GREENBAG RD AND DORSEY AVE INTERSECTION STUDY

APRIL, 2014

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1.0 Study Purpose

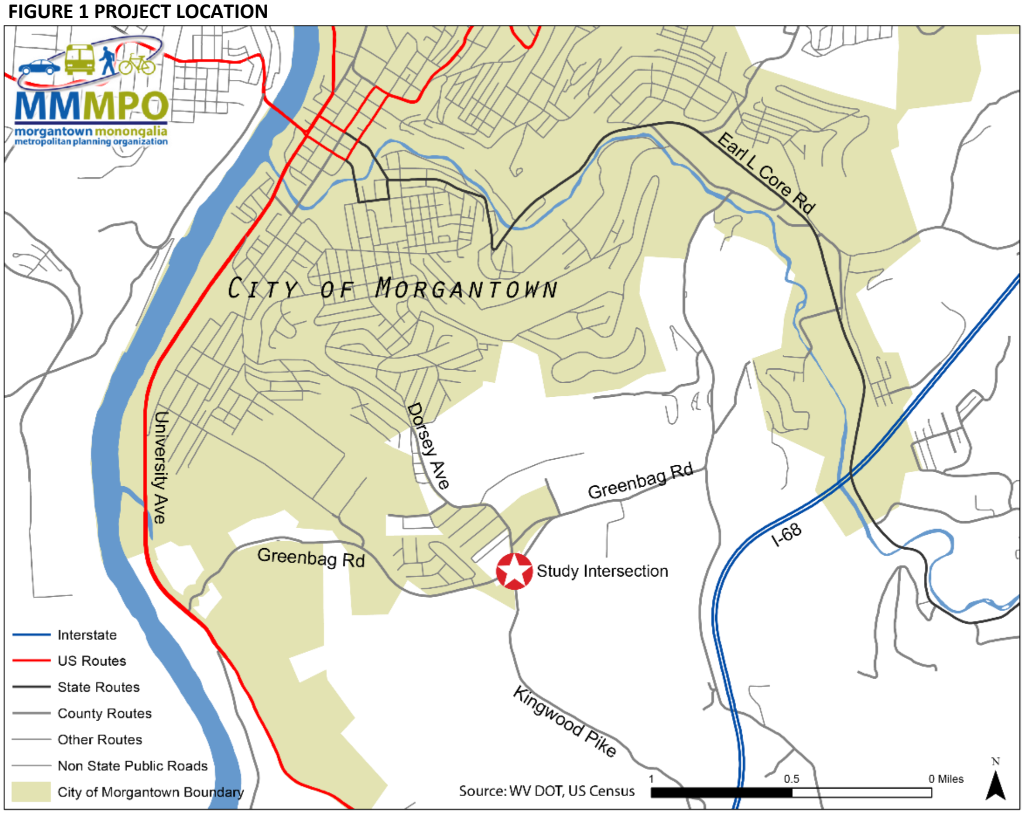
The purpose of this study is to understand the existing condition of Greenbag Rd and Dorsey Ave/Kingwood Pike intersection and to make preliminary recommendations to improve its safety and operational performance. It is also intended to establish a standard analytical procedure for similar studies to be conducted by MPO staff.

This study was prepared by MPO staff as a project of the MMMPO Unified Planning Work Program FY 2013-2014. This intersection was identified as a priority safety improvement location in the 2040 MMMPO Long Range Transportation Plan (LRTP), and was included in the Greenbag Road Improvements Project, a tier 1 project recommended in the LRTP.

2.0 Description of Location

The subject intersection is located at CR 875 (Greenbag Rd) and CR 81 (Dorsey Ave/Kingwood Pike) in Monongalia County, WV. Greenbag Rd is a minor arterial, two-lane facility that provides an important south-west connection in the southern Morgantown area linking US 119 (University Ave) and WV 7 (Earl L Core Road). Dorsey Ave/Kingwood Pike is a minor arterial, two-lane facility through the First Ward area and into the southeast end of the county. The project location is shown in Figure 1.

FIGURE 1 PROJECT LOCATION



3.0 EXISTING CONDITIONS

This section provides information that characterize the subject intersection. It includes review of geometric conditions, crash history, traffic operational status and other intersection related features. These key components form the basis for assessing the current physical and operating conditions of the subject intersection.

3.1 GEOMETRY & TOPOGRAPHY

The dimensions and topography include lane width, street width, turning radius, intersection length, and locations of utility/signal light poles, which are shown in Figure 2 and summarized in Table 1.

Geometric data was collected by using the Pictometry Aerial Map and Pictometry distance-measuring tool embedded in the ESRI Arch Map; topographic data came from 2 feet contour mappings from the Monongalia County GIS Database. Although every effort has been made to provide accurate geometric information for the intersection, the accuracy of the data used in this analysis cannot be guaranteed.

TABLE 1 INTERSECTION DIMENSION SUMMARY

	Southwest Leg	Northeast Leg	North Leg	South Leg
Approching lane width	12’5”	9’2”	9’10”	9’5”
Pavement width (edge to edge)	25’	21’9”	20’10”	20’9”
Right turn radius	41’10”	22’7”	56’5”	58’9”
Through movement travel length	147’2”	147’2”	168’4”	168’4”
Stop line offset	72’3”	43’10”	75’10”	70’7”

Following characteristics of the intersection have been identified:

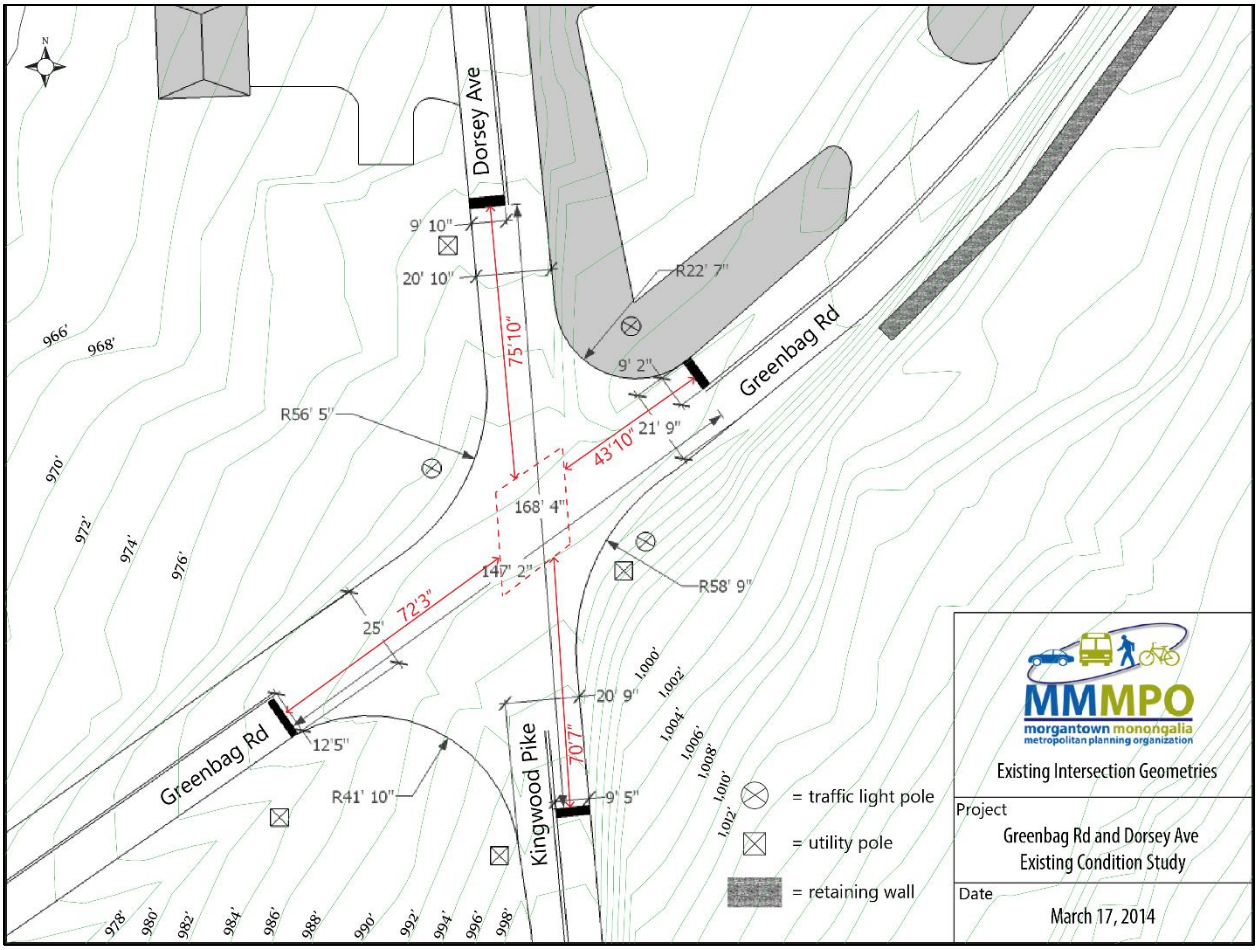
- **The Roadways intersect at skewed angels, which decrease the intersection’s safety and efficiency.** The skewed angle of this intersection results in 1) Longer traveling distance for vehicles entering the intersection, and therefore an increased time of exposure to the cross-street traffic; 2) limited vision for entering vehicles to observe opposing and crossing traffic; 3) the difficulty of aligning vehicles entering the cross street to make a right or left turn; and 4) acute-angle radius requiring large vehicles to encroach beyond their intended right of way to accomplish a turning movement.

- **The slopes of the intersection reduces its safety and efficiency.** The slope of the road increases when going southbound from Dorsey Ave to Kingwood Pike and when going northeast bound on Greenbag Rd. This obscures vision for vehicles entering the intersection, requiring a longer start up lost time for southbound and northeast bound vehicles.

- **Narrow lane width reduces intersection’s capacity.** The width of three of the approaching lanes at the intersection are under 10 feet, except for the southwest leg. This reduces the maximum rate at which vehicles can pass through the intersection under prevailing conditions.

- **Small turning radiuses make some turning movements difficult for large vehicles.** The small inside turning radii and design turning radii resulting from the skewed angle and slopes at this intersection make it difficult for large vehicles making turning movements. Large vehicles observed at the intersection during the field traffic count include Mountain Line Transit Authority buses, large school bus (S-BUS-12), interstate semitrailer (WB-20), and intermediate semitrailer (WB-12). Those vehicles have wider and longer wheelbases, which result in them requiring greater minimum turning radii than do passenger vehicles. The southwest bound right turn, southbound left turn, northbound left turn, and northeast bound right turn are all difficulte movements.

FIGURE 2 GEOMETRIC DIMENSIONS AND TOPOGRAPHY



3.2 Lane Designation and Signal Timing

The intersecting roads are both two-lanes with each approaching lane designated as a shared left-turn, through, and right-turn lane. Left turns operate under permissive only mode, which requires left-turning drivers to yield to a conflicting vehicle before completing the turn, that is, a green arrow for left turn traffic is never provided. The efficiency of this mode is dependent on the availability of gaps in the conflicting streams through which the turn can be safely completed[1].

The signal timing of the intersection is uncoordinated and is operated by pretimed control in which the cycle length, phase plan, and phase times are preset to repeat continuously. The southwest bound and northeast bound movements are allocated with 45 seconds green time and the southbound and northbound movements 35 seconds.

[1] Federal Highway administration, *Traffic Signal Timing Manual*, 2008, P 4-8

Figure 3 illustrates lane designation, speed limits and timing phrase. Figure 4 and 5 show the current signal timing plan. Data are summarized in Table 2. The intersection's operational status is discussed in 3.10 Traffic Operation.

FIGURE 3 INTERSECTION AERIAL PHOTOGRAPH

FIGURE 4 RING-AND-BARRIER DIAGRAM

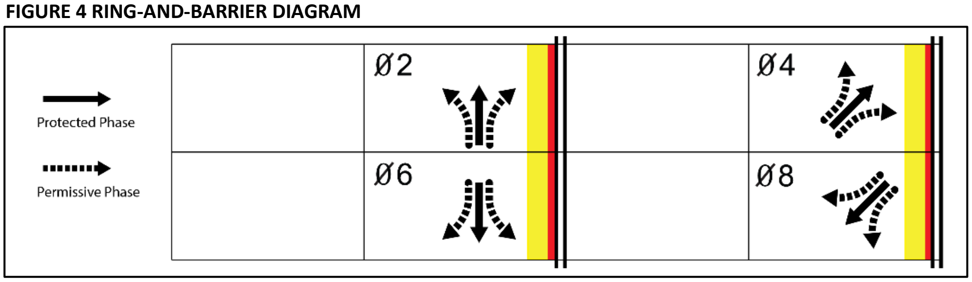


FIGURE 5 SPLITS AND PHASES DIAGRAM

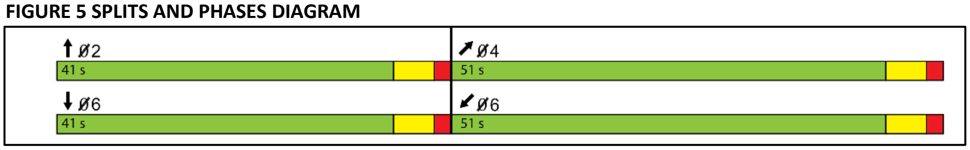


TABLE 2 SIGNAL TIMING SUMMARY

	Approaching Speed Limits	Phase Number	Effective Green Time	Yellow Time	All Red Time	Left-turn Mode
Northbound	45 MPH	2	35 s	4 s	2 s	Permissive
Southbound	35 MPH	6	35 s	4 s	2 s	Permissive
Northeast Bound	40 MPH	4	45 s	4 s	2 s	Permissive
Southwest Bound	40 MPH	8	45 s	4 s	2 s	Permissive

3.5 Land Use Pattern

The intersection is located outside of the City of Morgantown boundary, but it is included in the city's comprehensive plan for extended study and land management purposes. Currently, there is a one-floor business building located on the northeast corner of the intersection.

3.6 Pedestrian and Bicyclist Safety Assessment

This study uses both field assessment and the Pedestrian and Bicyclist Intersection Safety Indices (Ped ISI and Bike ISI) to assess the pedestrian and bicyclist safety at the subject intersection. The field assessment was conducted during the traffic count period and following characteristics was observed.

- **No pedestrians or bicyclist were observed during the field study period.** The lack of pedestrian and bicyclist traffic at this intersection may be largely because of the existing built environment of the surrounding area. It may be also because of the existence of alternative routes that divert any possible pedestrian and bicyclist traffic traveling in this area. Those routes include Luckey Ln and Richard Ave.
- **Some facilities beneficial to pedestrians and bicyclists are not provided.** Such facilities include, but not limited to, curb, wide shoulder, sidewalk pedestrian or bicyclist signal phase, crosswalk, and appropriate bicycle signage.
- **Uneven road surface constitute a hazard for bicyclists crossing this intersection.** Uneven surface exists in the center of the intersection, poses a potential hazard for bicyclists, especially when they travel through this intersection without any appropriate awareness or warning.

The Indices assign the subject intersection with values between 1 (safest) and 6 (least safe), which are produced by using method provided by Pedestrian and Bicyclist Intersection Safety Indices: User Guide[1]. Each leg of an intersection may have different characteristics affecting pedestrian or bicyclist safety; therefore the tools are intended to provide an evaluation of the safety of an individual crossing (Ped ISI) or approach leg (Bike ISI) rather than evaluating the intersection as a whole.

The Ped ISI and Bike ISI for the subject intersection are shown in Table 3 and Table 4[2].

TABLE 3 PEDESTRIAN SAFETY INDEX

Intersection Leg	Ped ISI Value
North	1.847
South	2.015
Northeast	1.955
Southwest	1.979

TABLE 4 BICYCLIST SAFETY INDEX

Intersection Leg	Bike ISI Value
North	TH 3.413
	RT 1.484
	LT 2.495
Northeast	TH 3.389
	RT 1.646
	LT 2.645
South	TH 3.413
	RT 1.484
	LT 2.495
Southwest	TH 3.389
	RT 1.646
	LT 2.645

[1] Detailed description of the method is provided in Appendix B Technical Support Document.
[2] ADT volume is estimated by using the data from nearby count stations in the 2013 MMMPO Traffic Count Report.

3.8 Transit Level of Service

This study uses the fixed-route transit service measures[1] to access the transit level of service for the subject intersection. The intersection has one bus stop served by Orange Line Route 4 and Mountain Heights Route 14, which are operated by the Mountain Line Transit Authority.

Service frequency determines how many times an hour a user has access to the transit mode, assuming that transit service is provided within acceptable walking distance and at the times the user wishes to travel. Service frequency LOS is determined by destination from a given transit stop. The service frequency LOS to major destinations from the stop at this intersection is summarized in Table 5.

TABLE 5 TRANSIT SERVICE FREQUENCY LOS

Destination from the intersection	Ave. Headway (min)	LOS
Morgantown Downtown/Bus Depot	31-60	E
Westover Terminal	31-60	E
WVU Evansdale Campus	31-60	E
Mountaineer Mall	>60	F

LOS E is service once per hour, which corresponds to the minimum service frequency applied when determining hours of service LOS. LOS F suggests service at frequencies greater than 1 hour, entailing highly creative planning or considerable wasted time on the part of passengers.

The passenger load LOS reflect the comfort level of the on-board vehicle portion of a transit trip. Based on field observation, the passenger load LOS for the stop at the intersection is at LOS A, suggesting passengers are able to spread out and can use empty seats to store parcels and bags.

3.9 Crash Analysis

This part provides an analysis of the characteristics of crashes that occurred at the subject intersection. Crash data used in this crash analysis are from the WV DOT Crash Database between 2009 and 2011, which was geocoded by MPO staff in 2013. Findings are summarized in Table 6 and 7.

TABLE 6 CRASH FREQUENCY AND RATE

Year	Crash Frequency	Number of vehicle involved	Injury[2]	Crash Rate[3]	State Average[4]
2009	3	5	3	521	380 (non- intersection)
2010	3	2	0		
2011	2	4	0		

TABLE 7 CRASH COLLISION TYPE

Collision Type	Frequency
Rear End	4
Single Vehicle Crash	2
Right Angle	2

[1] As provided by Transit Capacity and Quality of service Manual, Transportation Research Board
[2] No fatal crash reported during the study time period.
[3] Per hundred million entering vehicles. The ADT volume is estimated by compiling the peak hour ratio from the nearby count stations of the 2013 traffic report and the data from the field count. The crash rate should not be interpreted rigidly.
[4] Based on the state average crash rate for county route; - 2003 West Virginia Crash Data

3.10 TRAFFIC OPERATION

This section assesses the current operational status of the subject intersection by using three types of methods, which are the Quick Estimation Method (QEM), the Automobile Method, and the Intersection Capacity Utilization (ICU) Method. The characteristics of each method are summarized in Table 8.

TABLE 8 TRAFFIC OPERATION ANALYSIS METHOD DESCRIPTION

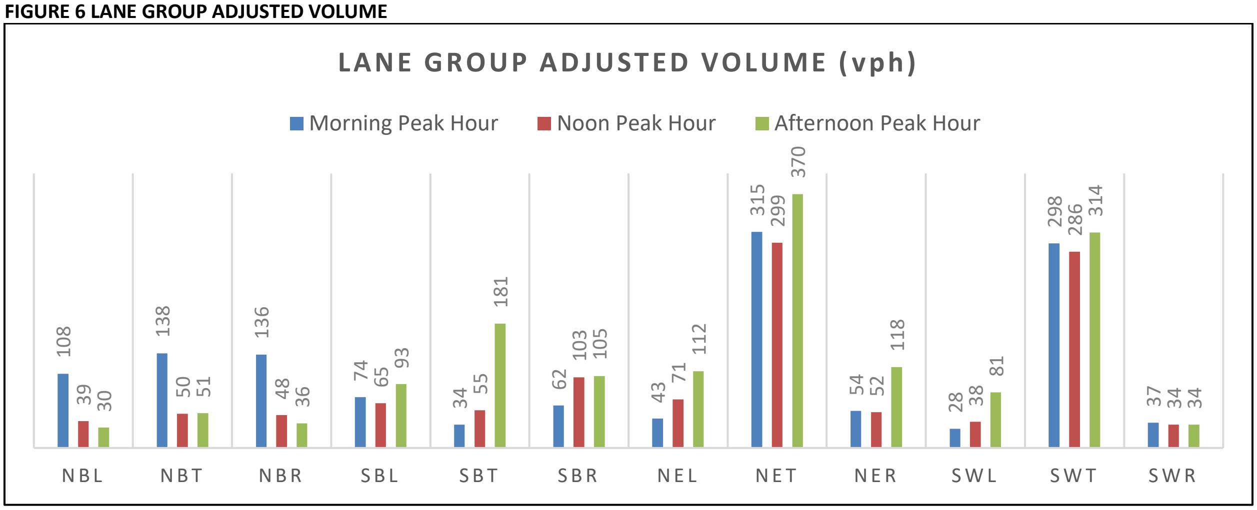
Method	Source	Application	LOS Focus	Process
Quick Estimation Method[1]	Highway Capacity Manual 2010	-- Preliminary left-turn treatment analysis -- Lane Group Adjusted Volume -- Control Delay -- Volume-to-Capacity Ratio -- Level of Service (LOS)	Delay	Manual
Automobile Method	Highway Capacity Manual 2010		Delay	Synchro
Intersection Capacity Utilization Method	Trafficware©		Capacity	Synchro

Traffic count data used in this analysis came from field counts conducted by MPO staff. The traffic count report presenting the raw traffic counts is provided in Appendix A. The complete result and detailed calculations for this analysis are discussed in Appendix B Technical Support Document.

The adjusted volume for each lane group (Table 9 and Figure 7) and the key indices of operational status of the subject intersection (Table 10, 11 and 12) are presented. The lane group adjusted volume is used to estimate the demand volume under prevailing condition. As the subject intersection involves shared left-turn operations, the effect of permissive left-turn is considered in the computation process of the adjusted volume. The indices include Volume-to-Capacity Ratio, Control Delay, and Level of Service.

No protected left turn is recommended for any approach of the subject intersection based on the preliminary left-turn treatment analysis by the QEM[2].

FIGURE 6 LANE GROUP ADJUSTED VOLUME



[1] The QEM is a simplified method for evaluating the performance of a signalized intersection at planning level. Comparing this with the operational level of analyses, only approximate results are desired from this method.
[2] The Left-turn treatment check provided in the QEM should not be used as the sole basis for determining the need for a left-turn phase.

TABLE 9 ADJUSTED FLOW RATE

Lane Group	NBL	NBT	NBR	SBL	SBT	SBR	NEL	NET	NER	SWL	SWT	SWR
Morning Peak Hour	108	138	136	74	34	62	43	315	54	28	298	37
Noon Peak Hour	39	50	48	65	55	103	71	299	52	38	286	34
Afternoon Peak Hour	30	51	36	93	181	105	112	370	118	81	314	34

TABLE 10 TRAFFIC OPERATION ANALYSIS SUMMARY BY QUICK ESTIMATION METHOD

Lane	NB			SB			NEB			SWB		
Criteria	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS
Morning Peak Hour	0.54	26.7	C	0.36	22.8	C	0.60	21.5	C	0.53	19.1	B
Noon Peak Hour	0.24	20.6	C	0.42	23.8	C	0.67	23.4	C	0.53	19.3	B
Afternoon Peak Hour	0.18	19.8	B	0.69	32.1	D	1.00	92	F	0.66	22.6	C

TABLE 11 TRAFFIC OPERATION ANALYSIS SUMMARY BY AUTOMOBILE METHOD

Lane	NB			SB			NEB			SWB		
Criteria	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS
Morning Peak Hour	0.72	33.9	C	0.43	25.0	C	0.56	19.8	B	0.50	18.5	B
Noon Peak Hour	0.26	20.8	C	0.41	23.4	C	0.60	21.0	C	0.50	18.5	B
Afternoon Peak Hour	0.25	20.6	C	0.68	31.0	C	1.12	101.4	F	0.85	36.6	D

TABLE 12 TRAFFIC OPERATION ANALYSIS SUMMARY BY INTERSECTION CAPACITY UTILIZATION METHOD

Lane	NB			SB			NEB			SWB		
Criteria	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS
Morning Peak Hour	0.85	43.0	D	0.45	21.3	C	0.61	21.3	C	0.57	20.7	C
Noon Peak Hour	0.30	16.9	B	0.48	20.3	C	0.66	23.3	C	0.58	21.1	C
Afternoon Peak Hour	0.26	16.6	B	0.82	40.7	D	1.00	62.1	E	0.81	33.9	C

The following concerns were identified, based on the traffic operation analyses and/or field observations.

- **Excessive delay on the northeast bound approach during the afternoon peak hour, as evidenced by its LOS F in two analysis methods.** This was likely caused by the effect of left-turning vehicles waiting to turn through a gap in the opposing traffic stream and blocking the through/right turn vehicles behind them, as Figure 7 shows that during the afternoon peak hour, there were 112 vehicles attempting to turn left on the northeast bound approach and 314 vehicle for through movement in the opposing lane. Observed delay ranged from 62.1 seconds to 101.4 seconds based the analysis.

- **High volume and low LOS occurred during the afternoon peak hour, except for the north bound approach, as evidenced by the LOS ranging from C to F during that time.** This was likely caused by the increased demand left-turns on both northeast bound and southwest bound approaches and by the relatively shorter green time allocated to the south bound traffic. It should be noted that The peak hour factor for the southbound traffic during the afternoon peak hour is 0.84, suggesting a high degree of traffic demand fluctuation in that hour, that is, a potentially higher degree of congestion for a peak 15-minutes flow rate in that hour.

- **Long northbound approach delay during the morning peak hour, as evidenced by its LOS C/D during that time.** This was likely caused by the high demand at that approach in that hour and the relatively short green time allocated to the north bound traffic at that time. Observed delay ranged from 26.7 seconds to 43.0 seconds, and occasionally one green cycle failed to clear the queue on that approach.

4.0 Future Conditions

The section provides information on the forecasted operational status of the intersection in 2020 and 2040. An average annual growth rate of 1.03%[1] has been used, and other variables, such as lane capacity, peak hour factor, and signal timing plan, remain unchanged.

The method of Intersection Capacity Utilization is used for this analysis. Forecasting details are provided in Appendix B Technical Support Document. Table 13 and 14 show the forecasted results in 2020 and 2040.

It can be found that

- In 2020, the northbound approach will operate at LOS D in the morning peak hour, and in 2040, it will operate at LOS F.
- In 2020, three approaches of the intersection will operate from LOS D to LOS F, and in 2040, they will all operate at LOS F.

TABLE 13 2020 INTERSECTION OPERATIONAL STATUS PROJECTION

Lane	NB			SB			NEB			SWB		
Criteria	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS
Morning Peak Hour	0.91	51.2	D	0.50	22.9	C	0.65	22.7	C	0.61	21.8	C
Noon Peak Hour	0.31	16.3	B	0.51	20.7	C	0.78	29.7	C	0.62	22.3	C
Afternoon Peak Hour	0.25	15.1	B	0.85	43.7	D	1.29	168.0	F	1.01	70.0	E

TABLE 14 2040 INTERSECTION OPERATIONAL STATUS PROJECTION

Lane	NB			SB			NEB			SWB		
Criteria	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS	V/C Ratio	Delay (s)	LOS
Morning Peak Hour	1.14	116.6	F	0.68	32.4	C	0.81	31.1	C	0.76	28.4	C
Noon Peak Hour	0.39	18.9	B	0.63	25.6	C	1.00	61.7	E	0.78	30.5	C
Afternoon Peak Hour	0.33	17.4	B	1.05	84.6	F	1.69	338.5	F	1.40	216.6	F

[1] The average annual growth rate is provided by the WVDOH District Office.

5.0 Alternatives Study

Four alternatives improve the safety and operational performance of the intersection were analyzed. These alternatives are summarized in Table 15 and illustrated through Figure 7 to Figure 10.

TABLE 15 DESCRIPTION OF ALTERNATIVES

	Geometry	Signal Timing
Alternative I	Add exclusive left turn lane on northeast bound and southwest bound approaches.	Add protected-permissive left turn phase for two exclusive left turn lanes and optimize cycle length.
Alternative II-a	Add exclusive turn lane on northeast bound approach.	Add protected-permissive left turn phase for one exclusive left turn lane and optimize cycle length.
Alternative II-b	Add exclusive turn lane on northeast bound approach.	Keep current signal timing pattern and optimize existing cycle length.
Alternative III	Unchanged.	Keep current signal timing pattern and optimize existing cycle length.

FIGURE 7 ALTERNATIVE I

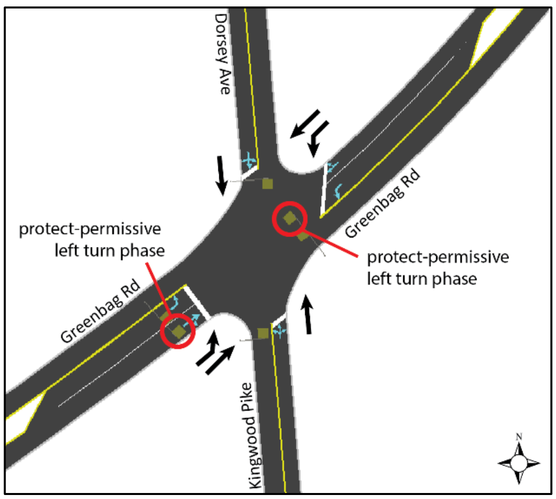


FIGURE 8 ALTERNATIVE II-A

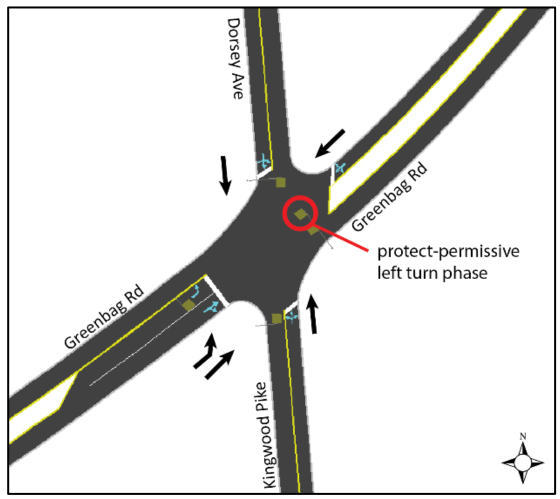


FIGURE 9 ALTERNATIVE II-B

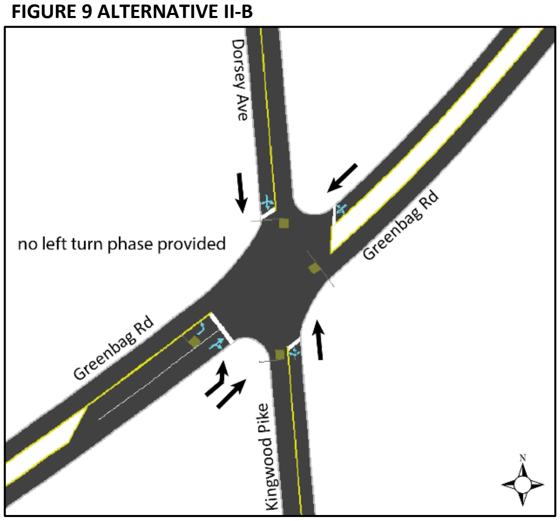
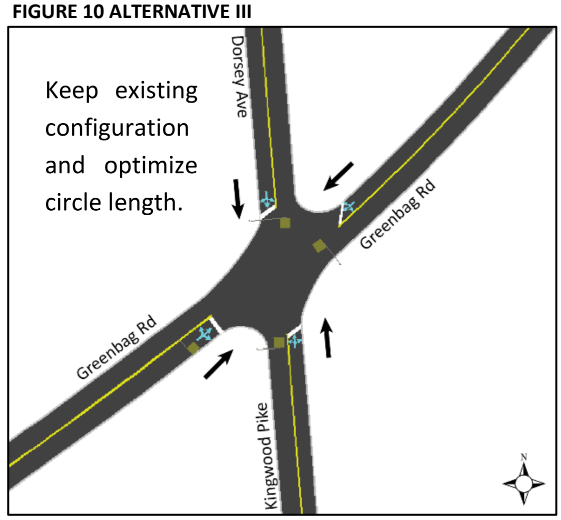


FIGURE 10 ALTERNATIVE III



5.1 Safety Analysis

Safety implications of proposed alternatives for motor vehicles, pedestrians, and bicycles were studied based on methods provided by the FHWA, which include the Interactive Highway Safety Design Model (IHSDM), the Crash Modification Factors Clearinghouse (CMF), and the Pedestrian/Bicycle Intersection Safety Indices (ISI). Existing traffic volume and pattern are used in the evaluation process.

With respect to motor vehicle safety, both the IHSDM and the CMF indicate that the Alternative I is the safest among all alternatives, followed by the Alternative II-a. Table X summaries the IHSDM and the CMF analysis.

TABLE 16 ALTERNATIVE SAFETY EVALUATION FOR MOTOR VEHICLES

	Crash Prediction Evaluation (2014-2020)[1]		Crash Reduction Factor[2, 3]
	Expected No. of crashes	Expected Crashes Rate (crashes/mi veh)	
Alternative I	13.7	1.96	42
Alternative II-a	15.3	2.19	24
Alternative II-b	15.5	2.22	21
Alternative III	17.2	2.46	N/A

With respect to pedestrian safety, the Ped ISI show that there is no significant change for pedestrian safety under the proposed alternatives.

With respect to bicycle safety, the Bike ISI shows that, compared with existing conditions, there is no significant change for bicycle safety under the proposed alternatives, except for bicyclists making left turn from the north leg and the south leg. The movements with decreased of safety are highlighted in red in Table 17.

TABLE 17 ALTERNATIVE SAFETY EVALUATION FOR BICYCLES

	Pedestrian Intersection Safety Index											
Intersection Leg	North (Dorsey)			South (Kingwood)			Northeast (Greenbag)			Southeast (Greenbag)		
Movement	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Alternative I	2.9	3.4	1.4	2.9	3.4	1.4	2.6	3.3	1.6	2.6	3.3	1.6
Alternative II-a	2.4	3.4	1.4	2.9	3.4	1.4	2.6	3.3	1.6	2.6	3.3	1.6
Alternative II-b	2.4	3.4	1.4	2.9	3.4	1.4	2.6	3.3	1.6	2.6	3.3	1.6
Alternative III	2.4	3.4	1.4	2.4	3.4	1.4	2.6	3.3	1.6	2.6	3.3	1.6

[1] Based on the FHWA Interactive Highway Safety Design Model (IHSDM) 2013.
[2] Based on the data from the Crash Modification Factors Clearinghouse, which is funded by the FHWA and maintained by the University of North Carolina Highway Safety Research Center.
[3] This value indicates a decrease in crashes.

5.2 Operational Analysis

The operational analysis uses the Intersection Capacity Utilization method and current afternoon peak hour volume to assess the proposed alternatives. Average annual growth rate of 1.1033 is used to forecast 2020 and 2040 conditions. The Synchro 8 is used in the computation process.

Table 18 summarizes the LOS, the signal delay, and the capacity utilization associated with each alternative at intersection level. Table 19 shows the LOS and the signal delay of two legs on Greenbag Rd. The LOS and the signal delay for each intersection leg are shown through Figure 12 to Figure 30. It is assumed that signal phasing is optimized based on projected volume.

This is a planning level study and is not a substitute for sound engineering judgment.

TABLE 18 ALTERNATIVE OPERATIONAL EVALUATION BY INTERSECTION

	Intersection LOS			Intersection Signal Delay (s)			Intersection Capacity Utilization		
	2014	2020	2040	2014	2020	2040	2014	2020	2040
Alternative I	C	D	E	31.5	37.1	75.0	69.1%	72.6%	86.4%
Alternative II-a	C	D	F	30.8	40.6	119.2	90.5%	95.3%	114.2%
Alternative II-b	C	D	F	28.2	35.1	119.7	90.5%	95.3%	114.2%
Alternative III	D	D	F	39.6	51.7	138.1	80.9%	85.4%	102.9%
No action	D	F	F	45.3	103.9	227.6	84.3%	88.8%	106.2%

From Table 18, it can be found that

- Alternative I provides the overall most efficient operation in 2014-2040 time frame.
- Alternative II-b provides the most efficient operation in 2014-2020 time frame.
- By optimizing existing signal timing, the Alternative III decreases the signal delay by 5.7 seconds in 2014, 52.2 seconds in 2020, and 89.5 in 2040.

TABLE 19 ALTERNATIVE OPERATIONAL EVALUATION ON GREENBAG RD

	Greenbag Rd Approach Delay and Level of Service						
	Northeast Bound (to Sabraton)			Southwest Bound (to Mountaineer Mall)			Total
	2014	2020	2040	2014	2020	2040	
Alternative I	35.6 (D)[1]	43.3 (D)	100.3 (F)	20.8(C)	22.9 (C)	30.5 (C)	254.3
Alternative II-a	17.2 (B)	16.2 (B)	25.1 (C)	42.5 (D)	57.5 (E)	244.7 (F)	403.2
Alternative II-b	19.1 (B)	19.7 (B)	30.8 (C)	40.7 (D)	51.6 (D)	270.7 (F)	432.6
Alternative III	43.3 (D)	54.7 (D)	185.3 (F)	19.3 (B)	23.4 (C)	74.9 (E)	400.9
No action	62.1 (E)	168.0 (F)	338.5 (F)	33.9 (C)	70.0 (E)	216.6 (F)	889.1

From Table 19, it can be found that

- Alternative I favors the southwest bound traffic and has the least amount of total delay in the analysis period.
- Alternative II-a and II-b favor the northeast bound traffic and keep the LOS of this approach at C in 2040
- Alternative III favors the southwest bound traffic and keeps the LOS of this approach at E in 2040.

[1] Where 35.6 = approach delay in seconds; (D) = approach LOS

Alternative I

The Alternative I recommends adding exclusive left turn lane on the northeast leg and the southwest leg and adding protected-permissive left turn phase for the two exclusive left turn lanes. Operational statuses and suggested signal phasing plan are summarized through Figure 11 to Figure 15.

FIGURE 11 ALTERNATIVE I SIGNAL PHASE

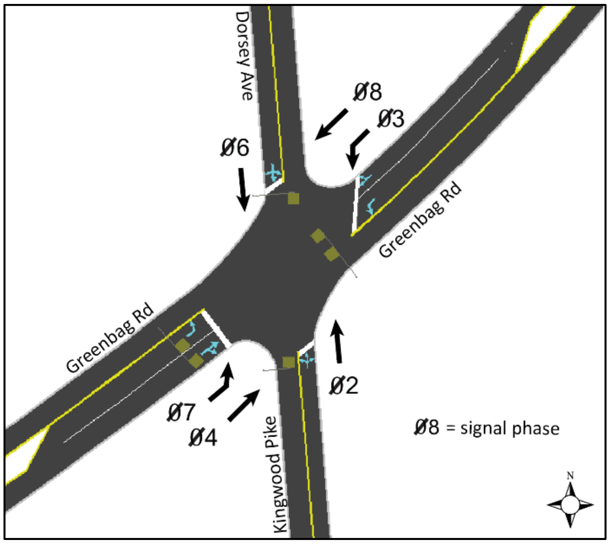


FIGURE 12 ALTERNATIVE I EXISTING CONDITION SCENARIO

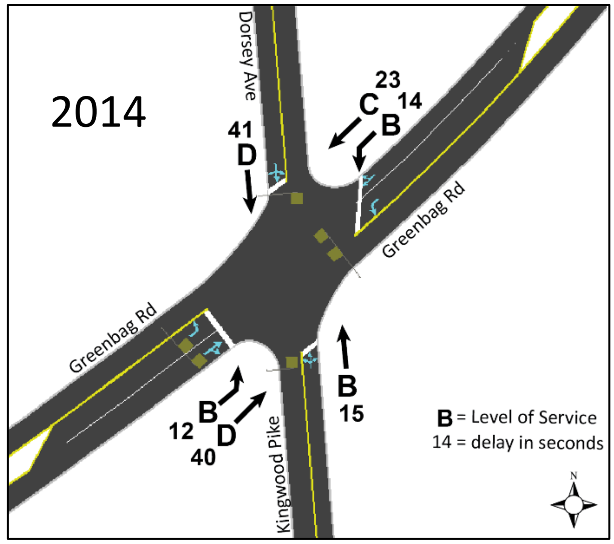


FIGURE 13 ALTERNATIVE I 2020 CONDITION SCENARIO

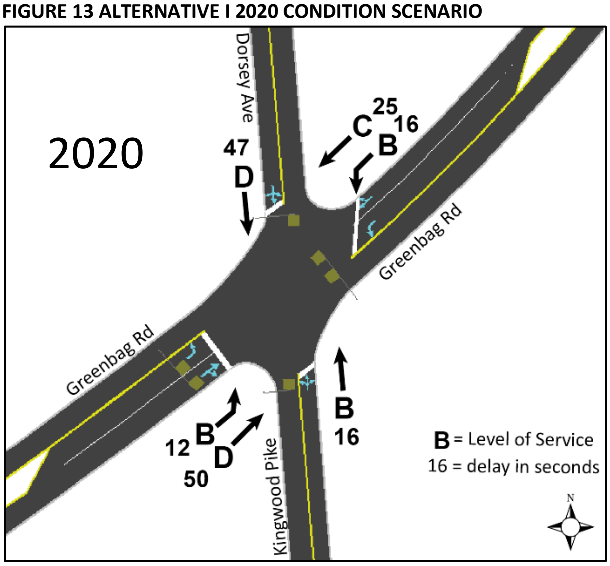
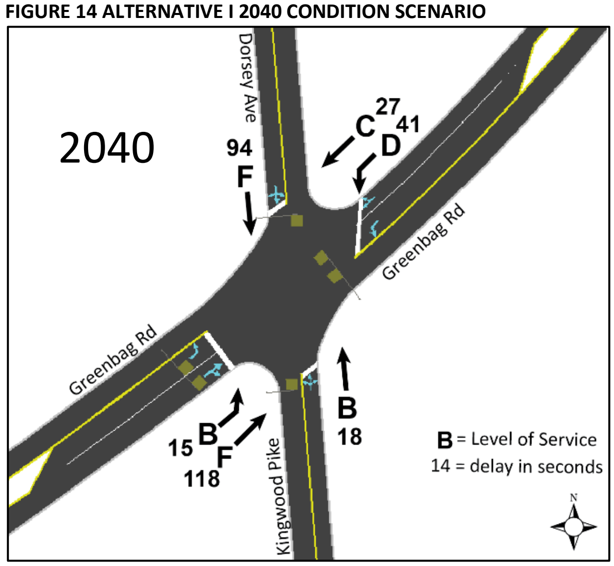


FIGURE 14 ALTERNATIVE I 2040 CONDITION SCENARIO



Alternative II-a

Alternative II-a recommends adding exclusive left turn lane on the southwest leg and adding protected-permissive left turn phase for that left turn lane. Operational statuses and suggested signal phasing plan are summarized through Figure 16 to Figure 20.

FIGURE 15 ALTERNATIVE II-A SIGNAL PHASE

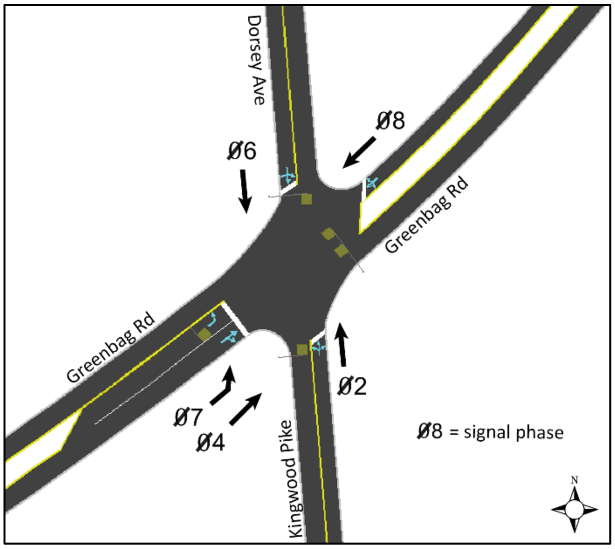


FIGURE 16 ALTERNATIVE II-A EXISTING CONDITION SCENARIO

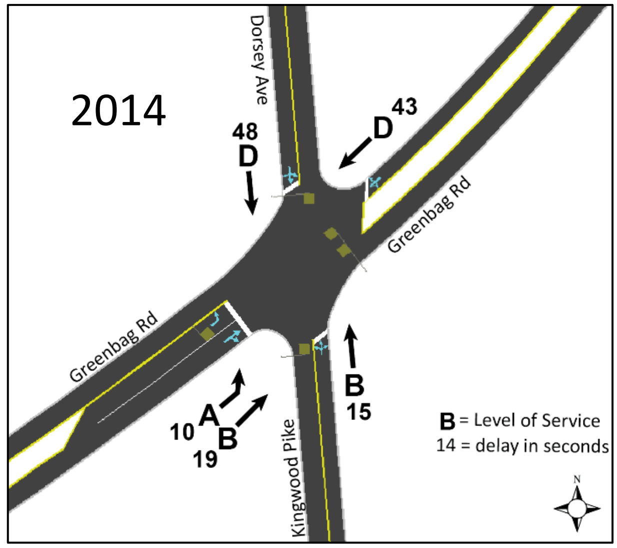


FIGURE 17 ALTERNATIVE II-A 2020 CONDITION SCENARIO

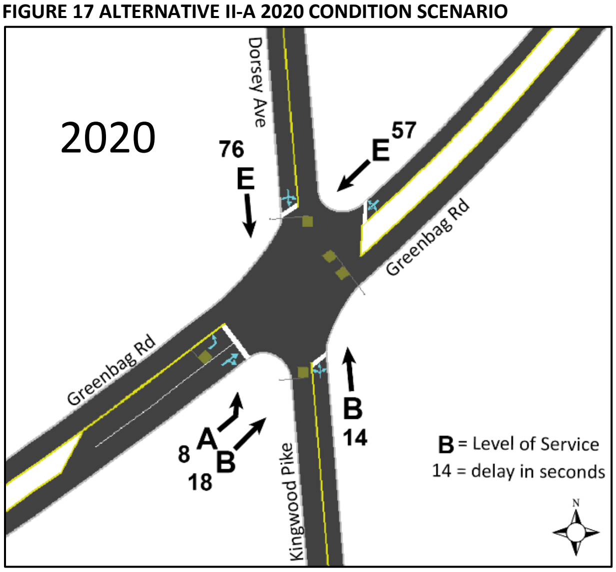
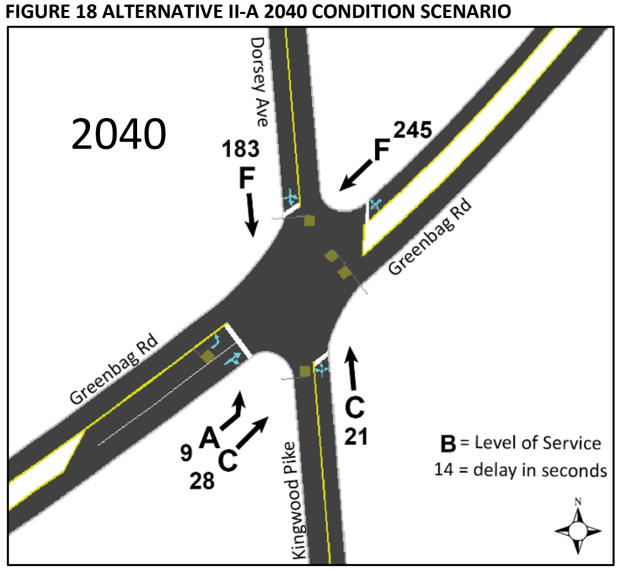


FIGURE 18 ALTERNATIVE II-A 2040 CONDITION SCENARIO



Alternative II-b

The Alternative II-a recommends adding an exclusive left turn lane on the southwest leg and optimizing existing cycle length based on existing traffic pattern; it does not recommend adding protected-permissive left turn phase for the proposed exclusive left turn lane. Operational statuses and suggested signal phasing plan are summarized through Figure 21 to Figure 25.

FIGURE 19 ALTERNATIVE II-B SIGNAL PHASE

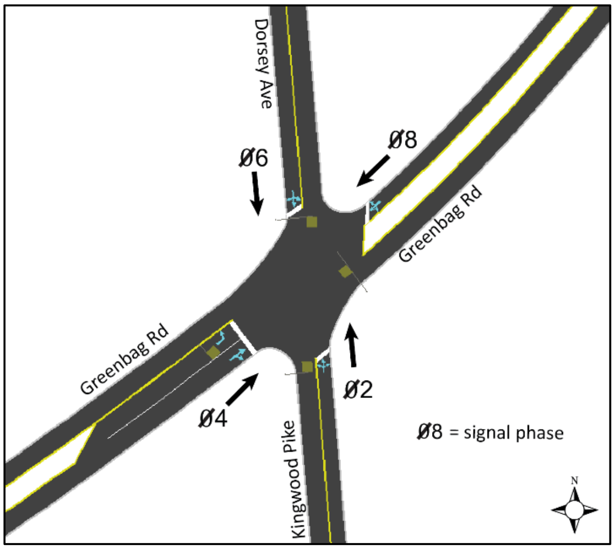


FIGURE 20 ALTERNATIVE II-B EXISTING CONDITION SCENARIO

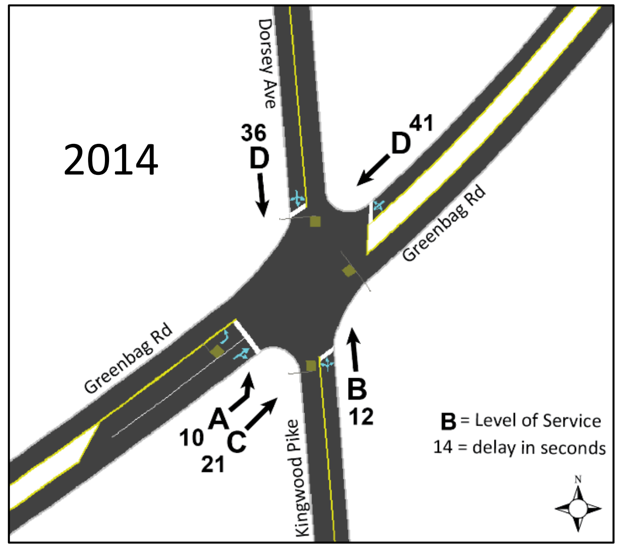


FIGURE 21 ALTERNATIVE II-B 2020 CONDITION SCENARIO

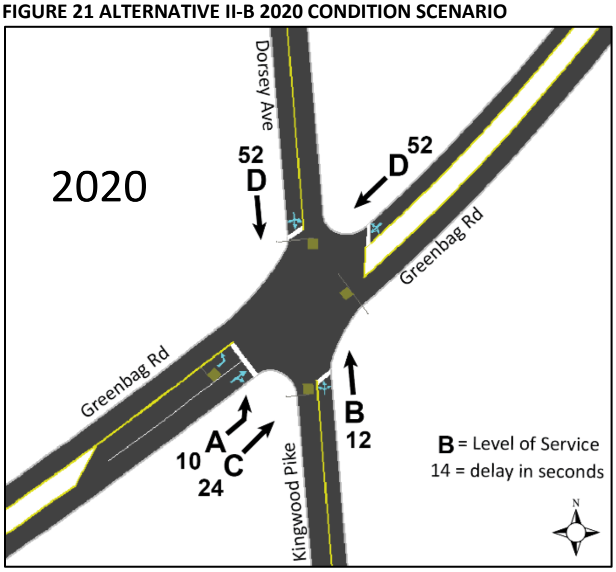
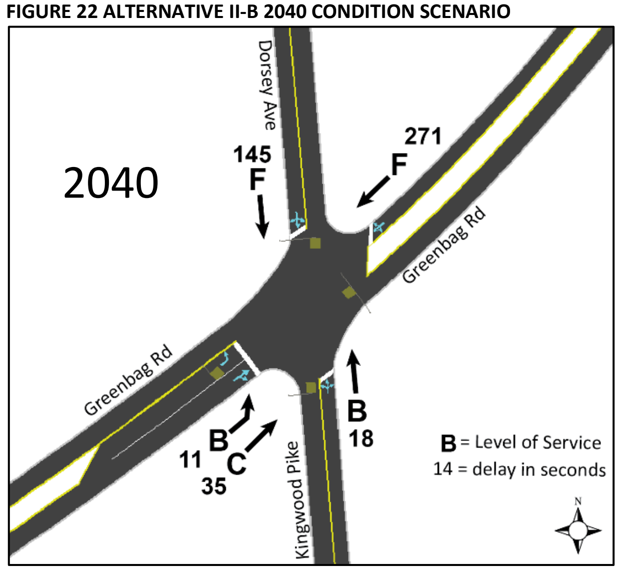


FIGURE 22 ALTERNATIVE II-B 2040 CONDITION SCENARIO



Alternative III

The Alternative III recommends optimizing the cycle length based on existing traffic pattern. Operational statuses and suggested signal phasing plan are summarized through Figure 26 to Figure 30.

FIGURE 23 ALTERNATIVE III SIGNAL PHASE

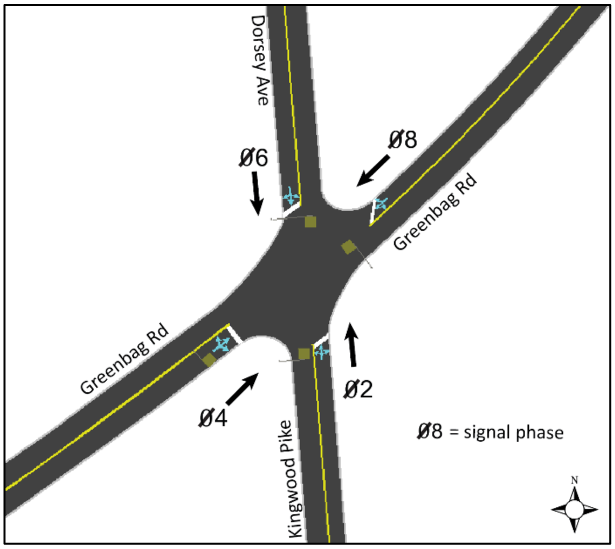


FIGURE 24 ALTERNATIVE III EXISTING CONDITION SCENARIO

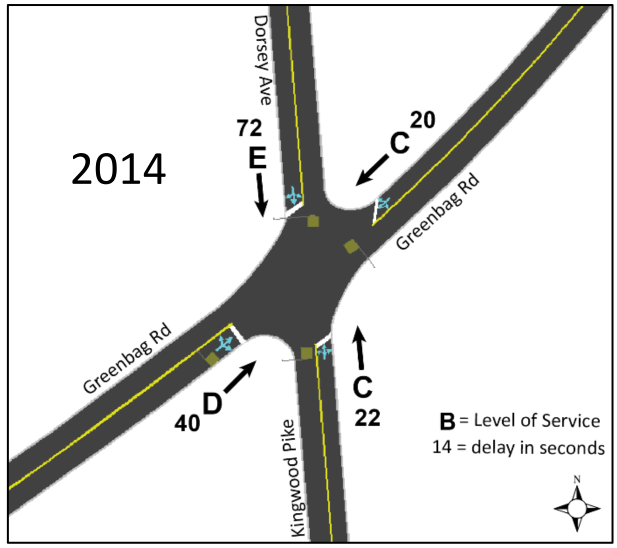


FIGURE 25 ALTERNATIVE III 2020 CONDITION SCENARIO

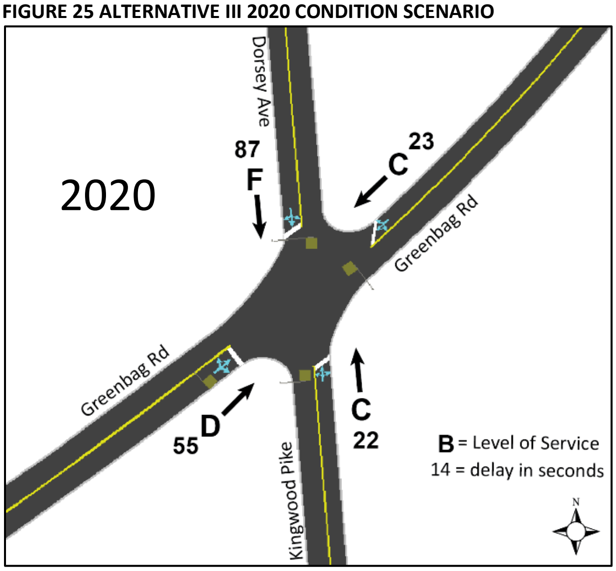
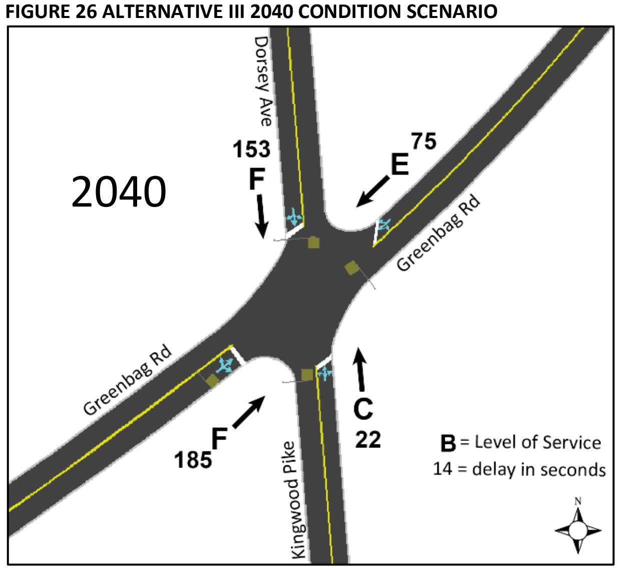


FIGURE 26 ALTERNATIVE III 2040 CONDITION SCENARIO



REFERENCES

AASHTO, Geometric Design of Highways and Streets (Green Book), 2004

Federal Highway Administration, Traffic Signal Timing Manual, 2008

Federal Highway Administration, Pedestrian and Bicyclist Intersection Safety Indices: User Guide, 2007

Federal Highway Administration, Crash Modification Factors Clearinghouse.

Transportation Research Board, Highway Capacity Manual, 2010

Transportation Research Board, Transit Capacity and Quality of Service Manual, 2nd Edition