C	D		E		E	C
American Ave	AM Peak Hour	Delay(sec)	1.8	3.4	0.3	3.1	4.1	4.6		4.6		31.9	27.2	29.7		29.6		29.6	6.2
		LOS	A	A	A	A	A	A		A		C	C	C		C		C	A
	PM Peak Hour	Delay(sec)	7.1	12.8	3.4	11.5	16.9	6.2		7.1		58.6	33.4	49.8		33.6		33.6	13.9
		LOS	A	B	A	B	B	A		A		E	C	D		C		C	B
Crossridge Blvd	AM Peak Hour	Delay(sec)	24.3	12.3	7.9	15.2	6.3	10.3		10.2		40.0	23.2	34.2	24.4	24.1	23.5	23.8	16.6
		LOS	C	B	A	B	A	B		B		D	C	C	C	C	C	C	B
	PM Peak Hour	Delay(sec)	66.5	5.5	0.6	17.3	6.7	13.9		13.5		52.5	39.0	41.9	48.8	40.5	59.8	56.3	23.2
		LOS	E	A	A	B	A	B		B		D	D	D	D	D	E	E	C
McIntosh St	AM Peak Hour	Delay(sec)	17.2	16.7		16.8	15.8	16.7	14.5	16.4	47.4	38.3	38.2	45.0	55.9	63.3	64.9	61.5	27.1
		LOS	B	B		B	B	B	B	B	D	D	D	D	E	E	E	E	C
	PM Peak Hour	Delay(sec)	49.4	28.0		30.4	57.6	13.8	1.8	17.6	45.5	66.0	45.2	54.4	45.6	43.4	34.7	42.4	28.6
		LOS	D	C		C	E	B	A	B	D	E	D	D	D	D	C	D	C

US 231 Intersection	Scenario	Criteria	EBL	EBT	EBR	Approach	WBL	WBT	WBR	Approach	NBL	NBT	NBR	Approach	SBL	SBT	SBR	Approach	Overall
Westen St	AM	Delay(sec)	22.8	1.6		6.7		38.5		38.5					79.4		57.8	69.0	25.2
	Peak Hour	LOS	C	A		A		D		D					E		E	E	C
	PM	Delay(sec)	38.7	1.2		6.3		16.9		16.9					82.1		47.4	61.3	17.1
	Peak Hour	LOS	D	A		A		B		B					F		D	E	B
Smallhouse Rd	AM	Delay(sec)	75.1	36.7	20.6	41.6	93.0	22.3	3.6	27.1	33.6	98.4	45.7	69.7	82.6	46.1	43.1	51.2	43.5
	Peak Hour	LOS	E	D	C	D	F	C	A	C	C	F	D	E	F	D	D	D	D
	PM	Delay(sec)	50.0	81.7	15.6	74.8	141.1	16.2	8.3	34.8	74.6	88.4	39.6	67.8	71.2	140.8	38.4	98.3	62.2
	Peak Hour	LOS	D	F	B	E	F	B	A	C	E	F	D	E	E	F	D	F	E
Thoroughbred Dr	AM	Delay(sec)	93.9	25.4		33.0	18.8	11.9		12.1		111.2	65.9	95.9		101.7	61.4	86.1	36.7
	Peak Hour	LOS	F	C		C	B	B		B		F	E	F		F	E	F	D
	PM	Delay(sec)	11.3	11.7		11.7	31.8	14.2		15.0		60.8	44.3	54.0		90.6	48.3	73.1	18.3
	Peak Hour	LOS	B	B		B	C	B		B		E	D	D		F	D	E	B
Harvard Dr	AM	Delay(sec)	15.1	19.3		19.2	12.6	7.0		7.5	71.2	56.0		60.6	84.8	54.6		72.3	18.3
	Peak Hour	LOS	B	B		B	B	A		A	E	E		E	F	D		E	B
	PM	Delay(sec)	21.5	28.6		28.2	27.9	8.6		11.5	63.8	40.0		49.4	105.8	39.0		70.1	25.2
	Peak Hour	LOS	C	C		C	C	A		B	E	D		D	F	D		E	C
Nashville Rd	AM	Delay(sec)	98.5	83.4	37.4	79.3	109.2	46.6	52.2	63.3	98.4	69.8	55.7	70.0	112.9	49.0	38.2	74.3	70.7
	Peak Hour	LOS	F	F	D	E	F	D	D	E	F	E	E	E	F	D	D	E	E
	PM	Delay(sec)	71.2	110.5	36.4	96.1	97.0	66.3	46.9	71.3	100.9	104.4	75.2	93.9	111.8	76.4	34.4	87.2	86.2
	Peak Hour	LOS	E	F	D	F	F	E	D	E	F	F	E	F	F	E	C	F	F
Industrial Dr	AM	Delay(sec)	18.5	21.6	12.3	20.8	22.6	23.2	17.4	22.5		57.1	25.9	52.9	29.6	26.8		28.0	25.8
	Peak Hour	LOS	B	C	B	C	C	C	B	C		E	C	D	C	C		C	C
	PM	Delay(sec)	17.1	13.9	9.3	13.6	12.7	13.7	8.4	13.3		51.0	33.4	46.3	57.5	35.3		48.1	18.8
	Peak Hour	LOS	B	B	A	B	B	B	A	B		D	C	D	E	D		D	B
Russellville Rd	AM Peak Hour	Delay(sec)	66.9	64.9	27.9	56.5	55.0	49.8		51.1	27.7	76.0	30.3	47.9	74.8	42.4		53.3	51.3
		LOS	E	E	C	E	D	D		D	C	E	C	D	E	D		D	D
	PM Peak Hour	Delay(sec)	40.2	55.8	32.4	45.9	68.5	51.7		56.7	34.0	67.5	28.0	45.9	68.5	49.9		55.9	51.3
		LOS	D	E	C	D	E	D		E	C	E	C	D	E	D		E	D

Table 9. 95th Queue Length (2020 Proposed System)

95th Queue Length (Feet)													
US 231 Intersection	Scenario	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Scottsville Rd	AM Peak	87	231	127	115	254	203	269*	449	173	166	209	55
	PM Peak	255	415	475*	246*	330	107	262*	277	56	419*	692	407*
Gary Farms Blvd	AM Peak	54	150	39	32	200	225	79		25	104		41
	PM Peak	165	235	72	287	515	529	357		69	284		151
Walton Ave	AM Peak	11	113	36	53	54	59	86		46	38		
	PM Peak	121	264	67	187	228	205	202		109	195		
American Ave	AM Peak	8	116	42	48	63	68	84		55	59		
	PM Peak	14	226	125	110	142	119	241		85	57		
Crossridge Blvd	AM Peak	137	147	19	41	132	152	181		70	62	62	79
	PM Peak	244	381	62	130	343	339	181		55	74	322	259
McIntosh St	AM Peak	120	253	258	61	220	40	344	671	120	127	103	103
	PM Peak	253	437	442	357	863	147	115	226	118	246	386	189
Westen St	AM Peak	334	332	NA	NA	462	472	NA	NA	NA	121	NA	140
	PM Peak	254	215	NA	NA	550	551	NA	NA	NA	167	NA	255
Smallhouse Rd	AM Peak	286	920	321*	229	291	178*	253*	750	311*	123	142	113
	PM Peak	278	509	237	473*	515	267*	144*	247	182	143	321	129
Thoroughbred Dr	AM Peak	166	433	415	76	244	261	279	121		185	91	
	PM Peak	90	361	360	92	139	148	180	87		103	77	
Harvard Dr	AM Peak	65	244	245	112	192	250	137	113		58	43	
	PM Peak	151	242	250	241	366	353	193	163		102	102	
Nashville Rd	AM Peak	301*	593	365*	242	322	351	253	490	296	198	166	11
	PM Peak	319*	636	459*	444*	503	341	220	354	426	479	418	45
Industrial Dr	AM Peak	138	236	46	73	221	43	204		51	75	70	
	PM Peak	79	250	61	89	308	96	156		52	139	110	
Russellville Rd	AM Peak	158	293	73	146	263	337	259	594	214	193	233	192
	PM Peak	78	306	198	252	402	442	276	430	192	225	546	446

*95th queue length exceeds the turn lane storage length.

Bicycle and Pedestrian Levels-of-Service Analysis

As part of this study RPM also conducted an inventory of the existing bicycle and pedestrian infrastructure along the corridor to determine bicycle and pedestrian levels-of-service (BLOS/PLOS) along the corridor using the 2010 Highway Capacity Manual (HCM) methodology.

PLOS

The PLOS analysis uses various traffic and roadway factors to measure the walking suitability of a roadway segment. The equation for this evaluation is shown below:

$$\mathrm{PLOS} = -1.2276 \ln\left(W_v + 0.5 * W_1 + 50 * p_{pk} + W_{buf} * f_b + W_{aA} * f_{sw}\right) + 0.0091 \left(\frac{v_m}{4 * N_{th}}\right) + 4 * \left(\frac{S_r}{100}\right)^2 + 6.0468$$

Where:

PLOS =Pedestrian Level-of-Service
W_v =Effective total width of outside through lane, bicycle lane, and shoulder as a function of traffic volume (ft) – dependent on hourly traffic volume
W_1 =Effective width of combined bicycle lane and shoulder (ft) – dependent on percent of occupied on-street parking
p_{pk} =Proportion of on-street parking occupied (no on-street parking exists in the corridor)
W_{buf} =Buffer width between roadway and available sidewalk (ft)
f_b =Buffer area coefficient (assumed to be 1.00)
W_{aA} =Adjusted available sidewalk width = min (W_A, 10) (ft)
W_A =Available sidewalk width (= 0 if sidewalk does not exist or W_T –W_{buf} if sidewalk exists) (ft)
W_T =Total width of outside through lane, bicycle lane, and paved shoulder (ft) – dependent on presence and occupancy of on-street parking
f_{sw} =Sidewalk width coefficient (= 6 - 0.3W_{aA})
W_s =Width of sidewalk (ft)
v_m =Mid-segment demand flow rate (direction nearest to the subject sidewalk) (vph)
N_{th} =Number of through lanes on the segment in the subject direction of travel
S_r =Motorized vehicle running speed (assumed to be the posted speed limit) (mph)

The PLOS calculation is based on the peak hour volume. Once the above equation was calculated, the PLOS score is converted into a LOS ranging from A to F as shown in Table 10.

Table 10: Pedestrian Level-of-Service Rating

PLOS	Scoring Criteria	Users' Perception of Quality
A	$\leq$ 2.00	Extremely High
B	> 2.00 and $\leq$ 2.75	Very High
C	> 2.75 and $\leq$ 3.50	Moderately High
D	> 3.50 and $\leq$ 4.25	Moderately Low
E	> 4.25 and $\leq$ 5.00	Very Low
F	> 5.00	Extremely Low

The pedestrian infrastructure is consistent along US 231 from Scottsville Rd to Industrial Dr which includes five foot wide sidewalks on both sides with a two foot grass buffer between the back of curb and the sidewalk. From Industrial Drive to Russellville Rd, the sidewalk only exists along the westbound direction of US 231. The PLOS for the study corridor was calculated and was determined to be LOS D.

The PLOS rating of D is consistent with a typical pedestrian's experience walking along Campbell Lane. High volumes of traffic drive the PLOS down, though the existing pedestrian infrastructure is reasonably safe, adequate, and typical given its role and context as a suburban commercial corridor.

BLOS

Using the 2010 HCM, a Bicycle Level-of-Service (BLOS) analysis was conducted along the study corridor. The BLOS analysis uses various traffic and roadway factors to measure the biking suitability of a roadway segment. The equation for this evaluation is shown below:

$$\mathrm{BLOS} = 0.507\ln\left(\frac{\mathrm{V}}{\mathrm{PHF}*\mathrm{N}}\right) + 0.1999\big(1.1199\ln(\mathrm{S_p}-20)+0.8103\big)*(1+10.38\mathrm{HV})^2 + 7.066\left(\frac{1}{\mathrm{P}}\right)^2 - 0.005(\mathrm{W_e})^2 + 0.057$$

Where:

BLOS =Bicycle Level-of-Service
V =Hourly directional volume (vph)
PHF =Peak hour factor
N =Number of directional lanes
S_p =Posted speed limit (mph)
HV =Percentage of heavy vehicles (decimal)
P =FHWA's 5-point pavement surface condition rating – assumed to be 3
W_e =Average effective width of the outside through lane (ft) – dependent on width of shoulder, width of outside thru lane, and percent of occupied on-street parking.

Once the above equation was calculated, the BLOS score was converted into a LOS ranging from A to F as shown in Table 11.

Table 11: Bicycle Level-Of-Service Rating

BLOS	Scoring Criteria	Users' Perception of Quality
A	≤ 1.50	Extremely High
B	> 1.50 and ≤ 2.50	Very High
C	> 2.50 and ≤ 3.50	Moderately High
D	> 3.50 and ≤ 4.50	Moderately Low
E	> 4.50 and ≤ 5.50	Very Low
F	> 5.50	Extremely Low

Similar to PLOS, BLOS for the study corridor was calculated and the results indicate a BLOS F along the corridor. This poor LOS is due to a combination of high traffic and lack of space in which to ride. No bike lanes or any dedicated bicycle infrastructure exists along the study corridor.

Pedestrian and Bicycle Recommendations

Without a major widening of U.S. 231, typical bike lane construction adjacent to the travel lanes appears cost prohibitive due to drainage, sidewalk, and right-of-way impacts. There are likely other ways that bicycle travel can be accommodated such as construction of a multi-use path in place of an existing sidewalk. Development of a multi-form bikeway including some cycle track appears possible along parallel streets and/or rear property lines along the corridor. Various facilities could be explored in future study.

Conversely, the existing pedestrian infrastructure is already reasonably safe and adequate for this type of roadway. Given the land use context as a suburban commercial corridor, significant improvement to the pedestrian experience is not likely without a major redevelopment strategy involving higher commercial density, reduced building setbacks, or other urban policies contributing to a more walkable environment. However, incremental improvements to pedestrian accommodations can be made with some cost-effective recommendations. These include:

- Consider restriping the existing crosswalks along the study corridor with a consistent, high-visibility pattern.
- Provide new crosswalks at the following signalized intersection approaches. Relocation of stop lines will be required at some locations and installation should follow MUTCD standards.
    - Westbound approach at the intersection of US 231/Gary Farms Blvd.
    - Westbound and southbound approaches at the intersection of US 231/Walton Ave.
    - Eastbound, westbound and southbound approaches at the intersection of US 231/American Ave.
    - Eastbound, westbound and southbound approaches at the intersection of US 231/Crossridge Blvd.
    - All approaches at the intersection of US 231/McIntosh St.
    - Eastbound and westbound approaches at the intersection of US 231/Westen St.
    - Northbound and southbound approaches at the intersection of US 231/Smallhouse Rd.
    - Westbound approaches at the intersection of US 231/Thoroughbred Dr.
    - Eastbound, northbound and southbound approaches at the intersection of US 231/Harvard Dr.
    - Southbound approach at the intersection of US 231/Industrial Dr.
- Provide new crosswalks across other major driveways or unsignalized streets along the corridor.
- Incorporate the provision of sidewalks as part of new developments to allow full pedestrian access between the development and Campbell Lane.

- Curb ramps exist at most intersections, though many appear to be out of compliance with ADA requirements. A detailed ADA assessment can establish a plan for prioritization and future replacement.
- Pedestrian push buttons and signal heads appear to be in good condition overall. A detailed ADA assessment would determine whether adjustments are needed to placement (height, etc.) or location.