Onondaga County Department of Transportation

Traffic Signal Optimization Project (Isolated Intersections)

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CHAPTER I

INTRODUCTION

Outdated traffic signal timings account for a significant amount of traffic delay on urban and suburban roadways across the country. Periodically updating traffic signal equipment and timings based on new technology and current traffic volumes can provide significant benefits at a relatively low cost, alleviate the need for additional infrastructure, and reduce time spent in traffic, fuel consumption, and emissions. This report summarizes the results of a Traffic Signal Timing Optimization study conducted at various county-owned and controlled intersections located throughout Onondaga County, New York.

A. Study Area

The study area intersections for this report include the following, as shown on Figure 1:

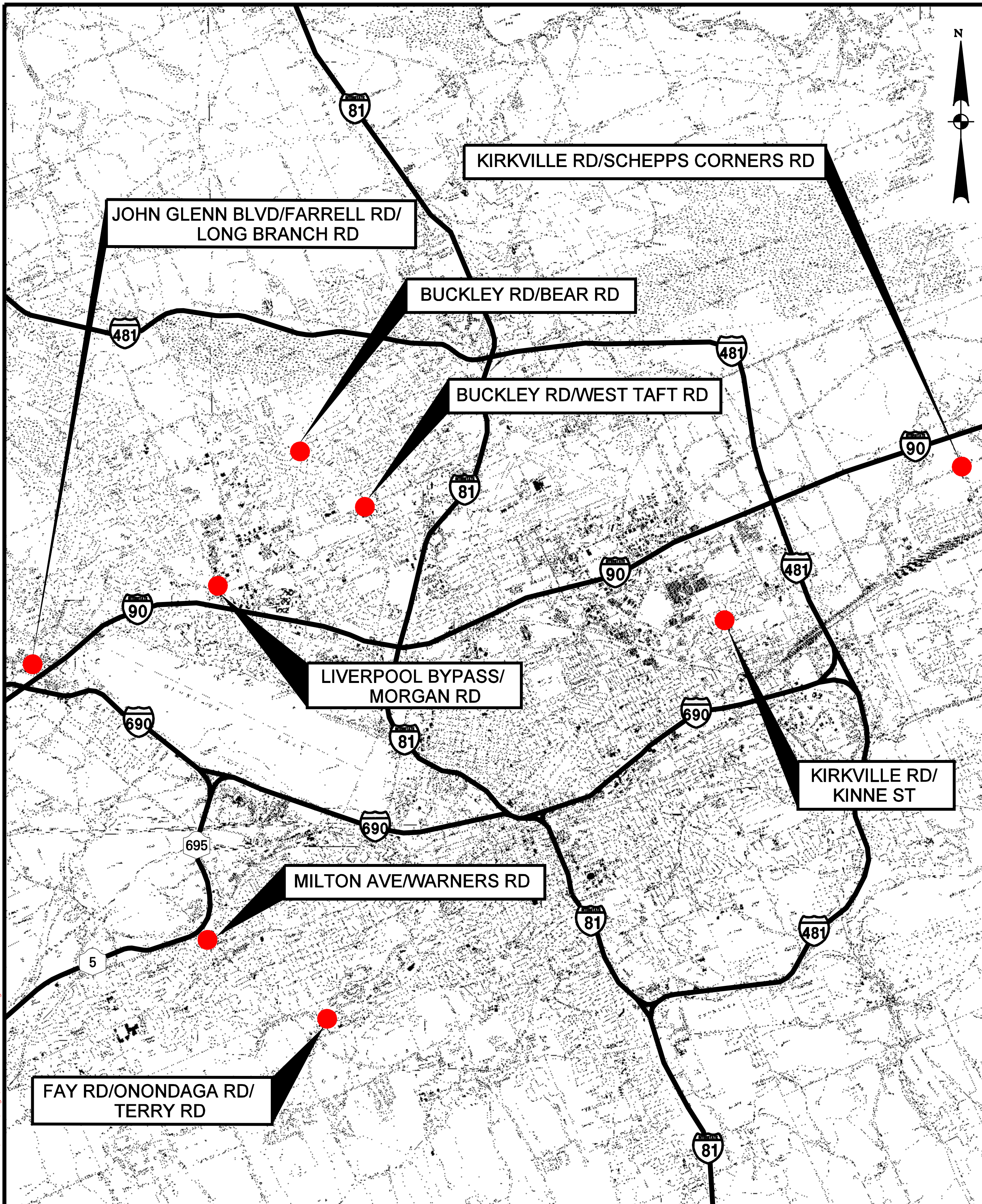
- Kirkville Rd (CR 53)/Kinne St (CR 86)
- Kirkville Rd (CR 53)/Schepps Corners Rd (CR 54)
- Liverpool Bypass (CR 88)/Morgan Rd (CR 47)
- Buckley Rd (CR 161)/Bear Rd (CR 191)
- Buckley Rd (CR 161)/West Taft Rd (CR 48)
- Fay Rd (CR 39)/Onondaga Rd (CR 240)/Terry Rd (CR 75)
- Milton Ave (CR 190)/Warners Rd (NY Rt. 173, CR 63)
- John Glenn Blvd (CR 81)/Long Branch Rd (CR 35)/Farrell Rd (Town)

B. Purpose and Methodology

The purpose of this study was to update intersection signal timings in order to maximize intersection capacity, reduce driver delays, reduce vehicle emissions, and improve the overall efficiency of traffic operations for the motoring public.

In order to accomplish this task, traffic count data, signal timing parameters, and intersection geometry was provided by the Syracuse Metropolitan Transportation Council (SMTC) and the Onondaga County Department of Transportation (OCDOT) to evaluate the current performance of the intersections. Adjustments in signal timings, off-sets, detection, and other parameters were made to improve intersection performance. Once adjustments were identified, changes to the field equipment could be made to implement improvements. Some adjustments, like converting from a leading protected left turn arrow to a lagging arrow will be easily noticed, while others, such as vehicle detection modifications or minor changes in the green time allocation, may not be realized by drivers.

Traffic simulation models of each intersection were developed using the Synchro 7 program. Existing traffic operations were documented and summarized and then optimization of the signals was performed. The changes in the signal timing parameters and the resulting performance changes were then documented to identify the net benefits for the actions.



PROJECT LOCATION

TRAFFIC SIGNAL OPTIMIZATION
ONONDAGA COUNTY
SYRACUSE, NEW YORK

CME
CREIGHTON MANNING ENGINEERING, LLP

PROJECT: 09-094d	DATE: 4/10	FIGURE: 1

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CHAPTER II

ANALYSIS

Traffic volume data, signal timings, intersection sketches, and photos of the study area intersections were gathered from data provided by the OCDOT and the SMTC. This information was used to create existing condition models of each intersection, which were then analyzed to determine their existing performance criteria. With the existing levels of service (LOS) established as the baseline condition, the signal was then optimized. The LOS definitions and a glossary of terms are included in Appendix A.

To maximize the efficiency and performance of each intersection, the traffic volumes for each peak hour were evaluated using a variety of cycle lengths and timing splits. In some cases, the optimized cycle lengths resulted in each signal phase operating at its maximum green time during each cycle of the peak hour. Given that traffic volumes will vary throughout the course of the peak hour, consideration was given to adjusting the cycle length to longer cycles, allowing the signal more flexibility to alter timings as traffic conditions warrant. For example, during low levels of traffic, the controller can reduce the cycle length and serve different approaches quicker. This is particularly useful during off-peak periods. During higher levels of traffic, most notably during peak hours, the cycle length can increase to provide longer green times on approaches that have higher volumes of traffic.

Changes to the existing timings, detection, or parameters such as minimums, maximums, recalls, clearance intervals, and vehicle extensions, are presented in this chapter along with the resulting intersection performance. Changes to these parameters are based on the Onondaga County Department of Transportation's *Traffic Signal Timing Standards* and the *Traffic Signal Timing Manual*, published by the Institute of Transportation Engineers (ITE), 2009. Appendix B includes detailed sketches, photos, controller settings, signal timings/splits, and level of service reports for each intersection.

A. Kirkville Road (CR 53)/Kinne Street (CR 86)

This four-leg intersection operates under a three-phase traffic signal with a 70-second cycle length. No recall is set, such that any phase can be skipped if no calls are placed on an approach. The eastbound Exeter Street approach and westbound Kirkville Road approach provide separate left-turn lanes with presence detection and shared through/right-turn lanes with point detection. The northbound and southbound approaches of Kinne Street each provide a single lane for shared turning movements with presence detection. The posted speed limit on Kirkville Road is 30 mph on the eastbound approach, 40 mph on the westbound approach and 35 mph on Kinne Street. Sidewalks exist on the northwest and southwest corners, but no crosswalks or pedestrian controls are provided. Table II.A.1 summarizes the detailed levels of service for existing and proposed conditions.

Table II.A.1 – Kirkville Road/Kinne Street LOS Summary

Intersection		AM Peak Hour		PM Peak Hour	
		Existing	Proposed	Existing	Proposed
Kirkville Rd/Kinne St					
Exeter St EB	L	B (16)	B (15)	B (14)	C (24)
	TR	C (20)	B (19)	C (30)	D (48)
Kirkville Rd WB	L	A (8)	A (8)	B (11)	C (33)
	TR	B (14)	B (14)	B (14)	C (25)
Kinne St NB	LTR	C (20)	C (21)	D (38)	B (20)
Kinne St SB	LTR	B (16)	B (16)	F (304)	D (49)
Overall		B (16)	B (16)	F (117)	D (37)

Key: NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y) = Level of Service (Delay, seconds per vehicle)

The intersection currently operates at good levels of service with low delays during the AM peak hour. During the PM peak hour, all approaches operate at LOS D or better with the exception of the southbound approach which operates at LOS F with considerable delays. However, by increasing the cycle length and the northbound/southbound green time, the southbound level of service is improved to LOS D and delays reduced by approximately 255 seconds.

To improve operations, the yellow/all-red clearance, minimum greens, and vehicle extension were modified. The signal was optimized using Synchro which resulted in a 55-second cycle length during the AM peak hour and a 90-second cycle length during the PM peak hour. The AM peak hour was adjusted to 60 seconds while the PM peak hour was adjusted to 100 seconds to minimize vehicle delays and minimize the volume to capacity (v/c) ratio. During the AM peak hour the intersection will operate at the 60-second cycle maximum during the higher percentiles of traffic and shorter cycle lengths during lower percentiles of traffic. During the PM peak hour, all phases will generally operate at their maximum with the exception of very low percentiles of traffic. Table II.A.2 summarizes the suggested changes in the signal timing parameters.

Table II.A.2 – Kirkville Road/Kinne Street Parameter Summary

Parameter	Existing	Proposed
Detection	NB, SB, EB, WB	No Change
Recall	None	No Change
Minimum Green	15-sec throughs; 13-sec lefts	10-sec throughs; 5-sec lefts
Yellow/All Red:	3/2-sec	3.5/2-sec for 35 mph speed
Vehicle Extension	3-sec	1-sec EB/WB lefts & NB/SB throughs[1]; 3.5-sec EB/WB through/right[2]
Cycle Length	70-sec	60-sec AM; 100-sec PM

B. Kirkville Road (CR53)/Schepps Corners Road (CR 54)

This four-leg intersection operates under a two-phase traffic signal with a 57-second cycle length. The eastbound and westbound approaches of Kirkville Road provide single lanes for shared turning movements with no known vehicle detection[3], operating with a maximum recall. The northbound and southbound approaches of Schepps Corners Road each provide a single lane for shared turning movements with presence detection presumed, given that no recall has been set, allowing the northbound and southbound phases to be skipped if no vehicles are detected. The posted speed limit on Kirkville Road is 45 mph and 30 mph on Schepps Corners Road. No sidewalks, crosswalks, or pedestrian controls are provided. Table II.B.1 summarizes the detailed levels of service for existing and proposed conditions.

Table II.B.1 – Kirkville Road/Schepps Corners Rd LOS Summary

Intersection		AM Peak Hour		PM Peak Hour	
		Existing	Proposed	Existing	Proposed
Kirkville Rd/Schepps Corners Rd					
Kirkville Rd EB	LTR	A (6)	A (6)	B (13)	B (10)
Kirkville Rd WB	LTR	A (9)	A (8)	A (8)	A (6)
Schepps Corners Rd NB	LTR	B (18)	B (11)	B (15)	B (11)
Schepps Corners Rd SB	LTR	B (16)	B (10)	B (14)	B (10)
Overall		B (11)	A (9)	B (12)	A (10)

Key: NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y) = Level of Service (Delay, seconds per vehicle)

The intersection currently operates at good levels of service with low delays. However, drivers on the Schepps Corners Road must wait for the Kirkville Road phase to finish even if no cars are present, given that it is set to the maximum recall. With the addition of detection on Kirkville Road and a minimum recall, drivers on Schepps Corners Road will be served more quickly when gaps appear in the Kirkville Road traffic. This will be particularly evident during off-peak hours.

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[1] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 30-35 mph.
[2] Max allowable headway = 3 sec, detection zone = 6 feet (placed 140 feet from the intersection), approach speed = 30-35 mph.
[3] No signal equipment diagram was available and controller parameters suggest detection is not provided.

To improve operations, presence detection was added to Kirkville Road and the yellow/all-red clearance, minimum greens, and vehicle extension were modified. The signal was optimized using Synchro which resulted in a 40 to 45-second cycle length. Review of the actuated green times indicated that each approach would operate at the maximum allowable split for all levels of traffic at this cycle length. Therefore, the cycle length was increased to 60 seconds during the AM and PM peak hours, allowing the intersection to operate at that maximum cycle length during the higher percentiles of traffic, and shorter cycle lengths during lower percentiles of traffic. Table II.B.2 summarizes the suggested changes in the signal timing parameters.

Table II.B.2 – Kirkville Road/Schepps Corners Rd Parameter Summary

Parameter	Existing	Proposed
Detection	NB/SB only	Presence added to EB/WB
Recall	Max EB/WB; none NB/SB	Min EB/WB
Minimum Green	20-sec EB/WB;10-sec NB/SB	10-sec on all approaches
Yellow/All Red:	3/3-sec	4/1-sec for 45 mph speed
Vehicle Extension	3-sec	1.5-sec all approaches[4]
Cycle Length	57-sec	60-sec AM & PM

C. Liverpool Bypass (CR88)/Morgan Road (CR 47)

This four-leg intersection operates under a four phase traffic signal with a 120-second cycle length and split phasing on the eastbound and westbound approaches. The eastbound approach of Liverpool Bypass and the westbound approach of Crown Road provide a single lane for shared turning movements with point detection presumed[5], and no recall. The northbound and southbound approaches of Morgan Road each provide a separate left turn lane, with presence detection presumed[5], and a shared through/right turn lane with no detection presumed[5] and a maximum recall. The posted speed limit on Liverpool Bypass and Crown Road is 30 mph and 35 mph on Morgan Road. No sidewalks, crosswalks, or pedestrian controls are provided. Table II.C.1 summarizes the detailed levels of service for existing and proposed conditions.

[4] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 30 mph.
[5] No signal equipment diagram was available and controller parameters suggested the described detection.

Table II.C.1 – Liverpool Bypass/Morgan Rd LOS Summary

Intersection		AM Peak Hour		PM Peak Hour		Sat Peak Hour	
		Existing	Proposed	Existing	Proposed	Existing	Proposed
Liverpool Bypass/Morgan Rd							
Liverpool Bypass EB	LTR	D (48)	C (23)	D (42)	C (33)	C (30)	B (14)
Crown Rd WB	LTR	D (50)	B (15)	D (43)	C (21)	C (34)	B (14)
Morgan Rd NB	L	C (22)	B (11)	B (10)	A (5)	A (6)	A (5)
	TR	D (40)	C (21)	C (29)	B (14)	A (10)	A (8)
Morgan Rd SB	L	C (21)	B (11)	B (18)	A (10)	A (9)	A (6)
	TR	D (38)	C (20)	C (21)	B (11)	B (12)	A (8)
Overall		D (40)	C (20)	C (27)	B (15)	B (15)	A (9)

Key: NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y) = Level of Service (Delay, seconds per vehicle)

During the AM and PM peak hours, all approaches operate at LOS D or better with overall intersection delays of 40 seconds or less. The intersection currently operates at good levels of service with low delays during the Saturday peak hour. With the addition of detection, removal of the split phasing, and a minimum recall, drivers on Liverpool Bypass and Crown Road will be served more frequently when gaps appear in the Morgan Road traffic. Also, removal of the split phasing reduces the overall cycle length, thereby allowing the signal to serve all approaches more frequently during the peak hour.

To improve operations, point detection was added to the Morgan Road through/right-turn lanes and the minimum greens and vehicle extension were modified. Split phasing was removed from the eastbound and westbound approaches and each phase maintained its existing yellow/all-red clearance. The signal was optimized using Synchro which resulted in a 75-second cycle length during the AM and PM peak hours and a 45-second cycle length during the Saturday peak hour. Review of the actuated green times indicated that each approach would operate at the maximum allowable split for most levels of traffic. Therefore, the cycle length was extended to 90 seconds during the AM and PM peak hours and 55 seconds during the Saturday peak hour, allowing the intersection to change cycle lengths according to traffic demands. Table II.C.2 summarizes the suggested changes in the signal timing parameters.

Table II.C.2 – Liverpool Bypass/Morgan Rd Parameter Summary

Parameter	Existing	Proposed
Detection	NB/SB lefts, EB/WB	Point added to NB/SB throughs
Recall	Max NB/SB; none EB/WB	Min NB/SB
Minimum Green	6-sec EB/WB & lefts;20-sec NB/SB	10-sec WB, NB/SB; 7-sec EB, 5-sec lefts
Yellow/All Red:	3/2-sec	No change
Vehicle Extension	4-sec WB & NB/SB lefts; 3-sec NB/SB throughs; 6-sec EB	1-sec NB/SB lefts[6]; 3.5-sec EB/WB & NB/SB throughs[7]
Cycle Length	120-sec	90-sec AM & PM; 55-sec Sat

D. Buckley Road (CR 161)/Bear Road (CR 191)

This four-leg intersection operates under a four phase traffic signal with a 126-second cycle length. For the purpose of this report, Bear Road will be referred to as having east/west orientation and Buckley Road will be north/south. The eastbound Bear Road approach provides an exclusive left-turn lane and a shared through/right-turn lane while the westbound Bear Road approach provides separate lanes for all three movements. All eastbound and westbound travel lanes operate with presence detection and no recall. The northbound Buckley Road approach provides separate left-turn, through, and right-turn lanes while the southbound approach of Buckley Road provides two exclusive left-turn lanes and a shared through/right-turn lane. The northbound and southbound travel lanes all operate with presence detection and a maximum recall. The posted speed limit on Buckley Road is 35 mph and 40 mph on Bear Road. Sidewalks are not provided at this intersection; however wide shoulders, crosswalks, and pedestrian controls are provided. Table II.D.1 summarizes the detailed levels of service for existing and proposed conditions.

Table II.D.1 – Buckley Rd/Bear Rd LOS Summary

Intersection		AM Peak Hour		PM Peak Hour		Sat Peak Hour	
		Existing	Proposed	Existing	Proposed	Existing	Proposed
Buckley Rd/Bear Rd							
Bear Rd EB	L	D (52)	D (36)	E (56)	D (54)	D (48)	C (30)
	TR	D (40)	C (25)	F (216)	F (92)	D (43)	C (25)
Bear Rd WB	L	F (144)	D (40)	F (469)	F (89)	D (50)	C (30)
	T	D (51)	C (21)	D (38)	B (17)	D (41)	C (24)
	R	B (17)	A (8)	B (18)	A (9)	C (25)	B (13)
Buckley Rd NB	L	D (49)	C (34)	E (56)	E (79)	D (52)	C (31)
	T	C (26)	C (32)	C (30)	D (46)	C (24)	C (22)
	R	C (23)	B (11)	C (32)	C (22)	C (20)	B (11)
Buckley Rd SB	L	D (48)	C (34)	D (52)	E (57)	D (46)	C (28)
	TR	C (27)	C (28)	C (33)	E (74)	B (20)	B (19)
Overall		D (53)	C (26)	F (148)	E (61)	C (33)	C (22)

Key: NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y.Y) = Level of Service (Delay, seconds per vehicle)

[6] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 30-35 mph.
[7] Max allowable headway = 3 sec, detection zone = 6 feet (placed 140 feet from the intersection), approach speed = 30-35 mph.

During the AM peak hour, all movements at the intersection operate at LOS D or better, with the exception of the westbound left-turn movement which operates at LOS F with over two minutes of delay. During the PM peak hour, this intersection operates at an overall LOS F with considerable delay due to the eastbound/westbound approaches and northbound left-turn movement. The intersection currently operates at LOS D or better, with overall intersection delay less than 35 seconds during the Saturday peak hour. With the addition of an overlapping northbound right-turn phase, minimum recall on the eastbound/westbound approaches, and no recall on the northbound/southbound approaches, drivers on Bear Road will be served more frequently when gaps appear in the Buckley Road traffic.

To improve operations, a northbound right-turn overlap phase was added, along with the modification of the yellow/all-red clearance, minimum greens, and vehicle extension. The signal was optimized using Synchro which resulted in an 80-second cycle length during the AM peak hour, a 130-second cycle length during the PM peak hour, and a 75-second cycle length during the Saturday peak hour. Review of the actuated green times indicated that each approach would operate at the maximum allowable split for most levels of traffic. Therefore, the cycle length was extended to 90 seconds during the AM and Saturday peak hours, allowing the controller to change cycle lengths in response to traffic demand. The cycle length during the PM peak hour was reduced to 120 seconds to comply with the OCDOT's *Traffic Signal Timing Parameters*. The shorter cycle length continues to result in poor levels of service during the PM peak hour; however, overall operations improve from LOS F with over two minutes of delay to LOS E with approximately one minute of delay. Geometry improvements are likely necessary to further improve operations at this intersection. Based on the traffic volumes, Bear Road lends itself to being considered the major street and Buckley Road being the minor street. Therefore, the recalls were set to reflect such. Table II.D.2 summarizes the suggested changes in the signal timing parameters.

Table II.D.2 – Buckley Rd/Bear Rd Parameter Summary

Parameter	Existing	Proposed
Detection	NB, SB, EB, WB	No change
Recall	Max NB/SB; none EB/WB	None NB/SB; Min EB/WB
Minimum Green	4.5-sec lefts; 7-sec EB/WB; 15-sec NB/SB	5-sec lefts; 10-sec throughs
Yellow/All Red:	4/2-sec	3.5/2.5-sec for 35-40 mph speed
Vehicle Extension	3-sec	1.5-sec all approaches[8]
Cycle Length	126-sec	90-sec AM & Sat; 120-sec PM

E. Buckley Road (CR 161)/West Taft Road (CR 48)

This four-leg intersection operates under a four phase traffic signal with a 144-second cycle length. The eastbound West Taft Road approach provides a left-turn lane, two through lanes, and a right-turn lane while the westbound West Taft Road

[8] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 35-40 mph.

approach provides a left-turn lane, a through lane, and a shared through/right-turn lane. All eastbound and westbound travel lanes operate with presence detection and minimum recall. The northbound and southbound Buckley Road approaches each provide a left-turn lane, through lane, and a shared through/right-turn lane, all of which operate with presence detection and no recall. The posted speed limit on Buckley Road is 35 mph and 40 mph on West Taft Road. No sidewalks, crosswalks, or pedestrian controls are provided at this intersection. Table II.E.1 summarizes the detailed levels of service for existing and proposed conditions.

Table II.E.1 – Buckley Rd/West Taft Rd LOS Summary

Intersection		AM Peak Hour		PM Peak Hour		Sat Peak Hour	
		Existing	Proposed	Existing	Proposed	Existing	Proposed
Buckley Rd/West Taft Rd							
West Taft Rd EB	L	D (47)	D (39)	D (51)	D (41)	D (38)	C (28)
	T	C (35)	C (27)	D (44)	C (31)	C (28)	C (22)
	R	C (20)	B (17)	B (18)	B (12)	B (14)	B (12)
West Taft Rd WB	L	D (44)	D (37)	D (54)	D (41)	C (32)	C (27)
	TR	C (24)	B (18)	D (39)	C (26)	C (23)	B (20)
Buckley Rd NB	L	D (42)	D (35)	E (56)	D (40)	C (31)	C (25)
	TR	D (39)	C (31)	D (46)	D (40)	C (31)	B (19)
Buckley Rd SB	L	D (42)	D (37)	E (55)	D (45)	C (31)	C (30)
	TR	D (39)	C (30)	D (39)	C (32)	C (31)	C (21)
Overall		C (34)	C (27)	D (44)	C (34)	C (28)	C (21)

Key: NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y.Y) = Level of Service (Delay, seconds per vehicle)

All movements at this intersection currently operate at LOS D or better during all three peak hours, with the exception of the northbound/southbound left-turn movements during the PM peak hour, which operate at LOS E. After optimization of the traffic signal, overall intersection delay will decrease by 5 to 10 seconds.

To improve operations, the yellow/all-red clearance, minimum greens, and vehicle extension were modified. The signal was optimized using Synchro which resulted in a 60-second cycle length during the AM peak hour, a 65-second cycle length during the PM peak hour, and a 55-second cycle length during the Saturday peak hour. Review of the actuated green times indicated that each approach would operate at the maximum allowable split for most levels of traffic. Therefore, the cycle length was extended to 90 seconds during the AM peak hour, 100 seconds during the PM peak hour, and 70 seconds during the Saturday peak hour allowing the controller to respond to changing traffic demands. Table II.E.2 summarizes the suggested changes in the signal timing parameters.

Table II.E.2 – Buckley Rd/West Taft Rd Parameter Summary

Parameter	Existing	Proposed
Detection	NB, SB, EB, WB	No change
Recall	Min EB/WB; none NB/SB	No change
Minimum Green	5-sec lefts; 8-sec NB/SB; 10-sec EB/WB	5-sec lefts; 10-sec throughs
Yellow/All Red:	4/2-sec	3.5/2-sec for 35-40 mph speed
Vehicle Extension	4-sec	1.5-sec all approaches[9]
Cycle Length	126-sec	90-sec AM; 100-sec PM; 70-sec Sat

F. Fay Road (CR 39)/Onondaga Blvd (CR 240)/Terry Road (CR 75)

This five-leg intersection operates under a four phase traffic signal with a 119-second cycle length. The eastbound and westbound Fay Road approaches each provide an exclusive left-turn lane and a shared through/right-turn lane with presence detection on all movements and no recall. The northbound Onondaga Boulevard approach provides a left-turn lane, a through lane, and a shared through/right-turn lane, while the southbound approach of Onondaga Boulevard provides an exclusive left-turn lane and a shared through/right-turn lane. The northbound and southbound travel lanes all operate with presence detection and a maximum recall. The south-westbound Terry Road approach provides a single lane for shared turning movements with presence detection and no recall. The posted speed limit on Fay Road and Onondaga Boulevard is 35 mph and 30 mph on Terry Road. No sidewalks, crosswalks, or pedestrian controls are provided at this intersection. Table II.F.1 summarizes the detailed levels of service for existing and proposed conditions.

Table II.F.1 – Fay Rd/Onondaga Blvd/Terry Rd LOS Summary

Intersection		AM Peak Hour		PM Peak Hour		Sat Peak Hour	
		Existing	Proposed	Existing	Proposed	Existing	Proposed
Fay Rd/Onondaga Rd/Terry Rd							
Fay Rd EB	L	D (39)	C (30)	F (109)	E (57)	D (36)	C (24)
	TR	E (69)	D (49)	D (37)	C (28)	D (35)	C (25)
Fay Rd WB	L	C (33)	C (26)	C (32)	C (24)	C (30)	C (21)
	TR	D (36)	C (29)	D (52)	D (41)	D (37)	C (26)
Onondaga Rd NB	L	E (56)	D (47)	D (50)	D (39)	D (43)	C (29)
	TR	D (37)	D (38)	D (35)	C (31)	C (28)	C (22)
Onondaga Rd SB	L	D (53)	D (47)	D (52)	D (40)	D (43)	C (30)
	TR	C (34)	D (35)	D (39)	C (34)	C (30)	C (24)
Terry Rd SWB	LTR	F (93)	D (50)	D (54)	D (42)	D (39)	C (30)
Overall		E (60)	D (43)	D (48)	D (37)	C (35)	C (26)

Key: NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y.Y) = Level of Service (Delay, seconds per vehicle)

During the AM peak hour, the intersection operates at an overall LOS E, with various movements operating at LOS E/F. The intersection operates at LOS D during the PM peak hour with the eastbound left-turn movement operating at LOS F with

[9] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 35-40 mph.

almost two minutes of delay. The intersection currently operates at LOS C during the Saturday peak hour.

To improve operations, the yellow/all-red clearance, minimum greens, and vehicle extension were modified. The signal was optimized using Synchro which resulted in a 90-second cycle length during the AM peak hour, a 75-second cycle length during the PM peak hour, and a 60-second cycle length during the Saturday peak hour. Review of the actuated green times indicated that each approach would operate at the maximum allowable split for most levels of traffic. Therefore, the cycle length was extended to 110 seconds during the AM peak hour, 100 seconds during the PM peak hour, and 75 seconds during the Saturday peak hour, allowing the controller to vary the cycle lengths according to traffic demands. Changing the recall to minimum on the northbound/ southbound approaches will allow drivers on Fay Road and Terry Road to be served more frequently when gaps appear in the Onondaga Boulevard traffic. At this particular intersection, it is necessary to use a longer all-red clearance phase because the alignment of the intersection's five legs create longer distances that drivers must travel in order to clear the conflicting approaches. Table II.F.2 summarizes the suggested changes in the signal timing parameters.

Table II.F.2 – Fay Rd/Onondaga Blvd/Terry Rd Parameter Summary

Parameter	Existing	Proposed
Detection	NB, SB, EB, WB	No change
Recall	Max NB/SB; none EB/WB/SWB	Min NB/SB
Minimum Green	8-sec lefts & EB/WB/SWB; 10-sec NB/SB	5-sec lefts; 10-sec throughs
Yellow/All Red:	4/2-sec	3.5/3-sec for 30-35 mph speed
Vehicle Extension	4-sec	1-sec all approaches[10]
Cycle Length	119-sec	110-sec AM; 100-sec PM; 75-sec Sat

G. Milton Avenue (CR 190)/Warners Road (NY 173/CR 63)

This four-leg intersection operates under a four phase traffic signal with a 104-second cycle length. The eastbound approach of Milton Avenue provides a single lane for shared turning movements while the westbound Milton Avenue approach provides an exclusive left-turn lane and a shared through/right-turn lane. The eastbound and westbound travel lanes operate with presence detection and no recall. The northbound and southbound approaches of Warners Road each provide a separate left-turn lane, with presence detection, and a shared through/right-turn lane with no detection and maximum recall. The posted speed limit on Milton Avenue and Warners Road is 30 mph. No sidewalks, crosswalks, or pedestrian controls are provided. Table II.G.1 summarizes the detailed levels of service for existing and proposed conditions.

[10] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 30-35 mph.

Table II.G.1 – Milton Ave/Warners Rd LOS Summary

Intersection		AM Peak Hour		PM Peak Hour		Sat Peak Hour	
		Existing	Proposed	Existing	Proposed	Existing	Proposed
Milton Ave/Warners Rd							
Milton Ave EB	LTR	C (33)	C (21)	D (44)	D (36)	D (44)	D (36)
Milton Ave WB	L	B (18)	B (12)	C (21)	B (18)	C (20)	B (16)
	TR	B (16)	B (11)	B (18)	B (16)	B (17)	B (14)
Warners Rd NB	L	B (15)	B (13)	C (21)	B (17)	B (16)	B (13)
	TR	C (28)	B (19)	D (42)	C (28)	D (41)	C (29)
Warners Rd SB	L	B (16)	B (12)	C (22)	B (17)	C (22)	B (17)
	TR	C (24)	B (17)	D (52)	C (33)	C (31)	C (22)
Overall		C (25)	B (17)	D (37)	C (27)	C (32)	C (24)

Key: S = Signal controlled intersection
NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y.Y) = Level of Service (Delay, seconds per vehicle)

The intersection currently operates at LOS D or better during all three peak hours, with overall intersection delays less than 40 seconds.

To improve operations, point detection was added to the Warners Road through lanes and the yellow/all-red clearance, minimum greens, and vehicle extension were modified. With the addition of detection and a minimum recall, drivers on Milton Avenue will be served more frequently when gaps appear in the Warners Road traffic. The signal was optimized using Synchro which resulted in a 60-second cycle length during the AM and PM peak hours and a 65-second cycle length during the Saturday peak hour. Review of the actuated green times indicated that each approach would operate at the maximum allowable split for most levels of traffic. Therefore, the cycle length was extended to 80 seconds during the AM peak hour, 100 seconds during the PM peak hour, and 90 seconds during the Saturday peak hour, allowing the controller to vary cycle lengths based on traffic demand. Table II.G.2 summarizes the suggested changes in the signal timing parameters.

Table II.G.2 – Milton Ave/Warners Rd Parameter Summary

Parameter	Existing	Proposed
Detection	NB/SB lefts, EB/WB	Point added to NB/SB throughs
Recall	Max NB/SB; none EB/WB	Min NB/SB
Minimum Green	6-sec lefts;7-sec EB/WB, NB/SB	10-sec throughs; 5-sec lefts
Yellow/All Red:	4/2-sec	3/2-sec for 30 mph speed
Vehicle Extension	4-sec	1-sec EB/WB throughs & NB/SB lefts[11]; 3.5-sec NB/SB throughs[12]
Cycle Length	104-sec	80-sec AM; 100-sec PM; 90-sec Sat

[11] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 30 mph.
[12] Max allowable headway = 3 sec, detection zone = 6 feet (placed 140 feet from the intersection), approach speed = 30 mph.

H. John Glenn Blvd (CR 81)/Long Branch Rd (CR 35)/Farrell Rd (Town)

This four-leg intersection operates under a four phase traffic signal with a 108-second cycle length. The eastbound and westbound John Glenn Boulevard approaches each provide an exclusive left-turn lane with presumed presence detection, an exclusive through lane, and a shared through/right-turn lane with no detection and maximum recall. The northbound Long Branch Road approach provides a single lane for shared turning movements with presence detection presumed and no recall. The southbound Farrell Road approach provides a shared left-turn/through lane and an exclusive right-turn lane with presence detection and no recall presumed[13]. The posted speed limit on John Glenn Boulevard is 55 mph and 30 mph on Long Branch Road and Farrell Road. No sidewalks, crosswalks, or pedestrian controls are provided at this intersection. Table II.H.1 summarizes the detailed levels of service for existing and proposed conditions.

Table II.H.1 – John Glenn Blvd/Long Branch Rd/ Farrell Rd LOS Summary

Intersection		AM Peak Hour		PM Peak Hour	
		Existing	Proposed	Existing	Proposed
John Glenn Blvd/Long Branch Rd/ Farrell Rd					
John Glenn Blvd EB	L	D (39)	D (46)	D (42)	D (50)
	TR	D (51)	C (30)	C (32)	C (26)
John Glenn Blvd WB	L	D (40)	D (45)	D (41)	D (48)
	TR	C (23)	B (19)	E (79)	D (37)
Long Branch Rd NB	LTR	C (33)	D (41)	D (38)	D (49)
Farrell Rd SB	LT	D (38)	D (42)	D (47)	D (52)
	R	C (34)	D (39)	C (33)	D (38)
Overall		D (40)	C (28)	E (57)	D (36)

Key: NB, SB, EB, WB = Northbound, Southbound, Eastbound, Westbound intersection approaches
L, T, R = Left-turn, through, and/or right-turn movements
X (Y.Y) = Level of Service (Delay, seconds per vehicle)

All approaches currently operate at LOS D or better during the AM peak hour. During the PM peak hour, all movements operate at LOS D or better with the exception of the westbound through/right-turn movement, which operates at LOS E with more than one minute of delay. However, by adding detection and optimizing the signal the levels of service will improve and overall delays will be reduced by 12 to 20 seconds.

To improve operations, the yellow/all-red clearance, minimum greens, and vehicle extension were modified. The signal was optimized using Synchro which resulted in an 80-second cycle length during the AM peak hour and a 90-second cycle length during the PM peak hour. The AM and PM peak hours were adjusted to 120 seconds to allow the controller to adjust cycle lengths according to traffic demands. Table II.H.2 summarizes the suggested changes in the signal timing parameters.

[13] No signal equipment diagram was available and controller parameters suggested the described detection.

Table II.H.2 – John Glenn Blvd/Long Branch Rd/ Farrell Rd Parameter Summary

Parameter	Existing	Proposed
Detection	EB/WB lefts; NB/SB	Point added to EB/WB throughs
Recall	Max EB/WB	Min EB/WB
Minimum Green	10-sec EB/WB; 8-sec NB/SB & lefts	15-sec EB/WB; 7-sec SB; 10-sec NB; 5-sec lefts
Yellow/All Red:	4/3-sec	4.5/3-sec for 30-55 mph speed
Vehicle Extension	4-sec	1-sec NB/SB[14]; 2.8-sec EB/WB[15]; 1.8-sec EB/WB lefts[16]
Cycle Length	108-sec	120-sec AM & PM

I. Optimization Summary

The recommendations discussed in the preceding sections are intended to develop consistency in the operations of each signal, improve responsiveness, and increase efficiency. The addition of vehicle detection is recommended for many of the intersections. The addition of detection on all approaches enables a signal to respond to changing traffic conditions, which increases the capacity of the intersection. Once vehicle detection is installed, recall settings in the signal controller can be used to create a minimum operating condition that the signal must serve. Beyond that, the controller can respond to the current demand.

Another key component of the recommendations is updating the vehicle extension times to accurately reflect the existing or proposed detection. The vehicle extension adds time to an approach that has already served the initial platoon of traffic with the minimum green, but continues to see additional vehicles arriving on the approach. The traffic signal will not start to “gap out” (i.e. end the current phase) until the vehicle has left the detection zone.

Many of the intersections included in this analysis currently have presence detection (long vehicle detection loops, typically 60 to 70 feet) and a 3 or 4 second vehicle extension time. Depending on the speed of approaching vehicles, this combination of presence detection and a 3 to 4 second vehicle extension will result in the continued extension of the green phase for a dwindling amount of vehicles, which increases the delay for drivers on conflicting approaches waiting for straggling vehicles to pass through the intersection. Therefore, for presence detection, this report generally recommends that the vehicle extension time be reduced to 1.0 to 1.8 seconds to allow the signal to serve all approaches more efficiently and reduce overall delay at the intersection.

In contrast, point detection uses a small detection zone, typically a 6-foot detector loop placed 100 to 200 feet from the intersection, and has a much shorter period of detection as a vehicle passes over it. Point detection requires longer vehicle extensions, since the detector has less time to detect a vehicle approaching the intersection.

[14] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 30 mph.
[15] Max allowable headway = 3 sec, detection zone = 6 feet (placed 100 feet from the intersection), approach speed = 55 mph.
[16] Max allowable headway = 3 sec, detection zone = 70 feet, approach speed = 55 mph.

Therefore, this report generally recommends that the vehicle extension time be reduced to 2.8 to 3.5 seconds for point detection to allow the signal to serve all approaches more efficiently and reduce overall delay at the intersection.

CHAPTER IV
CONCLUSIONS

Based on the results of this Traffic Signal Timing Optimization study, the following recommendations are offered for the study area intersections:

- Kirkville Rd/Kinne St: While this intersection currently operates at acceptable levels of service during the AM peak hour, the southbound approach experiences considerable delays during the PM peak hour. It is recommended that the timing parameters be adjusted as shown in Table II.A.2. These changes will result in a level of service D or better on all approaches during both peak hours and will allow the signal more flexibility to alter the timings as traffic conditions warrant.

- Kirkville Rd/Schepps Corners Rd: All movements at this intersection currently operate at a level of service B or better during the AM and PM peak hours, with approach delays less than 20 seconds. However, it is recommended that presence detection be added to the Kirkville Road approaches with the recall set to minimum. This will allow the signal to serve the Schepps Corners Road approaches more quickly when gaps appear in the Kirkville Road traffic. It is also recommended that the timing parameters be adjusted to that shown in Table II.B.2, which will results in excellent levels of service at this intersection.

- Liverpool Bypass/Morgan Rd: While this intersection currently operates at good levels of service during the AM, PM, and Saturday peak hours, it is recommended that point detection be added to the Morgan Road through/right-turn lanes, with minimum recall, to allow the signal to serve the Liverpool Bypass and Crown Road approaches as gaps appear in the Morgan Road traffic. Removal of the eastbound and westbound split phasing will shorten the overall cycle length allowing the signal to serve all approaches more frequently. Additional timing changes, shown in Table II.C.2, will result in improved intersection operations, with all approaches operating at a level of service C or better.

- Buckley Rd/Bear Rd: All movements at this intersection currently operate at a level of service D or better during the Saturday peak hour. However, during the AM and PM peak hours, several movements operate at LOS E/F, with long delays. It is recommended that an overlapping right-turn phase be added to the northbound approach to provide more green time for the right-turn movement. Additional timing changes will result in improved operations, with overall intersection delays of less than 30 seconds during the AM and Saturday peak hours. During the PM peak hour, this intersection will continue to operate at a poor level of service, even with the recommended timing adjustments. However, the recommended timing adjustments will result in an improvement in overall intersection operations during the PM peak hour from LOS F with over 2 minutes of delay to LOS E with about 1 minute of delay. Additional improvements, such as modifying the intersection geometry, are likely needed to further improve operations during the PM peak hour.

- Buckley Rd/West Taft Rd: While this intersection currently operates at acceptable levels of service during the AM, PM, and Saturday peak hours, recommended signal timing adjustments will result in an overall LOS C during all three peak hours and will allow the signal more flexibility to alter the timings as traffic conditions warrant.

- Fay Rd/Onondaga Rd/Terry Rd: This intersection currently operates at a level of service D or better during the PM and Saturday peak hours. However, the eastbound left-turn movement operates at LOS F with almost two minutes of delay during the PM peak hour. With the recommended timing changes, shown in Table II.F.2, the intersection will operate at an overall level of service D or better for all three peak hours, and the eastbound left-turn movement on Fay Road will improve from LOS F to LOS E during the PM peak hour. All other movements will operate at LOS D or better.

- Milton Ave/Warners Rd: While this intersection currently operates at LOS C/D during the AM, PM, and Saturday peak hours, it is recommended that point detection be added to the Warners Road through lanes with minimum recall. This will allow the signal to serve the Milton Avenue approaches more frequently as gaps appear in the Warners Road traffic. With the recommended timing changes, the intersection will operate at an overall LOS B/C with delays reduced by 8 to10 seconds.

- John Glenn Blvd/Long Branch Rd/Farrell Rd: This intersection currently operates at LOS D during the AM peak hour and LOS E during the PM peak hour. It is recommended that point detection be added to the eastbound/westbound through movements with a minimum recall. With the additional signal timing changes, this intersection will operate at an overall level of service C/D, with delays decreasing by 10 to 20 seconds.

Overall, most intersections can achieve better levels of service and reduced delays with the addition of vehicle detection, updated signal timings, and modified controller parameters. In some instances, these improvements will reduce delays on most movements, although some movements may still operate at the LOS E/F range. Additional physical improvements may be necessary to further reduce delays and congestion.

These recommendations are made solely on the basis of the information provided. Other engineering factors, such as sight distances, accident history, presumed detector locations, and previous experiences at these intersections need to be considered in the implementation or modification of these recommendations.