Delaware Valley Regional Planning Commission

2005 Regional Congestion and Accident Mitigation Program

September 2005

Created in 1965, the Delaware Valley Regional Planning Commission (DVRPC) is an interstate, intercounty and intercity agency that provides continuing, comprehensive and coordinated planning to shape a vision for the future growth of the Delaware Valley region. The region includes Bucks, Chester, Delaware, and Montgomery counties, as well as the City of Philadelphia, in Pennsylvania; and Burlington, Camden, Gloucester and Mercer counties in New Jersey. DVRPC provides technical assistance and services; conducts high-priority studies that respond to the requests and demands of member state and local governments; fosters cooperation among various constituents to forge a consensus on diverse regional issues; determines and meets the needs of the private sector; and practices public outreach efforts to promote two-way communication and public awareness of regional issues and the Commission.

Our logo is adapted from the official DVRPC seal, and is designed as a stylized image of the Delaware Valley. The outer ring symbolizes the region as a whole, while the diagonal bar signifies the Delaware River. The two adjoining crescents represent the Commonwealth of Pennsylvania and the State of New Jersey.

DVRPC is funded by a variety of funding sources including federal grants from the U.S. Department of Transportation's Federal Highway Administration (FHWA) and Federal Transit Administration (FTA), the Pennsylvania and New Jersey departments of transportation, as well as by DVRPC's state and local member governments. The authors, however, are solely responsible for its findings and conclusions, which may not represent the official views or policies of the funding agencies.

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DVRPC Staff took all photographs in the Spring of 2005.

1 INTRODUCTION

The Regional Congestion and Accident Mitigation Program (CAMP) is a program of the Delaware Valley Regional Planning Commission (DVRPC) to support the local counties and municipalities in both New Jersey and Pennsylvania in addressing the safety and mobility issues along their arterial road networks. Unlike a typical corridor study that examines a larger geographic area, the intent of the CAMP program is to examine individual intersections or specific problem sites. Assuring the efficient operation of the intersections is becoming an increasingly important issue as municipalities attempt to maximize vehicle roadway capacity to serve the growing demand for travel. These locations may experience high levels of congestion or have a high number of accidents. These accidents may not only result in injuries, but also add to the congestion and deficiency of the intersection. This report examines these congested areas with the goal of identifying potential cost-effective improvement strategies, which would improve the safety and mobility of goods and people.

DVRPC solicited input from each of the local county planning commissions in both Pennsylvania and New Jersey for potential problem locations. Working with the local county planning commissions, DVRPC selected six locations to study. Each of the locations is distinct from one another and has its own particular set of issues and problems. For example, three locations comprise of two intersecting streets at a signalized intersection. The differences are that they reside in either a rural, suburban, or urban setting. Other locations involve adjacent intersections where the operation of one affects the other. With each location being unique, there is no one cure-all solution. In fact, for each location, a combination of strategies may need to be implemented to have an impact on improving safety and reducing congestion.

The six study locations evaluated in this effort include three locations in Pennsylvania and three in New Jersey. They are as follows:

Pennsylvania:

West Fallowfield Township, Chester County
PA 41 (Gap Newport Pike) at PA 10 (Limestone Road)

Ridley Township Delaware County
PA 420 (Kedron Avenue) at MacDade Boulevard
PA 420 (Kedron Avenue) at Fourth Avenue/Academy Avenue

City of Philadelphia
34th Street at Grays Ferry Avenue

New Jersey:

Cinnaminson Township, Burlington County
Riverton Road (CR 603) at Branch Pike (CR 606)
Riverton Road (CR 603) at Parry Road
Branch Pike (CR 606) at Parry Road

<u>Voorhees Township, Camden County</u>
Haddonfield Road (CR 561) at White Horse Road (CR 673)

<u>West Windsor Township, Mercer County</u>
Old Trenton Road (CR 535) at Robbinsville-Edinburg Road (CR 526)
Old Trenton Road at Windsor Road (CR 641)/Edinburg Road (CR 526)

At the onset of this effort, multi-agency field views were conducted at each location with representatives from the local municipalities and county agencies. During these preliminary field views, a base set of problems was identified for further review. DVRPC staff conducted follow-up field views to better define the existing conditions, observe the operating conditions, and refine the problem identification. Subsequently, technical analysis was performed to quantify the identified transportation problem areas, formulate practical potential improvement scenarios, and document solutions.

The report is organized into six separate sections: one for each of the study locations. Within each section, the report is structured in a similar format that consists of: (1) Location Description, (2) Existing Conditions, (3) Opportunities and Constraints, (4) Potential Improvement Scenarios, and (5) Recommendations.

(1) The location description section provides an account of each location and examines the study area in terms of regional setting. This includes a general depiction of the local area surroundings, lane configuration and adjacent land uses.

(2) The existing conditions present additional background information for each site. For each location, turning movement counts were collected during the peak periods in 15-minute increments to determine the peak hour traffic volumes. Traffic signal timing and operation plans for each intersection were collected from either the local municipalities or the Pennsylvania Department of Transportation. At each location a crash analysis and a level of service (LOS) analysis was conducted.

The crash analysis was used to substantiate problems presented during the municipal field views and identify any probable causes and potential improvements. For each location, reportable crash records for at least a three-year period were collected from either the local municipalities or Pennsylvania or New Jersey Departments of Transportation. Reportable crashes typically involve an injury, fatality and/or significant property damage. In addition, one of the vehicles in the crash may be damaged to the point where it must be towed. In some of the locations, a significant number of non-reportable crashes occurred and data was collected on these crashes. Although, a non-reportable crash is one where there is no injury to the occupant(s) of the vehicle(s), and the vehicles involved do not need to be towed, the crash may have negative effects on the operation of the intersection.

For each location, a collision diagram was developed from the collected crash records. The purpose of the collision diagram is to pictorially represent different types of crashes that have occurred and is useful in identifying accident patterns and trends. With regards to PennDOT accident data, this is safety study and is confidential pursuant to 75 Pa. C.S. 3374 and 23 U.S.C. 409 and may not be disclosed or used in litigation without written permission from PennDOT.

The level of service analysis (LOS) is a common tool for assessment of transportation facilities and is used extensively in this report. For each location, the existing conditions and potential improvement scenarios LOS is evaluated. The concept of LOS, when applied to the performance of an intersection, has a precise meaning: it refers to the average delay experienced by a vehicle traveling through the intersection. The measure of effectiveness for signalized intersection LOS is the average control delay per vehicle. At each intersection, delay was estimated for each lane group and aggregated for each approach and for the intersection as a whole. This methodology does not take into account the potential impact of downstream congestion on intersection operation. **Table 1** shows level of service categories, from A to F, with associated criteria for each category.

Table 1 Level of Service (LOS) Designations and Associated Delays

Level of Service	Control Delay per Vehicle (seconds/vehicle)
A (Desirable)	< = 10
B (Desirable)	> 10 - 20
C (Desirable)	> 20 - 35
D (Acceptable)	> 35 - 55
E (Undesirable)	> 55 - 80
F (Unsatisfactory)	> 80

A general description taken from the *2000 Highway Capacity Manual* of the different LOS follows:

> *LOS A* – describes primarily free-flow operations at average travel speeds. There is very low vehicle delay. Vehicles are completely unimpeded in their ability to maneuver within the traffic stream.

LOS B - describes reasonably unimpeded operations at average travel speeds. Traffic still moves freely with few delays.

LOS C - describes stable operations. However, the ability to maneuver and change lanes in mid-block locations may be more restricted than at LOS B. Longer queues, adverse signal coordination, or both may contribute to lower average travel speeds and higher delays.

LOS D - borders on a range in which small increases in flow may cause substantial increases in delay and decreases in travel speeds. Longer delays may result from some combination of adverse signal progression, long cycle lengths, or high volumes.

LOS E - is characterized by significant delays. Delay may be great and up to several cycles. Such operations are caused by a combination of adverse progression, high signal density, high volumes, extensive delays at critical intersections, and inappropriate signal timing.

LOS F - is characterized by traffic flow at extremely low speeds. There are excessive delays that cause reduced capacity. Intersection congestion is likely at critical signalized locations, with high delays, high volumes, and extensive queuing.

For each of the locations, a review of the existing conditions and the various improvement scenarios was conducted using either McTrans Highway Capacity Software (*HCS*) or Synchro Software. The turning movement counts and traffic volume data, along with data from the traffic signal timing and operation plans, were analyzed using the software to determine the LOS.

HCS software implements the procedures defined in the Highway Capacity Manual (HCM 2000) for analyzing capacity and determining level of service (LOS) for signalized intersections. Synchro is a software application for optimizing traffic signal timing and performing capacity analysis. The software optimizes splits, offsets, and cycle lengths for individual intersections, arterials, or a complete network. Synchro performs capacity analysis using both the Intersection Capacity Utilization (ICU) and Highway Capacity Manual methods to evaluate signalized and unsignalized intersections and determine a level of service.

Both software methods take heavy vehicle percentages into consideration, which will have a slight effect on intersection level of service and volume-to-capacity ratios. Typically, both of the softwares assign a default value of 2 percent of the total traffic volumes to be heavy trucks, and unless otherwise noted, the typical 2 percent default value was used. However, in some of the study locations, local officials stated that heavy truck volume was a problem, Therefore, classification counts were collected to determine percentages.

(3) The opportunities and constraints section discusses specific issues or problems that may effect any potential improvements that have been identified. A typical issue may be the restriction of right-of-way expansion to increase capacity. Expansion may be cost prohibitive due to encroaching land uses or nearby bridge widths.

(4) Upon review of the identified problems, discussions with local stakeholders, and the existing conditions analysis, potential improvement scenarios were developed. These concepts are aimed at addressing operational and safety problems. Typical improvement scenarios range from optimizing signal timing, signal coordination, adding turning lanes and intersection redesign/reconstruction. These improvement concepts are aimed at addressing operational and safety problems of each location. For each scenario, an additional level of service analysis is conducted and compared to the existing LOS analysis. This process helps to determine if the scenario is implemented; whether there are any improvements to the efficiency and operations of the intersection.

(5) Based upon the LOS analysis, recommendations have been established on their ability to correct existing or potential problems or deficiencies. The potential improvement scenario concepts presented in this document have been categorized as short term, mid-range or long term. Short-term improvement recommendations typically considered a lower cost operational/safety improvement that can be completed with little lead time and no additional major studies. Long-range improvement concepts should only be pursued if the implemented set of short and mid-range improvements are evaluated and determined to be ineffective. These improvements, such as additional signing, resurfacing or enhancing pavement markings, may be completed primarily through maintenance activities. A mid-range improvement may require additional costs with regard to signal coordination and pedestrian enhancements. A long-term improvement may have a higher capital cost, and require the acquisition of right-of-way and construction of new infrastructure.

For each section of this document there is a corresponding Appendix that contains the detailed technical data documentation for crash records, turning movement counts and level of service analysis.

Delaware Valley Regional Planning Commission

CHESTER

2 GAP NEWPORT PIKE (PA 41) AND LIMESTONE ROAD (PA 10)

West Fallowfield Township, Chester County

Location Description

The intersection is located in a primarily rural area of southwest Chester County in West Fallowfield Township in the village of Cochranville. Gap Newport Pike (PA 41) is classified as a major arterial street that serves southwest Chester County from the Delaware state line to Lancaster County. It is also becoming more heavily traveled as it functions as an east-west corridor linking Wilmington and its port to areas to the west like Lancaster, York and Harrisburg. Agricultural lands dominate this section of PA 41 in the study area. Limestone Road (PA 10) is classified as a minor arterial. It runs through rural, agricultural, and natural areas. The study location is shown on **Map 1.**

Looking eastbound along Gap Newport Pike (PA 41)

Improvements to this location are a priority to the county because of increased population growth and development that has created additional traffic throughout the region. Also, to the south of this location, PennDOT is currently involved in a large-scale effort to study PA 41. They are developing solutions for a 9.5-mile section of PA 41 leading into Delaware. This is outside of the study area for this project, but any improvements to that stretch of PA 41 may have a future impact on the traffic in this location.

Gap Newport Pike (PA 41) and Limestone Rd. (PA 10)

West Fallowfield Township, Chester County, PA

2005 Regional Congestion and Accident Mitigation Program	Map 1	0 25 50 100 Feet N Delaware Valley Regional Planning Commission August 2005

Date of Aerial Photography: Spring, 2000

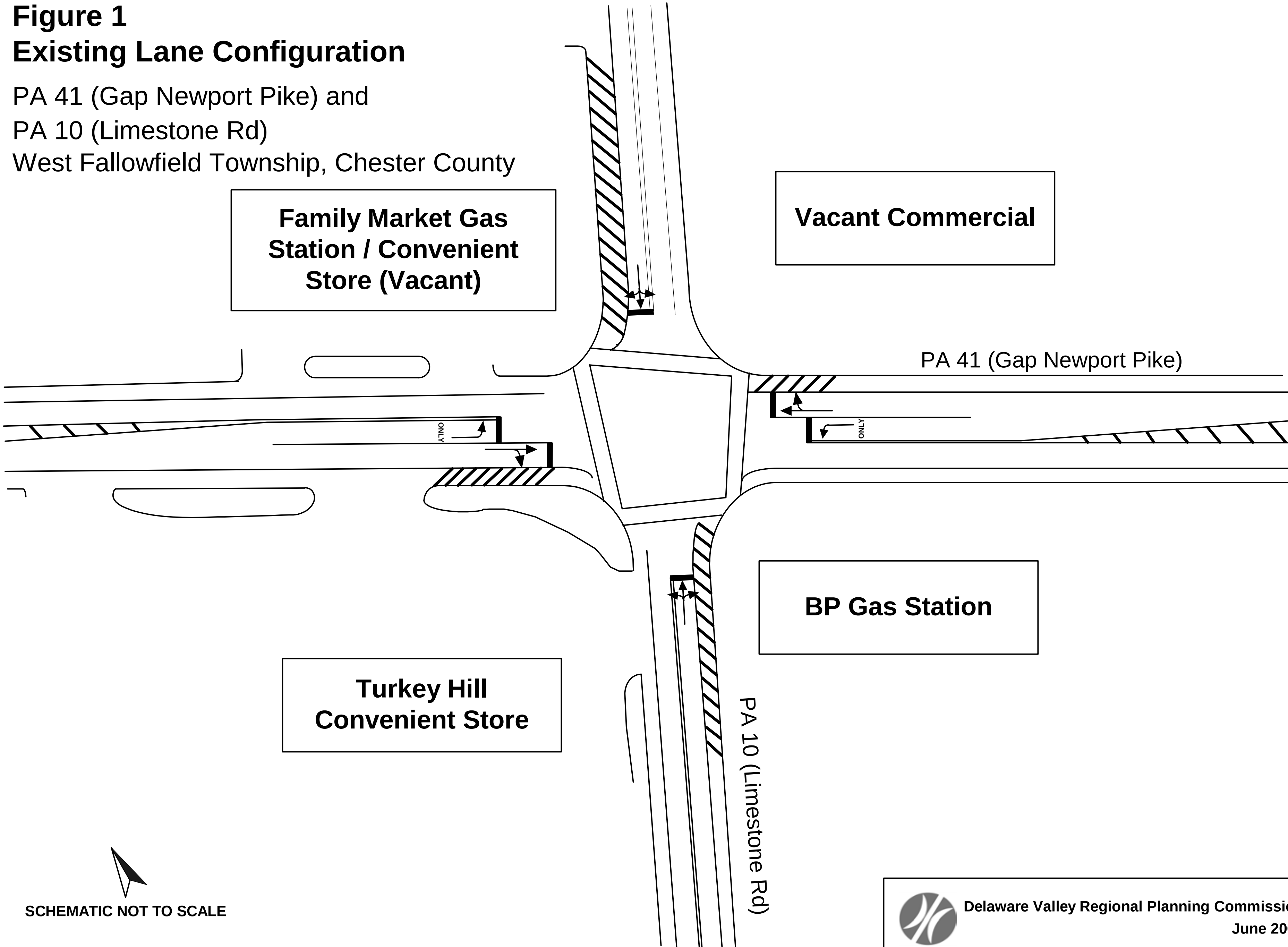
The intersection of Gap Newport Pike (PA 41) and Limestone Road (PA 10) is a four-leg intersection controlled by a traffic signal. **Figure 1** shows the existing lane configuration of the intersection. The signal is partially actuated providing protected left-turn phases. Both eastbound and westbound PA 41 contains a 12foot left-turn lane, a 12-foot shared through and right-turn lane and an 8-foot shoulder. The speed limit on Gap Newport Pike is 35 MPH. PA 10 northbound has one 11-foot travel lane with approximately 8-foot shoulders. PA 10 southbound also has one 11-foot travel lane. There is a 12-foot striped shoulder on the western side of the road and an 8-foot shoulder on the eastern side. The speed limit on Limestone Road is 35 MPH.

Looking northbound along Limestone Road (PA 10)

The surrounding land uses at this location are commercial in nature. There is a vacant lot located on the northeast corner of the intersection where local officials stated that an antique store is proposed. On the northwest corner, there is a closed Family Market gas station. A drainage basin for the market is adjacent to the corner of the intersection. The southwest corner of the intersection is a Turkey Hill Market. Each of these properties has access to/from both PA 41 and PA 10. The southeast corner has a BP gas station that provides unrestricted access to both PA 41 and PA10.

Figure 1
Existing Lane Configuration

PA 41 (Gap Newport Pike) and
PA 10 (Limestone Rd)
West Fallowfield Township, Chester County



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Existing Conditions

Identified Problems

In discussions with both township and county officials, it was stated that this intersection experiences heavy truck volumes. During each of the field views, many large trucks were noted, including large trucks carrying manufactured homes.

There are access management issues with the BP gas station access to PA 41 and PA 10 at the intersection. The more access points, especially those close to the intersection, the more conflict points there are and the potential for angle and rear-end crashes. On the other hand, the Turkey Hill Market is expanding and moving its gas pumps. They will be redesigning access into their lot and moving one of the access points on PA 41 farther away from the intersection. They are limited on what can be done along PA 10. There is also a septic field for the Turkey Hill Market located directly behind the building. This also limits any improvements on this side of the road.

Looking westbound at intersection along PA 41.
(Unlimited access into BP is located on left)

Vehicles use shoulders on PA 10 in both the north and southbound directions to pass left-turning vehicles from PA 10 to PA 41. Passing on the right presents a safety issue, because sight distance may be obstructed and opposing traffic making a left cannot be seen.

Pavement markings such as stop bars or lane markings have faded. These are needed to indicate where vehicles approaching the intersection should stop and sets them back from the intersection to avoid conflict with turning vehicles. This is especially helpful when large trucks are making a turn. The turning radii may also be a problem.

Field observations revealed deficient pavement surface. The PA 41 approaches have a concrete “white topping.” This is a bonded, fiber-reinforced cement concrete overlay that is used for road surfaces where traditional paving and asphalt materials have failed. Overlays made with this high-strength, fiber-reinforced concrete product require a thickness of two to six inches over asphalt; resulting in low cost and simple, fast installation. Streets and highways paved with white topping are more durable than asphalt and resist rutting under heavy traffic loads, particularly in intersections. The layer of concrete helps to prevent large trucks from tearing up the bituminous layer when either stopping or starting at the intersection. This cement layer has cracked and deteriorated.

Recently, PennDOT used a maintenance contract to eliminate the concrete and replace it with bituminous paving. This lower cost alternative may still turn out to be problematic with high levels of large truck traffic at this intersection. The pavement may need to be repaved more often than the concrete.


Deteriorated concrete pavement on PA 41

Crash Analysis

A crash analysis was performed in an effort to identify safety problems related to the operation of the intersection. Crash data was collected from the PennDOT Bureau of Highway Safety & Traffic Engineering Accident Records System for a five-year period from years 1998-2001 and 2003. Accident data for the year 2002 is unavailable from PennDOT.

During this five-year period, there were 17 accidents at this intersection. There were no fatalities recorded. However, there were seven recorded injuries because of these crashes. **Table 2** provides a breakdown by year for the number of accidents, the injuries, and the type of accident that occurred. The most common accidents in the intersection are rear end collisions that accounted for 40 percent of the total accidents.

Accidents are classified into three categories. These are angle, same direction rear-end, and fixed object. Rear-end crashes are typically comprised of vehicles traveling in the same direction, while angle crashes involve opposing traffic movements. A fixed object crash entails a vehicle striking an object such as a sign, telephone pole or light pole.

Table 2 Intersection Accident Summary (1998-2003)*

	1998	1999	2000	2001	2003	Total
Crashes	6	3	4	2	2	17
Severity						
Injuries	3	1	3	0	0	7
Fatalities	0	0	0	0	0	0
Accident Type						
Angle	66.7%	33.3%	0.0%	50.0%	50.0%	41.2%
Same Direction - Rear End	16.7%	66.7%	75.0%	50.0%	0.0%	41.2%
Fixed Object	16.7%	0.0%	25.0%	0.0%	50.0%	17.6%
Time of Day						
Midnight to 6 am	16.7%	0.0%	0.0%	0.0%	50.0%	11.8%
6 am to Noon	16.7%	0.0%	75.0%	0.0%	0.0%	23.5%
Noon to 6 pm	66.7%	66.7%	0.0%	100.0%	50.0%	52.9%
6 pm to Midnight	0.0%	33.3%	25.0%	0.0%	0.0%	11.8%
Rush Hours						
6 am to 9 am	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
11 am to 2 pm	66.7%	0.0%	50.0%	0.0%	0.0%	66.7%
4 pm to 7 pm	0.0%	33.3%	0.0%	50.0%	50.0%	33.3%
Light Conditions						
Daylight	83.3%	66.7%	75.0%	100.0%	50.0%	76.5%
Dark (Street Lights On)	16.7%	33.3%	0.0%	0.0%	50.0%	17.6%
Dark (Street Lights Off)	0.0%	0.0%	25.0%	0.0%	0.0%	5.9%
Weather Conditions						
No Adverse Conditions	66.7%	66.7%	100.0%	100.0%	100.0%	82.4%
Raining	33.3%	33.3%	0.0%	0.0%	150.0%	35.3%
Surface Conditions						
Dry	66.7%	66.7%	75.0%	100.0%	50.0%	70.6%
Wet	33.3%	33.3%	25.0%	0.0%	50.0%	29.4%
Contributing Factor						
Too Fast Combination	16.7%	0.0%	0.0%	50.0%	0.0%	11.8%
Red Light - Unknown	16.7%	0.0%	0.0%	0.0%	0.0%	5.9%
Other Driving Factors	16.7%	0.0%	50.0%	0.0%	50.0%	23.5%
Improper Turning	50.0%	0.0%	25.0%	0.0%	50.0%	29.4%
Tailgaiting	0.0%	33.3%	25.0%	0.0%	0.0%	11.8%
Driver Drinking	0.0%	33.3%	0.0%	0.0%	0.0%	5.9%
Improper Exit	0.0%	33.3%	0.0%	0.0%	0.0%	5.9%
Unknown	0.0%	0.0%	0.0%	50.0%	0.0%	5.9%

Source: PennDOT, Bureau of Highway Safety & Traffic Engineering Accident Records System, 1998-2003

** Accident data for the year 2002 is unavailable*

At this intersection, 76.5 percent of the accidents occurred during daylight hours. However, neither the morning (0 percent) nor evening (7.6 percent) peaks periods experienced a high rate of accidents. Conversely, the midday period from 11 AM to 2 PM had 35.2 percent of the total accidents. This helps to provide evidence that there are not typical AM/PM peak hours, but traffic occurring all day long. Weather and surface conditions also do not play a major role in causing collisions with 82.4 percent of the crashes occurring with no adverse weather conditions, 70.6 percent on dry surface road conditions.

According to the data, the 29.4% of the crashes can be contributed to improper turns. Other driving factors contribute to 23.5% of the crashes. These factors may range from anything such as driver behavior to mechanical errors. It is very difficult to make any conclusions as to what caused these crashes.

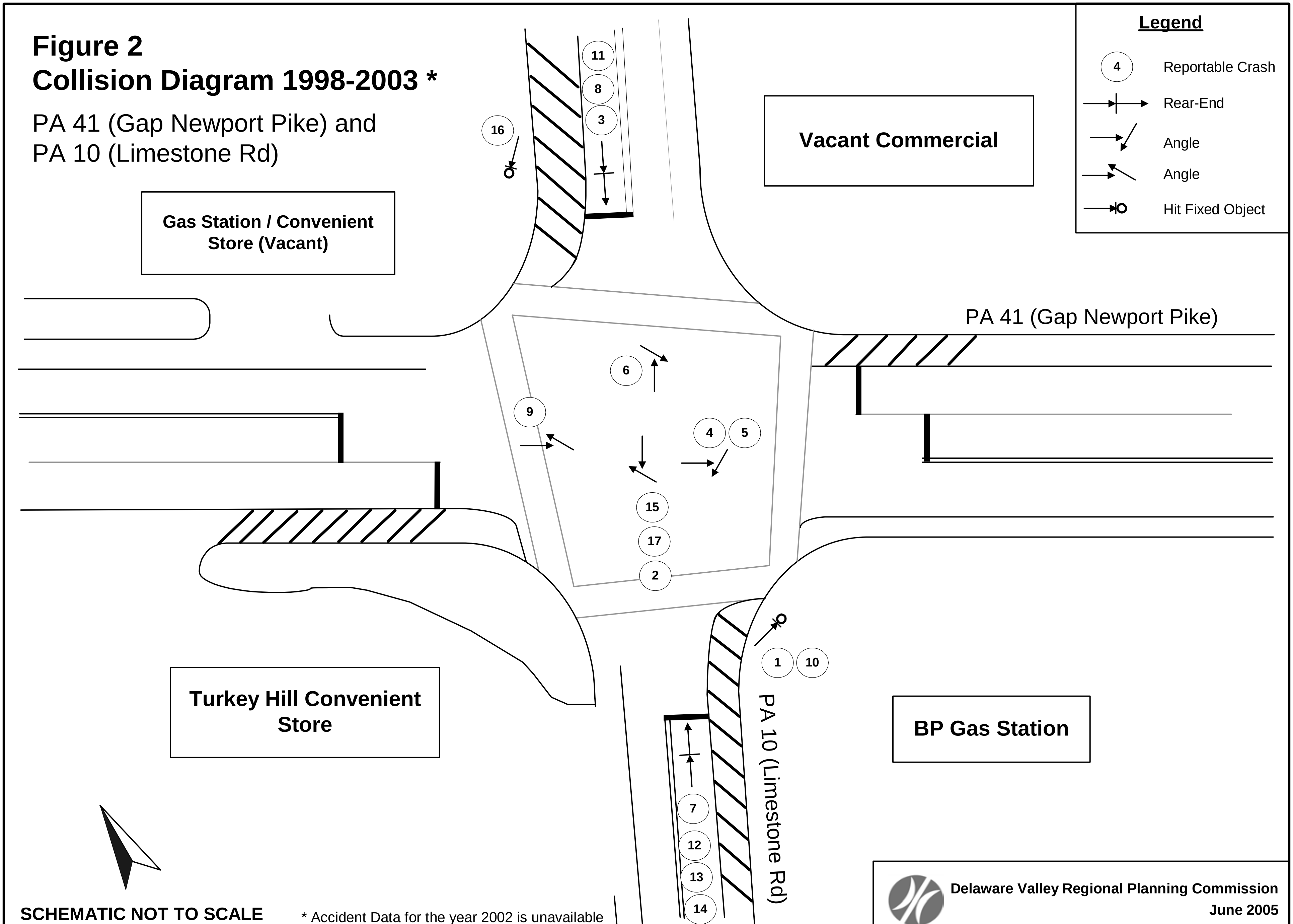
Figure 2 graphically displays a collision diagram that was prepared from the records obtained from PennDOT. A brief summary of the crash records is located in **Appendix A**. The purpose of the collision diagram is to pictorially represent different types of crashes that have occurred and is useful in identifying accident patterns and trends. The figure shows all the rear-end crashes that occur on PA 10. A contributing factor may be the lack of left-turn lanes. The high percentage of rear-end collisions may be related to through traffic attempting to bypass vehicles queuing to turn left at the intersection. The lack of left-turn lanes on PA 10 may also be a cause for the angle crashes.

Turning Movement Counts

A turning movement traffic count of the intersection was conducted during the AM and PM peak periods in January 2005. **Figure 3** displays the counts for the peak hours, which are 7:30-8:30 AM and 4:30-5:30 PM. Manual turning movement counts data for this location can be seen in **Appendix A**. The data revealed that PA 41 has higher volume of traffic than PA 10, with PA 41 westbound having the highest volume movement through the intersection with 444 vehicles in the AM and 465 in the PM. Throughout the intersection, total volumes tend to be slightly higher in the PM peak hours. PA 10 has a high percentage of left-turns that conflict with opposing traffic and there are currently no protected left-turns. In the AM, PA 10 southbound approach has 94 vehicles turning left that conflict with about 160 through movements on Gap Newport Pike northbound; while in the PM, 84 left-turns conflict with 257 through movements. On the northbound approach, there are 119 left-turns in conflict with 161 through movements in the AM, and 87 left-turns conflicting with 243 through movements in the PM. The left-turn volumes on both approaches of PA 10 also constrain vehicles attempting to go through the intersection in the same direction. There are no left-turn lanes, so vehicles are either prevented from going straight and have to wait in the queue, or pass on the right in the shoulder.

Figure 2
Collision Diagram 1998-2003 *

PA 41 (Gap Newport Pike) and
PA 10 (Limestone Rd)



SCHEMATIC NOT TO SCALE

* Accident Data for the year 2002 is unavailable

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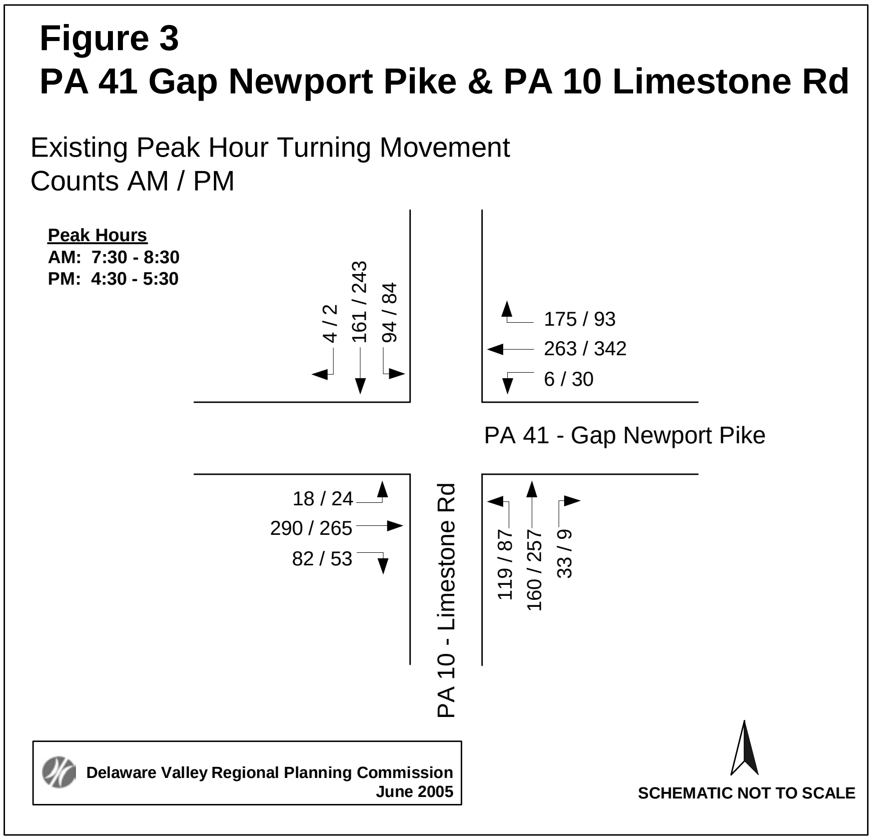


One of the concerns raised by the Chester County Planning Commission (CCPC) refers to the volume of trucks that traverse through this intersection. In June 2004, the CCPC conducted manual classification/turning movement counts for trucks and passenger vehicles from 7 AM to 1 PM. In 2004, their counts show that trucks account for 25.9 percent of the total volume. Gap Newport Pike (PA 41) eastbound approach had the highest volumes with over 30 percent.

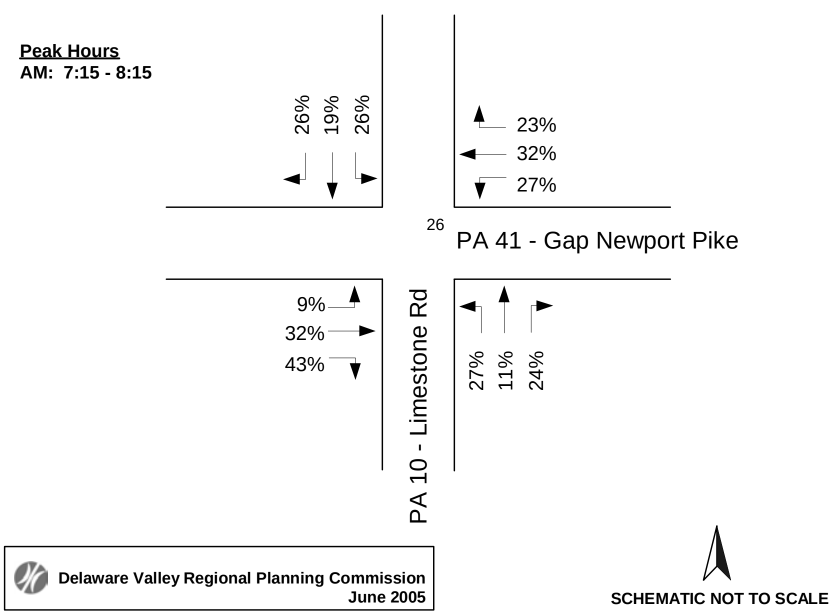
For this study, DVRPC revisited the intersection and conducted vehicle classification counts during the AM and PM peak periods. Vehicle classification counts from both CCPC and DVRPC are located in **Appendix A**. There is a discrepancy between the two agencies' counts. DVRPC's counts show a lower percentage of truck traffic through the intersection, with 14.8 percent and 9.6 percent in the AM and PM respectively. Variations in percentages between the counts may have resulted due to the difference in time of year the counts were made and the fact that Chester County counts took manual turning movement counts compared to directional automatic tube counts by DVRPC. In June 2005, the CCPC conducted another manual classification/turning movement count. This effort determined heavy truck volume in the AM peak is 24.4 percent of the total intersection with Gap Newport Pike eastbound again having volumes over 30 percent. Although the CCPC only took counts from 7 AM to 1 PM, their counts show that the percentage of trucks is consistent ranging, from 21.4 percent to 32.5 percent.

For the purpose of this study and the level of service analysis, it is important to get a handle on not only the volume of trucks, but also the turning movements of these trucks. Therefore, the AM peak-hour turning movement classification counts by CCPC for both 2004 and 2005 were averaged and applied to both the AM and PM periods. Although the actual counts for the PM peak were not taken, the uniformity of the truck volumes from 7 AM to 1 PM indicate that heavy trucks are present throughout the day. This was also confirmed during several field views to the study location.

Figure 4 displays the percentage of trucks for each of the turning movements. These percentages were used in the level of service analysis. Typically, during a Highway Capacity Manual level of service analysis, the default percentage of heavy trucks is 2 percent.

Figure 4
PA 41 Gap Newport Pike & PA 10 Limestone Rd

Average Percentage of Trucks for 2004-2005
AM Peak Hour Turning Movement Counts



Level of Service

The LOS analysis was performed on the intersection for the AM and PM peak periods using the Highway Capacity Software. The turning movement counts and vehicle classification counts were both collected during the peak periods in 15 minute increments. This data was used to determine the peak hour traffic volumes. The AM peak hour was 7:30 to 8:30, while the PM peak hour was 4:30 to 5:30.

The overall intersection LOS is C during both morning and afternoon peaks with average delays of 28.8 and 31.2 seconds respectively. Most of the approaches to the intersection also operate at a level of service C. Only the PA 41 westbound approach in the PM peak operates at a LOS D with 42.7 seconds in vehicle delay. This approach also has the largest delay in the AM peak period with 33.0 seconds. **Table 3** displays the existing level of service for the intersection.

Potential Improvement Scenarios

Based on the technical analysis of the existing conditions and suggestions from the local municipal officials, a set of potential improvement scenarios has been developed to determine the possible effective improvements that will reduce congestion and accidents created by limited capacity and design deficiencies of the intersections. For each improvement scenario, Synchro software analysis was used to determine the new level of service and amount of delay. **Table 3** displays each LOS data for improvement scenario. The existing level of service for the intersection is also listed for comparison purposes. Synchro LOS analysis data for both existing and potential improvement scenarios can be found in **Appendix A**.

Scenario 1 investigated optimizing the traffic signal at Gap Newport Pike (PA 41) And Limestone Road (PA 10). This low-cost improvement can be done immediately and does not require any right-of-way acquisition. By optimizing the signal at Gap Newport Pike and Limestone Road, the level of service in the intersection increases slightly. The cycle length changes from an existing cycle of 112 seconds to a 70-second cycle. The cycle length is the time in seconds required for one complete sequence of the signal.

In the AM peak, the LOS remains at a level C, but the delay decreases from 29 seconds to 25 seconds. All but one of the approaches experienced a reduction in delay during the AM, with the PA 41 eastbound approach reducing to a LOS B. The vehicle delay on the PA 10 northbound increases very slightly by four seconds. The reason for this is that the phasing for the intersection has changed and in order to optimize the entire intersection, the green light phase for this approach has been reduced from 44 seconds to 26 seconds per cycle.

In the PM peak, the LOS remains at a LOS C and the average delay per vehicle improves from 31 seconds to 24 seconds. The westbound approach on PA 41 improves from a LOS D to a

LOS C, with a reduction in vehicle delay by 12 seconds. The eastbound approach also experiences a 10-second reduction in vehicle delay. Signal optimization allows more vehicles to move through the intersection, reducing the backup.

TABLE 3
Peak Hour Level of Service (LOS) Analysis
PA 41 (Gap Newport Pike) and PA 10 (Limestone Rd)

Improvement Scenario	Direction of Travel	Peak AM Hour and Peak PM Hour LOS with Average Delay / Vehicle			
		AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)
Existing Conditions					
	PA 41 eastbound	C	27.4	C	30.6
	PA 41 westbound	C	33.0	D	42.7
	PA 10 northbound	C	28.6	C	23.8
	PA 10 southbound	C	24.1	C	23.7
	Intersection	C	28.8	C	31.2
1. Signal Optimization					
	PA 41 eastbound	B	19.9	C	20.6
	PA 41 westbound	C	24.6	C	30.8
	PA 10 northbound	C	32.3	C	22.4
	PA 10 southbound	C	23.4	C	22.3
	Intersection	C	24.8	C	24.6
2. Upgrade Signals to Demand Responsive Timing Plan					
	PA 41 eastbound	B	17.1	B	17.8
	PA 41 westbound	C	21.4	C	27.4
	PA 10 northbound	C	32.3	C	22.4
	PA 10 southbound	C	23.4	C	22.3
	Intersection	C	23.0	C	22.9
3. Add Left Turn Lanes on PA 10 northbound & southbound					
	PA 41 eastbound	A	9.4	B	13.1
	PA 41 westbound	B	11.0	B	18.7
	PA 10 northbound	C	21.6	B	16.0
	PA 10 southbound	C	20.2	B	15.9
	Intersection	B	14.6	B	16.1

Scenario 2 looks to build upon optimization of the traffic signal, by upgrading the traffic signals and implementing a fully actuated demand-responsive timing plan. To do so, the previously developed optimized timing plan is used; however, the PA 41 approaches are now also actuated. To do so, vehicle loop detection is needed along PA 41. There may also need to be upgrades to controllers, system software and signal heads. With these improvements to the intersection, the overall intersection LOS is C (23 seconds of vehicle delay) in both the AM and PM. When these results are compared to the existing conditions, the intersection remains at LOS C in the morning peak. The improvements are slight with the delay improving only six seconds in the AM and nine seconds in the PM.

Overall, this improvement scenario varies very little from the first scenario in which the signal-timing plan only is optimized. The LOS remains at a level C.

Scenario 3 seeks to improve the intersection by adding a left-turn lane on both the northbound and southbound approaches of Limestone Road (PA 10). The traffic signals are also updated to a fully actuated demand-responsive timing plan. Currently, there are left-turn lanes on the approaches of PA 41. Providing a dedicated left-turn lane reduces potential collisions between left-turning and through vehicles, increasing the capacity of the approach for both left and through traffic. The dedicated left-turn lane helps to alleviate congestion at the intersections. The turn lanes allow drivers to decelerate gradually out of the through lane and wait in a protected area. Vehicles traveling straight through the intersection will no longer pass left-turning vehicles on the right using the shoulder. This scenario requires right-of-way acquisition along PA 10 for additional capacity. However, there is a retaining wall along PA 10 just south of the intersection, which may restrict the placement of a new lane on the west side of the road. Portions of both shoulders may be used to accommodate the new left-turn lane. There may also be a need to upgraded controllers, system software and signal heads.

The HCS level of service analysis for this scenario indicates an improvement in the AM to a LOS B with 14.6 seconds of delay in the morning peak. This improvement is approximately 50 percent better than the existing conditions. Each of the approaches also experiences an improvement in LOS. PA 41 eastbound approach improves from LOS C to A with 9.4 seconds of delay, while the westbound approach improves from LOS C to B with 11 seconds of delay. Limestone Road southbound approach remains at a LOS C to 20.2 seconds of delay; while the northbound approach level of service remains at LOS C, but experiences a decrease in delay to 21.6 seconds.

In the afternoon peak period, the intersection improves from LOS C to B with a 15.1 second reduction in delay. Each of the approaches to the intersection also improves from a LOS C/D to a B. PA 41 benefits the most from adding a lane on PA 10. Compared to existing conditions, the vehicle delays for the east and westbound approaches of PA 41 could be reduced by 17.5 seconds and 24 seconds respectively.

Recommendations

Overall, this intersection currently operates at an acceptable level of service. The simple geometric configuration of this intersection is not confusing, which is reflected by the low number of accidents. Although the intersection currently operates a desirable LOS of C, Chester County is growing and there is the potential for heavier congestion in the future. Also any improvements to PA 41 east of the study location may likely have negative impact in this region. These factors and the large percentage of truck volumes that traverse through this intersection indicate a need for upgrading this intersection.

A short-term recommendation that can improve this intersection is to optimize the traffic signal timing of Gap Newport Pike (PA 41) and Limestone Road (PA 10). The LOS analysis shows that this has the potential to improve the delay of the overall intersection. Only the PA 10 northbound approach in the AM peak has an insignificant increase in delay. The afternoon peak also improves the traffic flow at the intersection and its individual approaches with the optimization of the signal.

Other short-term improvements and enhancements can be made to the location that would have a positive impact on the safety and operation of this intersection. These improvements are not incorporated into the HCS analysis, but can be integrated as part of any solution chosen for this intersection.

- Re-stripe pavement markings to identify turning movements and reduce driver confusion.
- Repainting the stop bars that are needed to help avoid conflicts in the intersection and create space for turning vehicles.
- Improve the turning radii to accommodate the large trucks.
- Promote proper access management at commercial properties adjacent to the intersection. This is the area where motorists are responding to the intersection, decelerating, and maneuvering into the appropriate lane to stop or complete a turn. Access connections too close to intersections may cause serious traffic conflicts that result in crashes and congestion.

Implementing Scenario 2 and upgrading the signal to install a fully actuated demand-responsive timing plan also improves the operation of the intersection. However, the benefits of this option over only optimizing the signal are relatively small and the cost of upgrading the system may outweigh the benefits. The upgrade to the signals and this new timing plan would be better implemented as part of a larger long-term effort.

The long-term recommendation (Scenario 3) would be to add left-turn lanes on each of the Limestone Road (PA 10) approaches. With the heavy volumes of trucks at this location, this option may become a priority. Providing a dedicated left-turn lane reduces potential collisions between left turning and through vehicles, increasing the capacity of the approach for both left and through traffic. To do so, the shoulders on PA 10 may be narrowed and re-striped to allow for an additional lane. If this were insufficient, then some right-of-way acquisition would be required. Widening PA 10 may be hampered due to a retaining wall just south of the intersection. To add the lane, property may have to be acquired on the east side of PA 10 and encroach on the gas station. If so, access to the gas station can be enhanced with proper access management and moving the curbs farther away from the middle of the intersection. As the LOS analysis indicates, the intersection is improved from a LOS C to a LOS B with only 14.6 and 16.1 second of delay in the AM and PM. More importantly, the addition of turning lanes will help improve the safety and efficiency of roadway intersections. Vehicles turning left will be removed from the queue, allowing through movements to pass.

With the physical improvements to the intersection, upgrades to the signal system can be made to make it fully actuated demand responsive.

Delaware Valley Regional Planning Commission

DELAWARE

3 KEDRON AVENUE (ROUTE 420) AND MACDADE BOULEVARD KEDRON AVENUE (ROUTE 420) & FOURTH AVENUE / ACADEMY AVENUE

Ridley Township, Delaware County

Location Description

The study area consists of two signalized intersections along Kedron Avenue (PA 420) in Ridley Township, Delaware County, Pennsylvania. The study location is shown on **Map 2.** The two intersections are at MacDade Boulevard and Fourth Avenue/Academy Avenue. Only 610 feet separate these two intersections along Kedron Avenue. There is a one-way access road, Plant Terrace, about halfway between the intersections. This provides access to several stores. Kedron Avenue is a major north-south roadway in Delaware County, which provides a link to I-95, while MacDade Boulevard connects directly with I-476 to the south of the study area. **Figure 5** shows the existing lane configuration of both intersections in the study area.

At the MacDade Boulevard intersection, both Kedron Avenue and MacDade Boulevard are classified as principal arterials. The eastbound MacDade approach has a 13-foot shared through/right-turn lane, a 12-foot through lane and a 12-foot left-turn lane. The westbound approach also has three lanes (15-foot shared through/right-turn lane, 11-foot through lane, 12-foot left-turn lane). Kedron Avenue northbound has a 15-foot shared through/right-turn lane and an 11-foot left-turn lane at the intersection; while the southbound approach has a 12-foot shared

Looking eastbound along PA 420 at MacDade Boulevard

Kedron Ave. (PA 420), MacDade Blvd., and Academy/4th Ave.

Ridley Township, Delaware County, PA

2005
Regional Congestion
and Accident
Mitigation Program

Map 2

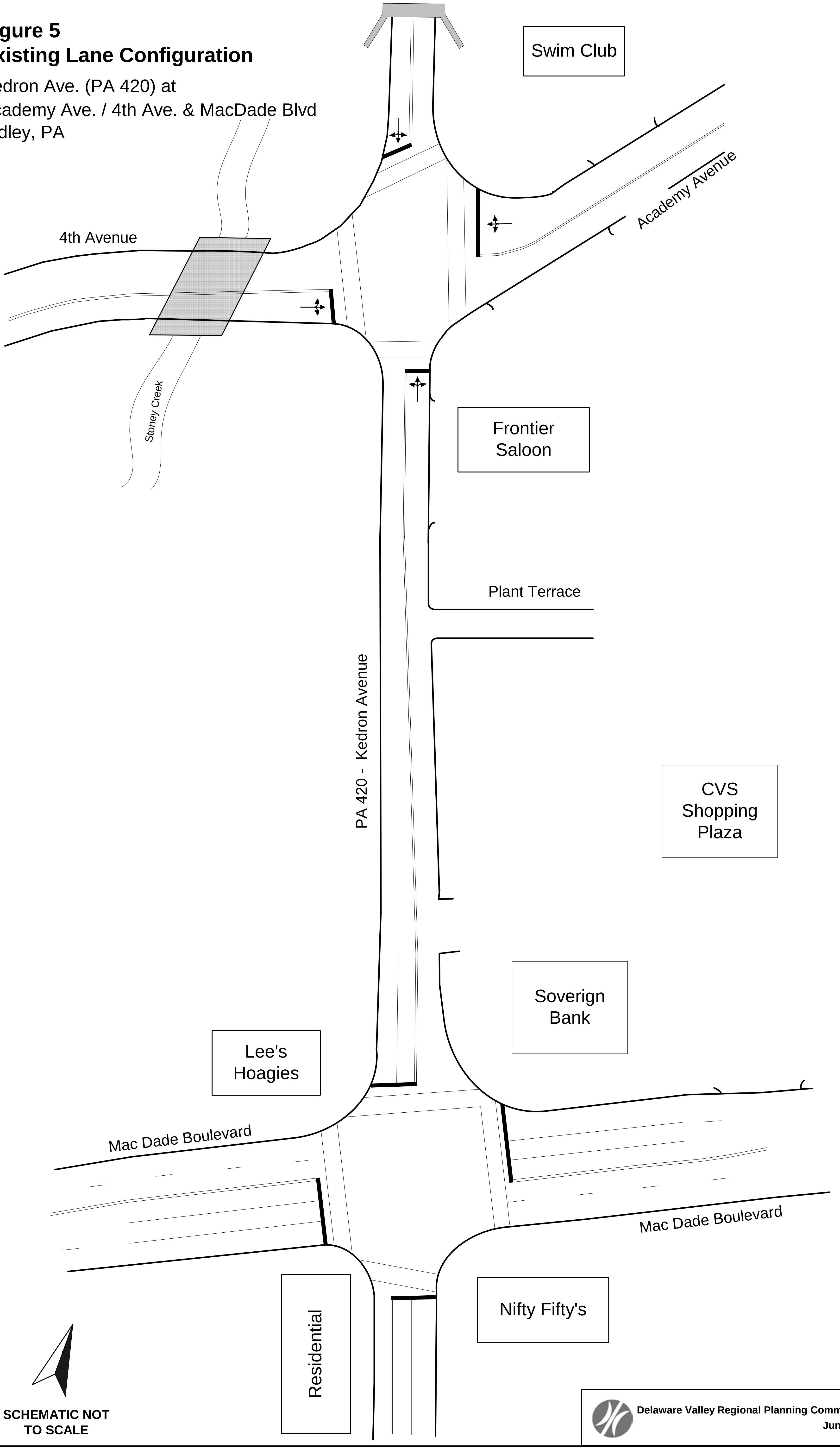
N
0 25 50 100
Feet

Delaware Valley
Regional Planning Commission
August 2005

Date of Aerial Photography: Spring, 2000

Figure 5
Existing Lane Configuration

Kedron Ave. (PA 420) at
Academy Ave. / 4th Ave. & MacDade Blvd
Ridley, PA



Delaware Valley Regional Planning Commission
June 2005

through/right-turn lane and an 11-foot left-turn lane. The left-turns are protected with separate signal phases. The intersection contains no shoulders, but there is a five-foot sidewalk along each approach.

The land use at the MacDade Boulevard intersection is mostly commercial in nature with a couple of residential housing units. On the northeast quadrant, a shopping plaza contains a CVS and a Sovereign Bank. Located across the street on the southeast corner are a Nifty Fifty's restaurant and other mixed commercial properties. On the northwest corner is a Lee's Hoagies deli that appears to have gone out of business. Residential housing is located along Kedron Avenue on the southwest corner. Municipal officials mentioned the future possibility of commercial development at this location.

North on PA 420 toward the Fourth Avenue/Academy Avenue intersection, there is a slight grade downhill. On the western side of Kedron Avenue, the land is recreational in nature. The land is wooded and belongs to the park located off Fourth Avenue. Kedron Avenue contains an 11-foot travel lane by direction with four-foot shoulders. Academy Avenue and Fourth Avenue also have one lane by direction with the approach lane of Fourth Avenue being 14 feet and Academy Avenue being 17 feet. Academy Avenue is also skewed at the intersection. There are no sidewalks, turning lanes or protected phasing at any of the approaches. Two bridges near Kedron and Fourth Avenue complicate this intersection. On the Kedron southbound approach, there is a bridge over Stoney Creek where the roadway narrows to 20 feet with no shoulder on the bridge. Along Fourth Avenue, the other bridge over the creek is 33 feet wide.

Looking west from Academy Avenue at Fourth Avenue & PA 420

Much of the land around the Fourth Avenue/Academy Avenue intersection is low-lying land associated with Stoney Creek. On the northeat corner of the intersection, the land is wooded with a seasonal swim club set back from the intersection with a parking lot located off Academy Avenue. The only commercial property is the Frontier Saloon and beer distributor on the southeast corner. The parking lot for the Saloon is adjacent to both Kedron Avenue and Fourth Avenue and access is unrestricted along these roads.

Existing Conditions

Identified Problems

Throughout the study area, there is congestion on Kedron Avenue (PA 420), notably in the southbound direction. Since the two signals are just over 600 feet from each other, the operation and efficiency of one intersection affects the other. If vehicles heading southbound on Kedron Avenue (PA 420) are unable to make it through the intersection at MacDade, there is a backup that restricts vehicles at the Fourth Avenue light to proceed through the intersection. At MacDade, the PA 420 southbound left-turn lane is approximately 75 feet long. During heavy congestion and peak periods, vehicles southbound on Kedron Avenue are queuing at the signal and often preventing other vehicles from getting into the left-turn lane, missing preferential treatment of the protected left-turn phases. To avoid congestion at MacDade Boulevard, vehicles travel a shortcut to avoid Kedron Avenue by making a right-turn at Fourth Avenue and then making a left-turn at Sutton Avenue to return to MacDade. The problem with this route is that these are local residential streets.

These two intersections are also uncoordinated. North of this location, there is a fiber optic coordination plan being developed, but these intersections are too distant to be included in that plan. However, it may be possible to coordinate these two locations.

At the intersection with Fourth Avenue/Academy Avenue, any future improvements that are made are constrained by the two bridges over Stoney Creek. The lanes narrow as they traverse the bridges, so any widening would have to consider bridge improvement costs.

Another issue with the intersection of Academy Avenue/Fourth Avenue is the unlimited access into the Saloon parking lot. Vehicles backing out of some of the spaces may veer into oncoming traffic along PA 420.

Along several approaches, there are difficult turning radii that cause problems for large trucks. For instance, large vehicles making a right-turn from MacDade Boulevard east to PA 420 south have trouble making this turn. They have to make a wide-angle approach and often enter into the oncoming lane. In addition, with Academy Avenue being skewed, the turning radii and sight distances are sometimes difficult.

Difficult turning radius from MacDade Blvd. on to PA 420

Crashes

Accident data was collected for both reportable and non-reportable accidents. Two separate sources are used representing a difference in periods. Reportable accident data was collected from the Pennsylvania Department of Transportation, Bureau of Highway Safety & Traffic Engineering Accident Records System, for a three-year period from 2000-2003 for this study. Accident data for the year 2002 was unavailable. Non-reportable accident data was collected from Call for Service Reports from the Ridley Township Police Department for 2002-2004. Brief summaries for both reportable and non-reportable crash records are located in **Appendix B**. This data does not present a full detailed account of non-reportable crashes. Although the accidents occurred in the study area, the precise location of an incident was difficult to determine and could not be plotted on a collision diagram. Therefore, only the type of accident was analyzed. **Table 4** presents a summary of the reportable crash data.

During these periods, there were 25 reportable and 53 non-reportable accidents. There were no fatalities recorded, however, there were 19 injuries due to reportable crashes, which typically involve an injury, fatality and significant property damage. In addition, one of the vehicles in the crash may be damaged to the point where it must be towed. A non-reportable crash involves a crash where there is no injury to the occupant(s) of the vehicle(s), and the vehicles involved do not need to be towed. A collision diagram depicts the location of the reportable accidents in **Figure 6**.

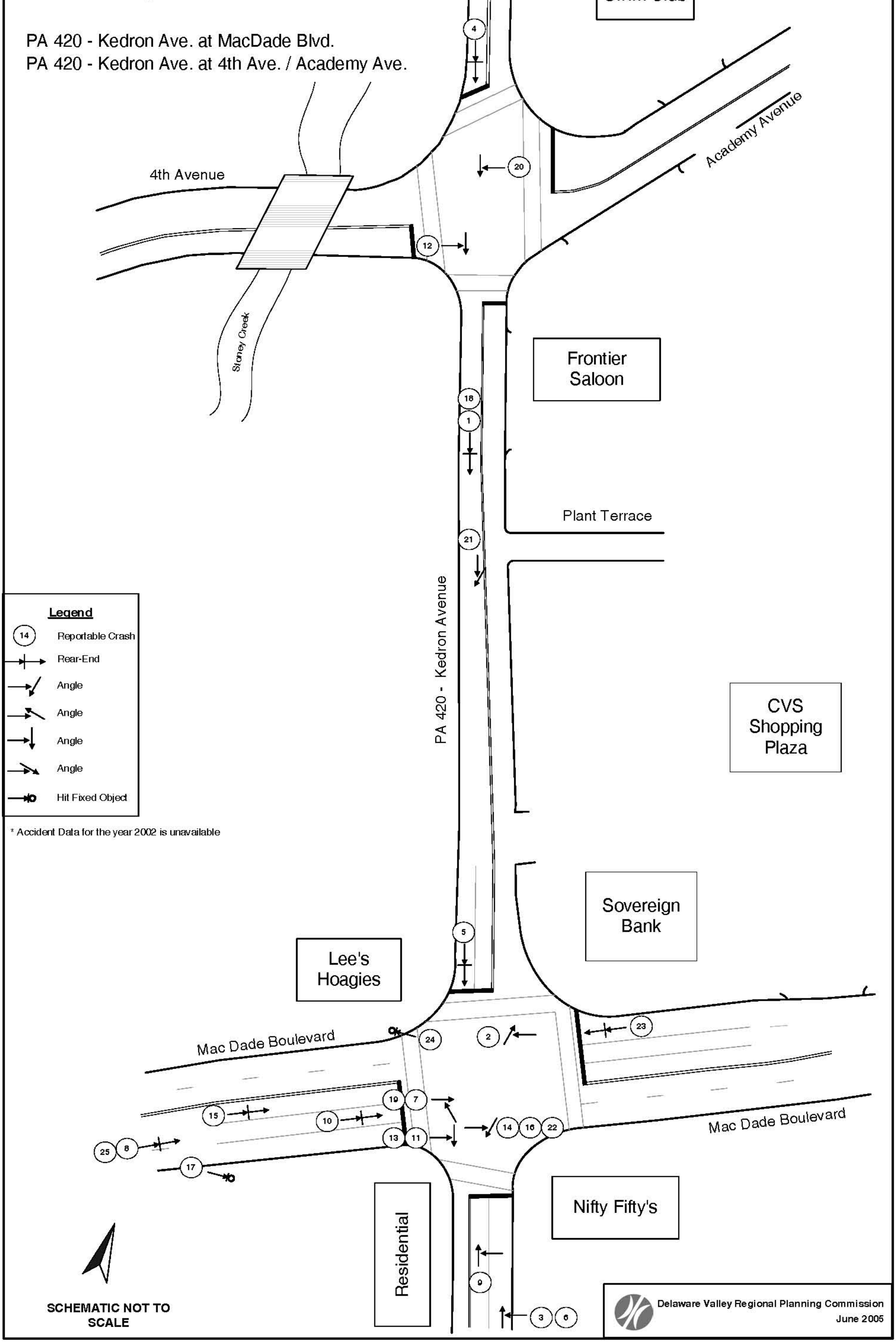
Table 4 Intersection Reportable Accident Summary (2000-2003)*

	2000	2001	2003	Total
Reportable Crashes	10	8	7	25
Severity				
Injuries	8	2	9	19
Fatalities	0	0	0	0
Accident Type				
Angle	50.0%	62.5%	57.1%	56.0%
Same Direction - Rear End	50.0%	25.0%	28.6%	36.0%
Fixed Object	0.0%	12.5%	14.3%	8.0%
Time of Day				
Midnight to 6 am	0.0%	12.5%	0.0%	4.0%
6 am to Noon	40.0%	25.0%	28.6%	32.0%
Noon to 6 pm	30.0%	37.5%	42.9%	36.0%
6 pm to Midnight	30.0%	25.0%	28.6%	28.0%
Rush Hours				
6 am to 9 am	0.0%	0.0%	0.0%	0.0%
11 am to 2 pm	10.0%	25.0%	0.0%	12.0%
4 pm to 7 pm	10.0%	25.0%	28.6%	20.0%
Light Conditions				
Daylight	70.0%	50.0%	57.1%	60.0%
Dark (Street Lights On)	20.0%	50.0%	42.9%	36.0%
Dark (Street Lights Off)	10.0%	0.0%	0.0%	4.0%
Weather Conditions				
No Adverse Conditions	60.0%	75.0%	100.0%	76.0%
Raining	30.0%	12.5%	0.0%	16.0%
Snowing	0.0%	12.5%	0.0%	4.0%
Unknown	10.0%	0.0%	0.0%	4.0%
Surface Conditions				
Dry	50.0%	75.0%	85.7%	68.0%
Wet	40.0%	25.0%	14.3%	28.0%
Unknown	10.0%	0.0%	0.0%	4.0%
Contributing Factors				
Improper Entrance	30.0%	0.0%	14.3%	16.0%
Improper Turning	10.0%	37.5%	0.0%	16.0%
Other Driving Factors	50.0%	0.0%	14.3%	24.0%
Police Pursuit	0.0%	0.0%	14.3%	4.0%
Red Light - Unknown	0.0%	25.0%	42.9%	20.0%
Sudden Slow Stop	0.0%	12.5%	0.0%	4.0%
Tailgating	10.0%	0.0%	0.0%	4.0%
Too Fast Combination	0.0%	12.5%	0.0%	4.0%
Unknown	0.0%	12.5%	14.3%	8.0%
* Accident Data for the year 2002 is unavailable				

Source: PennDOT, Bureau of Highway Safety & Traffic Engineering Accident Records System

Figure 6
Collision Diagram 2000-2003

PA 420 - Kedron Ave. at MacDade Blvd.
PA 420 - Kedron Ave. at 4th Ave. / Academy Ave.



Over this time frame, there has been a 30 percent decrease in reportable crashes from 10 accidents in 2000 to seven in 2004. Accidents are classified into three categories. These are angle, same direction rear end, and hit fixed object. Rear end crashes typically were comprised of vehicles traveling in the same direction, while angle crashes involved opposing traffic movements. A fixed object crash entails a vehicle striking an object such as a tree, sign, telephone pole or light pole. The most common reportable accident is angle crashes, which accounted for 56 percent. Other types of reportable crashes occurring are same direction rear end (36 percent) and hitting fixed objects (eight percent). Over this three-year period, 60 percent of the accidents occurred during daylight hours with 20 percent of the accidents occurring during the evening rush hours of 4 PM to 7 PM. Weather and surface conditions do not play a major role in causing collisions with 76 percent of the crashes occurring with no adverse weather conditions and 68 percent on dry surface road conditions.

The collision diagram (Figure 6) shows that more than half of the angled accidents occur as a result of left-turn movements at MacDade Boulevard. Rear-end crashes are common at the approaches to the intersections. Rear-end crashes typically occur during congested conditions with stop-and-go traffic, when vehicles in front of each other stop at the traffic light.

A congestion and rear-end accident problem is even more apparent when you look at the data for non-reportable accidents, displayed in **Table 5**. During the time frame from 2002-2004, there were 53 accidents. This is more than double the total reportable accidents and has increased dramatically over the last year. In 2004, accidents have increased 53 percent from 15 accidents in 2003 to 23. Of the total 53 non-reportable accidents, over half of these are rear-end collisions (53 percent). Many of these were reported to have taken place along Kedron Avenue between MacDade Boulevard and Fourth Avenue/Academy Avenue. Other non-reportable accident types were angled (23 percent), same direction sideswipes (11 percent) and hitting fixed objects (eight percent).

Table 5 Non-Reportable Accident Summary (2002-2004)

	2002	2003	2004	Total
Non Reportable Crashes	15	15	23	53
Accident Type				
Angle	27.0%	20.0%	22.0%	23.0%
Same Direction - Rear End	47.0%	53.0%	57.0%	53.0%
Same Direction - Sideswipe	13.0%	7.0%	13.0%	11.0%
Hit Fixed Object	13.0%	7.0%	0.0%	6.0%
Unknown	0.0%	13.0%	9.0%	8.0%

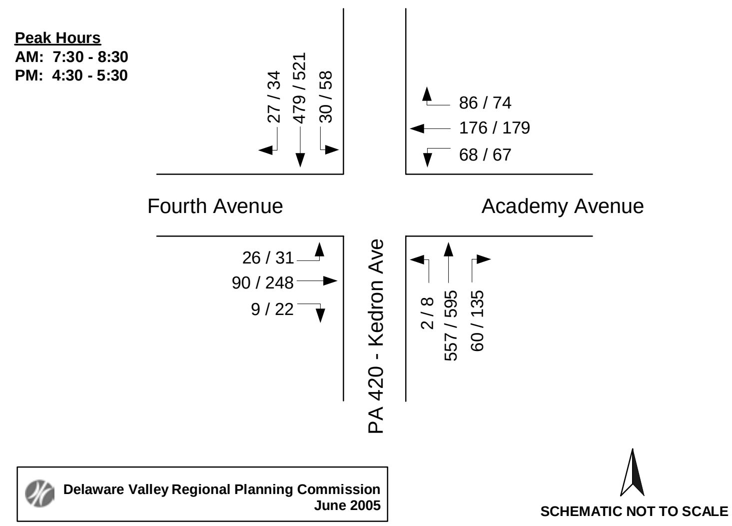
Source: Ridley Township Police Department Accident Reports

Turning Movement Counts

Turning movement counts were taken in January 2005 along Kedron Avenue at both the Fourth Avenue/Academy Avenue intersection and MacDade Boulevard.

Figure 7 PA 420 - Kedron Avenue and Fourth Ave / Academy Ave.

Existing Peak Hour Turning Movement Counts AM / PM

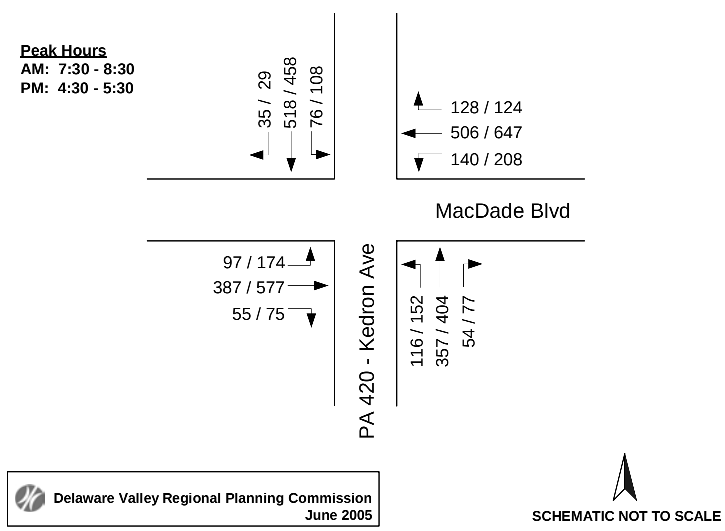


At the intersection of Fourth Avenue/Academy and PA 420, the peak hours are 7:30 to 8:30 in the morning and 4:30 to 5:30 in the afternoon. At Fourth Avenue/Academy and Kedron Avenue during both AM and PM peak periods, the dominant movements are through movements on Kedron Avenue in north and south directions. As **Figure 7** shows, there are 557 northbound vehicles in the morning and 595 in the afternoon making the through movement. In the southbound direction, there are 479 vehicles in the AM and 521 in the PM. The dominant moves on both Fourth Avenue and Academy Avenue are also through movements. Along Fourth Avenue, the afternoon peak carries 40 percent heavier volumes than in the morning, with about 300 vehicles into the intersection compared to only 125 in the morning. On Academy, the flow of travel is relatively constant with about 330 vehicles entering the intersection. In both peak periods, over half of the vehicles move straight through the intersection, while the remaining half is divided equally between those making left-turns or right-turns.

Figure 8 displays the turning movement counts for the intersection of MacDade Boulevard and Kedron Avenue (PA 420). The peak hours are 7:30 to 8:30 in the morning and 4:30 to 5:30 in the afternoon, which is the same as the adjacent intersection. MacDade Boulevard carries the heaviest traffic volumes of any of the roads in the study area with over 1,313 in the AM peak hour and 1,805 in the PM peak hour. The most dominant moves are vehicles traveling straight through the intersection with eastbound volumes of 387 and 577 for morning and afternoon peak hours respectively, and 506 and 647 westbound for morning and afternoon peak hour respectively. As the crash analysis suggested, these high through volumes on MacDade Boulevard conflict with opposing left-turns causing angle accidents. There are 140 and 204 vehicles making left-turns from MacDade Boulevard to Kedron Avenue south in both the AM and PM respectively. Left-turns from MacDade eastbound to Kedron Avenue north also cause a conflict with 97 making lefts in the AM and 174 in the PM.

Figure 8 PA 420 - Kedron Avenue and MacDade Boulevard

Existing Peak Hour Turning Movement Counts AM / PM



The north and southbound through traffic on Kedron Avenue also shows heavy volumes, with approximately 370 vehicles heading north and 475 vehicles heading south in both the AM and PM peak hours. Again, there are significant left-turns from Kedron Avenue to MacDade Boulevard. This is especially true in the PM peak with 108 left-turns from Kedron south to MacDade east and 152 vehicles from Kedron north to MacDade west. Complete manual turning movement counts for the study location can be seen in **Appendix B.**

Level of Service (LOS)

The LOS analysis was conducted using Synchro on both intersections for the AM and PM peak periods. The existing LOS for the Kedron Avenue (Route 420) and MacDade Boulevard intersection is an E in both morning and afternoon peak hours, with 74.1 seconds of delay in the morning and 73.3 seconds of delay in the afternoon. The PA 420 southbound is the worst approach to the intersection during the morning and afternoon peak hours, with a LOS F of 159.3 seconds of delay and 108.2 seconds of delay respectively. In the morning peak, each of the other approaches has a LOS D, while in the afternoon the level of services decline to a LOS E.

The intersection at Kedron Avenue and Fourth Avenue/Academy Avenue operates at a significantly better Level of Service than the intersection at MacDade. Overall, this intersection operates at a LOS B in both the morning and afternoon peak hours. There is only a delay of 11.1 seconds in the AM and 15.1 second in the afternoon. In the morning, both of the Kedron Avenue approaches operate at LOS A, with fewer than 10 seconds of vehicle delay. The other two approaches operate at a LOS B, with no more than 16 seconds of vehicle delay. In the afternoon, the LOS declines slightly with each approach operating at a LOS B and no more than 17 seconds of vehicle delay. **Table 6** presents the level of service data for existing conditions.

Potential Improvement Scenarios

Based on the technical analysis of the existing conditions and suggestions from the local municipal officials, a set of potential improvement scenarios has been developed to determine the possible effective improvements that will reduce congestion and accidents created by limited capacity and design deficiencies of the intersections. For each improvement scenario, Synchro software analysis was used to determine the new level of service and amount of delay. **Table 6** displays each scenario's LOS in comparison to the existing conditions. Complete Synchro LOS data for both existing and potential improvement scenarios can be found in **Appendix B**.

Scenario 1 investigated optimizing the traffic signal at MacDade Boulevard and Kedron Avenue. This low-cost improvement does not require any right-of-way acquisition and may be implemented immediately. By optimizing this signal, the operation and efficiency of this intersection improves dramatically. To do so, the cycle length changes from an existing cycle of 164 seconds to a 120 second cycle. In both the AM and PM peaks, the LOS improves from an E to a LOS D, with decreases in vehicle delays by over 20 seconds. Although the LOS on southbound Kedron remains at a level F, this approach improved the most in terms of delay; as it improved from 159.3 seconds to 81.4 in the AM peak. This approach improves at the expense of Kedron Avenue northbound, which declines from a LOS D to an E and increases in delay by 16 seconds. The improvement to Kedron Avenue southbound far outweighs this increase in delay to the northbound approach. In the morning peak, both MacDade Boulevard approaches also improves from an LOS D to a C, with vehicle delay decreasing over 8 seconds eastbound and 12 seconds westbound. In the PM peak, the intersection improves from LOS E to D, with a 23.2 second drop in delay to 50.1 seconds. Both of the MacDade Boulevard approaches improved to a LOS D. The Kedron Avenue southbound approach also has the largest improvement with a 35 second decrease in vehicle delay.

By optimizing the traffic signal at MacDade Boulevard, there is no effect on the LOS at Fourth Avenue/Academy and Kedron Avenue intersection.

Scenario 2 examined the option of coordinating and optimizing the two traffic signals at both Kedron Avenue/MacDade Boulevard and Fourth Avenue/Academy Avenue intersections.

TABLE 6
Peak Hour Level of Service (LOS) Analysis

Kedron Avenue (Route 420) and Mac Dade Boulevard

Improvement Scenario	Direction of Travel	AM Peak LOS	AM Peak Delay (sec)	PM Peak LOS	PM Peak Delay (sec)
Existing Conditions	Mac Dade Blvd. EB	D	37.2	E	58.6
	Mac Dade Blvd. WB	D	46.2	E	66.6
	PA 420 northbound	D	51.4	E	71.2
	PA 420 southbound	F	159.3	F	108.2
	Intersection	E	74.1	E	73.3
1. Signal Timing Optimization	Mac Dade Blvd. EB	C	29.5	D	35.6
	Mac Dade Blvd. WB	C	34.0	D	38.1
	PA 420 northbound	E	67.3	E	68.6
	PA 420 southbound	F	81.4	E	73.2
	Intersection	D	52.2	D	50.1
2. Signal Timing Optimization & Coordination	Mac Dade Blvd. EB	C	33.2	C	33.9
	Mac Dade Blvd. WB	D	40.3	D	37.2
	PA 420 northbound	E	62.7	E	70.1
	PA 420 southbound	D	52.6	E	69.2
	Intersection	D	46.7	D	49.5
3. Lengthen Left Turn Lane on Kedron SB (with optimization & coordination)	Mac Dade Blvd. EB	C	33.2	C	33.9
	Mac Dade Blvd. WB	D	40.3	D	37.2
	PA 420 northbound	E	62.7	E	70.1
	PA 420 southbound	D	52.6	E	69.2
	Intersection	D	46.7	D	49.5
4. Add Right Turn Lane on Kedron SB (with optimization & coordination)	Mac Dade Blvd. EB	C	31.6	C	31.9
	Mac Dade Blvd. WB	D	40.3	D	37.2
	PA 420 northbound	E	57.0	E	70.1
	PA 420 southbound	D	47.6	E	58.7
	Intersection	D	43.8	D	46.8

Kedron Avenue (Route 420) and Fourth Ave / Academy Ave

Improvement Scenario	Direction of Travel	AM Peak LOS	AM Peak Delay (sec)	PM Peak LOS	PM Peak Delay (sec)
Existing Conditions	Fourth Ave eastbound	B	14.0	B	16.7
	Academy Ave westbound	B	15.6	B	15.4
	PA 420 northbound	A	9.8	B	15.6
	PA 420 southbound	A	9.2	B	13.5
	Intersection	B	11.1	B	15.1
2. Signal Optimization & Coordination	Fourth Ave eastbound	D	40.1	D	43.0
	Academy Ave westbound	D	45.6	D	41.0
	PA 420 northbound	A	2.2	A	6.9
	PA 420 southbound	A	9.2	B	10.3
	Intersection	B	16.4	B	19.0
3. Lengthen Left Turn Lane on Kedron SB (with optimization)	Fourth Ave eastbound	D	40.1	D	43.0
	Academy Ave westbound	D	45.6	D	41.0
	PA 420 northbound	A	2.2	A	6.9
	PA 420 southbound	A	9.2	B	10.3
	Intersection	B	16.4	B	19.0
4. Add Right Turn Lane on Kedron SB (with optimization)	Fourth Ave eastbound	D	40.1	B	39.3
	Academy Ave westbound	D	45.6	B	40.2
	PA 420 northbound	A	2.7	A	8.0
	PA 420 southbound	A	9.2	A	12.6
	Intersection	B	16.5	B	19.4

There are no changes to the existing configuration of either of the intersections, so there is no need for right-of-way acquisition. This scenario option would require a fiber interconnect between the signals. Upgrades are needed to the controllers, system software and signal heads.

These two signals are just over 600 feet from each other and the operation of one affects the other. Under the right circumstances, coordination of adjacent traffic signals should improve the flow of traffic by synchronizing vehicle movement along a facility. On the other hand, the lack of traffic signal coordination may impede the flow of traffic and intensify congestion.

For this scenario, a traffic signal coordination analysis was performed using Synchro to analyze the potential for coordination, or “coordinatability,” of the two intersections. To assess coordinatability, Synchro evaluates the adjacent intersections and the roadway segment between them based on four measures: 1) travel time, 2) storage space, 3) proportion of traffic in platoon, and 4) main street volume. The four measures are used to calculate a coordinatability factor for each intersection pair. The coordinatability factor is on a scale of zero to 100, with 100 indicating maximum coordinatability. Generally, any score above 80 indicates that the intersections should be coordinated. Synchro also outputs a recommendation for or against coordination.

A coordinatability analysis was conducted on the existing conditions for both the AM and PM peak periods. For both the AM and PM, the Synchro analysis calculates that the travel time between intersections is sufficient for coordination. The traffic also exceeds 80 percent of its storage capabilities and there is a high volume of traffic. Generally, when there are short travel times and a high volume between the intersections, the potential for coordination is usually high. The associated coordinatability factors for these intersections is 91 for the AM and 80 for the PM. Based on these factors, Synchro recommends a need for signal coordination.

The LOS analysis performed with Synchro shows a dramatic improvement in the operation of the intersection at Kedron Avenue/MacDade Boulevard. In the morning, the operation of the intersection improved from a LOS E to D, with decrease in vehicle delay of 27.4 seconds. In the evening peak hour, the LOS improves to a LOS D, with a reduction in vehicle delay of about 24 seconds. The biggest improvement is the southbound Kedron Avenue, which improves from a LOS F to a D in the morning peak. The delay decreases by over 100 seconds from the existing conditions and by more than 30 seconds from just optimizing the signal timing. The evening peak improves to a LOS E, with over a 40 second decrease in vehicle delay.

To achieve the improvement in LOS at MacDade Boulevard, the LOS at Fourth Avenue / Academy Avenue experiences a slight increase in intersection delay. Though the LOS remains at level B, there is an increase in delay of about five seconds in both peak hours. The true negative effect of signal coordination occurs on both local roads of Fourth Avenue/Academy Avenue. These approaches declined to a LOS D, with an increase of over 25 seconds in delay. The reason is that the traffic signal cycle has increased from 60 seconds to 130 seconds with Kedron Avenue receiving preferential green phasing. This is evident with the Kedron northbound approach that remains at a LOS A, but improves to only 2.2 seconds of total delay in the morning peak. In the evening, this approach improves by almost 10 seconds to a LOS A and 6.9 seconds of delay. The southbound approach also remains at a LOS A in the morning. The southbound approach remains the same with only a 3 second decrease in vehicle delay in the evening peak.

Two other scenario options were examined to improve the efficiency of MacDade Boulevard and Kedron Avenue by adding lane capacity to the intersection. These scenarios build upon Scenario 2, including the improvements to coordinate and optimize the two traffic signals. **Scenario 3** looked at lengthening the left-turn lane on the southbound approach to MacDade Boulevard. By lengthening the left-turn lane, capacity is added to the intersection. Vehicles turning left are now able to access the turn lane and take advantage of protected left-turn phasing. The dedicated left-turn lane helps to alleviate congestion at the intersections by removing the vehicles from the southbound queue. **Scenario 4** looks at adding a right-turn lane for the southbound approach to MacDade Boulevard.

These changes to the capacity of the intersection help to increase the safety of the intersection by separating turning movement conflicts. Scenarios 3 and 4 would require right-of-way acquisition needed for additional capacity. Right-of-way can be acquired from the western edge of PA 420, where there is a vacant commercial property (Lee's Hoagies). There is also available land that is part of the park. In order to use this land, environmental issues would have to be further investigated to determine any impacts on Stoney Creek. These scenarios also require a fiber interconnect between the signals and upgrades to the controllers, system software and signal heads.

As seen in **Table 6,** neither of these two scenarios provides much improvement over the Scenario 2's signal timing optimization and coordination. The LOS remains at a level D with about 44 to 50 seconds of vehicle delay. Neither of these options adversely impact the Fourth Avenue/Academy Avenue intersection.

Recommendation

The congestion associated with Kedron Avenue (PA 420) southbound is the primary concern for this study location. This is an important north-south corridor in Delaware County. With the two intersections located very close to each other, a problem at one location negatively impacts the other. Congestion also augments the frequency of accidents in this location. The increase in non-reportable accidents by over 50 percent from 2003 to 2004 may be indicative that conditions are deteriorating at an alarming rate.

A short-term recommendation that can improve this intersection is to optimize the traffic signal timing of MacDade Boulevard and Kedron Avenue (PA 420). The LOS analysis shows that this has the potential to reduce the vehicle delay by 20 seconds in both peak periods. Though Kedron Avenue southbound approach continues to operate at a LOS F in the AM, the delay decreases by about 80 seconds. However, this scenario has no real effect on the Fourth Avenue/Academy Avenue intersection.

The two traffic signals with Kedron Avenue (PA 420) at MacDade Boulevard and Fourth Avenue/Academy Avenue should be optimized and coordinated as a long-term recommendation. The Kedron Avenue southbound approach LOS continues to improve in the AM to a LOS D with an additional decrease in delay by approximately 30 seconds. The benefits that are provided to Kedron Avenue compensate for added delays to the Fourth Avenue and Academy Avenue approaches.

Any other long-term effort that adds lane capacity to this section of PA 420 has many factors to take into consideration and the costs and benefits should be weighed. Right-of-way acquisition and bridge improvements to widen the lanes could be costly and time-consuming. The LOS analysis shows that beyond signal coordination and optimization, there are only marginal improvements, and therefore not recommended at this time.

Delaware Valley Regional Planning Commission

PHILADELPHIA

4 34th STREET AND GRAYS FERRY AVENUE

Philadelphia, Pennsylvania

Location Description

The study area is located in South Philadelphia at 34th Street and Grays Ferry Avenue. This is a four-legged intersection controlled with a traffic signal. There are protected left-turn phases for each approach. Both streets provide access into University City with 34th Street running in a north-south direction and Grays Ferry Avenue traveling in an east-west direction. The study area is shown on **Map 3**. Thirty-fourth Street provides direct access to the I-76 eastbound on-ramp and from the I-76 westbound off-ramp. To the north of the intersection along 34th Street there is the I-76 eastbound off-ramp. The entire intersection is elevated over a set of Conrail train tracks that bisect the intersection. Improvements to this location are a priority because of a future South Street bridge reconstruction project scheduled to begin in 2007. The existing South Street Bridge is being expanded from three lanes to four lanes with additional bike paths and improvement to I-76 off-ramps. Traffic at 34th Street and Grays Ferry Avenue is expected to increase, as both of these streets will be used as alternate routes into and out of University City during the closure of the South Street Bridge.

Looking southeast at 34th Street at Grays Ferry Avenue Intersection

Both 34th Street and Grays Ferry Avenue are classified as principal arterials. The speed limit is 30 MPH on 34th Street and Grays Ferry Avenue. The existing lane configuration of this

34th St. and Grays Ferry Ave.

Philadelphia, PA

2005
Regional Congestion
and Accident
Mitigation Program

Map 3

N
0 25 50 100
Feet

Delaware Valley
Regional Planning Commission
August 2005

Date of Aerial Photography: Spring, 2000

intersection is displayed on **Figure 9.** The 34th Street northbound approach has two through lanes, 1 left-turn lane, and one channelized right-turn lane; each 11 feet wide. The 34th Street southbound approach has a 10-foot through lane, a 10-foot shared through/right-turn lane with an island to channel right-turns immediately at the intersection, and an 11-foot left-turn lane. There are two 11-foot through lanes at the Grays Ferry Avenue eastbound approach, with one 10-foot left-turn lane, and a 5-foot bicycle lane. Westbound approaching Grays Ferry Avenue, there are two 13-foot through lanes and an 11-foot left-turn lane. There is also a concrete island in this approach to channel right-turns. These concrete islands help to allow pedestrians to cross the right-turn lane and wait on the refuge island for a green arrow. There is also a wide striped crosswalk for pedestrians.

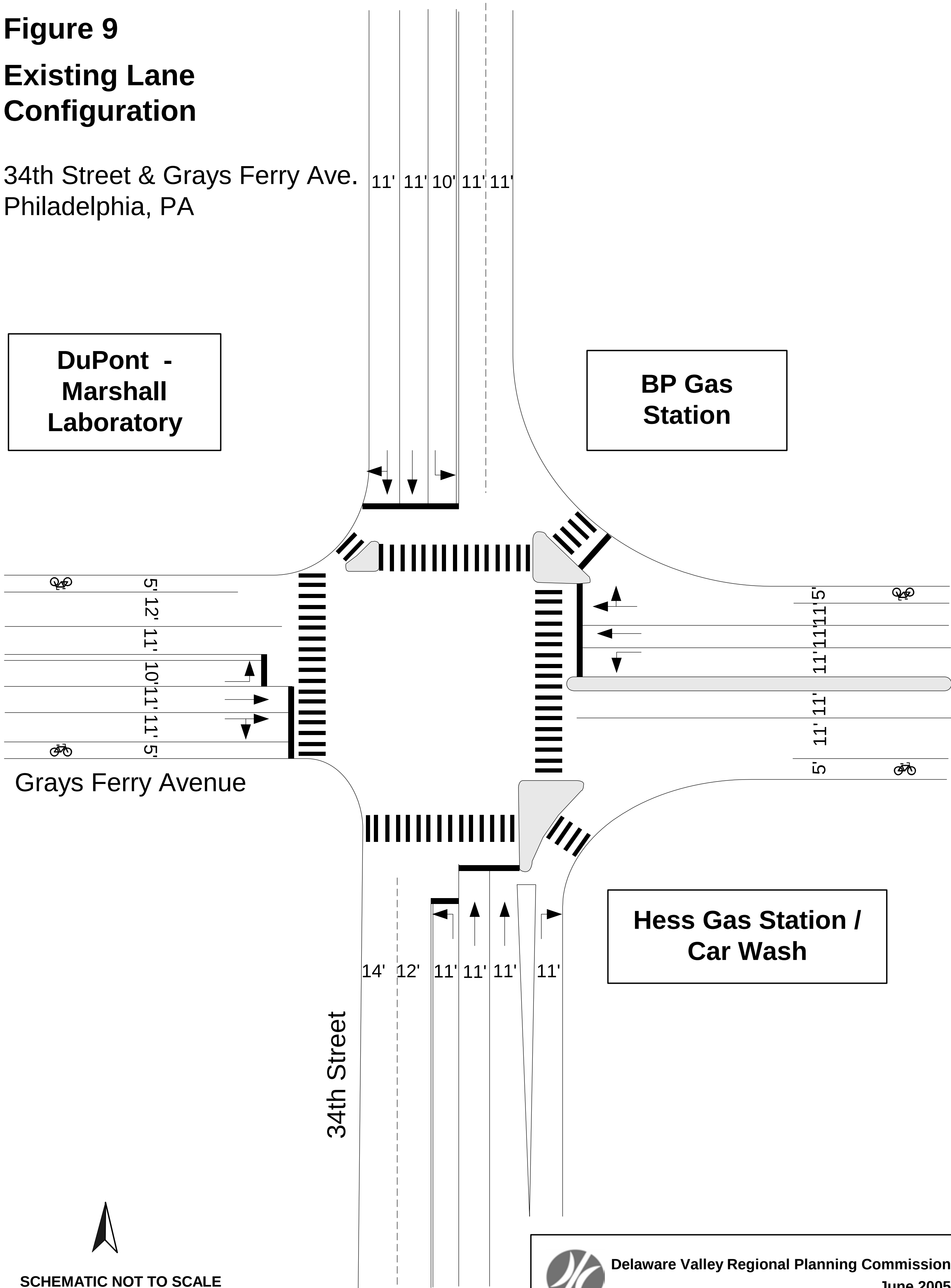
The intersection's surrounding land uses are classified as heavy manufacturing and commercial. On the northwest corner of the intersection is the Dupont Marshall Laboratory. The land immediately adjacent to the intersection is green space and fenced off with no direct access to the intersection. The entrance to this facility is located west of the intersection on Grays Ferry Avenue. At the northeast and southeast quadrants of the intersection, there is a BP gas station and a Hess gas station/carwash, respectively. The southwest quadrant is below grade and is part of the underpass to the Conrail railroad tracks. West of the study location on 34th Street, there is a Federal Express facility and a Waste Management Transfer and Recycling Center that help contribute to heavy truck volumes.

The signal timing on the local area grid is at 60 seconds; however, this intersection operates at a 90-second cycle length. With this difference, this traffic signal is not coordinated with other surrounding signals. The benefits of a coordinated network that connects a series of traffic lights together is to help traffic flow through a series of signals at a predetermined speed to minimize or avoid stops. In this area, there are existing signal interconnects along Grays Ferry Avenue, but these are old conduits and may no longer be functional. On the other hand, there are no interconnects between signals along 34th Street. The nearest signalized intersections on Grays Ferry Avenue are located to the west at 35th Street, Grove Street, and Federal Express Drive and are no more than approximately 1,600 feet from the study location. To the east, two more signals are located at 31st Street and 30th Street. Along 34th Street, the closest intersection is only 600 feet to the south at Wharton Street.

Figure 9

Existing Lane Configuration

34th Street & Grays Ferry Ave.
Philadelphia, PA



Delaware Valley Regional Planning Commission
June 2005

Existing Conditions

Identified Problems

At the onset of this effort, agency field views were conducted with representatives from the City of Philadelphia where the traffic and transportation issues in the study area were discussed. DVRPC staff conducted subsequent follow-up field views to observe the existing operating conditions of the study area. The following are descriptions of identified problems from this process.

The geometry of this intersection is straightforward, as the streets intersect at perpendicular angles. However, the turning radii at the northwest and southwest corners of the intersection cause problems for large trucks traveling from the west and making turns onto 34th Street south to access I-76. Trucks have to take a wide-angle approach to make the turn. They often have to avoid traffic in the oncoming lane. While making these turns, the trucks have a tendency to knock over signal mast arms as they try to maneuver around the corner. Traffic signal junction boxes have also been hit in the past and have been relocated. However, city officials stated that these continue to be struck and may need to be moved again. For large trucks making a right from 34th Street south onto Grays Ferry Avenue, they often drive over the channelized concrete island. This has actually caused the island to deteriorate and depress into the asphalt.

Turning Radius Problem: Tractor trailer making a wide right-turn from Grays Ferry Ave. eastbound to 34th St. south

In the peak AM periods, there are heavy volumes of northbound traffic along 34th Street as vehicles are heading toward University City. The high volumes of traffic have created congestion and long traffic queues. Traffic will often back up to the preceding intersection at Wharton Street. Another related problem is vehicles exiting I-76 westbound traveling at high rates of speed down the off-ramps. If there are long traffic queues then conflicts occur.

Pavement conditions at this intersection are deficient. In many locations, it is uneven and bulging. Pavement markings within the intersection, used to delineate turning movements, are faded and worn away.

The nature of this intersection is not conducive for safe pedestrian movements due to the high volumes of traffic and wide streets. Although this intersection may not have a high volume of pedestrian traffic, each field view identified some level of pedestrian activity. The sidewalks are dilapidated, and although there are push buttons for pedestrians on traffic signal poles, there are no pedestrian signals or other pedestrian amenities. All crosswalk striping is faded and striping on the southbound approach of 34th Street has been completely worn away.

Adding to the confusion at the intersection are vendors selling newspapers and bottles of water. These vendors tend to walk up and down the traffic lanes and in and out of the path of vehicles when they are stopped for a red light. This tends to cause driver distraction.

Crash Analysis

Accident data for the location of 34th Street and Grays Ferry Avenue was collected from the City of the Philadelphia Streets Department Crash Report for a three-year period from 2001-2003. According to Philadelphia Streets Department, Traffic Engineering Department, this intersection has the third most accidents at one location in the entire city. During this period, there were 48 reportable and 146 non-reportable accidents for a total of 194 accidents. There were no fatalities recorded; however, there were 56-recorded injuries because of these crashes.

Although a significant number of non-reportable accidents are occurring at this location, the data presented in **Table 7** refers only to the reportable accidents. A reportable crash typically involves an injury, fatality and/or significant property damage. In addition, one of the vehicles in the crash may be damaged to the point where it must be towed. A non-reportable crash is one where there is no injury to the occupant(s) of the vehicle(s), and the vehicles involved do not need to be towed. Data collected from the Philadelphia Streets Department Crash Report does not present a full detailed account of non-reportable crashes; only the location, date and time of

Table 7 Accident Summary (2001-2003)
34th Street & Grays Ferry Avenue Intersection

	2001	2002	2003	Total
Crashes				
Reportable	27	8	13	48
Non Reportable	47	64	35	146
Total	74	72	48	194
Severity*				
Injuries	41	10	5	56
Fatalities	0	0	0	0
Accident Type*				
Angle	29.6%	37.5%	23.1%	29.2%
Same Direction - Rear End	7.4%	25.0%	53.8%	22.9%
Head-On	22.2%	12.5%	7.7%	16.7%
Left Turn	14.8%	0.0%	0.0%	8.3%
Same Direction - Sideswipe	7.4%	0.0%	7.7%	6.3%
Other	11.1%	0.0%	0.0%	6.3%
Hit Fixed Object	0.0%	25.0%	7.7%	6.3%
Hit Pedestrian	7.4%	0.0%	0.0%	4.2%
Time of Day*				
Midnight to 6 am	18.5%	37.5%	7.7%	18.8%
6 am to Noon	11.1%	37.5%	30.8%	20.8%
Noon to 6 pm	44.4%	12.5%	30.8%	35.4%
6 pm to Midnight	25.9%	12.5%	30.8%	25.0%
Rush Hours*				
6 am to 9 am	3.7%	25.0%	15.4%	10.4%
11 am to 2 pm	25.9%	12.5%	23.1%	22.9%
4 pm to 7 pm	25.9%	12.5%	7.7%	18.8%
Light Conditions*				
Daylight	63.0%	50.0%	61.5%	60.4%
Dark (Street Lights On)	37.0%	50.0%	38.5%	39.6%
Weather Conditions*				
Clear	77.8%	87.5%	84.6%	81.3%
Rain	18.5%	12.5%	15.4%	16.7%
Snowy	3.7%	0.0%	0.0%	2.1%
Surface Conditions*				
Dry	70.4%	75.0%	84.6%	75.0%
Wet	25.9%	12.5%	15.4%	20.8%
Icy	0.0%	12.5%	0.0%	2.1%

* Severity, Accident Type, Time of Day, Rush Hours, Light, Weather, and Surface Conditions Data only refers to Reportable Crashes

Source: City of Philadelphia Streets Department Crash Report, 2001-2003 the incident are recorded. Therefore, this data could not be evaluated in detail. However, the fact that there were 146 crashes in a three-year period is very important to take into consideration when determining potential improvements to this location. The high number of non-reportable crashes, where both injury and property damage are at a minimum, may indicate a large number of low-speed crashes such as rear-end, sideswipes, or hitting fixed objects. These may be the result of high congestion, long traffic queues or inadequate turning radii. In 2001, there were 27 reportable accidents, which account for 56.2 percent of the entire period. There is a 70 percent decrease between 2001 and 2002. The dramatic decrease may be attributed to minor improvements in the intersection that occurred that year such as repaving and the addition of the channelized right-turn lane on 34th Street northbound. Although there was a 63 percent increase in accidents from 2002 to 2003, the 13 crashes in 2003 are still less than half of that in 2001.

The most common type of accident is angle accidents, which account for 29.2 percent of the total accidents. Angle crashes, left-turn crashes and head-on events involve opposing traffic. Same direction rear end (22.9 percent), head-on (16.7 percent), left-turn (8.3 percent,) and sideswipe (6.3 percent) collisions are other common types of incidents at this location. The remaining 16.6 percent include crashes involving animals, fixed objects, and pedestrians.

Figure 10 graphically displays a collision diagram of these accidents. A brief summary of the crash records is located in **Appendix C**. The purpose of the collision diagram is to pictorially represent different types of crashes that have occurred and is useful in identifying accident patterns and trends. The map clearly indicates the high number of angle and head-on crashes occurring within the middle of the intersection during turning movements. Rear-end accidents generally take place while vehicles are stopped and waiting at the traffic light.

At this intersection, 60.4 percent of the accidents occurred during daylight hours. Weather and surface conditions do not seem to play major roles in causing collisions, with 81.3 percent of the crashes occurring with clear conditions and 75.0 percent on dry surface road.

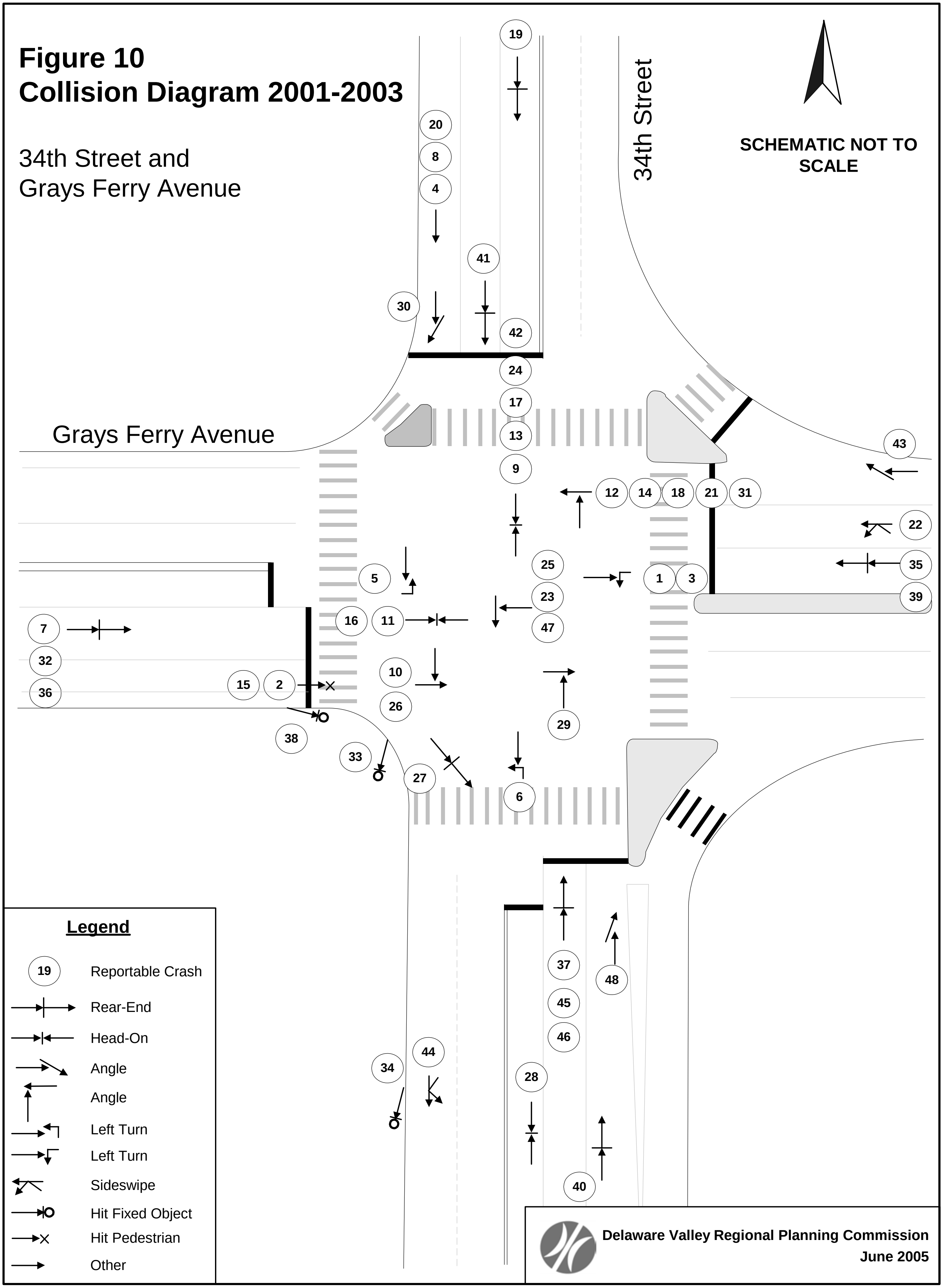
Both the evening (4 PM to 6 PM) and midday (11 AM to 2 PM) peak periods experienced high rates of accidents, with 18.4 percent and 22.9 percent respectively. Only 10.4 percent of the accidents occurred during the morning peak period.

Turning Movement Counts

A turning movement count of the intersection was conducted during the AM and PM peak periods in January 2005**. Figure 11** displays the counts for the peak hours, which are 7:45-8:45

Figure 10
Collision Diagram 2001-2003

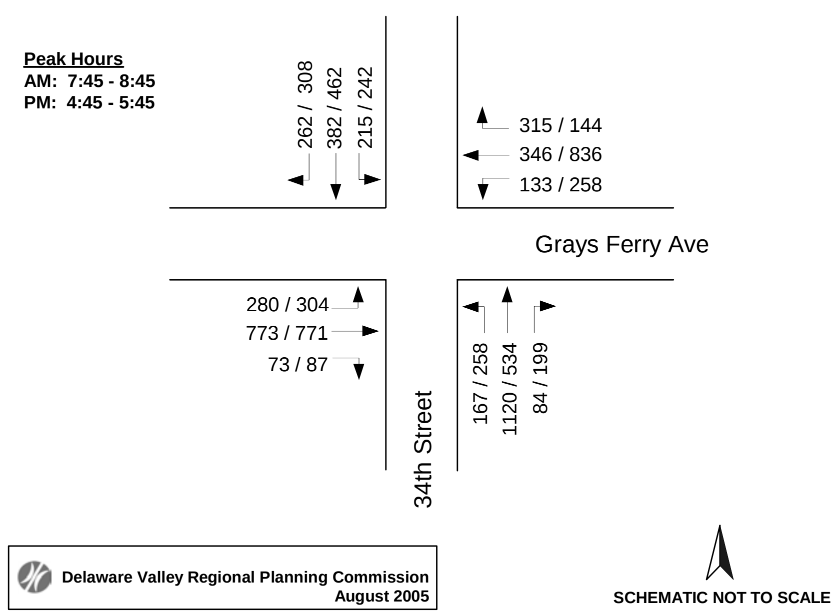
34th Street and
Grays Ferry Avenue



AM and 4:45-5:45 PM. The data shows that the AM peak period on the 34th Street northbound approach has the highest volume of traffic with 1,120 vehicles moving through the intersection. From this same direction, there are 167 vehicles making a left and 84 making a right-turn. Southbound on 34th there are 382 through movements, 262 right-turns and 215 left-turns that may conflict with the northbound traffic. The heavy volumes of northbound traffic are vehicles heading into University City, which is a major employment and academic center in Philadelphia. Additional traffic heading to University City include 280 left-turns from Grays Ferry Avenue eastbound and the 316 right-turns from Grays Ferry Avenue westbound. In addition to the traffic heading to University City, Grays Ferry Avenue eastbound has 773 vehicles moving through the intersection with 73 vehicles making a right. The through traffic conflicts with 133 left-turns from westbound Grays Ferry Avenue.

Figure 11 34th Street and Grays Ferry Avenue

Existing Peak Hour Turning Movement Counts AM / PM



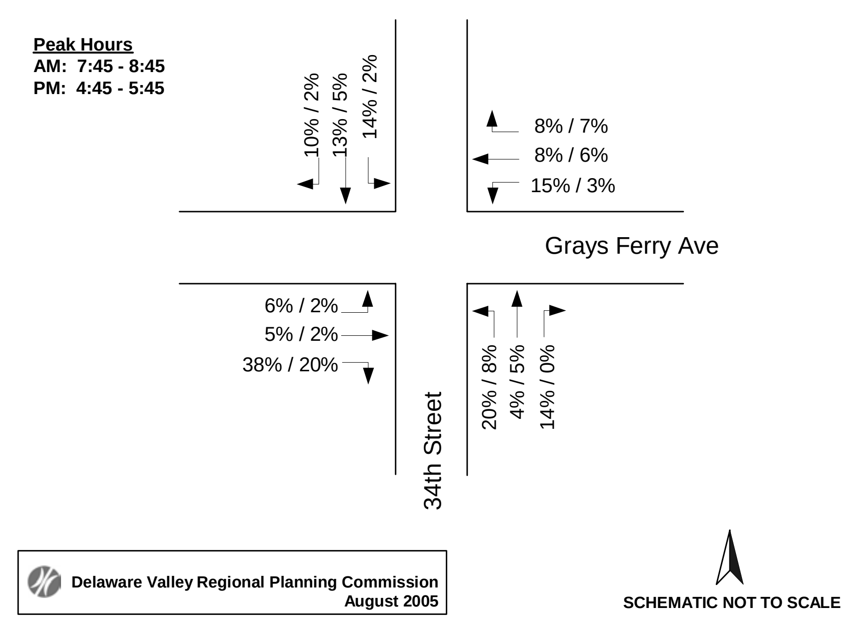
In the afternoon, Grays Ferry Avenue carries the majority of traffic. In the eastbound direction, there are 771 through movements, 304 left-turns and 87 right-turns. Westbound there are 836 through vehicles, 258 left-turns and144 right-turns. On 34th Street, each direction carries about 1,000 vehicles into the intersection. Throughout the intersection, there is a high volume of left-turns. These may contribute to the high number of angle accidents in the intersection. The left-turns also contribute to the rear end accidents, as other vehicles maneuver into adjacent travel lanes to avoid a stopped turning vehicle. Complete manual turning movement counts for this location can be seen in **Appendix C.**

Level of Service

A level of service (LOS) analysis was conducted for the Grays Ferry Avenue/34th Street intersection for the morning and afternoon peak hours. Traffic volume data, along with data from the traffic signal timing & operation plan, are analyzed to determine the LOS. All the data was input into Synchro Software, a traffic signal optimization program used to perform the level of service. For this location, the Philadelphia Streets Department - Traffic Engineering asked DVRPC to perform a LOS analysis and to obtain results in both Highway Capacity Software format and the Synchro software format. The data described below is from the Synchro Analysis; however **Table 8** displays the results for both Synchro and HCS. Complete analysis data for both Synchro and HCS for the existing conditions is located in **Appendix C**.

Heavy vehicle percentages have an effect on an intersection's level of service. Concerns were raised by the Philadelphia Streets Department regarding the percentages of trucks at this location. DVRPC revisited the intersection and conducted manual turning movement vehicle classification counts for this intersection during the AM and PM peak hours. **Figure 12** displays the percentage of trucks for each of the turning movements. These percentages were used in the level of service analysis.

Figure 12
34th Street and Grays Ferry Avenue
Percentage of Trucks
AM / PM Peak Hour Turning Movement Counts



The overall intersection LOS in the morning peak hour is LOS E, with 68.1 seconds of vehicle delay. Northbound 34th Street and Grays Ferry Avenue eastbound both experience an undesirable LOS F, with vehicle delays of 96.0 and 88.7 seconds respectively. The other opposite approaches, 34th

Street south and Grays Ferry Avenue west have LOS C with less than 35 seconds delay.

The intersection continues to operate at a LOS E, with 69.5 seconds of delay in the PM peak. Both directions of Grays Ferry Avenue have a LOS F, with 90.7 seconds of delay on the eastbound approach and 103.9 seconds of delay on the westbound approach. In the afternoon peak hour, 34th Street northbound has a LOS D, with 36.9 seconds of delay, while the southbound leg operates at a LOS C.

Opportunity and Constraints

Several factors limit the possibility of expanding the right-of-way of the intersection. First, this entire intersection is elevated over a set of Conrail railroad tracks, and much of the southwest corner is below grade. Next, there are vacated railroad tracks with accompanying right-of-way that flanks the south side of Grays Ferry Avenue.

Vacated railroad tracks and right-of-way along Grays Ferry Ave. eastbound approach.

Several years ago during a repaving project, a channelized right-turn lane was added to the northbound approach of 34th Street, and at that time, the Philadelphia Streets Department attempted to also channelize the eastbound Grays Ferry Avenue approach. However, they were unable to due to property ownership issues. Currently, the city has condemned the right-of-way in question. There is a legal dispute over the value of the property and an uncertainty about when the property will become available for city use. However, when this issue is finally solved, the city should be able to use this land for road improvements.

Another limiting factor at this intersection is the University Bridge that crosses the Schuylkill River. This is located approximately 880 feet north of the intersection on 34th Street. This bridge is a drawbridge with narrow lanes. This limits the prospect of adding capacity to the 34th Street southbound approach.

Potential Improvement Scenarios

Based on the technical analysis of the existing conditions and suggestions from the local municipal officials, a set of potential improvement scenarios has been developed to determine the possible effective improvements that will reduce congestion and accidents. For each improvement scenario, Synchro software analysis was used to determine the new level of service and amount of delay. As mentioned previously, the Philadelphia Streets Department - Traffic Engineering asked DVRPC to perform a LOS analysis and obtain results in both Highway Capacity Software format and the Synchro software format. The data described below is from the Synchro Analysis; however **Table 8** displays the results for both Synchro and Highway Capacity Software. Synchro LOS analysis and HCS data for potential improvement scenarios can be found in **Appendix C**.

Several short-term scenarios were considered for this intersection. Based on requests from the Philadelphia Streets Department - Traffic Engineering, several scenarios, which included optimizing the signal, were run at different cycle lengths. Each of the scenarios may be implemented quickly and efficiently, which is a priority of the city as the South Street Bridge Reconstruction project nears.

Scenario 1 examines optimizing the signal timing with the existing 90-second cycle length. This low-cost improvement does not require any right-of way acquisition and may be implemented immediately. By optimizing this signal, the operation and efficiency of this intersection is enhanced. In both the AM and PM peaks, the overall LOS of the intersection improves from LOS E to D, with decreases in vehicle delays by over 20 seconds. In the AM peak, 34th Street northbound shows the greatest improvement from a LOS F to a D and a decrease in vehicle delay from 96 seconds to 52.3 seconds. The Grays Ferry Avenue eastbound approach also improves in the AM from a LOS F to a D and experiences a decrease in delay by about 39 seconds. To achieve these improvements in the AM peak, both Grays

TABLE 8
Peak Hour Level of Service (LOS) Analysis
34th Street and Grays Ferry Avenue

Improvement Scenario	Direction of Travel	Peak AM Hour and Peak PM Hour LOS with Average Delay / Vehicle — SYNCHRO				Peak AM Hour and Peak PM Hour LOS with Average Delay / Vehicle — Highway Capacity Manual			
		AM Peak		PM Peak		AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)	LOS	Delay (sec)	LOS	Delay (sec)
Existing Conditions	Grays Ferry Ave EB	F	88.7	F	90.7	F	102.6	F	111.7
	Grays Ferry Ave WB	C	30.4	F	103.9	D	42.0	F	126.1
	34^{th} Street NB	F	96.0	D	36.9	F	107.1	D	40.3
	34^{th} Street SB	C	34.3	C	28.4	D	47.1	D	37.0
	Intersection	E	68.1	E	69.5	F	80.5	D	84.5
1. Signal Timing Optimization 90 Second Cycle Length	Grays Ferry Ave EB	D	49.8	D	38.1	E	57.5	D	45.2
	Grays Ferry Ave WB	D	49.4	D	48.7	E	69.4	E	57.3
	34^{th} Street NB	D	52.3	D	48.8	E	56.6	E	55.1
	34^{th} Street SB	D	36.0	E	64.9	D	46.6	F	88.0
	Intersection	D	47.6	D	49.7	E	57.2	E	60.9
2. Signal Timing Optimization Changes AM & PM cycle length to 75	Grays Ferry Ave EB	D	47.5	D	40.1	E	56.1	D	47.6
	Grays Ferry Ave WB	D	36.6	D	36.7	D	50.8	D	45.0
	34^{th} Street NB	E	56.1	D	49.8	E	61.6	E	56.8
	34^{th} Street SB	D	37.1	E	64.7	D	48.1	F	89.1
	Intersection	D	45.9	D	46.8	E	55.1	E	58.6
3. Signal Timing Optimization 120 Second Cycle Length	Grays Ferry Ave EB	D	53.5	D	49.4	E	63.6	E	59.1
	Grays Ferry Ave WB	E	64.7	D	50.2	F	90.9	E	60.9
	34^{th} Street NB	D	50.2	E	59.7	E	56.4	E	68.1
	34^{th} Street SB	D	36.9	E	57.0	D	49.5	E	78.7
	Intersection	D	51.1	D	53.4	E	63.7	E	66.0
4. Add Channelized Right Turn Lane on Grays Ferry Ave EB w/ existing	Grays Ferry Ave EB	E	55.7	E	67.6	E	62.1	F	83.1
	Grays Ferry Ave WB	C	29.1	F	103.9	D	39.7	F	126.0
	34^{th} Street NB	F	99.2	D	36.9	F	111.0	D	40.3
	34^{th} Street SB	C	30.1	C	28.4	D	41.8	D	37.0
	Intersection	E	58.9	E	63.0	E	69.0	E	76.5
5. Add Channelized Right Turn Lane on Grays Ferry Ave EB w/ 90 sec optimized cycle	Grays Ferry Ave EB	D	41.5	D	36.7	D	47.9	D	42.2
	Grays Ferry Ave WB	D	49.4	D	46.1	E	69.3	D	54.9
	34^{th} Street NB	D	52.3	D	48.8	E	56.6	E	55.1
	34^{th} Street SB	D	36.0	E	64.9	D	46.6	F	88.0
	Intersection	D	45.4	D	48.8	D	54.7	E	59.8

Ferry Avenue westbound and 34th Street southbound experience increases in delay of 19 seconds and 2 seconds respectively.

In the peak PM hour, Grays Ferry Avenue eastbound and westbound improves significantly with an improvement from a LOS F to a D and a decrease in delay by over 50 seconds in each direction. However, both approaches on 34th Street suffer increases in delay. The southbound approach deteriorates from a LOS C to an E with an increase in delay of over 36 seconds. The increases in LOS are attributed to change phasing that occurs in this new timing plan.

Although this scenario shows overall improvement in level of service, there are negative effects to some of the approaches. The benefit of this scenario comes from the major improvements in LOS to the most congested approaches that are currently failing under existing conditions. The Synchro software works to optimize the overall level of service for the intersection. It evens out the delay among all of the approaches, so that one approach might benefit at the expense of another.

Scenario 2 is another low-cost alternative that involves optimizing the signal timing, and allows Synchro to determine the appropriate cycle length. With this scenario, the cycle length changes in the AM and PM peak periods to 75 seconds from the existing 90 seconds. This scenario improves the operation of the intersection to a LOS D, with 45.9 seconds of delay in the AM and 46.8 seconds of delay in the PM. This is a decrease in vehicle delay by over 20 seconds in both of the peak periods. Although this scenario offers a slight improvement over Scenario 1, the overall results are very similar. In the AM peak, the two most congested legs, Grays Ferry Avenue eastbound and 34th Street northbound, have significant improvements in LOS and vehicle delay by over 40 seconds. However, the other two approaches have slight increases in delay. Grays Ferry Avenue westbound increases to a LOS D and by 6.2 seconds of vehicle delay, and 34th Street southbound increases to a LOS D and by 3 seconds vehicle delay.

In the peak PM period, both approaches of Grays Ferry Avenue are improved from a LOS F to a D. The eastbound approach decreases in vehicle delay by 50.6 seconds while the westbound approach decreases by 67.2 seconds. Conversely, both approaches on 34th Street suffer increases in delay. The southbound approach deteriorates from a LOS C to an E, with an increase in delay of over 36 seconds.

Although this scenario shows overall improvement in level of service over existing conditions and Scenario 1, there are negative effects to some of the approaches. The benefit of this scenario comes from the major improvements in LOS to those approaches that are currently failing in existing conditions.

Scenario 3 examines optimizing the signal timing with a 120-second cycle length. At neighboring intersections the cycle lengths are all 60 seconds, therefore the city is interested in looking at how the signal operates at twice that cycle length. The reason for this particular cycle length is that it is generally easier to coordinate signals if it is a multiple of the existing grid cycle length (60 seconds).

The results for this scenario are very similar to scenarios 1 and 2. The LOS of the overall intersection improves from a LOS E to a D, and the overall intersection delay is 51.1 seconds in the AM and 53.4 seconds in the PM. This delay is slightly greater than both scenarios 1 and 2. Like the other two scenarios, the approach LOS for Grays Ferry Avenue eastbound and 34th Street northbound improves in the AM peak. In the PM, both approaches of Grays Ferry Avenue improve, while 34th Street approaches declines in LOS.

Overall, this scenario improves the LOS of the intersection over the existing conditions. Although there is an improvement, the delay for both AM and PM periods is slightly greater (six seconds) than scenarios 1 and 2.

Scenario 4 adds a channelized right-turn lane on Grays Ferry Avenue for the eastbound approach using the existing signal timing. This scenario investigates a physical improvement that will potentially address the safety issues of this location. The use of a channelized right-turn can improve both the capacity and safety at an intersection by separating vehicle conflict points. Vehicles' turning right are cleared from the intersection and through traffic is able to move through the intersection without slowing down for right-turning vehicles. Right-of-way acquisition is needed for additional capacity. Although there appears to be ample room, widening along Grays Ferry Avenue may be limited due to possible encroachment on the railroad overpass. If widening is not a problem, the additional lane will also help to improve the turning radius to accommodate the trucks.

Using the existing signal timing, the overall operation of the intersection remains at a LOS E. There is an improvement in vehicle delay by 10 seconds in the AM and seven seconds in the PM. The only improvement to the intersection occurs on the Grays Ferry Avenue eastbound approach. Since right-turning vehicles are now removed from the through travel lanes, the LOS improves from a F to an E, with a decrease in vehicle delay by 33 seconds in the AM and by 23 seconds in the PM. All of the other approaches are unaffected by the addition of the channelized right-turn lane. During the AM peak hour, Northbound 34th Street continues to fail with LOS F and 99 seconds of delay, while southbound 34th Street operates at a C with 30 seconds of delay and Grays Ferry westbound operates at a LOS C, with 29 seconds of delay.

Scenario 5 looks at the same improvements as Scenario 4, channelized right-turn lane on Grays Ferry Avenue eastbound, but uses a 90-second optimized signal timing. The advantages of this scenario are physical improvements to the intersection make it safer.

Compared to existing conditions, the 34th Avenue and Grays Ferry Avenue intersection improve from LOS E to D, with delay falling from 68.1 seconds to 45.3 seconds during the morning peak. This improvement is primarily a factor of the optimized signal timing. As noted in scenarios 1, 2 and 3, the LOS of 34th Street northbound experiences the greatest improvement of all the approaches. There is an improvement from a LOS F to D and a 44-second drop in delay. The combination of the signal timing change and the channelized right-turn lane on eastbound Grays Ferry Avenue improves this approach from a LOS F to a LOS D, with a decrease in delay by 47.5 seconds. As with the scenarios 1, 2 and 3, the approach LOS for eastbound and westbound Grays Ferry Avenue and 34th Street southbound decline in the AM peak to accommodate for improvements of the other approaches.

During the afternoon peak, the intersection improves from LOS E to D with a 20.7-second fall in delay. Both approaches on Grays Ferry Avenue have significant improvements from a LOS F to D. The eastbound approach decreases in vehicle delay by 58 seconds, while the westbound approach declines by 54 seconds. Both approaches of 34th Street declined in LOS, with the southbound approach increasing by over 36 seconds.

Recommendations

The complexity of this location creates many challenges to improving both the safety and operation of this intersection. The future South Street Bridge reconstruction project will increase congestion at this location. Both 34th Street and Grays Ferry Avenue will be used as alternate routes as motorists head into and out of University City. This reconstruction project emphasizes the priority of improving this location.

A short-term recommendation that can improve this intersection is to optimize the traffic signal timing of 34th Street and Grays Ferry Avenue. This is a low-cost improvement that can be implemented quickly. Scenarios 1, 2, and 3 all look at optimizing the signal timing of the intersection. Each of the scenarios provides considerable improvement from the existing conditions. The Synchro software concentrated on improving the overall LOS of the intersection. The trade-off is that some approaches improve dramatically while other approaches increase vehicle delay. For example, in the AM peak, 34th Street northbound decreases vehicle delay by over 40 seconds, at the expense of Grays Ferry Avenue westbound that increases its delay from 6 to 19 seconds. Since each scenario improves the existing conditions, the Philadelphia Streets Department - Traffic Engineering will need to determine which cycle length they prefer and implement that timing plan.

Several short-term improvements and enhancements can be made to the location that would have a positive impact on the safety and operation of this intersection. These improvements are not incorporated into the Synchro analysis, but can be integrated as part of any solution chosen for this intersection, such as re-striping of all pavement markings to identify turning movements and reduce driver confusion. This includes repainting or possibly relocating the stop bars to help avoid conflicts in the intersection. By relocating the stop bars in advance of the intersection, it keeps the opposing lanes at intersections free, allowing trucks to turn wide and thereby allowing smaller curb radii. Advanced stop lines benefit pedestrians, as the pedestrians and drivers have a clearer view and more time to assess each other's intentions when the signal phase changes.

All pedestrian crosswalks and striping also need to be repainted. Other pedestrian amenities such as signal heads and push buttons should be upgraded at this location to assist pedestrians in making a reasonably safe crossing. This helps to minimize vehicle-pedestrian conflicts. The use of "Walk/Don't Walk" pedestrian signals is important with the complexity of the signal timing and the ability of vehicles to make dedicated left-turns. Also, with the width of these streets, pedestrian countdown signal indication would be helpful to indicate appropriate times for pedestrians to cross.

In addition to optimizing the signal, a longer-term approach of implementing physical improvements is needed to improve the efficiency and overall safety of the intersection. Although there were only 48 reportable crashes at this intersection from 2001-2003, there were an additional 146 non-reportable crashes. This indicates that a large number of these incidents occur under low speed, high congestion, long traffic queue conditions. Common crashes that may occur are same direction rear-end or sideswipes. To reduce these crashes, physical improvements are needed, such as improving the turning radii to accommodate large trucks. Turning radii should be designed to accommodate the turning path of a vehicle to avoid encroachment on pedestrian facilities and opposing lanes of travel. Existing channelized concrete islands should also be repaired.

In addition to improving turning radii, Scenario 5 should be implemented to add a channelized right-turn lane on Grays Ferry Avenue eastbound. This scenario includes the optimization of the traffic signal. This scenario will remove right-turn movements from the travel lanes on this approach of the intersection. By reducing the conflicts of vehicles stopping to make a right-turn, you also reduce the possibility of rear-end crashes. Once the legal dispute over the value of the property adjacent to Grays Ferry Avenue is resolved, this property can be used for this right-turn lane. A more in-depth technical engineering study of this right-of way and the impact of the below-grade railroad underpass is needed to determine the feasibility of this option.

To fully optimize the signal at this location, an appropriate coordinated closed-loop traffic signal system in this section of Philadelphia should be investigated. This option will require the city to upgrade the local grid system to a fiber network.

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BURLINGTON

5 RIVERTON ROAD (CR 603) AND BRANCH PIKE (CR 606) RIVERTON ROAD (CR 603) AND PARRY ROAD BRANCH PIKE (CR 606) AND PARRY ROAD

Cinnaminson Township, Burlington County

Location Description

The study location is the Riverton Road (CR 603)/Branch Pike (CR 606)/Parry Road intersections. The study location is shown on **Map 4.** Riverton Road and Branch Pike are county-owned and maintained roadways. Parry Road is municipal owned. The study location encompasses three unsignalized intersections (Riverton Road and Branch Pike, Riverton Road and Parry Road, and Branch Pike and Parry Road) forming a triangle with less than 90-degree angles. The intersections are stop sign controlled. The functional classification of the two county roads is urban minor arterials.

Looking west on Riverton Rd (CR 603) at Branch Pike (CR 606) intersection

CR 606 runs in an east-west direction from the Delaware River in Palmyra to US 130; from there it runs in a northeasterly direction to where it intersects with CR 603. CR 603 also runs in an east-west direction from the Delaware River in Riverton crossing US 130 past the study location into Moorestown and points east. Parry Road carries traffic in a north-south direction from Delran to where it ends at the intersection with CR 606. Traffic traveling along both county routes provides direct access to US 130 to the west of the study location and NJ 38 to the east, which intersects with I-295.

Riverton Rd. (CR 603), Branch Pike (CR 606), and Parry Avenue

Cinnaminson Township, Burlington County, NJ

2005
Regional Congestion
and Accident
Mitigation Program

Map 4

N

0 25 50 100
Feet

Delaware Valley
Regional Planning Commission
August 2005

Date of Aerial Photography: Spring, 2000

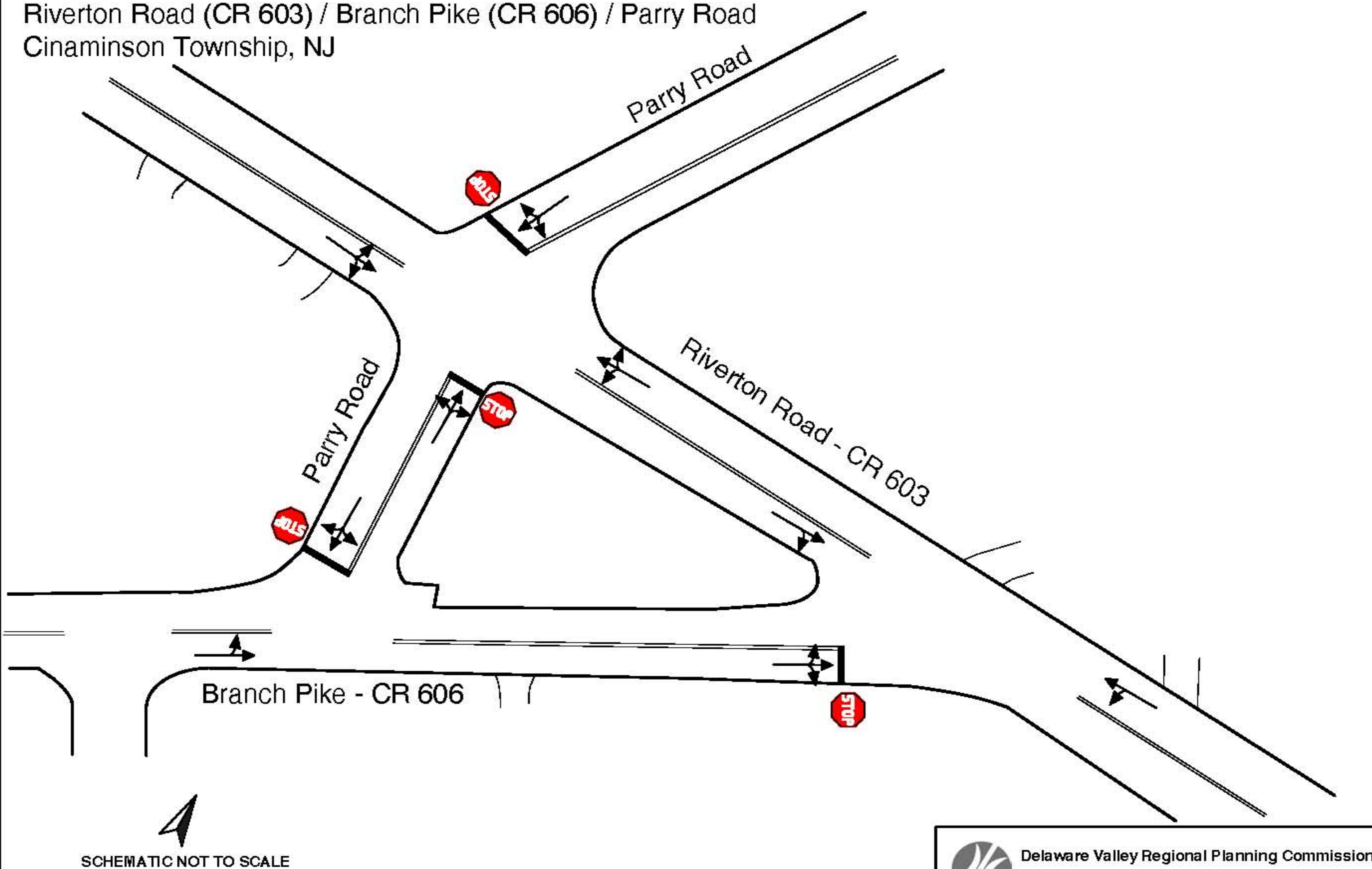
Looking west on Riverton Rd (CR 603) & Parry Rd intersections

Looking north at Parry Rd at Branch Pike (CR 606)

Figure 13 shows the existing lane configuration of the intersection. Riverton Road (CR 603) has one 12-foot lane in each direction with shoulders of approximately 5 feet. The speed limit is 40 MPH east of Parry Road and 35 MPH west of this location. Branch Pike (CR 606) has one 15-foot lane in each direction with no shoulders. The speed limit on this roadway is 35 MPH. Parry Road has one 20-foot lane in each direction with a speed limit of 25 MPH. The land use at, and immediately surrounding, the study location is predominantly single-family residential. Several schools are located along CR 606 and 603 and in close proximity to the location.

Figure 13
Existing Lane Configuration

Riverton Road (CR 603) / Branch Pike (CR 606) / Parry Road
Cinaminson Township, NJ



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June 2005

Industrial Center of Cinnaminson is located on the other side of US 130 and Moorestown Industrial Park is on the Moorestown border with Cinnaminson. The property between the three intersections is a drainage basin.

Existing Conditions

Identified Problems

All three intersections are angled or skewed. This leads to a number of existing problems and potential problems.

Traffic entering the intersection at Riverton Road from Parry Road experiences a problem with sight distance. This problem is caused by the angle of the roadway at the intersection and is compounded by a cement and stone column situated in the front of the property on the northwest corner of the intersection. Vegetation on the southwest corner impedes the sight of northbound motorists on Parry Road at this intersection.

Sun glare is a problem at the study location as observed on eastbound Riverton Road during the morning peak and westbound during the afternoon peak period.

Left-turning traffic from Riverton Road onto Branch Pike does not slow to make the turn. Due to the angle of the intersection, a right-turning vehicle from Branch Pike onto Riverton Road has to enter the intersection to see oncoming traffic.

Traffic volumes contribute to the problems associated with these intersections; there are a large number of vehicles making turn movements at both Riverton Road intersections.

Crash Analysis

According to the Burlington County Engineering Office database, there were 32 crashes recorded at the study location in the three-year period from 2000 to 2002. As shown in **Table 9**, the number of crashes varied over the three-year period – an increase of eight percent between 2000 and 2001; and a 25 percent decrease between 2001 and 2002. A brief summary of the crash records is located in **Appendix D**.

Table 9 Intersection Reportable Crash Summary (2000-2002)
Riverton Road (CR 603) and Branch Pike (CR 606) and Parry Road Intersection

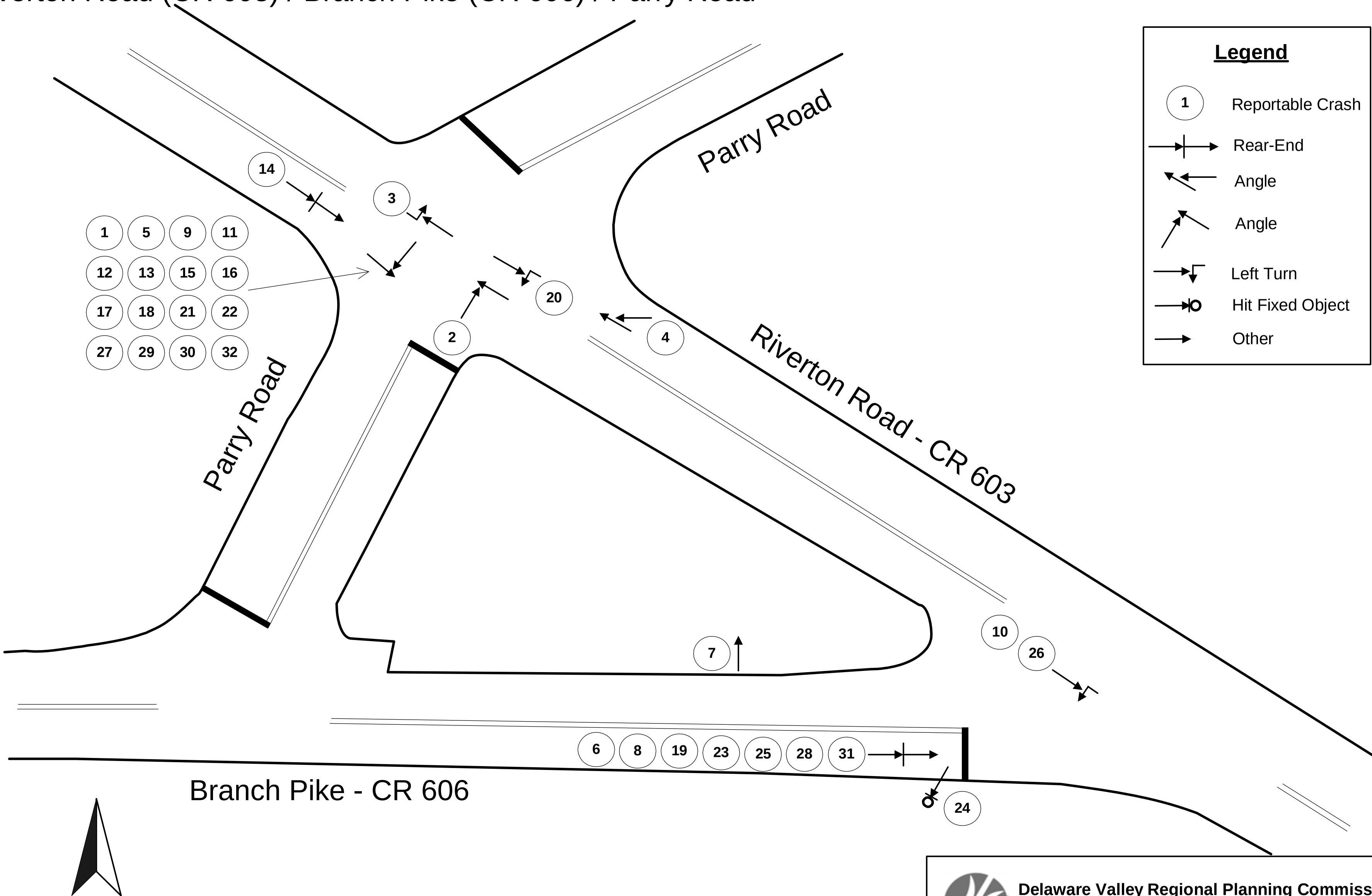
	2000	2001	2002	Total
Total	11	12	9	32
Severity				
Injuries	9	7	4	20
Fatalities	0	0	0	0
Accident Type				
Angle	54.5%	66.7%	44.4%	56.3%
Left Turn	18.2%	8.3%	11.1%	12.5%
Same Direction - Rear End	18.2%	25.0%	33.3%	25.0%
Fixed Object	0.0%	0.0%	11.1%	3.1%
Other	0.0%	0.0%	11.1%	3.1%
Time of Day				
Midnight to 6 am	9.1%	0.0%	11.1%	6.3%
6 am to Noon	18.2%	33.3%	22.2%	25.0%
Noon to 6 pm	54.5%	50.0%	33.3%	46.9%
6 pm to Midnight	18.2%	16.7%	33.3%	21.9%
Rush Hours				
6 am to 9 am	3.1%	3.1%	6.3%	12.5%
11 am to 2 pm	9.4%	3.1%	0.0%	12.5%
4 pm to 7 pm	15.6%	9.4%	12.5%	37.5%
Light Conditions				
Daylight	81.8%	66.7%	55.6%	68.8%
Dawn or Dusk	0.0%	8.3%	0.0%	3.1%
Dark (Street Lights On)	18.2%	25.0%	44.4%	28.1%
Weather Conditions				
Clear	90.9%	83.3%	88.9%	87.5%
Rain	9.1%	8.3%	11.1%	9.4%
Snowy	0.0%	8.3%	0.0%	3.1%
Surface Conditions				
Dry	90.9%	83.3%	88.9%	87.5%
Wet	9.1%	8.3%	11.1%	9.4%
Snowy	0.0%	8.3%	0.0%	3.1%

Source: Burlington County Engineering Office Database, 2000-2002

Twenty injuries were recorded over the three-year period with the highest number of nine in 2000 and the lowest four in 2002. **Figure 14** graphically displays a collision diagram of these accidents. Angle crashes makes up 56 percent of all crashes over the three-year period. All 18 angle crashes occurred at the Riverton Road / Parry Road intersection, this is most likely due to

Figure 14
Collision Diagram 2000 - 2002

Riverton Road (CR 603) / Branch Pike (CR 606) / Parry Road



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sight-distance problems experienced at the intersection, along with traffic volumes and motorist behavior. Same direction-rear end crashes had the second highest number recording eight, which is 25 percent of the total number. Of the eight crashes, seven occurred at or near the Riverton Road/Branch Pike intersection, a result of queuing at the intersection and poor geometry. Left-turn crashes occurred at both intersections on Riverton Road, two at each. Fifty percent of the crashes occurred during weekday morning (6 AM-9 AM) and afternoon (4 PM-7 PM) peak periods, with the afternoon peak three times higher than the morning peak. The possibility exists that sun glare played a role in these accidents. More than 68 percent of the crashes happened during daylight; and 87.5 percent happened on days with clear weather and dry road surface conditions.

Turning Movement Counts

Turning movement counts were taken at the study location in January 2005 for morning peak between the hours of 6 AM and 9 AM and afternoon peak between the hours of 4 PM and 7 PM. It was determined the peak morning hour is 7:30 AM to 8:30 AM and peak afternoon hour is 4:45 PM to 5:45 PM. Complete manual turning movement counts for each of the intersections in this study can be seen in **Appendix D.**

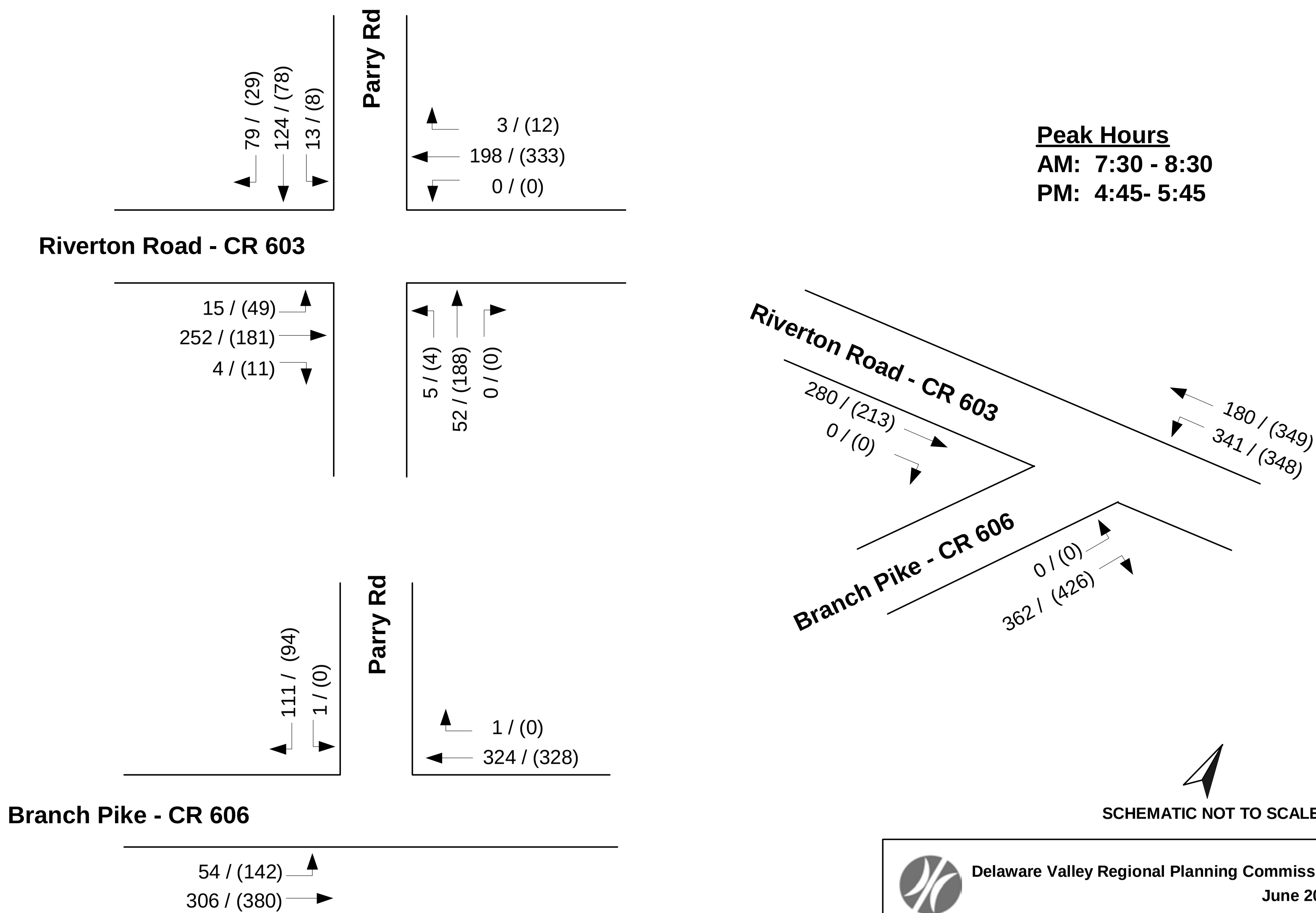
As shown in **Figure 15,** Riverton Road and Branch Pike each carries approximately 800 vehicles in the study location during the morning peak hour and over 900 during the afternoon peak hour. Parry Road carries an estimated 300 vehicles during the morning peak hour and approximately 400 vehicles for the afternoon peak hour.

At the Branch Pike/Riverton Road intersection, the right-turn from Branch Pike has the dominant movement with 362 vehicles in the morning peak hour and 426 vehicles in the afternoon hour. High volumes on Riverton Road constrain left-turn movements from westbound Riverton Road onto Branch Pike during the peak period. Approximately 350 vehicles make that left-turn during both morning and afternoon peak hour against eastbound through traffic volumes of 280 and 213 during morning and afternoon peak hours respectively.

At the Riverton Road/Parry Road intersection, the dominant movements are through movements on Riverton Road in both directions. Eastbound volumes are 252 and 181 for morning and afternoon peak hours respectively, and 198 and 333 westbound for morning and afternoon peak hours respectively. Parry Road movements through the intersection are the dominant movement and this presents conflicts with the high volume of traffic on Riverton Road, resulting in a large number of crashes at this location.

Figure 15
Existing Peak Hour Turning Movement Counts AM / (PM)

Riverton Road (CR 603) / Branch Pike (CR 606) / Parry Road



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The Branch Pike/Parry Road intersection has high through volumes on Branch Pike, 324 and 328 in the morning and afternoon peak hour westbound, and 306 and 380 eastbound morning and afternoon peak respectively. This conflicts with both left-turn traffic from Branch Pike onto Parry Road and right-turning traffic from Parry Road onto Branch Pike, with the potential for accidents at this location.

Level of Service

The Level of Service (LOS) analysis was conducted using Synchro on each of the intersections for the AM and PM peak periods. Currently, the intersections and their individual approaches appear to be operating efficiently according to the LOS analysis. **Table 10** shows the existing LOS analysis. At the intersection of Riverton Road and Parry Road, the overall intersection LOS is A for both morning and afternoon peaks with 6.2 seconds of delay in the morning peak and 9.5 seconds in the afternoon peak. The Parry Road approaches at Riverton Road operate at LOS C with 15 seconds of delay in the morning peak and LOS D with 29 seconds of delay in the afternoon peak northbound. Southbound operates at LOS C in both the morning and afternoon peaks with 16.9 seconds of delay in the morning peak and 19.5 seconds in the afternoon.

For the two other intersections of this study area, Synchro was able to produce a LOS using the Intersection Capacity Utilization (ICU) method. Because these are unsignalized intersections with only one approach that is stop-controlled, this method was chosen. The ICU LOS gives insight into how an intersection is functioning and how much extra capacity is available to handle traffic fluctuations and incidents. ICU is not a value that can be measured with a stopwatch, but it does give a good reading on the conditions that can be expected at the intersection. Under this method, the intersection level of service at Branch Pike at Riverton Road is LOS A in both the morning and afternoon peaks. At Branch Pike at Parry Road, the morning peak has a LOS A, while the afternoon peak is LOS B.

Synchro LOS analysis data for both existing and potential improvement scenarios can be found in **Appendix D**.

TABLE 10
Peak Hour Level of Service (LOS) Analysis
Riverton Road (CR 603) and Branch Pike (CR 606) and Parry Road

Improvement Scenario	Direction of Travel	Peak AM Hour and Peak PM Hour LOS with Average Delay / Vehicle				Direction of Travel	Peak AM Hour and Peak PM Hour LOS with Average Delay / Vehicle				Direction of Travel	Peak AM Hour and Peak PM Hour LOS with Average Delay / Vehicle			
		Riverton Road (CR 603) and Parry Road					**Riverton Road (CR 603) and Branch Pike (CR 606)**					**Branch Pike (CR 606) and Parry Road**			
Existing Conditions		AM Peak		PM Peak			AM Peak		PM Peak			AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)
	CR 603 EB	-	-	-	-	CR 606 EB	-	-	-	-	CR 606 EB	-	-	-	-
	CR 603 WB	-	-	-	-	CR 603 NB	-	-	-	-	CR 606 WB	-	-	-	-
	Parry Rd NB	C	15.0	D	29.3	CR 603 SB	-	-	-	-	Parry Rd SB	-	-	-	-
	Parry Rd SB	C	16.9	C	19.5										
	Intersection	A	6.2	A	9.5	Intersection	A		A		Intersection	A		B	
1. Add a traffic signal at the Parry Rd / Riverton Rd; Close Branch Pike to through traffic		AM Peak		PM Peak			AM Peak		PM Peak			AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)
	CR 603 EB	A	8.0	A	7.9	CR 606 EB	-	-	-	-	CR 606 EB	-	-	-	-
	CR 603 WB	A	9.6	A	9.5	CR 603 NB	-	-	-	-	CR 606 WB	-	-	-	-
	Parry Rd NB	B	19.3	B	17.0	CR 603 SB	-	-	-	-	Parry Rd SB	-	-	-	-
	Parry Rd SB	B	13.8	B	10.8										
	Intersection	B	12.7	B	12.0	Intersection	A		A		Intersection	B		C	
2. Actuated traffic signal at the Parry Rd / Riverton Rd; Branch Pike one-way eastbound traffic only		AM Peak		PM Peak			AM Peak		PM Peak			AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)
	CR 603 EB	A	6.3	A	5.6	CR 603 EB	-	-	-	-	CR 606 EB	-	-	-	-
	CR 603 WB	A	7.7	A	6.7	CR 603 WB	-	-	-	-	CR 606 WB	-	-	-	-
	Parry Rd NB	B	13.2	B	13.6	Parry Rd NB	-	-	-	-	Parry Rd SB	-	-	-	-
	Parry Rd SB	B	13.8	B	10.8										
	Intersection	A	8.8	A	7.4	Intersection	A		A		Intersection	B		C	
3. Pre-Timed traffic signal at the Parry Rd / Riverton Rd; Branch Pike one-way eastbound traffic only		AM Peak		PM Peak			AM Peak		PM Peak			AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)
	CR 603 EB	A	7.5	A	7.5	CR 603 EB	-	-	-	-	CR 606 EB	-	-	-	-
	CR 603 WB	A	9.0	A	8.9	CR 603 WB	-	-	-	-	CR 606 WB	-	-	-	-
	Parry Rd NB	B	14.7	B	16.0	Parry Rd NB	-	-	-	-	Parry Rd SB	-	-	-	-
	Parry Rd SB	B	16.4	B	15.3										
	Intersection	B	10.4	B	10.3	Intersection	A		A		Intersection	B		C	
4. Same as Scenario 2 with an estimated annual growth rate of 1.5% for 10 years		AM Peak		PM Peak			AM Peak		PM Peak			AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)		LOS	Delay (sec)	LOS	Delay (sec)
	CR 603 EB	A	6.6	A	5.9	CR 603 EB	-	-	-	-	CR 606 EB	-	-	-	-
	CR 603 WB	B	10.2	A	7.9	CR 603 WB	-	-	-	-	CR 606 WB	-	-	-	-
	Parry Rd NB	B	14.4	B	15.2	Parry Rd NB	-	-	-	-	Parry Rd SB	-	-	-	-
	Parry Rd SB	B	15.5	B	12.0										
	Intersection	B	10.6	A	9.0	Intersection	A		A		Intersection	C		D	

Opportunity and Constraints

With existing volumes, the intersections at the study location are not failing. However, a safety problem was also observed during the study team field view. Here is the opportunity to address this problem before it escalates.

Table 11 summarizes the crash data at the study location and compares it to the 2002 New Jersey Department of Transportation statewide accident summary for county road systems. The study location is above average for angle and left-turn collisions. Rear end collisions are comparable to the statewide average when aggregated. When taken by individual year, the 2002 percentage is higher than the state average.

Table 11 Statewide Crash Data Comparison

Type	Occurrence	Percentage (%)	NJDOT 2002 Statewide Average (%)
Angle	18	56.3	21.92
Same Direction – Rear End	8	25.0	29.04
Left Turn	4	12.5	5.89
Hit Fixed Object	1	3.1	11.13
Other	1	3.1	4.57
Total	**32**	**100.0**	

With increasing development in the region and the resulting traffic volume increase, there is the potential for an increase in the crash rate at the study location if not addressed. Many trips may be local, but Riverton Road and Branch Pike serve as the main east-west access arterials to major north-south highways in the region – US 130 and I-295. These roads also provide access to the River Line west of US 130. Land use in the area of the study location is predominantly residential; therefore, it can be assumed the majority of trips during peak periods are commuting trips.

Potential Improvement Scenarios

Conceptual location improvements were developed. A “do nothing” scenario is not being considered due to the dominance of safety issues at this location.

Scenario 1

Characteristics

- Add an actuated traffic signal at the Parry Road/Riverton Road intersection.
- Add a left-turn lane on westbound Riverton Road at Parry Road intersection.
- Add a right-turn lane at northbound Parry Road at Riverton Road intersection.
- Close the section of Branch Pike between Parry Road and Riverton Road to through traffic.
- Local traffic on closed section of Branch Pike access Riverton Road from Parry Road only.
- Relocate existing stop sign at Parry Road/Branch Pike intersection from southbound Parry Road approach to westbound Branch Pike (local traffic).

Advantages

- Eliminate uncontrolled left-turn from Riverton Road to Branch Pike.
- All right-turn movements from Branch Pike onto Riverton Road are eliminated which eliminates sight distance problems.
- All traffic on Riverton Road travels through a signalized intersection at Parry Road.
- Minimal right-of-way impacts.

Disadvantages

- Additional travel capacity needed for left-turn lanes.
- Potential transfer of problems associated with eastbound Branch Pike at the Riverton Road intersection to Parry Road/Riverton Road intersection.

Level of Service Analysis

A Synchro analysis was performed for this scenario using existing morning and afternoon peak hour data and can be found for each scenario in **Table 10**.

The overall intersection LOS for the Riverton Road/Parry Road intersection went from As in both the morning and afternoon peak for the existing conditions to Bs. There was improvement in all the approaches’ LOS. The intersection LOS at the Branch Pike/Parry Road intersection also deteriorated from A and B in the morning and afternoon peak respectively to B and C for this scenario. The results of the analysis for the northbound Parry Road approach is deceptive because the model is clearing just the traffic on northbound Parry Road and does not take into account the traffic queuing on Branch Pike waiting to go through the Riverton Road/Parry Road intersection. Although LOS deteriorated to Bs, it is still a more than acceptable LOS.

Scenario 2

Characteristics

- Add an actuated traffic signal at the Parry Road/Riverton Road intersection.
- Add a left-turn lane on westbound Riverton Road at Parry Road intersection.
- Convert the section of Branch Pike between Parry Road and Riverton Road to one-way eastbound traffic only.

  There is one house on this section with a driveway that would be affected by this change. However, this may be designed in such a way so that vehicles pulling out of this driveway could go in either direction. Therefore, a small segment of this section of Branch Pike up to the driveway could still be two-way traffic. If such a design is implemented, a stop sign at Branch Pike/Parry Road intersection would be needed on the westbound approach.)
- Reconfigure Branch Pike at the Riverton Road intersection to eliminate sight problems.
- Add stop sign for left-turning traffic only at southbound Parry Road approach at the Branch Pike intersection (local traffic).

Advantages

- Eliminate uncontrolled left-turns from Riverton Road to Branch Pike.
- Sight problems associated with making right-turns from Branch Pike onto Riverton Road are eliminated.
- All traffic on Riverton Road goes through signalized intersection at Parry Road.
- Minimal right-of-way impacts.

Disadvantages

- Additional travel capacity needed for left-turn lanes.
- Local residents on one-way section of Branch Pike may be inconvenienced.

Level of Service Analysis

A Synchro analysis was performed for this scenario using existing morning and afternoon peak hour data.

The overall intersection level of service for the Riverton Road/Parry Road intersection remained at LOS A for both morning and afternoon peaks. There was an increase in delay over the existing conditions in the morning peak and a decrease in the afternoon peak. All approaches will experience greater efficiency in both morning and afternoon peaks; LOS A and B are shown and seconds of delay are lower than existing. The Branch Pike/Parry Road intersection goes from LOS A to B in the morning peak and from LOS B to C in the afternoon peak, while at the Riverton Road/Branch Pike intersection the LOS remains A during both peaks.

Scenario 3

Characteristics

- Add a pre-timed traffic signal at the Parry Road/Riverton Road intersection.
- Add a left-turn lane on westbound Riverton Road at Parry Road intersection.
- Convert the section of Branch Pike between Parry Road and Riverton Road to one-way eastbound traffic only.

  There is one house on this section with a driveway that would be affected by this change. However, this may be designed in such a way so that vehicles pulling out of this driveway could go in either direction. Therefore, a small segment of this section of Branch Pike up to the driveway could still be two-way traffic. If such a design is implemented, a stop sign at Branch Pike/Parry Road intersection would be needed on the westbound approach.

- Reconfigure Branch Pike at the Riverton Road intersection to eliminate sight problems.
- Add stop sign for left-turning traffic only at southbound Parry Road approach at the Branch Pike intersection (local traffic).

Advantages

- Eliminate uncontrolled left-turn from Riverton Road to Branch Pike.
- Sight problems associated with making right-turn from Branch Pike on to Riverton Road is eliminated.
- All traffic on Riverton Road goes through signalized intersection at Parry Road.
- Minimal right-of-way impacts.
- Potential lower costs for adding a pre-timed signal than an actuated signal because loop detectors are not needed on each approach.

Disadvantages

- Additional travel capacity needed for left-turn lanes.
- Local residents on one-way section of Branch Pike may be inconvenienced.

Level of Service Analysis
A Synchro analysis was performed for this scenario using existing morning and afternoon peak hour data.

Where the signal is pre-timed not actuated as in Scenario 3 at the Riverton Road/Parry Road intersection level of service goes from an A to a B in both the morning and afternoon peak. Although the level of service for the approaches remains the same, the delay increases in all instances. Level of service for the other intersections remains the same for both scenarios.

<u>Scenario 4</u>

Characteristics

- Same as Scenario *2.*
- Traffic volumes were grown by annual growth rate of 1.5 percent annually per year for 10 years to estimate the level of service for the intersection in 2015.

Level of Service Analysis
A SYNCHRO analysis was performed for this scenario using existing morning and afternoon peak hour data with a growth rate as mentioned before.

In the future scenario, the Riverton Road/Parry Road intersection during the morning peak decreased from LOS A in Scenario 2 to B and remained at LOS A in the afternoon peak with increase in delay. All the approaches of this intersection will have a slightly longer delay. While the Riverton Road/Branch Pike intersection level of service will remain the same, Branch Pike/Parry Road intersection level of service changes from LOS B to C in the morning peak and from LOS C to D in the afternoon peak.

Recommendations

In terms of mobility, the intersections at the study location are operating efficiently, but there is a safety issue as evidenced by the type, severity and number of crashes. The congestion and safety problems identified at the study location need to be addressed. With increase in traffic through this location, the identified problems will increase. The potential also exists for the severity of crashes to increase. This location serves as an essential part of this important corridor in this region of Burlington County.

A more in-depth technical study of the combination of strategies as laid out in Scenario 2 should be pursued to determine feasibility and to address the problems identified at the study location.

From the results of our analysis, Branch Pike should be closed to westbound traffic between Riverton Road and Parry Road. This will eliminate the left-turn movement from Riverton Road onto Branch Pike, thus eliminating conflict with eastbound through traffic on Riverton Road. At the Riverton Road/Parry Road intersection a traffic signal could be warranted. The traffic signal should be installed with a left-turn phase for left-turning traffic on Parry Road. The roadway should be widened to include a westbound left-turn lane at this intersection. This traffic signal will control the movement of traffic at this intersection, reducing conflict between turning and through movement at this intersection.

As seen from the level of service analysis, closing Branch Pike to westbound traffic between Riverton Road and Parry Road and adding a traffic signal at the Riverton Road/Parry Road intersection does not adversely affect the flow of traffic through this location.

Delaware Valley Regional Planning Commission

CAMDEN

6 HADDONFIELD-BERLIN ROAD (CR 561) AT WHITE HORSE ROAD (CR 673)

Voorhees, Camden County

Location Description

The intersection of Haddonfield-Berlin Road and White Horse Road is located in Voorhees Township in the middle of Camden County. Haddonfield-Berlin Road (Route 561) is a major arterial that permits travel across the county from Winslow Township in the southeast to Camden City in the northeast. White Horse Road (Route 673) is a minor arterial that runs between Gloucester Township and Evesham Township in Burlington County. Both roads provide access to Cherry Hill Township, a major retail and employment center. Route 561 also provides access to the I-295 interchange in Cherry Hill, and to Philadelphia via I-676. The study location is shown on **Map 5**.

Looking east at Haddonfield-Berlin Road (Route 561)

Looking north at White Horse Road (Route 673)

Haddonfield-Berlin Rd. (CR 561) and White Horse Rd. (CR 673)

Voorhees Township, Camden County, NJ

2005 Regional Congestion and Accident Mitigation Program	Map 5	N 0 25 50 100 Feet Delaware Valley Regional Planning Commission August 2005

Date of Aerial Photography: Spring, 2000

In the vicinity of the intersection, retail units, shopping centers, and office parks front the two roads. The section of Route 673 between Route 561 and US 30 is heavily developed, and appears to draw a lot of trips. A bank, a shopping center, a garage, the Voorhees Township Municipal Building, and a church occupy the four corners of the intersection.

Existing Conditions

Identified Problems

There is severe traffic congestion on Route 561 at the intersection with Route 673, southbound and northbound. Traffic backs up on the southbound lanes for most of the day; vehicles typically wait at the traffic signal for several cycles to get through the intersection. The worst delays on the southbound lanes occur during the PM peak (5 PM to 6 PM). The northbound lanes also back up, but not as often and not as badly. The worst delays on the northbound lanes occur during the AM peak (8 AM to 9 AM). The Voorhees Township traffic safety officer reports incidents of road rage on the northbound lanes in the AM peak. The intensity of activity at the intersection is probably attributable to a large number of long-distance through trips combined with short trips to nearby retail establishments on Route 561 and Route 673.

Two possible explanations for the congestion on Route 561 were identified during field views: 1) Inadequate supply of lanes for through movements at the intersection with Route 673, and 2) Suboptimumal signal timing. The Route 561 northbound and southbound intersection approach configurations at the intersection with Route 673 are shown in **Figure 16**. The northbound approach has three lanes (designated left-turn, designated through, and designated right-turn). The southbound approach has two lanes (shared left-turn and through, and shared right-turn and through). The shared left-turn and through lane effectively functions as a left-turn only lane because it is difficult for vehicles to turn left. As a result, the shared right-turn and through lane must carry all through and right-turn movements.

Crash Analysis

To determine the safety of the intersection, crash data from the Voorhees Township Police Department was analyzed.

There were 92 reportable crashes recorded in the three-year period, 2002 to 2004, at the intersection. (The outside limit was 400 feet from the edge of the “box”) **Table 12** shows a

Figure 16
Existing Lane Configuration

Haddonfield-Berlin Road (CR 561)
& White Horse Road (CR 673)

Voorhees, NJ

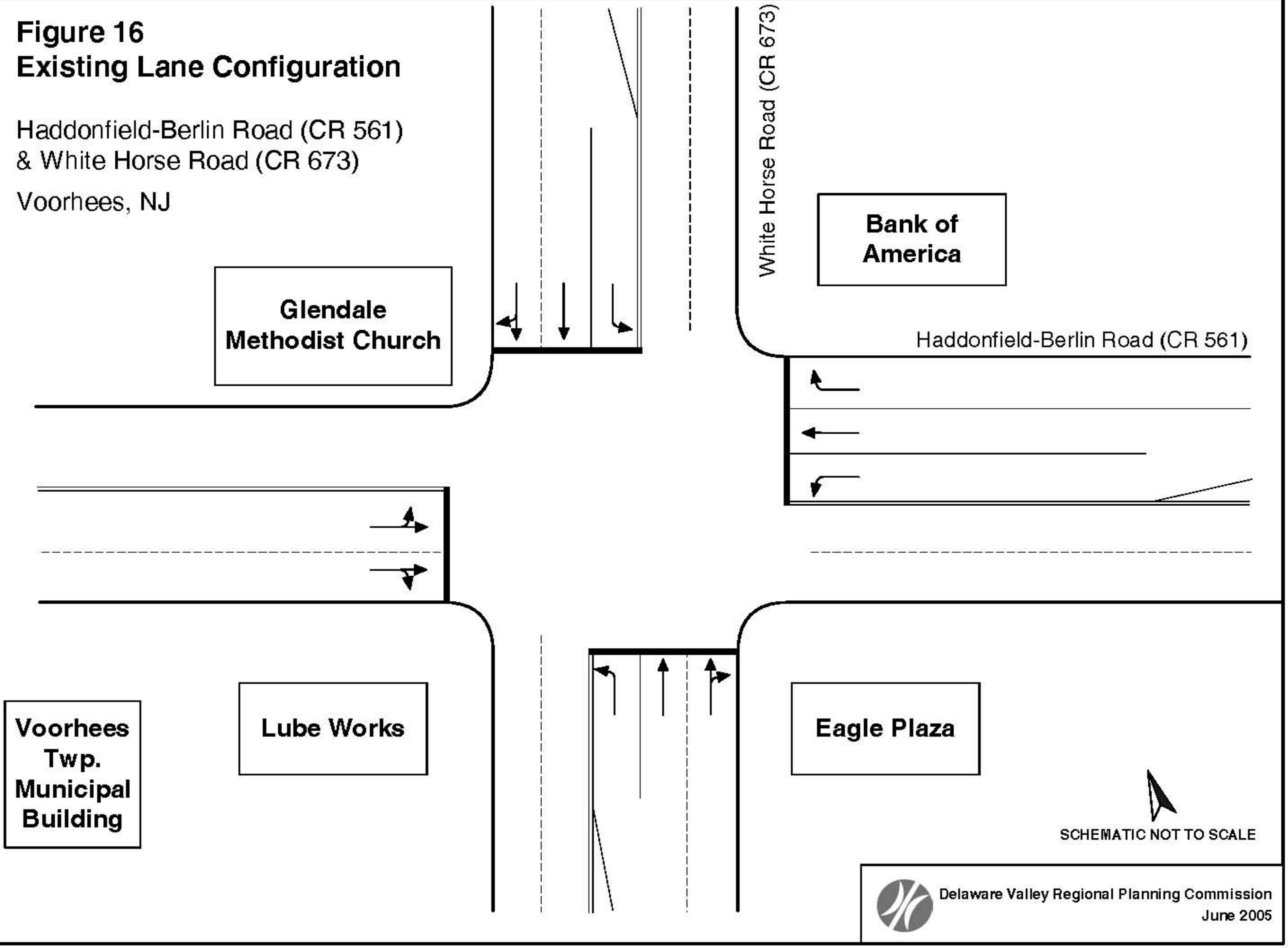


Table 12 Haddonfield-Berlin Road (CR 561) and White Horse Road (CR 673) Intersection Crash Summary (2002- 2004)

	2002	2003	2004	Total
Reportable Crashes	35	25	32	92
Severity				
Injuries	14	9	6	29
Fatalities	0	0	0	0
Accident Type				
Angle	20.0%	16.0%	21.9%	19.6%
Same Direction - Rear End	42.9%	24.0%	37.5%	35.9%
Left Turn	14.3%	48.0%	40.6%	32.6%
Same Direction - Sideswipe	22.9%	0.0%	0.0%	8.7%
Pedestrian	0.0%	4.0%	0.0%	1.1%
Fixed Object	0.0%	4.0%	0.0%	1.1%
Head-On	0.0%	4.0%	0.0%	1.1%
Time of Day				
Midnight to 6 am	0.0%	0.0%	0.0%	0.0%
6 am to Noon	20.0%	20.0%	34.4%	25.0%
Noon to 6 pm	77.1%	48.0%	59.4%	63.0%
6 pm to Midnight	2.9%	32.0%	6.3%	12.0%
Rush Hours				
6 am to 9 am	13.0%	7.1%	15.0%	12.3%
11 am to 2 pm	56.5%	28.6%	45.0%	45.6%
4 pm to 7 pm	30.4%	64.3%	40.0%	42.1%
Light Conditions				
Unknown	0.0%	4.0%	0.0%	1.1%
Daylight	88.6%	76.0%	78.1%	81.5%
Dawn or Dusk	5.7%	0.0%	3.1%	3.3%
Dark (Street Lights On)	5.7%	20.0%	18.8%	14.1%
Weather Conditions				
Clear	77.1%	72.0%	71.9%	73.9%
Rain	22.9%	20.0%	25.0%	22.8%
Snowy	0.0%	8.0%	0.0%	2.2%
Unknown	0.0%	0.0%	3.1%	1.1%
Surface Conditions				
Dry	77.1%	72.0%	71.9%	73.9%
Wet	22.9%	20.0%	25.0%	22.8%
Snowy	0.0%	8.0%	0.0%	2.2%
Unknown	0.0%	0.0%	3.1%	1.1%

Source: Voorhees Township Police Department, Accident Reports Data 2002-2004 breakdown of the crash data, and **Figure 17** is a collision diagram of all reportable crashes. Based on the data, the number of crashes was constant over the three-year period. In contrast, the number of injuries dropped from year to year; 14 out of 29 total injuries occurred in 2002. Nothing in the data appears to account for this trend. There were no fatalities.

Rear-end crashes were the most common (36 percent), and left-turn crashes the second most common (33 percent), crash type at the intersection. The rate of left-turn crashes far exceeds the county average (6 percent). The incidence of these types of crashes is explained, in part, by long queues, especially turning queues, at the intersection approaches. Most of the left-turn crashes were associated with access to and from commercial and institutional driveways in the vicinity of the intersection. The largest single source (10) was Country Club Diner. Other sources were Lube Works, Bank of America, and the Voorhees Township Municipal Building. (Voorhees Township has prohibited left-turns from the municipal building parking lot.) Vehicles turning left from the eastbound approach of Route 673 are also a large source (five) of left-turn crashes. The cause may be poor visibility of westbound vehicles due to the grade.

A full 63 percent of the crashes occurred between noon and 6 PM. Another 25 percent of the crashes occurred in the morning between 6 AM and noon. The large share of crashes in the midday and PM peak periods may reflect the intensity of activity at the intersection during those times. That activity is probably attributable to a large number of long-distance, through trips combined with short trips to nearby retail establishments on Route 561 and Route 673. A brief summary of the crash records is located in **Appendix E**.

Level of Service Analysis

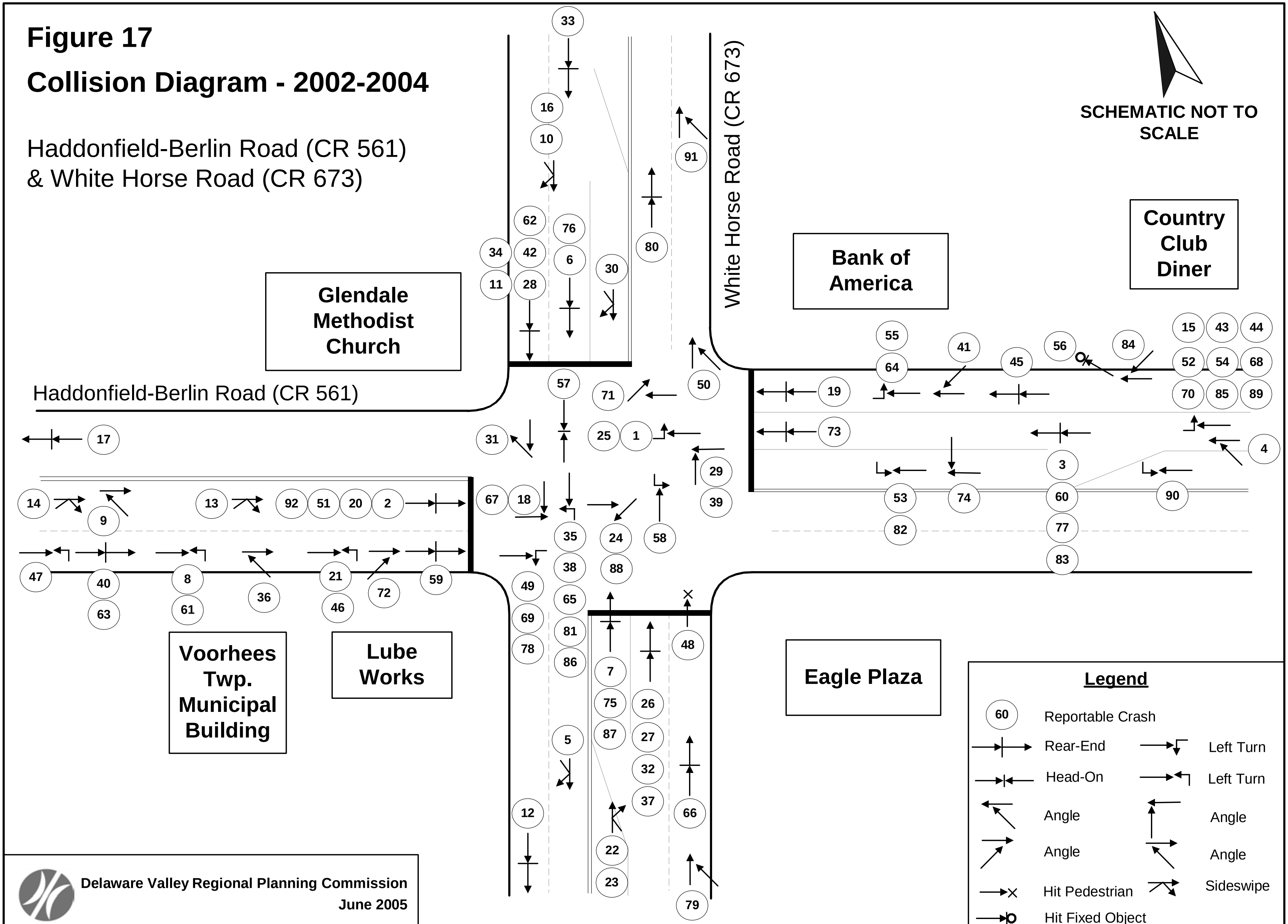
To determine the performance of the Route 561/Route 673 intersection, turning movement counts were collected and analyzed. DVRPC conducted manual turning movement counts at the intersection on Wednesday, January 12, 2005 from 6:00 AM to 9:00 AM, and from 4:00 PM to 7:00 PM. Peak hour turning-movement counts are shown in **Figure 18**. The Complete manual turning movement count data can be seen in **Appendix E.** In addition, the signal plan and timing permit for the intersection were obtained from the Camden County Department of Public Works. Highway Capacity Software (HCS), a traffic signal optimization program that performs level of service calculations, was used to analyze the data.

HCS simulated traffic on Route 561 and Route 673, and the performance of the intersection under existing road conditions was analyzed. Under existing road conditions, Route 673 performs well, but Route 561 performs poorly. Furthermore, delays at the southbound approach of Route 561 are much worse than those at the other three approaches. Vehicles at the Route

Figure 17

Collision Diagram - 2002-2004

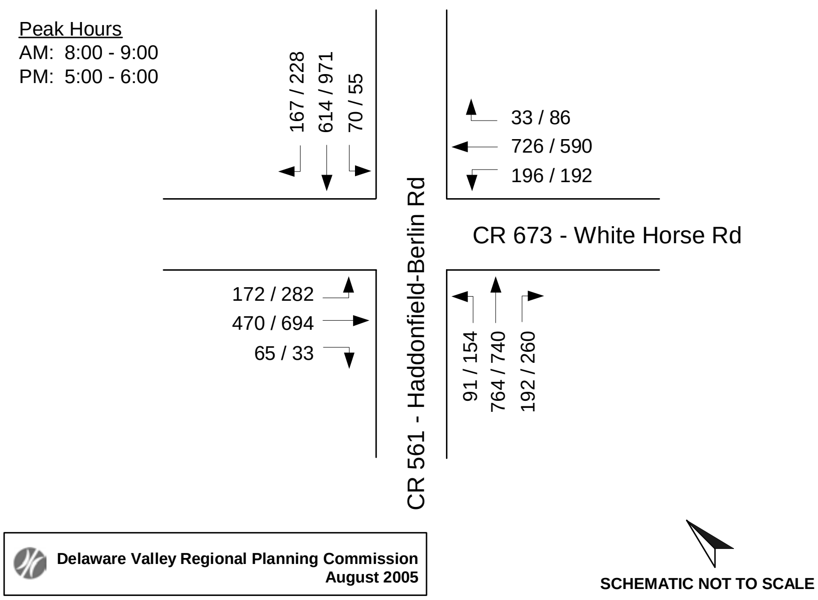
Haddonfield-Berlin Road (CR 561)
& White Horse Road (CR 673)



Delaware Valley Regional Planning Commission
June 2005

Figure 18
CR 561 Haddonfield-Berlin Rd & CR 673 White Horse Rd

Existing Peak Hour Turning Movement Counts AM / PM



561 southbound approach are experiencing delays of five and one-half minutes in the AM peak, and ten minutes in the PM peak. Field views suggest that extreme delays at the approach are not confined to the peak period and may prevail during the non-peak period. In the AM peak, Route 561 northbound operates at an unacceptable level of service (LOS E). In the PM peak, Route 673 eastbound also operates at LOS E.

Overall, the intersection operates at LOS F during the AM and PM peaks. Vehicles experience an average delay of approximately 110 seconds in the AM peak and approximately 205 seconds in the PM peak.

The results of the level of service analysis, with associated average delay per vehicle, are summarized in **Table 13**.

Opportunities and Constraints

In this section, opportunities and constraints on future transportation improvements at the intersection of Route 561 and Route 673 are identified. These include the Glendale Methodist Church property, the storm drain network under the intersection, and the acquisition of Route 561 right-of-way.

TABLE 13
Peak Hour Level of Service (LOS) Analysis
Haddonfield-Berlin Road (CR 561) and White Horse Road (CR 673)

Improvement Scenario	Direction of Travel	Peak AM Hour and Peak PM Hour LOS with Average Delay / Vehicle			
		AM Peak		PM Peak	
Existing		LOS	Delay (sec)	LOS	Delay (sec)
	CR 561 northbound	E	55.9	D	47.3
	CR 561 southbound	F	327.5	F	584.7
	CR 673 eastbound	C	30.1	E	58.2
	CR 673 westbound	C	32.5	C	34.3
	Intersection	F	110.9	F	205.6
1. Signal optimization		AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)
	CR 561 northbound	C	30.8	D	36.8
	CR 561 southbound	D	49.6	E	76.1
	CR 673 eastbound	D	41.0	F	136.1
	CR 673 westbound	E	62.2	F	102.8
	Intersection	D	45.8	F	84.7
2A. Add southbound travel lane		AM Peak		PM Peak	
(northbound lead)		LOS	Delay (sec)	LOS	Delay (sec)
	CR 561 northbound	C	24.6	D	37.5
	CR 561 southbound	C	20.1	D	45.7
	CR 673 eastbound	D	39.8	E	58.9
	CR 673 westbound	E	58.1	D	43.6
	Intersection	D	35.4	D	46.1
2B. Add northbound travel lane		AM Peak		PM Peak	
		LOS	Delay (sec)	LOS	Delay (sec)
	CR 561 northbound	B	14.2	C	27.1
	CR 561 southbound	D	42.9	E	69.1
	CR 673 eastbound	C	26.4	E	73.8
	CR 673 westbound	D	35.3	D	51.6
	Intersection	C	29.2	E	55.2
3. Add southbound travel lane and		AM Peak		PM Peak	
northbound travel lane		LOS	Delay (sec)	LOS	Delay (sec)
	CR 561 northbound	B	17.1	C	23.4
	CR 561 southbound	C	20.7	D	44.4
	CR 673 eastbound	C	22.0	E	57.5
	CR 673 westbound	C	27.1	D	37.2
	Intersection	C	21.6	D	40.2

The Glendale Methodist Church[1]

Glendale Methodist Church, which dates from 1855, is located on the northeast corner of the intersection of Route 561 and Route 673. The property includes the church building and the surrounding yard and wood (0.45 acre total). The Route 561 right-of-way in front of the church is owned by Camden County, which purchased it from the church in the 1930s. Two tall tulip trees, which also date from the 1850s and frame the front entrance of the church, stand in the right-of-way. There are other examples from the nineteenth century of tulip trees being used for architectural decoration in this way.

The church is listed in both the National Register of Historic Places and the New Jersey Register of Historic Places. Because the church is a historic property, transportation improvement projects at the intersection are subject to review. Federal law and New Jersey state law each define a review process, although they impose different requirements.

At the federal level, the National Historic Preservation Act of 1966 requires federal agencies to consider the effects of their undertakings on historic properties. The process, which is commonly known as *Section 106* review, applies only to federally funded, licensed, or authorized projects. Section 106 calls on federal agencies to identify and seek to avoid adverse

Glendale Methodist Church on Haddonfield-Berlin Rd.

[1] The summary is taken from the New Jersey Historic Preservation Office (HPO) Web site, documents provided by New Jersey HPO, and conversations with New Jersey HPO staff.

effects. The process is consultive. In addition, Section 4F of the US Department of Transportation Act states than a historic property cannot be destroyed to make way for a transportation project unless there is no prudent and feasible alternative. Both the USDOT and NJDOT would have to evaluate any transportation improvement project falling under Section 4F.

A stronger set of reviews comes into play when a historic property is listed in the New Jersey Register. Once listed in the New Jersey Register, a historic property enjoys protections from any public undertakings that would encroach upon, damage, or destroy it. An encroachment is defined as a public undertaking that impacts the historic characteristics for which a property is listed in the New Jersey Register. Prior authorization is required before the project may go forward.

HPO staff reviews projects that may pose a threat to a historic property. If the project is determined not to be an encroachment, it will be authorized within 45 days of receipt of an application. If the project is determined to constitute an encroachment, it will be submitted to the Historic Sites Council, a gubernatorially appointed body of 11 citizens created to advise the commissioner of the Department of Environmental Protection. The council reviews proposed encroachments at an open public meeting, and makes a recommendation to the Commissioner for final action.

In 1998, Camden County made application to the New Jersey HPO to implement a transportation improvement project on Route 561 in the right-of-way in front of Glendale Methodist Church. The project called for widening northbound Route 561 to construct a second northbound through lane. Had it gone forward, the tulip trees in front of the church would have been removed and the right-of-way would have been paved over, moving the curb line 15 feet closer to the church.

The church argued that the project would destroy what was left of the historic setting, a rural crossroads. Much of the natural landscape had already disappeared. The church property stood alone, and was surrounded by commercial development. In this context, the tulip trees were a living connection to the natural landscape that had once existed. They had also been placed in front of the church as architectural decoration. Therefore, removal of the trees would diminish the historic character of the church.

New Jersey HPO staff and the Historic Sites Council accepted the argument, and the commissioner ruled in favor of the church, denying the application to widen the roadway.

Based on conversations with New Jersey HPO staff, there appear to be no circumstances under which an improvement project in front of the church would be allowed. The logic of the original ruling would cover other cases. For example, death or disease of the tulip trees leading to their removal would not alter the situation significantly, because the remnant front lawn would also have historic value. Pursuing an improvement project would also be likely to spur opposition from those in the community who had supported the original historic designation.

In conclusion, a transportation improvement project in the right-of-way in front of Glendale Methodist Church would face overwhelming legal and political obstacles.

Storm Drain Network

A storm drain network under the intersection of Route 561 and Route 673 collects runoff from an area west of Route 561. One source of runoff is Eagle Plaza. Water enters the network from a concrete drainage way on the edge of the Eagle Plaza parking lot near the intersection. There are large inlets under the Voorhees Township parking lot, from which the flow continues down the grade. Camden County engineers think that the storm drain network would not be a significant barrier to widening the southbound lanes of Route 561, and would probably require only minor reconstruction.

Acquisition of Route 561 Right-of-Way

To widen Route 561 at the intersection with Route 673 would require acquisition of right-of-way. Except for the property in front of Glendale Methodist Church, which is owned by Camden County, the land abutting the existing roadway is private property. There are also narrow public sidewalks on all corners of the intersection. The property on the southeast corner is owned by Bank of America; the property on the southwest corner by Eagle Plaza; and the properties on the northwest corner by Lube Works and Voorhees Township. Most of the land abutting the roadway is occupied by parking and includes several driveways. The land abutting Eagle Plaza is occupied by decorative landscaping and a concrete drainage way. The drainage way runs behind the Mattress Giant and Blockbuster Video building in the Eagle Plaza parking lot. There appears to be enough room between Route 561 and the building to add one traffic lane; to add more than one lane would probably require acquisition of the building.

Potential Improvement Scenarios

Despite the limits imposed by the New Jersey Historic Preservation Office ruling, improvement projects on the other (southbound) side of Route 561 would be allowable. Adding another lane for through movements, southbound or northbound, would be feasible.
A northbound lane would require reconstruction of the roadway. Somewhere before Glendale Methodist Church, the road would leave the existing alignment to create the new lane. Somewhere after the church and Route 673, the road would return to the existing alignment.

In addition, traffic signal optimization could be an effective alternative to physical reconstruction of the intersection approaches.

Improvement Scenarios: Level of Service Analysis

Based on analysis of existing conditions at the intersection, four improvement scenarios were developed and tested. The focus was on improving the performance of Route 561, but the consequences for the performance of Route 673 were also important. One scenario alters the signal timing of the intersection; the other three scenarios alter the Route 561 intersection approach configurations. The improvement scenarios are listed in **Table 14**.

Table 14 Route 561 Improvement Scenarios

Scenario		Description
1	Signal optimization	Signal timing was modified with the goal of reducing delay on CR 561 to acceptable levels.
2A	Add southbound through lane (northbound lead)	The new approach configuration has three lanes (a shared right-turn and through lane, a through lane, and an exclusive left-turn lane).
2B	Add northbound through lane	The new approach configuration has four lanes (an exclusive right-turn lane, two through lanes, and an exclusive left-turn lane).
3	Add southbound through lane and northbound through lane	See 2A and 2B above.

The performance of the intersection under the four improvement scenarios was analyzed using HCS. The results of the level of service analysis, with the associated average delay per vehicle, are shown in the previous **Table 13**. HCS analysis data for both existing and potential improvement scenarios can be found in **Appendix E**.

Scenario 1 - Signal Optimization

The existing signal timing plan was analyzed by comparing it to an optimal signal timing plan generated by HCS. A significant difference between the performances of the two plans became evident, suggesting that the existing plan could be improved by retiming the signal.

Signal optimization would reduce delay on Route 561 compared to existing conditions. The benefits would include a significant reduction in the extreme delay on Route 561 southbound. But to reduce delay on Route 561, the cost would be increased delay on Route 673. In the PM peak, both the eastbound and westbound intersection approaches would fail (LOS F), with an average delay of approximately two minutes. In the AM peak, average delay on Route 673 westbound would increase from 33 seconds to 62 seconds, and average delay on Route 673 eastbound would increase from 30 seconds to 41 seconds. In general, delays on Route 673 would double or triple. The intersection overall would operate at LOS D in the AM peak and at LOS F in the PM peak. Vehicles would experience an average delay of approximately 45 seconds in the AM peak and 85 seconds in the PM peak.

Scenario 2A - Add Southbound Through Lane (Northbound Lead)

Adding another southbound through lane would reduce delay on Route 561 compared to existing conditions. The benefits would include a significant reduction in the extreme delay on Route 561 southbound. But performance at the Route 673 intersection approaches would be impacted negatively in the AM peak and the PM peak. The most significant change would be in the AM peak. Average delay on Route 673 westbound would increase from 33 seconds to 58 seconds, and average delay on Route 673 eastbound would increase from 30 seconds to 40 seconds. In the PM peak, average delay on Route 673 westbound would increase from 34 seconds to 44 seconds. The intersection, overall, would operate at LOS D in the AM peak and the PM peak. Vehicles would experience an average delay of 35 seconds in the AM peak and approximately 45 seconds in the PM peak.

Scenario 2B - Add Northbound Through Lane

Adding a northbound through lane would reduce delay on Route 561 compared to existing conditions. The benefits would include a significant reduction in the extreme delay on Route 561 southbound. However, performance at the Route 673 intersection approaches would be impacted negatively in the PM peak. Average delay on Route 673 westbound would increase from 34 seconds to 52 seconds. Average delay on Route 673 eastbound would increase from 58 seconds to 74 seconds. The performance of Route 673 in the AM peak would not change significantly from existing conditions. The intersection overall would operate at LOS C in the AM peak and at LOS E in the PM peak. Vehicles would experience an average delay of approximately 30 seconds in the AM peak and 55 seconds in the PM peak.

Scenario 3 - Add Southbound Through Lane and Northbound Through Lane

Adding both southbound and northbound through lanes would reduce delay on Route 561 compared to existing conditions. The benefits would include a significant reduction in the extreme delay on Route 561 southbound. All intersection approaches would improve at all times of the day compared to existing conditions, except Route 673 eastbound in the PM peak. It would be operating at the same level of service (LOS E) as under existing conditions. The intersection overall would operate at LOS C in the AM peak and at LOS D in the PM peak. Vehicles would experience an average delay of approximately 20 seconds in the AM peak and 40 seconds in the PM peak.

Summary

The performance of Route 561 would be improved by each of the four improvement scenarios, but the performance of Route 673 would be different depending on the improvement scenario. The results of the level of service analysis are summarized in **Table 15**.

Table 15 Summary of Route 561 Improvement Scenario LOS Analysis

Scenario		Scenario Compared to Existing Conditions
1	Signal optimization	Route 561 much better. Route 673 worse. In the PM peak, Route 673 fails.
2A	Add southbound through lane (northbound lead)	Route 561 much better. Route 673 slightly worse but acceptable.
2B	Add northbound through lane	Route 561 much better. Route 673 slightly worse but acceptable.
3	Add southbound through lane and northbound through lane	Route 561 much better. Route 673 slightly better or the same.

Improvement Scenarios: Cost and Alignment

The four improvement scenarios were compared for three factors: Performance, cost, and impact on Route 561 alignment.

Performance

The performance of Route 561 and Route 673 under each of the four improvement scenarios was analyzed in the previous section.

Cost

The cost of Scenario 1, altering the signal timing of the intersection, would be negligible.

Scenarios 2A, 2B, and 3 would alter one or both Route 561 intersection approach configurations. The costs of these scenarios would have two major components: Right-of-way acquisition and reconstruction of the existing roadway. It was beyond the scope of this study to determine precise costs, but it was possible to make judgments about relative costs.

Scenarios 2A, 2B, and 3 would all require acquisition of right-of-way, but the size of the footprint would vary. Scenario 2A would probably require right-of-way only on the northwest corner of the intersection, the location of a garage and the Voorhees Township Municipal Building. Scenarios 2B and 3 would require right-of-way on the northwest corner and the southwest corner, the location of Eagle Plaza.

Furthermore, scenarios 2A and 2B would require right-of-way to construct one lane, but Scenario 3 would require right-of-way to construct two lanes. The second lane would probably require acquisition of the Mattress Giant and Blockbuster Video building in the Eagle Plaza parking lot. It would also consume some of the Voorhees Township Municipal Building parking lot.

Finally, scenarios 2B and 3 would also require reconstruction of the roadway. Somewhere before Glendale Methodist Church, the road would leave the existing alignment to create the new lane(s). Somewhere after the church and Route 673, the road would return to the existing alignment. Scenarios 2A and 2B are illustrated in **Figures 19 and 20**, respectively.

Impact on Route 561 Alignment

Maintenance of a straight alignment is desirable for reasons of efficiency, safety, and aesthetics. Scenario 1 would have no impact on the Route 561 alignment. Scenario 2A would have an impact on the southbound lanes of Route 561. Scenarios 2B and 3 would have an impact on the southbound and northbound lanes of Route 561.

The four improvement scenarios were ranked for each factor. Three rankings were possible: 1, 2, or 3, with 1 being the highest. The results appear in **Table 16**.

Table 16 Ranking of Route 561 Improvement Scenarios By Three Factors

Scenario		Performance	Cost	Alignment
1	Signal optimization	3	1	1
2A	Add southbound through lane (northbound lead)	2	2	1
2B	Add northbound through lane	2	3	3
3	Add southbound through lane and northbound through lane	1	3	3

Figure 19

Add Southbound Lane on CR 561

Haddonfield-Berlin Road (CR 561)
& White Horse Road (CR 673)

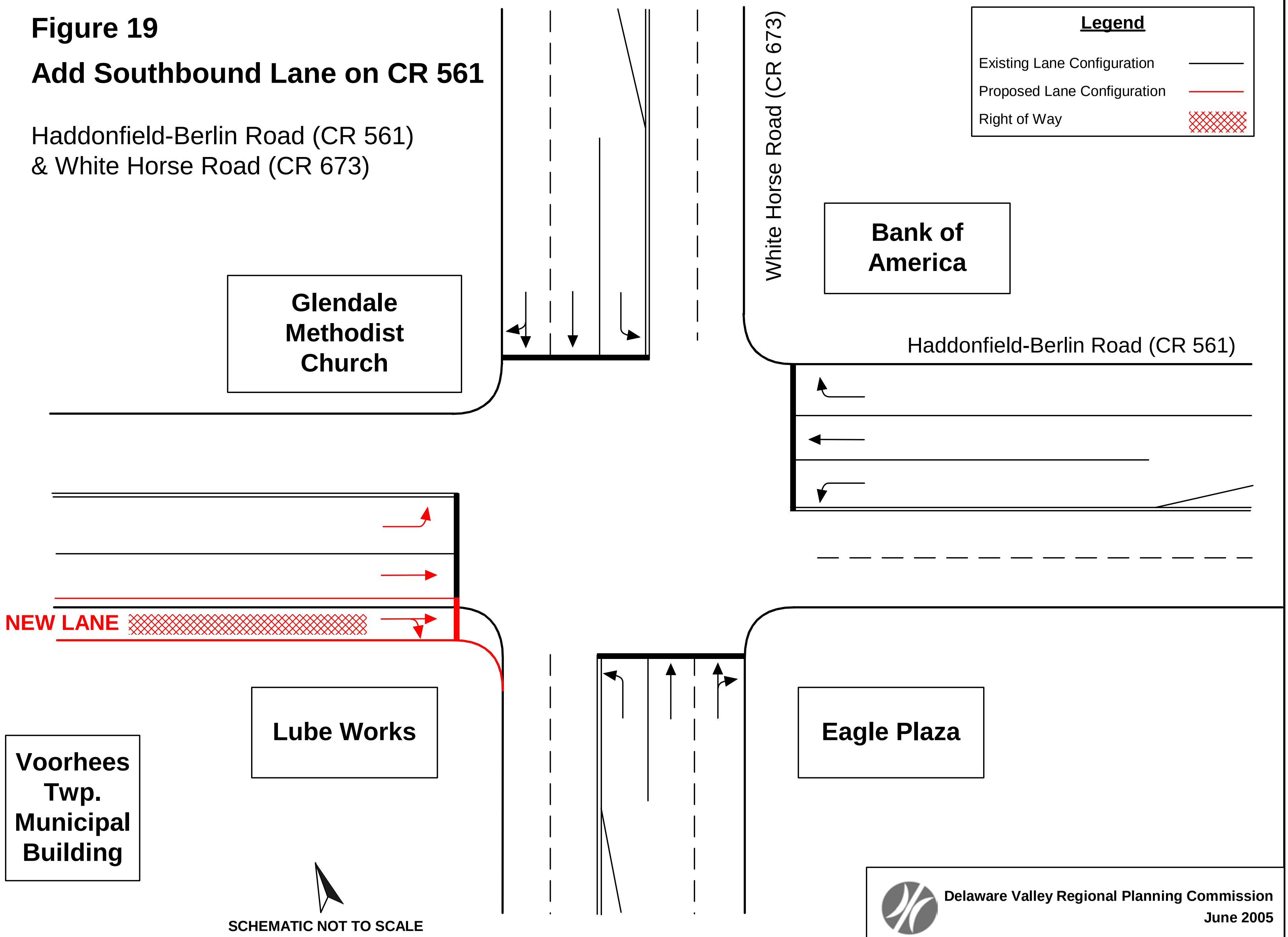
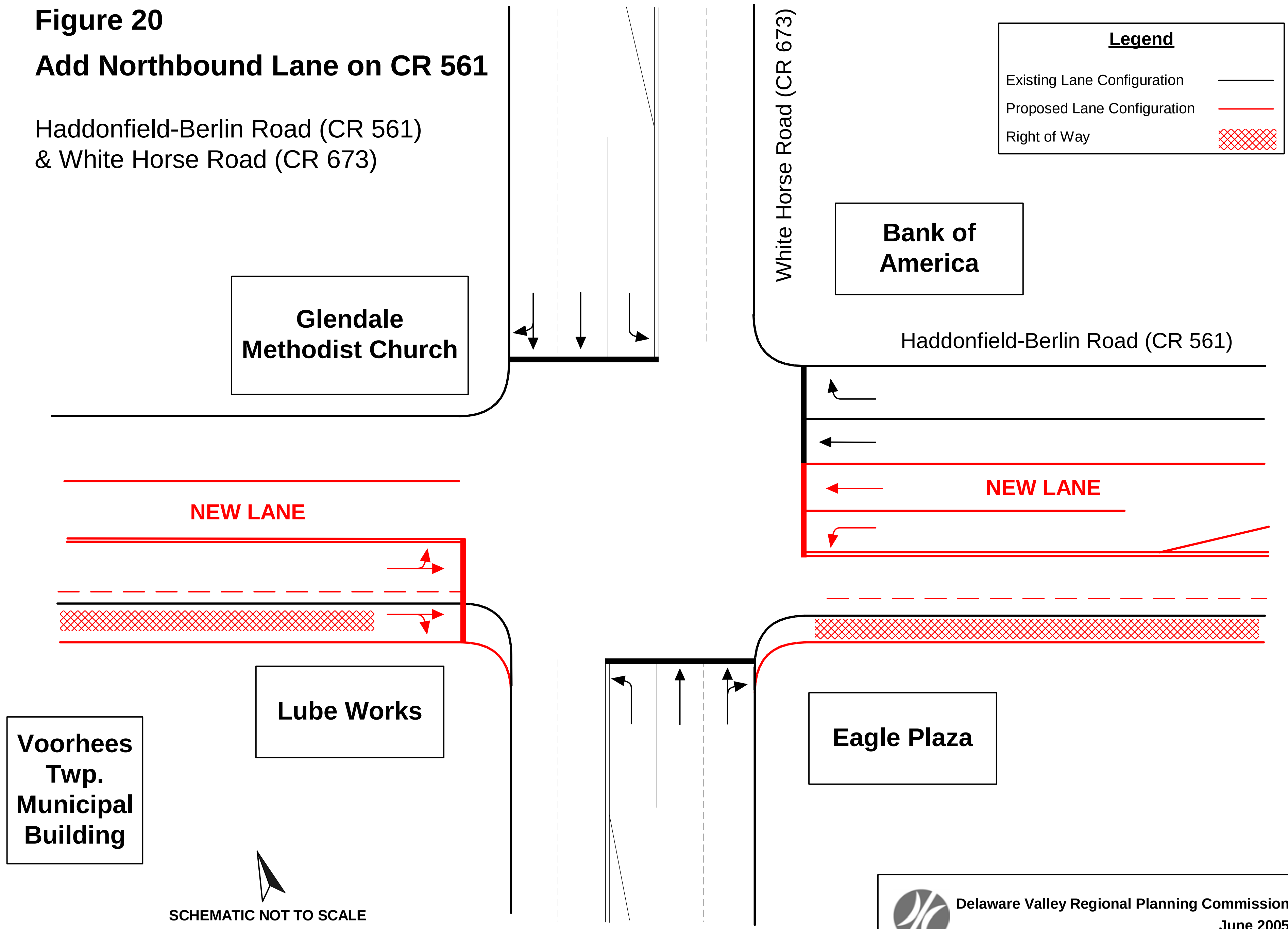


Figure 20

Add Northbound Lane on CR 561

Haddonfield-Berlin Road (CR 561)
& White Horse Road (CR 673)



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Improvement Scenarios: Conclusions

1. There are two explanations for the congestion on Route 561 at the intersection with Route 673: Inadequate supply of lanes for through movements at the intersection, and suboptimumal signal timing.

2. A transportation improvement project in the right-of-way in front of Glendale Methodist Church would face overwhelming legal and political obstacles.

3. Despite the limits imposed by the New Jersey Historic Preservation Office ruling, widening on the other (southbound) side of Route 561 would be allowable. Adding another lane for through movements, southbound or northbound, would be feasible. In addition, traffic signal optimization could be an effective alternative to physical reconstruction of the intersection approaches.

4. Based on analysis of existing conditions at the intersection, four improvement scenarios were developed and tested. The focus was on improving the performance of Route 561, but the consequences for the performance of Route 673 were also important. The four improvement scenarios were also ranked for two other factors: cost and impact on the Route 561 alignment.

Recommendations

Each improvement scenario has been evaluated as a long-term solution, and a short-term solution, to the congestion on Route 561.

Long-Term Solution

Although Scenario 1, signal optimization, would improve dramatically the overall performance of the intersection, the costs it would impose would be unacceptable. Scenario 3 would cost more than scenarios 2A or 2B, mostly because of the need to acquire the Mattress Giant and Blockbuster Video building in the Eagle Plaza parking lot. Therefore, compared to scenarios 2A or 2B, the marginal costs of Scenario 3 would be higher, but the marginal benefits, from enhanced intersection performance, would be small. Between scenarios 2A and 2B, there would be almost no difference in performance except that Scenario 2A would perform better in the PM peak (LOS D compared to LOS E), when the worst congestion occurs. Scenario 2A ranks higher than Scenario 2B for the other two factors: cost and impact on the Route 561 alignment. Therefore, with performance the same or slightly better, and with a better alignment and lower costs, Scenario 2A is the recommended long-term solution.

Short-Term Solution

Only Scenario 1 would be eligible as a short-term solution. Nevertheless, the same factors that make it unacceptable as a long-term solution also argue against it as a short-term solution.

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7 OLD TRENTON RD. (CR 535) AT ROBBINSVILLE-EDINBURG RD. (CR 526) OLD TRENTON RD. AT WINDSOR ROAD (CR 641)/EDINBURG RD. (CR 526)

West Windsor Township, Mercer County

Location Description

The study location consists of two adjacent intersections located in the southernmost section of West Windsor Township. The intersections are Robbinsville-Edinburg Road (CR 526)/Old Trenton Road (CR 535) and Edinburg Road (CR 526)/Windsor Road (CR 641)/Old Trenton Road (CR 535). The study location is shown on **Map 6.** All roadways at this location are county-owned and maintained. The functional classification of the roadways is urban minor arterial. The study location encompasses one unsignalized intersection: Robbinsville-Edinburg Road/Old Trenton Road, and one signalized intersection: Edinburg Road/Windsor Road/Old Trenton Road, which are 0.23 miles apart. Between both intersections, Old Trenton Road carries a bridge over Assunpink Creek. Robbinsville-Edinburg Road/Old Trenton Road is a T-intersection that is stop sign controlled at the northbound approach of Robbinsville-Edinburg Road. Edinburg Road/Windsor Road/Old Trenton Road is a four-leg intersection controlled by a 4-phased traffic signal.

CR 535 runs in a northeast-southwest direction from Plainsboro Township in Middlesex County and points east to NJ 33 in Hamilton Township, continuing into Trenton. CR 526 runs in a north-south direction from Monmouth County and points south through NJ 33 in Washington Township, going north to join with CR 571 where it bears west and continues into Princeton

Looking west on Old Trenton Road at Windsor Road/Edinburg Road

Old Trenton Rd. (CR 535), Robbinsville Rd. (CR 526), Edinburg Rd. (CR 526), and Windsor Rd. (CR 641)

West Windsor Township, Mercer County, NJ

2005 Regional Congestion and Accident Mitigation Program	Map 6	N 0 25 50 100 Feet Delaware Valley Regional Planning Commission August 2005

Date of Aerial Photography: Spring, 2000

Borough. Between the two intersections, Old Trenton Road is designated as CR 526. CR 641 runs in a northwest-southeast direction from CR 539 in Monmouth County to CR 535. All the roadways in the study location have access to multiple east-west and north-south major highways in the area. CR 535 has direct access to I-295 via NJ 33. To the north of the study area, Edinburg Road (CR 526) has direct access to US 1. To the south, Robbinsville (CR 526) has direct access to US 130, I-195 and the New Jersey Turnpike. CR 641 also has direct access to US 130.

Figure 21 shows the existing lane configuration of the study area. Robbinsville-Edinburg Road (CR 526) has one 12-foot lane in each direction with shoulders approximately 3 feet. The speed limit on this roadway is 45 MPH. Old Trenton Road also has one12-foot lane in each direction with shoulders approximately 3 feet at the Robbinsville-Edinburg Road intersection. East of this intersection, the speed limit of this roadway is 35 MPH, and east of the intersection with Edinburg Road/Windsor Road and Old Trenton Road the speed limit is 50 MPH. Edinburg Road and Windsor Road has one 12-foot lane in each direction with no shoulders at the intersection with Old Trenton Road. The speed limit on Edinburg Road is 40 MPH. Sidewalks are provided on Old Trenton Road east of the intersection, however, no pedestrian activity was observed.

Looking west on Old Trenton Road at Windsor Road /Edinburg Road

The land uses at, and immediately surrounding the study location is varied. At the study location, the land uses are comprised of commercial, residential, agricultural and wooded uses. In the surrounding area, residential development and a number of parks, both county and local, dominate the land uses. Mercer County Community College and Mercer County Vo-Tech School are located west of the study location. Several business parks are located in close proximity of the study location.

Existing Conditions

Identified Problems

Commercial and residential developments have been rapidly growing in the area over the last decade. As a result, traffic volumes have increased and the intersections at the study location have been experiencing congestion during morning and afternoon peak periods. At the Robbinsville-Edinburg Road/Old Trenton Road intersection, backups are experienced on eastbound Old Trenton Road in the PM peak period due to left-turning traffic on Robbinsville-Edinburg Road. Old Trenton Road is one lane in each direction with no dedicated left-turn lane. A similar problem exists at the northbound approach to this intersection on Robbinsville-Edinburg Road. Traffic turning in both directions (left and right) during peak periods conflicts with through traffic on Old Trenton Road. At the Edinburg Road/Windsor Road/Old Trenton Road intersection there is also congestion from high traffic volumes. There is traffic congestion at the Old Trenton Road eastbound approach of the intersection during the morning peak period. This is due mainly to the volume of left-turning traffic during this period and the lack of a dedicated left-turn lane.

Sight distance problems also exist at the two intersections. At the Robbinsville-Edinburg Road/Old Trenton Road intersection, an embankment creates this problem on Old Trenton Road to the west of the intersection. The embankment makes it difficult for motorists to see on-coming traffic. Removal of the embankment is on the *Mercer County Capital Improvement Program*. Sight problems are also associated with the alignment of Robbinsville-Edinburg Road at the intersection.

At the Edinburg Road/Windsor Road/Old Trenton Road intersection, sight problems are experienced on three of the four legs. This is a result of buildings located on the corners of the roadway with little setback from the curb.

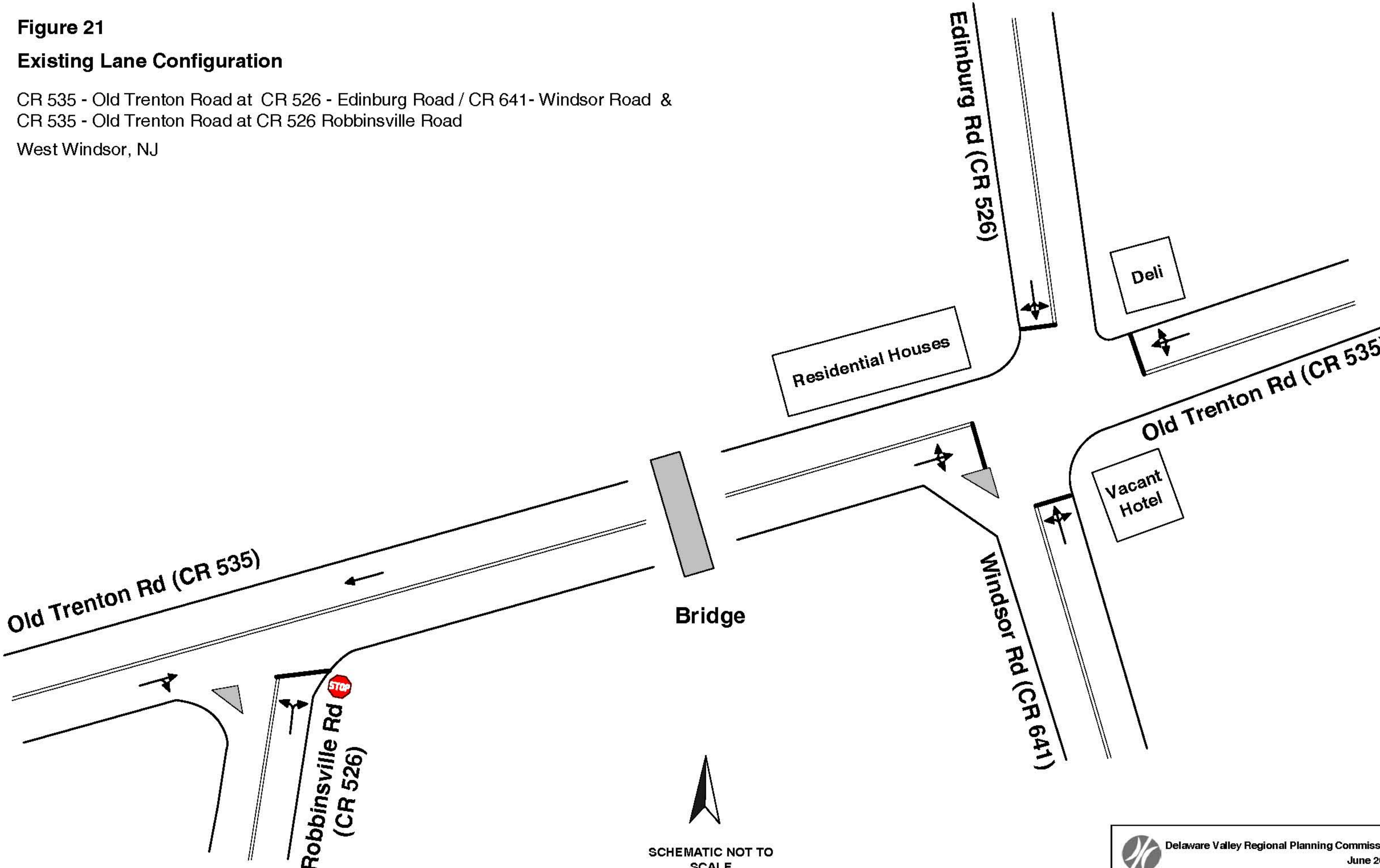
The Edinburg Road/Windsor Road/Old Trenton Road intersection is skewed with the north and south approaches being offset from one another. As a result, traffic on the northbound approach to intersection on Windsor Road experiences problems with the curb when continuing straight onto Edinburg Road or making a right-turn on Old Trenton Road. This curb also creates

Figure 21

Existing Lane Configuration

CR 535 - Old Trenton Road at CR 526 - Edinburg Road / CR 641- Windsor Road &
CR 535 - Old Trenton Road at CR 526 Robbinsville Road

West Windsor, NJ



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problems for through traffic on eastbound Old Trenton Road. The existing alignment of the intersection also leads to problems with optimizing the phasing of the traffic signal.

The bridge on Old Trenton Road over the Assunpink Creek is deficient, with an overall rating of 0.455 on the New Jersey Department of Transportation Bridge Management System. Federal funds may be used to replace or rehabilitate this bridge.

Crash Analysis

West Windsor Township Police accident reports for the years 2002, 2003 and 2004 were gathered. Reportable and non-reportable accidents were analyzed for the study location. Reportable accidents were those that involved an injury, fatality or more than five hundred dollars worth of property damage.

At Edinburg Road/Windsor Road/Old Trenton Road intersection, there were 64 crashes over the three-year period. Twenty-seven percent, or 17 crashes, were non-reportables. Seventeen injuries were recorded over the study period out of the 64 crashes and no fatalities were recorded. In 2002, 22 crashes were recorded. The number of crashes fell in 2003 by 18 percent to 18 crashes and rose in 2004 to 24, a 33 percent increase over 2003. Injuries also rose in 2004 from three in the previous years to 10. As shown in **Table 17** and **Figure 22**, same direction rear end accidents constituted almost half of the total accidents, 40.9 percent. This could be attributed to congestion at the intersection. In 2003, 61.1 percent of total accidents were of same direction rear end accidents. This represents the highest percentage of accident type within the database. Accidents involving a fixed object were 17.2 percent, or 11 accidents, of the total. Those accidents rose from one in 2002 to four in 2003 and rose again in 2003 to six accidents. Angle accidents were 12.5 percent of the database for the three-year period. There were seven angle accidents in 2002. In 2003, there were no recorded angle accidents and one was recorded for 2004. Over the three-year period there was only one accident involving a pedestrian. For the study location, it was determined that morning peak is 6 AM-9 AM and afternoon peak is 4 PM-7 PM on weekdays. Fifty-two percent of the accidents over the study period occurred during both peak periods. Afternoon peak accidents were consistent through the study period at seven, while for morning peak they increased from three in 2002 and 2003 to six in 2004. For all three years, accidents were more prevalent during the daylight period, 65.6 percent; clear weather conditions, 78.1 percent; and on dry road surface, 68.8 percent.

Seventy-six crashes were associated with the Robbinsville-Edinburg Road/Old Trenton Road intersection for the three-year period. There were 22 injuries resulting from these accidents reported during the study period. Of the 76 crashes, 16 were non-reportable. The number of accidents has progressively increased over the study period. In 2003, there was a 21 percent increase over 2002 and in 2004, there was a 48 percent increase over 2003. The number of injuries also progressively increased: 40 percent between 2002 and 2004, and 43 percent between 2003 and 2004.

Table 17 Intersection Accident Summary (2002-2004)
Edinburg Rd (CR 526)/Windsor Rd (CR 641) and Old Trenton Rd (CR 535)

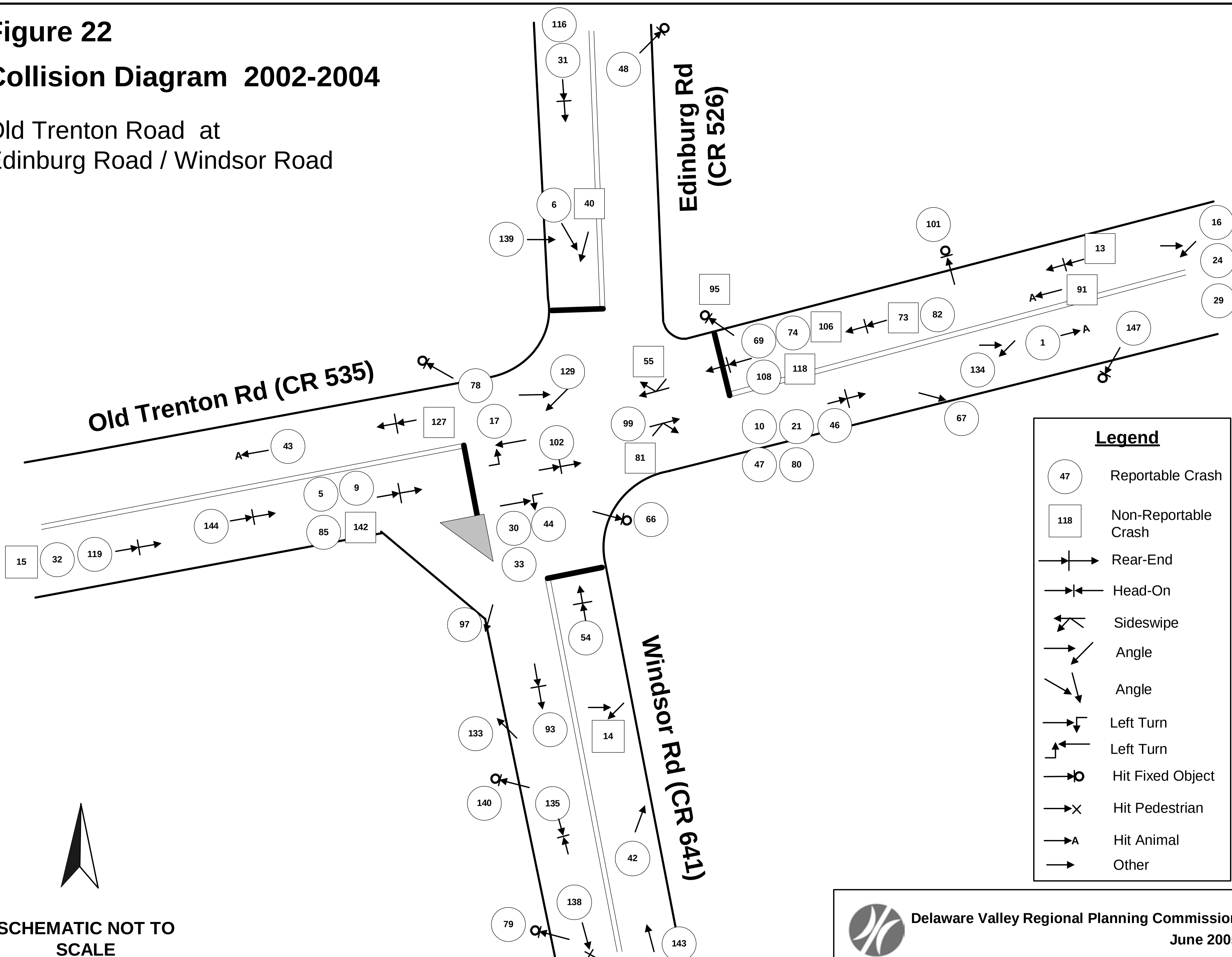
	2002	2003	2004	Total	2002	2003	2004	Total
Crashes								
Reportable	16	14	17	47	34.0%	29.8%	36.2%	73.4%
Non Reportable	6	4	7	17	35.3%	23.5%	41.2%	26.6%
Total	22	18	24	64	34.4%	28.1%	37.5%	100%
Severity								
Injuries	3	3	11	17	17.6%	17.6%	64.7%	
Fatalities	0	0	0	0	0	0	0	
Accident Type								
Same Direction - Rear End	9	11	10	30	40.9%	61.1%	41.7%	46.9%
Fixed Object	1	4	6	11	4.5%	22.2%	25.0%	17.2%
Angle	7	0	1	8	31.8%	0.0%	4.2%	12.5%
Left Turn	3	0	0	3	13.6%	0.0%	0.0%	4.7%
Same Direction - Sideswipe	0	2	1	3	0.0%	11.1%	4.2%	4.7%
Head-On	0	0	1	1	0.0%	0.0%	4.2%	1.6%
Pedestrian	0	0	1	1	0.0%	0.0%	4.2%	1.6%
Animal	1	0	0	1	4.5%	0.0%	0.0%	1.6%
Other	1	1	4	6	4.5%	0.0%	4.2%	3.1%
Time of Day								
Midnight to 6 am	0	0	0	0	0.0%	0.0%	0.0%	0.0%
6 am to Noon	4	5	8	17	18.2%	27.8%	33.3%	26.6%
Noon to 6 pm	12	8	10	30	54.5%	44.4%	41.7%	46.9%
6 pm to Midnight	6	5	6	17	27.3%	27.8%	25.0%	26.6%
Rush Hours								
6 am to 9 am	3	3	6	12	21.4%	27.3%	40.0%	30.0%
11 am to 2 pm	4	1	2	7	28.6%	9.1%	13.3%	17.5%
4 pm to 7 pm	7	7	7	21	50.0%	63.6%	46.7%	52.5%
Light Conditions								
Daylight	15	11	16	42	68.2%	61.1%	66.7%	65.6%
Dark (Street Lights On)	5	6	4	15	22.7%	33.3%	16.7%	23.4%
Dawn or Dusk	2	1	1	4	9.1%	5.6%	4.2%	6.3%
Unknown	0	0	3	3	0.0%	0.0%	12.5%	4.7%
Weather Conditions								
Clear	19	14	17	50	86.4%	77.8%	70.8%	78.1%
Rain	3	4	6	13	13.6%	22.2%	25.0%	20.3%
Fog	0	0	1	1	0.0%	0.0%	4.2%	1.6%
Surface Conditions								
Dry	19	12	13	44	86.4%	66.7%	54.2%	68.8%
Wet	3	5	9	17	13.6%	27.8%	37.5%	26.6%
Icy	0	1	2	3	0.0%	5.6%	8.3%	4.7%

Source: West Windsor Police Department, Accident Reports Data 2002-2004

Figure 22

Collision Diagram 2002-2004

Old Trenton Road at
Edinburg Road / Windsor Road



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As seen in **Table 18** and **Figure 23,** same direction rear end accidents make up the majority of the crashes with 51.3 percent. A brief account of the crash records is located in **Appendix F**. These accidents are a result of congestion at the intersection and sight distance problems associated with the intersection as mentioned earlier. Left-turn accidents account for 19.7 percent of the total incidents for the study period. Between 2002 and 2003, there was a 67 percent reduction in left-turn accidents from six to two, which increased in 2004 to seven accidents of this type. This may be due to conflicting traffic to and from Robbinsville-Edinburg Road and poor sight distance. Angle accidents are 15.8 percent of the three-year total. There was a progressive increase in this accident type over the three years. There were no accidents involving pedestrians or bicycles at this intersection. Forty-nine percent of the accidents over the study period occurred during the morning and afternoon peak periods. The morning peak experienced a steady increase in accidents over the three-year period from two in 2002 to six in 2004. The afternoon peak experienced a decrease between 2002 and 2003 from seven to four, then an increase in 2004 to twelve. Seventy-one percent of total accidents occurred in daylight conditions and 14.5 percent occurred at night. Of total accidents, 73.7 percent occurred in clear weather conditions, but in 2004, 85.3 percent occurred during this weather condition. Rain had the next highest weather condition with 21.1 percent of the total. In 2002, 26.3 percent of accidents occurred during rainy weather conditions while in 2003, it was 34.8 percent and in 2004, it was only 8.8 percent. Dry road surface condition recorded the highest number of accidents, 52. In 2004, 29 accidents occurred with dry surface conditions, 85.3 percent of that year total. The remaining five crashes occurred with wet surface conditions. In 2003, 11 of the 23 accidents for that year occurred during dry conditions, and of the remainder, 10 crashes occurred with wet surface conditions and two with icy conditions. In 2002, 63.2 percent of crashes occurred on dry surface conditions and the remaining 36.8 percent occurred on wet surface conditions.

Turning Movement Counts

At the Robbinsville-Edinburg Road/Old Trenton Road intersection, the through movements are dominant on Old Trenton Road in both morning and afternoon peak hours. The counts were taken on January 20, 2005 in morning from 6 AM to 9 AM and the evening from 4 PM to 7 PM. The morning peak hour is 7:45 AM to 8:45 AM and the afternoon peak hour is 5 PM to 6 PM. **Figure 24** displays the peak hour turning movement counts. There are 876 vehicles traveling westbound on Old Trenton Road at this intersection with only 91 making the left-turn onto Robbinsville-Edinburg Road. The 91 left-turning vehicles are doing so in conflict with 324 through vehicles traveling eastbound on Old Trenton Road during the morning peak hour. Thirty-three vehicles make a right-turn from Old Trenton Road onto Robbinsville-Edinburg Road during the morning peak hour. The morning peak hour has 412 vehicles traveling northbound through this intersection, 91 making a left-turn and 321 going right on Old Trenton Road.

Table 18 Intersection Accident Summary (2002-2004)
Old Trenton Rd (CR 535) and Robbinsville Rd (CR 526)

	2002	2003	2004	Total	2002	2003	2004	Total
Crashes								
Reportable	17	19	24	60	28.3%	31.7%	40.0%	78.9%
Non Reportable	2	4	10	16	12.5%	25.0%	62.5%	21.1%
Total	19	23	34	76	25.0%	30.3%	44.7%	100%
Severity								
Injuries	5	7	10	22	22.7%	31.8%	45.5%	
Fatalities	0	0	0	0	0	0	0	
Accident Type								
Same Direction - Rear End	9	14	16	39	47.4%	60.9%	47.1%	51.3%
Left Turn	6	2	7	15	31.6%	8.7%	20.6%	19.7%
Angle	2	3	7	12	10.5%	13.0%	20.6%	15.8%
Hit Fixed Object	2	3	2	7	10.5%	13.0%	5.9%	9.2%
Head-On	0	1	0	1	0.0%	4.3%	0.0%	1.3%
Same Direction - Sideswipe	0	0	1	1	0.0%	0.0%	2.9%	1.3%
Hit Animal	0	0	1	1	0.0%	0.0%	2.9%	1.3%
Time of Day								
Midnight to 6 am	0	0	0	0	0.0%	0.0%	0.0%	0.0%
6 am to Noon	4	7	12	23	21.1%	30.4%	35.3%	30.3%
Noon to 6 pm	9	11	14	34	47.4%	47.8%	41.2%	44.7%
6 pm to Midnight	6	5	8	19	31.6%	21.7%	23.5%	25.0%
Rush Hours								
6 am to 9 am	2	4	6	12	13.3%	33.3%	27.3%	24.5%
11 am to 2 pm	6	4	4	14	40.0%	33.3%	18.2%	28.6%
4 pm to 7 pm	7	4	12	23	46.7%	33.3%	54.5%	46.9%
Light Conditions								
Daylight	14	18	22	54	73.7%	78.3%	64.7%	71.1%
Dark (Street Lights On)	3	3	5	11	15.8%	13.0%	14.7%	14.5%
Dawn or Dusk	2	2	2	6	10.5%	8.7%	5.9%	7.9%
Unknown	0	0	5	5	0.0%	0.0%	14.7%	6.6%
Weather Conditions								
Clear	12	15	29	56	63.2%	65.2%	85.3%	73.7%
Rain	5	8	3	16	26.3%	34.8%	8.8%	21.1%
Snowy	1	0	1	2	5.3%	0.0%	2.9%	2.6%
Fog	1	0	1	2	5.3%	0.0%	2.9%	2.6%
Surface Conditions								
Dry	12	11	29	52	63.2%	47.8%	85.3%	68.4%
Wet	7	10	5	22	36.8%	43.5%	14.7%	28.9%
Icy	0	2	0	2	0.0%	8.7%	0.0%	2.6%

Source: West Windsor Police Department, Accident Reports Data 2002-2004

Figure 23

Collision Diagram   2002-2004

Old Trenton Road at Robbinsville Road

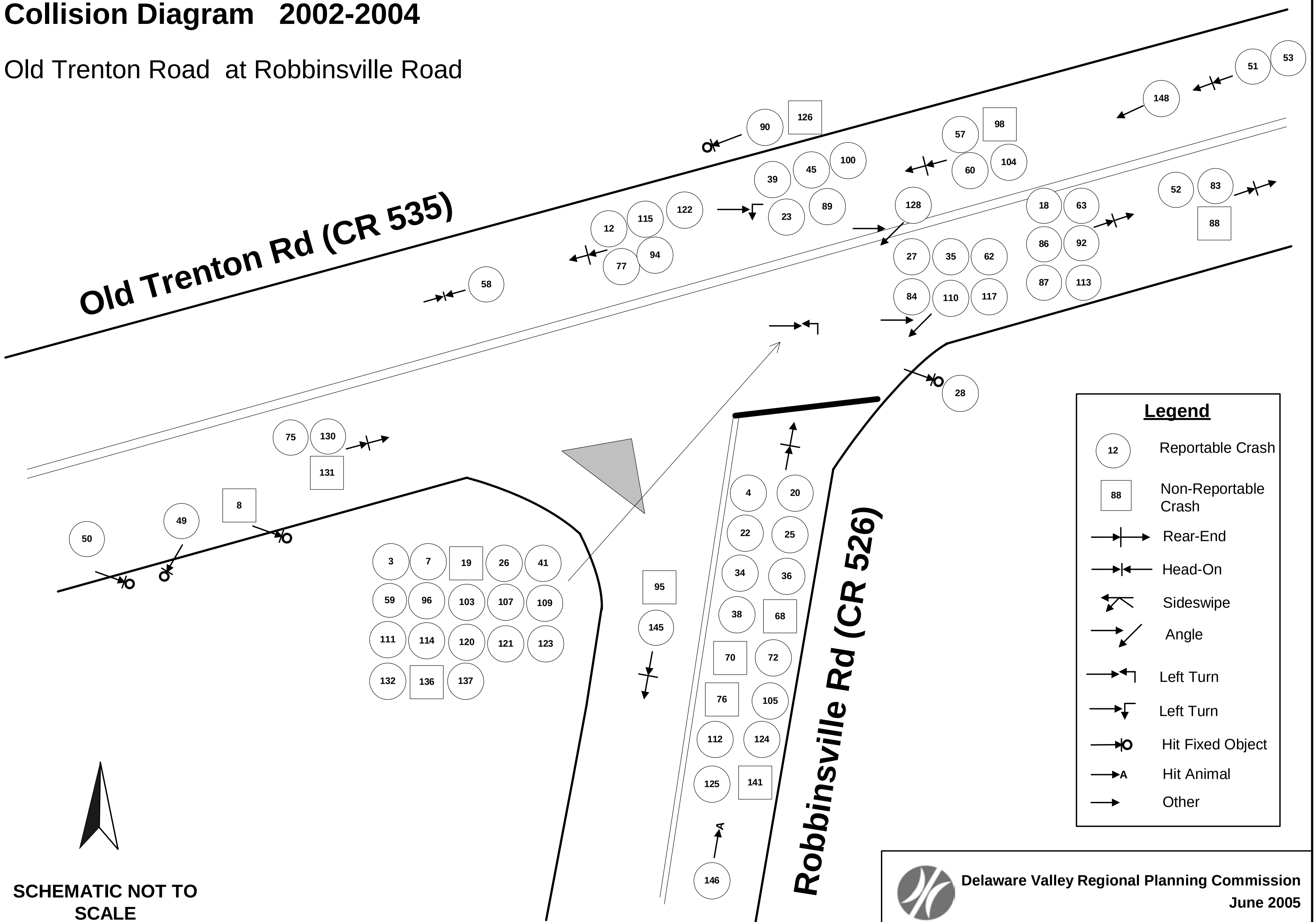

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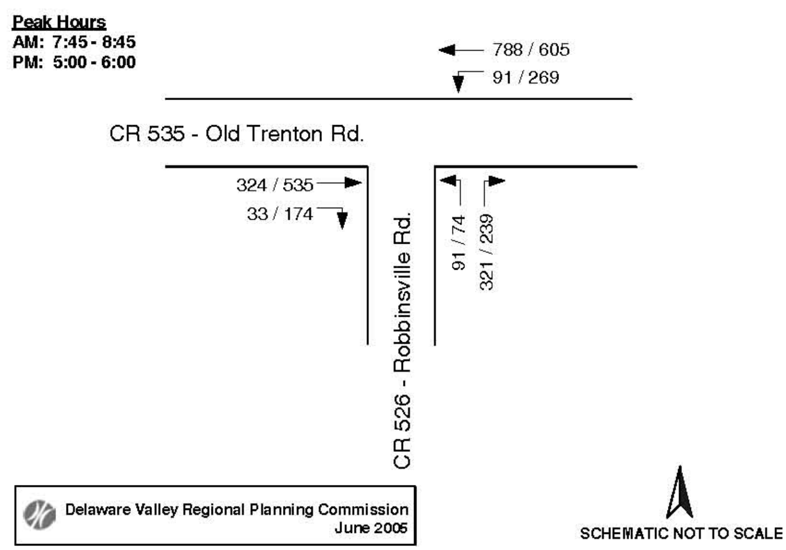
Figure 24 CR 535 - Old Trenton Rd. and CR 526- Robbinsville Rd.

Existing Peak Hour Turning Movement Counts AM / PM



Conflict from left-turning vehicles increase in the afternoon peak hour, with 269 vehicles traveling from westbound Old Trenton Road onto Robbinsville-Edinburg Road. This is three times that of the morning peak hour. The dominant movement at the intersection is still the through movement on Old Trenton Road, 605 traveling westbound and 535 eastbound during the afternoon peak hour. There are 174 vehicles make the right-turn from eastbound Old Trenton Road to Robbinsville-Edinburg Road in the afternoon peak hour, while 74 vehicles make the left-turn from Robbinsville-Edinburg Road onto Old Trenton Road and 239 make the right.

Figure 25 CR 535 - Old Trenton Rd. and CR 526- Edinburg Rd. / CR 641 Windsor Rd

Existing Peak Hour Turning Movement Counts AM / PM

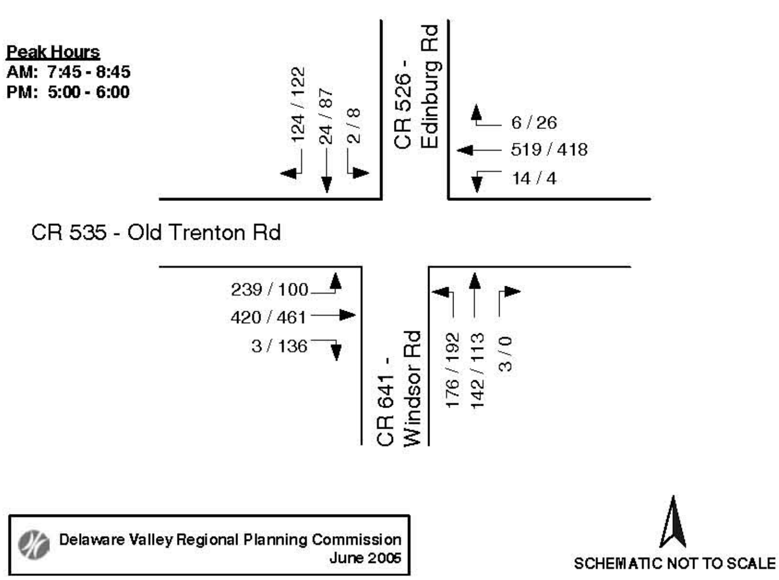


Figure 25 displays the peak hour turning movement counts for this location. The counts were taken on January 18, 2005 in morning from 6 AM to 9 AM and the evening from 4 PM to 7 PM. The morning peak hour is 7:45 AM to 8:45 AM and the afternoon peak hour is 5 PM to 6 PM. At the Edinburg Road/Windsor Road/Old Trenton Road intersection, through movement on Old Trenton Road is the dominant with 519 vehicles westbound during the morning peak hour and 418 in

the afternoon peak hour. Eastbound through movements are 420 vehicles during the morning peak hour and 461 vehicles in the afternoon peak hour. There are 253 vehicles making left-turns in conflict with through movement in both directions during the morning peak hour, however, it is the eastbound that has the most problems with 239 vehicles. From Windsor Road, the dominant movement is the left-turn onto Old Trenton Road during morning and afternoon peak hour, with 174 vehicles and 192 vehicles, respectively. From Edinburg Road, the dominant movement is right-turns for both morning and afternoon peak hours, with 124 and 122 vehicles, respectively. There are 24 vehicles making the through movement from Edinburg Road onto Windsor Road in the morning peak hour and 87 in the afternoon peak hour.

Level of Service

Table 19 shows the existing level of service (LOS) analysis. Edinburg Road/Windsor Road/Old Trenton Road intersection is failing during the morning peak hour with LOS F and 114.2 seconds of delay and LOS D in the afternoon peak hour with 37.5 seconds of delay. The Old Trenton Road eastbound approach also fails during the morning peak hour with LOS F and 229.6 seconds of delay; in the afternoon peak it is LOS C with 27.4 seconds of delay. Old Trenton Road westbound has LOS C with 21.7 seconds of delay in the morning peak hour and LOS B in the afternoon peak hour. Windsor Road approach to the intersection experiences LOS E with 61.7 seconds of delay in the morning peak hour and LOS D with 45.8 seconds of delay in the afternoon peak. Edinburg Road southbound had LOS D with 51.1 seconds of delay in the morning peak hour, and LOS F with 92.1 seconds of delay in the afternoon peak. With the exception of Edinburg Road southbound approach, all approaches have better level of service in the afternoon peak than the morning peak.

For the Robbinsville-Edinburg Road/Old Trenton Road intersection, Synchro was able to produce a LOS using the Intersection Capacity Utilization (ICU) method. Because this is an unsignalized intersection with only one approach that is stop-controlled, this method was chosen. The ICU LOS gives insight into how an intersection is functioning and how much extra capacity is available to handle traffic fluctuations and incidents. ICU is not a value that can be measured with a stopwatch, but it does give a good reading on the conditions that can be expected at the intersection. Under this method, the intersection level of service at Branch Pike at Riverton Road is LOS F in both the morning and afternoon peaks. The Robbinsville-Edinburg Road approach to the intersection is LOS F with 160.8 seconds of delay in the morning peak hour; in the afternoon peak hour, this approach fails with LOS F and 624.5 seconds of delay. Synchro LOS analysis data for each intersection for both the existing and potential improvement scenarios can be found in **Appendix F**.

TABLE 19
Peak Hour Level of Service (LOS) Analysis

CR 535 – Old Trenton Rd & CR 526 - Edinburg Rd / CR 641- Windsor Rd

Improvement Scenario	Direction of Travel	AM Peak LOS	AM Peak Delay (sec)	PM Peak LOS	PM Peak Delay (sec)
Existing Conditions	Old Trenton Rd. EB	F	229.6	C	27.4
	Old Trenton Rd. WB	C	21.7	B	17.9
	Windsor Rd. NB	E	61.7	D	45.8
	Edinburg Rd. SB	D	51.1	F	92.1
	Intersection	F	114.2	D	37.5
Signal Timing Optimization	Old Trenton Rd. EB	E	72.0	C	22.3
	Old Trenton Rd. WB	B	15.9	B	17.4
	Windsor Rd. NB	F	101.3	D	49.9
	Edinburg Rd. SB	F	88.6	E	56.3
	Intersection	E	61.0	C	31.2
1. Optimize & coordinate with new signal at CR 535 / CR 526 during AM peak periods & uncoordinated during PM peiods	Old Trenton Rd. EB	E	67.3	C	22.3
	Old Trenton Rd. WB	B	15.9	B	17.5
	Windsor Rd. NB	F	101.9	D	49.9
	Edinburg Rd. SB	F	88.6	E	56.3
	Intersection	E	59.2	C	31.3
2. Add left-turn lane to the eastbound CR 535 approach	Old Trenton Rd. EB	B	15.1		18.7
	Old Trenton Rd. WB	C	32.2		27.6
	Windsor Rd. NB	E	72.1		39.8
	Edinburg Rd. SB	D	48.4		44.9
	Intersection	C	34.5		29.2
3. Add left-turn lane to the eastbound CR 535 approach; Optimize & coordinate with new signal during AM peak periods & uncoordinated during PM	Old Trenton Rd. EB	B	14.2	B	17.7
	Old Trenton Rd. WB	C	28.8	C	28.6
	Windsor Rd. NB	F	97.1	D	44.6
	Edinburg Rd. SB	D	53.9	D	49.7
	Intersection	D	38.3	C	30.8
4. Add left-turn lane to the eastbound CR 535 & northbound CR 641	Old Trenton Rd. EB	B	13.1	B	17.6
	Old Trenton Rd. WB	C	28.5	C	26.5
	Windsor Rd. NB	D	42.1	D	33.9
	Edinburg Rd. SB	D	43.0	D	35.7
	Intersection	C	26.4	C	26.0
5. Re-align the intersection; Add left-turn lane to the eastbound CR 535 & northbound CR 641; Optimize & coordinate with new signal during AM & PM peak periods; Rehabilitate/replace bridge	Old Trenton Rd. EB	B	11.5	B	14.6
	Old Trenton Rd. WB	C	25.8	C	24.5
	Windsor Rd. NB	C	33.4	D	54.1
	Edinburg Rd. SB	D	51.4	D	53.6
	Intersection	C	23.9	C	30.9

CR 535 – Old Trenton Rd & CR 526 Robbinsville Rd

Improvement Scenario	Direction of Travel	AM Peak LOS	AM Peak Delay (sec)	PM Peak LOS	PM Peak Delay (sec)
Existing Conditions	Old Trenton Rd. EB		N/A		N/A
	Old Trenton Rd. WB		N/A		N/A
	Robbinsville Rd. NB	F	160.8	F	624.5
	Intersection	F	41.3	F	106.8
1. Signalize Intersection; Add left turn lane on CR 535 westbound; Add left turn lane on CR 526 northbound	Old Trenton Rd. EB	A	7.8	A	5.2
	Old Trenton Rd. WB	B	13.3	A	9.5
	Robbinsville Rd. NB	C	25.5	A	9.2
	Intersection	B	15.1	A	7.9
2. Add left turn lane on CR 535 westbound	Old Trenton Rd. EB		N/A		N/A
	Old Trenton Rd. WB		N/A		N/A
	Robbinsville Rd. NB	D	30.2	F	137.6
	Intersection	A	8.0	C	24.3
3. Signalize Intersection; Add left turn lane on CR 535 westbound; Add left turn lane on CR 526 northbound	Old Trenton Rd. EB	A	4.4	A	5.1
	Old Trenton Rd. WB	A	2.6	A	8.8
	Robbinsville Rd. NB	A	4.8	A	6.7
	Intersection	A	3.5	A	7.1
4. Add left turn lane on CR 535 westbound; Add left turn lane on CR 526 northbound	Old Trenton Rd. EB		N/A		N/A
	Old Trenton Rd. WB		N/A		N/A
	Robbinsville Rd. NB	D	30.2	F	137.6
	Intersection	A	8.0	C	24.3
5. Signalize Intersection; Add left turn lane on CR 535 westbound; Add left turn lane on CR 526 northbound	Old Trenton Rd. EB	A	4.1	A	2.9
	Old Trenton Rd. WB	A	3.6	A	1.7
	Robbinsville Rd. NB	A	5.2	B	10.5
	Intersection	A	4.1	A	3.6

Opportunities and Constraints

A major constraint for the study location is a lack of additional right-of-way. Buildings are close to the curb with little or no setback at the Edinburg Road/Windsor Road/Old Trenton Road intersection.

Potential capacity increase at the Edinburg Road/Windsor Road/Old Trenton Road intersection will require the removal of buildings located on the intersection. As the hotel on the southeast corner of the Edinburg Road/Windsor Road/Old Trenton Road intersection is currently vacant, this presents an opportunity to acquire this property.

Looking west at Edinburg Rd. / Windsor Rd. /Old Trenton Rd. intersection

The number of accidents recorded in the study location represents a safety problem. The study location exceeds the statewide percentage for same direction rear end accidents, left-turn accidents and fixed object accidents that are 28.79 percent, 5.9 percent and 13.56 percent, respectively, according to the NJDOT Accident Summary for Statewide County Road Systems.

The bridge on Old Trenton Road over the Assunpink Creek also presents a constraint in any proposal for capacity increase. Currently, the bridge is deficient and will only get worse with time, however, federal funds may be used to replace or rehabilitate this bridge.

Assunpink Creek Bridge on Old Trenton Road

Potential Improvement Scenarios

Several improvement scenarios were considered for the study location. Synchro was used to determine level of service and amount of delay and can be found for each scenario in **Table 19**. A schematic diagram of the proposed lane configurations for each scenario is illustrated in **Figure 26.**

As well as the five scenarios detailed below, a scenario was run with just the optimization of the traffic signal at the Windsor Road/Edinburg Road/Old Trenton Road intersection. There was no change in the LOS or delay for the Edinburg-Robbinsville Road/Old Trenton Road intersection or their approaches in both morning and afternoon peak. At the signalized intersection for the morning peak hour, it improved from a LOS F to E with a 53-second fall in delay to 61 seconds. All approaches improved except westbound Old Trenton Road, which deteriorated from LOS B to C with a 5.8-second increase in delay. Eastbound Old Trenton Road improved the most in terms of delay; it improved from 229.6 seconds to 72 seconds, from LOS F to E. Northbound Windsor Road and southbound Edinburg Road improved from LOS F to E and from LOS F to D respectively, and reducing seconds of delay by 39.6 and 37.5 seconds respectively. In the afternoon peak hour, the intersection improved from LOS D to C with a 6.3 second drop in delay to 31.2 seconds. All approaches have the same LOS with small improvement in delay except southbound Edinburg Road, which went from LOS F to E and from 91.7 seconds of delay to 56.3.

Scenario 1

Characteristics

- Signalize Edinburg-Robbinsville Road/Old Trenton Road intersection.
- Add left-turn lane at the westbound Old Trenton Road approach.
- Optimize signal at Windsor Road/Edinburg Road/Old Trenton Road intersection and coordinate with the new signal during the morning peak and uncoordinated during the afternoon peak.

Advantages

- Added capacity with left-turn lane at westbound Old Trenton Road approach.
- Protected left-turn from signal helps to alleviate congestion.
- Conflict with turning traffic from Edinburg-Robbinsville Road on to Old Trenton Road is controlled.
- Minimal right-of way acquisition, county-owned land at this intersection can be utilized.

Disadvantages

- Right-of-way needed for widening
- Widening may cause encroachment on wooded/farmland that may be environmentally sensitive.
- Unless bridge is rehabilitated or replaced, it restricts widening.

LOS Analysis

A SYNCHRO analysis was performed for this scenario using existing morning and afternoon peak hour data.

The Edinburg Road/Windsor Road/Old Trenton Road intersection improves from LOS F to E, with delay improving from 114.2 seconds to 59.2 seconds during the morning peak. Old Trenton Road eastbound has the most dramatic change of all the approaches, from LOS F to E with a more than 160 seconds drop in delay to 67.3 seconds. Northbound Windsor Road and southbound Edinburg Road deteriorates from LOS E and D, respectively, to LOS F. Westbound Old Trenton Road also improves from LOS C to B. During the afternoon peak, the intersection improves from LOS D to C, with a six second fall in delay. Edinburg Road southbound has the most change, improving from LOS F to E with a 35-second fall in delay. The other approach has the same LOS with minor changes in the seconds of delay.

The Edinburg-Robbinsville Road/Old Trenton Road intersection improves from LOS F to B in the morning peak and from LOS F to A in the afternoon peak. The improvement or deterioration in level of delay for an intersection from adding a signal is not a straight comparison. Therefore,

Figure 26

Potential Improvement Scenarios - Lane Configuration

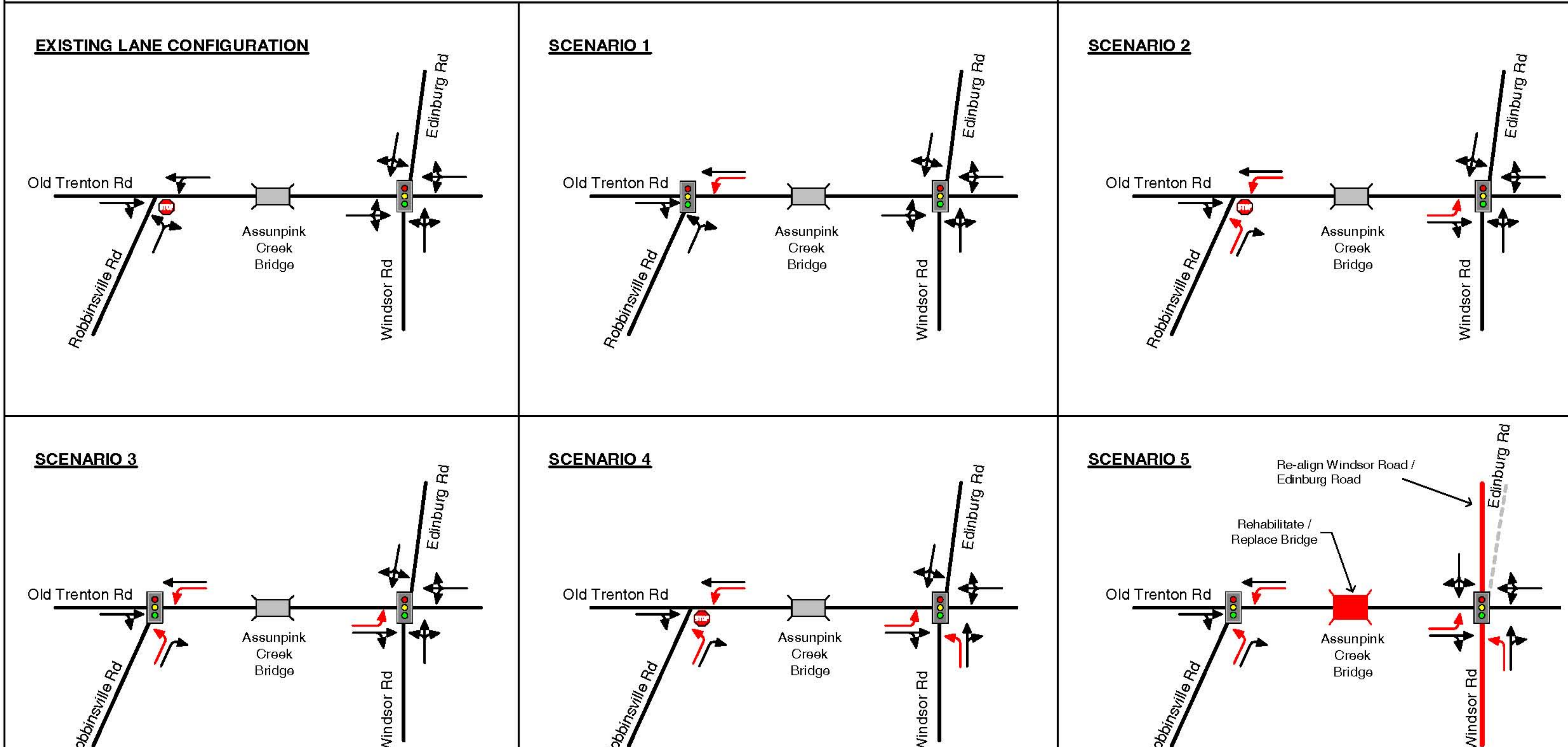
CR 535 - Old Trenton Road at CR 526 - Edinburg Road / CR 641- Windsor Road &
CR 535 - Old Trenton Road at CR 526 Robbinsville Road

West Windsor, NJ

Legend

- Existing Lane
- Added Dedicated Left Turn Lane
- Signalized Intersection
- Stop sign controlled intersection
- Existing Alignment

SCHEMATIC NOT TO SCALE



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the study will not consider delay at the Edinburg-Robbinsville Road/Old Trenton Road intersection before and after the proposed addition of the signal.

Scenario 2

Characteristics

- Add a dedicated left-turn lane to the eastbound approach of the Windsor Road/Edinburg Road/Old Trenton Road intersection.
- Program signal for protected left-turn for eastbound traffic to account for turning lane.
- Add left-turn lanes at the westbound Old Trenton Road approach at the Edinburg-Robbinsville Road/Old Trenton Road intersection and northbound Edinburg-Robbinsville Road. Intersection remains unsignalized with stop sign control.

Advantages

- Dedicated left-turn lanes helps to alleviate congestion at the intersections.
- Protected left-turn signal will potentially reduce the conflict between left-turning vehicles and through traffic.
- Minimal right-of way acquisition required. The county-owned land at southwest quadrant of this intersection could be utilized.

Disadvantages

- Right-of-way acquisition needed for additional capacity.
- Widening needs to be extended to southeast quadrant of the intersection for alignment – limited/no available right-of-way.
- Widening may cause encroachment on wooded/farmland that may be environmentally sensitive.
- Unless bridge is rehabilitated or replaced, it restricts the length of widening.

LOS Analysis

A Synchro analysis was performed for this scenario using existing morning and afternoon peak hour data.

The Edinburg Road/Windsor Road/Old Trenton Road intersection improves from LOS F to C, with delay falling from 114.2 seconds to 34.5 seconds during the morning peak. Old Trenton Road eastbound has the most dramatic change of all the approaches. The LOS improves from an F to B, with a more than 200 second drop in delay to 15.1 seconds. The other approaches' LOS remain the same with changes in the delay. Northbound Windsor Road and westbound Old Trenton Road experience an increase in delay while southbound Edinburg Road falls slightly. During the afternoon peak, the intersection improves from LOS D to C, with an eight

second fall in delay. Edinburg Road southbound has the most change, improving from LOS F to D with a 47-second fall in delay. Old Trenton Road eastbound also improves LOS, from C to B, and westbound improves from C to B with more than 10 seconds of delay decrease.
The Edinburg-Robbinsville Road/Old Trenton Road intersection improves dramatically from LOS F to A in the morning peak and from LOS F to C in the afternoon peak.

Scenario 3

Characteristics

- Signalize the Edinburg-Robbinsville Road/Old Trenton Road intersection.
- Add left-turn lanes in the westbound Old Trenton Road approach at the Edinburg-Robbinsville Road/Old Trenton Road intersection and for northbound Edinburg-Robbinsville Road.
- Add a dedicated left-turn lane to the eastbound approach of the Windsor Road/Edinburg Road/Old Trenton Road intersection.
- Signal at Windsor Road/Edinburg Road/Old Trenton Road intersection with protected left-turn for eastbound traffic is optimized and coordinated with the new signal in the morning peak, but uncoordinated with it in the afternoon.

Advantages

- Added capacity with left-turn lane at westbound Old Trenton Road approach at the Edinburg-Robbinsville Road/Old Trenton Road intersection and eastbound Old Trenton Road approach at Windsor Road/Edinburg Road/Old Trenton Road intersection.
- Protected left-turn from signal helps to alleviate congestion.
- Conflict with turning traffic from Edinburg-Robbinsville Road on to Old Trenton Road is controlled.
- Protected left-turn signal will potentially reduce the conflict between left-turning vehicles and through traffic.
- Minimal right-of way acquisition, county-owned land at intersections can be utilized.

Disadvantages

- Right-of-way acquisition needed for additional capacity.
- Widening needs to be extended to southeast quadrant of the intersection for alignment – limited/no available right-of-way.
- Widening may cause encroachment on wooded/farmland that may be environmentally sensitive.
- Unless bridge is rehabilitated or replaced, it restricts the length of widening.

LOS Analysis

A Synchro analysis was performed for this scenario using existing morning and afternoon peak hour data.

The Edinburg Road/Windsor Road/Old Trenton Road intersection improves from LOS F to D, with delay falling from 114.2 seconds to 38.3 seconds during the morning peak. Old Trenton Road eastbound has the most dramatic change of all the approaches. There is an improvement from a LOS F to B and a 215-second drop in delay to 14.2 seconds. Northbound Windsor Road approach is failing with LOS F and 97.1 seconds of delay, an increase of 35.4 seconds. Southbound Edinburg Road and westbound Old Trenton Road remains at the same LOS as existing, with slight increases in delay. During the afternoon peak, the intersection improves from LOS D to C, with a 6.7-second fall in delay. Edinburg Road southbound has the most change, improving from LOS F to D with a 42-second fall in delay. Old Trenton Road eastbound also improves from LOS C to B, with approximately 10 seconds drop in delay, while westbound deteriorates from LOS B to C, with 10 seconds of increase in delay.

The Edinburg-Robbinsville Road/Old Trenton Road intersection improves dramatically from LOS F to A in the morning peak and from LOS F to A in the afternoon peak.

Scenario 4

Characteristics

- Add a dedicated left-turn lane to the eastbound approach of the Windsor Road/Edinburg Road/Old Trenton Road intersection.
- Add a dedicated left-turn lane to the northbound approach of the Windsor Road/Edinburg Road/Old Trenton Road intersection.
- Optimize and program signal for protected left-turns for eastbound and northbound traffic.
- Add left-turn lanes at the westbound Old Trenton Road approach at the Edinburg-Robbinsville Road/Old Trenton Road intersection and at northbound Edinburg-Robbinsville Road. Intersection remains unsignalized with stop sign control.

Advantages

- Dedicated left-turn lanes help to alleviate congestion at the intersections.
- Protected left-turn signal will potentially reduce the conflict between left-turning vehicles and through traffic.
- Minimal right-of way acquisition required. It is possible to utilize county-owned land at southwest quadrant of this intersection.

Disadvantages

- Right-of-way acquisition needed for additional capacity.
- Widening needs to be extended to southeast quadrant of the intersection for alignment – limited/no available right-of-way.
- Widening may cause encroachment on wooded/farmland that may be environmentally sensitive.
- Unless bridge is rehabilitated or replaced, it restricts the length of widening.

LOS Analysis

A SYNCHRO analysis was performed for this scenario using existing morning and afternoon peak hour data.

The Edinburg Road/Windsor Road/Old Trenton Road intersection goes from LOS F to C, with delay falling from 114.2 seconds to 26.4 seconds during the morning peak. Old Trenton Road eastbound has the most dramatic change of all the approaches with an improvement of LOS F to B. There is a 216 seconds drop in delay to 13.1 seconds. Northbound Windsor Road improves from LOS E to D, from 61.7 seconds of delay to 42.1. LOS for the other approaches remains the same with some change in delay. During the afternoon peak, the intersection improves from LOS D to C with 11.5 seconds fall in delay. Edinburg Road southbound has the most change going from LOS F to D with 56 seconds fall in delay. Old Trenton Road eastbound also improves LOS, from C to B; westbound deteriorates slightly from B to C with 8.6 seconds of delay increase. Windsor Road northbound approach improves from LOS D to C with 11.9 seconds decrease in delay.

The Edinburg-Robbinsville Road/Old Trenton Road intersection increased dramatically from LOS F to A in the morning peak and from LOS F to C in the afternoon peak.

Scenario 5

Characteristics

- Signalize the Edinburg-Robbinsville Road/Old Trenton Road intersection.
- Add left-turn lanes at the westbound and northbound approaches of the Edinburg-Robbinsville Road/Old Trenton Road intersection.
- Add a dedicated left-turn lane to the eastbound and northbound approaches of the Windsor Road/Edinburg Road/Old Trenton Road intersection.
- Optimize the signal at the Windsor Road/Edinburg Road/Old Trenton Road intersection with protected left-turn for eastbound and northbound traffic, and coordinate with new signal during both morning and afternoon peaks.

- Re-align the Windsor Road/Edinburg Road/Old Trenton Road intersection. Align Old Trenton Road and Windsor Road with Edinburg Road. This can be accomplished through acquisition of the vacant building (formerly hotel) on the southeast quadrant of the intersection and the home on the northwest quadrant of the intersection.
- Rehabilitate/replace bridge with a wider structure for added lane capacity.

Advantages

- Additional roadway capacity with left-turn lanes and new bridge.
- Dedicated left-turn helps to alleviate congestion.
- Protected left-turn signal will potentially reduce the conflict between left-turning vehicles and through traffic.
- Re-alignment of the intersection will relieve some of the sight-distance problems.
- Right-turn on red allowed at the intersection that may help to relieve congestion.

Disadvantages

- Right-of-way acquisition for geometric improvements could be costly.
- Widening may cause encroachment on wooded/farmland that may be environmentally sensitive.
- Though the buildings on the intersection are not registered as historic with the state, the possibility exists that they are. There may be neighborhood objections to their removal.
- Bridge rehabilitated or replaced could be costly and time-consuming.

LOS Analysis

A SYNCHRO analysis was performed for this scenario using existing morning and afternoon peak hour data.

The Edinburg Road/Windsor Road/Old Trenton Road intersection improves from LOS F to C, with delay falling from 114.2 seconds to 23.9 seconds during the morning peak. Old Trenton Road eastbound has the most dramatic change of all the approaches of LOS F to B with a 218.1-second drop in delay to 11.5 seconds. Northbound Windsor Road improves from LOS E to C, from 61.7 seconds of delay to 33.4 seconds. Westbound Old Trenton Road and Edinburg Road southbound LOS remain the same with 4.1 and 0.3 seconds of delay increase respectively. During the afternoon peak, the intersection improves from LOS D to C with a 6.6-second fall in delay. Edinburg Road southbound has the most change, improving from a LOS F to D with a 38.1-second fall in delay. Old Trenton Road eastbound approach improves from LOS C to B with 12.8 seconds of delay, while the westbound approach decreases from LOS B to C with 6.6 seconds of increase in delay. Windsor Road northbound approach has the same LOS with 8.3 seconds of increase in delay.

The Edinburg-Robbinsville Road/Old Trenton Road intersection improves dramatically from LOS F to A in the morning peak and from LOS F to A in the afternoon peak.

Recommendations

With increasing development in the area surrounding the study location, there will be an increase in traffic through and in this location. The congestion and safety problems identified will potentially multiply with traffic increase. Therefore, problems need to be addressed immediately. These arterials carry both local and regional traffic; therefore, strategies will effect regional movement.

In the near term, the possibility of optimizing the traffic signal at the Edinburg Road/Windsor Road/Old Trenton Road intersection should be pursued. Our analysis shows that this has the potential to improve the delay of the overall intersection and all but one of its approaches during the morning peak. The benefits to the intersection from optimization as shown in our analysis far exceeds its costs, since the westbound Old Trenton Road approach, which is adversely affected, will only have minor delays, as seen in the LOS analysis. During the afternoon peak, optimization of the signal also improves the traffic flow at the intersection and its individual approaches.

In the short term, due to the safety problems that exist from conflicting movements at the Edinburg-Robbinsville Road/Old Trenton Road intersection, the possibility of installing a traffic signal to control traffic movement at the intersection should be further evaluated. In addition, a left-turn lane should be added to the westbound Old Trenton Road approach of the intersection. This signal can be coordinated with the signal at the Edinburg Road/Windsor Road/Old Trenton Road intersection to provide safe, efficient flow of traffic through the region.

In the long term, to alleviate congestion and improve safety at the study location, a more in-depth study of the combination of strategies of Scenario *5* should be pursued. Windsor Road needs to be realigned with Edinburg Road at the Edinburg Road/Windsor Road/Old Trenton Road intersection. This will require at least the removal of the building (hotel) on the southeast corner of the intersection and the building at the northwest corner. It is also recognized that any widening of Old Trenton Road utilizing south-side county property to provide an eastbound left-turning lane will require removal of the hotel. This is necessary to provide adequate right-of-way for through traffic on Old Trenton Road at this intersection. In addition, the Old Trenton Road Bridge between the two intersections should be replaced, as currently it is deficient. It should be replaced with a wider bridge that will provide additional roadway width for left-turning lanes and bike and pedestrian facilities.

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APPENDIX

APPENDIX A

Chester County PA 41 (Gap Newport Pike) at PA 10 (Limestone Road)

- Crash Data
- Turning Movement Counts
- Classification Counts
- Level of Service Analysis Worksheets

APPENDIX B

Delaware County PA 420 (Kedron Avenue) at MacDade Boulevard
PA 420 (Kedron Avenue) at Academy Avenue / 4th Avenue

- Crash Data
- Turning Movement Counts
- Level of Service Analysis Worksheets

APPENDIX C

Philadelphia 34th Street at Grays Ferry Avenue

- Crash Data
- Turning Movement Counts
- Level of Service Analysis Worksheets

APPENDIX D

Burlington County	Riverton Road (CR 603) at Branch Pike (CR 606) Riverton Road (CR 603) at Parry Road Branch Pike (CR 606) at Parry Road

– Crash Data

– Turning Movement Counts

– Level of Service Analysis Worksheets

APPENDIX E

Camden County Haddonfield Road (CR 561) at White Horse Road (CR 673)

- Crash Data
- Turning Movement Counts
- Level of Service Analysis Worksheets

APPENDIX F

Mercer County — Old Trenton Road (CR 535) at Robbinsville-Edinburg Road (CR 526)
Old Trenton Road at Windsor Road (CR 641) / Edinburg Road (CR 526)

- Crash Data
- Turning Movement Counts
- Level of Service Analysis Worksheets

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Geographic Area Covered: West Fallowfield Township, Chester County; Ridley Township, Delaware County; and the City of Philadelphia in Pennsylvania. Cinnaminson Township, Burlington County; Voorhees Township, Camden County; West Windsor Township, Mercer County in New Jersey.

Key Words: intersection analysis, traffic congestion, potential improvement scenarios, accidents, crash analysis, level of service analysis

Abstract: This report represents a planning effort to support the local counties and municipalities in both New Jersey and Pennsylvania in addressing the safety and mobility issues along their arterial road network. This network can typically experience congested conditions due to high traffic volumes and or limited capacity. Accidents occurring along these congested facilities not only result in injuries but also add to the congestion. The goal is to identify potential cost-effective improvement strategies, which will reduce congestion and accidents and improve the safety and mobility of goods and people.

Working with the local county planning commissions, DVRPC selected six locations to study. For each of these locations, field views to review transportation problem locations were undertaken, and consequently technical analysis to quantify the identified transportation problem areas and document practical solutions. Level of service analyses and accident analyses were conducted for each selected area.

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