KAW VALLEY ENGINEERING

TRAFFIC IMPACT STATEMENT
MIXED USE DEVELOPMENT
FAIRVIEW CROSSING NORTH AT S. COMMERICAL STREET
SMITHVILLE, MISSOURI

Prepared For:

KANSAS CITY PROPERTIES & INVESTMENTS LLC
13530 Mount Olivet Road
Smithville, Missouri 64089

Prepared By:

KAW VALLEY ENGINEERING, INC.
8040 N Oak Trafficway
Kansas City, Missouri 64118

January 31, 2023

STATE OF MISSOURI
Matthew A. Cross
MATTHEW A.
CROSS
NUMBER
PE-2020008364
1/31/23
PROFESSIONAL ENGINEER

Project No. B20D4001

Mixed Use Development
Fairview Crossing North at S. Commercial Street
Smithville, Missouri
Project No. B20D4001

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EXHIBITS:

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January 31, 2023 **B20D4001**

Mr. Shane Crees
Kansas City Properties & Investments LLC
13530 Mount Olivet Road
Smithville, MO 64089

RE: TRAFFIC IMPACT STATEMENT
MIXED USE DEVELOPMENT
FAIRVIEW CROSSING NORTH AT S. COMMERICAL STREET
SMITHVILLE, MISSOURI

Dear Mr. Crees:

At the request of the City of Smithville, Missouri, Kaw Valley Engineering, Inc. (KVE) has had an opportunity to review the impacts of developing a 7.3-acre, mixed use project at southeast corner of the US Highway 169 and S. Commercial Street intersection may have on the adjacent street network. The purpose of this study is to provide discussion and document the potential intersection connection point at S. Commercial Street and geometric improvements that may be warranted by the development. There are three other connection points that were not evaluated. This report will be submitted to the City of Smithville to support the proposed Development Plan.

DEVELOPMENT DESCRIPTION

Kansas City Properties & Investments intends to develop a mixed-use project with 35,000 SF of leasable commercial space on an undeveloped Parcel on the south side of Smithville, Missouri. The Clay County parcel number included in the project boundary is 05917000700800. As illustrated on the attached site plan, the commercial lots will front US Highway 169 and access will be provided via northbound US Highway 169, S. Commercial Street, and the proposed Dorothy Street. For clarity of this report, the proposed roadway at the southern property, Fairview Crossing, is referred to as Dorothy Street. The proposed roadway running through this site, Fairview Crossing North, is referred to as Fairview Drive. In the future, both proposed roadways will be named Fairview Drive.

The subject property is bound by US Highway 169 to the west and S. Commercial Street to the north. The south end of the property abuts both commercial and undeveloped property. Adjacent properties to the east have been developed as large lot single family homes. S. Commercial Street is currently accessible by a 30’ wide drive which is approximately 120’ from the US Highway 169 intersection with S. Commercial Street. This drive will be removed and a new 28’ wide roadway, referred to as Fairview Drive, is proposed approximately 350’ east of the US 169 Highway intersection with S. Commercial Street. This roadway will intersect S. Commercial Street and will align with the opposing drive to the gas station. The (intent is to dedicate this access drive as public right of way and the assumed design speed for Fairview Drive was 15 mph. The proposed intersection with S. Commercial drive is approximately 350’ from US Highway 169 which complies with minimum intersection spacing per *Kansas City Metro American Public Works Association Section 5200*.

EXISTING ROAD CONDITIONS AND TRAFFIC VOLUMES

Within the corridor between US Highway 169 and Shamrock Way, S. Commercial Street is a two-lane improved roadway with curb and gutter. Four drives for commercial properties exist on the north side within a single 250’ span. These drives do not comply with spacing requirements provided by *American Public Works Association Section 5200* of 100’. This deviation makes any attempt to align the proposed Fairview Drive roadway with the adjacent fueling station drive also out of compliance. One drive exists on the south side of the roadway that will be removed as part of the development. A traffic signal exists at the intersection of US Highway 169 and S. Commercial Street. During on site observation between the hours of 11am to 1pm and 4pm to 6pm, westbound queuing was noted to be a maximum of 150’ behind the stop bar.

Tabulated traffic volume and distribution diagrams for the existing conditions are included in the appendix. Figure 1 presents the Existing Peak Hour Traffic Volume and Distribution Diagram.

SITE TRAFFIC PROJECTIONS

A trip generation analysis was completed to predict the number of vehicular trips that the proposed mixed used development may generate during a typical weekday. This analysis focuses on typical mid-day and evening peak hours that may coincide with peak hour traffic on the adjacent roadways. Vehicle trip generation estimates for retail stores, restaurants, and small offices were determined using the Institute of Transportation Engineers (ITE) Trip Generation Manual, *10th Edition*. Mid-day peak hour of generator estimates are not provided in the manual however, the manual does discuss in the 820 land use section “the overall highest vehicle volumes during the AM and PM on a weekday were counted between 11:45 a.m. and 12:45 p.m. and 12:15 and 1:15 p.m., respectively”. Since those periods fell within the analyzed mid-day peak hour span, their values were average together to provide a mid-day peak hour estimate.

ITE Trip Generation: 10th Edition					
Land Use	**GFA (1000)**	**ITE Code**	**Daily Traffic**	**Mid-day Peak Hour of Generator (11 AM-1 PM)**	**PM Peak Hour of Adjacent (4-6 PM)**
Shopping Center	**35.2**	**820**	**1182**	**126**	**469**
	Enter		591	66	225
	Exit		591	60	244

Trips to and from the proposed entryway will be adjusted to account for pass-by traffic associated with the commercial portion of the development. Pass-by traffic is understood to be traffic already on the adjacent street network that may divert to the proposed development. The ITE Manual indicates that a Pass-by rate of 27% may be realized for the Shopping Center portion of the generated by the project.

PROPOSED TRIP DISTRIBUTION

Since multiple points of access are proposed within the development, a distribution was applied across trip generation estimates to assign trips to each access point. See the table below for the access distribution for the entire development.

Proposed Overall Trip Distribution	
Access	Distribution
Fairview Drive at S. Commercial Street	38%
US Central Bank Shared Drive at S. US Highway 169	19%
Dorothy Street	5%
Fairview Drive at S. US Highway 169	38%
Total	100%

It was assumed that 38% of trips would enter and exit through the intersection of Fairview Drive at S. Commercial Street. This is based on the expectation that this access, along with the intersection of Fairview Drive at S. US Highway 169 would serve as the primary access points for the development. The shared drive at US Central Bank provides relatively close access for a large portion of the development but involves maneuvering through various parking lots to reach different areas of the site and the intersection. Thus, it was assumed that this access would see a low distribution from the total development. A previous traffic impact analysis for the development directly south was conducted by KVE. This analysis showed that 34 trips total, 25 entering, 9 exiting would occur northbound through Dorothy Street. These values were accounted for in the total distribution traveling through the Fairview Crossing North development. By applying the proposed overall trip distribution to the generated mid-day and PM peak hour totals, following table showing assigned trips at the intersection of Fairview Drive and S. Commercial Street can be used.

Generated Trips Assigned to S. Commercial Street at Fairview Drive			
Development		**Mid-day Peak Hour of Generator (11 AM-1 PM)**	**PM Peak Hour of Adjacent (4-6 PM)**
Fairview Crossing North	**Total**	50	187
	Enter	26	90
	Exit	24	97
Fairview Crossing To/From Dorothy Street	**Total**	34	34
	Enter	25	25
	Exit	9	9
Aggregate	**Total**	84	221
	Enter	51	115
	Exit	33	106

These totals at the intersection were applied against a second distribution to assign directional traffic. It is assumed that shopping center pass-by traffic generated by the mixed-use development would have a similar distribution pattern to the existing traffic on S. Commercial Street.

Trip Distribution (Existing and Proposed)		
Direction (To/From)	Mid-Day	PM
Private Drive @ Phillips 66	6%	9%
East Via S. Commercial Street	48%	53%
West Via S. Commercial Street	46%	38%
Total	100%	100%

This analysis only focuses on impacts at this intersection. However, there are three drives to the west within 200’ of the proposed Fairview Drive intersection that would also be impacted. These drives provide access to a fast-food restaurant so a portion of trips entering from the west and exiting to the west could be assigned to those drives. Two of the four total drives on the north side of S. Commercial Street provide thru access to Shamrock Way and other businesses. One of which is the private drive at Phillips 66. Therefore, trip distribution may appear skewed as the private drive could be used by drivers as a thru route as well as for access to the gas station.

Refer to Figure 2 for the Existing Plus Proposed Peak Hour Traffic Volume and Distribution Diagram.

SIGHT DISTANCE

The guidelines for sight distance analysis are based upon the design guidelines for intersection sight distance for at-grade intersections published in the American Association of State Highway and Transportation Officials (AASHTO), <u>A Policy on Geometric Design of Highway and Streets,</u> *2011 Edition.* Refer to the following table for the recommended sight distances for driveways with stop control on the minor approach.

Intersection (Driveway) Sight Distance				
Access Location and Turning Movement		Design Speed	AASHTO Recommended Distance Passenger Car	AASHTO Recommended Distance Single Unit Truck/Bus
Driveway | S. Commercial Street (Two Lane Section)	Left (Case F) (One Lane)	35	285'	335'
Driveway | Fairview Drive (Two Lane Section)	Left (Case B1) (Two Lane)	15*	170'	210'
	Right (Case B2)	15*	145'	190'
	Cross Maneuver (Case B1)	15*	170'	210'

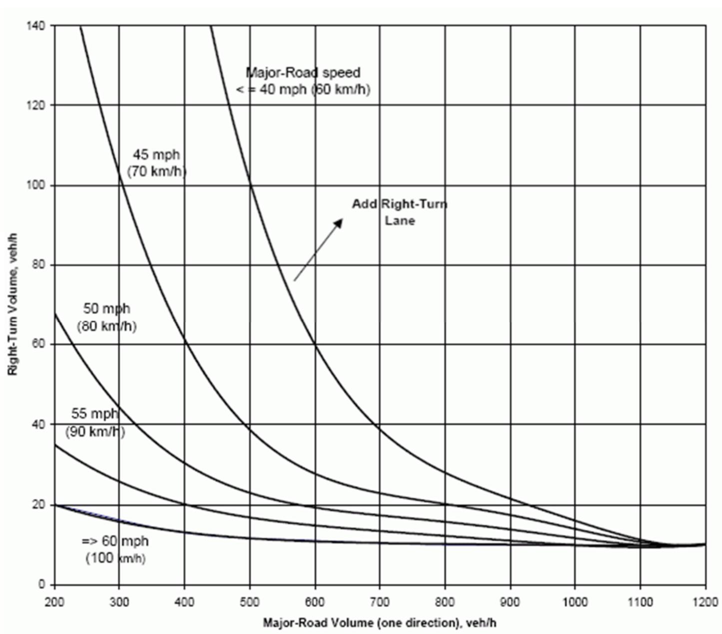
* Assumed Speed for Fairview Drive

The recommended intersection sight distance needs to be considered for all existing and proposed driveways. The S. Commercial Street profile along the property would be classified as level to and the horizonal alignment lends itself to good sight lines. The sight distance appears to exceed the sight distance criteria at the intersection at Fairview Drive. There is a crest in the roadway approximately 300’ east of the intersection which is beyond the requisite sight distance for a turning maneuver out of

Fairview Drive. The approach grade on Fairview Drive should be limited to 3% or less within 25' of the roadway. KCPI should be advised that care should be taken by the landscape architect/landscaper to not locate landscaping or signage within the sight triangles at each driveway as well.

AUXILIARY TURN LANE WARRANT ANALYSIS

Auxiliary turn lanes are recommended to improve safety as well as capacity at on-grade intersections. Several Transportation Research Board (TRB) reports include guidance on establishing a warrant analysis to determine if auxiliary turn lanes and tapers at driveways and intersections are needed based upon speed as well as opposing, advancing, and turning volumes on the primary roadway. The safety benefit associated with auxiliary turn lanes is realized by removing slow or stopped vehicles from through lanes. Auxiliary turn lanes were considered for the intersection of Fairview Drive at South Commercial Street due to the estimated trips that may be generated by the proposed development. A turn lane warrant analysis was completed for all potential right turn movements into the development from S. Commercial Street due to the volume of traffic traveling eastbound. The criterion utilized to evaluate the warrant for right turn lanes is based upon the following graph provided in the MoDOT Engineering Policy Guide Section 940.9.9. Advancing and Turning volumes presented on Figure 2, the Existing plus Proposed Peak Hour Traffic Volume and Distribution Diagram were used for this assessment.

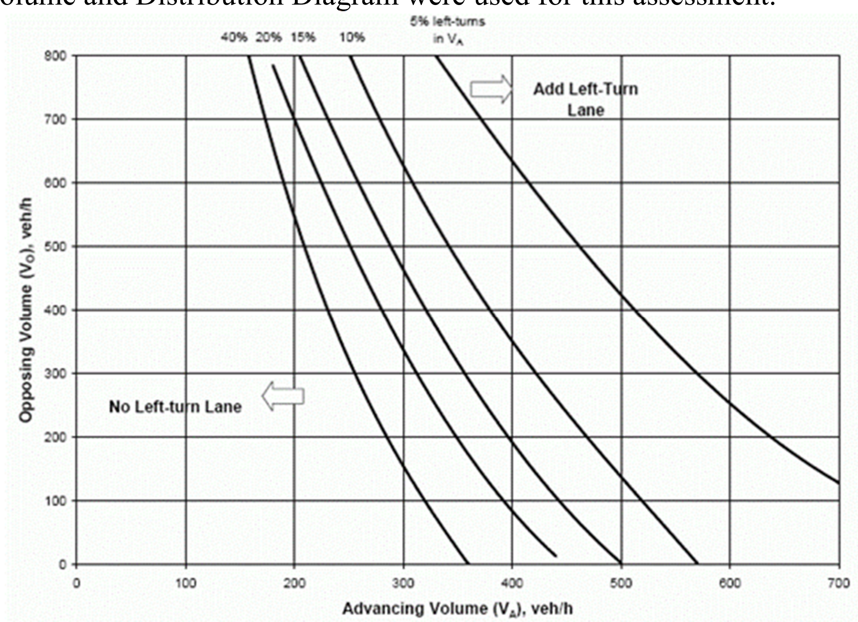


The following tables summarize the results of that analysis.

Right Turn Lane Warrant Analysis (MoDOT EPG 940.9.9)					
		Turning	Advancing	Speed	Turn Lane Warranted
EB South Commercial Street at Fairview Drive	Mid-Day Peak	24	81	35 mph	No
	PM Peak	44	205	35 mph	No
NB Fairview Drive at South Commercial Street	Mid-Day Peak	16	33	15 mph	No
	PM Peak	56	106	15 mph	No

As defined in the table above, auxiliary right turn lanes are not warranted at the eastbound approach at S. Commercial Street as well as the northbound approach at Fairview Drive. Due to good sightlines and the reduced speeds as US Highway 169 transitions from divided highway to urban arterial, an auxiliary right turn lane at this location was not recommended.

A westbound left turn lane and a northbound left turn lane at the intersection of S. Commercial Street and Shane's was considered. The criterion utilized to evaluate the warrant for left turn lanes is based upon the following graph provided in the MoDOT Engineering Policy Guide Section 940.9.6. Advancing, Opposing and Turning volumes presented on Figure 2, the Existing plus Proposed Peak Hour Traffic Volume and Distribution Diagram were used for this assessment.



The following tables summarize the results of that analysis.

Left Turn Lane Warrant Analysis (MoDOT EPG 940.9.1)							
		Turning	Advancing	Percent Left Turns	Opposing	Speed	Turn Lane Warranted
WB South Commercial Street at Fairview Drive	Mid-Day Peak	24	62	39%	76	<40 mph	No
	PM Peak	48	135	36%	173	<40 mph	
NB Fairview Drive at South Commercial Street	Mid-Day Peak	15	33	45%	3	<40 mph	No
	PM Peak	40	106	38%	10	<40 mph	

CAPACITY AND QUEUING ANALYSIS (CAQA)

The peak hour traffic volumes at the study intersection were analyzed to determine the operational characteristics during the peak hours of a typical weekday for the following scenarios:

- Existing Conditions
- Existing Plus Proposed Conditions

The Mid-Day and PM Peak hours analyzed corresponded to the peak hours of the adjacent street network.

The study intersections were analyzed using procedures and methodology outlined in the *Highway Capacity Manual 2010* (HCM) produced by the Transportation Research Board. The intersections were evaluated using Level-of-Service (LOS). An intersection operational analysis is typically quantified by the LOS experienced by drivers. LOS ranges from 'A' to 'F'. LOS 'A' represents free flow movement of traffic and minimal delay. LOS 'F' is usually an indication of congested conditions and excessive delay. Intermediate grades indicate incremental increases in congestion and delay. The LOS deemed acceptable varies by community and facility type. LOS of 'E' of 'F' are often acceptable for low or moderate traffic volumes such as private driveways or intersections where signalization warrants are not met or a traffic signal may be undesirable. The criteria for LOS as defined by delay for an unsignalized and signalized intersection is listed in the following table.

Level-of-Service: Delay Threshold		
Level-of-Service (LOS)	Unsignalized Intersection	Signalized Intersection
	Approach Delay (Sec/Veh)	Approach Delay (Sec/Veh)
A	0-<10	0-<10
B	10-<15	10-<20
C	15-<25	21-<35
D	25-<35	35-<55
E	35-<50	55-<80
F	>50	>80

A queuing analysis was also performed for the intersection of study. This type of analysis is used to determine if the proposed driveways are adequately spaced, if there are potential conflicts with the access points, and determine if dedicated turning lanes or additional length to turning lanes may be required. Results are presented as the 95th Percentile Queue. Moreover, the 95th Percentile Queue is a measurement of the expected length (queue) of traffic that would develop because of traffic control at an intersection. The 95th percentile was used to account for traffic fluctuations. The results of the analyses as well as recommendations for improvements to the study intersections are listed below for each of the scenarios evaluated.

CAQA: EXISTING CONDITIONS

The existing geometrics and peak hour traffic volumes as presented on Figure 1 were modeled to determine the baseline LOS and delay at the location of the proposed intersection. The results of the analysis are summarized in the following Table.

Summary of Results: Capacity & Queuing Analysis							
Existing Conditions:							
		Mid-Day Peak Hour			PM Peak Hour		
Intersection	Movement	LOS	Delay (sec/veh)	95% Queuing (veh)	LOS	Delay (sec/veh)	95% Queuing (veh)
S. Commercial Street at Private Drive @Phillips 66	Intersection	A	0.8	--	A	1.2	--
	Eastbound Left	A	7.4	0	A	1.4	0.1
	Southbound Left/Right	A	8.6	0	A	9.3	0

All approaches operate at an acceptable LOS during Peak Hours.

CAQA: EXISTING PLUS PROPOSED CONDITIONS

S. Commercial Street was modeled with the proposed drive and traffic volumes presented in Figure 2. All minor approaches were assumed to be stop controlled. The results of the analysis are summarized in the following Table.

Summary of Results: Capacity & Queuing Analysis							
Existing + Proposed Conditions:							
		Mid-Day Peak Hour			PM Peak Hour		
Intersection	Movement	LOS	Delay (sec/veh)	95% Queuing (veh)	LOS	Delay (sec/veh)	95% Queuing (veh)
S. Commercial Street at Private Drive @Phillips 66 and Fairview Drive	Intersection	A	3.2	--	A	6.5	--
	Northbound Thru/Left/Right	A	8.9	0.1	B	12.8	1.2
	Eastbound Left	A	7.3	0	A	7.5	0.1
	Westbound Left	A	7.4	0	A	8.2	0.4
	Southbound Thru/Left/Right	B	9.0	0.1	C	16.6	0.3

The proposed roadway intersecting S. Commercial Street and adjacent to the commercial drive should operate at an acceptable LOS with limited queueing. Any effect on the adjacent street network by the proposed development will be minimal. Traffic exiting the gas station will experience increased delay but existing plus proposed peak hour traffic at the approach is low enough to be assumed negligible.

CONCLUSION

As outlined above the following improvements are recommended to support the proposed development.

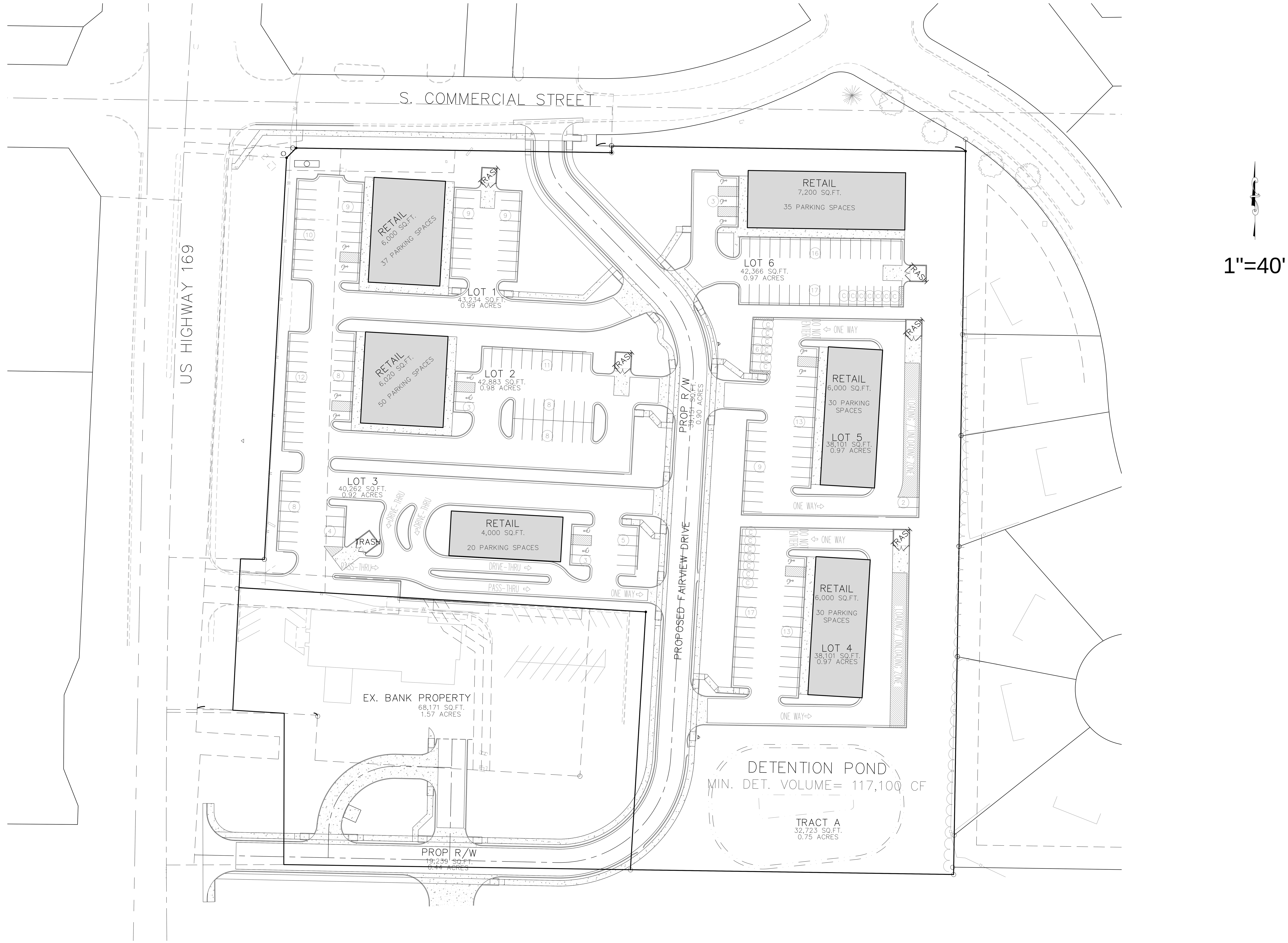
- Construct Fairview Drive as a two-lane roadway as shown on the proposed development plan. Horizontal and Vertical Alignments should comply with Kansas City Metropolitan Chapter of APWA Specifications, Section 5200. Adequate sight distance shall be provided at all proposed intersections and driveways. The design speed shall not exceed 15 mph.
- Pedestrian access to public right of way in compliance with ADA standards should be provided.
- All signage should be mounted in accordance with the Manual of Uniform Traffic Control Devices.

We appreciate the opportunity to be of service to you on this project. If you have any questions or require additional information, please contact me via email at aaronm@kveng.com or at (816) 468-5858.

Respectfully submitted,
Kaw Valley Engineering, Inc.

Aaron R. Moore, EIT
Staff Engineer

Matthew A. Cross, P.E.
Project Manager



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KAW VALLEY ENGINEERING

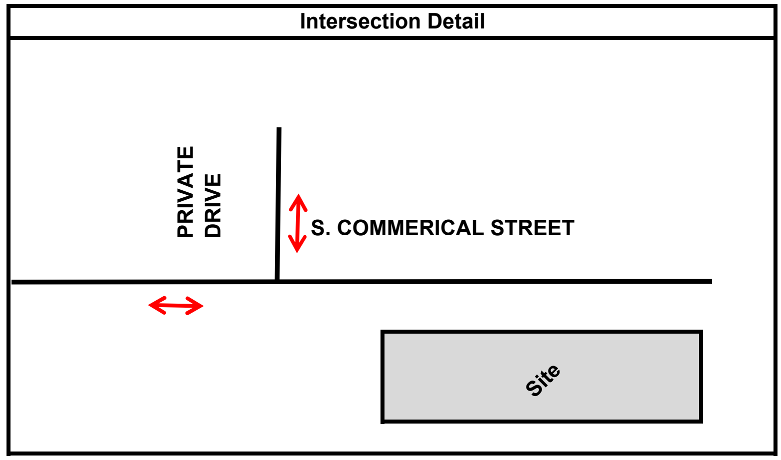
PROPOSED SITE PLAN
01/31/2023
B20D4001

Traffic Impact Study: Fairview Crossing North
Kaw Valley Engineering, Inc.

Project #: B20D4001

Volume and Distribution Diagram

Intersection: S. Commerical Street & Private Drive
Date: 01-18-23



Wednsday 01-18-2023

AM	Traffic Movement								
Time	**EBL**	**EBthru**	**SBL**	**SBR**	**WBthru**	**WBR**			**Total**
11:00	3	17	0	0	7	0			27
11:15	2	10	1	1	14	1			29
11:30	1	7	0	1	11	0			20
11:45	2	16	0	2	14	0			34
12:00	0	17	0	0	17	0			34
12:15	0	12	0	0	4	0			16
12:30	3	11	0	2	15	2			33
12:45	2	18	0	4	13	0			37
Peak Hour Counts	**5**	**58**	**0**	**6**	**49**	**2**			120
Peak Hour Factor	**0.42**	**0.81**	**#####**	**0.38**	**0.72**	**0.25**			**0.88**
Distribution	**8%**	**92%**	**0%**	**100%**	**96%**	**4%**			

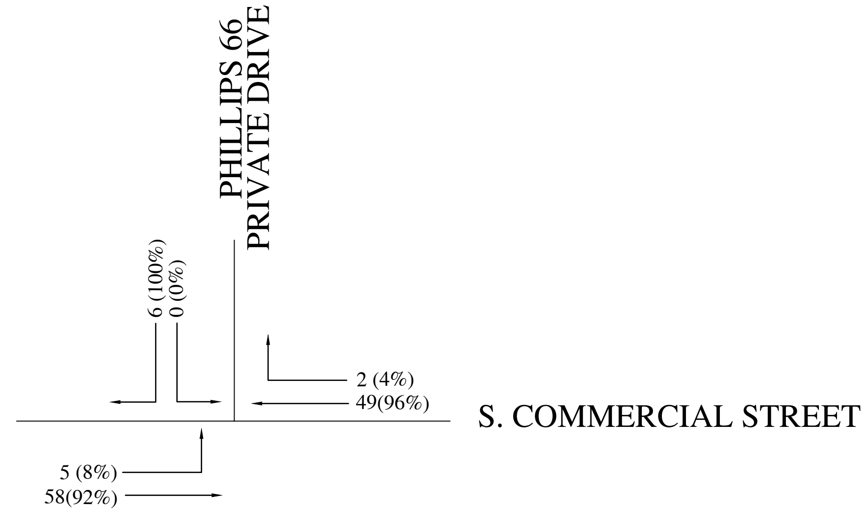
PM	Traffic Movement								
Time	**EBL**	**EBthru**	**SBL**	**SBR**	**WBthru**	**WBR**			**Total**
4:00	4	27	1	3	15	1			51
4:15	7	28	2	2	11	1			51
4:30	3	41	0	2	23	2			71
4:45	2	25	1	1	14	1			44
5:00	7	42	0	4	28	2			83
5:15	1	29	0	2	19	0			51
5:30	11	49	1	3	21	0			85
5:45	6	28	1	1	30	0			66
Peak Hour Counts	**25**	**148**	**2**	**10**	**98**	**2**			285
Peak Hour Factor	**0.57**	**0.76**	**0.50**	**0.63**	**0.82**	**0.25**			**0.86**
Distribution	**14%**	**86%**	**17%**	**83%**	**98%**	**2%**			

LEGEND

EXISTING PEAK HOUR VOLUME

245 (40%) DISTRIBUTION

WEEKDAY MID-DAY PEAK (11AM-1PM)



WEEKDAY PM PEAK(4-6PM)

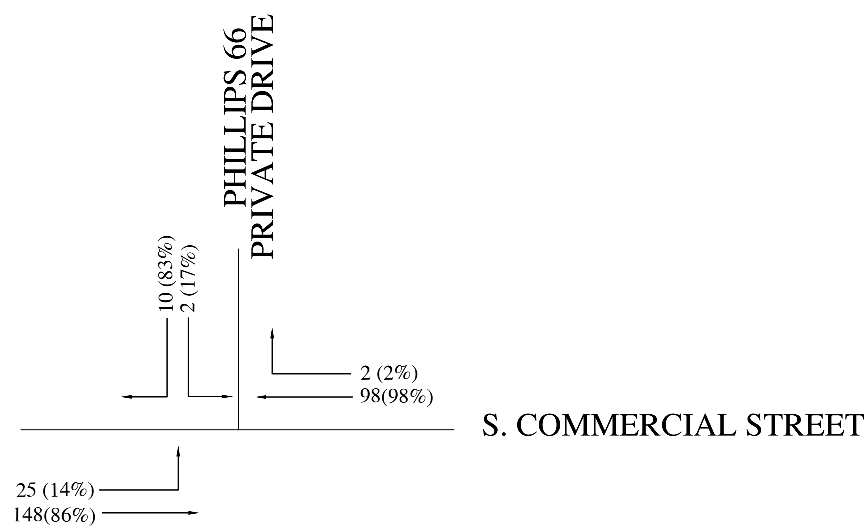


FIGURE "1"

PROJECT #: B20D4001

FAIRVIEW CROSSING NORTH AT S. COMMERCIAL STREET

SMITHVILLE, MISSOURI

EXISTING PEAK HOUR TRAFFIC VOLUME AND DISTRIBUTION DIAGRAM

SCALE: N.T.S.

FIGURE-1.dwg

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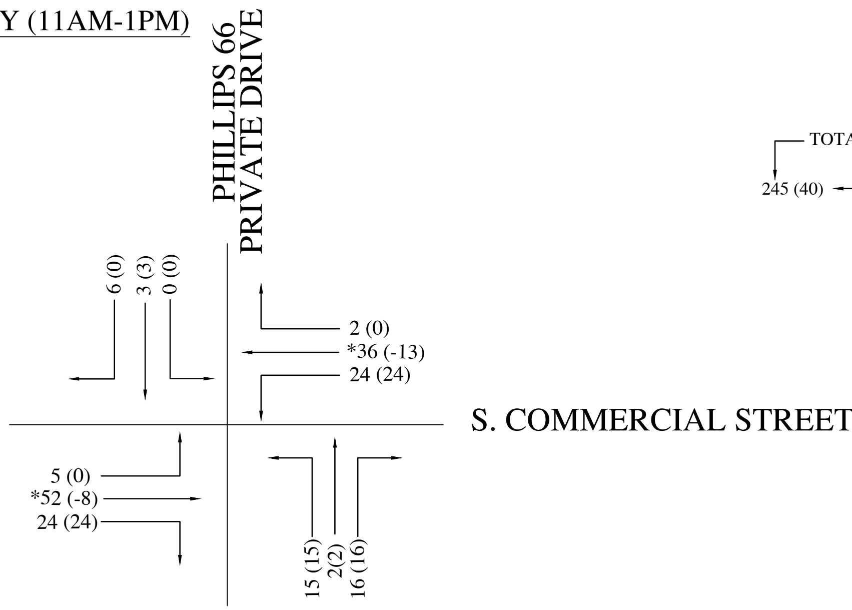
Generated Trips Assigned to S. Commercial Street at Fairview Drive			
Development		Mid-day Peak Hour of Generator (11 AM-1 PM)	PM Peak Hour of Adjacent (4-6 PM)
Fairview Crossing North	Total	50	187
	Enter	26	90
	Exit	24	97
Fairview Crossing To/From Dorothy Street	Total	34	34
	Enter	25	25
	Exit	9	9
Aggregate	Total	84	221
	Enter	51	115
	Exit	33	106

Trip Distribution (Existing and Proposed)		
Direction (To/From)	Mid-Day	PM
Private Drive @ Phillips 66	6%	9%
East Via S. Commercial Street	48%	53%
West Via S. Commercial Street	46%	38%
Total	100%	100%

WEEKDAY MID-DAY (11AM-1PM)

PROPOSED DEVELOPMENT
TOTAL: 84
ENTER: 51
EXIT: 33



LEGEND

245 (40) — TOTAL PROPOSED PEAK HOUR VOLUME; (40) — PROPOSED TO/FROM DEVELOPMENT

NOTES:
1.PROPOSED DISTRIBUTION ASSUMES 38% OF TRAFFIC GENERATED FROM THE ENTIRE DEVELOPMENT ENTERS/EXITS FAIRVIEW DRIVE,
2.*INCLUDES PASS-BY REDUCTION OF 27% FROM NEW TRIPS TURNING INTO THE PROPOSED DEVELOPMENT

WEEKDAY PM PEAK HOUR (4PM-6PM)

PROPOSED DEVELOPMENT
TOTAL: 221
ENTER: 115
EXIT: 106

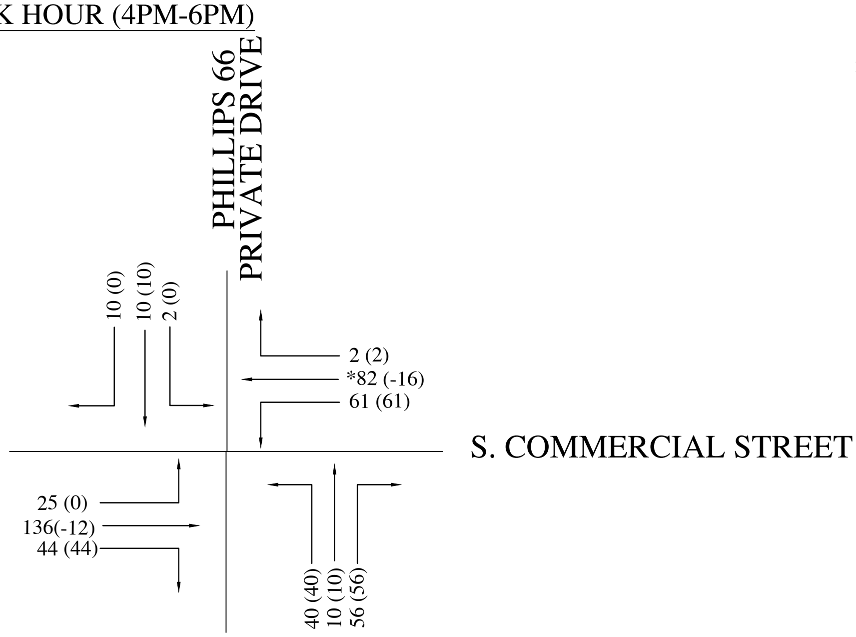


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FIGURE "2"

PROJECT #: B20D4001

FAIRVIEW CROSSING NORTH AT S. COMMERCIAL STREET

SMITHVILLE, MISSOURI

EXISTING PLUS PROPOSED PEAK HOUR TRAFFIC VOLUME AND DISTRIBUTION DIAGRAM

SCALE: N.T.S.

FIGURE-2.dwg

Intersection	
Int Delay, s/veh	0.8

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	5	58	49	2	0	6
Future Vol, veh/h	5	58	49	2	0	6
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	57	76	82	25	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	9	76	60	8	0	6

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	68	0	-	0	158	64
Stage 1	-	-	-	-	64	-
Stage 2	-	-	-	-	94	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1533	-	-	-	833	1000
Stage 1	-	-	-	-	959	-
Stage 2	-	-	-	-	930	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1533	-	-	-	828	1000
Mov Cap-2 Maneuver	-	-	-	-	828	-
Stage 1	-	-	-	-	953	-
Stage 2	-	-	-	-	930	-

Approach	EB	WB	SB
HCM Control Delay, s	0.8	0	8.6
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1533	-	-	-	1000
HCM Lane V/C Ratio	0.006	-	-	-	0.006
HCM Control Delay (s)	7.4	-	-	-	8.6
HCM Lane LOS	A	-	-	-	A
HCM 95th %tile Q(veh)	0	-	-	-	0

Intersection	
Int Delay, s/veh	1.2

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations		↑	↑		↰	
Traffic Vol, veh/h	25	148	98	2	2	10
Future Vol, veh/h	25	148	98	2	2	10
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	57	76	82	25	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	44	195	120	8	2	10

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	128	0	-	0	407	124
Stage 1	-	-	-	-	124	-
Stage 2	-	-	-	-	283	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1458	-	-	-	600	927
Stage 1	-	-	-	-	902	-
Stage 2	-	-	-	-	765	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1458	-	-	-	580	927
Mov Cap-2 Maneuver	-	-	-	-	580	-
Stage 1	-	-	-	-	871	-
Stage 2	-	-	-	-	765	-

Approach	EB	WB	SB
HCM Control Delay, s	1.4	0	9.3
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1458	-	-	-	843
HCM Lane V/C Ratio	0.03	-	-	-	0.014
HCM Control Delay (s)	7.5	-	-	-	9.3
HCM Lane LOS	A	-	-	-	A
HCM 95th %tile Q(veh)	0.1	-	-	-	0

Intersection	
Int Delay, s/veh	3.2

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Vol, veh/h	5	52	24	24	36	2	15	2	16	0	3	6
Future Vol, veh/h	5	52	24	24	36	2	15	2	16	0	3	6
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Free	Free	Stop	Stop	Stop	Stop	Stop	Stop
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	-	-	-	-	-	-	-	-	-	-	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	42	81	88	88	72	25	88	88	88	88	88	38
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	12	64	27	27	50	8	17	2	18	0	3	16

Major/Minor	Major1			Major2			Minor1			Minor2		
Conflicting Flow All	58	0	0	91	0	0	220	214	78	-	223	54
Stage 1	-	-	-	-	-	-	102	102	-	-	108	-
Stage 2	-	-	-	-	-	-	118	112	-	-	115	-
Critical Hdwy	4.12	-	-	4.12	-	-	7.12	6.52	6.22	-	6.52	6.22
Critical Hdwy Stg 1	-	-	-	-	-	-	6.12	5.52	-	-	5.52	-
Critical Hdwy Stg 2	-	-	-	-	-	-	6.12	5.52	-	-	5.52	-
Follow-up Hdwy	2.218	-	-	2.218	-	-	3.518	4.018	3.318	-	4.018	3.318
Pot Cap-1 Maneuver	1546	-	-	1504	-	-	736	684	983	0	676	1013
Stage 1	-	-	-	-	-	-	904	811	-	0	806	-
Stage 2	-	-	-	-	-	-	887	803	-	0	800	-
Platoon blocked, %		-	-		-	-						
Mov Cap-1 Maneuver	1546	-	-	1504	-	-	707	666	983	-	658	1013
Mov Cap-2 Maneuver	-	-	-	-	-	-	707	666	-	-	658	-
Stage 1	-	-	-	-	-	-	897	805	-	-	791	-
Stage 2	-	-	-	-	-	-	853	788	-	-	794	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	0.8	2.4	8.9	9
HCM LOS			A	A

Minor Lane/Major Mvmt	NBLn1	EBL	EBT	EBR	WBL	WBT	WBR	SBLn1
Capacity (veh/h)	934	1546	-	-	1504	-	-	924
HCM Lane V/C Ratio	0.022	0.008	-	-	0.018	-	-	0.021
HCM Control Delay (s)	8.9	7.3	-	-	7.4	-	-	9
HCM Lane LOS	A	A	-	-	A	-	-	A
HCM 95th %tile Q(veh)	0.1	0	-	-	0.1	-	-	0.1

Intersection	
Int Delay, s/veh	6.5

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Vol, veh/h	25	136	44	61	82	2	40	10	56	2	10	10
Future Vol, veh/h	25	130	103	140	78	2	98	22	136	2	22	10
Conflicting Peds, #/hr	0	0	0	0	0	0	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Free	Free	Stop	Stop	Stop	Stop	Stop	Stop
RT Channelized	-	-	None	-	-	None	-	-	None	-	-	None
Storage Length	-	-	-	-	-	-	-	-	-	-	-	-
Veh in Median Storage, #	-	0	-	-	0	-	-	0	-	-	0	-
Grade, %	-	0	-	-	0	-	-	0	-	-	0	-
Peak Hour Factor	57	76	86	86	82	25	86	86	86	86	92	86
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	44	171	120	163	95	8	114	26	158	2	24	12

Major/Minor	Major1			Major2			Minor1			Minor2		
Conflicting Flow All	103	0	0	291	0	0	762	748	231	836	804	99
Stage 1	-	-	-	-	-	-	319	319	-	425	425	-
Stage 2	-	-	-	-	-	-	443	429	-	411	379	-
Critical Hdwy	4.12	-	-	4.12	-	-	7.12	6.52	6.22	7.12	6.52	6.22
Critical Hdwy Stg 1	-	-	-	-	-	-	6.12	5.52	-	6.12	5.52	-
Critical Hdwy Stg 2	-	-	-	-	-	-	6.12	5.52	-	6.12	5.52	-
Follow-up Hdwy	2.218	-	-	2.218	-	-	3.518	4.018	3.318	3.518	4.018	3.318
Pot Cap-1 Maneuver	1489	-	-	1271	-	-	322	341	808	287	316	957
Stage 1	-	-	-	-	-	-	693	653	-	607	586	-
Stage 2	-	-	-	-	-	-	594	584	-	618	615	-
Platoon blocked, %		-	-		-	-						
Mov Cap-1 Maneuver	1489	-	-	1271	-	-	259	284	808	188	263	957
Mov Cap-2 Maneuver	-	-	-	-	-	-	259	284	-	188	263	-
Stage 1	-	-	-	-	-	-	668	629	-	585	506	-
Stage 2	-	-	-	-	-	-	483	505	-	460	593	-

Approach	EB	WB	NB	SB
HCM Control Delay, s	1	5	12.8	16.6
HCM LOS			B	C

Minor Lane/Major Mvmt	NBLn1	EBL	EBT	EBR	WBL	WBT	WBR	SBLn1
Capacity (veh/h)	643	1489	-	-	1271	-	-	345
HCM Lane V/C Ratio	0.286	0.029	-	-	0.128	-	-	0.103
HCM Control Delay (s)	12.8	7.5	-	-	8.2	-	-	16.6
HCM Lane LOS	B	A	-	-	A	-	-	C
HCM 95th %tile Q(veh)	1.2	0.1	-	-	0.4	-	-	0.3