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ASHMAN AND RODD CONVERSION STUDY

Draft Report

City of Midland

October 2022

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1. INTRODUCTION

Ashman and Rodd Streets currently operate as a one-way pair from Main Street to Cambridge Street, with Ashman primarily one-way south-westbound and Rodd one-way north-eastbound. This traffic pattern is generally well established in the community, but at times leads to some congestion and adverse travel as local residents navigate these streets. A capacity analysis review was conducted as part of a two-way conversion study to provide the City of Midland with additional information as the future configuration of the road network in the study area is considered. This analysis examines the impacts of changing both roadways from one-way to two-way over their entire lengths. The study area is highlighted in Figure 1.

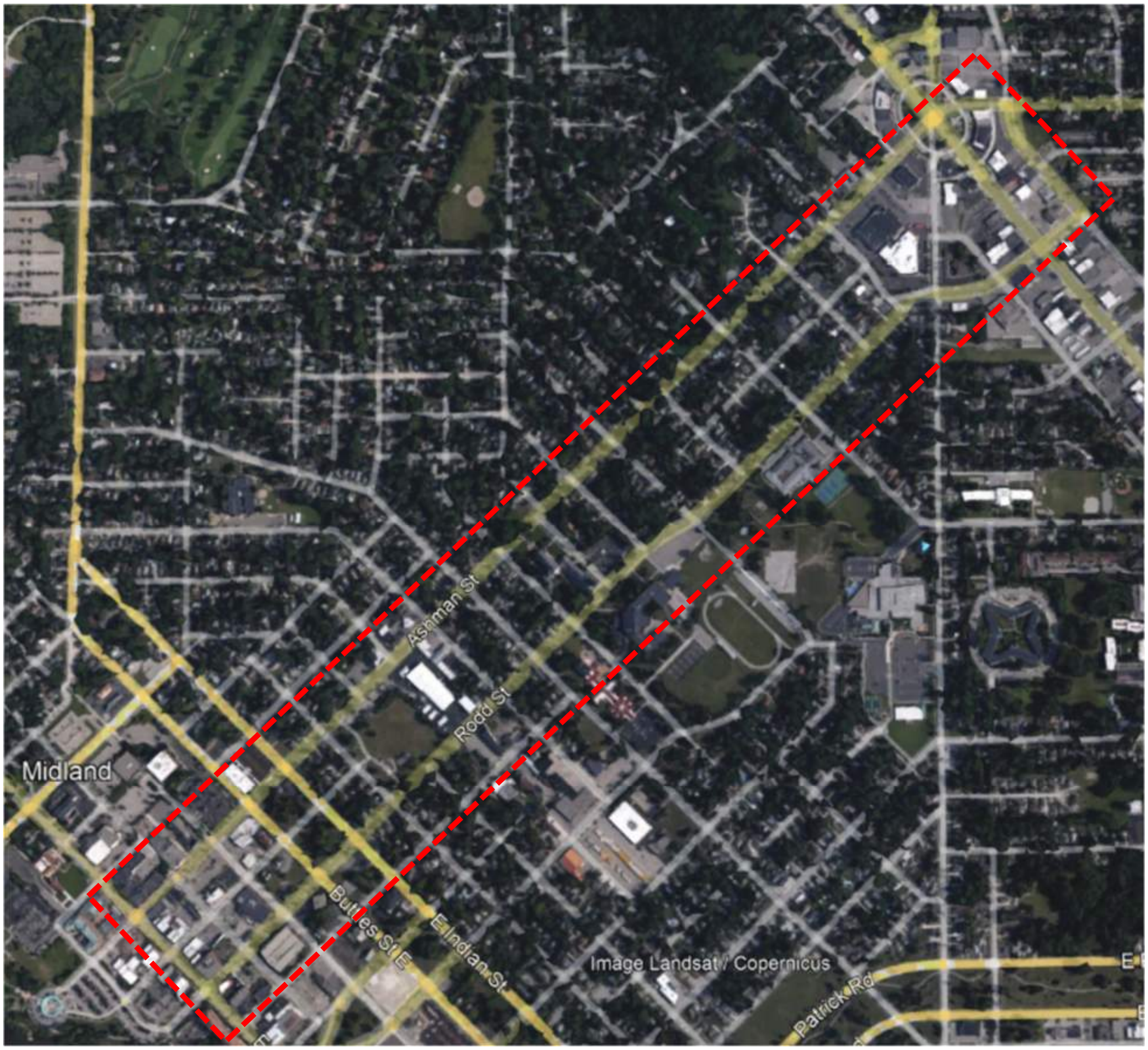

Figure 1: Study Area

In order to evaluate how these changes will affect traffic operations throughout the study network, it was necessary to make certain assumptions to best approximate how motorists would re-route through the network to reach their preferred travel destinations. These assumptions were based on existing traffic patterns at the various intersections, an understanding of the primary traffic generators in the area and finally the inherent tendency for motorists to seek the path of least delay.

2. TRAFFIC INFORMATION

TDC (Traffic Data Collection, LLC) conducted video turning movement counts for 4-hours on a weekday from 7:00 AM to 9:00 AM and 4:00 PM to 6:00 PM at the study intersections: Ashman/Main, Ashman/Buttles, Ashman/Indian, Ashman/Carpenter, Ashman/Nelson, Ashman/Saginaw/Jefferson, Ashman/Cambridge, Rodd/Main, Rodd/Buttles, Rodd/Indian, Rodd/Carpenter, Rodd/Nelson, Rodd/Jefferson, Rodd/Saginaw, and Rodd/Cambridge. TDC LLC further conducted turning movement counts for 6-hours on a Saturday from 6:00 AM to 12:00 PM during COVID-19. The counts collected pedestrian, passenger vehicle and truck data. Turning movement counts were recorded and logged with Miovision video scout cameras and the data, together with traffic reports & video files, were uploaded to the Miovision Central cloud platform.

At the time that the counts were collected school was not in session. Morning peak hour volumes were adjusted to account for trips related to school arrival. In the immediate vicinity of Central Park Elementary traffic volumes were increased by 25% at the Carpenter and Nelson intersections. Further removed from the school site, volumes were increased 10% at the Indian and Buttles intersections, and 5% everywhere else during the AM peak. The PM peak was not adjusted as school dismissal times do not correspond to the PM peak hour. Additionally, Main Street from Ashman Street to Rodd Street was closed at the time counts were taken. Volumes in the vicinity of Main Street were adjusted to allow for the analysis of the movements that were closed during data collection.

Operational Analysis

The study intersections were analyzed according to the methodologies published in the Highway Capacity Manual. For this project, Synchro Version 11 software was used to conduct the analysis. Software printouts for the evaluations of intersections have been included in Appendix B. This software package computes delay values based on factors such as number and type of lanes, intersection controls such as STOP signs or traffic signals, traffic volumes, pedestrian volumes, geometric characteristics, signal timing characteristics, roadway grade, speed limit, etc. This analysis determines the average delay experienced by vehicles. This value is an average across the entire peak hour, vehicles arriving during the busiest portion of the peak hour or arriving in a clustered group of vehicles instead of in a random pattern could experience longer delays. On the other hand, vehicles arriving during a lighter portion of the peak hour could experience a shorter delay. The average delay is used to determine the corresponding level of service (LOS) values for each intersection movement as well as the intersection as a whole.

The LOS of an intersection is based on factors such as number and types of lanes, intersection controls such as STOP signs or traffic signals, traffic volumes, pedestrian volumes, etc. LOS is expressed as a letter grade, in a range from A through F. In this context, ‘A’ represents the best conditions, with very little or no average delay to vehicles. LOS ‘F’ is the worst of conditions, equated with very large average delays and few gaps of acceptable length. The following tables identify level of service criteria for un-signalized and signalized intersections.

Table 1: Level of Service Criteria for Unsignalized Intersections

Level of Service	Average Delay/Vehicle (seconds)	Description
A	0 to 10	Little or no delay, very low main street traffic
B	> 10 to 15	Short traffic delays, many acceptable gaps
C	> 15 to 25	Average traffic delays, frequent gaps still occur
D	> 25 to 35	Longer traffic delays, limited number of acceptable gaps
E	> 35 to 50	Very long traffic delays, very small number of acceptable gaps
F	>50	Extreme traffic delays, virtually no acceptable gaps in traffic

SOURCE: Transportation Research Board, Highway Capacity Manual 2010.

Table 2: Level of Service Criteria for Signalized Intersections

Level of Service	Average Delay/Vehicle (seconds)	Description
A	0 to 10	Most vehicles do not stop at all. Most arrive during the green phase. Little or no delay.
B	> 10 to 20	More vehicles stop than for LOS A. Still good progression thru lights. Short traffic delays.
C	> 20 to 35	Significant number of vehicles stop, although many pass thru without stopping.
D	> 35 to 55	Many vehicles stop. Individual signal cycle failures are noticeable. Progression is intermittent.
E	> 55 to 80	Considered to be the limit of acceptable delay. Individual cycle failures are frequent, and progression is poor.
F	>80	Extreme and unacceptable traffic delays.

SOURCE: Transportation Research Board, Highway Capacity Manual 2010.

An intersection LOS 'D' is considered by many traffic safety professionals to be the minimum acceptable condition in an urban/suburban area. For rural areas, most highway agencies consider LOS 'C' the minimum. Given the location of the study intersection, within an urbanized area, LOS 'D' was utilized as the study goal.

3. ASHMAN STREET

Existing Roadway Cross Section

Ashman Street is one-way south-westbound except east of Cambridge Street. Ashman has a speed limit of 30 mph. Ashman Street has three travel lanes north of Buttles Street and two travel lanes south of Buttles Street. On-street parking is available south of Ellsworth Street. The intersection of Ashman Street, Saginaw Road, and Jefferson Avenue has six legs. The two Jefferson Avenue legs are one-way roads carrying traffic away from the intersection. Surrounding this intersection is a roadway circle dedicated to parking for nearby businesses. For Ashman, the existing configuration and three conversion alternatives were considered.

2-Way Conversion with Center Left Turn Lane

This alternative includes a full conversion from one-way to two-way streets along with providing a center left turn lane from the intersection at Buttles through the intersection at Cambridge. The majority of this alternative is accomplished within the existing pavement limits with the exception of modest widening to align the left turn lanes at Buttles and reconfiguration at the intersection with Cambridge.

2-Way Conversion 2-Lane

This alternative also considers a full conversion from one-way to two-way streets for the entire length of Ashman. A center left turn lane is provided between the intersections with Saginaw and Cambridge. Elsewhere along the corridor this alternative will provide excess pavement that has the potential to be repurposed. Potential uses for this additional pavement could include on-street parking on one side or an on-street bicycle facility in both directions.

2-Way Conversion Combination

This alternative also considers a full conversion from one-way to two-way streets for the entire length of Ashman. A center left turn lane is provided from the intersection at Buttles through the intersection at Cambridge on Ashman Street. Along Ashman Street, this alternative is most similar to the 2-way conversion with center left turn lane alternative. This alternative utilizes differing road sections for Ashman and Rodd within the study area. It is expected for a larger portion of traffic to shift from Rodd to Ashman since Ashman will have more lanes and no bicycle facilities in this alternative.

As shown in Tables 3 through 9 below, anticipated intersection delay and level of service (LOS) values are comparable between the existing condition and the two evaluated alternatives. The existing north-westbound approach for Ashman Street at Carpenter Street is LOS D; however, both evaluated alternatives provide a reduction in delay for that approach. At the intersection of Ashman Street and Saginaw Road, the north-eastbound approach is a LOS D for both 2-way alternatives. At this intersection the south-westbound approach in the PM peak does change from LOS C to LOS D; however, there is improvement in the north-westbound approach during this peak period.

Table 3: Ashman and Main Unsignalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	-	-	6.9	A	7.3	A	7.9	A	7.2	A
	2-Way & Left Turn Lane	7.4	A	7.3	A	7.1	A	7.2	A	7.2	A
	2-Way 2-Lane	7.4	A	7.3	A	7.1	A	7.2	A	7.2	A
	2-Way Combination	7.5	A	7.3	A	7.1	A	7.2	A	7.2	A
PM Peak	Existing	-	-	7.3	A	7.7	A	8.2	A	7.5	A
	2-Way & Left Turn Lane	7.8	A	7.9	A	7.4	A	7.6	A	7.7	A
	2-Way 2-Lane	7.8	A	7.9	A	7.4	A	7.6	A	7.7	A
	2-Way Combination	8.0	A	8.0	A	7.4	A	7.7	A	7.8	A

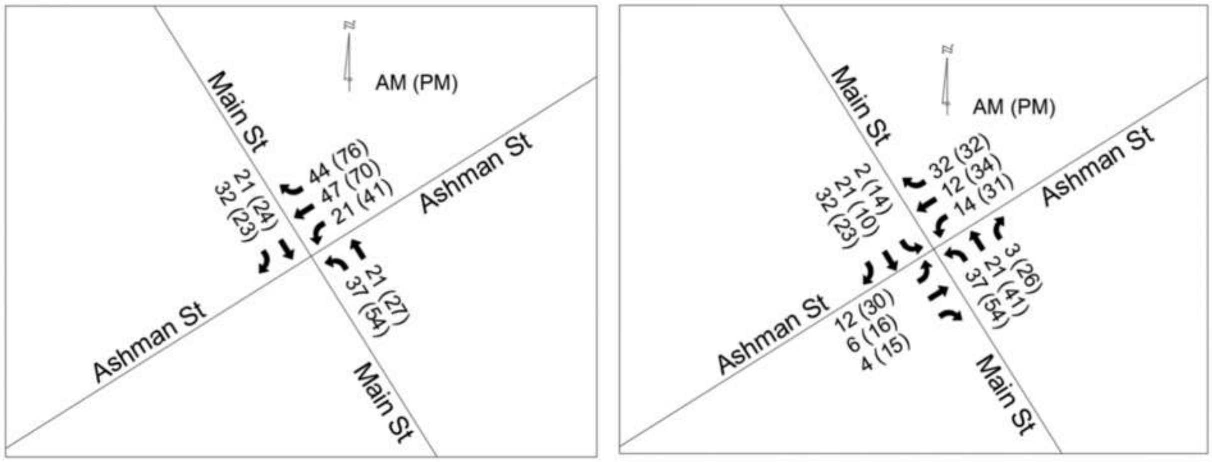

Figure 2: Ashman and Main Existing vs. Proposed Traffic Volumes

Table 4: Ashman and Buttles Signalized Delay and Level of Service

		North-eastbound		South-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	-	-	7.4	A	8.7	A	8.4	A
	2-Way & Left Turn Lane	18.6	B	11.6	B	9.8	A	10.1	B
	2-Way 2-Lane	18.6	B	17.0	B	9.8	A	10.7	B
	2-Way Combination	16.1	B	9.6	A	12.3	B	12.0	B
PM Peak	Existing	-	-	9.3	A	8.5	A	8.6	A
	2-Way & Left Turn Lane	17.7	B	34.7	C	9.1	A	12.7	B
	2-Way 2-Lane	15.8	B	26.4	C	10.8	B	13.0	B
	2-Way Combination	17.7	B	14.2	B	12.8	B	13.4	B

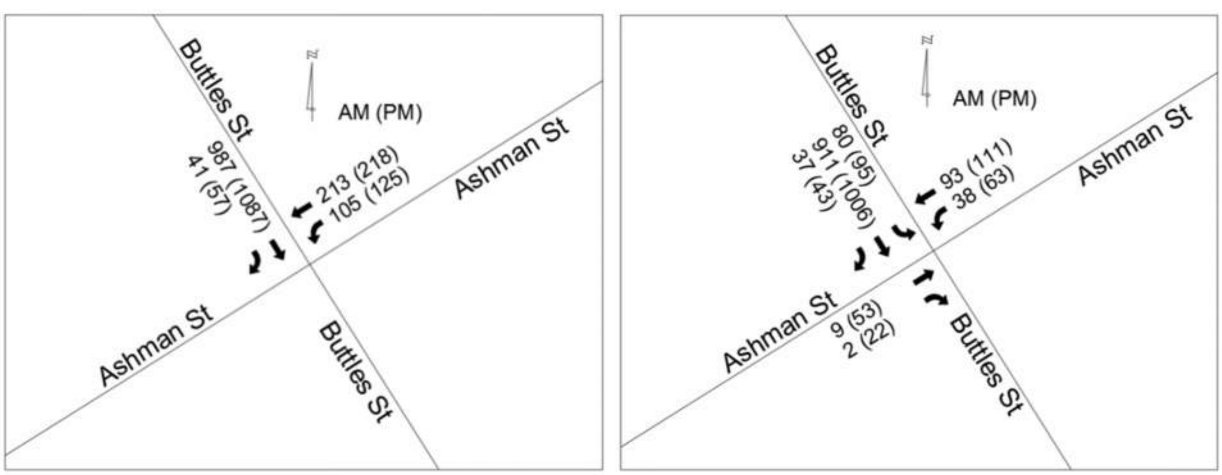

Figure 3: Ashman and Buttles Existing vs. Proposed Traffic Volumes

Table 5: Ashman and Indian Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	-	-	20.2	C	6.5	A	10.7	B
	2-Way & Left Turn Lane	34.5	C	20.7	C	4.1	A	8.4	A
	2-Way 2-Lane	35.0	C	15.9	B	4.1	A	7.8	A
	2-Way Combination	28.4	C	17.6	B	4.7	A	8.3	A
PM Peak	Existing	-	-	10.5	B	7.1	A	8.1	A
	2-Way & Left Turn Lane	32.8	C	14.5	B	2.2	A	7.1	A
	2-Way 2-Lane	32.9	C	13.0	B	2.3	A	6.9	A
	2-Way Combination	21.3	C	10.9	B	5.0	A	8.1	A

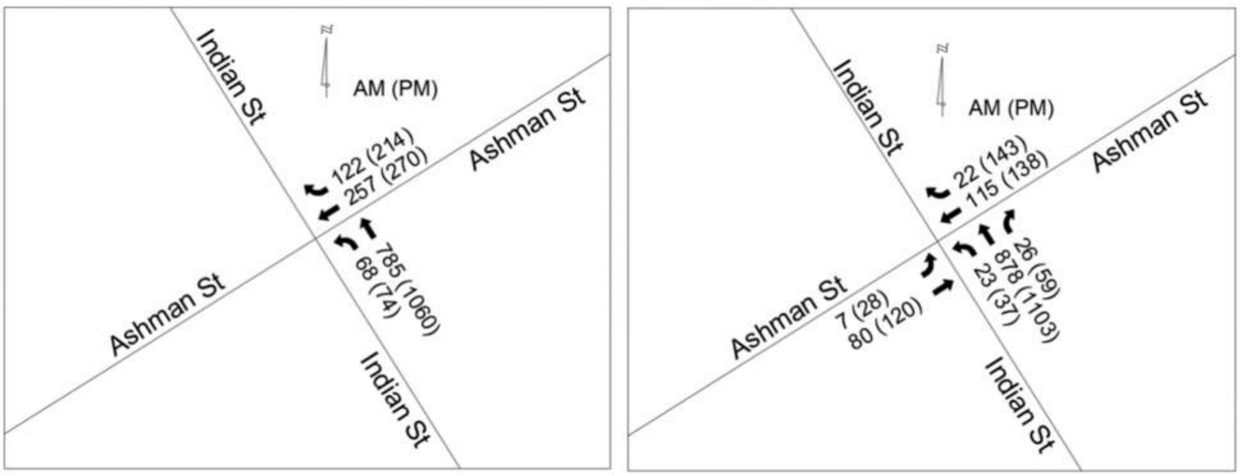

Figure 4: Ashman and Indian Existing vs. Proposed Traffic Volumes

Table 6: Ashman and Carpenter Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	-	-	-	-	**38.3**	**D**	23.1	C	8.4	A
	2-Way & Left Turn Lane	3.8	A	1.4	A	31.0	C	24.6	C	8.7	A
	2-Way 2-Lane	8.4	A	4.5	A	8.7	A	20.3	C	8.6	A
	2-Way Combination	4.0	A	2.5	A	31.5	C	22.8	C	8.9	A
PM Peak	Existing	6.2	A	6.7	A	7.8	A	15.9	B	7.9	A
	2-Way & Left Turn Lane	7.8	A	7.9	A	7.4	A	7.6	A	7.7	A
	2-Way 2-Lane	5.7	A	7.0	A	10.9	B	19.6	B	8.8	A
	2-Way Combination	6.2	A	3.6	A	9.0	A	24.1	C	7.3	A

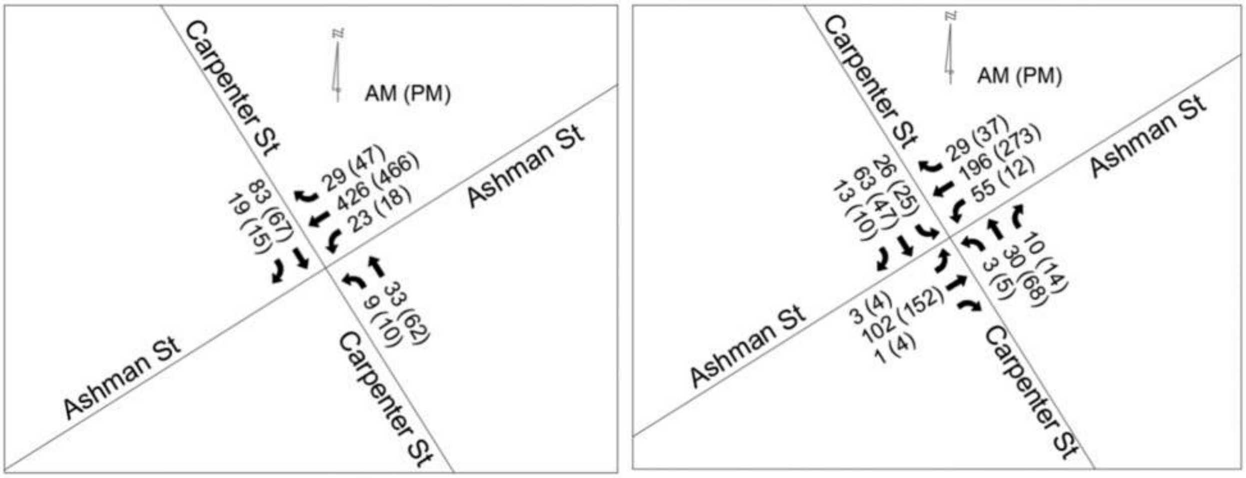

Figure 5: Ashman and Carpenter Existing vs. Proposed Traffic Volumes

Table 7: Ashman and Nelson Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	-	-	8.9	A	31.0	C	20.6	C	11.6	B
	2-Way & Left Turn Lane	1.1	A	8.2	A	32.3	C	23.7	C	8.1	A
	2-Way 2-Lane	5.1	A	5.5	A	12.0	B	19.2	B	6.3	A
	2-Way Combination	1.1	A	8.0	A	33.5	C	20.1	C	8.3	A
PM Peak	Existing	-	-	6.1	A	18.5	B	21.7	C	8.3	A
	2-Way & Left Turn Lane	4.8	A	10.8	B	13.6	B	18.7	B	9.5	A
	2-Way 2-Lane	4.3	A	7.7	A	16.1	B	23.1	C	7.9	A
	2-Way Combination	6.0	A	7.7	A	7.5	A	23.1	C	7.5	A

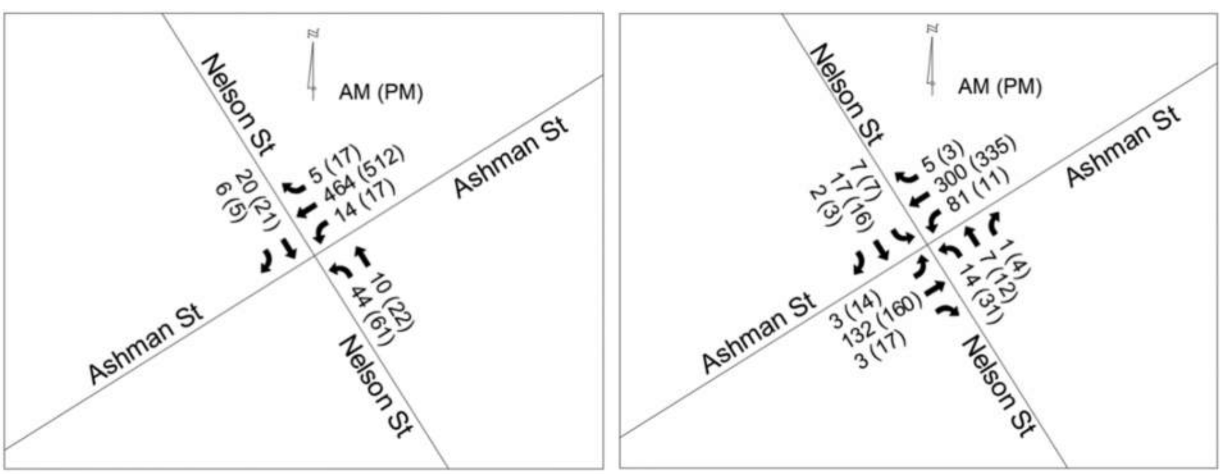

Figure 6: Ashman and Nelson Existing vs. Proposed Traffic Volumes

Table 8: Ashman and Saginaw Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	-	-	24.8	C	18.1	B	32.4	C	25.8	C
	2-Way & Left Turn Lane	**39.2**	**D**	31.2	C	22.0	C	25.4	C	26.4	C
	2-Way 2-Lane	**38.6**	**D**	31.2	C	17.9	B	25.4	C	25.0	C
	2-Way Combination	**43.2**	**D**	33.3	C	21.0	C	24.9	C	26.5	C
PM Peak	Existing	-	-	26.9	C	**46.4**	**D**	22.9	C	34.4	C
	2-Way & Left Turn Lane	**40.6**	**D**	**40.6**	**D**	26.1	C	32.5	C	31.4	C
	2-Way 2-Lane	**40.6**	**D**	**40.6**	**D**	26.1	C	32.5	C	31.4	C
	2-Way Combination	**42.4**	**D**	**48.0**	**D**	31.5	C	**35.5**	**D**	**36.7**	**D**

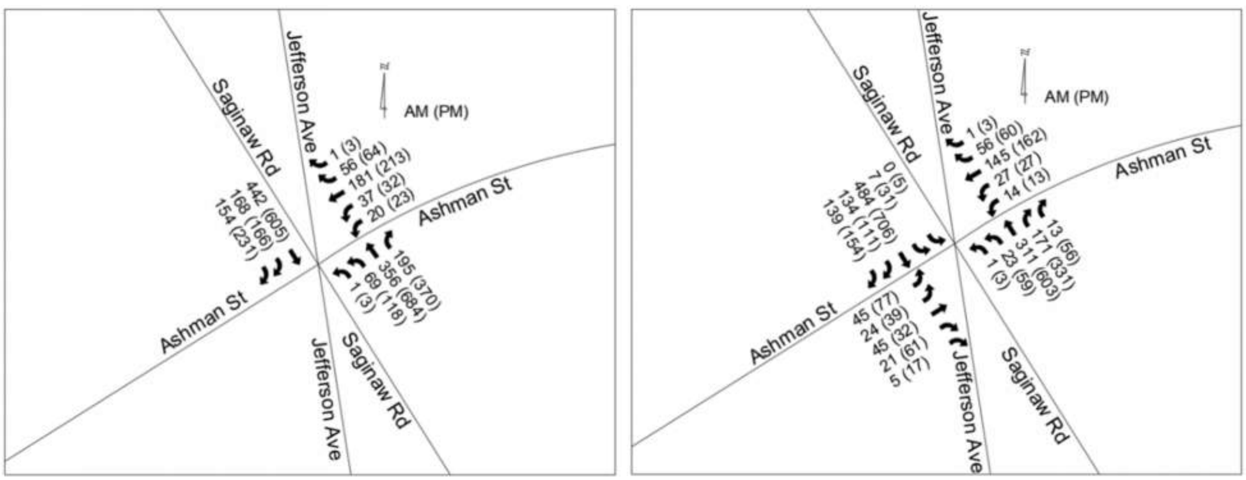

Figure 7: Ashman and Saginaw Existing vs. Proposed Traffic Volumes

Table 9: Ashman and Cambridge Unsignalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	-	-	0.2	A	12.4	B	11.6	B	2.6	A
	2-Way & Left Turn Lane	1.0	A	1.3	A	10.6	B	12.9	B	3.5	A
	2-Way 2-Lane	1.0	A	1.3	A	10.6	B	12.9	B	3.5	A
	2-Way Combination	1.0	A	0.9	A	10.5	B	12.2	B	3.2	A
PM Peak	Existing	-	-	0.2	A	13.5	B	12.1	B	5.3	A
	2-Way & Left Turn Lane	1.3	A	1.3	A	14.0	B	17.8	C	7.4	A
	2-Way 2-Lane	1.3	A	1.3	A	14.0	B	17.8	C	7.4	A
	2-Way Combination	0.8	A	0.6	A	15.1	C	15.9	C	5.8	A

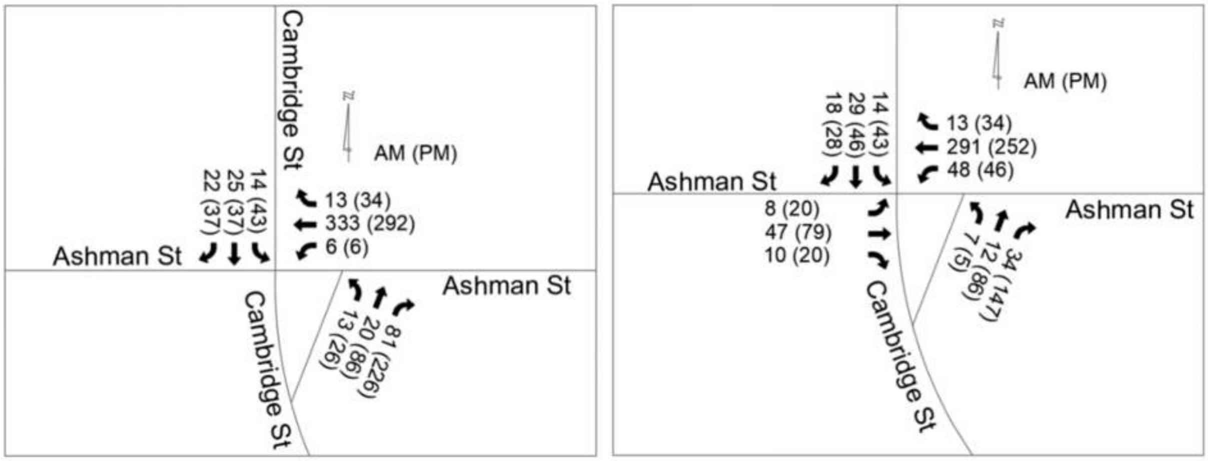

Figure 8: Ashman and Cambridge Existing vs. Proposed Traffic Volumes

4. RODD STREET

Existing Roadway Cross Section

Rodd Street is one-way north-eastbound over its entire length north of Main Street. Rodd runs from Main Street to Cambridge Street. Rodd Street has a speed limit of 30 mph. Rodd Street is two lanes south of Buttles Street, three lanes south of Jefferson Ave, four lanes from Jefferson Ave to Saginaw Road, and three lanes from Saginaw Road to Cambridge Street. Central Park Elementary School is located along Rodd St between Reardon St and Collins St.

2-Way Conversion with Center Left Turn Lane

This alternative includes a full conversion from one-way to two-way streets along with providing a center left turn lane from the intersection at Buttles through the intersection at Cambridge. The majority of this alternative is accomplished within the existing pavement limits with the exception of reconfiguration at the intersections with Jefferson and Nickels.

2-Way Conversion 2-Lane

This alternative also considers a full conversion from one-way to two-way streets for the entire length of Rodd. A center left turn lane is provided at the intersection with Saginaw. Along the corridor this alternative will provide excess pavement that has the potential to be repurposed. Potential uses for this additional pavement could include on-street parking on one side or an on-street bicycle facility in both directions.

2-Way Conversion Combination

This alternative also considers a full conversion from one-way to two-way streets for the entire length of Rodd. One lane would be provided in each direction. Excess pavement has the potential to be repurposed. Potential uses for this additional pavement include on-street bicycle facilities. Along Rodd Street, this alternative is most similar to the 2-way conversion 2-lane alternative. This alternative utilizes differing road sections for Ashman and Rodd within the study area. It is expected for a larger portion of traffic to shift from Rodd to Ashman since Ashman will have more lanes and no bicycle facilities in this alternative.

As shown in Tables 10 through 17 below, anticipated intersection delay and level of service (LOS) values are comparable between the existing condition and the two evaluated alternatives. Both two-way alternatives have little to no impact on level of service (LOS) or delay. The existing and proposed conditions on Rodd Street all operate with a LOS C or better.

Table 10: Rodd and Main Unsignalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	7.3	A	-	-	6.3	A	6.3	A	6.7	A
	2-Way & Left Turn Lane	6.6	A	8.4	A	6.6	A	6.5	A	7.6	A
	2-Way 2-Lane	7.2	A	7.6	A	6.6	A	6.5	A	7.2	A
	2-Way Combination	7.1	A	7.6	A	6.6	A	6.4	A	7.2	A
PM Peak	Existing	8.1	A	-	-	6.8	A	6.9	A	7.5	A
	2-Way & Left Turn Lane	7.1	A	8.4	A	7.2	A	7.0	A	7.5	A
	2-Way 2-Lane	7.8	A	7.6	A	7.1	A	6.9	A	7.4	A
	2-Way Combination	7.6	A	7.6	A	7.0	A	6.9	A	7.3	A

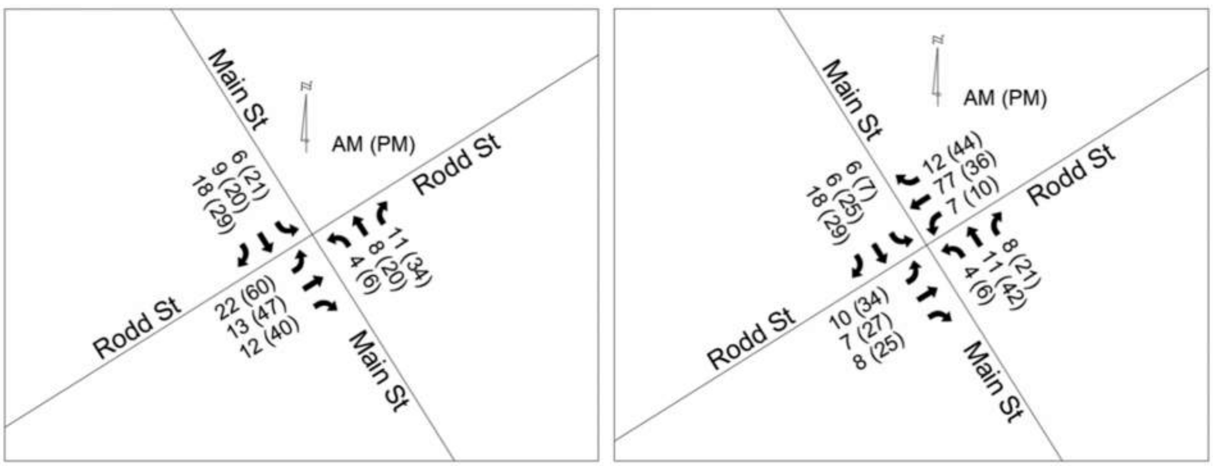

Figure 9: Rodd and Main Existing vs. Proposed Traffic Volumes

Table 11: Rodd and Buttles Signalized Delay and Level of Service

		North-eastbound		South-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	13.2	B	-	-	4.5	A	4.9	A
	2-Way & Left Turn Lane	11.9	B	14.3	B	3.1	A	4.9	A
	2-Way 2-Lane	11.9	B	16.7	B	3.2	A	5.3	A
	2-Way Combination	9.6	A	16.9	B	4.7	A	6.1	A
PM Peak	Existing	15.4	B	-	-	5.5	A	6.9	A
	2-Way & Left Turn Lane	9.0	A	10.3	B	13.8	B	12.9	B
	2-Way 2-Lane	16.1	B	14.2	B	2.3	A	5.0	A
	2-Way Combination	7.9	A	11.0	B	13.8	B	13.2	B

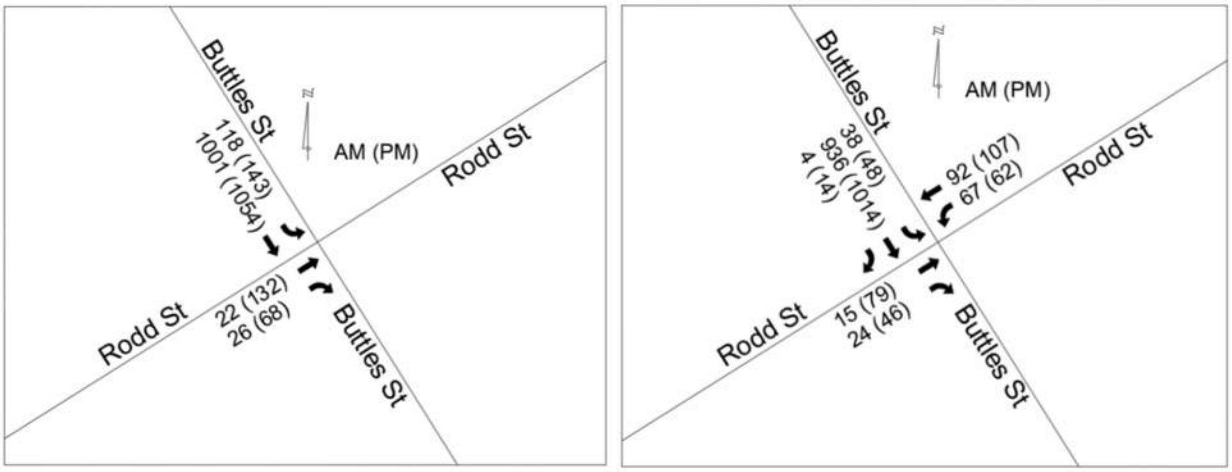

Figure 10: Rodd and Buttles Existing vs. Proposed Traffic Volumes

Table 12: Rodd and Indian Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	20.7	C	-	-	8.3	A	10.1	B
	2-Way & Left Turn Lane	20.1	C	6.7	A	9.5	A	9.6	A
	2-Way 2-Lane	21.4	C	5.6	A	9.5	A	9.5	A
	2-Way Combination	16.8	B	2.0	A	13.8	B	12.2	B
PM Peak	Existing	14.8	B	-	-	9.1	A	10.2	B
	2-Way & Left Turn Lane	23.9	C	5.0	A	9.3	A	9.9	A
	2-Way 2-Lane	17.5	B	3.9	A	11.1	B	10.7	B
	2-Way Combination	18.3	B	2.6	A	13.2	B	12.1	B

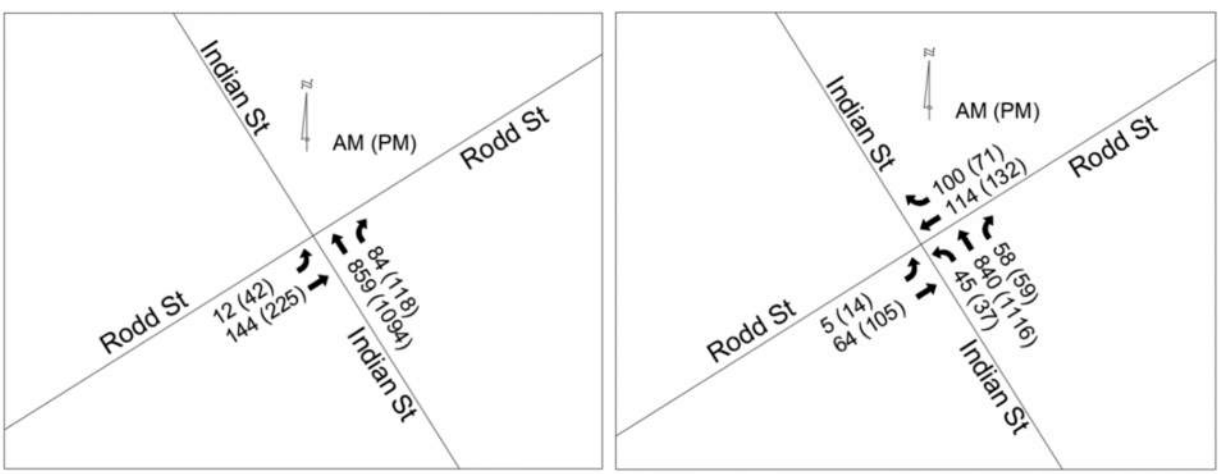

Figure 11: Rodd and Indian Existing vs. Proposed Traffic Volumes

Table 13: Rodd and Carpenter Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	8.5	A	-	-	12.2	B	8.9	A	9.2	A
	2-Way & Left Turn Lane	11.3	B	10.6	B	14.1	B	6.0	A	10.3	B
	2-Way 2-Lane	11.2	B	12.1	B	14.1	B	11.1	B	11.9	B
	2-Way Combination	12.8	B	10.2	B	14.2	B	8.4	A	11.2	B
PM Peak	Existing	9.4	A	-	-	10.5	B	28.8	C	12.5	B
	2-Way & Left Turn Lane	7.6	A	3.7	A	14.3	B	12.5	B	7.9	A
	2-Way 2-Lane	8.4	A	3.7	A	14.3	B	10.7	B	8.0	A
	2-Way Combination	10.7	B	4.4	A	14.7	B	28.5	C	11.5	B

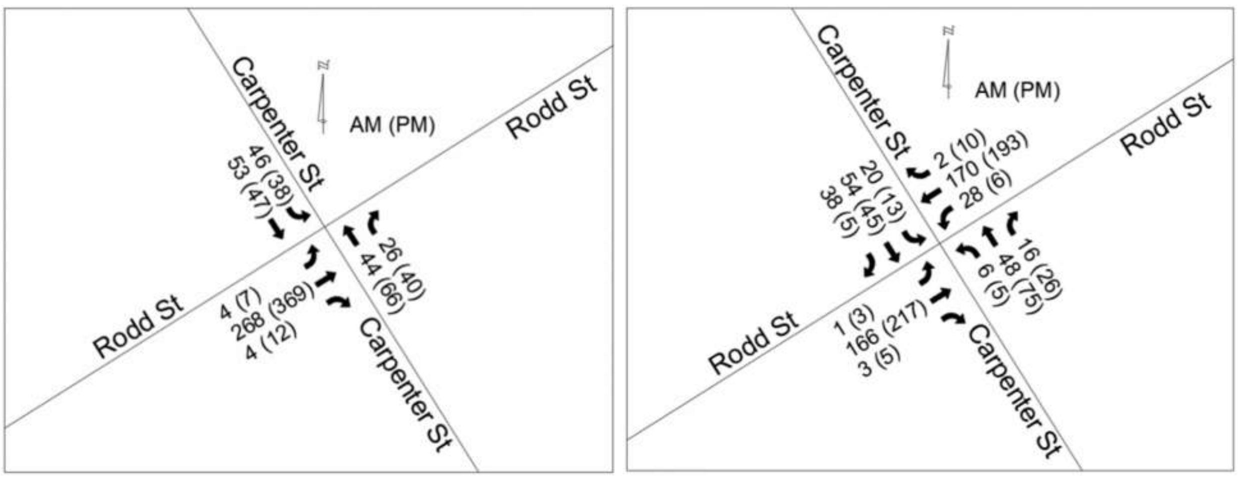

Figure 12: Rodd and Carpenter Existing vs. Proposed Traffic Volumes

Table 14: Rodd and Nelson Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	5.8	A	-	-	15.8	B	10.6	B	7.5	A
	2-Way & Left Turn Lane	8.4	A	17.2	B	16.0	B	3.7	A	10.1	B
	2-Way 2-Lane	7.1	A	15.7	B	16.0	B	6.4	A	9.7	A
	2-Way Combination	9.1	A	17.1	B	15.9	B	4.1	A	10.4	B
PM Peak	Existing	5.2	A	-	-	15.5	B	15.8	B	7.2	A
	2-Way & Left Turn Lane	8.4	A	20.5	C	15.9	B	11.8	B	13.3	B
	2-Way 2-Lane	8.5	A	19.3	B	15.9	B	11.7	B	13.0	B
	2-Way Combination	7.6	A	20.0	B	16.7	B	14.4	B	14.8	B

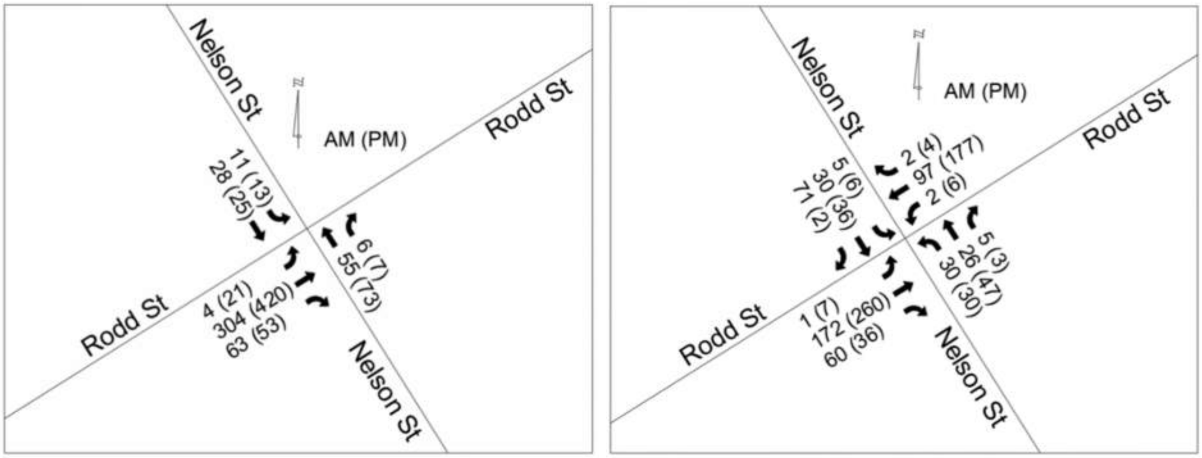

Figure 13: Rodd and Nelson Existing vs. Proposed Traffic Volumes

Table 15: Rodd and Jefferson Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	4.5	A	-	-	5.3	A	2.4	A	3.9	A
	2-Way & Left Turn Lane	5.8	A	11.7	B	5.6	A	3.5	A	6.5	A
	2-Way 2-Lane	5.6	A	14.3	B	5.8	A	2.7	A	7.0	A
	2-Way Combination	6.1	A	12.8	B	5.6	A	6.7	A	7.6	A
PM Peak	Existing	4.8	A	-	-	7.0	A	16.7	B	8.7	A
	2-Way & Left Turn Lane	2.5	A	13.0	B	7.2	A	17.3	B	9.9	A
	2-Way 2-Lane	2.6	A	14.7	B	7.2	A	17.3	B	10.4	B
	2-Way Combination	5.1	A	14.9	B	9.2	A	16.2	B	12.8	B

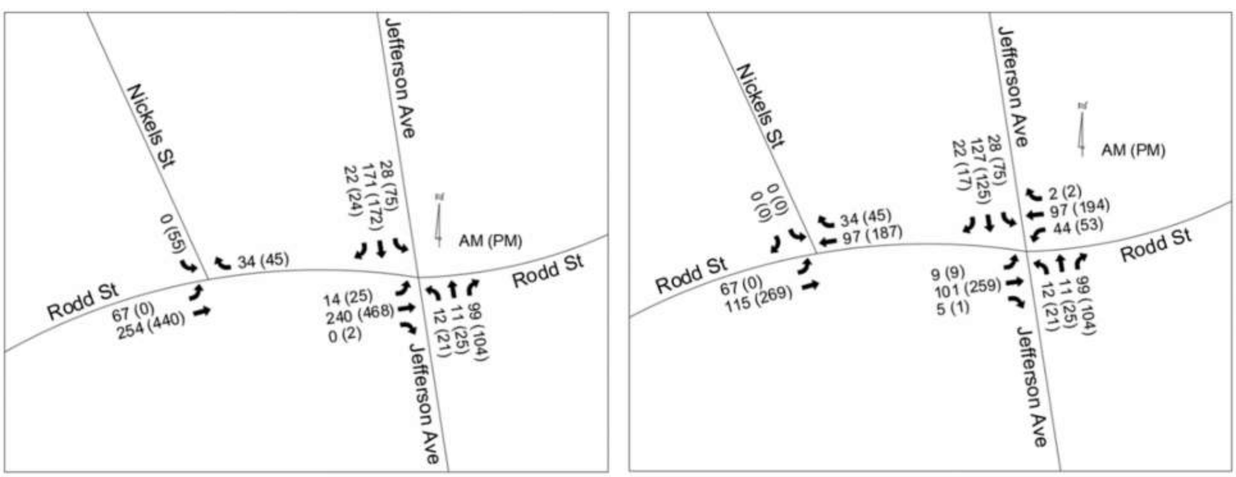

Figure 14: Rodd and Jefferson Existing vs. Proposed Traffic Volumes

Table 16: Rodd and Saginaw Signalized Delay and Level of Service

		North-eastbound		South-westbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	23.8	C	-	-	8.5	A	0.3	A	10.7	B
	2-Way & Left Turn Lane	11.6	B	14.2	B	14.8	B	9.6	A	12.1	B
	2-Way 2-Lane	17.2	B	14.3	B	14.8	B	8.4	A	12.8	B
	2-Way Combination	11.9	B	14.2	B	14.1	B	9.2	A	11.7	B
PM Peak	Existing	27.7	C	-	-	9.9	A	4.4	A	14.0	B
	2-Way & Left Turn Lane	20.5	C	15.6	B	21.3	C	12.0	B	17.6	B
	2-Way 2-Lane	20.6	C	15.7	B	21.3	C	12.0	B	17.7	B
	2-Way Combination	22.1	C	18.9	B	14.4	B	2.8	A	10.9	B

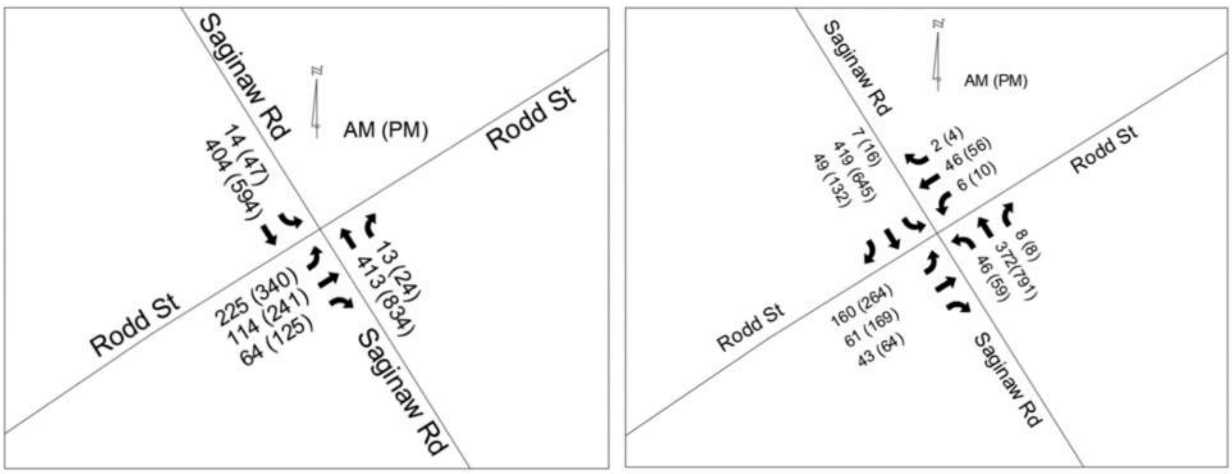

Figure 15: Rodd and Saginaw Existing vs. Proposed Traffic Volumes

Table 17: Rodd and Cambridge Unsignalized Delay and Level of Service

		North-eastbound		North-westbound		South-eastbound		Intersection	
		Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS	Delay (Sec.)	LOS
AM Peak	Existing	6.1	A	11.0	B	11.4	B	7.5	A
	2-Way & Left Turn Lane	5.9	A	10.0	A	9.3	A	8.0	A
	2-Way 2-Lane	5.9	A	10.1	B	9.3	A	8.1	A
	2-Way Combination	5.9	A	10.0	A	9.6	A	7.9	A
PM Peak	Existing	6.6	A	16.8	C	18.6	C	9.3	A
	2-Way & Left Turn Lane	6.7	A	14.6	B	12.9	B	9.6	A
	2-Way 2-Lane	6.8	A	14.8	B	12.9	B	9.7	A
	2-Way Combination	6.2	A	11.8	B	12.3	B	8.8	A

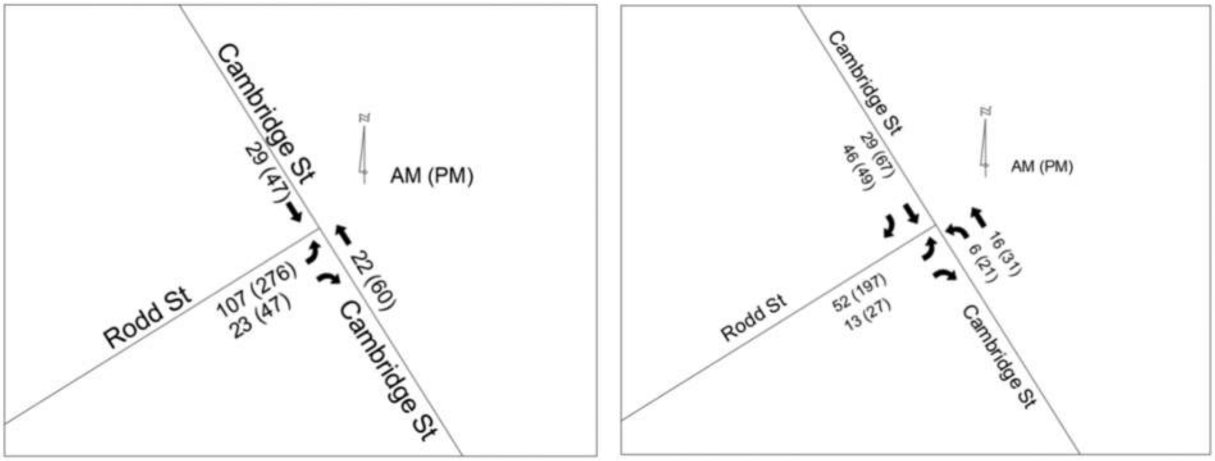

Figure 16: Rodd and Cambridge Existing vs. Proposed Traffic Volumes

5. CONVERSION CONSIDERATIONS

Projected intersection operations play a critical component of a transportation network review. When large changes to the network are being considered, there are other factors that can be equally important to consider in the evaluation of the alternatives.

Roadway Capacity

The factor most closely related to the operational analysis; the capacity of the study roadways would be modified by the conversion from one-way to two-way flow. The creation of one-way pair road networks was traditionally implemented to maximize the capacity of the road system. This was especially the case in downtown areas where one-way systems were used to get a high number of people quickly in and out of a commercial district.

The implementation of any of the conversion alternatives in this area would decrease the available capacity of these roadways by reducing the number of through travel lanes. The operational analysis of the project area indicates that there is excess capacity in the existing one-way pair road network. The reduction in capacity associated with a conversion to two-way traffic provides for a more right-sized roadway system.

Roadway Speeds

In addition to providing for a large number of vehicles, one-way pair systems were also historically implemented to expedite travel through a community. By providing multiple lanes in one direction, one-way roadways allow for more aggressive drivers to frequently switch lanes to maintain higher speeds. When one-way roadways have excessive capacity and multiple lanes there is significant opportunity for drivers to travel at excessive speeds along the corridor. This can be mitigated somewhat by close attention to the signal timing offsets to control the 'green band' of progression between signalized intersections, penalizing speeding traffic by having them have to slow or stop to get back in the synchronization of the signals.

The implementation of a conversion alternative in the study area would eliminate the opportunity for faster moving vehicles to weave around slower vehicles. This type of conversion has the potential to serve as a traffic calming treatment, narrowing the speed gaps between the fastest and slowest vehicles on the roadway. Narrowing the speed gaps typically results in an increase in safety. This includes a reduction in severe type crashes.

System Navigation

Many communities throughout the country have one-way street networks. Drivers who are local to and are familiar with these networks find navigation simple and are used to circling the block to get to their destination. Drivers who are less familiar with the area often find two-way streets to be more intuitive. A one-way system does not allow less familiar drivers to return on the same street they arrived on.

The implementation of a conversation alternative would represent a change in what local drivers are familiar with. By providing a more direct route to the destination, two-way streets can reduce travel distances and decrease the number of vehicles using side streets to circle the block.

Roadway Access and Safety

The analysis of potential conflict points is frequently used as a proxy for determining safety differences between intersection configurations. With a one-way pair system, all of the conflict points involving the eliminated approach are removed. With fewer conflict points, vehicles entering or crossing the roadway only have to find gaps in traffic from one direction. This benefit is partially offset both by the multiple lanes provided in the one-way configuration and the potential for higher speeds. Wrong-way movements are a safety concern unique to one-way streets.

Two-way traffic in the study area will require drivers entering the roadway to monitor for traffic in both directions, resulting in more conflict points. Drivers may have modestly increased waits for bi-directional gaps to turn left from driveways or side streets. It should be noted that roughly half of these left turns would not be allowed under the current traffic flow pattern.

Non-Motorized Mobility and Safety

Road speed and conflict points are also key factors in pedestrian and cyclist safety. One-way streets can provide simplified crossings with pedestrians primarily concerned with risks coming from a single direction. This is complicated by the potential for wrong way drivers, high speeds and volume increased by adverse travel in the existing one-way network. An additional concern with multi lane crossings is the ability of a vehicle in the first lane to block the line of site between the pedestrian and oncoming vehicles in subsequent lanes.

The two-way conversion alternatives not providing a center left turn lane offer the potential for additional non-motorized accommodations. These alternatives could use excess pavement to provide bicycle facilities or on-street parking. In addition to the potential for designated bicycle facilities, these alternatives could allow for the shortening of pedestrian crossings by way of curb extensions.

Emergency and Maintenance Considerations

Transportation systems have a wide variety of users. These roadways are key components of the city's infrastructure and changes have the potential to impact multiple areas. Emergency response plans are typically optimized for the existing roadway network. Any changes to the network should be considered for impacts to and adjustment to emergency routes. With Police, Fire and EMS all located in the vicinity of the study area, careful consideration of how emergency response could be modified in the event of 2-way conversion is critical. In many cases, first responders are balancing concerns of navigating around stopped vehicles during a call with the benefit of a more direct route provided by two-way traffic.

Maintenance is another often overlooked consideration with a potential two-way conversion. Multiple lanes in one direction offer the potential to close one lane for repair with minimal disruption to the remaining lanes. Winter plowing and leaf removal strategies my also need to be adjusted as one-way streets are often approached differently than two-way streets for this activity.

Implementation Considerations

The roadway within this network would need to be adjusted to accommodate two-way traffic. This includes modifying existing pavement markings, signals, and signage. Depending on the selected alternative, a center left turn lane, regular travel lanes and/or bicycle facilities would need to be striped. With the age and design life of the existing traffic signals, adding signal heads for the new traffic direction and adjusting existing heads for lane alignment is likely to lead to full rebuilds of the traffic signals. While this comes at a substantial cost, the age of the signals may require replacement in the near future providing opportunity to implement a different traffic pattern at that time. The existing one-way and do not enter signs would need to be removed. If the bicycle facility option is selected, additional signage and pavement markings will be needed to alert drivers of this shift in conditions.

Additional Opportunity

The potential conversion of this network from a one-way pair to a set of two-way streets provides the city with some unique opportunities. Removing some of the excess capacity of the roadway would allow for the development of additional features, either within or outside the roadway. As demonstrated by the Capacity analysis, this space could be used for the establishment of a two-way center left-turn lane to facilitate easier movements to side streets and driveways. The space could also be repurposed to allow for a designated bicycle facility in both directions or on-street parking along one side.

Any of the two-way alternatives provide some additional flexibility to the road network. Once two-way travel is established on both roadways, detour routes in the study area become much simpler than under the one-way configuration. This could allow for more aggressive construction schedules for future projects. It could also allow for more special event road closures.

6. CONCLUSIONS AND FINDINGS

Ashman and Rodd Streets currently operate as a one-way pair from Main Street to Cambridge Street, with Ashman primarily one-way south-westbound and Rodd one-way north-eastbound. The modification of this traffic pattern to establish two-way flow provides similar operational results to the existing one-way configuration. The intersection operational results are similar for the two-way conversion both with and without a center left turn lane.

Overall, the existing and ***all proposed alternatives*** have acceptable levels of service and delay. Additionally, there is little variation between proposed alternatives when it comes to operations. The combination alternative has some operational tradeoffs since Rodd is being used as a multimodal corridor. The analysis of the combination alternative indicates acceptable level of service on both corridors.

Table 18: Anticipated Post Conversion Operational Delay Change (Seconds)

	2-Way & Left Turn Lane		2-Way 2-Lane		2-Way Combination	
	AM	PM	AM	PM	AM	PM
Ashman and Main	0	0.2	0	0.2	0	0.3
Ashman and Buttles	1.7	4.1	2.3	4.4	3.6	4.8
Ashman and Indian	-2.3	-1	-2.9	-1.2	-2.4	0
Ashman and Carpenter	0.3	-0.2	0.2	0.9	0.5	-0.6
Ashman and Nelson	-3.5	-5.3	1.2	-0.4	-3.3	-0.8
Ashman and Saginaw	0.6	-0.8	-3	-3	0.7	2.3
Ashman and Cambridge	0.9	2.1	0.9	2.1	0.6	0.5
Rodd and Main	0.9	0	0.5	-0.1	0.5	-0.2
Rodd and Buttles	0	6.0	0.4	-1.9	1.2	6.3
Rodd and Indian	-0.5	-0.3	-0.6	0.5	2.1	-1.9
Rodd and Carpenter	1.1	-4.6	2.7	-4.5	2	-1
Rodd and Nelson	2.6	6.1	2.2	5.8	2.9	7.6
Rodd and Jefferson	2.6	1.2	3.1	1.7	3.7	4.1
Rodd and Saginaw	1.4	3.6	2.1	3.7	1	-3.1
Rodd and Cambridge	0.5	0.3	0.6	0.4	0.4	-0.5

When contrasting alternatives, factors outside operation and delay are significant considerations. The existing one-way pair and each of the studied two-way alternatives provide for a unique character of the roadway. The existing configuration provides a high-capacity road network that is familiar to local drivers and does not require additional capital expenditure to continue its use. The two-way alternatives provide opportunity to change the character of these roadways and allow for more direct travel routes and consistent speeds. These alternatives include the potential to include a left turn lane for a network more focused on vehicle mobility or bicycle facilities could provide non-motorized features that do not currently exist.

A cost estimate was prepared to evaluate the potential price differences for each alternative. After preparing each alternative, it was determined that the cost of each alternative is relatively comparable. The biggest factors when determining price were permanent signing and pavement markings, traffic signals, and maintenance of traffic (MOT). The overall project cost would be around 3.7 million dollars. Within that estimate, it is assumed that all signals would be rebuilt due to their current age and overall life expectancy. It is also assumed that roadway resurfacing is not included in this estimate, outside of minor geometric adjustments at a few select intersections.

Table 19: One-Way vs. Two-Way Comparisons

	One-Way	Two-Way
Operations	Acceptable Operations	Acceptable Operations
Roadway Capacity	Adequate Excess Reserve	Adequate Somewhat Less Reserve
Roadway Speeds	Higher Speeds Quick Movement Through Town	Lower Speeds More Consistent Speeds
System Navigation	Familiar for Locals Adverse Travel Required	Friendly for Visitors Direct Route Travel
Roadway Access and Safety	Conflicts from One Direction Higher Speeds	More Conflict Points Full Access Driveways
Non-Motorized Mobility	Conflicts from One Direction Potential for Higher Vehicle Volumes	Bike Facility Opportunity Shorter Ped Crossing Opportunity
Emergency and Maintenance	No Change Required Multiple Lanes to Maneuver	Direct Route Travel Requires Revised Planning
Implementation	No Change Required	Signs, pavement markings, and signals will need to be changed
Additional Opportunity	No Change Required	Parking or Bike Facilities Special Event Closures
Anticipated Costs	No Immediate Expenditures Signal Replacement at Design Life	Estimated 3.7 million