Traffic and Safety Engineering Services to Saginaw County Metropolitan Planning Commission

Final Draft Report

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Traffic and Safety Engineering Services to MPOs

A Study of Selected Intersections in Saginaw County, MI

Final Draft Report

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Date: October 31, 2006

The opinions, findings, and conclusions expressed in this publication are those of the author(s) and not necessarily those of the Michigan Office of Highway Safety and Planning of the U.S. Department of Transportation, Federal Highway Administration. This report was prepared in cooperation with the Michigan Office of Highway Safety Planning and U.S. Department of Transportation, Federal Highway Administration.

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1.0 INTRODUCTION

Efficient transportation systems provide safe and convenient movement of people and goods. Efficiency of traffic flow and highway safety are key factors in any short or long-range transportation planning process. The optimization of safety and mobility are generally achieved through planning, design, construction, operation, and maintenance of the area-wide transportation system. Metropolitan Planning Organizations (MPOs), which consist of various local transportation agencies including cities, county road commissions, public transit agencies, as well as the Michigan Department of Transportation (MDOT), coordinate transportation planning activities in the urban areas. One important responsibility of an MPO is to identify and prioritize the traffic operational and safety needs of the highway network, and invest available resources in a cost-effective manner in projects and programs, which maximize societal benefits.

The Wayne State University-Transportation Research Group (WSU-TRG), through a grant provided by the Office of Highway Safety Planning (OHSP) has performed a comprehensive traffic crash and operational analysis to assist the Saginaw Metropolitan Planning Commission in Michigan, in identifying safety needs and mitigating existing crash hazards.

In October 2005, OHSP sponsored a project for the WSU-TRG to perform a comprehensive crash and operational analysis at a site-specific level within the boundaries of the Saginaw Metropolitan Planning Commission (MPO).

The following provides the purpose and objectives of the Traffic and Safety Engineering Services to MPO's project, as well as a brief description of the study area along with an overview of the methodology used for the study.

1.1 Study Purpose and Objectives

The purpose of this study is to conduct a review of the traffic conditions and crash characteristics of high crash locations within the MPO's boundary, to identify operational and driver behavior issues that may be affecting safety and to develop potential mitigation measures to reduce the potential crash risk at selected locations. The study includes implementation, staging and evaluation plans for assessing future changes in crash patterns attributable to the implemented safety improvements.

The specific objectives of this study are as follows:

- Perform an MPO-wide crash analysis to identify candidate sites for detailed crash analysis and on-site investigations.
- Conduct a site-specific detailed traffic crash analysis of selected locations.
- Collect and analyze traffic operational data at the selected study sites.
- Identify operational and safety deficiencies, and develop engineering and behavioral countermeasures.
- Recommend site-specific and policy-type engineering and driver behavioral measures.
- Suggest an implementation and an evaluation plan for assessing the effectiveness of the countermeasures.
- Prepare a study report of the results.

1.2 Report Outline

This report contains eight main sections, covering details of data collection, analysis and site-specific recommendations.

Section 1.0: Contains general background information, study purpose and objectives, overview of the study area, methodologies adopted to perform the various analyses and a brief outline of the structure of the report.

Section 2.0: Contains county-level crash characteristics of the Saginaw County Metropolitan Commission.

Section 3.0: Describes the selection process for the 40 study intersections.

Section 4.0: Describes the physical characteristics of the major corridors including general traffic volumes and traffic control, results of signal progression analyses, and crash patterns of driver behavior-related crashes related to drinking and driving, lack of seatbelt usage, speeding and red light running.

Section 5.0: Contains a list of suggested site-specific countermeasures for implementation.

Section 6.0: Provides a thorough discussion of each of the study intersections as well as the results of detailed site-specific analysis including physical characteristics, traffic volumes,

traffic control, crash analysis, safety deficiencies, recommended improvements and operational (LOS) evaluations.

Section 7.0: Provides a procedural guide for conducting safety evaluations, once the countermeasures have been installed.

Section 8.0: List of policy-type and driver behavior-related recommendations to improve safety in the Saginaw MPO.

1.3 Study Area

The Saginaw Metropolitan Planning Commission supports, augments and advises the planning efforts conducted by the cities and townships in Saginaw County. This MPO is located near Bay County to its north, Shiawassee County to its south, Tuscola County to its east and Gratiot County to its west.

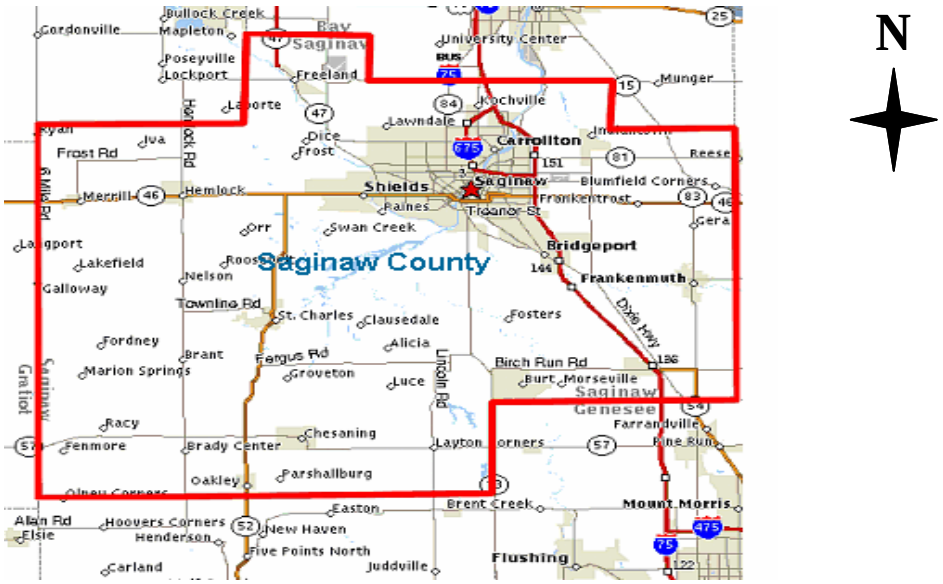


Figure 1.1. Saginaw Metropolitan Planning Commission Area

The preliminary steps of this study consisted of the analysis of the top 100 intersections within Saginaw County to determine the crash and injury frequencies for a two-year study period (2003-2004). Based on this review, 40 intersections were identified as having 'high' to 'moderate' crash experiences and were included in the detailed site-specific analysis. The majority of these selected intersections were located in the City of Saginaw, Saginaw Township, Thomas Township and Kochville Township. The candidate intersections included in this project are shown in Figure 1.2 and are as follows:

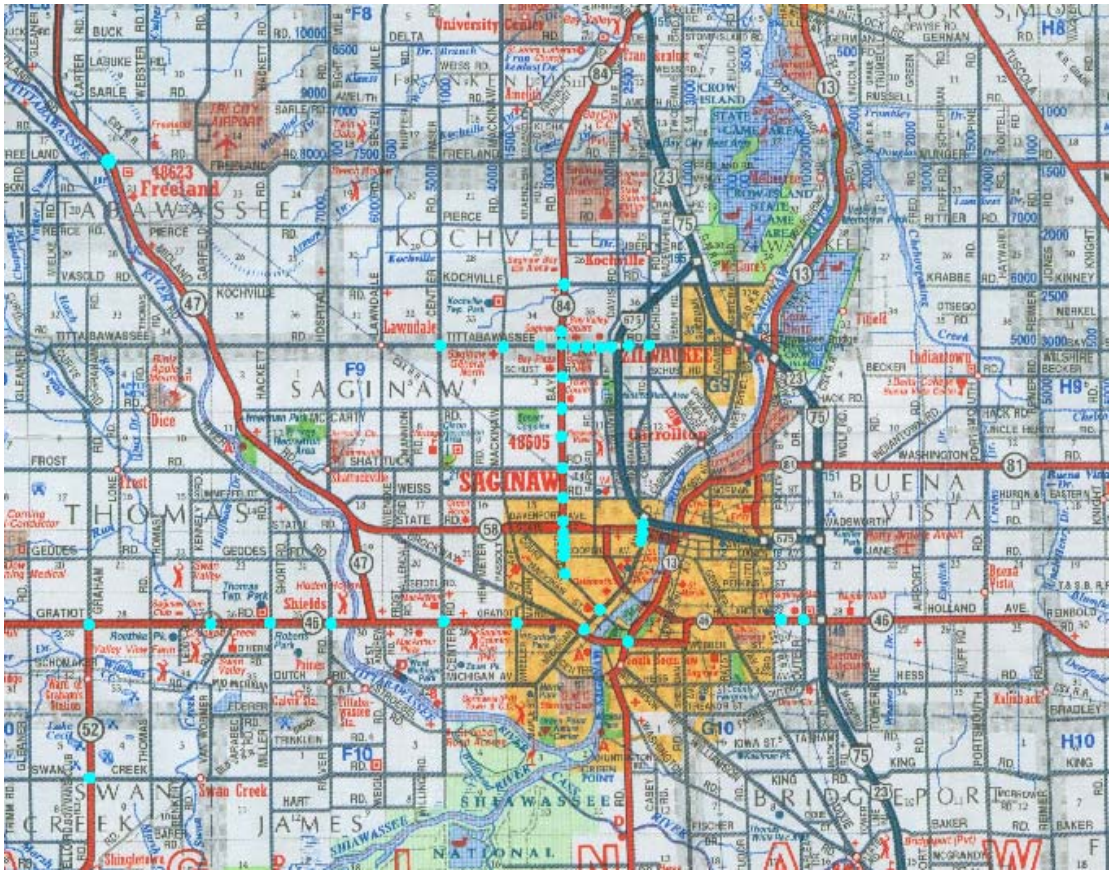


Figure 1.2 Location of the Study Intersections for Saginaw MPO

1. Bay Road and Kochville Road
2. Bay Road and Bueker Drive
3. Bay Road and Tittabawassee Road
4. Bay Road and Schust Road
5. Bay Road and McCarty Road
6. Bay Road and Universal Drive
7. Bay Road and Shattuck Road
8. Bay Road and Weiss Road
9. Bay Street and Davenport Avenue
10. Bay Street and State Street
11. Bay Street and West Genesee Street
12. Bay Street and Congress Avenue
13. Bay Street and Cooper Avenue

14. Bay Street and Court Street
15. Tittabawassee Road and North Center Road
16. Tittabawassee Road and Mackinaw Road
17. Tittabawassee Road and Fortune Boulevard
18. Tittabawassee Road and Fashion square Boulevard
19. Tittabawassee Road and Berl Drive
20. Tittabawassee Road and Towne Center Road
21. Tittabawassee Road and SB I675
22. Tittabawassee Road and NB I675
23. Tittabawassee Road and North Michigan Avenue
24. Gratiot Road and Graham Road
25. Gratiot Road and Kennely Road
26. Gratiot Road and Van Warmer Road
27. Gratiot Road and Miller Road
28. Gratiot Road and River Road
29. Gratiot Road and Center Road
30. Gratiot Road and Wheeler Road
31. North Michigan Avenue and Davenport Avenue
32. North Michigan Avenue and State Street
33. North Michigan Avenue and West Genesee Avenue
34. Michigan Avenue and Court Street
35. Michigan Avenue, Mackinaw Street and Gratiot Avenue
36. East Holland Avenue and South Outer Drive
37. East Holland Avenue and Cumberland Street
38. Rust Avenue and Washington Avenue
39. West Freeland Road and Midland Road
40. South Graham Road and Swan Creek Road

1.4 Study Methodology

In order to provide comprehensive traffic safety and engineering services for the Saginaw County MPO, this study consisted of three main activities (as presented below), which encompassed a variety of sub-tasks. They are as follows:

1) Investigation of the crash characteristics at an MPO level to determine the annual crash trends using charts, graphs and Geographic Information System (GIS) maps to identify crash patterns in Saginaw County. The crash data for this analysis was obtained by querying the computerized Statewide Traffic Crash Database of the Michigan State Police (MSP) for the years 2003 and 2004.
2) Identification of the crash experience at candidate sites and the selection of intersections for detailed analysis based on the most recent traffic crash data for two years (2003- 2004).
3) Detailed intersection and corridor analyses to review physical, traffic and crash characteristics, and to identify safety and operational deficiencies, as well as driver behavioral issues in order to develop mitigation strategies, the following methodology was used:

   - Conducted on-site investigations to collect intersection geometry, lane use and designation, adjacent land uses, location of curb cuts, traffic control devices and all relevant features within an approximate 150-foot radius of the intersection.
   - Reviewed traffic characteristics by collecting manual turning and through movement counts during a 'typical' weekday (Tuesday, Wednesday, Thursday) PM peak period (4:00 to 6:00 PM) using a one or two-person team, depending on the intersection size and volume of traffic that the intersection carried.
   - Evaluated the intersections' capacity/level of service (LOS) using the most recent version of Highway Capacity Software (HCS), which applies the techniques described in the *Highway Capacity Manual*, published by the Transportation Research Board in 2000.
   - Analyzed traffic crash data for selected high crash locations using data obtained from the Michigan State Police for the years 2003 and 2004 (downloaded UD-10 traffic crash reports from MSP website). In order to

compile the crash reports for the study intersections, the downloaded UD-10 forms were carefully reviewed, sorted and printed. The number of crashes obtained from the UD-10 reports and the number contained in the Statewide Traffic Crash Database were compared for each intersection by year as a check for accuracy. The hardcopies of the UD-10 reports were then used in causation/pattern analysis to prepare summary tables and collision diagrams.

- Analyzed traffic crash data to identify driver behavioral issues related to alcohol use, seatbelt use, speeding and red light running. Recommendations for enforcement and educational-type safety measures were made based on this analysis.
- Used the results of the detailed traffic crash analysis, operational analysis, analysis of crashes related to driver behavioral issues and safety-related issues at a corridor and intersection-specific level. Safety improvements were then recommended based on the predominant crash types, identified safety deficiencies and probable cause of crashes, as outlined in various publications [*Highway Safety Improvements Program (HSIP) Procedural Guide* (DOT-FH-11-9679), Federal Highway Administration, 1981; National Cooperative Highway Research Program (NCHRP) Report 457, *Evaluating Intersection Improvements: An Engineering Study Guide*, Transportation Research Board, National Research Council, 2001; *Manual on Identification, Analysis and Correction of High-Crash Locations*, 3rd Edition, Missouri Department of Transportation Technology Transfer Assistance Program , 1999]. It should be noted that the crash causation analysis focused on identifying specific crash types that could be mitigated by the installation of a countermeasure. This type of crash is known as a 'correctable' crash. For example, if a crash analysis reveals a predominance of left-turn head-on crashes, a countermeasure such as the installation of a left turn lane and/or left turn signal is expected to correct the crash problem. However, if a crash occurs due to driver distraction, such as stopping at a green signal to read a map (or any other type of distraction), specific engineering countermeasures could not be suggested to mitigate this type of crash. Nevertheless, driver awareness campaigns

may be organized to spread a general message about the hazards of distracted driving. Thus, this study focused on identifying correctable crashes, determining the probable causes and selecting well-known cost-effective countermeasures to mitigate the crash problems.

2.0 CRASH CHARACTERISTICS IN SAGINAW COUNTY

As a part of this study, the WSU-TRG conducted various levels of traffic crash analyses for the Saginaw County area in order to select locations to be studied in a comprehensive manner.

2.1 Comparison of Crash Trends in Michigan and Saginaw County

The analysis began with a comparative review of crash trends for the State of Michigan and Saginaw County. The overall crash, injury and fatal crash frequencies were determined for an eight-year period (1997 to 2004), and the crash trends were identified at a state and county level, as shown in Figures 2.1 and 2.2. The overall/total crash frequency in Michigan initially showed a decreasing trend from 1997 to 1998, followed by an increasing trend from 1998 to 2000, and then decreased again from 2000 to 2004. Similar trends were observed in the overall crash experience in Saginaw County, which showed a decreasing trend from 1997 to 1998, an increasing trend from 1998 to 2000, and a drastic decreasing trend from 2000 to 2001, followed by a moderate decreasing trend from 2001 to 2004. The trend of injury crashes continually decreased over the years 1997 to 2004 in both the State and Saginaw County. The trend of fatal crashes in Michigan has been steady over the past three years (2002-2004) since a considerable decrease from 2001 to 2002. The trend in Saginaw County is sporadic, but appears to be decreasing.

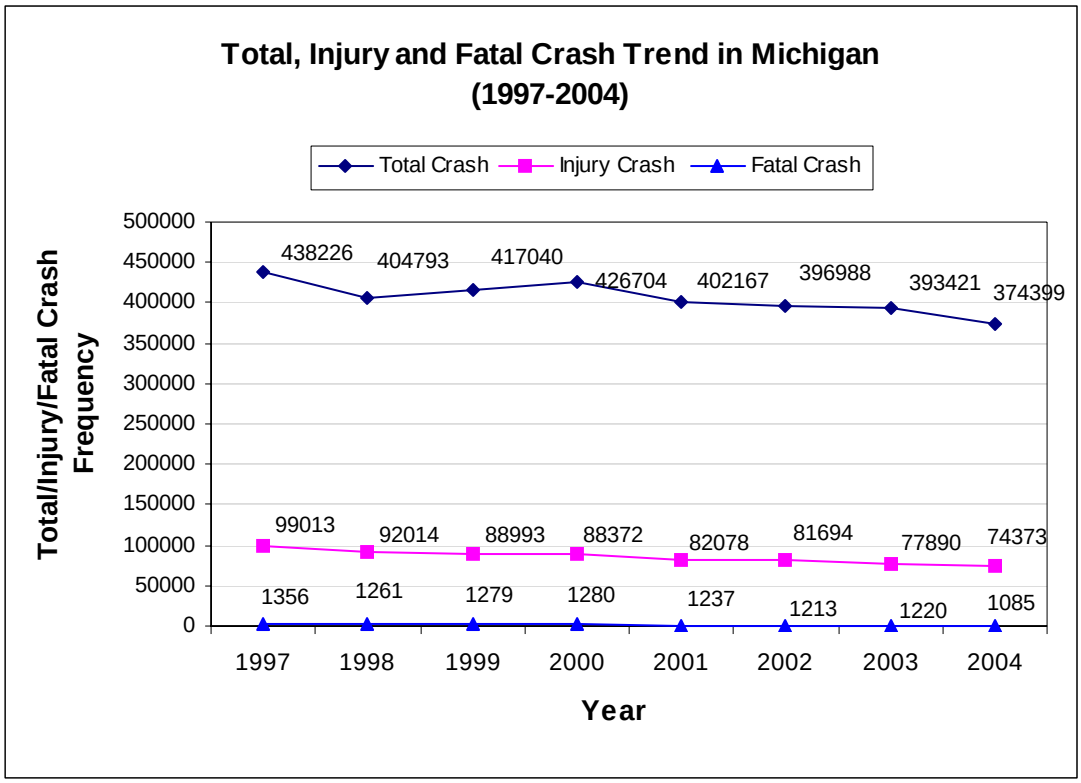


Figure 2.1 Yearly Crash Trend in Michigan (1997-2004)

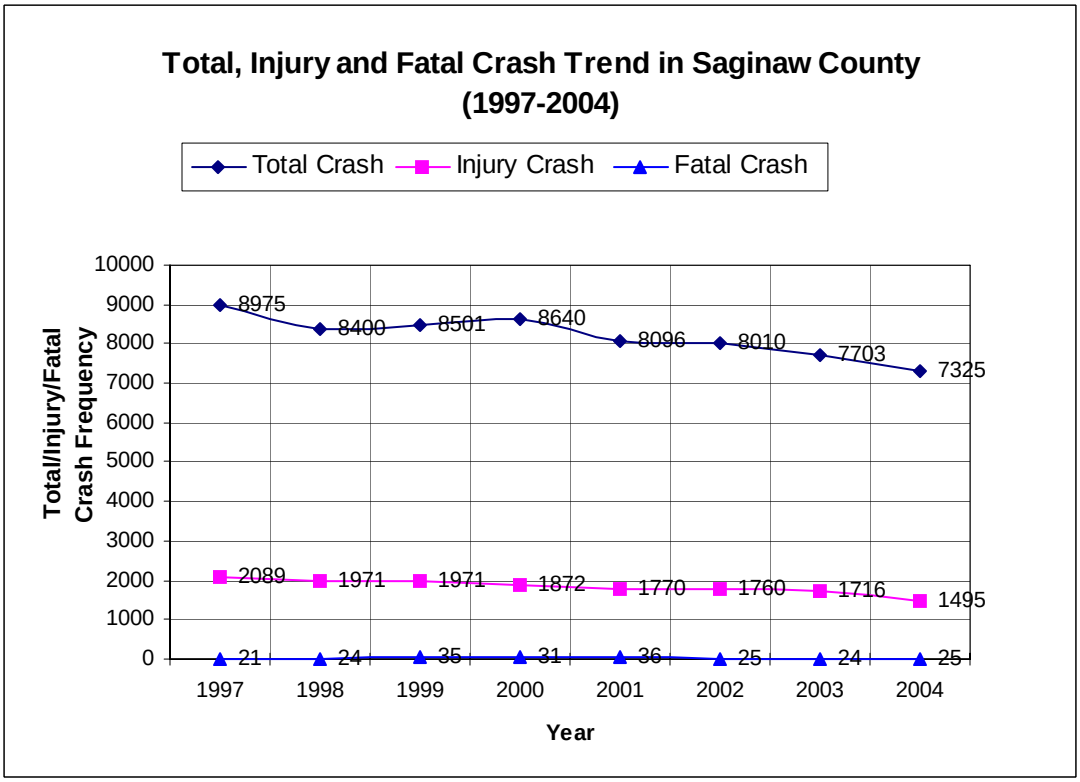


Figure 2.2 Yearly Crash Trend in Saginaw County (1997-2004)

2.2 Temporal Crash Patterns in Saginaw County

The temporal crash patterns in Saginaw County based on the month of the year, day of the week and time of the day are presented in Figures 2.3, 2.4 and 2.5 respectively, for the years 1997 to 2004. For the 'month of the year' analysis, it was found that for total crashes, a steady trend prevails during 10 months of the year, with the exception of the months of December and January, in which the number of crashes are higher. This may be attributable to the hazardous driving conditions due to snowfall in the winter season. Also, the hunting season covers the months of October and November, thereby increasing the volume of traffic on the road by a considerable amount leading to an increased frequency of crashes. The crash frequency is the lowest during the month of April due to the fact that all the accumulated snow is melted, leaving the pavement dry and driving conditions favorable for motorists.

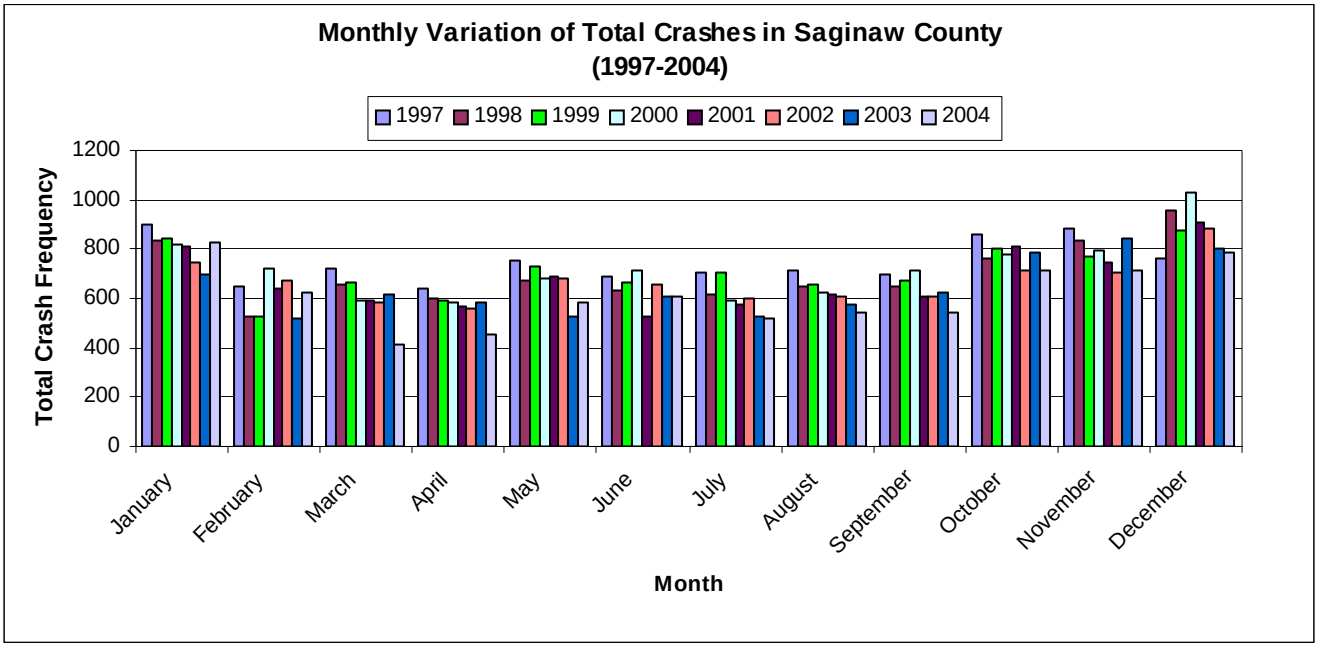


Figure 2.3. Monthly Variation of Total Crashes in Saginaw County (1997-2004)

In terms of the temporal crash trends by day of the week, the total crash frequencies are higher on Friday, as shown in Figure 2.4. The obvious reason for this increased number of crashes on a typical Friday is the greater rate of travel during the weekend. A considerable percentage of the people start travel for the weekend during the latter part of the day on Friday, creating various types of traffic congestion and crashes.

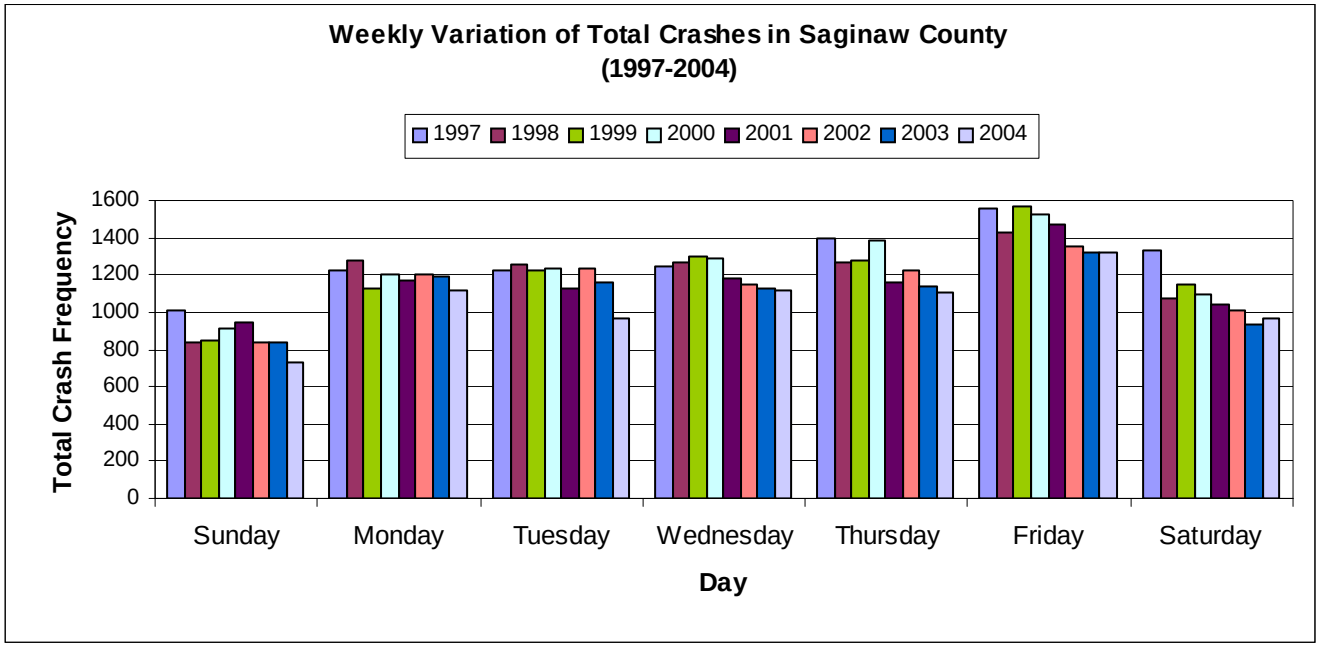


Figure 2.4. Weekly Variation of Total Crashes in Saginaw County (1997-2004)

Time of day crash trends were analyzed for four time intervals including 6:00-9:00 AM (3-hour period), 11:00 AM-1:00 PM (2-hour period), 4:00-6:00 PM (2-hour period), and 10:00 PM-5:00 AM (7-hour period), as shown in Figure 2.5. The average crashes per hour during the listed time intervals indicate that the highest number of crashes occur during the evening peak hour (4:00-6:00 PM). The evening peak hour is considered to experience a maximum amount of traffic during a typical weekday. Consequently, the average number of crashes is the highest at this time. The average frequency of crashes per hour, per year is found to be 614 for the evening peak hour.

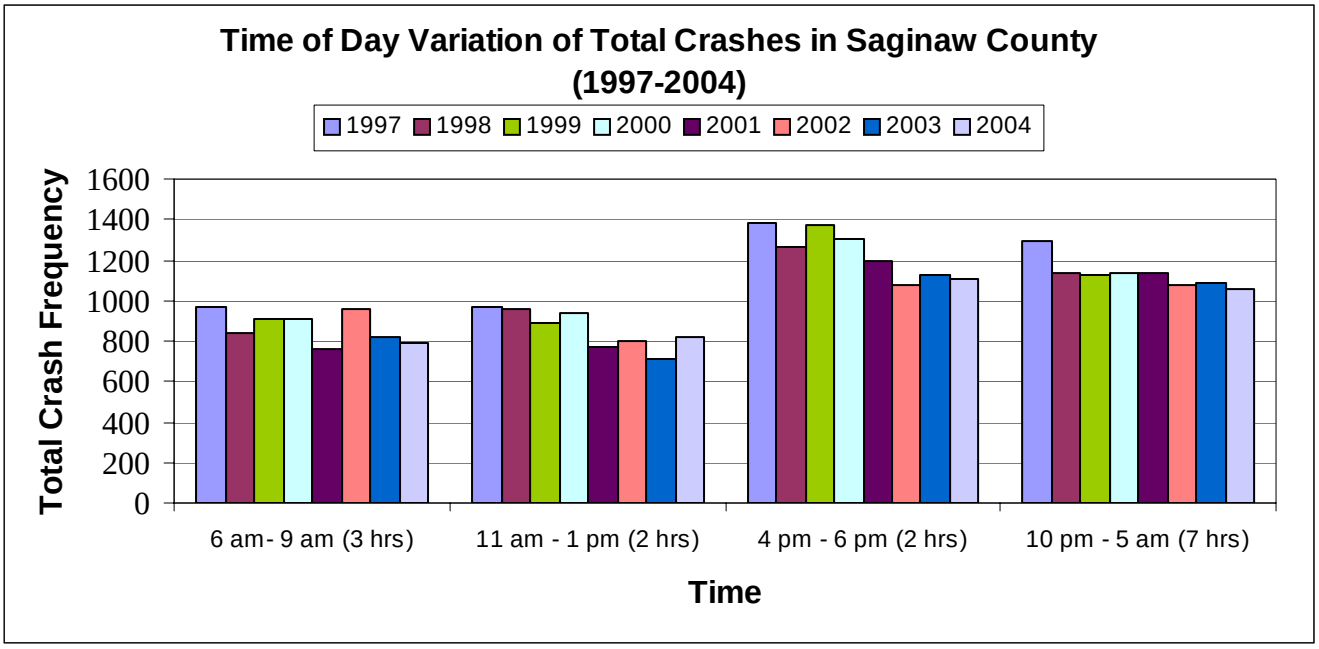


Figure 2.5. Time of Day Variation of Total Crashes in Saginaw County (1997-2004)
Note: (Average Indicates Average Crashes per Hour per Year)

2.3 Crash Patterns in Saginaw County by Weather Condition

Categorization of the crashes based on the weather conditions can reveal important facts and play an important role in traffic crash analysis. The majority of traffic crashes in Saginaw County occurred during clear or cloudy weather conditions (79.7 percent of all crashes), as shown in Figure 2.6.

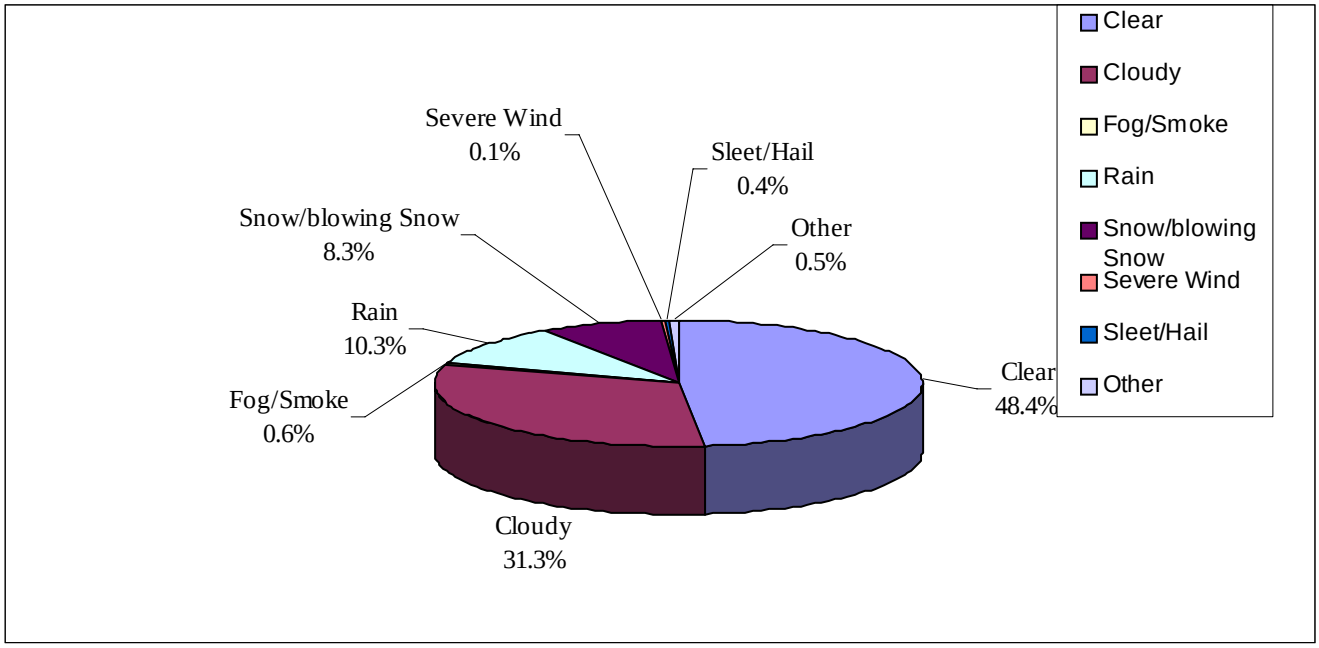


Figure 2.6 Weather Condition and Total Crashes in Saginaw County (1997-2004 Average)

Approximately 10.3 percent of the crashes occurred during rainy conditions and 8.3 percent of crashes during snowy conditions. The majority of crashes occured under clear/cloudy weather conditions, since this is the predominant weather condition throughout a typical year in Michigan.

2.4 Crash Patterns in Saginaw County for a Target Age Group of Drivers

The trend of total crashes from 1999 to 2004 for various target age groups of drivers is shown in Figure 2.7. Younger drivers (ages 16-20 years) typically have a higher crash risk due to their limited driving experience. Drivers between the ages of 21 and 34 years are considered as a high-risk driver group, since they are more susceptible to speeding, drinking and driving, and aggressive driving. Elderly drivers of age 70 years and older are another high risk driving group due to physical and visual challenges related to the aging process, which makes the driving task more complex. The trend in crashes involving these high-risk driver groups in Saginaw County has decreased in recent years, from 2002 to 2004. Especially in the year 2002, the crashes within these three vulnerable groups were considerably low.

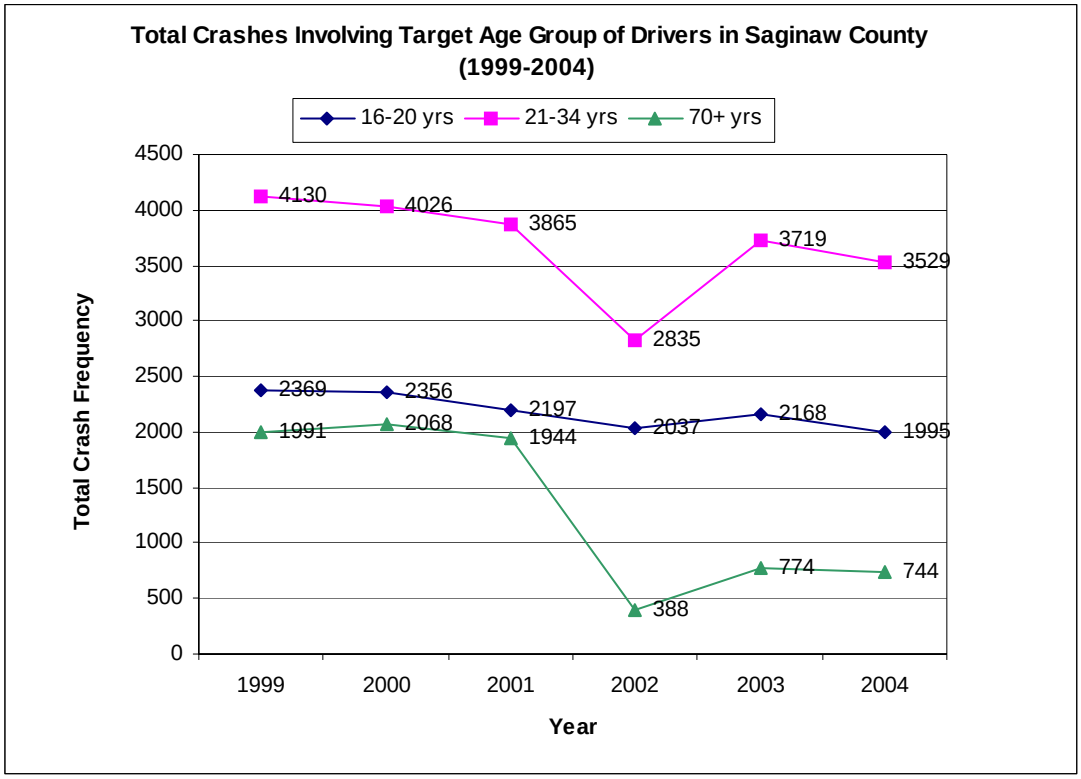


Figure 2.7 Target Age Group of Drivers in Saginaw County (1999-2004)

2.5 Trend of Driver Behavior-Related Crashes in Saginaw County

Driver behavior such as driving under the influence of alcohol, speeding, red light running and not using a seatbelt, are considered to be dangerous driving actions and typically result in a high proportion of crashes and injuries. A comprehensive study of such hazardous driver actions was performed as a part of this study to highlight the trend of crashes resulting from such behaviors, as well as to identify the most vulnerable locations to be targeted for treatment.

Driving while intoxicated is dangerous, and drivers with high blood alcohol content (BAC) levels are at an increased risk for crashes, injuries and death. Driving under the influence of alcohol is a serious national problem that tragically affects thousands of victims annually. The annual trend of crashes involving drivers under the influence of alcohol in Saginaw County is shown in Figure 2.8.

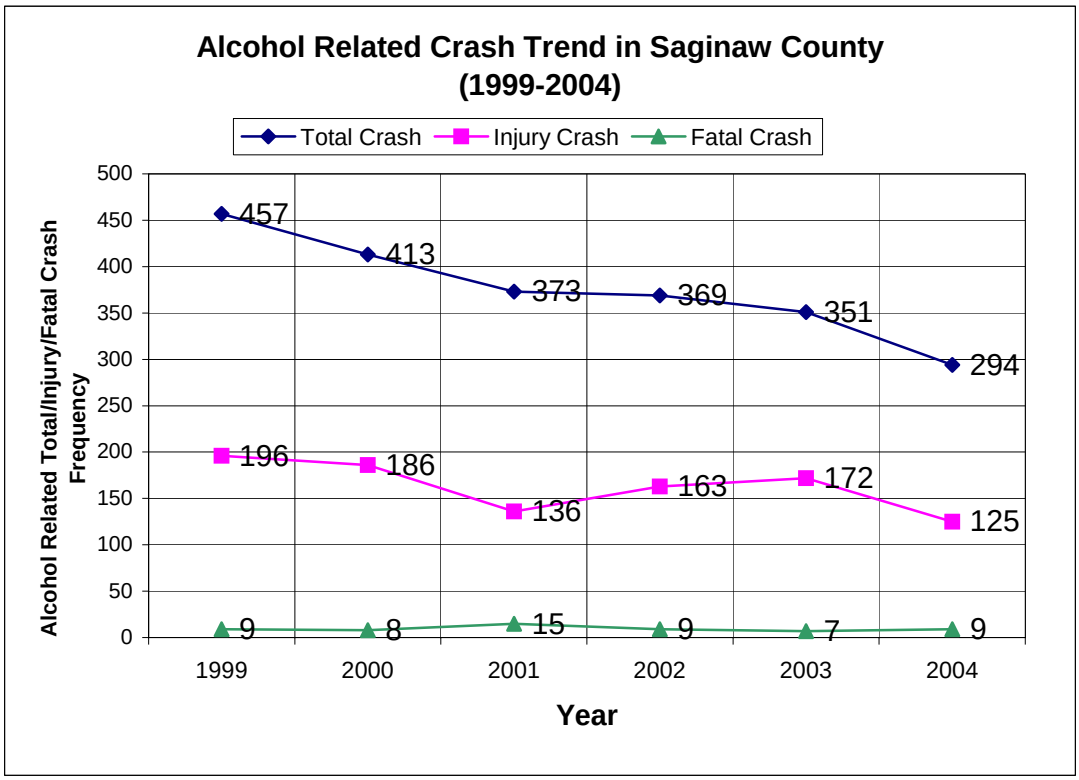


Figure 2.8. Annual Trend of Total, Injury and Fatal Crashes Involving Drivers Under the Influence of Alcohol (1999-2004)

The number of alcohol-related total crashes and injury crashes exhibit an overall decreasing trend in Saginaw County over the years 1999 to 2004. In 1999, the number of alcohol-related crashes was 450, which reduced to 300 in 2004, resulting a 33 percent reduction. Injury crashes reduced from 200 in 1999 to 125 in 2004, which indicates a 37 percent reduction.

The crash frequency of speeding-related crashes in Saginaw County was the lowest in the year 1999, with 793 total crashes and 214 injury crashes. In the year 2002, the number of speeding related crashes was high (963 crashes). The annual trend of speeding related crashes in Saginaw County is shown in Figure 2.9.

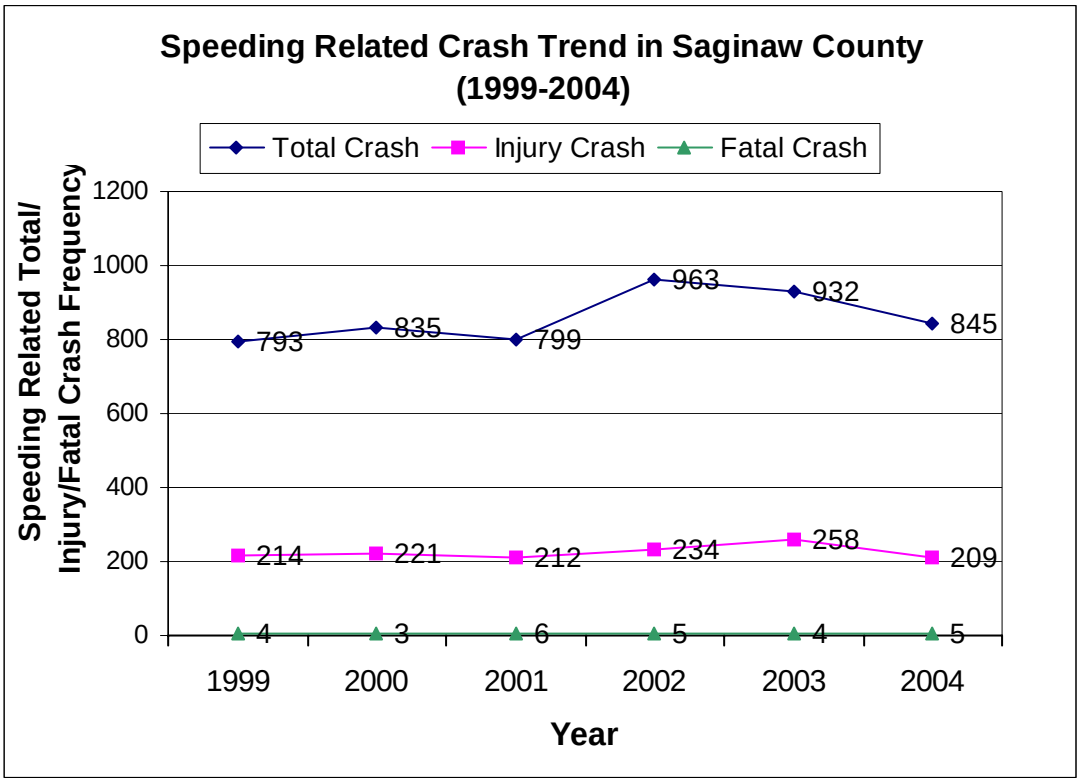


Figure 2.9. Annual Trend of Speeding Related Total, Injury and Fatal Crashes in Saginaw County (1999-2004)

A portion of the crashes that occur at signalized intersections include drivers who disobey traffic signals and run red lights. This type of crash is associated with high injuries and fatalities and has become a national concern over the past several years. The trend of total. crashes involving red light running (RLR), as shown in Figure 2.10, is essentially constant from 1999 to 2003 followed by a decrease in 2004.

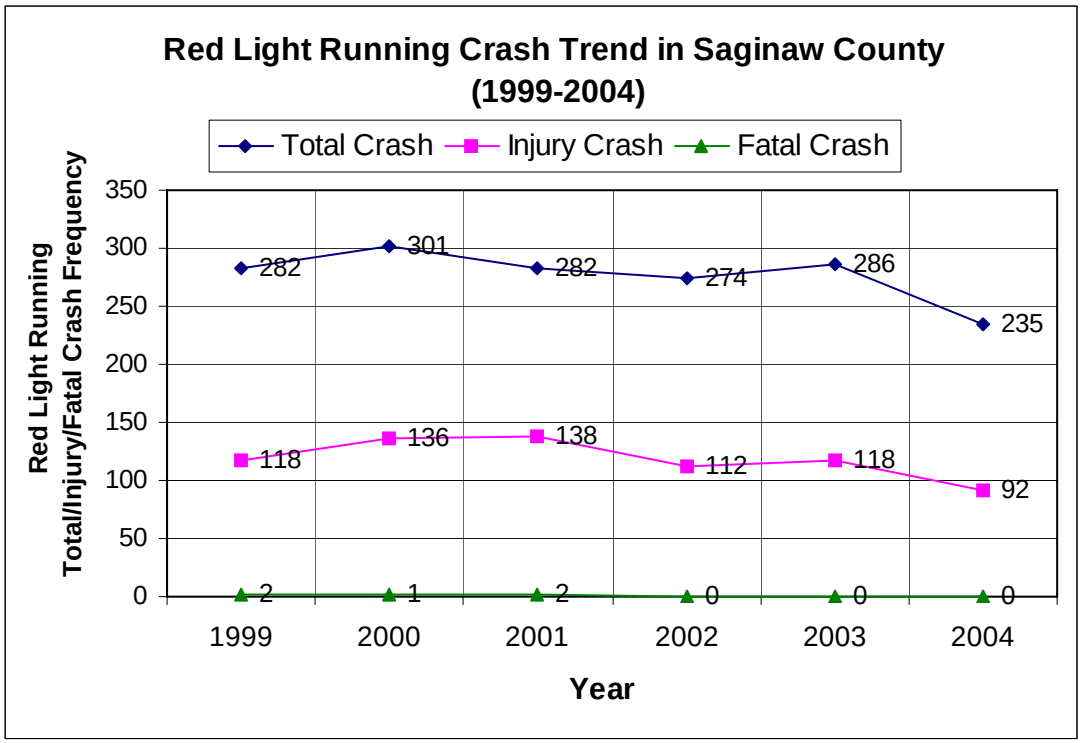


Figure 2.10. Annual Trend of Red Light Running Total, Injury and Fatal Crashes in Saginaw County (1999-2004)

The use of safety belts helps prevent fatalities and reduces the incidence of serious injuries in all types of crashes. The overall number of crashes involving drivers who were unbelted, in Saginaw County, has drastically decreased from 1999 to 2004, about 57 percent, as shown in Figure 2.11. Injury crashes have also shown an overall decreasing trend. This may be attributable to statewide seatbelt use awareness campaigns and enforcement programs.

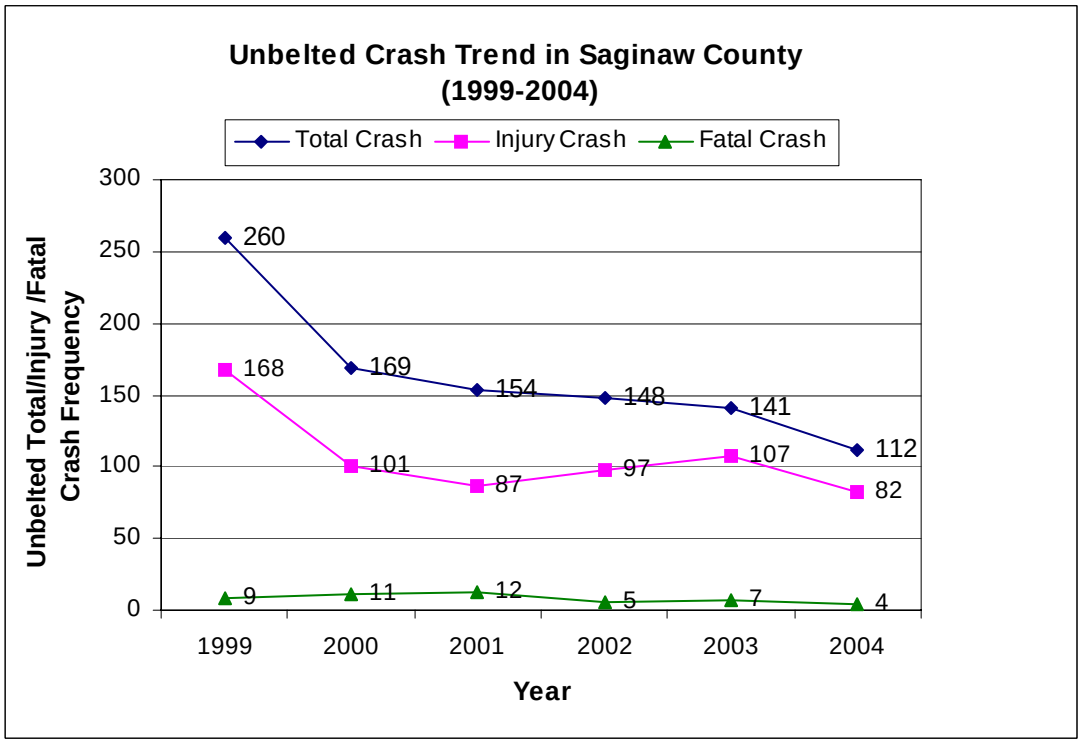


Figure 2.11. Annual Trend of Total, Injury and Fatal Crashes Involving Unbelted Drivers (1999-2004)

2.6 Spatial Analysis of Intersection Crashes in Saginaw County

An analysis was performed at the city/township level to identify the areas in Saginaw County that experience a high number of crashes and injuries at signalized intersections. When this analysis was done, the latest crash database available to WSU–TRG was the 2004 computerized crash database from the Michigan State Police. In 2004, crashes occurring at all intersections in Saginaw County represented a significant portion (18.7 percent) of the total crashes in the county (1,369 crashes at signalized intersections/7,325 total crashes = 18.7 percent). The crashes at signalized intersections in Saginaw County show that a total of 1369 crashes occurred within the boundaries of 23 cities and townships in the year 2004. The spatial distribution of crash frequencies at signalized intersections at a city/township level is shown in Figure 2.12.

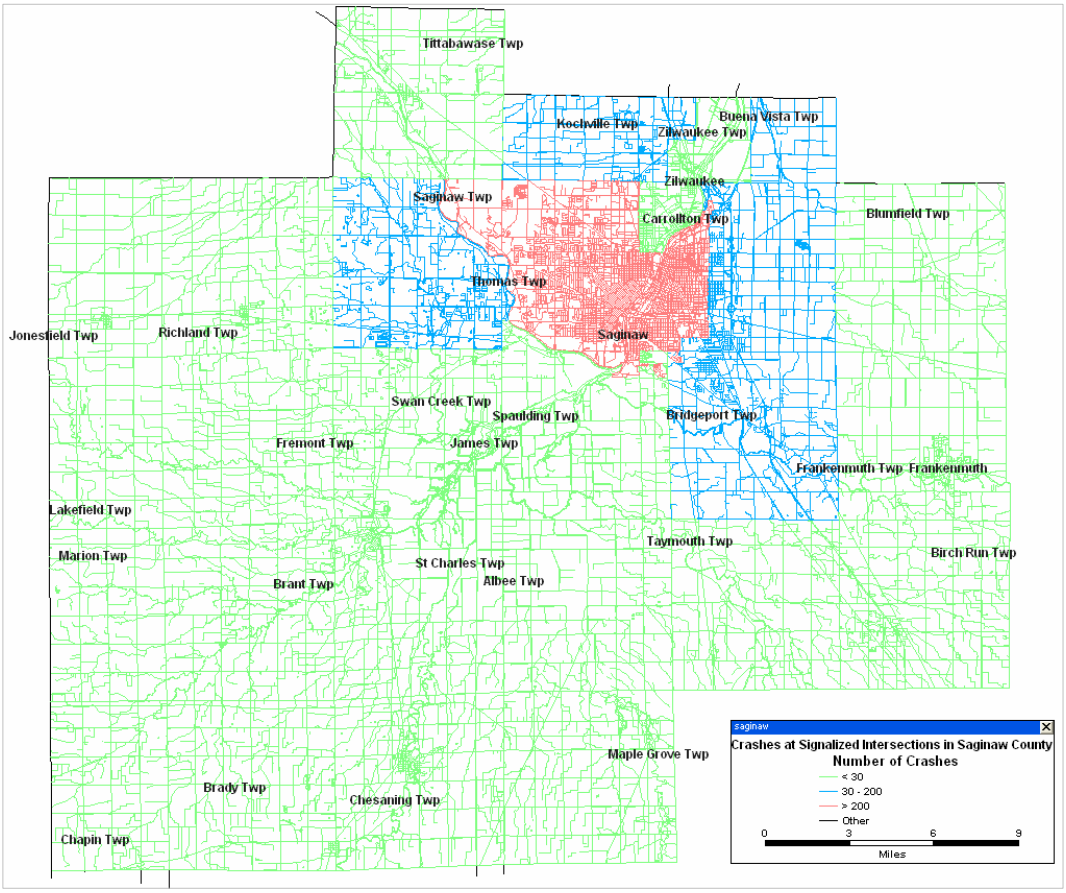


Figure 2.12 Spatial Distributions of Total Crashes at Signalized Intersections in Saginaw County

The total number of crashes at signalized intersections in the City of Saginaw is the highest among signalized intersections in all the cities and townships in Saginaw County, followed by Saginaw Township. A total of 490 crashes occurred at signalized intersections in the City of Saginaw (35.8 % of total) and 486 crashes (35.5 %) in Saginaw Township. In Kochville and Thomas Townships the number of crashes at signalized intersections was 84 and 53 respectively.

Signalized intersections, mainly in these three cities/townships of Saginaw County, were the focus of this study since crashes occurring at these intersections represent a significant portion of the total crashes in the area, and also because the type of crashes that occur at these locations have a higher probability of being 'corrected' by engineering and driver behavioral countermeasures, as compared to crashes occurring at other locations.

3.0 SELECTION OF INTERSECTIONS INCLUDED IN A SITE-SPECIFIC ANALYSIS

Queries from the statewide crash database were performed for a maximum of 100 high and moderate crash intersections to determine the total number of crashes and injuries for three consecutive years (2002, 2003 and 2004). However, for the detailed analysis of the selected study intersections, the most recent crash data forms for the years 2003 and 2004 were used. The total and injury crash frequencies for the detailed intersections are shown in Table 3.1 (a).

Table 3.1 (a) Total and Injury Crash Frequencies at Signalized Intersections in Saginaw County Queried from the Michigan State Police Computerized Crash Database

Rank of Intersection	*Primary street*	*Intersecting street*	*City/Township*	*Average Annual Crash Frequency For years 2002,2003 and 2004*		
				TOTAL	*INJURY*	*FATAL*
1	State	Center	Saginaw Twp	63	13	0
2	Bay	Tittabawassee	Saginaw Twp	36	6	0
3	Gratiot	Center	Saginaw Twp	31	6	0
4	State	Wieneke	Saginaw Twp	30	8	0
5	Bay	Shattuck	Saginaw Twp	30	5	0
6	Hemmeter	State	Saginaw Twp	29	9	0
7	Mackinaw	Tittabawassee	Saginaw Twp	29	7	0
8	Bay	Mccarty	Saginaw Twp	28	5	0
9	Rust	Washington	Saginaw	24	10	0
10	Tittabawassee	Town centre	Saginaw Twp	24	4	0
11	Bay	Schust	Saginaw Twp	23	7	0
12	Holland	Outer dr	Buena Vista Twp	23	3	0
13	Bay	Weiss	Saginaw Twp	21	3	0
14	Bay	Davenport	Saginaw	20	4	0
15	Gratiot	Miller	Thomas Twp	20	5	0
16	Gratiot	River	Thomas Twp	19	7	0
17	Michigan	Genesee	Saginaw	19	6	0
18	Michigan	State	Saginaw	18	6	0
19	Michigan	Davenport	Saginaw	17	4	0
20	Midland	Titta bawassee	Tittabawase Twp	17	4	0
21	Bay	State	Saginaw	16	4	0
22	Center	Shattuck	Saginaw Twp	16	3	0
23	Davenport	Hill	Saginaw	16	5	0
24	Birchrun	Dixie	Birch Run Twp	16	4	0
25	Tittabawassee	Fashion square	Saginaw Twp	16	3	0
26	Washington	I75	Buena Vista Twp	15	2	0
27	Hill	State	Saginaw	15	5	0
28	Midland	State	Saginaw Twp	15	2	0
29	Davenport	Mackinaw	Saginaw	14	4	0
30	Center	Tittabawassee	Saginaw Twp	14	2	0
31	Mackinaw	Mccarty	Saginaw Twp	13	5	0
32	Genesee	Holland	Saginaw	13	3	0
33	Gratiot	Midland	Saginaw Twp	13	2	0
34	Birchrun	I 75	Birch Run	13	2	0
35	Mackinaw	State	Saginaw	12	2	0
36	Tittabawassee	Berl	Saginaw Twp	12	4	0
37	Court	Michigan	Saginaw	12	3	0
38	Gratiot	Graham	Thomas Twp	12	5	0
39	Sheridan	Holland	Saginaw	12	3	0

Table 3.1 (a) Total and Injury Crash Frequencies at Signalized Intersections in Saginaw County Queried from the Michigan State Police Computerized Crash Database (Continued…)

Rank of Intersection	*Primary street*	*Intersecting street*	*City/Township*	*Average Annual Crash Frequency For years 2002,2003 and 2004*		
				TOTAL	*INJURY*	*FATAL*
40	Tittabawassee	Davis	Kochville Twp	12	1	0
41	Center	Brockway	Saginaw Twp	11	3	0
42	Center	Weiss	Saginaw Twp	11	3	0
43	Dixie	Hess	Buena Vista Twp	11	4	0
44	I75	Holland	Buena Vista Twp	11	2	0
45	Main	Washington	Tittabawase Twp	11	3	0
46	Washington	Genesee	Saginaw	11	2	0
47	Beyer	Main	Birch Run	11	2	0
48	Davenport	Warwick	Saginaw	11	1	0
49	Mackinaw	Shattuck	Saginaw Twp	10	3	0
50	Midland	M47	Tittabawase Twp	10	2	0
51	Gratiot	Wheeler	Saginaw	10	2	0
52	Bay	Enterprise	Saginaw Twp	10	2	0
53	River	State	Thomas Twp	10	3	0
54	Michigan	Tittabawassee	Carrollton Twp	10	2	0
55	Veterans memorial	Washington	Saginaw	10	2	0
56	Bay	Pierce	Kochville Twp	10	1	0
57	State	Warwick	Saginaw	9	4	0
58	Tittabawassee	Fortune	Saginaw Twp	9	2	0
59	Holland	Cumberland	Saginaw	9	1	0
60	Dixie	Junction	Bridgeport Twp	9	3	0
61	Remington	Genesee	Saginaw	9	3	0
62	Shattuck	Hermansau	Saginaw Twp	9	3	0
63	Brady	Oakley	Chesaning Twp	9	2	0
64	I675	I75	Buena Vista Twp	9	2	0
65	Mason	State	Saginaw	9	2	0
66	Freeland	Lawndale	Kochville Twp	9	2	0
67	I 75	Dixie	Bridgeport Twp	9	2	0
68	Tittabawassee	I675	Carrollton Twp	9	1	0
69	Remington	Warren	Saginaw	8	3	0
70	Holland	20th	Saginaw	8	3	0
71	Rust	Sheridan	Saginaw	8	3	0
72	Bay	Freeland	Kochville Twp	8	2	0
73	Holland	17th	Saginaw	8	3	0
74	Bay	Congress	Saginaw	8	3	0
75	Michigan	Stephens	Saginaw	8	3	0
76	Michigan	Shattuck	Carrollton Twp	8	3	0
77	Court	Congress	Saginaw	8	3	0
78	State	Lawndale	Saginaw Twp	8	2	0

Table 3.1 (a) Total and Injury Crash Frequencies at Signalized Intersections in Saginaw County Queried from the Michigan State Police Computerized Crash Database (Continued…)

Rank of Intersection	*Primary street*	*Intersecting street*	*City/Township*	*Average Annual Crash Frequency For years 2002,2003 and 2004*		
				TOTAL	*INJURY*	*FATAL*
79	**Bay**	**Genesee**	**Saginaw**	**8**	**2**	**0**
80	**Center**	**Seidel**	**Saginaw Twp**	**8**	**1**	**0**
81	**Washington**	**Gallagher**	**Saginaw**	**8**	**0**	**0**
82	**Court**	**Mason**	**Saginaw**	**8**	**4**	**0**
83	**Genesee**	**Hess**	**Saginaw**	**8**	**3**	**0**
84	**Brockway**	**Malzahn**	**Saginaw**	**8**	**2**	**0**
85	**Remington**	**Sheridan**	**Saginaw**	**8**	**1**	**0**
86	**Michigan**	**Union**	**Saginaw**	**7**	**2**	**0**
87	**Michigan**	**Vermont**	**Saginaw**	**7**	**2**	**0**
88	**Mason**	**Cooper**	**Saginaw**	**7**	**3**	**0**
89	**Barnard**	**Mccarty**	**Saginaw Twp**	**7**	**3**	**0**
90	**Sheridan**	**Webber**	**Saginaw**	**7**	**2**	**0**
91	**Main**	**Genesee**	**Frankenmuth**	**7**	**1**	**0**
92	**Michigan**	**Cooper**	**Saginaw**	**7**	**1**	**0**
93	**Sheridan**	**Washington**	**Bridgeport Twp**	**7**	**2**	**0**
94	**Court**	**Hamilton**	**Saginaw**	**7**	**2**	**0**
95	**Elm**	**Gratiot**	**Saginaw**	**7**	**2**	**0**
96	**Michigan**	**Williams**	**Saginaw**	**7**	**2**	**0**
97	**Rust**	**Fordney**	**Saginaw**	**7**	**2**	**0**
98	**I75**	**Burt**	**Birch Run Twp**	**7**	**1**	**0**
99	**Genesee**	**Mason**	**Saginaw**	**6**	**2**	**0**
100	**Genesee**	**Webber**	**Saginaw**	**6**	**2**	**0**

Table 3.1 (b) represents the candidate intersections identified for inclusion in this study. These intersections were selected based on the recommendations from the Saginaw MPO. The majority of these intersections are located within the boundaries of the City of Saginaw and Saginaw Township.

Table 3.1 (b) Total, Injury and Fatal Crash Frequency at Study Intersections in Saginaw County Queried from the Michigan State Police Computerized Crash Database

Serial No	*Intersection*	*City/Township*	*Average Annual Crash Frequency for Two Years (2003 and 2004)*		
			TOTAL	*INJURY*	*FATAL*
1	Bay Road and Kochville Road	Kochville Twp.	4	2	0
2	Bay Road and Bueker Drive	Kochville Twp.	5	2	0
3	Bay Road and Tittabawassee Road	Saginaw Twp	35	5	0
4	Bay Road and Schust Road	Saginaw Twp	26	7	0
5	Bay Road and McCarty Road	Saginaw Twp	26	4	0
6	Bay Road and Universal Drive	Saginaw Twp	2	1	0
7	Bay Road and Shattuck Road	Saginaw Twp	30	4	0
8	Bay Road and Weiss Road	Saginaw Twp	21	4	0
9	Bay Street and Davenport Avenue	Saginaw	22	5	0
10	Bay Street and State Street	Saginaw	18	4	0
11	Bay Street and West Genesee Street	Saginaw	7	2	0
12	Bay Street and Congress Avenue	Saginaw	6	1	0
13	Bay Street and Cooper Avenue	Saginaw	2	0	0
14	Bay Street and Court Street	Saginaw	3	0	0
15	Tittabawassee Road and North Center Road	Saginaw Twp	15	2	0
16	Tittabawassee Road and Mackinaw Road	Saginaw Twp	31	5	0
17	Tittabawassee Road and Fortune Boulevard	Saginaw Twp	11	3	0
18	Tittabawassee Road and Fashion square Boulevard	Saginaw Twp	15	2	0
19	Tittabawassee Road and Berl Drive	Saginaw Twp	14	4	0
20	Tittabawassee Road and Towne Center Road	Saginaw Twp	23	3	0
21	Tittabawassee Road and SB I675	Zilwauke Twp	4	1	0
22	Tittabawassee Road and NB I675	Zilwauke	2	1	0
23	Tittabawassee Road and North Michigan Avenue	Carrollton Twp	9	3	0

Table 3.1 (b) Total, Injury and Fatal Crash Frequency at Study Intersections in Saginaw County Queried from the Michigan State Police Computerized Crash Database (Continued…)

Serial No	*Intersection*	*City/Township*	*Average Annual Crash Frequency for Two Years (2003 and 2004)*		
			TOTAL	*INJURY*	*FATAL*
24	Gratiot Road and Graham Road	Thomas Twp	10	5	0
25	Gratiot Road and Kennely Road	Thomas Twp	8	3	0
26	Gratiot Road and Van Warmer Road	Thomas Twp	3	1	1
27	Gratiot Road and Miller Road	Thomas Twp	20	5	0
28	Gratiot Road and River Road	Thomas Twp	23	6	0
29	Gratiot Road and Center Road	Saginaw Twp	35	7	0
30	Gratiot Road and Wheeler Road	Saginaw	13	2	0
31	North Michigan Avenue and Davenport Avenue	Saginaw	19	3	0
32	North Michigan Avenue and State Street	Saginaw	21	5	0
33	North Michigan Avenue and West Genesee Avenue	Saginaw	18	4	0
34	Michigan Avenue and Court Street	Saginaw	13	3	0
35	Michigan Avenue, Mackinaw Street and Gratiot Avenue	Saginaw	11	2	0
36	East Holland Avenue and South Outer Drive	Buena Vista	23	2	0
37	East Holland Avenue and Cumberland Street	Saginaw	9	1	0
38	Rust Avenue and Washington Avenue	Saginaw	24	8	0
39	West Freeland Road and Midland Road	Tittabawassee Twp.	2	1	0
40	South Graham Road and Swan Creek Road	Thomas Twp.	2	7	0

4.0 MAJOR CORRIDOR CHARACTERISTICS

As a part of this study, the characteristics of four major corridors (Bay Road, Tittabawassee Road, Gratiot Road and Michigan Avenue) were investigated. The following provides a description of the physical characteristics, traffic volumes, traffic control, signal progression analysis and an analysis of traffic crashes related to driver behavioral issues such as alcohol, seatbelt use, speeding and red light running. The progression analysis and signal timing optimization have been performed for the PM peak (4:00-6:00 PM), which represents the worst-case scenario for the traffic volume. However, similar optimization technique may be applied for the morning peak and off peak periods. Schematic diagrams depicting the general appearance of the major corridors are shown in Figure 4.1 through Figure 4.3.

Among the four major corridors, Bay Corridor runs through the City of Saginaw, Saginaw Township and Kochville Township, while Tittabawassee Corridor is mainly located at the northern boundary of Saginaw Township. As shown in Figure 4.1 the Bay Corridor runs in the north-south direction and intersects with Tittabawassee corridor, which runs in the east-west direction.

The Michigan Avenue Corridor, with all study intersections located in the City of Saginaw's incorporated area, runs in the northeast and southwest direction and intersects with the Gratiot Corridor.

The Gratiot Corridor extends in the east-west direction from Thomas Township on its west to the City of Saginaw's incorporated area on its east. As we have a mutually intersecting network of arterials in this area, it is necessary to prioritize the arterials to achieve desired progression. Considering the volume of traffic and the functional classification, Bay Corridor is ranked as the most important corridor, followed by the Tittabawassee and Gratiot Corridor. As a result, after optimizing the Bay Corridor, Tittabawassee Road was taken into consideration to attain the progression for the entire network. Similarly, the Gratiot Road Corridor and Michigan Avenue Corridor were optimized as an integrated network.

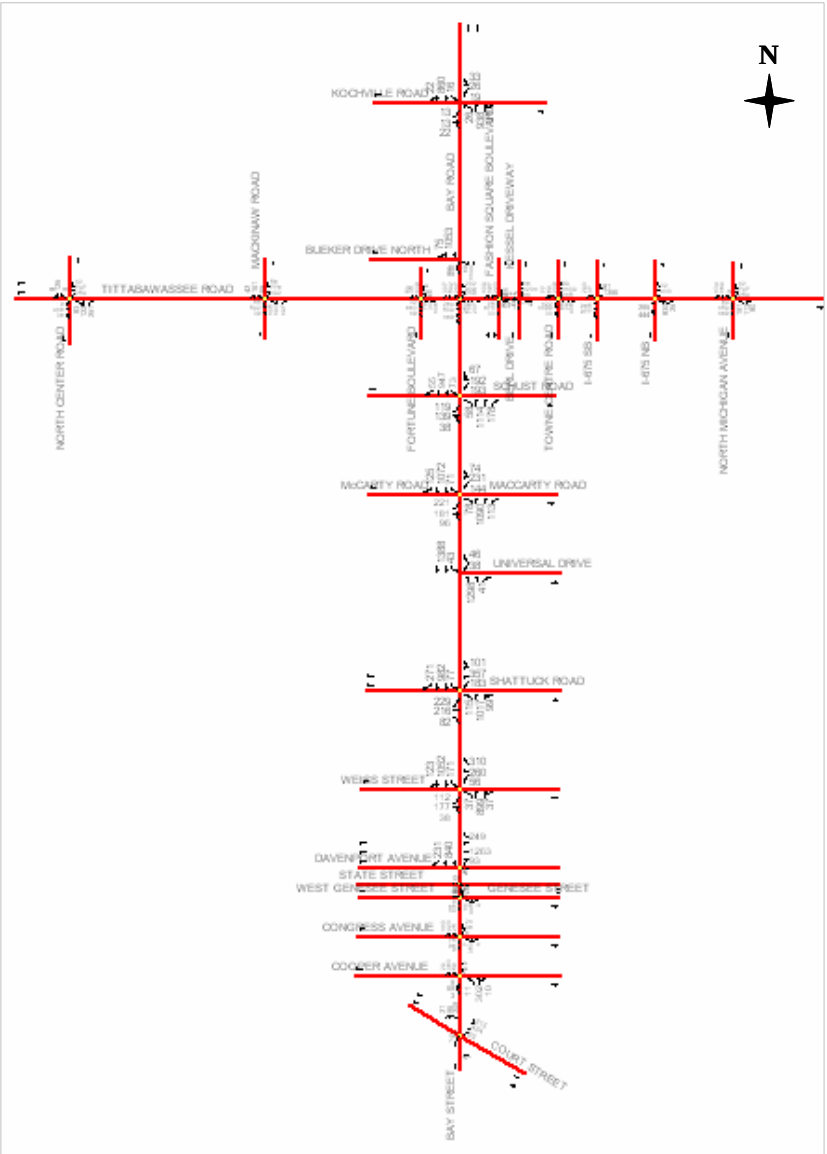


Figure 4.1. Line Diagram of Bay and Tittabawassee Road Network

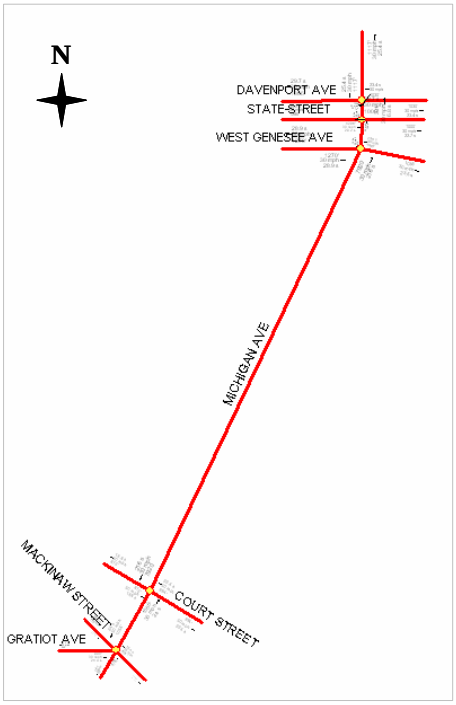


Figure 4.2 Line Diagram of North Michigan Avenue Corridor

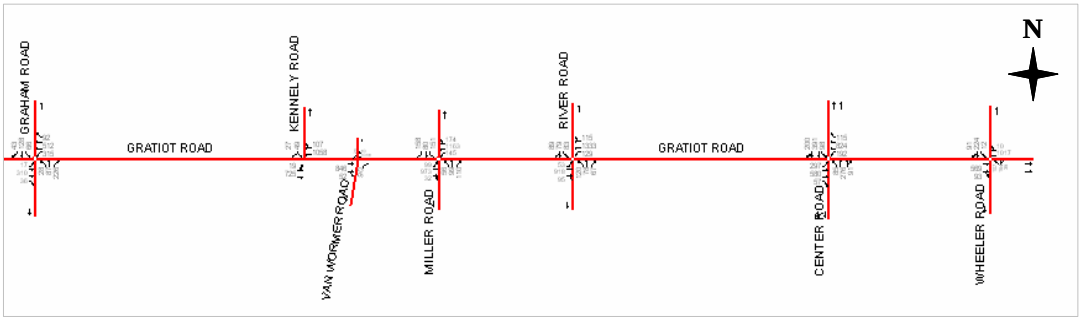


Figure 4.3 Line Diagram of Gratiot Road Corridor

4.1 Bay Road Corridor – Physical Characteristics

Bay Road is a major arterial in Saginaw County, which runs North-South (N-S) through the townships of Kochville and Saginaw and the City of Saginaw's incorporated area. To the south, it merges at the intersection with Court Street. Fourteen intersections from north to south over a 4.75-mile long segment of Bay Road from Kochville Road to Court Street constitutes the study intersections, as shown in Figure 4.4, and are as follows:

- Bay Road and Kochville Road
- Bay Road and Bueker Drive
- Bay Road and Tittabawassee Road
- Bay Road and Schust Road
- Bay Road and McCarty Road
- Bay Road and Universal Drive
- Bay Road and Shattuck Road
- Bay Road and Weiss Road
- Bay Street and Davenport Avenue
- Bay Street and State Street
- Bay Street and West Genesee Street
- Bay Street and Congress Avenue
- Bay Street and Cooper Avenue
- Bay Street and Court Street

Bay Road is a State Trunk line (M-84) from north up to its intersection with Weiss Road. After that, Bay Road turns into a county major arterial, changes its name to Bay Street and continues to the south where it intersects with Court Street. A schematic diagram of the entire Bay Road Corridor is provided in Figure 4.4, which depicts the prevailing conditions in the field. The directional arrows indicate the PM Peak hour volume and also show the number of lanes for each approach and their usage. Existing signal timing plans are shown with each intersection of the corridor including the cycle length and the phasing diagram.

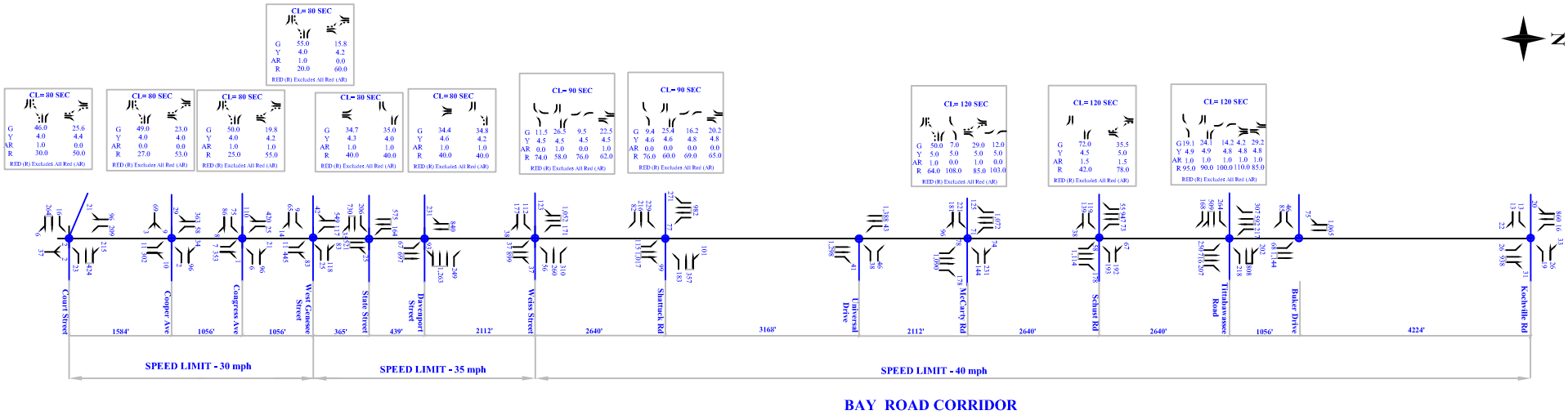


Figure 4.4 Schematic Diagram of Bay Road Corridor

The horizontal alignment of the Bay Road Corridor is generally straight and the surrounding land uses are commercial in nature. From Kochville Road to Weiss Road, Bay Road consists of a five-lane cross-section with two lanes for northbound traffic, two lanes for southbound traffic and a center lane for left turns.

However, there is an extra right-turn lane, for both approaches, at all the intersections with Tittabawassee Road to Weiss Road. From Davenport Road to Court Street the number of lanes reduces to four; two lanes for each direction of traffic.

At the intersection of Bay Street and Davenport Avenue, there is an extra left turning lane for both directions of traffic. Left turn movements for southbound traffic are prohibited, since Davenport Avenue is a one-way street. Similarly, at the intersection of Bay and State, there is an extra left turn lane for southbound traffic and left turns for northbound traffic are prohibited. Bay Street has no extra lane at its intersections with West Genesee Street, Congress Avenue, Cooper Avenue and Court Street. The posted speed limit on Bay Road from Kochville Road to Tittabawassee Road is 50 mph. From the intersection with Tittabawassee Road to North of Weiss Road the posted speed limit is 40 mph. From the intersection with Weiss Road up to north of the intersection with West Genesee Avenue the speed limit is 35 mph. From the intersection with West Genesee Street up to the south end of the corridor the speed limit is 30 mph. The study intersections on the Bay Corridor are shown on the map in Figure 4.5.

4.2 Bay Road Corridor – Traffic Volumes

The Average Daily Traffic (ADT) volume data along the Bay Road Corridor was obtained from the Traffic Count Maps published by MDOT for the year 2004, and is presented in Figure 4.6. The traffic volumes on Bay Road range from 29,100 vehicles per day (vpd) near McCarty and Shattuck Road, to 20,000 vpd near Kochville Road.

4.3 Bay Road Corridor – Intersection Traffic Control

The Bay Road study corridor includes fourteen intersections, three of which are unsignalized. The intersections with Kochville Road, Universal Drive and Bueker Drive are unsignalized. The left turn movements at the intersection with Tittabawassee are actuated

with a total cycle length of 120 seconds. The intersection with Schust Road is pre-timed with a total cycle length of 120 seconds during the PM Peak hour. Both of the intersections with McCarty Road and Shattuck Road are actuated for left-turn movement with cycle lengths of 120 seconds and 90 seconds during the PM peak period, respectively. Both of the intersections of Bay and Weiss Road and Bay and State are fully actuated with a cycle length of 80 seconds, whereas the intersections of Bay and Davenport are pre-time controlled with a cycle length of 80 seconds during PM peak period. All of the remaining four intersections are pre-time controlled with a cycle length of 80 seconds. In conclusion, the observed cycle lengths, during peak hours at the study intersections, vary along the corridor. However along the corridor, from the intersection of Bay Street and Davenport Avenue to the Bay Street and Court Street intersection, all 6 intersections have the same cycle length of 80 seconds. Most of the intersections lack the adequate clearance interval and hence require signal-timing modifications. Also, it is possible to achieve progression through the entire Bay Corridor if the cycle lengths for each intersection are maintained.

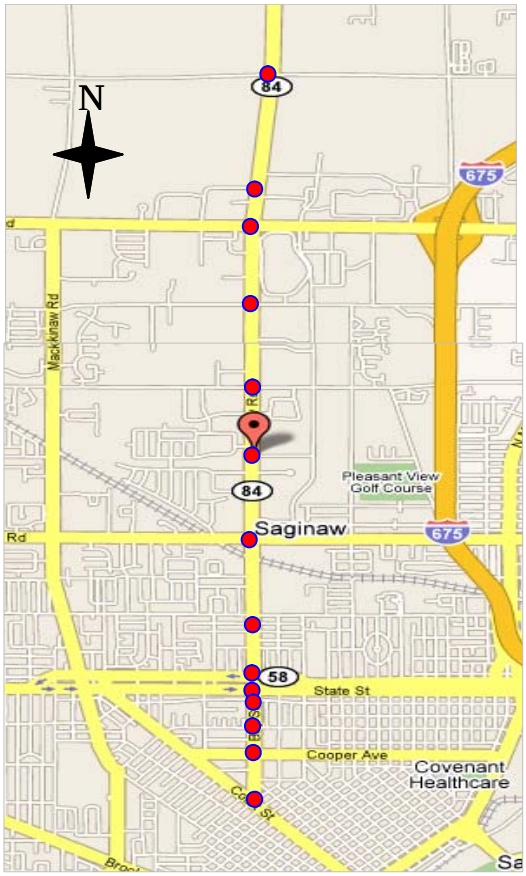


Figure 4.5 Bay Corridor with the Study Intersections

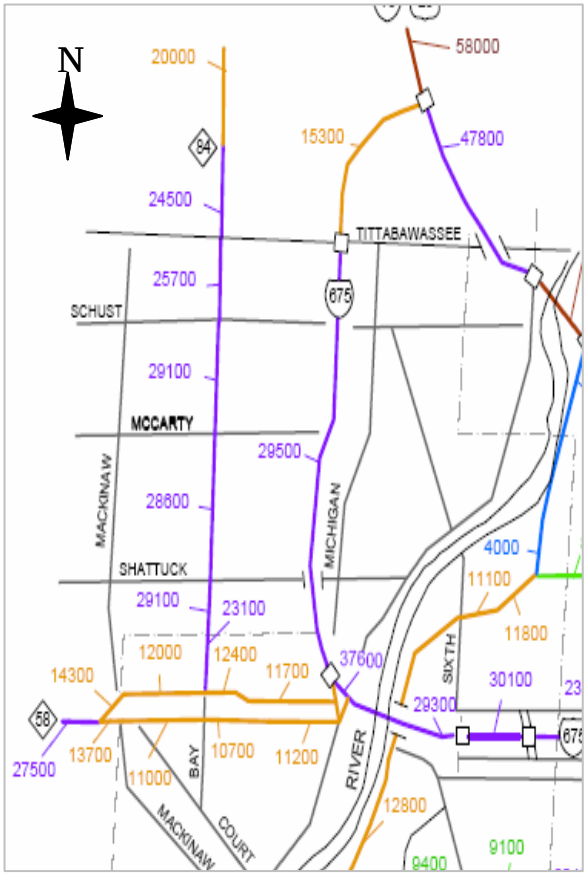


Figure 4.6. ADT Volumes along Bay Corridor
(Source: MDOT ADT Map, 2004)

4.4 Signal Progression for the Bay Road (M-84) Corridor

A progression analysis was performed for the Bay Corridor (M-84) from Kochville Road to Court Street, including twelve intersections using Synchro and SimTraffic programs. The cycle length of the network was optimized to be 90 seconds. Bay Road changes its functionality from Weiss Road to the south direction, where it is known as Bay Street. It also has speed limits varying from 50 mph to 30 mph on the stretch of road considered for progression. The results of this analysis are shown on the time space diagram in Figure 4.7 and are as follows:

- NB Bay Road:
  - -Bandwidth = 24 sec
  - -Progression Speed: 30 mph (from Court Street to West Genesee Street)
    35 mph (from West Genesee Street to Weiss Road)
    40 mph (from Weiss Road to Tittabawassee Road)
    50 mph (from Tittabawassee Road to Kochville Road)
- SB Bay Road:
  - -Bandwidth = 24 sec
  - -Progression Speed: 50 mph (from Kochville Road to Tittabawassee Road)
    40 mph (from Kochville Road to Weiss Road)
    35 mph (from Weiss Road to West Genesee Street)
    30 mph (from West Genesee Street to Court Street)

The data obtained from the existing signal timing permits indicate that under the current signal phasing and timing setting no progression exists along the Bay Corridor. After optimizing the signal timing of all intersections in the corridor with an optimized signal cycle length and appropriate offsets, a good progression band can be achieved, which will considerably improve the traffic flow. Even if some of the intersections are operated by actuated or semi-actuated signals, progression can still be achieved if the cycle length and offsets remain constant; minimum and maximum splits are set up and the signal controllers are synchronized to adjust the split within a given threshold value. The minimum green time for through traffic should always be provided and any addition of green time should be varied depending on the traffic volumes, keeping the cycle length and offsets constant.

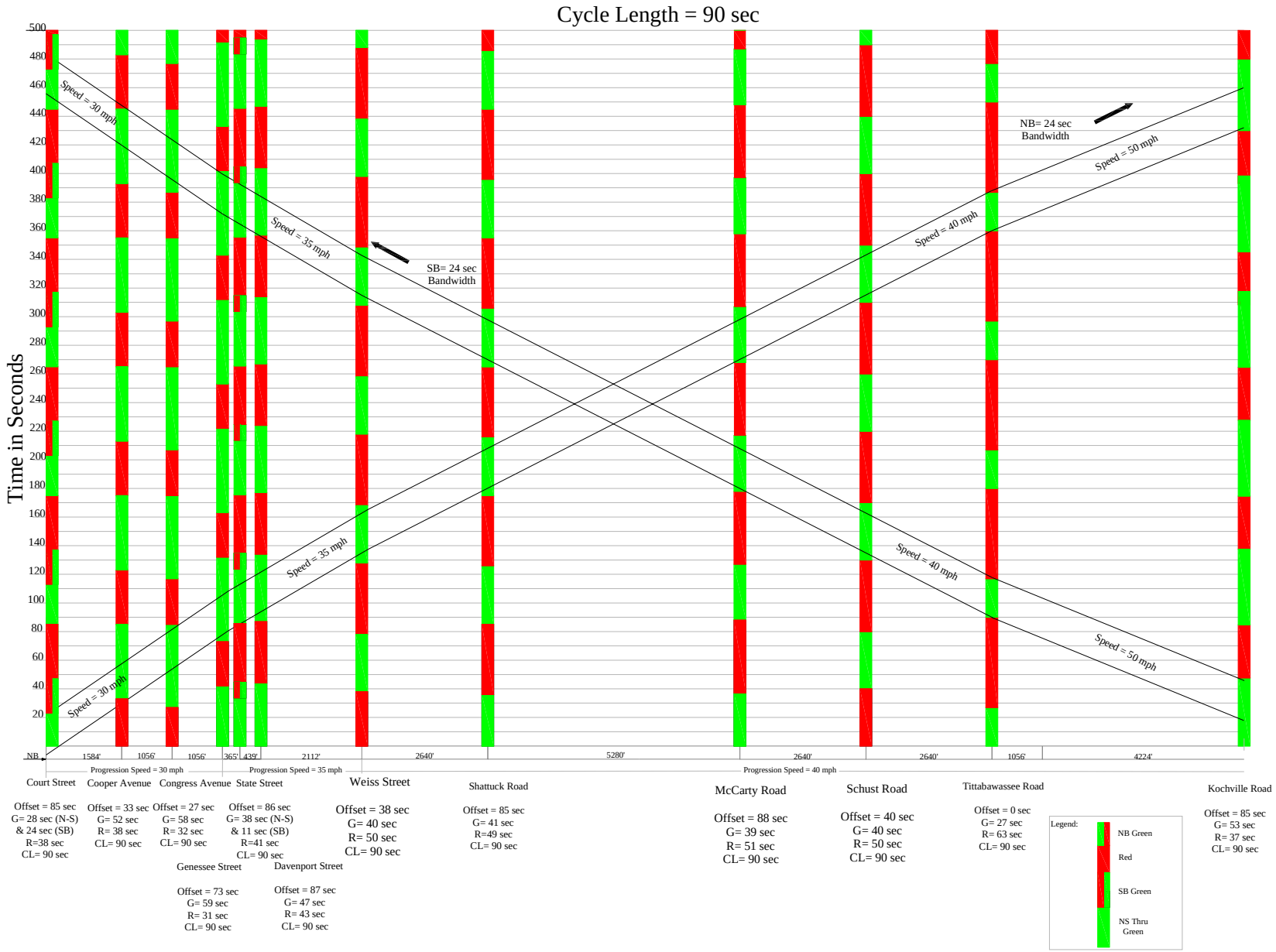


Figure 4.7. Optimized Time Space Diagram for Proposed PM Peak Hour Progression- Bay Road Corridor from Court to Kochville

4.5 Bay Road Corridor – Measure of Effectiveness

An analysis of the Bay Road Corridor for existing conditions showed no progression through the study intersections being operated at varying cycle lengths for the intersections. For the proposed scenario a new progression system was designed, based on an optimum cycle length for all intersections. The existing and proposed signal timing settings were modeled by Synchro and simulated by Sim Traffic to identify the change in performance parameters for the moving traffic between the existing and proposed scenario. Data for different performance parameters, including total delay, fuel consumption, emissions etc., was recorded for a typical peak hour. The data obtained for the proposed improvements showed favorable changes in the measure of effectiveness, as presented in Table 4.1.

Table 4.1. SimTraffic Output for Existing and Proposed Conditions – Bay Road

Performance Parameters (For Peak One Hour)	Existing Conditions	Proposed Conditions	Percentage Improvement
Delay (Sec/Veh)	198.7	112.1	43.6%
Total Delay (Veh-Hour)	852.4	490.0	42.5%
Total Number of Vehicle Stops	61,786	40,428	34.6%
Average Speed (mph)	18	22	22.2%
Fuel Used for the Entire Corridor (Gallons)	2,171	1,961	9.7%
HC Emissions (grams)	7,161	6,270	12.4%
CO Emissions (grams)	333,335	294,588	11.6%
NOx Emissions (grams)	21,459	19,365	9.8%

Table 4.1 shows the change in observed measure of effectiveness, attributable to the proposed improvements. Among the different parameters, the reduction in total delay in the peak hour is specifically significant, about 42.5 percent with the proposed conditions. Also, the average speed of vehicular traffic improved from 18 miles per hour to 22 miles per hour, facilitating enhanced flow. Pollutant emission has also decreased with the proposed conditions. All these improvements in traffic performance parameters involve societal benefits. The tangible benefits include reduction in fuel consumption and travel time, which may be directly equated to a dollar value. The major intangible benefit is the reduction of emission of greenhouse gases.

4.6 Bay Road Corridor – Characteristics of Driver Behavior-Related Crashes

A crash analysis was performed for the study intersections along Bay Road to quantify the number of crashes involving four driver behavior related problems, such as driving under the influence of alcohol, drivers not wearing their seatbelts, crashes involving speeding and red light running. The crash frequency information was obtained by querying the MSP's Statewide Traffic Crash Database for the years 2001-2004. The data available for all four years was analyzed to identify any unusual trends in driver behavior.

For the Bay Road Corridor, thirty-four crashes occurred in the four-year period (2001 to 2004) involving drivers who were under the influence of alcohol, as shown in Figure 4.8. The highest alcohol-related crash frequency observed was six, which occurred at the intersection of Bay Street and Davenport Avenue in 2001.

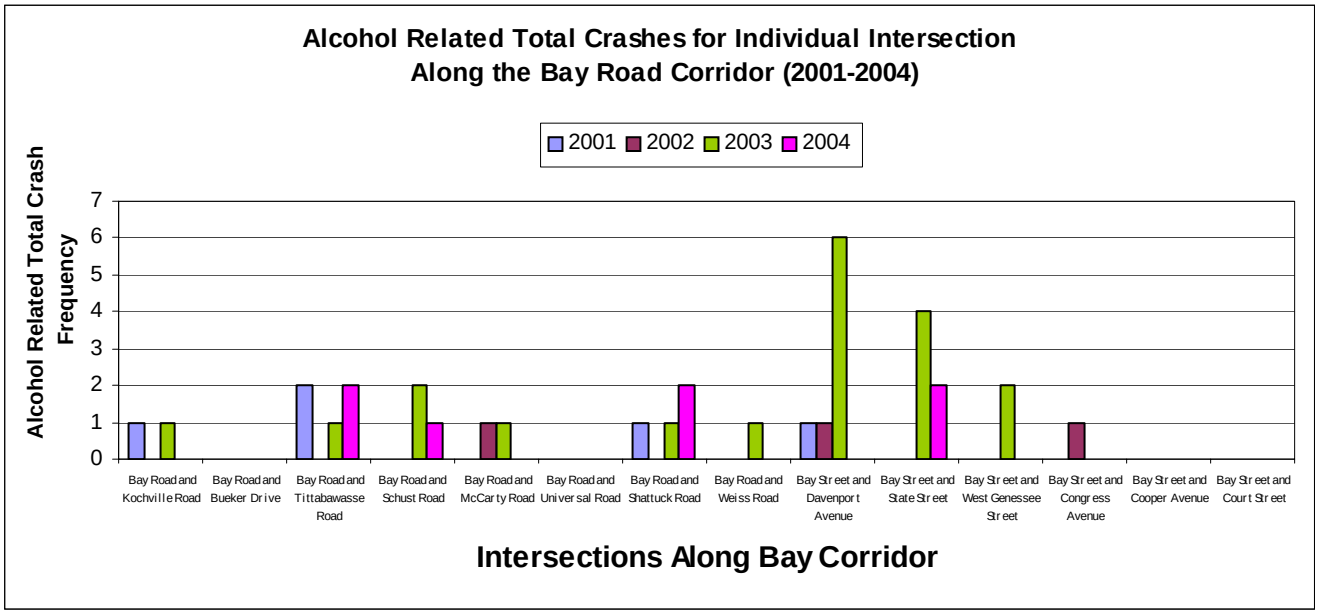


Figure 4.8 Alcohol Related Total Crashes along Bay Road Corridor

There were 10 crashes reported at the fourteen study intersections on Bay Road during the four-year period, which involved drivers who were unbelted at the time of crash as shown in Figure 4.9. The highest crash frequency was in Bay Road and Tittabawassee Road intersection in the year 2003.

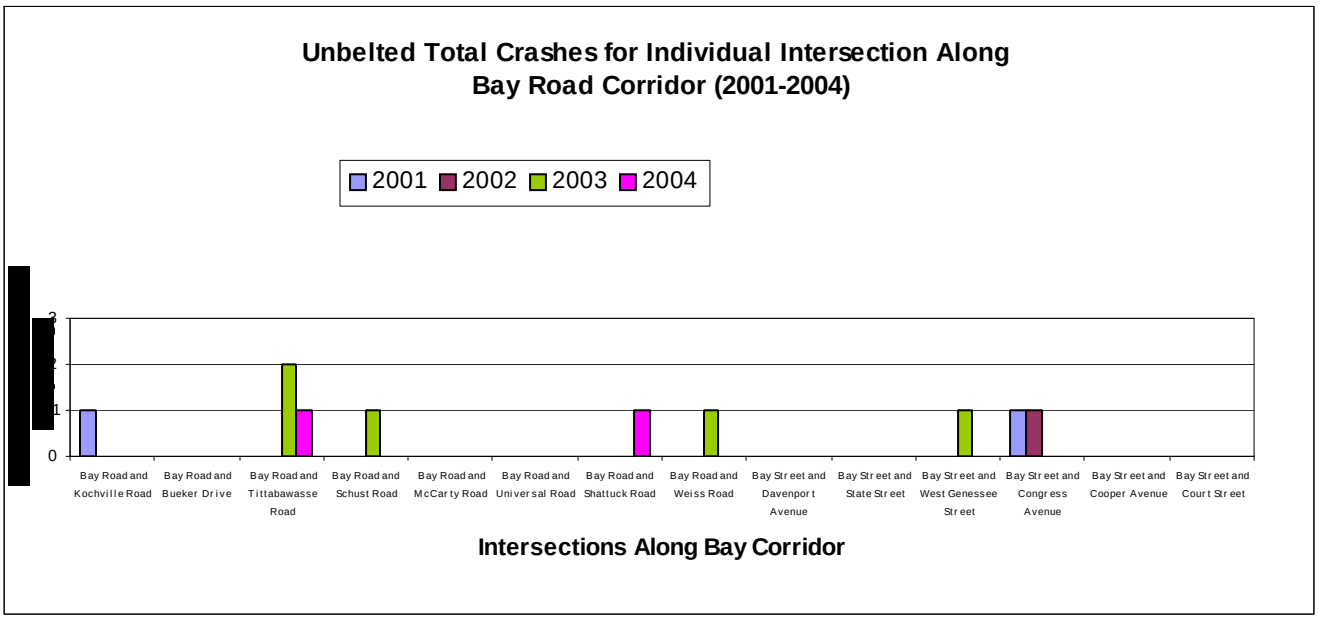


Figure 4.9 Unbelted Total Crashes along Bay Road Corridor

A total of 34 speeding-related crashes were observed at the Bay Road Corridor intersections during the four-year period, as shown in Figure 4.10. The intersection of Bay Road and Tittabawassee Road experienced the highest speeding-related crash frequency (four crashes) in the year 2003. Also during four-year period this intersection experienced the highest crash frequency (11 crashes), followed by the intersection of Bay Road and Shattuck Road (six crashes).

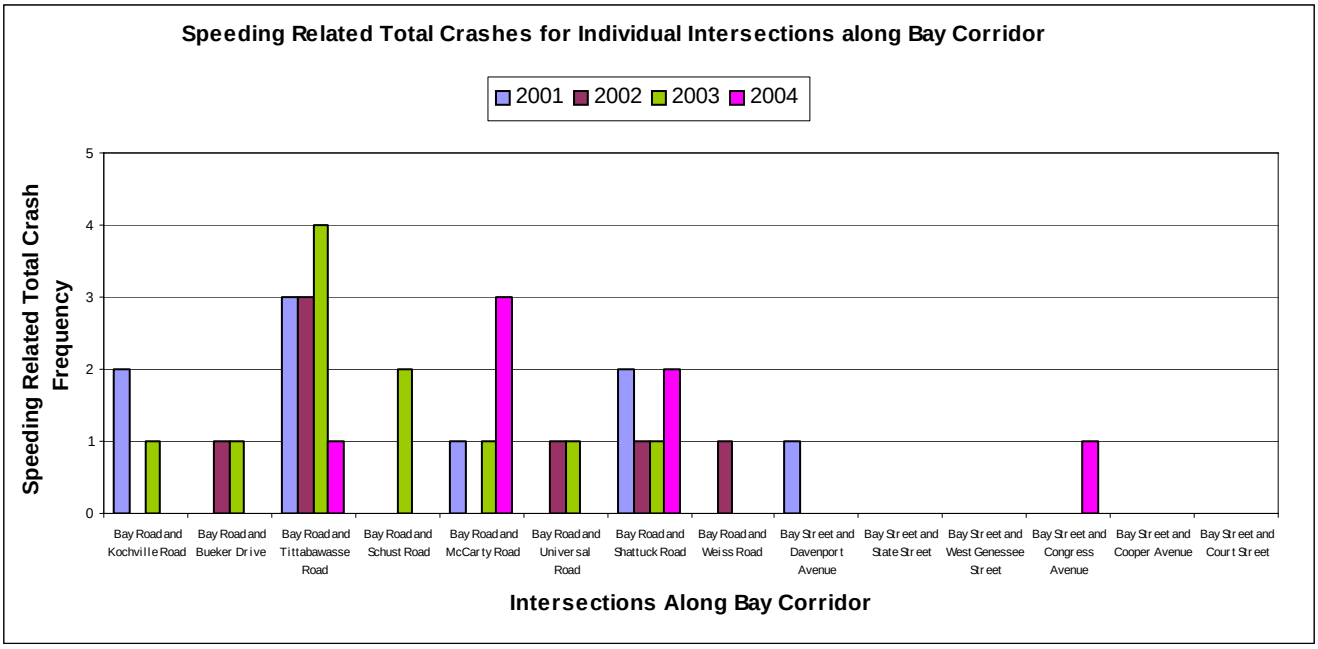


Figure 4.10 Speeding Related Total Crashes along Bay Road Corridor

A total of 95 red light running crashes occurred at the Bay Road Corridor intersections over the four-year period (Figure 4.11). Higher red light violations were observed at the intersection of Bay Street and Davenport Avenue with a crash frequency of nine in the year 2002. A total 27 red light running crashes were observed in this intersection during the four year period, followed by the intersection of Bay Street and State Street with 15 crashes in the four year period.

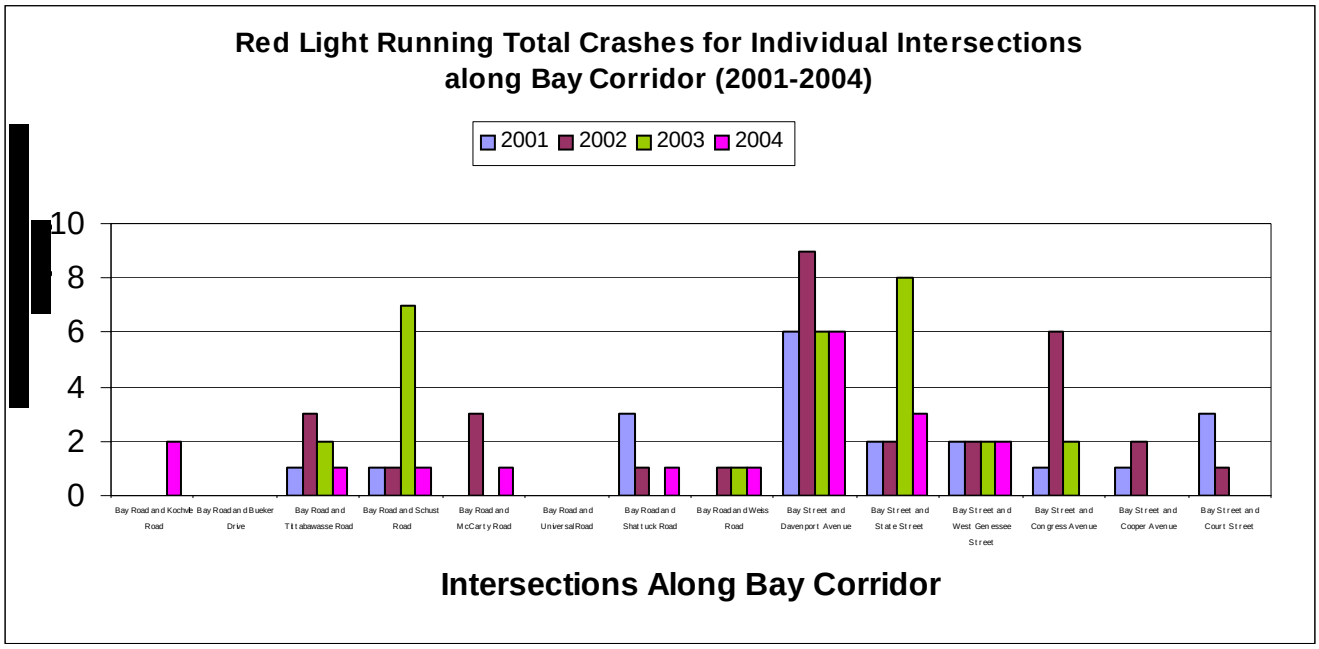


Figure 4.11 Red Light Running Total Crashes along Bay Road Corridor

In summary, the Bay Road Corridor seemed to have a higher proportion of driver behavior-related crashes attributable to red light running, alcohol and speeding related crashes. There were insignificant crashes related to unbelted driver conditions. The intersection of Bay and Davenport Street shows a predominance of red-light running and alcohol related crashes, whereas the intersection of Bay Road and Tittabawassee Road experienced more speeding related crashes.

4.7 Tittabawassee Road Corridor – Physical Characteristics

Tittabawassee Road is a major arterial in Saginaw County, which runs east west (E-W) through Saginaw Township. Five signalized intersections over a 3 mile segment of Tittabawassee Road from North Center Road in the west to North Michigan Avenue in the east includes the study intersections, as shown in Figure 4.12 and are as follows:

- Tittabawassee Road and North Center Road
- Tittabawassee Road and Mackinaw Road
- Tittabawassee Road and Fortune Boulevard
- Tittabawassee Road and Bay Road
- Tittabawassee Road and Fashion Square Boulevard
- Tittabawassee Road and Berl Drive
- Tittabawassee Road and Towne Center Road
- Tittabawassee Road SB I 675
- Tittabawassee Road and NB I 675
- Tittabawassee Road and North Michigan Avenue

From the west, Tittabawassee Road continues to move through the majority of commercial areas. There is a mall near the Tittabawassee Road and Fashion Square Boulevard intersection.

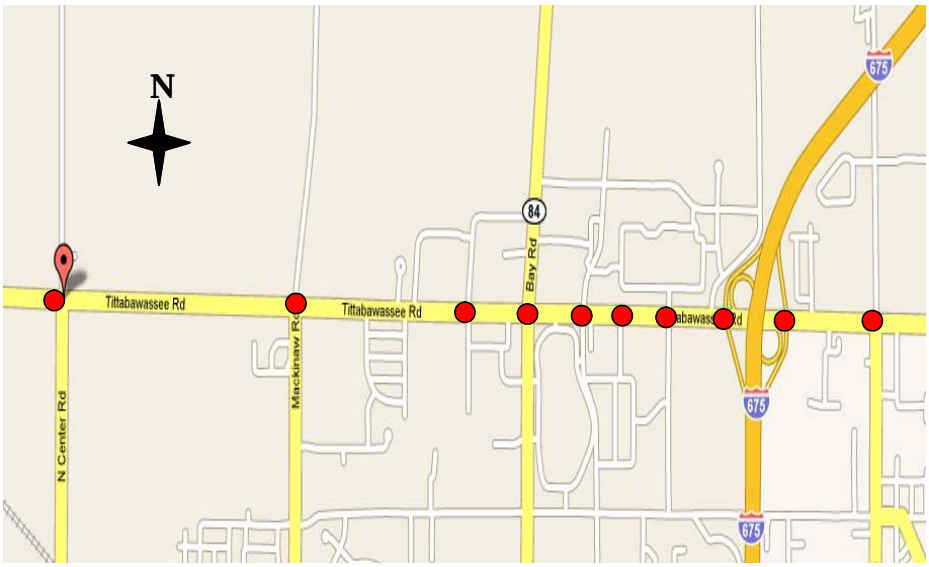


Figure 4.12 Tittabawassee Road Corridor with Study Intersections

A schematic diagram of the Tittabawassee Road Corridor is shown in Figure 4.13, which represents the existing conditions in the field. The directional arrows indicate the PM Peak hour volume and also show the number of lanes for each approach and their usage. Existing signal timing plans are shown above each intersection of the corridor including the cycle length and the phasing diagram.

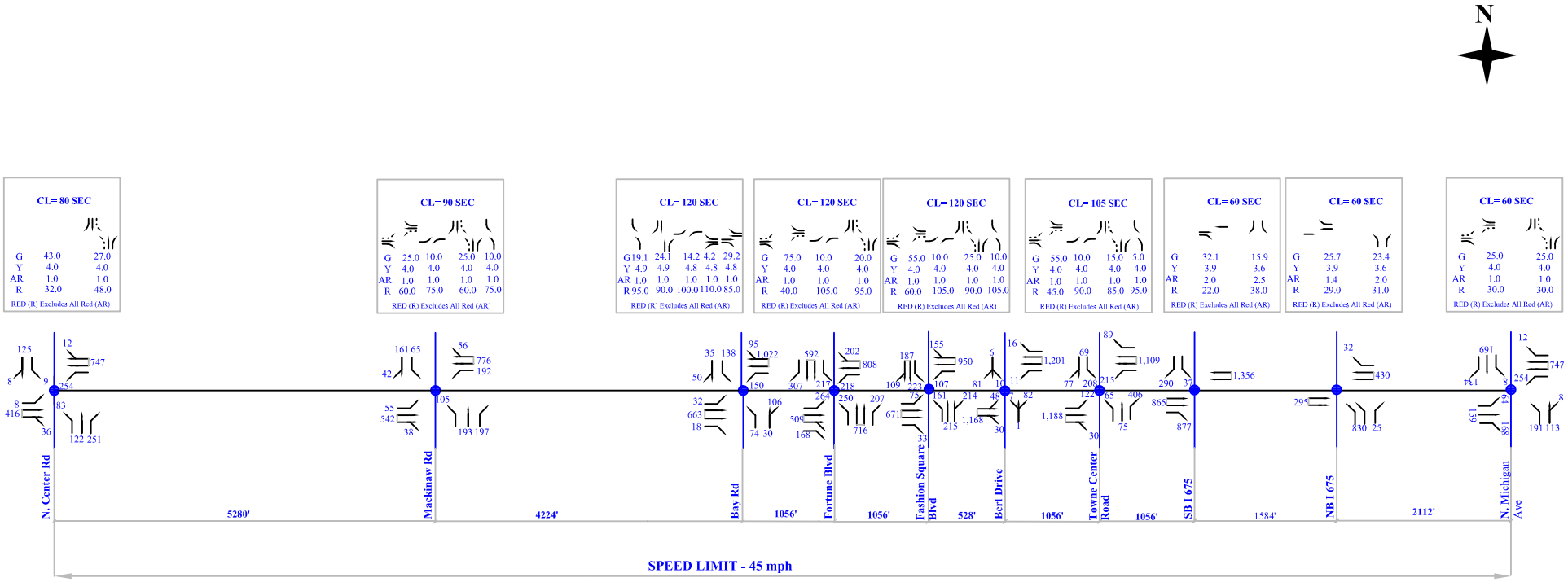


Figure 4.13 Schematic Diagram of Tittabawassee Road Corridor

4.8 Tittabawassee Road Corridor – Intersection Traffic Control

The study corridor on Tittabawassee Road consists of nine signalized intersections and one unsignalized intersection. The Tittabawassee Road intersection at North Center Road operates with an 80 second cycle length using a two phase signal plan during the PM peak hour. Mackinaw Road operates with a 90-second cycle length using a four-phase signal plan. This is a semi-actuated signal where all the left turn phases and N-S Mackinaw through lanes have loop detectors and function as a semi actuated signal. The intersection at Fortune Boulevard works with a 120 second cycle length with a three-phase signal (E-W through, E-W left turn and N-S through) in the PM peak hour. The intersection with Bay Road operates with a cycle length of 120 seconds and has five phases (Lead protected left turn in North-South direction, lead protected left turn in East-West direction and Westbound split). All the left turn phases are actuated in this intersection. The intersection with Fashion Square Boulevard has four signal phases and operates with a 120 second cycle length and is a pre-timed signal. The intersection with Towne Center Road has four phases and works with a 105 second cycle length in the PM peak hour and is fully actuated. The intersection of Tittabawassee Road and SB I 675 operates with a 60 second cycle length with a two-phase signal plan. The intersection with NB I 675 also has a 60 second cycle length with a two-phase signal. The intersection of North Michigan Avenue and Tittabawassee Road operates with a 60 second cycle length and has a simple two-phase signal plan.

4.9 Signal Progression on the Tittabawassee Road Corridor

A progression analysis was performed for the Tittabawassee Road Corridor from North Center Road to North Michigan Avenue, which includes nine intersections. As the Tittabawassee Corridor has one intersection in common with the Bay Corridor, both of the intersections were modeled and optimized as a single network by Synchro. Hence, the same cycle length (90 seconds) as the Bay Corridor was chosen for designing a progression system. The results of the analysis are shown in the time-space diagram in Figure 4.14 and are as follows:

- EB Tittabawassee Road

  -Bandwidth = 18 sec

  -Progression Speed = 45 mph

- WB Tittabawassee Road

  -Bandwidth = 18 sec

  -Progression Speed = 45 mph

The existing signal timing settings in this corridor do not exhibit any progression. After optimizing the corridor for an optimum cycle length, the results indicate that a good bandwidth at feasible peak hour speeds can be achieved, which will eventually improve the through traffic movement in the Tittabawassee Corridor. It should be noted that the progression can still be maintained along the corridor, even if some of the intersections are operated with actuated or semi-actuated control. As long as the cycle length and offsets remain constant, minimum and maximum splits can be set up so they would not interfere with the progression of the main corridor.

4.10 Tittabawassee Road Corridor – Measure of Effectiveness

After optimizing the signal timing for progression by Synchro, the corridor was simulated using SimTraffic software. Data for different performance parameters including total delay, fuel consumption, emissions etc. were recorded for a typical peak hour traffic condition (4.00-6.00 PM). The data obtained for the proposed improvements showed significant changes in the measures of effectiveness, as presented in Table 4.2.

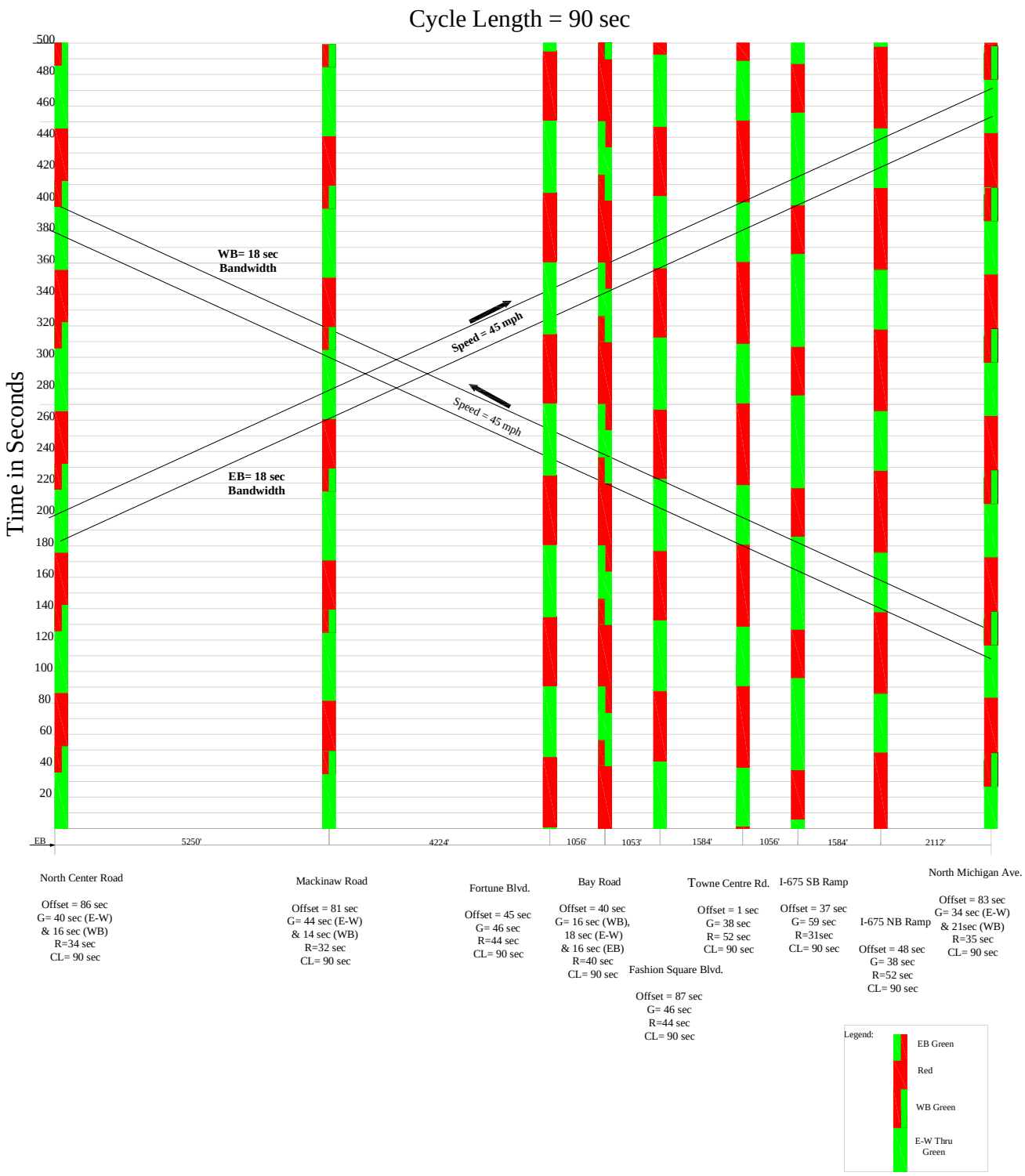


Figure 4.14 Optimized Time Space Diagram for Proposed PM Peak Hour Progression - Tittabawassee Road Corridor from North Center to North Michigan

Table 4.2. SimTraffic Output for Existing and Proposed Conditions-Tittabawassee Corridor

Performance Parameters (For Peak One Hour)	Existing Conditions	Proposed Conditions	Percentage Improvement
Delay (Sec/Veh)	126.6	76.5	39.6%
Total Delay (Veh-Hour)	643.8	394.5	38.7%
Total Number of Vehicle Stops	38,344	34,789	9.3%
Average Speed (mph)	18	22	22.2%
Fuel Used for the Entire Corridor (Gallons)	1628.5	1496.1	8.1%
HC Emissions (grams)	5,381	5,006	7.0%
CO Emissions (grams)	266,629	258,025	3.2%
NOx Emissions (grams)	17,029	16,433	3.5%

Table 4.2 shows the changes observed in the measures of effectiveness, attributable to the proposed improvements. Among the parameters the reduction in total delay and delay per vehicle are significant (nearly 40%). The number of vehicle stops has also decreased by about 9.3 percent, together with an increase in the average speed from 18 mph to 22 mph. The reduction of delay results in the economic benefits.Fuel usage for the entire corridor reduced to 8.5 percent. Additionally, some environmental benefits can be achieved, such as the reduction of emission of harmful gases like hydrocarbons (HC), carbon monoxide (CO) and nitrogen oxides (NOx).

4.11 Tittabawassee Road – Characteristics of Driver Behavior-Related Crashes

The four-year crash frequency (2001-2004) for the intersections along the Tittabawassee Road was determined using the MSP's Statewide Traffic Crash Database for the following driver behavior categories:

- Crashes involving drivers who were under the influence of alcohol
- Crashes involving drivers not wearing their seatbelts

- Speeding-related crashes
- Red light running crashes

In general, the number of crashes involving driver behavioral issues has a relatively low frequency for a given intersection during a given year. For the Tittabawassee Corridor, there were 13 crashes that occurred in a four-year period involving drivers under the influence of alcohol, as shown in Figure 4.15. Two alcohol-related crashes in four intersections are the highest within the corridor.

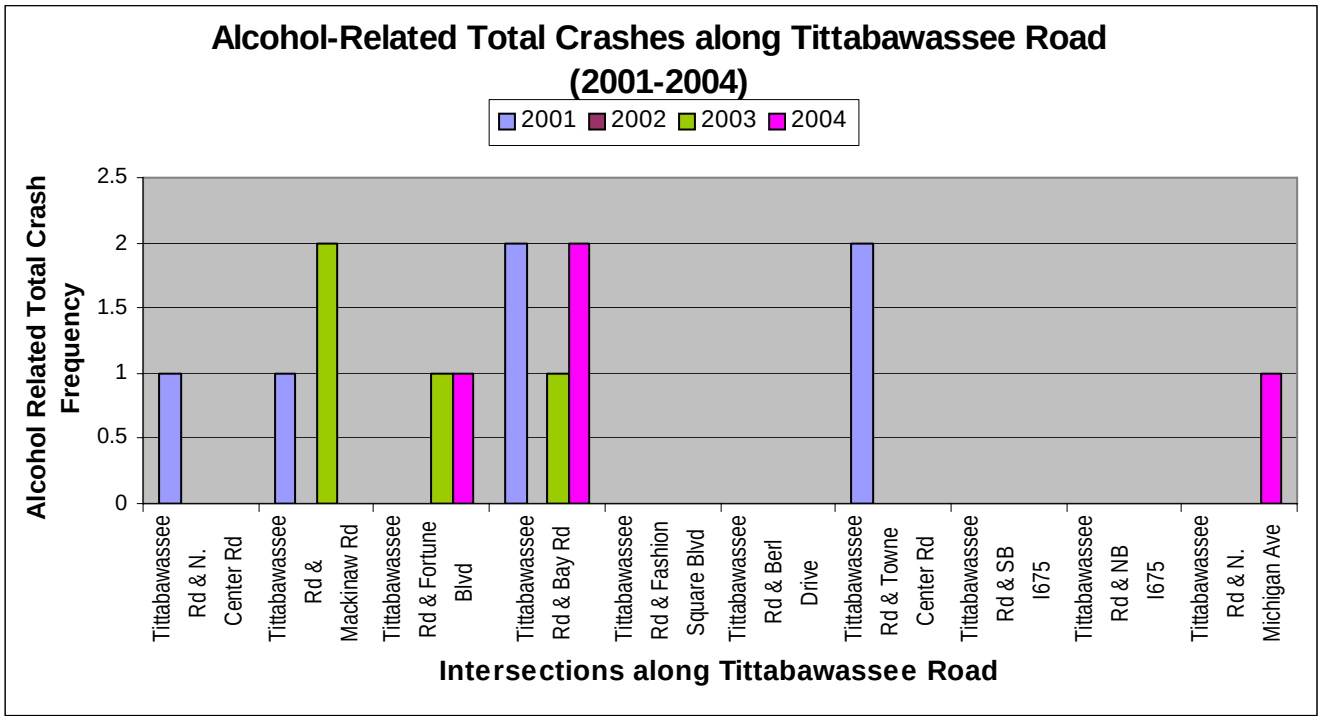


Figure 4.15 Alcohol-Related Total Crashes along Tittabawassee Road Corridor

Crashes involving drivers who were unbelted accounted for a total of eight crashes in the four-year period at the Tittabawassee Road Corridor intersections, which can be considered a low frequency (Figure 4.16).

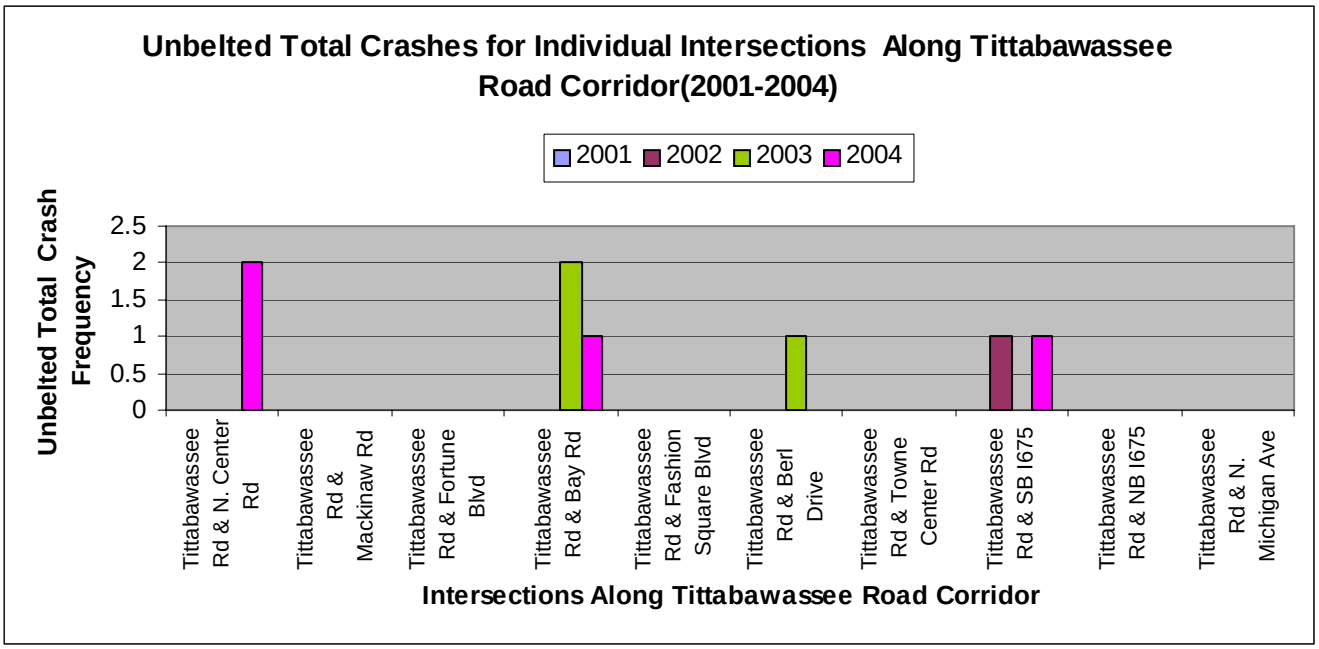


Figure 4.16 Crashes Involving Unbelted Drivers along Tittabawassee Road Corridor

A total of 65 speeding-related crashes occurred at the intersections of the Tittabawassee Road Corridor during the four-year period, as shown in Figure 4.17. The intersection with the highest frequency of speeding-related crashes was Tittabawassee Road and NB I-675 (fourteen crashes in a four-year period).

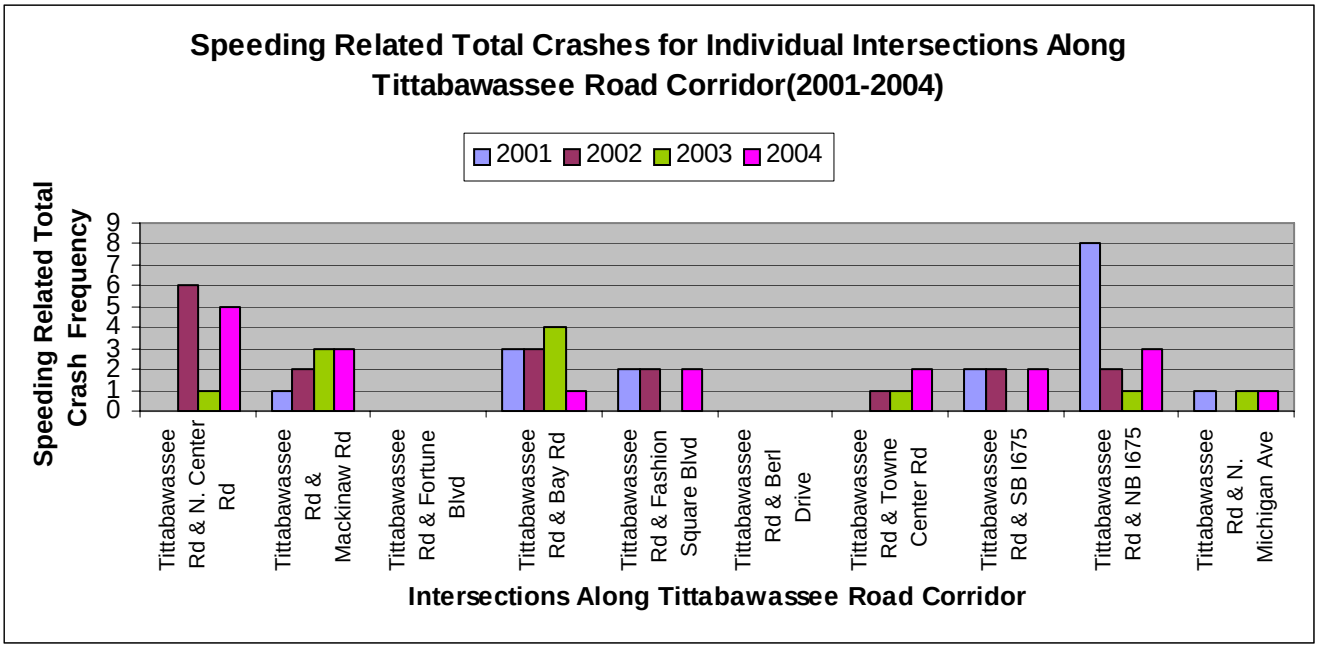


Figure 4.17 Speeding Related Total Crash along Tittabawassee Road Corridor

A total of 42 red light running (RLR) crashes occurred along the Tittabawassee Road Corridor during the four-year study period, as shown in Figure 4.18. The intersection with the highest frequency of RLR crashes is Tittabawassee Road and Mackinaw Road with 16 crashes in a four-year period.

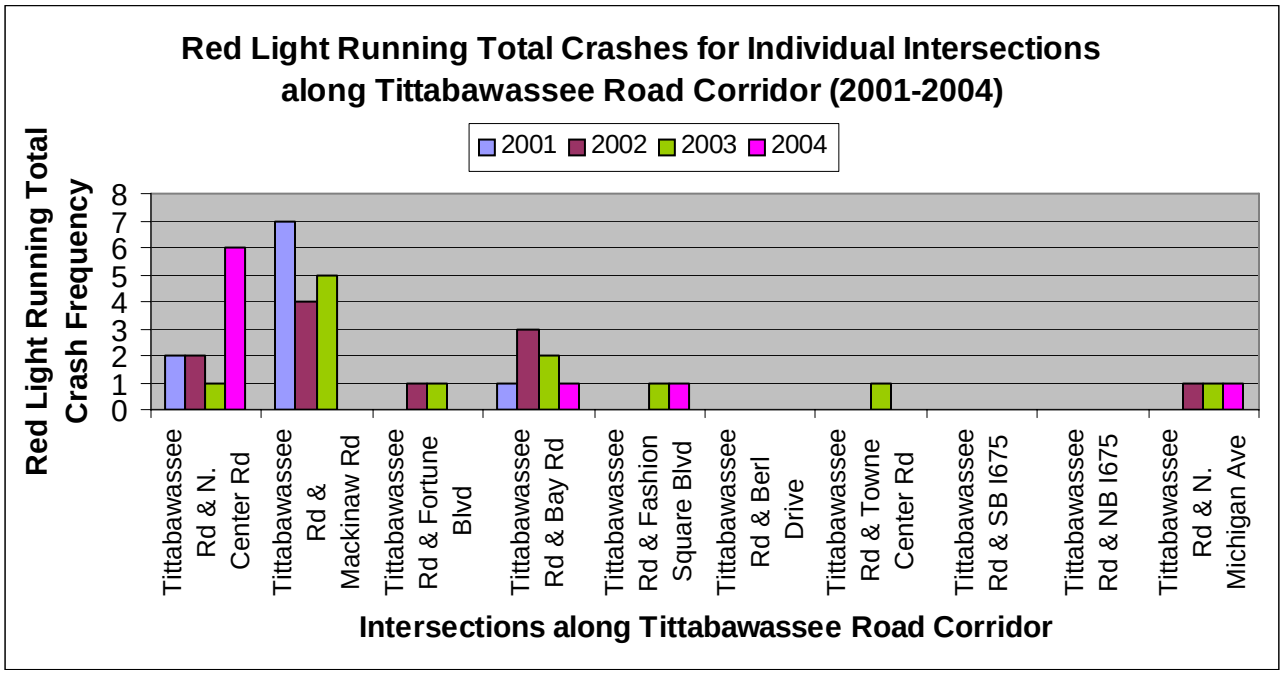


Figure 4.18 Red Light Running Total Crashes along the Tittabawassee Road Corridor

Overall, the predominant driver behavior problem for the Tittabawassee Road Corridor is related to speeding and red light running at intersection approaches. It should be noted that the increased red light running experience might be related to the signal timing design along Tittabawassee Road. In addition to the potential aggressive nature of the drivers traveling along this corridor, specifically the provision of an improper length of the yellow and all red intervals is a problem.

4.12 Gratiot Road Corridor – Physical Characteristics

Gratiot Road is a major arterial in Saginaw County, which runs east west (E-W) through Thomas Township and Saginaw Township. Gratiot Road is a state trunk line named M-46. Five signalized intersections and two unsignalized intersections over a 7.1-mile long segment of Gratiot Road from Graham Road in the west to Wheeler Road in the east includes the study intersections, as shown in Figure 4.19 as follows:

- Gratiot Road and Graham Road
- Gratiot Road and Kennely Road
- Gratiot Road and Van Warmer Road
- Gratiot Road and Miller Road
- Gratiot Road River Road
- Gratiot Road and Center Road
- Gratiot Road and Wheeler Road

From the east, Gratiot Road continues to move through commercialized areas until intersecting with Van Warmer Road and then runs through residential neighborhoods. A schematic diagram of the Gratiot Road Corridor is shown in Figure 4.20, which represents the existing conditions in the field. The directional arrows indicate the PM Peak hour volume and also show the number of lanes for each approach with their usage. Existing signal timing plans are shown above each intersection of the corridor including the cycle length and the phasing diagram.

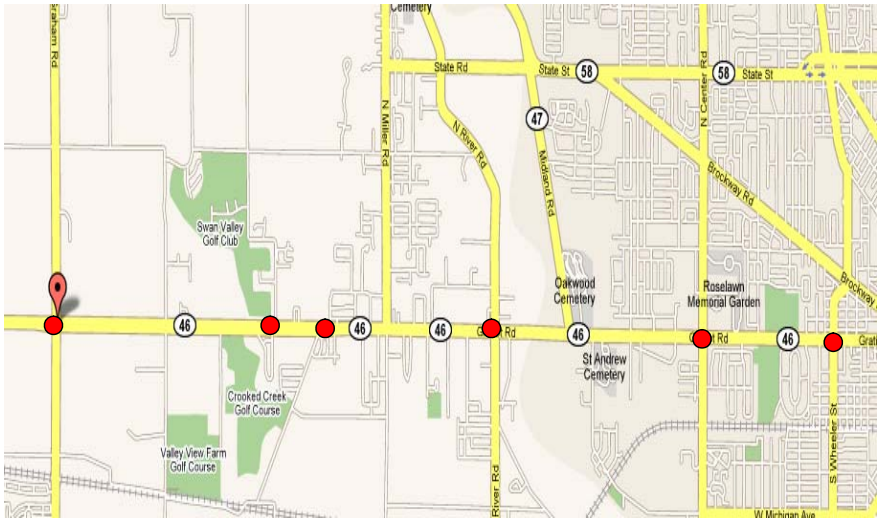
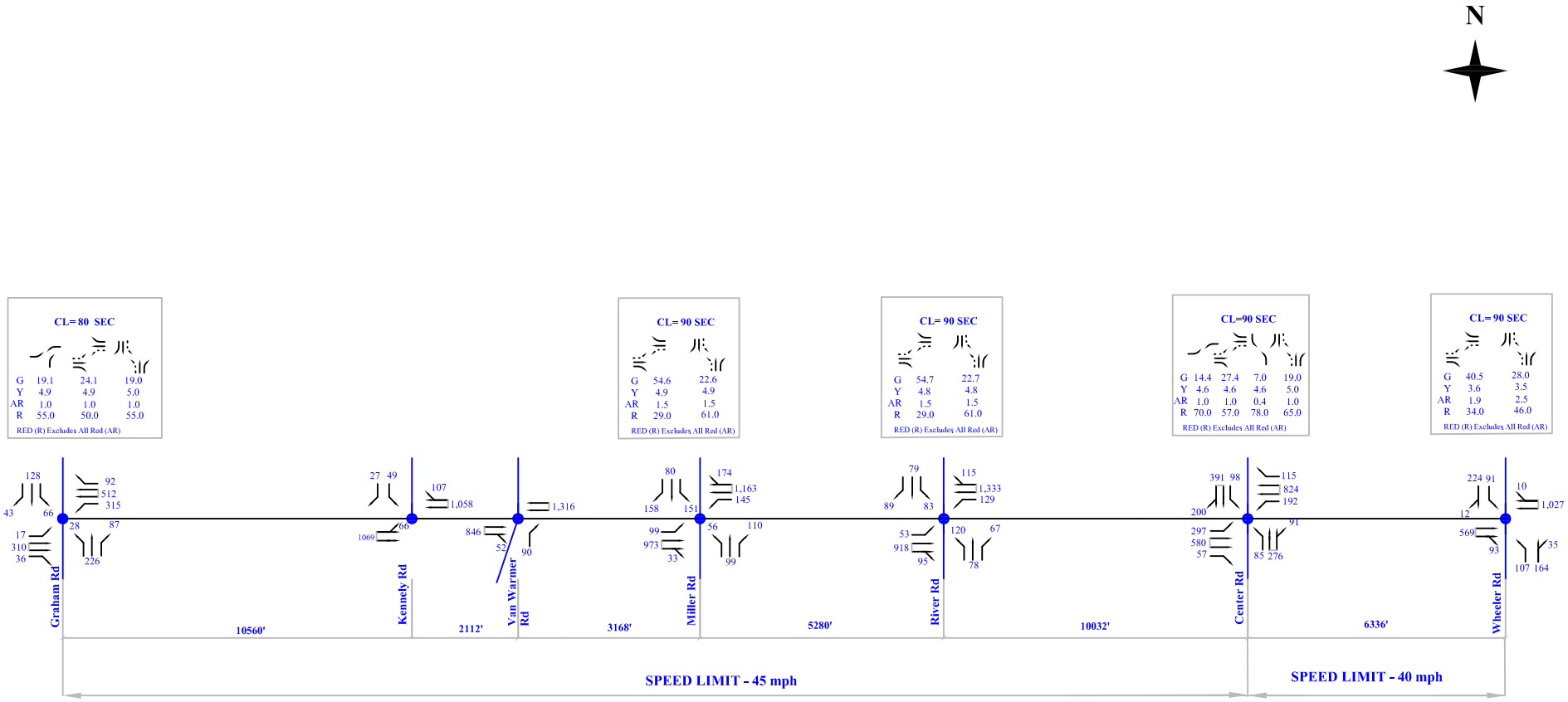


Figure 4.19 Gratiot Road Corridor with Study Intersections



GRATIOT ROAD CORRIDOR

Figure 4.20 Schematic Diagram of Gratiot Road Corridor

The horizontal alignment of Gratiot Road is straight throughout the corridor. At Graham Road, Gratiot Road consists of four lanes for eastbound movements and four lanes for westbound movement with two through lanes, a right-turn lane and a left-turn lane. Near the intersection with Kennely Road, eastbound and westbound movements are separated by the grass median. Gratiot Road has two lanes for through movements in each approach. The same four-lane cross section is followed for the Van Warmer intersection also. The posted speed limit for the Gratiot Road Corridor from Graham Road to Center Road is 45 mph and is reduced to 40 mph further east from Center Road.

4.13 Gratiot Road Corridor – Traffic Volumes

The ADT volume data along the Gratiot Road (M-46) Corridor was obtained from Traffic Count Maps published by the Michigan Department of Transportation for 2004 and is shown in Figure 4.21.

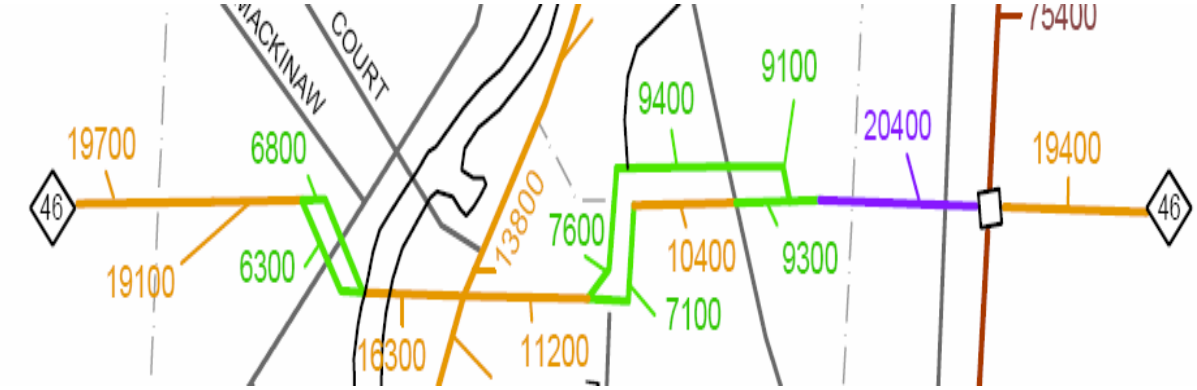


Figure 4.21 ADT Volumes along Gratiot Road Corridor (Source: MDOT ADT Map, 2004)

4.14 Gratiot Road Corridor – Intersection Traffic Control

The study corridor on Gratiot Road consists of five signalized intersections and two unsignalized intersections. Gratiot Road at Graham Road intersection operates with a cycle length of 80 seconds using a three-phase design (E-W left turn, E-W through, N-S through) and the intersection at Miller Road operates with a 90-second cycle length using a two-phase design. The Gratiot Road and Miller Road intersection is semi actuated where detectors are placed in N-S Miller Road. The Gratiot Road and River Road intersection has a simple two-phase signal with a 90 second cycle length and is semi actuated, where

detectors are placed in N-S River Road. At the Gratiot Road and Center Road intersection, the cycle length is 90 seconds with a four phase signal plan where the left turn bays in every approach are actuated and the Gratiot Road and Wheeler Road intersection has a simple two phase signal with a cycle length of 80 seconds. The signals along the corridor are supported by diagonal span wire configurations at all the intersections. The intersections with Kennely Road and Van Warmer Road are controlled by a stop control sign.

4.15 Signal Progression on Gratiot Road Corridor

A progression analysis was performed for the Gratiot Corridor (M-46) from Graham Road to Michigan Avenue that includes seven intersections using Synchro and SimTraffic programs. The cycle length of the network was optimized to be 90 seconds. As the distances among different intersections along this corridor are quite high, it might not be suitable to get adequate progression along this corridor. The speed limit along the Gratiot Corridor is 45 mph and it was considered for progression. The results of this analysis are shown on the time space diagram in Figure 4.22 and are as follows:

- EB Gratiot Road:
  - -Bandwidth = 18 sec
  - -Progression Speed= 45 mph
- WB Bay Road:
  - -Bandwidth =18sec
  - -Progression Speed= 45 mph

The existing signal timing settings in this corridor do not exhibit any progression. After optimizing the signal timing of all intersections in the corridor with a unique cycle length and appropriate offsets, moderate bandwidth can be achieved, which will considerably improve the traffic flow. Even if some of the intersections are operated by actuated or semi-actuated signals, progression can still be achieved if the cycle length and offsets remain constant; minimum and maximum splits are set up and the signal controllers are synchronized to adjust the split with a given threshold value. The minimum green time for through traffic should always be provided and any addition of green should be varied depending on the traffic volumes, keeping the cycle length and offset constant.

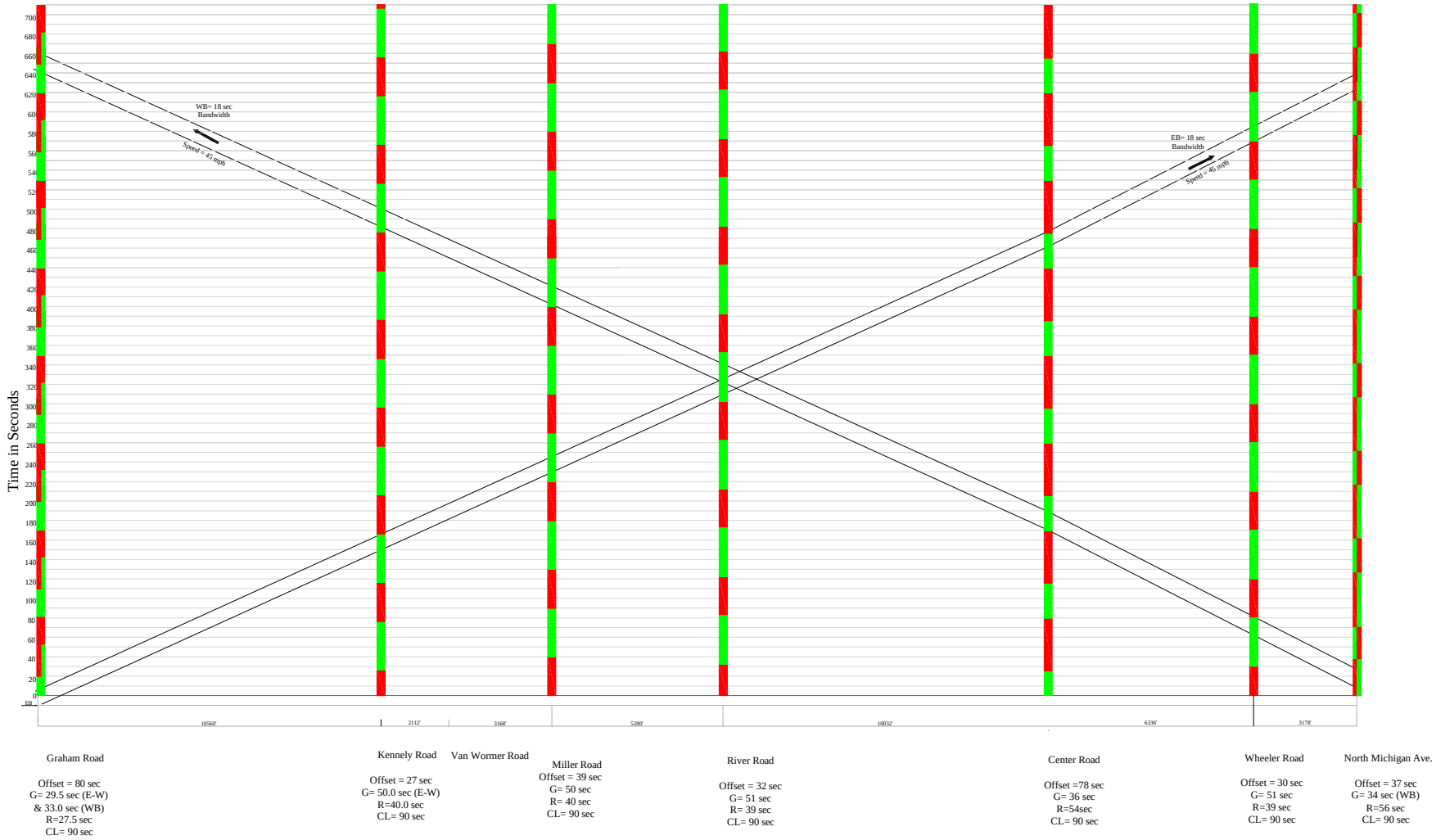


Figure 4.22. Optimized Time Space Diagram for Proposed PM Peak Hour Progression - Gratiot Road Corridor from Graham Road to North Michigan Avenue

4.16 Gratiot Road Corridor – Measure of Effectiveness

After optimizing the signal timing for progression by Synchro, the corridor was simulated by using SimTraffic software. Data for different performance parameters including total delay, fuel consumption, emissions etc. were recorded for a typical peak hour condition (4.00-6.00 PM). The data obtained for the proposed improvements showed significant changes in the measures of effectiveness, as presented in Table 4.3.

Table 4.3. SimTraffic Output for Existing and Proposed Conditions – Gratiot Road

Performance Parameters (For Peak One Hour)	Existing Conditions	Proposed Conditions	Percentage Improvement
Delay (Sec/Veh)	88.1	88.9	-0.91%
Total Delay (Veh-Hour)	211.7	212.8	-0.52%
Total Number of Vehicle Stops	19,139	16,187	15.42%
Average Speed (mph)	33	33	0%
Fuel Used for the Entire Corridor (Gallons)	1,737	1,690	2.67%
HC Emissions (grams)	4,824	4,632	3.98%
CO Emissions (grams)	214,329	197,937	7.65%
NOx Emissions (grams)	14,817	14,237	3.91%

Table 4.3 shows the changes observed in the measures of effectiveness, attributable to the proposed improvements. Among the different parameters, there have been some improvements and some deterioration. For example, the delay parameters like total delay and delay per vehicle have been slightly increased. However, pollutant emission has been decreased in the proposed conditions. In addition, the number of vehicles stopped has been reduced. There have been some economic benefits in terms of less consumption of fuel. The average speed along the corridor has remained unchanged.

4.17 Gratiot Road Corridor – Characteristics of Driver Behavior-Related Crashes

The four-year crash frequency (2001-2004) for the intersections along the Gratiot Road Corridor was obtained from the MSP's Statewide Traffic Crash Database for the following driver behavior related crash categories:

- Crashes involving drivers who were under the influence of alcohol
- Crashes involving drivers not wearing their seatbelts
- Speeding-related crashes
- Red light running crashes

For the Gratiot Road Corridor, there were twenty crashes that occurred in a four-year period involving drivers under the influence of alcohol, as shown in Figure 4.23.

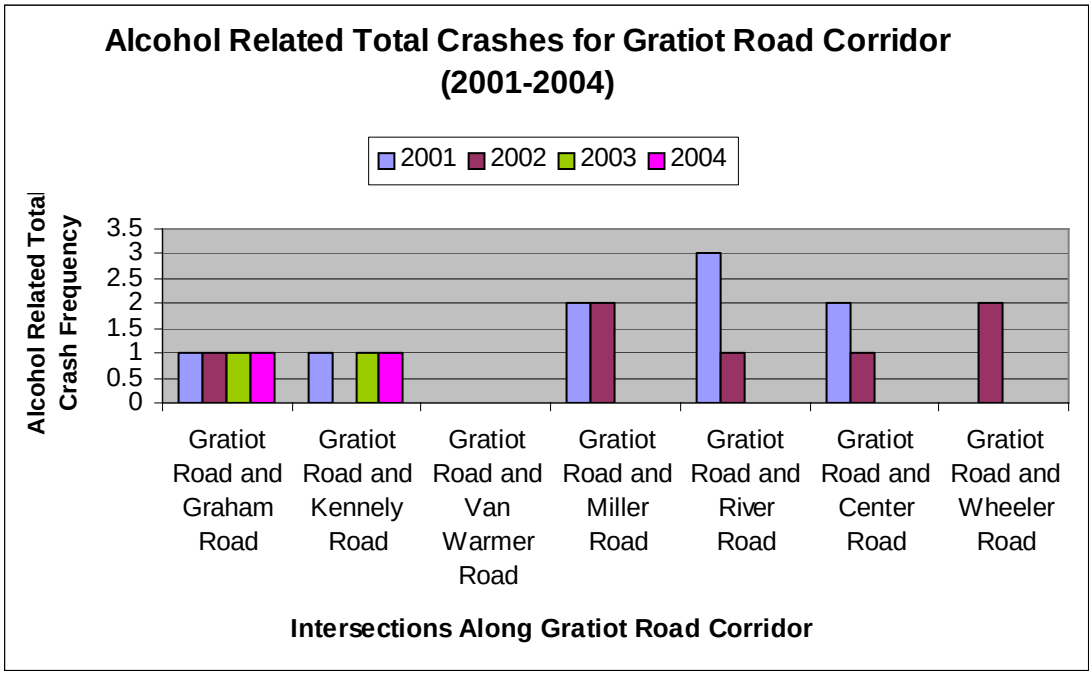


Figure 4.23 Alcohol Related Total Crashes along Gratiot Road Corridor

The highest number of alcohol related crashes (3 crashes) occurred at the intersection of Gratiot and River road in the year 2001. On the other hand the intersection of Gratiot and Graham Road has one alcohol related crash every year.

Crashes involving unbelted drivers accounted for only two crashes in the four-year period at the Gratiot Road Corridor intersections, which can be considered a very low frequency.

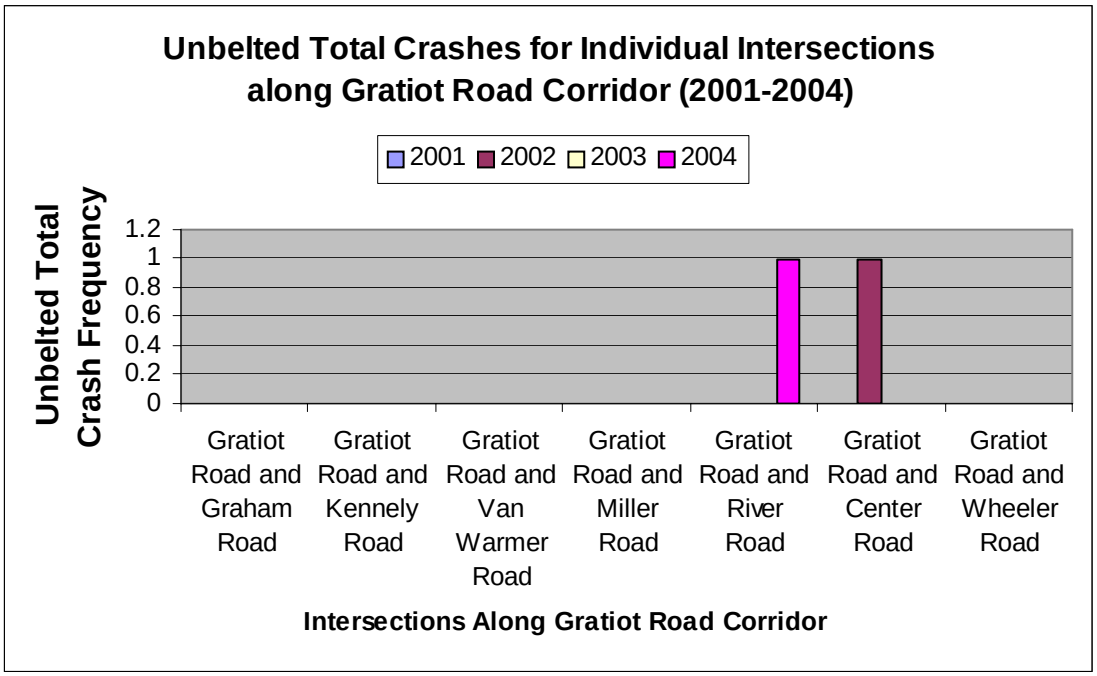


Figure 4.24 Unbelted Total Crashes along Gratiot Road Corridor

A total of thirty-seven speeding-related crashes occurred at the Gratiot Road Corridor intersections during the four-year period, as shown in Figure 4.25. The intersection with the highest frequency of speeding-related crashes was Gratiot Road and Center Road with a total of fourteen crashes.

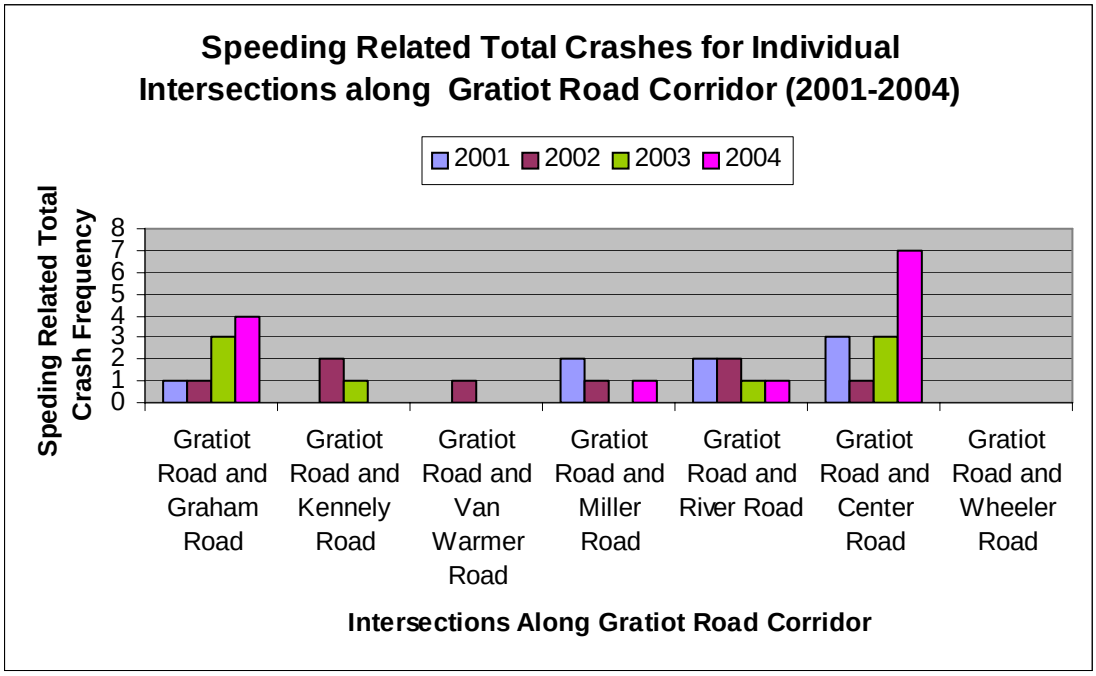


Figure 4.25 Speeding Related Total Crashes along Gratiot Road Corridor

A total of fifty three red light running (RLR) crashes occurred along the Gratiot Road Corridor during the four-year study period, as shown in Figure 4.26. The intersection with the highest frequency of RLR crashes was Gratiot Road and Center Road with sixteen crashes in a four-year period.

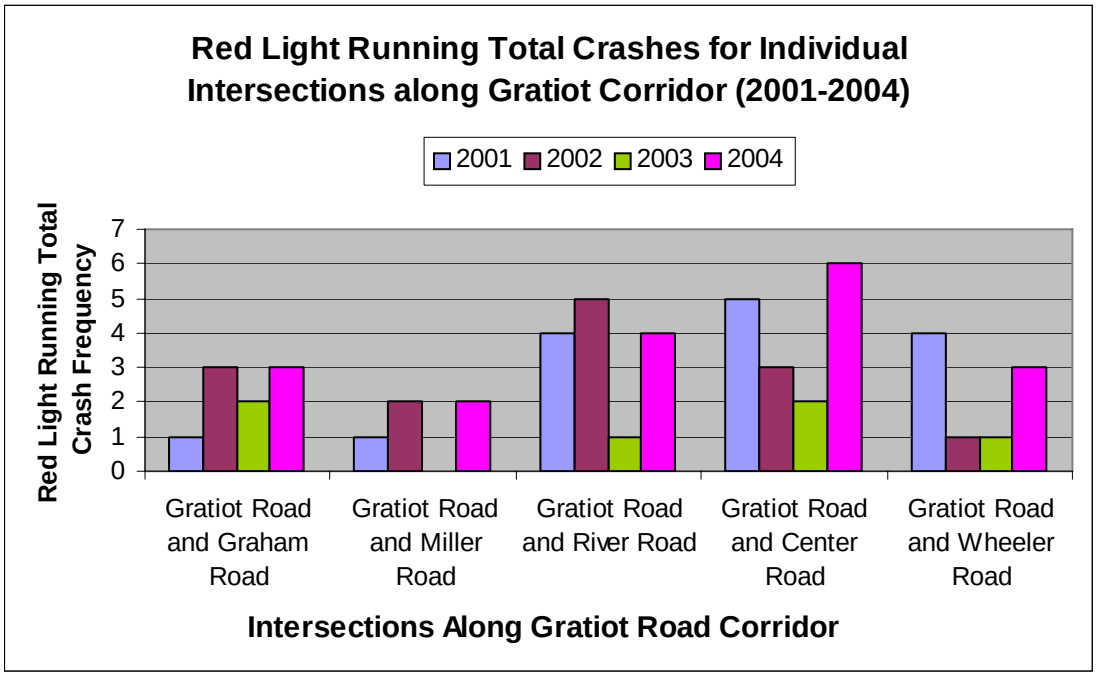


Figure 4.26 Red Light Running Total Crashes along Gratiot Road Corridor

Overall, the predominant driver behavior problem for the Gratiot Road Corridor is related to red light running. It should be noted that the high red light running experience may be related to the signal timing design along Gratiot Road, specifically the inadequacy of yellow and all-red intervals, which is in addition to the potential aggressive nature of the drivers traveling along this corridor.

4.18 North Michigan Avenue – Physical Characteristics

North Michigan Avenue is a major arterial in Saginaw County. North Michigan Avenue starts from Tittabawassee Road in the north and runs in the north-south direction until it reaches State Road where it turns in the southwest direction. Five signalized intersections over a 1.8 mile long segment of North Michigan Avenue, from Davenport Avenue in the north to Gratiot and Mackinaw Avenue in the south, were considered the study intersections, as shown in Figure 4.27, which are as follows:

- North Michigan Avenue and Davenport Avenue
- North Michigan Avenue and State Street.
- North Michigan Avenue and West Genesee Street.
- Michigan Avenue and Court Street.
- Michigan Avenue, Mackinaw Street and Gratiot Avenue

North Michigan Avenue generally includes commercial developments along its corridor. However, at the intersection with Davenport Avenue there are some residential homes. A schematic diagram of the North Michigan Corridor is provided in Figure 4.28, which depicts the prevailing conditions. The directional arrows shown in this figure indicate the PM Peak hour volume and also show the number of lanes for each approach and their usage. Existing signal timing plans are shown above each intersection of the corridor including the cycle length and the phasing diagram.

The horizontal alignment of the Michigan Corridor is generally straight. North of the intersection with Davenport Avenue, Michigan Avenue is a two-lane one-way road for northbound movement. South of the intersection, it is a two-way road with one lane going in the southbound direction. For northbound traffic, near the intersection, there is one left-turn lane and two through lanes. In between the intersections with Davenport Avenue and State Street, North Michigan Avenue is a four-lane road. The stretch of the corridor between State Street and West Genesee is a four-lane road with two lanes for both directional movements. At the intersection with West Genesee Avenue, there is an extra right turn lane for northbound traffic. South of this intersection North Michigan Avenue is a 5-lane road with one center lane for left-turning movement. At the intersection with Court Street the intersection has exclusive left-turn lanes for both north and southbound traffic. The stretch

between the Michigan Avenue and Court Street intersection and Michigan, Mackinaw and Gratiot has a five-lane uniform lane configuration with one center turning lane. The southern-most study intersection in the North Michigan Corridor is a five-legged intersection comprised of Gratiot Avenue, Mackinaw Street and Michigan Avenue. Near this intersection Michigan Avenue is a six-lane road with an extra right-turn lane. The posted speed limit for the entire study corridor of North Michigan Avenue is 25 mph.

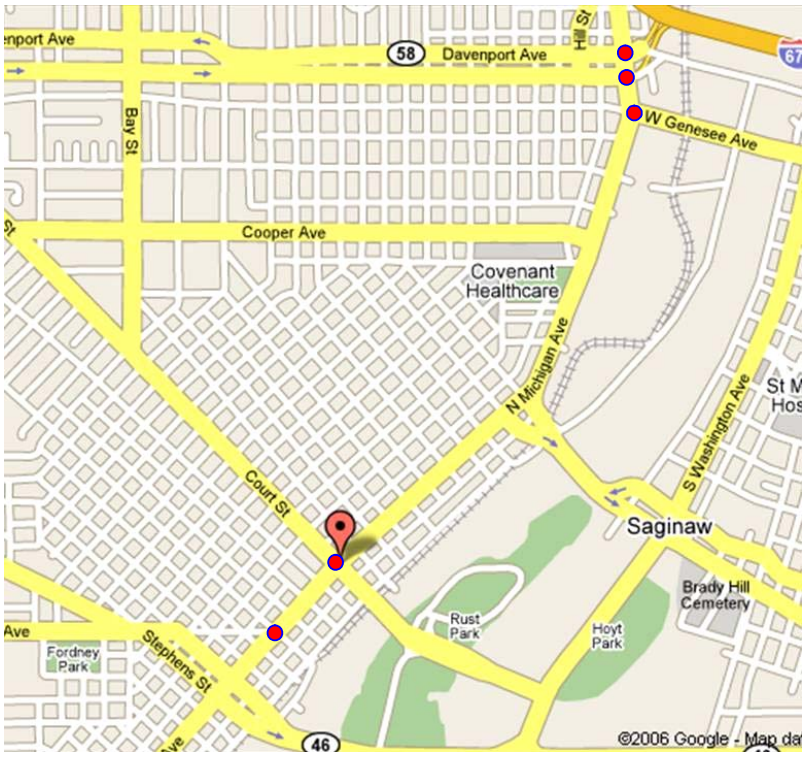
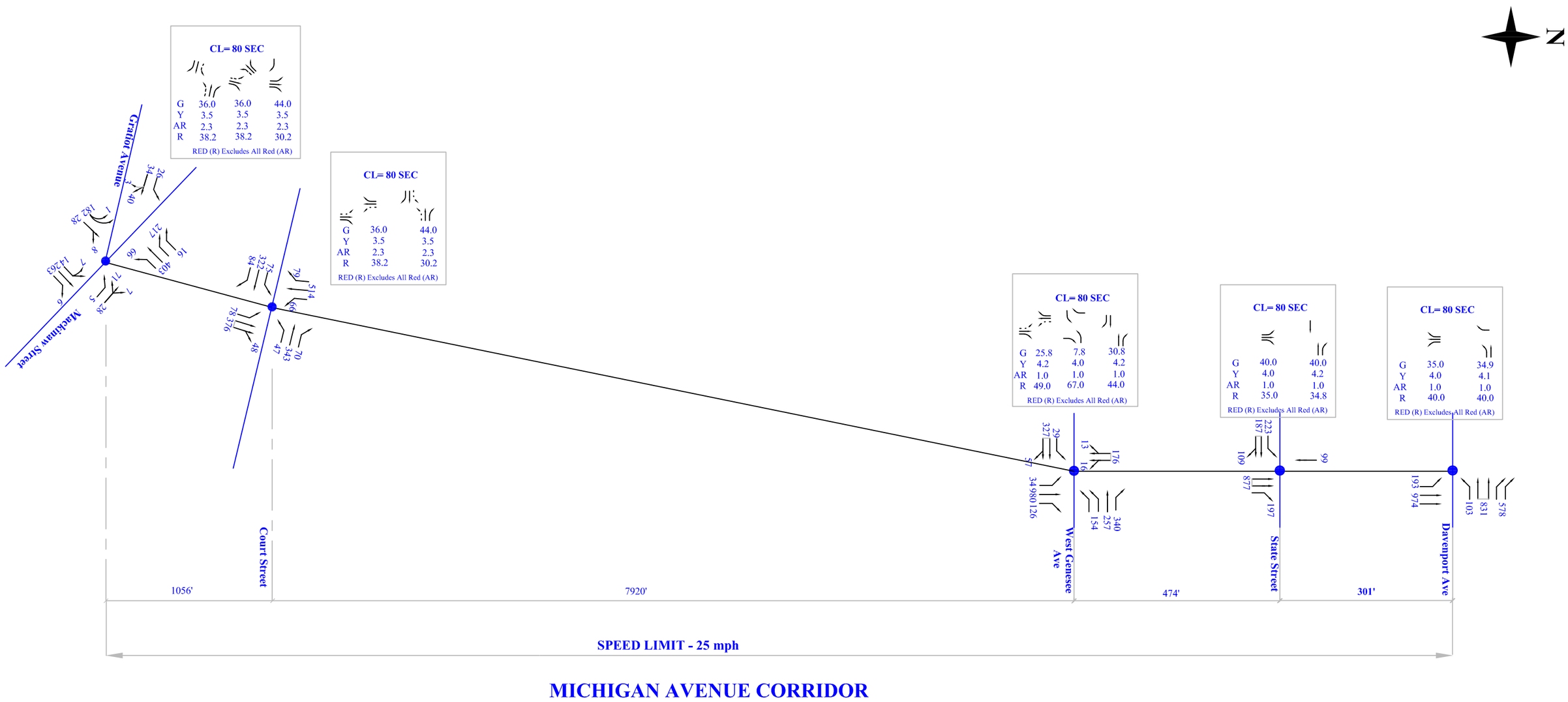


Figure 4.27 North Michigan Avenue Corridor with Study Intersections

4.19 North Michigan Avenue Corridor – Intersection Traffic Control

The study corridor on North Michigan Avenue consists of five signalized intersections. Both intersections of North Michigan with Davenport Avenue and State Street are pre-time controlled with a cycle length of 80 seconds for the PM peak hour. The intersection of North Michigan Avenue and West Genesee is fully actuated with a cycle length of 80 seconds for the PM peak period. Both intersections at Court Street and at Gratiot/Mackinaw Street are pre-time controlled with a cycle length of 80 seconds during the PM peak period. The signals along the corridor are supported by diagonal span wire configurations at all the intersections. All the study intersections in the North Michigan Corridor have the same cycle length of 80 seconds during the PM peak period.



MICHIGAN AVENUE CORRIDOR

Figure 4.28 Schematic Diagram of North Michigan Avenue Corridor

4.20 Signal Progression on North Michigan Avenue

A progression analysis was performed for Michigan Avenue from Gratiot Road to Davenport Avenue, including five intersections using Synchro and SimTraffic programs. The cycle length of the network was optimized to be 90 seconds. The cycle length for the Gratiot Corridor was also chosen for the North Michigan Corridor, as there is a common intersection between these two corridors. The speed limit along the Gratiot Corridor is 30 mph and it was considered for progression. The results of this analysis are shown on the time space diagram in Figure 4.29 and are as follows:

- EB Gratiot Road:
  - -Bandwidth = 16 sec
  - -Progression Speed= 30 mph
- WB Bay Road:
  - -Bandwidth =16 sec
  - -Progression Speed= 30 mph

The data obtained from the existing signal timing permits indicate that under the current signal phasing and timing setting no progression exists along the Michigan corridor. After optimizing the signal timing of all intersections in the corridor with an optimized cycle length and appropriate offsets a moderate progression band can be achieved, which will considerably improve the traffic flow. Even if some of the intersections are operated by actuated or semi-actuated signals, the progression can still be achieved if the cycle length and offsets remain constant; minimum and maximum splits are set up and the signal controllers are synchronized to adjust the split within a given threshold value. The minimum green time for through traffic should always be provided and any addition of green time should be varied depending on the traffic volumes, keeping the cycle length and offsets constant.

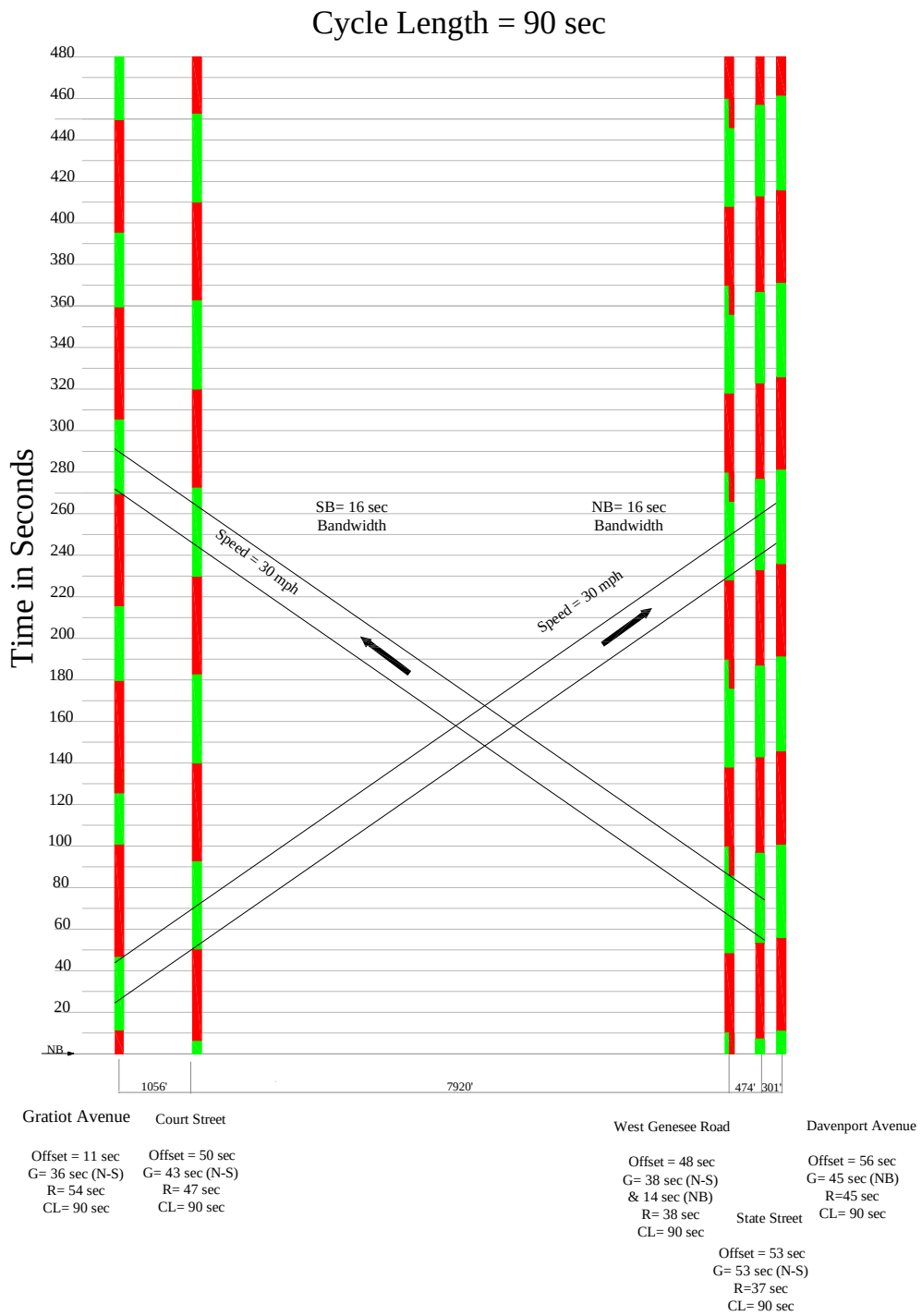


Figure 4.29. Optimized Time Space Diagram for Proposed PM Peak Hour Progression - Michigan Avenue Corridor from Davenport Avenue to Gratiot Avenue

4.21 North Michigan Avenue Corridor – Measure of Effectiveness

An analysis of the North Michigan Avenue Corridor for existing conditions showed no progression as the component intersections operated at varying cycle lengths for all the intersections. For the proposed scenario a new progression was designed, based on an optimum cycle length for all intersections. The existing and proposed signal timing settings were modeled by Synchro and simulated by Sim Traffic to identify the change in performance parameters of the moving traffic between the existing and proposed scenario. Data for different performance parameters including total delay, fuel consumption, emissions etc. was recorded for a typical peak hour. The data obtained for the proposed improvements showed favorable changes in the measure of effectiveness, as presented in Table 4.3.

Table 4.4. SimTraffic Output for Existing and Proposed Conditions – Gratiot Road

Performance Parameters (For Peak One Hour)	Existing Conditions	Proposed Conditions	Percentage Improvement
Delay (Sec/Veh)	39.8	38.8	2.51
Total Delay (Veh-Hour)	78.2	74.9	4.22
Total Number of Vehicle Stops	6354	6090	4.15
Average Speed (mph)	21	21	0.00
Fuel Used for the Entire Corridor (Gallons)	377.2	374.1	0.82
HC Emissions (grams)	931	923	0.86
CO Emissions (grams)	27503	27039	1.69
NOx Emissions (grams)	2437	2411	1.07

Table 4.4 shows the changes in observed measure of effectiveness, attributable to the proposed improvements. Among the parameters the reduction in total delay and delay per vehicle are nominal. The number of vehicle stops has also been decreased by about 4.2 percent, although there is no change in the average speed along the corridor. The reduction of delays results in economic benefits in terms of less fuel usage for the entire corridor, although this benefit is insignificant (only 0.82 %). Some negligible environmental benefits can be achieved, such as the reduction of emission of harmful gases likes hydrocarbons (HC), carbon monoxide (CO) and nitrogen oxides (NO_x).

4.22 Michigan Avenue – Characteristics of Driver Behavior-Related Crashes

The four-year crash frequency (2001-2004) for the intersections along the Michigan Avenue Corridor was determined using the MSP's Statewide Traffic Crash Database for the following driver behavior categories:

- Crashes involving drivers who were under the influence of alcohol
- Crashes involving drivers not wearing their seatbelts
- Speeding-related crashes
- Red light running crashes

For the Michigan Avenue Corridor, 6 crashes occurred in a four-year period involving drivers under the influence of alcohol, as shown in Figure 4.30. The crash frequency is not significant in this case.

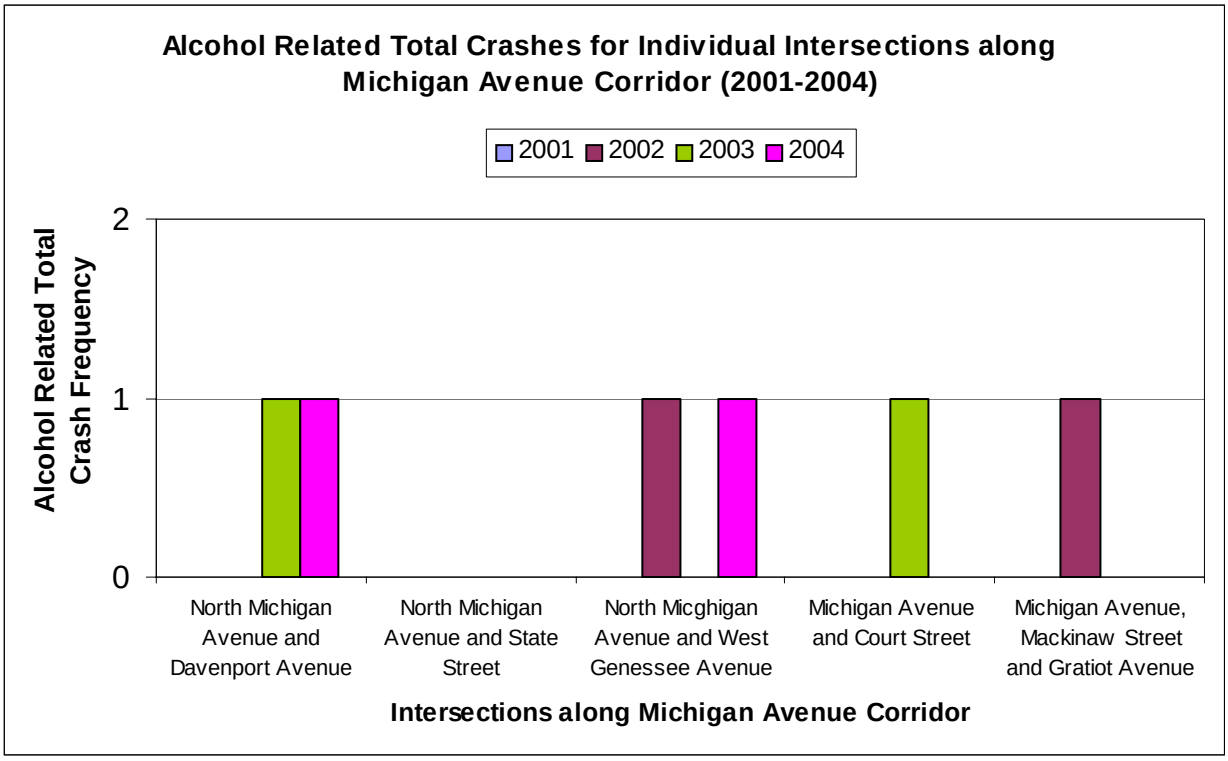


Figure 4.30 Alcohol Related Total Crashes along Michigan Avenue Corridor

Crashes involving drivers who were unbelted accounted for only one crash in the four-year period at the Michigan Avenue Corridor intersections, which is considered a low frequency.

A total of 13 speeding-related crashes occurred at the Michigan Corridor intersections during the four-year period, as shown in Figure 4.31. The intersections with the highest frequency of speeding-related crashes were North Michigan Avenue and Davenport Avenue (five crashes in a four-year period), and Michigan Avenue and Court Street (three crashes in a four-year period).

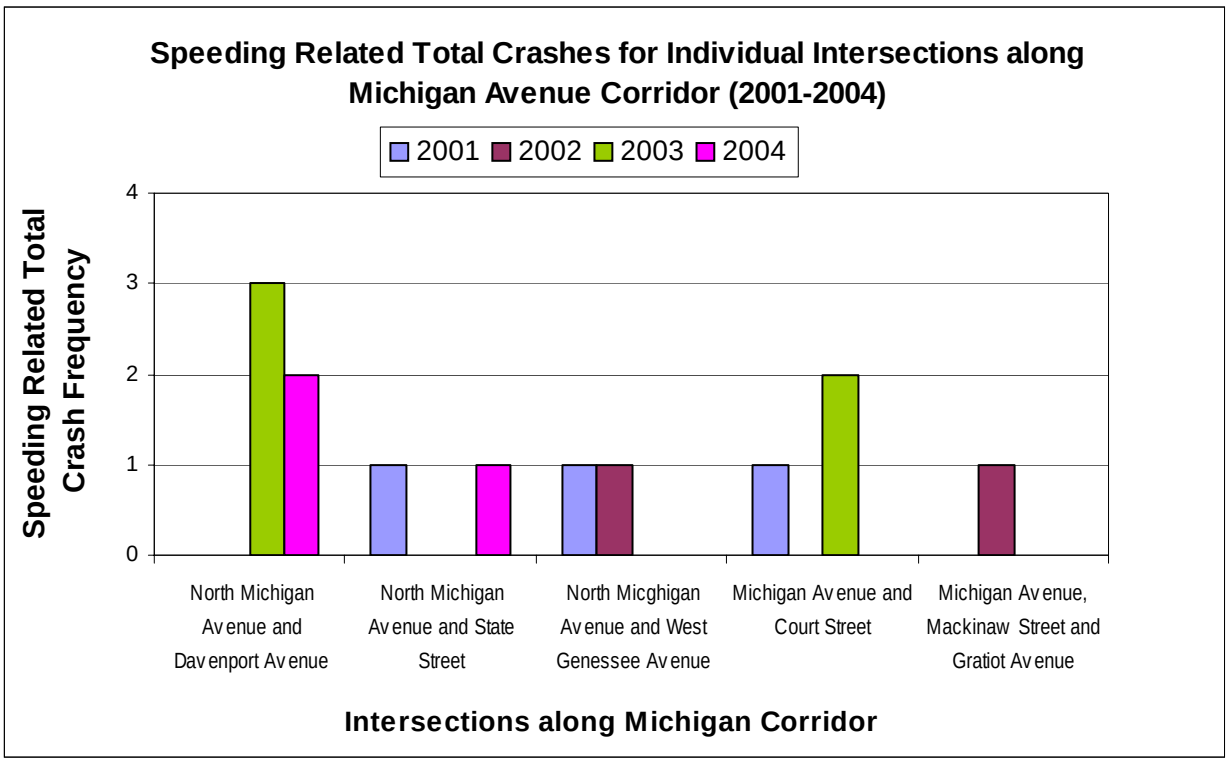


Figure 4.31 Speeding Related Total Crashes along the Michigan Avenue Corridor

A total of 64 red light running (RLR) crashes occurred along the Michigan Avenue Corridor during the four-year study period, as shown in Figure 4.32. The intersections with the highest frequency of RLR crashes were North Michigan Avenue and Davenport Avenue

Road with 18 crashes in a four-year period followed by North Michigan Avenue and State Street with 17 crashes in a four-year period.

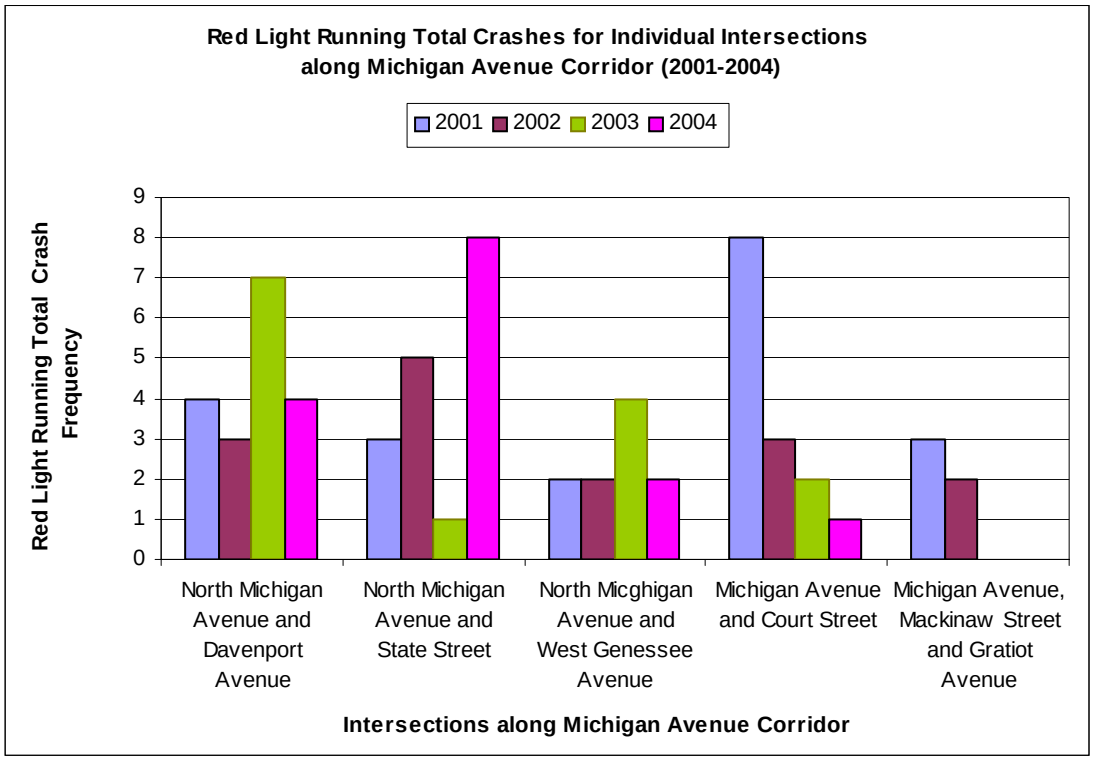


Figure 4.32 Red Light Running Total Crashes along Michigan Avenue Corridor

Overall, the predominant driver behavior problem for the Michigan Corridor is related to red light running and speeding at intersection approaches. It should be noted that the heavy red light running experience may be related to the signal timing design along Michigan Avenue, specifically the provision of improper length of the yellow and all-red interval, which is in addition to the potential aggressive nature of the drivers traveling along this corridor.

5.0 SUMMARY OF RECOMMENDATIONS

This section summarizes the engineering countermeasures suggested at the individual intersection level for all the 40 study intersections. Some of the improvements listed are immediate improvements that can be implemented including the changes in signal timing or phasing plan, lane usage or modifications and others. Some of the improvements involve physical improvements of the highway infrastructure to be implemented at a suitable time.

The location of the stop bars at the intersections is an important factor to prevent possible hazardous situations for motorists and pedestrians. To ensure the safety of both, the stop bars should be located at an appropriate distance from the nearest crosswalk or the edge of the curb. So, it is recommended to follow the Michigan Manual of Uniform Traffic Control Devices (MMUTCD) standards as close as possible. Although MMUTCD recommends 4-foot gap between crosswalk and stop bar, certain geometric conditions like turning radius, existing sidewalk features etc. should also be considered to make the improvements feasible.

5.1 Suggested Improvements for All Intersections:

1. **Bay Road and Kochville Road**

- Perform additional signal warrant analysis
- Introduce a two-phase semi-actuated signal with a skip phase and a cycle length of 90 seconds during the PM peak period.
- Provide a yellow interval of 5.0 seconds for the north-south phase and 4.7 seconds for the east-west phase.
- Provide all-red interval of 1.0 sec and 1.4 seconds for north south and east-west phase respectively.
- Upgrade the existing span-wire signal to a mast arm signal installation.
- Provide stop bars for north-south directional traffic.

- Provide a crosswalk in the east leg of the intersection across Kochville Road.
- Post a speed limit sign for the eastbound traffic downstream of the road.
- Provide stop signs for all the driveways located near the intersection. Individual landowner should provide these stop signs.
- Relocate the stop bar closer to intersection to provide more visibility.
- Optimize the signal timings for the AM peak and off-peak hours.

2. Bay Road and Bueker Drive

- Provide additional pavement markings on the northbound and southbound approaches.
- Provide a left-only marking on the eastbound left-turn lane of Bueker Drive.
- Provide a stop bar and crosswalk on the eastbound approach of Bueker Drive.
- Post a stop sign and stop bar on the driveway east of the intersection opposite to Bueker Drive.
- Post speed limit signs for the northbound and westbound traffic downstream of the intersection.

3. Bay Road (M-84) and Tittabawassee Road

- Revise the yellow interval for Bay Road to 4.7 seconds and for Tittabawassee Road to 4.3 seconds.
- Increase the all-red interval for Bay Road to 2.0 seconds and the all-red interval for Tittabawassee Road to 1.8 seconds.
- Provide a post-mounted signal for left-turn movements.
- Post speed limit signs for the southbound traffic downstream of the intersection.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Relocate the crosswalks on the northbound and southbound approaches of Bay Road with some modifications of sidewalk configuration on the southeast corner of the intersection.

- Relocate the stop bars on the northbound, southbound and eastbound approaches closer to the intersection.
- Provide stop signs for all the driveways located near the intersection. Individual landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours compatible with the progressive movement of traffic on both corridors.

4. Bay Road (M-84) and Schust Road

- Reduce the yellow interval to 3.9 seconds for Bay Road (M-84) and 4.3 seconds for Schust Road.
- Increase the all-red interval for Bay Road (M-84) to 1.7 seconds and for Schust Road to 2.2 seconds.
- Provide exclusive left turn phases for both directions and thereby revise the signal-timing plan with a four-phase operation and a 90-second cycle length, compatible with the progression along the Bay Corridor.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Provide a post-mounted signal for left-turning vehicles.
- Relocate the stop bars and crosswalks on all the approaches.
- Upgrade pavement markings on all approaches.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

5. Bay Road (M-84) and McCarty Road

- Reduce the yellow interval of Bay Road to 3.9 seconds and McCarty Road to 4.3 seconds.
- Increase the all-red interval of Bay Road to 1.5 seconds and McCarty Road to 2.0 seconds.
- Post speed limit signs downstream of the intersection for southbound traffic.

- Provide upgraded lane markings with proper directional arrows.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign on all the corners of the intersection for the left-turns.
- Relocate stop bars and crosswalks as per MMUTCD guidelines.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

6. Bay Road and Universal Drive

- Provide additional pavement markings with proper directional arrows for turning movements.
- Provide a pedestrian crosswalk on Universal Drive.
- Post speed limit signs downstream of the intersection for northbound and southbound traffic.
- Provide lane markings for universal drive.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

7. Bay Road (M-84) and Shattuck Road

- Reduce the yellow interval for Bay Road to 3.9 seconds and Shattuck Road to 3.6 seconds.
- Increase the all-red clearance interval for Bay Road to 1.8 seconds and Shattuck Road to 2.6 seconds.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at all the corners for left-turns.
- Replace the existing span-wire signal by box-span traffic signal installation.

- Relocate the stop bar and crosswalk as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel to guide the motorist to drive within the speed limit.
- Optimize the signal timings for the AM peak and off-peak hours. Provide a permissive protected left-turn phase instead of lead protected phases.
- Provide two sets of left only pavement markings on the westbound left-turn lane.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

8. Bay Road (M-84) and Weiss Street

- Reduce the yellow interval for Bay Road to 3.9 seconds and for Weiss Street to 3.2 seconds.
- Increase the all-red interval for the Bay Road to 1.7 seconds and for the Weiss Street to 2.2 seconds.
- Relocate the stop bar on the northbound, southbound and eastbound approaches, and the crosswalk on the northbound and eastbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for left-turns.
- Upgrade pavement markings for the northbound, southbound and eastbound approaches with proper directional arrows.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.
- As a part of the access management plan close, move or modify some driveways that have potential safety hazards.

9. Bay Street and Davenport Avenue

- Decrease the yellow interval for Bay Street and Davenport Avenue to 3.6 seconds.
- Increase the all-red interval for Bay Street to 1.7 seconds and for Davenport Avenue to 2.4 seconds.
- Relocate the stop bar on the southbound approach, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade pavement markings for the northbound and southbound approaches with proper directional arrows.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

10. Bay Street (M-84) and State Street (M-58)

- Reduce the yellow interval for Bay Street to 3.6 seconds and for State Street to 3.5 seconds.
- Increase the all-red interval for Bay Street to 1.4 seconds and for State Street to 2.1 seconds.
- Relocate the stop bar on the southbound and eastbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade pavement markings for the northbound, southbound and eastbound approaches with proper directional arrows.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

- Optimize the signal timings for the AM peak and off-peak hours.

11. Bay Street (M-84) and West Genesee Street

- Reduce the yellow interval for Bay Street to 3.6 seconds and for West Genesee Street to 3.5 seconds.
- Increase the all-red interval for Bay Street to 1.5 seconds and for West Genesee Street to 2.2 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Provide a modified three-phase signal system.
- Post speed limit signs downstream of the intersection for all directions of travel excluding eastbound.
- Provide a new stop bar and pedestrian crossing markings on the eastbound approach.
- Relocate stop bars on the southbound and westbound approaches and crosswalk on the westbound approach as per MMUTCD guidelines.
- Upgrade pavement markings for the southbound, eastbound and westbound approaches with proper directional arrows.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

12. Bay Street (M-84) and Congress Avenue

- Reduce the yellow interval for Bay Street and Congress Avenue to 3.5 seconds.
- Increase the all-red interval for Bay Street to 1.9 seconds and for Congress Avenue to 1.7 seconds.
- Relocate the stop bar on the all approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade pavement markings for the all approaches with proper directional arrows.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

- Optimize the signal timings for the AM peak and off-peak hours.

13. Bay Street (M-84) and Cooper Avenue

- Reduce the yellow interval for Bay Street and Cooper Avenue to 3.5 seconds.
- Increase the all-red interval for Bay Street to 1.6 seconds and for Cooper Avenue to 2.0 seconds.
- Relocate the stop bar on the northbound, southbound and westbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade pavement markings for the northbound, southbound and westbound approaches with proper directional arrows.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

14. Bay Street (M-84) and Court Street

- Reduce the yellow interval for Bay Street and Court Street phase to 3.5 seconds.
- Increase the all red interval for Bay Street to 2.5 seconds and for Court Street to 1.7 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade the existing pavement markings for eastbound, westbound and southbound approaches.
- Relocate the stop bar on the northbound, southbound and westbound approaches and crosswalk on the northbound approach, as per MMUTCD guidelines.
- Provide a modified three-phase signal system.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

- Optimize the signal timings for the AM peak and off-peak hours.

15. Tittabawassee Road and North Center Road

- Increase the yellow interval for Tittabawassee Road and North Centre Road to 4.3 seconds.
- Increase the all-red interval for Tittabawassee Road and North Centre Road to 1.5 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade pavement markings on all the approaches.
- Provide a modified three-phase signal system.
- Relocate stop bars on all the approaches, as per MMUTCD guidelines.
- Optimize the signal timings for the AM peak and off-peak hours

16. Tittabawassee Road and Mackinaw Road

- Increase the yellow interval for Tittabawassee Road to 4.3 seconds and for the Mackinaw Road to 4.7 seconds.
- Increase the all-red interval for Tittabawassee Road to 1.3 seconds and Mackinaw Road to 1.6 seconds.
- Eliminate the exclusive left-turn phase for the north-south and east-west movement.
- Provide a modified three-phase signal system.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade the existing pavement markings with proper directional arrows on all the approaches.
- Relocate the stop bars and crosswalks on the southbound and eastbound approaches, as per MMUTCD guidelines.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

- Optimize the signal timings for the AM peak and off-peak hours.

17. Tittabawassee Road and Fortune Boulevard

- Increase the yellow interval for Tittabawassee Road to 4.3 seconds and reduce it to 3.5 seconds for fortune Boulevard.
- Increase the all red interval for Tittabawassee Road to 1.3 seconds and Fortune Boulevard to 3.6 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade the existing pavement markings with proper directional arrows for the all approaches.
- Relocate the stop bar on the all approaches and crosswalks on the southbound, eastbound and westbound approaches, as per MMUTCD guidelines.
- Install post-mounted traffic signals with a "LEFT" illuminated sign at the northeast and southwest corners of the intersection for north and southbound left-turns.

18. Tittabawassee Road and Fashion Square Boulevard

- Increase the yellow interval for Tittabawassee Road to 4.3 seconds and reduce it to 3.6 seconds for Fashion Square Boulevard.
- Increase the all-red interval for Tittabawassee Road to 1.4 seconds and for Fashion Square Boulevard to 2.0 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade the existing pavement markings with proper directional arrows for all the approaches.
- Relocate the crosswalk on all the approaches and stop bar for the northbound, southbound and eastbound approaches, as per MMUTCD guidelines.
- Install post-mounted traffic signals with a "LEFT" illuminated sign at all the corners of the intersection for left-turns.

- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

19. Tittabawassee Road and Berl Drive

Stage-I:

- Upgrade pavement markings on the westbound approach.
- Provide stop bars at the northbound and southbound approaches provide a crosswalk on northbound approach as per MMUTCD guidelines.
- Realign sidewalk at the south of the intersection as shown in Figure 6.19.6.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

Stage-II:

- Provide a raised median of 15 feet wide.
- Upgrade pavement markings on the all approaches.
- Realign the sidewalk at the south of the intersection as shown in Figure 6.19.7.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

20. Tittabawassee Road and Towne Center Road

- Increase the yellow interval for Tittabawassee Road and Towne Centre Road to 4.3 seconds.

- Increase the all-red interval for Tittabawassee Road to 1.3 seconds and for Towne Centre to 1.8 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade the existing pavement markings for all the approaches.
- Relocate the stop bars and crosswalks on the northbound, eastbound and westbound approaches, as per MMUTCD guidelines.
- Provide new stop bar, centerline and pavement markings on the northern leg of the intersection.
- Install post-mounted traffic signals with a "LEFT" illuminated sign at the all corners of the intersection for left-turns.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

21. Tittabawassee Road and I 675 SB Ramp

- Increase the yellow interval for Tittabawassee Road to 4.3 seconds and reduce it to 3.5 seconds for the SB I-675 Ramp.
- Reduce the all-red interval for Tittabawassee Road to 1.0 second.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Relocate stop bars at the southbound, eastbound and westbound approaches, as per MMUTCD guidelines.
- Provide upgraded pavement markings with proper directional arrows for the southbound and eastbound approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

- Optimize the signal timings for the AM peak and off-peak hours.

22. Tittabawassee Road and NB I-675 Ramp

- Increase the yellow interval to 4.5 seconds for all phases of Tittabawassee Road and reduce it to 3.5 seconds for movements along NB I-675.
- Increase the all-red interval for Tittabawassee Road to 1.5 seconds and to 2.5 seconds for NB-I675.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Relocate the stop bars at the northbound, westbound and eastbound approaches, as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide upgraded pavement markings with proper directional arrows for the all approaches.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

23. Tittabawassee Road and North Michigan Avenue

- Increase the yellow interval for Tittabawassee Road and North Michigan Avenue to 4.3 seconds.
- Increase the all-red interval for Tittabawassee Road to 1.4 seconds and for North Michigan Avenue to 1.5 seconds.
- Upgrade pavement markings with proper directional arrows on all the approaches.
- Provide a modified three-phase signal system.
- Relocate stop bars on eastbound and westbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.

- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

24. Gratiot Road (M-46) and Graham Road (M-52)

- Increase the yellow interval for Gratiot Road to 5.0 seconds
- Increase the all-red interval for Gratiot Road to 1.2 seconds and for Graham Road to 1.5 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade pavement markings on all the approaches.
- Relocate the stop bars at all four approaches, as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

25. Gratiot Road (M-46) and Kennely Road

- Introduce a two-phase semi-actuated signal with a skip phase and a base cycle length of 90 seconds during the PM peak period.
- Provide mast-arm supported signal heads.
- Set the yellow interval of 4.3 seconds for Gratiot Road and 3.6 seconds for Kennely Road.
- Provide an all-red interval of 1.3 seconds for Gratiot Road and 1.8 seconds for Kennely Road.
- Provide Stop Bars for eastbound and westbound directional traffic.

- Relocate the stop bar closer to the intersection for southbound traffic to provide more visibility.
- Provide a centerline and exclusive left-turn lane for southbound traffic.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs
- Upgrade pavement markings for all approaches.

26. Gratiot Road (M-46) and Van Wormer Road

- Install an advance warning sign indicating "Stop Ahead" along Van Wormer Road.
- Provide upgraded pavement markings with proper directional arrows for all the approaches.
- Provide a stop bar, crosswalk and a centerline for the northbound approach. Relocate the stop bar and provide a crosswalk for the southbound Home Depot driveway, as per MMUTCD guidelines.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

27. Gratiot Road (M-46) and Miller Road

- Reduce the yellow interval for Gratiot Road to 4.3 seconds and Miller Road to 3.9 seconds.
- Increase the all-red interval for Miller Road to 1.9 seconds and reduce it to 1.4 seconds for Gratiot Road.
- Introduce the exclusive left-turn phase for the east-west movement.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Relocate the stop bars and crosswalk at all the approaches, as per MMUTCD guidelines.

- Provide upgraded pavement markings with proper directional arrows for the all approaches.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for east and westbound left-turns.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

28. Gratiot Road (M-46) and River Road

- Reduce the yellow interval for Gratiot Road and River Road to 4.3 seconds.
- Reduce the all-red interval for Gratiot Road to 1.4 seconds.
- Provide a modified three-phase signal system with the inclusion of an exclusive left-turn phase for the east west Gratiot Road movements.
- Relocate stop bars and crosswalks at all the approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for east and westbound left turning vehicles.
- Upgrade pavement markings with proper directional arrows for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

29. Gratiot Road (M- 46) and Center Road

- Reduce the yellow interval for Gratiot Road to 4.3 seconds and Center Road to 3.9 seconds.
- Increase the all-red interval for the Gratiot Road phase to 1.6 seconds and for the Center Road phase to 1.9 seconds.
- Relocate stop bars and crosswalks on the northbound, southbound and eastbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Install post-mounted traffic signals with a "LEFT" illuminated sign for all corners of the intersection to assist left turning vehicles.
- Upgrade pavement markings with proper directional arrows for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

30. Gratiot Avenue (M-46) and Wheeler Street

- Increase the yellow interval for Gratiot Avenue to 3.9 seconds and reduce it for Wheeler Street to 3.2 seconds.
- Reduce the all-red interval for Gratiot Avenue to 1.5 seconds and 2.2 seconds for Wheeler Street.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Relocate stop bars at the eastbound and westbound approaches, as per MMUTCD guidelines.

- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

31. Davenport Avenue and North Michigan Avenue

- Reduce the yellow interval for Davenport Avenue and North Michigan Avenue to 3.5 seconds.
- Increase the all-red interval for Davenport Avenue to 1.4 seconds and North Michigan Avenue to 2.2 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Relocate stop bars and crosswalks on the westbound and northbound approaches, as per MMUTCD guidelines.
- Upgrade pavement markings on all the approaches
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

32. North Michigan Avenue and State Street (M-58)

- Reduce the yellow interval for N. Michigan Avenue and State Street to 3.5 seconds.
- Increase the all-red interval for State Street to 1.6 seconds and for N. Michigan Avenue to 1.5 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Relocate stop bars on the northbound, southbound and eastbound approaches, as per MMUTCD guidelines.
- Upgrade pavement markings on the southbound and eastbound approaches.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

33. North Michigan Avenue and West Genesee Avenue

- Reduce the yellow interval for North Michigan Avenue and West Genesee Avenue to 3.5 seconds.
- Increase the all-red interval for West Genesee Avenue to 1.7 seconds and for North Michigan Avenue to 2.5 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Change lane width for eastbound and westbound approaches as shown in proposed diagram in figure 6.33.6.
- Relocate stop bars and crosswalks on the all approaches, as per MMUTCD guidelines.
- Upgrade pavement markings on the all approaches.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

34. North Michigan Avenue and Court Street

- Increase the all-red interval for North Michigan Avenue and Court Street to 2.5 seconds.

- Relocate the stop bars and crosswalks on all the approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Provide a modified three-phase signal system.
- Install post-mounted traffic signals with a "LEFT" illuminated sign at the northeast and southwest corners of the intersection for the left-turn movements.
- Upgrade pavement markings for the all approaches with proper directional arrows.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

35. Mackinaw Street, South Michigan Avenue and Gratiot Avenue

- Reduce the yellow interval to 3.5 seconds for all phases for North Michigan Avenue, Mackinaw Street and Gratiot Avenue.
- Reduce the all-red interval for Michigan Avenue to 1.7 seconds and increase it to 2.5 seconds for Mackinaw Street and Gratiot Avenue.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Relocate the stop bars at the northwest and eastbound approaches, as per MMUTCD guidelines.
- Provide upgraded pavement markings for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

36. East Holland Avenue (M-46) and South Outer Drive

- Reduce the yellow interval for East Holland Avenue to 4.3 seconds and reduce it for South Outer Drive to 3.5 seconds.
- Introduce the all-red interval for East Holland Avenue to 1.5 seconds and for South Outer Drive to 2.0 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade the existing pavement markings for all the approaches.
- Relocate the stop bars and crosswalks on all the approaches, as per MMUTCD guidelines.
- Install post-mounted traffic signals with a "LEFT" illuminated sign at the northeast and southwest corners of the intersection for east and westbound left-turns.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Optimize the signal timings for the AM peak and off-peak hours.

37. East Holland Avenue and Cumberland Street

- Reduce the yellow interval for Holland Avenue to 3.6 seconds and Cumberland Street to 3.5 seconds.
- Introduce the all-red interval for Holland Avenue to 1.8 seconds and Cumberland Street to 2.0 seconds.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Relocate stop bars and provide new crosswalks at all the approaches, as per MMUTCD guidelines.
- Provide upgraded pavement markings for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

38. Rust Avenue (M-46) and South Washington Avenue (M-13)

- Reduce the yellow interval for Rust Avenue and South Washington Avenue to 3.6 seconds.
- Increase the all-red interval for Rust Avenue to 1.9 seconds and for the South Washington Avenue to 2 seconds.
- Relocate stop bars at all the approaches and crosswalk for the eastbound approach as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Upgrade pavement markings for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

39. West Freeland Road and Midland Road (M-47)

- Reduce the yellow interval for West Freeland Road to 3.9 seconds and for Midland Road to 4.3 seconds.
- Increase the all-red interval for West Freeland Road to 2.1 seconds and Midland Road to 2.0 seconds.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at all corners of the intersection for left-turning vehicles.
- Upgrade pavement markings for all the approaches.

- Relocate stop bars and crosswalks on the northbound and southbound approach, as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

40. South Graham Road (M- 52) and Swan Creek Road

- Increase the yellow interval for Swan Creek Road and for South Graham Road to 5.0 seconds.
- Increase the all-red interval for Swan Creek Road to 1.2 seconds.
- Provide an excusive left turn phase for east west Swan Creek Road movement.
- Relocate stop bars and at all the approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for north and southbound left-turning vehicle.
- Upgrade pavement markings for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.
- Optimize the signal timings for the AM peak and off-peak hours.

6.0 DETAILED INTERSECTION ANALYSIS

The selected study intersections are investigated individually and extensively to determine the probable causal factors for the crashes and to suggest appropriate countermeasures. The detailed analyses of the 40 study intersections include a critical review of the existing physical and traffic characteristics of the roadway including Average Daily Traffic volumes (ADT) based on data obtained from the Michigan Department of Transportation, and peak hour traffic data collected by the WSU staff as a part of this study. In addition, detailed assessments on traffic control measures, crash analysis, safety deficiencies, recommended improvements, and operational evaluations are provided for each of the study intersections.

Physical Characteristics

As a part of this study, a field survey was conducted by the WSU staff for each of the study intersections. The field survey included visiting the study intersections, drawing condition diagrams, taking photographs in order to capture existing lane use and other potential physical characteristics in the vicinity of the intersections, and assessing the existing traffic control devices. The computerized condition diagrams and photographs of the intersection approaches are provided later in this section for all the study intersections.

Traffic Volumes

PM peak period traffic counts were collected for each intersection from 4:00 PM to 6:00 PM on typical weekdays between January of 2006 and April of 2006. An analysis of daily traffic volumes at key locations indicated that the highest traffic volumes occur during the PM peak period. Traffic volume diagrams, including the PM peak hour turning movement counts, have been prepared for each intersection and are provided later in this section.

Crash Analysis

Two years of site-specific traffic crash data (2003 and 2004) were analyzed as a part of this study to identify safety deficiencies and to develop countermeasures. Crash information was derived from the UD-10 crash reports obtained from the Michigan State Police database. Crash data summaries and collision diagrams were prepared for each intersection for the two-year study period. Collision diagrams for each year, 2003 and 2004, are provided in this section for each of the intersections.

The collision diagrams were analyzed to determine predominant crash patterns for each intersection, supported by figures indicating the same. Crash types are indicative of inadequacies and specific deficiencies of safety measures at individual intersections. Rear-end crashes at a signalized intersection are generally related to excessive speeds, poor visibility of signal heads, signal timing deficiencies, excessive pedestrian crossings, unexpected stopping of vehicles, and slippery surfaces on the roadways. Right-angle crashes at a signalized intersection are generally related to an inadequate clearance interval, poor visibility of signal heads, unexpected stopping of vehicles and lack of driver information. Left-turn head-on crashes are generally related to excessive speed, an inadequate clearance interval, inadequate gaps in the opposing traffic stream and inadequate signal timing for left turning movements. Sideswipe crashes are generally related to excessive speed, unexpected lane changing, low visibility of signal heads and inadequate pavement markings.

Safety Deficiencies and Recommended Improvements

Based on the crash analysis, safety deficiencies for each of the selected 40 intersections have been identified and suggested improvements have been recommended as well. The recommended improvements include engineering measures and behavioral measures such as public awareness campaigns, enforcement and education strategies. A summary of some of the countermeasures recommended as a part of this study include the following:

Engineering Countermeasures

- Signal timing and phasing modifications including:
  - Modifying cycle lengths along the major corridors of Bay Road, Tittabawassee Road, Gratiot Road and Michigan Avenue to be consistent and the same in order to facilitate signal coordination and progression. Currently, the intersections along these corridors operate at various cycle lengths.
  - Calculating clearance intervals (yellow and all-red intervals) according to MDOT's revised guidelines and the Institute of Transportation Engineers (ITE) recommended practice. In many instances the yellow intervals included in the existing signal timing plans exceeded the values recommended by ITE and MDOT, while all-red intervals were either excluded from the existing timings or were too short. Excessive yellow intervals may encourage some motorists to proceed through the intersection, even if they had sufficient time and distance to stop comfortably. Rear-end crashes often result due to the unexpected stopping of some motorists, while others intend to travel through the intersection during the signal

change interval. The lack of or insufficient length of all-red intervals in traffic signal timing may not provide enough time for drivers to clear the intersection before opposing traffic receives a green signal indication. All-red intervals should be provided and should have a duration, which allows vehicles to traverse through the intersection safely. The lack of all-red intervals may lead to a high frequency of left-turn head-on, angle and injury crashes.

- Providing exclusive left-turn phases at locations with high left-turn volumes and a predominance of left-turn head-on crashes.
- Providing protected-only left-turn phasing at locations where an existing permitted/protected left-turn phase exists, but still experiences a high number of left-turn head-on crashes.

- Geometric modifications including the addition of turning lanes and receiving lanes.
- Signage modifications including the installation of overhead lane designation signs to reduce driver confusion, particularly for the significant number of tourists/recreational travelers.
- Installation of traffic signs including 'No Turn On Red' signs, speed limit signs at the downstream of the intersections, and others where appropriate.
- Routine maintenance of pavement markings.
- Relocation of stop bars and crosswalks closer to the intersection, as per MMUTCD guidelines.
- Improving safety and traffic flow on roads that have angle on-street parking by reducing the posted speed limit, modifying on-street angle parking to parallel parking, or providing a raised median barrier to separate parking traffic from through traffic.
- Construction of raised islands in the center lane for left turns to mitigate driveway conflicts in close proximity of the intersection.

Operational Evaluation

The Level of service (LOS) analyses for the existing and improved traffic conditions for the PM peak hour were performed for the 40 intersections. According to the most recent edition of the Highway Capacity Manual, level of service is a qualitative measure describing operational conditions of a traffic stream or intersection. Level of service ranges from A to F, with LOS A being the best. LOS D is generally considered to be acceptable. Table 6.1 and Table 6.2 present the criteria for defining the various levels of service for unsignalized and signalized intersections.

Table 6.1. Level of Service Criteria (Unsignalized Intersection)

Level of Service	Average Stopped Vehicle Delay (seconds)
A	≤10
B	>10 and ≤ 15
C	>15 and ≤ 25
D	>25 and ≤ 35
E	>35 and ≤ 50
F	> 50

Note: LOS "D" is considered acceptable in urban/suburban areas.

Table 6.2. Level of Service Criteria (Signalized Intersection)

Level of Service	Average Stopped Delay/Vehicle (seconds)
A	≤10
B	>10 and ≤20
C	>20 and ≤35
D	>35 and ≤55
E	>55 and ≤80
F	>80

Note: LOS "D" is considered acceptable in urban/suburban areas.

Behavioral Countermeasures

In general, the crashes involving driver behavior issues have a relatively low frequency for a given intersection during a given year. General engineering countermeasures are not applicable to mitigate these crashes. However, critical locations should be selected on the basis of historical crash data pertaining to driver behavioral issues. These locations may be subjected to strict observation and enforcement. Also, local educational institutions may be approached to organize public awareness campaigns providing general information about the consequences of drinking and driving, speeding, red-light running and inadequate use of safety belts. Mass awareness is the most effective way to counteract crashes involving improper driving actions or driver behavior.

A crash analysis was performed for the 40 study intersections to quantify the number of crashes involving driver behavior problems, such as driving under the influence of alcohol, drivers not wearing their seatbelts and crashes involving speeding and/or red light running. The crash frequency information was obtained by querying the MSP's Statewide Traffic Crash Database for the years 2001, 2002, 2003 and 2004, as shown in Table 6.3.

From the percentage of crashes related to driver behavioral issues, as obtained from Table 6.3, the critical intersections are selected. Depending upon the specific type of driver behavior (aggressive driving, driving under the influence of alcohol or improper seat-belt usage) observed at these intersections, appropriate countermeasures are suggested. Along with other countermeasures, mass awareness would be the most effective way to communicate the consequences of driver behavioral issues and to mitigate the resulting crashes.

Table 6.3. Crashes Related to Driver Behavioral Issues

Name of Corridors	Intersection Name	Total Crashes for Four Years of Data Combined (2001-2004)			
		Alcohol-Related Crashes	Crashes Involving Unbelted Drivers	Speeding Related Crashes	Red Light Running Crashes
Bay Road	Bay Road & Kochville Road	2	1	3	2
	Bay Road & Bueker Drive	0	0	2	0
	Bay Road & Tittabawassee Road	5	3	11	7
	Bay Road & Schust Road	3	1	2	10
	Bay Road & McCarty Road	2	0	5	4
	Bay Road & Universal Drive	0	0	2	0
	Bay Road & Shattuck Road	4	1	6	5
	Bay Road & Weiss Road	1	1	1	3
	Bay Street & Davenport Avenue	8	0	1	27
	Bay Street & State Street	6	0	0	15
	Bay Street & West Genesee Street	2	1	0	8
	Bay Street & Congress Avenue	1	2	1	9
	Bay Street & Cooper Avenue	0	0	0	3
	Bay Street & Court Street	0	0	0	4
Tittabawassee Road	Tittabawassee Road & N. Center Road	1	2	12	11
	Tittabawassee Road & Mackinaw Road	5	0	9	16
	Tittabawassee Road & Fortune Boulevard	2	0	0	2
	Tittabawassee Road & Fashion Square Blvd	0	0	6	2
	Tittabawassee Road & Berl Drive	0	1	0	0
	Tittabawassee Road & Towne Center Road	4	0	4	1
	Tittabawassee Road & NB I 675	1	2	6	0
	Tittabawassee Road & SB I 675	0	0	14	0
	Tittabawassee Road & N. Michigan Ave	1	0	3	3

Name of Corridors	Intersection Name	Total Crashes for Four Years of Data Combined (2001-2004)			
		Alcohol-Related Crashes	Crashes Involving Unbelted Drivers	Speeding Crashes	Red Light Running Crashes
Gratiot Road	Gratiot Road & Graham Road	4	0	9	9
	Gratiot Road & Kennely Road	3	0	3	0
	Gratiot Road & Van Wormer Road	0	0	1	0
	Gratiot Road & Miller Road	4	0	4	5
	Gratiot Road & River Road	4	1	6	14
	Gratiot Road & Center Road	3	1	14	16
	Gratiot Road & Wheeler Road	2	0	0	9
Michigan Avenue	North Michigan Ave & Davenport Ave	2	0	5	18
	North Michigan Ave & State Street	0	0	2	17
	North Michigan Ave & West Genesee Ave	2	0	2	10
	Michigan Ave & Court Street	1	0	3	14
Other Intersections	Michigan Ave, Mackinaw St & Gratiot Ave	1	1	1	5
	East Holland Avenue & South Outer Drive	6	1	5	5
	East Holland Avenue & Cumberland Street	3	1	0	2
	Rust Avenue & Washington Avenue	2	0	2	55
	West Freeland Road & Midland Road	1	1	2	3
	South Graham Road & Swan Creek Road	3	0	7	1
	TOTALS	**89**	**21**	**154**	**315**

Alcohol-Related Crashes

A total of 89 crashes occurred at the 40 study intersections involving a driver who was driving under the influence of alcohol during the four-year study period. The following study intersections experienced four or more alcohol-related crashes in the four-year analysis period and would be candidate locations for **enforcement programs**:

- Bay Road & Tittabawassee Road (Five crashes in a four year analysis period)
- Bay Road & Shattuck Road (Four crashes in a four year analysis period)
- Bay Street & Davenport Avenue (Eight crashes in a four year analysis period)
- Bay Street & State Street (Six crashes in a four year analysis period)
- Tittabawassee Road & Mackinaw Street (Six crashes in a four year analysis period)
- Tittabawassee Road & Towne Center Road (Four crashes in a four year analysis period)
- Gratiot Road & Graham Road (Four crashes in a four year analysis period)
- Gratiot Road & Miller Road (Four crashes in a four year analysis period)
- Gratiot Road & River Road (Four crashes in a four year analysis period)
- East Holland Avenue & South Outer Drive (Six crashes in a four year analysis period)

Unbelted Crashes

A total of 21 crashes occurred at the 40 study intersections during the four-year study period that involved unbelted drivers, which is a moderate frequency. Thus, no specific countermeasures are recommended at the study intersections related to seat belt use.

It should be noted that the number of crashes involving drivers who were not wearing their seat belt at the time of the crash has reduced considerably in Saginaw County over the past few years. As shown in Section 2.5, Figure 2.11, there were a total of 112 crashes in the entire county involving unbelted drivers in the year 2004, while in 1999 this crash frequency was 260, a reduction of 57%. This reduction in crashes may be attributed to seatbelt awareness campaigns and enforcement programs conducted in the area, which have received heightened publicity in recent years. It is recommended that the public awareness campaigns and enforcement programs be continued in the area.

Speeding-Related Crashes

A total of 154 speeding-related crashes occurred at the 40 study intersections during the four-year study period. The following sixteen study intersections experienced four or more speeding-related crashes in the four-year analysis period and would be candidate locations for **enforcement programs**:

- Bay Road & Tittabawassee Road (Eleven crashes in a four year analysis period)
- Bay Road & McCarty Road (Five crashes in a four year analysis period)
- Bay Road & Shattuck Road (Six crashes in a four year analysis period)
- Tittabawassee Road & North Center Road (Twelve crashes in a four year analysis period)
- Tittabawassee Road & Mackinaw Road (Nine crashes in a four year analysis period)
- Tittabawassee Road & Fashion Square Boulevard (Six crashes in a four year analysis period)
- Tittabawassee Road & Towne Center Road (Four crashes in a four year analysis period)
- Tittabawassee Road & NB I 675 (Six crashes in a four year analysis period)
- Tittabawassee Road & SB I 675 (Fourteen crashes in a four year analysis period)
- Gratiot Road & Graham Road (Nine crashes in a four year analysis period)

- Gratiot Road & Miller Road (Four crashes in a four year analysis period)
- Gratiot Road & River Road (Six crashes in a four year analysis period)
- Gratiot Road & Center Road (Fourteen crashes in a four year analysis period)
- North Michigan Avenue & Davenport Avenue (Five crashes in a four year analysis period)
- East Holland Avenue & South Outer Drive (Five crashes in a four year analysis period)
- South Graham Road & Swan Creek Road (Seven crashes in a four year analysis period)

Midblock speed studies were conducted as a part of this study on Bay Road, Tittabawassee Road, Gratiot Road, and Michigan Avenue to verify the magnitude of speeding problems on these roads. The speed studies along all these corridors were done during off-peak hours to capture the free flow speed of traffic. Midblock locations were selected between two intersections on the corridor such that the moving traffic is free from any turning or stopping. The results of the speed studies are shown in Table 6.4. The observed data indicates that some of portion of the major corridors have a posted speed limit higher than the 85th percentile speed. For the Bay Road Corridor, the difference between the 85th % Speed and posted speed limit is about 8 mph and 2mph and for the Tittabawassee Road Corridor, it is about 9 mph in some places. This difference signifies that the vehicles are traveling at a lower speed than the posted limit. One of the reasons for this phenomenon may be the presence of highly commercial areas on both sides of the corridors for a considerable distance. Heavy turning movement to and from these commercial establishments through the existing driveways is a major cause for queuing up and slow movement of through traffic. Especially, at intersections where exclusive turning lanes are not provided, the through traffic faces longer delays when the turning volume is high. After analyzing the results of the observed speed studies, it is highly recommended to bring down the posted speed limit.

Table 6.4. Results of Midblock Speed Studies at Selected Locations

Corridor	Midblock Location	Speed Limit (mph)	Mean Speed (mph)	Standard Deviation	85th Percentile Speed (mph)
Bay Road	5275 Bay Road	50mph	36mph	5.07	42mph
	5300 Bay Road	50mph	35mph	5.07	41mph
	3783 Bay Road	40mph	33mph	4.87	38mph
	3750 Bay Road	40mph	35mph	4.48	39mph
Tittabawassee Road	4430 Tittabawassee Rd	45mph	49mph	4.02	52mph
	4241 Tittabawassee Rd	45mph	46mph	4.68	51mph
	2235 Tittabawassee Rd	45mph	30mph	3.65	34mph
	2627 Tittabawassee Rd	45mph	31mph	3.29	36mph
Gratiot Road	Gratiot Rd & Graham Rd	55 mph	57mph	4.47	62mph
	9535 Gratiot Road	55mph	58mph	4.42	62mph
	5355 Gratiot Road	45mph	43mph	4.07	46mph
	5300 Gratiot Road	45mph	37mph	4.02	41mph
Michigan Avenue	517 Michigan Avenue	30mph	35mph	4.50	39mph
	515 Michigan Avenue	30mph	33mph	4.72	38mph
	1721 Michigan Avenue	30mph	30mph	5.95	37mph
	1740 Michigan Avenue	30mph	34mph	3.87	39mph

Red Light Running Crashes

A total of 315 crashes occurred at the 40 study intersections during the four-year study period, which were related to red light running. The following twenty-three intersections experienced four or more red light running crashes in the four-year analysis period:

- Bay Road & Tittabawassee Road (Seven crashes in a four year analysis period)
- Bay Road & Schust Road (Ten crashes in a four year analysis period)
- Bay Road & McCarty Road (Four crashes in a four year analysis period)
- Bay Road & Shattuck Road (Five crashes in a four year analysis period)
- Bay Street & Davenport Avenue (Twenty seven crashes in a four year analysis period)
- Bay Street & State Street (Fifteen in a four year analysis period)
- Bay Street & West Genesee Street (Eight crashes in a four year analysis period)
- Bay Street & Congress Avenue (Nine crashes in a four year analysis period)

- Bay Street & Court Street (Four crashes in a four year analysis period)
- Tittabawassee Road & North Center Road (Eleven crashes in a four year analysis period)
- Tittabawassee Road & Mackinaw Road (Sixteen crashes in a four year analysis period)
- Gratiot Road & Graham Road (Nine crashes in a four year analysis period)
- Gratiot Road & Miller Road (Five crashes in a four year analysis period)
- Gratiot Road & River Road (Fourteen crashes in a four year analysis period)
- Gratiot Road & Center Road (Sixteen crashes in a four year analysis period)
- Gratiot Road & Wheeler Road (Nine crashes in a four year analysis period)
- North Michigan Avenue & Davenport Avenue (Eighteen crashes in a four year analysis period)
- North Michigan Avenue & State Street (Seventeen crashes in a four year analysis period)
- North Michigan Avenue & West Genesee Avenue (Ten crashes in a four year period)
- Michigan Avenue & Court Street (Fourteen crashes in a four year period)
- Michigan Avenue, Mackinaw Street & Gratiot Avenue (Five crashes in a four year period)
- East Holland Avenue & South Outer Drive (Five crashes in a four year period)
- Rust Avenue & Washington Avenue (Fifty five crashes in a four year period)

The recommended engineering countermeasures (modifying the traffic signals' yellow and all-red intervals) are expected to alleviate some of the red light running caused by drivers, who did not intend to run the red light, but might have been caught in a 'dilemma zone' when approaching the intersection during the yellow light. In order to alleviate problems with red light running due to aggressive driving behavior, it is recommended to enact increased enforcement at the critical locations as marked above. It should be noted that it might be risky for police officers to cite offenders for red light running, based on traditional enforcement techniques, as they must follow them through the red light. However, there are some new technologies, which are currently being tested that allow police officers to safely observe red light runners downstream of the intersection.

Another method to discourage red light running would be to participate in a national **public awareness campaign**. The Federal Highway Administration sponsors an annual event entitled "Stop on Red" Week to discourage such behavior. More information can be found on the Internet at http://safety.fhwa.dot.gov/fourthlevel/pro_res_srlr_week.htm. This type of campaign could impact red light running throughout the entire county.

1. Bay Road and Kochville Road

The intersection of Bay Road (M-84) and Kochville Road is located in the central part Kochvile Township. Kochville Road runs in the east-west direction and Bay Road runs in the north-south direction. Bay Road is a state trunk line Road (M-84).

Bay Road has an average daily traffic volume of approximately 20,000 vpd, as per MDOT 2004 data.

Bay Road is 61 feet wide on both the north and south leg of the intersection with Kochville Road. Bay Road is a five-lane road with two lanes in each direction and a center left-turn lane. Kochville Road is 39 feet wide east of the intersection with one exclusive left-turn and one through lane in each direction. On the west side of the intersection Kochville Road is 38 feet wide having a similar lane configuration.

The speed limit on Bay Road (M-84) is 55 miles per hour north of Kochville Road and 50 miles per hour for southbound traffic. Kochville Road has a speed limit of 50 mph.

Land Use

Kochville Road primarily functions as a collector road as well as a link between the two state trunk line roads, Bay Road (M-84) and Midland Road (M-47). On the northeast corner of the intersection there is Kochville United Methodist Church and on the northwest corner there is a piece of empty land for sale. The Southwest and southeast corners have a cemetery and a Bird's Nest shop, respectively. There is only one driveway on the east leg of Kochville Road within 150 ft of the intersection. The driveway is 42 feet wide.

Field Data

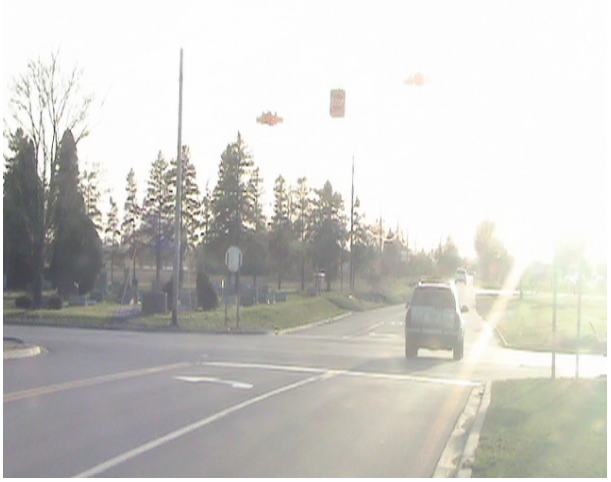
Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.1.1. Details of the intersection approaches are shown in photographs 1a through 1d. Photographs shown below depict the flashing beacon hanging from the span wire installation.

Photograph 1a. NB Bay Road at Kochville Road

Photograph 1b. SB Bay Road at Kochville Road

Photograph 1c. EB Kochville Road at Bay Road



Photograph 1d. WB Kochville Road at Bay Road

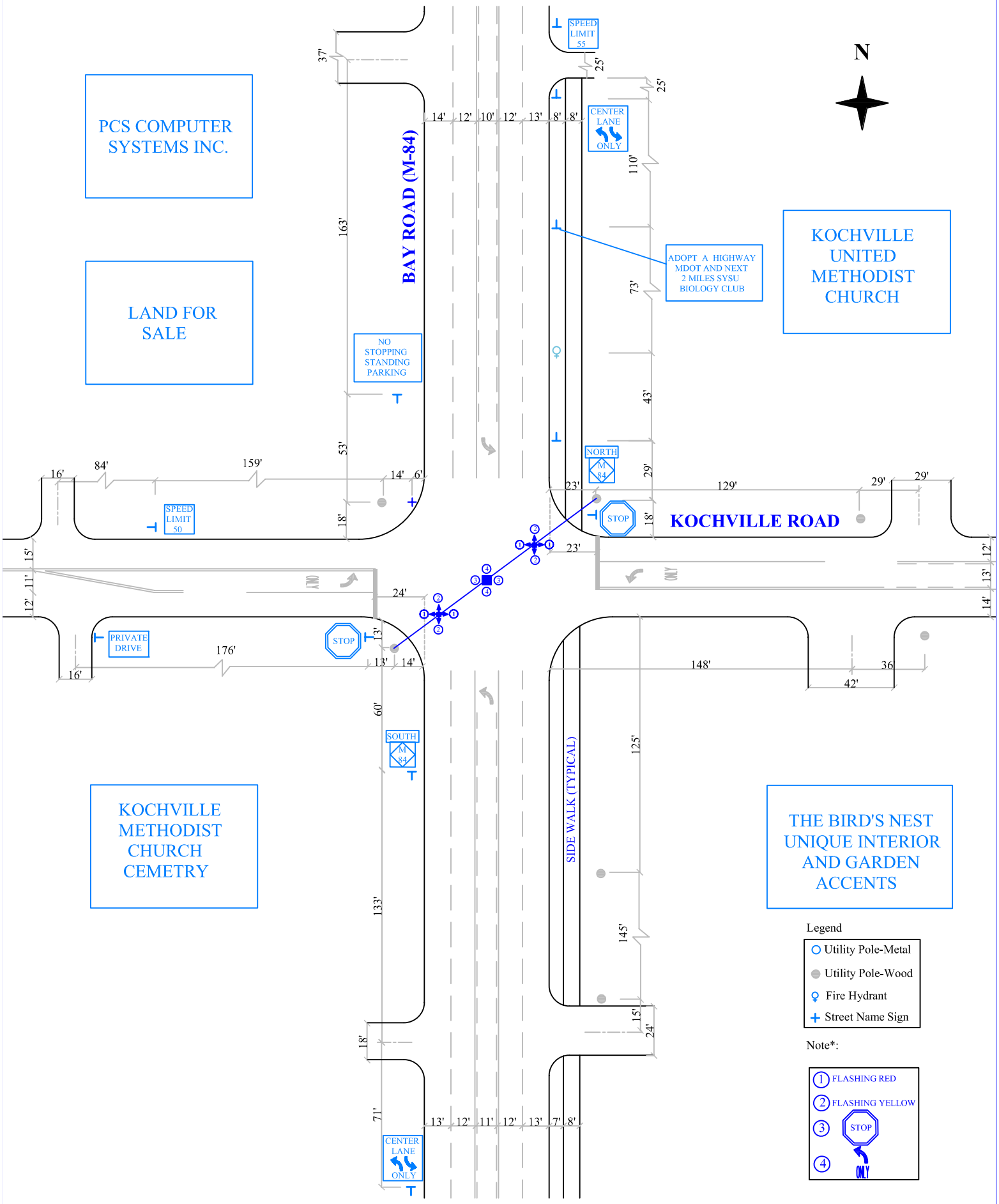


Figure 6.1.1. Existing Condition Diagram of Bay Road (M-84) and Kochville Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on a clear day on February 7, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at suitable locations around the intersection and covering two intersecting approaches each. The results of the data are presented in Figure 6.1.2.

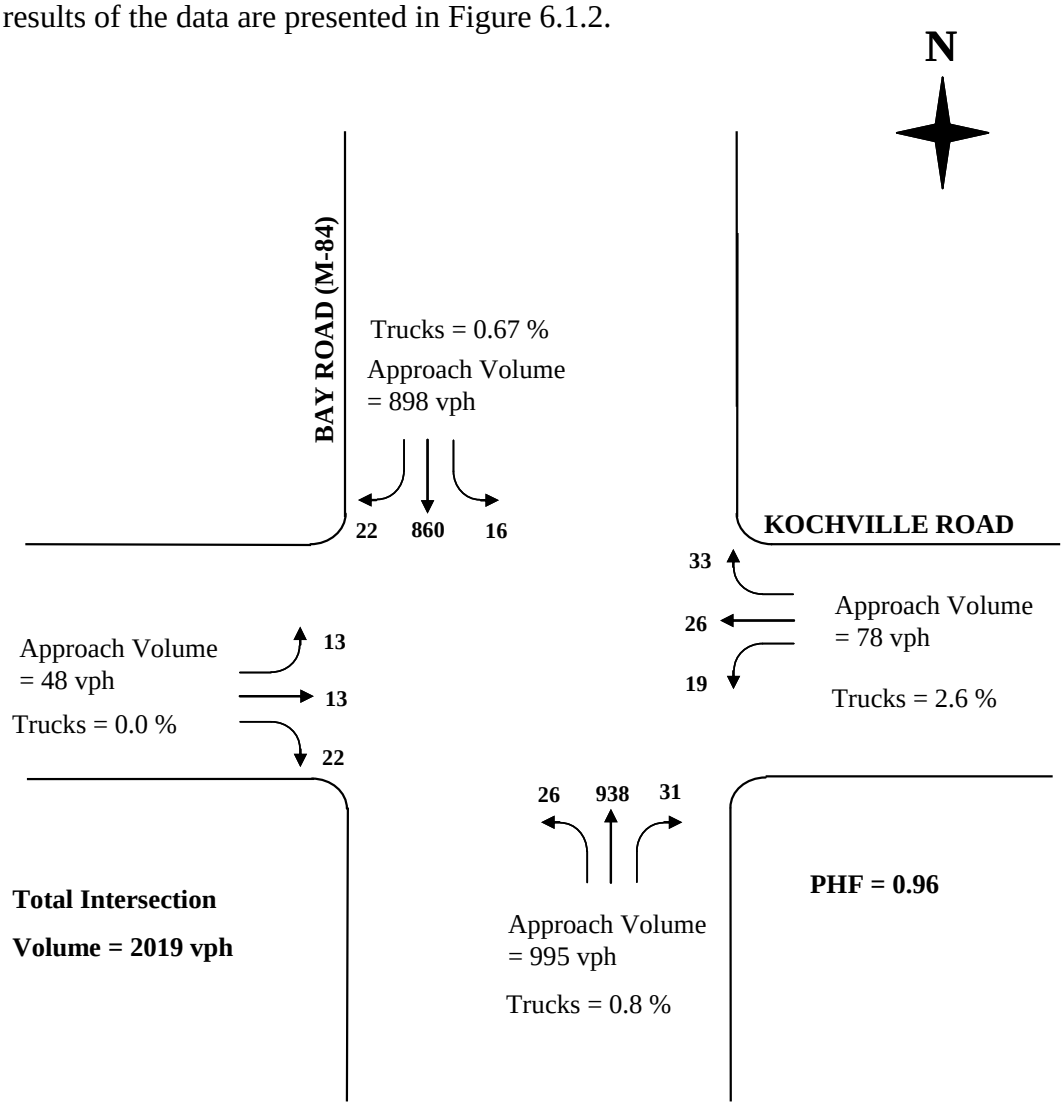


Figure 6.1.2 Existing PM Peak Hour (4:00–5:00 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and Kochville Road`

Traffic Control

The intersection is a two-way stop controlled intersection and controlled by a flashing red and yellow traffic signal. The major road (M-84) vehicles are controlled by a flashing yellow while the minor road (Kochville Road) vehicles are controlled by a flashing red

light. In addition to this, there is a center hung case sign stating, " Left Only" for north-south traffic and "Stop" for east-west traffic.

Crash Analysis

A total of eight crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of four crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. Crashes that have occurred within 150 feet around the intersection were considered only. Figures 6.1.3 and 6.1.4 show the collision diagrams for the years 2003 and 2004, respectively.

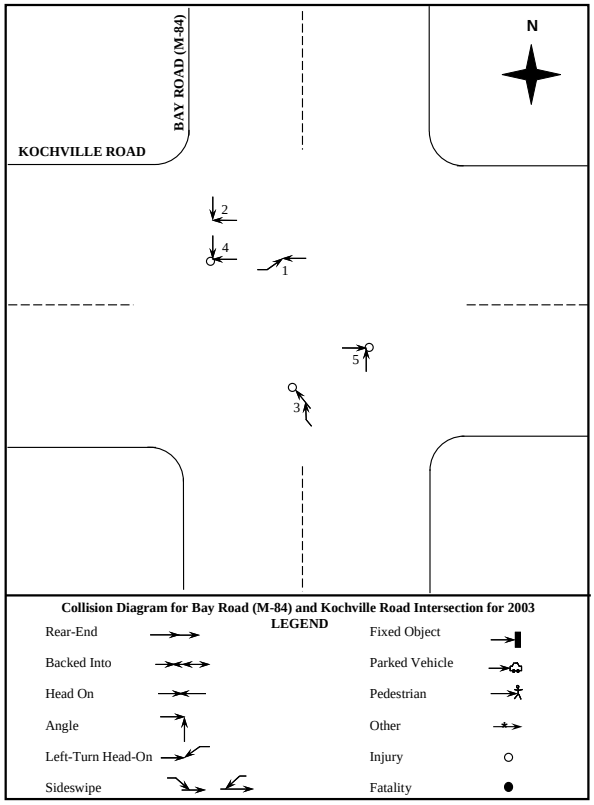


Figure 6.1.3. Collision Diagram for Bay Road and Kochville Road for 2003

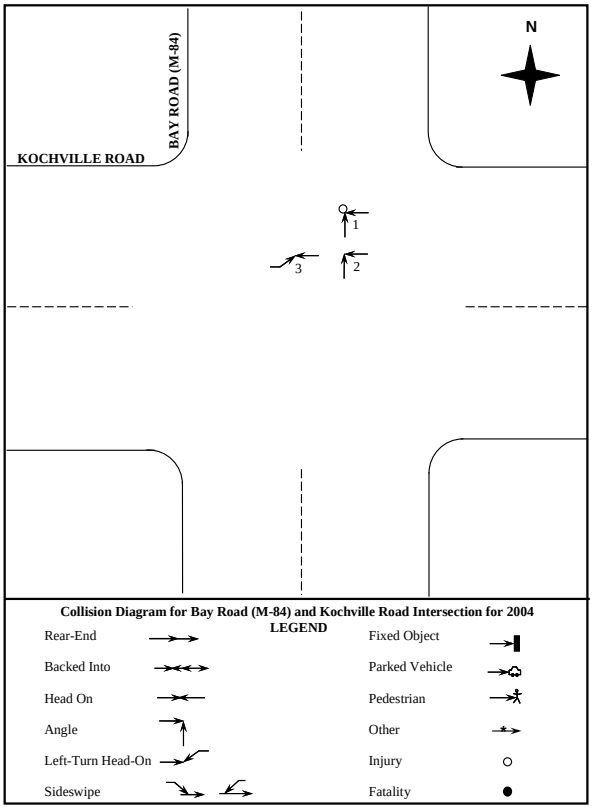


Figure 6.1.4. Collision Diagram for Bay Road and Kochville Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.43 per million entering vehicles, which is nominal, given the approach volume at the intersection. The number of right-angle crashes occurring at the intersection is a cause of major concern, which may

involve the inadequate gap for the minor road vehicles to cross the intersection. Similarly, there are some left-turn head-on crashes between minor road vehicles.

Safety Deficiencies and Recommended Improvements

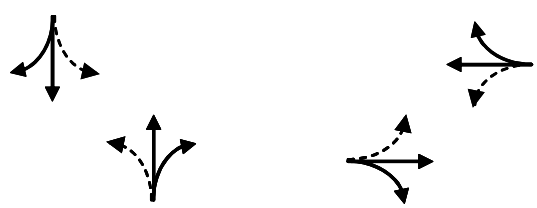
The crashes reported in this intersection are right angle and left-turn-head-on crashes, which may have occurred due to inadequate available gap for the minor road traffic. A two-phase traffic signal with a 90 second cycle length during the PM peak period is recommended in place of the existing flashing red and yellow signal. The signal should be semi-actuated considering a skip phase when there is no traffic in the minor street. The signal should also be mast arm supported or box-span type to provide better visibility to drivers. The All red and yellow intervals for the recommend traffic signal are shown in Table 6.1.1.

Table 6.1.1 Clearance Interval of Proposed Signal

Interval/Phase	Yellow	All Red
N-S Phase	5.0	1.0
E-W Phase	4.7	1.4

The signal-timing plan for the PM peak hour for the intersection is shown below:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 seconds



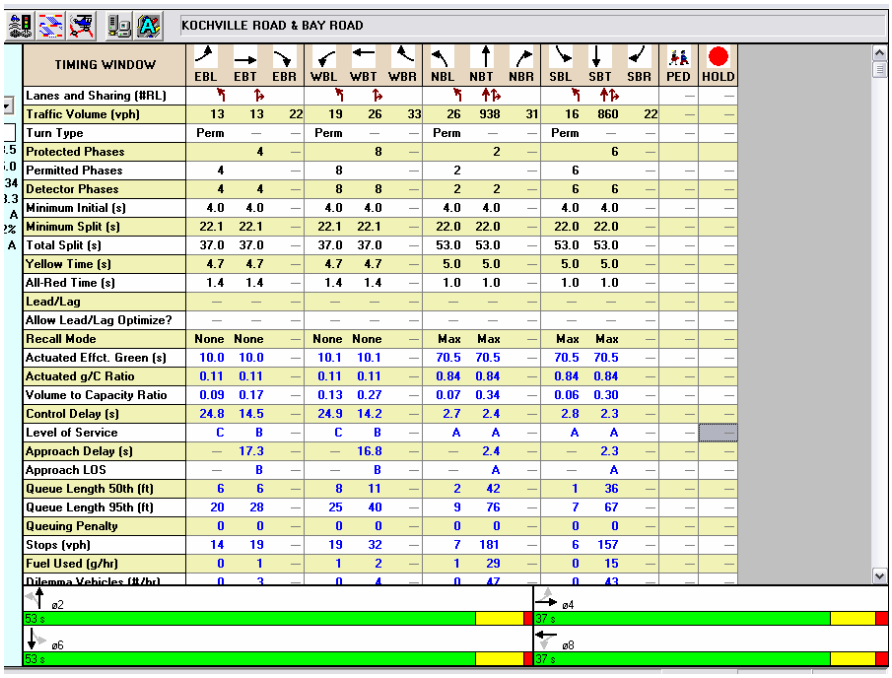
N-S Bay	E-W Kochville
G = 47.0 sec	G = 30.9 sec
Y =5.0 sec	Y =4.7 sec
*R = 37.0 sec	*R = 53.0 sec
AR = 1.0 sec	AR = 1.4 sec

* Excludes All Red

The intersection is a part of the Bay Road Corridor considered for progression. Hence, it should have the same cycle length as the rest of the intersections in the corridor. The

green time shown in the signal timing plan for Kochville Road is the threshold maximum value. If there is no traffic in the minor road the green phase can be skipped. Also, the green time can be varied according to the traffic demand. In any case, the signal controller should be synchronized such that the subtraction of green from the threshold maximum is adjusted with additional green for the major road, keeping the cycle length and offset for progression constant. Thus, progression on Bay Road will not be impacted. A snapshot of the Synchro Timing Window shows an actuated effective green of 10 seconds for Kochville Road and 70.5 seconds for Bay Road.

Figure 6.1.5 Snapshot of the Signal Timing Window in Synchro for Bay and Kochville



KOCHVILLE ROAD & BAY ROAD

TIMING WINDOW	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	PED	HOLD
Lanes and Sharing (#RL)													—	—
Traffic Volume (vph)	13	13	22	19	26	33	26	938	31	16	860	22	—	—
Turn Type	Perm	—	—	Perm	—	—	Perm	—	—	Perm	—	—	—	—
Protected Phases		4	—		8	—		2	—		6	—		
Permitted Phases	4		—	8		—	2		—	6		—	—	—
Detector Phases	4	4	—	8	8	—	2	2	—	6	6	—	—	—
Minimum Initial (s)	4.0	4.0	—	4.0	4.0	—	4.0	4.0	—	4.0	4.0	—	—	—
Minimum Split (s)	22.1	22.1	—	22.1	22.1	—	22.0	22.0	—	22.0	22.0	—	—	—
Total Split (s)	37.0	37.0	—	37.0	37.0	—	53.0	53.0	—	53.0	53.0	—	—	—
Yellow Time (s)	4.7	4.7	—	4.7	4.7	—	5.0	5.0	—	5.0	5.0	—	—	—
All-Red Time (s)	1.4	1.4	—	1.4	1.4	—	1.0	1.0	—	1.0	1.0	—	—	—
Lead/Lag	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Allow Lead/Lag Optimize?	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Recall Mode	None	None	—	None	None	—	Max	Max	—	Max	Max	—	—	—
Actuated Effct. Green (s)	10.0	10.0	—	10.1	10.1	—	70.5	70.5	—	70.5	70.5	—	—	—
Actuated g/C Ratio	0.11	0.11	—	0.11	0.11	—	0.84	0.84	—	0.84	0.84	—	—	—
Volume to Capacity Ratio	0.09	0.17	—	0.13	0.27	—	0.07	0.34	—	0.06	0.30	—	—	—
Control Delay (s)	24.8	14.5	—	24.9	14.2	—	2.7	2.4	—	2.8	2.3	—	—	—
Level of Service	C	B	—	C	B	—	A	A	—	A	A	—	—	—
Approach Delay (s)	—	17.3	—	—	16.8	—	—	2.4	—	—	2.3	—	—	—
Approach LOS	—	B	—	—	B	—	—	A	—	—	A	—	—	—
Queue Length 50th (ft)	6	6	—	8	11	—	2	42	—	1	36	—	—	—
Queue Length 95th (ft)	20	28	—	25	40	—	9	76	—	7	67	—	—	—
Queuing Penalty	0	0	—	0	0	—	0	0	—	0	0	—	—	—
Stops (vph)	14	19	—	19	32	—	7	181	—	6	157	—	—	—
Fuel Used (g/hr)	0	1	—	1	2	—	1	29	—	0	15	—	—	—
Dilemma Vehicles (ft/hr)	0	3	—	0	4	—	0	47	—	0	43	—	—	—

A list of the recommended improvements is summarized below:

- Introduce a two-phase semi-actuated traffic signal with a skip phase and a cycle length of 90 seconds during the PM peak period.
- Provide a yellow interval of 5.0 seconds for North-South phase and 4.7 seconds for east-west phase.
- Provide all-red interval of 1.0 second and 1.4 seconds for north-south and east-west phase respectively.

- Install mast arm supported traffic signals.
- Provide stop bars for north-south traffic.
- Provide a crosswalk on the east leg of the intersection across Kochville Road.
- Post a speed limit sign for the eastbound traffic downstream of the intersection.
- Provide stop signs for all the driveways located near the intersection.
- Relocate the stop bars closer to the intersection to provide more visibility on Kochville Road.
- Implement the optimized signal timings for AM peak and off-peak hours.

The proposed intersection diagram is shown in Figure 6.1.7 and shows the recommended installations to be implemented at the intersection.

Operational Evaluation:

The results of the level of service analysis by HCS for the intersection of Bay Road and Kochville Road indicate that for the existing unsignalized conditions during the PM peak hour, Bay Road operates at level of service is A with 5.3 seconds/vehicle of delay, and the Kochville Road operates at level of service F with considerable delay.

With the proposed improvements in the signal timing, including signal progression along Bay Road, the overall intersection level of service will be A with a considerable reduction of delay for Kochville Road (LOS D). The levels of service for the existing conditions and for the proposed improvements are shown in Table 6.1.2.

Table 6.1.2 Bay Road and Kochville Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions (unsignalized)		LOS with Proposed Improvements Including Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
NB Bay Road	1.0	A	2.0	A
SB Bay Road	1.0	A	1.9	A
EB Kochville Road	60.0	F	40.1	D
WB Kochville Road	78.8	F	41.8	D
Overall Intersection	**NA**	**NA**	**4.0**	**A**

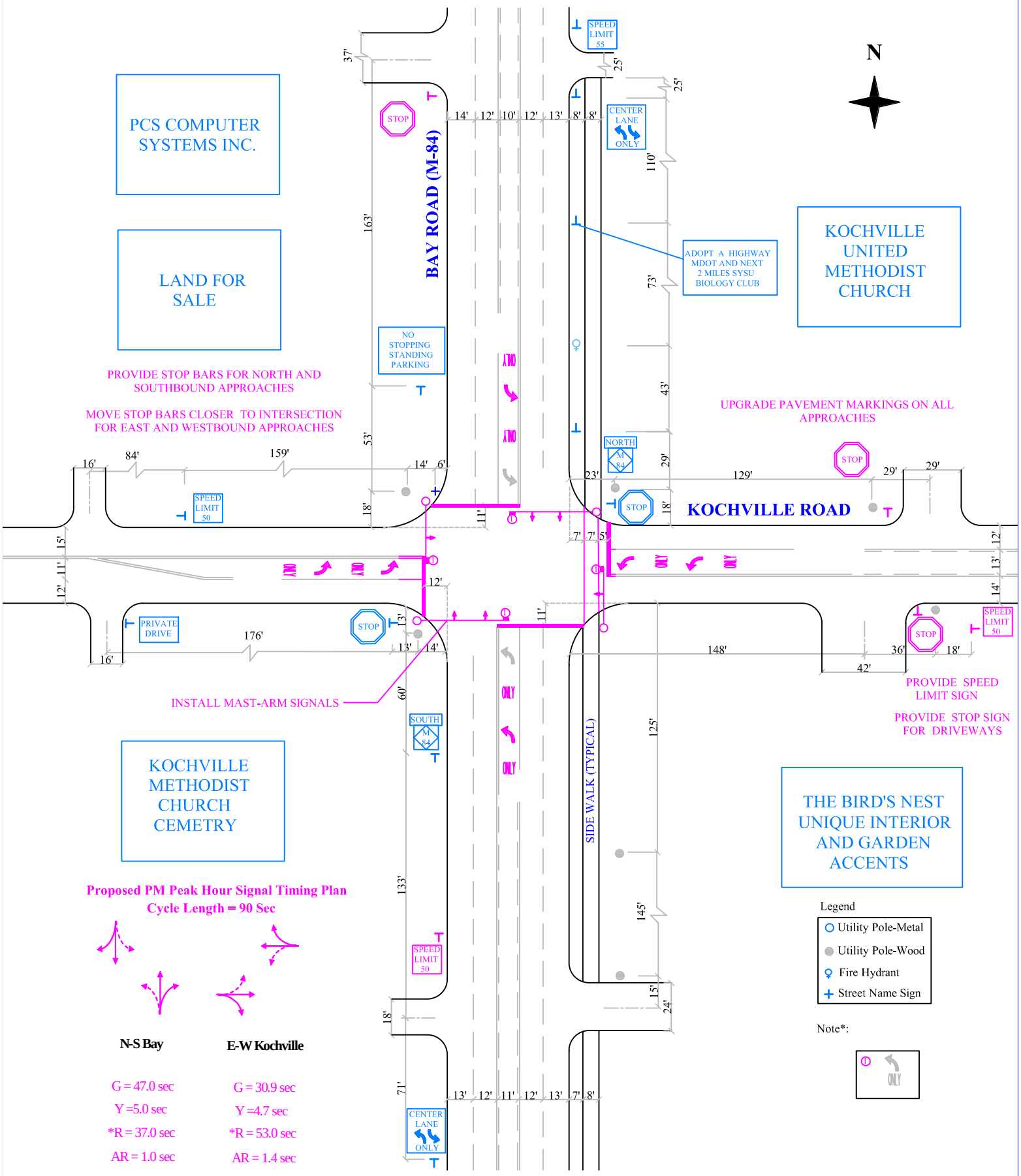


Figure 6.1.6 Proposed Diagram of Bay Road (M-84) and Kochville Road Intersection (Not to Scale)

2. Bay Road and Bueker Drive

The intersection of Bay Road and Bueker Drive is located in the southern part of Kochville Township. Bay Road runs in the north-south direction and makes a T-intersection with Bueker Drive. Bay Road is a State Trunk line Road and it is also known as (M-84).

Bay Road has an average daily traffic volume of approximately 24,500 vpd at this location, as per MDOT 2004 data.

Bay Road is 60 feet wide and has two through lanes in each direction and a center left turn lane. Bueker Drive is 37 feet wide. It has an exclusive left turn and a right turn lane at the eastbound approach and has a receiving lane in the west bound direction.

The posted speed limit for Bay Road is 50 miles per hour and for Bueker Drive it is 25 miles per hour.

Land Use

There is a Food Market and a Home Depot store on the northwest corner of the intersection. A bank and a pizza shop are located in the southwest corner. The northeast corner of the intersection has some residential homes whereas the southeast corner has a Buffet Max, a Staples store and a Boat Outboard Marine shop.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are shown in Figure 6.2.1. Details of the intersection approaches are shown in photographs 2a through 2c. Photographs 2a and 2b show the Bay Road approaches having significant right of way on both sides. Pavement markings are also depicted in

these photographs. Photograph 2c shows the eastbound approach and the stop control sign for this unsignalized intersection.

Photograph 2a. NB Bay Road at Bueker Drive

Photograph 2b. SB Bay Road at Bueker Drive

Photograph 2c. EB Bueker Drive at Bay Road

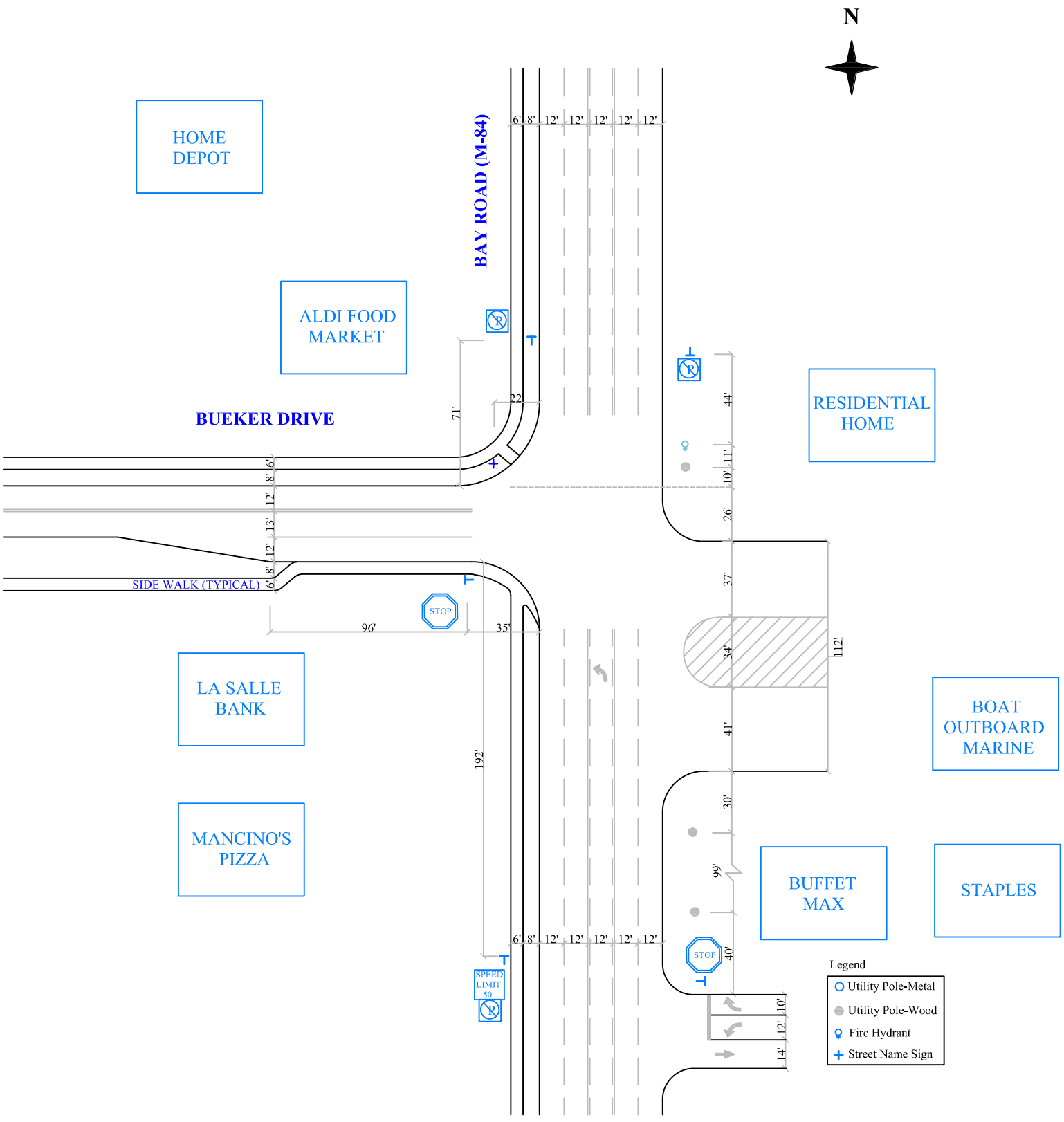


Figure 6.2.1. Existing Condition Diagram of Bay Road (M-84) and Bueker Drive Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on a snowy day on February 7, 2006 from 4 PM to 6 PM by two people standing at one suitable corner of the intersection and covering all the approaches. The results of the data are presented in Figure 6.2.2, which show a significant amount of through volume in the north-south Bay Road and very few turning movements to and from Bueker Drive.

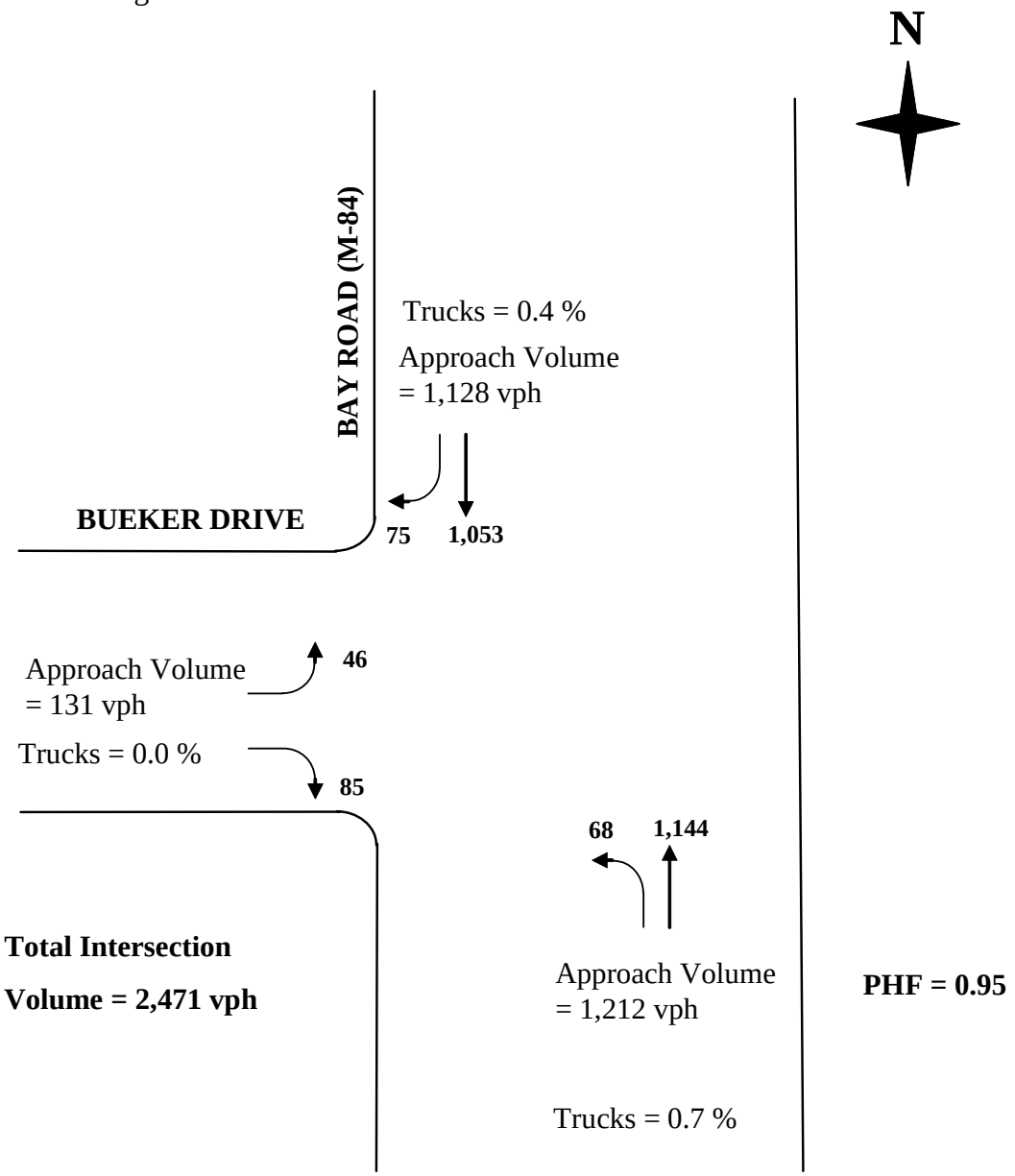


Figure 6.2.2 Existing PM Peak Hour (4:15–5:15 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and Bueker Drive

Traffic Control

This intersection is unsignalized and controlled by a one-way stop control sign for the eastbound approach.

Crash Analysis

A total of ten crashes for the years 2003 and 2004 were recorded for this intersection with an average crash frequency of five crashes per year. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 report information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Bay Road and Bueker Drive are shown in Figure 6.2.3. and Figure 6.2.4 respectively.

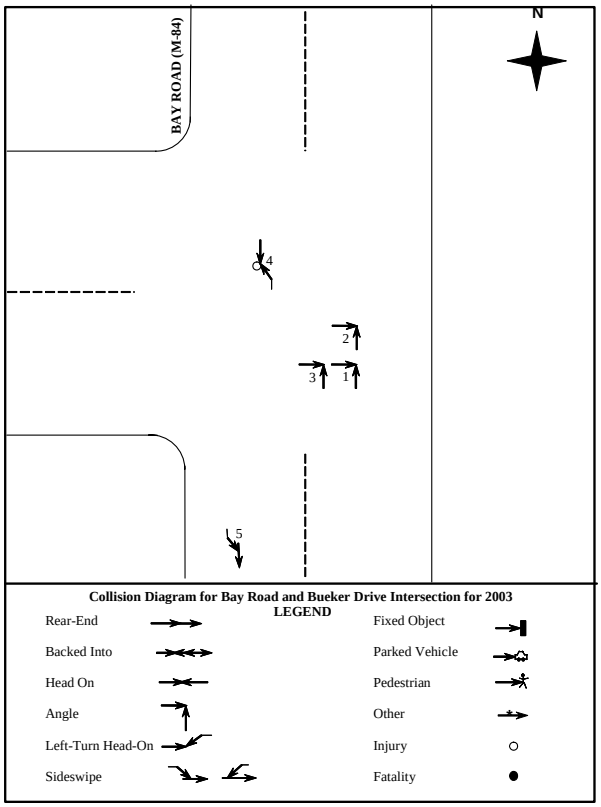


Figure 6.2.3 Collision Diagram for Bay Road and Bueker Drive for 2003

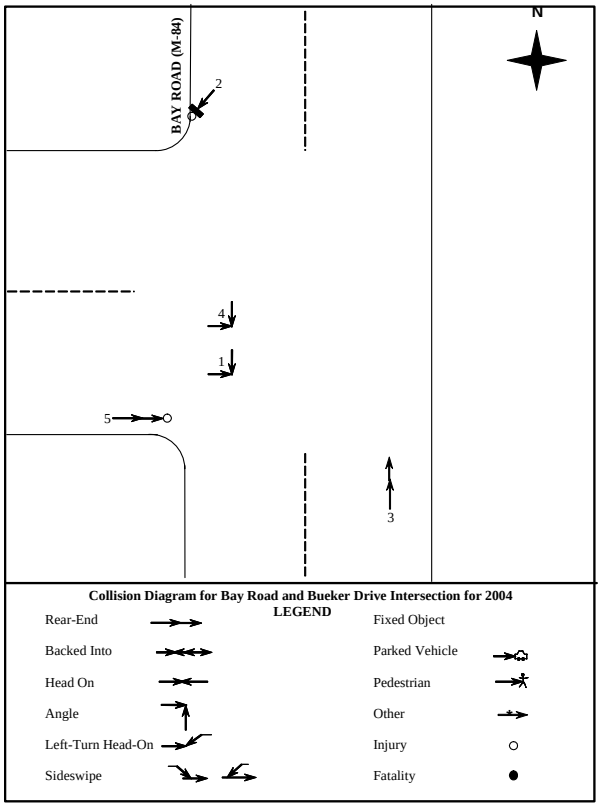


Figure 6.2.4 Collision Diagram for Bay Road and Bueker Drive for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.44 per million entering vehicles, which is not significant, considering the approach volume at the intersection.

Safety Deficiencies and Recommended Improvements

This intersection has some right angle crashes, which might have occurred, due to an inadequate gap for the minor road traffic. The pavement markings should be improved to provide better guidance to the drivers. A stop bar and crosswalk should be provided on Bueker Drive coupled with a set of left-only marking. Improved pavement markings should also be provided for northbound and southbound traffic. There is a wide driveway on the east side of the intersection almost opposite to Bueker Drive. The driveway should have stop bar and stop sign for the traffic exiting out from the driveway.

Speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

A list of the recommended improvements is summarized below:

- Provide additional pavement markings on the northbound and southbound approaches.
- Provide left-only markings on eastbound left-turn lane of Bueker Drive.
- Provide stop bar and crosswalk on the eastbound approach of Bueker Drive.
- Post a stop sign and stop bar on the driveway east of the intersection opposite to Bueker Drive.
- Post speed limit signs downstream of the intersection for all approaches.

The proposed intersection diagram is shown in Figure 6.2.5 and shows the recommended installations to be implemented at the intersection.

Operational Evaluation:

The results of the level of service analysis for the intersection of Bay Road and Bueker Drive indicate that, for the existing conditions during the PM peak hour, Bay road operates at level of service A while Bueker drive operates at level of service F. Since the proposed recommendations do not affect the traffic control, the level of service will not change.

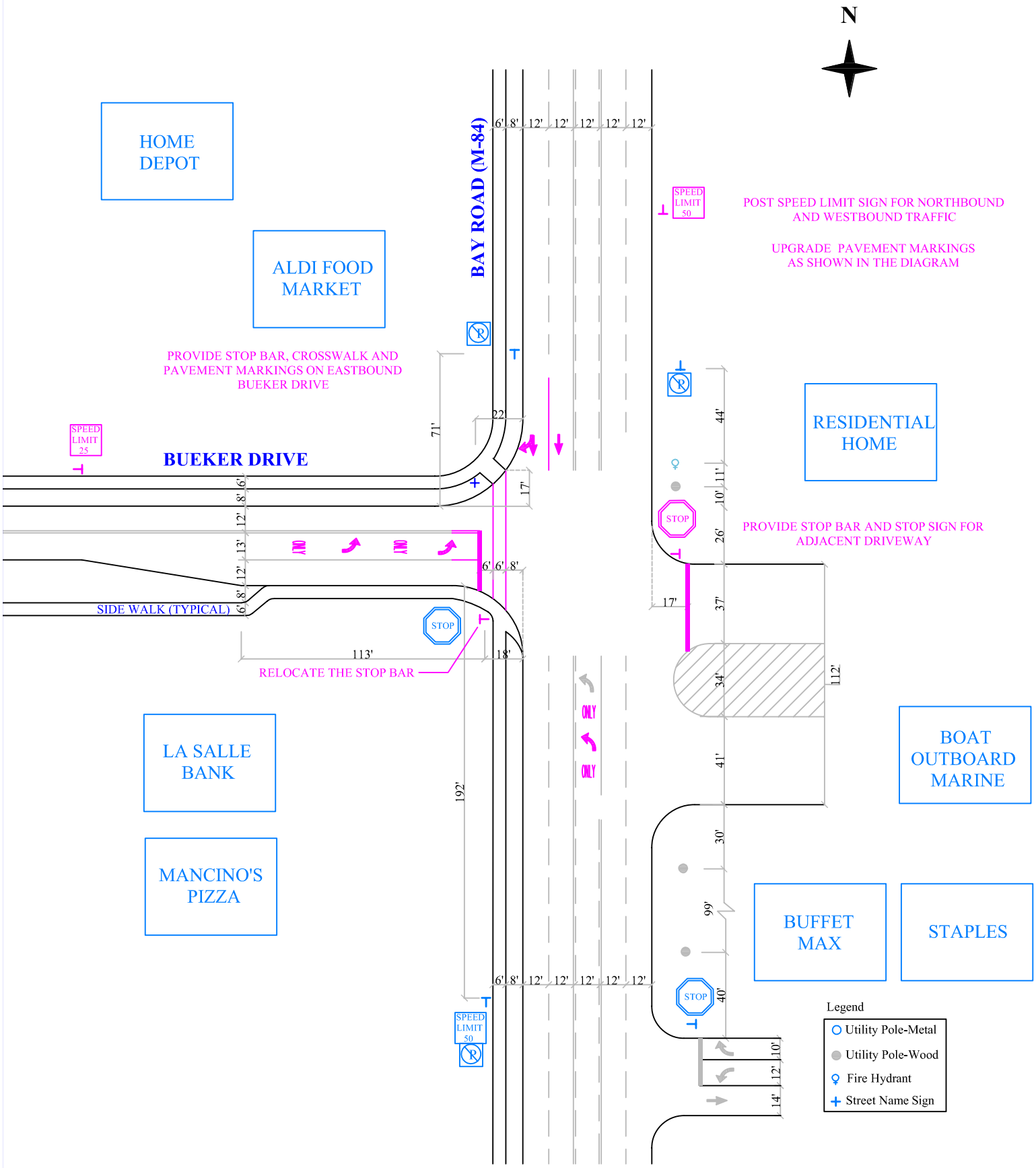


Figure 6.2.5. Proposed Diagram of Bay Road (M-84) and Bueker Drive Intersection (Not to Scale)

The level of service results for the existing condition is shown in Table 6.2.1.

Table 6.2.1
Bay Road and Bueker Drive
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions	
	Delay (Sec/Vehicle)	LOS
NB Bay Road	0.7	A
SB Bay Road	0	A
EB Bueker Drive	81.4	F
Overall Intersection	**4.6**	**A**

3. Bay Road (M-84) and Tittabawassee Road

The intersection of Bay Road and Tittabawassee Road is located in the northern part of the Saginaw Township. Both roads are arterials and Bay Road is also a state trunk line (M-84).

Bay Road has average daily traffic volumes of approximately 24,500 vpd and 25,700-vpd north and south of the intersection respectively at this location as per MDOT Average Daily Traffic Map (2004).

Bay Road is 71 feet wide with one exclusive left-turn lane, one exclusive right- turn lane and two through lanes at both northbound and southbound approaches. Tittabawassee Road is about 60 feet wide with one exclusive left-turn lane, one through lane and one shared through and right-turn lane for westbound traffic; while for eastbound traffic, the road is 70 feet wide with an additional exclusive right turn lane.

The speed limit for Bay Road is 50 miles per hour north of Tittabawassee Road and 40 miles per hour south, and for the Tittabawassee Road; it is 45 miles per hour.

Land Use

The intersection of Bay Road and Tittabawassee Road has a Payless Shoe shop on the northeast corner; an Alltel shop on the northwest corner, a Sears Auto Center in the southwest corner and a Flagstar Bank on the southeast and southwest corners respectively. There are four driveways within 150 ft of the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.3.1. Photographs of each approach leg of the intersection are shown in Photographs 3a through 3d. All the photographs show the signal heads hanging from the span wire installation and the pavement markings.

Photograph 3a. NB Bay Road at Tittabawassee Road

Photograph 3b. SB Bay Road at Tittabawassee Road

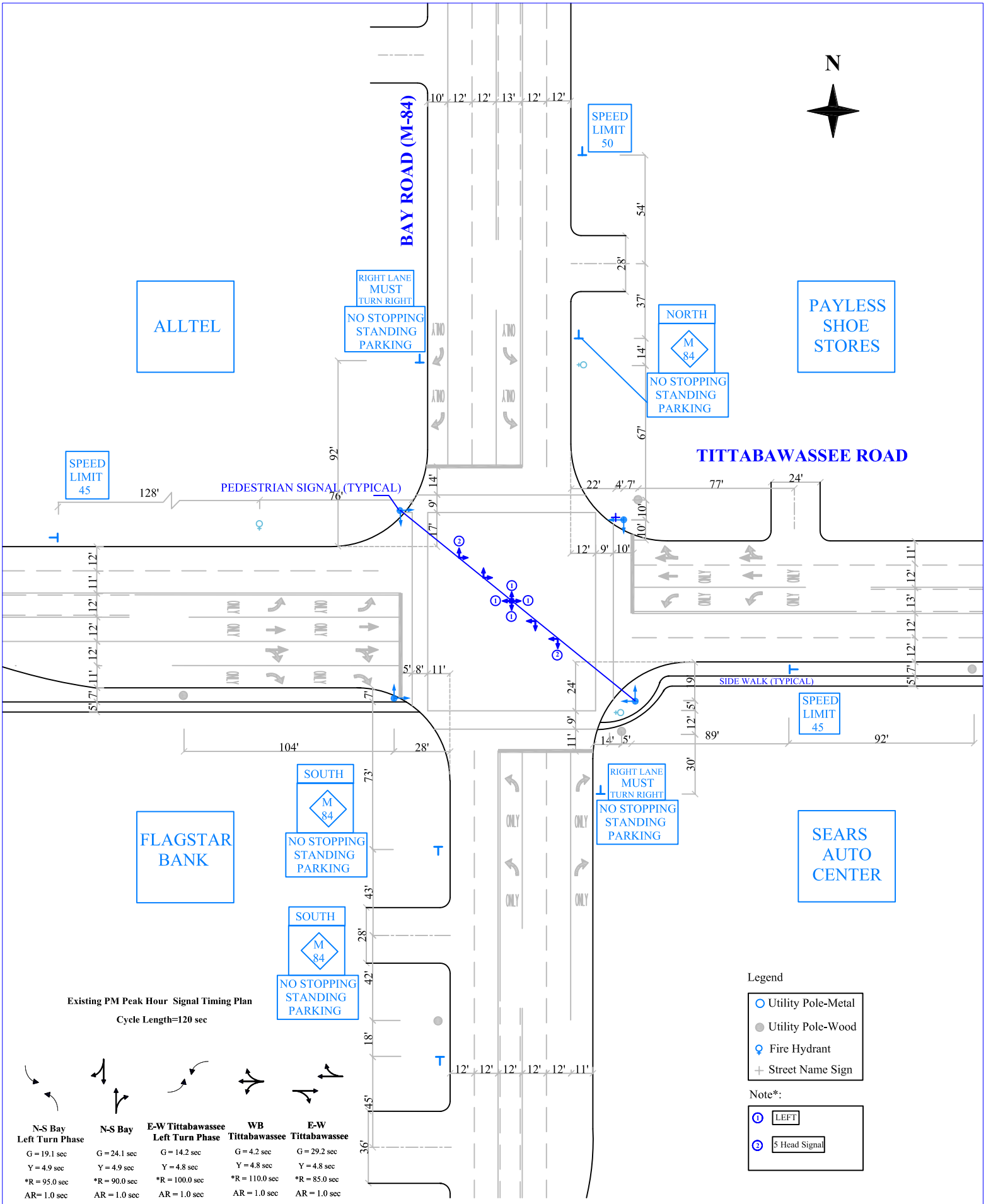


Figure 6.3.1 Existing Condition Diagram of Bay Road (M-84) and Tittabawasee Road Intersection (Not to Scale)

Photograph 3d. WB Tittabawassee Road at Bay Road

Photograph 3c. EB Tittabawassee Road at Bay Road

Peak Hour Volumes

PM peak hour traffic volumes were collected on a clear day on February 7, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The results of the data are presented in Figure 6.3.2.

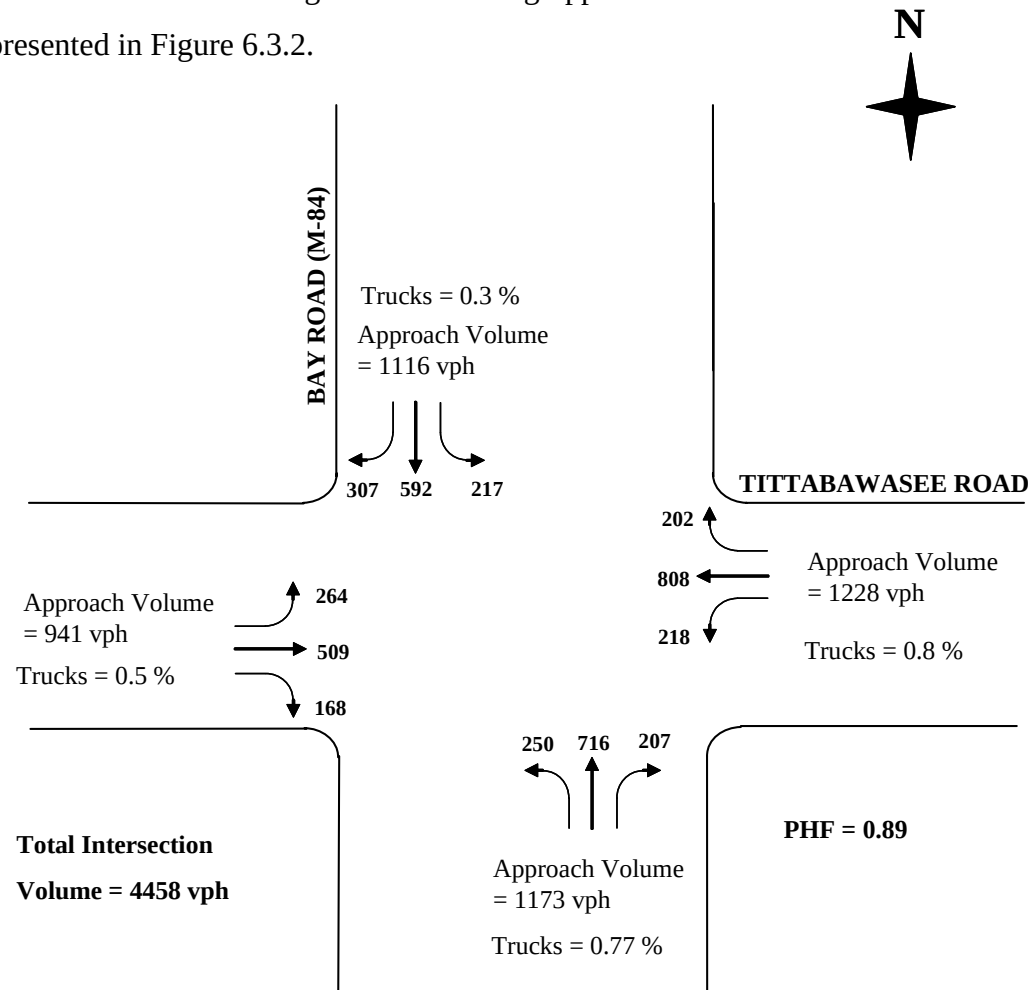


Figure 6.3.2 Existing PM Peak Hour (4:30–5:30 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and Tittabawassee Road

Traffic Control

The basic signal timing plan is given by a five phase design including protected left turn phases for both Bay Road and Tittabawassee Road and a WB split phase for Tittabawassee Road and operates on a 120-second cycle length during the PM peak hour. The existing clearance interval at this intersection includes a yellow interval of 4.9 seconds and an all-red interval of 1.0 second for the northbound and southbound approaches. For the eastbound and westbound approaches, there is a yellow interval of 4.8 seconds and an all-red interval of 1.0 second. The existing signal-timing plan for this intersection is accompanied with the existing condition diagram in Figure 6.3.1. The intersection has pedestrian signals and crosswalks on all approaches.

Crash Analysis

A total of 70 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of thirty-five crashes per year. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. Figures 6.3.3 and 6.3.4 show the collision diagrams for the years 2003 and 2004, respectively.

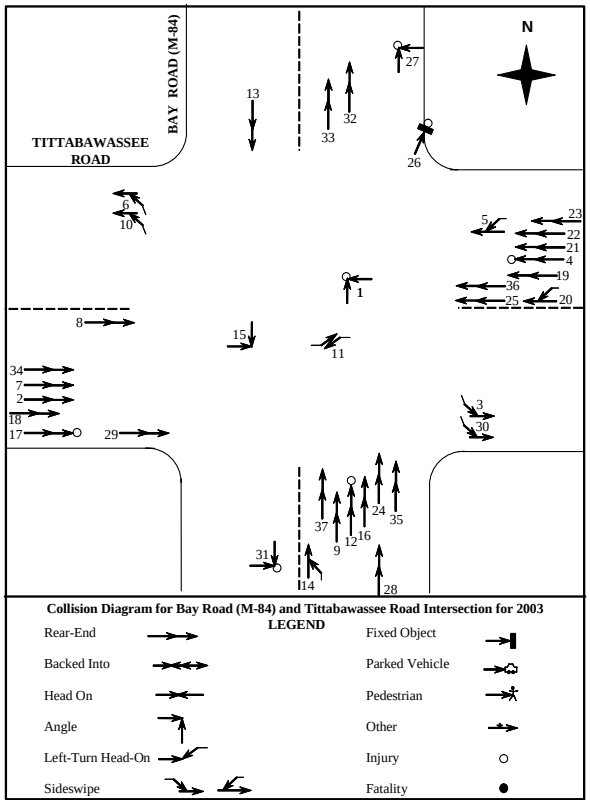


Figure 6.3.3 Collision Diagram for Bay Road and Tittabawassee Road for 2003

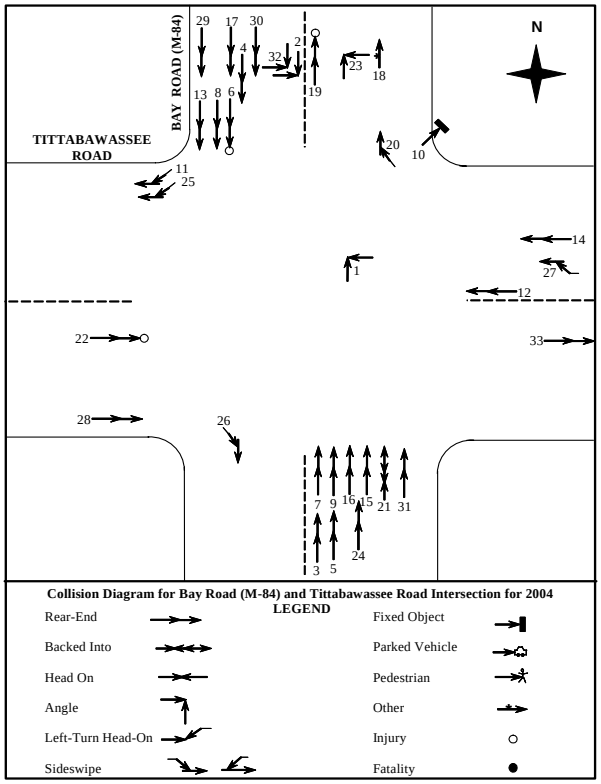


Figure 6.3.4 Collision Diagram for Bay Road and Tittabawassee Road for 2004

The annual crash rate is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.72 per million entering vehicles, which is high, taking into account the approach volume at the intersection. From the collision diagrams, as provided in the figures above, a significant number of rear-end crashes occurred at the intersection due to inadequate clearance intervals and signal timing. Some driveway related right-angle crashes have also occurred at the intersection.

The crash distribution, based upon type of crashes, is shown in Figure 6.3.5. Rear-end, sideswipe same and angle crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes include 45 rear-end crashes, 12 sideswipe same crashes and eight angle crashes.

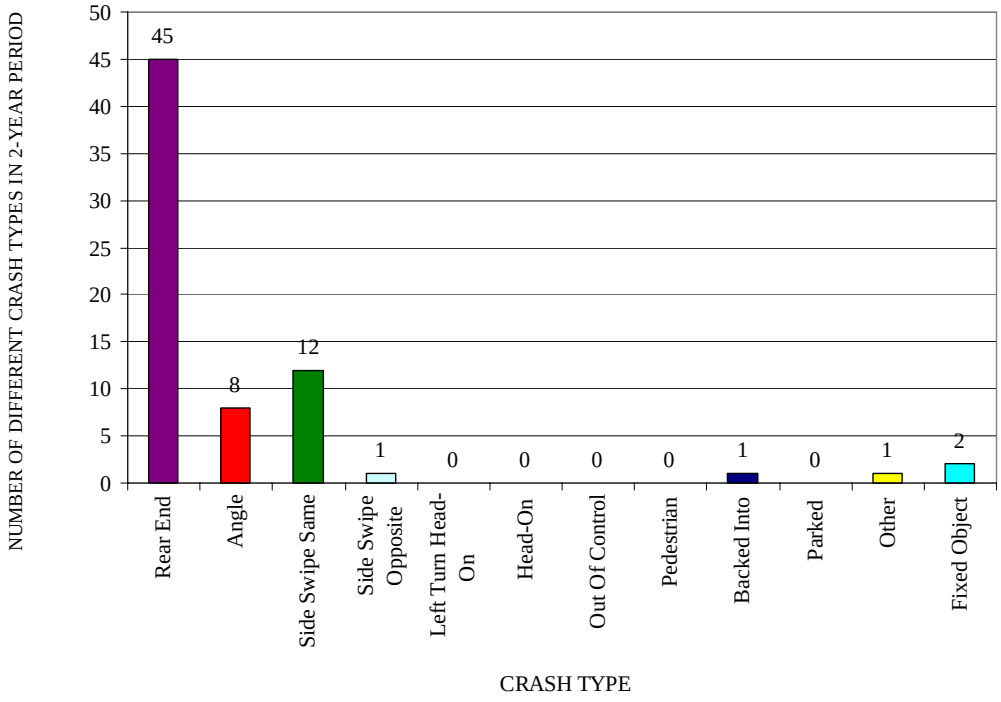


Figure 6.3.5. Two year (2003 & 2004) Crash Experience at Bay Road (M-84) and Tittabawassee Road Intersection

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of Bay Road and Tittabawassee Road could be attributed to inadequate signal timing, signal phasing and clearance interval; and the presence of driveways close to the intersection.

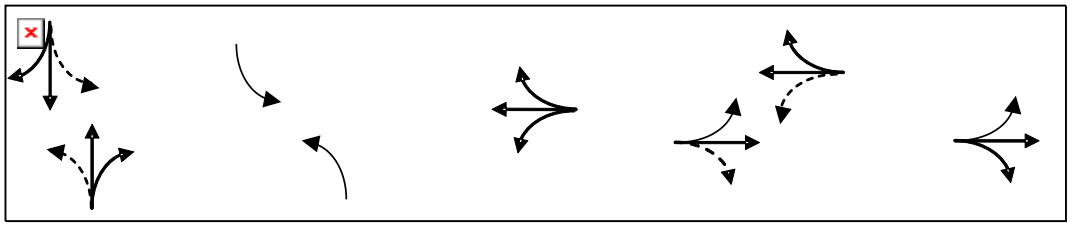
Being a pivotal intersection of two major roads, it is required to maintain progressions for both the roads. The phasing plan should be changed accordingly to accomplish the desired progressions. Hence the existing five-phase, 120-second signal timing plan should be replaced by a five-phase, 90-second timing plan including two split phases, two permissive phases and one left-turn protected phase during the PM peak hour. To improve visibility, the existing span wire installation should be replaced by mast arm or box span signal installation, as shown in the proposed intersection diagram in Figure 6.3.6. The existing yellow and all-red intervals are recalculated and shown in Table 6.3.1, along with their potential benefits.

Table 6.3.1. Existing and Proposed Clearance Intervals for the Bay Road (M-84) and Tittabawassee Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Left Turn	4.8	1.0	-	-	Less rear end, angle, left-turn head-on and injury crashes
WB Split Phase	4.8	1.0	4.3	1.8	
E-W Through Phase	4.8	1.0	4.3	1.8	
EB Split Phase	-	-	4.3	1.8	
N-S Through Phase	4.9	1.0	4.7	2.0	
N-S Left Turn	4.9	1.0	4.7	2.0	

The proposed PM peak hour signal timing plan for these improvements is as follows:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



N-S Bay	N-S Bay Left Turn Phase	WB Tittabawassee	E-W Tittabawassee	EB Tittabawassee
G = 20.3	G = 6.3	G = 9.9	G = 11.9	G = 9.9
Y = 4.7	Y = 4.7	Y = 4.3	Y = 4.3	Y = 4.3
*R = 69.7	*R = 83.7	*R = 80.1	*R = 78.1	*R = 80.1
AR = 2.0 sec	AR= 2.0 sec	AR = 1.8 sec	AR = 1.8 sec	AR = 1.8 sec

* Excludes All Red

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for north-south phase to 4.7 seconds and for east-west phase to 4.3 seconds.
- Increase the all-red interval for north-south phase to 2.0 seconds and for east-west phase to 1.8 seconds.
- Provide new signal-timing plan with 90 seconds cycle length, which includes permissive-protected phases for north-south , EB split phase, E-W permissive phase and WB split phase.
- Post speed limit signs downstream of the intersection for all approaches.
- Upgrade the existing span-wire configuration to a mast arm or box span signal installation.

- Relocate the crosswalks on northbound and southbound approaches of Bay Road with some modifications of sidewalk configuration on the southeast corner of the intersection.
- Relocate the stop bars on northbound, southbound and westbound approaches closer to the intersection.
- Provide stop sign for all the driveways located near the intersection. Individual landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours compatible with the progressive movement of traffic on both corridors.

The proposed intersection diagram is shown in Figure 6.3.6 and shows the improvements such as the installation of a mast-arm signal support, relocating the crosswalk, posting speed limit signs, etc.

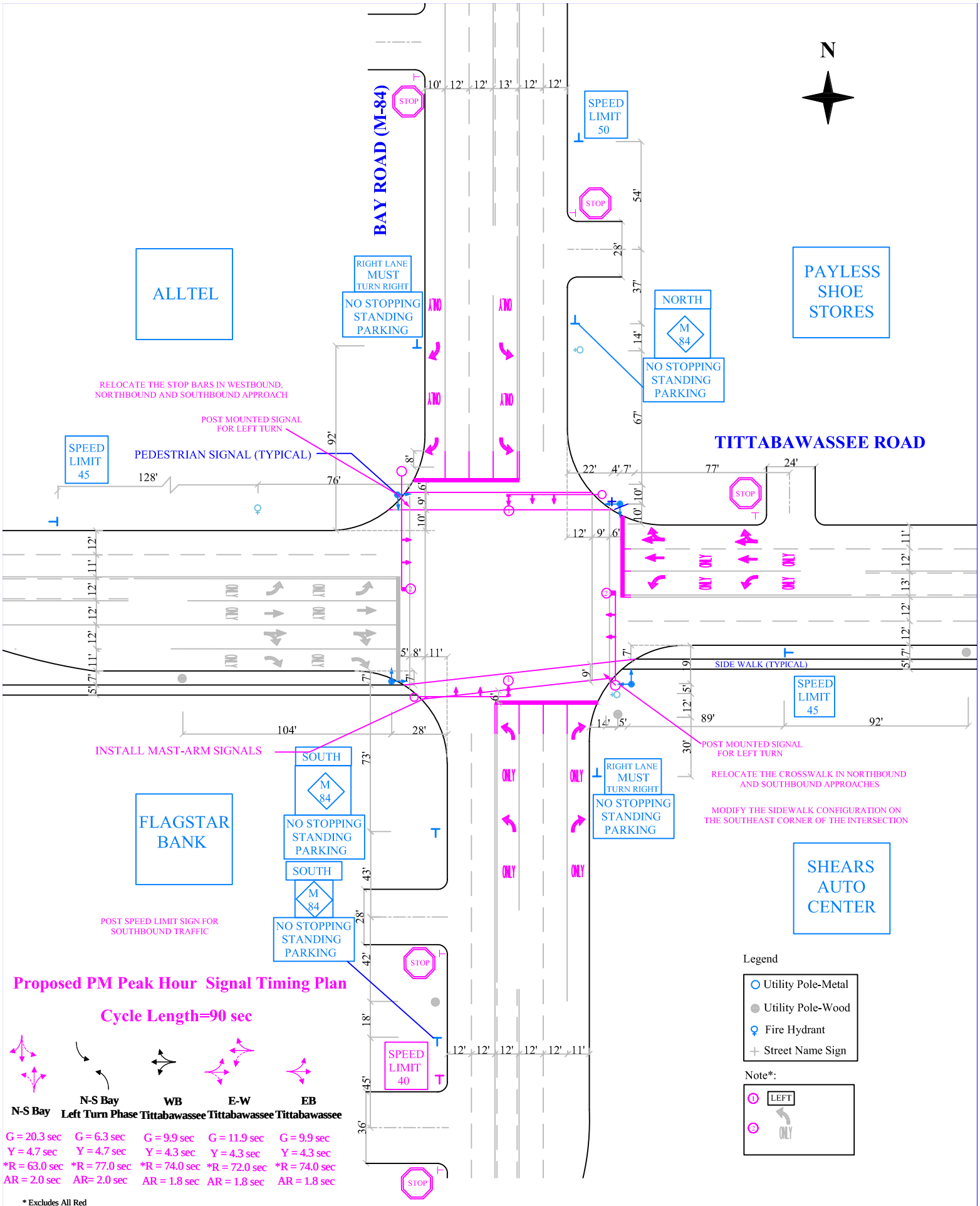
Operational Evaluation:

The results of the level of service analysis for the intersection of Bay Road and Tittabawassee Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is E with a delay of 68.2 seconds per vehicle. With the noted improvements and modifications to the signal timing, the overall intersection LOS will be D, with a reduced delay of 37.8 seconds per vehicle.

The level of service results for the existing conditions and with the proposed improvements are shown in Table 6.3.2.

Table 6.3.2. Bay Road (M-84) and Tittabawassee Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Veh.)	LOS	Delay (Sec/Veh.)	LOS
EB Tittabawassee Road	85.9	F	28.9	C
WB Tittabawassee Road	49.8	D	43.7	D
NB Bay Road	80.8	F	34.3	C
SB Bay Road	59.9	E	42.4	D
Overall Intersection	**68.2**	**E**	**37.8**	**D**



Proposed PM Peak Hour Signal Timing Plan

Cycle Length=90 sec

N-S Bay	N-S Bay Left Turn Phase	WB Tittabawassee	E-W Tittabawassee	EB Tittabawassee
G = 20.3 sec	G = 6.3 sec	G = 9.9 sec	G = 11.9 sec	G = 9.9 sec
Y = 4.7 sec	Y = 4.7 sec	Y = 4.3 sec	Y = 4.3 sec	Y = 4.3 sec
*R = 63.0 sec	*R = 77.0 sec	*R = 74.0 sec	*R = 72.0 sec	*R = 74.0 sec
AR = 2.0 sec	AR= 2.0 sec	AR = 1.8 sec	AR = 1.8 sec	AR = 1.8 sec

* Excludes All Red

Figure 6.3.6. Proposed Diagram of Bay Road (M-84) and Tittabawasee Road Intersection (Not to Scale)

4. Bay Road (M-84) and Schust Road

The intersection of Bay Road (M-84) and Schust Road is located in the northeastern part of Saginaw Township. Bay Road runs in the north-south direction while Schust Road runs in the east-west direction. Bay Road is a major state trunk line and it is also known as M-84. Schust Road is a collector.

Bay Road, near Schust Road has an average daily traffic volume of approximately 25,700 vpd in the north of this intersection and 29,100 vpd in the south of the intersection as per MDOT 2004 data.

Bay Road is 70 feet wide north of the intersection and 72 feet wide south of the intersection with one exclusive left-turn lane, two through lanes and one dedicated right-turn lane. Schust Road is about 48 feet wide, with one dedicated left-turn lane and one shared through/right-turn lane on the east leg of the intersection. The west leg of Schust Road is 35 feet wide with one exclusive left turn lane and one shared through/right-turn lane. The speed limit for Bay Road is 40 miles per hour and for Schust Road, it is 45 miles per hour west of Bay Road and 40 mph for east.

Land Use

On the northeast corner of the intersection, there is 'Lifestyles' (a spa and pool dealer shop). On the northwest corner there are 'Discount Health Foods' and 'Comfort Zone Water Beds'. The intersection also has a gas station and a K-mart on the southwest corner and a shopping plaza in the southeast corner of the intersection. There are multiple driveways to serve these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.4.1. Details of the intersection approaches are shown in photographs 4a through 4d. The photographs show the signal heads hanging from the span-wire supported installation.

Photograph 4a. NB Bay Road at Schust Road

Photograph 4b. SB Bay Road at Schust Road

Photograph 4c. EB Schust Road at Bay Road

Photograph 4d. WB Schust Road at Bay Road

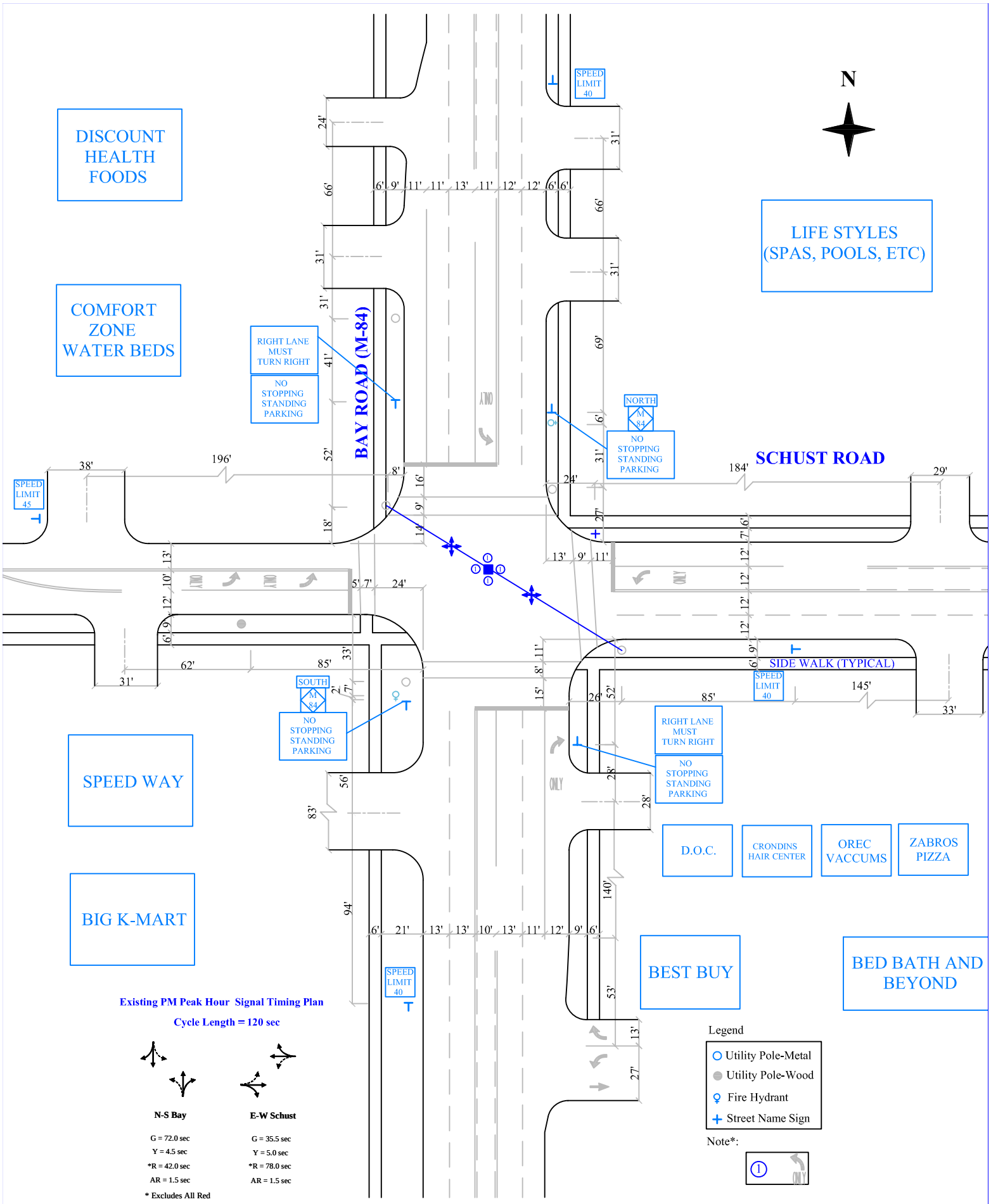


Figure 6.4.1 Existing Condition Diagram of Bay Road (M-84) and Schust Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were collected on a clear day on February 7, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at suitable locations around the intersection and covering two intersecting approaches each. The results of the data are presented in Figure 6.4.2. The traffic volume on Bay Road is relatively higher than the traffic volume on Schust Road.

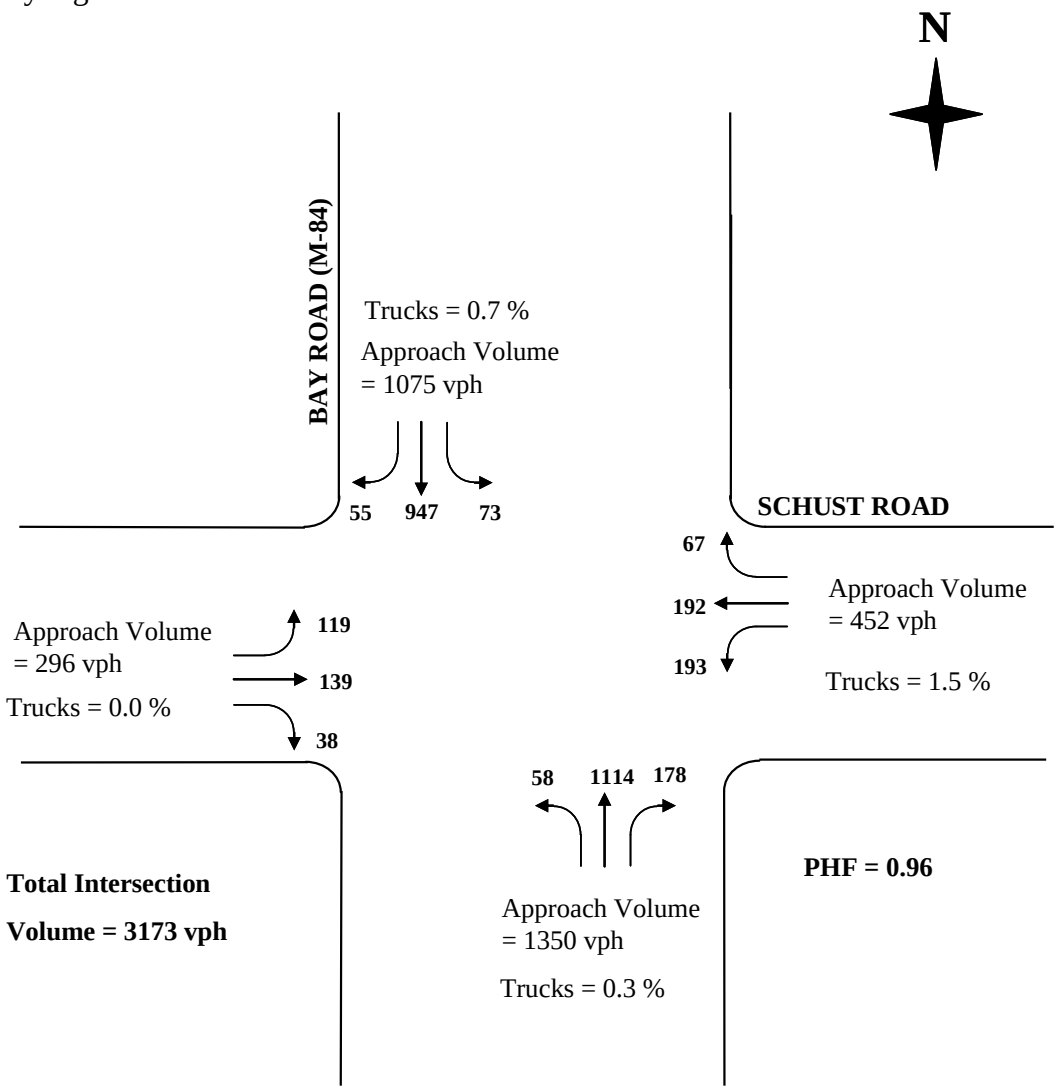


Figure 6.4.2. Existing PM Peak Hour (4:15-5:15 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and Schust Road

Traffic Control

This intersection is controlled by a two-phase, pre-timed signal with a 120-second cycle length during PM Peak Hour. The existing clearance interval includes a yellow interval of 4.5 seconds for the north-south phase and 5.0 seconds for the east-west phase. It has an all-red interval of 1.5 seconds for both the phases. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.4.1.

Crash Analysis

A total of 52 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 26 crashes. Collision diagrams were prepared based on a thorough investigation of the UD-10 reports to identify crash patterns. The collision diagrams for this intersection are shown in Figures 6.4.3 and 6.4.4.

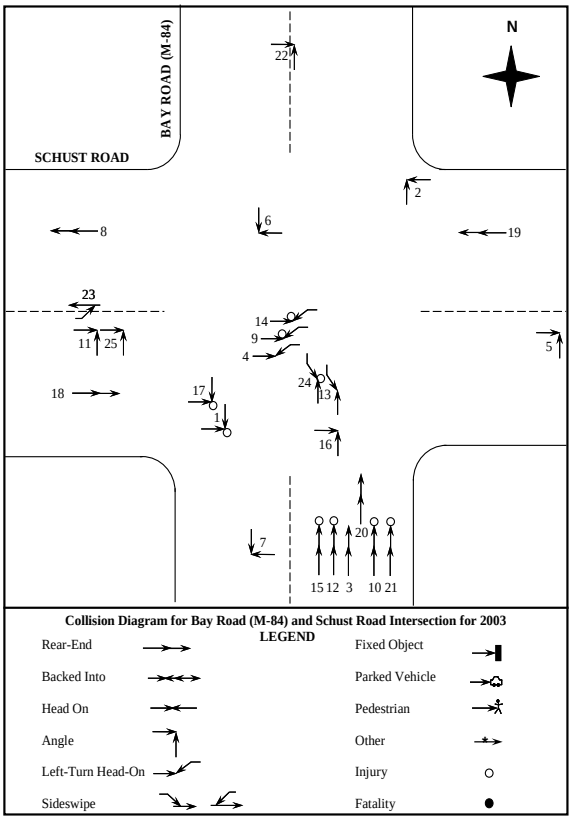


Figure 6.4.3 Collision Diagram for Bay Road (M-84) and Schust Road for 2003

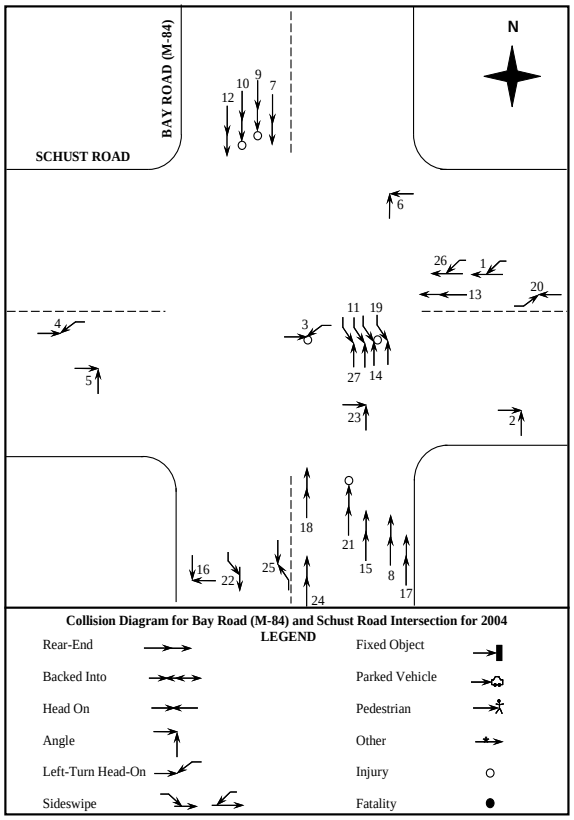


Figure 6.4.4 Collision Diagram for Bay Road (M-84) and Schust Road for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.80 per million entering vehicles, which is nominal, considering the approach volume at the intersection. There are seven left-turn head-on crashes, which may have occurred due to the absence of protected left turn phases. There are also a number of rear-end crashes possibly caused by inadequate clearance intervals and driveway activities near the intersection.

The crash distribution, based upon types of crashes, is shown in Figure 6.4.5. Rear-end, left-turn-head-on and angle crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes include 20 rear-end crashes, 15 angle crashes and 13 left-turn head-on crashes.

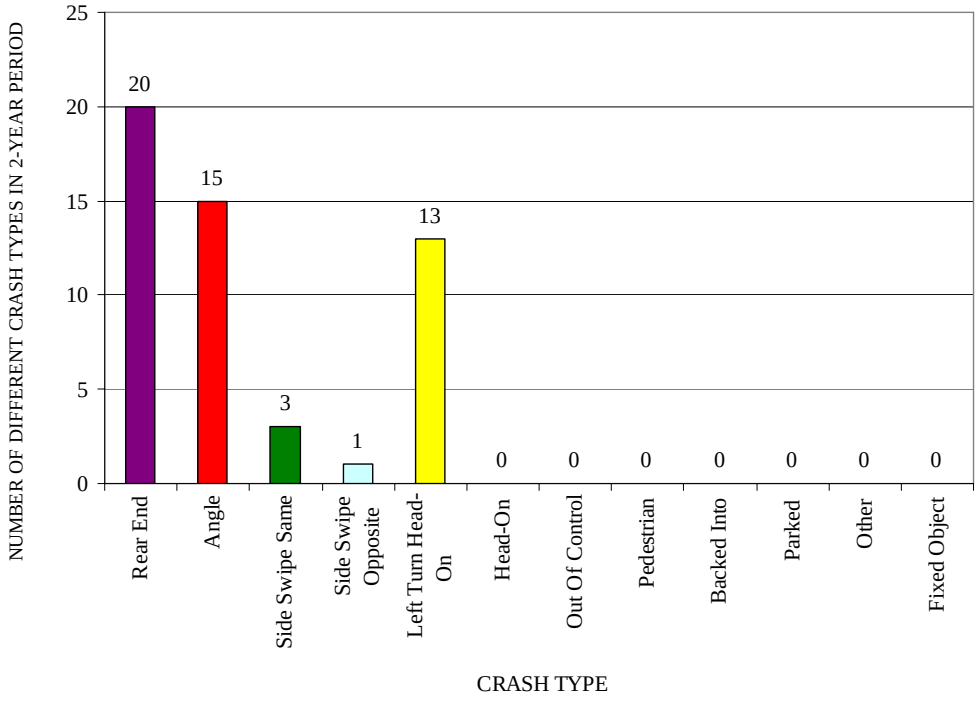


Figure 6.4.5. Two year (2003 & 2004) Crash Experience at Bay Road (M-84) and Schust Road Intersection

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of Bay Road and Schust Road may be attributed to inadequate clearance intervals, absence of protected left-turn phases causing left-turn head-on crashes, driveway activities near the intersection and poor visibility of traffic signals. Eight of the crashes occurred due to driveway movements. To protect the potential left-turn head on crashes, the signal-timing plan should have exclusive left-turn phases. A four-phase signal with a 90-second cycle length is suggested. Improper clearance intervals like excessive yellow and insufficient all red is another cause of rear end crashes at the intersection. Hence, the existing yellow and all-red intervals are revised and shown in Table 6.4.1, along with their potential benefits.

Proper access management plans should be taken to avoid the driveway related crashes. Stops signs should be posted for driveways.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram is shown on Figure 6.4.5.

Table 6.4.1 Existing and Proposed Clearance Intervals for the Bay Road (M-84) and Schust Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S left-turn	-	-	3.9	1.7	Less rear end, angle, left-turn head-on and injury crashes
N-S through and right	4.5	1.5	3.9	1.7	
E-W left-turn	-	-	4.3	2.2	
E-W through and right	5.0	1.5	4.3	2.2	

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds

N-S Bay	N-S Bay Left Turn	E-W Schust	E-W Schust Left Turn
G = 36.4 sec	G = 5.4 sec	G = 18.5 sec	G = 5.5 sec
Y = 3.9 sec	Y = 3.9 sec	Y = 4.3 sec	Y = 4.3 sec
*R = 48.0 sec	*R =79.0 sec	*R = 65.0 sec	*R = 78.0 sec
AR = 1.7 sec	AR = 1.7 sec	AR = 2.2 sec	AR = 2.2 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval to 3.9 seconds for Bay Road (M-84) and 4.3 seconds for Schust Road.
- Increase the all-red interval for Bay Road (M-84) to 1.7 seconds and for Schust Road to 2.2 seconds.
- Provide permissive-protected left turn phases for both north-south and east-west approaches and a 90 second cycle length for PM peak hour to maintain the progression along the Bay Road corridor.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Provide post-mounted signals for left-turning vehicles.
- Relocate the stop bar and crosswalk on all the approaches.
- Upgrade pavement markings on all approaches.
- Provide stop sign for all the driveways located near the intersection.

- Implement the Optimized signal timings for AM peak and off-peak hours

Operational Evaluation:

The results of the level of service analysis for the intersection of Bay Road (M-84) and Schust Road indicate that, for the existing conditions during the PM peak hour, the overall intersection level of service is B with 14.3 seconds of delay per vehicle.

With the modifications of the signal phasing and timing and the noted improvements, the overall intersection level of service becomes C with an increased delay of 22.4 seconds per vehicle. However, the proposed signal timing plan will ensure a better progression along Bay Road (M-84), and provide enhanced safety because of the provision of adequate all-red intervals and protected left-turn phases.

The levels of service results for the existing conditions and with the proposed improvements are shown in Table 6.4.2.

Table 6.4.2
Bay Road (M-84) and Schust Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Schust Road	35.4	D	31.2	C
WB Schust Road	38.5	D	34.0	C
NB Bay Road	7.2	A	19.0	B
SB Bay Road	7.2	A	19.4	B
Overall Intersection	14.3	B	22.4	C

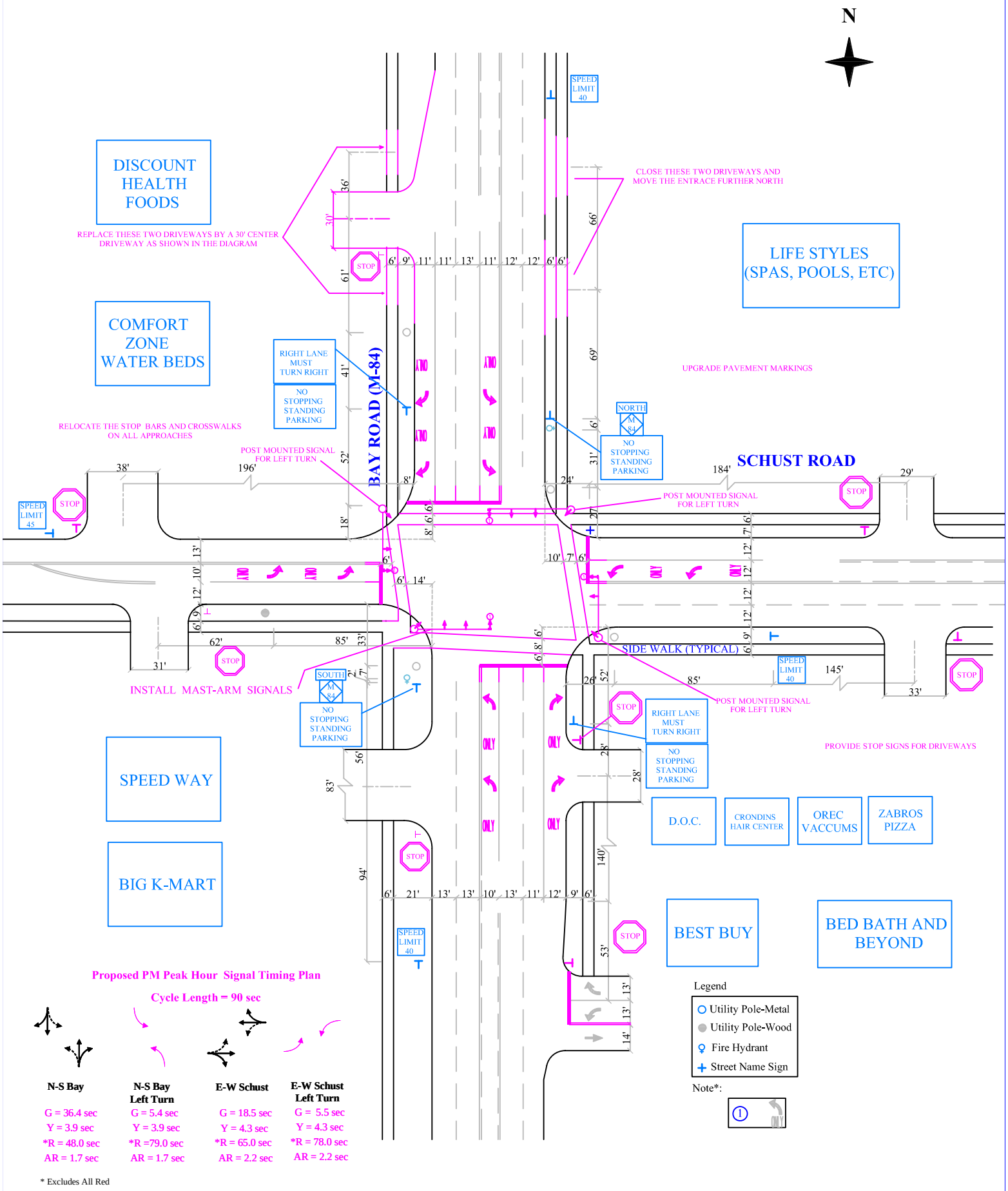


Figure 6.4.5. P roposed Diagram of Bay Road (M-84) and Schust Road Intersection (Not to Scale)

5. Bay Road (M-84) and McCarty Road

The intersection of Bay Road (M-84) and McCarty Road is located in the northeastern part of Saginaw Township. McCarty Road runs in the east-west direction. Bay Road is a state trunk line and it is also known as M-84. Bay Road runs in the north-south direction. McCarty Road is a collector.

Bay Road (M-84), north of the intersection with McCarty Road, has an average daily traffic volume of approximately 29,100 vpd, as per MDOT 2004 data. South of McCarty Road, Bay Road has an average daily traffic volume of approximately 28,600 vpd.

Bay Road is 72 feet wide with one dedicated left-turn lane, two through lanes and one exclusive right-turn lane at both northbound and southbound approaches.

McCarty Road consists of one exclusive left-turn lane and one shared through/right-turn lane in both eastbound and westbound directions. The width of McCarty Road is 37 feet east of the intersection and 38 feet west of the intersection.

The posted speed limit for McCarty Road is 45 mph and for Bay Road it is 40 mph.

Land Use

The intersection of Bay Road and McCarty road has a Mexican restaurant in the northeast corner, some shops in the southeast corner and an SBC office in the northwest corner. There are multiple driveways close to the intersection, which serve the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.5.1. Details of the intersection approaches are shown in photographs 5a through 5d. Photographs 5a and 5b show the northbound and southbound approaches of Bay Road with hanging signal heads from the existing span-wire installation. Photograph 5c shows the eastbound approach of McCarty Road with a wide driveway on its right. Photograph 5d shows the westbound approach of McCarty Road.

Photograph 5a. NB Bay Road at McCarty Road

Photograph 5b. SB Bay Road at McCarty Road

Photograph 5c. EB McCarty Road at Bay Road

Photograph 5d. WB McCarty Road at Bay Road

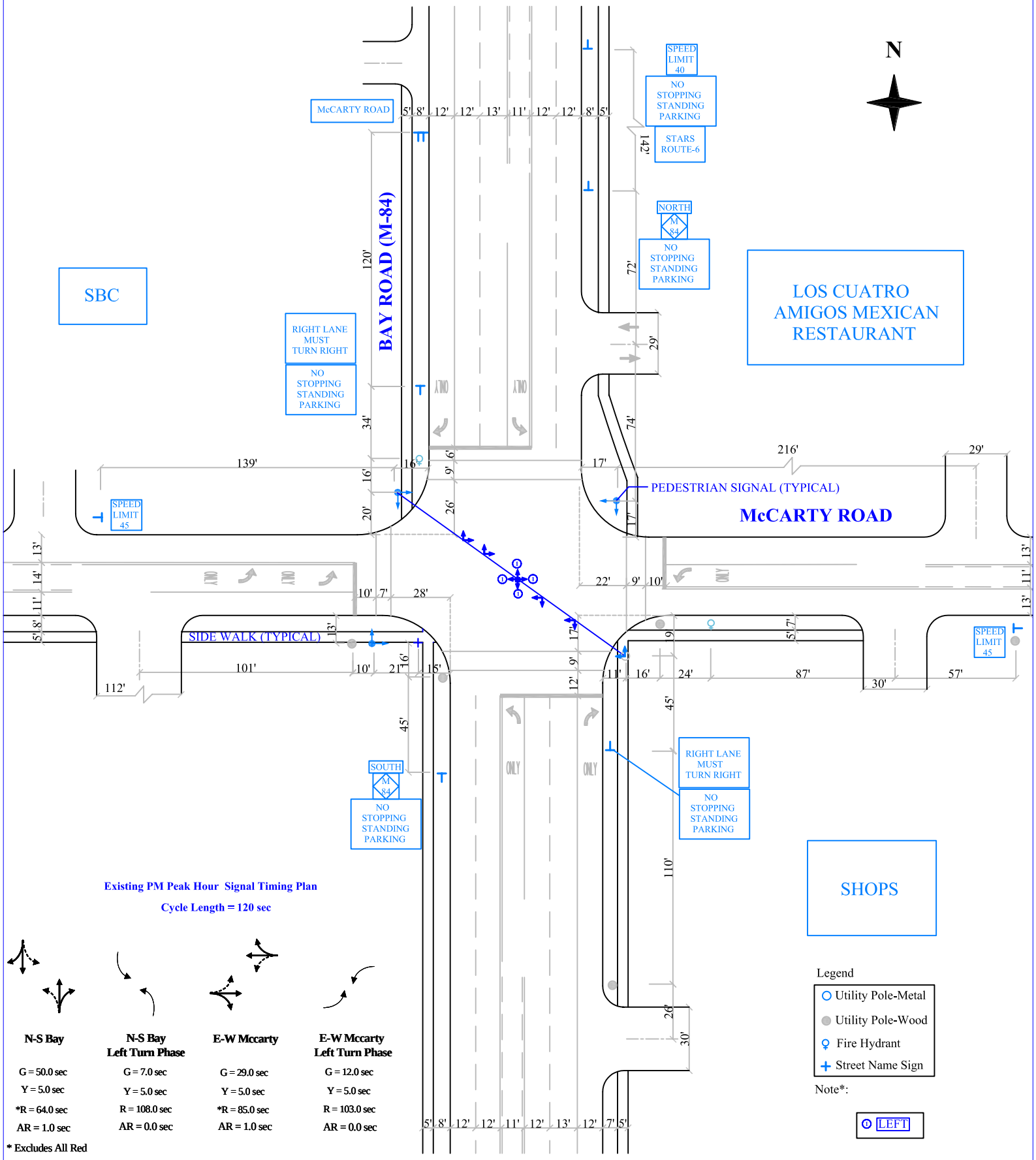


Figure 6.5.1 Existing Condition Diagram of Bay Road (M-84) and McCarty Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes including turning movement counts were manually collected on February 7, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The highest one-hour volumes during the peak period are shown in Figure 6.5.2.

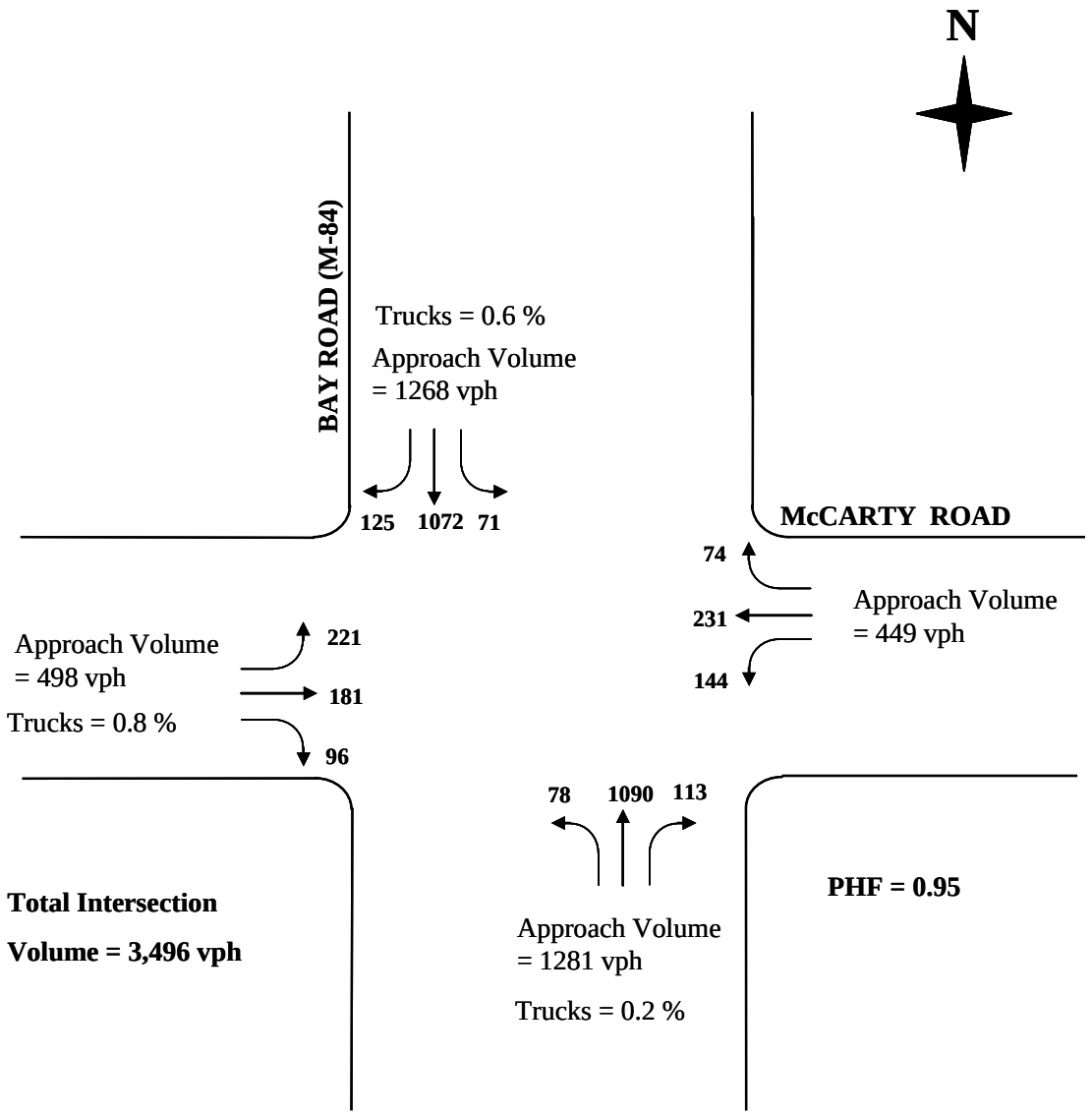


Figure 6.5.2. Existing PM Peak Hour (4:45-5:45 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and McCarty Road

Traffic Control

The intersection is controlled by the traffic signal with a four-phase 120-second cycle length during PM peak hour and the left turn movements are actuated. The existing clearance interval includes a yellow interval of 5.0 seconds for each phases and an all-red interval of 1.0 second for the north-south and east-west through phases. The existing PM peak hour signal timing plan is shown with the existing condition diagram in Figure 6.5.1.

Crash Analysis

A total of 53 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 27 crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. Figures 6.5.3 and 6.5.4 show the collision diagrams for the years 2003 and 2004, respectively.

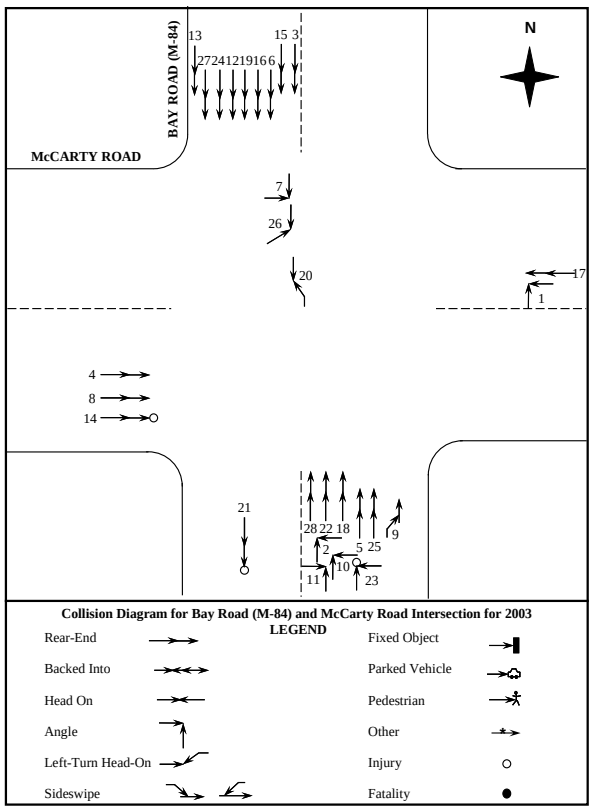


Figure 6.5.3. Collision Diagram for Bay Road and McCarty Road for 2003

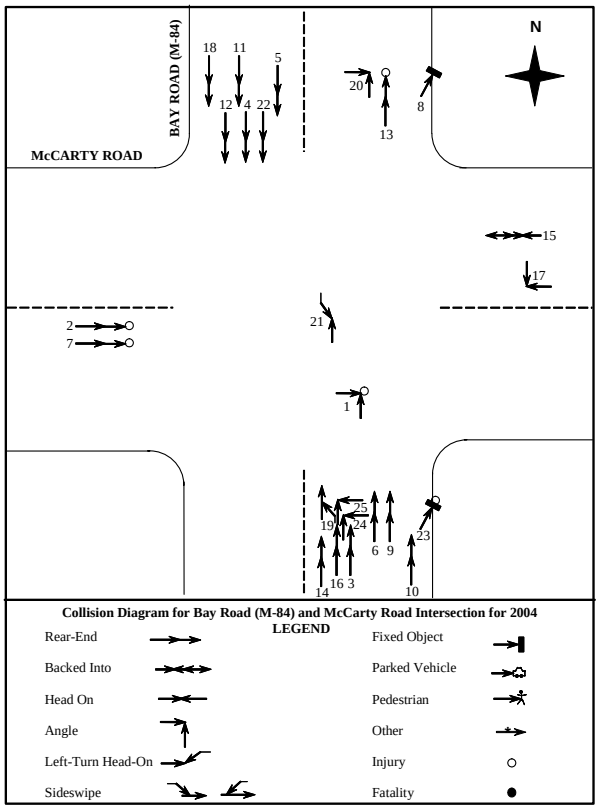


Figure 6.5.4. Collision Diagram for Bay Road and McCarty Road for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.66 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The rear end crashes occurring at the intersection are of specific significance, which may have been caused due to inadequate clearance intervals. There are also nine driveway related angle crashes near the intersection.

The crash distribution, based upon types of crashes, is shown in Figure 6.5.5. Rear-end and angle crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes include 34 rear-end crashes, and 12 angle crashes.

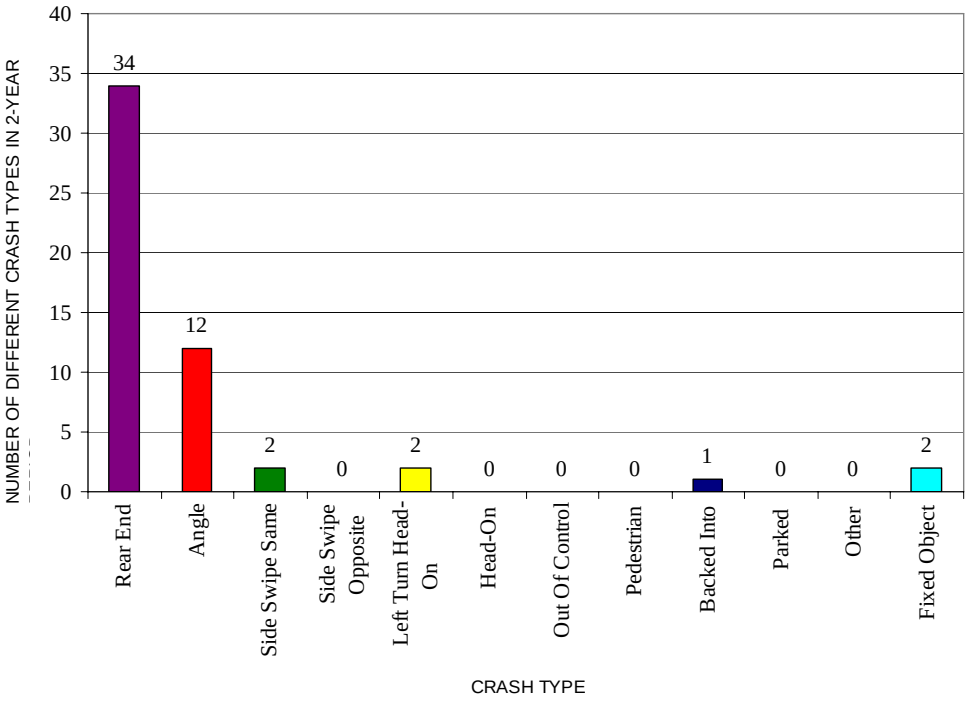


Figure 6.5.5 Two year (2003 & 2004) Crash Experience at Bay Road (M-84) and McCarty Road Intersection

Safety Deficiencies and Recommended Improvements

The crash distribution of this intersection shows a considerable number of rear-end crashes at the northbound and southbound approaches. Also, there are some left-turn-head-on and angle crashes within the intersection. The majority of rear-end and right-angle crashes may have been caused by insufficient clearance intervals, movements from driveways, poor visibility and improper pavement markings. The adequate clearance intervals have been computed and incorporated in the proposed signal-timing plan. To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.5.6. Post-mounted traffic signals with "LEFT" illuminated signs should be installed at all corners to assist left turning vehicles. Long-term policy improvements may include closure of the driveways located near the intersection mitigating hazardous situations for drivers and can be integrated with the access management plan for the Bay Road corridor. The driveways on the north and south legs closer to the intersection may be closed for enhanced safety with the help of local agencies.

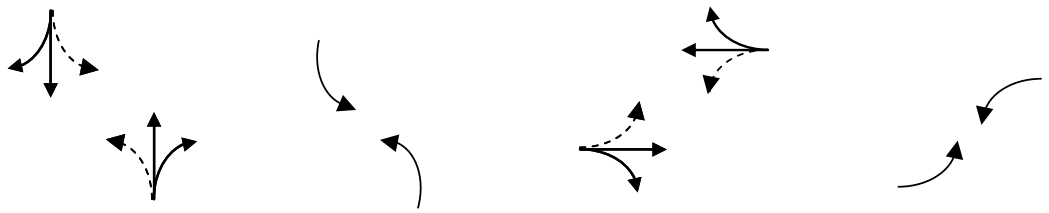
The existing yellow and all-red intervals are shown in Table 6.5.1, along with the proposed intervals and their potential benefits.

Table 6.5.1. Existing and Proposed Clearance Intervals for the Bay Road (M-84) and McCarty Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	5.0	1.0	4.3	2.0	Less rear end, angle, left-turn head-on and injury crashes
E-W Left Turn Phase	5.0	0.0	4.3	2.0	
N-S Through Phase	5.0	1.0	3.9	1.5	
N-S Left Turn Phase	5.0	0.0	3.9	1.5	

The proposed PM peak period signal timing plan based on a 90 second cycle length is shown as follows:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



N-S Bay	N-S Bay Left Turn Phase	E-W McCarty	E-W McCarty Left Turn Phase
G = 34.6 sec	G = 5.6 sec	G = 19.7 sec	G = 6.7 sec
Y = 3.9 sec	Y = 3.9 sec	Y = 4.3 sec	Y = 4.3 sec
*R = 50.0 sec	*R = 79.0 sec	*R = 64.0 sec	*R = 77.0 sec
AR = 1.5 sec	AR = 1.5 sec	AR = 2.0 sec	AR = 2.0 sec

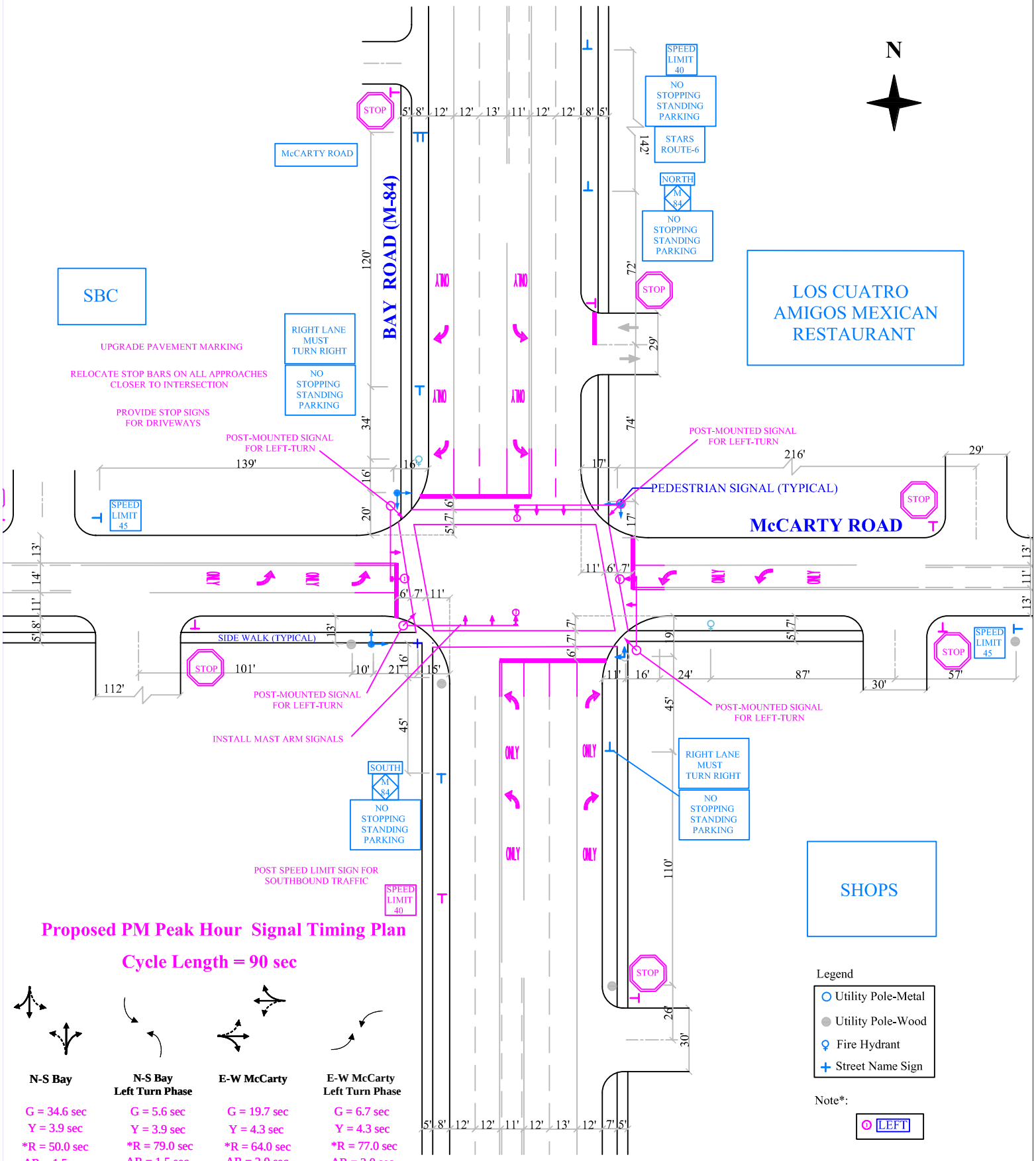
* Excludes All Red

The cycle length of the proposed condition has been changed to 90 seconds to maintain the progression along Bay Corridor. Speed limit signs should also be posted for downstream of the intersection for all approaches where they do not currently exist.

Figure 6.5.6 depicts the proposed improvements including the installation of mast-arm supported signals, upgraded pavement markings and post-mounted traffic signals for the left-turns along with the proposed signal timing plan.

A list of the recommended improvements is summarized below:

- Reduce the yellow interval of Bay Road to 3.9 seconds and that of McCarty Road to 4.3 seconds.
- Increase the all-red interval of Bay Road to 1.5 seconds and that of McCarty Road to 2.0 seconds.
- Post speed limit signs downstream of the intersection all approaches where they do not currently exist.
- Provide upgraded lane markings with proper directional arrows.



Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 sec

N-S Bay	N-S Bay Left Turn Phase	E-W McCarty	E-W McCarty Left Turn Phase
G = 34.6 sec	G = 5.6 sec	G = 19.7 sec	G = 6.7 sec
Y = 3.9 sec	Y = 3.9 sec	Y = 4.3 sec	Y = 4.3 sec
*R = 50.0 sec	*R = 79.0 sec	*R = 64.0 sec	*R = 77.0 sec
AR = 1.5 sec	AR = 1.5 sec	AR = 2.0 sec	AR = 2.0 sec

* Excludes All Red

Figure 6.5.6 Proposed Diagram of Bay Road (M-84) and McCarty Road Intersection (Not to Scale)

Upgrade the existing span-wire configuration to a mast-arm supported or box-span type signal installation.

- Install post-mounted traffic signals with a "LEFT" illuminated sign on all the corners of the intersection for the left-turns.
- Relocate stop bars and crosswalks as per MMUTCD guidelines.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Road (M-84) and McCarty Road indicate that, under the existing conditions during the PM peak hour, the overall intersection level of service is C with a delay of 31.0 seconds per vehicle.

With the modifications implemented to the signal phasing and timing and the noted improvements, the overall intersection level of service will be C with a delay of 28.6 seconds per vehicle. The level of service results for the existing conditions and with the proposed modifications is shown in Table 6.5.2.

Table 6.5.2
Bay Road (M-84) and McCarty Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB McCarty Road	44.2	D	37.9	D
WB McCarty Road	42.6	D	36.7	D
NB Bay Road	26.8	C	27.4	C
SB Bay Road	26.1	C	23.3	C
Overall Intersection	**31.0**	**C**	**28.6**	**C**

6. Bay Road and Universal Drive

The intersection of Bay Road and Universal Drive is located in the eastern part of Saginaw Township. This is a T-intersection where Bay Road and Universal Drive run in the north-south and east-west directions respectively. Bay Road (M-84) is a state trunk line and Universal Drive is classified as a local road.

Bay Road (M-84), near Universal Drive, has an average daily traffic volume of approximately 28,600 vpd per MDOT data (2004).

Bay Road has a left turn center lane, two through lanes and one exclusive right turn lane for the northbound approach, and for the southbound approach, it has one left turn center lane, one through lane, and one shared through and right-turn lane. Universal Drive has one dedicated left turn lane and one right turn lane. Bay Road is 59 feet wide north of the intersection and 71 feet wide south of the intersection. Universal Drive is about 36 feet wide.

The speed limit for Universal Drive is 25 miles per hour and for Bay Road (M-84) it is 40 miles per hour.

Land Use

There is a Blockbuster Video in the northeast corner and a Moving Supply Company in the southeast corner. There is a Mid Michigan Oriental Rugs shop and a Church Supply store along the west side of the intersection. There are multiple driveways close to the intersection, which serve the vehicles going to and from these establishments

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.6.1. Details of the intersection approaches are shown in photographs 6a through 6c. Photograph 6a shows the additional right-turn lane at the northbound approach of Bay Road. Similarly Figure 6c shows the exclusive left turn lane of Universal Drive. The photographs also show the absence of any traffic signal system in this intersection. Certain driveways are also visible in these photographs, which are located close to the intersection.

Photograph 6a. NB Bay Road at Universal Drive

Photograph 6b. SB Bay Road at Universal Drive

Photograph 6c. WB Universal Drive at Bay Road

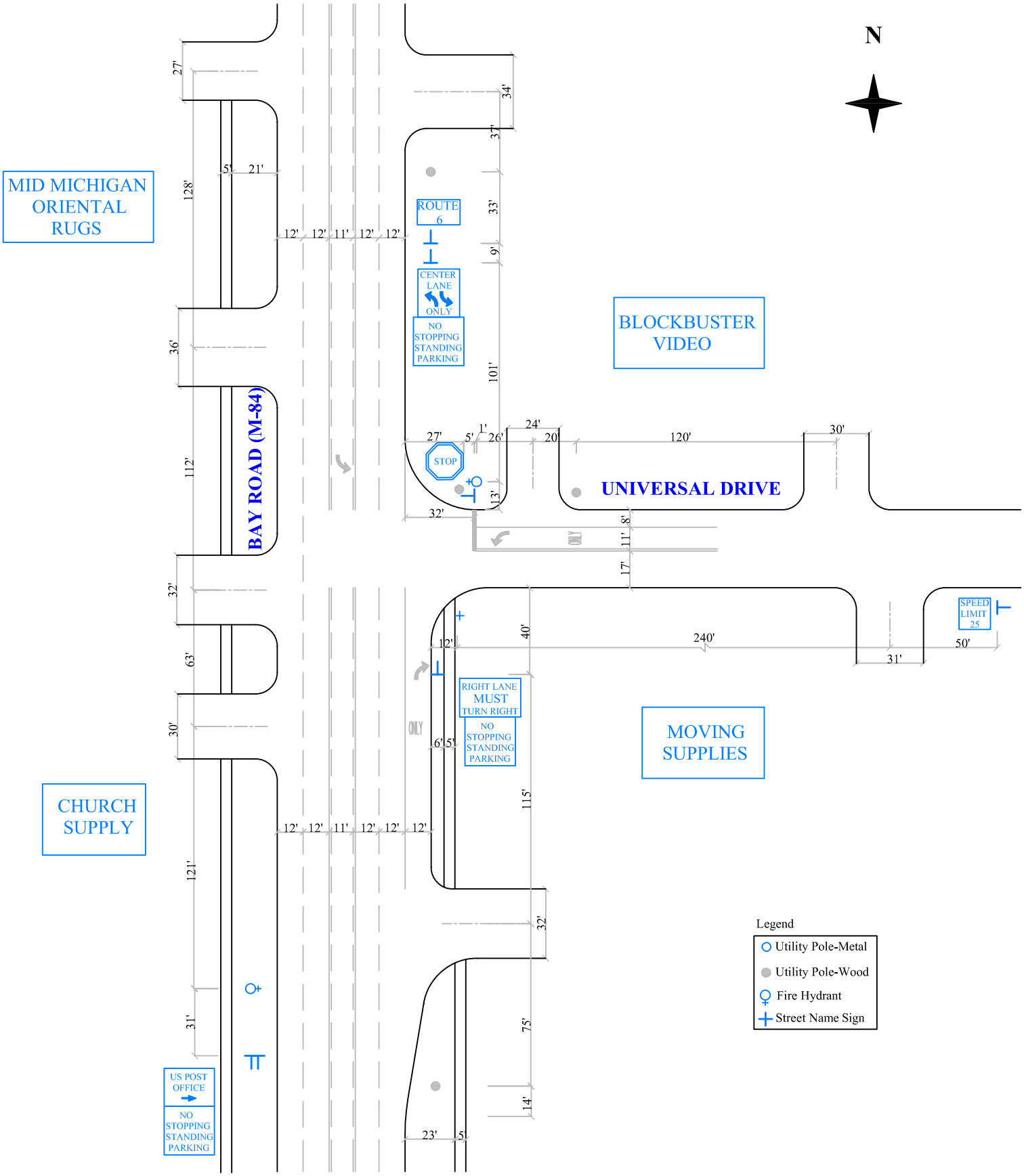


Figure 6.6.1 Existing Condition Diagram of Bay Road (M-84) and Universal Drive Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on January 31, 2006 from 4 PM to 6 PM by two people standing at two suitable corners of the intersection. The results of the data are presented in Figure 6.6.2. The data clearly indicates a considerable traffic volume in the north-south directions and very insignificant flow of traffic to and from the Universal Drive.

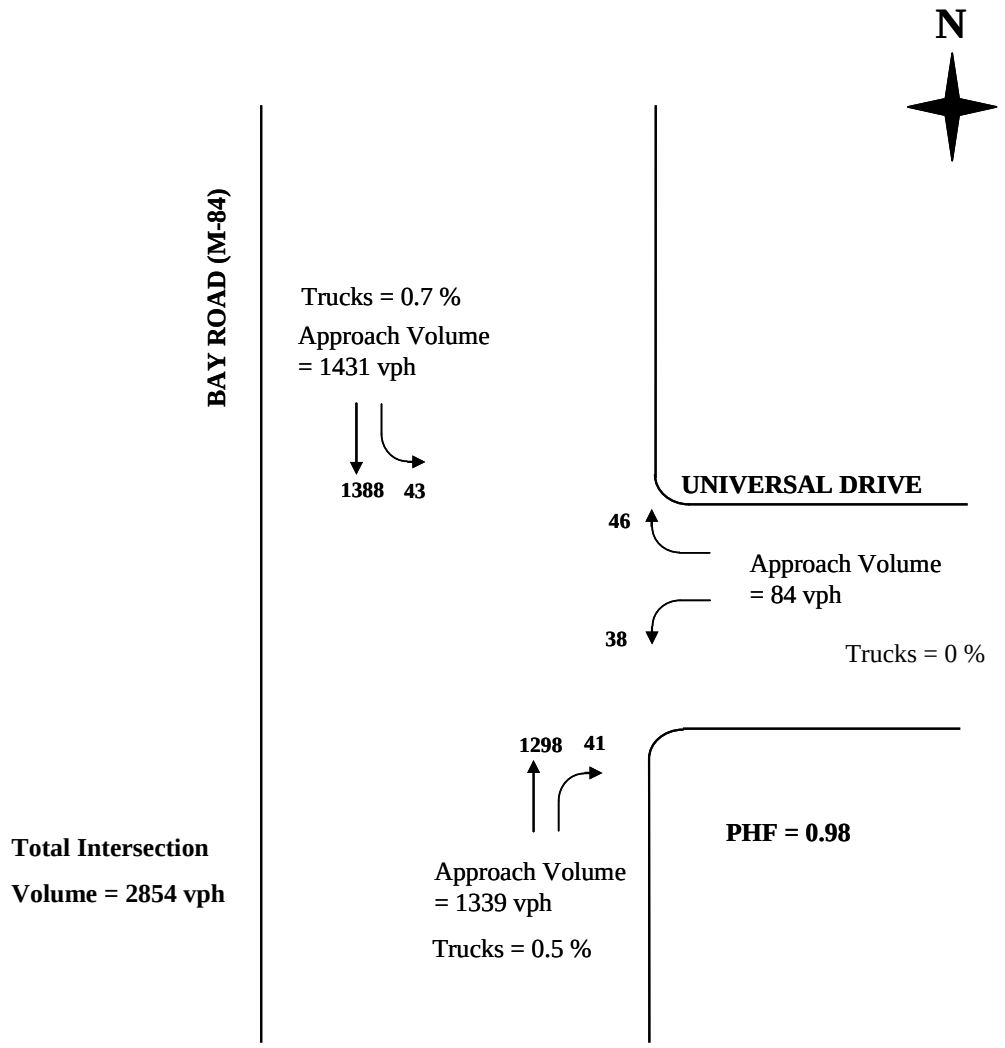


Figure 6.6.2 Existing PM Peak Hour (4:30–5:30 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and Universal Drive

Traffic Control

This is an unsignalized intersection and is controlled by a one-way stop control sign. The stop sign is for all traffic entering Bay Road from Universal Drive and is placed on the northeast corner of the intersection.

Crash Analysis

A total of 3 crashes in the year 2003 were recorded. In 2004 there were no crashes recorded. A Collision diagram was prepared based on the UD-10 reports to identify crash patterns. The collision diagram for the intersection based on 2003 data is shown in Figure 6.6.3.

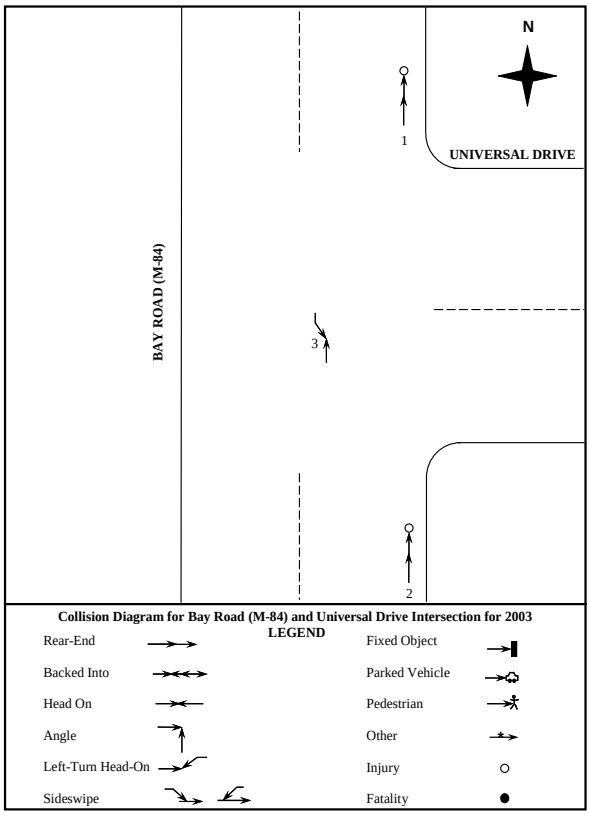


Figure 6.6.3. Collision Diagram for Bay Road and Universal Drive for 2003

Safety Deficiencies and Recommended Improvements

The intersection of Bay Road and Universal Drive had two rear-end and one head-on left-turn crashes in 2003. The traffic movement from Universal Drive to Bay Road is insignificant as per the volume count data. A pedestrian crosswalk should be provided on

Universal Drive. Speed limit signs should be provided downstream of the intersection for northbound and southbound traffic to provide better guidance. The recommended improvements are shown in the proposed intersection diagram as shown in Figure 6.6.6.

A list of the recommended improvements is summarized below:

- Provide additional pavement markings with proper directional arrows for turning traffic.
- Provide pedestrian crosswalk on Universal Drive.
- Post speed limit signs downstream of the intersection for northbound and southbound traffic.
- Provide lane markings for Universal Drive.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

Operational Evaluation:

The level of service analysis by HCS for the intersection of Bay Road and Universal Drive indicates that for the existing conditions during the PM peak hour, Bay Road operates at Level of Service A,where as Universal Drive operates at Level of Service F. The level of service results for the existing condition is shown in Table 6.6.1.

Table 6.6.1 Bay Road and Universal Drive
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions	
	Delay (Sec/Vehicle)	LOS
NB Bay Road	0	A
SB Bay Road	0.38	A
WB Universal Drive	134.9	F
Overall Intersection	**NA**	**NA**

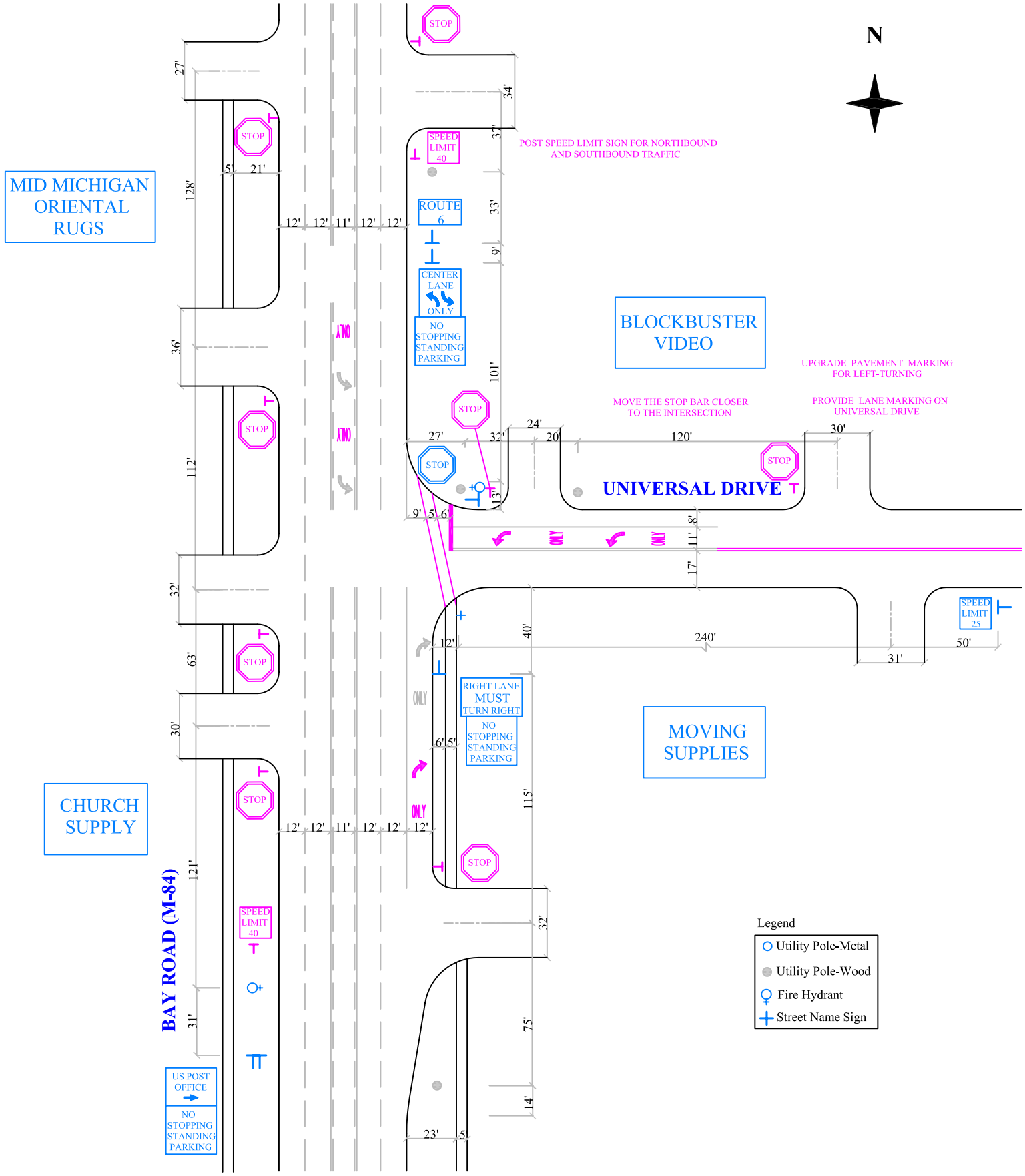


Figure 6.6.6 Proposed Diagram of Bay Road (M-84) and Universal Drive Intersection (Not to Scale)

7. Bay Road (M-84) and Shattuck Road

The intersection of Bay Road (M-84) and Shattuck Road is located in the southern part of Saginaw Township. Bay Road runs in the north-south direction and Shattuck Road runs in the east-west direction. Bay Road is a state trunk line (M-84) and Shattuck Road is classified as a collector.

Bay Road has an average daily traffic volume of approximately 28,600 vpd north of the intersection and 29,100 vpd south of the intersection at this location, as per MDOT (2004).

Bay Road is 75 feet wide on the north leg of the intersection and it is 82 feet on the south leg of the intersection. At the southbound approach, Bay Road has one dedicated left turn lane, one dedicated right-turn lane and two through lanes. At the northbound approach, it has one exclusive left turn lane, one through lane and one shared through and right-turn lane. East of the intersection, Shattuck Road is 59 feet wide with one exclusive left-turn lane, one through lane and one shared through and right turn lane. West of the intersection, Shattuck Road has one exclusive left turn, one through lane and one exclusive right turn lane with a total width of 59 feet.

The speed limit for Bay Road is 40 miles per hour and for Shattuck Road it is 35 miles per hour.

Land Use

There is a Rite Aid Pharmacy and a Walgreens Pharmacy on the northeast and southeast corners of the intersection, respectively. On the northwest corner of the intersection, there are some commercial buildings and on the southwest corner there is a Burger King and a Redmond's. There are multiple driveways close to the intersection, which serve the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside

obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.7.1. Photographs of each approach leg of the intersection are shown in Photographs 7a through 7d. All the photographs show the span wire signal installation and pavement markings.

Photograph 7a. EB Shattuck Road at Bay Road

Photograph 7b. WB Shattuck Road at Bay Road

Photograph 7c. NB Bay Road at Shattuck Road

Photograph 7d. SB Bay Road at Shattuck Road

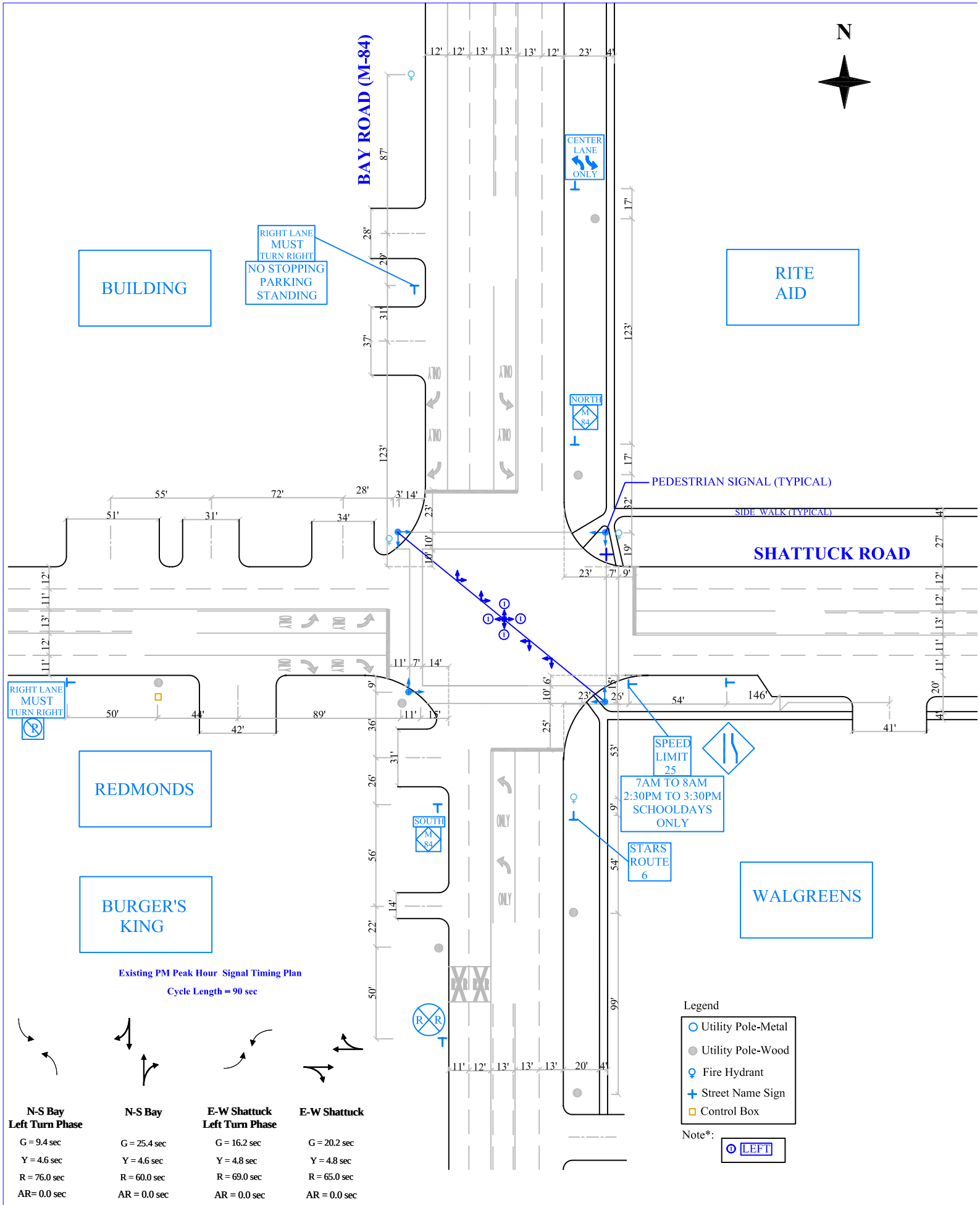


Figure 6.7.1 Existing Condition Diagram of Bay Road (M-84) and Shattuck Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on a clear day on January 31, 2006 from 4:00 PM to 6:00 PM by two people standing at suitable corners of the intersection and covering all the approaches. The results of the data are presented in Figure 6.7.2. There are high through traffic volumes in the north-south directions along with heavy right-turning traffic from the southbound approach. The left turning volume from east-west Shattuck Road to Bay Road is also quite high.

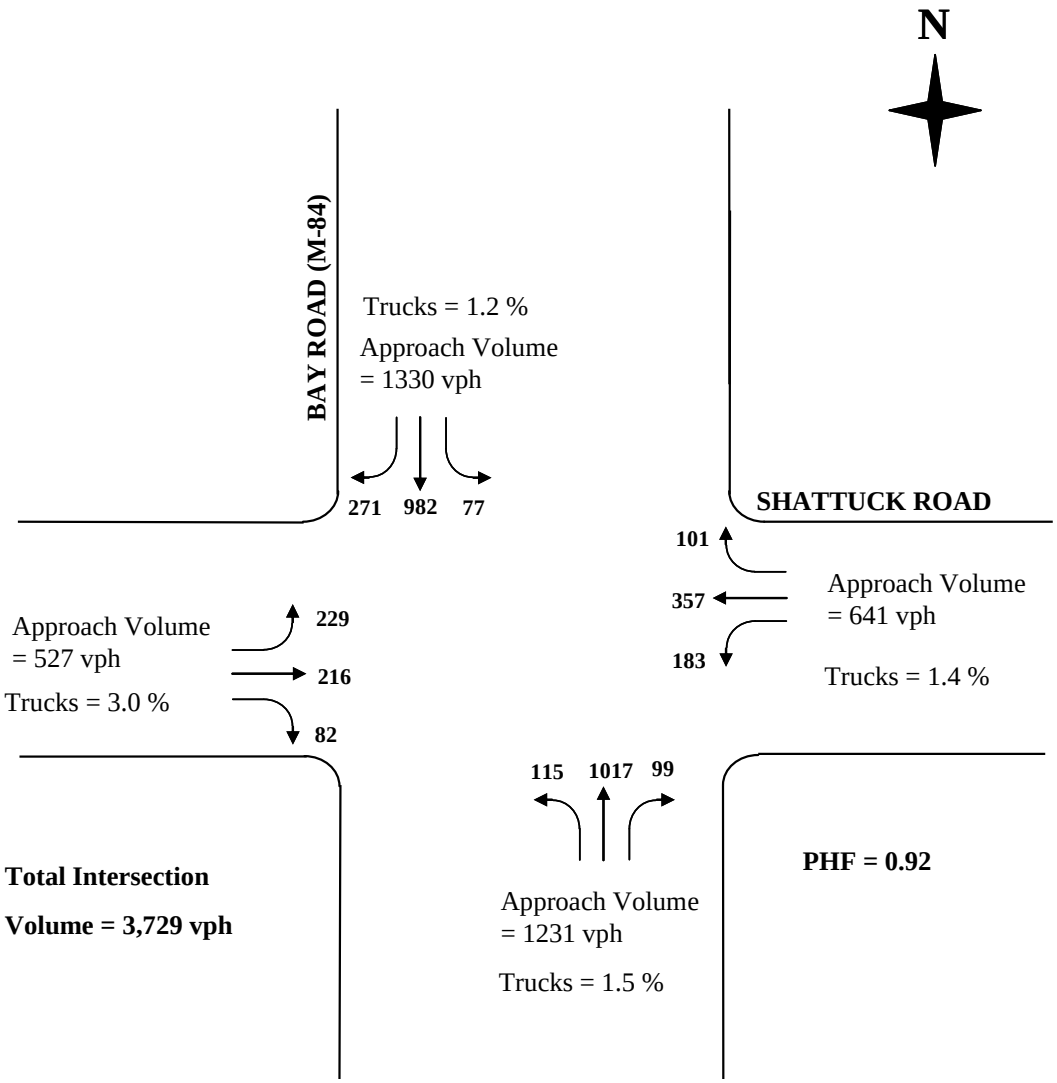


Figure 6.7.2. Existing PM Peak Hour (4:00-5:00 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and Shattuck Road

Traffic Control

The basic signal timing plan is given by a four phase design for Bay Road and Shattuck Road which operates on a 90-second cycle length during the PM peak hour. The north-south phases have a yellow interval of 4.6 seconds and the east-west phases have a yellow interval of 4.8 seconds. There is no all-red interval in the signal timing plan. The intersection has pedestrian signals and marked crosswalks on all approach legs.

Crash Analysis

A total of 58 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of twenty-nine crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. Figures 6.7.3 and 6.7.4 show the collision diagrams for the years 2003 and 2004, respectively.

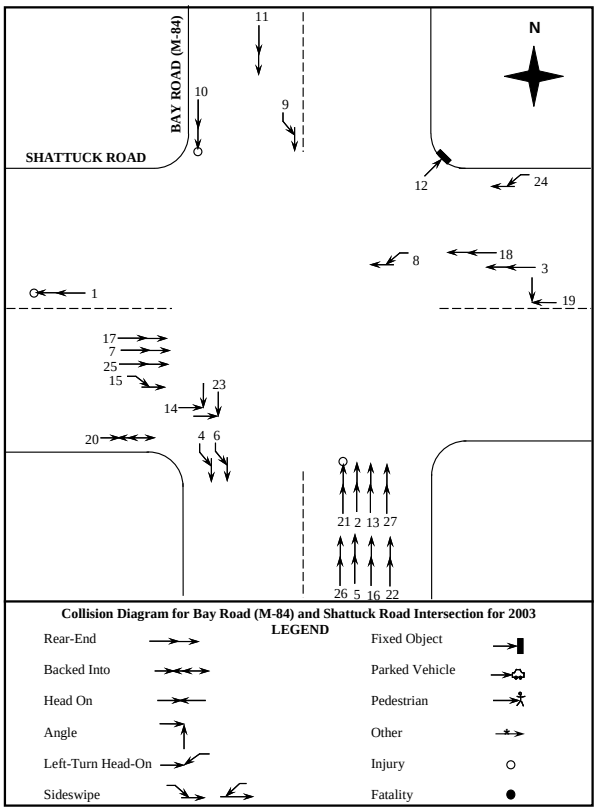


Figure 6.7.3 Collision Diagram for Bay Road (M-84) and Shattuck Road for 2003

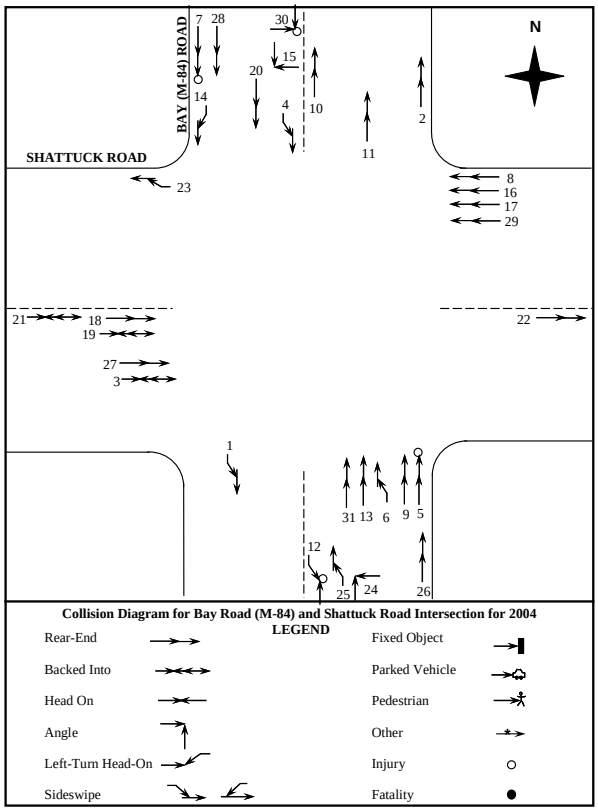


Figure 6.7.4 Collision Diagram for Bay Road (M-84) and Shattuck Road for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.7 per million entering vehicles, which is nominal, taking into account the approach volume at the intersection. Rear-end collisions are predominant at the intersection. They may happen due to inadequate signal timings and clearance intervals.

The crash distribution, based upon type of crashes, is shown in Figure 6.7.5. Rear-end, side-swipe-same and angle crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes include 34 rear-ends crashes, 13 sideswipe crashes and six angle crashes.

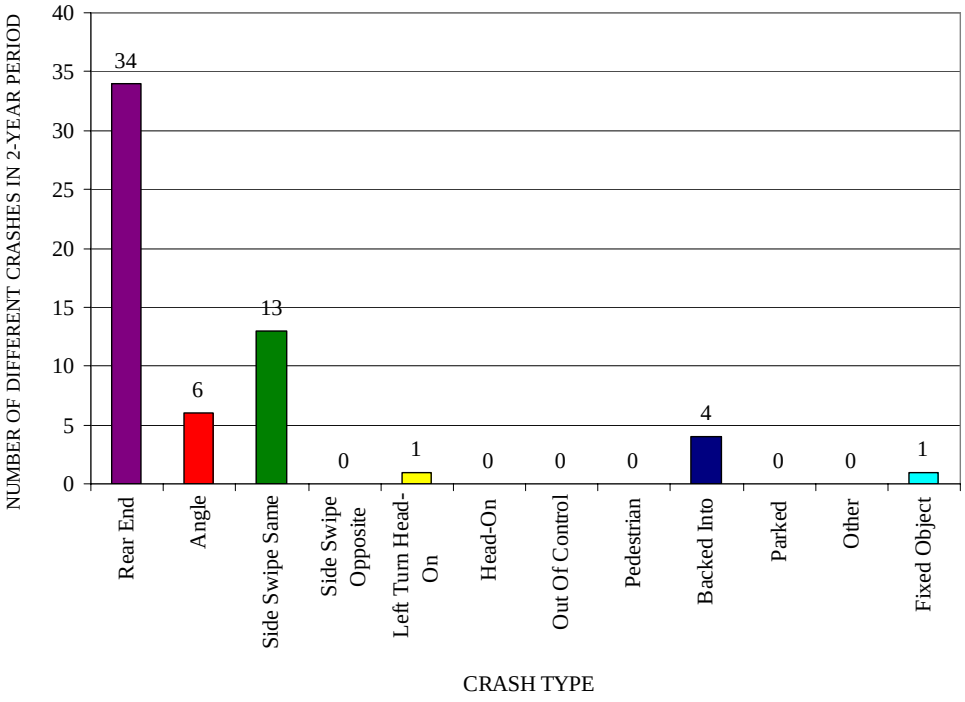


Figure 6.7.5 Two year (2003 & 2004) Crash Experience at Bay Road (M-84) and Shattuck Road Intersection

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of Bay Road and Shattuck Road could be attributed to signal timing and clearance interval inadequacies and the presence of driveways near the intersection. There has been no all red interval for the existing signal-timing plan. So, the recommended clearance intervals are calculated and provided in the proposed signal-timing plan. The existing signal timing plan is revised for desired signal progression. A cycle length of 90 second is recommended. The eastbound exclusive through lane and westbound exclusive left turn lane should be marked with two sets of 'through only' and 'left only' pavement markings respectively. The existing span-wire signal should be replaced with a mast-arm traffic signal installation. Additionally, left-turn post-mounted signals with left-illuminated sign are recommended. Speed limit signs should also be posted downstream of the intersection for all approaches where they do not currently exist. The modified signal timing plan is shown in the proposed intersection diagram as shown in Figure 6.7.6.

The existing yellow and all-red intervals are shown in Table 6.7.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.7.1. Existing and Proposed Clearance Intervals for the Bay Road and Shattuck Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Left-Turn Phase	4.8	0.0	3.9	1.8	Less rear end, angle and injury crashes
E-W Through Phase	4.8	0.0	3.9	1.8	
N-S Left-Turn Phase	4.6	0.0	3.6	2.6	
N-S Through Phase	4.6	0.0	3.6	2.6	

The proposed PM Peak hour signal timing plan for this intersection is shown below:

Cycle Length = 90 Seconds

Proposed PM Peak Hour Signal Timing Plan

N-S Bay	N-S Bay Left Turn Phase	E-W Shattuck	E-W Shattuck Left Turn Phase
G = 35.3 sec	G = 4.3 sec	G = 16.8 sec	G = 9.8 sec
Y = 3.9 sec	Y = 3.9 sec	Y =3.6 sec	Y = 3.6 sec
*R = 49.0 sec	*R = 80.0 sec	*R = 67.0 sec	*R = 74.0 sec
AR = 1.8 sec	AR= 1.8 sec	AR = 2.6 sec	AR = 2.6 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Bay Road to 3.9 seconds and for Shattuck Road to 3.6 seconds.
- Increase the all-red clearance interval for Bay Road to 1.8 seconds and for Shattuck Road to 2.6 seconds.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at all the corners for the left-turns.
- Replace the existing span-wire signal with a box-span type or mast-arm supported system.
- Relocate the stop bars and crosswalks as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel to guide the motorist to drive within the speed limit.
- Optimize the signal timings for AM peak and off-peak hours.
- Provide permissive protected left-turn phase instead of lead protected phases.

- Provide two sets of left only pavement markings on the westbound left-turning lane.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.

The proposed intersection diagram is shown in Figure 6.7.6 and shows various proposed recommendations.

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Road and Shattuck Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is E with a delay of 77.8 seconds per vehicle.

With the modifications to the signal timing and phasing including a change of cycle length to 90 seconds and the noted improvements, the overall intersection level of service will be improved to C with a delay of 28.9 seconds per vehicle. This reduced delay will result in enhanced safety and better progression.

The levels of service results for the existing conditions and with the proposed improvements are shown in Table 6.7.2.

Table 6.7.2.

Bay Road and Shattuck Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements with Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Shattuck Road	75.5	E	36.8	D
WB Shattuck Road	44.0	D	35.7	D
NB Bay Road	113.8	F	26.2	C
SB Bay Road	61.1	E	25.0	C
Overall Intersection	**77.8**	**E**	**28.9**	**C**

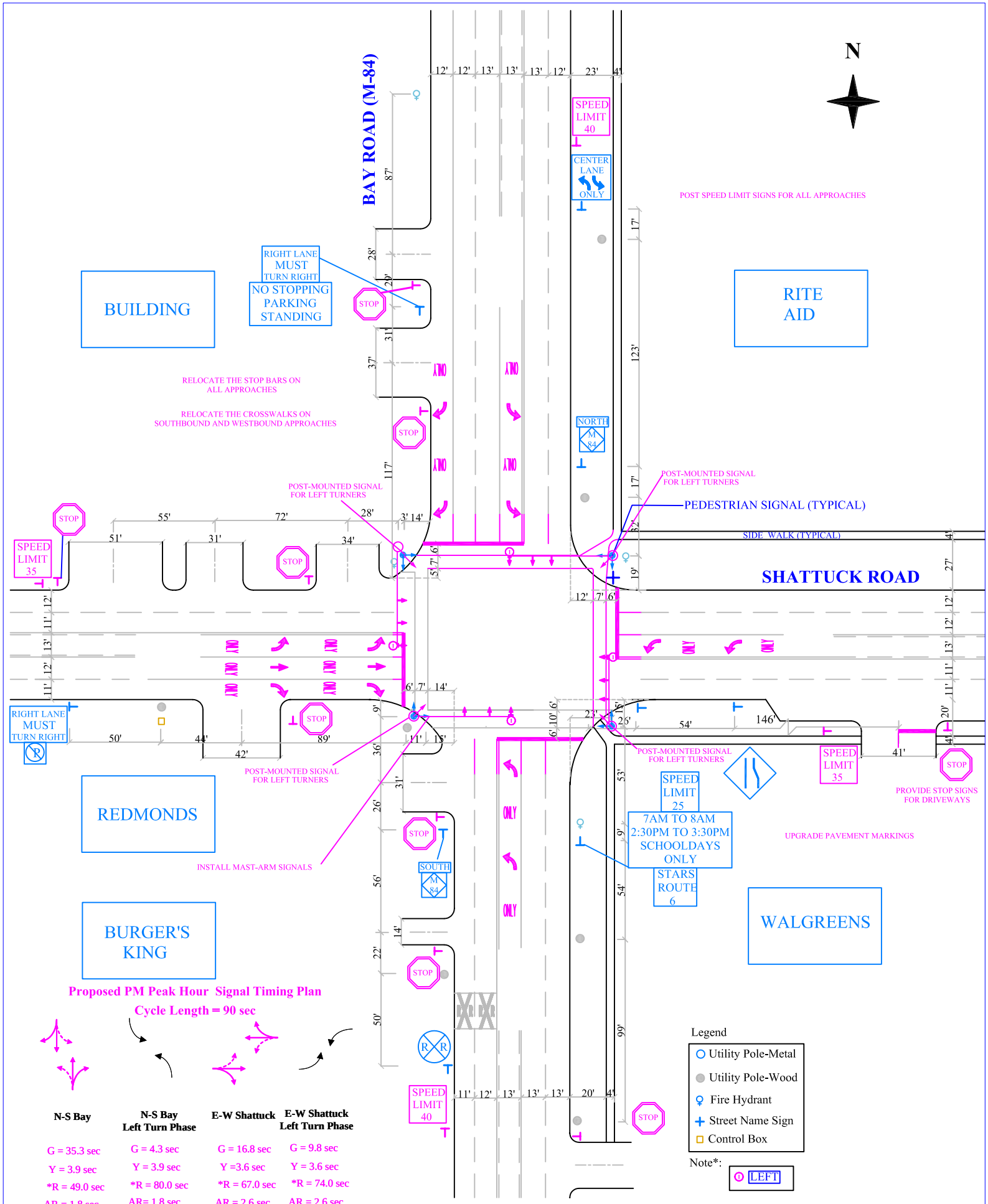


Figure 6.7.6 Proposed Diagram of Bay Road (M-84) and Shattuck Road Intersection (Not to Scale)

8. Bay Road (M-84) and Weiss Street

The intersection of Bay Road and Weiss Street is located in the northern part of the City of Saginaw. Bay Road runs in the north-south direction and Weiss Street runs in the east-west direction. Bay Road is a state trunk line and it is known as M-84, whereas Weiss Street is classified as a collector.

Bay Road, near the intersection with Weiss Street has an average daily traffic volume of approximately 29,100 vpd north of the intersection and 23,100 vpd south of the intersection as per MDOT data (2004).

Bay Road is 59 feet wide, north of the intersection with one dedicated left-turn lane, one through lane and one shared through/right-turn lane. South of the intersection, it is 56 feet wide. Weiss Street is 44 feet wide, east of the intersection and has three exclusive lanes for right turning, through and left turning traffic. West of the intersection Weiss Street is 45 feet wide with one exclusive left-turn lane and one shared through/right-turn lane.

The speed limit of Bay Road is 40 miles per hour of Weiss Street and 35 miles per hour south. For Weiss Road, it is 30 miles per hour.

Land Use

There is a Family Video Shop on the southeast corner of the intersection. On the southwestern corner of the intersection, there is a Total Solutions Group. Patisseries and Acdelco are located on the northwestern corner of the intersection. An income tax office is located on the northeastern corner of the intersection. There are multiple driveways located close to the intersection, which serve the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.8.1. Details of the intersection approaches are shown in photographs 8a through 8d. Photographs show the hanging signal heads with span-wire installations as well as the pavement markings in place.

Photograph 8a. NB Bay Road at Weiss Street

Photograph 8b. SB Bay Road at Weiss Street

Photograph 8c. EB Weiss Street at Bay Road

Photograph 8d. WB Weiss Street at Bay Road

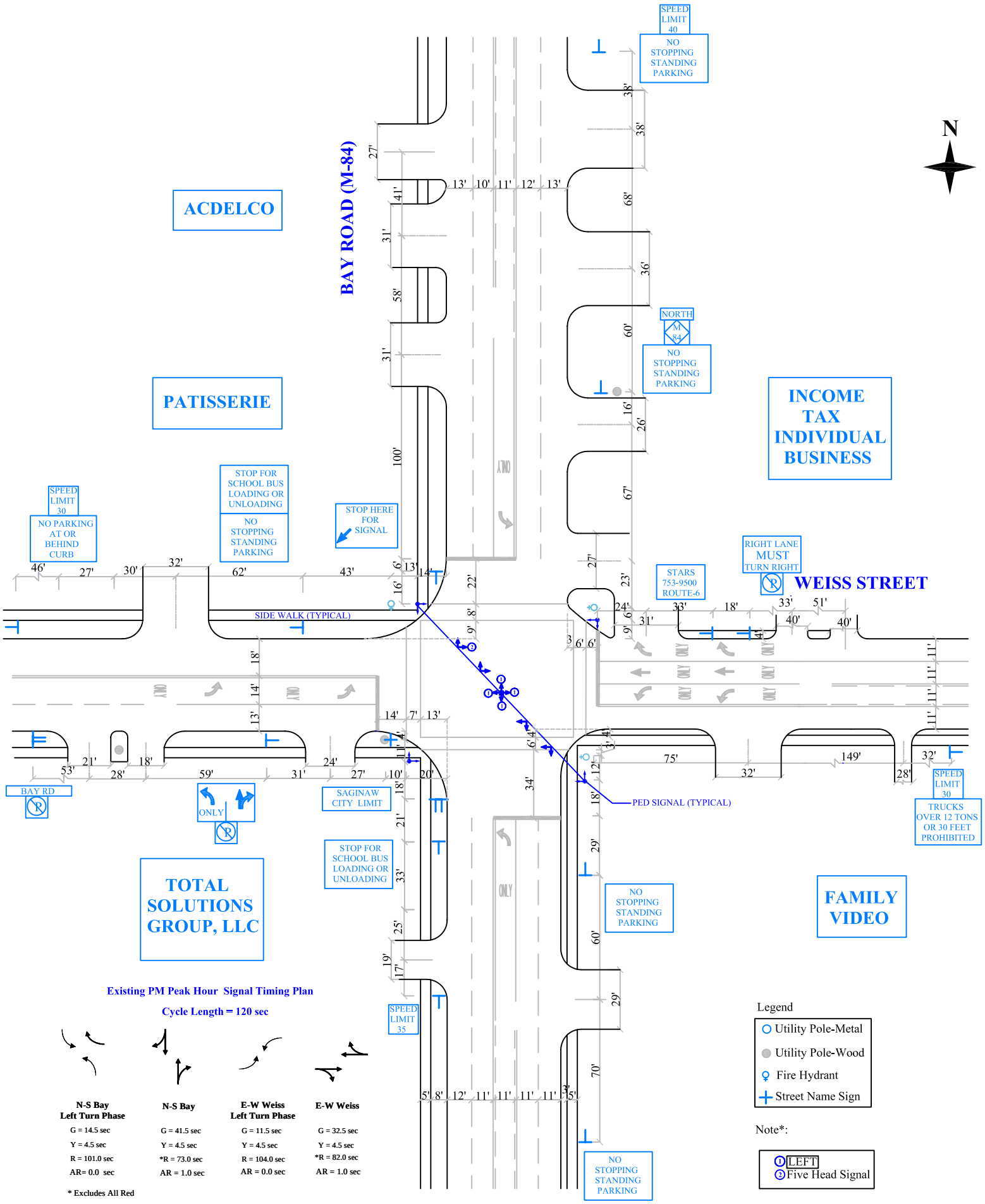


Figure 6.8.1 Existing Condition Diagram of Bay Road (M-84) and Weiss Street Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were collected manually on a clear day on January 31, 2006 from 4 PM to 6 PM by two people standing at a suitable corner of the intersection and covering two intersecting approaches each. The results of the data are presented in Figure 6.8.2.

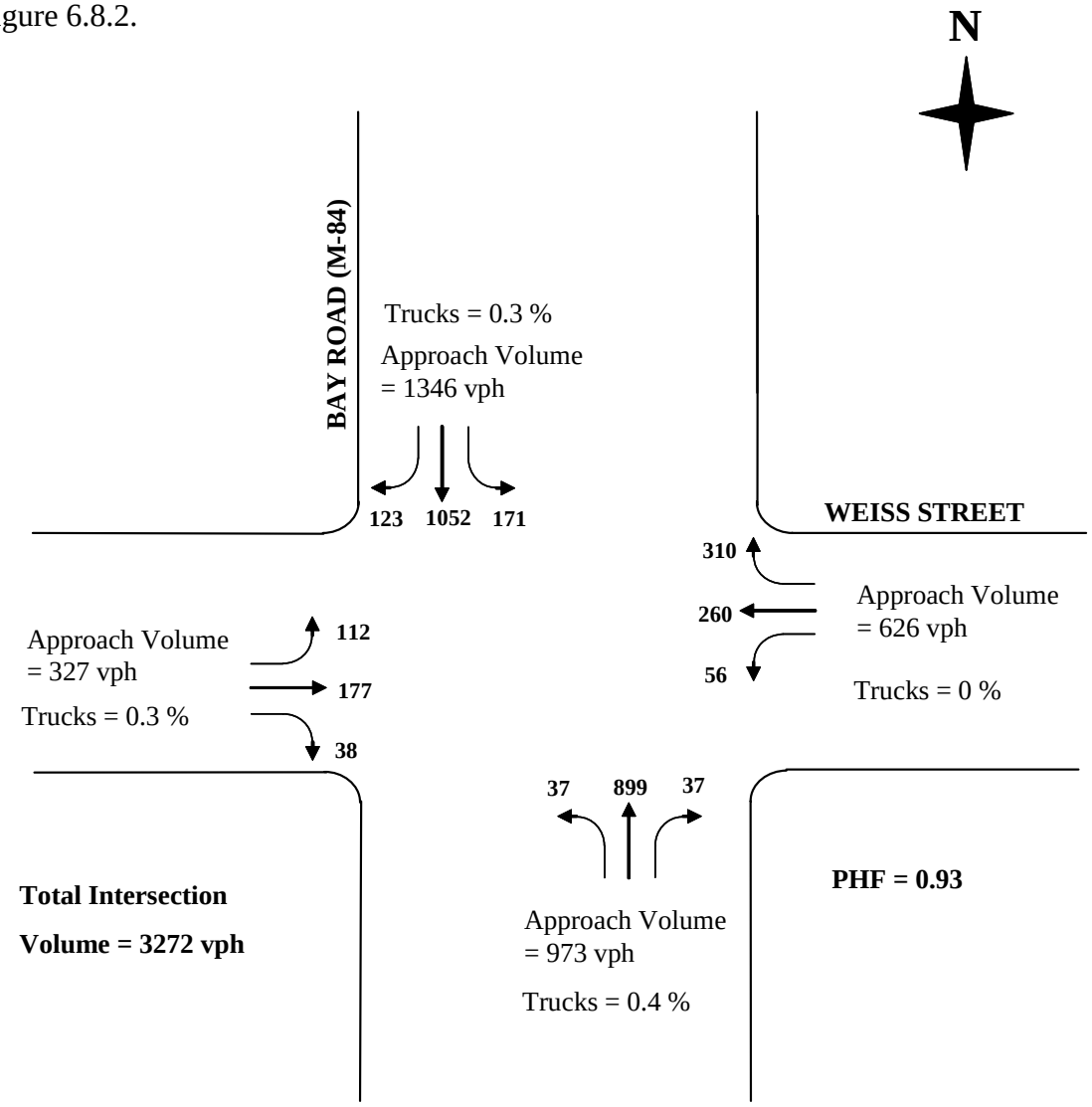


Figure 6.8.2 Existing PM Peak Hour (4:30–5:30 PM) Traffic Volumes for the Intersection of Bay Road (M-84) and Weiss Street

Traffic Control

The basic signal timing plan is given by a four phase design for Bay Road and Weiss Road which operates on a 90-second cycle length during the PM peak hour. The existing clearance interval includes a yellow interval of 4.5 seconds for each phase and an all-red interval of 1.0 second for the north-south and east-west through phases. An all-red interval is not provided for the left-turn phases. The existing PM peak hour signal timing plan is shown on the existing condition diagram in figure 6.8.1.

Crash Analysis

A total of 41 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 21 crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. Figures 6.8.3 and 6.8.4 show the collision diagrams for the years 2003 and 2004, respectively.

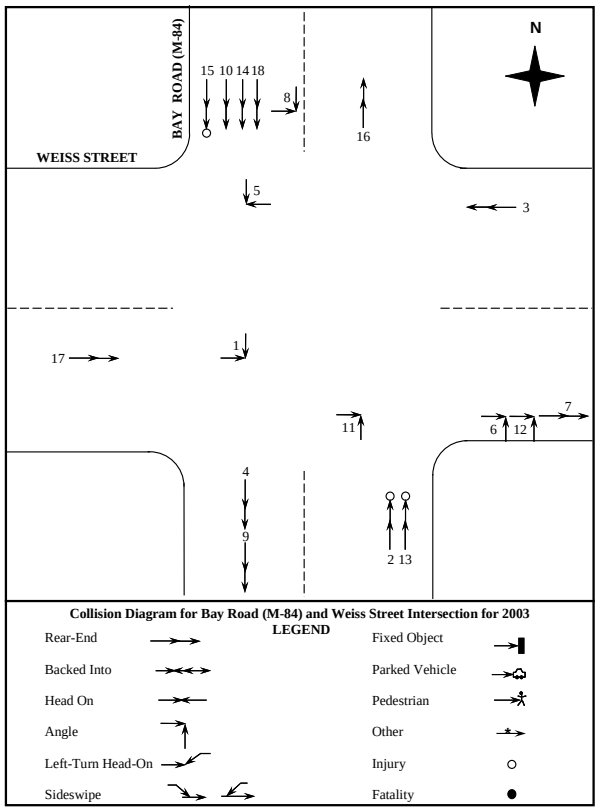


Figure 6.8.3. Collision Diagram for Bay Road (M-84) and Weiss Street for 2003

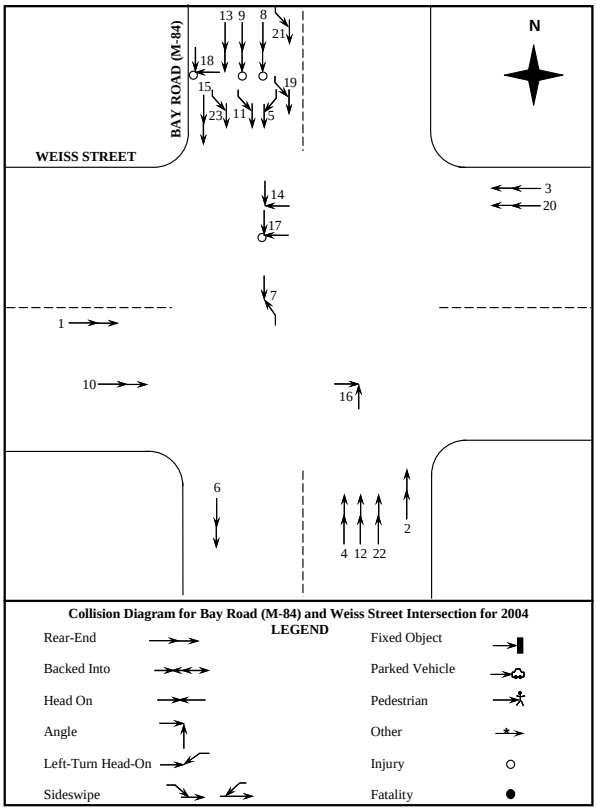


Figure 6.8.4. Collision Diagram for Bay Road (M-84) and Weiss Street for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.41 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The rear end crashes occurring at the intersection are of specific significance, which may have been caused by the inadequate signal timing and sudden stops near the intersection. There are also some angle crashes at the intersection, which might have occurred due to an improper clearance interval and red-light running. Some of the angle crashes might have occurred due to traffic movements at driveways.

The crash distribution, based upon types of crashes, is shown in Figure 6.8.5. Rear end and angle crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequencies for these crashes include 25 rear-end crashes and ten angle crashes.

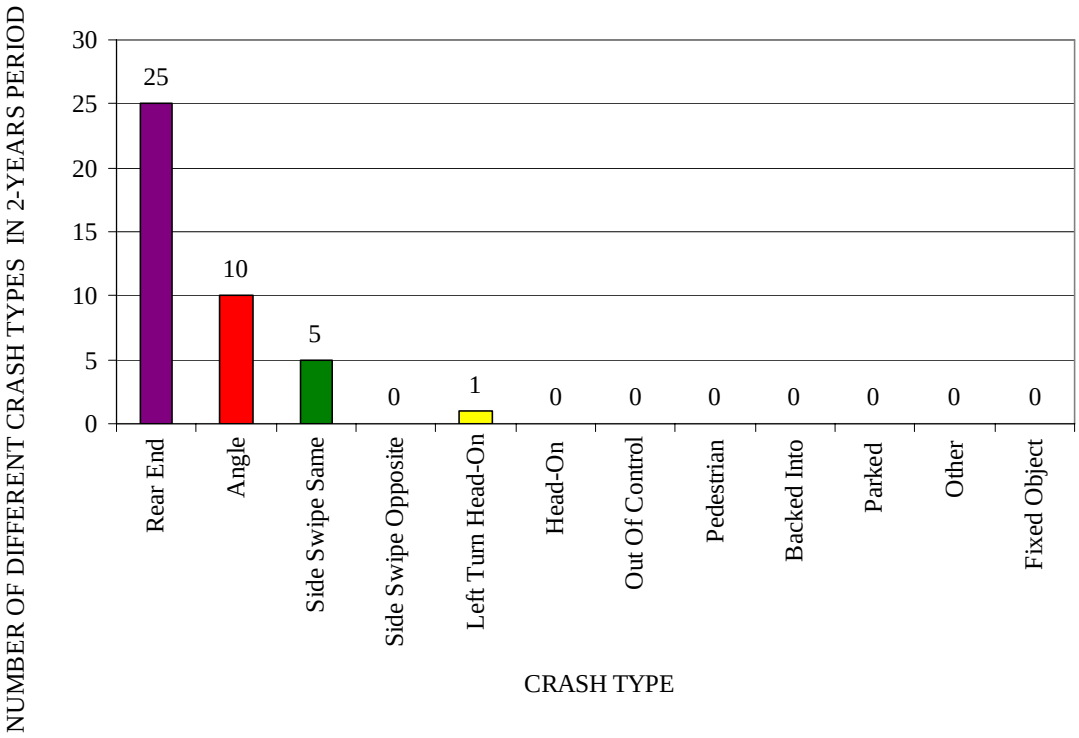


Figure 6.8.5 Two year (2003 & 2004) Crash Experience at Bay Road (M-84) and Weiss Street Intersection

Safety Deficiencies and Recommended Improvements

The majority of the crashes occurring at the intersection of Bay Road and Weiss Street are of the rear-end, angle and side swipe same types and occurred due to several possible reasons including restricted or poor visibility of traffic signals, improper lane changing and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan.

Moreover, an analysis of the UD-10 forms reveal that some of the rear end and angle crashes occurred due to improper movements regarding the driveways of the Family Video and Patisserie close to the intersection.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm supported box-span signal installation, as shown in the proposed intersection diagram in Figure 6.8.6.

The cycle length of the proposed condition has been changed to 90 seconds to maintain the progression along the Bay Road Corridor.

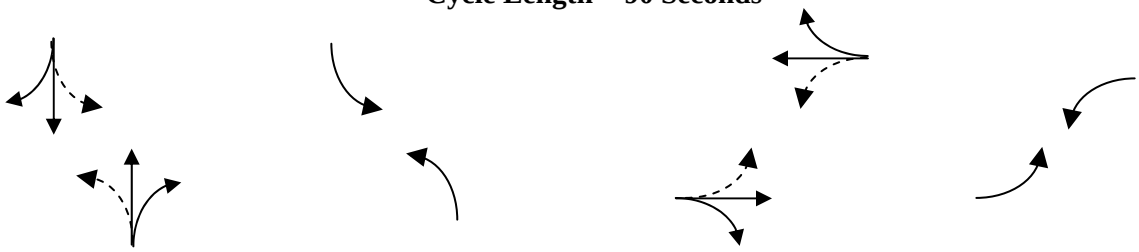
Post-mounted traffic signals with "LEFT" illuminated signs should be installed at all corners of the intersection to assist left turn vehicles. The stop bars on the northbound, southbound and eastbound approaches should be relocated. Pavement markings on all the approaches should be upgraded. The existing yellow and all-red intervals are shown in Table 6.8.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.8.1. Existing and Proposed Clearance Intervals for the Bay Road (M-84) and Weiss Street Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Left Turn Phase	4.5	0.0	3.9	1.7	Less rear end, angle and injury crashes
N-S Through Phase	4.5	1.0	3.9	1.7	
E-W Left Turn Phase	4.5	0.0	3.2	2.2	
E-W Through Phase	4.5	1.0	3.2	2.2	

The proposed PM peak hour signal timing based on a 90 second cycle length is shown below:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



N-S Bay	N-S Bay Left Turn Phase	E-W Weiss	E-W Weiss Left Turn Phase
G = 34.4 sec	G = 8.4 sec	G = 20.6 sec	G = 4.6 sec
Y = 3.9 sec	Y = 3.9 sec	Y = 3.2 sec	Y = 3.2 sec
*R = 50.0 sec	R = 76.0 sec	*R = 64.0 sec	R = 80.0 sec
AR = 1.7 sec	AR= 1.7 sec	AR = 2.2 sec	AR = 2.2 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for the Bay Road to 3.9 seconds and for the Weiss Street to 3.2 seconds.
- Increase the all-red interval for the Bay Road to 1.7 seconds and for the Weiss Street to 2.2 seconds.
- Relocate stop bar on the northbound, southbound and eastbound approaches, and crosswalk on the northbound and eastbound approaches as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for the left-turns.
- Upgrade pavement markings for the northbound, southbound and eastbound approaches with proper directional arrows.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.

- Optimize the signal timings for AM peak and off-peak hours.

The proposed intersection diagram in Figure 6.8.6 shows the proposed improvements which include the installation of a mast-arm signal, provision of post-mounted traffic signals with "LEFT" illuminated signs, upgrading the pavement markings on all approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Road and Weiss Street indicate that, for the existing conditions during the PM peak hour, the overall intersection level of service is E with a delay of 56.1 seconds per vehicle.

With the modifications to the signal timings, phasing and the noted improvements, the overall intersection level of service will be improved to C with a reduced delay of 27.1 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic along with improved progression and enhanced safety.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.8.2.

Table 6.8.2
Bay Road (M-84) and Weiss Street
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Weiss Street	36.4	D	31.7	C
WB Weiss Street	27.5	C	45.0	D
NB Bay Road	43.1	D	22.8	C
SB Bay Road	82.7	F	20.8	C
Overall Intersection	**56.1**	**E**	**27.1**	**C**

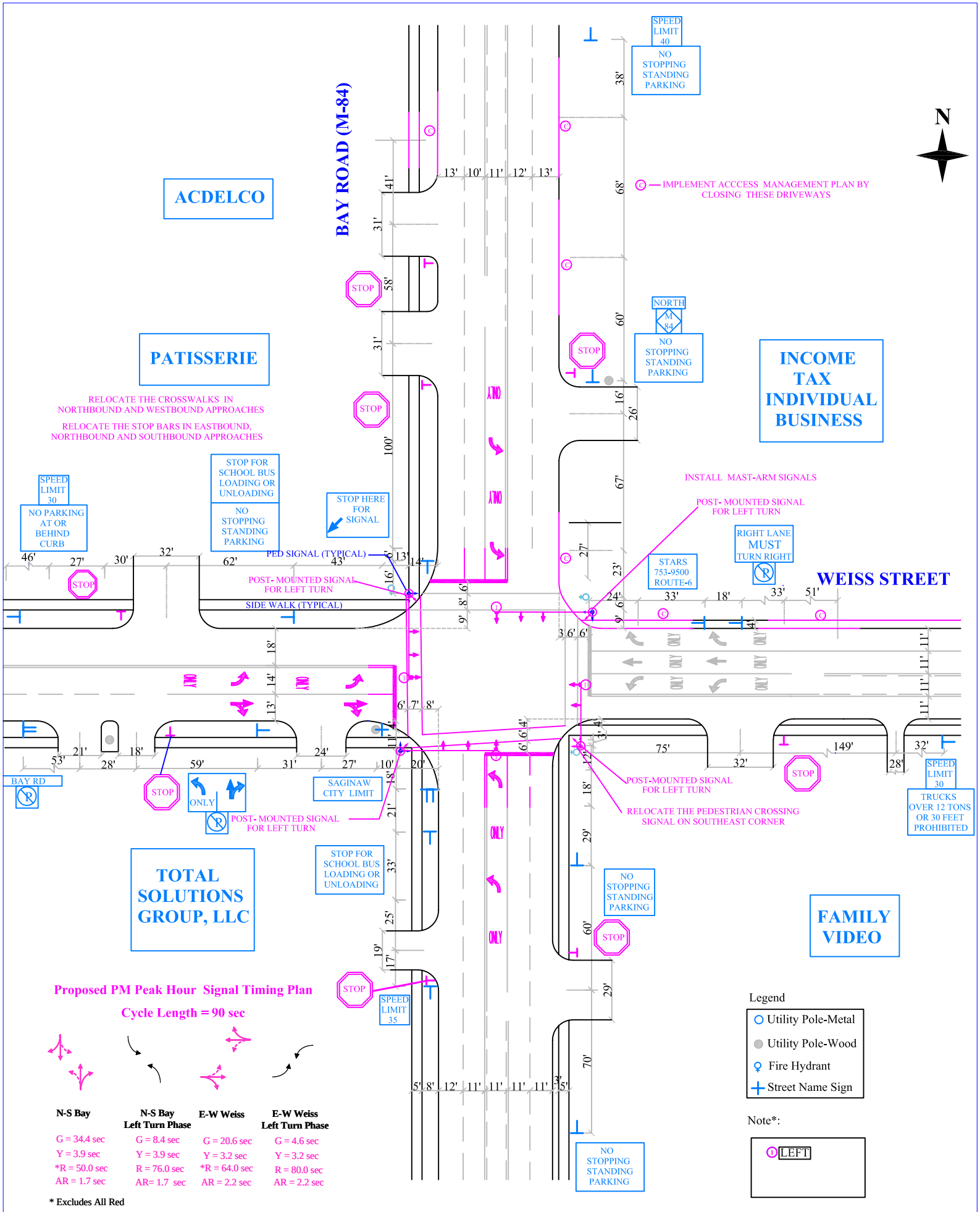


Figure 6.8.6 Proposed Diagram of Bay Road (M-84) and Weiss Street Intersection (Not to Scale)

9. Bay Road and Davenport Avenue

The intersection of Bay Street and Davenport Avenue is located in the northern part of Saginaw Township. Bay Street runs in the north-south direction and Davenport Avenue runs in the east-west direction. Both Bay Street and Davenport Avenue are state trunk lines and known as M-84 and M-58 respectively.

Bay Street, near the intersection with Davenport Avenue, has an average daily traffic volume of approximately 23,100 vpd as per Michigan Department of Transportation Traffic Count Maps (2004 Data). Davenport Avenue, at the intersection with Bay Street, has an average daily traffic volume of approximately 12,400 vpd as per Michigan Department of Transportation Traffic Count Maps (2004 Data).

Bay Street is 62 feet wide on the north of the intersection and 56 feet wide on the south of the intersection. It has one dedicated left-turn lane, one through lane, and one through and right-shared lane for the northbound approach. At the southbound approach, it has one dedicated right-turn lane and two through lanes.

Davenport Avenue is 46 feet wide on the east leg of the intersection with one exclusive right turn lane, two through lanes and one through and left shared lane. West of the intersection, Davenport Avenue is 38 feet wide with three receiving lanes. The speed limit for Bay Street is 35 miles per hour.

Land Use

There is a Realty office and Property Management in the northeast corner and an 'After Hour Tuxedo World' shop in the northwest corner of the intersection. Admiral is located in the southeast corner of the intersection. An 'Advance Auto Part' shop is in the southwest corner of the intersection. There are multiple driveways close to the intersection to serve these establishments. The driveways to Admiral are located very close to the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.9.1. Details of the intersection approaches are shown in photographs 9a through 9c. The photographs show the existing span wire signal installation at the intersection along with the existing pavement markings

Photograph 9a. NB Bay Street at Davenport Avenue

Photograph 9b. SB Bay Street at Davenport Avenue

Photograph 9c. WB Davenport Avenue at Bay Street

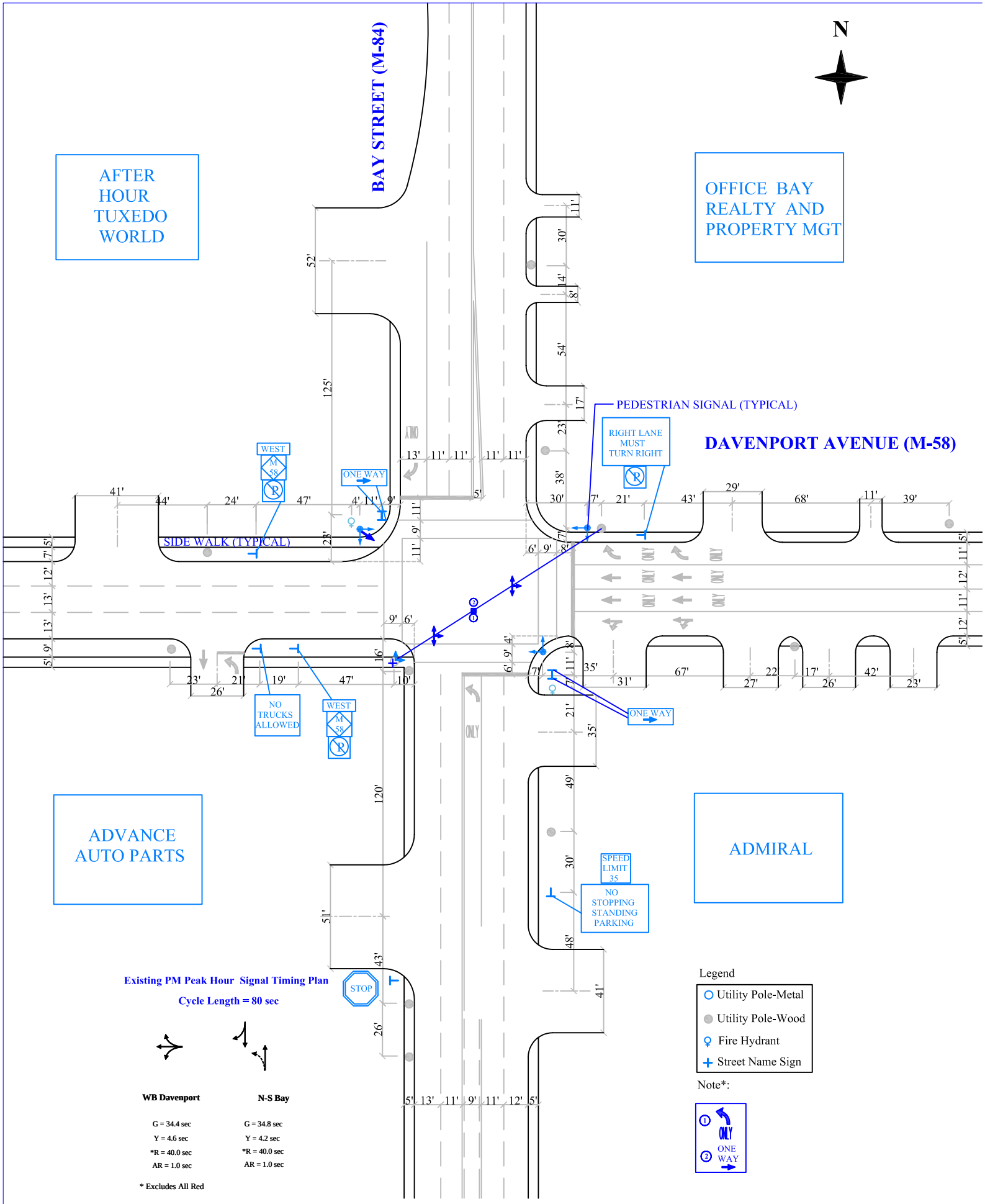


Figure 6.9.1 Existing Condition Diagram of Bay Street (M-84) and Davenport Avenue (M-58) Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were collected manually on a clear day on January 31, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:30 to 5:30 PM and is presented in Figure 6.9.2, which indicates the total intersection volume is considerably high with 3,440 vehicles per hour.

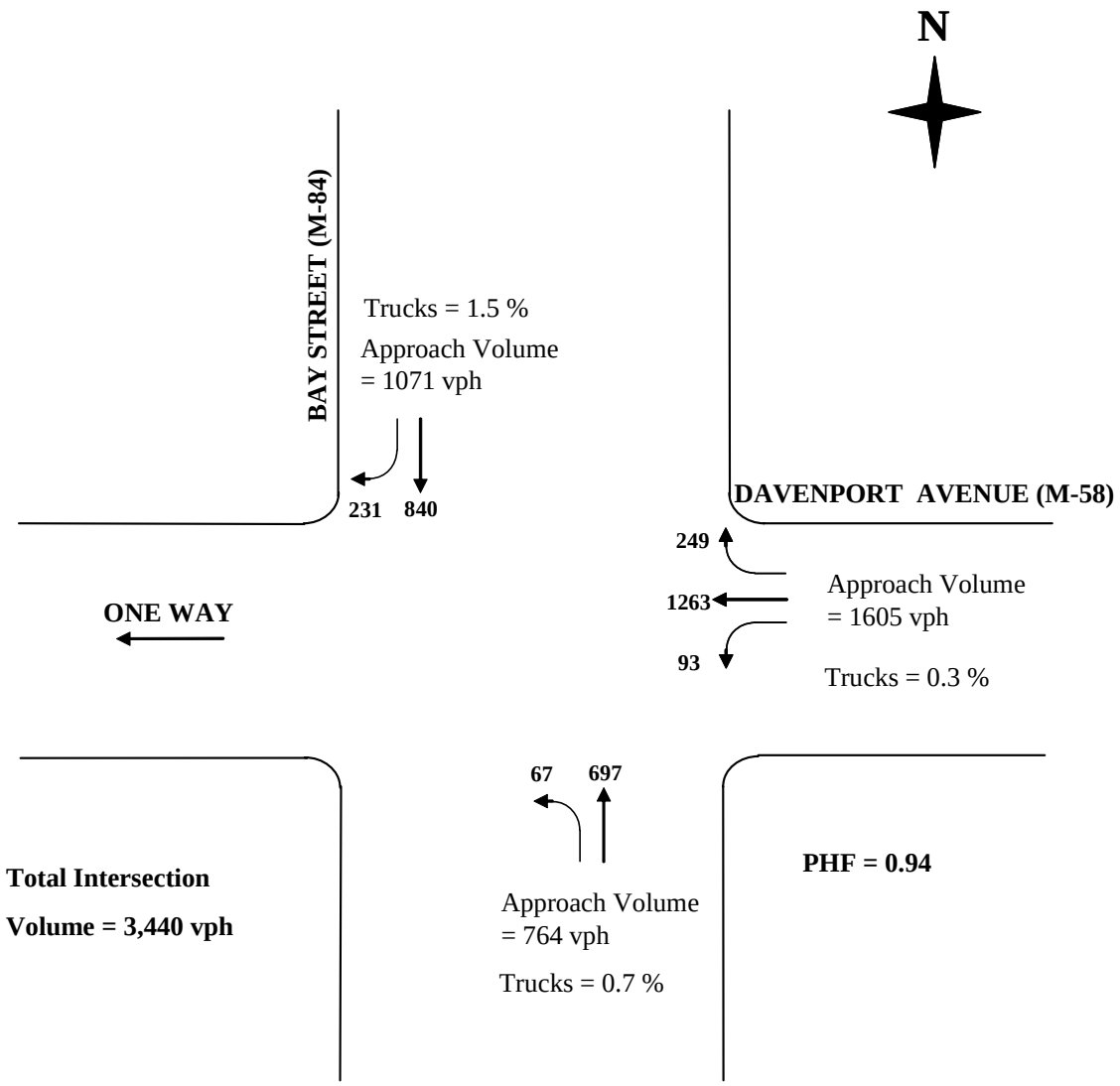


Figure 6.9.2. Existing PM Peak Hour (4:30–5:30 PM) Traffic Volumes for the Intersection of Bay Street (M-84) and Davenport Avenue (M-58)

Traffic Control

During peak hours, the intersection is controlled by a two-phase span wire signal installation with an 80-second cycle length. The existing clearance interval includes a yellow interval of 4.6 seconds and an all-red interval of 1.0 second for the westbound phase. The north-south phase has a yellow interval of 4.2 seconds and an all-red interval of 1.0 second. The existing signal-timing plan for this intersection is shown in the existing condition diagram in figure 6.9.1.

Crash Analysis

A total of 45 crashes were recorded for the years 2003 and 2004 with an average annual crash frequency of 23 crashes. Collision diagrams were prepared based on the UD-10 report's information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Bay Street and Davenport Avenue are shown in Figures 6.9.3 and 6.9.4 respectively.

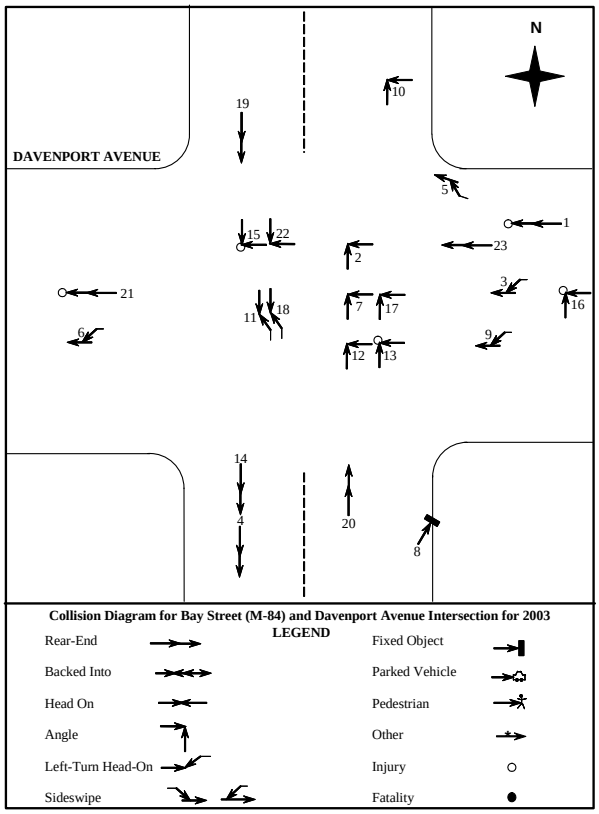


Figure 6.9.3. Collision Diagram for Bay Street and Davenport Avenue for 2003

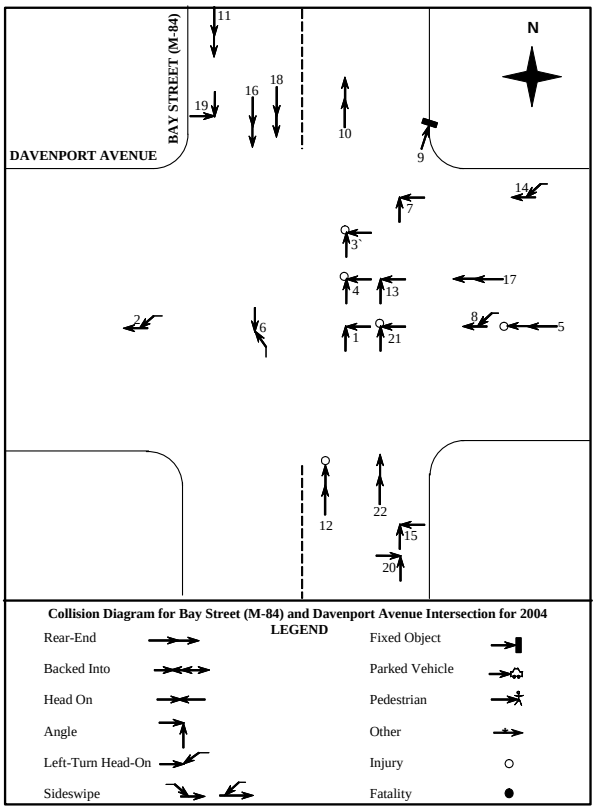


Figure 6.9.4. Collision Diagram for Bay Street and Davenport Avenue for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.47 per million entering vehicles which is high, considering the approach volume at the intersection. Angle crashes are of specific significance, which may have occurred due to insufficient clearance interval and improper driveway movements. There are three left-turn head-on crashes at the intersection, which may have occurred due to an interference of northbound left turns with the southbound through traffic during the permissive phase.

The crash distribution, based upon types of crashes, is shown in Figure 6.9.5. Angle and Rear End crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes includes 18 angle crashes, and 15 rear end crashes.

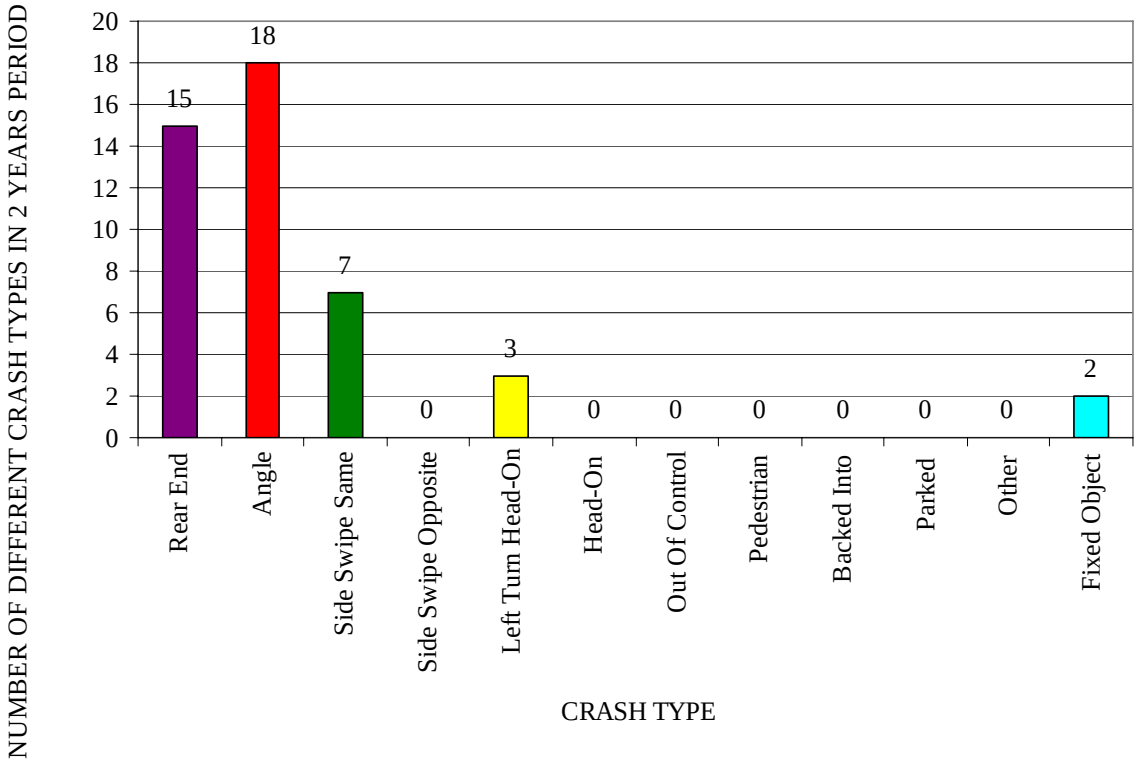


Figure 6.9.5 Two year (2003 & 2004) Crash Experience at Bay Street (M-84) and Davenport Avenue Intersection

Safety Deficiencies and Recommended Improvements

The majority of the crashes occurring at this intersection of Bay Street and Davenport Avenue are of the angle, rear-end, side swipe same and left-turn head-on type, happening due to several possible reasons including restricted or poor visibility of traffic signals, improper lane changing and inadequate clearance interval. The required clearance interval is computed and provided in the proposed signal-timing plan.

Due to the excessive distance between the crosswalks and stop bars, some drivers may find themselves in the 'dilemma zone' in the intersection for an extended period of time. The Michigan Manual of Uniform Traffic Control Devices recommends that stop bars at intersections be placed four feet in advance of the nearest crosswalk line. On the southbound approach of Bay Street, this distance is 11 feet. It appears that the stop bar for this approach could be moved closer to the crosswalk. Pavement markings on the northbound and southbound approaches should also be upgraded.

The cycle length of the proposed signal timing has been changed to 90 seconds to be compatible with the progression along the Bay Road Corridor.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The frequently occurring angle and rear-end crashes will be prevented by the provision of adequate clearance intervals. To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.9.6.

The existing yellow and all-red intervals are shown in Table 6.9.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.9.1. Existing and Proposed Clearance Intervals for the Bay Street (M-84) and Davenport Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
WB Split Phase	4.6	1.0	3.6	2.4	Less rear end, angle, left-turn head-on and injury crashes
N-S Through Phase	4.2	1.0	3.6	1.7	

The proposed PM peak period signal timing plan based on a 90 second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds

WB Davenport	**N-S Bay**
G = 37.0	G = 41.7
Y = 3.6	Y = 3.6
*R = 47.0	*R = 43.0
AR = 2.4	AR = 1.7

* Excludes All

A list of the recommended improvements is summarized below:

- Decrease the yellow interval for Bay Street and Davenport Avenue to 3.6 seconds.
- Increase the all-red interval for Bay Street to 1.7 seconds and for the Davenport Avenue to 2.4 seconds.
- Relocate the stop bar on the southbound approach, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade pavement markings for the northbound and southbound approaches with proper directional arrows.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours.

The proposed diagram in Figure 6.9.6 shows the proposed improvements which include the new installation of a mast-arm support for the signals, upgrading the pavement markings on all approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Street and Davenport Avenue indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 16.0 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will remain at B with an increased delay of 16.6 seconds per vehicle. The modified signal timing and changed cycle length along with the adjusted clearance intervals will ensure enhanced safety.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.9.2.

Table 6.9.2.

Bay Street (M-84) and Davenport Avenue
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
WB Davenport Avenue	17.7	B	21.2	C
NB Bay Street	14.0	B	7.2	A
SB Bay Street	14.8	B	16.5	B
Overall Intersection	**16.0**	**B**	**16.6**	**B**

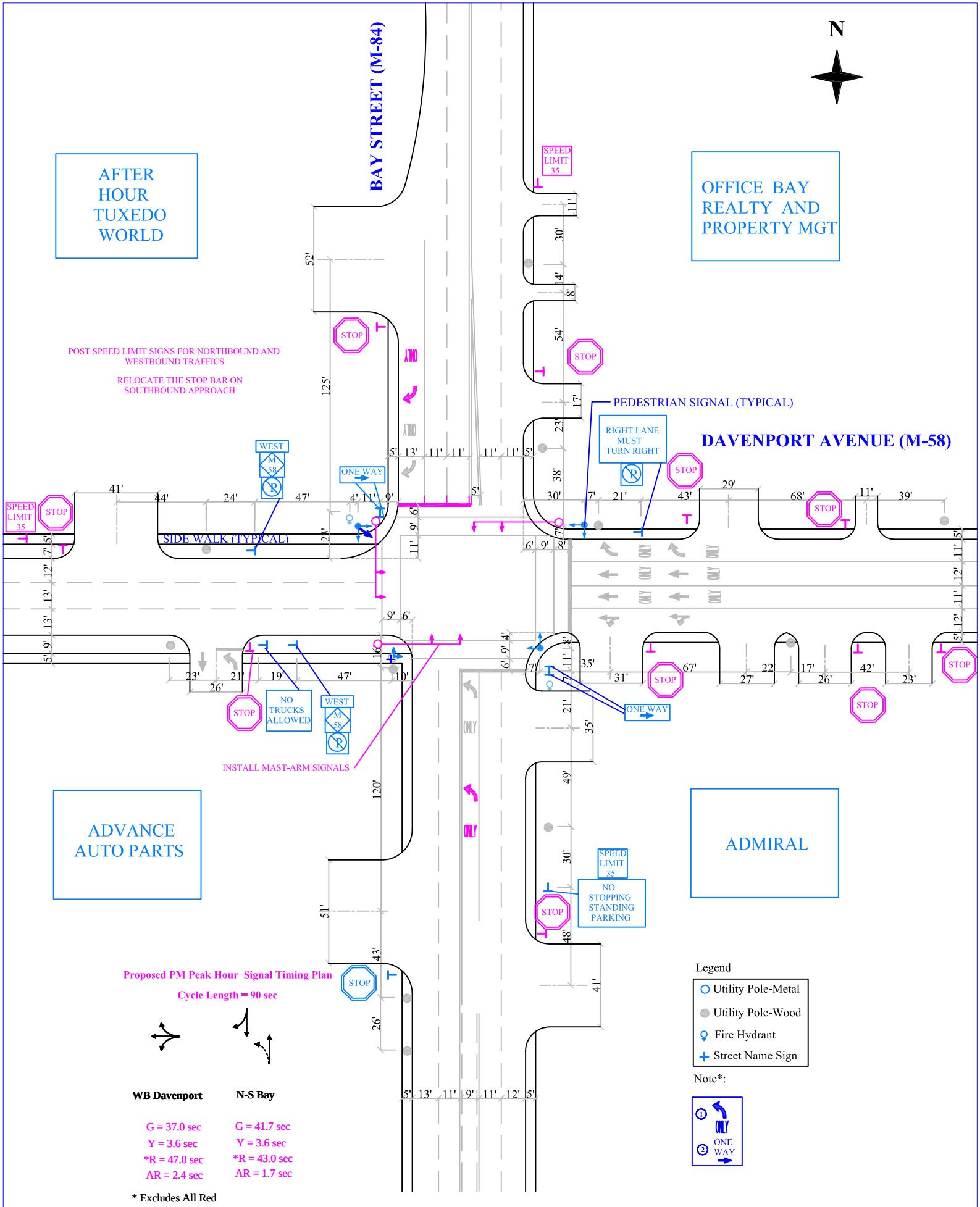


Figure 6.9.6 Proposed Diagram of Bay Street (M-84) and Davenport Avenue (M-58) Intersection (Not to Scale)

10. Bay Street (M-84) and State Street (M-58)

The intersection of Bay Street (M-84) and State Street (M-58) is located within the northwestern part of the City of Saginaw. Bay Street is a major state trunk line. On the other hand, State Street is an eastbound one-way street, which is one of the bifurcated parts of the major state trunk line State Street (M-58). State Street (M-58) is divided into two one-way roads between its intersection with Coolidge Avenue near the western boundary of the City of Saginaw and Davenport Avenue near the I-675 interchange.

Bay Street on north of the intersection has an average daily traffic volume of approximately 23,100 vpd as per MDOT 2004 data. East of the intersection State Street has an average daily traffic volume of approximately 10,700 vpd, whereas on the western side, the average daily traffic volume is 11,000 vpd as per MDOT 2004 data.

Bay Street (M-84) south of the intersection is 59 feet wide with one through lane and one shared through and right-turn lane. Bay Street (M-84) north of the intersection, is 57 feet wide with one exclusive left turn lane and two through lanes. On the east leg of the intersection, State Street is 31 feet wide with two receiving lanes. On the west leg of the intersection, it is 37 feet wide with one exclusive left turn lane and two through lanes.

The speed limit for Bay Street (M-84) is 35 miles per hour, while for State Street; it is 30 miles per hour.

Land Use

There is a Pizza shop in the northwest corner of the intersection. There are some commercial developments in the northeast corner of the intersection. 7-Eleven is located in the southeast corner of the intersection. A law office and a state farm insurance office are located in the southwest corner of the intersection. There are multiple driveways very close to the intersection, which serve these establishments. The driveways for the commercial developments are located very close to the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the Bay Street and State Street intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.10.1. Details of the intersection approaches are shown in photographs 10a through 10c. The photographs show the existing span wire signal installation along with the pavement markings.

Photograph 10a. NB Bay Street at State Street

Photograph 10b. SB Bay Street at State Street

Photograph 10c. EB State Street at Bay Street

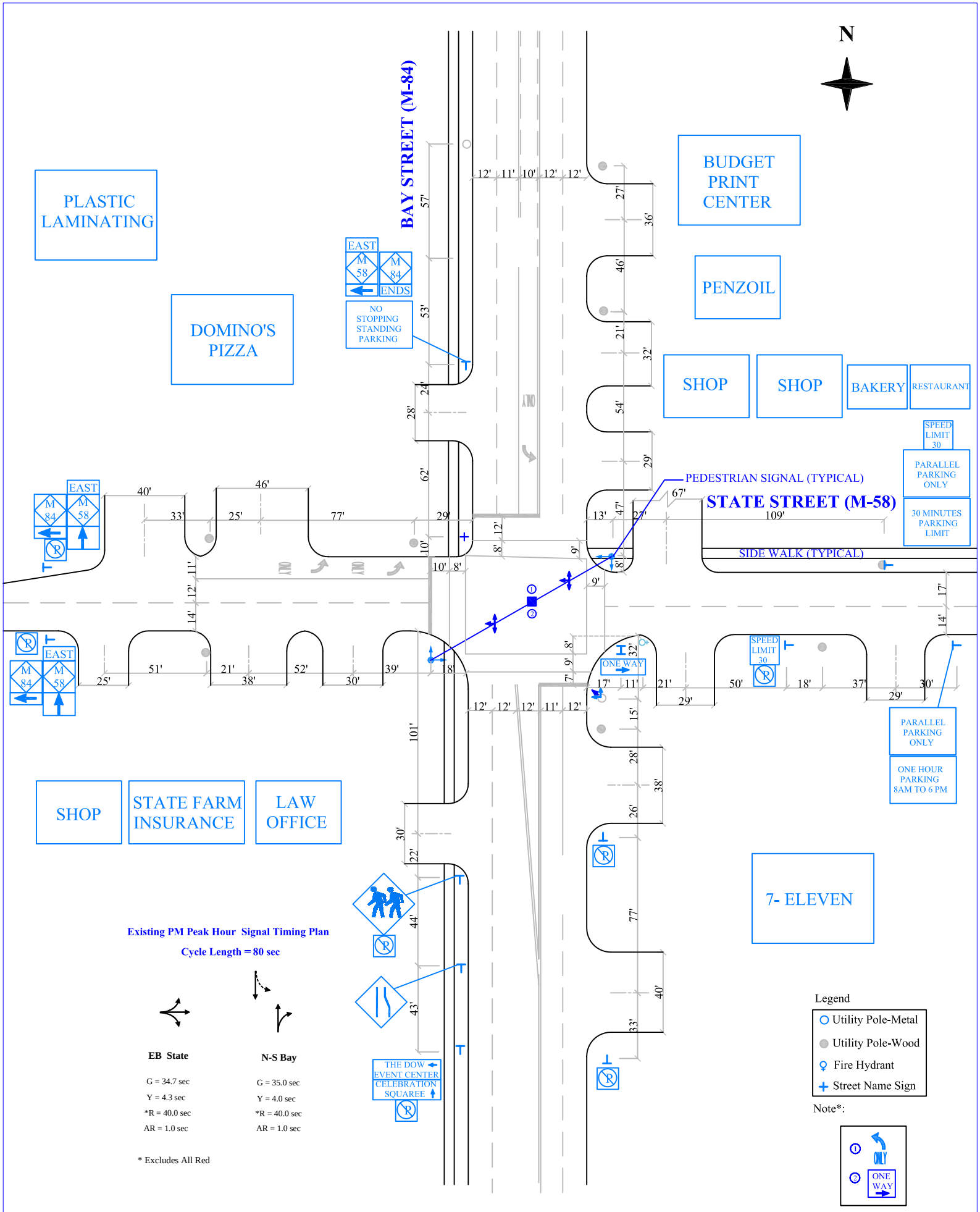


Figure 6.10.1 Existing Condition Diagram of Bay Street (M-84) and State Street (M-58) Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on a clear day on January 31, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:30 to 5:30 PM and is presented in Figure 6.10.2.

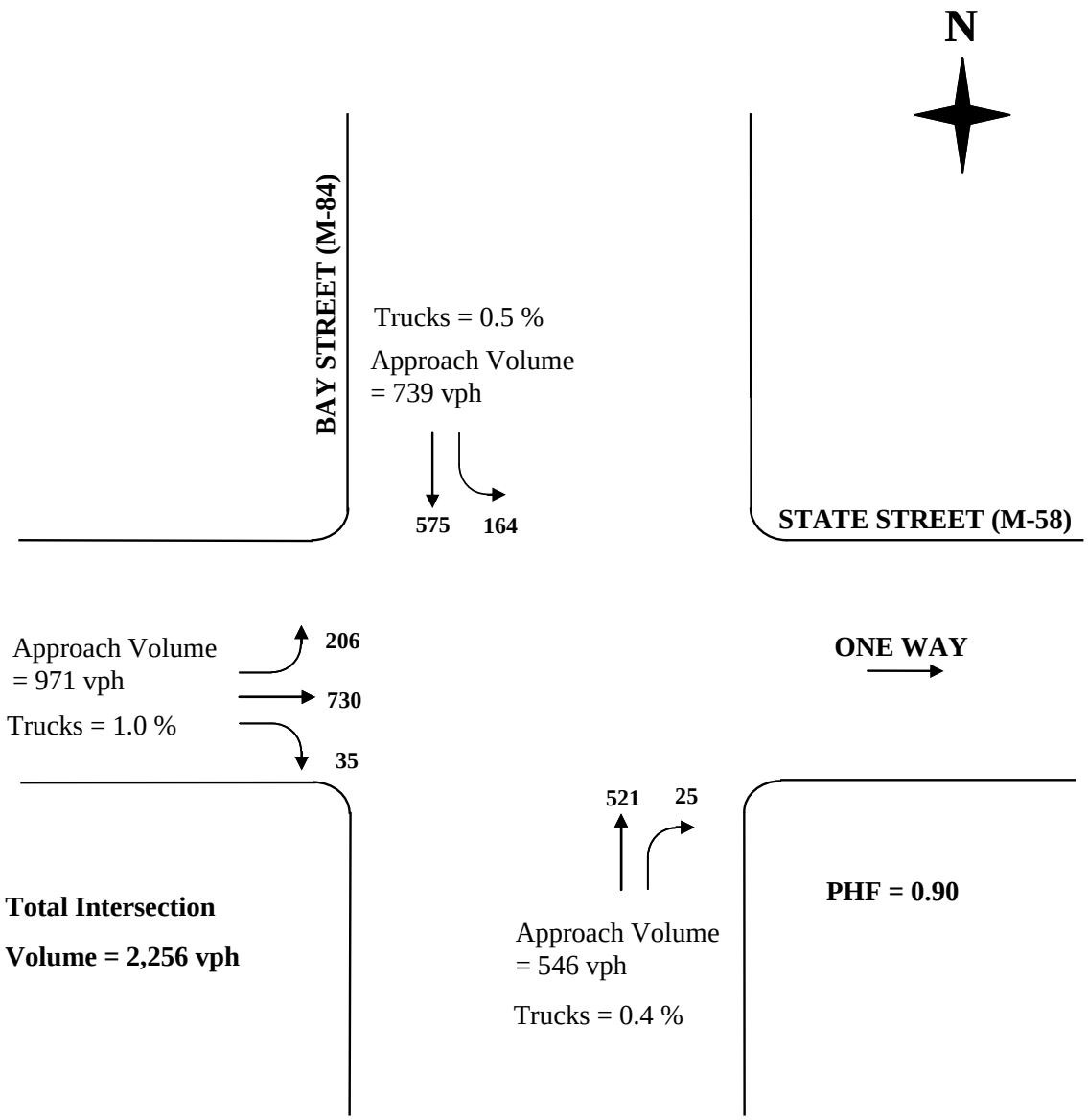


Figure 6.10.2 PM Peak Hour (4:45–5:45 PM) Traffic Volumes for the Intersection of Bay Street (M-84) and State Street (M-58)

Traffic Control

This intersection is controlled by a two-phase span-wire traffic signal with an 80-second cycle length. For both southbound and northbound M-84, the existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second. For eastbound State Street, the yellow interval is 4.3 seconds and all-red interval is 1.0 second. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.10.1.

Crash Analysis

For the intersection of Bay Street and State Street, a total of 36 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of eighteen crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Bay Street and State Street are shown in Figure 6.10.3 and Figure 6.10.4.

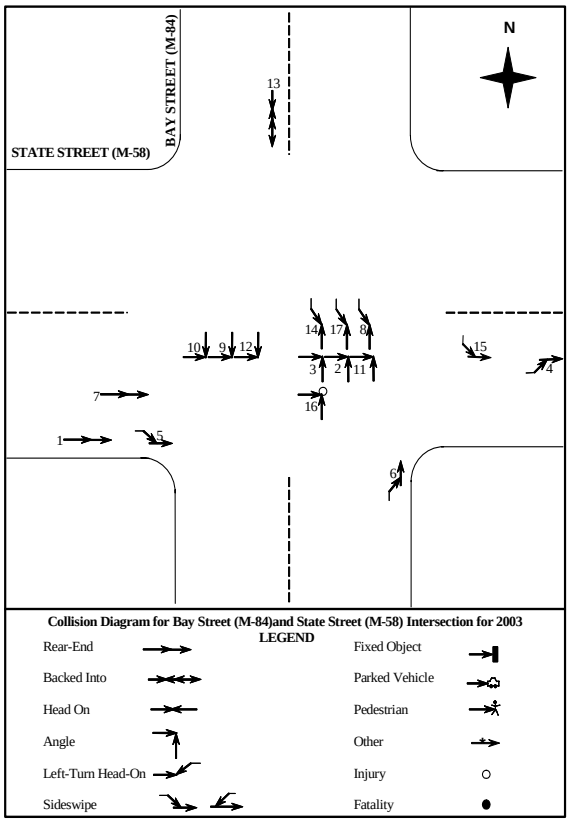


Figure 6.10.3 Collision Diagram for Bay Street and State Street for 2003

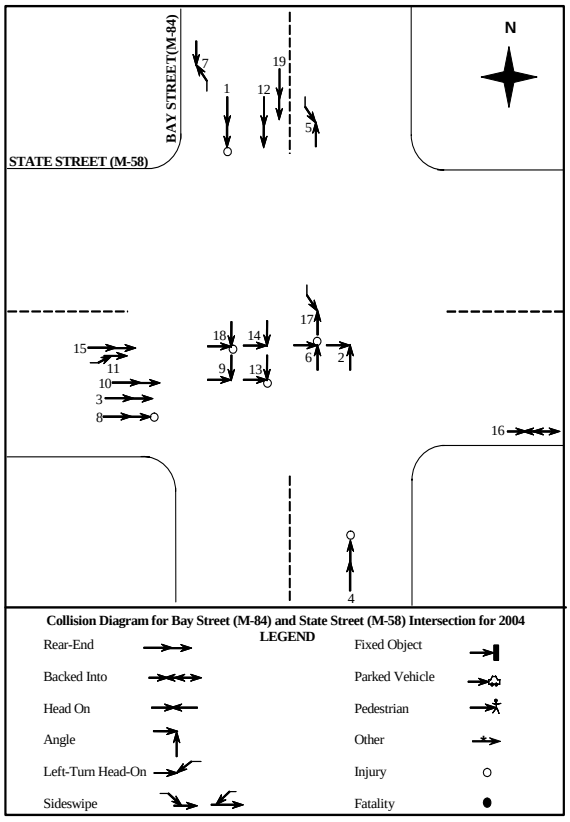


Figure 6.10.4 Collision Diagram for Bay Street and State Street for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The crash rate is 1.75 per million entering vehicles, which is nominal, considering the approach volume at the intersection.

The crash distribution, based upon types of crashes, is shown in Figure 6.10.5. Angle and rear end crashes are the predominant crashes reported at this intersection. The frequencies of left-turn head-on crashes and sideswipe same crashes are also very high. The corresponding two-year crash frequencies for these crashes include 13 angle crashes, ten rear-end crashes, six left turn head on crashes and five sideswipe same crashes.

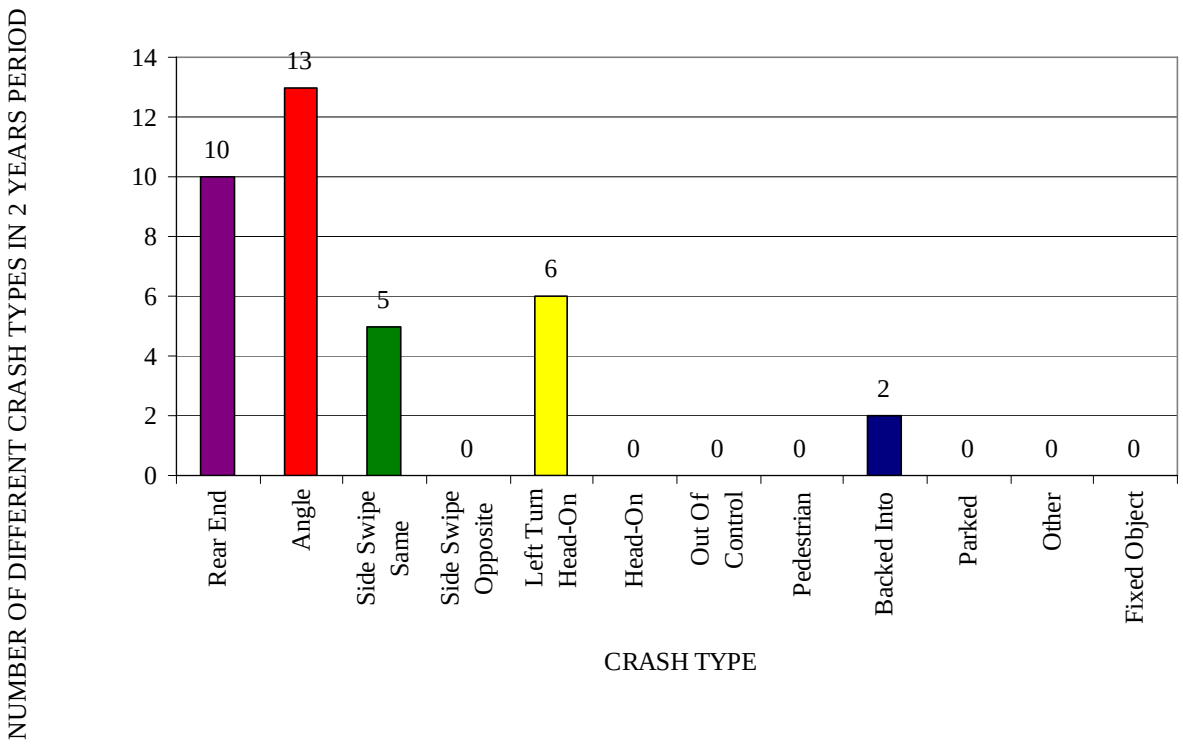


Figure 6.10.5 Two year (2003 & 2004) Crash Experience at Bay Street (M-84) and State Street (M-58) Intersection

Safety Deficiencies and Recommended Improvements

The majority of the crashes occurring at this intersection of Bay Street and State Street are of the left-turn head-on, angle and rear-end types and occur due to several possible reasons including restricted or poor visibility of traffic signals, a left turn permissive phase for Bay Street and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan.

Due to the excessive distance between the crosswalks and stop bars, some drivers may find themselves in the 'dilemma zone' in the intersection for an extended period of time. The Michigan Manual of Uniform Traffic Control Devices recommends that stop bars at intersections be placed four feet in advance of the nearest crosswalk line. On the eastbound approach of State Street, this distance is 10 feet and 12 feet on the southbound approach. It appears that the stop bar for both approaches could be moved closer to the crosswalk. Pavement markings on the northbound, southbound and westbound approaches should be upgraded.

The cycle length of the proposed signal plan has been changed to 90 seconds to be compatible with the progression along the Bay Road Corridor.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The frequently occurring angle and rear-end crashes will be prevented by the provision of adequate clearance intervals. To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation.

The existing yellow and all-red intervals are shown in Table 6.10.1, along with the proposed lengths of these intervals and their potential benefits.

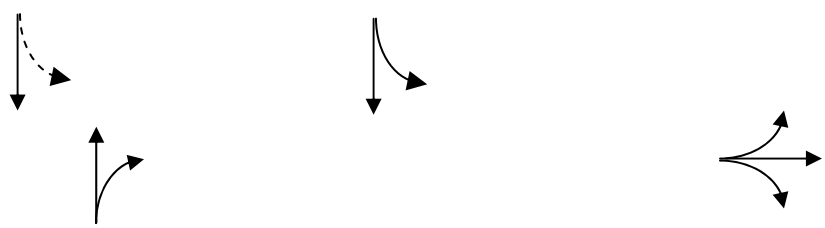
Table 6.10.1 Existing and Proposed Clearance Intervals for the Bay Street (M-84) and State Street (M-58) Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
EB Split Phase	4.3	1.0	3.5	2.1	Less rear end, left-turn head-on, angle and injury crashes
N-S Through Phase	4.0	1.0	3.6	1.4	
SB Split Phase	-	-	3.6	1.4	

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



N-S Bay	N-S Bay	EB State
G = 33.0 sec	G = 6.0 sec	G = 35.4 sec
Y = 3.6 sec	Y = 3.6 sec	Y = 3.5 sec
*R = 52.0 sec	*R = 79.0 sec	*R = 40.0 sec
AR = 1.4 sec	AR = 1.4 sec	AR = 2.1 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Bay Street to 3.6 seconds and for State Street to 3.5 seconds.
- Increase the all-red interval for Bay Street to 1.4 seconds and for State Street to 2.1 seconds.

- Relocate stop bar on the southbound and eastbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade pavement markings for the northbound, southbound and eastbound approaches with proper directional arrows.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours.

The proposed diagram in Figure 6.10.6 shows the proposed improvements which include the installation of mast-arm signals, upgrading the pavement markings on the all approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Street (M-84) and State Street (M-58) indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 14.9 seconds per vehicle.

With the modifications to the signal timings, phasing and the noted improvements, the overall intersection level of service will remain at B with a delay of 13.5 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic along with improved progression

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.10.2 (page 188).

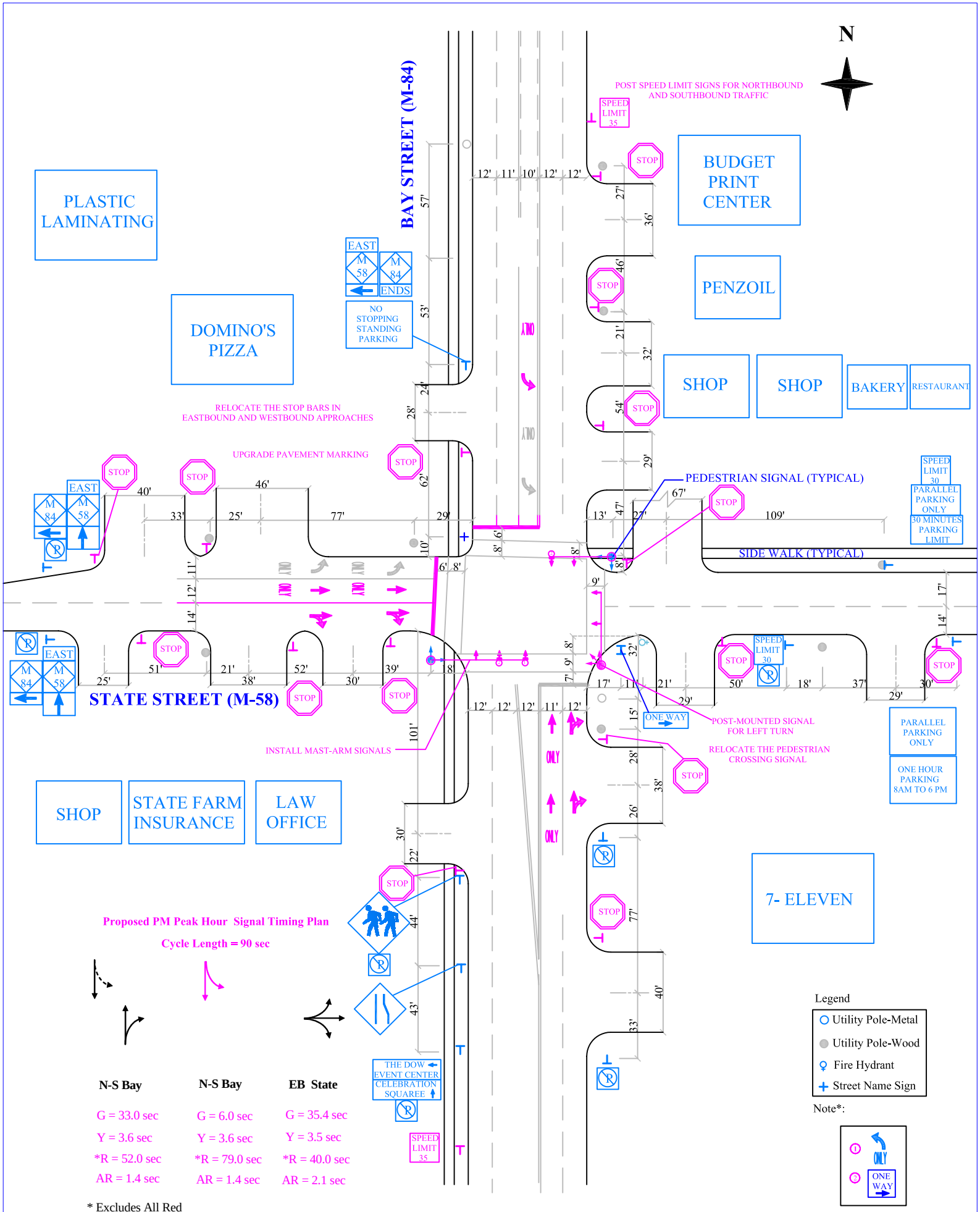


Figure 6.10.6 Proposed Diagram of Bay Street (M-84) and State Street (M-58) Intersection (Not to Scale)

Table 6.10.2
Bay Street (M-84) and State Street (M-58)
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB State Street	16.4	B	21.3	C
NB Bay Street	12.8	B	13.5	B
SB Bay Street	14.6	B	3.3	A
Overall Intersection	**14.9**	**B**	**13.5**	**B**

11. Bay Street (M-84) and West Genesee Street

The intersection of Bay Street (M-84) and West Genesee Street is located in the northwest part of the City of Saginaw. Bay Street, runs in the north-south direction and intersects West Genesee Road, which runs in the east-west direction. Bay Street serves as a state trunk line (M-84), whereas West Genesee Street can be classified as a collector street.

Bay Street, near the intersection of West Genesee Street, has an average daily traffic volume of approximately 23,000 vpd as per MDOT Data (2004).

Bay Street is about 42 feet wide on the either side of the intersection with one shared through and left turn lane, and one shared through and right turn lane at both approaches.

West Genesee Street is 36 feet wide on the either side of the intersection with one exclusive left-turn lane and one shared through and right-turn lane.

The speed limit for Bay Street is 35 miles per hour. West Genesee Street has a speed limit of 30 miles per hour of Bay Street and 25 miles per hour for west.

Land Use

West Genesee Street is a collector road, which runs primarily through a commercial area. A Burtrum Furs and a dentist office occupy the northeast and southeast corners of the intersection respectively. There is a Bay Connection in the northwest corner of the intersection. There are multiple driveways close to the intersection, which serve these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses are schematically shown in Figure 6.11.1. Details of the intersection approaches are shown in photographs 11a through 11d. Photographs 11a and 11b indicate the east and westbound approaches of Genesee Road with marked pavement cracks. The photographs show the existing span wire signal installation along with the pavement markings.

Photograph 11a. EB Genesee Street at Bay Street

Photograph 11b. WB Genesee Street at Bay Street

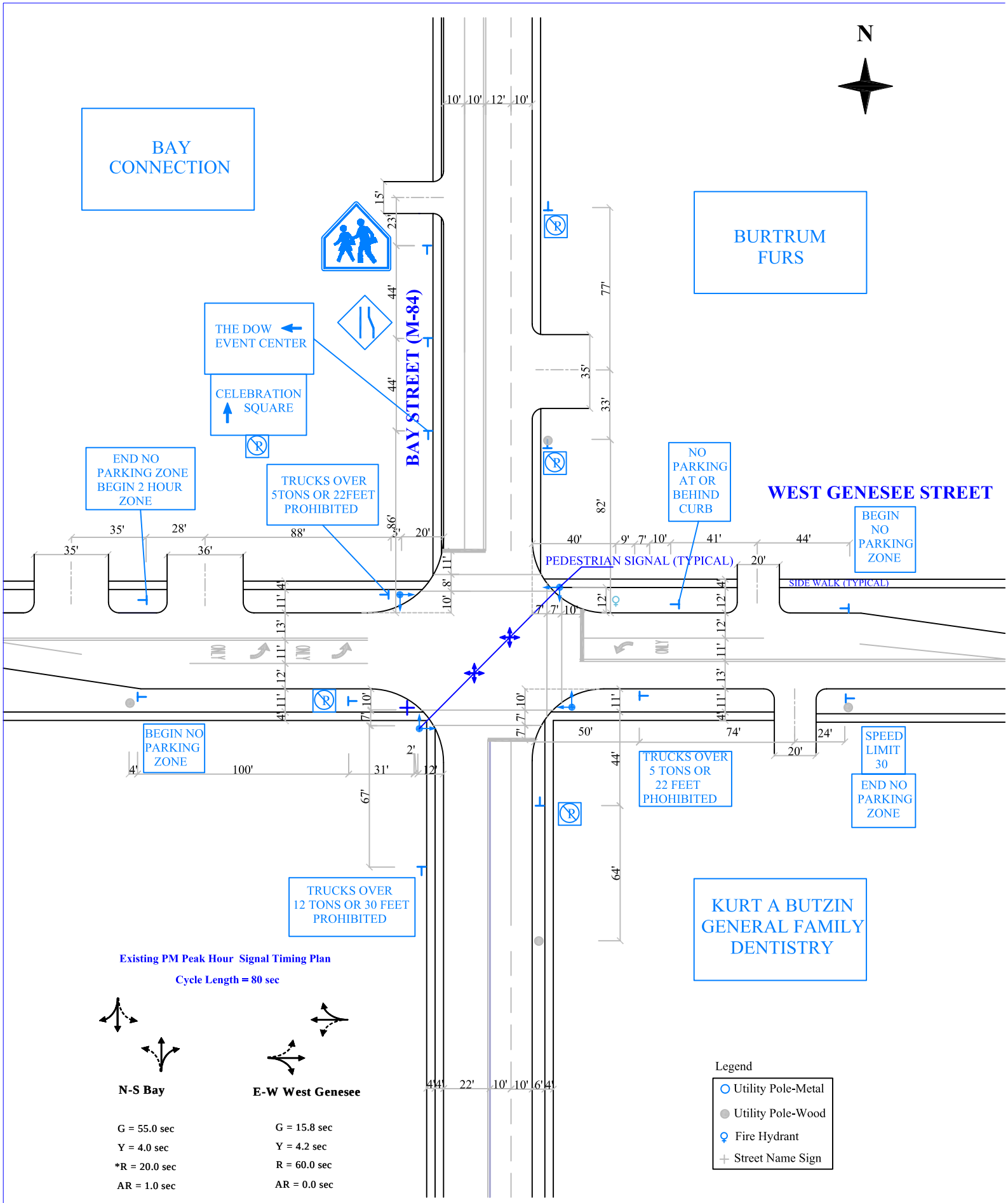


Figure 6.11.1 Existing Condition Diagram of Bay Street (M-84) and West Genesee Street Intersection (Not to Scale)

Photograph 11c. NB Bay Street at W Genesee Street

Photograph 11d. SB Bay Street at W Genesee Street

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on a clear day on January 24, 2006 from 4 PM to 6 PM by two people standing at a suitable corner of the intersection and covering all the approaches together. The results of the data are presented in Figure 6.11.2. The volume along north-south Bay Street is relatively higher than that of West Genesee Street.

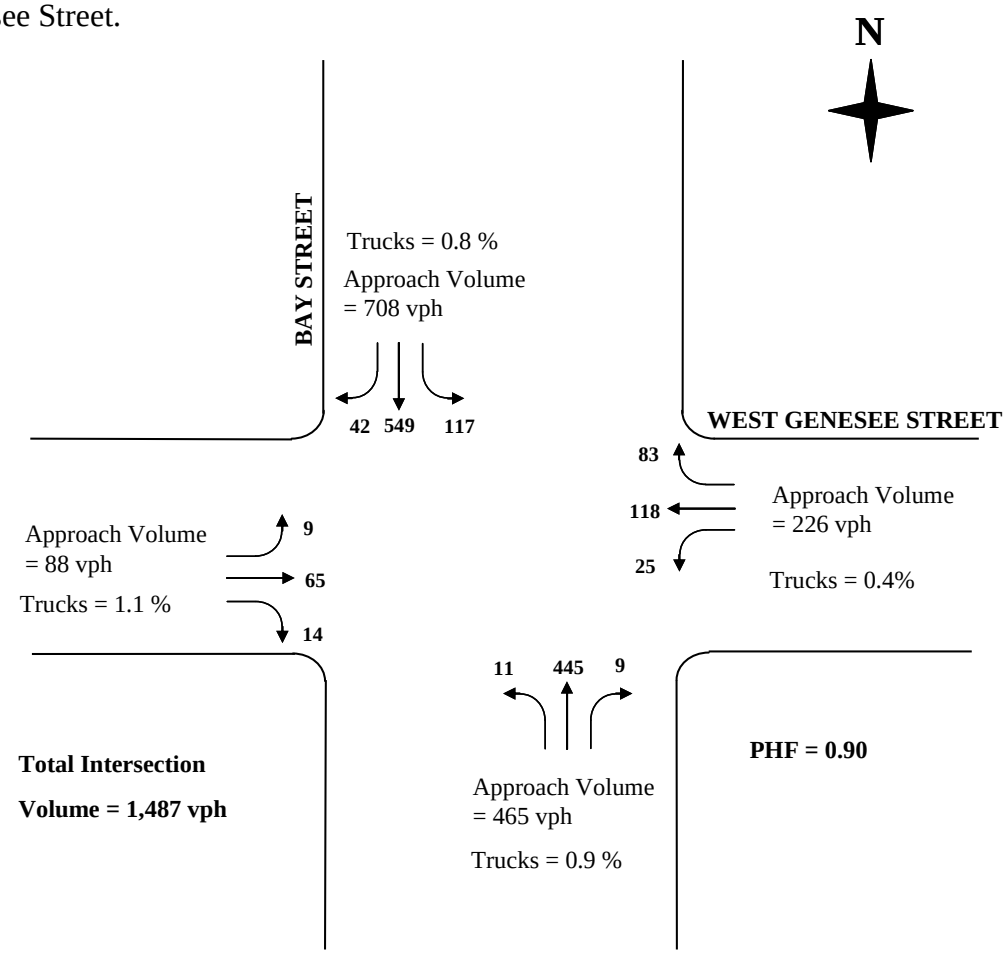


Figure 6.11.2. Existing PM Peak Hour (4:15–5:15 PM) Traffic Volumes for the Intersection of Bay Street (M-84) and West Genesee Street

Traffic Control

The intersection is controlled by a two-phase traffic signal with an 80-second cycle length during the PM peak hour. The existing clearance interval includes a yellow interval of 4.0 seconds in the north-south direction, 4.2 second in the east-west direction and an all-red interval of 1.0 second in the north-south direction. The signal timing for this intersection is shown in the existing condition diagram in Figure 6.11.1.

Crash Analysis

A total of fourteen crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of seven crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. The collision diagrams for 2003 and 2004 can be found in Figures 6.11.3 and Figure 6.11.4 respectively.

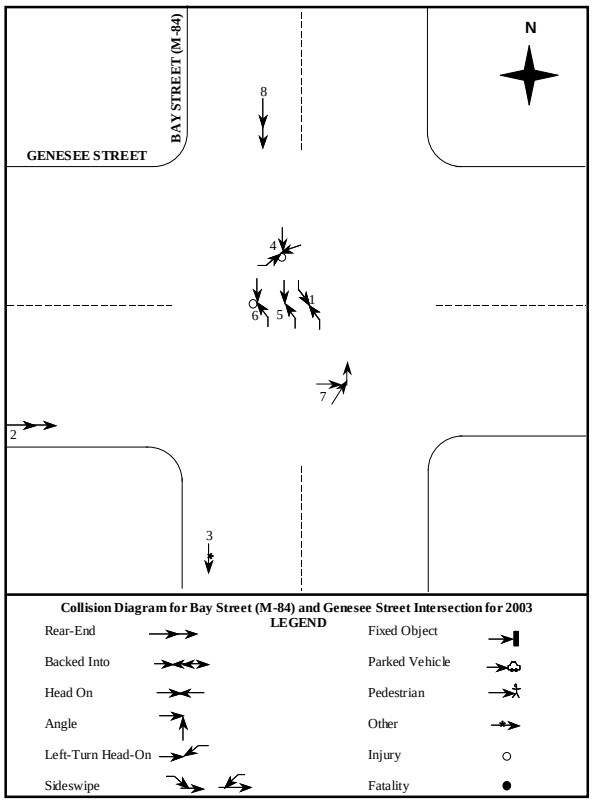


Figure 6.11.3 Collision Diagram for Bay Street and West Genesee Street for 2003

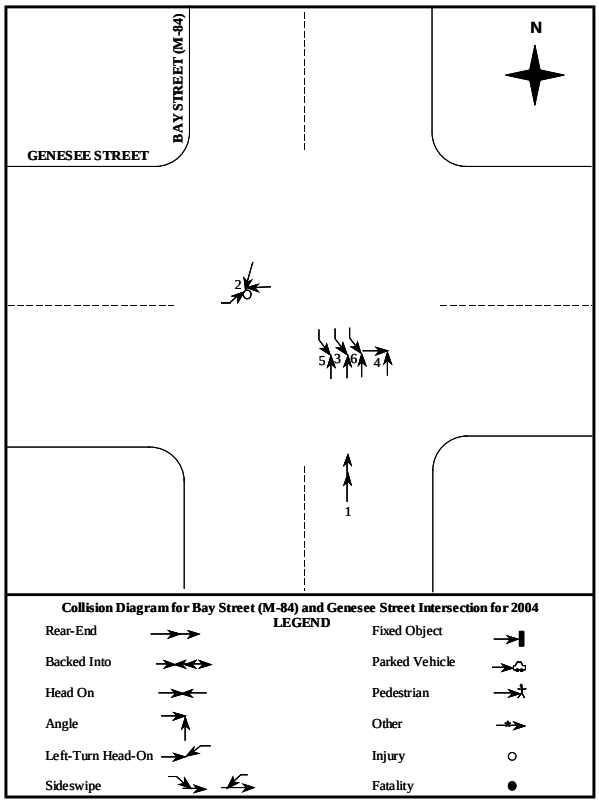


Figure 6.11.4 Collision Diagram for Bay Street and West Genesee Street for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.03 per million entering

vehicles, which is nominal, considering the approach volume at the intersection. There are some left-turn head-on crashes reported at the intersection which might have been caused by interference of oncoming through traffic with the northbound and south bound left-turn during the permissive phase.

The crash distribution, based upon type of crashes, is shown in Figure 6.11.5. Left-turn head-on, rear-end, angle and other crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequencies for these crashes include three rear-end crashes, six left-turn head-on crashes, four angle crashes and one other form of crash.

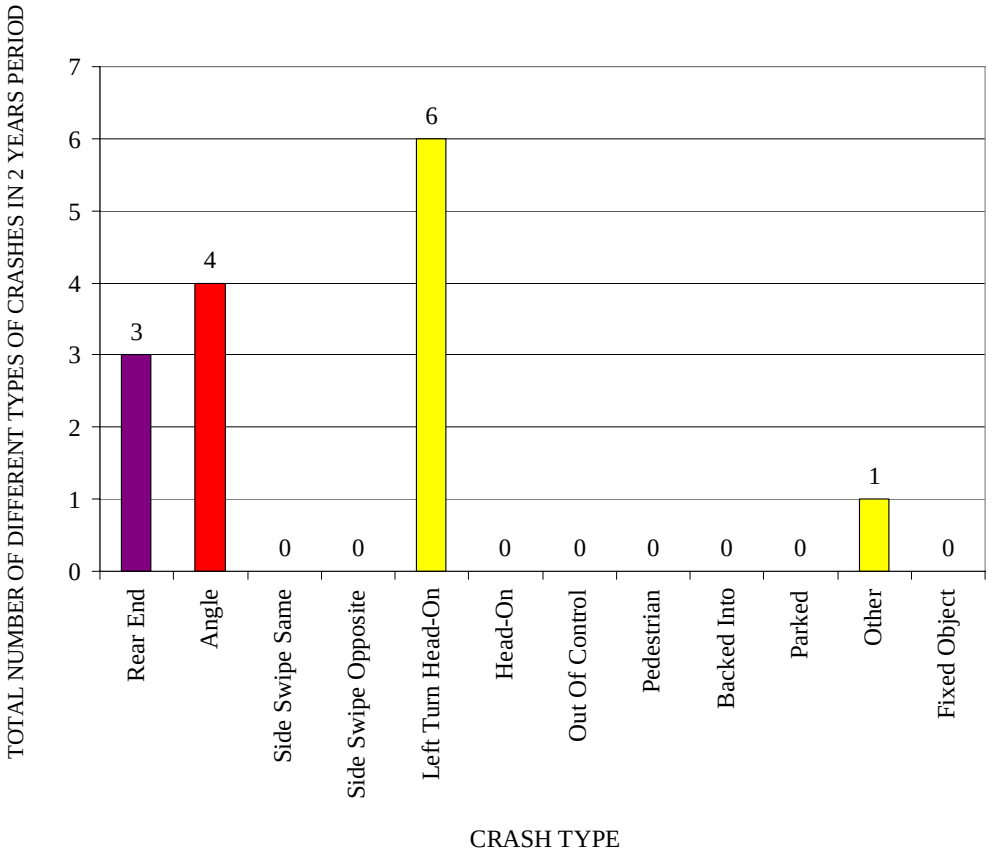


Figure 6.11.5 Two Year (2003 & 2004) Crash Experience at Bay Street (M-84) and West Genesee Street Intersection

Safety Deficiencies and Recommended Improvements

The majority of the crashes occurring at the intersection of Bay Street and West Genesee Street are of the left-turn head- on, angle and rear end types, which occur due to several

possible reasons including permissive left turn movement, poor visibility of traffic signals and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan. The left turn volume is high on the southbound approach compared to the northbound approach. The existing signal-timing plan is modified with a split movement for southbound Bay Street and a changed cycle length of 90 seconds for desired progression along the Bay Road Corridor.

Due to the excessive distance between the crosswalks and stop bars, some drivers may find themselves in the 'dilemma zone' in the intersection for an extended period of time. The Michigan Manual of Uniform Traffic Control Devices recommends that stop bars at intersections be placed four feet in advance of the nearest crosswalk line. On the westbound approach of West Genesee Street, this distance is 10 feet and 11 feet on the southbound approach of Bay Street. It appears that the stop bar for both approaches could be moved closer to the crosswalk. The eastbound approach of West Genesee Street does not have a stop bar or crosswalk, so it is recommended to add both.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.11.6.

The crosswalk should be relocated on the westbound approach. A new stop bar and cross walk should be provided on the eastbound approach. Pavement markings should be upgraded on the southbound, eastbound and westbound approaches with proper directional arrows.

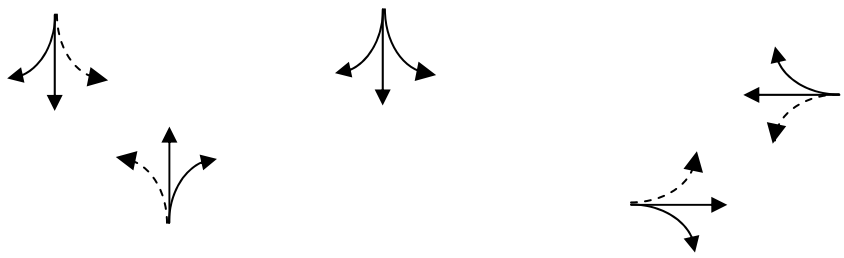
The existing yellow and all-red intervals are shown in Table 6.11.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.11.1. Existing and Proposed Clearance Intervals for the Bay Street (M-84) and West Genesee Street Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
SB Only Phase	-	-	3.6	1.5	Less rear end, left-turn head-on and injury crashes
N-S Through Phase	4.0	1.0	3.6	1.5	
E-W Through Phase	4.2	0.0	3.5	2.2	

The proposed PM peak hour signal timing plan based on a cycle length of 90 seconds is as follows:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



N-S Bay	SB Bay	E-W West Genesee
G = 37.9 sec	G = 10.9 sec	G = 25.3 sec
Y = 3.6 sec	Y = 3.6 sec	Y = 3.5 sec
*R = 47.0 sec	*R = 74.0 sec	*R =59.0 sec
AR = 1.5 sec	AR = 1.5 sec	AR = 2.2 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Bay Street to 3.6 seconds and for West Genesee Street reduce it to 3.5 seconds.
- Increase the all-red interval for Bay Street to 1.5 seconds and for the West Genesee Street to 2.2 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Provide a modified three-phase signal system.
- Post speed limit signs downstream of the intersection for all directions of travel excluding eastbound.
- Provide a new stop bar and pedestrian crossing markings on the eastbound approach.
- Relocate stop bars on the southbound and westbound approaches and crosswalk on the westbound approach as per MMUTCD guidelines.
- Upgrade pavement markings for the southbound, eastbound and westbound approaches with proper directional arrows.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours.

The proposed diagram in Figure 6.11.6 shows the proposed improvements which include the installation of mast-arm signals, upgrading the pavement markings on the southbound, eastbound and westbound approaches and the proposed signal timing plan.

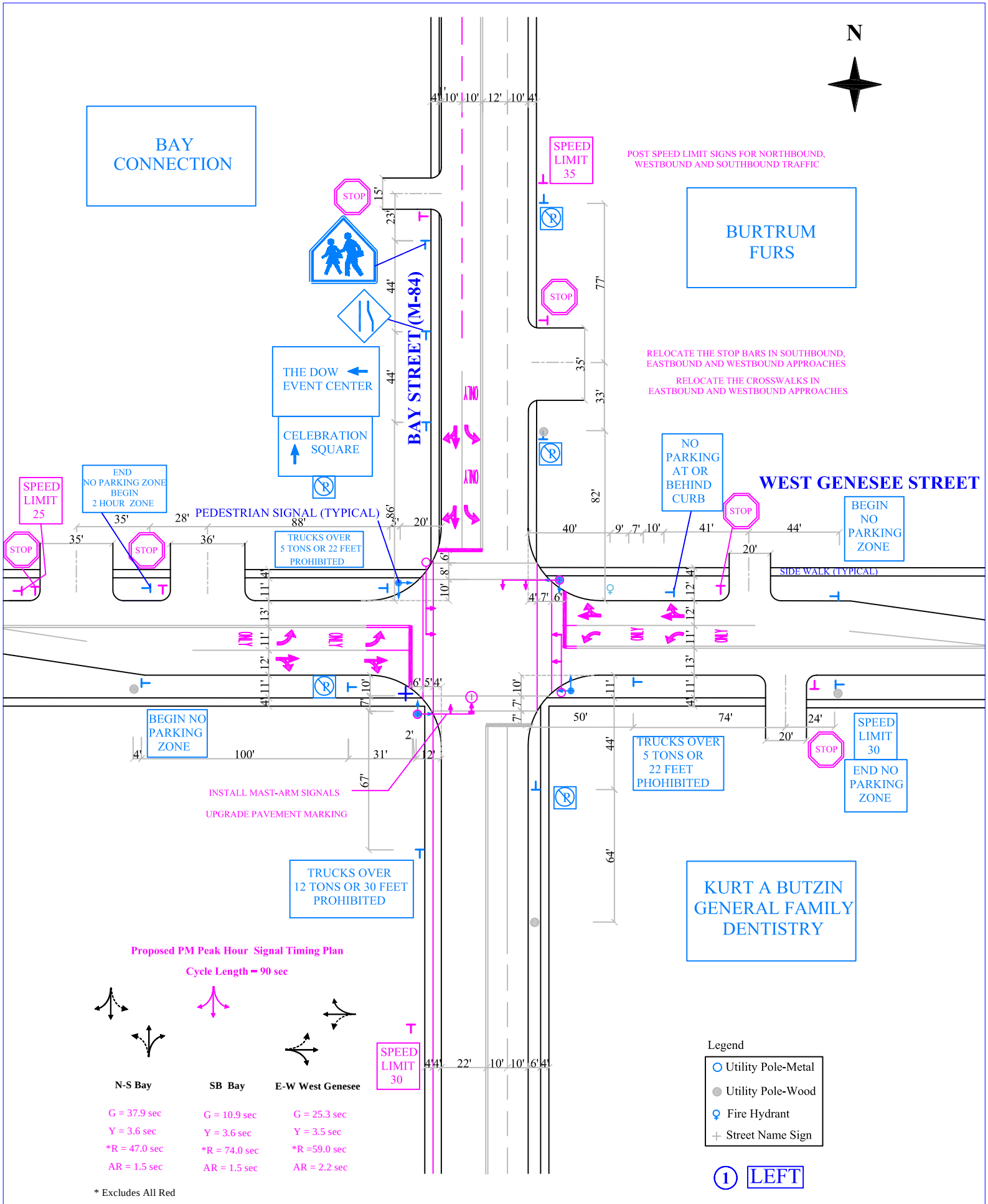


Figure 6.11.6 Proposed Diagram of Bay Street (M-84) and West Genesee Street Intersection (Not to Scale)

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Street and West Genesee Street indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is A with a delay of 8.6 seconds per vehicle.

With the noted improvements and modifications to the signal timing and phasing including signal progression, the overall intersection will operate at Level of Service B with an increased delay of 18.9 seconds per vehicle. The overall delay at the intersection will increase due to the inclusion of a split phase and the provision of adequate yellow and all red intervals. However, the safety at the intersection will be improved along with the traffic progression.

The level of service results for the existing conditions and with the proposed improvements are shown in Table 6.11.2.

Table 6.11.2.

Bay Street (M-84) and West Genesee Street
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB West Genesee Street	28.1	C	23.7	C
WB West Genesee Street	34.2	C	27.1	C
NB Bay Street	2.2	A	21.6	B
SB Bay Street	2.9	A	14.0	B
Overall Intersection	**8.6**	**A**	**18.9**	**B**

12. Bay Street (M-84) and Congress Avenue

The intersection of Bay Street and Congress Avenue is located in the northwest part of the City of Saginaw. Bay Street runs in the north south direction and Congress Avenue runs in the east west direction. Bay Street is a state trunk line also known as M-84 and Congress Avenue is classified as an arterial.

Bay Street is 36 feet wide on the south leg of the intersection with one shared through and left turn lane and one shared through and right turn lane. It is 37 feet wide and has an exclusive left turn lane and one shared through and right turn lane for the southbound approach.

Congress Avenue is 42 feet wide with one exclusive left-turn lane and one shared through and right-turn lane for both eastbound and westbound approaches.

The speed limit for both Bay Street and Congress Avenue is 30 miles per hour.

Land Use

The intersection of Bay Street and Congress Avenue is located in a residential area. There are residential homes on the all four corners of the intersection. There are multiple driveways close to the intersection to serve these residential homes.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.12.1. Details of the intersection

approaches are shown in photographs 12a through 12d. The photographs show the span-wire signal installation at the intersection along with the existing pavement markings.

Photograph 12a. NB Bay Street at Congress Ave

Photograph 12b. SB Bay Street at Congress Ave

Photograph 12c. EB Bay Street at Congress Ave

Photograph 12d. WB Bay Street at Congress Ave

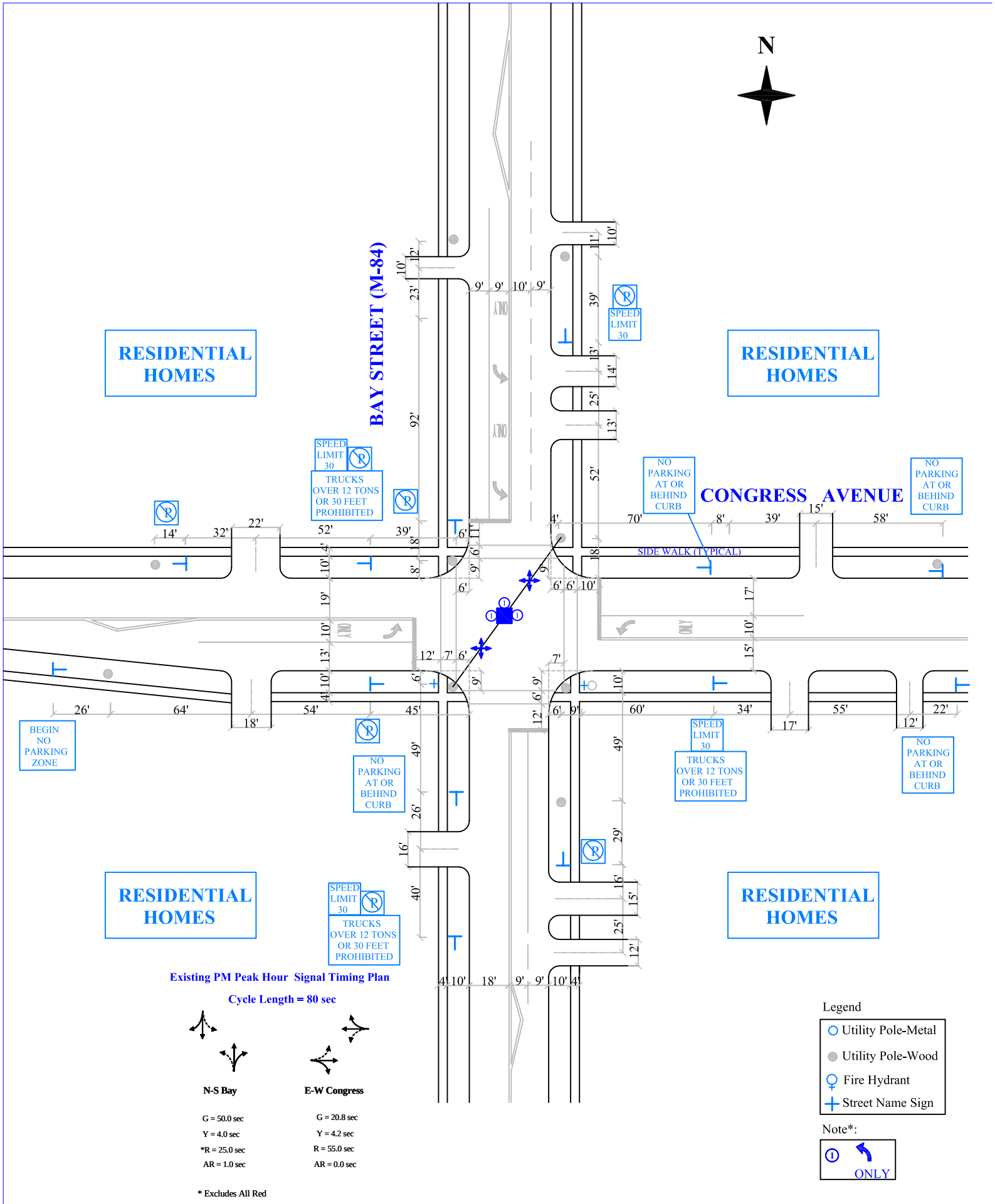


Figure 6.12.1 Existing Condition Diagram of Bay Street and Congress Avenue Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on a clear day on January 24th, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:30 to 5:30 PM and is presented in Figure 6.12.2. The traffic patterns indicate that the total intersection volume is moderately low with 1,208 vehicles per hour.

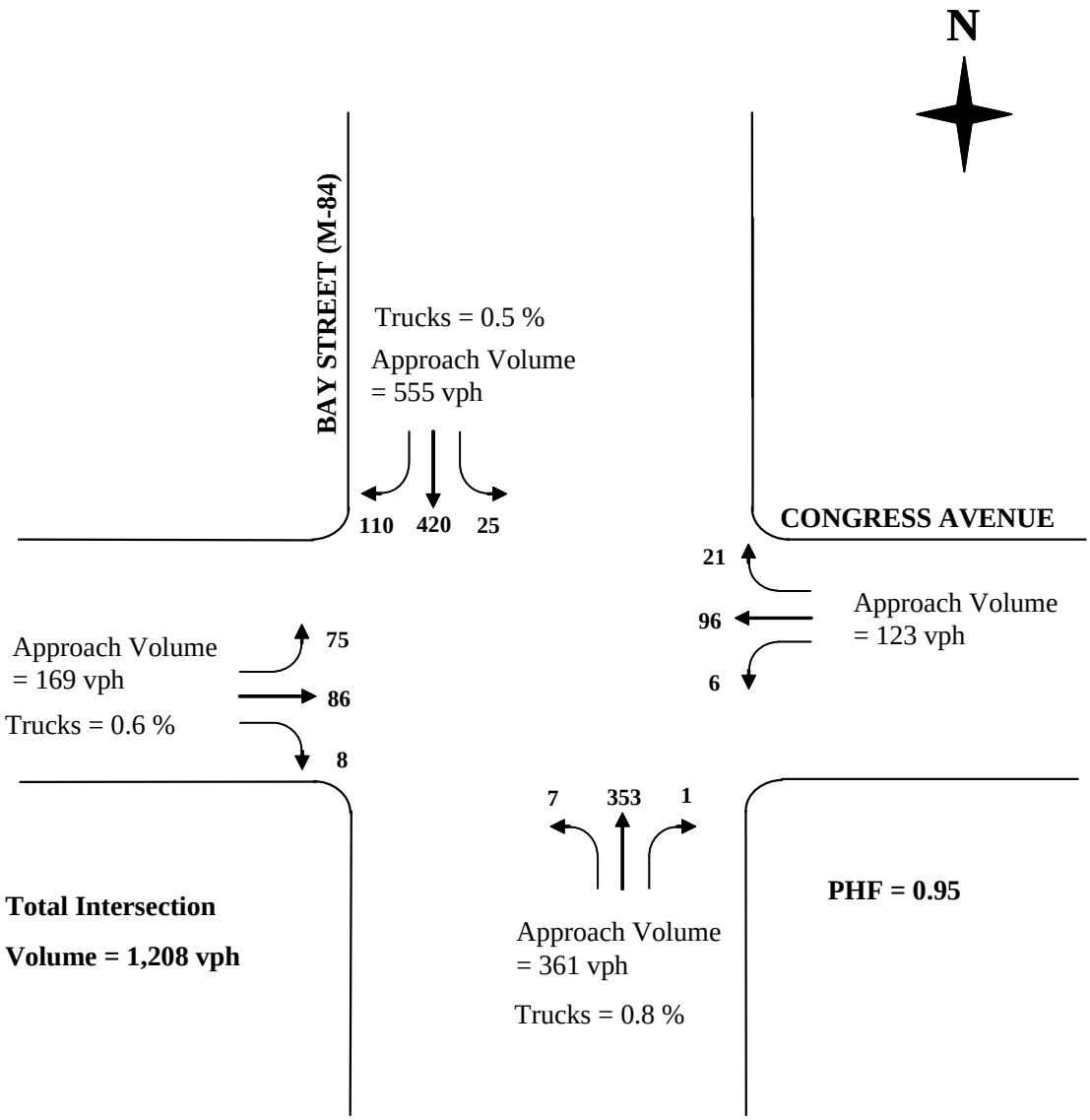


Figure 6.12.2. Existing PM Peak Hour (4:30–5:30 PM) Traffic Volumes for the Intersection of Bay Street (M-84) and Congress Avenue

Traffic Control

This intersection is controlled by a two-phase span-wire traffic signal with an 80-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for the north-south phase and a yellow interval of 4.2 seconds and an all-red interval of 1.0 second for the east-west phase. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.12.1.

Crash Analysis

A total of eleven crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of six crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Bay Street and Congress Avenue are shown in Figures 6.12.3 and 6.12.4.

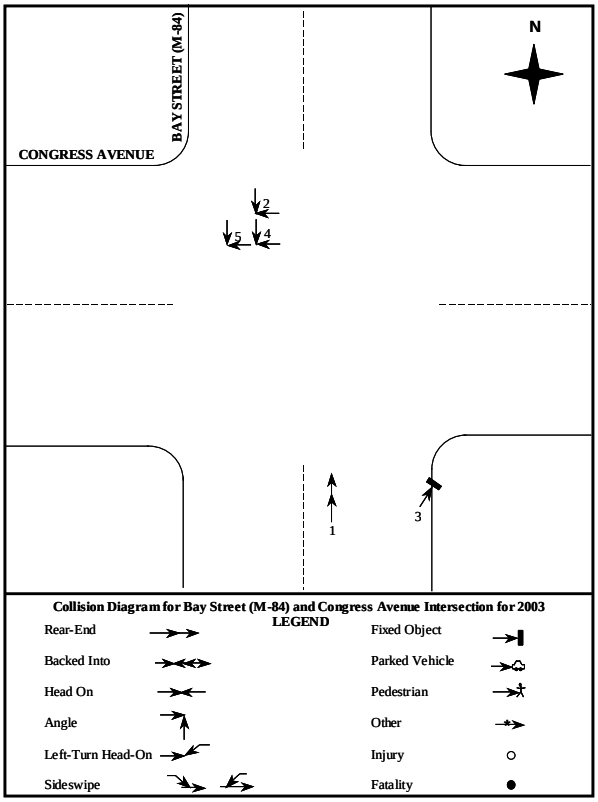


Figure 6.12.3. Collision Diagram for Bay Street and Congress Avenue for 2003

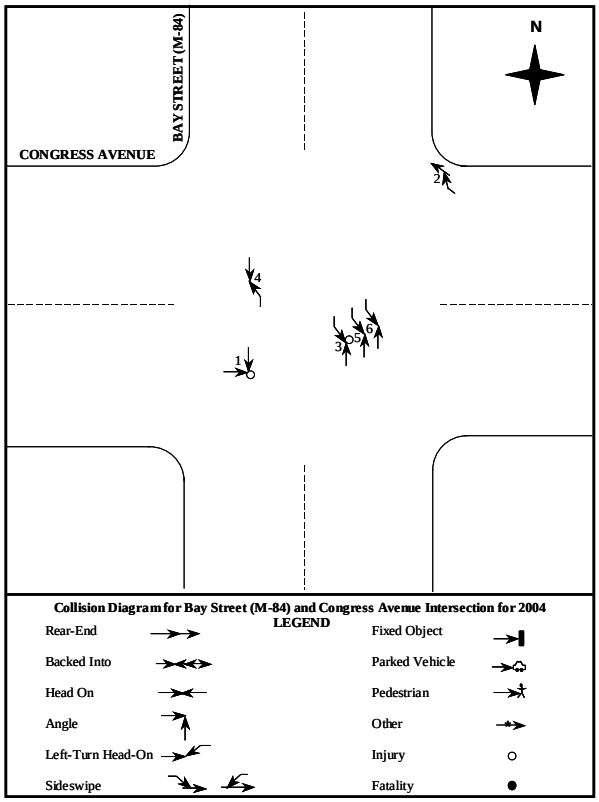


Figure 6.12.4. Collision Diagram for Bay Street and Congress Avenue for 2004

The crash data shows the annual frequency of six, which indicates that the annual crash rate at the intersection per million entering vehicles is also very nominal, considering the approach volume. Angle crashes and left-turn head-on crashes are of specific significance, from which angle crashes may have occurred due to insufficient clearance intervals, and left-turn head-on collisions which may have occurred due to the interference of oncoming northbound through traffic with the southbound left turns during the permissive phase.

The crash distribution, based upon type of crashes, is shown in Figure 6.12.5. Left-turn head-on, angle, rear-end, sideswipe same and fixed object crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes includes four left-turn head-on crashes and four angle crashes.

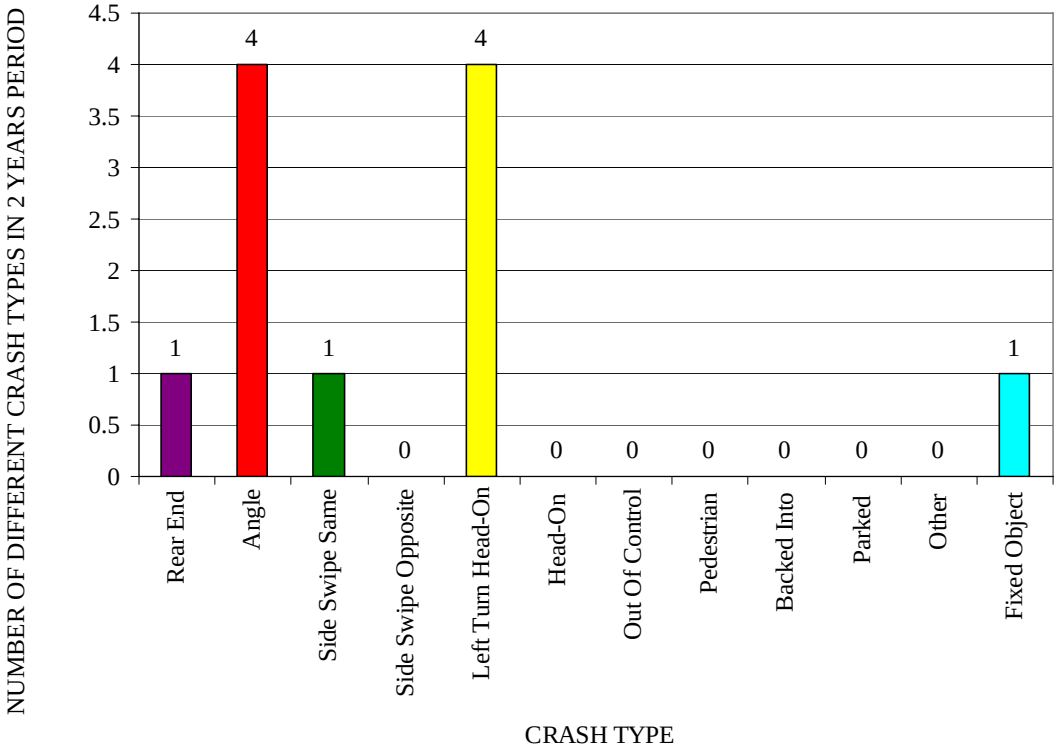


Figure 6.12.5 Two Year (2003 & 2004) Crash Experience at Bay Street (M-84) and Congress Avenue Intersection

Safety Deficiencies and Recommended Improvements:

The majority of the crashes occurring at this intersection of Bay Street and Congress Avenue are of the left-turn head-on and angle types, which occur due to several possible reasons including permissive left turn movement and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan.

Due to the excessive distance between the crosswalks and stop bars, some drivers may find themselves in the 'dilemma zone' in the intersection for an extended period of time. The Michigan Manual of Uniform Traffic Control Devices recommends that stop bars at intersections be placed four feet in advance of the nearest crosswalk line. On the westbound approach of Congress Avenue, this distance is 10 feet and 12 feet on the eastbound approach. On the northbound approach of Bay Street, this distance is 12 feet and 11 feet on the southbound approach. It appears that the stop bar for each approach could be moved closer to the crosswalk.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.12.6.

The cycle length of the proposed signal-timing plan has been changed to 90 seconds to be compatible with the progression along the Bay Road Corridor.

The stop bar should be relocated on each approach. Pavement markings should be upgraded with proper directional arrows on all approaches. Provide two approach and two receiving lanes for westbound through traffic.

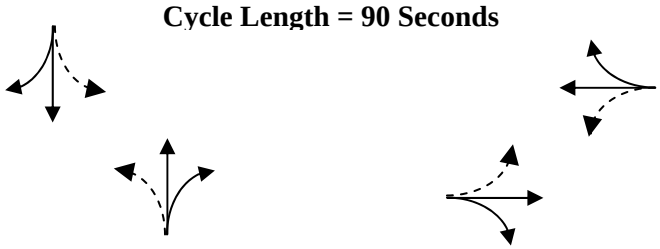
The existing yellow and all-red intervals are shown in Table 6.12.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.12.1 Existing and Proposed Clearance Intervals for the Bay Street (M-84) and Congress Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	4.0	1.0	3.5	1.9	Less angle, left-turn head-on and injury crashes
E-W Through Phase	4.2	1.0	3.5	1.7	

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



N-S Bay	**E-W Congress**
G = 52.6 sec	G = 26.8 sec
Y =3.5 sec	Y = 3.5 sec
*R = 32.0 sec	*R = 58.0 sec
AR = 1.9 sec	AR = 1.7 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Bay Street and Congress Avenue to 3.5 seconds.
- Increase the all-red interval for Bay Street to 1.9 seconds and for the Congress Avenue to 1.7 seconds.
- Relocate the stop bar on each approach, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the pavement markings for all approaches with proper directional arrows.

- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours.

The proposed condition diagram in Figure 6.12.6 (on page 206) shows the proposed improvements which include the installation of mast-arm signals, upgrading the pavement markings on the all approaches and the proposed signal timing plan with adequate clearance intervals.

Operational Evaluation

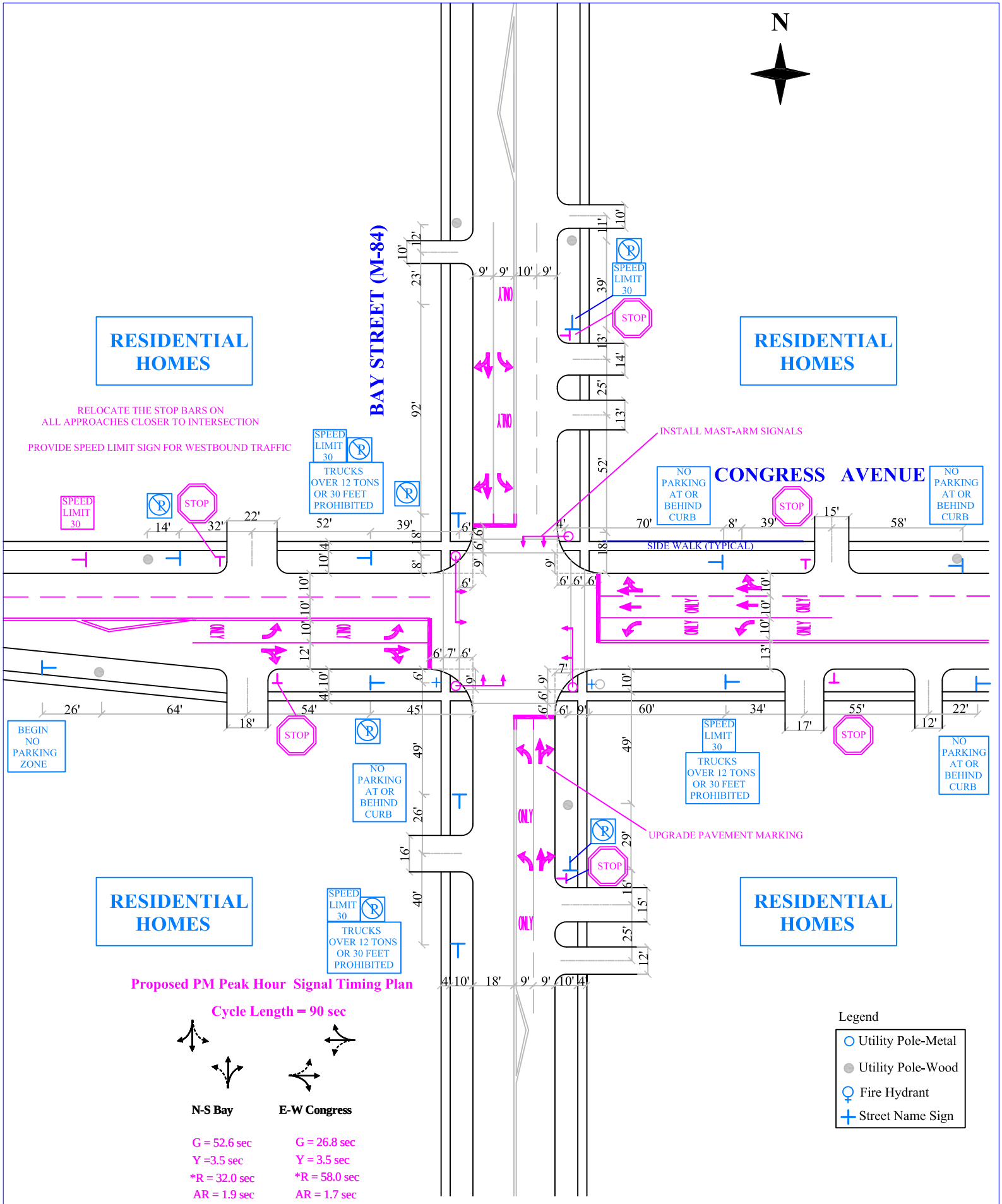
The results of the level of service analysis for the intersection of Bay Street and Congress Avenue indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is A with a delay of 9.4 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection will operate at the level of service A with a reduced delay of 7.8 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic and enhanced safety along with improved progression.

The levels of service results for the existing conditions and with the proposed improvements are summarized in Table 6.12.2.

Table 6.12.2.
Bay Street (M-84) and Congress Avenue
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Congress Avenue	24.4	C	23.6	C
WB Congress Avenue	23.9	C	23.3	C
NB Bay Street	4.0	A	2.1	A
SB Bay Street	5.1	A	3.2	A
Overall Intersection	**9.4**	**A**	**7.8**	**A**



Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 sec

N-S Bay	E-W Congress
G = 52.6 sec	G = 26.8 sec
Y =3.5 sec	Y = 3.5 sec
*R = 32.0 sec	*R = 58.0 sec
AR = 1.9 sec	AR = 1.7 sec

* Excludes All Red

Figure 6.12.6 Proposed Diagram of Bay Street and Congress Avenue Intersection (Not to Scale)

13. Bay Street (M-84) and Cooper Avenue

The intersection of Bay Street (M-84) and Cooper Avenue is located in the northwestern part of the City of Saginaw. Bay Street runs in the north-south direction and Cooper Avenue is in the east-west direction. Bay Road is a state trunk line, M-84 but at this intersection, it is called Bay Street and has the nature of providing more accessibility than mobility. Cooper Avenue is classified as a local street.

At the northbound and southbound approaches, Bay Street is 35 feet wide and consists of one exclusive left turn lane and one shared through and right turn lane. Cooper Avenue is 26 feet wide with one approach lane for eastbound traffic. The road is 31 feet wide with one exclusive right turn lane and one shared through and left turn lane for westbound traffic.

The speed limit for Bay Street is 30 miles per hour and Cooper Avenue has a speed limit of 25 miles per hour.

Land Use

The intersection of Bay Street and Cooper Avenue is occupied by residential homes on all the corners except the northwest corner of the intersection. There is a party shop on the northwest corner of the intersection. There are multiple driveways close to the intersection, which serve the residential homes and other establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Bay Street and Cooper Avenue.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.13.1.

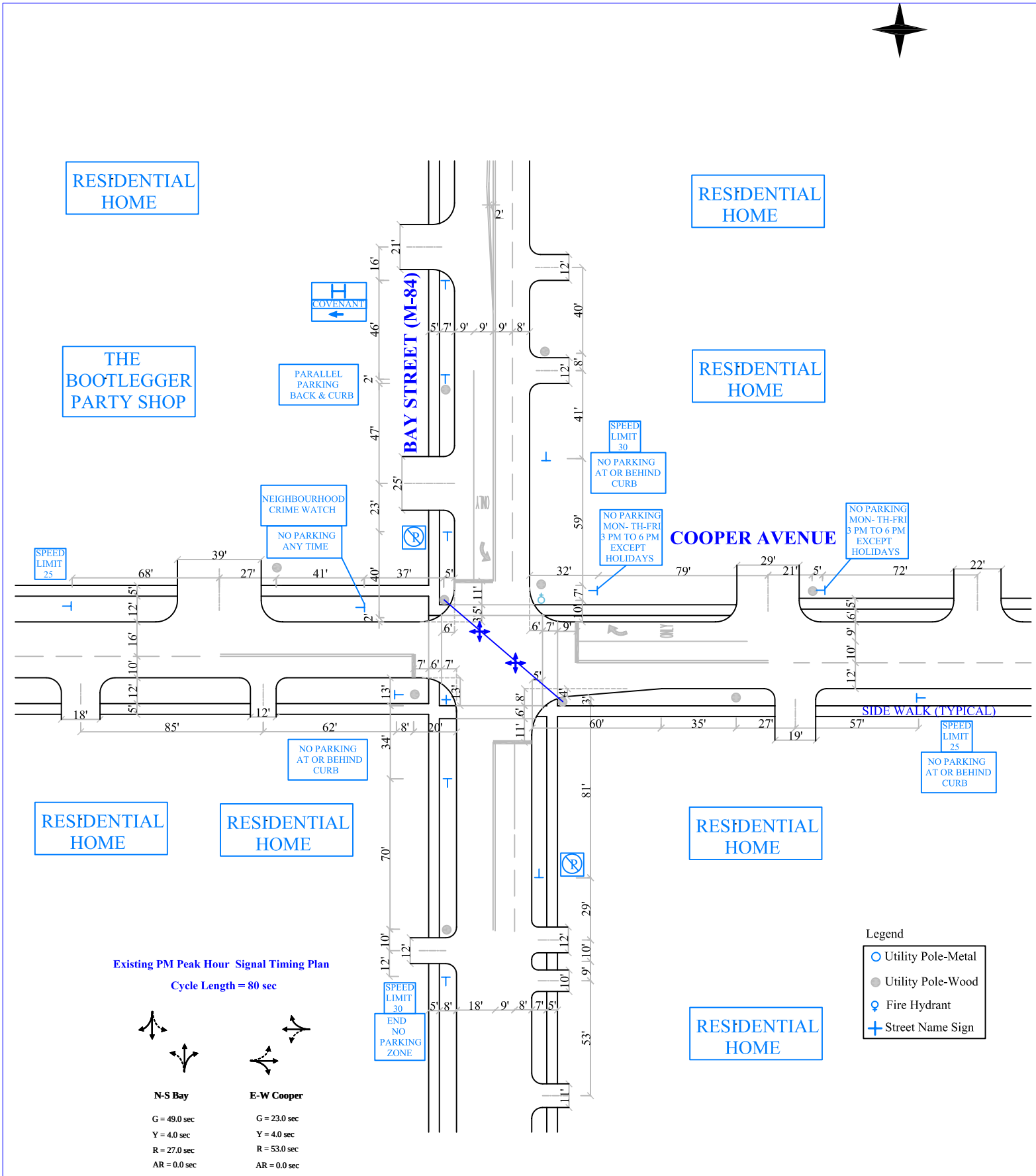


Figure 6.13.1 Existing Condition Diagram of Bay Street (M-84) and Cooper Avenue Intersection (Not to Scale)

Details of the intersection approaches are shown in photographs 13a through 13d. These photographs show the existing span-wire installation. Pavement markings are clearly shown on the photographs.

Photograph 13a. EB Cooper Avenue at Bay Street

Photograph 13b. WB Cooper Avenue at Bay Street

Photograph 13c. NB Bay Street at Cooper Avenue

Photograph 13d. SB Bay Street at Cooper Avenue

Peak Hour Volumes

PM peak period traffic volumes were collected manually on a clear day on January 24, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:00 to 5:00 PM and is presented in Figure 6.13.2. The traffic patterns indicate that Bay Street carries substantially higher volumes than Cooper Avenue.

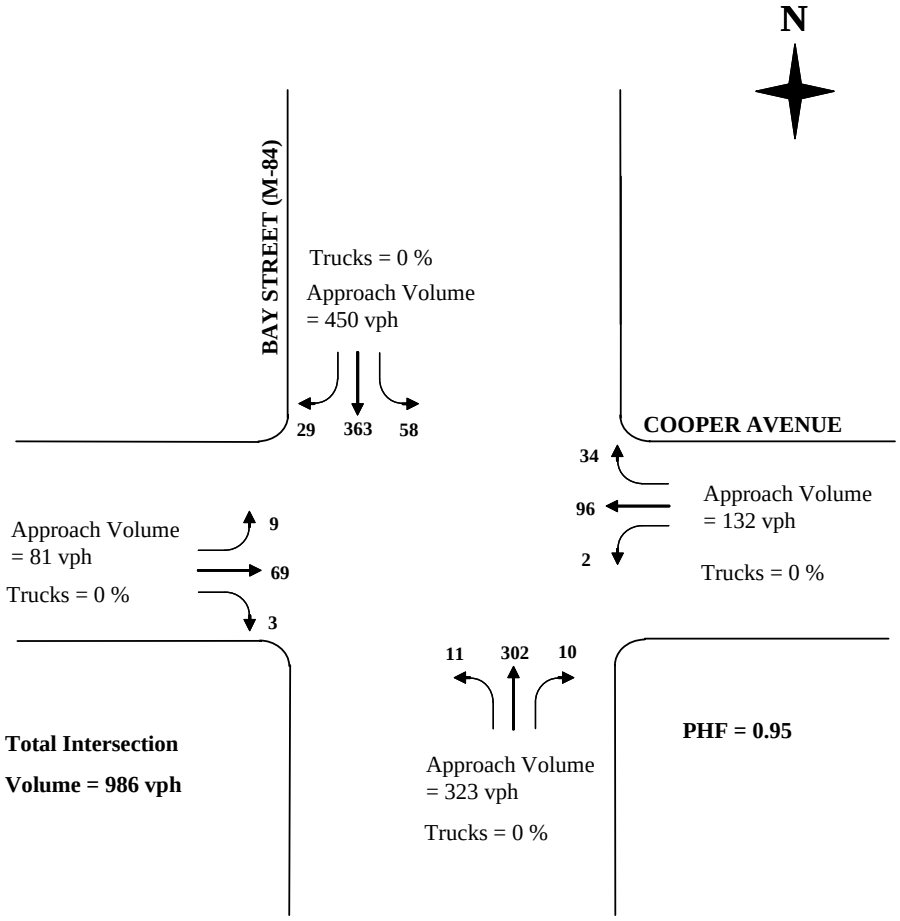


Figure 6.13.2. Existing PM Peak Hour (4:00–5:00 PM) Traffic Volumes for the Intersection of Bay Street (M-84) and Cooper Avenue

Traffic Control

This intersection is controlled by a two-phase span-wire traffic signal with an 80-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds for each phase. An all-red interval is not provided for either. The existing PM peak hour signal timing plan is shown in the existing condition diagram in Figure 6.13.1 (on page 210).

Crash Analysis

For the intersection of Bay Street and Cooper Avenue, only four crashes were recorded for the year 2004. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. The collision diagram for the year 2004 for the intersection is shown in Figure 6.13.3.

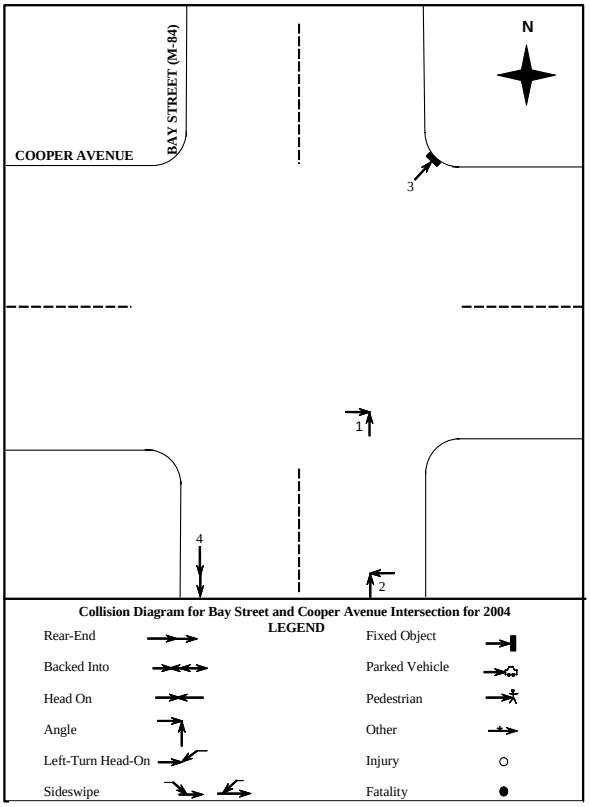


Figure 6.13.3 Collision Diagram for Bay Street and Cooper Avenue for 2004

Safety Deficiencies and Recommended Improvements

Crashes occurring at this intersection of Bay Street and Cooper Avenue are of the angle and rear end type, happening due to several possible reasons including poor visibility of signals and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan.

Due to the excessive distance between the crosswalks and stop bars, some drivers may find themselves in the 'dilemma zone' in the intersection for an extended period of time. The Michigan Manual of Uniform Traffic Control Devices recommends that stop bars at

intersections be placed four feet in advance of the nearest crosswalk line. On the westbound approach of Cooper Avenue, this distance is nine feet. On the northbound and southbound approach of Bay Street, this distance is 11 feet. It appears that the stop bar for all these approaches could be moved closer to the crosswalk.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation.

The cycle length of the proposed signal-timing plan has been changed to 90 seconds to be compatible with the progression along Bay Corridor.

The stop bar should be relocated on the northbound, southbound and westbound approaches. Pavement markings should be upgraded with proper directional arrows on the northbound, southbound and westbound approaches.

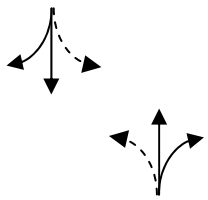
The existing yellow and all-red intervals are shown in Table 6.13.1, along with the proposed lengths of these intervals and their potential benefits.

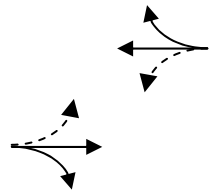
Table 6.13.1 Existing and Proposed Clearance Intervals for the Bay Street (M-84) and Cooper Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.0	0.0	3.5	2.0	Reduced probability of occurrence of rear-end and angle crashes
N-S Through Phase	4.0	0.0	3.5	1.6	

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds





N-S Bay	**E-W Cooper**
G = 46.9	G = 32.5
Y = 3.5	Y = 3.5
R = 38.0	R = 52.0
AR = 1.6	AR = 2.0

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for the Bay Street and Cooper Avenue to 3.5 seconds.
- Increase the all-red interval for the Bay Street to 1.6 seconds and for the Cooper Avenue to 2.0 seconds.
- Relocate the stop bar on the northbound, southbound and westbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade pavement markings for the northbound, southbound and westbound approaches with proper directional arrows.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours.

The proposed intersection diagram in Figure 6.13.4 shows the proposed improvements which include the installation of mast-arm signals, upgrading the pavement markings on the all approaches and the proposed signal timing plan.

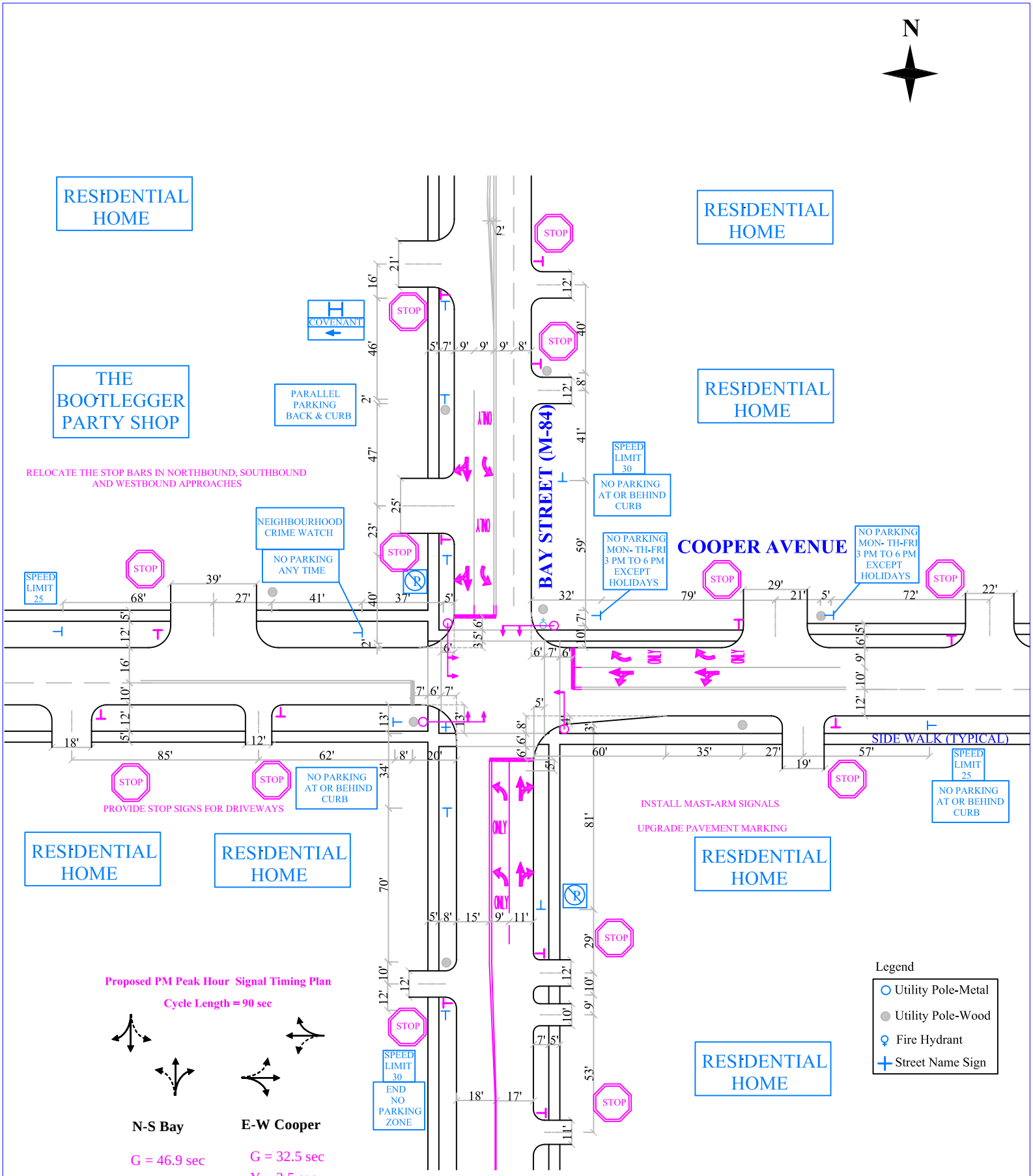


Figure 6.13.4 Proposed Diagram of Bay Street (M-84) and Cooper Avenue Intersection (Not to Scale)

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Street and Cooper Avenue indicate that, for the existing conditions during the PM peak hour, the overall intersection level of service is A with a delay of 8.6 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall level of service of the intersection will remain at A with a marginally increased delay of 9.3 seconds per vehicle. The modified signal timing and changed cycle length along with the adjusted clearance intervals will ensure enhanced safety and improved progression.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.13.2.

Table 6.13.2
Bay Street (M-84) and Cooper Avenue
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Cooper Avenue	22.1	C	18.7	B
WB Cooper Avenue	22.2	C	18.8	B
NB Bay Street	4.9	A	9.9	A
SB Bay Street	5.1	A	4.3	A
Overall Intersection	**8.6**	**A**	**9.3**	**A**

14. Bay Street (M-84) and Court Street

The intersection of Bay Street (M-84) and Court Street is located in the northwestern part of the City of Saginaw. Bay Street runs in the north-south direction and Court Street runs in the east-west direction. Bay Street is a state trunk line and is also known as M-84. The south portion of the intersection is named as South Alexander Street. Court Street is classified as an arterial street.

At the northbound approach, Bay Street is 35 feet wide with one approach lane. At the south bound approach, it is 54 feet wide with one dedicated left-turn lane and one shared through and right turn lane.

At the eastbound approach, Court Street is 53 feet wide with one left-turn lane, one through lane and one shared through and right turn lane. At the westbound approach, Court Street is 56 feet wide with two through lanes and one exclusive right turn lane. Three is an 8 foot wide area which allows parallel parking on the westbound approach of the intersection.

The speed limit for Bay Street is 25 miles per hour and for Court Street, it is 30 miles per hour.

Land Use

There is a "Page-Tel" Cellular shop on the northeast corner and a Pizza shop on the northwest corner of the intersection. There are some shops and residential homes on the southeast and southwest corners of the intersection. There are multiple driveways close to the intersection which serve these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.14.1. Details of the intersection approaches are shown in photographs 14a through 14d. The photographs show the existing span-wire installation with the hanging signal heads and the pavement markings on the intersection approaches.

Photograph 14a. NB Bay Street at Court Street

Photograph 14b. SB Bay Street at Court Street

Photograph 14c. EB Court Street at Bay Street

Photograph 14d. WB Court Street at Bay Street

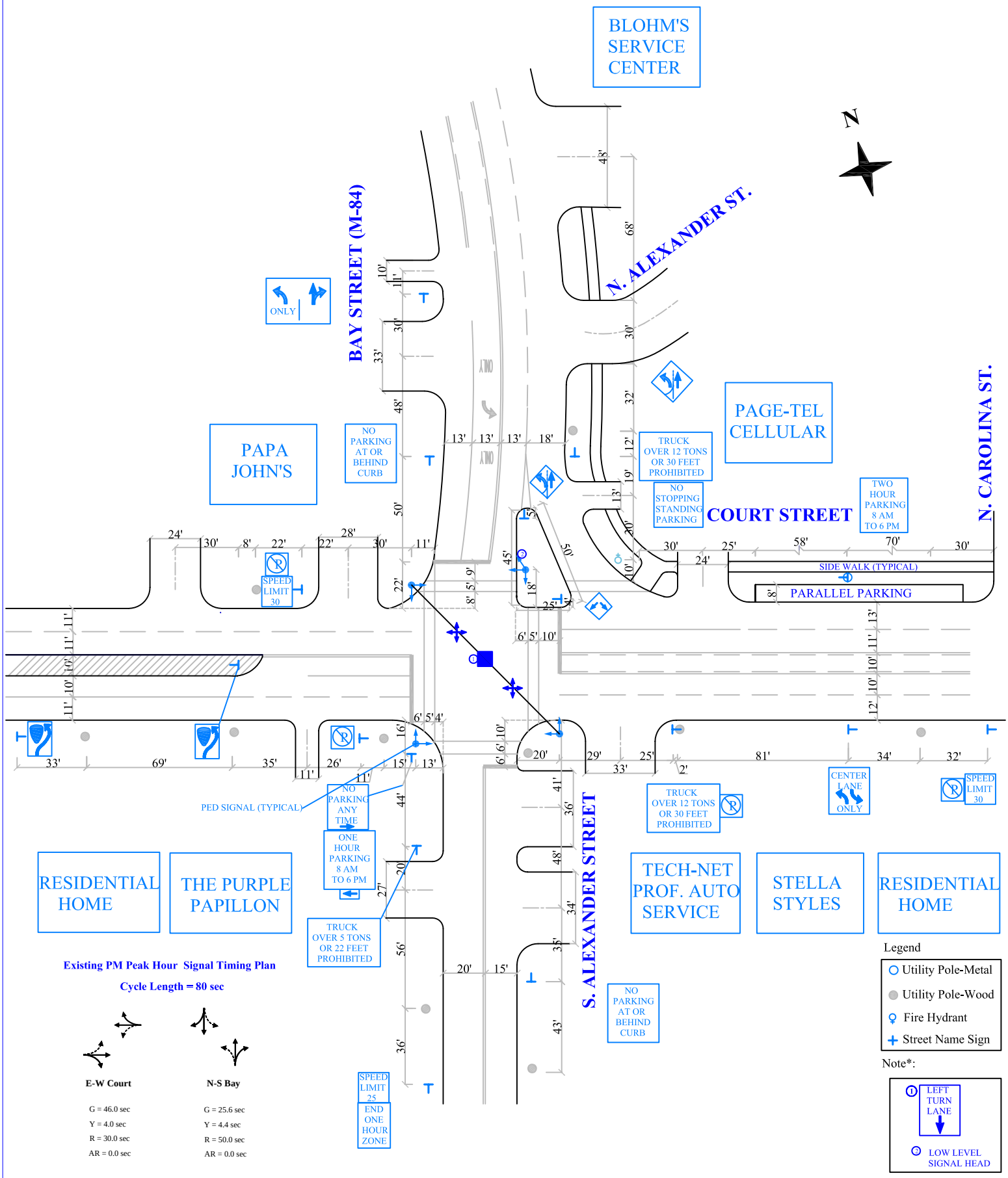


Figure 6.14.1 Existing Condition Diagram of Bay Street (M-84) and Court Street Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected on a snowy day on January 24, 2006 from 4 PM to 6 PM by two people standing at a suitable corner of the intersection and covering two intersecting approaches each. The results of the data are presented in Figure 6.14.2. The traffic patterns indicate that the volumes in the east-west direction are much higher than the north-south direction. Overall, the intersection carries 1,315 vehicles during the PM peak hour.

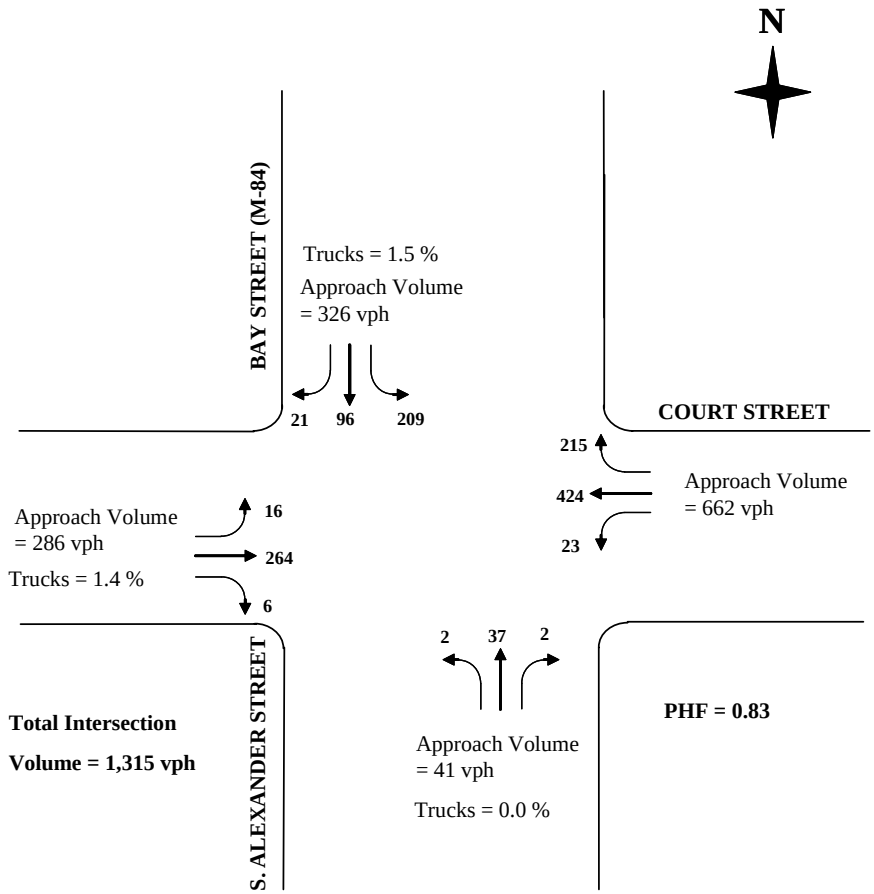


Figure 6.14.2. Existing PM Peak Hour (4:30–5:30 PM) Traffic Volumes for the Intersection of Bay Street (M-84) and Court Street

Traffic Control

This intersection is controlled by a two-phase span-wire signal installation with an 80 second cycle length. The existing clearance interval includes a 4.0 second yellow interval for the east-west phase. The north-south phase has a 4.4 second yellow interval. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.14.1 on page 220.

Crash Analysis

A total of five crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of three crashes. Collision diagrams were prepared based on the UD-10 report's information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Bay Street and Court Street are shown in Figures 6.14.3 and 6.14.4.

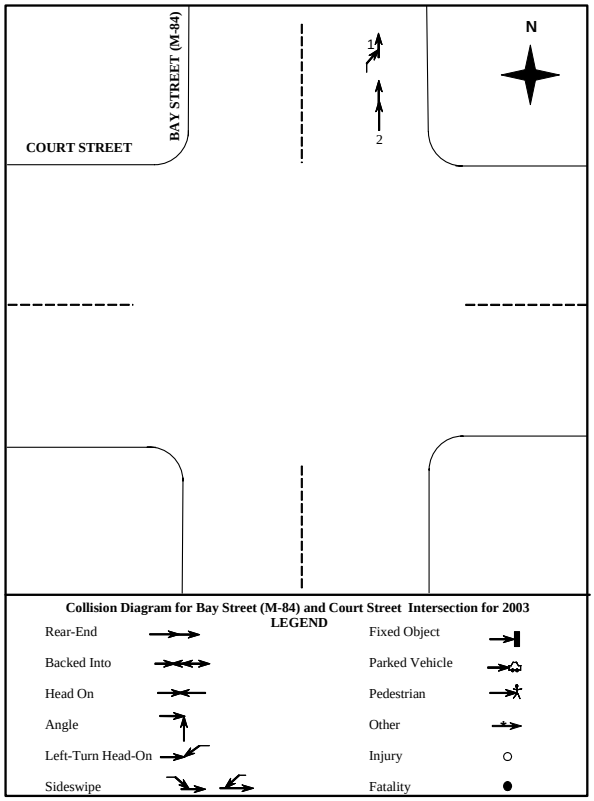


Figure 6.14.3. Collision Diagram for Bay Street and Court Street for 2003

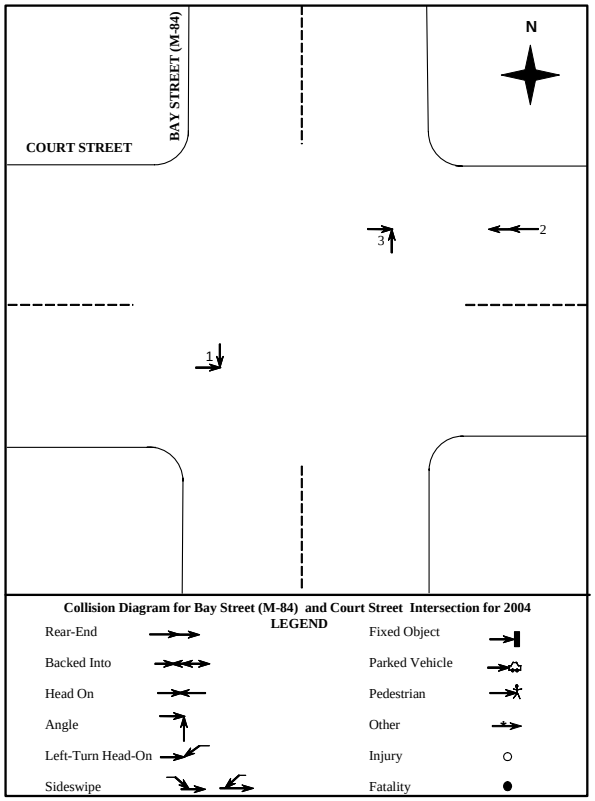


Figure 6.14.4. Collision Diagram for Bay Street and Court Street for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.50 per million entering vehicles, which is low, considering the approach volume at the intersection. There are two angle crashes reported at the intersection.

The crash distribution, based upon type of crashe, is shown in Figure 6.14.5. Rear-end, angle, and sideswipe same crashes are the predominant crashes reported at this

intersection. The corresponding two-year crash frequency for these crashes include, two rear end crashes, two angle crashes and one sideswipe same crash.

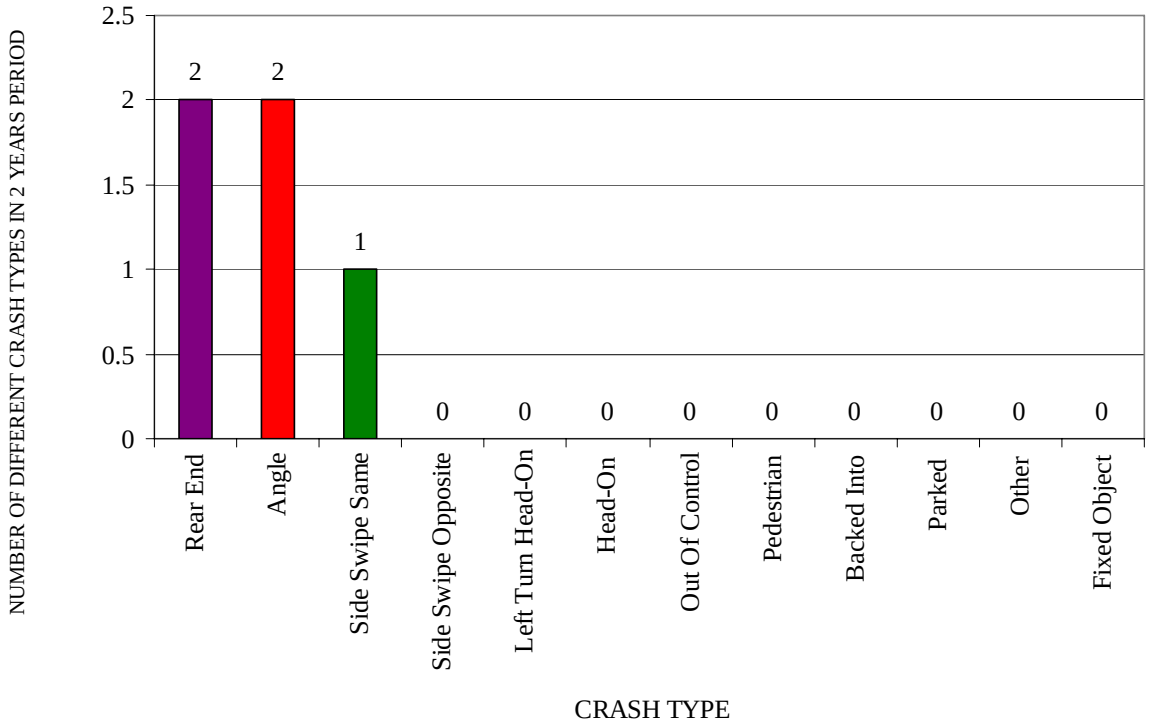


Figure 6.14.5 Two year (2003 & 2004) Crash Experience at Bay Street (M-84) and Court Street Intersection

Safety Deficiencies and Recommended Improvements

Crashes occurring at this intersection of Bay Street and Cooper Avenue are of the angle and rear end type, which occur due to several possible reasons including poor visibility of traffic signals and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan. The left turn volume is high at the southbound approach compared to the northbound approach. The existing signal-timing plan was modified to include a split phase for southbound Bay Street and has an adjusted cycle length of 90 seconds to allow desired progression along the Bay Road Corridor.

Due to the excessive distance between the crosswalks and stop bars, some drivers may find themselves in the 'dilemma zone' in the intersection for an extended period of time. The Michigan Manual of Uniform Traffic Control Devices recommends that stop bars at intersections be placed four feet in advance of the nearest crosswalk line. On the

southbound approach of Bay Street, this distance is nine feet and for the northbound approach it is ten feet. On the westbound approach of Court Street, this distance is ten feet. It appears that the stop bar for these approaches could be moved closer to the crosswalk.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.14.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The stop bar should be relocated on the northbound, southbound and westbound approaches. The crosswalk should be relocated on the northbound approach. Pavement markings should be upgraded with proper directional arrows on the eastbound, southbound and westbound approaches.

The existing yellow and all-red intervals are shown in Table 6.14.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.14.1. Existing and Proposed Clearance Intervals for the Bay Street (M-84) and Court Street Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.0	0.0	3.5	1.7	Less rear end, angle, and injury crashes
N-S Through Phase	4.4	0.0	3.5	2.5	
SB Only Phase	-	-	3.5	2.5	

The proposed PM peak hour signal timing plan based on a cycle length of 90 seconds is shown on page 226.

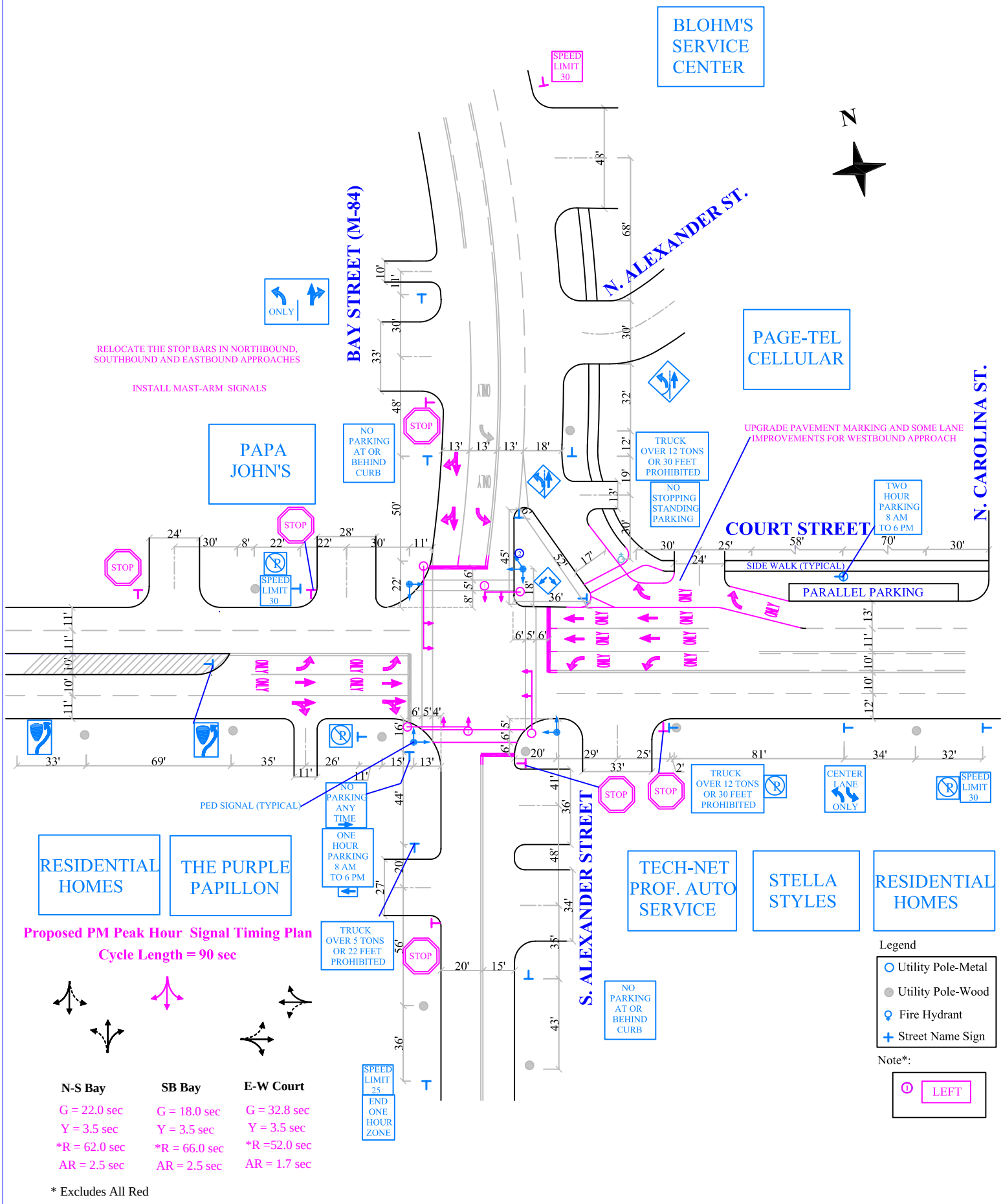
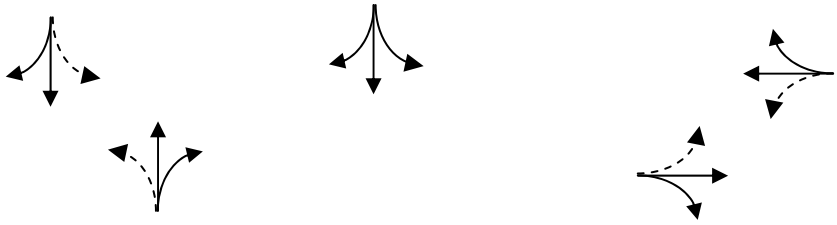


Figure 6.14.6 Proposed Diagram of Bay Street (M-84) and Court Street Intersection (Not to Scale)

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



N-S Bay	S-B Bay	E-W Court
G = 22.0	G = 18.0	G = 32.8
Y = 3.5	Y = 3.5	Y = 3.5
*R = 62.0	*R = 66.0	*R =52.0
AR = 2.5	AR = 2.5	AR = 1.7

* Excludes All

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Bay Street and the Court Street phase to 3.5 seconds.
- Increase the all red interval for Bay Street to 2.5 seconds and for the Court Street to 1.7 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the existing pavement markings for the eastbound, westbound and southbound approaches.
- Relocate the stop bar on the northbound, southbound and westbound approaches and crosswalk on the northbound approach, as per MMUTCD guidelines.
- Provide a modified three-phase signal system.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours.

The proposed intersection diagram in Figure 6.14.6 (on page 225) shows the proposed improvements which include the installation of mast-arm signals, upgrading the pavement markings on the all approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Bay Street and Court Street indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 13.2 seconds per vehicle.

With the modifications to the signal timings, phasing, and the change of the cycle length from 80 seconds to 90 seconds, the overall intersection level of service will be changed to C with an increased delay of 20.1 seconds per vehicle. The effect of this marginally increased delay is accepted to achieve progression and enhance safety for traffic. The clearance intervals are changed from the existing conditions of the intersection to enhance safety.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.14.2.

Table 6.14.2.
Bay Street (M-84) and Court Street
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Court Street	8.1	A	19.7	B
WB Court Street	9.6	A	24.4	C
NB Bay Street	18.2	B	25.1	C
SB Bay Street	23.8	C	11.0	B
Overall Intersection	**13.2**	**B**	**20.1**	**C**

15. Tittabawassee Road and North Center Road

The intersection of Tittabawassee Road and North Center Road is located within the northern boundaries of Saginaw Township. North Center Road runs in the north-south direction and Tittabawassee Road runs in the east-west direction. Both Tittabawassee Road and North Center Road are functionally classified as arterial roads.

Tittabawassee Road is 59 feet wide and has one dedicated left turn lane, one through lane and one shared through and right-turn lane for the eastbound approach. West of the intersection, Tittabawassee Road is 58 feet wide with one exclusive left turn lane, one through lane and one shared through and right turn lane for the westbound approach. North of the intersection, North Center Road is 33 feet wide with an exclusive left turn lane and a shared through and right turn lane for the southbound approach. South of the intersection, North Center Road is 59 feet wide with a dedicated left turn lane, one through lane and an exclusive right-turn lane for the northbound approach.

The speed limit for both Tittabawassee Road and North Center Road is 45 miles per hour.

Land Use

The intersection of Tittabawassee Road and North Center Road has. There is a psychological center with a 56-foot driveway connecting on northwest corner of the intersection. There are empty lands in the other corners of the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and North Center Road.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.15.1. Details of the intersection

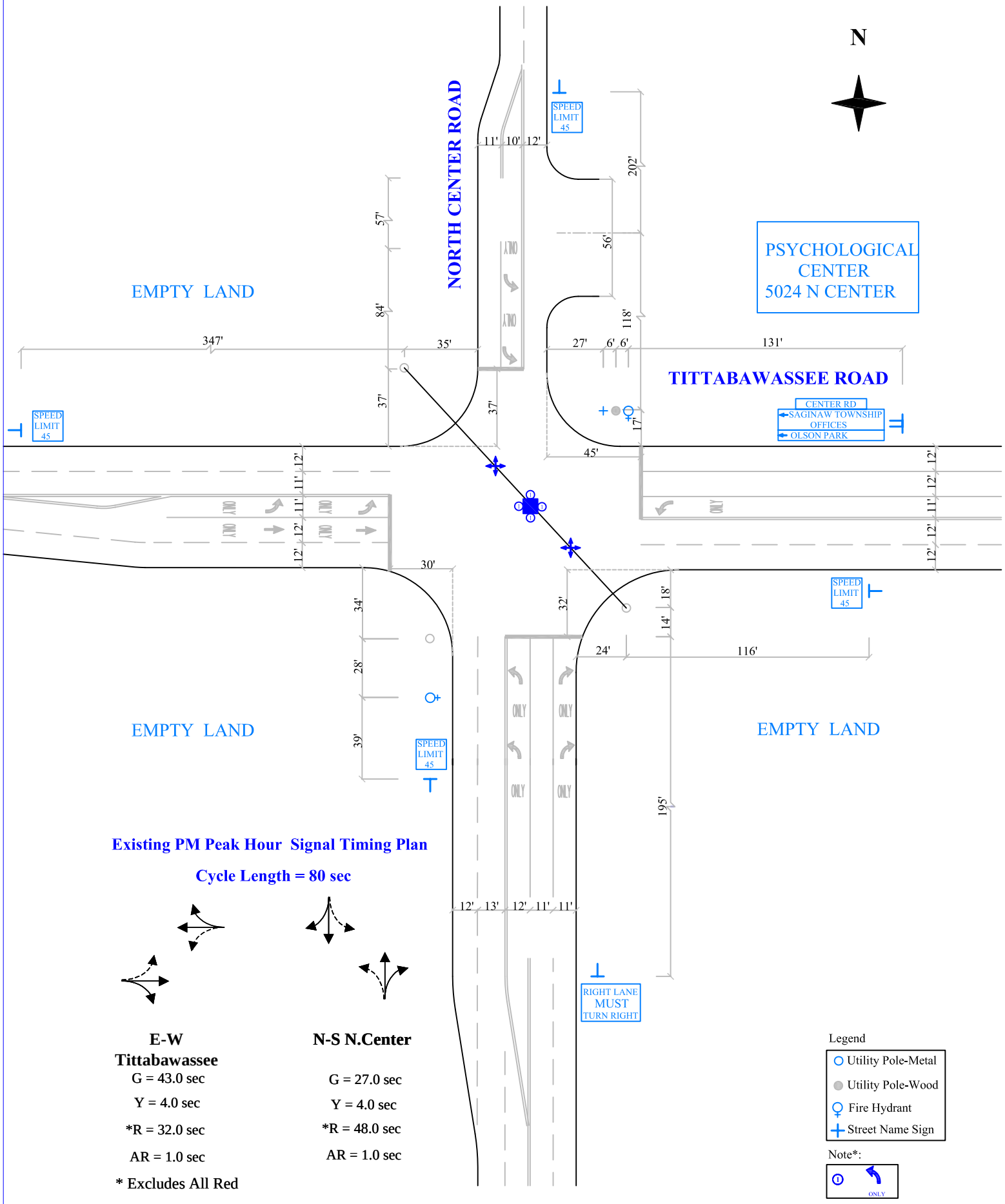
approaches are shown in photographs 15a through 15d. The photographs show the existing span wire signal installation at the intersection along with the existing pavement markings.

Photograph 15a. NB North Center Road at Tittabawassee Road

Photograph 15b. SB North Center Road at Tittabawassee Road

Photograph 15c. EB Tittabawassee Road at North Center Road

Photograph 15d. WB Tittabawassee Road at North Center Road



Existing PM Peak Hour Signal Timing Plan

Cycle Length = 80 sec

E-W Tittabawassee	N-S N.Center
G = 43.0 sec	G = 27.0 sec
Y = 4.0 sec	Y = 4.0 sec
*R = 32.0 sec	*R = 48.0 sec
AR = 1.0 sec	AR = 1.0 sec

* Excludes All Red

Figure 6.15.1 Existing Condition Diagram of North Center Road and Tittabawassee Road Intersection (N ot to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on a clear day on March 28, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two suitable corners of the intersection and covering two intersecting approaches each. The peak hourly volume from 4:45 PM to 5:45 PM is presented in Figure 6.15.2 and represents the highest peak hour (one-hour) volume in the PM peak period.

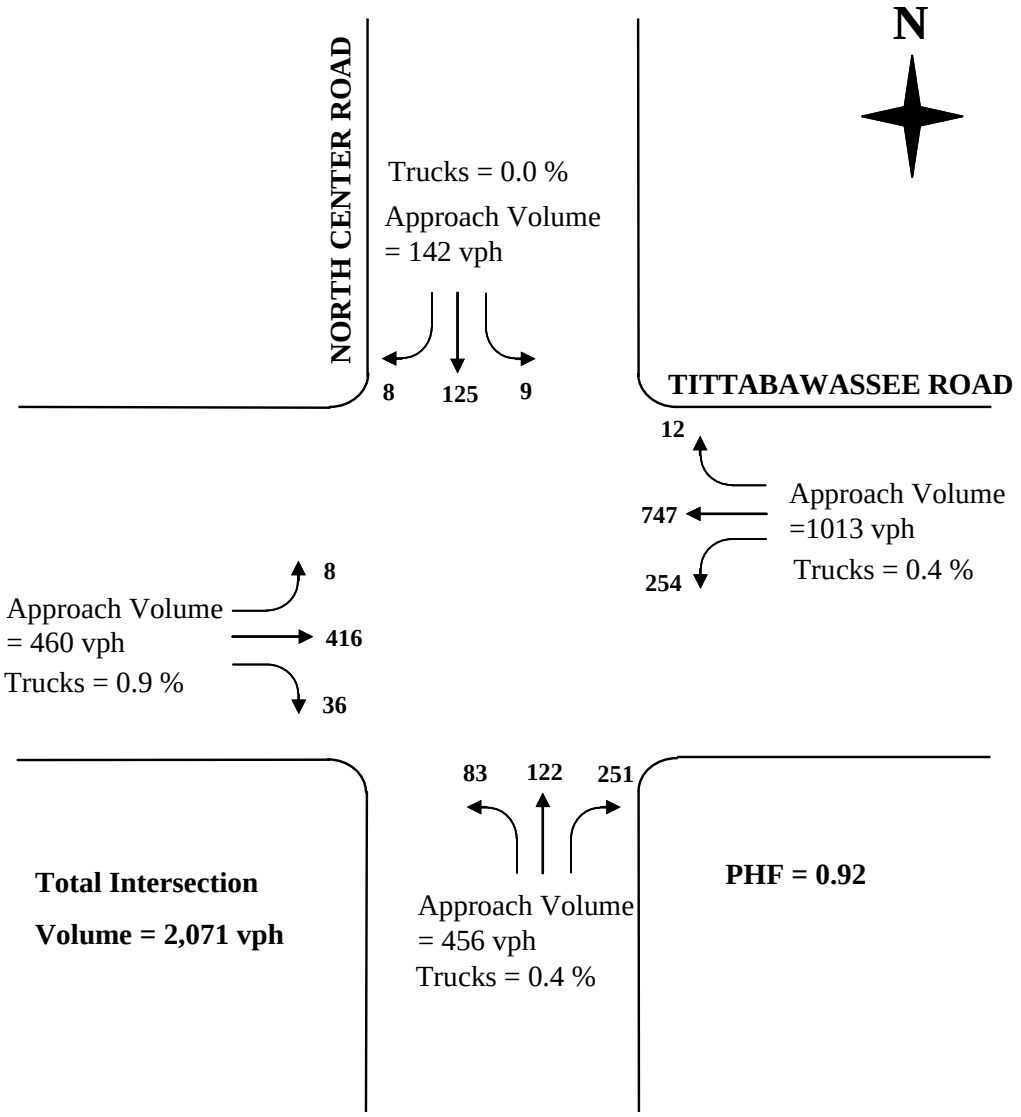


Figure 6.15.2. PM Peak Hour (4:45–5:45 PM) Traffic Volumes for the Intersection of Tittabawassee Road and North Center Road

Traffic Control

This intersection is controlled by a two-phase span-wire traffic signal with an 80-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for both east-west and north-south phases. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.15.1 (on page 230).

Crash Analysis

A total of 30 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 15 crashes. Collision diagrams were prepared based on the UD-10 report's information to identify crash patterns within an approximate radius of 150 feet around the intersection. The collision diagrams for the years 2003 and 2004 for the intersection of Tittabawassee Road and North Center Road are shown in Figures 6.15.3 and 6.15.4 respectively.

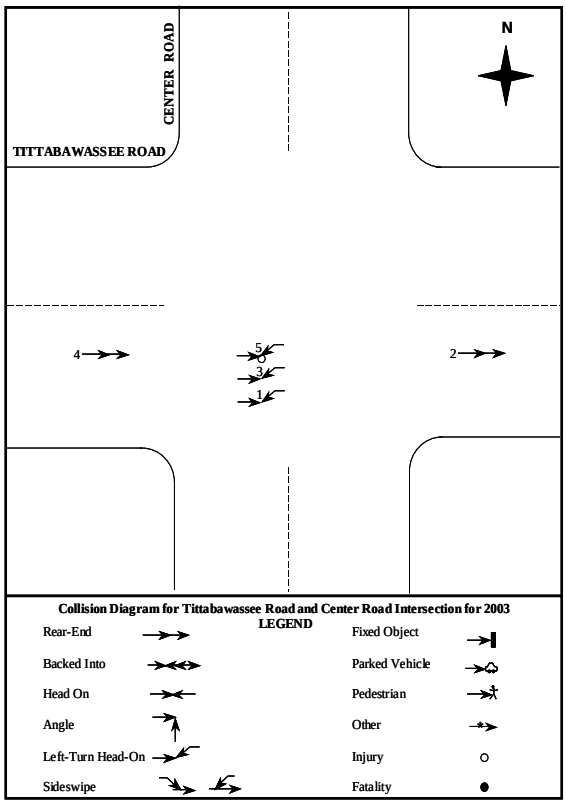


Figure 6.15.3 Collision Diagram for Tittabawassee Road and North Center Road for 2003

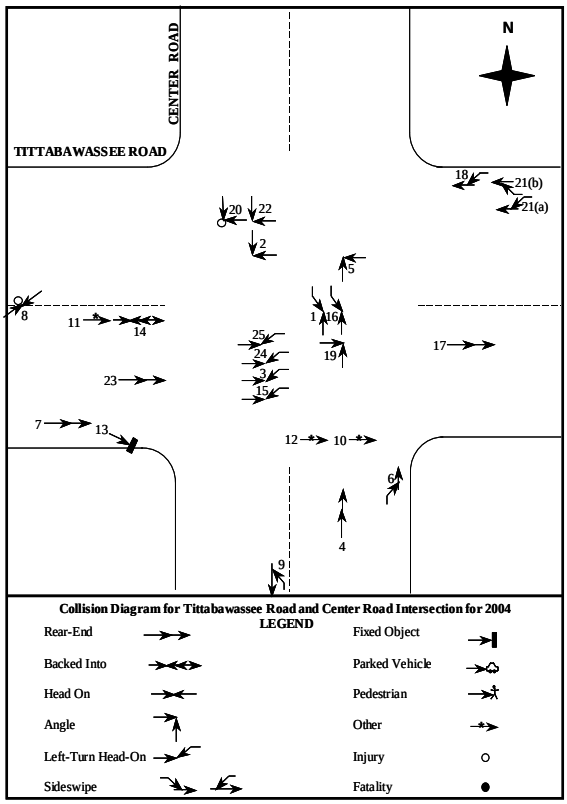


Figure 6.15.4 Collision Diagram for Tittabawassee Road and North Center Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.59 per million entering vehicles, which is considerable since the approach volume at the intersection is very low. Most of the crashes are left-turn head-on crashes, and are possibly due to permissive left-turn movements.

The crash distribution, based upon type of crash, is shown in Figure 6.15.5. Left-turn head-on, rear-end, angle and sideswipe same crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequencies for these crashes include nine left-turn head-on, six rear-end, five angle and three sideswipe same crashes.

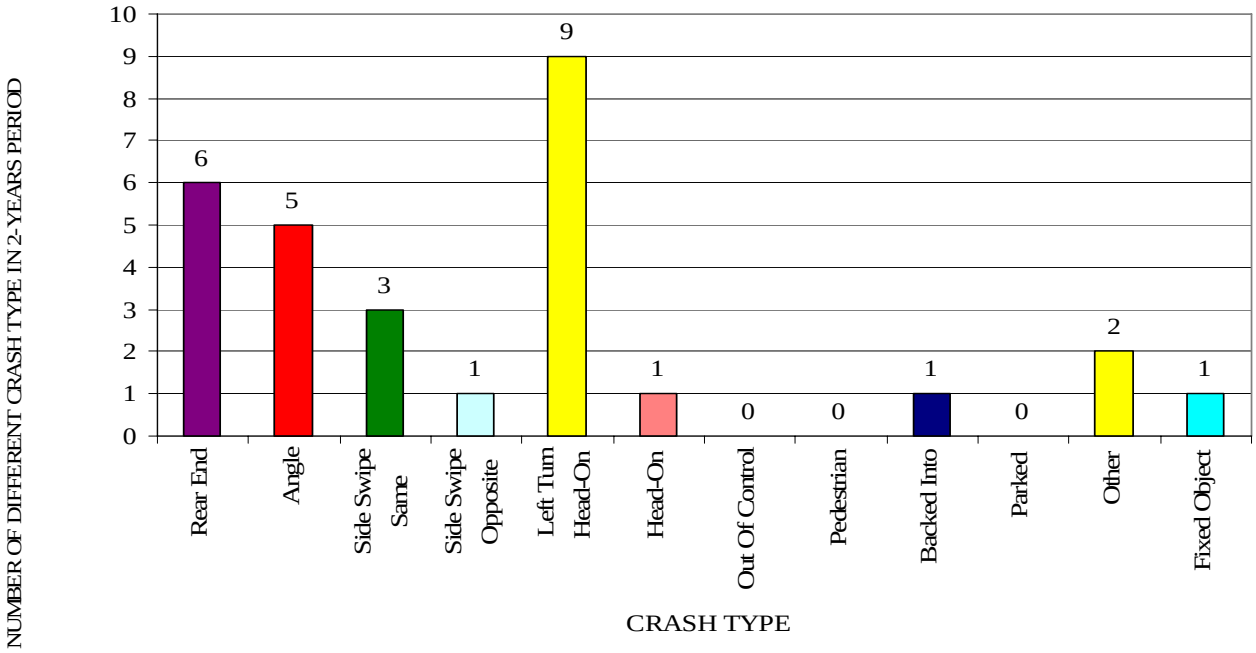


Figure 6.15.5 Two Year (2003-2004) crash experience at Tittabawassee Road and North Center Road Intersection

Safety Deficiencies and Recommended Improvements

Frequently occurring left turn head on, angle and rear end crashes at the intersection of Tittabawassee Road and North Center Road could be attributed to improper signal timing, phasing design and clearance interval inadequacies. The required clearance interval was computed and is provided in the proposed signal-timing plan. The left turn volume is very high at the westbound approach but there is no separate left turn phase. The existing signal-timing plan was modified to include a split phase for westbound Tittabawassee Road and a changed cycle length of 90 seconds to allow progression.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation.

The stop bar should be relocated on all approaches as per the Michigan Manual of Uniform Traffic Control Devices guidelines. The pavement markings should be upgraded with proper directional arrows on the all approaches.

The existing yellow and all-red intervals are shown in Table 6.15.1, along with the proposed lengths of these intervals and their potential benefits.

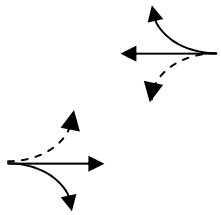
Table 6.15.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and North Center Road Intersection

Interval/Phase	**Existing Interval Length (sec)**		Proposed Interval Length (sec)		Potential Benefit
	Yellow	**All-Red**	**Yellow**	**All-Red**	
E-W Through Phase	4.0	1.0	4.3	1.5	Less rear end, angle, left-turn head-on and injury crashes
N-S Through Phase	4.0	1.0	4.3	1.5	
WB Split Phase	-	-	4.3	1.5	

The proposed PM peak hour signal timing plan, provided with adequate clearance intervals, is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Tittabawassee

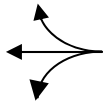
G = 34.2 sec

Y = 4.3 sec

*R = 50.0 sec

AR = 1.5 sec

* Excludes All Red



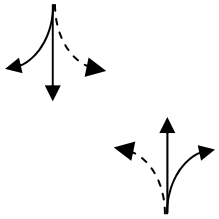
WB Tittabawassee

G = 10.2 sec

Y = 4.3 sec

*R = 74.0 sec

AR = 1.5 sec



N-S N.Center

G = 28.2sec

Y = 4.3 sec

*R = 56.0 sec

AR = 1.5 sec

A list of the recommended improvements is summarized below:

- Increase the yellow interval for the Tittabawassee Road and North Center Road to 4.3 seconds.
- Increase the all-red interval for the Tittabawassee Road and North Center Road to 1.5 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the pavement markings on all approaches with proper directional arrows.
- Provide a modified three-phase signal plan.
- Relocate the stop bar on the all approaches, as per MMUTCD guidelines.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for AM peak and off-peak hours.
- Implement the optimized PM signal timing.

The proposed intersection diagram is provided in Figure 6.15.6, which shows the recommended improvements including the installation of a mast-arm signal, an upgrade of pavement markings, the repositioning of the stop bar on all approaches and the proposed signal timing plan.

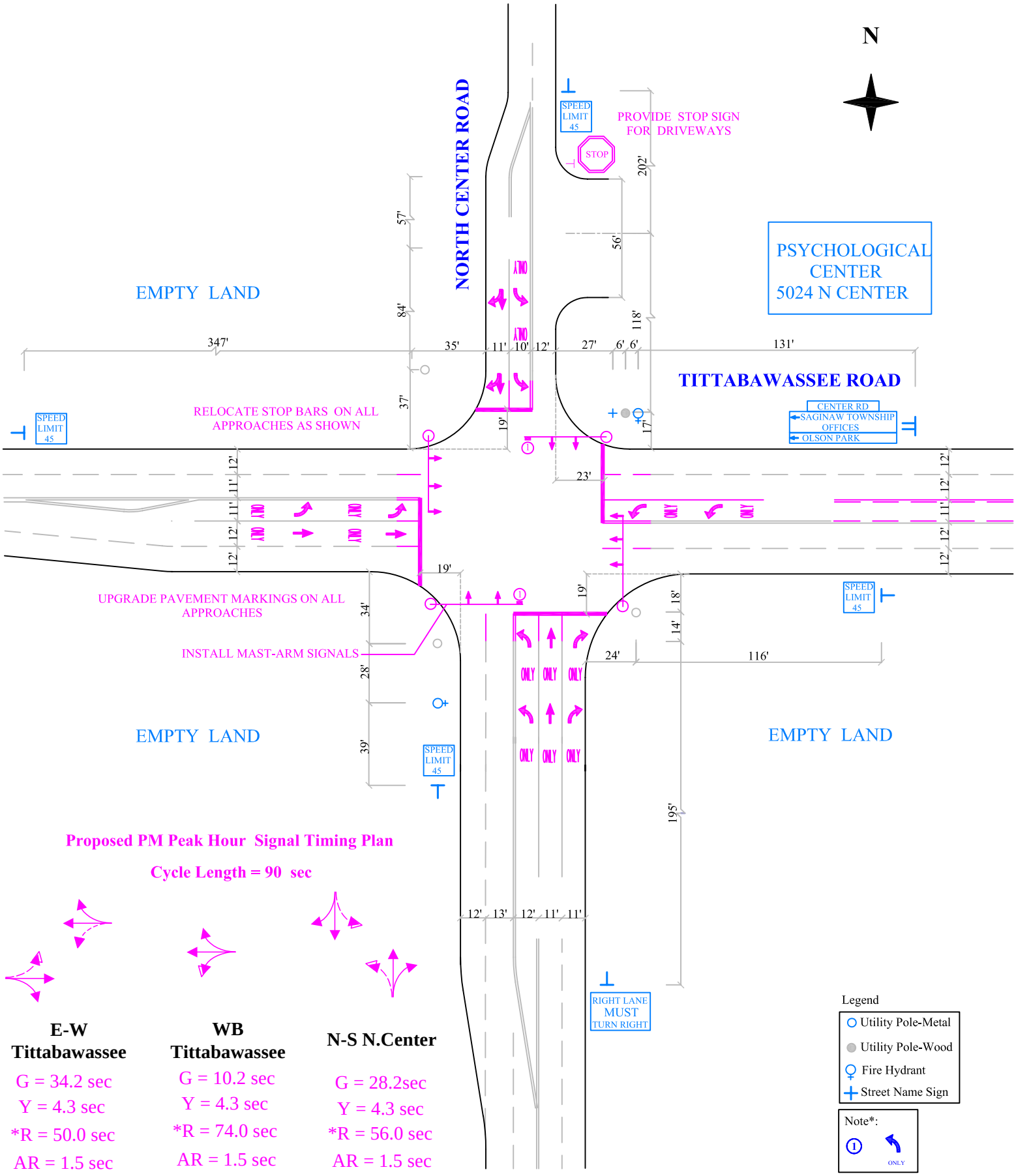
Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and North Center Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 11.5 seconds per vehicle.

With the modification to the signal timings and noted improvements, the overall intersection level of service will remain at B with a delay of 14.8 seconds per vehicle. The overall delay at the intersection will increase due to the inclusion of a split phase and the provision of adequate yellow and all red interval. However, the safety at the intersection will be improved along with the traffic progression. The levels of service results for the existing conditions and with the proposed improvements are summarized in table 6.15.2.

Table 6.15.2.
Tittabawassee Road and North Center Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	6.7	A	19.2	B
WB Tittabawassee Road	9.1	A	8.0	A
NB North Center Road	20.4	C	24.0	C
SB North Center Road	19.0	B	22.3	C
Overall Intersection	**11.5**	**B**	**14.8**	**B**



Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 sec

E-W Tittabawassee	WB Tittabawassee	N-S N.Center
G = 34.2 sec	G = 10.2 sec	G = 28.2sec
Y = 4.3 sec	Y = 4.3 sec	Y = 4.3 sec
*R = 50.0 sec	*R = 74.0 sec	*R = 56.0 sec
AR = 1.5 sec	AR = 1.5 sec	AR = 1.5 sec

* Excludes All Red

Figure 6.15.6 Proposed Diagram of North Center Road and Tittabawassee Road Intersection (Not to Scale)

16. Tittabawassee Road and Mackinaw Road

The intersection of Tittabawassee Road and Mackinaw Road is located within the northern central boundary of Saginaw Township. Tittabawassee Road runs in the east-west direction and Mackinaw Road runs in the north-south direction. Tittabawassee Road is a major arterial street and Mackinaw Road on the north of the intersection, acts as a collector while south of it acts as an arterial.

Tittabawassee Road is 55 feet wide and has one dedicated left-turn lane, one through lane and one shared through and right turn lane for westbound traffic. Similarly, the eastbound approach is 56 feet wide with the same laneage as the westbound approach. Mackinaw Road is 34 feet wide with one exclusive left-turn lane, one through lane and one shared through and right-turn lane for the southbound movement. Similarly the northbound approach is 36 feet wide with one exclusive left turn lane, one through lane and one dedicated right turn lane.

The posted speed limit for Tittabawassee Road is 45 miles per hour for both approaches. Mackinaw Road has a speed limit of 50 miles per hour on north of the intersection and 40 miles per hour for the south of the intersection.

Land Use

The intersection of Tittabawassee Road and Mackinaw Road is occupied by a Citgo Gas station in the northeast corner. There is a covenant Health Care in the southwest corner and empty land in the northwest and southeast corners. There are two driveways close to the intersection which serve the Citgo Gas Station.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and Mackinaw Road.

The existing characteristics of the intersection including traffic control and surrounding land uses are schematically shown in Figure 6.16.1. Details of the intersection approaches are shown in photographs 16a through 16d. The photographs show the existing conditions at each approach with the span-wire signal installation and pavement markings.

Photograph 16a. NB MackinawRoad at Tittabawassee Road

Photograph 16b. SB Mackinaw Road at Tittabawassee Road

Photograph 16c. EB Tittabawassee Road at Mackinaw Rd

Photograph 16d. WB Tittabawassee Road at Mackinaw Road

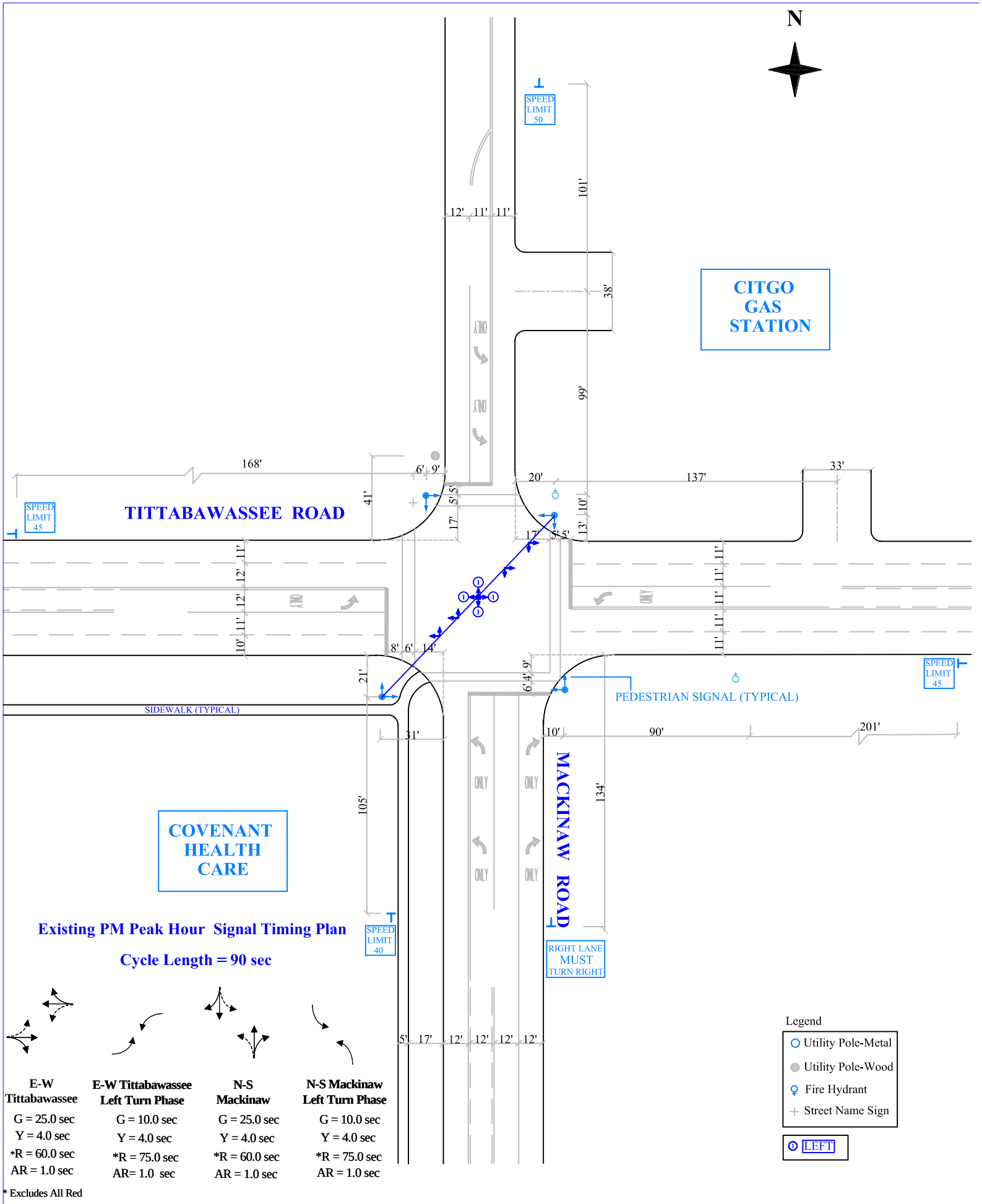


Figure 6.16.1 Existing Condition Diagram of Tittabawassee Road and Mackinaw Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on a clear day on March 28, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at suitable locations around the intersection and covering all the approaches. The peak hourly volume from 4:30 to 5:30 PM is presented in Figure 6.16.2 and represents the highest peak hour (one-hour) volumes in the PM peak period. The traffic patterns indicate that the volume at the westbound approach is almost double than the volume at any other approach.

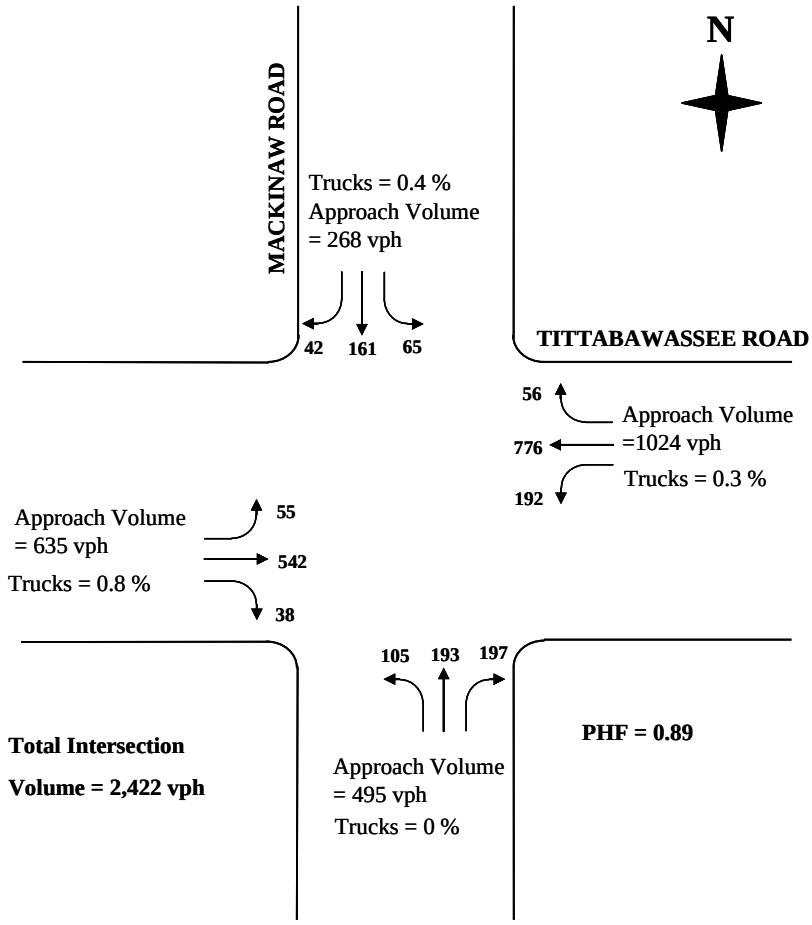


Figure 6.16.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes for the Intersection of Tittabawassee Road and Mackinaw Road

Traffic Control

This intersection is controlled by a four-phase span-wire traffic signal with 90 second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for all east-west and north-south phases. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.16.1 (page 240).

Crash Analysis

A total of 62 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 31 crashes. Collision diagrams were prepared based on the UD-10 reports, information to identify crash patterns within an approximate radius of 150 feet around the intersection. Figures 6.16.3 and 6.16.4 show the collision diagrams for the years 2003 and 2004 respectively.

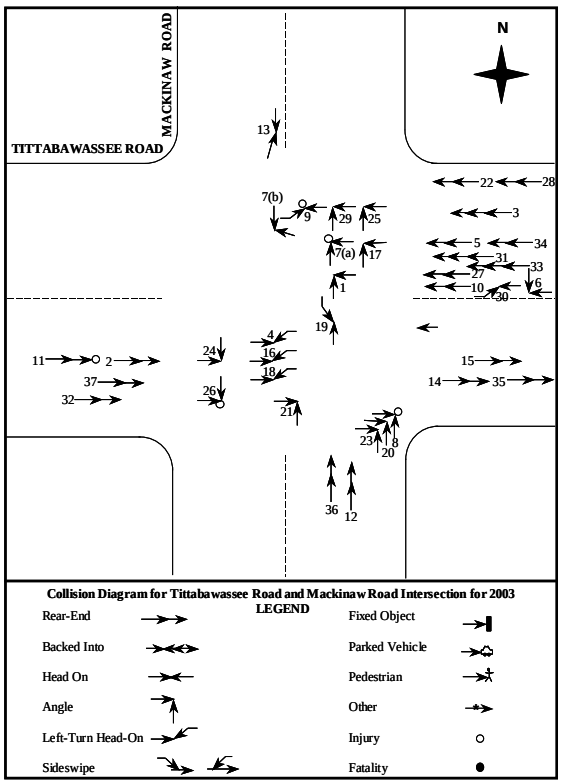


Figure 6.16.3. Collision Diagram for Tittabawassee Road and Mackinaw Road for 2003

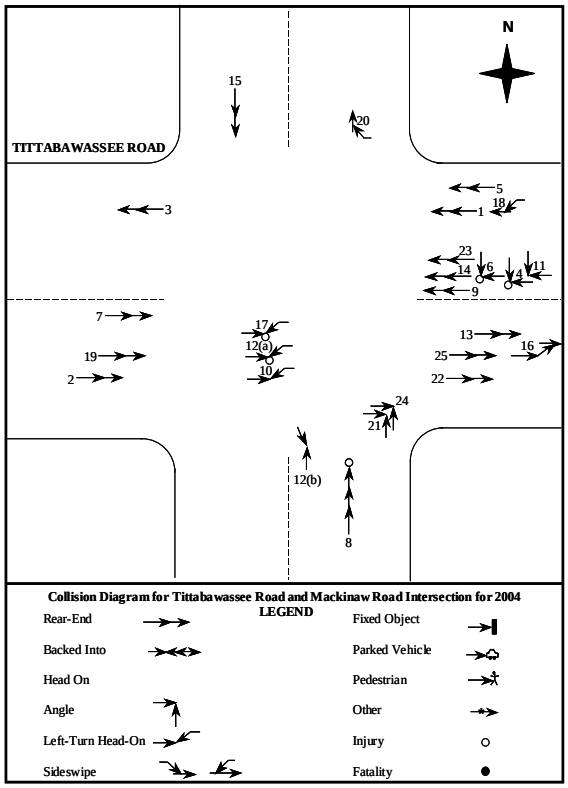


Figure 6.16.4. Collision Diagram for Tittabawassee Road and Mackinaw Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The crash rate is 2.8 per million entering vehicles, which is high, considering the approach volume at the intersection. Rear end, angle and left-turn head- on crashes are of specific concern as they are found in significant numbers across all the approaches. Also, there are two sideswipe same crashes and one sideswipe opposite near the northeast corner of the intersection. The significant number of crashes may be due to inadequate signal timings and insufficient clearance

intervals. The interference of heavy oncoming through traffic with the left turn movements and a high speed limit might be the cause of left- turn head-on crashes.

The crash distribution, based upon type of crash, is shown in Figure 6.15.5. Rear-end, angle and left-turn head-on crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes includes 33 rear end, 17 angle and nine left-turn head-on crashes.

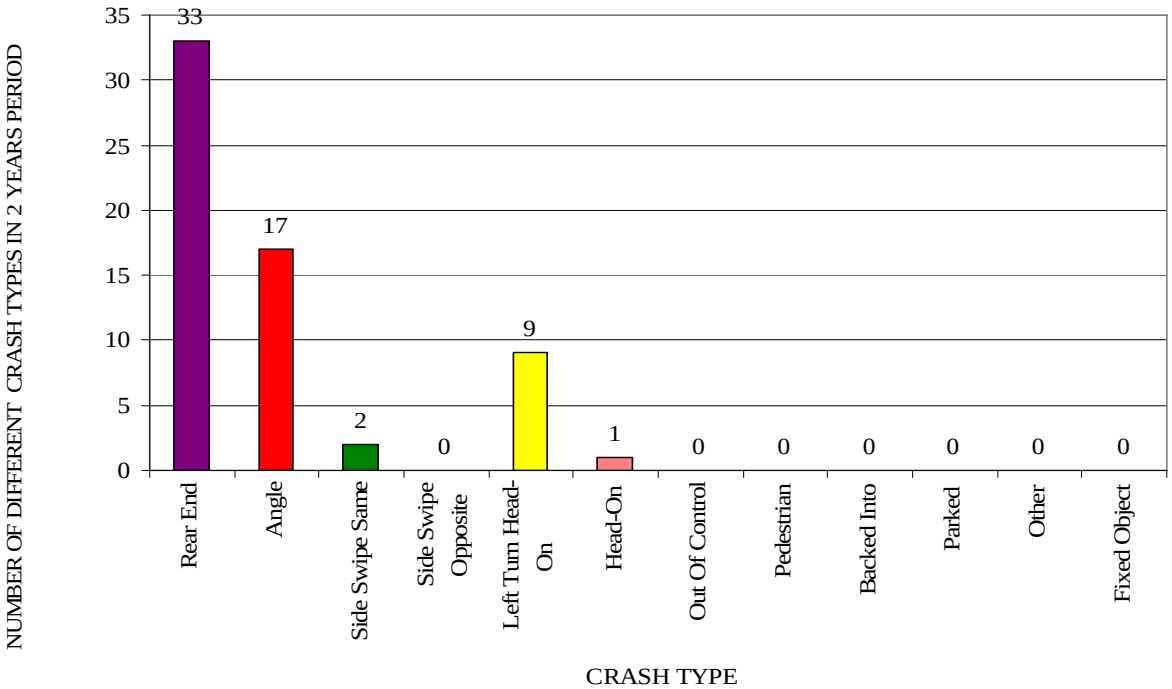


Figure 6.16.5 Two year (2003 & 2004) Crash Experience at Tittabawassee Road and Mackinaw Road Intersection

Safety Deficiencies and Recommended Improvements

Frequently occurring rear end, left- turn head- on and angle crashes at the intersection of Tittabawassee Road and Mackinaw Road could be attributed to several possible reasons including poor visibility of traffic signals and inadequate clearance intervals. The required clearance interval was computed and is provided in the proposed signal-timing plan.

When analyzing the existing volumes at the intersection, the westbound volume is higher than the eastbound movement. The existing signal-timing plan was modified and includes

a split phase for westbound Tittabawassee Road and a changed cycle length of 90 second to allow progression.

Due to the excessive distance between the crosswalks and stop bars, some drivers may find themselves in the 'dilemma zone' in the intersection for an extended period of time. The Michigan Manual of Uniform Traffic Control Devices recommends that stop bars at intersections be placed four feet in advance of the nearest crosswalk line. On the southbound approach of Mackinaw Road, this distance is 17 feet and on the eastbound approach of Tittabawassee Road it is 14 feet. It appears that the stop bar for these approaches could be moved closer to the crosswalk. Pavement markings should be upgraded with proper directional arrows on the southbound, eastbound and westbound approaches.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.16.6.

The existing yellow and all-red intervals are shown in Table 6.16.1, along with the proposed lengths of these intervals and their potential benefits.

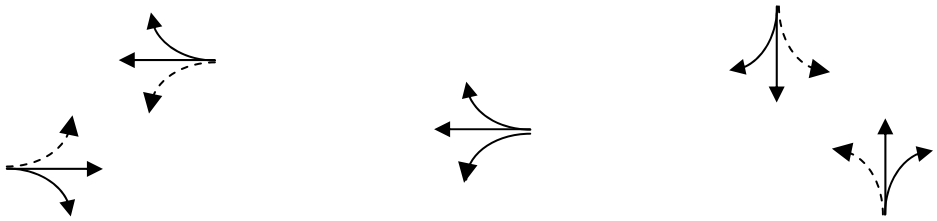
Table 6.16.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and Mackinaw Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.0	1.0	4.3	1.3	Less rear end, angle, left-turn head-on and injury crashes
E-W Left-Turn Phase	4.0	1.0	-	-	
N-S Through Phase	4.0	1.0	4.7	1.6	
N-S Left-Turn Phase	4.0	1.0	-	-	
WB Only Phase	-	-	4.3	1.3	

The proposed PM peak period signal timing plan based on a 90 second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Tittabawassee	**WB Tittabawassee**	**N-S Mackinaw**
G = 38.4 sec	G = 8.4 sec	G = 25.7 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 4.7sec
*R = 46.0 sec	*R = 76.0 sec	*R = 58.0 sec
AR = 1.3 sec	AR= 1.3 sec	AR = 1.6 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Increase the yellow interval for the Tittabawassee Road to 4.3 seconds and for the Mackinaw Road to 4.7 seconds.
- Increase the all-red interval for the Tittabawassee Road to 1.3 seconds and the Mackinaw Road to 1.6 seconds.
- Eliminate the exclusive left-turn phase for the north-south and east-west movement.
- Provide a modified three-phase signal system.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the existing pavement markings with proper directional arrows on all the approaches.
- Relocate the stop bars and cross walks on the southbound and eastbound approaches, as per MMUTCD guidelines.
- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.

- Optimize the signal timings for AM peak and off-peak hours.
- Implement the optimized PM peak hour signal timing.

The proposed intersection diagram in Figure 6.16.6 shows the improvements including a mast-arm signal installation, upgrading the pavement markings and the proposed signal timing plan.

Operational Evaluation

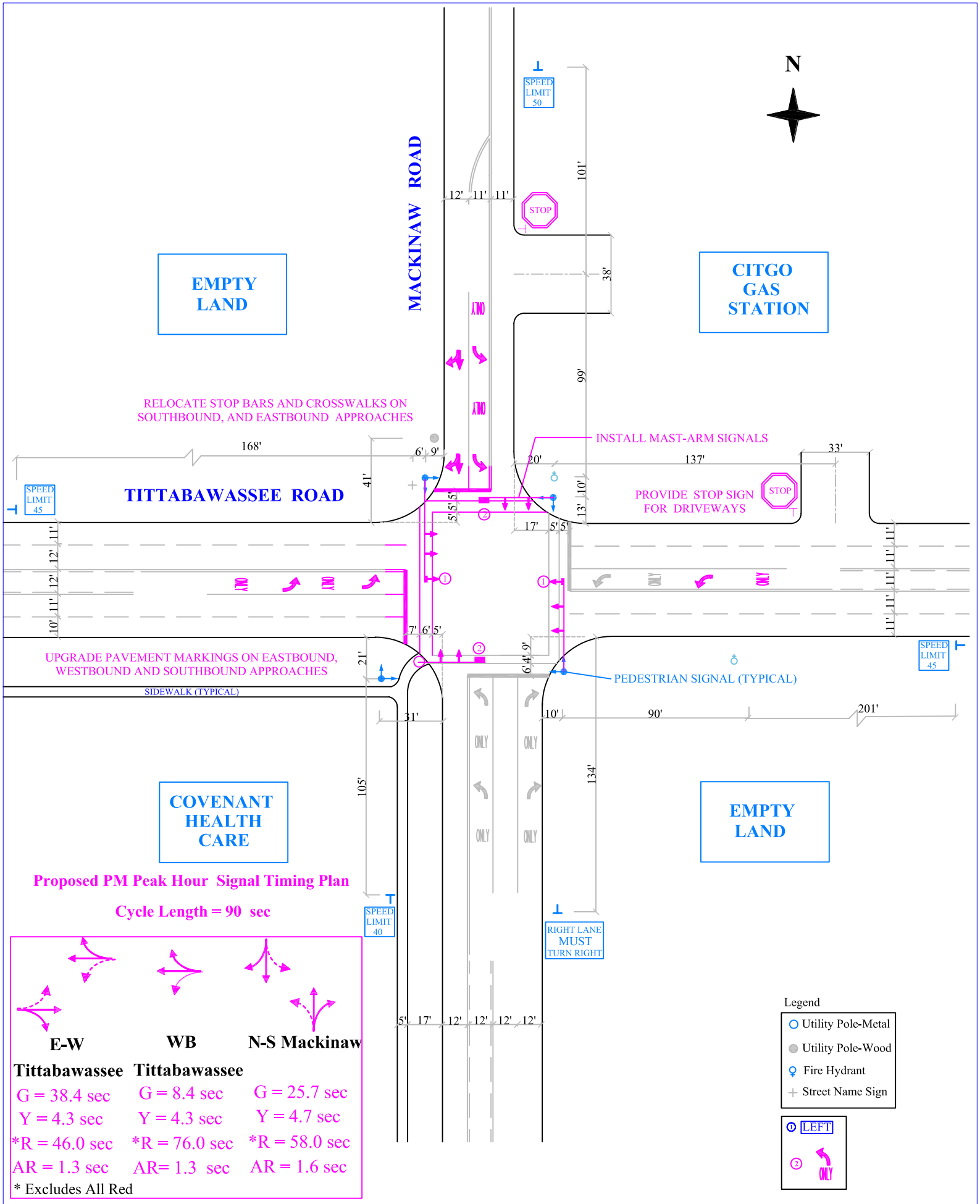
The results of the level of service analysis for the intersection of Tittabawassee Road and Mackinaw Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is C with a delay of 34.0 seconds per vehicle.

With the modification to the signal timings, phasing and the noted improvements, the overall intersection level of service will reduce to B with a delay of 12.7 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic along with improved progression. The calculated clearance interval is provided for greater safety.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.16.2.

Table 6.16.2.
Tittabawassee Road and Mackinaw Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	30.5	C	14.9	B
WB Tittabawassee Road	41.8	D	2.0	A
NB Mackinaw Road	26.2	C	26.3	C
SB Mackinaw Road	25.5	C	25.6	C
Overall Intersection	**34.0**	**C**	**12.7**	**B**



Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 sec

E-W Tittabawassee	WB Tittabawassee	N-S Mackinaw
G = 38.4 sec	G = 8.4 sec	G = 25.7 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 4.7 sec
*R = 46.0 sec	*R = 76.0 sec	*R = 58.0 sec
AR = 1.3 sec	AR= 1.3 sec	AR = 1.6 sec

* Excludes All Red

Figure 6.16.6 Proposed Diagram of Tittabawassee Road and Mackinaw Road Intersection (Not to Scale)

17. Tittabawassee Road and Fortune Boulevard

The intersection of Tittabawassee Road and Fortune Boulevard is located in the northeastern part of Saginaw Township. Tittabawassee Road runs in the east-west direction and Fortune Boulevard runs in the north-south direction. The north leg of the intersection is called Bueker Drive. Tittabawassee Road functions as a principal arterial while Fortune Boulevard functions as a local road.

At the westbound approach, Tittabawassee Road consists of one dedicated left-turn lane, one through lane, and one shared through and right turn lane. At the eastbound approach, Tittabawassee Road consists of one exclusive left-turn lane, two through lanes and a dedicated right-turn lane. Tittabawassee Road is approximately 56 feet wide on the east leg and 85 feet wide on the west leg of the intersection.

At the north and southbound approaches, Fortune Boulevard consists of one exclusive left-turn lane and a shared through/right-turn lane. Fortune Boulevard is approximately 38 feet wide on the north leg and 36 feet wide on the south leg of the intersection.

The speed limit on Tittabawassee Road is 45 miles per hour, while on Fortune Boulevard; it is 25 miles per hour.

Land Use

The intersection of Tittabawassee Road and Fortune Boulevard is surrounded by commercial establishments on all of its corners. Cash State, Speedy Sew and Allstate Insurance occupy the northeast corner. Two driveways are connected to these establishments at the westbound approach. Meijer occupies the northwest corner of the intersection. There is one driveway within 150 feet of the intersection at the southbound approach to reach this establishment. Burlington Coat Factory occupies the southeast corner and FedEx Kinko's, Baycom Nextel and a shopping center is on the southwest corner. There are no driveways very close to intersection to reach the southeast and southwest establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and Fortune Boulevard.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.17.1. Details of the intersection approaches are shown in photographs 17a through 17d. The photographs show the span-wire signal installation and existing pavement markings.

Photograph 17a. NB Fortune Boulevard at Tittabawassee Road

Photograph 17b. SB Bueker Drive at Tittabawassee Road

Photograph 17c. EB Tittabawassee Road at Fortune Boulevard

Photograph 17d. WB Tittabawassee Road at Fortune Boulevard

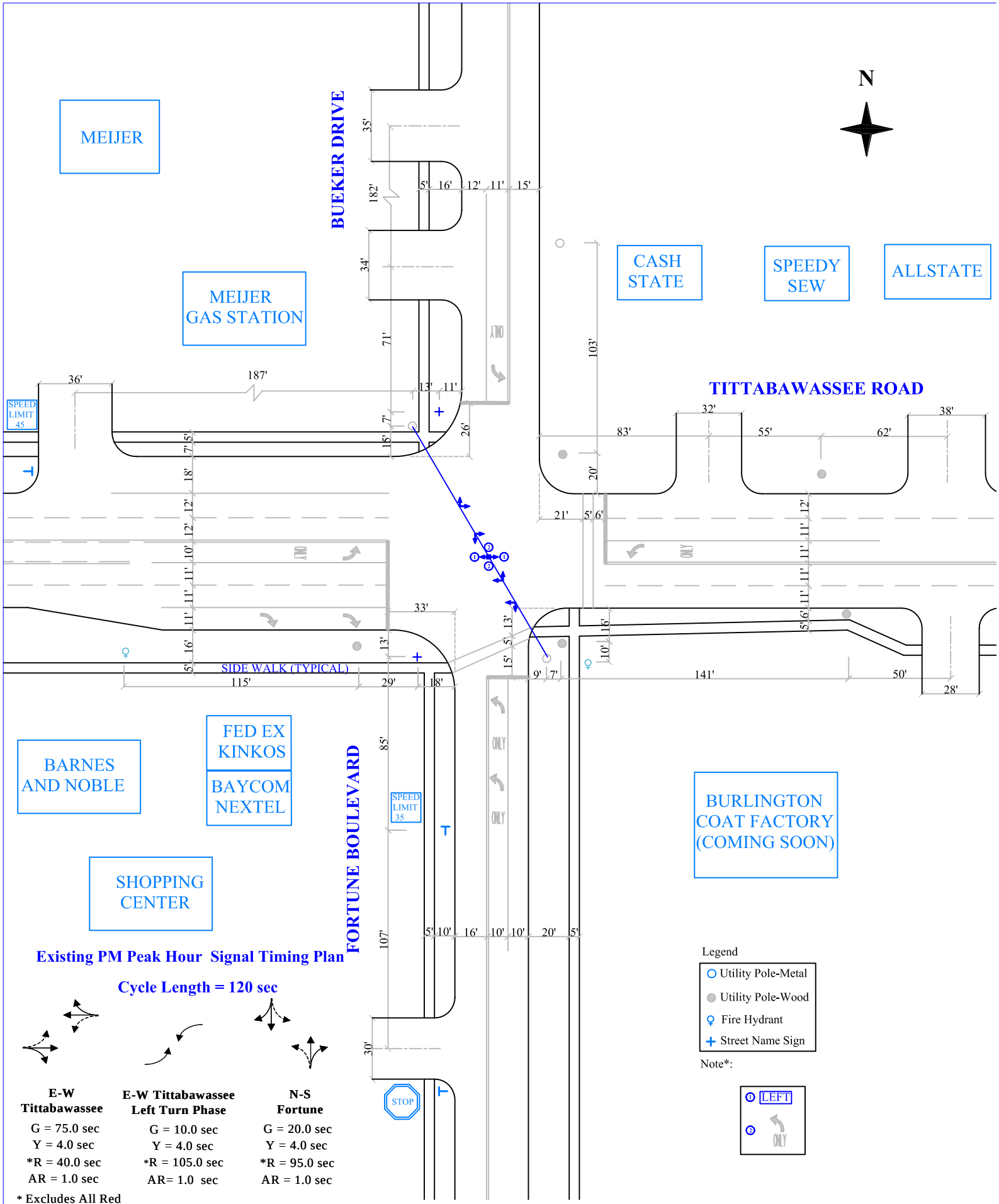


Figure 6.17.1 Existing Condition Diagram of Tittabawassee Road and Fortune Boulevard Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on March 28, 2006 from 4 PM to 6 PM by two people standing at a suitable corner of the intersection and covering all of the approaches. The peak hourly volume from 4:15 PM to 5:15 PM is presented in Figure 6.17.2 and represents the highest peak hour (one-hour) volumes in the PM peak period. The traffic patterns indicate that the volumes in the east-west direction are relatively higher than the north-south direction. Also, the westbound and southbound left-turn volumes are considerably high.

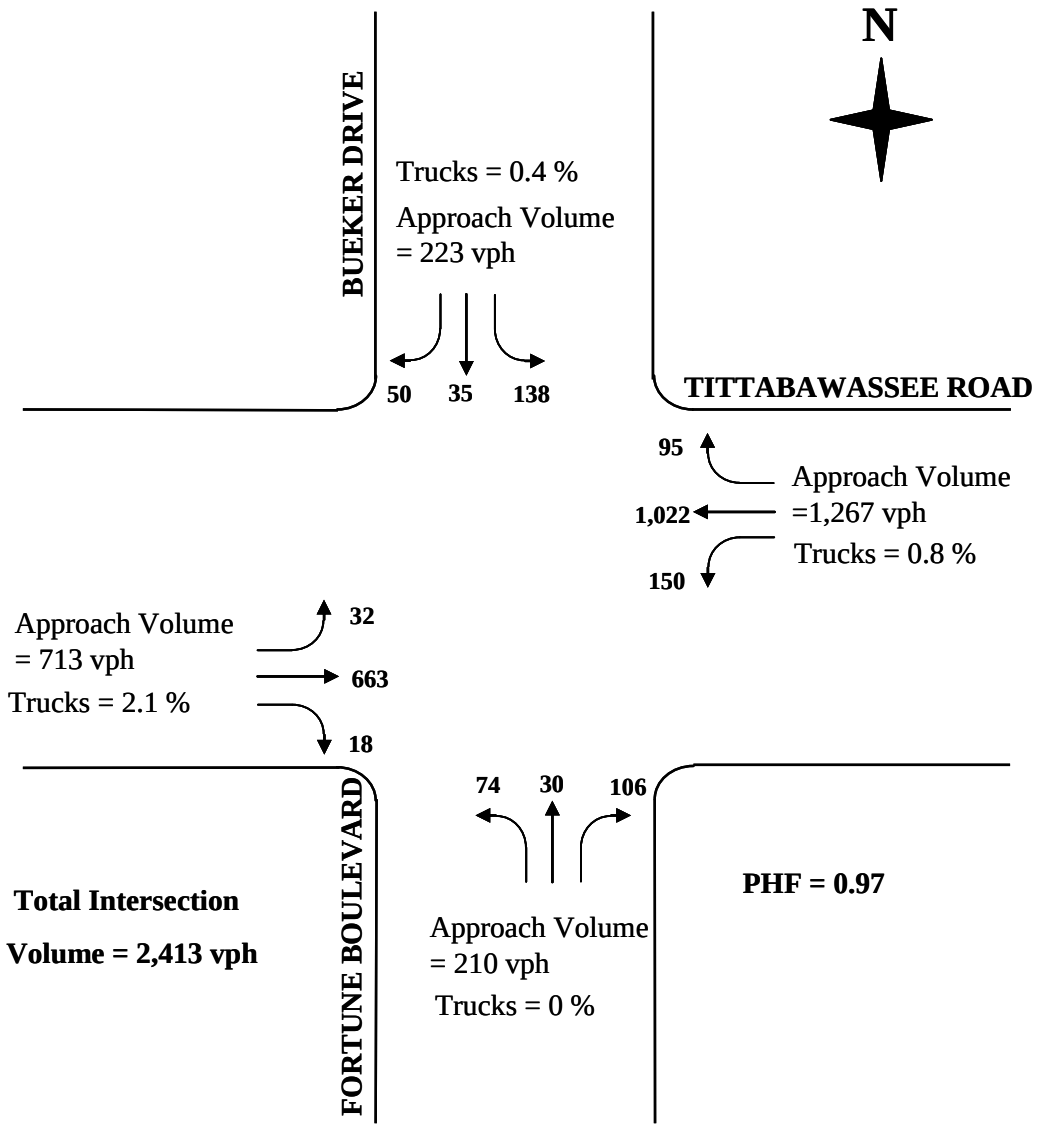


Figure 6.17.2. Existing PM Peak Hour (4:15–5:15 PM) Traffic Volumes at Tittabawassee Road and Fortune Boulevard

Traffic Control

This intersection is controlled by a three-phase span-wire traffic signal with a 120-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for each phase. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.17.1 (on page 250).

Crash Analysis

A total of 22 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 11 crashes. Collision diagrams were prepared based on the UD-10 reports' information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Tittabawassee Road and Fortune Boulevard are shown on Figures 6.17.3 and 6.17.4 respectively.

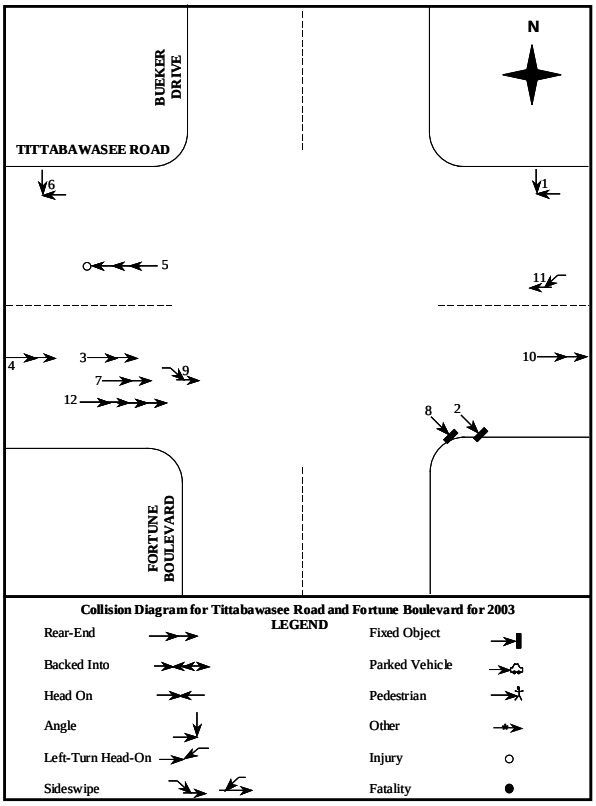


Figure 6.17.3 Collision Diagram for Tittabawassee Road and Fortune Boulevard for 2003

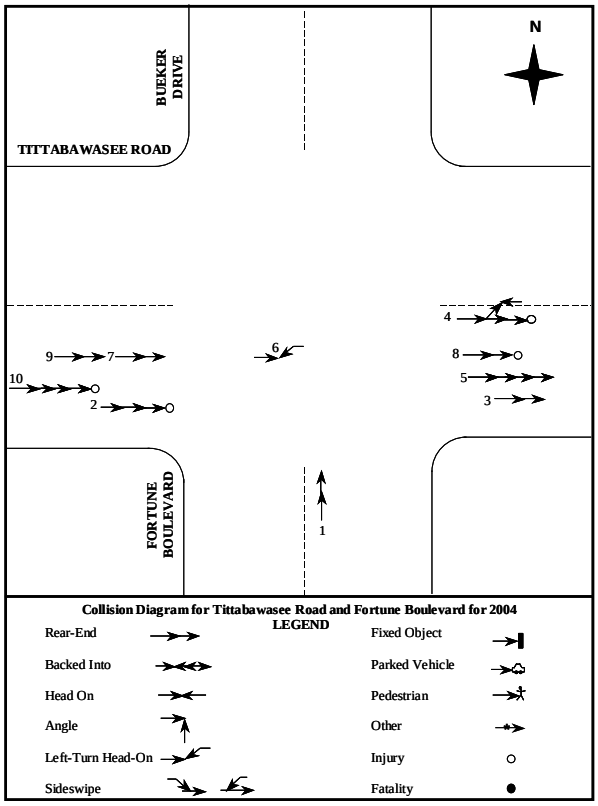


Figure 6.17.4 Collision Diagram for Tittabawassee Road and Fortune Boulevard for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.00 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The majority of the crashes are of the rear-end type and occur at the east and westbound approaches. These are possibly caused by some major driveway activities near the intersection, inadequate signal timing or insufficient clearance interval.

The crash distribution, based upon type of crash, is shown in Figure 6.17.5. Rear-end crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for this crash type includes 15 rear-end crashes.

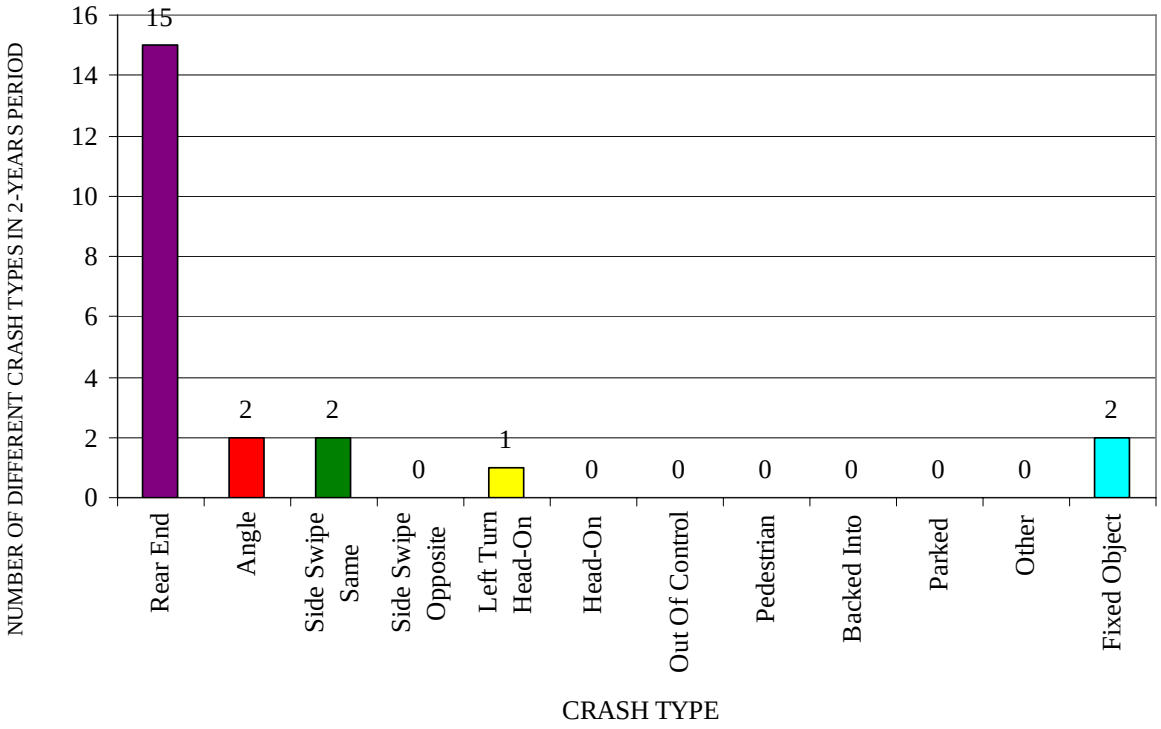


Figure 6.17.5. Two year (2003 & 2004) Crash Experience at Tittabawassee Road and Fortune Boulevard Intersection

Safety Deficiencies and Recommended Improvements

The majority of the crashes occurring at the intersection of Tittabawassee Road and Fortune Boulevard are of the rear-end type and occur due to several possible reasons including restricted or poor visibility of traffic signals and inadequate clearance intervals.

The required clearance interval was computed and is provided in the proposed signal-timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram is shown in Figure 6.17.6.

The cycle length of the proposed signal-timing plan has been changed to 90 seconds to be compatible with the progression along the Tittabawassee Road Corridor.

The stop bars should be relocated at each approach. The crosswalks should be relocated at the southbound, eastbound and westbound approaches. Pavement markings should be upgraded with proper directional arrows on the all approaches.

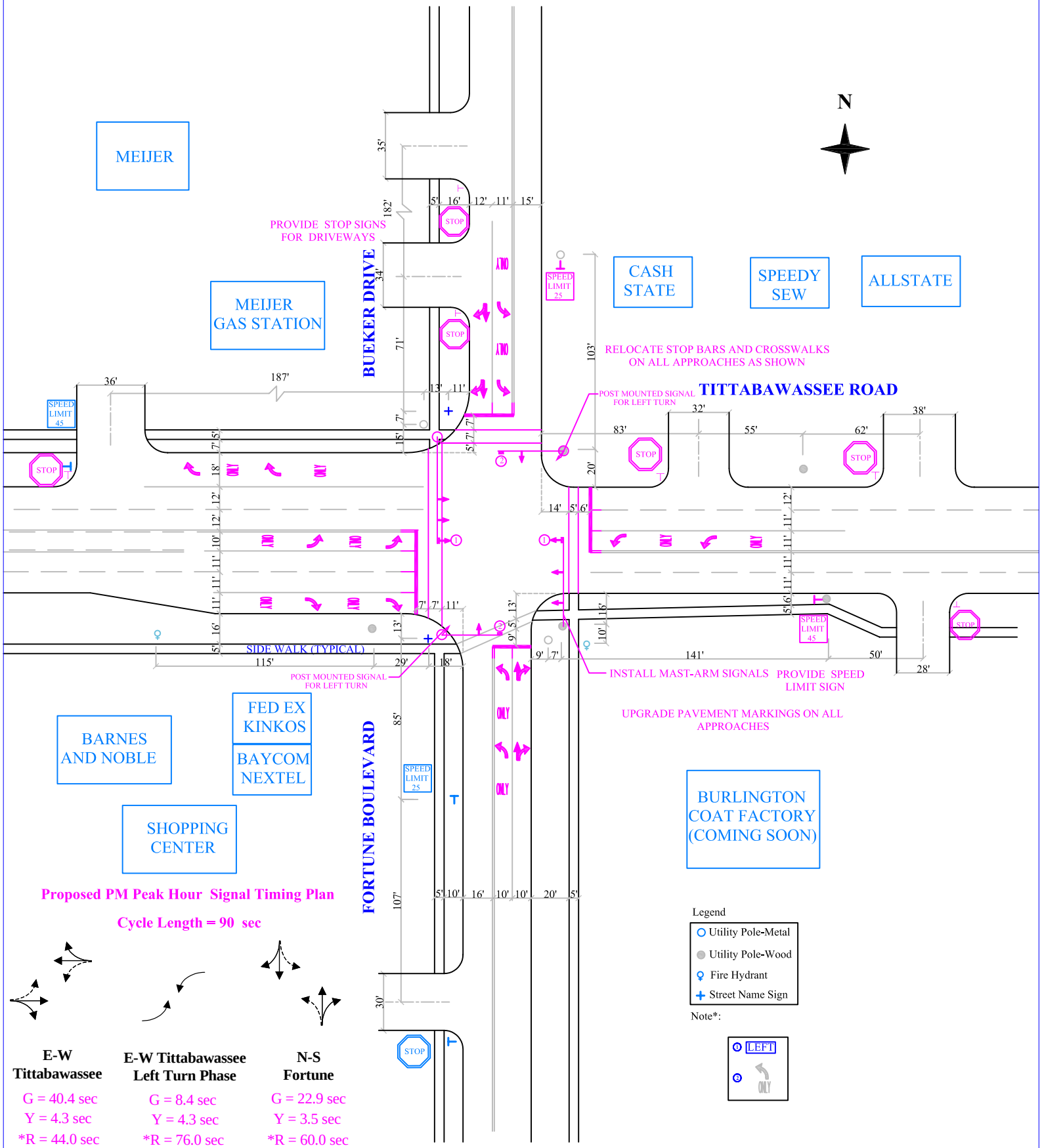
Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Post-mounted traffic signals with "LEFT" illuminated signs should be provided at the northeast and southwest corners of the intersection to assist north and southbound left turning vehicles.

The existing yellow and all-red intervals are shown in Table 6.17.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.17.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and Fortune Boulevard Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.0	1.0	4.3	1.3	Less rear-end and injury crashes
E-W Left-Turn Phase	4.0	1.0	4.3	1.3	
N-S Through Phase	4.0	1.0	3.5	3.6	



Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 sec

E-W Tittabawassee	E-W Tittabawassee Left Turn Phase	N-S Fortune
G = 40.4 sec	G = 8.4 sec	G = 22.9 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 3.5 sec
*R = 44.0 sec	*R = 76.0 sec	*R = 60.0 sec
AR = 1.3 sec	AR= 1.3 sec	AR = 3.6 sec

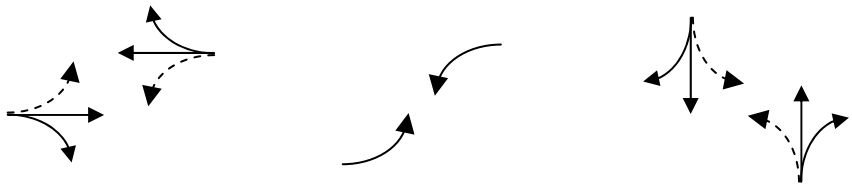
* Excludes All Red

Figure 6.17.6 Proposed Diagram of Tittabawassee Road and Fortune Boulevard Intersection (Not to Scale)

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Tittabawassee	E-W Tittabawassee Left Turn Phase	N-S Fortune
G = 40.4 sec	G = 8.4 sec	G = 22.9 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 3.5 sec
*R = 44.0 sec	*R = 76.0 sec	*R = 60.0 sec
AR = 1.3 sec	AR= 1.3 sec	AR = 3.6 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Increase the yellow interval for Tittabawassee Road to 4.3 seconds and reduce it to 3.5 seconds for Fortune Boulevard.
- Increase the all red interval for Tittabawassee Road to 1.3 seconds and for Fortune Boulevard to 3.6 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the existing pavement markings with proper directional arrows for the all approaches.
- Relocate the stop bar on the all approaches and crosswalk on the southbound, eastbound and westbound approaches, as per MMUTCD guidelines.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for north and southbound left-turns.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Provide stop sign for all the driveways located near the intersection. Landowner should provide this stop sign.
- Optimize the signal timings for A.M. and off-peak hours.
- Implement the optimized PM signal timing.

The proposed intersection diagram is shown in Figure 6.17.6 (on page 255) and shows the proposed improvements which include the installation of a mast-arm support for the signals, a provision for post-mounted signals for left-turns, an upgrade of the pavement markings on all approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and Fortune Boulevard indicate that under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 14.2 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will remain at B with a reduced delay of 12.8 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic along with improved progression.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.17.2.

Table 6.17.2.
Tittabawassee Road and Fortune Boulevard Intersection
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	5.1	A	13.2	B
WB Tittabawassee Road	6.0	A	7.7	A
NB Fortune Boulevard	48.6	D	26.8	C
SB Bueker Drive	62.3	E	28.4	C
Overall Intersection	**14.2**	**B**	**12.8**	**B**

18. Tittabawassee Road and Fashion Square Boulevard

The intersection of Tittabawassee Road and Fashion Square Boulevard is located in the northeastern part of Saginaw Township. Tittabawassee Road runs in the east-west direction and Fashion Square Boulevard runs in the north-south direction. Tittabawassee road functions as a principal arterial street while Fashion Square Boulevard functions as a collector street.

At the east and westbound approaches, Tittabawassee Road consists of an exclusive left-turn lane, one through lane and one shared through and right-turn lane. Tittabawassee Road is approximately 60 feet wide.

At the north and southbound approaches, Fashion Square Boulevard consists of an exclusive left-turn lane, one through lane and one shared through and right-turn lane. Fashion Square Boulevard is approximately 49 feet wide on the south leg and 48 feet wide on the north leg of the intersection.

The speed limit on Tittabawassee Road is 45 miles per hour and on Fashion Square Boulevard it is 35 miles per hour.

Land Use

The intersection of Tittabawassee Road and Fashion Square Boulevard has Toys'R'Us on the northwest corner, Good Year on the northeast corner, TGIF on the southwest corner and Mattress Connection on the southeast corner. Fashion Square Mall is located very close to the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and Fashion Square Boulevard.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.18.1. Details of the intersection approaches are shown in photographs 18a through 18d. North and southbound approaches are shown by photographs 18a and 18b with the signal heads hanging from the span wire installation. All approaches of the intersection as shown in photographs 18b through 18d have faded pavement markings.

Photograph 18a. NB Fashion Square Blvd at Tittabawassee Road

Photograph 18b. SB Fashion Square Blvd at Tittabawassee Road

Photograph 18c. EB Tittabawassee Road at Fashion Square Boulevard

Photograph 18d. WB Tittabawassee Road at Fashion Square Boulevard.

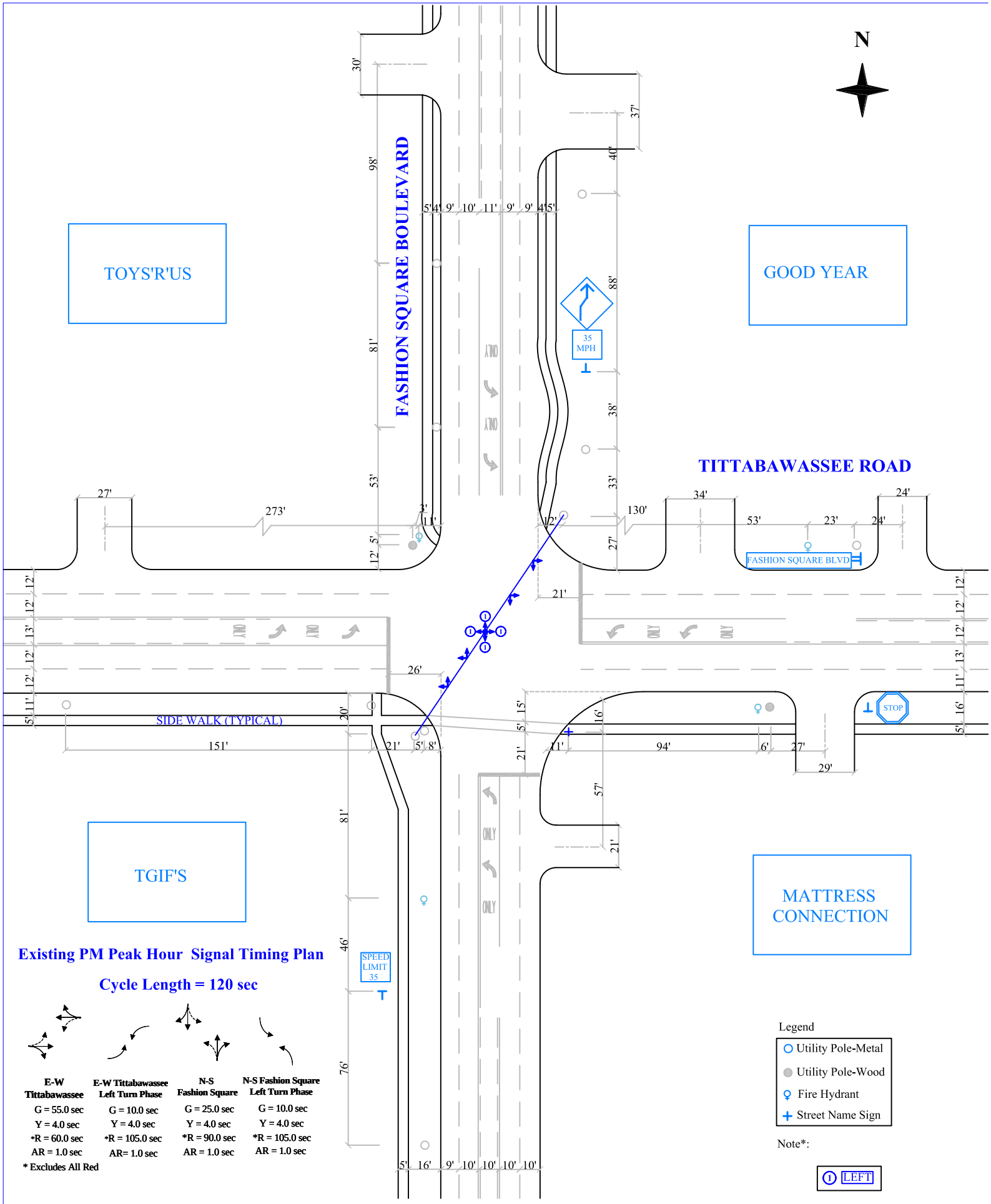


Figure 6.18.1 Existing Condition Diagram of Tittabawassee Road and Fashion Square Boulevard Intersection
(Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were manually collected on a clear day on March 28, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering all the approaches. The peak hourly volume from 4:30 PM to 5:30 PM is presented in Figure 6.18.2, which indicates the highest peak hour (one-hour) volumes in the PM peak period.

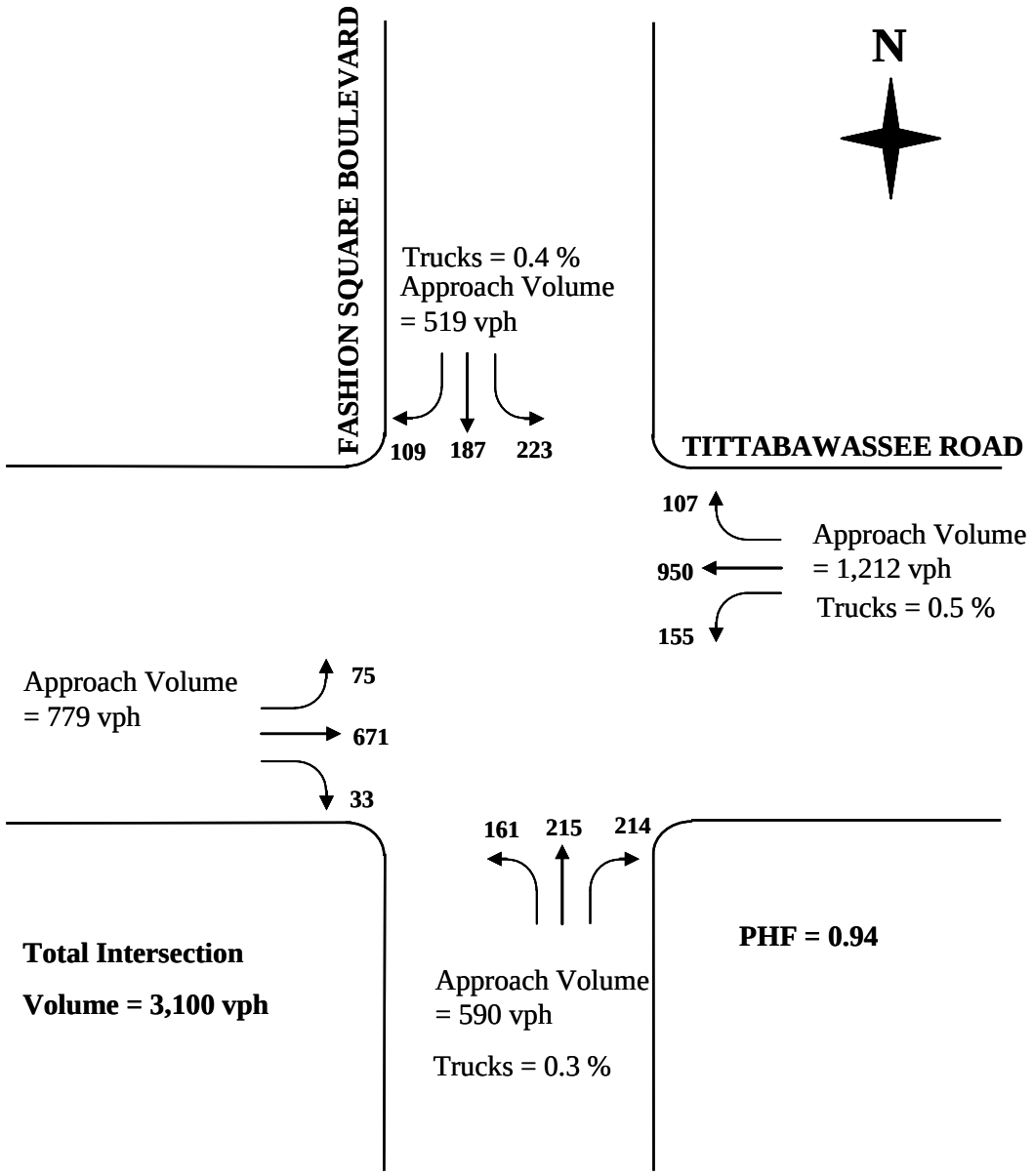


Figure 6.18.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes at Tittabawassee Road and Fashion Square Boulevard

Traffic Control

This intersection is controlled by a four-phase span-wire traffic signal with 120-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for all east-west and north-south phases.

Crash Analysis

A total of 30 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 15 crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. Figures 6.18.3 and 6.18.4 show the collision diagrams for these years respectively.

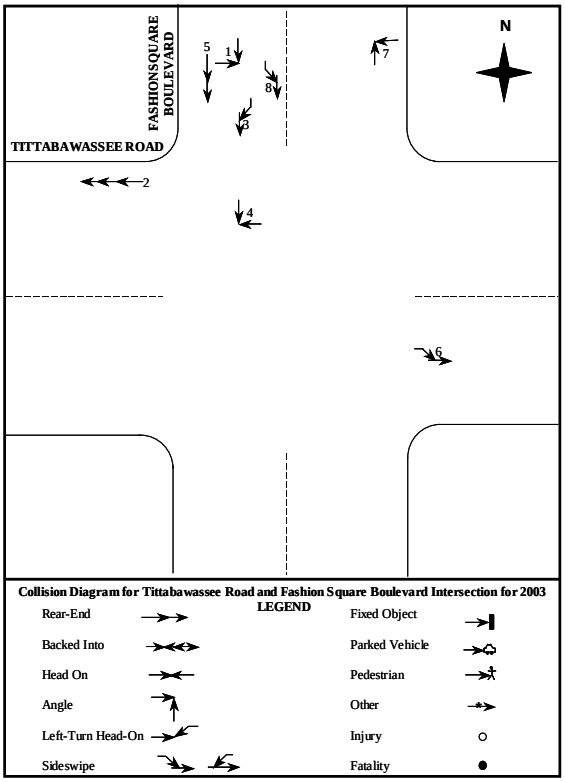


Figure 6.18.3. Collision Diagram for Tittabawassee Road and Fashion Square Boulevard for 2003

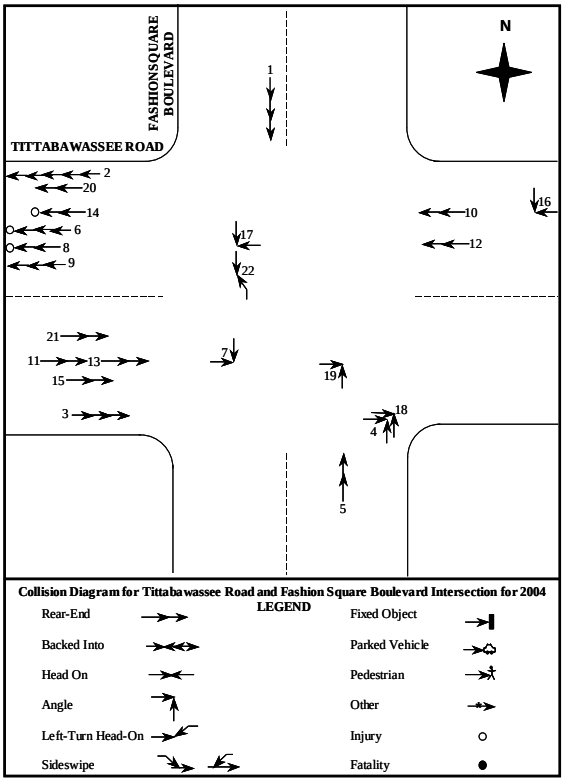


Figure 6.18.4. Collision Diagram for Tittabawassee Road and Fashion Square Boulevard for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The crash rate is 1.06 per million entering

vehicles, which is nominal, considering the approach volume at the intersection. Rear-end crashes at the intersection are of specific significance especially in east-west direction, which may have occurred due to inadequate signal timing, insufficient clearance interval and driveway activities. Angle crashes are also quite significant especially on the southeast and northwest side of the intersection, which may have occurred due to improper clearance interval. Also, there are two sideswipe crashes at the southbound approach and one at the eastbound approach of the intersection.

The crash distribution based upon type of crashes is shown in Figure 6.18.5. Rear-end, angle and side swipe same are the predominant type of crashes reported at the intersection. The corresponding two-year crash frequency includes 17 rear-end crashes, nine angle crashes and three sideswipe same crashes.

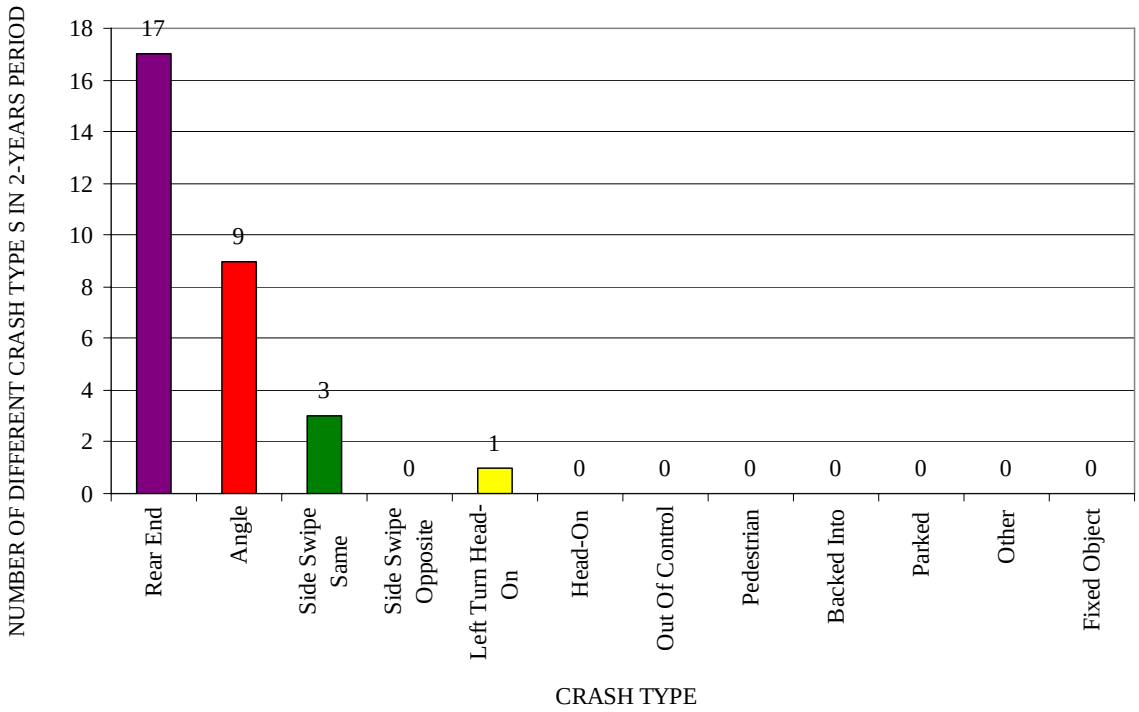


Figure 6.18.5: Two year (2003 & 2004) Crash Experience at Tittabawassee Road and Fashion Square Boulevard Intersection

Safety Deficiencies and Recommended Improvements

The majority of crashes occurring at the intersection of Tittabawassee Road and Fashion Square Boulevard are rear-end and angle crashes. These crashes may be occurring due to

several possible reasons including restricted or poor visibility of traffic signals and inadequate clearance intervals. The required clearance intervals are computed and provided in the proposed signal-timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.18.6.

The crosswalks should be relocated on all approaches and the stop bars should be relocated on the northbound, southbound and eastbound approaches. Pavement markings should be upgraded with proper directional arrows on all approaches.

The cycle length of the proposed signal-timing plan has been changed to 90 seconds to be compatible with the progression along the Tittabawassee Road Corridor.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Post-mounted traffic signals with “LEFT” illuminated signs should be provided at the all corners of the intersection to assist left-turning vehicles.

The existing yellow and all-red intervals are shown in Table 6.18.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.18.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and Fashion Square Boulevard Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.0	1.0	4.3	1.4	Less rear-end, angle and injury crashes
E-W Left Turn Phase	4.0	1.0	4.3	1.4	
N-S Through Phase	4.0	1.0	3.6	2.0	
N-S Left Turn Phase	4.0	1.0	3.6	2.0	

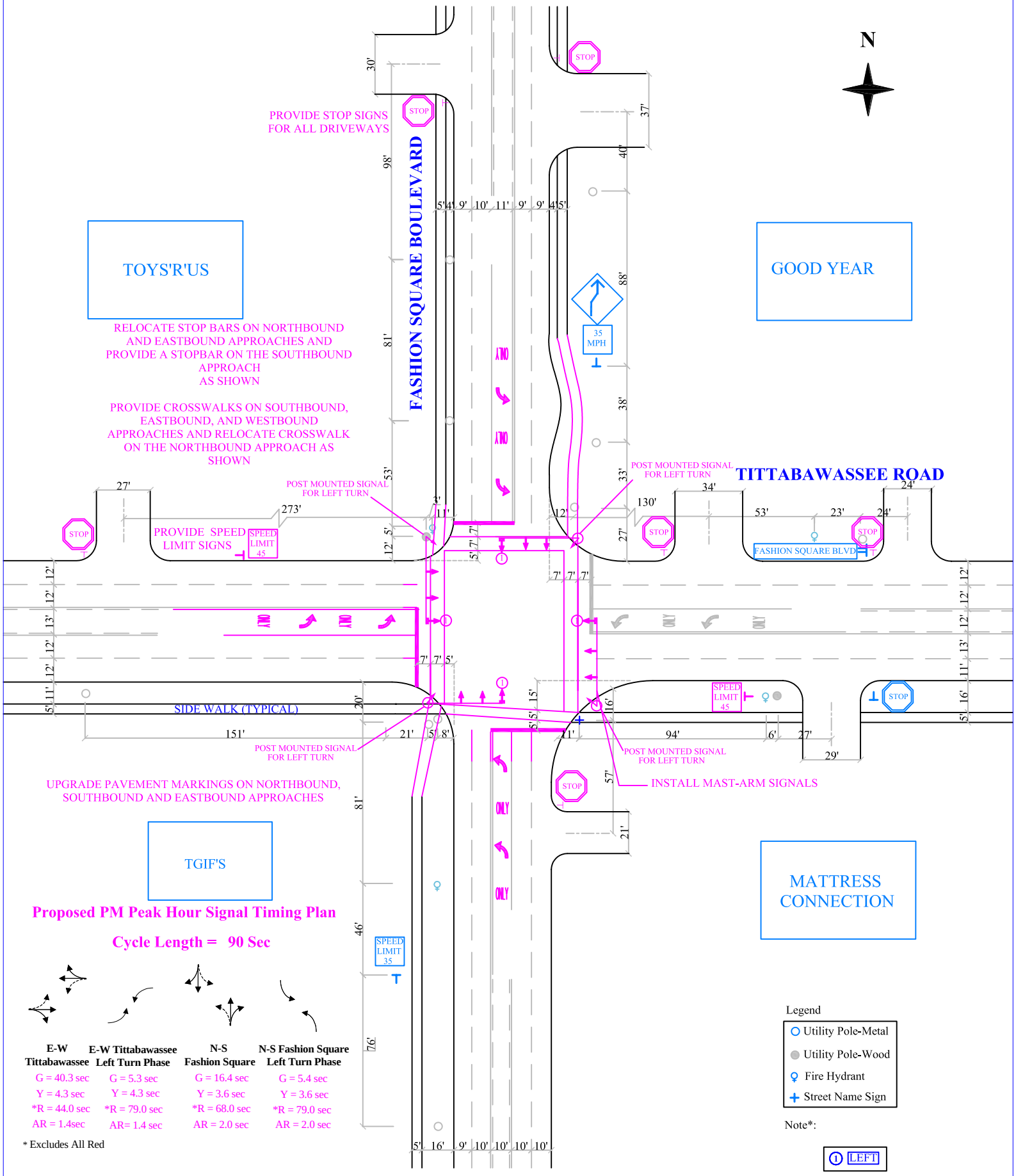
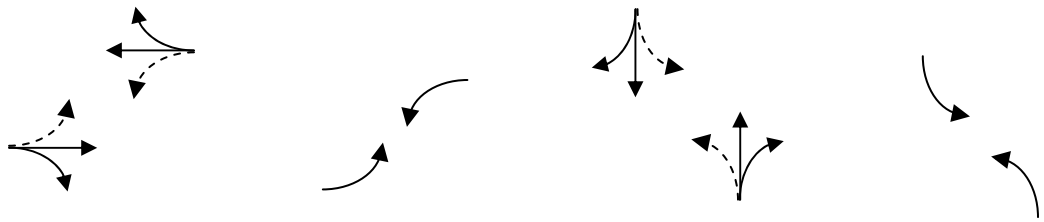


Figure 6.18.6 Proposed Diagram of Tittabawassee Road and Fashion Square Boulevard Intersection (Not to Scale)

The proposed PM peak period signal timing plan based on a 90 second cycle length is shown as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Tittabawassee	E-W Tittabawassee Left Turn Phase	N-S Fashion Square	N-S Fashion Square
G = 40.3	G = 5.3	G = 16.4	G = 5.4
Y = 4.3	Y = 4.3	Y = 3.6	Y = 3.6
*R = 44.0	*R = 79.0	*R = 68.0	*R = 79.0
AR = 1.4sec	AR= 1.4 sec	AR = 2.0 sec	AR = 2.0 sec

* Excludes All

A list of the recommended improvements is summarized below:

- Increase the yellow interval for Tittabawassee Road to 4.3 seconds and reduce it to 3.6 seconds for Fashion Square Boulevard.
- Increase the all-red interval for Tittabawassee Road to 1.4 seconds and for the Fashion Square Boulevard to 2.0 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the existing pavement markings with proper directional arrows for all the approaches.
- Relocate the crosswalks on all the approaches and stop bars for the northbound, southbound and eastbound approaches, as per MMUTCD guidelines.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the all corners of the intersection for left-turns.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all driveways located near the intersection. Landowners should provide these stop signs.

- Optimize the signal timings for A.M. and off-peak hours.
- Implement the optimized PM signal timing.

The proposed intersection diagram shown in Figure 6.18.6 (on page 265) shows the proposed improvements which include new installation of mast-arm support for the signals, a provision for post-mounted signals for left-turns, upgrading the pavement markings on all approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and Fashion Square Boulevard indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is C with a delay of 28.7 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will remain at C with a delay of 24.1 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic along with improved progression.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.18.2.

Table 6.18.2.
Tittabawassee Road and Fashion Square Boulevard
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	18.2	B	13.7	B
WB Tittabawassee Road	21.2	C	19.8	B
NB Fashion Square Boulevard	44.0	D	35.6	D
SB Fashion Square Boulevard	46.5	D	37.9	D
Overall Intersection	**28.7**	**C**	**24.1**	**C**

19. Tittabawassee Road and Berl Drive

The intersection of Tittabawassee Road and Berl Drive is located in the northeast part of Saginaw Township and forms a "T" intersection. Tittabawassee Road runs in the east-west direction and Berl Drive runs in the north-south direction. Berl Drive functions as a local street while Tittabawassee Road functions as a principal collector.

At the eastbound approach, Tittabawassee Road consists of one through lane, and one shared through and right-turn lane. At the westbound approach, Tittabawassee road consists of two through lanes and one exclusive right-turn lane for traffic going to the Kessel Driveway. There is a 12-foot wide center lane for turning movements. Tittabawassee Road is approximately 61 feet wide on the west leg and 75 feet wide on the east leg of the intersection.

At the northbound approach, Berl Drive is 27 feet wide with one shared left-turn/through/right-turn lane and one receiving lane. Kessel Driveway, north of the intersection, is 62 feet wide.

Land Use

The intersection of Tittabawassee Road and Berl Drive has a Burger King on the southwest corner, Hello Sushi Japanese Cuisine and Feice's Interior on the southeast corner and Art Van Furniture and Clearance Center and Pier 1 Imports on the north side.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and Berl Drive.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.19.1. Details of the intersection approaches are shown in photographs 19a through 19c. The northbound approach of the intersection is shown in photograph 19c where driveways located very close to the intersection are visible. The pavement markings are not clear on all three approaches as shown in photographs.

Photograph 19a. EB Tittabawassee Road at Berl Drive

Photograph 19b. WB Tittabawassee Road at Berl Drive

Photograph 19c. NB Berl Drive at Tittabawassee Road

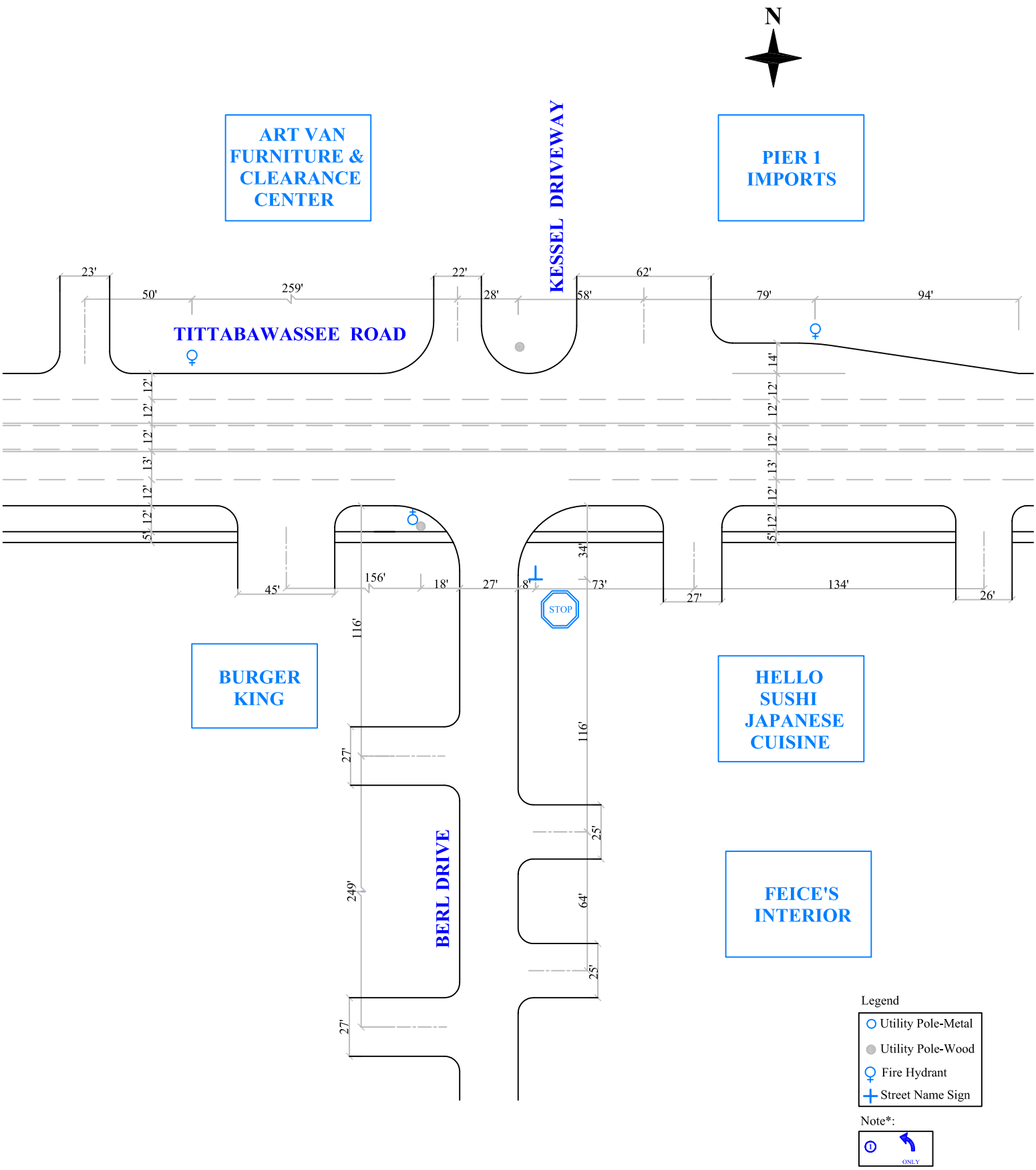


Figure 6.19.1 Existing Condition Diagram of Titabawassee Road and Berl Drive Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were manually collected on a clear day on April 4, 2006 from 4 PM to 6 PM by two people standing at appropriate locations around the intersection and covering all the approaches. The peak hourly volume from 4:30 PM to 5:30 PM is presented in Figure 6.19.2, which represent the highest peak hour (one-hour) volumes in the PM peak period. The traffic patterns show that volumes at the eastbound and westbound approaches are considerably higher than the volumes at the southbound approach.

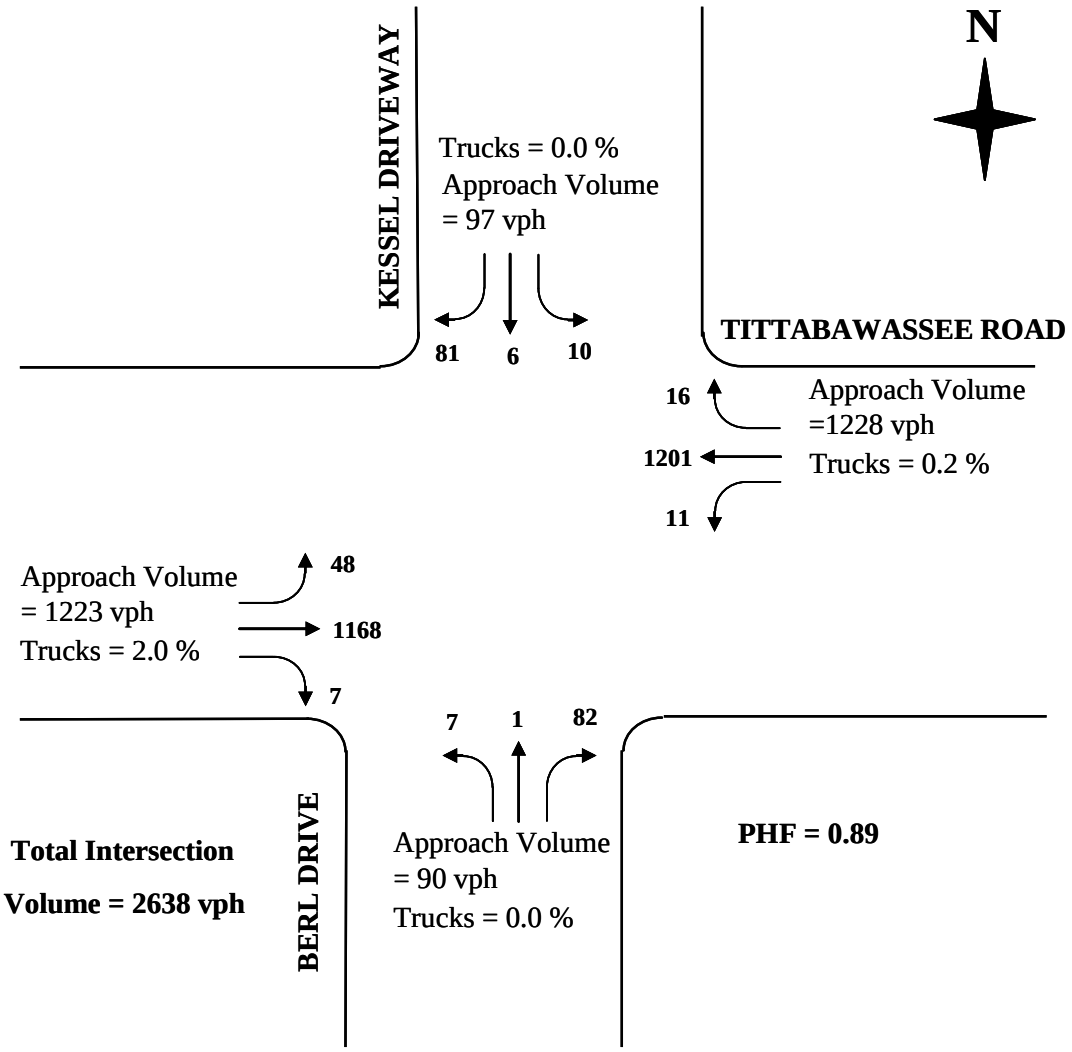


Figure 6.19.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes at Tittabawassee Road and Berl Drive

Traffic Control

Three is no traffic signal at the intersection of Tittabawassee Road and Berl Drive. The intersection is a one-way stop controlled intersection. Tittabawassee Road has the right of the way while northbound Bueker Drive is controlled by stop sign.

Crash Analysis

A total of 27 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 14 crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. Figures 6.19.3 and 6.19.4 show the collision diagrams for these years respectively.

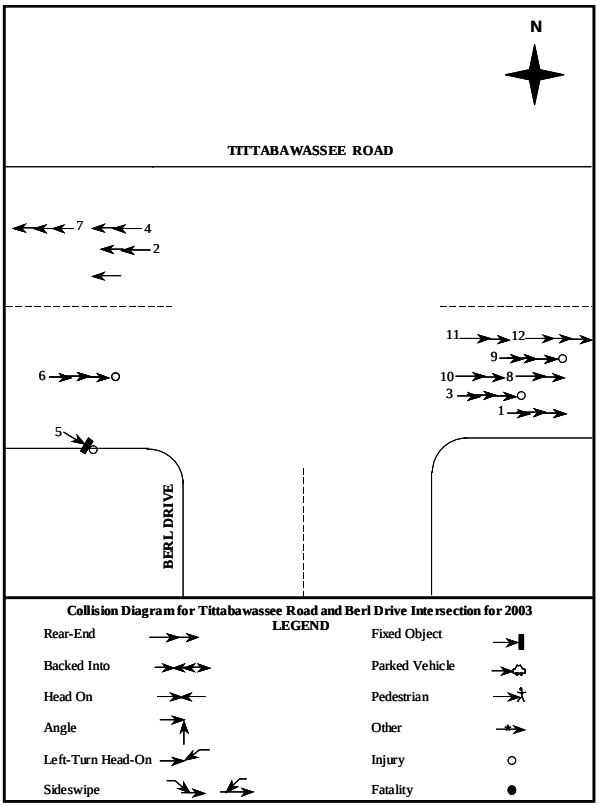


Figure 6.19.3. Collision Diagram for Tittabawassee Road and Berl Drive for 2003

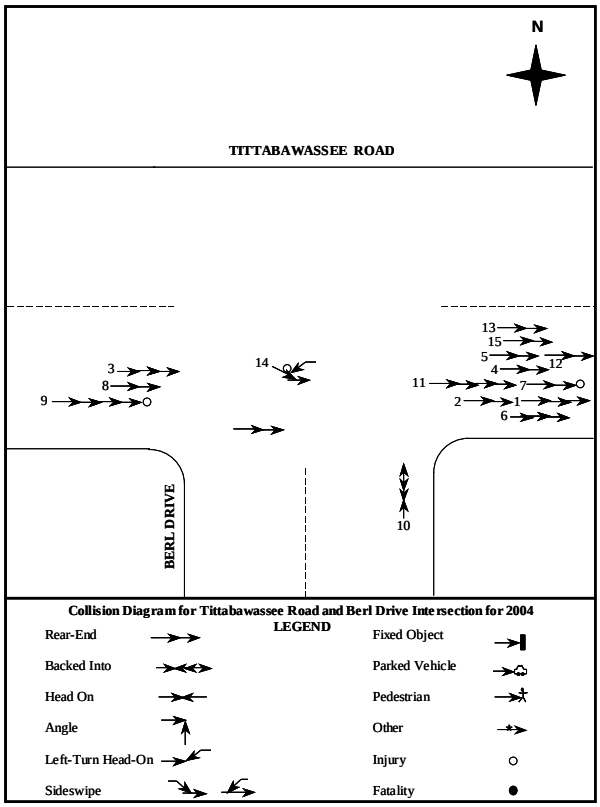


Figure 6.19.4. Collision Diagram for Tittabawassee Road and Berl Drive for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.16 per million entering vehicles, which is nominal, considering the approach volume at the intersection. Rear-end crashes are of specific significance at the intersection, which may have occurred due to major driveway activities near the intersection and lack of traffic signal control. There is one left-turn head-on crash on Tittabawassee Road. Also, there is one single motor vehicle fixed object crash which may be due to a driver losing control.

The crash distribution based upon type of crashes is shown in figure 6.19.5. Rear-end is the predominant type of crash reported at the intersection. The corresponding two-year crash frequency includes 24 rear-end crashes.

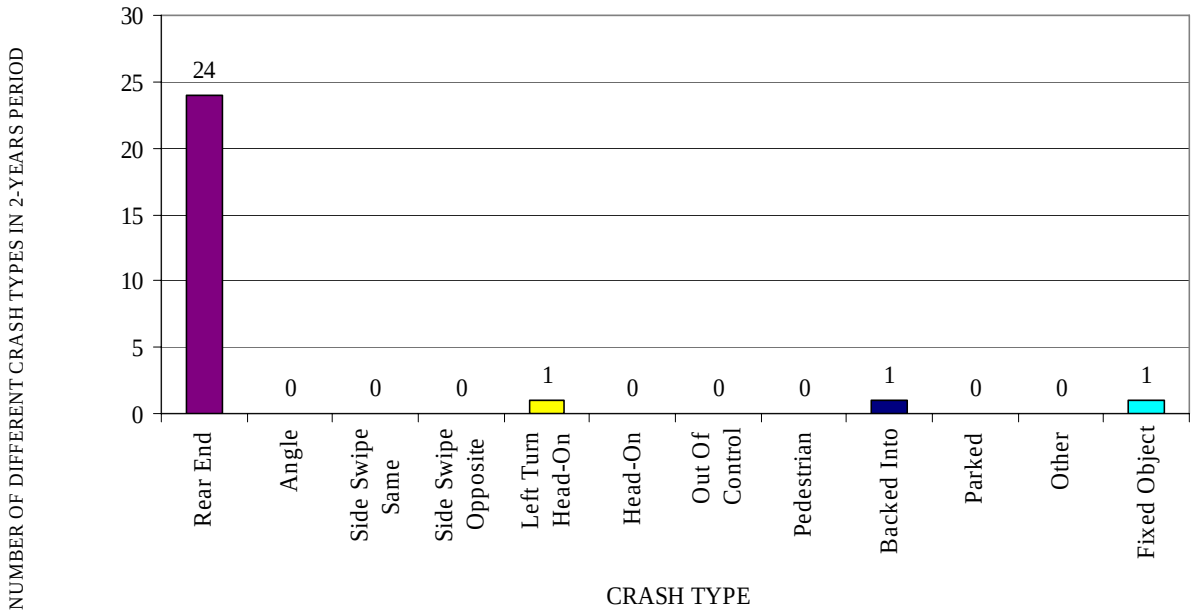


Figure 6.19.5: Two Year (2003 & 2004) Crash Experience at Tittabawassee Road and Berl Drive Intersection

Safety Deficiencies and Recommended Improvements

The majority of crashes occurring at this intersection of Tittabawassee Road and Berl Drive are of rear-end crashes. These crashes may due to several possible reasons including unsafe movements into and out of the driveways close to the intersection, inadequate gaps for the minor road traffic or the high speed along Tittabawassee Road.

Two different options are recommended for the intersection of Tittabawassee Road and Berl Drive.

Option 1

A stop bar, crosswalk and centerline should be provided for northbound Berl Drive and a stop bar should be provided for the southbound Kessel Driveway. Advance 'Stop Ahead' warning signs should be provided along Berl Drive. Sidewalk should be realigned with the crosswalk of the northbound Berl Drive. Pavement markings should be upgraded for the westbound approach.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The proposed intersection diagram for Option 1 improvements is shown in Figure 6.19.6 along with the upgraded pavement markings.

Option 2

Option 2 includes installing a 15-foot wide raised median, for Michigan left-turns, in the Tittabawassee Road left-turn lane. This would prohibit left turns at the intersection with Berl Drive and force vehicles to make a Michigan left-turn east and west of Tittabawassee Road.

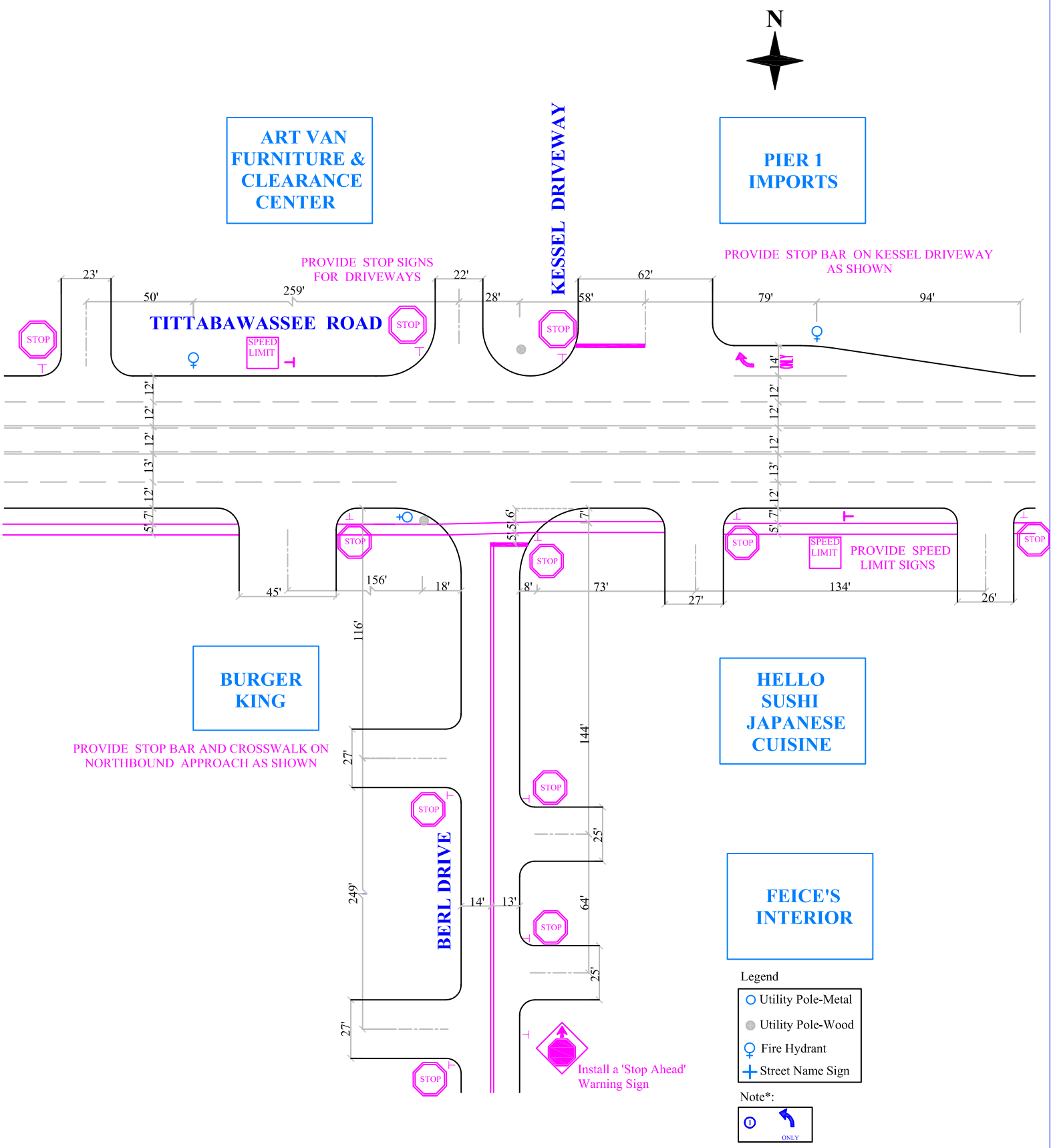


Figure 6.19.6 Proposed Diagram (Alternative 1) of Titabawassee Road and Berl Drive Intersection (Not to Scale)

A stop bar, crosswalk and centerline should be provided at northbound Berl Drive and a stop bar should be provided at the southbound Kessel Driveway. Advance 'Stop Ahead' warning signs should be provided along Berl Drive. The sidewalk should be realigned with the crosswalk of Berl Drive. Pavement markings should be upgraded for all approaches.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The proposed diagram for Option 2 in Figure 6.19.7 shows the proposed improvements, which includes the installation of a raised median and upgrading the pavement markings on all approaches.

A list of the recommended improvements is summarized below

Option 1:

- Upgrade pavement markings on the westbound approach.
- Provide the stop bars at the northbound and southbound approaches, provide crosswalk on the northbound approach as per MMUTCD guidelines.
- Realign sidewalks south of the intersection as shown in Figure 6.19.6.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all driveways located near the intersection. Landowners should provide these stop signs.

Option 2:

- Provide a 15-foot raised median in the center lane of Tittabawassee Road.
- Upgrade pavement markings on the all approaches.
- Realign sidewalks south of the intersection as shown in Figure 6.19.7.

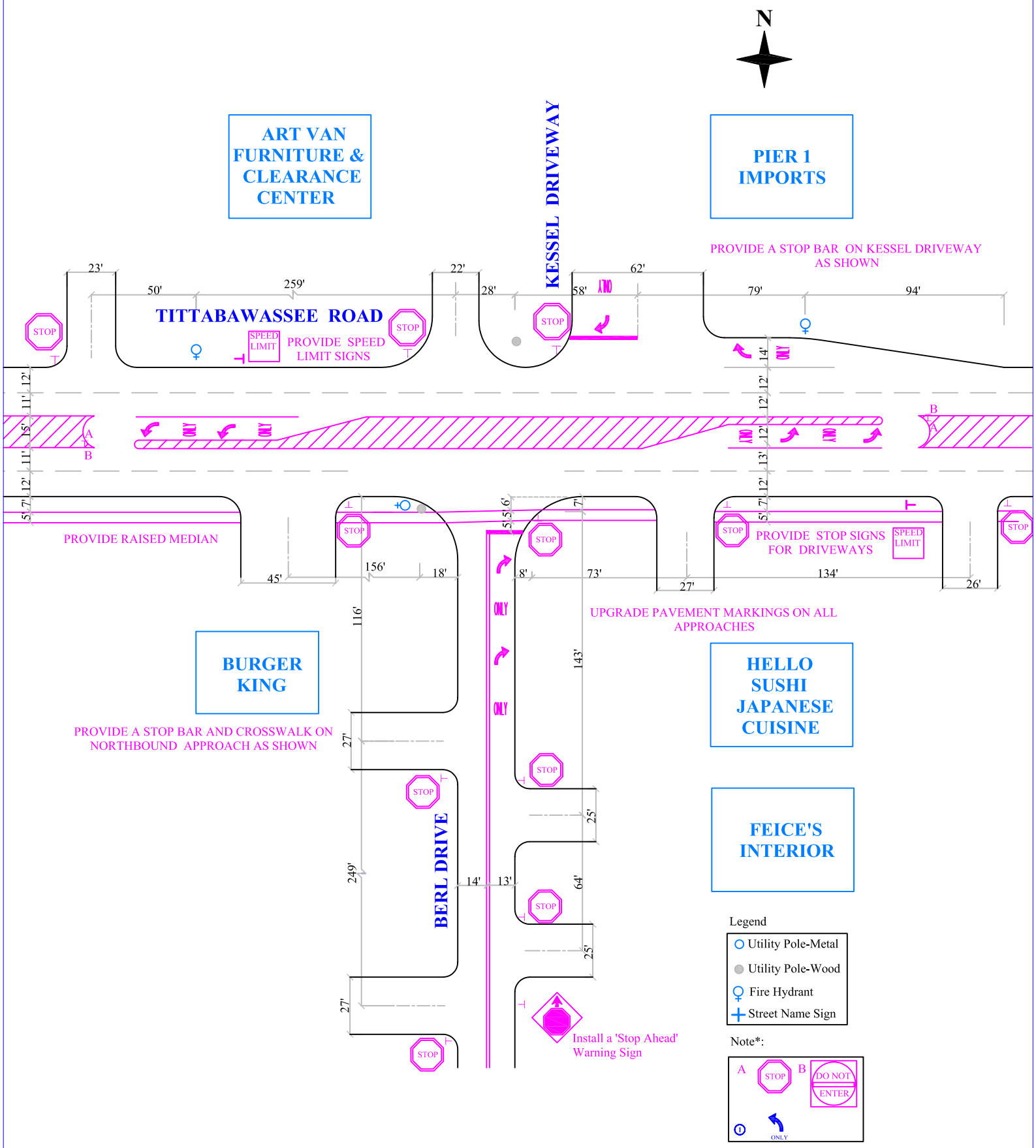


Figure 6.19.7 Proposed Diagram (Alternative 2) of Titabawassee Road and Berl Drive Intersection (Not to Scale)

- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all driveways located near the intersection. Landowners should provide these stop signs.

Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and Berl Drive indicate that, under existing conditions during the PM peak hour, northbound Berl Drive level of service is F with a delay of 87 seconds per vehicle and the southbound Kessel Driveway level of service is F with a delay of 226.8 seconds per vehicle.

The levels of service results for the existing conditions are summarized in Table 6.19.2.

Table 6.19.2.

Tittabawassee Road and Berl Drive
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions	
	Delay (Sec/Vehicle)	LOS
EB Tittabawassee	NA	NA
WB Tittabawassee	NA	NA
NB Berl Drive	87	F
SB Kessel Driveway	226.8	F

20. Tittabawassee Road and Towne Center Road

The intersection of Tittabawassee Road and Towne Center Road is located in the northeastern part of Saginaw Township. Tittabawassee Road runs in the east-west direction and Towne Center Road runs in the north-south direction. Tittabawassee Road functions as a principal arterial while Towne Center Road functions as a local business road.

At the westbound approach, Tittabawassee Road consists of one dedicated left-turn lane, two through lanes and one exclusive right turn lane for westbound traffic. At the eastbound approach, Tittabawassee Road consists of one exclusive left-turn lane, one through lane, and one shared through and right turn lane. Tittabawassee Road is approximately 70 feet wide on the east leg and 62 feet wide on the west leg of the intersection.

At the westbound approach, Towne Center Road consists of an exclusive left-turn lane, one through lane and an exclusive right-turn lane for northbound traffic and a single approach lane for southbound traffic. Tittabawassee Road is approximately 46 feet wide on both the approaches.

The speed limit on both Tittabawassee Road and Towne Center Road is 45 miles per hour.

Land Use

The intersection of Tittabawassee Road and Towne Center Road is occupied by business centers at all the corners. The northwest corner has a Taco Bell restaurant with a 29 foot driveway from the Towne Center Road. Across the Towne Center Road, there is another 29 foot driveway to access the National City Bank, which is located on the northeast corner of the intersection. The Southern part of the intersection is occupied by a Denny's Restaurant and a Damon's Grill located on the southwest and the southeast corners respectively. Three more driveways located reasonably far from the intersection provide access to these two centers. Two of them are on the south leg of Towne Center Road and the other is on the west leg of Tittabawassee Road.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and Towne Center Road.

The existing characteristics of the intersection are schematically depicted in the condition diagram shown in Figure 6.20.1 and in the details of the approaches contained in the following photographs 20a through 20d.

Photograph 20a. NB Towne Center Road at Tittabawassee Road

Photograph 20b. SB Towne Center Road at Tittabawassee Road

Photograph 20c. EB Tittabawassee Road at Towne Center Road

Photograph 20d. WB Tittabawassee Road at Towne Center Road

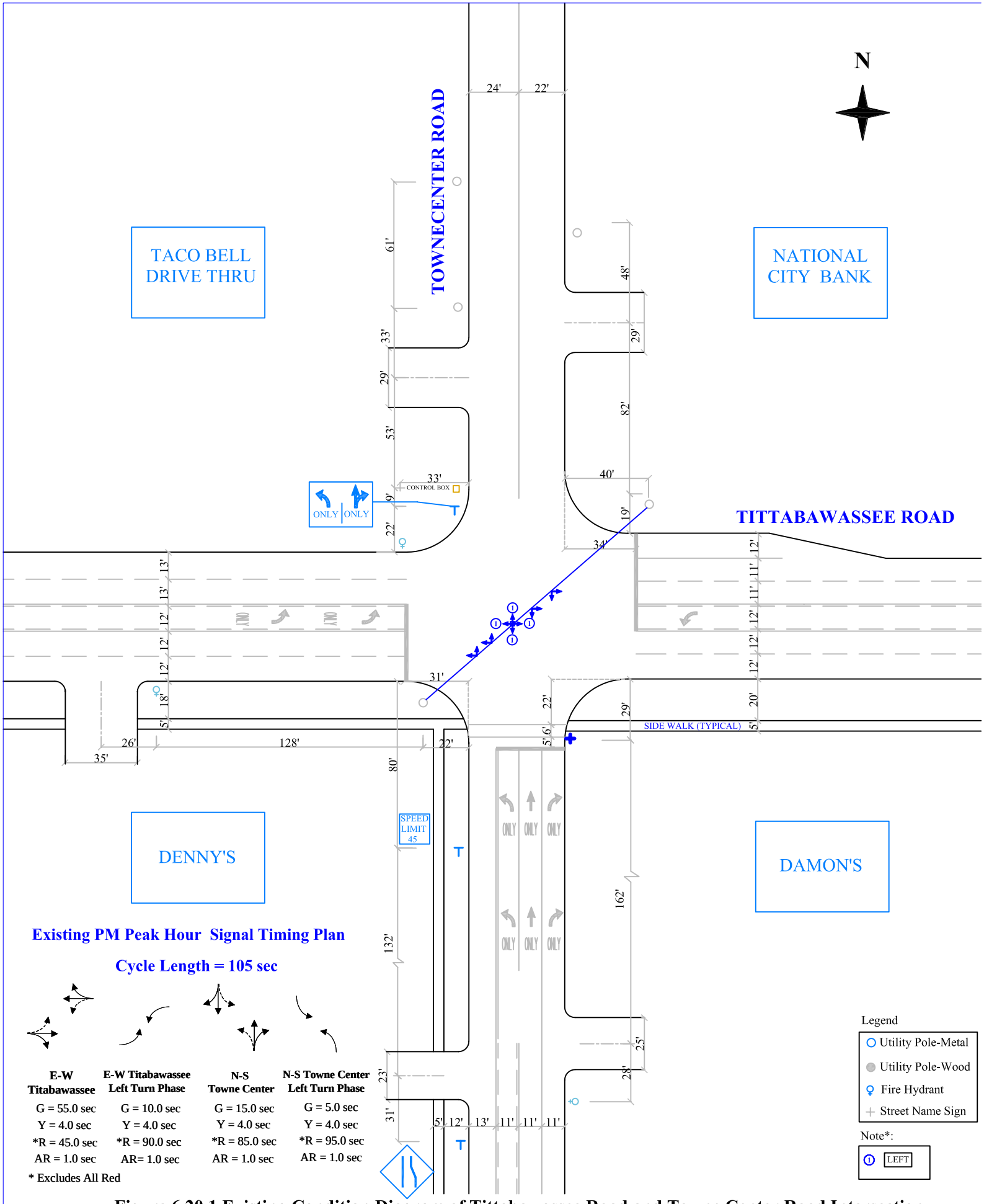


Figure 6.20.1 Existing Condition Diagram of Tittabawassee Road and Towne Center Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on April 4, 2006 from 4 PM to 6 PM by two people standing at a suitable corner of the intersection and covering all the approaches. The peak hourly volume from 4:45 PM to 5:45 PM is presented in Figure 6.20.2, which represent the highest peak hour (one-hour) volumes in the PM peak period. The traffic patterns indicate that the volumes in the east-west direction are relatively higher than the north-south direction. Also, the eastbound, westbound and southbound left turn volumes are considerably high.

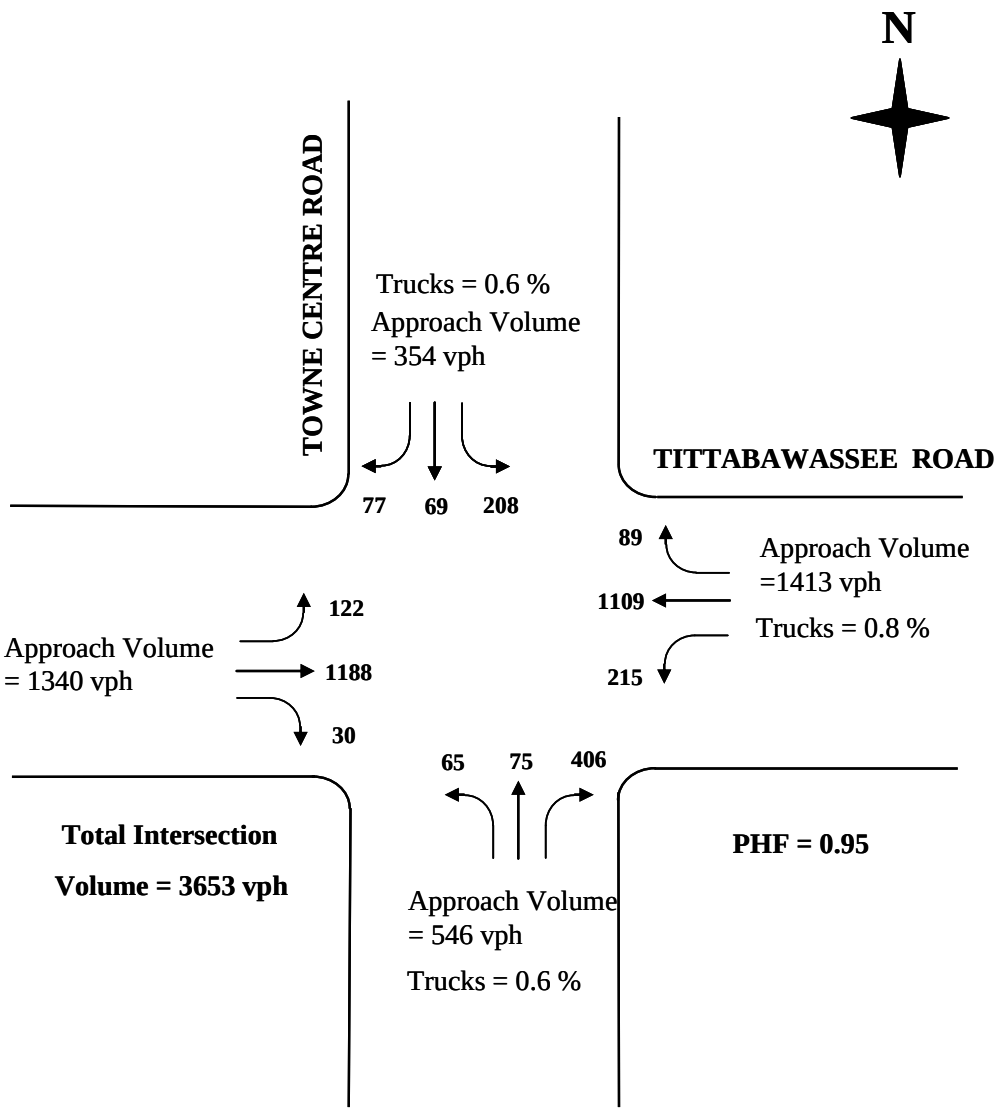


Figure 6.20.2 Existing PM Peak Hour (4:45–5:45 PM) Traffic Volumes at Tittabawassee Road and Towne Centre Road

Traffic Control

This intersection is controlled by a four-phase span-wire traffic signal with a 105-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for the north-south and east-west phases. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.20.1.

Crash Analysis

A total of 46 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 23 crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Tittabawassee Road and Towne Center Road are shown in Figures 6.20.3 and 6.20.4, respectively.

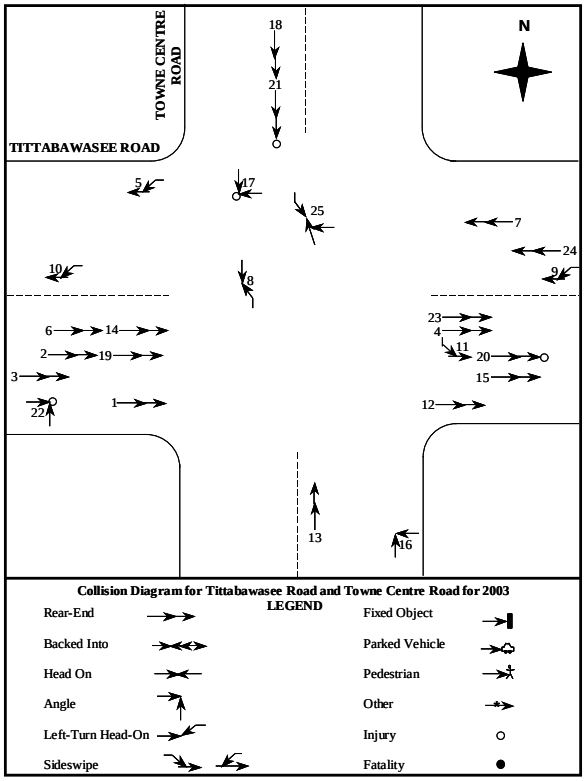


Figure 6.20.3 Collision Diagram for Tittabawassee Road and Towne Center Road for 2003

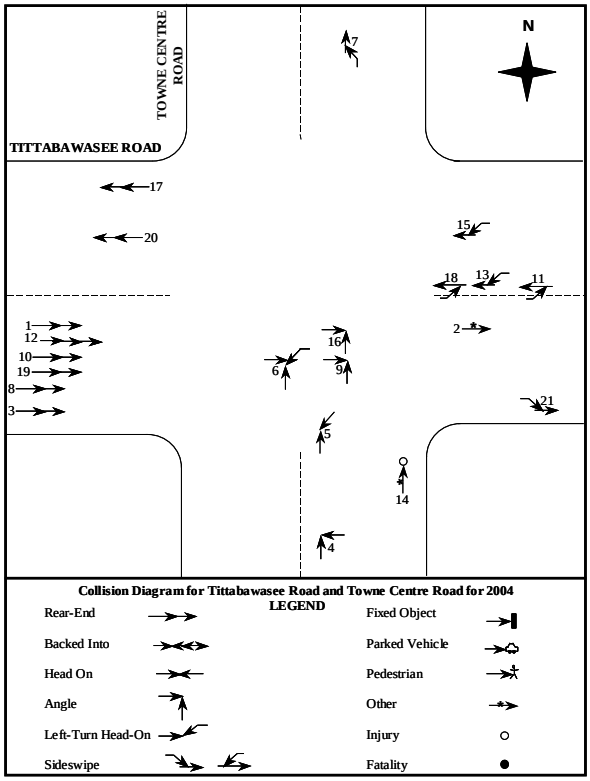


Figure 6.20.4 Collision Diagram for Tittabawassee Road and Towne Center Road for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The crash rate is 1.38 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The rear end crashes occurring specifically on the east-west approaches at the intersection are of specific significance, which may have occurred due to inadequate signal timing and insufficient clearance interval. Angle crashes are also quite significant in number, which may have occurred due to improper clearance intervals.

The crash distribution, based upon type of crashes, is shown in Figure 6.20.5. Rear-end, sideswipe same and angle crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequencies for these crashes include 24 rear-end crashes; eight sideswipe same crashes and eight angle crashes.

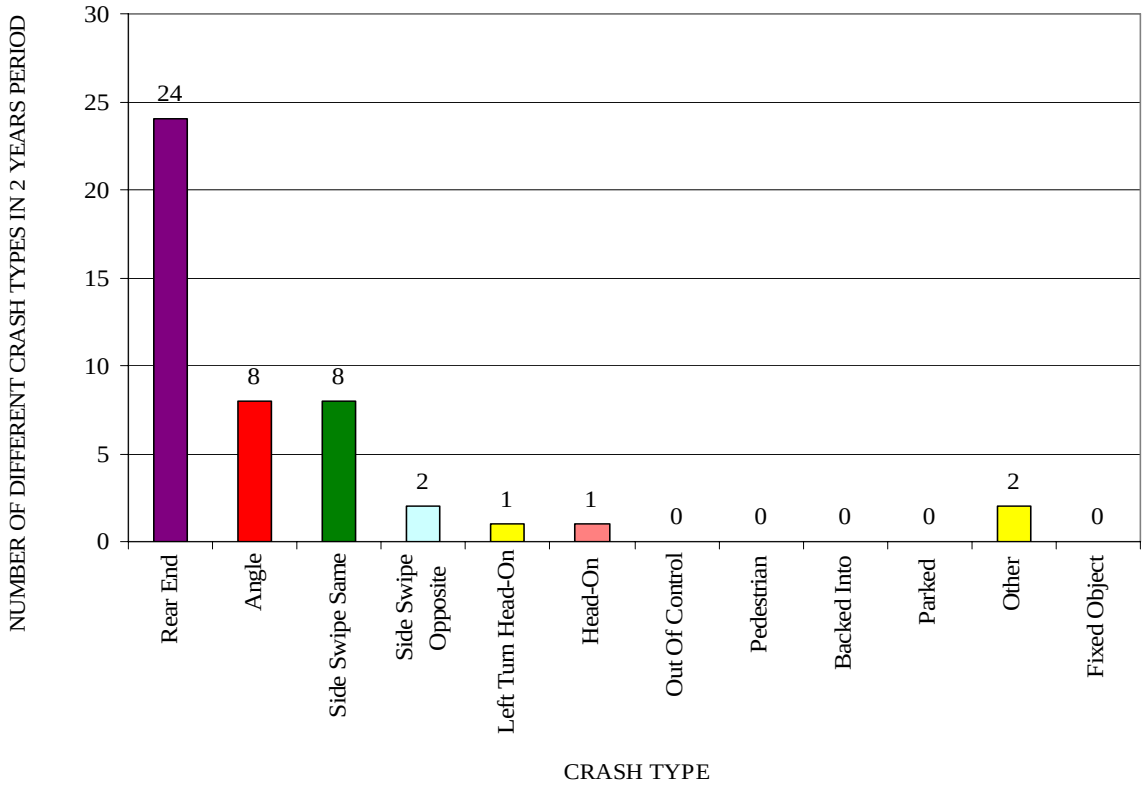


Figure 6.20.5 Two Year (2003 & 2004) Crash Experience at Tittabawasee Road and Towne Center Road Intersection

Safety Deficiencies and Recommended Improvements

The majority of the crashes occurring at the intersection of Tittabawassee Road and Towne Center Road are rear-end and angle. These crashes may be a result of several possible reasons including restricted or poor visibility of traffic signals and improper clearance intervals. Moreover, analysis of UD-10 forms reveals that a considerable number of these crashes have occurred due to unsafe movements from and into the driveways situated close to the intersection.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.20.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Stop bars and crosswalks should be relocated on all approaches. Pavement markings should be upgraded with proper directional arrows on all approaches. Lane usage should be changed for the southbound approach as shown in the proposed diagram in Figure 6.20.6.

Post-mounted traffic signals with “LEFT” illuminated signs should be provided at all corners of the intersection to assist left turning vehicles. Provide a new stop bar, centerline and pavement markings on the northern leg of the intersection.

The existing yellow and all-red intervals are shown in Table 6.20.1, along with the proposed lengths of these intervals and their potential benefits.

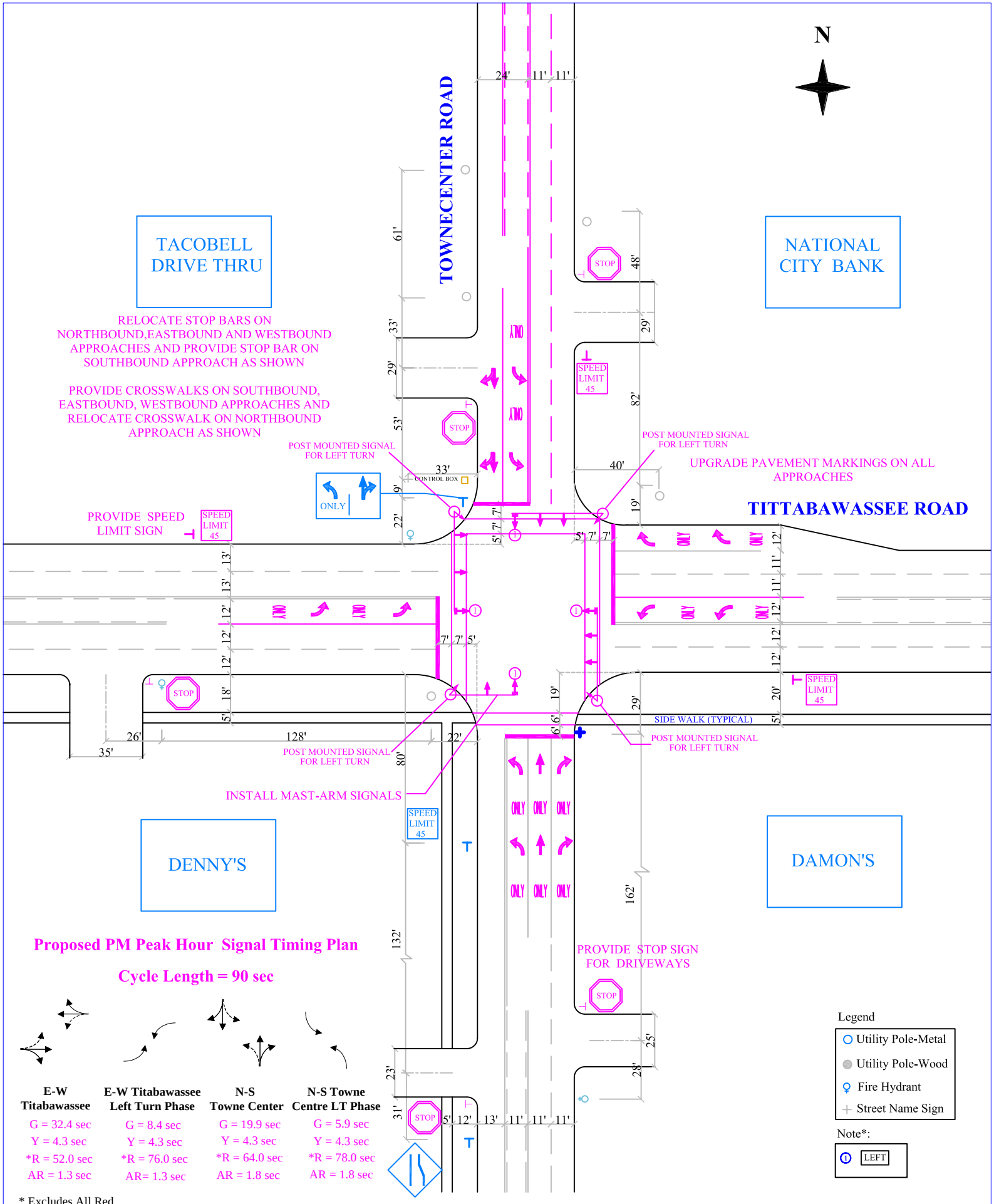


Figure 6.20.6 Proposed Diagram of Tittabawassee Road and Towne Center Road Intersection (Not to Scale)

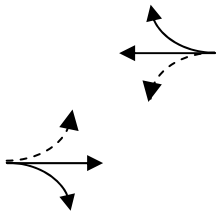
Table 6.20.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and Towne Center Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.0	1.0	4.3	1.3	Less rear-end, angle and injury crashes
E-W Left-Turn Phase	4.0	1.0	4.3	1.3	
N-S Through Phase	4.0	1.0	4.3	1.8	
N-S Left-Turn Phase	4.0	1.0	4.3	1.8	

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



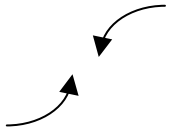
E-W Titabawassee

G = 32.4 sec

Y = 4.3 sec

*R = 52.0 sec

AR = 1.3 sec



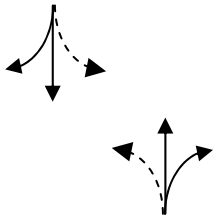
E-W Titabawassee Left Turn Phase

G = 8.4 sec

Y = 4.3 sec

*R = 76.0 sec

AR= 1.3 sec



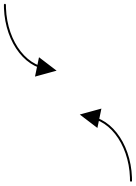
N-S Towne Center

G = 19.9 sec

Y = 4.3 sec

*R = 64.0 sec

AR = 1.8 sec



N-S Towne Center Left Turn Phase

G = 5.9 sec

Y = 4.3 sec

*R = 78.0 sec

AR = 1.8 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Increase the yellow interval for Tittabawassee Road and Towne Center Road to 4.3 seconds.
- Increase the all-red interval for Tittabawassee Road to 1.3 seconds and for Towne Center Road to 1.8 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the existing pavement markings for all the approaches.
- Relocate the stop bars and crosswalks on northbound, eastbound and westbound approaches, as per MMUTCD guidelines.
- Provide a new stop bar, centerline and pavement markings on the northern leg of the intersection.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at all corners of the intersection for left-turns.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. The landowners should provide these stop signs.
- Optimize the signal timings for A.M. and off-peak hours.
- Implement the optimized PM signal timing.

The proposed intersection diagram in Figure 6.20.6 (on page 286) shows the proposed improvements which include new installation of mast-arm support for the signals, a provision for post-mounted signal for left-turns, upgrading the pavement markings on all the approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and Towne Center Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is D with a delay of 41.3 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will remain at D with a reduction in delay of 40.8 seconds per vehicle

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.20.2.

Table 6.20.2.

Tittabawassee Road and Towne Center Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	14.0	B	42.3	D
WB Tittabawassee Road	13.8	B	40.2	D
NB Towne Center Road	199.8	F	49.6	D
SB Towne Center Road	44.6	D	25.6	C
Overall Intersection	**41.3**	**D**	**40.8**	**D**

21. Tittabawassee Road and I-675 SB Ramp

The intersection of Tittabawassee Road and I-675 SB Ramps is located in northwestern part of Zilwaukee Township. Tittabawassee Road runs in the east-west direction and the I-675 SB Ramps run in the north-south direction. Tittabawassee Road functions as a principal arterial.

At the westbound approach, Tittabawassee Road consists of two through lanes and one center lane, with no left turn for westbound traffic. At the eastbound approach, Tittabawassee Road consists of two through lanes, one dedicated right turn lane for the SB I-675 on Ramp and centre of 12-foot median. Tittabawassee Road is approximately 60 feet wide on the east leg and 85 feet wide on the west leg of the intersection.

The I-675 SB off Ramp is 23 feet wide north of the intersection with two exit lanes for southbound exit traffic to Tittabawassee Road. I-675 SB on Ramp is 25 feet wide south of the intersection for traffic entering onto the SB I-675. The speed limit for Tittabawassee Road is 45 miles per hour.

Land Use

The intersection of Tittabawassee Road and I-675 Ramp has no significant land use, as the land near the intersection is used to access the on and off ramps of I-675.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and I-675 SB Ramps.

The existing characteristics of the intersection including traffic control and surrounding land uses are schematically shown in Figure 6.21.1. Details of the intersection approaches are shown in photographs 21a through 21d. Photographs show the signal heads hanging from the span wire installation and also show the pavement markings.

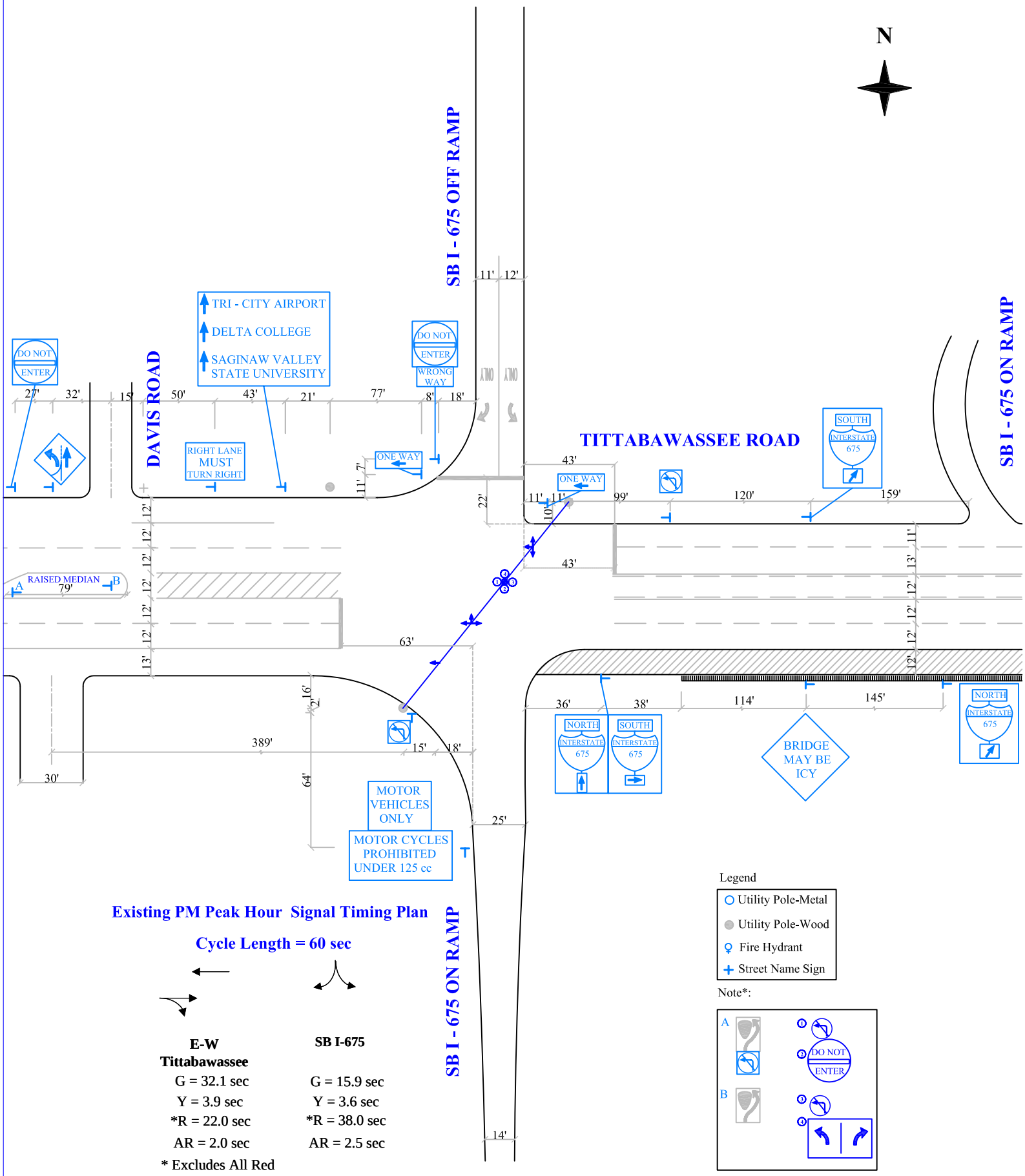


Figure 6.21.1 Existing Condition Diagram of Tittabawassee Road & SB I-675 Ramp Intersection (Not to Scale)

Photograph 21a. EB Tittabawasse Road at I-675 Ramp

Photograph 21b. WB Tittabawassee Road at I-675 Ramp

Photograph 21c. SB I-675 Off Ramp at Tittabawasse Road

Photograph 21d. SB I-675 On Ramp at Tittabawassee Road

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on April 4, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume from 4:30 PM to 5:30 PM is presented in Figure 6.21.2, which represent the highest peak hour (one-hour) volumes in the PM peak period. The traffic patterns indicate that

approach volumes of Tittabawassee Road are much higher than the volume of the I-675 SB off Ramp.

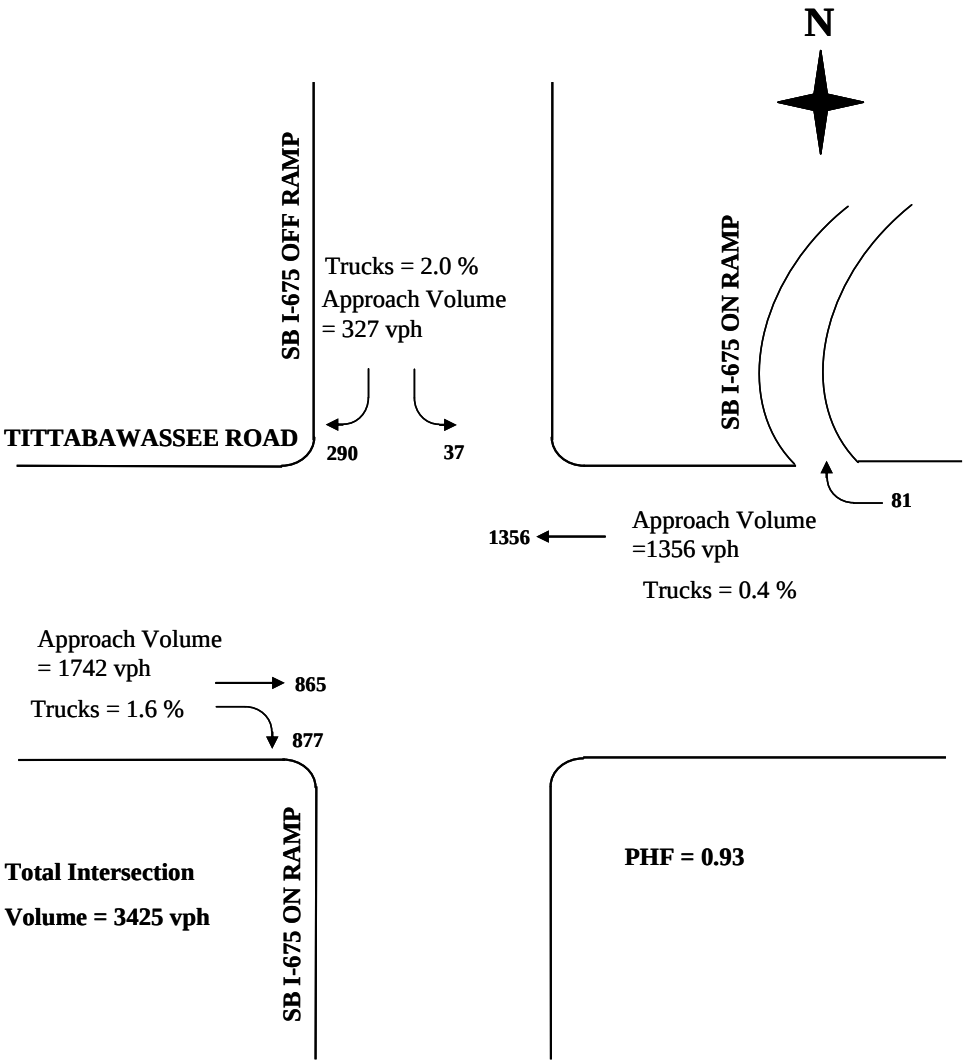


Figure 6.21.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes at Tittabawassee Road and SB I-675 Ramp

Traffic Control

This intersection is controlled by two-phase span-wire traffic signal with 60-second cycle length. The existing clearance interval includes a yellow interval of 3.9 seconds and an all-red interval of 2.0 seconds for the east-west phase. Similarly the southbound phase includes a yellow interval of 3.6 seconds and an all-red interval of 2.5 seconds. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.21.1.

Crash Analysis

A total of eight crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of four crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Tittabawassee Road and I-675 SB Ramp are shown in Figure 6.21.3 and 6.21.4, respectively.

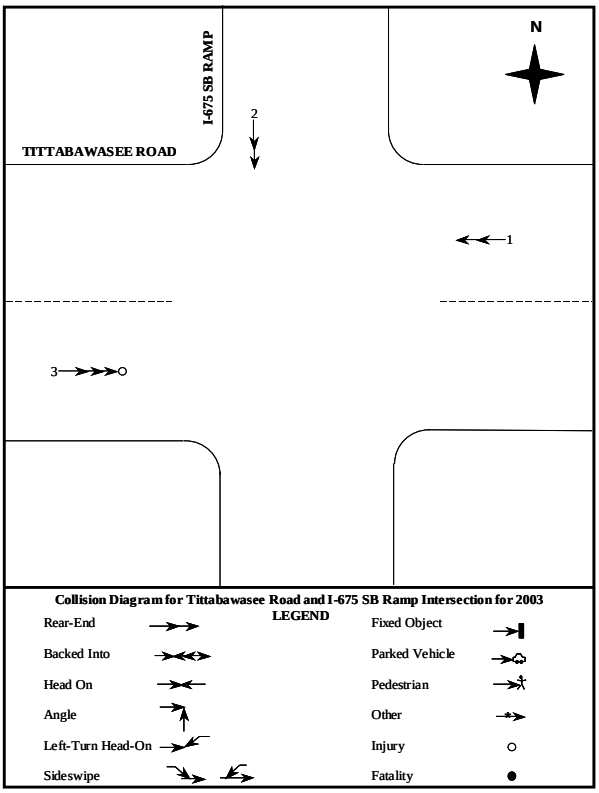


Figure 6.21.3 Collision Diagram for Tittabawassee Road and SB I-675 for 2003

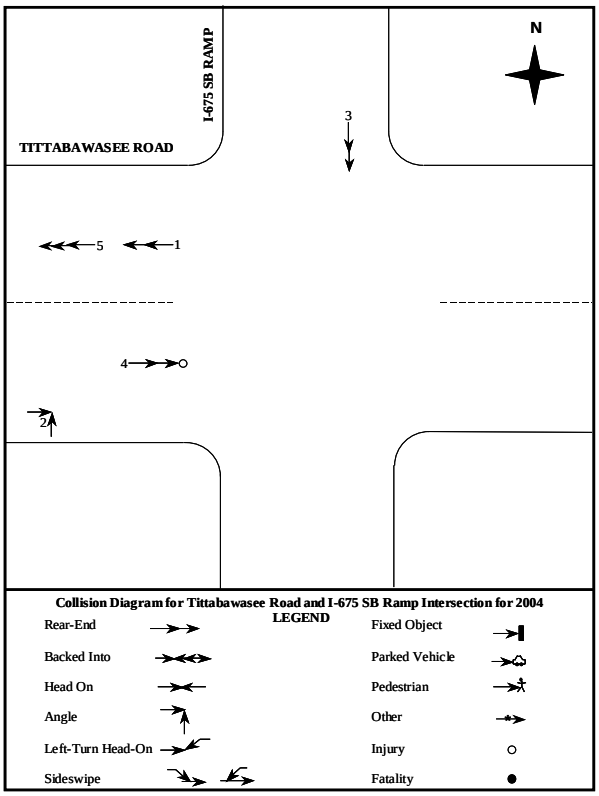


Figure 6.21.4 Collision Diagram for Tittabawassee Road and SB I-675 for 2004

The annual crash rate per million vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.26 per million entering vehicles, which is nominal, considering the approach volume at the intersection. There are significant crashes occurring near the intersection as seen from the collision diagrams. There are some rear end crashes occurring across each approach near the intersection, which may have occurred due to inadequate signal timing and insufficient clearance interval.

Safety Deficiencies and Recommended Improvements

The majority of crashes occurring at the intersection of Tittabawassee Road and SB I-675 Ramps are rear-end crashes These crashes may be due to several possible reasons including restricted or poor visibility of traffic signals and improper clearance intervals. Adequate clearance intervals have been provided and signal timings modified for better progression and safety as shown in the proposed signal timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed condition intersection diagram shown in Figure 6.21.5.

The cycle length of the proposed condition has been changed to 90 seconds to be compatible with the progression along Tittabawassee corridor.

Stop bars should be relocated for the southbound, eastbound and westbound approaches. Pavement markings should be upgraded for the southbound and eastbound approaches.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.21.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.21.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and SB I-675 Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	3.9	2.0	4.3	1.0	Less rear end, angle and injury crashes
SB Split Phase	3.6	2.5	3.5	2.5	

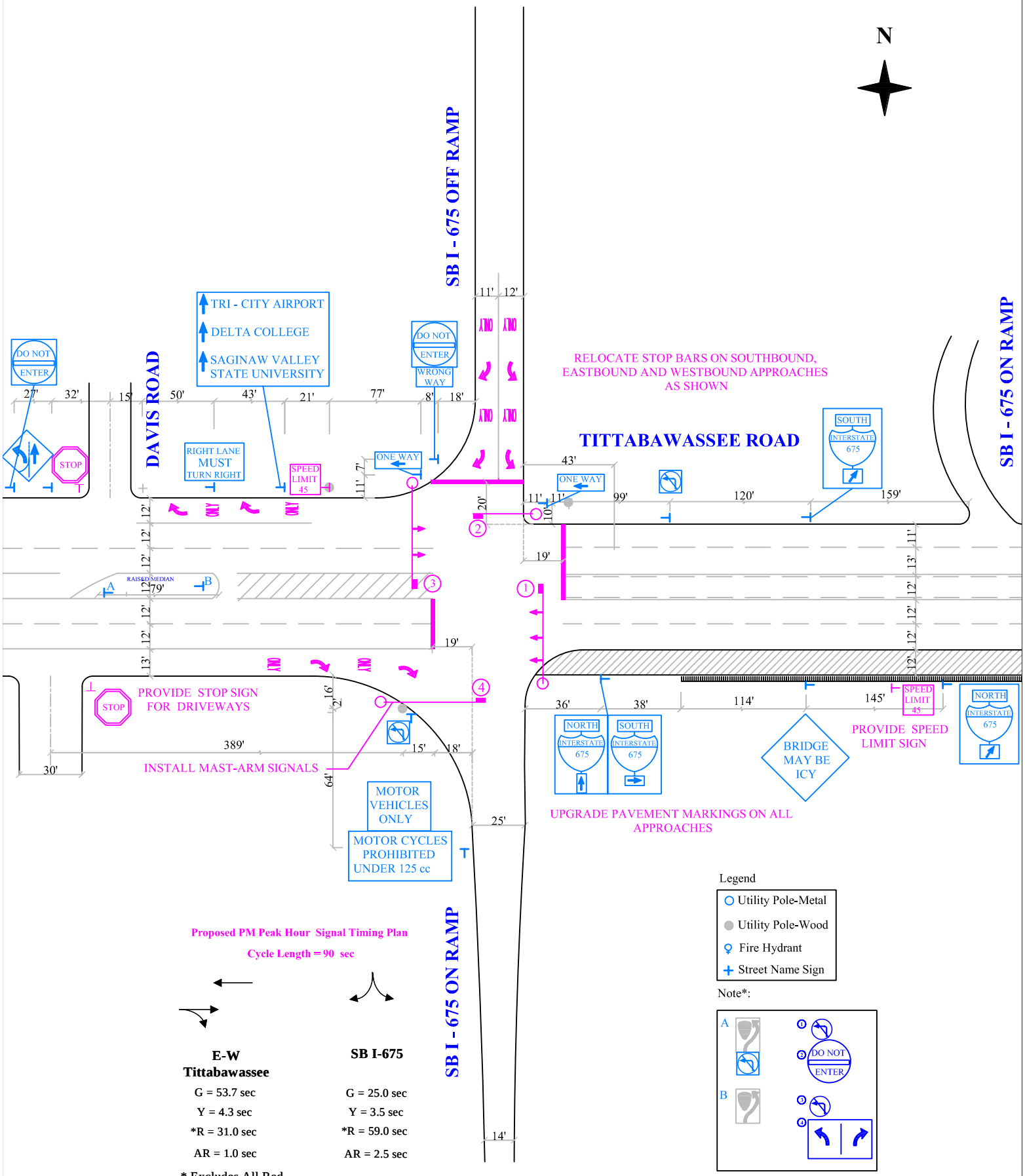


Figure 6.21.5 Proposed Diagram of Tittabawassee Road & SB I-675 Ramp Intersection (Not to Scale)

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds

E-W Tittabawassee	**SB I-675**
G = 53.7 sec	G = 25.0 sec
Y = 4.3 sec	Y = 3.5 sec
*R = 31.0 sec	*R = 59.0 sec
AR = 1.0 sec	AR = 2.5 sec

* Excludes All Red

A list of the recommended improvements and implementation stages are summarized below:

- Increase the yellow interval for Tittabawassee Road to 4.3 seconds and reduce it to 3.5 seconds for SB I-675 Ramp.
- Reduce the all-red interval for Tittabawassee Road to 1.0 second.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Relocate stop bars at the southbound, eastbound and westbound approaches, as per MMUTCD guidelines.
- Provide upgraded pavement markings with proper directional arrows for the southbound and eastbound approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. The landowners should provide these stop sign.

- Optimize the signal timings for A.M. and off-peak hours.
- Implement the optimized PM signal timing.

Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and SB I-675 Ramp indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 10.2 seconds per vehicle.

With the modifications to the signal timings, phasing and the noted improvements, the overall intersection level of service will remain at B with an increased delay of 19.2 seconds per vehicle.

The overall delay at the intersection will increase due to the increased cycle length and the provision of adequate yellow and all-red intervals. However, the safety at the intersection will be improved along with the traffic progression.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.21.2.

Table 6.21.2.
Tittabawassee Road and SB I675
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	9.7	A	25.4	C
WB Tittabawassee Road	8.7	A	8.3	A
SB I-675 Off Ramp	21.2	C	34.3	C
Overall Intersection	**10.2**	**B**	**19.2**	**B**

22. Tittabawassee Road and NB I-675 Ramps

The intersection of Tittabawassee Road and NB I-675 Ramps is located in the northwestern part of Zilwaukee City. NB I-675 Ramps run in the north-south direction and Tittabawassee Road runs in the east-west direction. The NB I-675 on Ramp of the intersection is located at the north leg of the intersection and NB I-675 off ramp in located at the south leg. Tittabawassee Road functions as a principal arterial.

At the westbound approach, Tittabawassee Road consists of one exclusive right-turn lane, two through lanes and one center lane. At the eastbound approach, Tittabawassee Road consists of two through lanes and a center lane. Tittabawassee Road is approximately 71 feet wide on the east leg and 60 feet wide on the west leg of the intersection.

The NB I-675 off Ramp is 34 feet wide with two exclusive left-turn lanes and one exclusive right-turn lane. The On Ramp of NB I-675 is about 25 feet wide.

Along Tittabawassee Road, the posted speed limit is 45 miles per hour.

Land Use

The intersection of Tittabawassee Road and NB I-675 Exit Ramps has no significant land use around the intersection, except a church on the southeast corner. Tittabawassee Road crosses over I-675 by bridge west of the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and NB I-675 Ramp.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.22.1. Details of the intersection approaches are shown in photographs 22a through 22c. Photograph 22a and 22b show the eastbound and westbound approaches of Tittabawassee Road while photograph 22c shows the I-675 NB off Ramp with signal heads hanging from the span wire installation and also show the pavement markings.

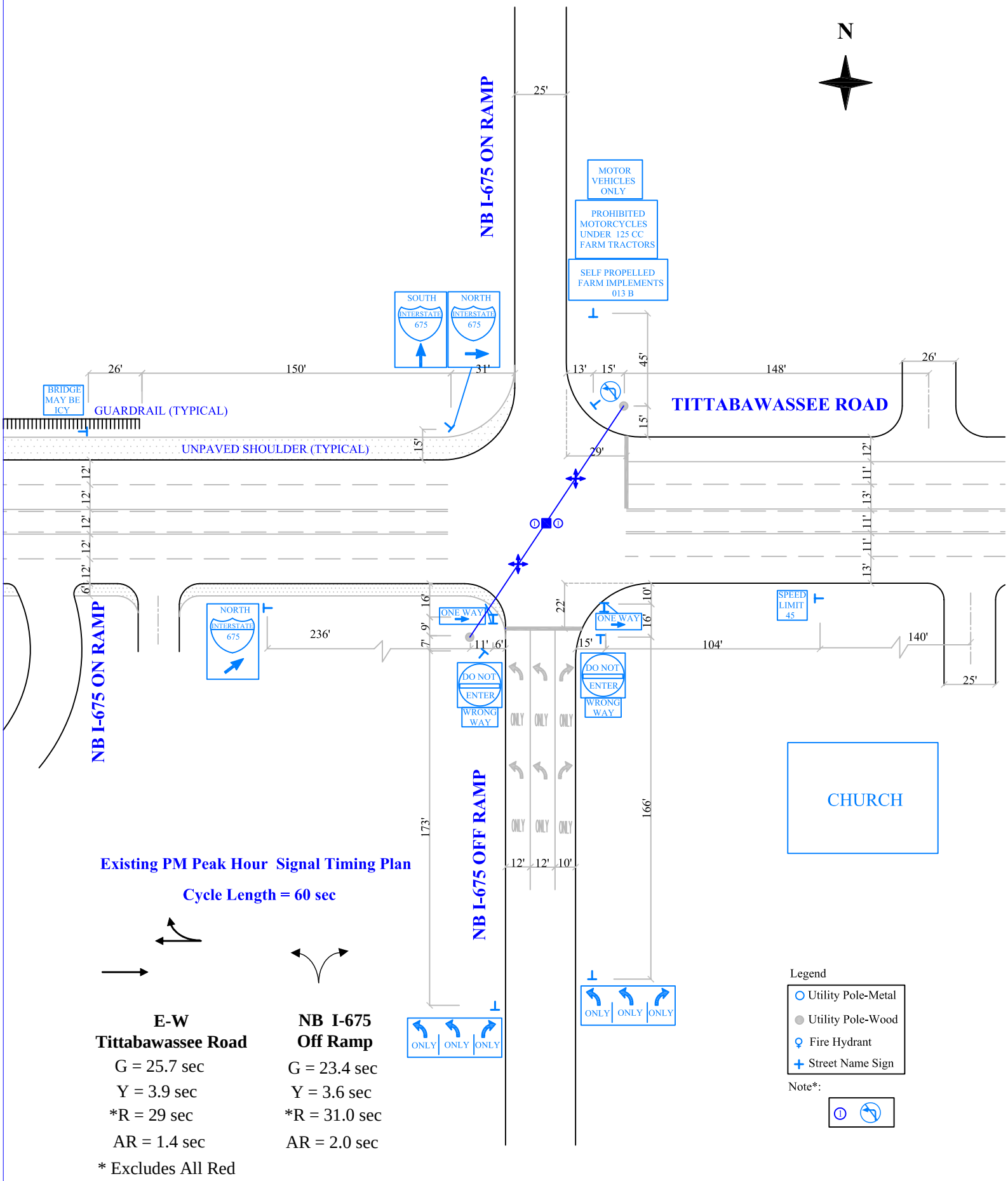


Figure 6.22.1. Existing Condition Diagram of Tittabawassee Road and NB I-675 Intersection (Not to Scale)

Photograph 22a. EB Tittabawassee Road at NB I-675 Ramp

Photograph 22b. WB Tittabawassee Road at NB I-675 Ramp

Photograph 22c. NB I-675 Off Ramp at Tittabawassee Road

Peak Hour Volumes

The PM peak hour traffic volumes were manually collected on April 4, 2006 from 4 PM to 6 PM by two people standing at a suitable corner of the intersection and covering two intersecting approaches each. The peak hourly volume from 4:30 PM to 5:30 PM is presented in Figure 6.22.2, which represent the highest peak hour (one-hour) volume in

the PM peak period. Left turning movements at the northbound approach of this intersection is much higher than the right-turning movement and the westbound traffic is much higher than eastbound traffic.

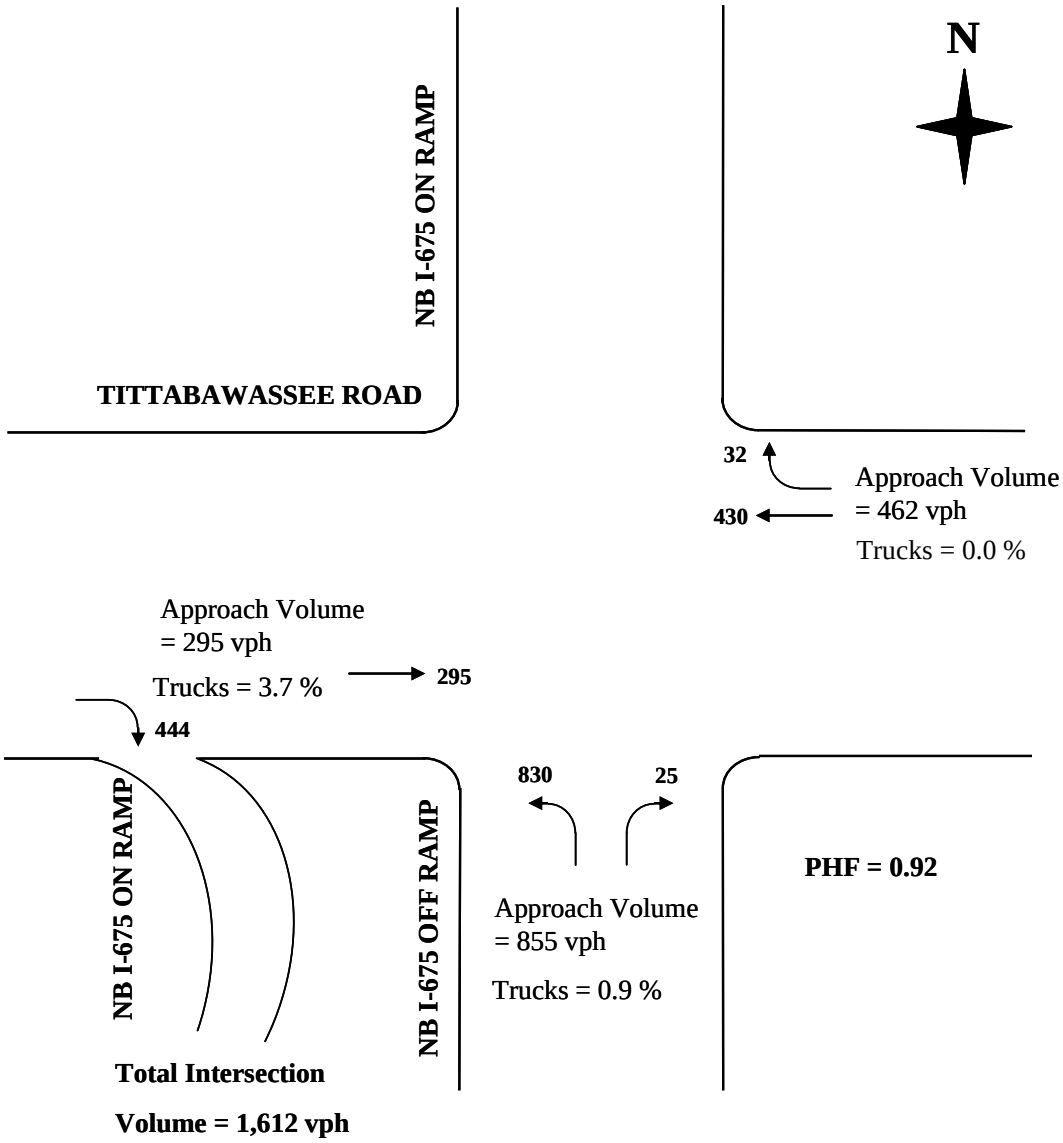


Figure 6.22.2 Existing PM Peak Hour (4:30 –5:30 PM) Traffic Volumes at Tittabawassee Road and NB I-675 Ramp

Traffic Control

This intersection is controlled by a two-phase span-wire traffic signal with a 60-second cycle length. The existing clearance interval includes a yellow interval of 3.9 seconds and an all-red interval of 1.4 seconds for the east-west phase and a yellow interval of 3.6 seconds and an all-red interval of 2.0 seconds for the NB I-675 Ramp phase. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.22.1.

Crash Analysis

A total of four crashes for the years 2004 were recorded. There were no crashes in the year 2003. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. Figure 6.22.3 shows the collision diagram for the year 2004.

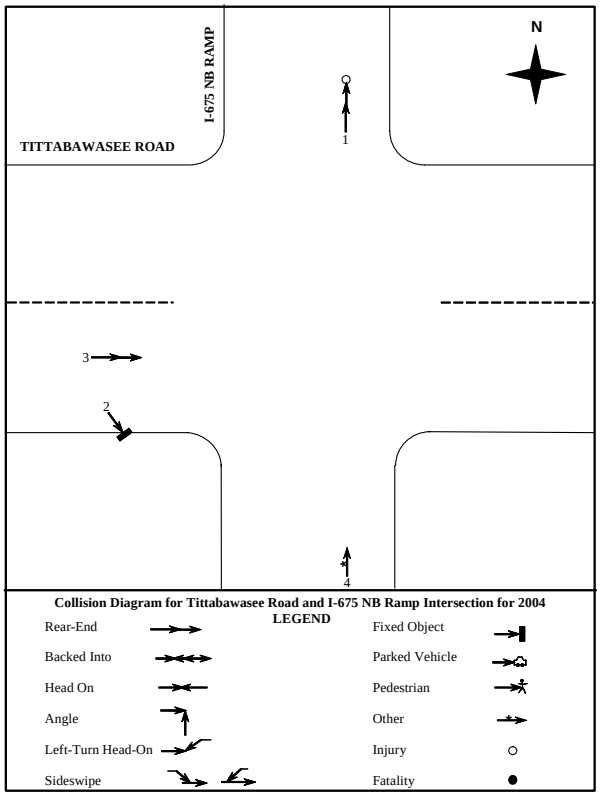


Figure 6.22.3 Collision Diagram for Tittabawassee Road and I-675 NB Ramps for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.54 per million entering vehicles, which is nominal, considering the approach volume at the intersection.

There are no significant crashes occurring near the intersection as seen from the collision diagrams. There are few rear end crashes occurring across each approach near the intersection, which may have occurred due to inadequate signal timing and insufficient clearance interval.

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of Tittabawassee Road and NB I-675 Ramp could be attributed to inadequate clearance interval. Adequate clearance intervals have been provided and signal timings modified for better progression and safety as shown in the proposed signal timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.22.4.

The cycle length has been changed to 90 seconds to be compatible with the progression along Tittabawassee corridor.

Stop bars should be relocated for northbound, eastbound and westbound approaches. Pavement markings should be upgraded for all approaches. Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.22.1, along with the proposed lengths of these intervals and their potential benefits.

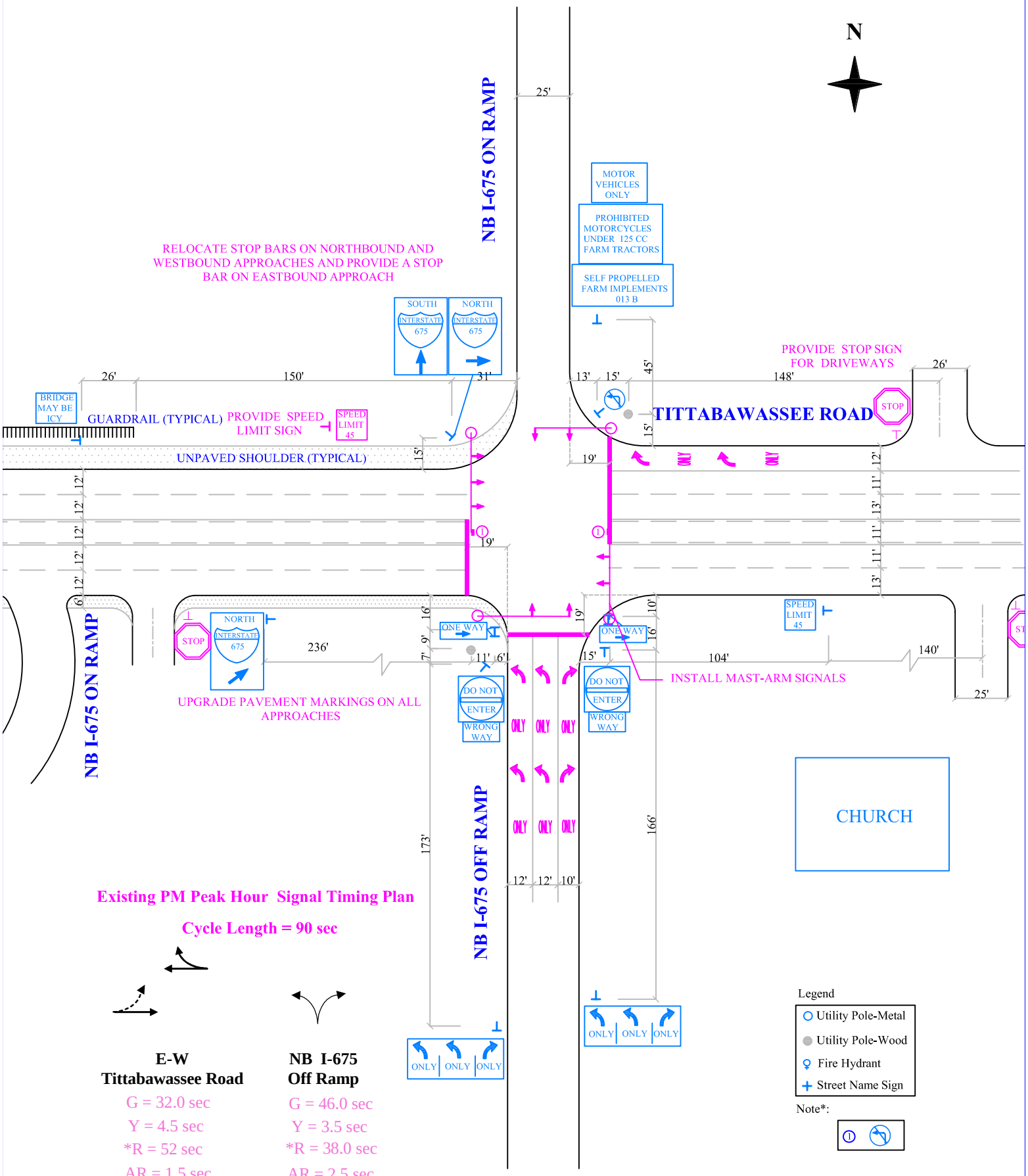


Figure 6.22.4 Proposed Diagram of Tittabawassee Road and NB I-675 Intersection (Not to Scale)

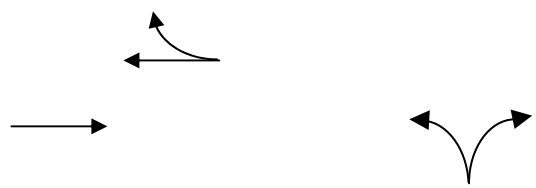
Table 6.22.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and NB-I 675 Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	Less rear end, angle, left-turn head-on and injury crashes
E-W Through Phase	3.9	1.4	4.5	1.5	
NB Split Phase	3.6	2.4	3.5	2.5	

The proposed PM peak period signal timing plan based on a 90 second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W
Tittabawassee Road

G = 32.0 sec

Y = 4.5 sec

*R = 52 sec

AR = 1.5 sec

* Excludes All Red

NB I-675
Off Ramp

G = 46.0 sec

Y = 3.5 sec

*R = 38.0 sec

AR = 2.5 sec

A list of the recommended improvements and implementation stages are summarized below:

- Increase the yellow interval to 4.5 seconds for all phases for Tittabawassee Road and reduce it to 3.5 seconds for movements along NB I-675.
- Increase the all-red interval for Tittabawassee Road to 1.5 seconds and to 2.5 seconds for NB I-675.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Relocate the stop bars at the northbound, westbound and eastbound approaches, as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide upgraded pavement markings with proper directional arrows for the all approaches.
- Provide stop signs for all driveways located near the intersection. The landowner should provide these stop signs.
- Optimize the signal timings for A.M. and off-peak hours.
- Implement the optimized PM signal timing.

In Figure 6.22.5 (on page 305), the proposed condition diagram includes the improvements to the intersection including the installation of mast-arm signals, upgrading the pavement markings for some of the approaches along with the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and NB I-675 Ramps indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 12.6 seconds per vehicle.

With the modifications to the signal timing, phasing and the noted improvements, the overall intersection level of service will be reduced to C with an increased delay of 26.3 seconds per vehicle. The overall delay at the intersection will increase due to the increased cycle length and the provision of adequate yellow and all-red interval. However, the safety at the intersection will be improved along with the traffic progression.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.22.2.

Table 6.22.2.
Tittabawassee Road and NB I- 675 Ramp
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	11.0	B	16.7	B
WB Tittabawassee Road	9.0	A	20.4	C
NB I 675	15.7	B	37.7	D
Overall Intersection	**12.6**	**B**	**26.3**	**C**

23. Tittabawassee Road and North Michigan Avenue

The intersection of Tittabawassee Road and North Michigan Avenue is located in the northwestern part of Zilwaukee City. Tittabawassee Road runs in the east-west direction and North Michigan Avenue runs in the north-south direction. Both Tittabawassee Road and North Michigan Avenue function as arterial roads. North of the intersection, North Michigan Avenue is known as North Michigan Road.

At the westbound approach, Tittabawassee Road consists of an exclusive left turn lane, an exclusive through lane and one dedicated right turn lane. At the eastbound approach, Tittabawassee Road consists of the same lane configuration but has an additional receiving lane. Tittabawassee Road is approximately 50 feet wide on the east leg and 60 feet wide on the west leg of the intersection.

At the southbound approach, North Michigan Road consists of one exclusive left turn lane, one exclusive through lane and one dedicated right-turn. At the northbound approach, North Michigan Road consists of one exclusive left-turn and one shared through and right turn lane. North Michigan Road is approximately 48 feet wide on the north leg and 45 feet wide on the south leg of the intersection.

The speed limit for both the roadways is 45 miles per hour.

Land Use

The intersection of Tittabawassee Road and North Michigan Avenue has minimal land uses around the intersection. The northeast and southeast corners of the intersections are empty. On northwest corner of the intersection there is Lake Huron Credit Union and on the southwest corner of the intersection there is The Roofing Man. There are several driveways near the intersection to access the various land uses.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside

obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Tittabawassee Road and North Michigan Avenue.

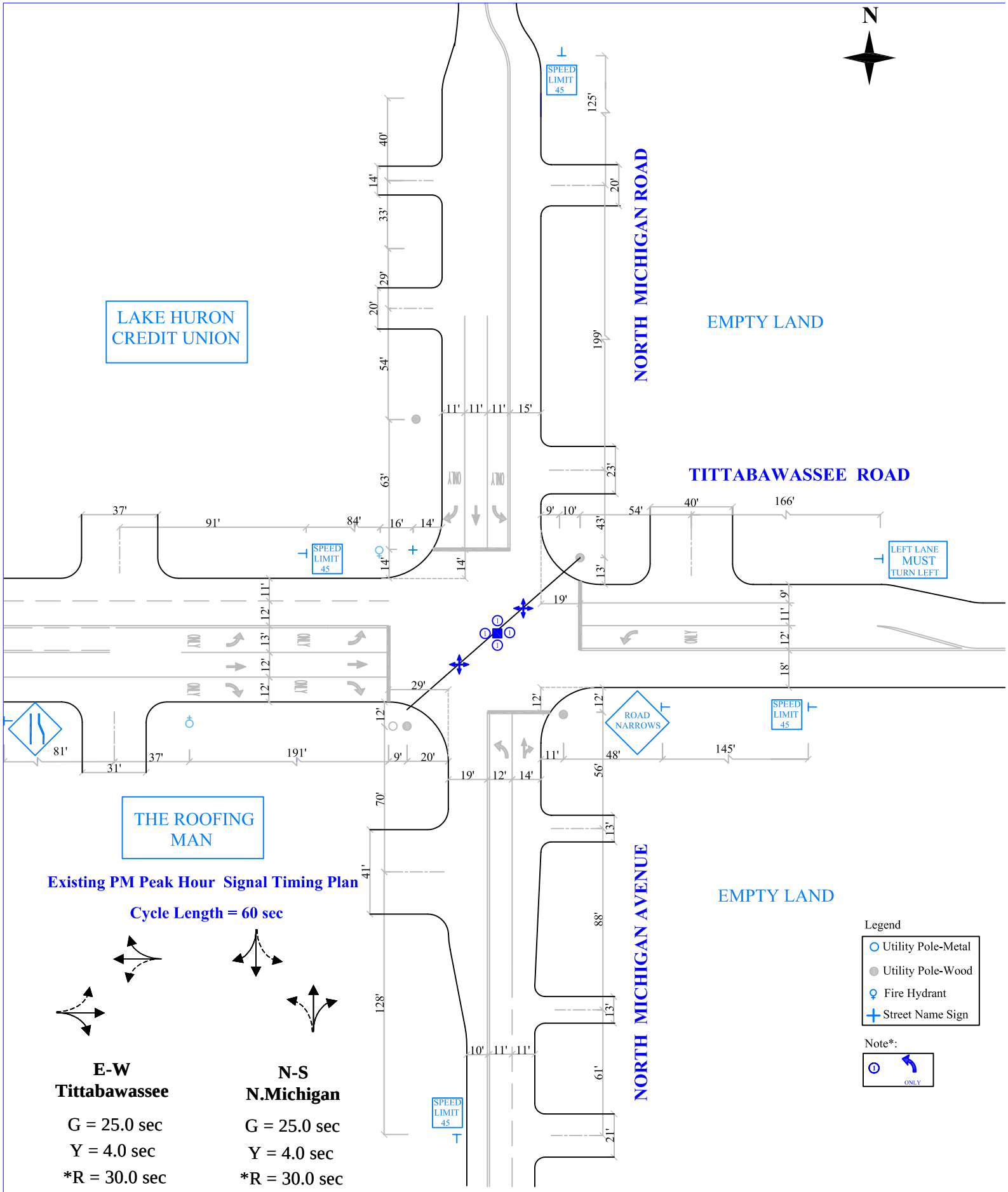
The existing characteristics of the intersection including traffic control and surrounding land uses are schematically shown in Figure 6.23.1. Details of the intersection approaches are shown in photographs 23a through 23d. Photographs show the hanging signal heads from the existing span wire installation. Photographs 23c and 23d show driveways located very near to the intersection for the northbound and southbound approaches respectively.

Photograph 23a. EB Tittabawassee Road at North Michigan Avenue

Photograph 23b. WB Tittabawassee Road at North Michigan Avenue

Photograph 23c. NB North Michigan Avenue at Tittabawassee Road

Photograph 23d. SB North Michigan Avenue at Tittabawassee Avenue



Existing PM Peak Hour Signal Timing Plan

Cycle Length = 60 sec

E-W Tittabawassee	**N-S N.Michigan**
G = 25.0 sec	G = 25.0 sec
Y = 4.0 sec	Y = 4.0 sec
*R = 30.0 sec	*R = 30.0 sec
AR = 1.0 sec	AR = 1.0 sec

* Excludes All Red

Figure 6.23.1.Existing Condition Diagram of North Michigan Avenue and Tittabawassee Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on clear day on April 4, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two suitable corners of the intersection and covering two intersecting approaches each. The peak hourly volume from 4:45 PM to 5:45 PM is presented in Figure 6.23.2, which represent the highest peak hour (one-hour) volume in the PM peak period. The traffic patterns indicate that volumes in the east-west direction are considerably higher than the north-south direction.

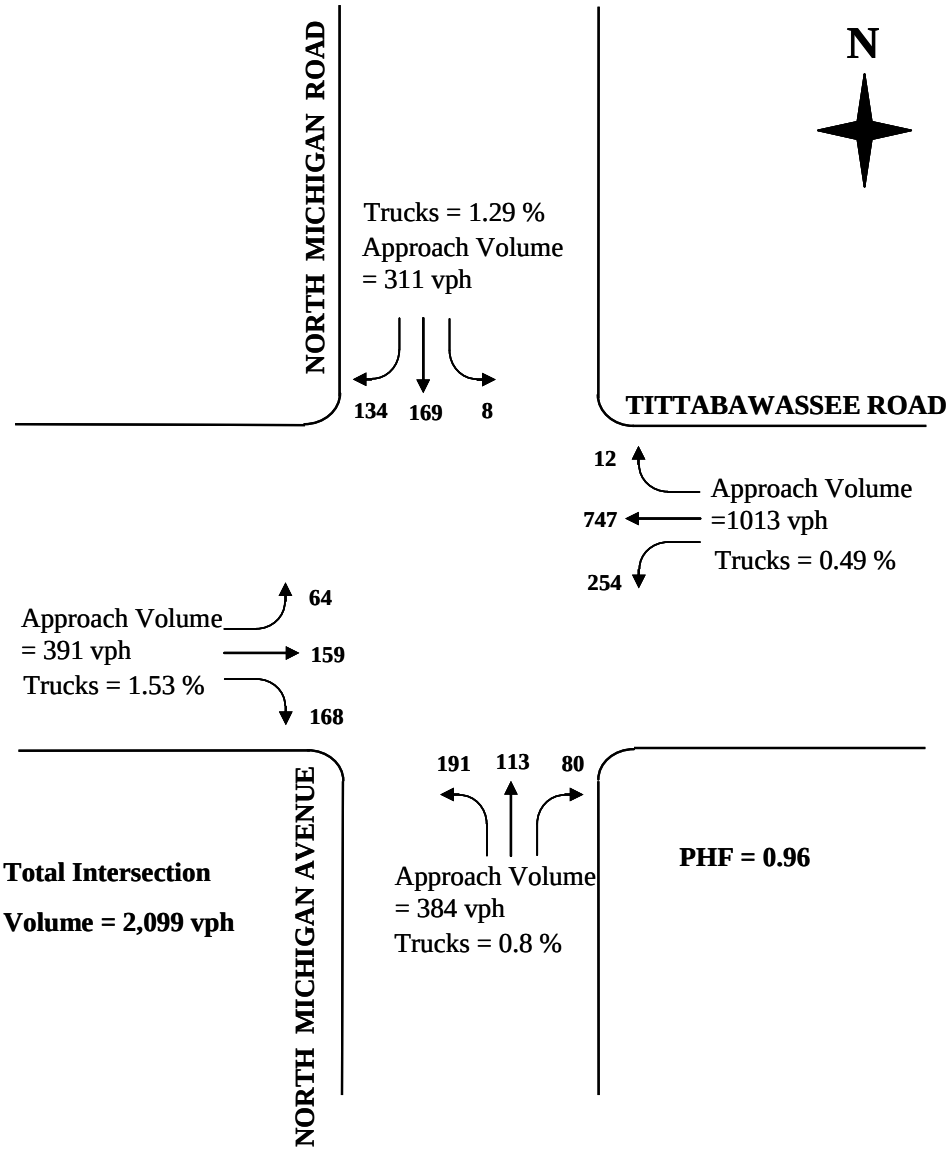


Figure 6.23.2. Peak Hour (4:45–5:45 PM) Traffic Volumes at Tittabawassee Road and North Michigan Avenue

Traffic Control

This intersection is controlled by a two-phase span-wire traffic signal with a 60-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for both the phases. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.23.1.

Crash Analysis

A total of 19 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of ten crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Tittabawassee Road and North Michigan Avenue are shown in Figures 6.23.3 and 6.23.4, respectively.

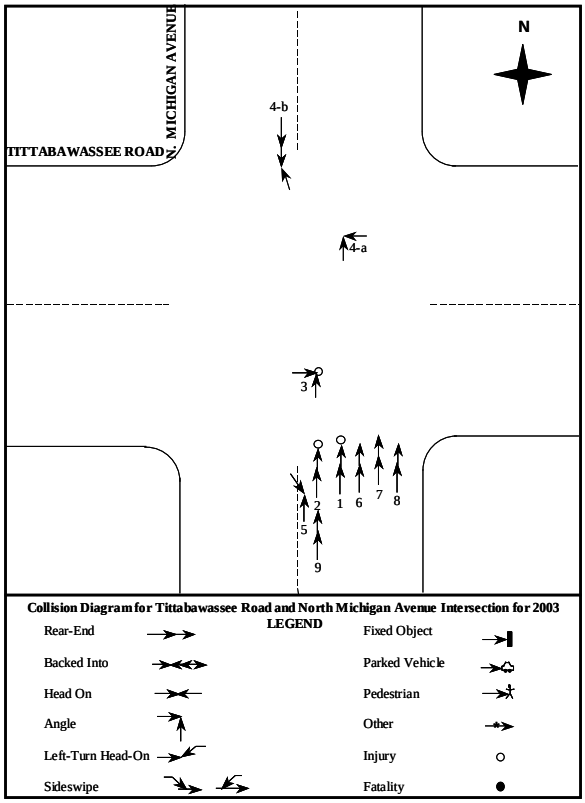


Figure 6.23.3. Collision Diagram for Tittabawassee Road and North Michigan Avenue for 2003

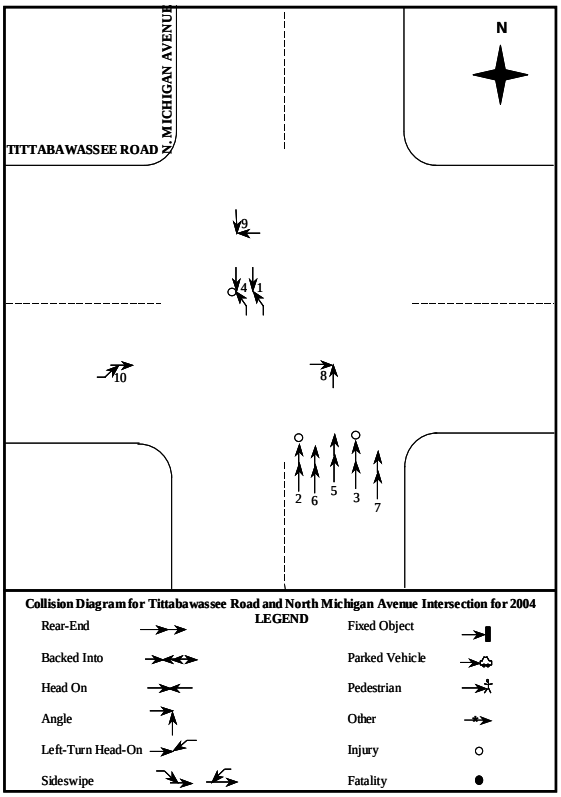


Figure 6.23.4. Collision Diagram for Tittabawassee Road and North Michigan Avenue for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.04 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The rear end crashes occurring on the northbound approach are of specific significance and may have occurred due to inadequate signal timing and insufficient clearance intervals.

The crash distribution, based upon type of crashes, is shown in Figure 6.23.5. Rear-end, angle and left-turn head-on crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequencies for these crashes include 11 rear-end crashes, four angle crashes and two left-turn head-on crashes.

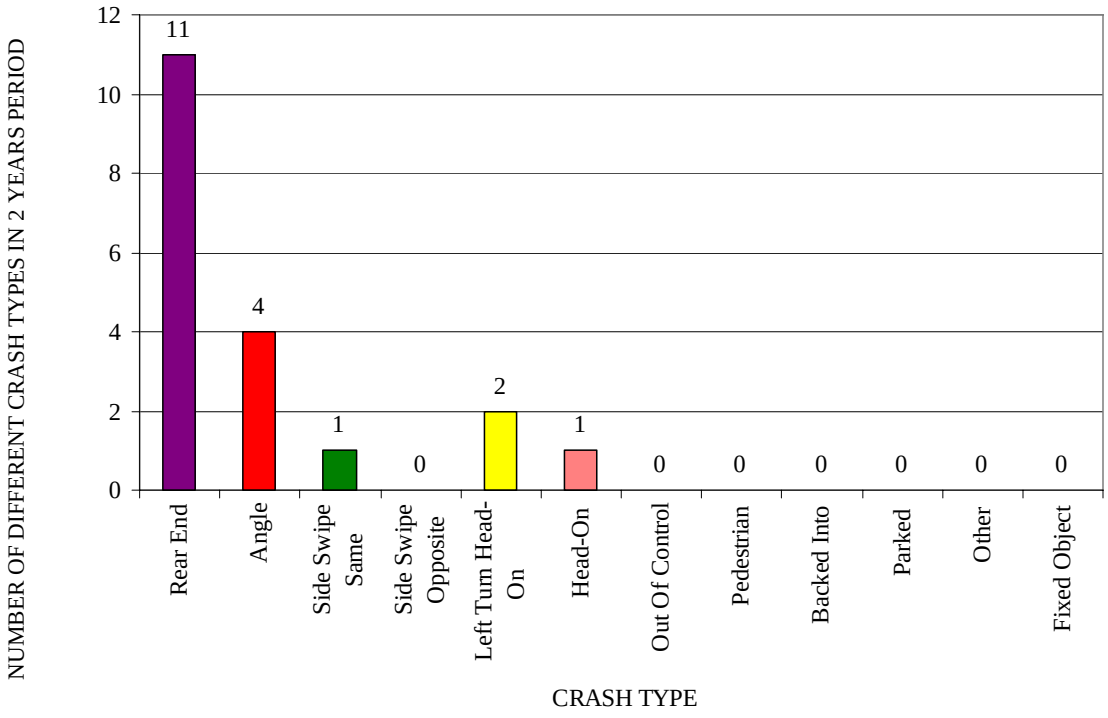


Figure 6.23.5: Two Year (2003 & 2004) Crash Experience at Tittabawassee Road and North Michigan Avenue Intersection

Safety Deficiencies and Recommended Improvements

Frequently occurring rear-end, left-turn head-on and angle crashes at the intersection of Tittabawassee Road and North Michigan Avenue could be attributed to poor visibility of traffic signals, improper phasing design and clearance interval inadequacies. The required clearance interval is computed and provided in the proposed signal-timing plan. The left turn volume is very high on the westbound approach but there is no separate left turn phase. The existing signal-timing plan has modified with addition of split movement for westbound Tittabawassee Road and a changed cycle length of 90 -second for desired progression.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.23.6.

Stop bar should be relocated for eastbound and westbound approaches. Pavement markings should be upgraded with proper directional arrows for the all approaches.

The existing yellow and all-red intervals are shown in Table 6.23.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.23.1. Existing and Proposed Clearance Intervals for the Tittabawassee Road and North Michigan Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	4.0	1.0	4.3	1.5	Less rear end, angle and injury crashes
E-W Through Phase	4.0	1.0	4.3	1.4	
WB Tittabawassee	-	-	4.3	1.4	

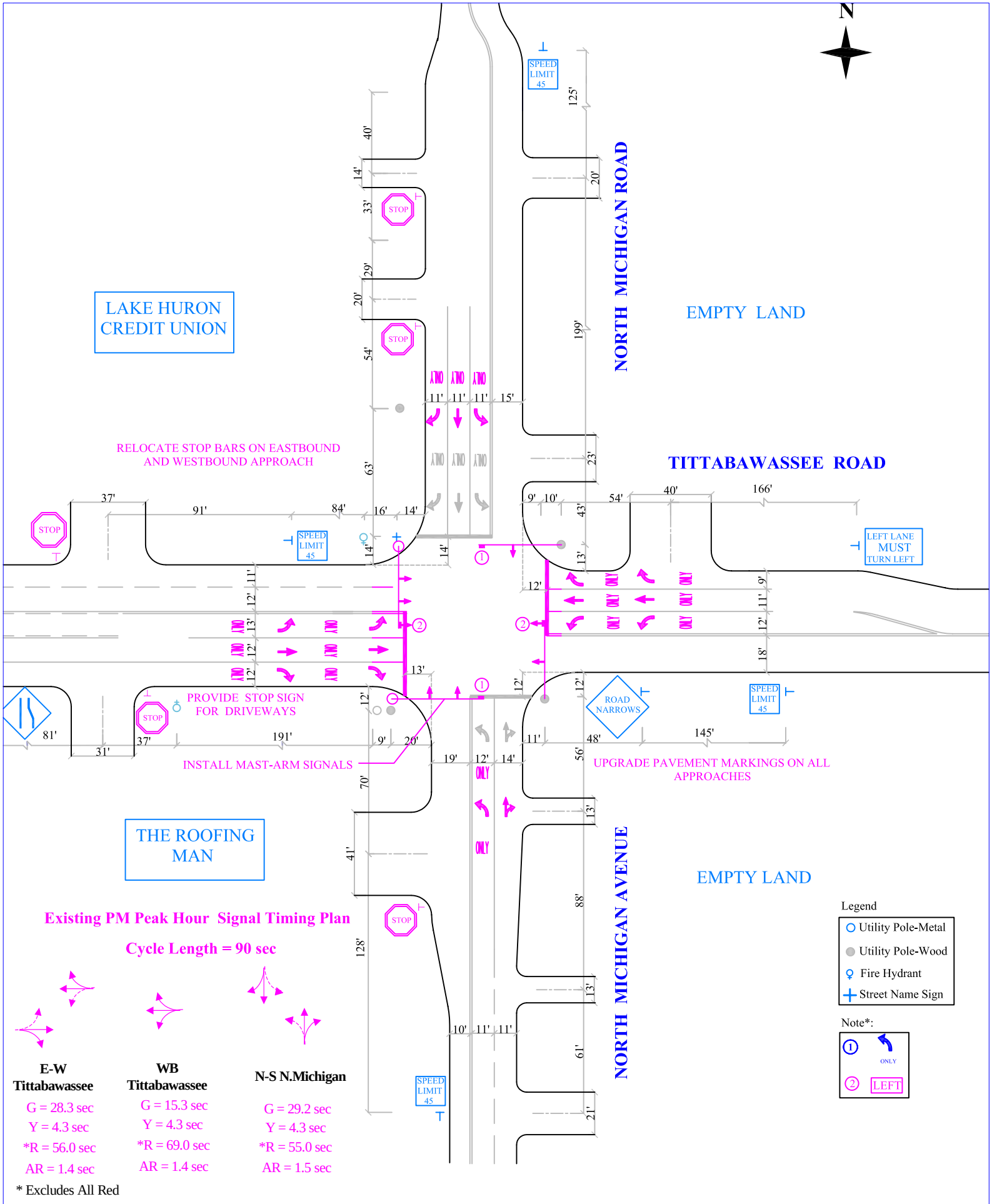
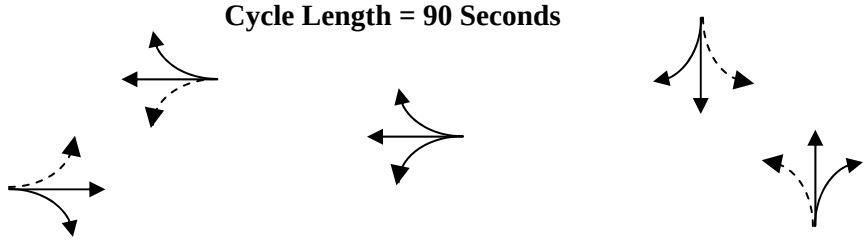


Figure 6.23.6 Proposed Diagram of North Michigan Avenue and Tittabawassee Road Intersection (Not to Scale)

The proposed PM peak period signal timing plan based on a 90 second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Tittabawassee	**WB Tittabawassee**	**N-S N. Michigan**
G = 28.3 sec	G = 15.3 sec	G = 29.2 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 4.3 sec
*R = 56.0 sec	*R = 69.0 sec	*R = 55.0 sec
AR = 1.4 sec	AR = 1.4 sec	AR = 1.5 sec

* Excludes All Red

A list of the recommended improvements and implementation stages are summarized below:

- Increase the yellow interval for Tittabawassee Road and North Michigan Avenue to 4.3 seconds.
- Increase the all-red interval for Tittabawassee Road to 1.4 seconds and for North Michigan Avenue to 1.5 seconds.
- Upgrade pavement markings with proper directional arrows on all the approaches.
- Provide a modified three-phase signal system.
- Relocate stop bars on eastbound and westbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Provide stop signs for all the driveways located near the intersection. Landowners should provide these stop signs.
- Optimize the signal timings for A.M. and off-peak hours.
- Implement the optimized PM signal timing.

Figure 6.23.6 shows the proposed improvements such as installing the mast-arm support for the signals, upgrading pavement marking and also shows the proposed signal timing plan

Operational Evaluation

The results of the level of service analysis for the intersection of Tittabawassee Road and North Michigan Avenue indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 12.0 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will remain at B with an increased delay of 17.2 seconds per vehicle. The overall delay at the intersection will increase due to the inclusion of a split phase and the provision of adequate yellow and all red intervals. However, the safety at the intersection will be improved along with the traffic progression.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.23.2.

Table 6.23.2.
Tittabawassee Road and North Michigan Avenue
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Tittabawassee Road	10.1	B	20.0	B
WB Tittabawassee Road	12.7	B	11.9	B
NB North Michigan Avenue	12.5	B	25.4	C
SB North Michigan Avenue	11.1	B	22.0	C
Overall Intersection	**12.0**	**B**	**17.2**	**B**

24. Gratiot Road (M-46) and Graham Road (M-52)

The intersection of Gratiot Road (M-46) and Graham Road (M-52) is located in the southwestern part of Thomas Township in Saginaw County. Gratiot Road runs in the east-west direction and Graham Road runs in the north-south direction. Both Gratiot Road and Graham Road function as principal arterials, also known as M-46 and M-52 respectively.

At the eastbound approach, Gratiot Road consists of an exclusive left-turn lane, two through lanes and one dedicated right turn lane. At the westbound approach, Gratiot Road consists of one exclusive left turn lane, two through lanes and a dedicated right-turn lane. Gratiot Road is approximately 87 feet wide on the east leg and 86 feet wide on the west leg of the intersection.

At the east and westbound approaches, Graham Road consists of one exclusive left turn lane, one through lane and one dedicated right turn lane. Graham Road is approximately 59 feet wide on the north leg and 58 feet wide on the south leg of the intersection.

The speed limits on Gratiot Road and Graham Road is 55 miles per hour.

Land Use

The intersection of Gratiot Road (M-46) and Graham Road (M-52) has a Citgo 7-11 on the northeast corner and Green Stone on the northwest corner. There are three driveways, serving these establishments on the north leg of the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Gratiot Road and Graham Road.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.24.1. Details of the intersection approaches are shown in photographs 24a through 24d. North and southbound approaches are shown by photographs 24a and 24b with the signal heads hanging from the span wire installation. All approaches of the intersection as shown in photograph 24a through 24d have firm pavement markings.

Photograph 24a. NB Graham Road at Gratiot Road

Photograph 24b. SB Graham Road at Gratiot Road

Photograph 24c. EB Gratiot Road at Graham Road

Photograph 24d. WB Gratiot Road at Graham Road

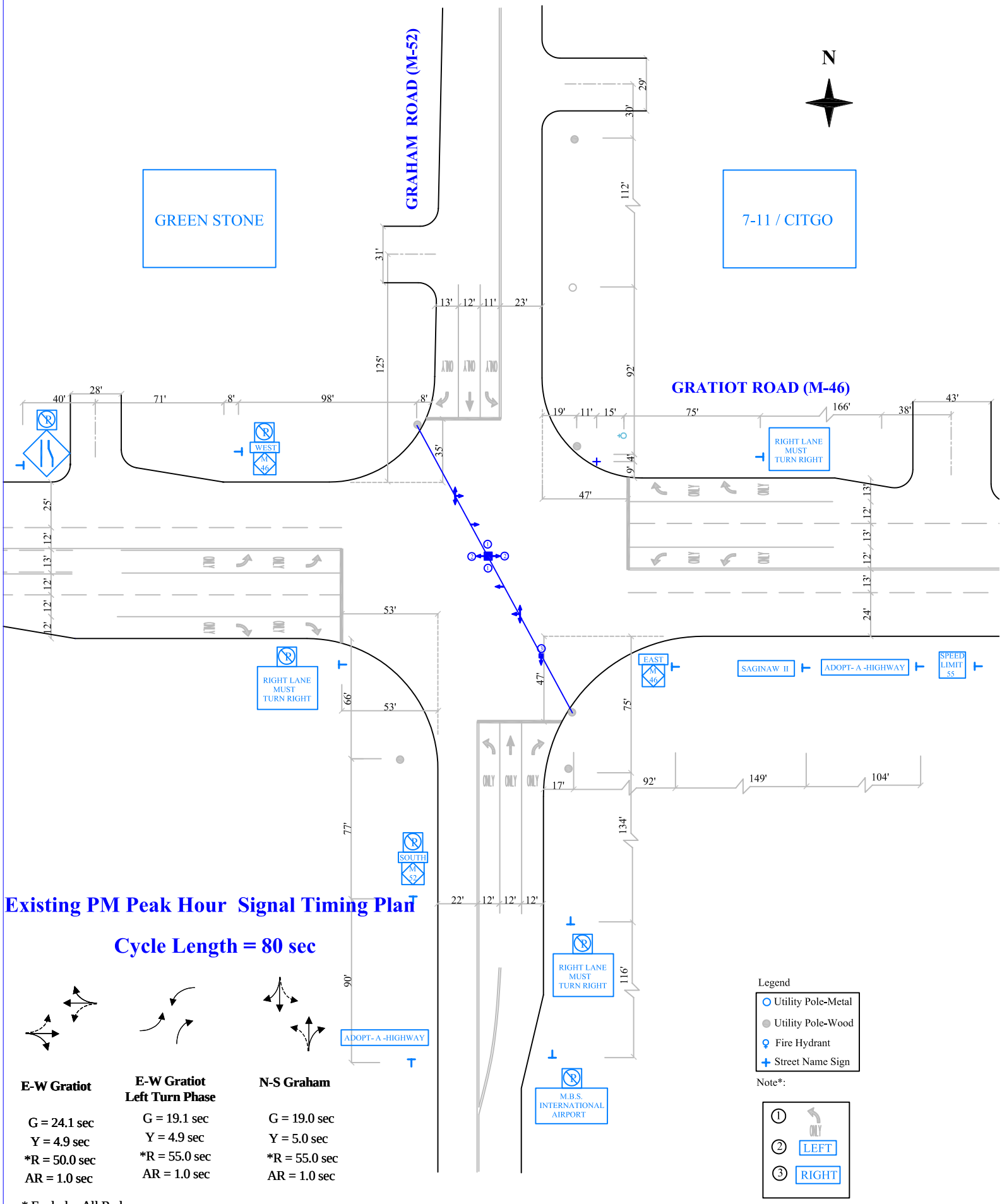


Figure 6.24.1.Existing Condition Diagram of Gratiot Road (M-46) and Graham Road (M-52) Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were manually collected on a clear day on April 11, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at suitable locations around the intersection and covering two intersecting approaches individually. The peak hourly volume from 5:00 PM to 6:00 PM is presented in Figure 6.24.2, which represents the highest peak hour (one-hour) volumes in the PM peak period. The traffic patterns indicate that the volumes in the east-west direction are relatively higher than the north-south direction. Also, the westbound left-turn and northbound right turn volumes are considerably high.

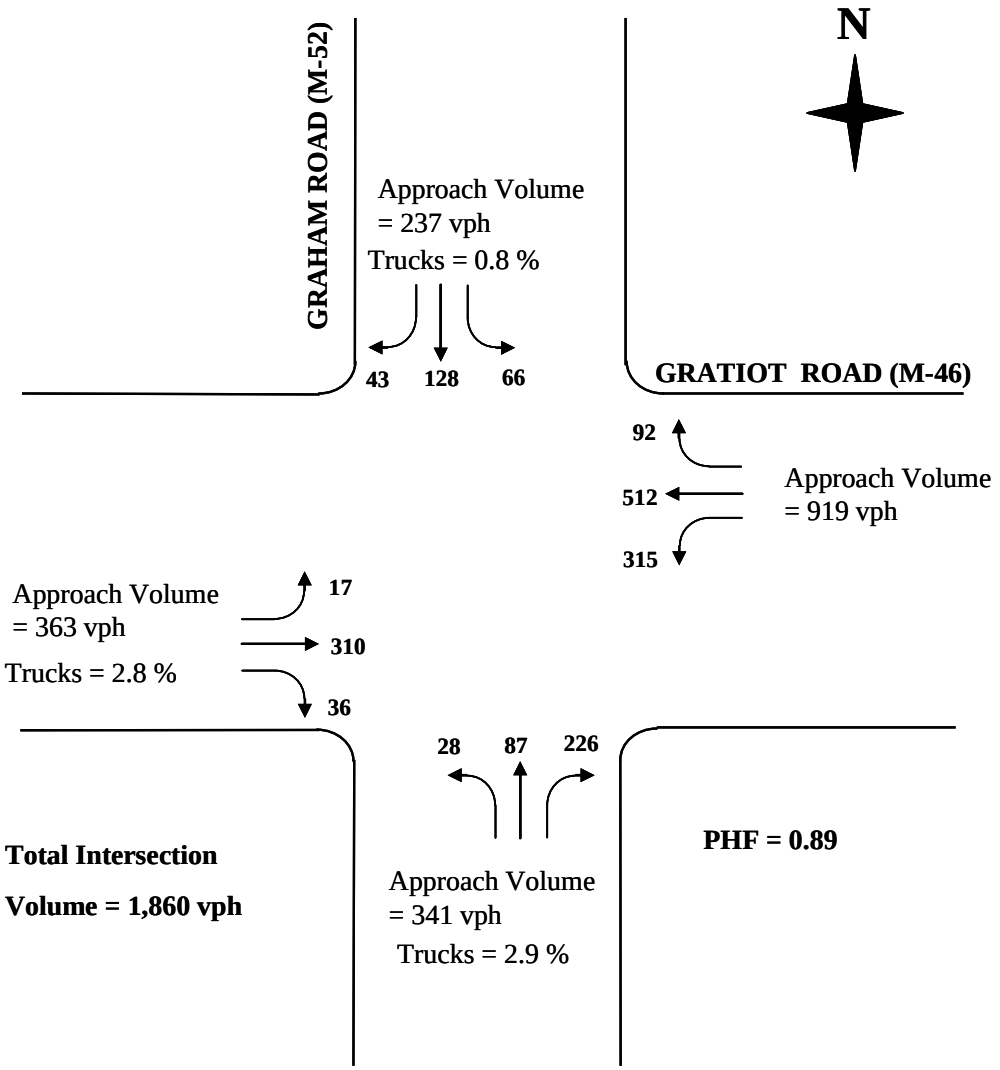


Figure 6.24.2. Peak Hour (5:00–6:00 PM) Traffic Volumes at Gratiot Road (M-46) and Graham Road (M-52)

Traffic Control

This intersection is controlled by a three-phase span-wire traffic signal with an 80-second cycle length. The existing clearance interval includes a yellow interval of 4.9 seconds and an all-red interval of 1.0 second for the east and westbound phases. For the north and southbound phase, it includes a yellow interval of 5.0 seconds and an all-red interval of 1.0 second.

Crash Analysis

A total of 20 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of ten crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for this intersection of Gratiot Road and Graham Road are shown in Figures 6.24.3 and 6.24.4 respectively.

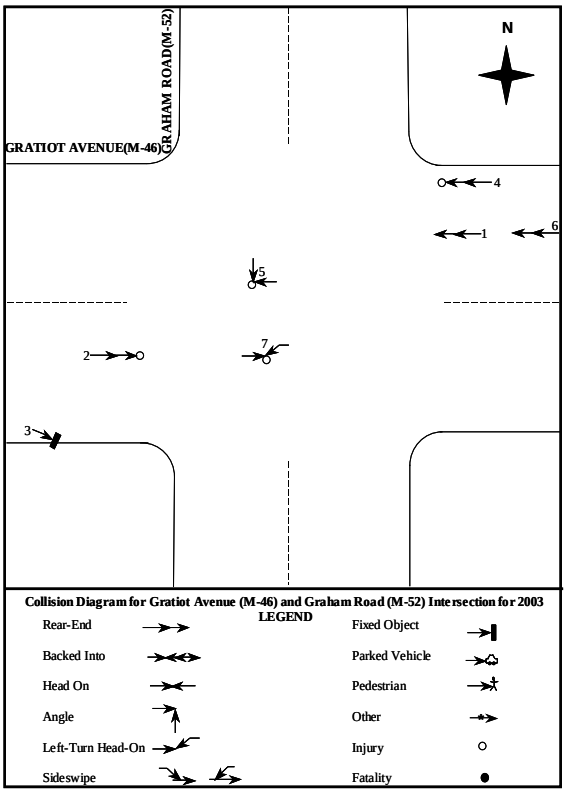


Figure 6.24.3. Collision Diagram for Gratiot Road and Graham Road for 2003

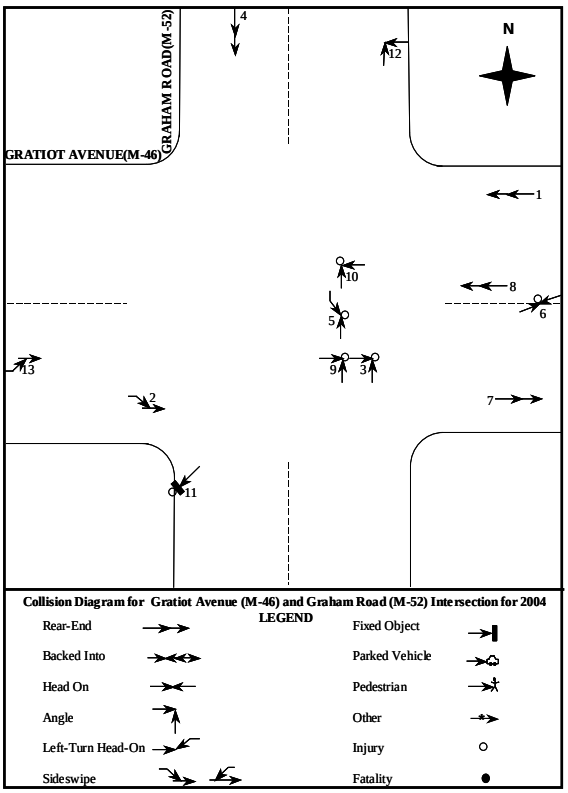


Figure 6.24.4. Collision Diagram for Gratiot Road and Graham Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.18 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The rear- end crashes occurring specifically on the east-west approaches of the intersection are of specific significance, which may have occurred due to inadequate signal timing and insufficient clearance interval. There are some angle crashes, which may have occurred due to improper clearance interval.

The crash distribution, based upon type of crashes, is shown in Figure 6.24.5. Rear-end and angle crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes include eight rear-end crashes and five angle crashes.

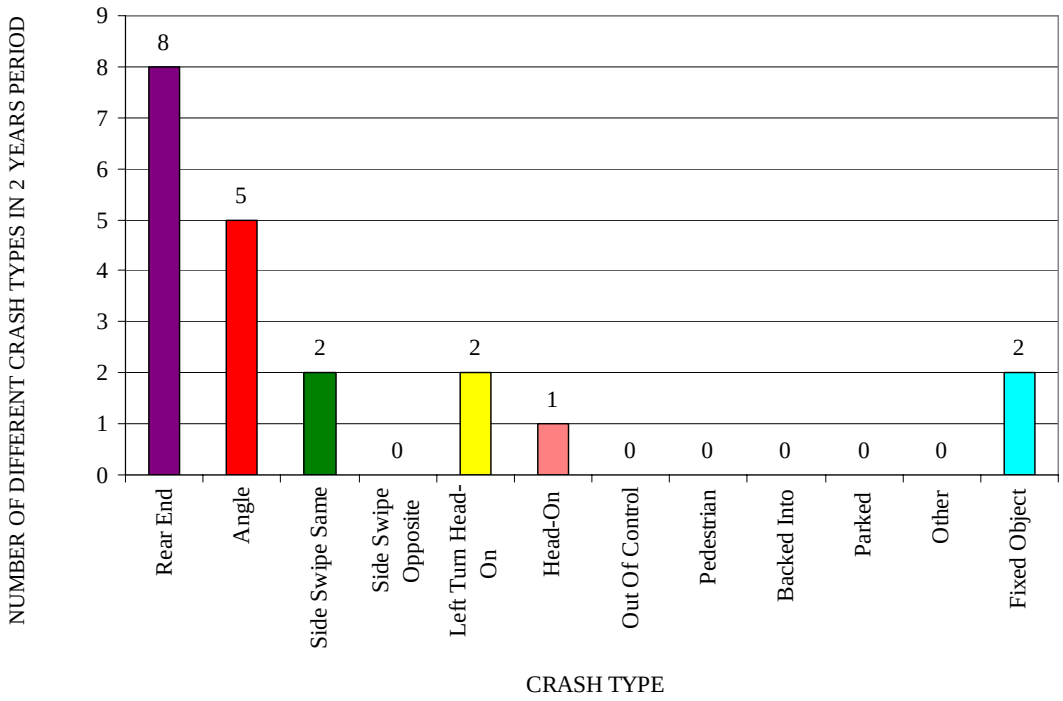


Figure 6.24.5: Two Year (2003 & 2004) Crash Experience at Gratiot Road (M-46) and Graham Road (M-52) Intersection

Safety Deficiencies and Recommended Improvements

Majority of crashes occurring at the intersection of Gratiot Road and Graham Road are of rear end, left turn head on and angle crashes which could be attributed to poor visibility

of traffic signals, improper phasing design and clearance interval inadequacies. The required clearance interval is computed and provided in the proposed signal-timing plan.

Analyzing the existing volumes at the intersection, the westbound volume is higher than the eastbound movement. The existing signal-timing plan has been modified with the addition of a split phase for westbound Tittabawassee Road and a changed cycle length of 90-second to ensure desired progression.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed intersection diagram in Figure 6.24.6.

Stop bars should be relocated for all approaches. Pavement markings should be upgraded with proper directional arrows for all approaches. Lane markings for the receiving lanes should be upgraded for all approaches as shown in the proposed diagram in Figure 6.24.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.24.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.24.1. Existing and Proposed Clearance Intervals for the Gratiot Road (M-46) and Graham Road (M-52) Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	5.0	1.0	5.0	1.5	Less rear-end, left-turn head-on and injury crashes
E-W Left-turn Phase	4.9	1.0	-	-	
E-W Through Phase	4.9	1.0	5.0	1.2	
WB Only phase	-	-	5.0	1.2	

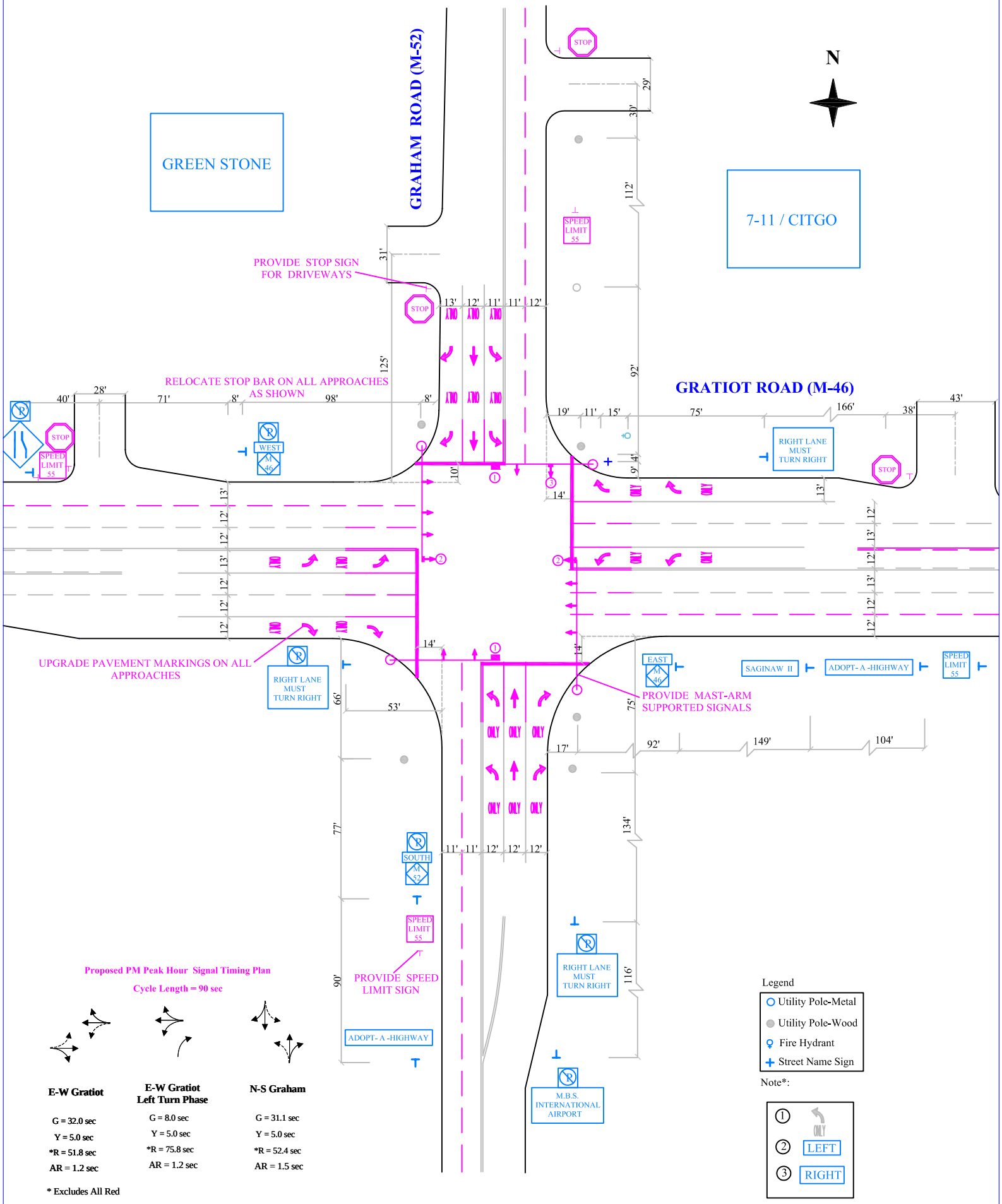
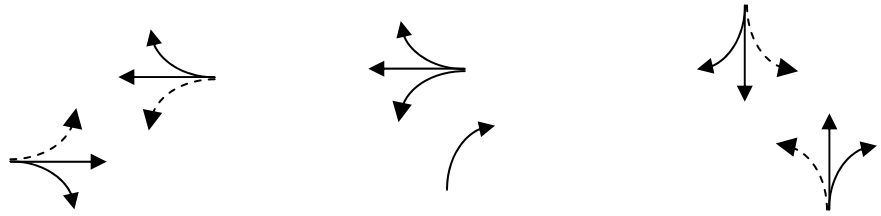


Figure 6.24.6 Proposed Diagram of Gratiot Road (M-46) and Graham Road (M-52) Intersection (Not to Scale)

The proposed PM peak period signal timing plan based on a 90 second cycle length is shown as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Gratiot	**WB Gratiot**	**N-S Graham**
G = 28.0 sec	G = 12.0 sec	G = 31.1 sec
Y = 5.0 sec	Y = 5.0 sec	Y = 5.0 sec
*R = 55.8 sec	*R = 71.8 sec	*R = 52.4 sec
AR = 1.2 sec	AR = 1.2 sec	AR = 1.5 sec

* Excludes All Red

A list of the recommended improvements and implementation stages are summarized below:

- Increase the yellow interval for Gratiot Road to 5.0 seconds
- Increase the all-red interval for Gratiot Road to 1.2 seconds and for Graham Road to 1.5 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Provide a modified three-phase signal system by introducing a split phase for westbound Gratiot Road and removing the protected left-turn phase for Gratiot Road.
- Upgrade pavement markings with proper directional arrows on all the approaches.
- Relocate the stop bar at all four approaches, as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Provide stop signs for all the driveways located near the intersection. Landowners should provide these stop signs.
- Optimize the signal timings for AM and off-peak hours.
- Implement the optimized PM signal timing.

The proposed intersection diagram in Figure 6.24.6 shows the improvements including a mast-arm signal installation, upgrading the pavement markings and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Gratiot Road (M-46) and Graham Road (M-52) indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 19.6 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will remain at B with a delay of 14.2 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic along with enhanced safety and improved progression. The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.24.2.

Table 6.24.2.
Gratiot Road (M-46) and Graham Road (M-52)
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Gratiot Road	19.2	B	21.5	C
WB Gratiot Road	16.1	B	10.3	B
NB Graham Road	27.0	C	13.4	B
SB Graham Road	24.6	C	19.5	B
Overall Intersection	**19.6**	**B**	**14.2**	**B**

25. Gratiot Road (M-46) and Kennely Road

The intersection of Gratiot Road and Kennely Road is located in the southeastern part of Thomas Township and forms a "T" intersection. Gratiot Road runs in the east-west direction and north of Gratiot Road, Kennely Road runs in the north-south direction. Kennely Road can be classified as a local street and Gratiot Road is an arterial. Gratiot Road is also known as M-46.

At the eastbound approach Gratiot Road consists of two lanes including one through lane and one shared through/left-turn lane. The westbound approach consists of two lanes including one through lane and one shared through/right-turn lane. The pavement width of Gratiot Road is approximately 68 feet including a 22 foot grass median.

Kennely Road north of the intersection with Gratiot Road is 31 feet wide with one dedicated right-turn lane, and one receiving lane.

The speed limit for Gratiot Road is 45 miles per hour and for Kennely Road is 35 miles per hour.

Land Use

The intersection of Gratiot Road and Kennely Road has a La Salle Bank on northwest corner, and a community Baptist Church on northeast corner. The southern side of the intersection is occupied by residential homes.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Gratiot Road (M-46) and Kennely Road.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.25.1. Details of the intersection approaches are shown in photographs 25a through 25c.

Photograph 25a. EB Gratiot Road at Kennely Road

Photograph 25b. WB Gratiot Road at Kennely Road

Photograph 25c. SB Kennely Road at Gratiot Road

Peak Hour Volumes

PM peak period traffic volumes were manually collected on a clear day on April 11, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the

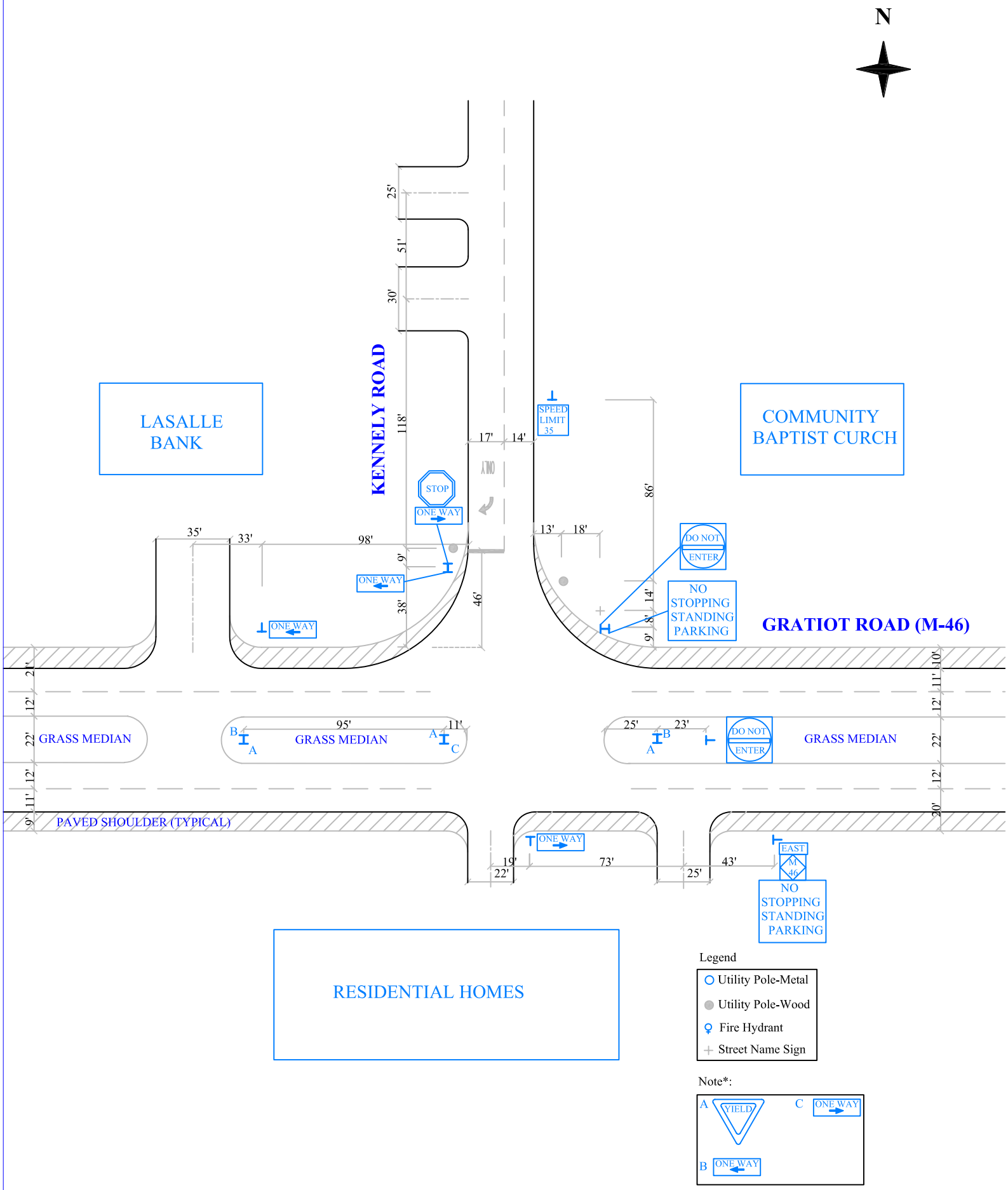


FIgure 6.25.1. Existing Condition Diagram of Gratiot Road (M-46) and Kennely Road Intersection (Not to Scale)

intersection and covering all intersecting approaches. The peak hourly volume occurred from 4:30 PM to 5:30 PM and is presented in Figure 6.25.2. The traffic patterns indicate that volumes of Gratiot Road are considerably higher than Kennely Road.

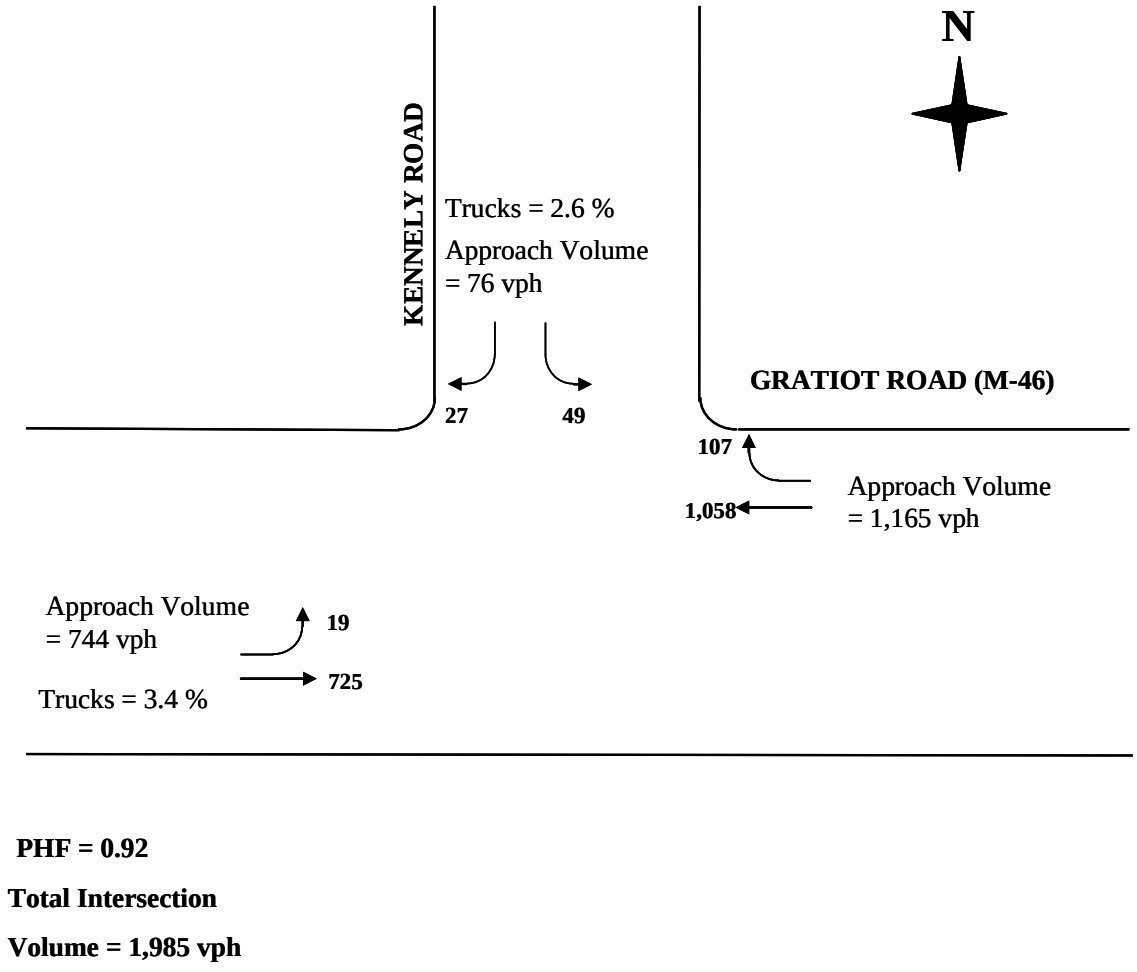


Figure 6.25.2. Existing PM Peak Hour (4:30 –5:30 PM) Traffic Volumes for the Intersection of Gratiot Road (M-46) and Kennely Road

Traffic Control

The intersection of Gratiot Road and Kennely Road is a one-way stop-controlled intersection. Gratiot Road has the right-of-way while Kennely Road is controlled by a stop sign. There are yield signs for the eastbound and westbound traffic of Gratiot Road.

Crash Analysis

A total of 16 crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of eight crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. Figures 6.25.3 and 6.25.4 show the collision diagrams for these years respectively.

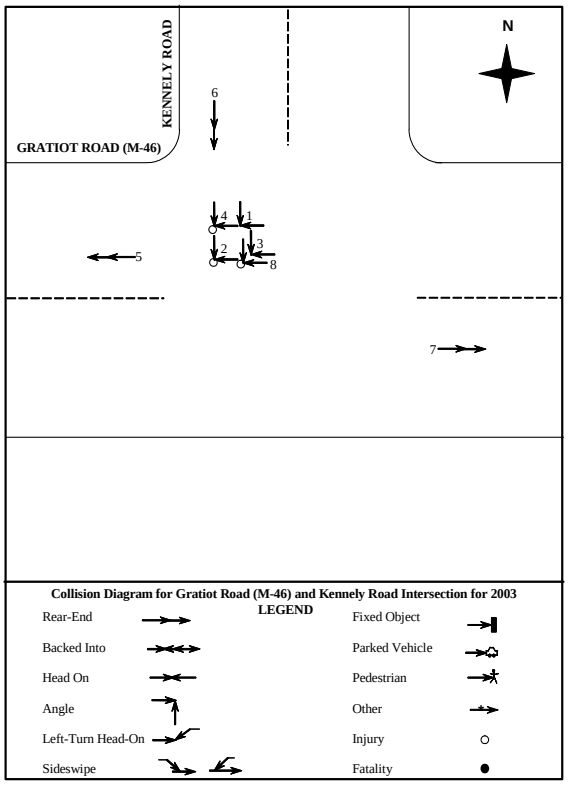


Figure 6.25.3. Collision Diagram for Gratiot Road and Kennely Road for 2003

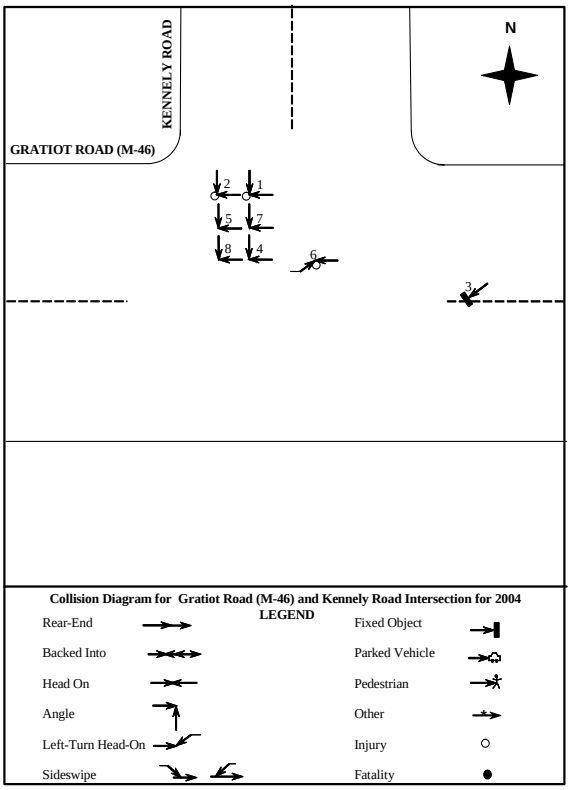


Figure 6.25.4. Collision Diagram for Gratiot Road and Kennely Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.88/million entering vehicles, which is

nominal, considering the approach volume at the intersection. There is considerable number of angle crashes at the intersection, possibly due to the one-way stop control.

Safety Deficiencies and Recommended Improvements

Frequently occurring angle crashes at the intersection of Gratiot Road and Kennely Road could be due to inadequate gap for the minor road traffic and high approach speed along Gratiot Road. In order to mitigate this problem, this intersection should be checked for signal warrants. Considering the right angle crash trends a semi-actuated signal can be installed to solve the problem. This will also increase the level of service of Kennely Road.

Apart from the planning for safety improvements, the pavement markings should be improved to provide better guidance to the drivers. Stop bars should be provided for eastbound and westbound traffic. An exclusive left turn lane should be provided for Kennely Road using the existing pavement width. Also, lane use arrow markings with lane use control word "ONLY" should be provided in pairs for the southbound approach of Kennely Road for the left-turn and right-turn lane to prevent entrapment and to help road users select the appropriate lane in advance of reaching the stop bar. Additionally solid yellow line demarcation should be provided to separate the outgoing and incoming traffic for the southbound approach of Kennely Road.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

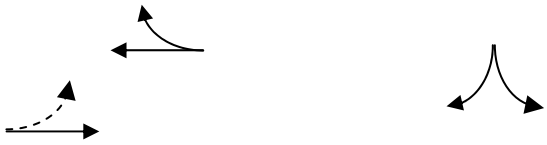
Also, it is proposed to provide a semi-actuated signal with a cycle length of 90 seconds.
The proposed yellow and all-red intervals are shown in Table 6.25.1

Table 6.25.1. Proposed Clearance Intervals for the Gratiot Road and Kennely Road Intersection

Interval/Phase	Proposed Interval Length (sec)		Potential Benefit
	Yellow	**All-Red**	
E-W Through Phase	4.3	1.3	Less rear-end, left-turn head-on and injury crashes
SB- Split Phase	3.6	1.8	

The proposed PM peak period signal timing plan based on a 90-second cycle length is shown below.

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



E-W Gratiot	**SB Kennely**
G = 53.4	G = 25.6
Y = 4.3	Y = 3.6
*R = 31.0	*R = 59.0
AR = 1.3	AR = 1.8

* Excludes All

A list of the recommended improvements is summarized below:

- Provide a two-phase semi-actuated signal with a cycle length of 90 seconds for the PM peak period.
- Provide mast-arm signal installation.
- Provide a yellow interval of 4.3 seconds for Gratiot Road and 3.6 seconds for Kennely Road.
- Provide an all-red interval of 1.3 seconds for Gratiot Road and 1.8 seconds for Kennely Road.
- Provide stop bars for the eastbound and westbound approaches of Gratiot Road.
- Relocate the stop bar closer to the intersection for southbound Kennely Road.
- Provide an exclusive left-turn lane for southbound traffic using the existing width of the pavement.
- Provide two sets of left-turn only and right-turn only arrows for the south-bound approach.
- Provide solid yellow line demarcation to separate the outgoing and incoming traffic for the south-bound approach of Kennely Road.

- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection.
- Re-stripe the existing lane markings on all approaches.
- Implement optimized PM signal timing

The proposed condition diagram in Figure 6.25.5 shows the recommended improvements

Operational Evaluation

The results of the level of service analysis for the intersection of Gratiot Road and Kennely Road indicate that, under existing conditions during the PM peak hour, southbound level of service is F with a delay of 81.9 seconds per vehicle.

After the installation of semi-actuated signal the level of service of Kennely Road is expected to improve to C with a delay of 24.4 seconds per vehicle and the overall intersection level of service will improve to A with a delay of 10 seconds per vehicle.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.25.2.

Table 6.25.2.
Gratiot Road and Kennely Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Gratiot Road	-	-	5.7	A
WB Gratiot Road	-	-	11.8	B
SB Kennely Road	81.9	F	24.4	C
Overall Intersection	**81.9**	**F**	**10.0**	**A**

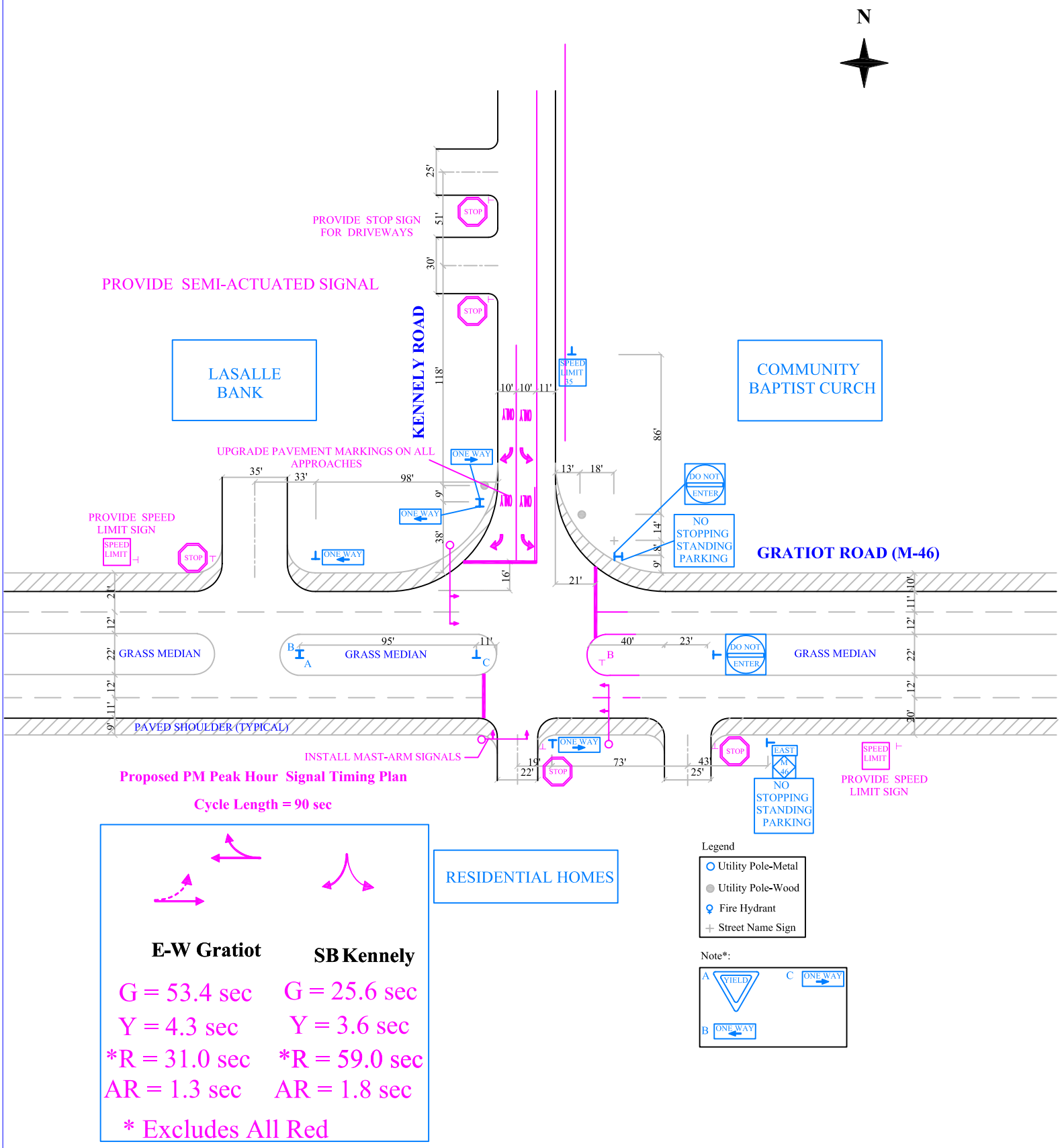


Figure 6.25.6 Proposed Diagram of Gratiot Road (M-46) and Kennely Road Intersection (Not to Scale)

26. Gratiot Road (M-46) and Van Wormer Road

The intersection of Gratiot Road (M-46) and Van Wormer Road is located in the southern part of Thomas Township and forms a T-shaped intersection. Gratiot Road runs in the east-west direction and to the south of Gratiot Road, Van Wormer Road runs in the north-south direction. Gratiot Road (M-46) is a state trunk line and Van Wormer Road can be classified as a local street.

Gratiot Road consists of two lanes including one through lane and one shared through and right turn lane for both approaches. The pavement width of Gratiot Road is 79 feet including a 31 foot grass median.

Van Wormer Road consists of two lanes, one for outgoing traffic and one for incoming traffic. The pavement width of Van Wormer Road is 24 feet.

The speed limit for Gratiot Road is 45 miles per hour and Van Wormer Road is 40 miles per hour.

Land Use

The intersection of Gratiot Road and Van Wormer Road has an Ascenscon Lutheran Church on the southwest corner and a Home Depot and Carter Lumber on the northern side. There is a 35 foot wide driveway serving the vehicles going to and from the Home Depot.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Gratiot Road (M-46) and Van Wormer Road.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.26.1.

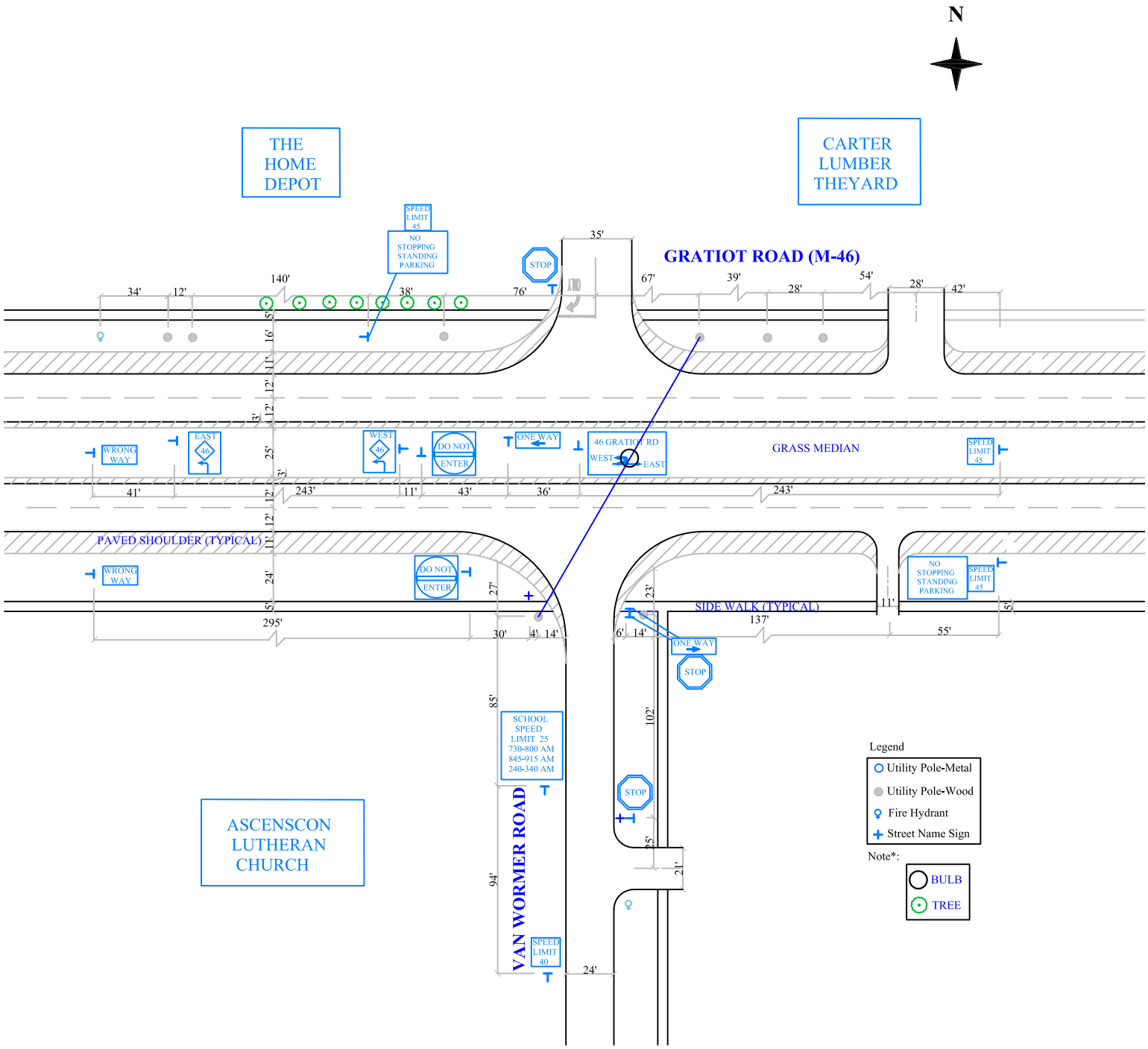


Figure 6.26.1. Existing Condition Diagram of Gratiot Road (M-46) and Van wormer Road Intersection (Not to Scale)

Details of the intersection approaches are shown in photographs 26a through 26d. Photograph 26b shows the stop controlled driveway for the Home depot.

Photograph 26a. NB Van Wormer Road at Gratiot Road

Photograph 26b. Home Depot Driveway at Gratiot Road

Photograph 26c. EB Gratiot Road at Van Wormer Road

Photograph 26d. WB Gratiot Road at Van Wormer Road

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on April 11, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other and covering all the approaches. The peak hourly volume from 4:30 PM to 5:30 PM is presented in Figure 6.26.2. The traffic patterns indicate that volumes along Gratiot Road are much higher compared to the volumes along Van Wormer Road and the Home Depot driveway.

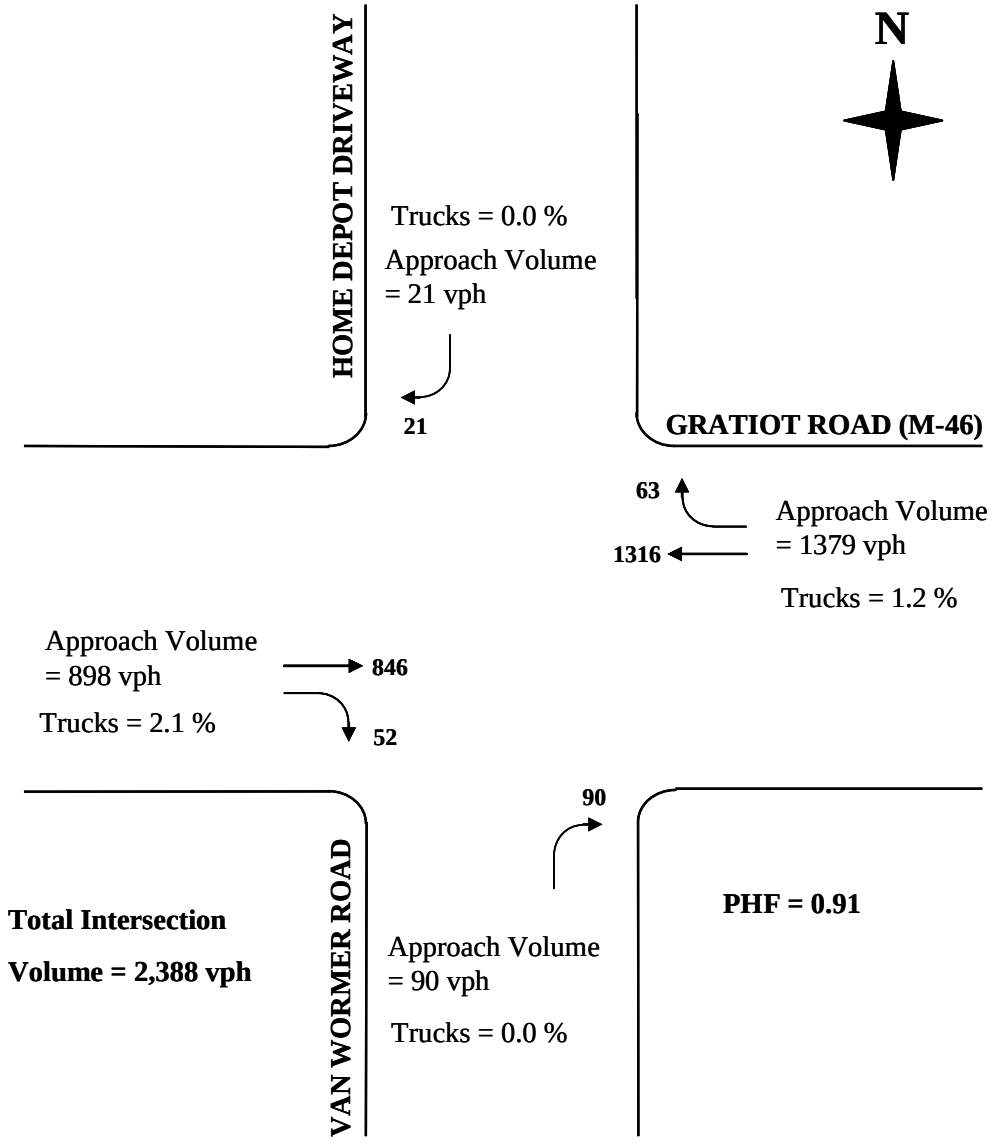


Figure 6.26.2. Existing PM Peak Hour (4:30 –5:30 PM) Traffic Volumes for the Intersection of Gratiot Road (M-46) and Van Wormer Road

Traffic Control

This intersection of Gratiot Road and Van Wormer Road is a two-way stop controlled intersection. Gratiot Road has the right-of-way while the Van Wormer Road and the Home Depot driveway are controlled by stop signs.

Crash Analysis

A total of six crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of three crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams prepared for the years 2003 and 2004 for the intersection of Gratiot Road and Van Wormer Road are shown in Figures 6.26.3 and 6.26.4. respectively.

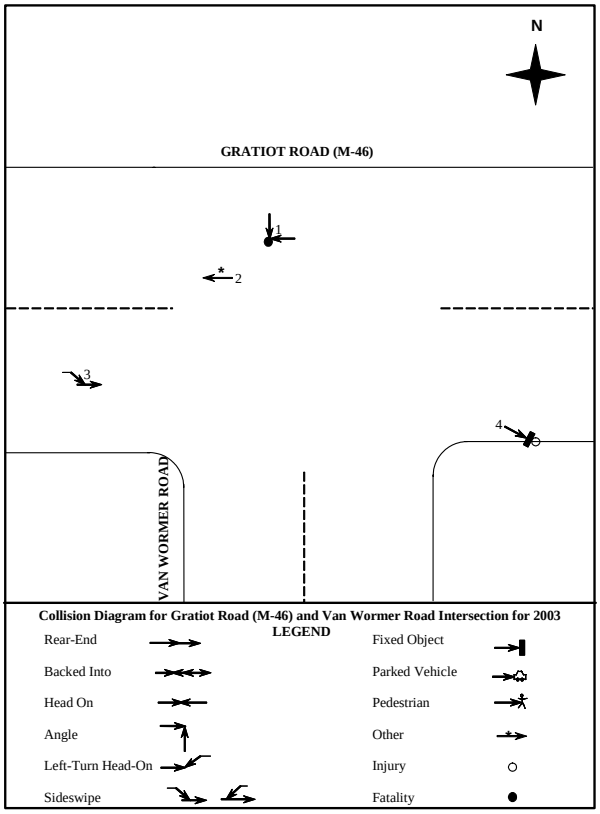


Figure 6.26.3. Collision Diagram for Gratiot Road and Van Wormer Road for 2003

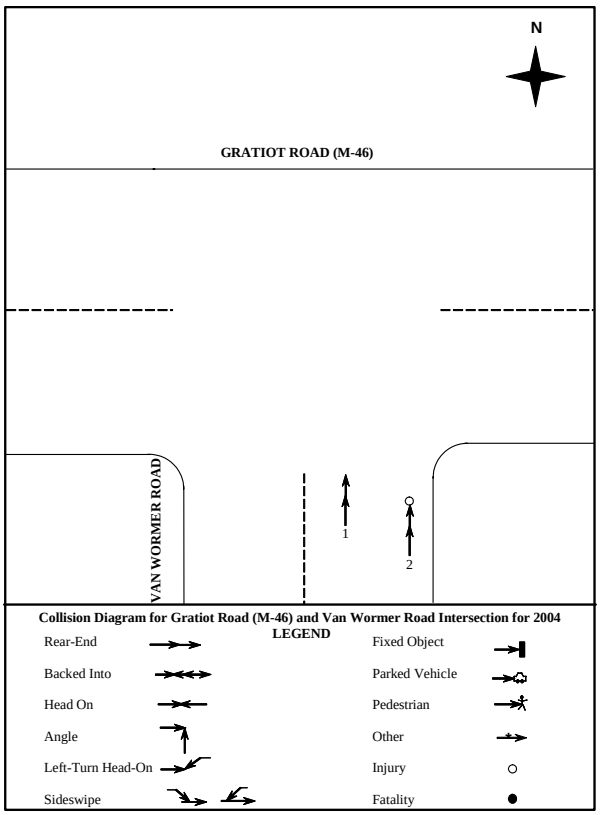


Figure 6.26.4. Collision Diagram for Gratiot Road and Van Wormer Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.27 per million entering vehicles, which is nominal, considering the approach volume at the intersection.

Safety Deficiencies and Recommended Improvements

Crashes occurring at the Gratiot Road (M-46) and Van Wormer Road intersection could be attributed to poor visibility, inadequate gap for the minor road traffic and high approach speed along Gratiot Road.

A stop bar and crosswalk should be provided for the northbound approach. Relocate the stop bar and provide a crosswalk for southbound Home Depot driveway. The pavement markings should be upgraded for all approaches. Advance warning signs should be provided along Van Wormer Road.

A list of the recommended improvements is summarized below:

- Install advance warning sign indicating “Stop Ahead” along Van Wormer Road.
- Provide upgraded pavement markings with proper directional arrows for all the approaches.
- Provide a stop bar and crosswalk for the northbound approach.
- Provide solid yellow line demarcation to separate the outgoing and incoming traffic for the northbound approach of Van Wormer Road.
- Relocate the stop bar and provide a crosswalk for southbound Home Depot driveway as per MMUTCD guidelines.
- Provide stop signs for all the driveways located near the intersection.

The proposed diagram in Figure 6.26.5 shows the recommended improvements.

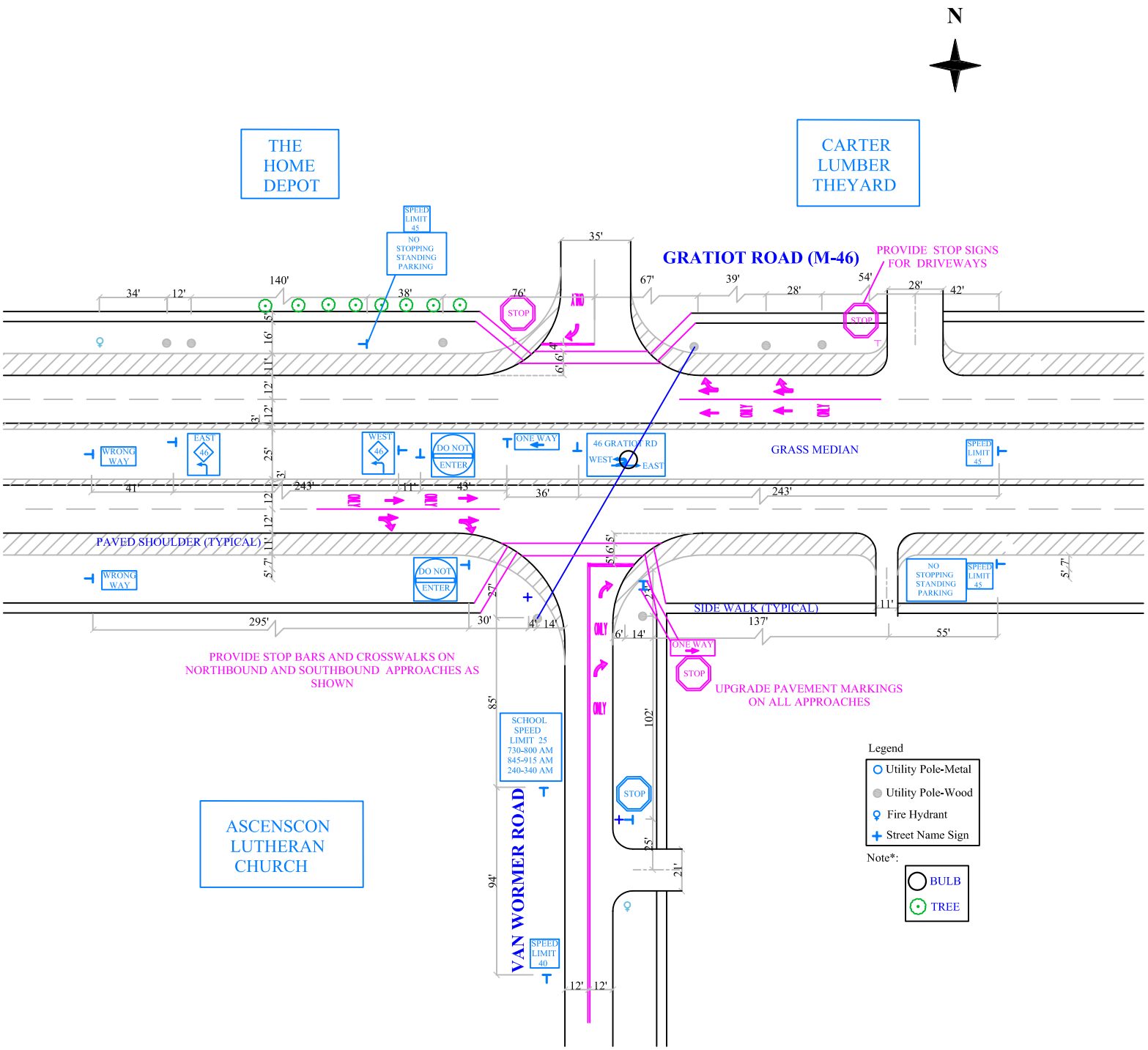


Figure 6.26.6 Proposed Diagram of Gratiot Road (M-46) and Van Wormer Road Intersection (Not to Scale)

Operational Evaluation

The results of the level of service analysis for the intersection of Gratiot Road and Van Wormer Road indicate that, under existing conditions during the PM peak hour, northbound Van Wormer Road has a level of service B with a delay of 13.4 seconds per vehicle and southbound Home Depot driveway has a level of service C with a delay of 15.9 seconds per vehicle.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.26.2.

Table 6.26.2
Gratiot Road and Van Wormer Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions	
	Delay (Sec/Vehicle)	LOS
EB Gratiot Road	NA	A
WB Gratiot Road	NA	A
NB Van Wormer Road	13.4	B
SB Home Depot Driveway	15.9	C
Overall Intersection	**NA**	**NA**

27. Gratiot Road (M-46) and Miller Road

The intersection of Gratiot Road (M-46) and Miller Road is located on the eastern side of Thomas Township. Miller Road runs in the north-south direction and Gratiot Road runs in the east-west direction. Gratiot road is a state trunk line (M-46) and Miller Road can be classified as an arterial street.

Gratiot Road consists of one exclusive left-turn lane, one through lane and one shared through and right turn lane on both eastbound and westbound approaches. The pavement width of Gratiot Road is 60 feet.

Miller Road consists of one exclusive left-turn lane, one through lane and one dedicated right-turn lane on both northbound and southbound approaches. Miller Road is 47 feet wide to the north of the intersection and to the south it is 44 feet wide.

The speed limit for Gratiot Road (M-46) is 45 miles per hour and for Miller Road is 40 miles per hour.

Land Use

The intersection of Gratiot Road and Miller Road, is occupied by Taco Bell on the northwest corner, Citgo Gas Station and Nascar on northeast corner, Tuffy Service Center, Wendys and Print Express on the southeast corner and Little Caesars on the southwest corner. Additionally, the northeast and southwest corners are also occupied by some residential homes. There are a considerable number of driveways serving vehicles going to and from these establishments and residential homes.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Gratiot Road and Miller Road.

The existing characteristics of the intersection including traffic control and surrounding land uses are schematically shown in Figure 6.27.1. Details of the intersection approaches are shown in photographs 27a through 27d. Photographs show the existing mast-arm signal installation along with the existing pavement markings. There are several driveways very close to the intersection as shown by these photographs.

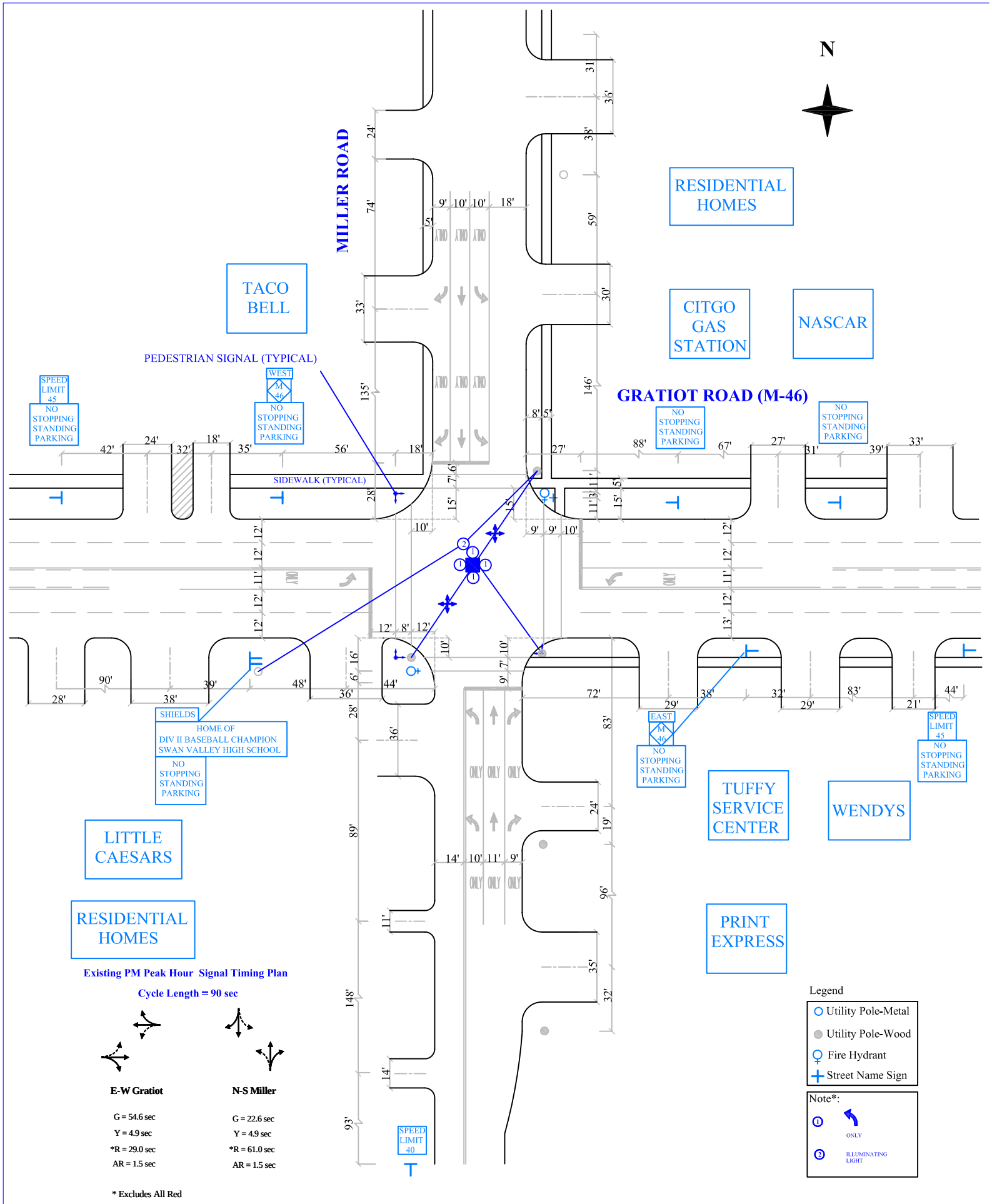


Figure 6.27.1.Existing Condition Diagram of Gratiot Road (M-46) and Miller Road Intersection (Not to Scale)

Photograph 27a. EB Gratiot Road at Miller Road

Photograph 27b. WB Gratiot Road at Miller Road

Photograph 27c. NB Miller Road at Gratiot Road

Photograph 27d. SB Miller Road at Gratiot Road

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on April 11, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches individually. The peak hour volume occurred from 4:30 PM to 5:30 PM is presented in Figure 6.27.2. The traffic patterns indicate that approach volumes along Gratiot Road are much higher compared to the volumes along Miller Road.

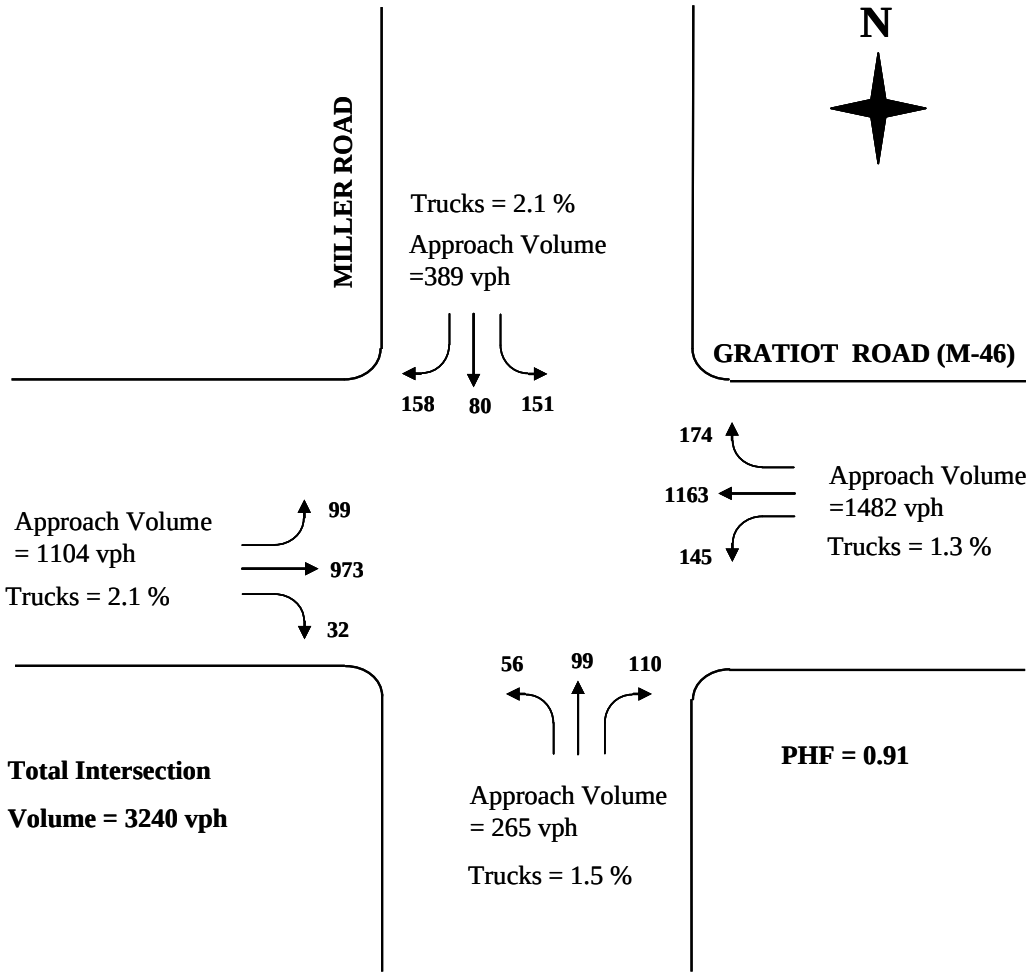


Figure 6.27.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes for the Intersection of Gratiot Road (M-46) and Miller Road

Traffic Control

This intersection is controlled by a two-phase traffic signal with a 90-second cycle length. The existing clearance interval includes a yellow interval of 4.9 seconds and an all-red interval of 1.5 seconds for all phases in all directions. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.27.1.

Crash Analysis

A total of 40 crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of 20 crashes. Collision diagrams were prepared based on the UD-10

reports information to identify crash patterns. The collision diagrams prepared for the years 2003 and 2004 for the intersection of Gratiot Road and Miller Road are shown as in Figure 6.27.3 and 6.27.4.

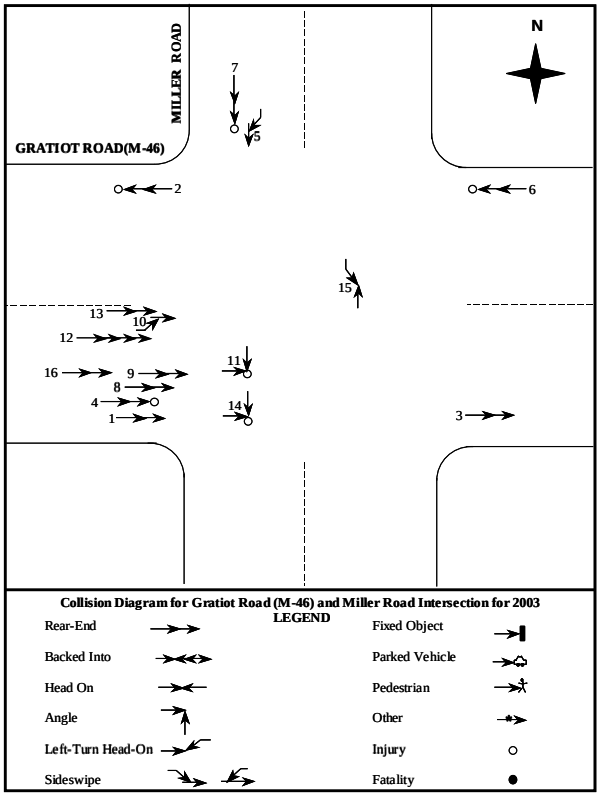


Figure 6.27.3. Collision Diagram for Gratiot Road and Miller Road for 2003

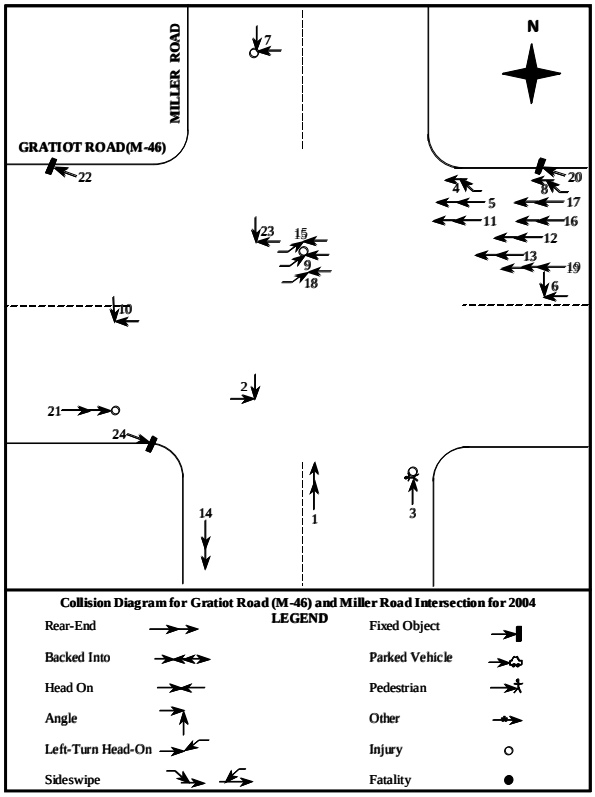


Figure 6.27.4. Collision Diagram for Gratiot Road and Miller Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.35 per million entering vehicles, which is nominal, considering the approach volume at the intersection. There are a significant number of crashes occurring at the intersection as seen from the collision diagrams. Rear-end crashes are of specific significance, which may have occurred due to inadequate signal timing and insufficient clearance intervals. Most of the rear-end crashes, which occurred at the eastbound and westbound approaches of Gratiot Road may be due to the traffic going into and out of the driveways, located close to the intersection. Also, the intersection has a considerable number of

left-turn head-on and angle crashes at the intersection which may also be due to the inadequate clearance intervals and signal timing plan.

The crash distribution, based upon type of crashes, is shown in Figure 6.27.5. Rear-end (21) crashes and angle (7) crashes are the predominant crashes reported at this intersection.

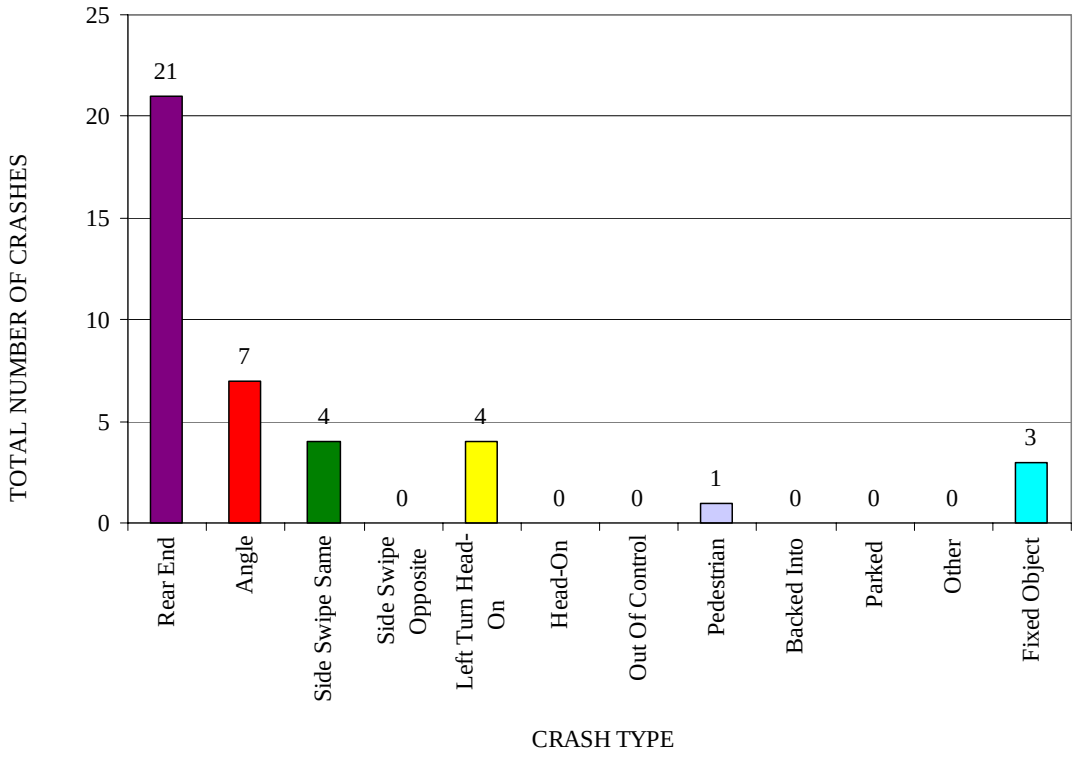


Figure 6.27.5: Two year (2003 & 2004) crash experience at Gratiot Road (M-46) and Miller Road Intersection

Safety Deficiencies and Recommended Improvements

Majority of crashes at the intersection of Gratiot Road (M-46) and Miller Road could be attributed to poor visibility of traffic signals and inadequate clearance intervals. Movements into and out of the driveways located close to the intersection may also be a potential cause for the frequent rear-end crashes at this intersection.

Adequate clearance intervals have been provided and signal timings are modified with a changed cycle length of 90-second to attain progression along the Gratiot corridor. An analysis of the left-turn warrants indicate that an exclusive left-turn phase is warranted for the east-west left-turn movement. So the proposed signal timing plan consists of three phases including a protected left-turn phase.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.27.6.

Post-mounted traffic signals with a "LEFT" illuminated sign should be provided at the northeast and southwest corners of the intersection to assist eastbound and westbound left turning vehicles. Stop bars and crosswalks should be relocated on the all approaches as per MMUTCD guidelines. Pavement markings should be upgraded for all approaches with proper directional arrows. Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.27.1, along with the proposed lengths of these intervals and their potential benefits.

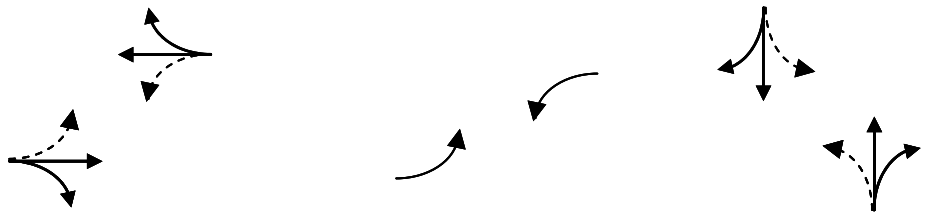
Table 6.27.1. Existing and Proposed Clearance Intervals for the Gratiot Road (M-46) and Miller Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.9	1.5	4.3	1.4	Less rear-end, angle and injury crashes.
E-W Left Turn Phase	-	-	4.3	1.4	
N-S Through Phase	4.9	1.5	3.9	1.9	

The proposed PM peak period signal timing plan with a 90 second cycle length is shown below:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Gratiot	**E-W Gratiot**	**N-S Miller**
G = 44.3 sec	G = 6.3 sec	G = 22.2 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 3.9 sec
*R = 40.0 sec	*R = 78.0 sec	*R = 62.0 sec
AR = 1.4 sec	AR = 1.4 sec	AR = 1.9 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Gratiot Road to 4.3 seconds and Miller Road to 3.9 seconds.
- Increase the all-red interval for Miller Road to 1.9 seconds and reduce it to 1.4 seconds for Gratiot Road.
- Provide an exclusive left-turn phase for the east-west movement.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Relocate stop bars and crosswalks at all the approaches, as per MMUTCD guidelines.
- Restripe the pavement markings with proper directional arrows for all approaches.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for east and westbound left-turns.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all driveways located near the intersection.
- Optimize the signal timings for AM peak and off-peak hours and implement optimized signal timing for PM peak hour.

The proposed diagram in Figure 6.27.5 shows the recommended improvements

Operational Evaluation

The results of the level of service analysis for the intersection of Gratiot Road (M-46) and Miller Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 15.8 seconds per vehicle.

With the modifications to the signal timings, phasing and the noted improvements, the overall intersection level of service will remain B with an increased delay of 19.1 seconds per vehicle. Although the overall delay has slightly increased due to the inclusion of a protected left-turn phase for the east-west movements and consistent cycle length along the G corridor, the effect of this marginally increased delay is acceptable due to the achieved progression and enhanced safety for traffic.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.27.2.

Table 6.27.2.
Gratiot Road (M-46) and Miller Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Gratiot Road	12.9	B	21.2	C
WB Gratiot Road	12.9	B	13.2	B
NB Miller Road	26.2	C	27.1	C
SB Miller Road	29.6	C	30.9	C
Overall Intersection	**15.8**	**B**	**19.1**	**B**

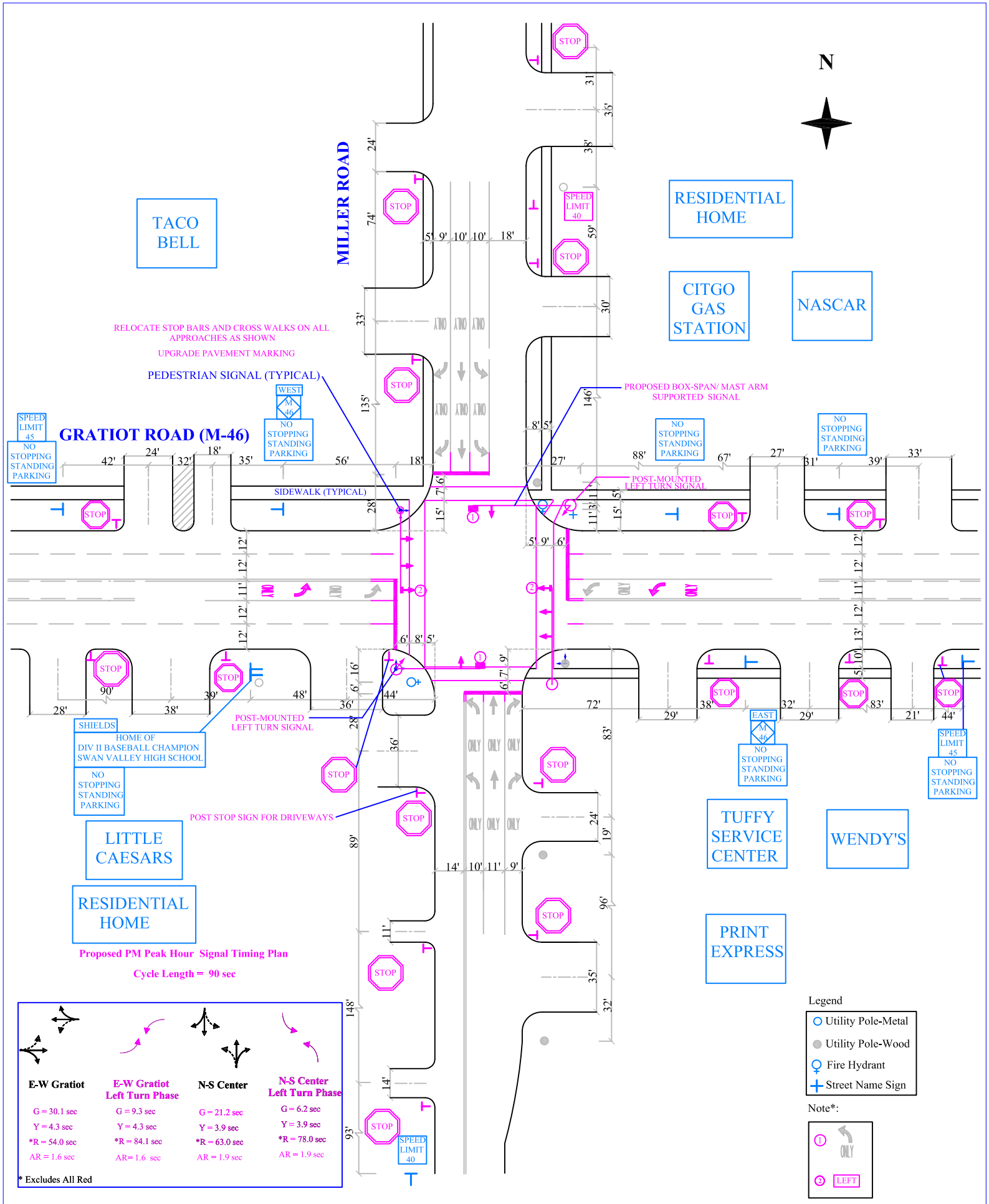


Figure 6.27.5 Proposed Diagram of Gratiot Road (M-46) and Miller Road Intersection (Not to Scale)

28. Gratiot Road (M-46) and River Road

The intersection of Gratiot Road (M-46) and River Road is located in the southeastern part of Thomas Township, close to Tittabawassee River. Gratiot Road (M-46) runs along the north-south direction and River Road runs in the east-west direction. Gratiot Road is a state trunk line (M-46) and River Road serves as an arterial road and runs parallel to the Tittabawassee River

Gratiot Road consists of one exclusive left-turn lane, one through lane and one shared through and right turn lane on both the approaches. The pavement width of Gratiot Road is 59 feet.

River Road to the north of the intersection with Gratiot Road consists of one exclusive left-turn lane and one shared through and right turn lane and to the south it consists of one exclusive left turn lane, one through lane and one dedicated right turn lane. River Road is 38 feet wide to the north of the intersection with Gratiot Road and to the south it is 48 feet wide.

The speed limit for Gratiot Road and River Road is 45 miles per hour.

Land Use

The intersection of Gratiot Road (M-46) and River Road has a Family Video on the southeast corner, a gas station on the southwest corner, a Jack's Fruit and Meat Market on the northwest corner and a drive thru party store and residential homes on the northeast corner. There are multiple driveways close to the intersection serving the vehicles going to and from these establishments and residential homes.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Gratiot Road and River Road.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.28.1. Details of the intersection approaches are shown in photographs 28a through 28d. Photographs 28a and 28b show the north and southbound approaches of River Road with driveways close to the intersection.

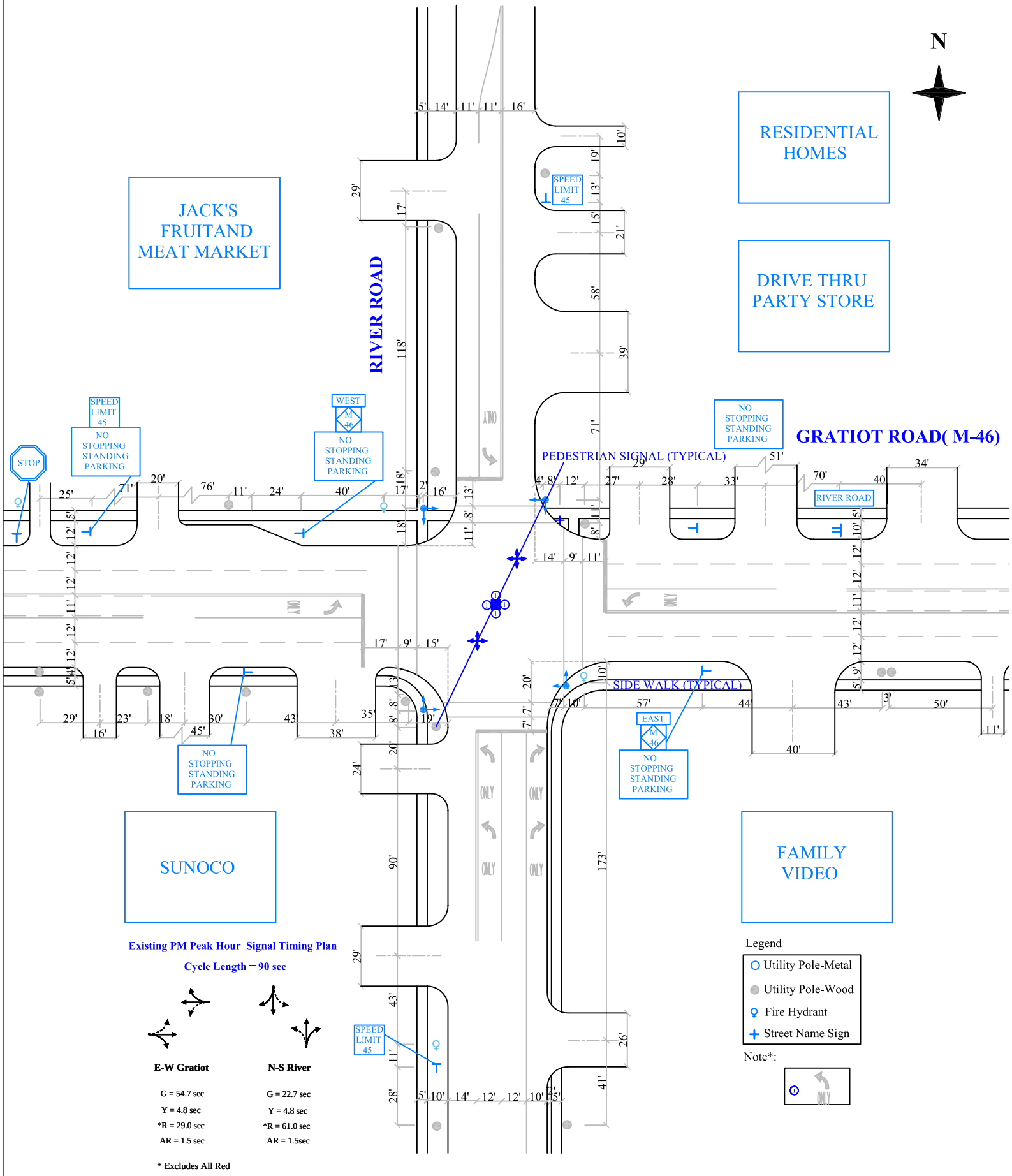


Figure 6.28.1.Existing Condition Diagram of Gratiot Road (M-46) and River Road Intersection (Not to Scale)

Photographs 28c and 28d show the east and westbound approaches with the pavement markings and existing span-wire installation of signals.

Photograph 28a. NB River Road at Gratiot Road

Photograph 28b. SB River Road at Gratiot Road

Photograph 28c. EB Gratiot Road at River Road

Photograph 28d. WB Gratiot Road at River Road

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on a clear day on April 11, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other and covering two intersecting approaches individually. The peak hourly volume occurred from 4:45 to 5:45 PM and is presented in Figure 6.28.2. The traffic patterns indicate that the volumes in the east-west direction are considerably higher as compared to the north-south direction.

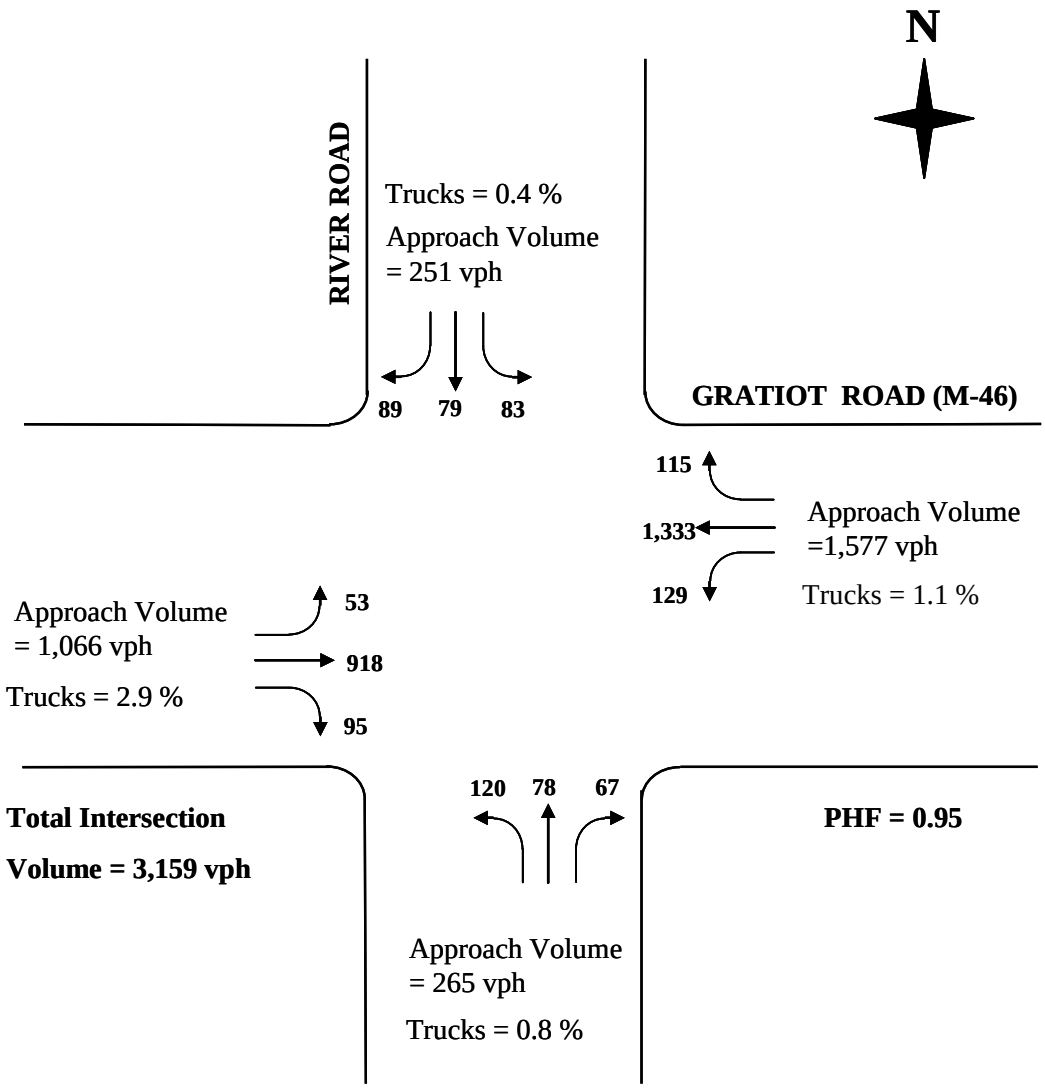


Figure 6.28.2. Existing PM Peak Hour (4:45-5:45 PM) Traffic Volumes for the Intersection of Gratiot Road (M-46) and River Road

Traffic Control

The intersection of Gratiot Road and River Road is controlled by a two-phase traffic signal with a 90-second cycle length. The existing clearance interval includes a yellow interval of 4.8 seconds and an all-red interval of 1.5 seconds for north-south and east-west phases. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.28.1.

Crash Analysis

A total of 38 crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of 19 crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Gratiot Road (M-46) and River Road are shown in Figures 6.28.3 and 6.28.4 respectively.

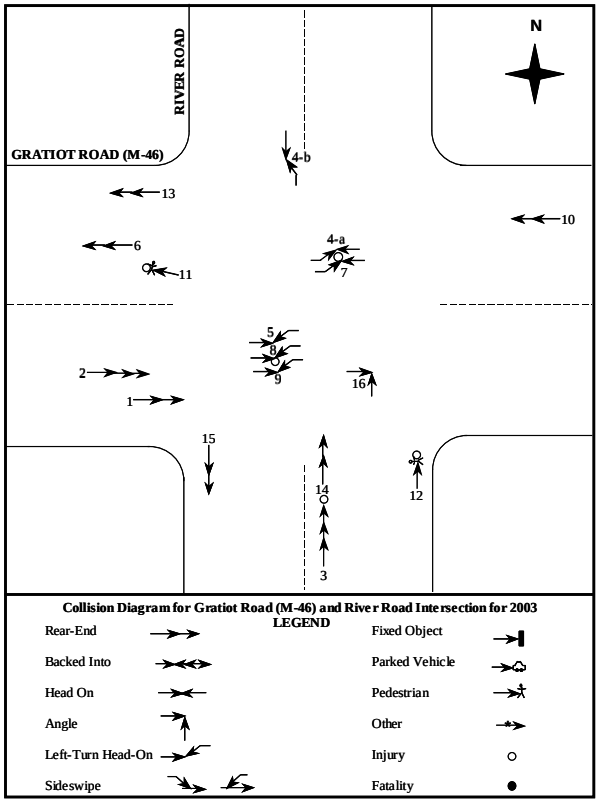


Figure 6.28.3. Collision Diagram for Gratiot Road and River Road for 2003

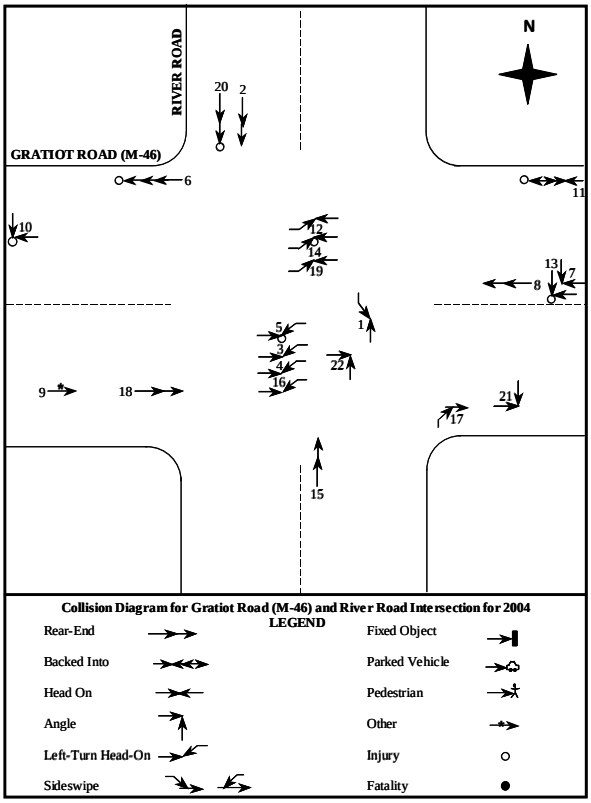


Figure 6.28.4. Collision Diagram for Gratiot Road and River Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.32 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The left-turn head-on crashes are of specific significance, which may have occurred due to heavy volume of oncoming through traffic during the permissive phase and the high speed of vehicles along Gratiot Road. Rear-end crashes are also quite significant in number, which may have occurred due to inadequate signal

timing and insufficient clearance intervals. There are some angle crashes, which may have occurred due to improper clearance intervals.

The crash distribution, based upon type of crashes, is shown in Figure 6.28.5. Rear-end, left-turn head-on and angle is the predominant crashes reported at this intersection. The corresponding two-year crash frequencies for these crashes include 14 rear-end crashes, 13 left-turn head-on crashes and six angle crashes.

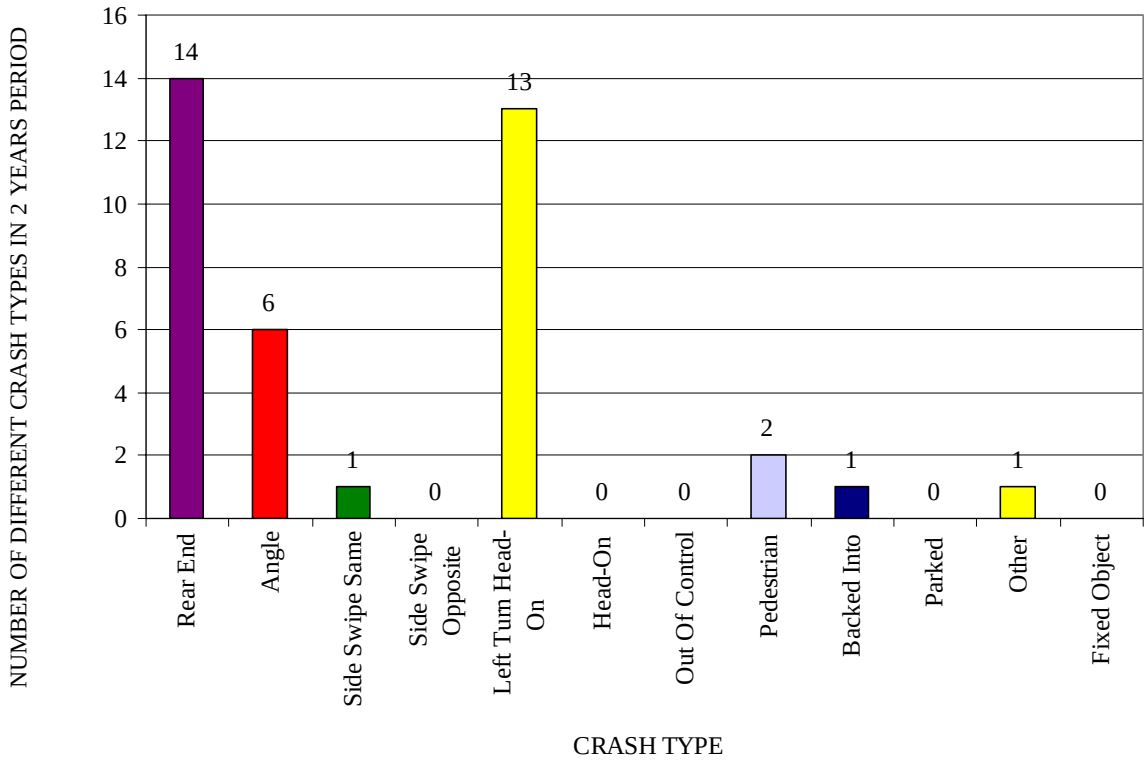


Figure 6.28.5: Two year (2003 & 2004) crash experience at Gratiot Road (M-46) and River Road Intersection

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of Gratiot Road (M-46) and River Road could be attributed to inadequate clearance intervals, signal timing and phasing.

The collision diagram indicates the frequent occurrence of left-turn head-on crashes for the east and westbound movements. An analysis of left-turn warrants show that an exclusive left turn phase is warranted for the east-west left-turn movement. The proposed signal timing plan consists of three phases including a protected left-turn phase for the east-west movement. To improve the signal visibility, the existing span wire installation should be replaced by mast-arm signal installation, as shown in the proposed diagram in Figure 6.28.6.

Post-mounted traffic signals with "LEFT" illuminated signs should be provided at the northeast and southwest corners of the intersection to assist east and westbound left turning vehicles.

Stop bars and crosswalks should be relocated on all approaches. Pavement markings should be upgraded at all approaches with proper directional arrows. Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.28.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.28.1. Existing and Proposed Clearance Intervals for the Gratiot Road (M-46) and River Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	4.8	1.5	4.3	1.5	Less rear end, angle, left-turn head-on and injury crashes
E-W Through Phase	4.8	1.5	4.3	1.4	
E-W Left Turn Phase	-	-	4.3	1.4	

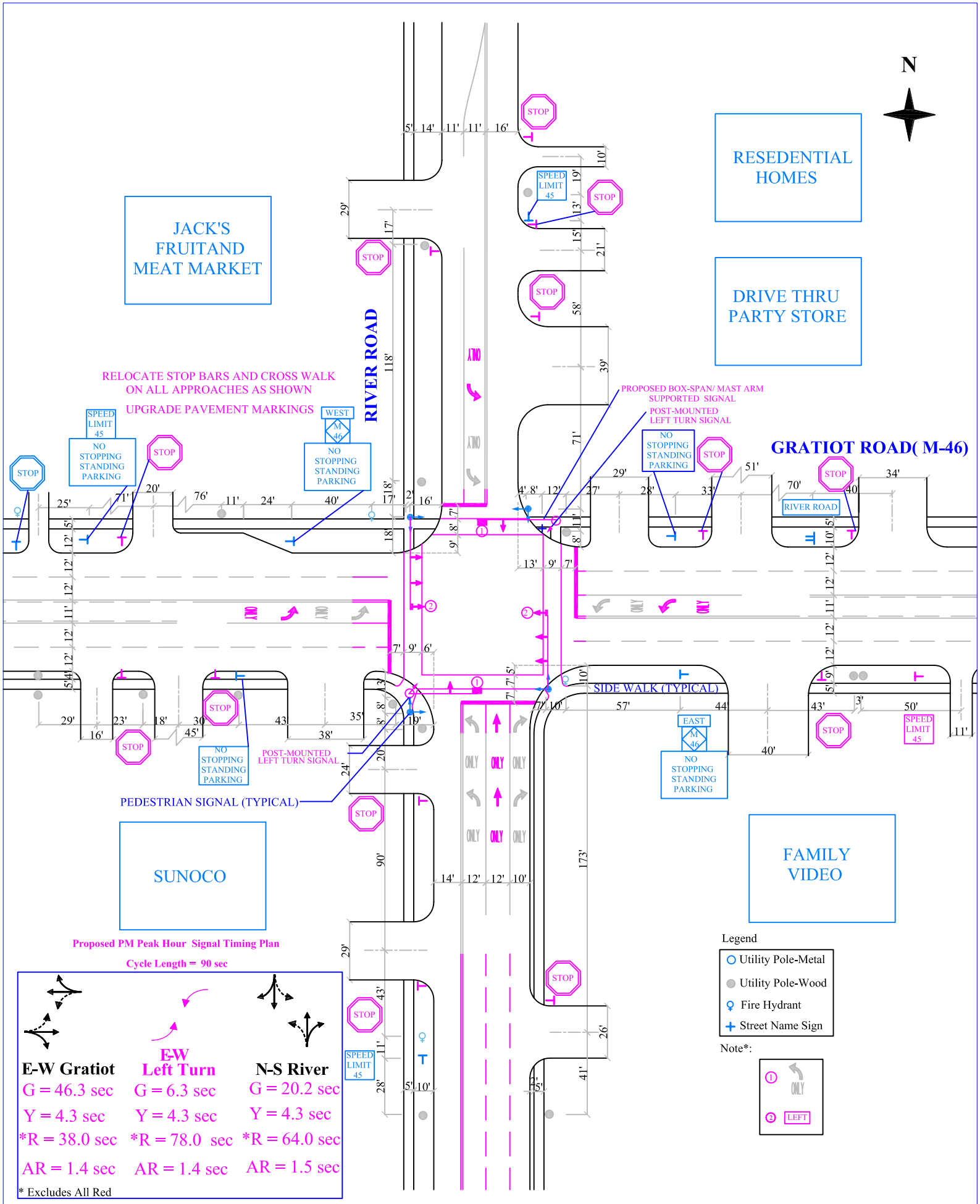
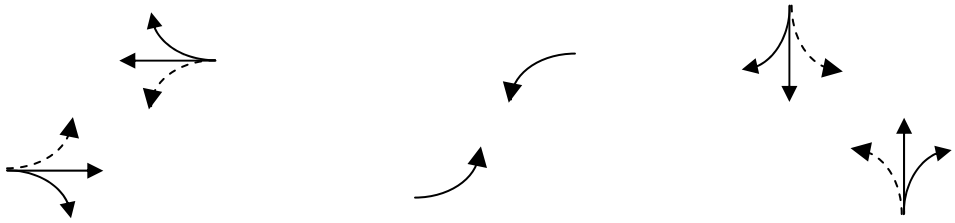


Figure 6.28.6 Proposed Diagram of Gratiot Road (M-46) and River Road Intersection (Not to Scale)

The proposed PM peak hour signal timing plan with a cycle length of 90 second is shown below:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



E-W Gratiot	E-W Left Turn	N-S River
G = 46.3 sec	G = 6.3 sec	G = 20.2 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 4.3 sec
*R = 38.0 sec	*R = 78.0 sec	*R = 64.0 sec
AR = 1.4 sec	AR = 1.4 sec	AR = 1.5 sec

* Excludes All Red

The proposed diagram in Figure 6.28.6 indicates the improvements including a mast-arm signal installation, the post-mounted traffic signals for left-turns, the upgraded pavement markings for all approaches and the proposed signal timing plan.

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Gratiot Road and River Road to 4.3 seconds.
- Reduce the all-red interval for Gratiot Road to 1.4 seconds.
- Provide an exclusive left-turn phase for Gratiot Road.
- Relocate stop bars and crosswalks at all the approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for east and westbound left turning vehicles.
- Upgrade pavement markings with proper directional arrows for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Provide stop signs for all the driveways located near the intersection.
- Optimize the signal timings for AM peak and off-peak hours and implement the optimized signal timings for PM peak hour.

Operational Evaluation

The results of the level of service analysis for the intersection of Gratiot Road and River Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is A with a delay of 9.5 seconds per vehicle.

With the modifications to the signal timings and phasing and the noted improvements, the overall intersection level of service will be C with a delay of 22.4 seconds per vehicle. Although the overall delay has slightly increased due to the inclusion of the protected left-turn phase for east-west movements and the consistent cycle length along the Michigan corridor, it is acceptable since progression has been achieved and the safety has been enhanced.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.28.2.

Table 6.28.2.
Gratiot Road (M-46) and River Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Gratiot Road	5.1	A	4.7	A
WB Gratiot Road	6.9	A	31.3	C
NB River Road	27.6	C	31.7	C
SB River Road	27.4	C	31.2	C
Overall Intersection	**9.5**	**A**	**22.4**	**C**

29. Gratiot Road (M- 46) and Center Road

The intersection of Gratiot Road (M-46) and Center Road is located in the southern part of Saginaw Township. Gratiot Road runs in the east-west direction and Center Road runs in the north-south direction. Gratiot Road (M-46) is a state trunk street and Center Road serves as a major Arterial Street.

Gratiot Road, east of the intersection with Center Road has an average daily traffic volume of approximately 19,700 vpd as per MDOT 2004 data.

Gratiot Road consists of one dedicated left-turn lane, two through lanes and a dedicated right-turn lane on both eastbound and westbound approaches. Gratiot Road is 72 feet wide to the east of the intersection and to the west it is 69 feet.

Center Road consists of one dedicated left turn lane, one through lane and one shared through and right-turn lane on both northbound and southbound approaches. Center Road is 57 feet wide to the north of the intersection and to the south it is 56 feet wide.

The speed limit for Gratiot Road is 45 miles per hour west of the intersection and 40 miles per hour east of the intersection. The speed limit for Center Road is 40 miles per hour.

Land Use

The intersection of Gratiot Road and Center Road has a Shell gas station on the southwest corner, a Rite Aid Pharmacy on the northwest corner, Struts Brakes on the southeast corner and American Video and H&R Block on the northeast corner. There are multiple driveways, serving the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Gratiot Road and Center Road.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.29.1. Details of the intersection approaches are shown in photographs 29a through 29d. Photographs 29a and 29b show the north and southbound approaches of Center Road with driveways close to the intersection on the northbound approach. Photographs 29c and 29d show the east and westbound approaches with existing span-wire installation and pavement markings. There are driveways close to the intersection on both eastbound and westbound approaches.

Photograph 29a. NB Center Road at Gratiot Road

Photograph 29b. SB Center Road at Gratiot Road

Photograph 29c. EB Center Road at Gratiot Road

Photograph 29d. WB Center Road at Gratiot Road

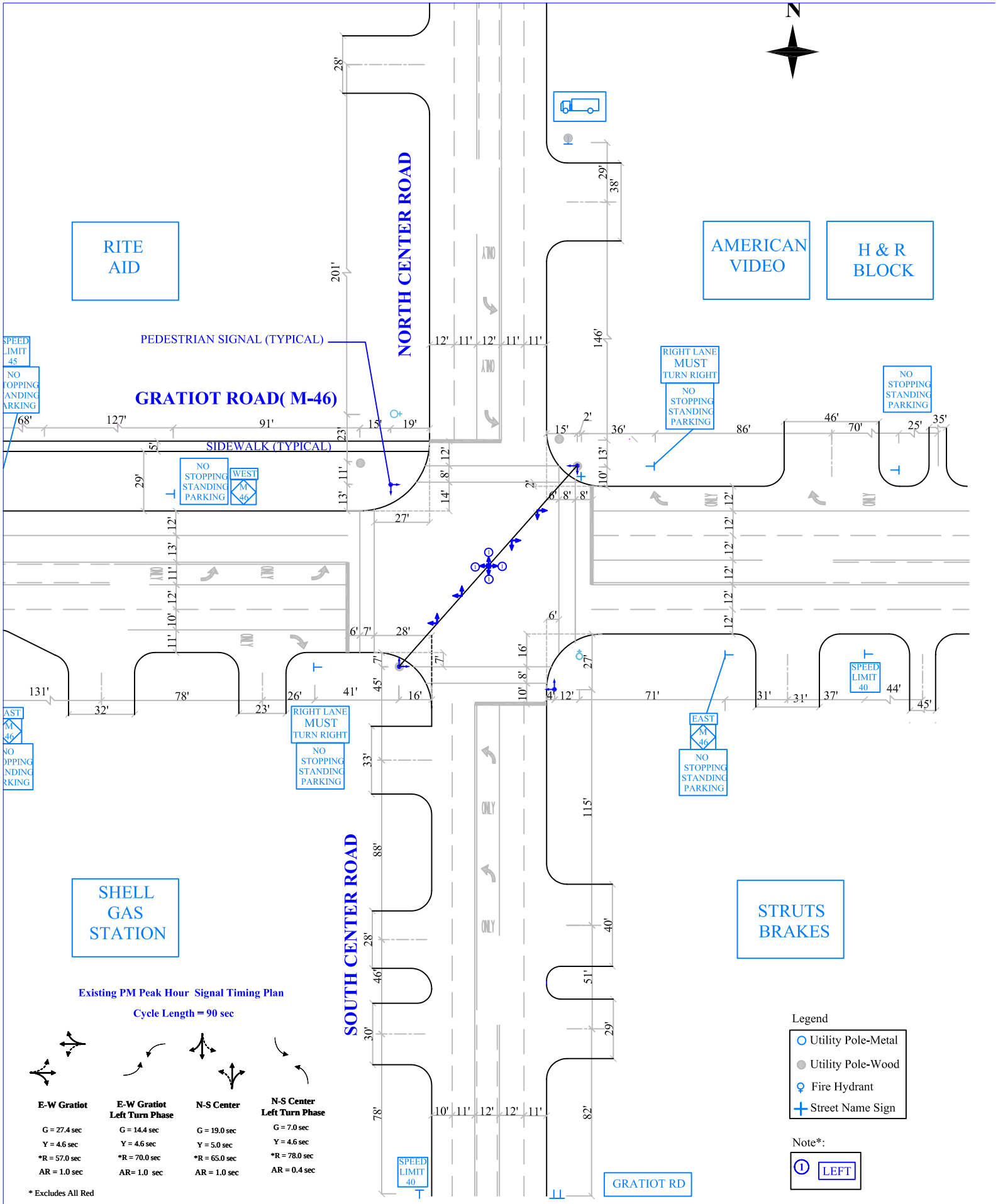


Figure 6.29.1.Existing Condition Diagram of Gratiot Road (M-46) and Center Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on a clear day on April 25, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:30 to 5:30 PM and is presented in Figure 6.29.2. The traffic patterns indicate that the volumes in the east-west direction are relatively higher than the north-south direction. Also, the eastbound and westbound left-turn volumes are considerably high.

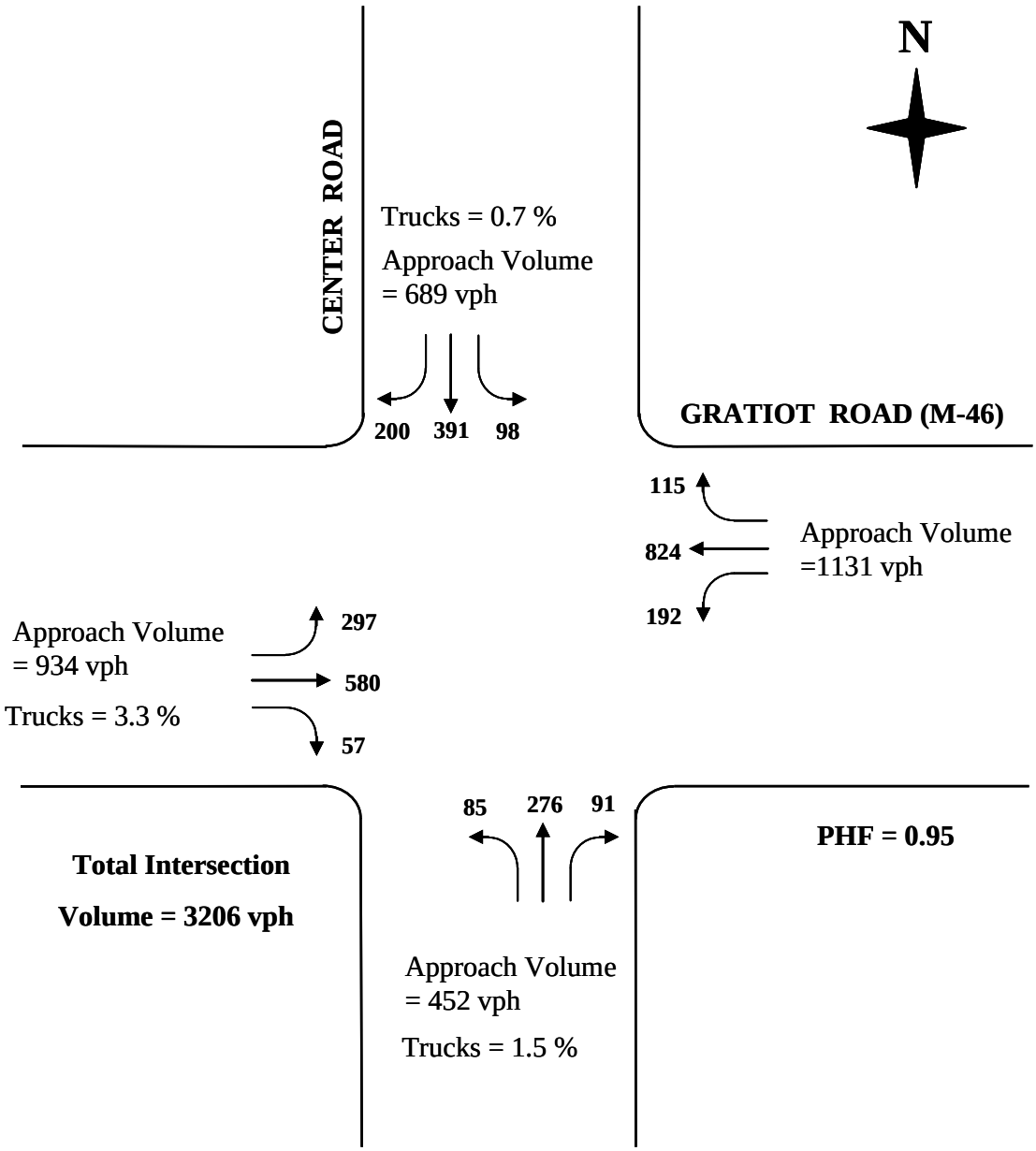


Figure 6.29.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes for the Intersection of Gratiot Road (M-46) and Center Road

Traffic Control

This intersection is controlled by a four-phase traffic signal with a 90 second cycle length. The existing clearance interval includes a yellow interval of 4.6 seconds and an all red interval of 1.0 second for the east-west through and left-turn phase. The north-south through phase has a yellow interval of 5.0 seconds and all-red interval of 1.0 second while the north-south left-turn phase has yellow interval of 4.6 seconds and all red interval of 0.4 seconds. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.29.1.

Crash Analysis

A total of 69 crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of 35 crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Gratiot Road and Center Road are shown in Figures 6.29.3 and 6.29.4 respectively.

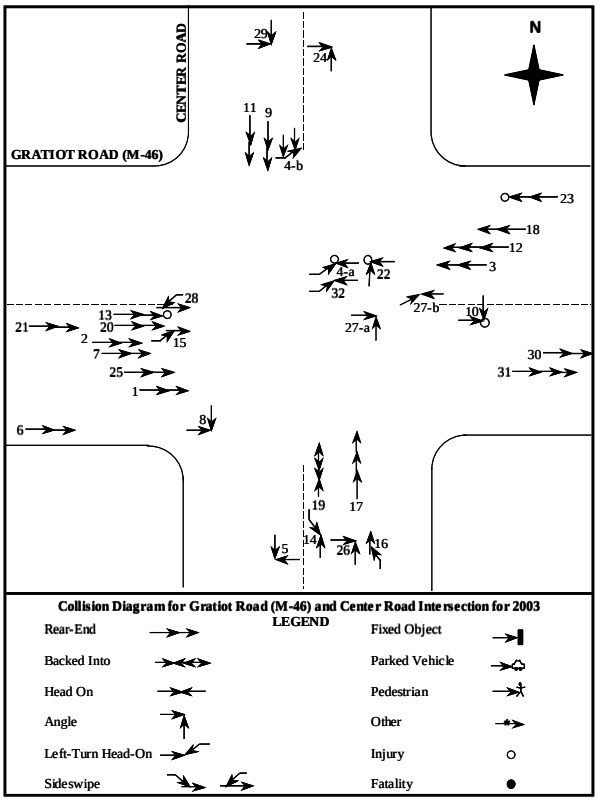


Figure 6.29.3. Collision Diagram for Gratiot Road and Center Road for 2003

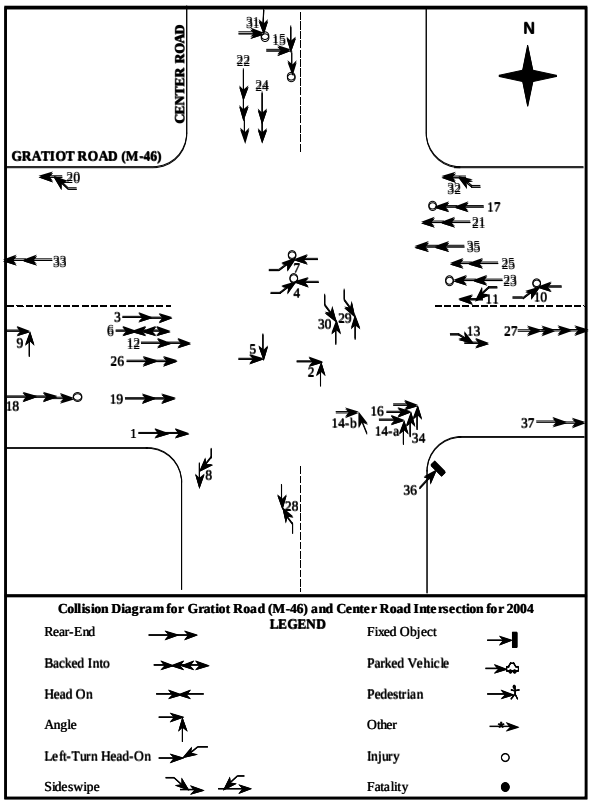


Figure 6.29.4. Collision Diagram for Gratiot Road and Center Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 2.39 per million entering vehicles, which is a significant, considering the approach volume at the intersection. The intersection had significant number of rear-end crashes and some angle and left-turn head-on crashes which may be due to inadequate signal timing plan and insufficient clearance intervals.

The crash distribution based upon type of crashes is shown in Figure 6.29.5. Rear-end, angle and left-turn head-on crashes are the predominant type of crash reported at the intersection. The corresponding two year crash frequency includes 33 rear-end crashes, 16 angle crashes and nine left turn head-on crashes.

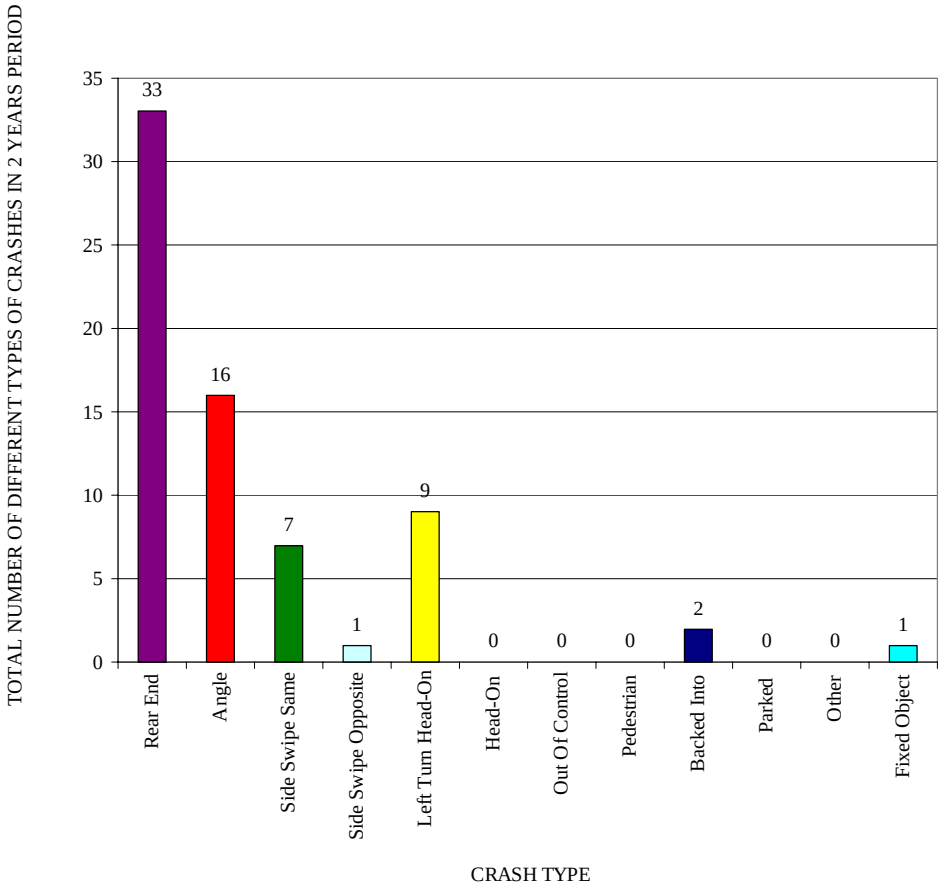


Figure 6.29.5. Two year (2003 & 2004) crash experience at Gratiot Road(M-46) and Center Road Intersection

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of Gratiot Road (M-46) and Center Road could be attributed to poor visibility of traffic signals and signal timing inadequacies related to the clearance interval, as revealed by a relatively high occurrence of rear-end, angle and left-turn head-on crashes.

To improve signal visibility, the existing span wire installation should be replaced by mast-arm signal installation, as shown in the proposed diagram in Figure 6.29.6.

Post-mounted traffic signals with "LEFT" illuminated signs should be provided at the northwest and southeast corners to assist north and southbound left turning vehicles and northeast and southwest corners to assist east and westbound left turning vehicles.

Stop bar and cross walk should be relocated on the northbound, southbound and eastbound approaches. Pavement markings should be upgraded at all the approaches.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.29.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.29.1. Existing and Proposed Clearance Intervals for the Gratiot Road (M-46) and Center Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	4.6	1.0	4.3	1.6	Less rear end, angle, left-turn head-on and injury crashes
E-W Left Turn Phase	4.6	1.0	4.3	1.6	
N-S Through Phase	5.0	1.0	3.9	1.9	
N-S Left Turn Phase	4.6	0.4	3.9	1.9	

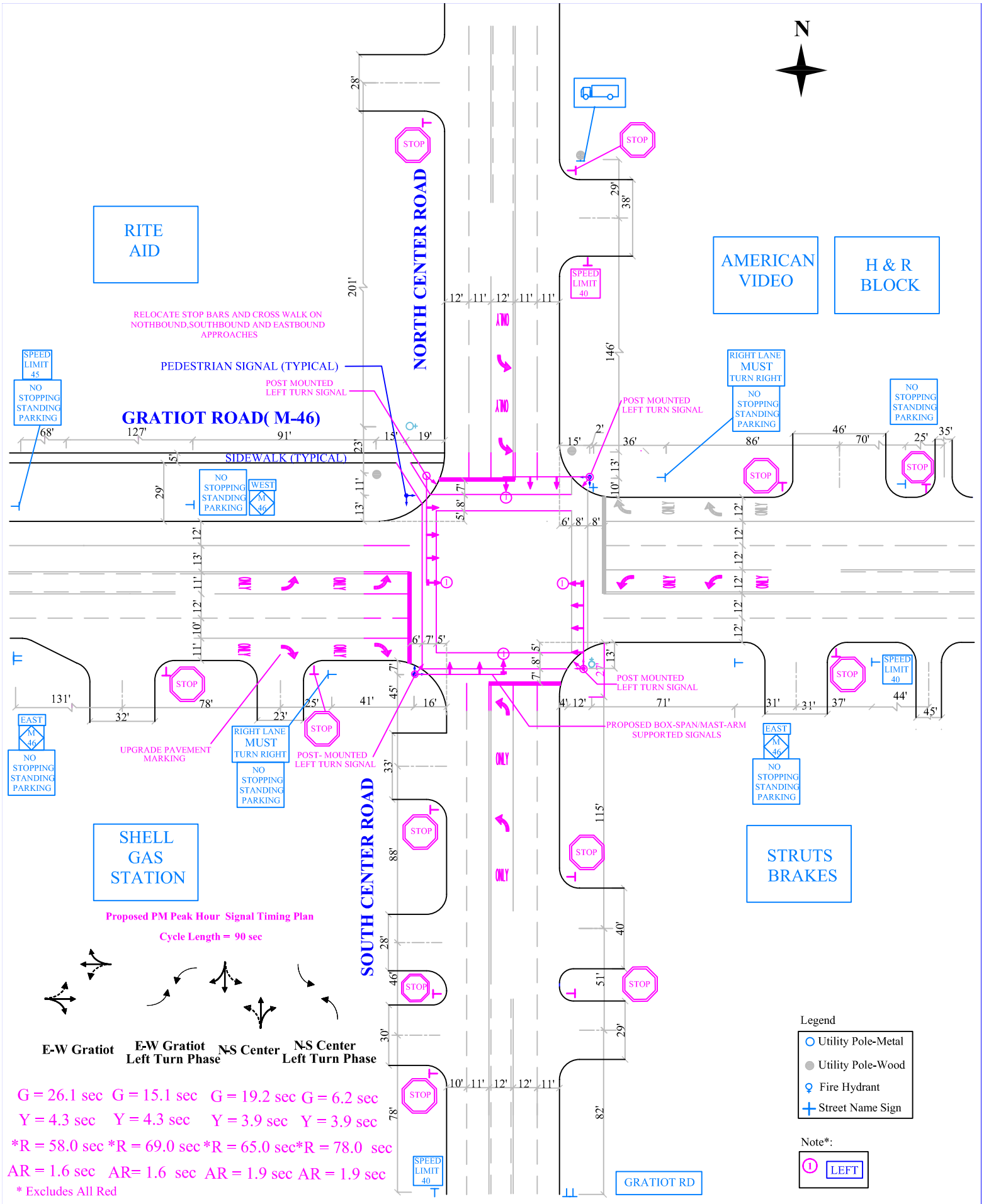
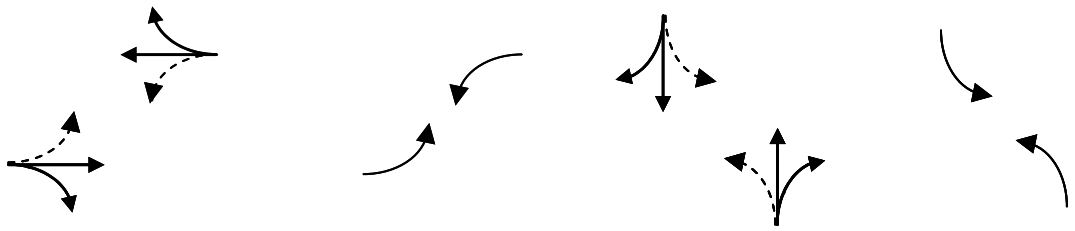


Figure 6.29.6 Proposed Diagram of Gratiot Road (M-46) and Center Road Intersection (Not to Scale)

The proposed PM peak hour signal timing plan with a cycle length of on a 90-second is shown below:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



E-W Gratiot	E-W Gratiot Left Turn Phase	N-S Center	N-S Center Left Turn Phase
G = 30.1 sec	G = 9.3 sec	G = 21.2 sec	G = 6.2 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 3.9 sec	Y = 3.9 sec
*R = 54.0 sec	*R = 84.1 sec	*R = 63.0 sec	*R = 78.0 sec
AR = 1.6 sec	AR= 1.6 sec	AR = 1.9 sec	AR = 1.9 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Gratiot Road to 4.3 seconds and for Center Road to 3.9 seconds.
- Increase the all-red interval for Gratiot Road to 1.6 seconds and for Center Road to 1.9 seconds.
- Relocate stop bars and crosswalks on the northbound, southbound and eastbound approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at all corners of the intersection to assist left turning vehicles.
- Upgrade pavement markings with proper directional arrows for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Provide stop signs for all the driveways located near the intersection.
- Optimize the signal timings for AM peak and off-peak hours and implement the optimized signal timing for PM peak hour.

The proposed diagram in Figure 6.29.6 indicates the improvements including a mast-arm signal installation, post-mounted traffic signals for left-turns, the upgraded pavement markings for all approaches and the proposed signal timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Gratiot Road (M-46) and Center Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is C with a delay of 29.0 seconds per vehicle.

With the modifications to the signal timing and the noted improvements, the overall intersection level of service will remain C with a delay of 32.0 seconds per vehicle. Although the overall delay has slightly increased due to the provision of adequate yellow and all-red intervals and the consistent cycle length along the Michigan corridor, the benefits of the achieved progression and enhanced safety outweigh the increased delay.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.29.2.

Table 6.29.2.
Gratiot Road (M-46) and Center Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Gratiot Road	26.9	C	28.5	C
WB Gratiot Road	26.6	C	31.2	C
NB Center Road	30.2	C	31.9	C
SB Center Road	35.3	D	38.3	D
Overall Intersection	**29.0**	**C**	**32.0**	**C**

30. Gratiot Avenue (M-46) and Wheeler Street

The intersection of Gratiot Avenue (M-46) and Wheeler Street is located in the western part of Saginaw. Gratiot Avenue runs in the east-west direction and Wheeler Street runs in the north-south direction. Gratiot Avenue is a state trunk line and also known as M-46 and Wheeler Street is classified as a major arterial.

Gratiot Avenue, west of the intersection with Wheeler Street has an average daily traffic volume of approximately 19,700 vpd and 19,100 vpd to the east of the intersection as per MDOT 2004 Data.

Gratiot Avenue consists of one through lane and one shared through and right turn lane for both eastbound and westbound approaches. The pavement width of Gratiot Avenue is 45 feet to the east of the intersection and to the west it is 48 feet.

Wheeler Street consists of one shared through and right-turn lane and one dedicated left-turn lane for both southbound and northbound approaches. The pavement width of Wheeler Street is 45 feet to the north of the intersection and to the south it is 48 feet.

The posted speed limit for Gratiot Avenue is 35 miles per hour east of the intersection and 40 miles per hour west of the intersection. The speed limit for N Wheeler Street, is 25 miles per hour and for S Wheeler Street it is 30 miles per hour.

Land Use

The intersection of Gratiot Avenue (M-46) and Wheeler Street is mainly occupied by residential homes on all sides. There are multiple driveways closer to the intersection which serves the vehicles going to and form these residential homes.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of Gratiot Avenue and Wheeler Street.

The existing characteristics of the intersection including traffic control and surrounding land uses are schematically shown in Figure 6.30.1. Details of the intersection approaches are shown in photographs 30a through 30d. these photographs show the existing span-wire signal installation and pavement markings. The westbound approach has two driveways located very close to the intersection.

Photograph 30a. NB Wheeler Street at Gratiot Avenue

Photograph 30b. SB Wheeler Street at Gratiot Avenue

Photograph 30c. EB Gratiot Avenue at Wheeler Street

Photograph 30d. WB Gratiot Avenue at Wheeler Street

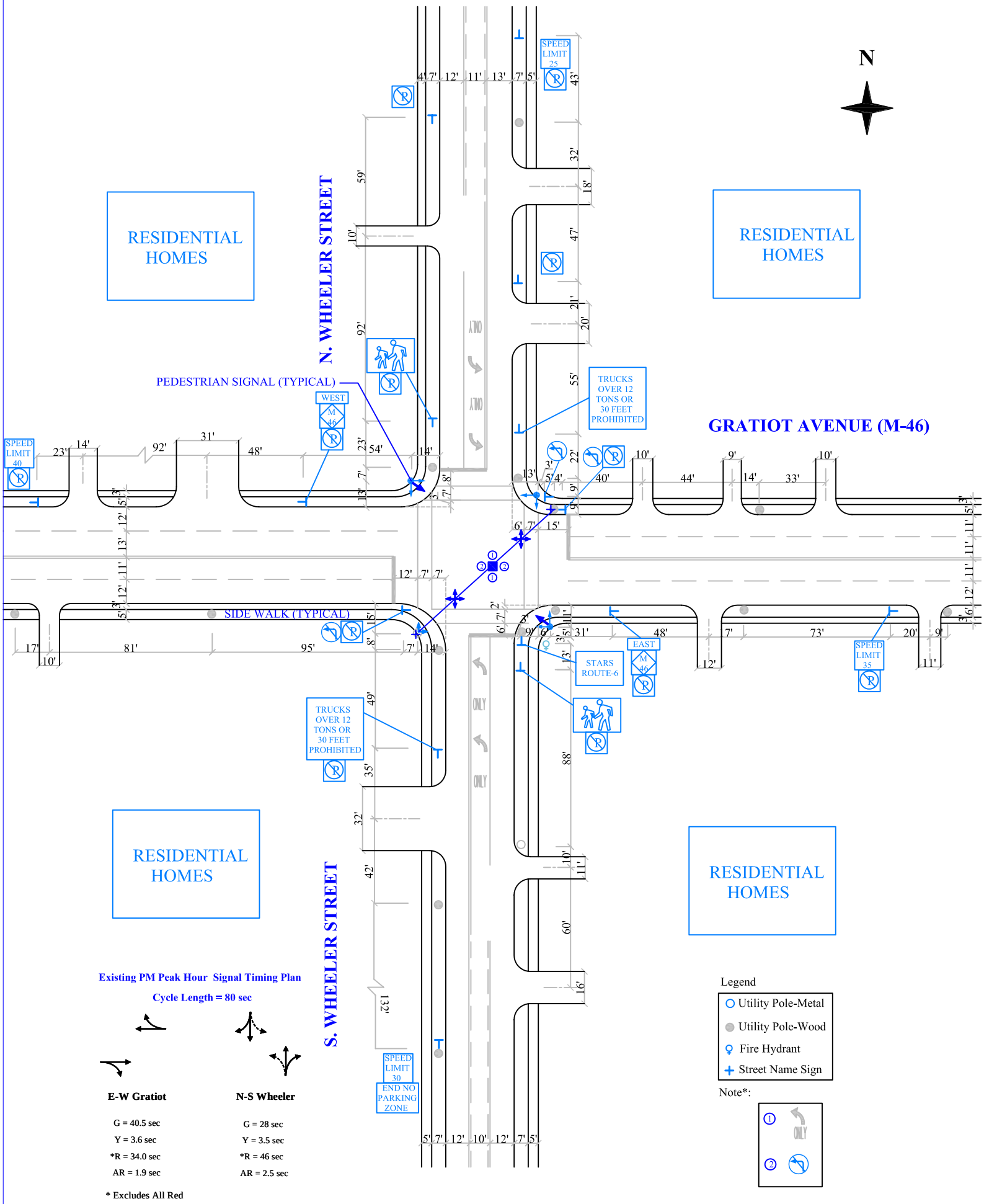


Figure 6.30.1. Existing Condition Diagram of Gratiot Avenue (M-46) and Wheeler Street Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on a clear day on April 25, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at suitable locations around the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:15 PM to 5:15 PM and is presented in Figure 6.30.2. The traffic patterns indicate that volumes in the east-west direction are considerably higher than the north-south direction.

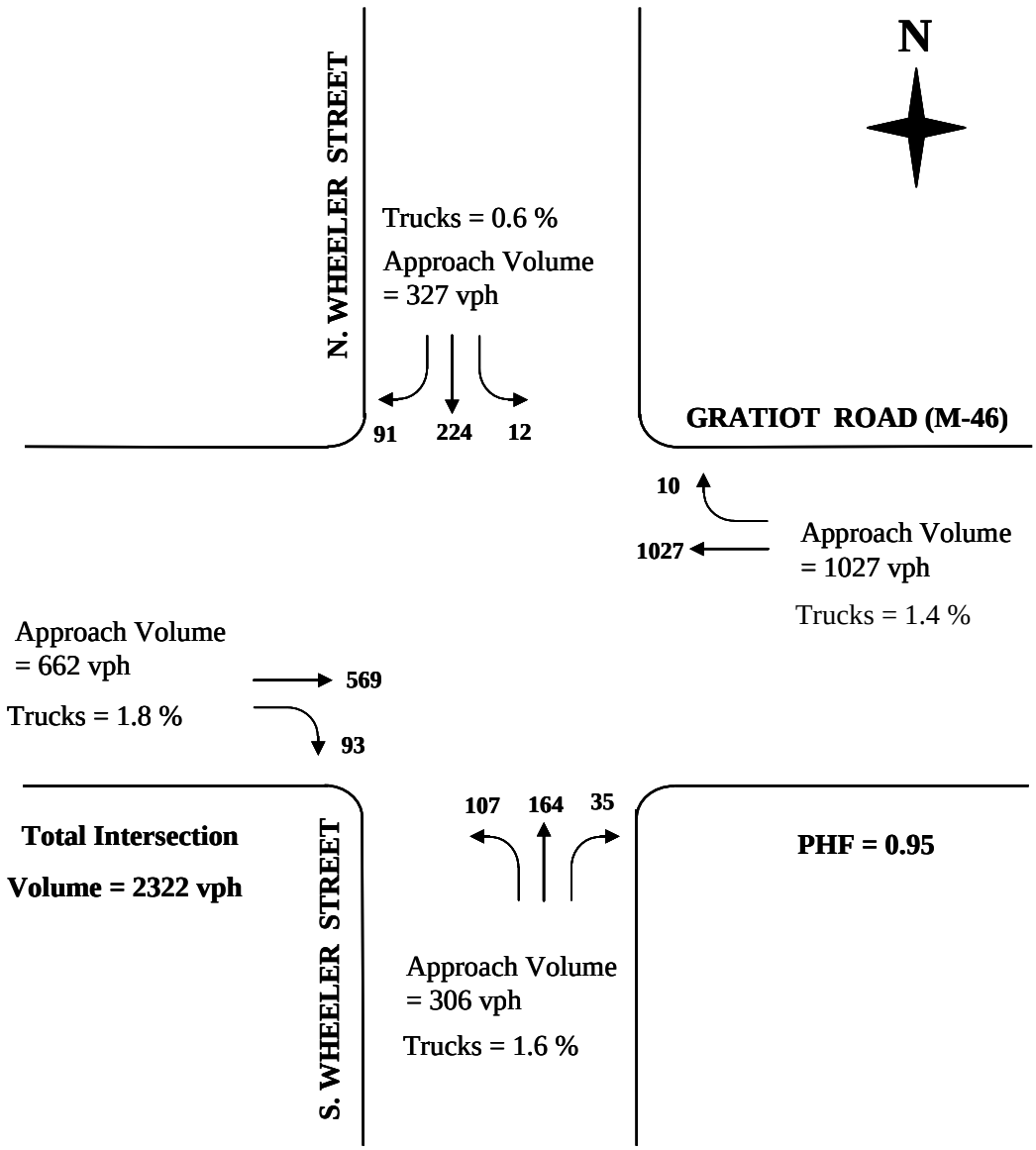


Figure 6.30.2. Existing PM Peak Hour (4:15-5:15 PM) Traffic Volumes for the Intersection of Gratiot Avenue (M-46) and Wheeler Street

Traffic Control

This intersection is controlled by a two-phase traffic signal with an 80-second cycle length. The existing clearance interval includes a yellow interval of 3.6 seconds and an all-red interval of 1.9 seconds for the east-west phase. The north-south phase has a yellow interval of 3.5 seconds and an all red interval of 2.5 seconds. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.30.1.

Crash Analysis

A total of 26 crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of 13 crashes. Collision diagrams were prepared based on the UD-10 reports to identify crash patterns. Figures 6.30.3 and 6.30.4 show the collision diagrams for the years 2003 and 2004, respectively.

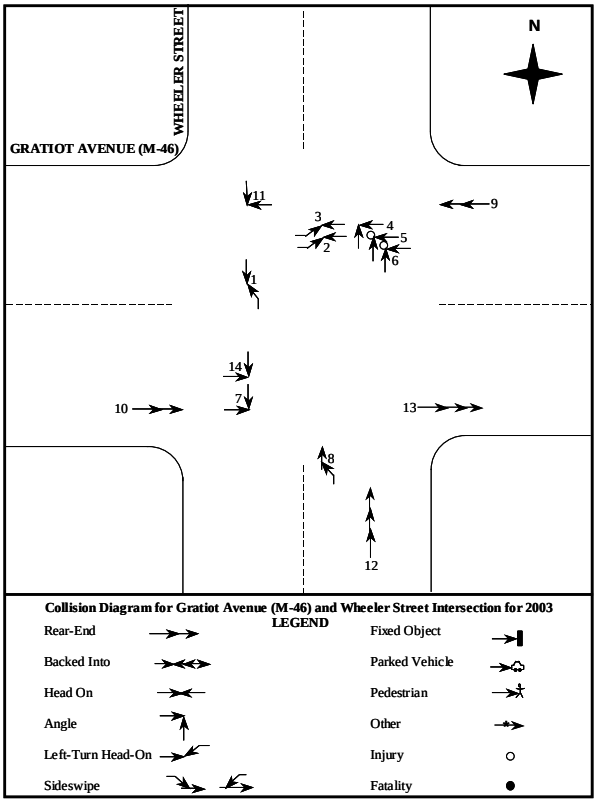


Figure 6.30.3. Collision Diagram for Gratiot Avenue and Wheeler Street for 2003

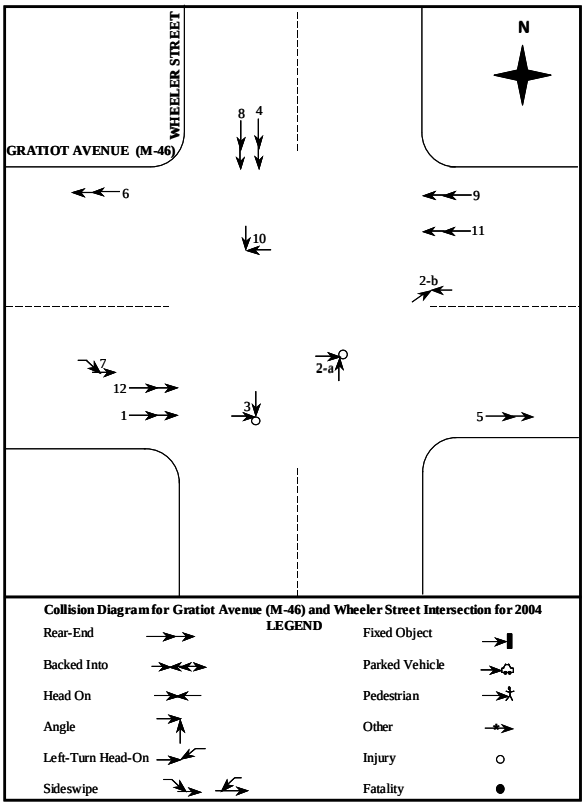


Figure 6.30.4. Collision Diagram for Gratiot Avenue and Wheeler Street for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.23 per million entering vehicles, which is nominal, considering the approach volume at the intersection.

The crash distribution, based upon type of crashes, is shown in Figure 6.30.5. Rear-end and angle crashes are the predominant type of crashes reported at this intersection. The corresponding two-year crash frequencies for these crashes include 12 rear-end crashes and nine angle crashes.

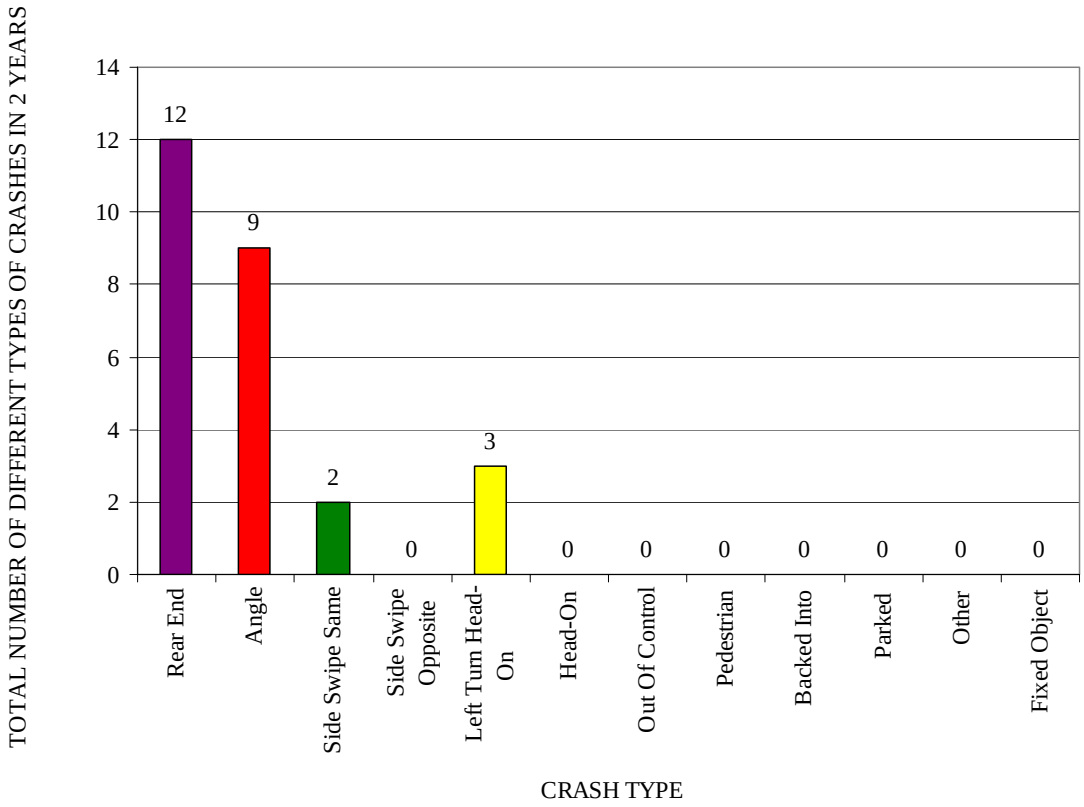


Figure 6.30.5: Two year (2003 & 2004) crash experience at Gratiot Avenue (M-46) and Wheeler Street Intersection

Safety Deficiencies and Recommended Improvements

An analysis of collision diagrams show that considerable number of angle and rear-end crashes occurred at the intersection of Gratiot Avenue (M-46) and Wheeler Street. This may be due to poor visibility of traffic signals and inadequate clearance intervals and signal timings.

The existing cycle length of 80 seconds is changed to 90 seconds for progression along the Gratiot Corridor. Adequate clearance intervals have been provided and are shown in the proposed signal-timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.30.6.

Stop bar should be relocated on the east and westbound approaches.

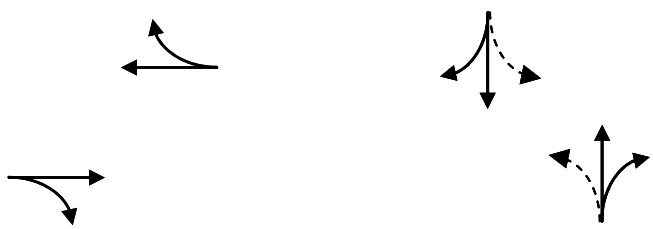
The existing yellow and all-red intervals are shown in Table 6.30.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.30.1. Existing and Proposed Clearance Intervals for the Gratiot Avenue (M-46) and Wheeler Street Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Through Phase	3.6	1.9	3.9	1.5	Less rear-end, angle, left-turn head-on and injury crashes
N-S Through Phase	3.5	2.5	3.2	2.2	

The proposed PM peak period signal timing plan with a cycle length of 90 seconds is shown below.

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



E-W Gratiot	N-S Wheeler
G = 45.6 sec	G = 33.6 sec
Y = 3.9 sec	Y = 3.2 sec
*R = 39.0 sec	*R = 51.0 sec
AR = 1.5 sec	AR = 2.2 sec

* Excludes All Red

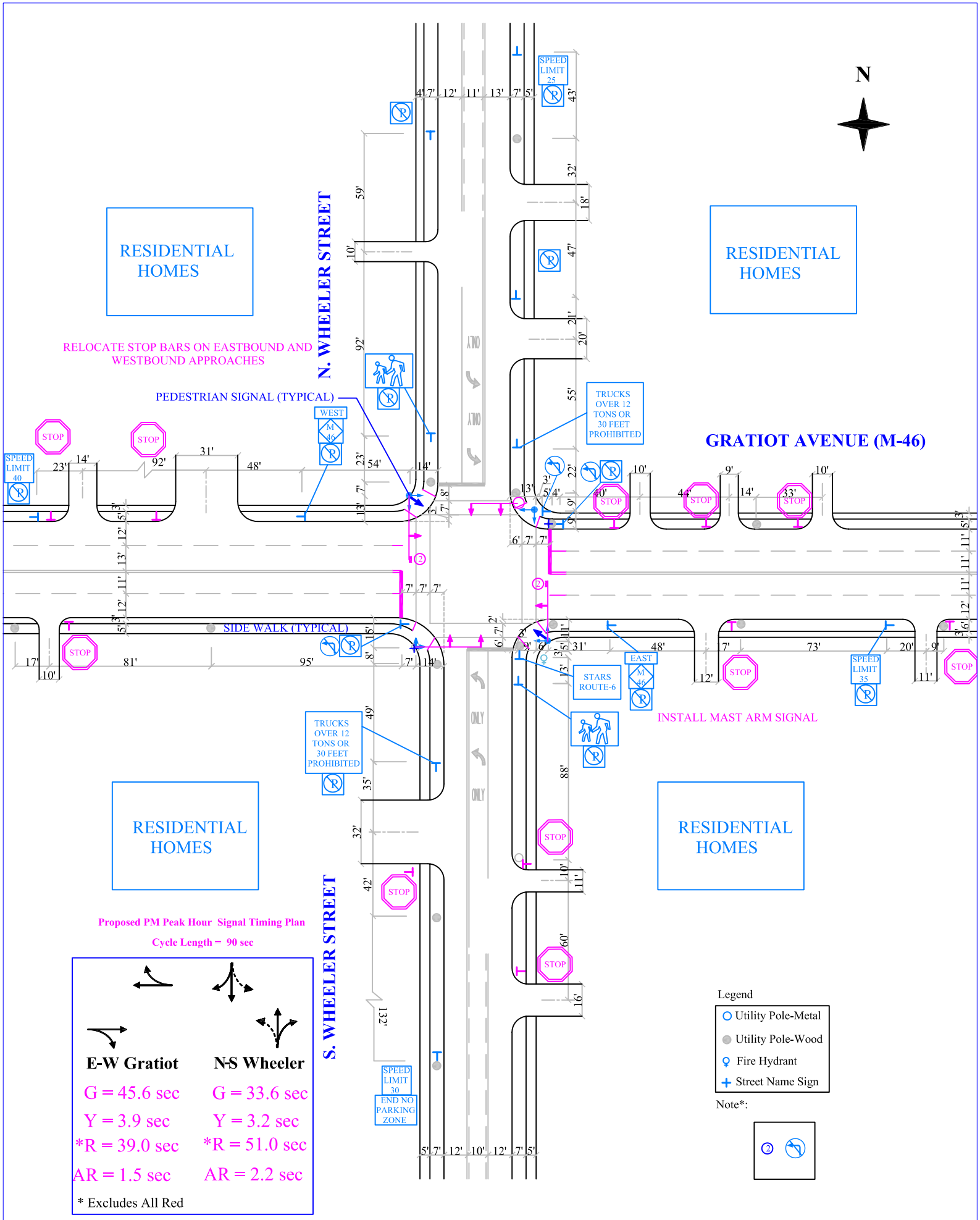


Figure 6.30.6 Proposed Diagram of Gratiot Avenue (M-46) and Wheeler Street Intersection (Not to Scale)

A list of the recommended improvements is summarized below:

- Increase the yellow interval for Gratiot Avenue to 3.9 seconds and reduce it for Wheeler Street to 3.2 seconds.
- Reduce the all-red interval for Gratiot Avenue to 1.5 seconds and 2.2 seconds for Wheeler Street.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Relocate stop bars at eastbound and westbound approaches, as per MMUTCD guidelines.
- Provide stop signs for all the driveways located near the intersection.
- Optimize the signal timings for AM peak and off-peak hours and implement the optimized signal timings for PM peak.

The proposed diagram of the intersection in Figure 6.30.6 shows the improvements including a mast-arm signal installation and the proposed signal-timing plan.

Operational Evaluation

The results of the level of service analysis for the intersection of Gratiot Avenue (M-46) and Wheeler Street indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 12.9 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will be B with an increased delay of 14.5 seconds per vehicle. This LOS will ensure better operational performance along with enhanced safety and improved progression.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.30.2.

Table 6.30.2.
Gratiot Avenue (M-46) and Wheeler Street
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Gratiot Avenue	8.7	A	4.0	A
WB Gratiot Avenue	11.1	B	16.6	B
NB Wheeler Street	20.0	C	21.5	C
SB Wheeler Street	21.0	C	22.8	C
Overall Intersection	**12.9**	**B**	**14.5**	**B**

31. Davenport Avenue and North Michigan Avenue

The intersection of Davenport Avenue and North Michigan Avenue is located in the northern part of Saginaw City. Davenport Avenue runs in the east-west direction and North Michigan Avenue runs in the north-south direction. Davenport Avenue is a state trunk line classified as M-58 and North Michigan Avenue is classified as an arterial.

Davenport Avenue, near the intersection with North Michigan Avenue, has an average daily traffic volume of approximately 11,700 vpd as per Michigan Department of Transportation Traffic Count Maps (2004 Data).

Davenport Avenue is a one-way street which consists of one dedicated left-turn lane, one through lane and one shared through and right-turn lane to the east of the intersection and to the west it consists of three receiving lanes. Davenport Avenue is 41 feet wide to the east of the intersection and to the west it is 38 feet wide.

North Michigan Avenue is a one-way street which consists of one dedicated left-turn lane and two through lanes to the south of the intersection and to the north it consists of two receiving lanes. The pavement width of North Michigan Avenue is 45 feet to the south of the intersection and to the north it is 36 feet wide.

The speed limit for Davenport Avenue is 35 miles per hour, while along North Michigan Avenue; it is 25 miles per hour

<u>Land Use</u>

The intersection of Davenport Avenue and North Michigan Avenue has Great Lake Auto Sale on the northwest corner, a Thrift Store on the southeast corner and a parking lot on the southwest corner. The northeast corner of the intersection is mainly occupied by residential homes. There are multiple driveways close to the intersection which serve the vehicles going to and from these establishments and residential homes.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.31.1. Details of the intersection approaches are shown in photographs 31a through 31d. Photograph 31b shows the existing span wire installation and the pavement markings on the south leg of the intersection.

Photograph 31a. NB N. Michigan Ave. at Davenport Avenue (North Leg)

Photograph 31b. NB N. Michigan Ave. at Davenport Ave. (South Leg)

Photograph 31c. WB Davenport Ave at N. Michigan Ave.

Photograph 31d. WB Davenport Ave. at N. Michigan Ave

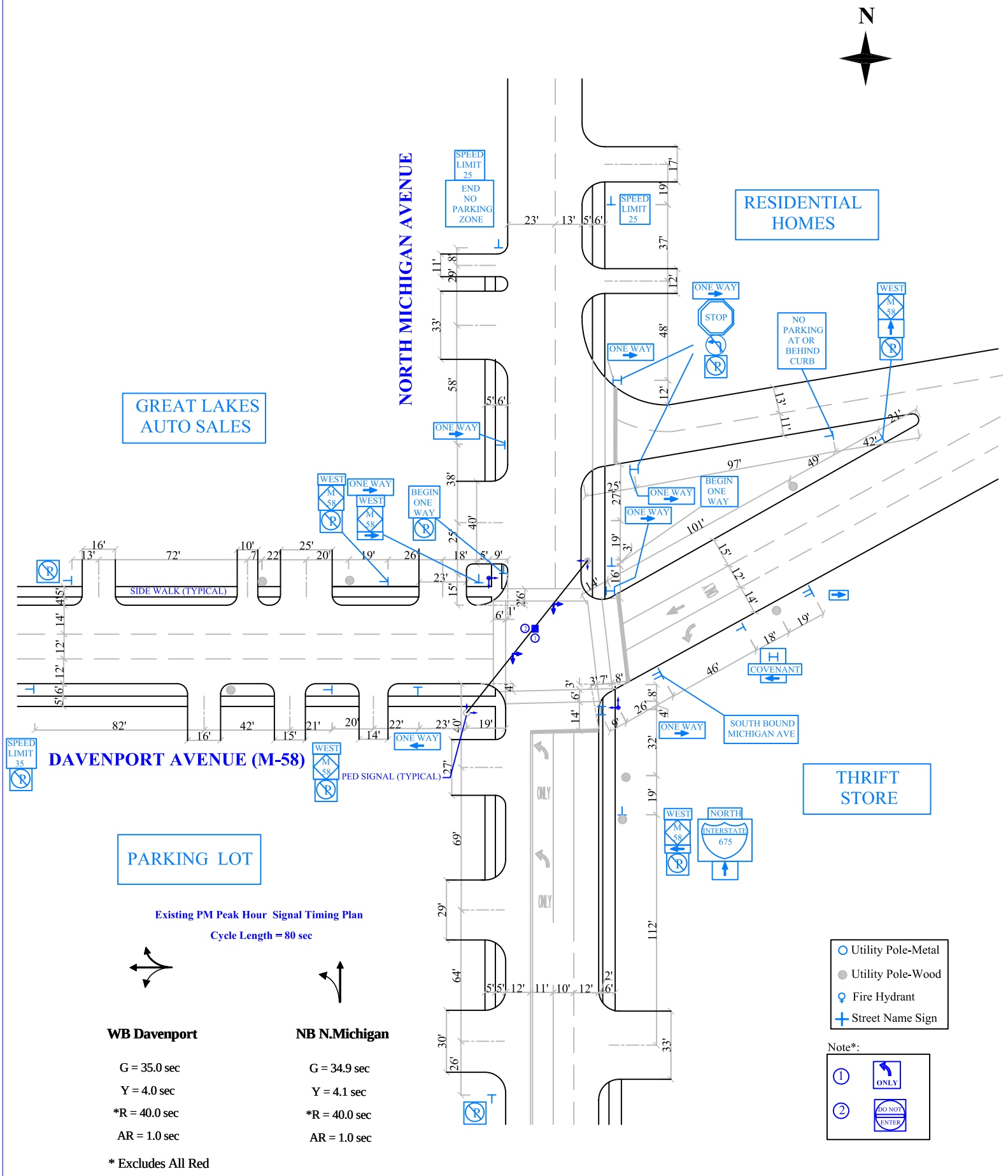


Figure 6.31.1.Existing Condition Diagram of Davenport Avenue (M-58) and North Michigan Avenue Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were collected manually on a clear day on April 18, 2006 from 4 PM to 6 PM by two people standing diagonally opposite at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:30 to 5:30 PM and is presented in Figure 6.31.2. The traffic patterns indicate that there is no significant difference between volumes in the northbound and westbound direction.

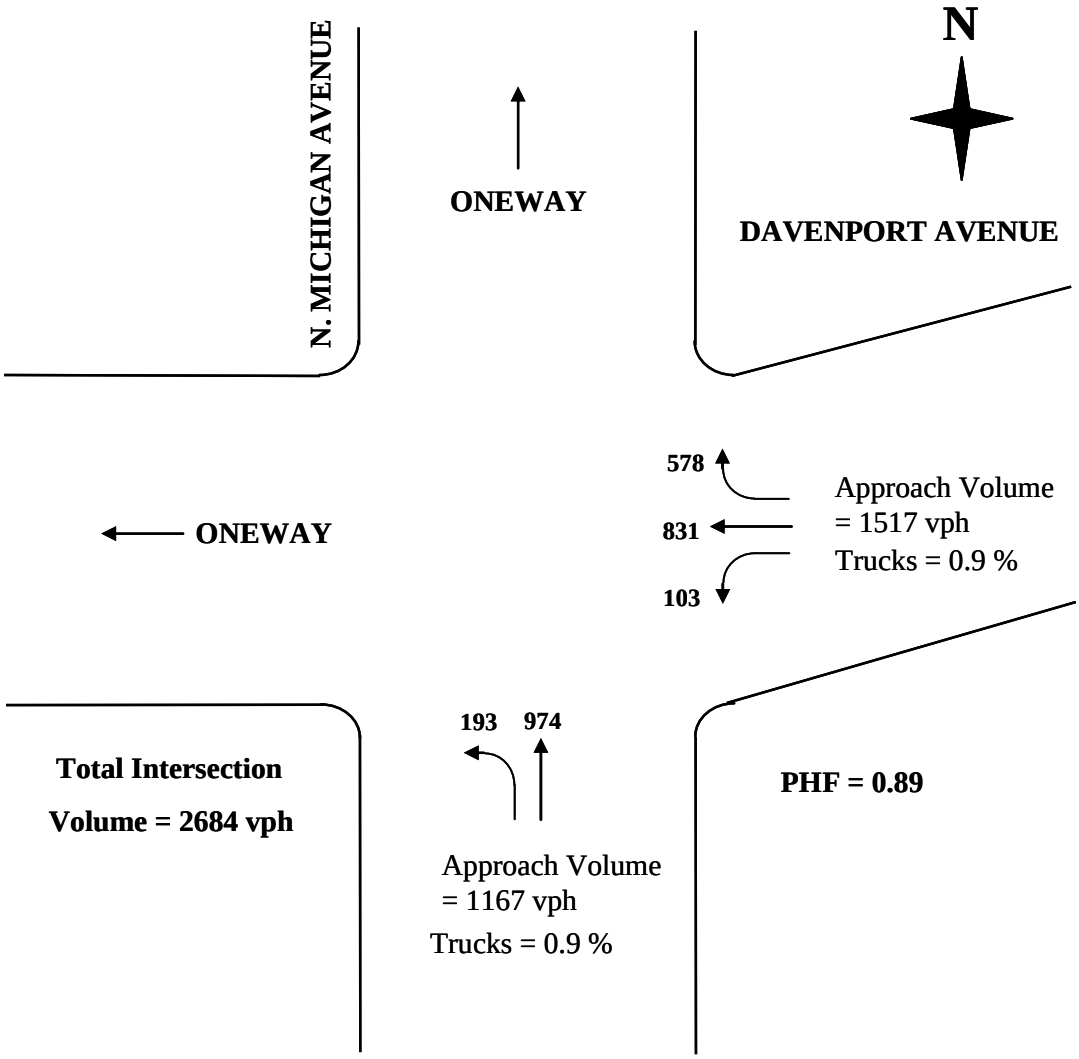


Figure 6.31.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes for the Intersection of North Michigan Avenue and Davenport Avenue

Traffic Control

This intersection is controlled by a two-phase span-wire traffic signal with an 80-second cycle length. The existing clearance interval consists of a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for the Davenport Avenue and a yellow interval of 4.1 seconds and an all-red interval of 1.0 second for the N. Michigan Avenue. The existing PM peak hour signal timing plan is shown in the existing condition diagram in Figure 6.31.1.

Crash Analysis

A total of 37 crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of 19 crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of Davenport Avenue and Congress Avenue are presented in Figures 6.31.3 and 6.31.4, respectively.

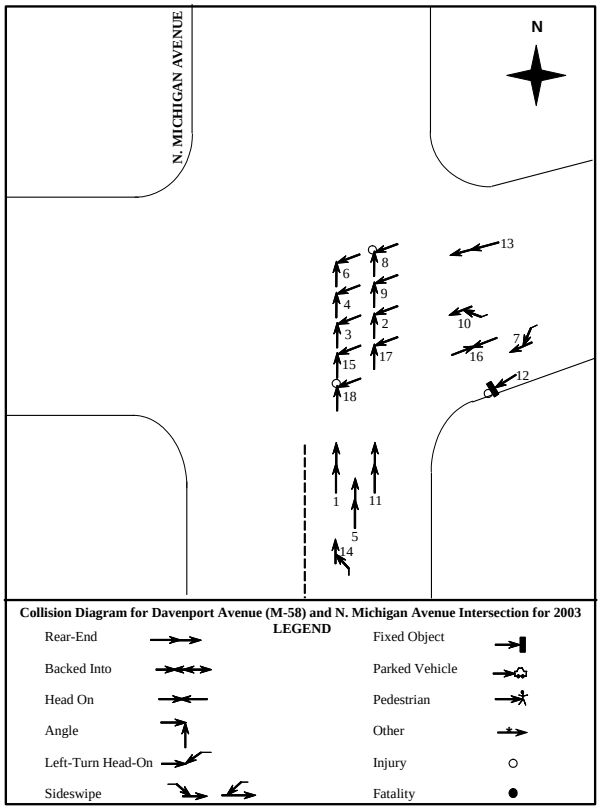


Figure 6.31.3. Collision Diagram for N. Michigan Avenue and Davenport Ave. for 2003

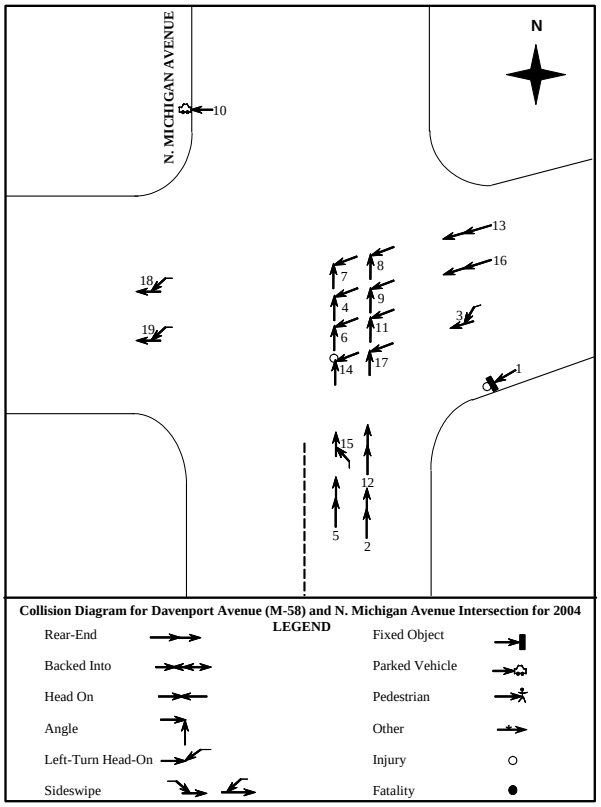


Figure 6.31.4. Collision Diagram for N. Michigan Avenue and Davenport Ave. for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.55 per million entering vehicles, which is nominal, given the approach volume at the intersection.

The crash distribution, based upon types of crashes, is shown in Figure 6.31.5. Angle and rear-end crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes includes 17 angle crashes, and nine rear end crashes.

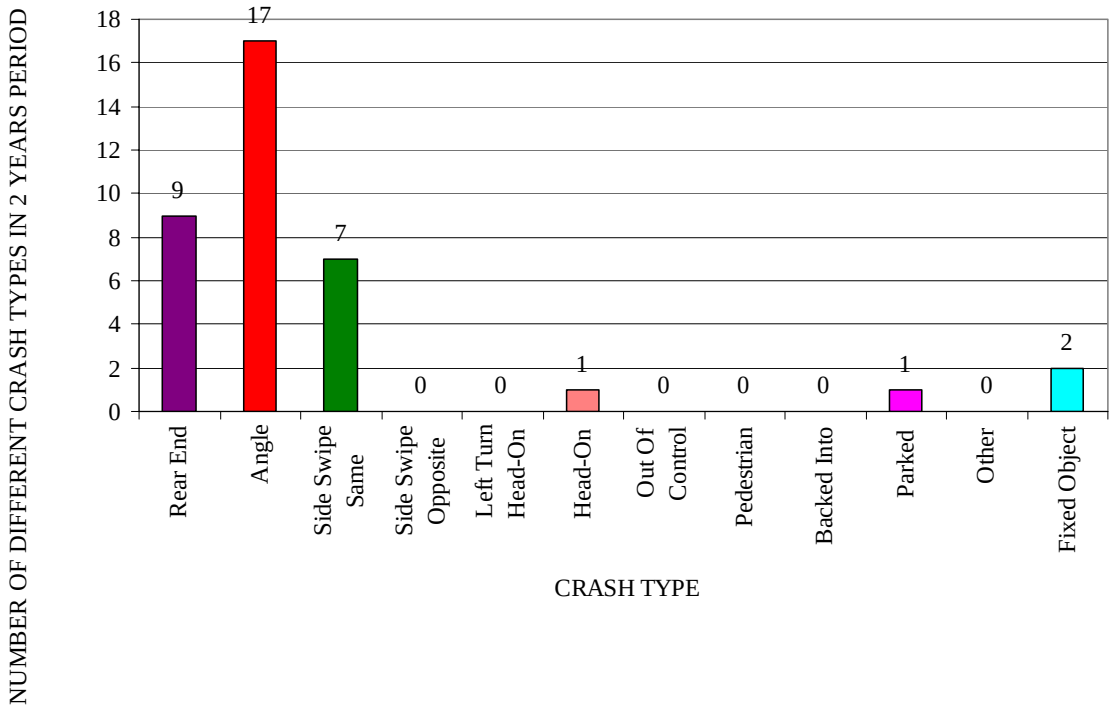


Figure 6.31.5 Two Year (2003-2004) crash experience at North Michigan Avenue and Davenport Avenue Intersection

Safety Deficiencies and Recommended Improvements

Majority of the crashes occurred at the intersection of North Michigan Avenue and Davenport Avenue are of angle and rear-end type. This may be due to the poor visibility of traffic signals

and inadequate clearance intervals and signal timings. The required clearance intervals are computed and provided in the proposed signal-timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed condition diagram in Figure 6.31.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Relocate stop bars and crosswalks on the northbound and westbound approaches. Upgrade pavement markings on all the approaches.

The existing yellow and all-red intervals are shown in Table 6.31.1, along with the proposed lengths of these intervals and their potential benefits.

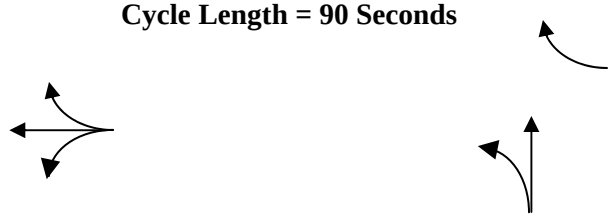
Table 6.31.1 Existing and Proposed Clearance Intervals for the North Michigan Avenue and Davenport Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
WB Protected Phase	4.0	1.0	3.5	1.4	Less rear end, angle, and injury crashes
NB Protected Phase	4.1	1.0	3.5	2.2	

The proposed PM peak hour signal timing plans based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



WB Davenport	**NB N. Michigan**
G = 40.1 sec	G = 39.3 sec
Y = 3.5 sec	Y = 3.5 sec
*R = 45.0 sec	*R = 45.0 sec
AR = 1.4 sec	AR = 2.2 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for Davenport Avenue and N. Michigan Avenue to 3.5 seconds.
- Increase the all-red interval for Davenport Avenue to 1.4 seconds and for N. Michigan Avenue to 2.2 seconds.
- Upgrade the existing span-wire configuration to mast-arm signal installation.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Relocate stop bars and crosswalks on the westbound and northbound approaches, as per MMUTCD guidelines.
- Upgrade pavement markings on all the approaches.
- Provide stop signs for all the driveways located near the intersection.
- Optimize the signal timings for AM peak and off-peak hours and implement the optimized signal timing plan for PM peak hour.

The proposed diagram in Figure 6.31.6 shows the proposed improvements which include installation of mast-arm support for the signals, upgrading the pavement markings on all the approaches and the proposed signal timing plan.

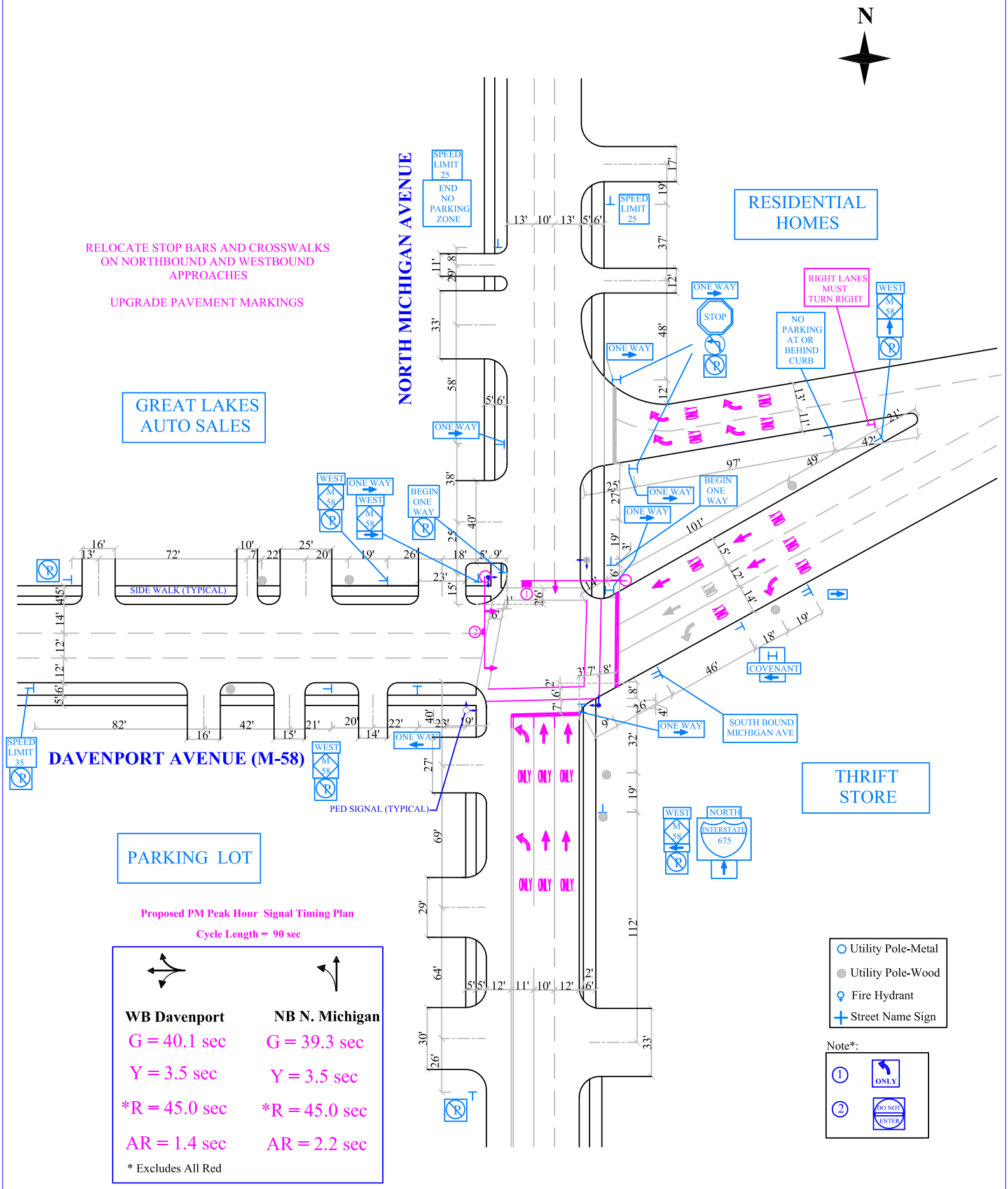


Figure 6.31.6 Proposed Diagram of Davenport Avenue (M-58) and North Michigan Avenue Intersection (Not to Scale)

Operational Evaluation

The results of the level of service analysis for the intersection of North Michigan Avenue and Davenport Avenue indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 18.9 seconds per vehicle.

With the modified signal timings and the proposed improvements, the overall intersection level of service will remain at B with a delay of 18.3 seconds per vehicle.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.31.2.

Table 6.31.2.
North Michigan Avenue and Davenport Avenue
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
NB North Michigan Avenue	20.5	C	17.8	B
WB Davenport Avenue	17.6	B	18.7	B
Overall Intersection	**18.9**	**B**	**18.3**	**B**

32. North Michigan Avenue and State Street (M-58)

The intersection of North Michigan Avenue and State Street (M-58) is located in the central northern part of Saginaw. State Street (M-58) runs in the east direction and North Michigan Avenue runs in the north-south direction. State Street is a state trunk line, also known as M-58 and North Michigan Avenue is a major arterial.

State Street near the intersection with North Michigan Avenue, has an average daily traffic volume of approximately 11,200 vpd as per MDOT 2004 data.

State Street is a one way street which consists of two lanes including a through lane and a shared through and left-turn lane. East of the intersection, the street is divided into a 23 foot two-lane road which functions as an entrance ramp for South I-675 and a 17 foot one-lane road connecting to Davenport Avenue. State Street is 28 feet wide west of the intersection.

North Michigan Avenue consists of three through lanes and one receiving lane to the south of the intersection and to the north it consists of one through lane and three receiving lanes. The pavement width of Michigan Avenue is 43 feet to the south of the intersection and to the north it is 44 feet.

The speed limit for Michigan Avenue is 25 miles per hour.

Land Use

The intersection of North Michigan Avenue and State Street has a car sales center on the northwest corner, Tim Horton's on the southwest corner and an Action Thrift Store on the northeast corner. There are multiple driveways near the intersection which serve the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of North Michigan Avenue and State Street.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.32.1. Details of the intersection approaches are shown in photographs 32a through 32d.

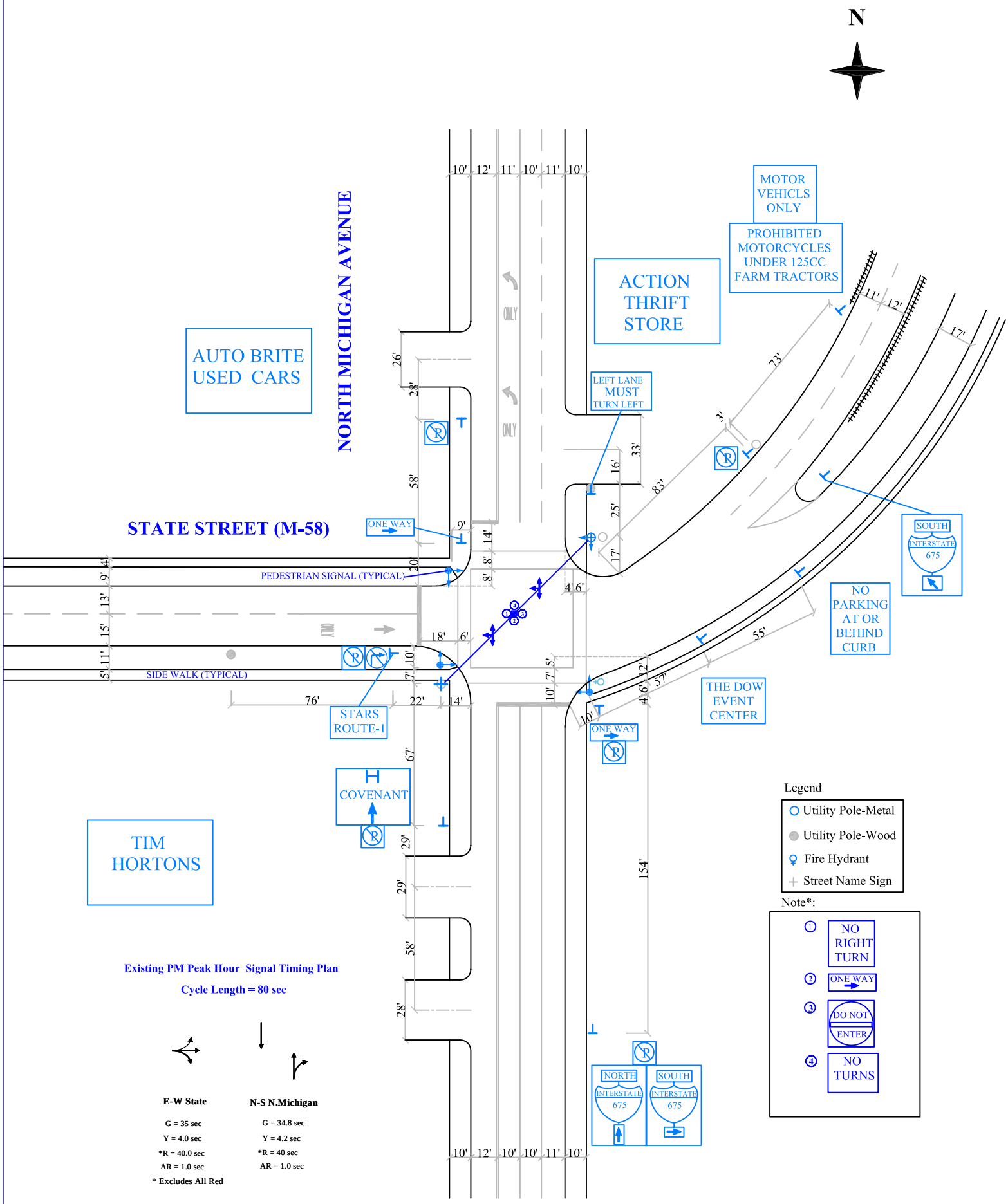


Figure 6.32.1.Existing Condition Diagram of State Street (M-58) and North Michigan Avenue Intersection (Not to Scale)

Photograph 32a. NB North Michigan Avenue at State Street

Photograph 32b. SB North Michigan Avenue at State Street

Photograph 32c. EB State Street at North Michigan Avenue

Photograph 32d. EB I-675 S Ramp from State Street

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on a clear day on April 18, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:30 to 5:30 PM and is presented in Figure 6.32.2.

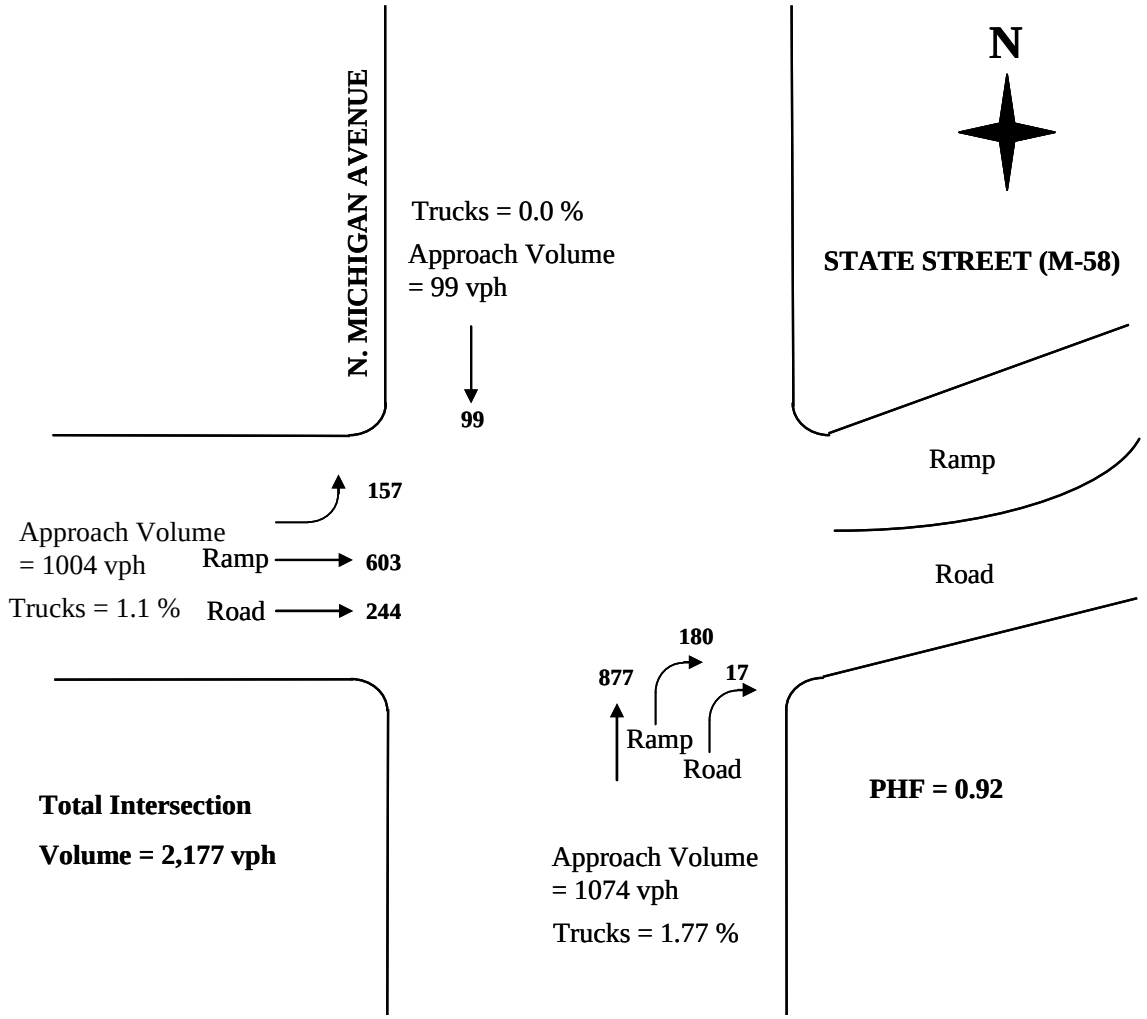


Figure 6.32.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes for the Intersection of North Michigan Avenue and State Street

Traffic Control

This intersection is controlled by a two-phase traffic signal with an 80-second cycle length. The existing clearance interval includes a yellow interval of 4.0 seconds and an all-red interval of 1.0 second for State Street and a yellow interval of 4.2 seconds and an all red interval of 1.0 second for N. Michigan Avenue. The existing PM peak hour signal timing plan is shown on the existing condition diagram in Figure 6.32.1.

Crash Analysis

A total of 41 crashes were recorded for the years 2003 and 2004 combined, with an average annual crash frequency of 21 crashes. Collision diagrams were prepared based on the UD-10 reports information to identify crash patterns. The collision diagrams for the years 2003 and 2004 for the intersection of North Michigan Avenue and State Street are shown in the following Figures 6.32.3 and 6.32.4, respectively.

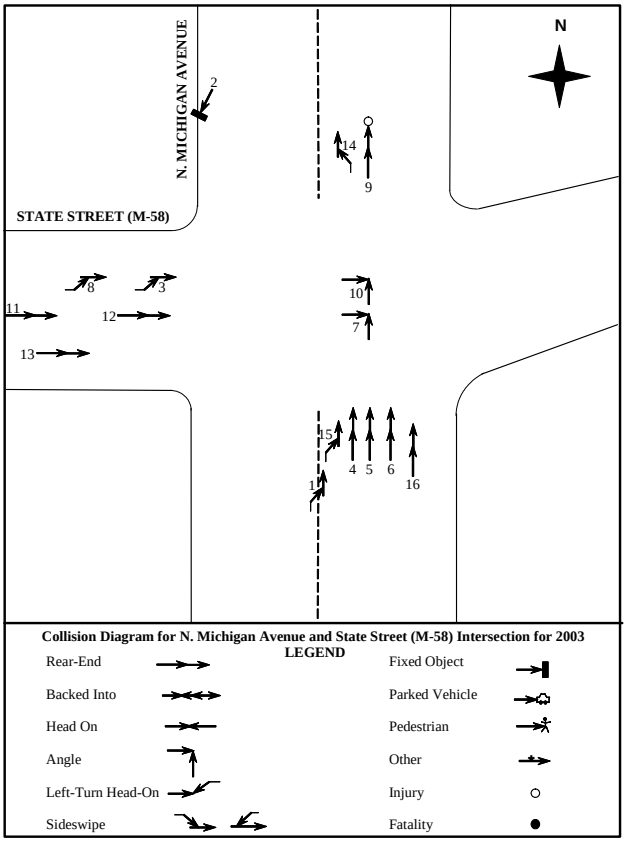


Figure 6.32.3 Collision Diagram for North Michigan Avenue and State Street for 2003

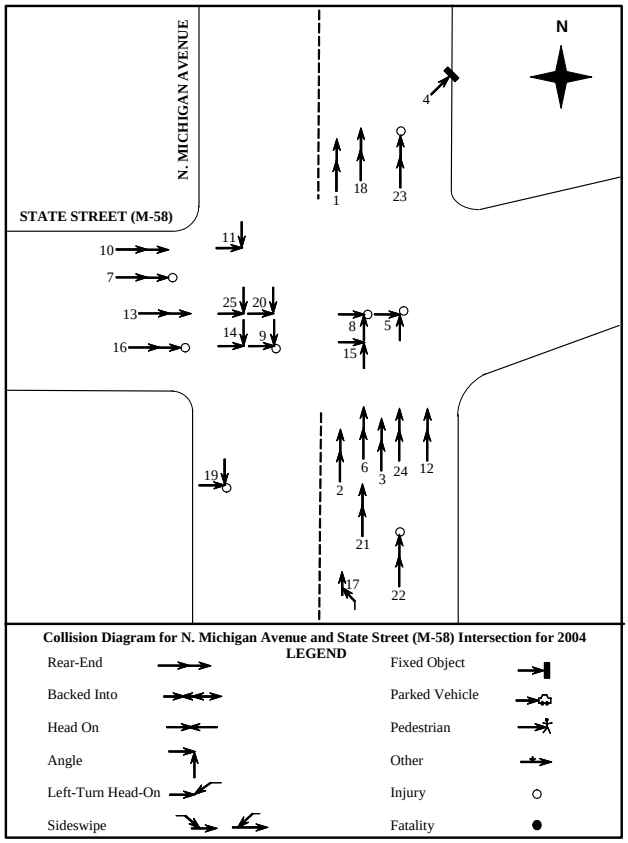


Figure 6.32.4 Collision Diagram for North Michigan Avenue and State Street for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 2.11 per million entering vehicles, which is significant considering the approach volume at the intersection.

The crash distribution, based upon types of crashes, is shown in Figure 6.32.5 Rear-end, angle and side swipe same crashes are the predominant crashes reported at this intersection. The corresponding two-year crash frequency for these crashes includes 22 rear-end crashes, 11 Angle and six side swipe same crashes.

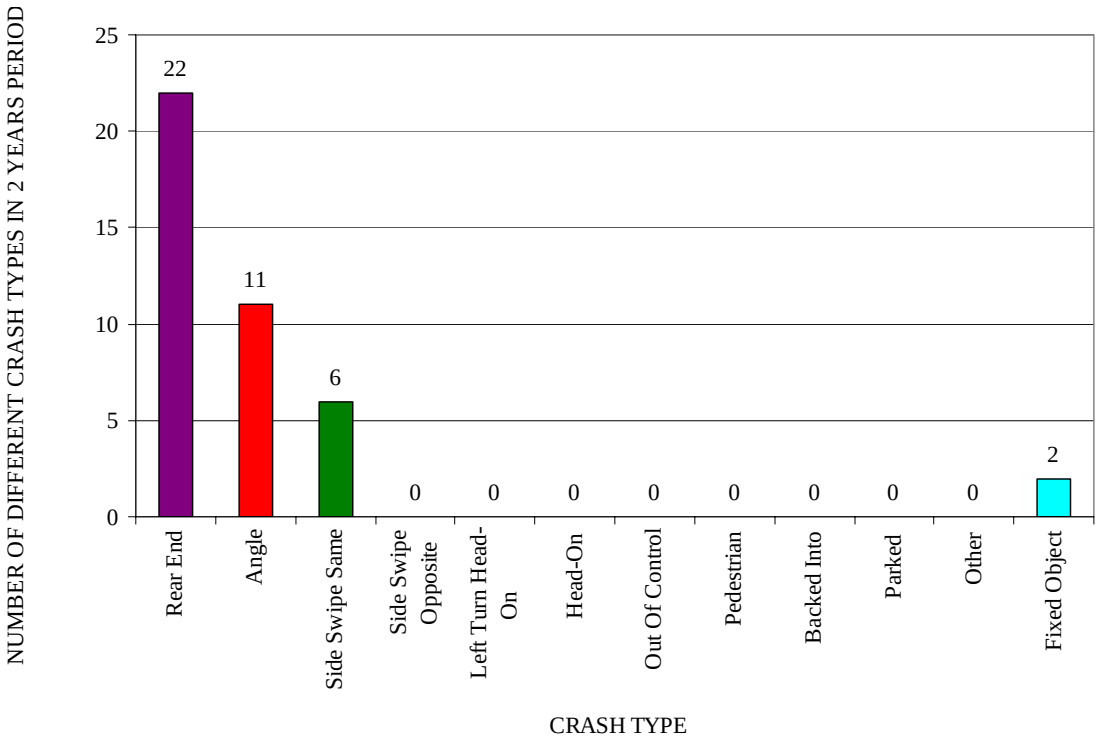


Figure 6.32.5 Two Year (2003-2004) crash experience at North Michigan Avenue and State Street Intersection

Safety Deficiencies and Recommended Improvements

Majority of the crashes occurring at the intersection of North Michigan Avenue and State Street (M-58) are angle and rear-end. This may be due to the poor visibility of traffic signals and inadequate clearance intervals and signal timings. The required clearance intervals are computed and provided in the proposed signal-timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.32.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Relocate stop bars on the northbound, southbound and eastbound approaches. Upgrade pavement markings on the southbound and eastbound approaches.

The proposed condition diagram in Figure 6.32.6 shows the proposed improvements which include new installation of mast-arm support for the signals, upgrading the pavement markings on the southbound and eastbound approaches and the proposed signal timing plan.

The existing yellow and all-red intervals are shown in Table 6.32.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.32.1. Existing and Proposed Clearance Intervals for the North Michigan Avenue and State Street Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
EB Protected Phase	4.0	1.0	3.5	1.5	Less rear end, angle, left-turn head-on and injury crashes
N-S Through Phase	4.2	1.0	3.5	1.6	

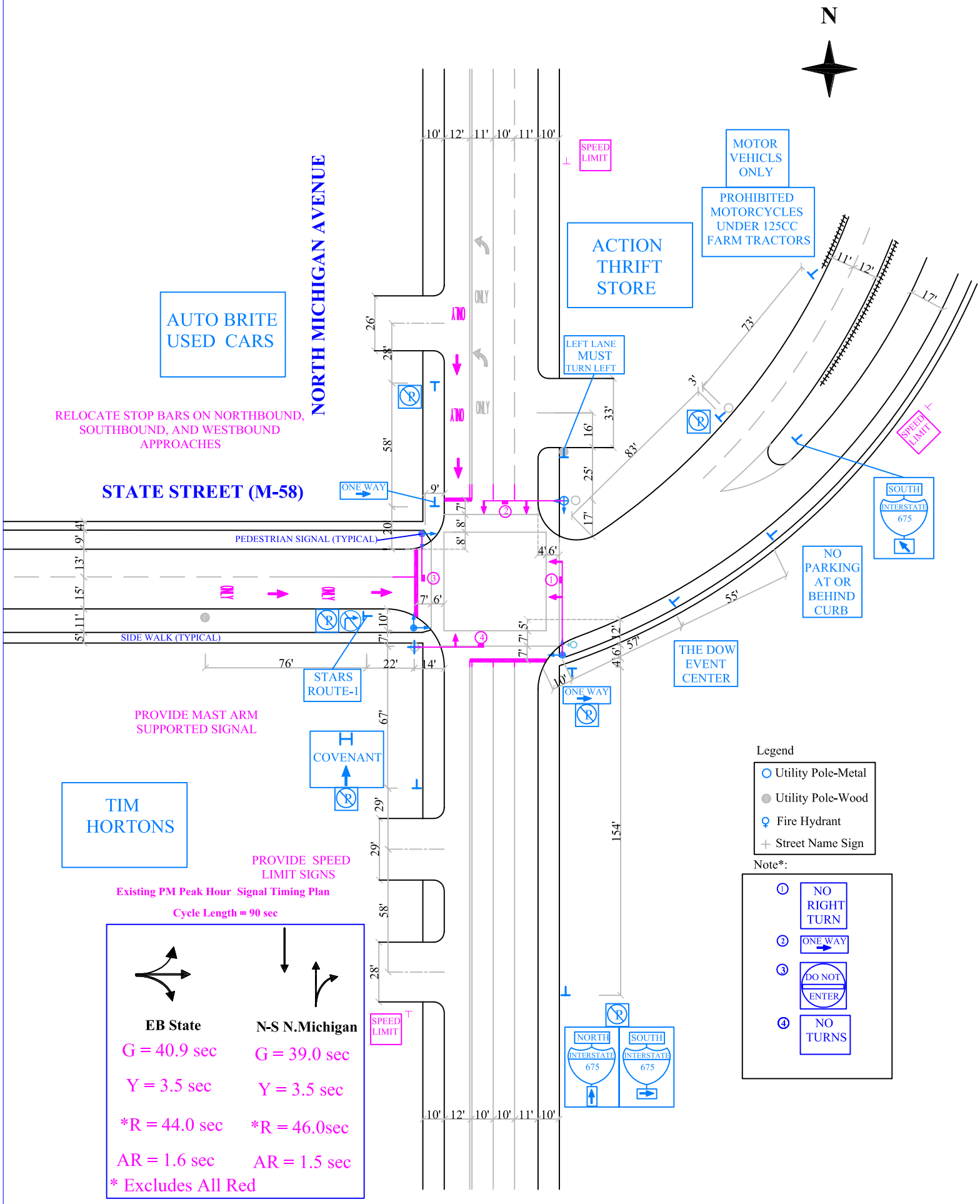
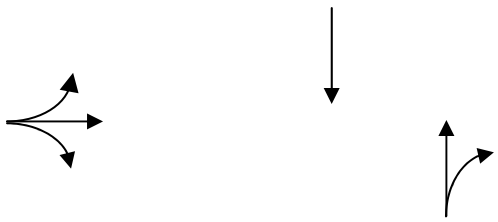


Figure 6.32.6 Proposed Diagram of State Street (M-58) and North Michigan Avenue Intersection (Not to Scale)

The proposed PM peak hour signal timing plans based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



EB State	**N-S N.Michigan**
G = 40.9 sec	G = 39.0 sec
Y = 3.5 sec	Y = 3.5 sec
*R = 44.0 sec	*R = 46.0sec
AR = 1.6 sec	AR = 1.5 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval for N. Michigan Avenue and State Street to 3.5 seconds.
- Increase the all-red interval for State Street to 1.6 seconds and for N. Michigan Avenue to 1.5 seconds.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Relocate stop bars on the northbound, southbound and eastbound approaches, as per MMUTCD guidelines.
- Upgrade pavement markings on the southbound and eastbound approaches.
- Provide stop signs for all the driveways located near the intersection. Landowner should provide these stop signs.

- Optimize the signal timings for AM peak and off-peak hours and implement the optimized signal timing for PM peak hour.

Operational Evaluation

The results of the level of service analysis for the intersection of North Michigan Avenue and State Street indicate that, under existing conditions during the PM peak hour, overall intersection level of service is B with a delay of 16.1 seconds per vehicle.

With the modifications to the signal timings, changed cycle length and the noted improvements, the overall level of service of the intersection will remain at B with a delay of 17.9 seconds per vehicle. Although the overall delay has slightly increased due to inclusion of an adequate all-red interval and consistent cycle length along the Michigan corridor, this increased delay is acceptable since progression and enhanced safety have been achieved.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.32.2.

Table 6.32.2.
North Michigan Avenue and State Street
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB State Street	18.6	B	19.3	B
NB N. Michigan Avenue	14.2	B	16.6	B
SB N. Michigan Avenue	11.1	B	17.7	B
Overall Intersection	**16.1**	**B**	**17.9**	**B**

33. North Michigan Avenue and West Genesee Avenue

The intersection of North Michigan Avenue and West Genesee Avenue is located north of the City of Saginaw. North Michigan Avenue runs in the north-south direction and West Genesee Avenue runs in the east-west direction. Both North Michigan Avenue and West Genesee Avenue may be classified an arterials.

At the southbound approach, North Michigan Avenue consists of one exclusive left-turn lane, one exclusive through lane and one shared through and right-turn lane. The northbound approach consists of two through lanes. North Michigan Avenue is approximately 56 feet wide at the southbound approach and 42 feet wide at the northbound approach of the intersection.

West Genesee Avenue, east of intersection consists of two exclusive left-turn lanes, one exclusive through lane, and one exclusive right-turn lane. West of intersection, West Genesee Avenue has one exclusive left-turn lane, one exclusive through lane and one shared through and right-turn lane. West Genesee Avenue is approximately 68 feet wide at the east leg and 54 feet wide at the west leg of the intersection.

The speed limit for North Michigan Avenue is 25 mph and for West Genesee Avenue, it is 30 mph.

Land Use

The intersection of North Michigan Avenue and West Genesee Avenue has a Bakery Outlet on the northeast corner, a Walgreen pharmacy on the southeast corner and a Rite Aid Pharmacy on the southwest corner. There is Bringer Inn on the northwest corner of the intersection. There are multiple driveways very close to the intersection (within 150 feet from the edge), which serve the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all

other relevant information within an approximate radius of 150 feet around the intersection of North Michigan Avenue and West Genesee Avenue.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.33.1. Details of the intersection approaches are shown in photographs 33a through 33d.

Photograph 33a. EB West Genesee Avenue at North Michigan Avenue

Photograph 33b. WB West Genesee Avenue at North Michigan Avenue

Photograph 33c. NB North Michigan Avenue at West Genesee Avenue

Photograph 33d. SB North Michigan Avenue at West Genesee Avenue

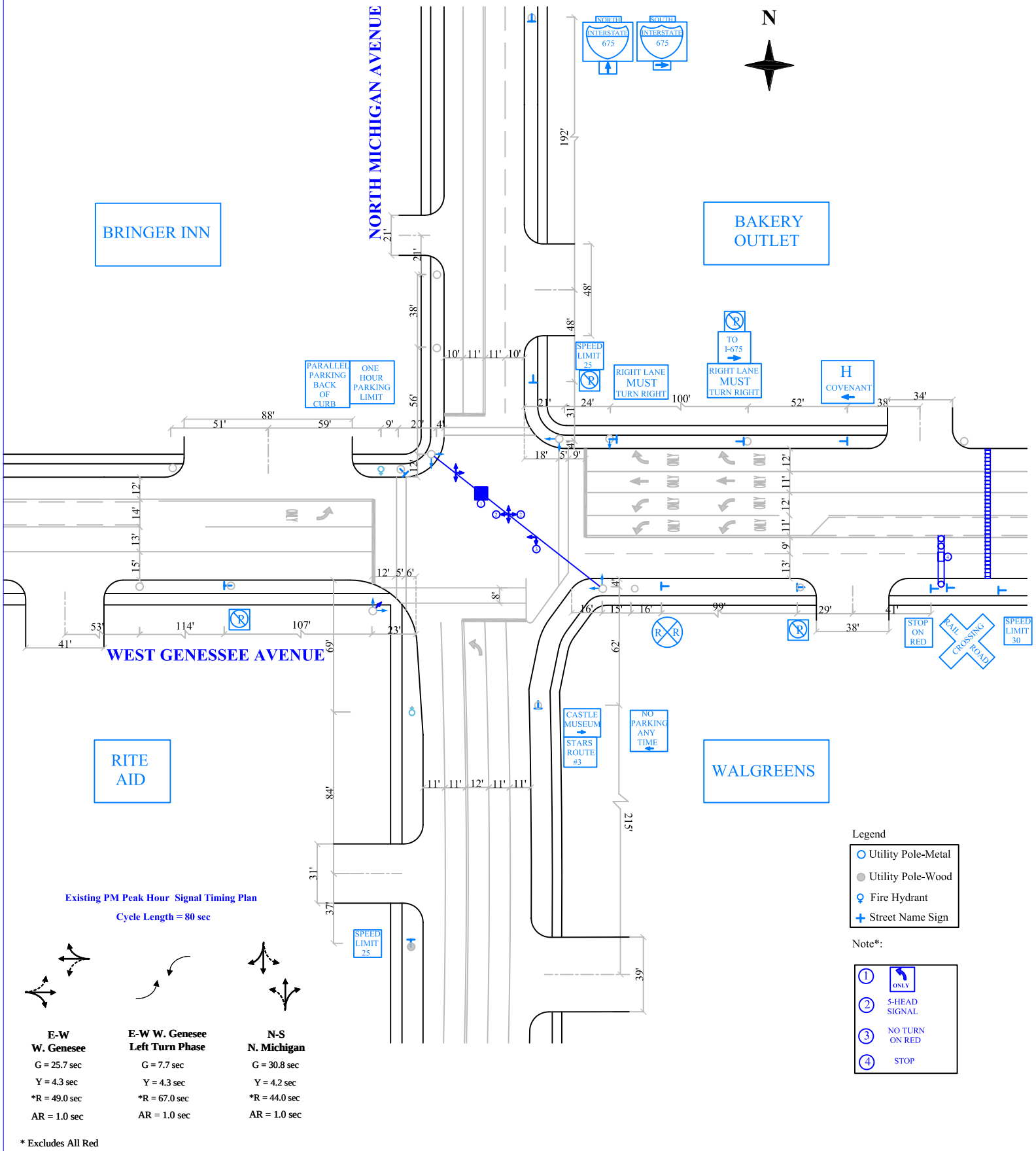


Figure 6.33.1. Existing Condition Diagram of North Michigan Avenue and West Genesee Avenue Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were collected manually on clear day on April 25, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4.30 PM to 5:30 PM and is presented in Figure 6.33.2.

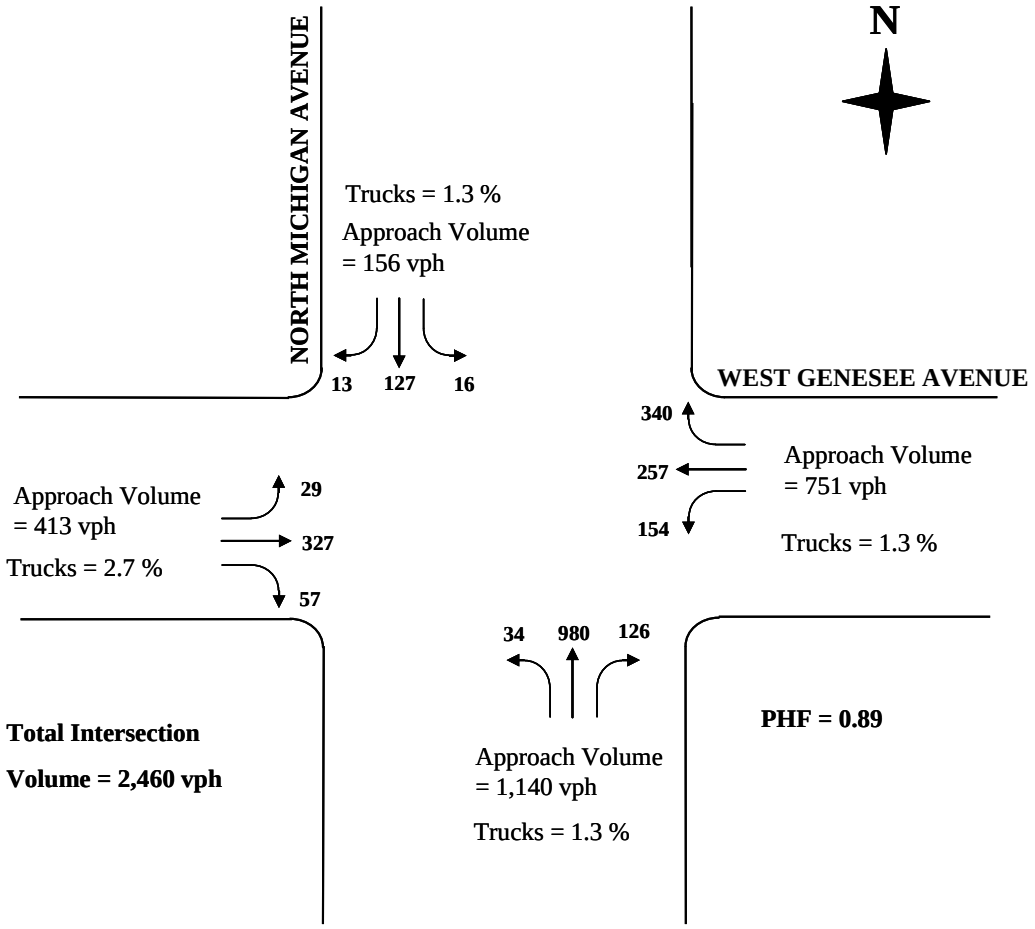


Figure 6.33.2. Existing PM Peak Hour (4:30-5:30 PM) Traffic Volumes at North Michigan Avenue and West Genesee Avenue

Traffic Control

This intersection is controlled by a three-phase span-wire traffic signal. The PM peak hour operates on a 80-second cycle. The existing clearance interval consists of a yellow interval of 4.2 seconds and an all-red interval of 1.0 second for each movement. The existing PM peak hour signal timing plan is shown in the existing condition diagram in Figure 6.33.1 (on page 407).

Crash Analysis

A total of 36 crashes were recorded for the years 2003 and 2004 combined with an average annual crash frequency of 18 crashes. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 reports to identify crash patterns. Figures 6.33.3 and 6.33.4 show the collision diagrams for these years respectively.

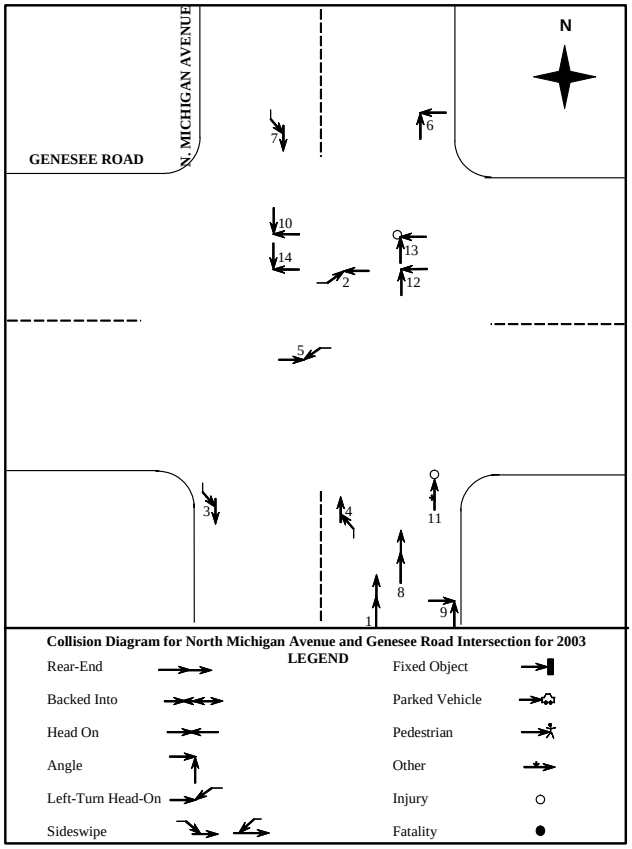


Figure 6.33.3 Collision Diagram for North Michigan Avenue and West Genesee Avenue for 2003

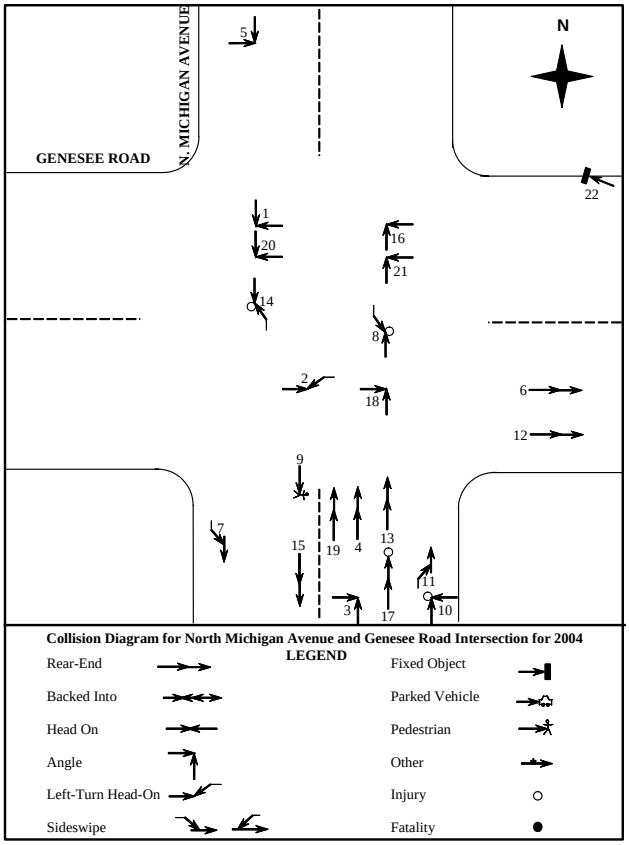


Figure 6.33.4 Collision Diagram for North Michigan Avenue and West Genesee Avenue for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.6 per million entering vehicles, which is nominal, considering the approach volume at the intersection. Angle crashes and rear-end crashes are of specific concern and might be due to improper clearance intervals. Left-turn head-on crashes also exist and might be due to interference of oncoming through traffic during the permissive phase.

The crash distribution, based upon types of crashes, is shown in Figure 6.33.5. Angle crashes, rear-end crashes, side swipe same crashes and left-turn head-on crashes are the predominant type of crashes reported at the intersection. The corresponding two year crash frequency for these crashes includes nine rear end crashes, fourteen Angle crashes, five side swipe same crashes and five left-turn head-on crashes.

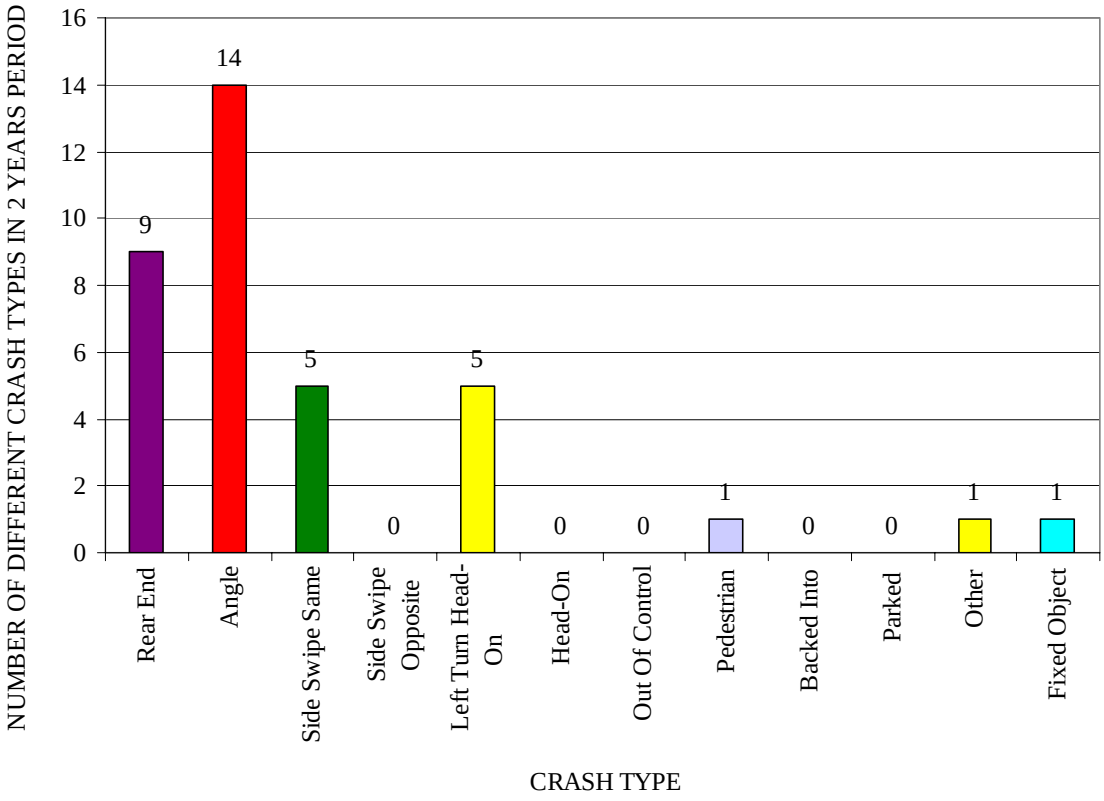


Figure 6.33.5 Two Year (2003-2004) Crash Experience at North Michigan Avenue and West Genesee Avenue Intersection

Safety Deficiencies and Recommended Improvements

Majority of the crashes occurring over the intersection are of the angle, rear-end, side swipe same and left-turn head-on type. These might be occurring due to several possible reasons including restricted or poor visibility of traffic signals, improper lane changing and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed condition diagram in Figure 6.33.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Relocate stop bars on the all approaches as per MMUTCD guidelines. Upgrade pavement markings on all the approaches.

The proposed diagram in Figure 6.33.6 shows recommended improvements including upgrading the existing span-wire signal support system to a mast-arm support signal installation, upgrading the pavement markings on all approaches and the proposed signal timing plan.

The existing yellow and all-red intervals are shown in Table 6.33.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.33.1. Existing and Proposed Clearance Intervals for the North Michigan Avenue and West Genesee Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Left turn Phase	4.3	1.0	-	-	Less left-turn head-on, angle, rear-end and injury crashes
E-W Through Phase	4.3	1.0	3.5	1.7	
N-S Through Phase	4.2	1.0	3.5	2.5	
WB Split phase	-	-	3.5	1.7	
NB Split phase	-	-	3.5	2.5	

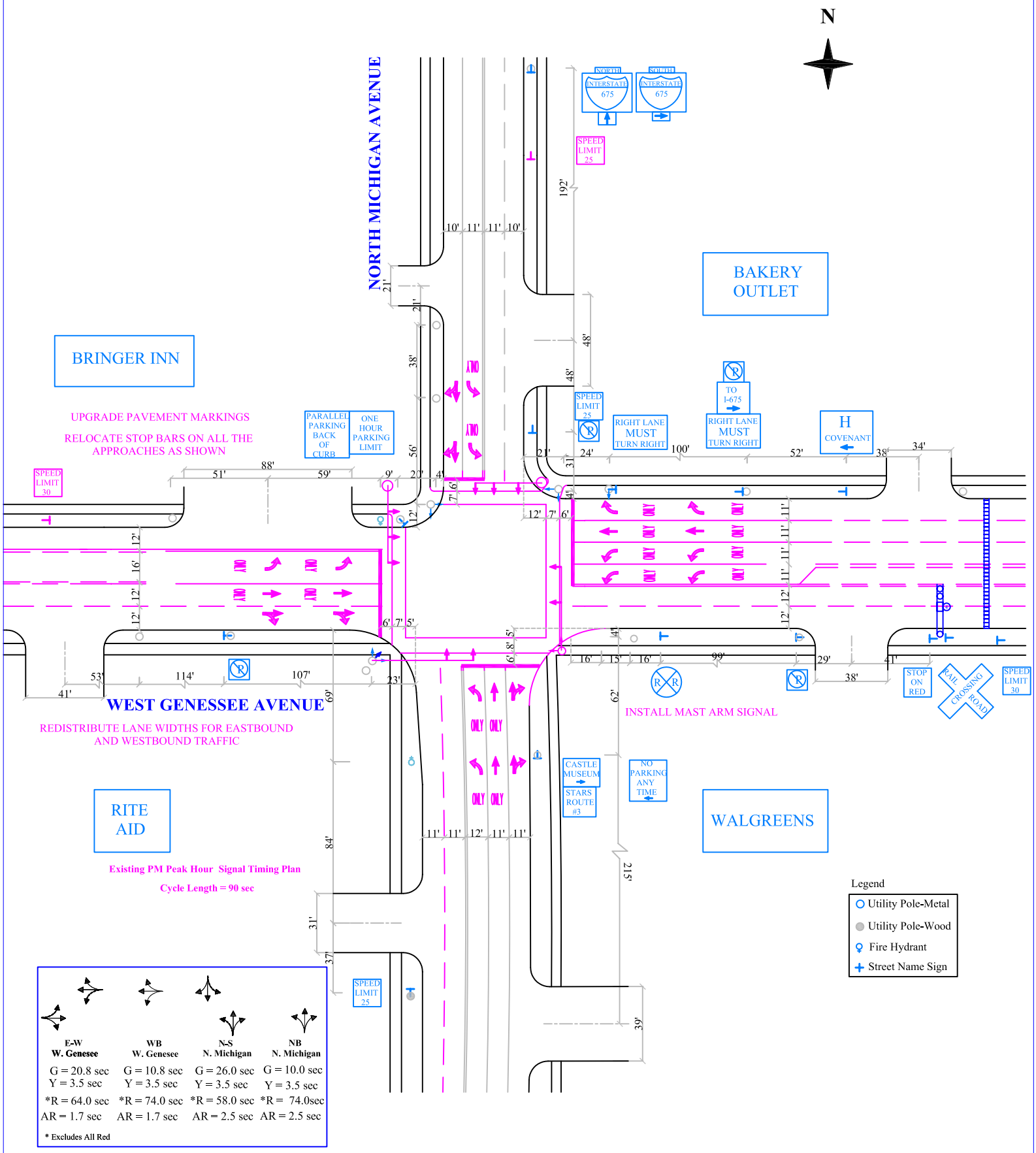
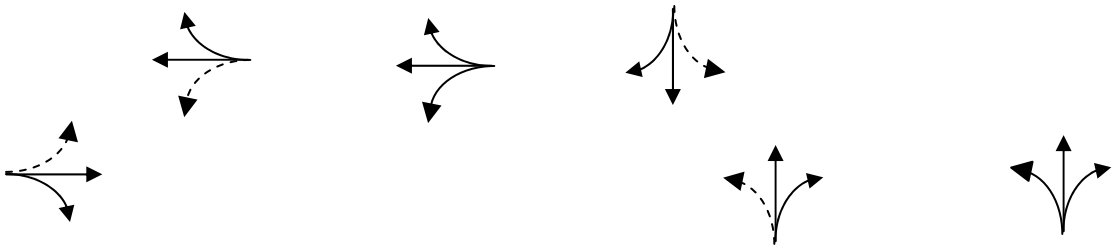


Figure 6.33.6 Proposed Diagram of North Michigan Avenue and West Genesee Avenue Intersection (Not to Scale)

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



E-W **W. Genesee**	**WB** **W. Genesee**	**N-S** **N. Michigan**	**NB** **N. Michigan**
G = 18.8 sec	G = 8.8 sec	G = 32.0 sec	G = 8.0 sec
Y = 3.5 sec	Y = 3.5 sec	Y = 3.5 sec	Y = 3.5 sec
*R = 66.0 sec	*R = 76.0 sec	*R = 52.0 sec	*R = 76.0sec
AR = 1.7 sec	AR = 1.7 sec	AR = 2.5 sec	AR = 2.5 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval to 3.5 seconds for all approaches.
- Increase the all-red interval to 1.7 seconds for West Genesee Avenue and 2.5 seconds for N. Michigan Avenue.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Redistribute lane widths for eastbound and westbound approaches as shown in the proposed diagram in figure 6.33.6.
- Relocate stop bars and crosswalks on the all approaches, as per MMUTCD guidelines.
- Upgrade pavement markings on the all approaches.
- Provide stop signs for all the driveways located near the intersection. Landowners should provide these stop signs.

- Optimize signal timing for AM and off-peak hours.
- Implement the optimized PM signal timings.

Operational Evaluation

The results of the level of service analysis for the intersection of North Michigan Avenue and West Genesee Street indicate that, for the existing conditions during the PM peak hour, the overall intersection level of service is C with 23.9 seconds per vehicle of delay.

With the modifications to the signal timing, phasing and the noted improvements, the overall intersection level of service will remain at C with a delay of 23.1 seconds per vehicle. The modified signal timing and changed cycle length along with the adjusted clearance intervals will ensure enhanced safety.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.33.2.

Table 6.33.2.
North Michigan Avenue and West Genesee Avenue
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Genesee Avenue	21.9	C	30.5	C
WB Genesee Avenue	19.5	B	20.1	C
NB Michigan Avenue	28.4	C	20.5	C
SB Michigan Avenue	15.5	B	33.4	C
Overall Intersection	**23.9**	**C**	**23.1**	**C**

34. North Michigan Avenue and Court Street

The intersection of North Michigan Avenue and Court Street is located in the central part of the City of Saginaw. North Michigan Avenue runs in the northeast and southwest direction, while Court Street at an approximate 60-degree angle to North Michigan Avenue runs in the northwest and southeast direction. North Michigan Avenue functions as an arterial while Court Street is a collector road.

At the southwest leg, North Michigan Avenue consists of one through lane, one shared through/right-turn lane and one center lane for left-turning movement. The northeast part of North Michigan Avenue consists of one exclusive through lane, one shared through/right-turn lane and one center lane. The pavement width of southwest North Michigan Avenue is approximately 60 feet while northeast North Michigan Avenue is approximately 62 feet wide.

The northwest and southeast legs of Court Street consist of one exclusive left-turn lane, one through lane and one shared through/right-turn lane. The northwest part of Court Street is approximately 52 feet wide while the southeast leg is approximately 61 feet wide. The rightmost receiving lane of Court Street in the southeast leg is used for car parking.

The posted speed limit for both North Michigan Avenue and Court Street is 25 miles per hour.

Land Use

The intersection of North Michigan Avenue and Court Street has two parking lots on the north and south corners. Some local shops are located on the eastern corner. City hall is located on the western corner. There are two driveways within 150 feet of the intersection which serves the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection..

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.34.1. Details of the intersection approaches are shown in photographs 34a through 34d. The photographs show on-street parking at the northwest bound and southwest bound approaches. The photographs also show the span-wire signal installation at the intersection along with cracks on the pavement surface at the northeast approach.

Photograph 34a. SE bound Court Street at North Michigan Avenue Ave

Photograph 34b. NW bound Court Street at North Michigan Avenue Ave

Photograph 34c. NE bound North Michigan Avenue at Court Street

Photograph 34d. SW bound North Michigan Avenue at Court Street

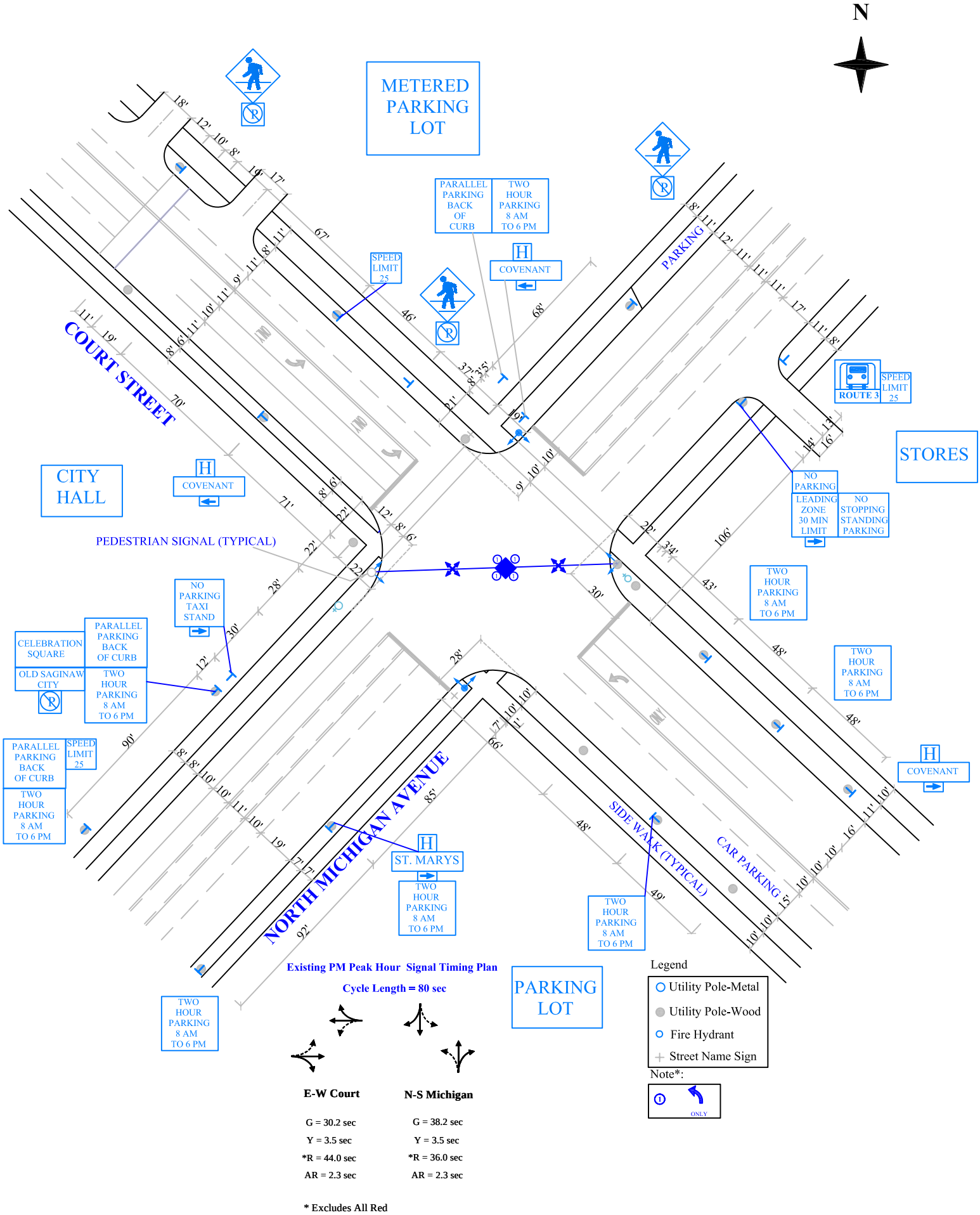


Figure 6.34.1. Existing Condition Diagram of North Michigan Avenue and Court Street Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were collected on a clear day on April 18, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The peak hourly volume occurred from 4:15 PM to 5:15 PM and is presented in Figure 6.34.2. The traffic patterns indicate no significant difference in the volume in the NE-SW and NW-SE direction.

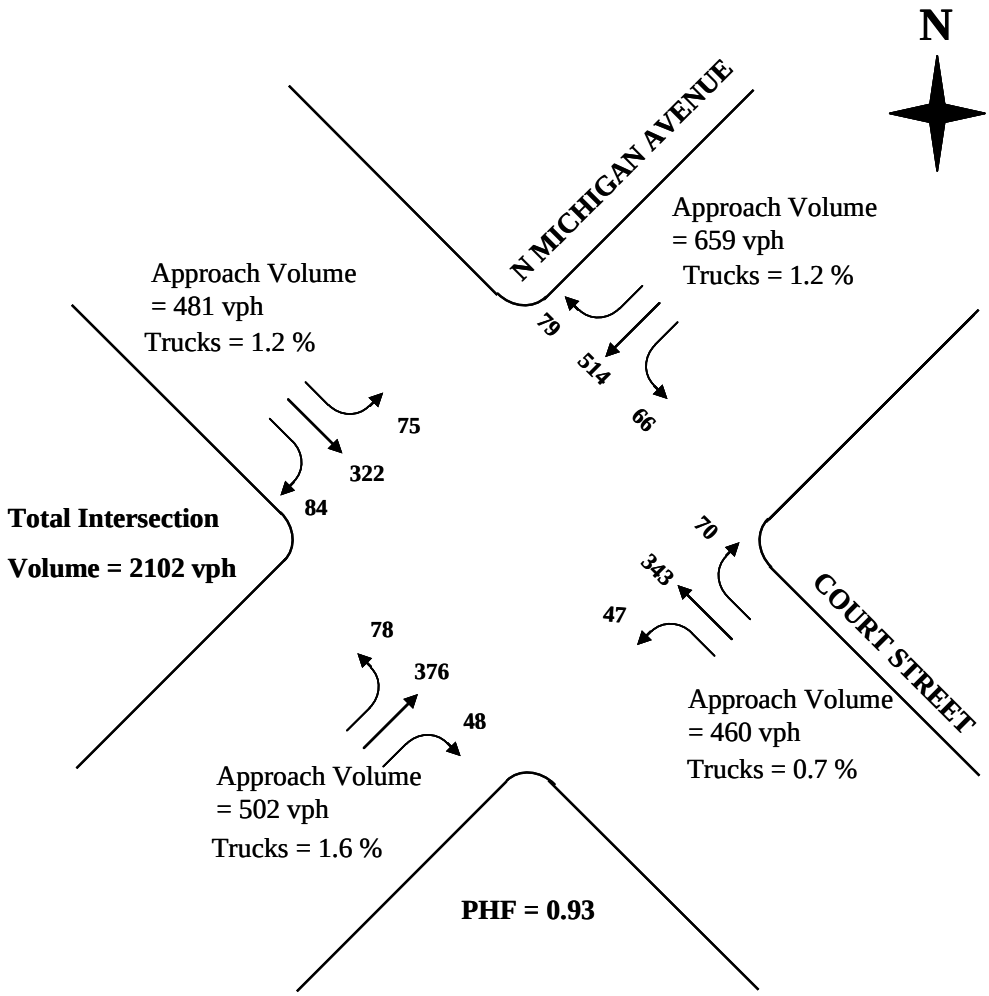


Figure 6.34.2. Existing PM Peak Hour (4:15-5:15 PM) Traffic Volumes for the Intersection of North Michigan Avenue and Court Street

Traffic Control

This basic signal timing plan is given by a two-phase design operating on an 80-second cycle length during the PM peak hour. The existing yellow interval is 3.5 seconds for both phases and

an all-red interval of 2.3 seconds for both phases. The existing PM peak hour signal timing plan is shown in Figure 6.34.1.

Crash Analysis

A total of 26 crashes were recorded for the years 2003 - 2004 combined, with an average annual crash frequency of 13 crashes. Collision diagrams for these years (2003 and 2004) were prepared based on the UD-10 reports to identify crash patterns. Figures 6.34.3 and 6.34.4 show the collision diagrams for these years respectively.

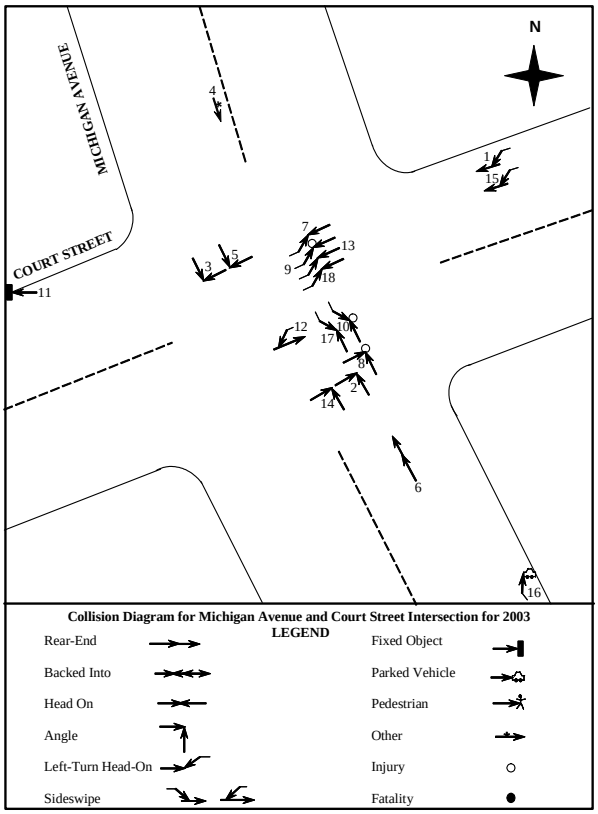


Figure 6.34.3. Collision Diagram for N. Michigan Avenue and Court Street for 2003

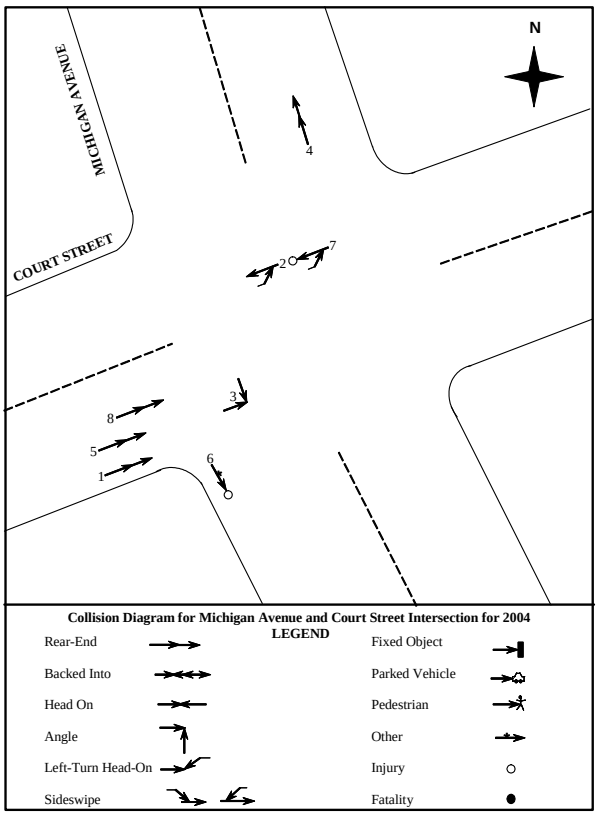


Figure 6.34.4. Collision Diagram for N. Michigan Avenue and Court Street for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.35 per million entering vehicles, which is nominal, considering the approach volume at the intersection. From the collision diagrams as provide in the figures above, the intersection area seems to have angle, left-turn head-on and a few rear-end crashes.

The crash distribution, based upon types of crashes, is shown in Figure 6.34.5. Angle, left-turn head-on, rear-end and side-swipe opposite crashes are the predominant type reported at the intersection. The corresponding two-year crash frequency for these crashes include six angle crashes, six left-turn head-on crashes, five rear end crashes and three side-swipe opposite

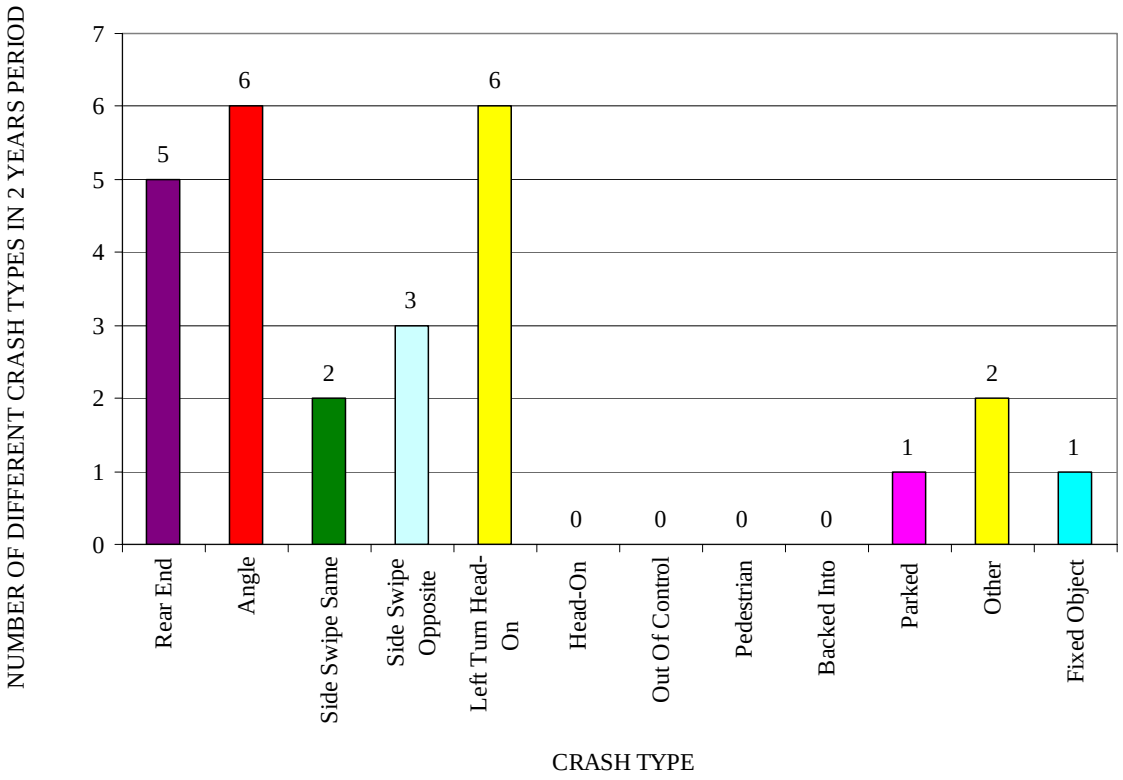


Figure 6.34.5 Two Year (2003-2004) crash experience at North Michigan Avenue and Court Street Intersection

Safety Deficiencies and Recommended Improvements

Majority of the crashes occurring at the intersection are of the left-turn head-on, angle, rear-end and side-swipe opposite type. This may be occurring due to possible reasons including restricted or poor visibility of traffic signals, the left-turn permissive phase and inadequate clearance intervals. The required clearance interval is computed and provided in the proposed signal-timing plan.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.34.6.

The proposed cycle length is 90 seconds and is compatible with the progression along the Michigan Corridor.

Post-mounted traffic signals with "LEFT" illuminated signs should be installed at the northeast and southwest corners of the intersection to assist left turning vehicles.

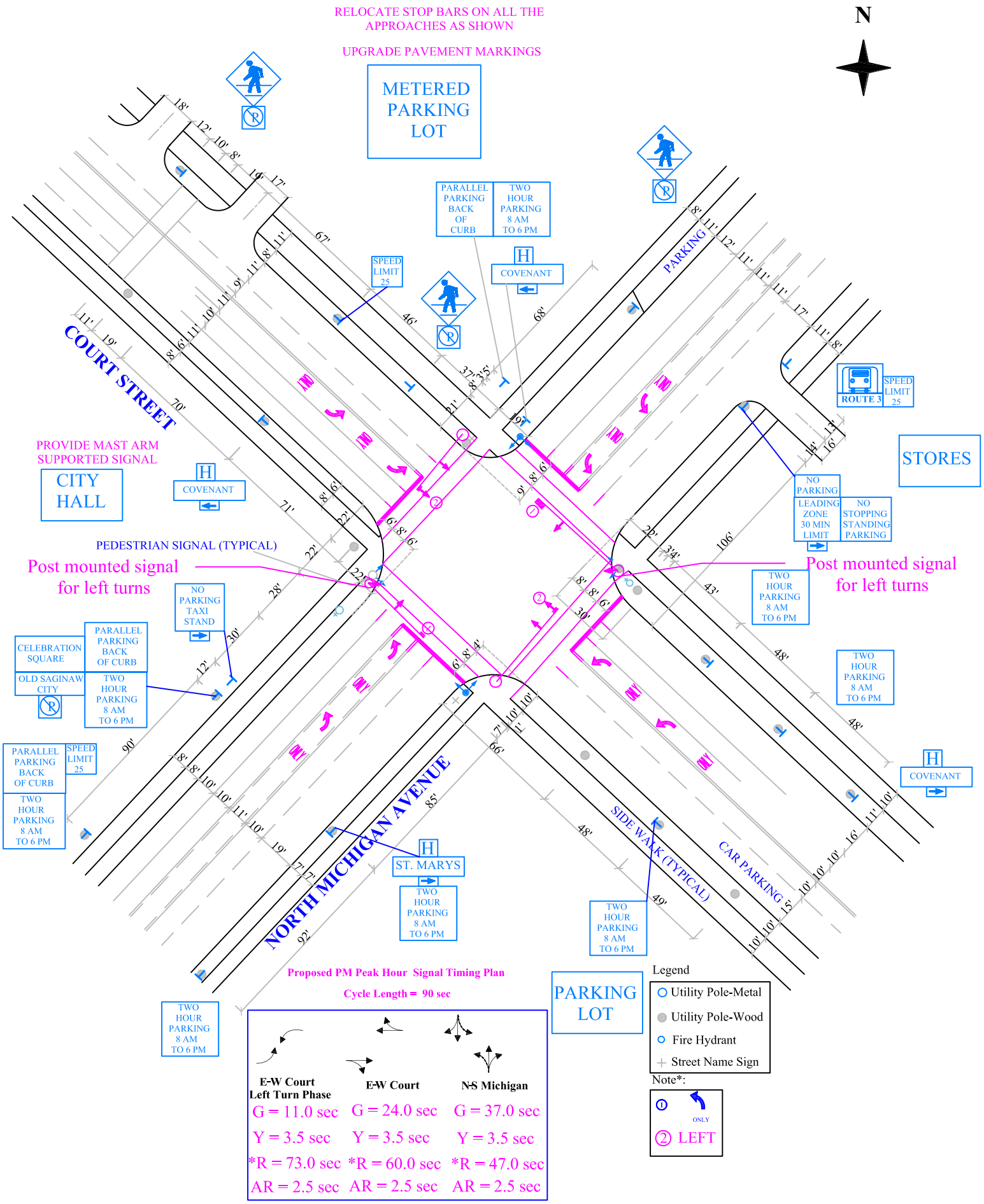
Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The proposed diagram in Figure 6.34.6 shows the recommended improvements which include installation of mast-arm support for the signals, post-mounted traffic signals for the left-turns, upgrading the pavement markings on the all approaches, relocation of stop bar and crosswalks on all approaches and the proposed signal timing plan.

The existing yellow and all-red intervals are shown in Table 6.34.1, along with proposed lengths of these intervals and their potential benefits.

Table 6.34.1. Existing and Proposed Clearance Intervals for the North Michigan Avenue and Court Street Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	3.5	2.3	3.5	2.5	Less left-turn head-on, angle, rear-end and injury crashes
E-W Through Phase	3.5	2.3	3.5	2.5	
E-W Left Turn Phase	-	-	3.5	2.5	

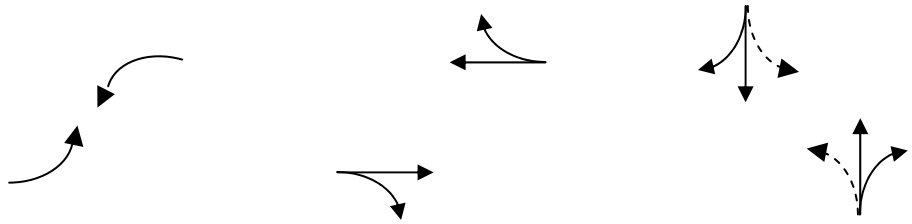


* Excludes All Red

Figure 6.34.6 Proposed Diagram of North Michigan Avenue and Court Street Intersection (Not to Scale)

The proposed PM peak hour signal timing plan with a cycle length of 90 second cycle is shown below:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 90 Seconds



E-W Court Left Turn Phase	E-W Court	N-S Michigan
G = 11.0 sec	G = 24.0 sec	G = 37.0 sec
Y = 3.5 sec	Y = 3.5 sec	Y = 3.5 sec
*R = 73.0 sec	*R = 60.0 sec	*R = 47.0 sec
AR = 2.5 sec	AR = 2.5 sec	AR = 2.5 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Increase the all-red interval to 2.5 seconds for North Michigan Avenue and Court Street.
- Relocate stop bars and crosswalks on all approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Provide a modified three-phase signal timing plan.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for the left-turns.
- Upgrade pavement markings for all approaches with proper directional arrows.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowners should provide these stop sign.
- Optimize signal timing for AM and off-peak hours.
- Implement the optimized PM signal timings.

Operational Evaluation

The results of the level of service analysis for the intersection of North Michigan Avenue and Court Street indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 16.2 seconds per vehicle.

With the modifications to the signal timings, phasing and the noted improvements, the overall intersection level of service will increase to C with a delay of 27.7 seconds per vehicle. The overall delay at the intersection will increase due to the inclusion of a protected left-turn phase and the provision of adequate all-red interval. However, the safety at the intersection will be improved as well as progression will occur down Michigan Avenue.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.34.2.

Table 6.34.2.
North Michigan Avenue and Court Street
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Court Street	19.1	B	33.1	C
WB Court Street	20.0	C	28.8	C
NB N. Michigan Avenue	13.0	B	19.5	B
SB N. Michigan Avenue	13.8	B	29.2	C
Overall Intersection	**16.2**	**B**	**27.7**	**C**

35. Mackinaw Street, South Michigan Avenue and Gratiot Avenue

The intersection of Mackinaw Street, South Michigan Avenue and Gratiot Avenue is located in the central region of the City of Saginaw. Gratiot Avenue is considered as a state trunk line (M-46) west of the intersection until its major share of vehicular movement is divided into Stephens and Williams Street. Hence, Gratiot Avenue near the intersection is not recognized as M-46 and eventually ends at this intersection. South Michigan Avenue runs northeast and southwest and Mackinaw Street runs in northwest and southeast. South Michigan Avenue works as an arterial whereas Mackinaw Street works as a local road.

On the westside of the intersection, Gratiot Avenue consists of one exclusive left-turn lane and one shared through and right-turn lane. The southwest approach of South Michigan Avenue consists of an exclusive right turn lane, two exclusive through lanes and a center lane for left turn. The northeast approach of South Michigan Avenue has one exclusive through lane, a right and through shared lane and a center lane for left-turns.

The pavement width of Gratiot Avenue is approximately 33 feet wide at the west leg. Mackinaw Street is approximately 28 feet wide. South Michigan Avenue is approximately 57 feet wide at the southwest leg and at the southwest leg it is approximately 64 feet wide.

The speed limit for Gratiot Avenue is 25 miles per hour and it is 30 miles per hour for South Michigan Avenue. Speed limit of Mackinaw Street is 25 miles per hour.

Land Use

The intersection is surrounded by commercial establishments at the corners. A liquor store and gas station is on the northeast corner, a law office, liquor store and Goodwill Industries office is on the northwest corner, Huron Fish Company and Rally's Hamburgers is on the southeast corner and a McDonalds on the southeast corner. There are three driveways within 150 feet of the intersection which serve the vehicles going to and from these establishments

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.35.1. Details of the intersection approaches are shown in photographs 35a through 35e. The photographs show the laneage and pavement markings on the approaches.

Photograph 35a. NE Bound S. Michigan Avenue at Gratiot Avenue and Mackinaw Street

Photograph 35b. SW Bound S. Michigan Avenue at Gratiot Avenue and Mackinaw Street

Photograph 35c. NW Bound Mackinaw Street at Gratiot Avenue and S. Michigan Avenue

Photograph 35d. SE Bound Mackinaw Street at Gratiot Avenue and S. Michigan Avenue

Photograph 35e. EB Gratiot Avenue at South Michigan Avenue and Mackinaw Street

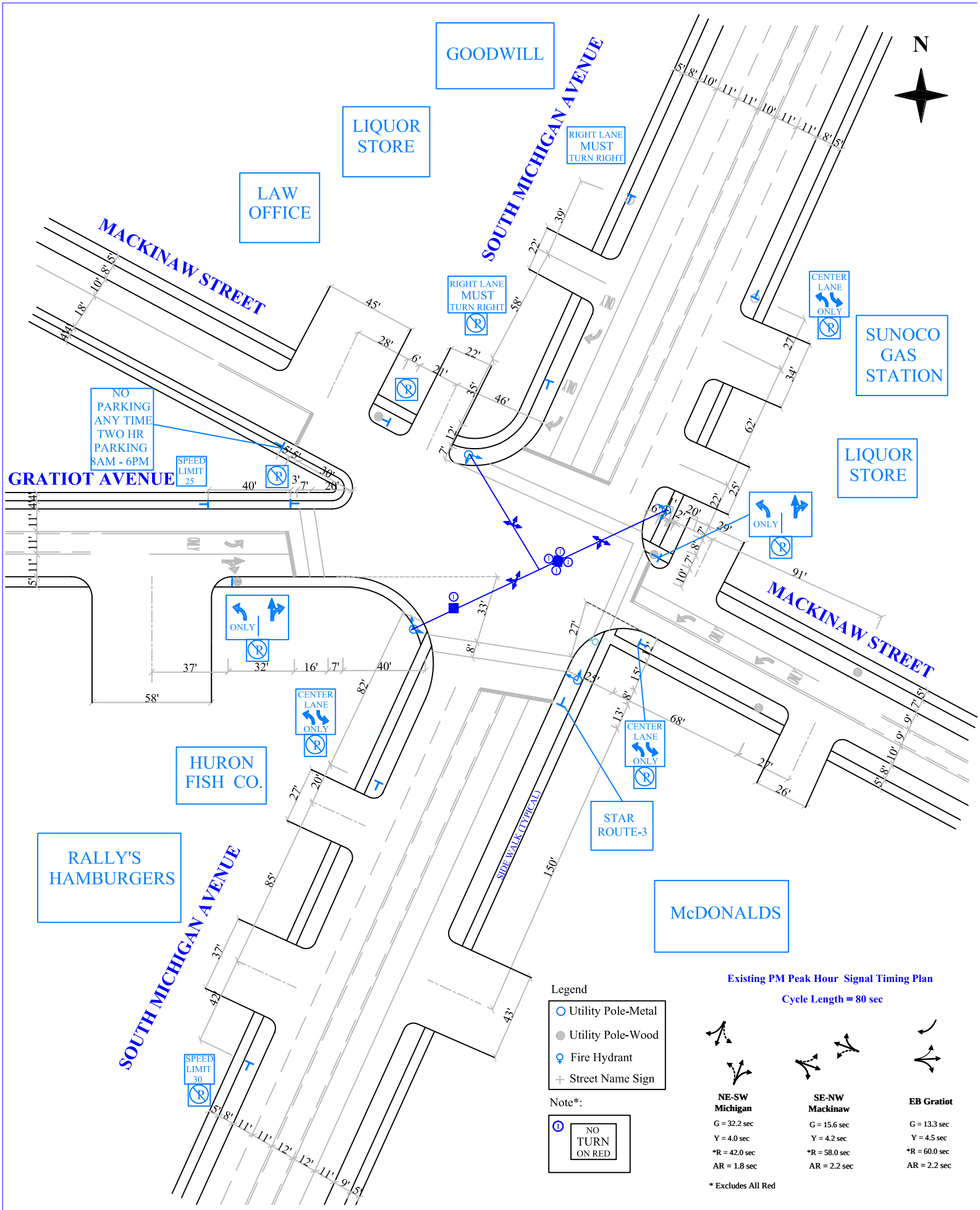


Figure 6.35.1.Existing Condition Diagram of Mackinaw Street, South Michigan Avenue and Gratiot Avenue Intersection (Not to Scale)

Peak Hour Volumes

The PM peak period traffic volumes were manually collected on April 18, 2006 from 4 PM to 6 PM by three people standing at a corner of the intersection and covering all intersecting approaches. The peak hourly volume occurred from 4:30 to 5:30 PM and is presented in Figure 6.35.2. The traffic patterns indicate that the volume of South Michigan Avenue is higher than the volume of Gratiot Avenue and Mackinaw Street.

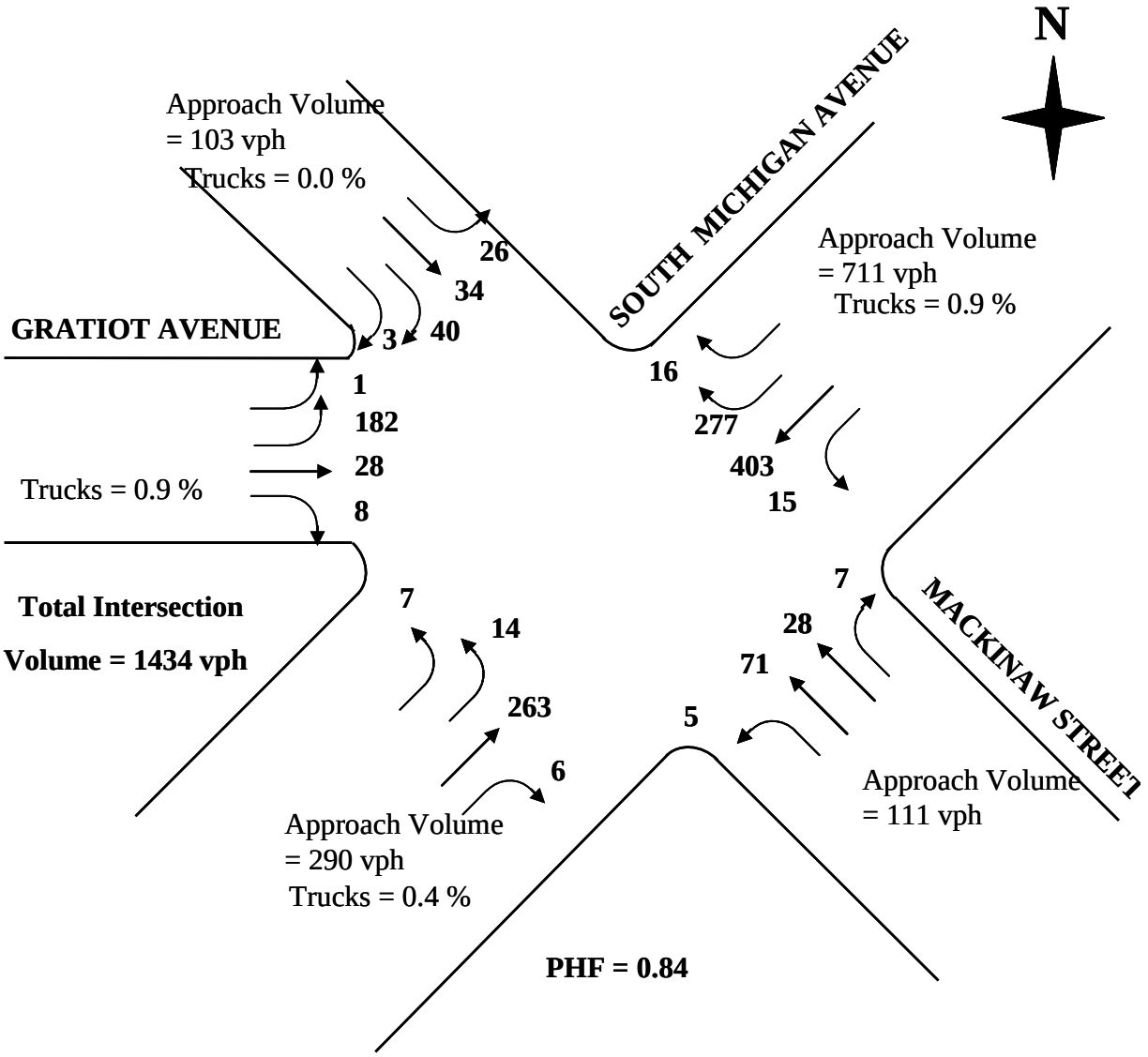


Figure 6.35.2. Existing PM Peak Hour (4:30-5:30PM) Traffic Volumes for the Intersection of Mackinaw Street, South Michigan Avenue and Gratiot Avenue

Traffic Control

The basic signal timing plan is given by a three-phase design which operates on a 80-second cycle length during the PM peak hour. The signals are supported by diagonal span-wire configuration. The existing clearance interval consists of 4.0 seconds yellow interval and 1.8 seconds all-red interval for northeast and southwest through phase. Southeast and northwest through phase has 4.2 seconds yellow interval and 2.2 seconds all-red interval. Eastbound Gratiot Avenue phase has 4.5 seconds yellow interval and 2.2 seconds all-red intervals. The existing PM peak hour signal timing plan is shown in the condition diagram in Figure 6.35.1.

Crash Analysis

A total of 21 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 11 crashes. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 reports information to identify crash patterns. Figures 6.35.3 and 6.35.4 show the collision diagrams for these years respectively.

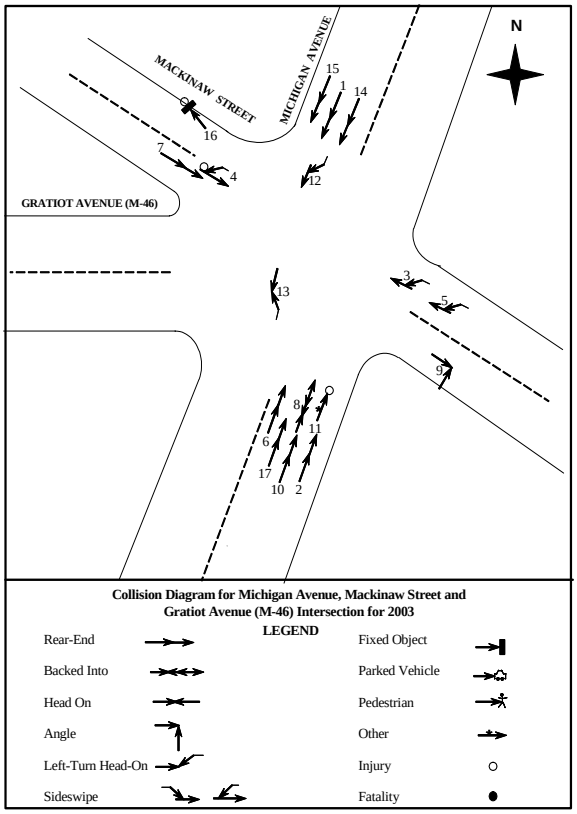


Figure 6.35.3. Collision Diagram for Michigan Avenue, Mackinaw Street and Gratiot Avenue Intersection for 2003

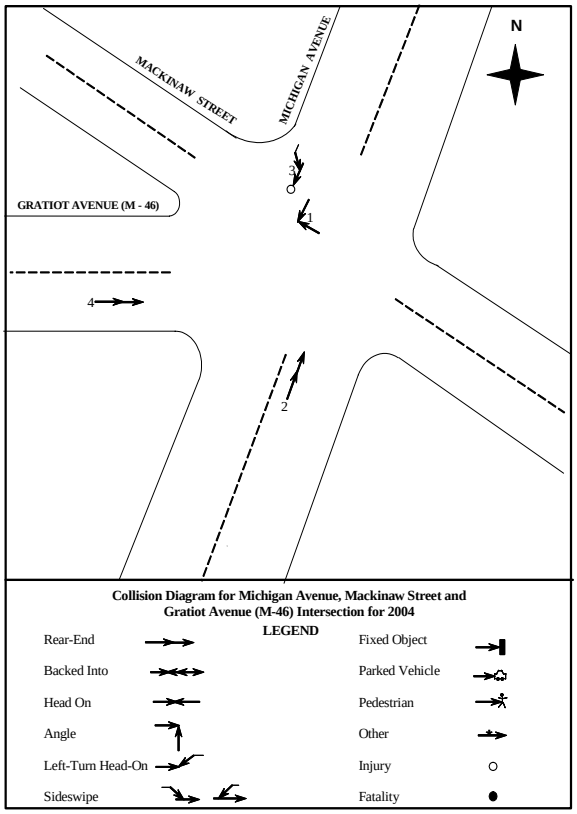


Figure 6.35.4. Collision Diagram for Michigan Avenue, Mackinaw Street and Gratiot Avenue Intersection for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.68 per million vehicles entering vehicles, which is nominal, considering the approach volume at the intersection. Rear end crashes are of major concern for both years.

The crash distribution, based upon types of crashes, is shown in Figure 6.35.5. Rear-end and side-swipe same crashes are the predominant type of crashes reported at the intersection. The corresponding two-year crash frequency for these crashes includes ten Rear end crashes and four side-swipe same crashes.

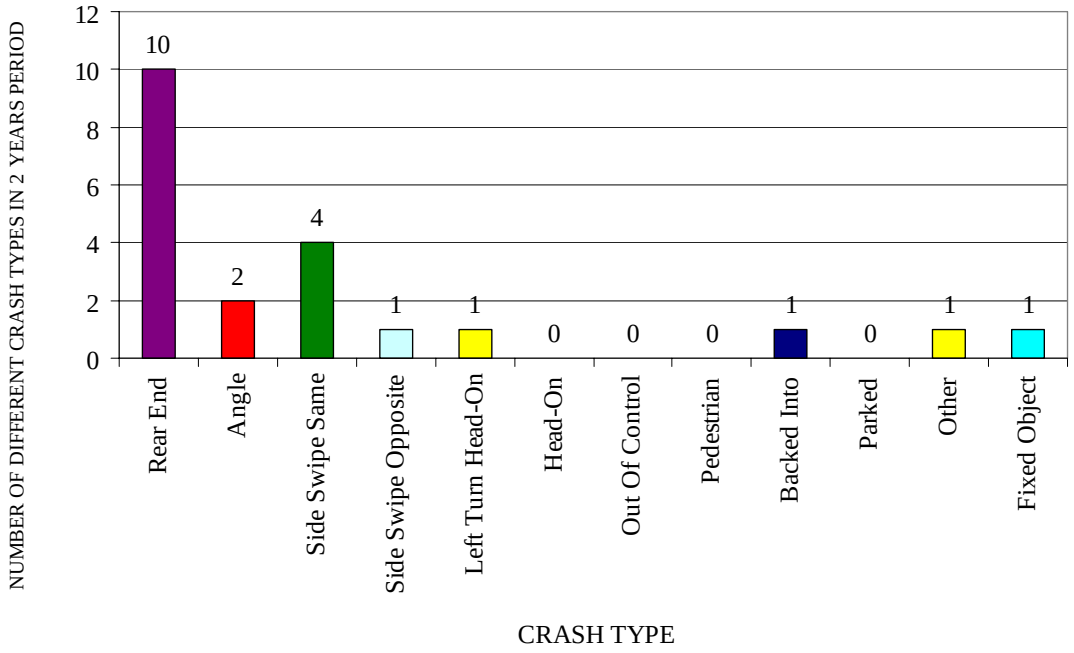


Figure 6.35.5 Two Year (2003-2004) Crash Experience at Mackinaw Street, South Michigan Avenue and Gratiot Avenue Intersection

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of Mackinaw Street, South Michigan Avenue and Gratiot Avenue could be attributed to inadequacies in signal timing and improper visibility.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation.The existing yellow and all-red intervals are shown in Table 6.35.1, along with the proposed lengths of these intervals and their potential benefits. In the proposed signal timing, the cycle length is modified to 90 seconds for safe traffic movement.

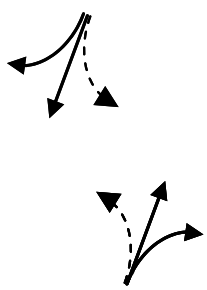
Table 6.35.1. Existing and Proposed Clearance Intervals for the Mackinaw Street, South Michigan Avenue and Gratiot Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
NE-SW Through phase	4.0	1.8	3.5	1.7	Less rear-end, angle, left-turn head-on and injury crashes
SE-NW Through Phase	4.2	2.2	3.5	2.5	
EB Protected Phase	4.5	2.2	3.5	2.5	

The proposed PM peak period signal timing plan based on a 90 second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



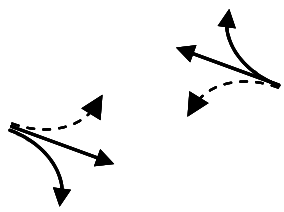
NE-SW Michigan

G = 31.8 sec

Y = 3.5 sec

*R = 53.0 sec

AR = 1.7 sec



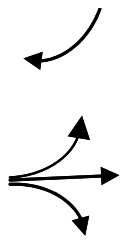
SE-NW Mackinaw

G = 16.0 sec

Y = 3.5 sec

*R = 68.0 sec

AR = 2.5 sec



EB Gratiot

G = 25.0 sec

Y = 3.5 sec

*R = 59.0 sec

AR = 2.5 sec

* Excludes All Red

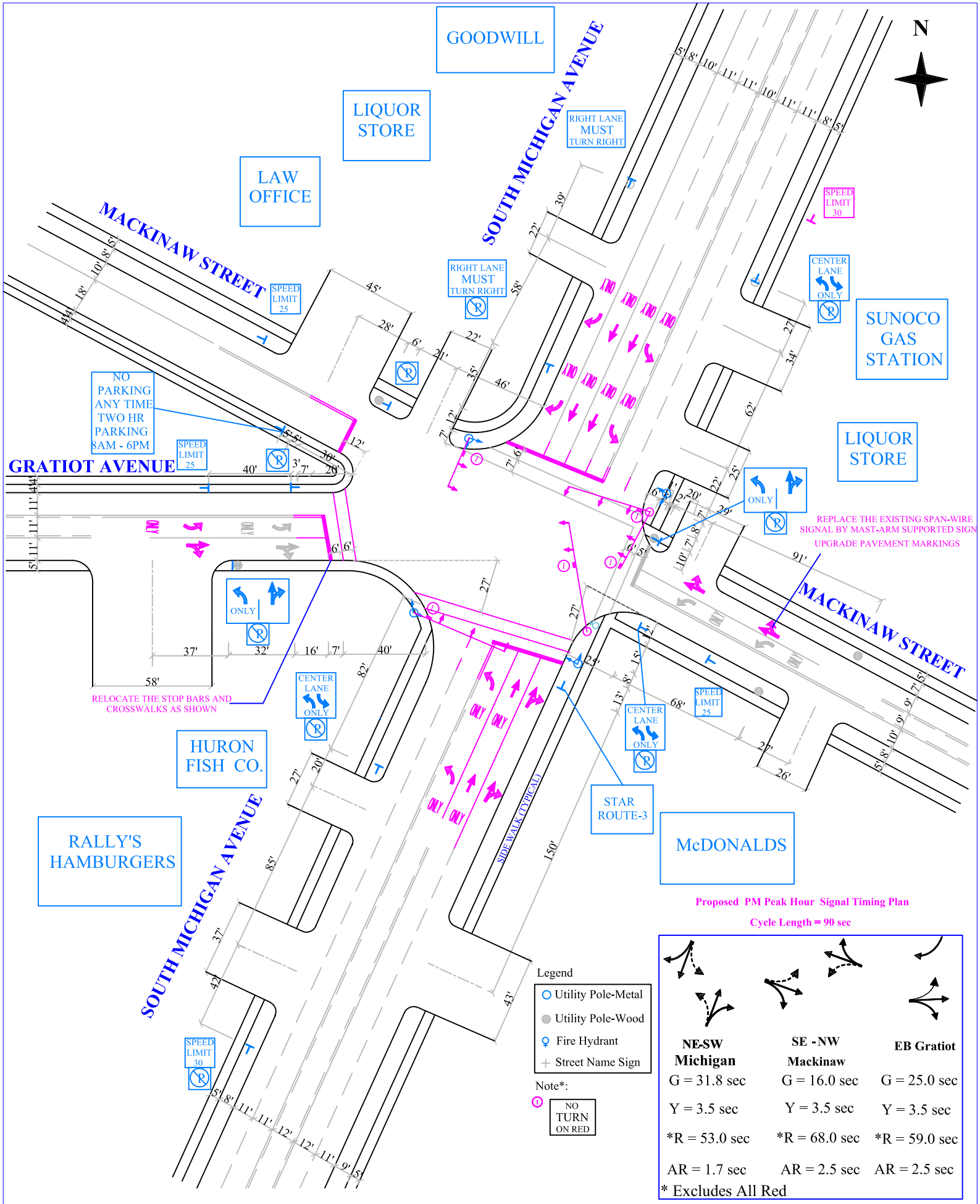


Figure 6.35.6 Proposed Diagram of Mackinaw Street, South Michigan Avenue and Gratiot Avenue Intersection (Not to Scale)

A list of the recommended improvements is summarized below:

- Reduce the yellow interval to 3.5 seconds for all phases.
- Reduce the all-red interval to 1.7 seconds for Michigan Avenue; increase to 2.5 seconds for Mackinaw Street and Gratiot Avenue.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Relocate the stop bars at the northwest and eastbound approaches, as per MMUTCD guidelines.
- Provide upgraded pavement markings for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowners should provide these stop signs.
- Optimize signal timing for AM and off-peak hours.
- Implement the optimized PM signal timings.

Operational Evaluation

The results of the level of service analysis for the intersection of Michigan Avenue, Mackinaw Street and Gratiot Avenue indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is D with a delay of 35.3 seconds per vehicle.

With the modifications to the signal timings and phasing and the noted improvements, the overall intersection level of service will improve to C with a reduced delay of 27.9 seconds per vehicle. The change of cycle length is included as an improvement to the existing signal timing along with changed all-red and yellow intervals.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.35.2.

Table 6.35.2.
South Michigan Avenue, Gratiot Avenue and Mackinaw Street
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Gratiot Avenue	113.5	F	41.3	D
NB Michigan Avenue	16.6	B	22.9	C
SB Michigan Avenue	21.4	C	24.5	C
SE Mackinaw Street	26.4	C	31.8	C
NW mackinaw Street	27.1	C	32.7	C
Overall Intersection	**35.3**	**D**	**27.9**	**C**

36. East Holland Avenue (M-46) and South Outer Drive

The intersection of East Holland Avenue and South Outer Drive is located at the southwestern part of Buena Vista Township. East Holland Avenue runs in the east-west direction and south Outer Drive runs in the north-south direction. East Holland Avenue is a state trunk line also known as M-46 and South Outer Drive is a local street in between East Washington Avenue and Hess Avenue.

East Holland Avenue, near the intersection with South Outer Drive, has an average daily traffic volume of approximately 20,400 vpd as per MDOT 2004 Data.

East Holland Avenue consists of one dedicated left-turn lane, one through lane, and one shared through/right-turn lane for both the approaches. The pavement width of East Holland Avenue is approximately 60 feet wide.

Outer Drive consists of one exclusive left-turn lane, a one through lane and one shared through/right-turn lane. Outer Drive is approximately 61 feet wide at the southbound approach and is approximately 56 feet wide at northbound approach.

The speed limit for East Holland Avenue is 45 miles per hour and for South Outer Drive, it is 35 miles per hour.

Land Use

The intersection is surrounded by commercial establishments across all sides. A Best Western Hotel is located at the southeast corner; Subway, Saginaw Inn, Hooly's Pizza, and Sunoco gas station are on the southwest corner; Welcome Inn and Suites on the north-east corner and a Speedway gas station and Wendy's across north-west corner. There are driveways within 150 feet of the intersection which serves the vehicles going to and from these establishments except from the westbound approach of East Holland Avenue.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection of East Holland Avenue and South Outer Drive.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.36.1. Details of the intersection approaches are shown in photographs 36a through 36d. These photographs show the span-wire signal installation and existing pavement markings. The southbound approach is shown in photograph 36b, which has a major driveway close to the intersection. The northbound and eastbound approaches in photographs 36a and 36c respectively show close proximity of driveways near the intersection.

Photograph 36a. NB Outer Drive at East Holland Avenue

Photograph 36b. SB Outer Drive at East Holland Avenue

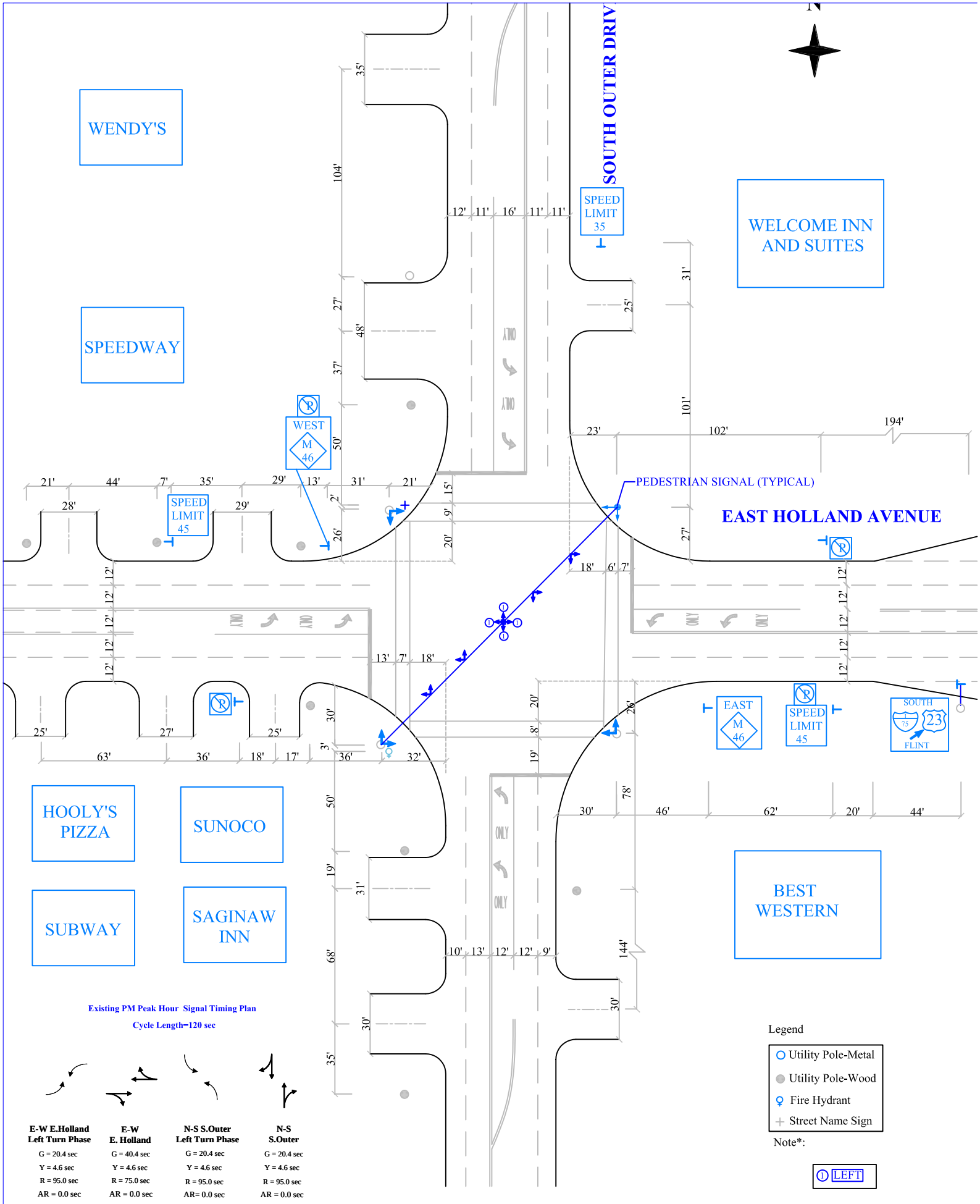


Figure 6.36.1.Existing Condition Diagram of East Holland Avenue and South Outer Drive Intersection (Not to Scale)

Photograph 36c. EB South Outer Drive at East Holland Avenue

Photograph 36d. WB South Outer Drive at East Holland Avenue

Peak Hour Volumes

PM peak period traffic volumes were manually collected on April 26, 2006 from 4 PM to 6 PM by two people standing at a corner of the intersection and covering all the approaches together. The results of the PM peak hour (5:00 - 6:00PM) traffic volume are presented in Figure 6.36.2. The traffic patterns indicate that there is no considerable difference between the traffic volume on East Holland Avenue and South Outer Drive.

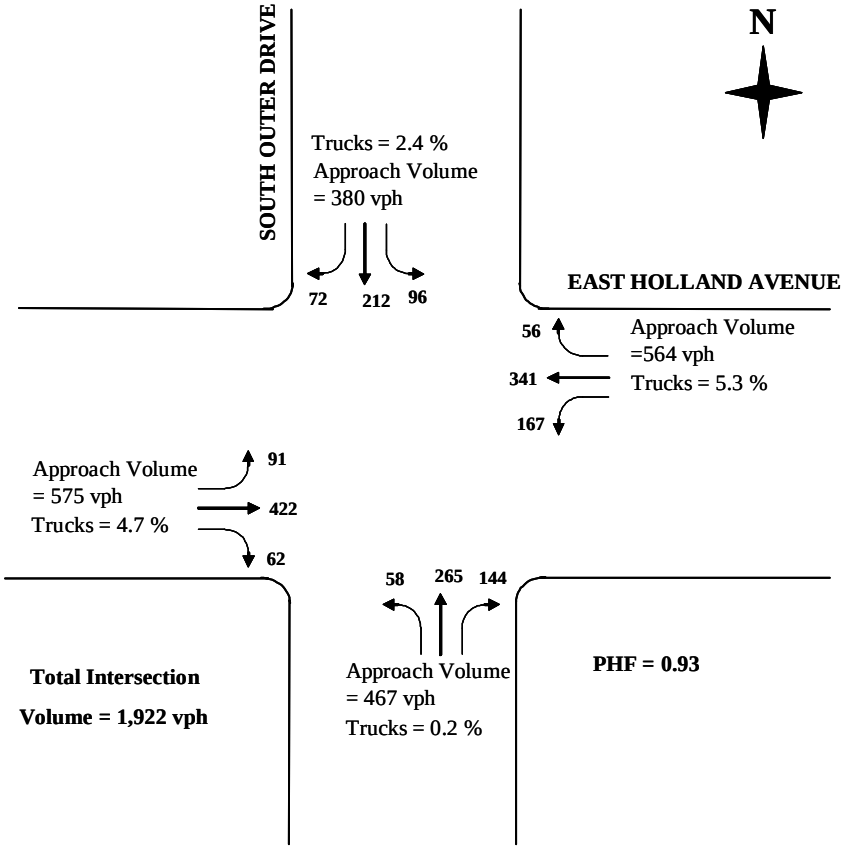


Figure 6.36.2. Existing PM Peak Hour (5:00-6:00PM) Traffic Volumes for the Intersection of East Holland Avenue and South Outer Drive

Traffic Control

This intersection is controlled by a four-phase traffic signal operating on a 120-second cycle length. The existing clearance interval includes a yellow interval of 4.6 seconds and no all-red interval for all four phases. The existing PM peak hour signal timing plan is shown in Figure 6.36.1.

Crash Analysis

A total of 46 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of 23 crashes. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 reports to identify crash patterns. Figures 6.36.3 and 6.36.4 show the collision diagrams for these years respectively.

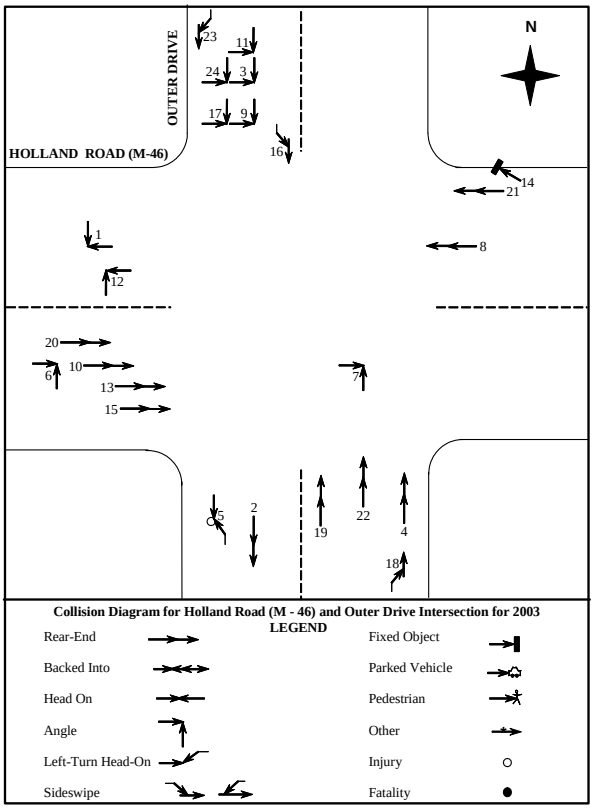


Figure 6.36.3. Collision Diagram for E. Holland Ave and S. Outer Dr. for 2003

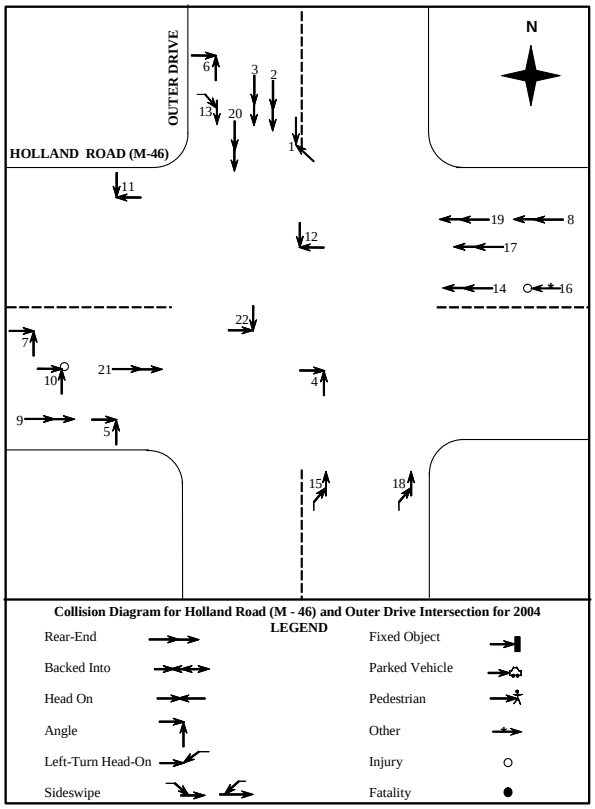


Figure 6.36.4. Collision Diagram for E. Holland Ave and S. Outer Dr. for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 2.62 per million entering vehicles, which is significant, considering the approach volume at the intersection. Rear-end and angle crashes are of specific concern that might have occurred due to inadequate signal timing, insufficient clearance intervals and major driveway activities near the intersection.

The crash distribution, based upon types of crashes, is shown in Figure 6.36.5. Rear-end (19), angle (17) and side-swipe same (6) crashes are the predominant type of crashes reported at the intersection.

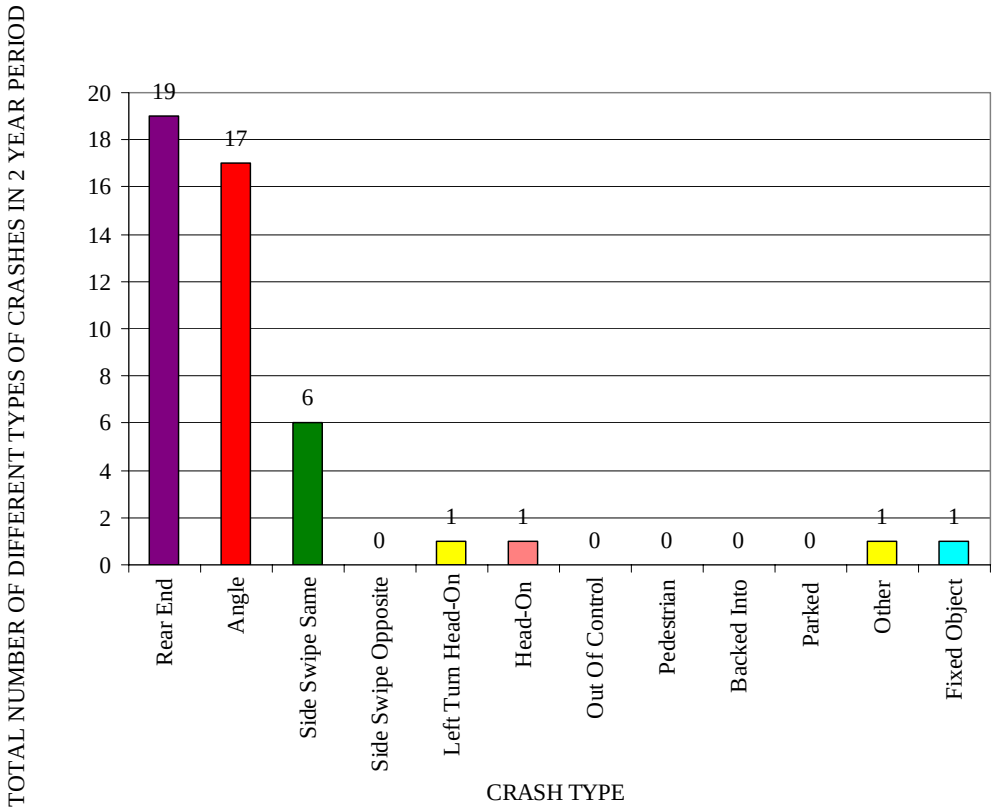


Figure 6.36.5 Two Year (2003-2004) crash experience at East Holland Avenue and South Outer Drive Intersection

Safety Deficiencies and Recommended Improvements

The majority of crashes occurring at the intersection are rear-end and angle crashes. This may be occurring due to several possible reasons including restricted visibility or inadequate clearance

intervals. Analysis of UD-10 forms reveals that some of these crashes have occurred due to unsafe driveway activities around the intersection.

Proposed improvements may consist of provision of adequate yellow and all-red clearance intervals. To improve signal visibility, the existing span-wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.36.5. Post-mounted traffic signals with "LEFT" illuminated signs should be provided at the northeast and southwest corners of the intersection to assist east and westbound left-turn vehicles.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.36.1, along with the proposed lengths of these intervals and their potential benefits.

There are significant angle crashes involving driveway movements on the north leg of South Outer Drive. Therefore a raised median at this approach is recommended to restrict the driveway movement which is shown in the proposed intersection diagram in Figure 6.35.5.

The proposed diagram in Figure 6.35.5 also shows other recommended improvements including the mast-arm installation, upgrading the pavement markings and the proposed signal timing plans.

The N-S left-turn phase has been eliminated in the proposed signal timing plan as the left-turn traffic is not high enough to warrant a separate left-turn phase.

Table 6.36.1. Existing and Proposed Clearance Intervals for the East Holland Avenue and South Outer Drive Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
E-W Left Turn Phase	4.6	0.0	4.3	1.5	Less rear-end, angle, left-turn head-on, and injury crashes
E-W Through Phase	4.6	0.0	4.3	1.5	
N-S Left Turn Phase	4.6	0.0	-	-	
N-S Through Phase	4.6	0.0	3.5	2.0	

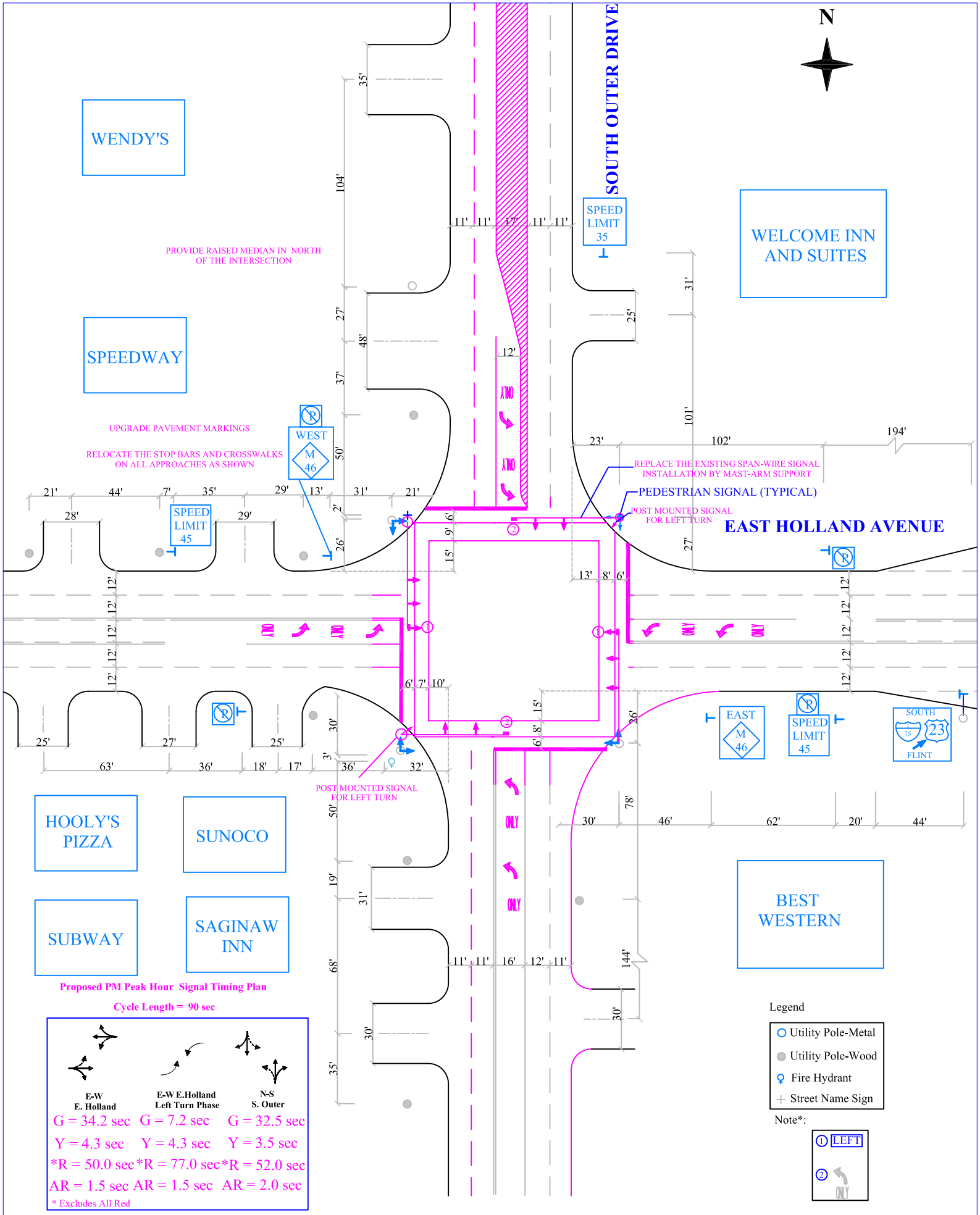
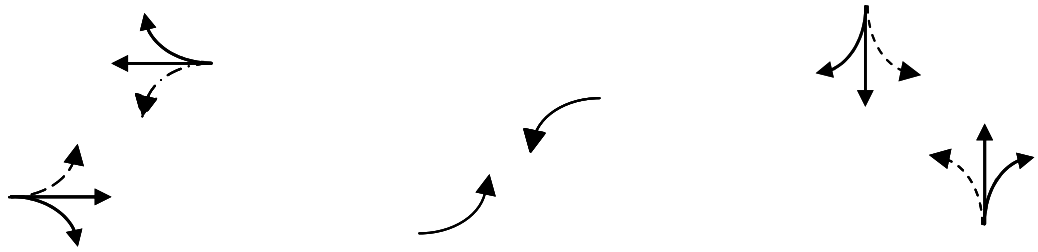


Figure 6.36.6 Proposed Diagram of East Holland Avenue and South Outer Drive Intersection (Not to Scale)

The existing cycle length is changed to 90 seconds for safety benefits and to achieve desired progression along the corridor. The proposed PM peak period signal timing plan with a cycle length of 90-second is shown below:

Proposed PM Peak Hour Signal Timing
Cycle Length = 90 Seconds



E-W E. Holland	E-W E.Holland Left Turn Phase	N-S S. Outer
G = 34.2 sec	G = 7.2 sec	G = 32.5 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 3.5 sec
*R = 50.0 sec	*R = 84.2 sec	*R = 52.0 sec
AR = 1.5 sec	AR = 1.5 sec	AR = 2.0 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval to 4.3 seconds for East Holland Avenue and 3.5 seconds for South Outer Drive.
- Introduce the all-red interval of 1.5 seconds for East Holland Avenue and 2.0 seconds for South Outer Drive.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade the existing pavement markings for all the approaches.
- Relocate the stop bar and crosswalks on all the approaches, as per MMUTCD guidelines.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for east and westbound left-turns.

- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Optimize the signal timings for AM and off-peak hours.
- Implement the optimized PM signal timings.

Operational Evaluation

The results of the level of service analysis for the intersection of East Holland Avenue and South Outer Drive indicate that under existing conditions during the PM peak hour, the overall intersection level of service is D with a delay of 39.4 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will remain at C with a reduced delay of 21.4 seconds per vehicle. This reduced overall delay will facilitate better movement of traffic.

The level of service results for the existing conditions and with the proposed improvements are summarized in Table 6.36.2.

Table 6.36.2.
E. Holland Avenue and S. Outer Drive
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB E. Holland Avenue	31.6	C	19.0	B
WB E. Holland Avenue	35.4	D	19.6	B
NB S. Outer Drive	49.6	D	20.7	C
SB S. Outer Drive	45.2	D	21.4	C
Overall Intersection	**39.4**	**D**	**21.4**	**C**

37. East Holland Avenue and Cumberland Street

The intersection of East Holland Avenue and Cumberland Street is located at the eastern boundary of the City of Saginaw. Cumberland Street forms a “T” intersection with East Holland Avenue at the south leg. Cumberland Street runs in the east-west direction. East Holland Avenue is a state trunk line and also known as M-46. Cumberland Street works as a local street.

East Holland Avenue, near the intersection with Cumberland Street, has an average daily traffic volume of approximately 20,400 vehicles per day (vpd) east of the intersection as per Michigan Department of Transportation 2004 Data.

East Holland Avenue at the eastbound approach consists of one dedicated right-turn lane, two through lanes and a center lane while at the westbound approach, East Holland Road consists of an exclusive left-turn lane and two through lanes. East Holland Avenue is approximately 65 feet wide at the eastbound approach and is approximately 56 feet wide at the westbound approach.

Cumberland Street at the northbound approach consists of one exclusive right-turn lane and one exclusive left-turn lane. The pavement width of Cumberland Street is approximately 36 feet wide.

The speed limit for East Holland Avenue is 35 miles per hour.

Land Use

The intersection of East Holland Avenue and Cumberland Street is located in a sparsely populated area with very few business activities around the corners. A gas station is on the south west corner and a US Post Office is on the southeast corner. The gas station has four driveways from all its adjacent streets. There is no significant land use at the north side of the intersection.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.37.1. Details of the intersection approaches are shown in photographs 37a through 37c. Photographs show the span-wire signal installation and existing pavement markings

Photograph 37a. EB East Holland Avenue at Cumberland Street

Photograph 37b. WB East Holland Avenue at Cumberland Street

Photograph 37c. NB Cumberland Street at East Holland Avenue

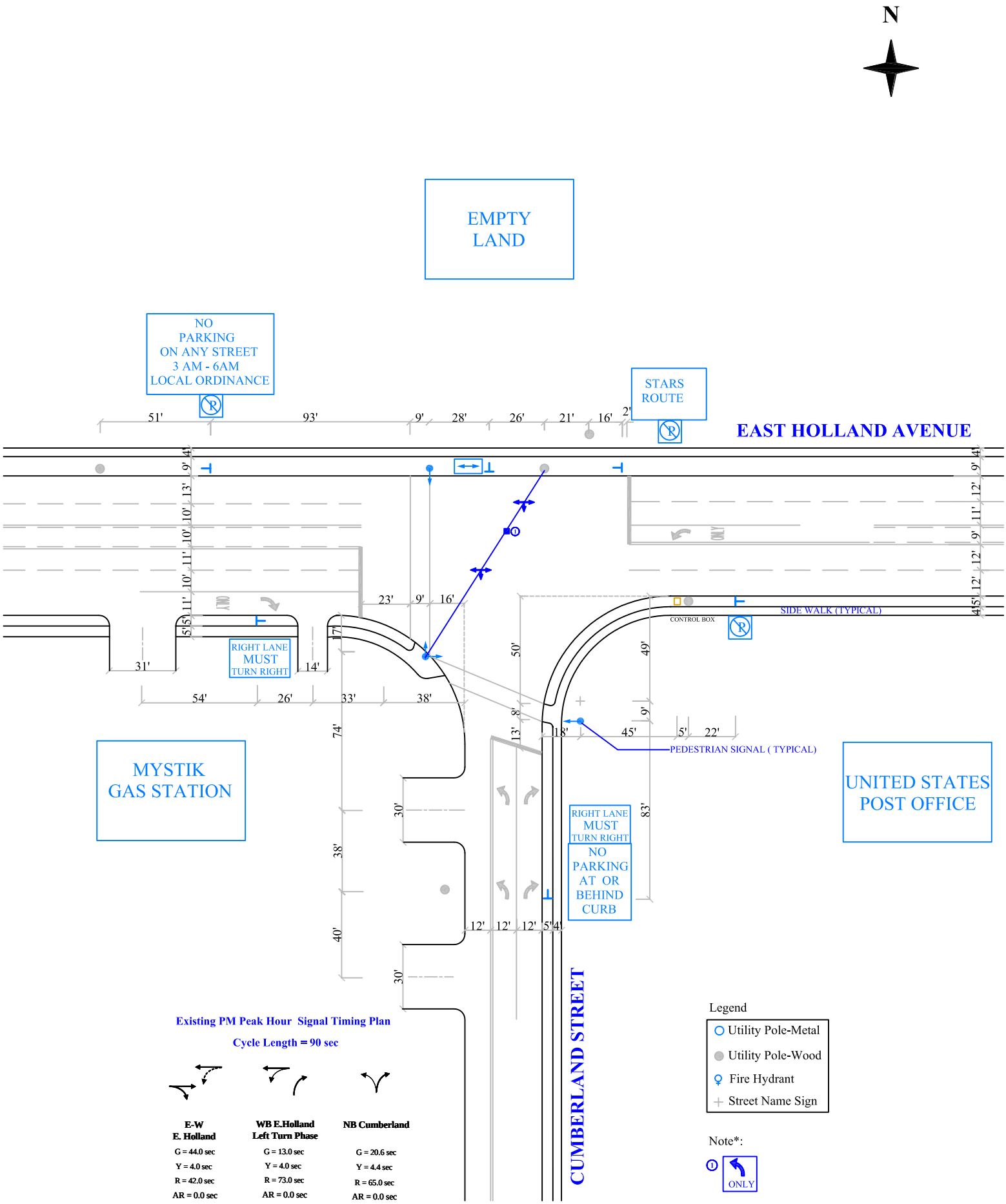


Figure 6.37.1.Existing Condition Diagram of East Holland Avenue and Cumberland Street Intersection (Not to Scale)

Peak Hour Volumes

PM peak period traffic volumes were manually collected on a clear day on April 26, 2005 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering all the approaches. The peak hourly volume occurred from 4:00PM to 5:00PM and is presented in Figure 6.37.2. The traffic patterns indicate that the volume on East Holland Avenue is considerably higher than the volume on Cumberland Street.

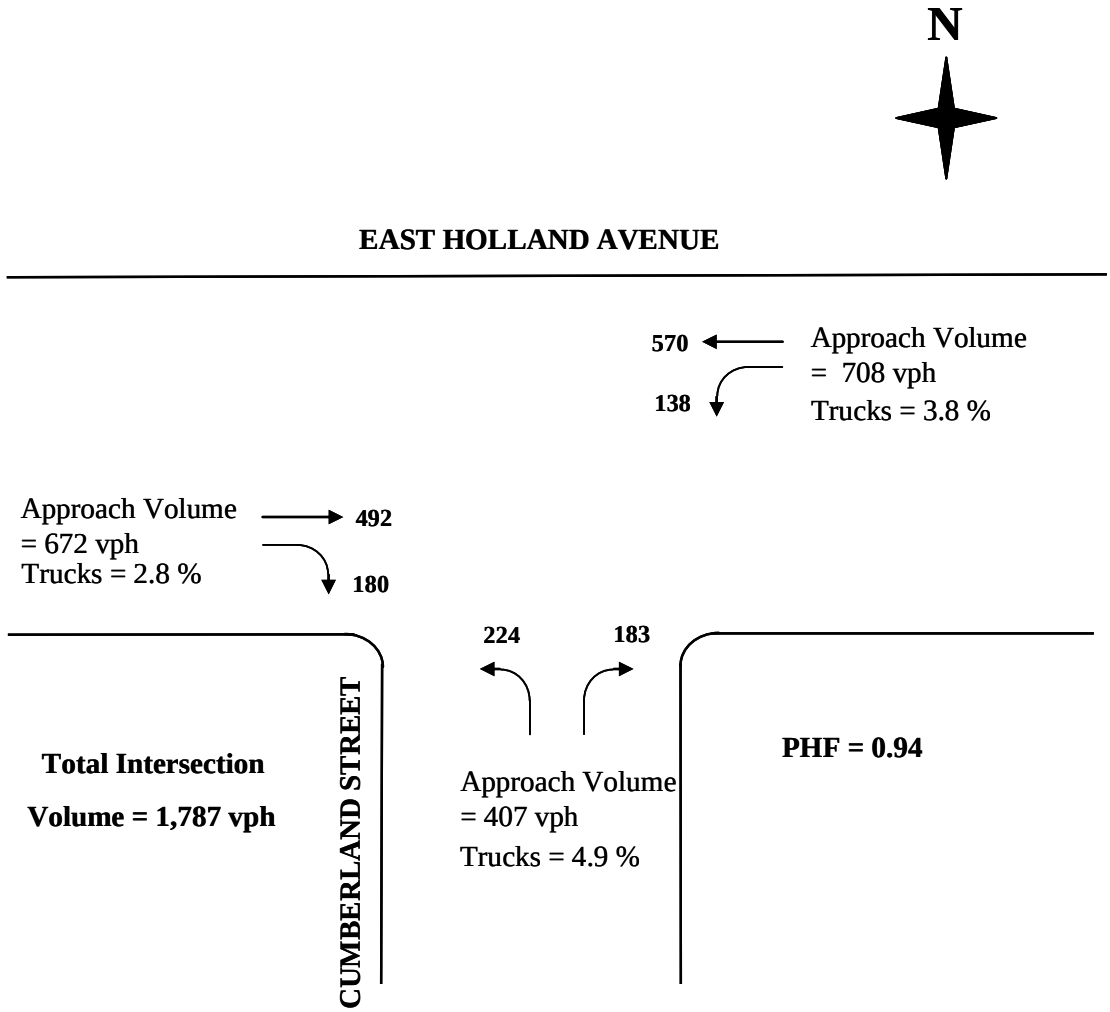


Figure 6.37.2. Existing PM Peak Hour (4:00-5:00 PM)) Traffic Volumes for the Intersection of East Holland Avenue and Cumberland Street

Traffic Control

This intersection is controlled by a three-phase traffic signal operating on 90-second cycle length. The existing clearance interval includes 4.0 seconds yellow interval for the east-west through, westbound left-turn and northbound right-turn phases and 4.4 seconds yellow interval

for the northbound Cumberland Street phase. There is no all-red intervals in any phases. The existing PM peak hour signal timing plan is shown in Figure 6.37.1. Left turning phase of E. Holland Avenue is actuated while east-west through and right phase of E. Holland Avenue is coordinated. Right and left turning movement from northbound Cumberland Street is dual coordinated.

Crash Analysis

A total of eighteen crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of nine crashes. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 reports to identify crash patterns. Figures 6.37.3 and 6.37.4 show the collision diagrams for these years respectively.

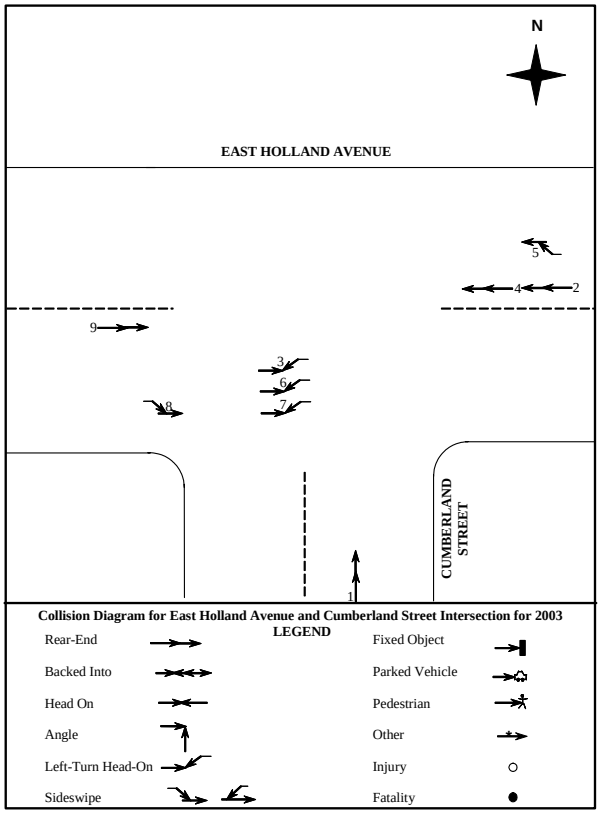


Figure 6.37.3. Collision Diagram for Holland Avenue and Cumberland Street for 2003

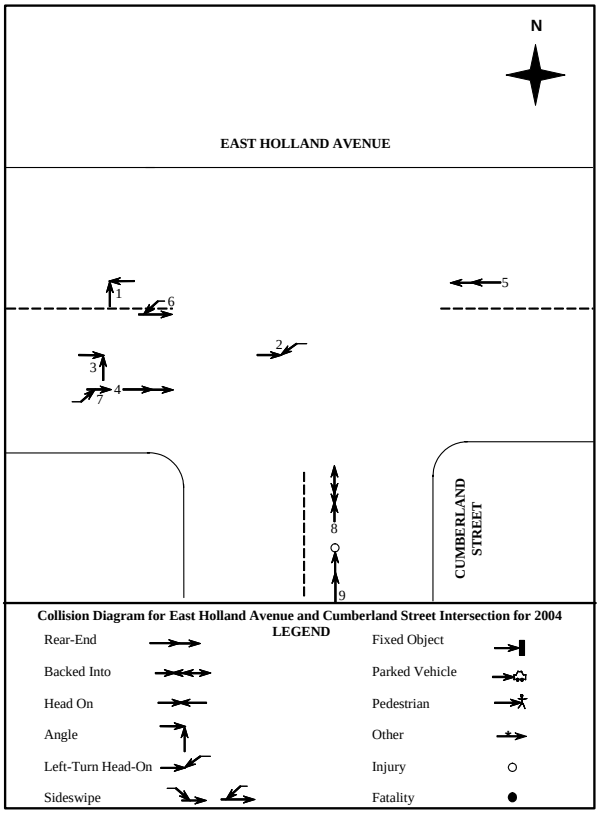


Figure 6.37.4. Collision Diagram for Holland Avenue and Cumberland Street for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.10 per million entering vehicles, which is nominal, considering the approach volume at the intersection. Left-turn head-on crashes are of main concern, which might have occurred due to interference of oncoming through traffic of eastbound East Holland Avenue with westbound left-turn during permissive phase.

The crash distribution, based upon types of crashes, is shown in Figure 6.37.5. Rear-end, left-turn head-on and side swipe same crashes are the predominant crashes reported at the intersection. The corresponding two-year crash frequency for these crashes includes seven rear-end crashes, four left-turn head-on crashes and three side-swipes same crashes.

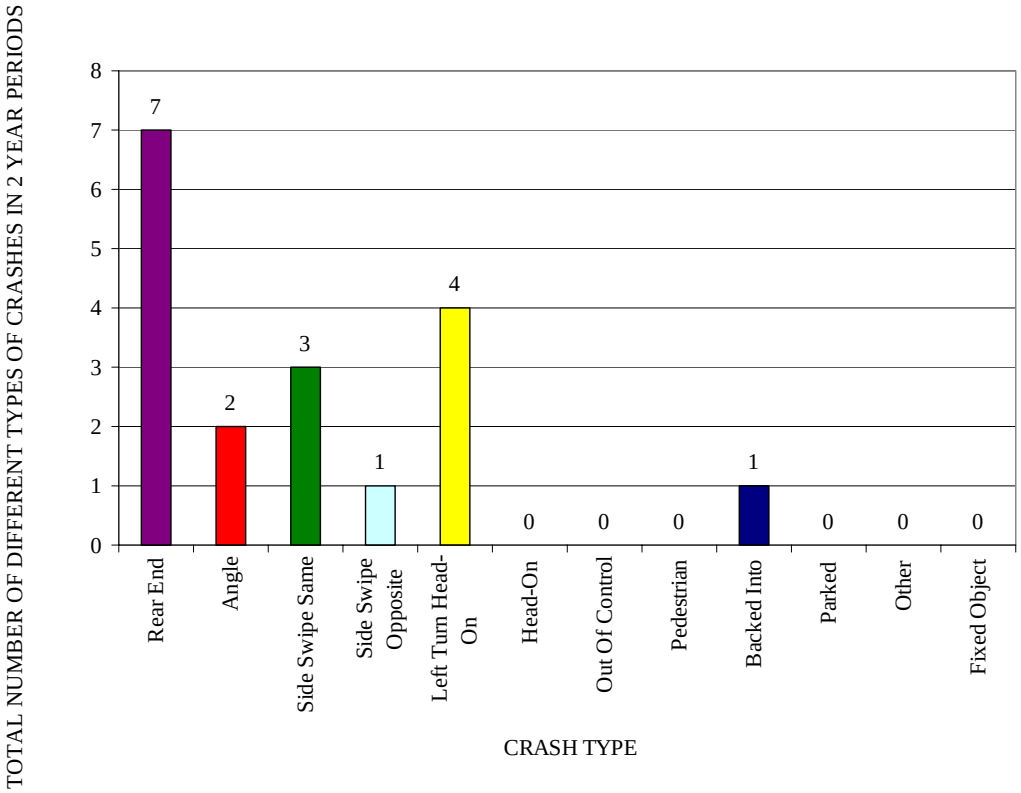


Figure 6.37.5 Two Year (2003-2004) Crash Experience at Holland Avenue and Cumberland Street Intersection

<u>Safety Deficiencies and Recommended Improvements</u>

The majority of crashes occurring at the intersection are left-turn head-on and rear-end crashes. These are occurring due to several possible reasons including restricted or poor visibility and inadequate clearance intervals.

Mitigation measures may consist of provision of adequate yellow and all-red clearance intervals. To improve signal visibility, the existing span-wire installation should be replaced by a mast-arm signal installation, as shown in the proposed diagram in Figure 6.37.6. The pavement markings are upgraded across all the approaches.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

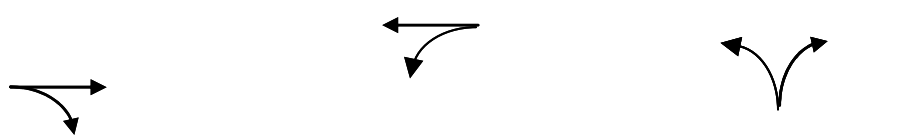
The existing yellow and all-red intervals are shown in Table 6.37.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.37.1. Existing and Proposed Clearance Intervals for the East Holland Avenue and Cumberland Street Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
WB Phase	4.0	0.0	3.6	1.8	Less rear-end, angle, left-turn head-on and injury crashes
EB Phase	-	-	3.6	1.8	
E-W Through Phase	4.0	0.0	-	-	
NB Phase	4.4	0.0	3.5	2.0	

The proposed PM peak period signal timing plan based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length =90 Seconds



EB Holland	WB Holland	NB Cumberland
G = 23.6 sec	G = 25.6 sec	G = 24.5 sec
Y = 3.6 sec	Y = 3.6 sec	Y = 3.5 sec
*R = 61.0 sec	*R = 59.0 sec	*R = 60.0 sec
AR = 1.8 sec	AR = 1.8 sec	AR = 2.0 sec

* Excludes All Red

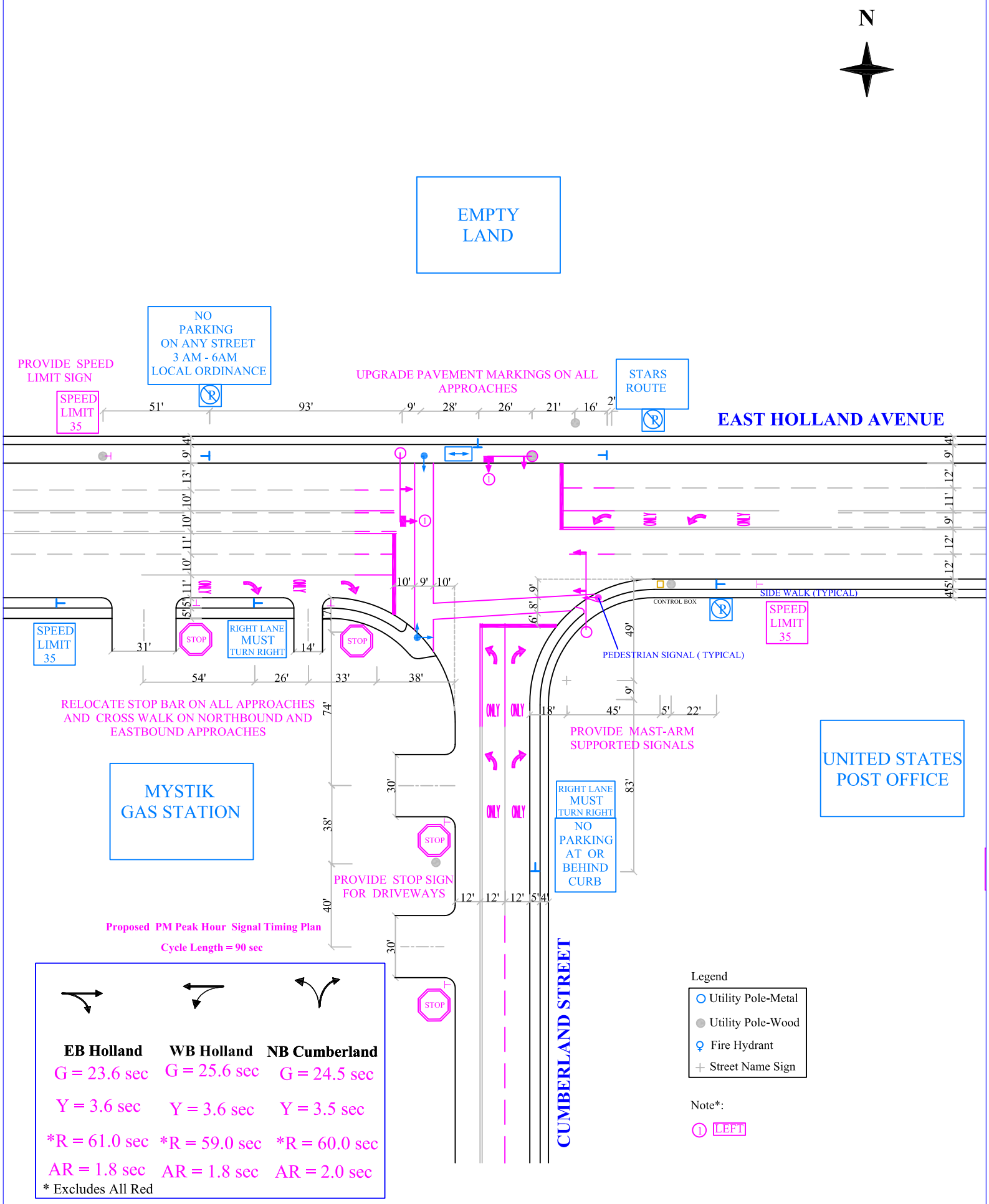


Figure 6.37.6 Proposed Diagram of East Holland Avenue and Cumberland Street Intersection (Not to Scale)

A list of the recommended improvements is summarized below:

- Reduce the yellow interval to 3.6 seconds for Holland Avenue and 3.5 seconds for Cumberland Street.
- Introduce the all-red interval of 1.8 seconds for Holland Avenue and 2.0 seconds for Cumberland Street.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Relocate stop bars and provide new crosswalks at all the approaches, as per MMUTCD guidelines.
- Provide upgraded pavement markings for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowners should provide these stop signs.
- Optimize signal timings for AM and off-peak hours.
- Implement the optimized PM signal timings.

The proposed diagram in Figure 6.37.5 shows the improvements including mast-arm installation for the signals, post-mounted signals for left-turns and upgraded pavement markings.

Operational Evaluation

The results of the level of service analysis for the intersection of East Holland Avenue and Cumberland Street indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 10.5 seconds per vehicle.

With the modifications to the signal timings and the noted improvements, the overall intersection level of service will be reduced to C with an increased delay of 29.2 seconds per vehicle. This improvement includes appropriate all-red intervals to provide safety benefits. The increased safety benefits outweigh the impact on the increased delay.

Table 6.37.2. East Holland Avenue and Cumberland Street Level of Service Analysis PM Peak Hour				
Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements and Signal Progression	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB E. Holland Avenue	10.2	B	29.9	C
WB E. Holland Avenue	1.7	A	28.5	C
NB Cumberland Street	28.1	C	29.5	C
Overall Intersection	**10.5**	**B**	**29.2**	**C**

38. Rust Avenue (M-46) and South Washington Avenue (M-13)

The intersection of Rust Avenue and South Washington Avenue is located in the central part of Saginaw. South Washington Avenue runs in the north-south direction and Rust Avenue runs in the east-west direction. Both Rust Avenue and South Washington Avenue are state trunk lines known as M-46 and M-13 respectively.

Rust Avenue, east of the intersection with South Washington Avenue, has an average daily traffic volume of approximately 11,200 vehicles per day (vpd) and west of the intersection with South Washington Avenue it has an average daily traffic volume of 16,300 vpd as per MDOT data (2004). South Washington Avenue has an average daily traffic of 13,800 vpd near the intersection.

Right-turn movement of westbound traffic from Rust Avenue is provided with a separate right-turn leg 174 feet east of the intersection. Similarly right-turn from South Washington Avenue is segregated by another wing 159 feet north of the intersection. On the east leg, Rust Avenue consists of one exclusive right-turn lane, two through lanes and one exclusive left-turn lane. Rust

Avenue is approximately 59 feet wide at the east leg of intersection. The approach is divided by a raised concrete median with 2 feet width near the intersection and a gradual increase of width reaching 6 feet near the right-turning leg. On the west leg, Rust Avenue consists of one shared through/right-turn lane, one through lane and a raised median of 14 feet wide. At the west leg Rust Avenue is approximately 60 feet wide.

At the north leg, South Washington Avenue consists of two through lanes and a raised median of variable width from 7 feet near the intersection to 14 feet where channelization starts. South Washington Avenue is approximately 59 feet wide at the north leg of the intersection. At the south leg of the intersection, South Washington Avenue consists of one exclusive left-turn lane, one through lane and one shared through right-turn lane. The pavement width at the south leg of the intersection for South Washington Avenue is approximately 57 feet wide.

The speed limit for Rust Avenue is 35 miles per hour.

Land Use

The intersection of Rust Avenue and South Washington Avenue is located in a residential area. Residential homes are located at the southeast and southwest corners of the intersection. While north-east and north-west corners are triangular traffic islands created due to channelization.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.38.1. Details of the intersection approaches are shown in photographs 38a through 38f.

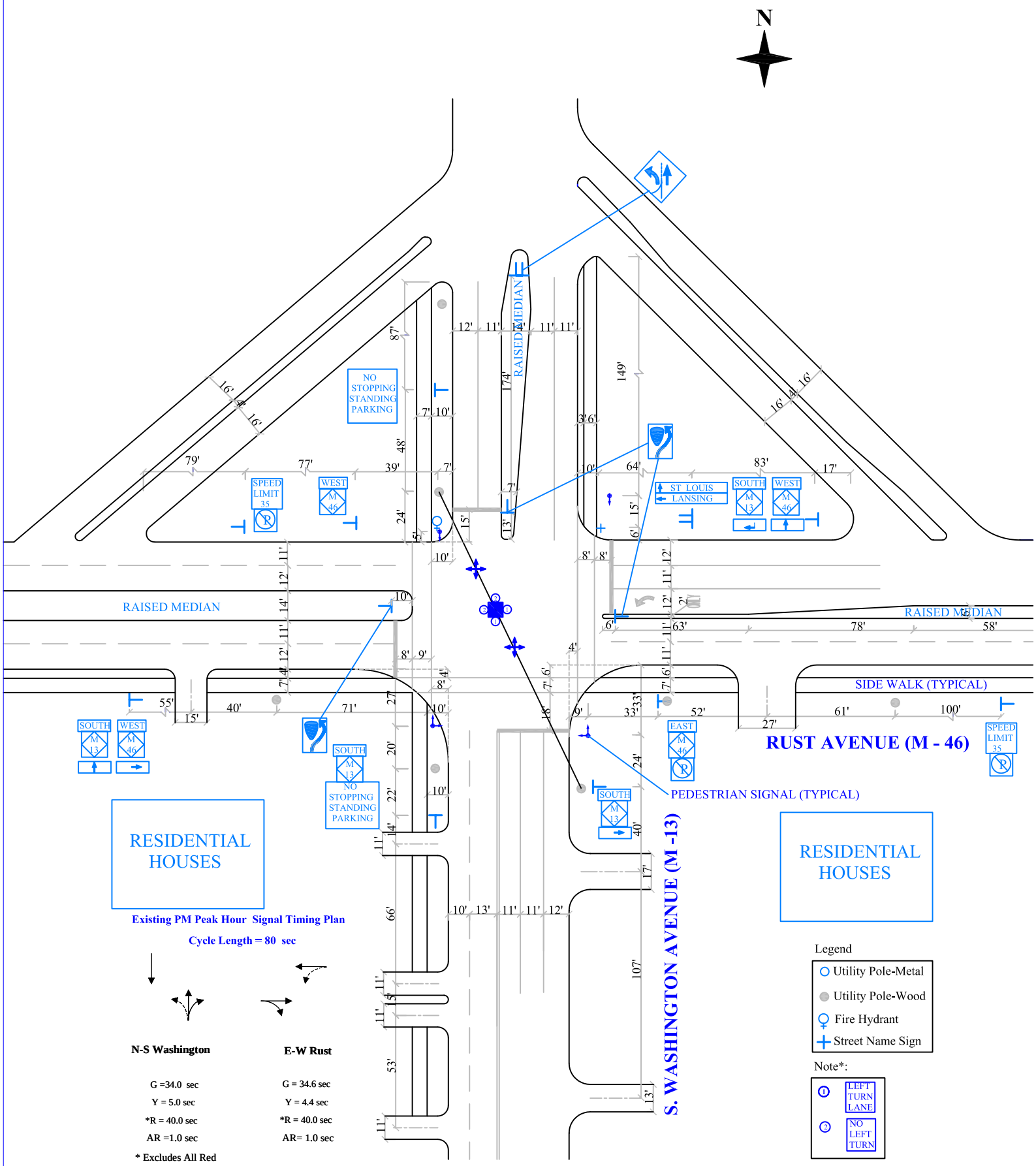


Figure 6.38.1 Existing Condition Diagram of Rust Avenue (M-46) and S. Washington Avenue (M-13) Intersection (Not to Scale)

Photograph 38a. EB Rust Avenue at Washington Avenue

Photograph 38b. WB Rust Avenue at Washington Avenue

Photograph 38c. NB Washington Avenue at Rust Avenue

Photograph 38d. SB Washington Avenue at Rust Avenue

Photograph 38e. NE Rust Avenue at Washington Avenue

Photograph 38f. NW Rust Avenue at Washington Avenue

Peak Hour Volumes

PM peak period traffic volumes were manually collected on a clear day on April 25, 2006 from 4 PM to 6 PM by three people standing at corner locations of the intersection and covering all intersecting approaches together. The peak hour volume occurred from 4:15 PM to 5:15 PM and is presented in Figure 6.38.2. The traffic patterns indicate that there is no considerable difference in the traffic volume between Rust Avenue and Washington Avenue.

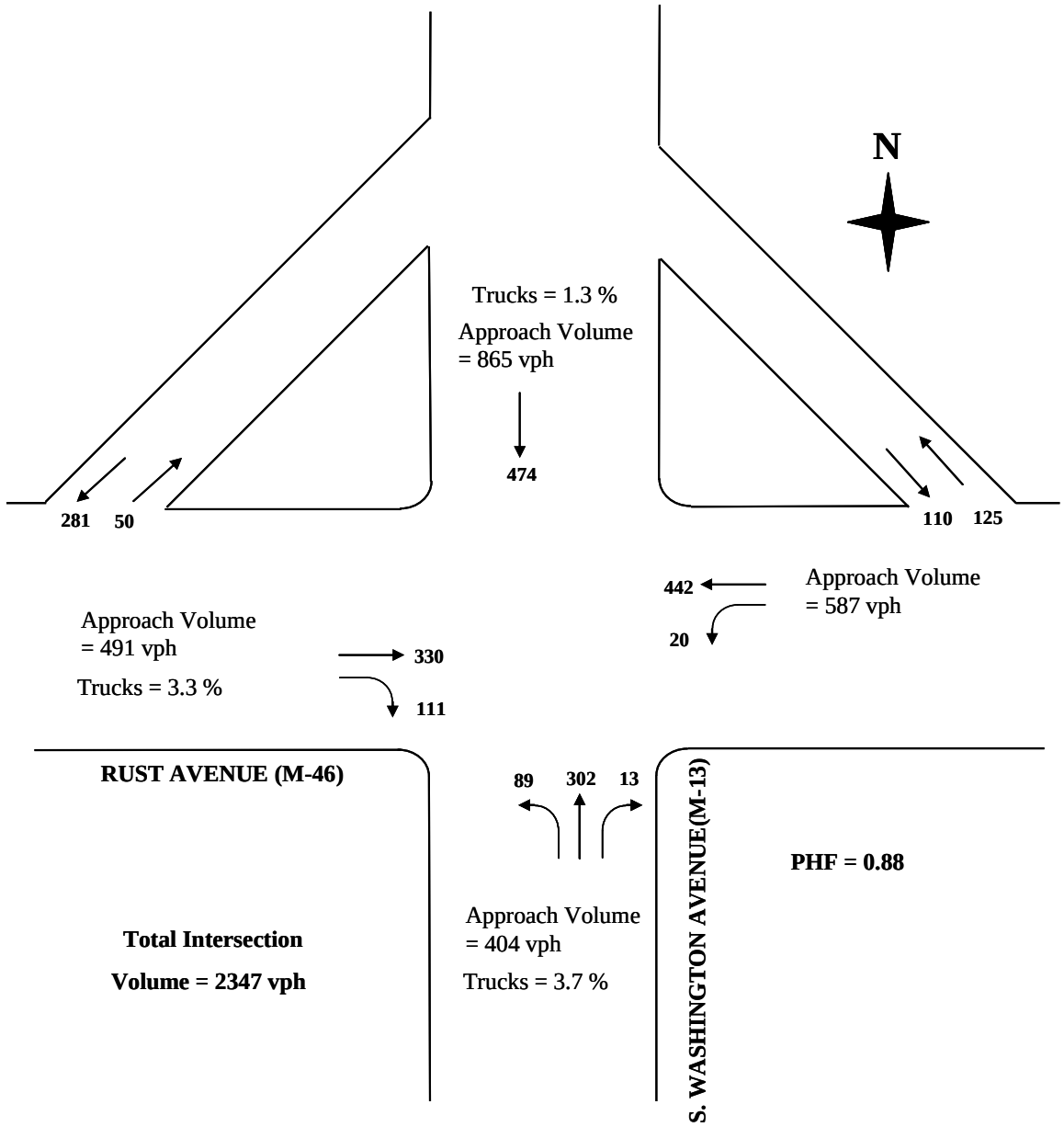


Figure 6.38.2. Existing PM Peak Hour (4:15-5:15 PM) Traffic Volumes for the Intersection of Rust Avenue (M-46) and South Washington Avenue (M-13)

Traffic Control

This intersection is controlled by a four-phase traffic signal operating on 120-second cycle length. The existing clearance interval includes 4.7 seconds of yellow interval and 1.0 second all-red interval for north-south left-turn and through phases. The east-west left-turn and through phases have 4.8 seconds yellow interval and 1.0 second all-red Interval. The existing PM peak hour signal timing plan is shown in Figure 6.38.1.

Crash Analysis

A total of 48 crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of twenty crashes. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 reports to identify crash patterns. Figures 6.38.3 and 6.38.4 show the collision diagram for these years respectively. Angle crashes at the intersection are of specific concern, which may might have occurred due to improper clearance interval..

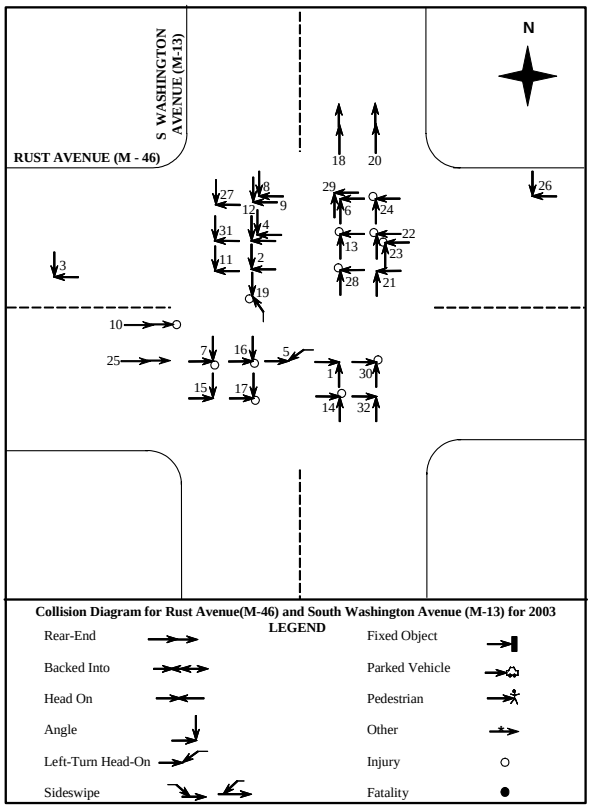


Figure 6.38.3 Collision Diagram for Rust Ave. and S. Washington Ave. for 2003

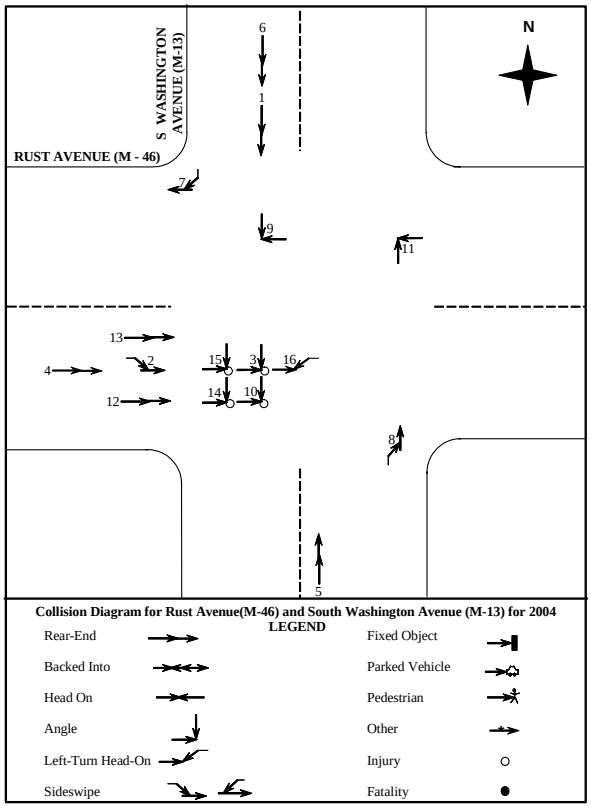


Figure 6.38.4 Collision Diagram for Rust Ave. and S. Washington Ave. for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.87 per million entering vehicles, which is nominal, considering the approach volume at the intersection.

The crash distribution, based upon types of crashes, is shown in Figure 6.38.5. Angle and Rear end crashes are the predominant type of crash reported at th intersection. The corresponding two-year crash frequency for these crashes includes 32 angle crashes and 10 rear-end crashes.

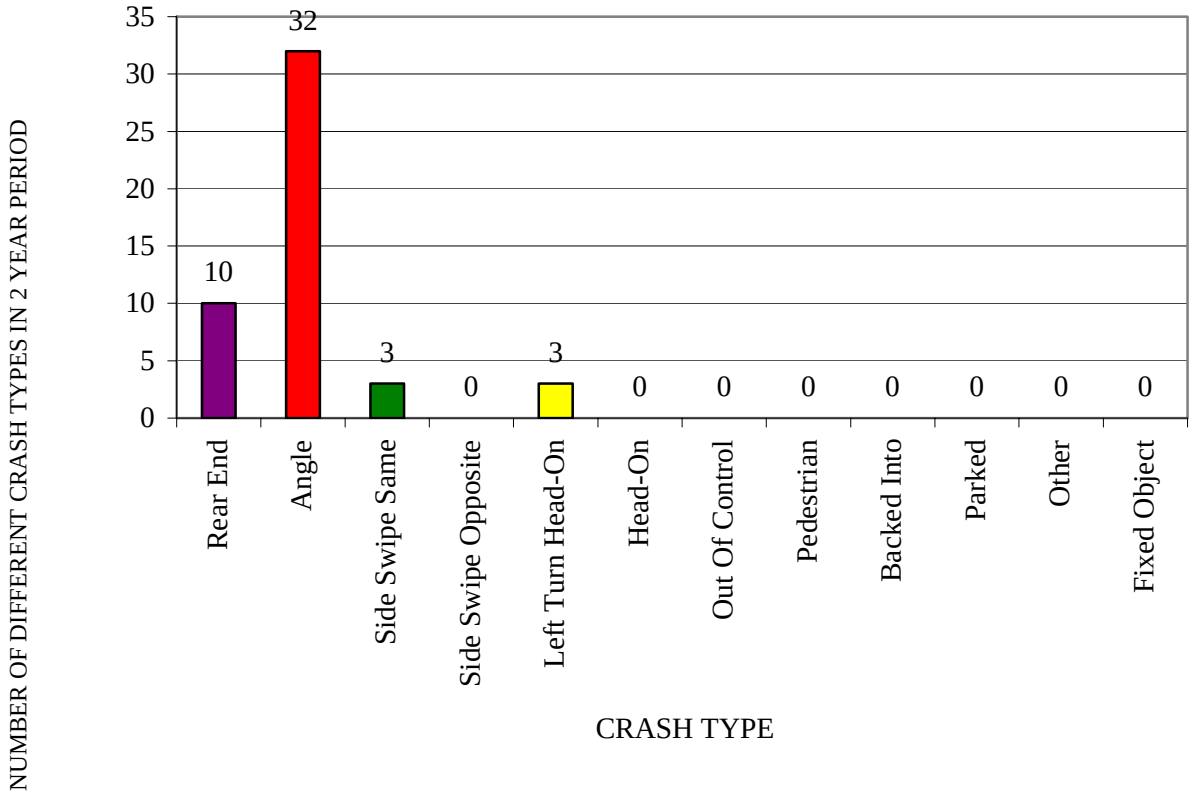


Figure 6.38.5 Two Year (2003-2004) crash experience at Rust Avenue and South Washington Avenue Intersection

Safety Deficiencies and Recommended Improvements

Frequently occurring angle and rear-end crashes at the intersection of Rust Avenue and South Washington Avenue could be attributed to poor visibility of traffic signals and inadequate clearance interval. The required clearance interval is computed and provided in the proposed signal-timing plan.

The existing signal-timing plan is modified to a cycle length of 60 seconds. The pavement markings should be upgraded for all approaches.

To improve signal visibility, the existing span-wire installation should be replaced by mast-arm signal installation, as shown in the proposed diagram in Figure 6.38.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

The existing yellow and all-red intervals are shown in Table 6.38.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.38.1. Existing and Proposed Clearance Intervals for the Rust Avenue and South Washington Avenue Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	5.0	1.0	3.6	2.0	Less angle, rear-end and injury crashes
E-W Through Phase	4.4	1.0	3.6	1.9	

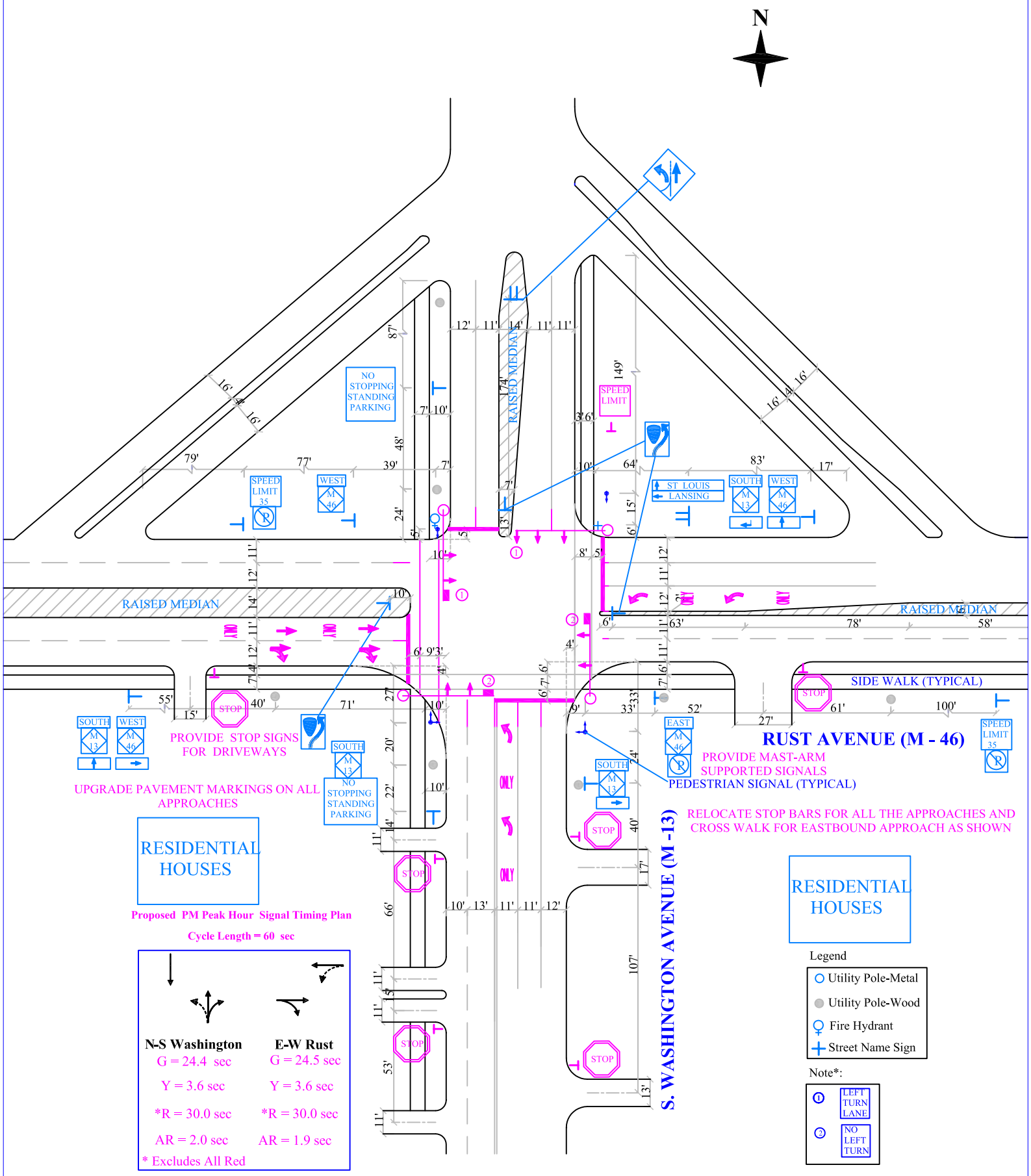
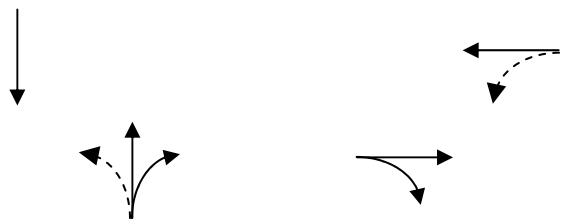


Figure 6.38.6 Proposed Diagram of Rust Avenue (M-46) and S. Washington Avenue (M-13) Intersection (Not to Scale)

The proposed PM peak hour signal timing plans based on a 60-second cycle length is as follows

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 60 Seconds



N-S Washington	**E-W Rust**
G = 24.4 sec	G = 24.5 sec
Y = 3.6 sec	Y = 3.6 sec
*R = 30.0 sec	*R = 30.0 sec
AR = 2.0 sec	AR = 1.9 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Reduce the yellow interval to 3.6 seconds for all phases.
- Increase the all-red interval to 1.9 seconds for Rust Avenue and 2.0 seconds for South Washington Avenue.
- Relocate stop bars at all the approaches and crosswalks for eastbound approach as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Upgrade pavement markings for all the approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowners should provide these stop signs.

- Optimize the signal timings for AM peak and off-peak hours.
- Implement the optimized PM signal timings.

Operational Evaluation

The results of the level of service analysis for the intersection of Rust Avenue and South Washington Avenue indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is B with a delay of 14.9 seconds per vehicle.

With the modifications to the existing signal timings including change in cycle length and the noted improvements, the overall intersection level of service will remain B with a delay of 12.0 seconds per vehicle. This reduction in overall delay will improve traffic flow through the intersection. The modified signal timing along with the adjusted clearance intervals will ensure enhanced safety.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.38.2.

Table 6.38.2.
Rust Avenue and South Washington Avenue
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
NB South Washington Avenue	14.8	B	11.8	B
SB South Washington Avenue	15.3	B	12.4	B
EB Rust Avenue	14.5	B	11.7	B
WB Rust Avenue	14.8	B	12.0	B
Overall Intersection	**14.9**	**B**	**12.0**	**B**

39. West Freeland Road and Midland Road (M-47)

The intersection of West Freeland Road and Midland Road is located at the northwestern boundary of Tittabawassee Township. West Freeland Road runs in the east-west direction and Midland Avenue runs in the north-south direction. West Freeland Road works as a collector road and Midland Road is a state trunk line also named as M-47.

West Freeland Road consists of one dedicated left-turn lane, one through lane and one dedicated right-turn lane at the eastbound approach. For thewestbound approach, West Freeland Road consists of one exclusive left-turn lane and one shared through/right-turn lane. West Freeland Road is approximately 50 feet wide at the eastbound approach and approximately 45 feet wide at the westbound approach.

Midland Road consists of an exclusive left turn lane, a through lane and one shared through/right-turn lane for both the southbound and northbound approach. Midland Road is approximately 57 feet wide at both approaches.

The speed limit for Midland Avenue is 35 miles per hour.

Land Use

The intersection of West Freeland Road and Midland Road has commercial establishments at all the four corners. There is a Burger King in the northwest corner, Freeland Chiropractic Center and Zentx Media Group Inc. in the northeast corner, a Marathon gas station on the southeast corner and a Tobacco Discount shop on the southwest corner.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.39.1. Details of the intersection approaches are shown in photographs 39a through 39d. Photographs show the mast-arm signal installation in place and

the land uses around the intersection. The east, north and southbound approaches are shown by photographs 39a, 39c and 39d.

Photograph 39a. EB W Freeland Road at Midland Road

Photograph 39b. WB W Freeland Road at Midland Road

Photograph 39c. NB Midland Road at W Freeland Road

Photograph 39d. SB Midland Road at W Freeland Road

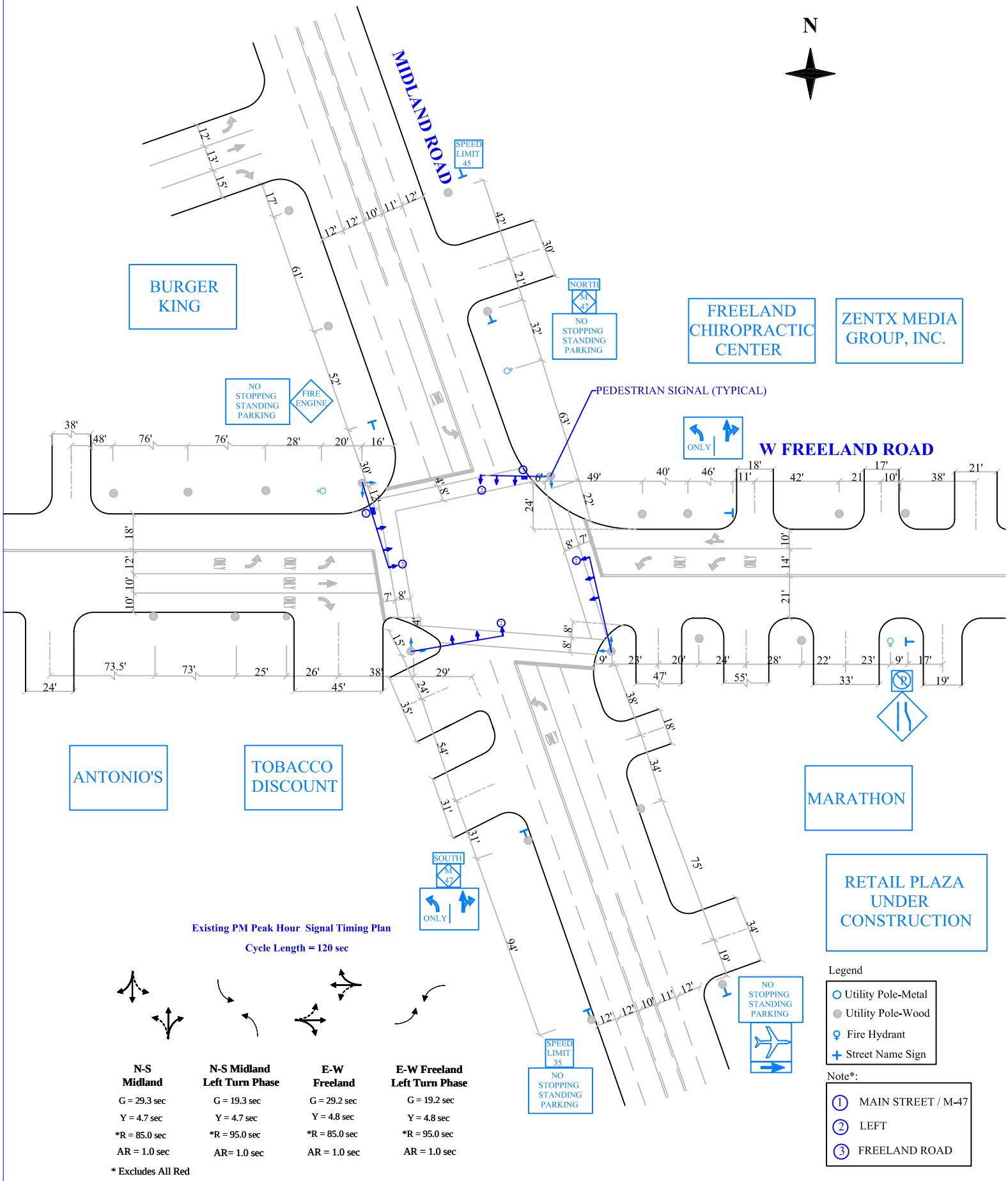


Figure 6.39.1.Existing Condition Diagram of Freeland Road and Midland Road Intersection
(Not To Scale)

Peak Hour Volumes

PM peak period traffic volumes were collected manually on a clear day on April 26, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other at two corners of the intersection and covering two intersecting approaches each. The results of the PM peak hour (4:45PM-5:45PM) traffic volumes are presented in Figure 6.39.2. The traffic patterns indicate volume in the north-south direction much higher than the east-west direction. The northbound left-turn volume is also considerably high.

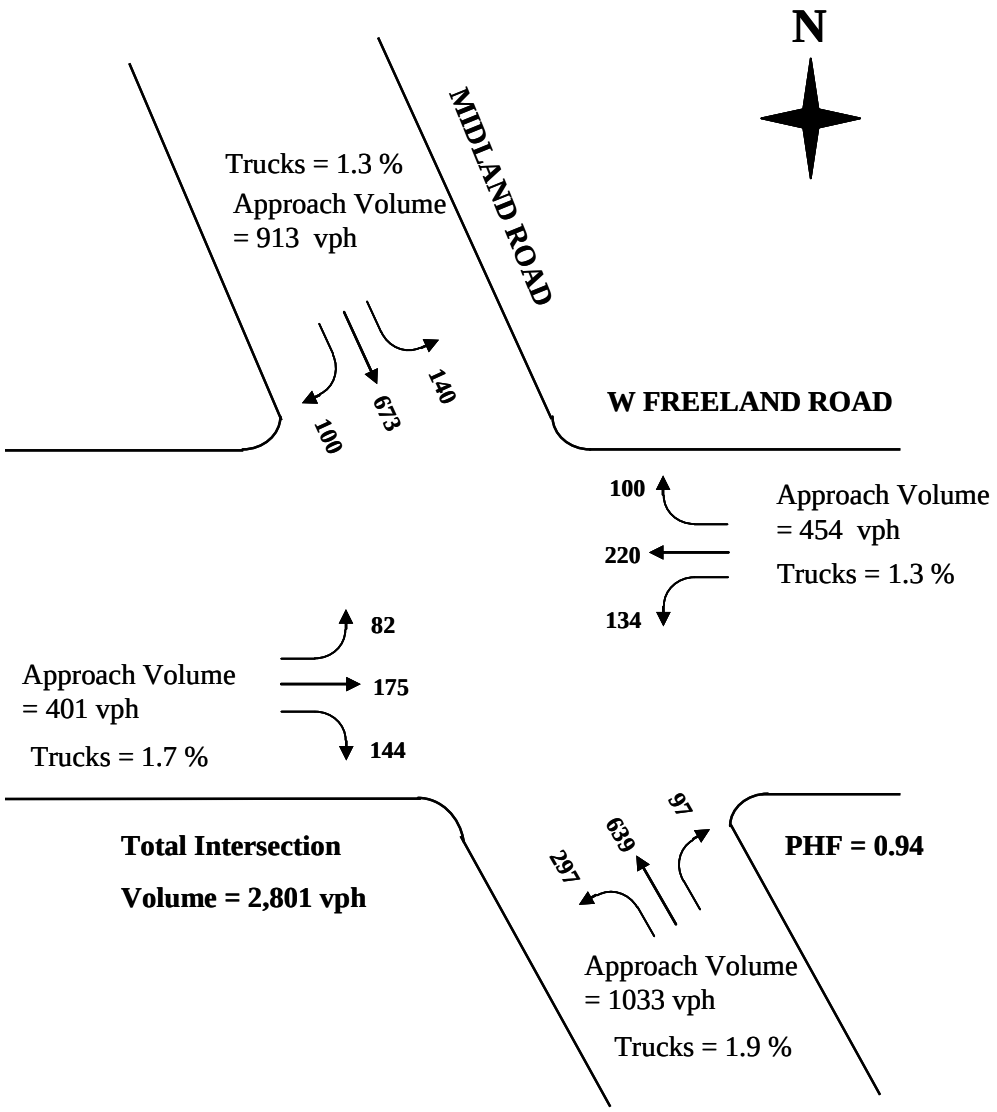


Figure 6.39.2. Existing PM Peak Hour (4:45-5:45 PM) Traffic Volumes for the Intersection of West Freeland Road and Midland Road

Traffic Control

This intersection is controlled by a four-phase traffic signal operating on 120-second cycle length. The existing clearance interval includes 4.8 seconds of yellow interval and 1.0 second of all-red interval for the east-west through and left-turn phases. For north-south through and left turn phases, it includes 4.7 seconds of yellow interval and 1.0 second of all-red intervals. The existing PM peak hour signal timing plan is shown in Figure 6.39.1.

Crash Analysis

A total of four crashes for the years 2003 and 2004 were recorded with an average annual crash frequency of two crashes. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 reports to identify crash patterns. Figures 6.39.3 and 6.39.4 show the collision diagram for these years respectively.

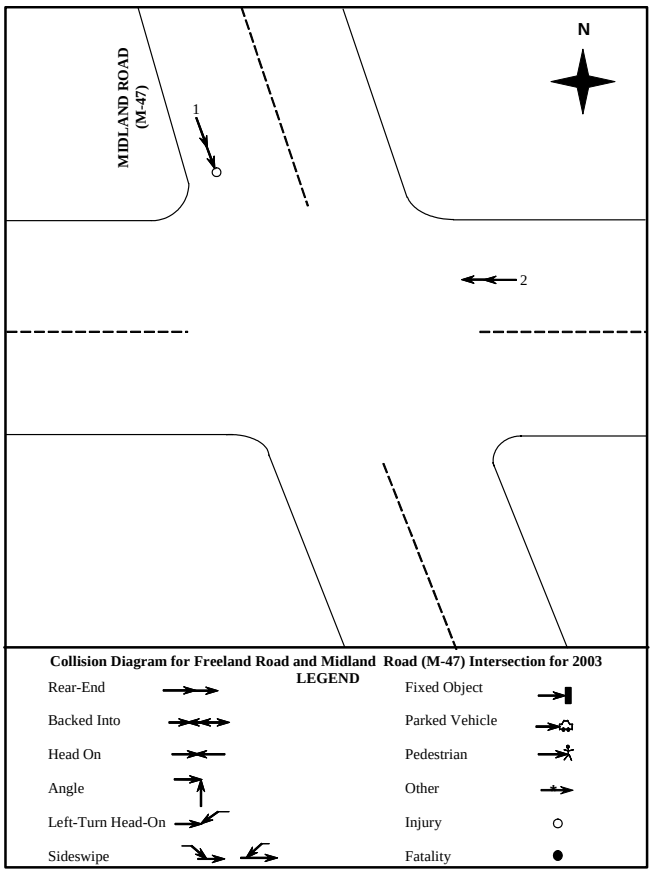


Figure 6.39.3. Collision Diagram for W. Freeland Road and Midland Road for 2003

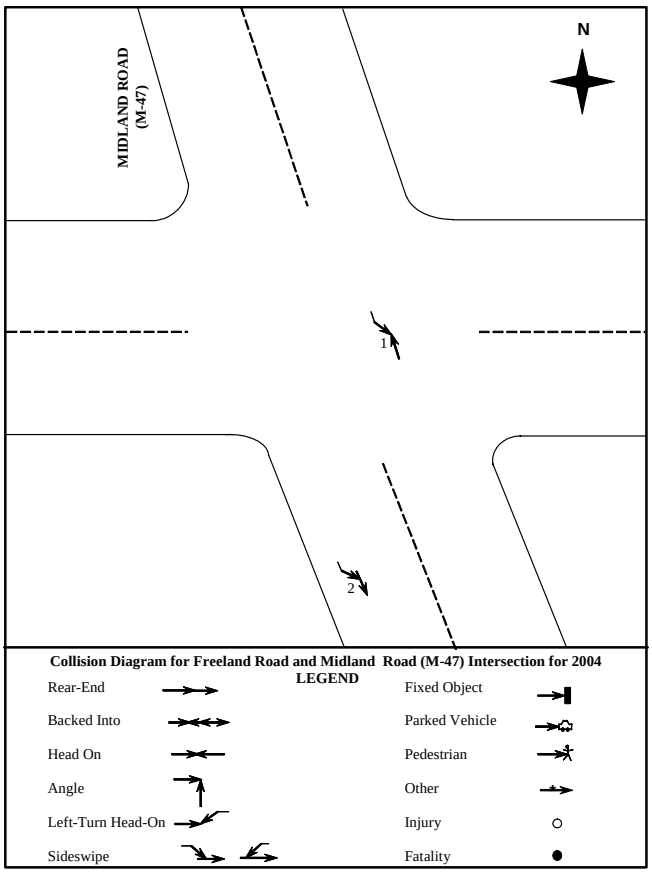


Figure 6.39.4. Collision Diagram for W. Freeland Road and Midland Road for 2004

The crash distribution, based upon types of crashes, is shown in Figure 6.39.5.

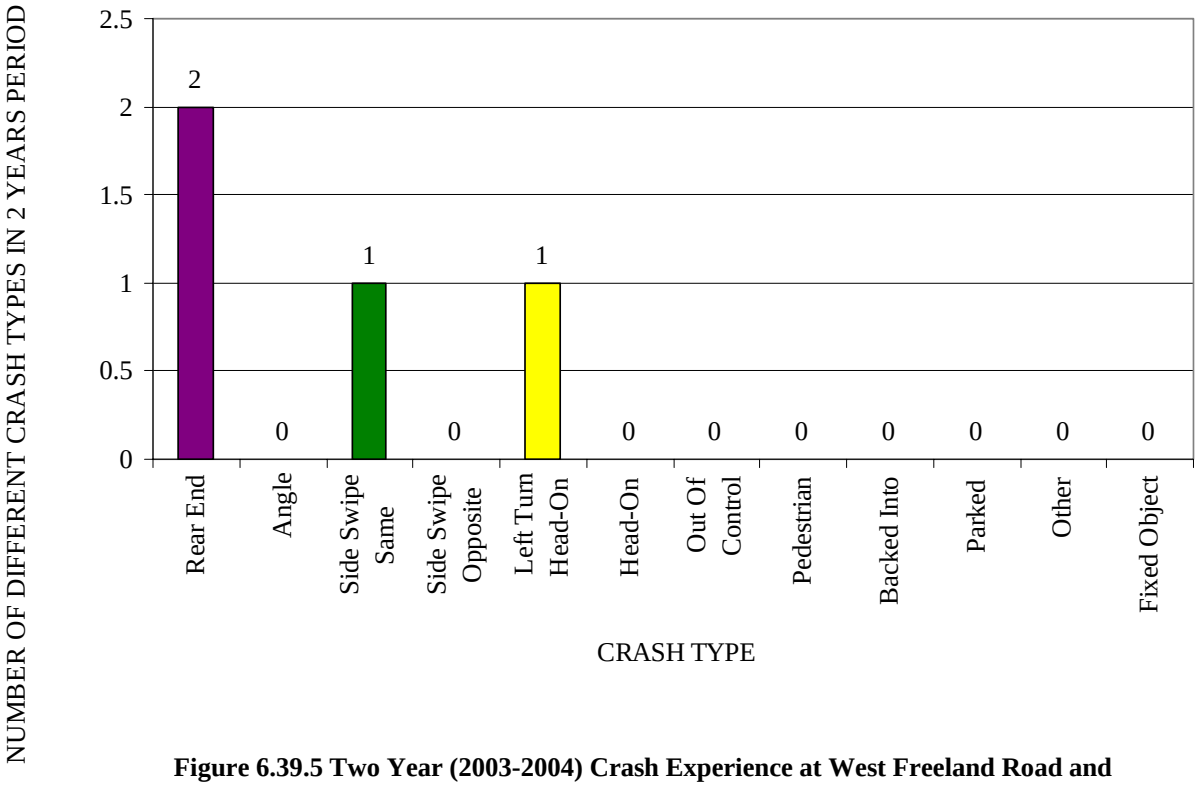


Figure 6.39.5 Two Year (2003-2004) Crash Experience at West Freeland Road and Midland (M-47) Road Intersection

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 0.16 per million entering vehicles, which is very low, considering the approach volume at the intersection.

Safety Deficiencies and Recommended Improvements

The crashes at the intersection of West Freeland Road and Midland Road could be attributed to poor visibility of traffic signals and inadequate clearance interval. The required clearance interval is computed and provided in the proposed signal-timing plan.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Post-mounted traffic signals with “LEFT” illuminated signs should be provided at the all corners of the intersection to assist left turning vehicles.

The proposed diagram in Figure 6.39.6 shows the improvements including a provision for post-mounted traffic signal for the left-turns along with upgrading the pavement markings for all the approaches and the proposed signal timing plan.

The existing yellow and all-red intervals are shown in Table 6.39.1, along with the proposed lengths these intervals and their potential benefits

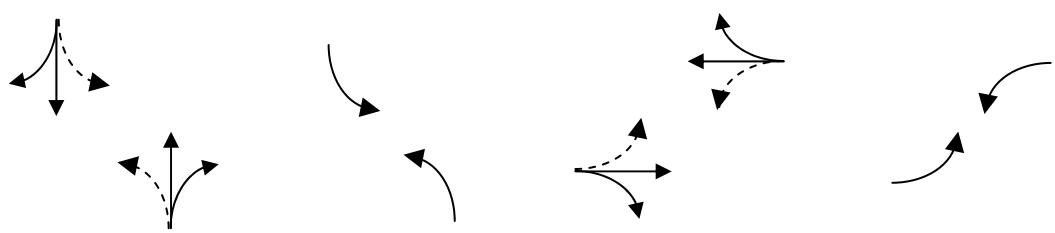
Table 6.39.1. Existing and Proposed Clearance Intervals for the West Freeland Road and Midland Road (M-47) Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	4.7	1.0	4.3	2.0	Less rear-end, left-turn head-on and injury crashes
N-S Left-turn Phase	4.7	1.0	4.3	2.0	
E-W Through Phase	4.8	1.0	3.9	2.1	
E-W Left-turn Phase	4.8	1.0	3.9	2.1	

The proposed PM peak hour signal timing plans based on a 90-second cycle length is as follows:

Proposed PM Peak Hour Signal Timing Plan

Cycle Length = 90 Seconds



N-S Midland	N-S Midland Left Turn Phase	E-W Freeland	E-W Freeland Left Turn Phase
G = 25.7 sec	G = 12.7 sec	G = 21.0 sec	G = 6.0 sec
Y = 4.3 sec	Y = 4.3 sec	Y = 3.9 sec	Y = 3.9 sec
*R = 58.0 sec	*R = 71.0 sec	*R = 63.0 sec	*R = 78.0 sec
AR = 2.0 sec	AR= 2.0 sec	AR = 2.1 sec	AR = 2.1 sec

* Excludes All Red

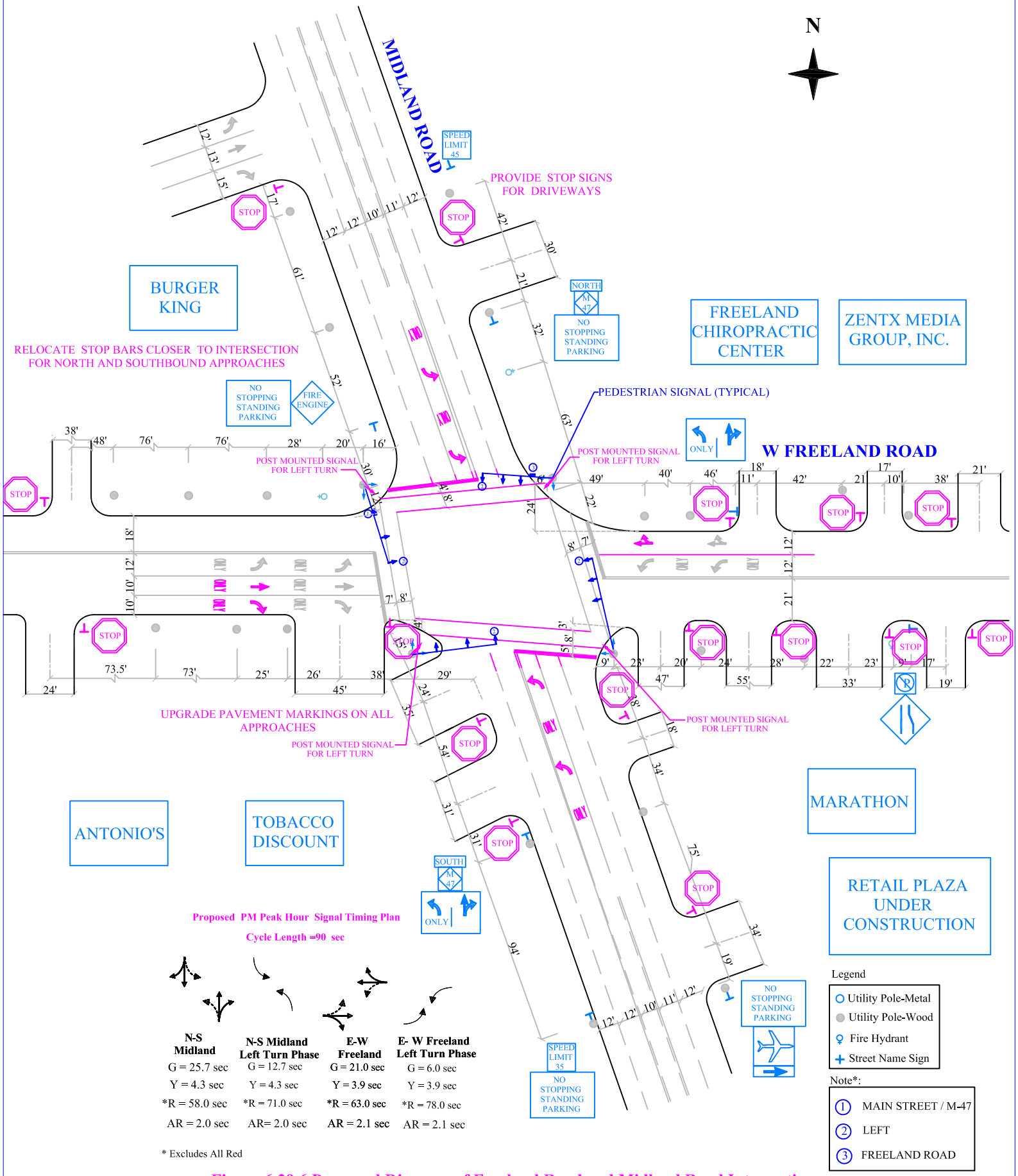


Figure 6.39.6 Proposed Diagram of Freeland Road and Midland Road Intersection (Not To Scale)

A list of the recommended improvements is summarized below:

- Reduce the yellow interval to 3.9 seconds for the West Freeland Road and 4.3 seconds for the Midland Road.
- Increase the all-red interval to 2.1 seconds for the West Freeland Road and to 2.0 seconds for the Midland Road.
- Install post-mounted traffic signals with a "LEFT" illuminated sign at all corners of the intersection for left-turning vehicles.
- Upgrade pavement markings for all the approaches.
- Relocate stop bars and crosswalks on the northbound and southbound approach, as per MMUTCD guidelines.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.
- Provide stop signs for all the driveways located near the intersection. Landowners should provide this stop signs.
- Optimize signal timings for AM and off-peak hours.
- Implement the optimized PM signal timings.

Operational Evaluation

The results of the level of service analysis for the intersection of West Freeland Road and Midland Road (M-47) indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is D with a delay of 47.0 seconds per vehicle.

With the modifications to the existing signal timings including changing the cycle length to 90-second and the noted improvements, the overall intersection level of service will be improved to C with a reduced delay of 32.3 seconds per vehicle. This reduction in overall delay will improve traffic flow through the intersection.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.39.2.

Table 6.39.2.
Freeland Road and Midland Road (M-47)
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
NB Midland Road	37.4	D	29.6	C
SB Midland Road	42.4	D	33.8	C
EB W. Freeland Road	50.4	D	34.0	C
WB W. Freeland Road	49.6	D	30.9	C
Overall Intersection	**47.0**	**D**	**32.3**	**C**

40. South Graham Road (M- 52) and Swan Creek Road

The intersection of Graham Road and Swan Creek Road is located in the southwestern part of Thomas Township. Swan Creek Road runs along the east-west direction and Graham Road runs along the north-south direction. Graham Road is a state trunk line and Swan Creek Road is classified as a major arterial street.

Graham Road consists of one dedicated left-turn lane, one through lane and an exclusive right-turn lane at the north and southbound approaches. Graham Road is approximately 60 feet wide at the north leg and is approximately 44 feet wide at the south leg of the intersection.

Swan Creek Road consists of one dedicated left-turn lane and one shared through/right-turn lane at east and westbound approaches. Swan Creek Road is approximately 38 feet wide at both legs of the intersection.

Land Use

The intersection of Graham Road and Swan Creek Road has a fire station on the southeast corner, Cookies Bar and Restaurant on the southwest corner, Gulliver Auto Repair on the northeast corner and Image Artz on the northwest corner. There are driveways within 150 feet of the intersection which serves the vehicles going to and from these establishments.

Field Data

Field investigations were carried out to collect data related to intersection geometry, lane use and designation, adjacent land uses and curb cuts, traffic control devices, roadside obstacles and all other relevant information within an approximate radius of 150 feet around the intersection.

The existing characteristics of the intersection, including traffic control and surrounding land uses, are schematically shown in Figure 6.40.1. Details of the intersection approaches are shown in photographs 40a through 40d.

Photograph 40a. NB Graham Road at Swan Creek Road

Photograph 40b. SB Graham Road at Swan Creek Road

Photograph 40c. EB Swan Creek Road at Graham Road

Photograph 40d. WB Swan Creek Road at Graham Road

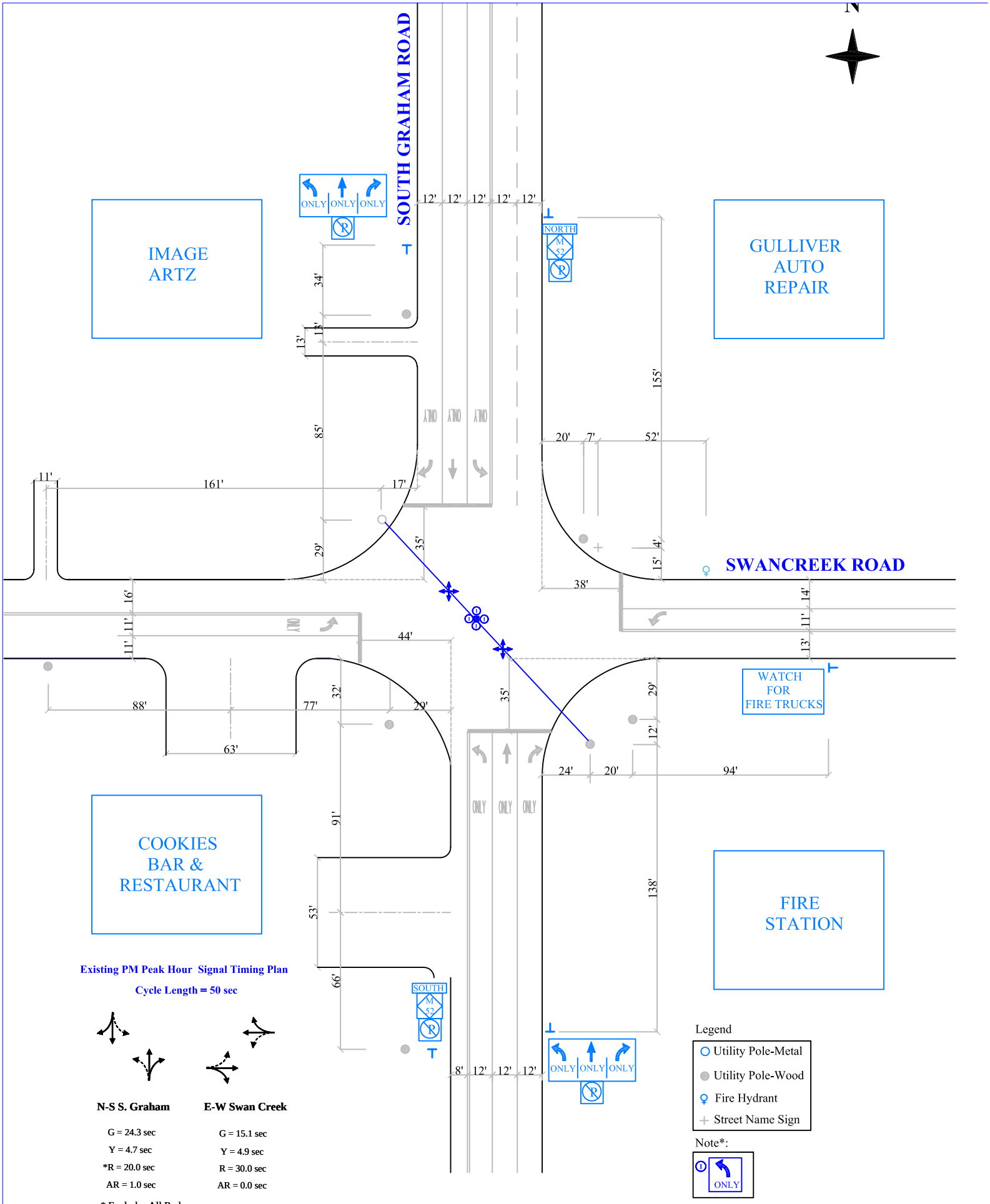


Figure 6.40.1.Existing Condition Diagram of South Graham Road and Swan Creek Road Intersection (Not to Scale)

Peak Hour Volumes

PM peak hour traffic volumes were manually collected on a clear day on April 26, 2006 from 4 PM to 6 PM by two people standing diagonally opposite to each other and covering two intersecting approaches each. The peak hourly volume occurred from 5:00PM to 6:00 PM and is presented in Figure 6.40.2. The traffic patterns indicate that there is no significant difference between volume in the east-west and north-south direction except the westbound left-turn volume is relatively higher.

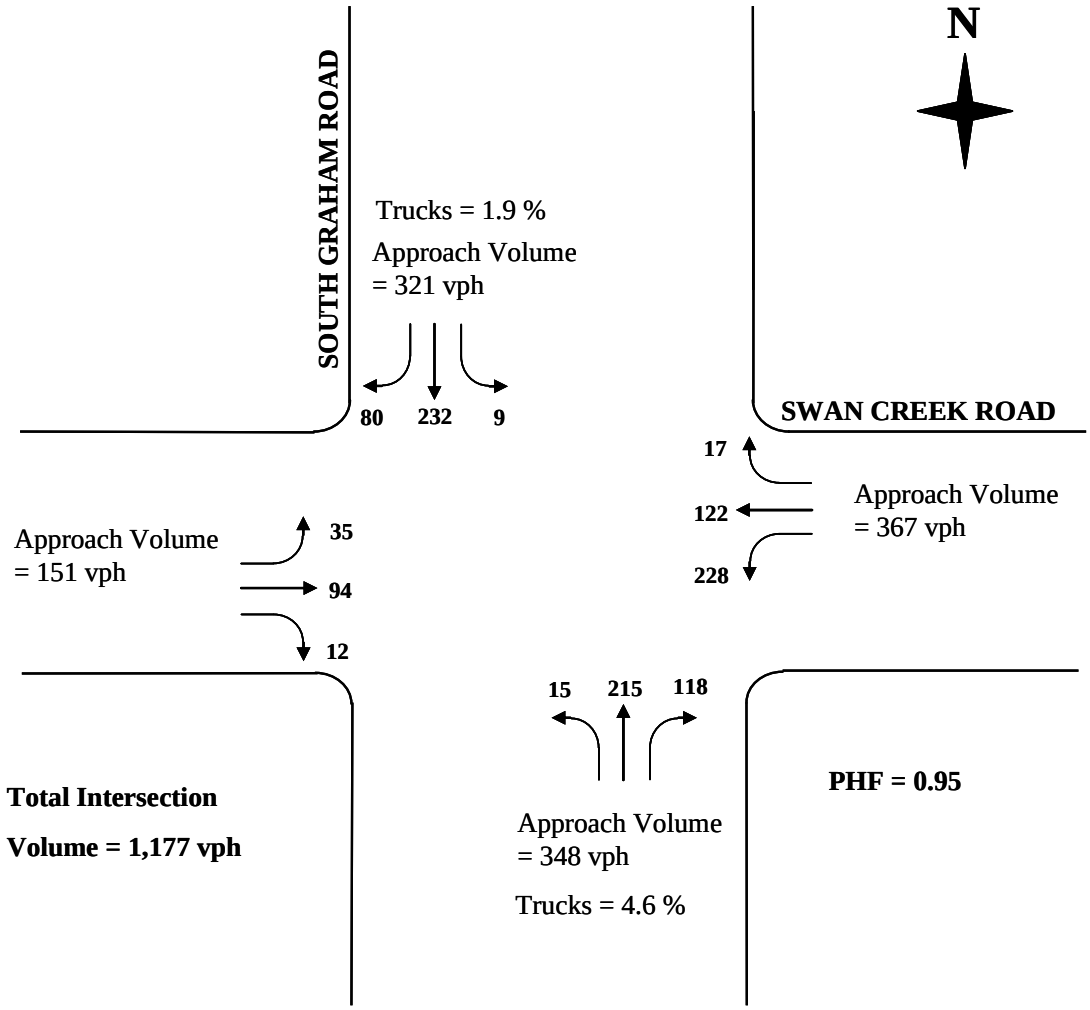


Figure 6.40.2. Existing PM Peak Hour (5:00-6:00 PM) Traffic Volumes for the Intersection of South Graham Road and Swan Creek Road

Traffic Control

This intersection is controlled by a two-phase traffic signal operating on 50-second cycle length. The existing clearance interval includes a 4.7 seconds yellow interval and 1.0 second all-red

interval for the north-south phase while the east-west phase has a 4.9 seconds yellow interval and no all-red interval. The existing PM peak hour signal timing plan is shown on the Figure 6.40.1. The intersection does not have pedestrian signals nor marked crosswalks on any approaches.

Crash Analysis

A total of 13 crashes for the years 2003 and 2004 combined were recorded, with an average annual crash frequency of seven crashes. Collision diagrams for these years (2003-2004) were prepared based on the UD-10 reports to identify crash patterns. Figures 6.40.3 and 6.40.4 show the collision diagrams for these years respectively.

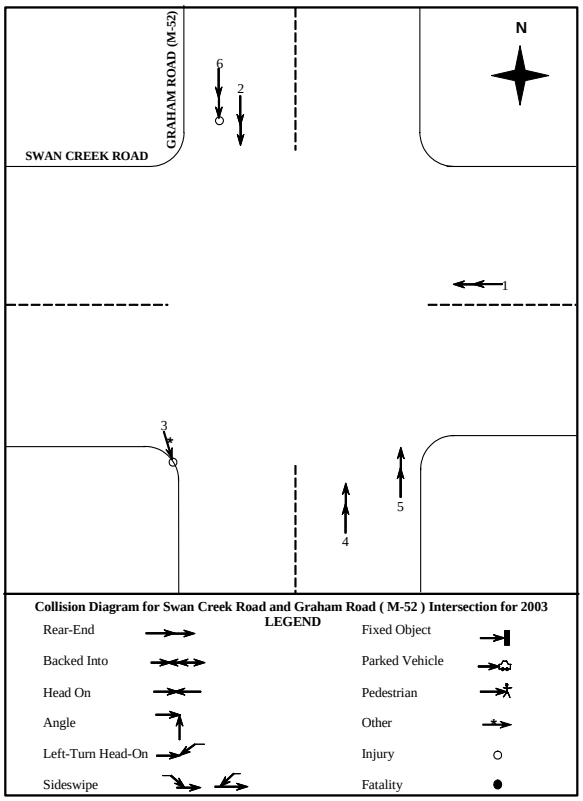


Figure 6.40.3. Collision Diagram for Graham Road and Swan Creek Road for 2003

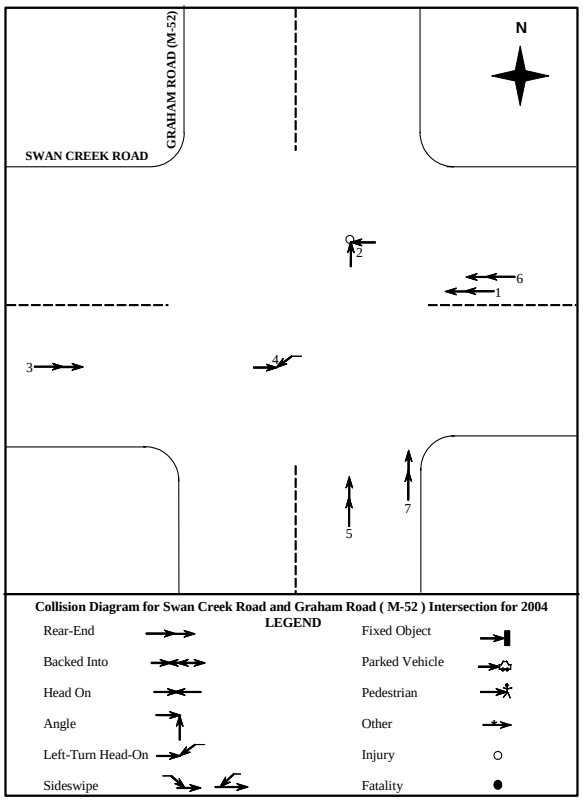


Figure 6.40.4. Collision Diagram for Graham Road and Swan Creek Road for 2004

The annual crash rate per million entering vehicles is computed by considering the peak hour total entering volume to be 8% of the ADT. The rate is 1.30 per million entering vehicles, which is nominal, considering the approach volume at the intersection. The majority of crashes

occurring near the intersection are rear-end. These crashes might have occurred due to inadequate signal timing and insufficient clearance interval.

The crash distribution, based upon types of crashes, is shown in Figure 6.40.5. Rear-end crashes are the predominant type of crashes reported at the intersection. The corresponding two-year crash frequency for these crashes includes 10 rear-end crashes, one left-turn head-on crashes, one angle crash, and one other crash.

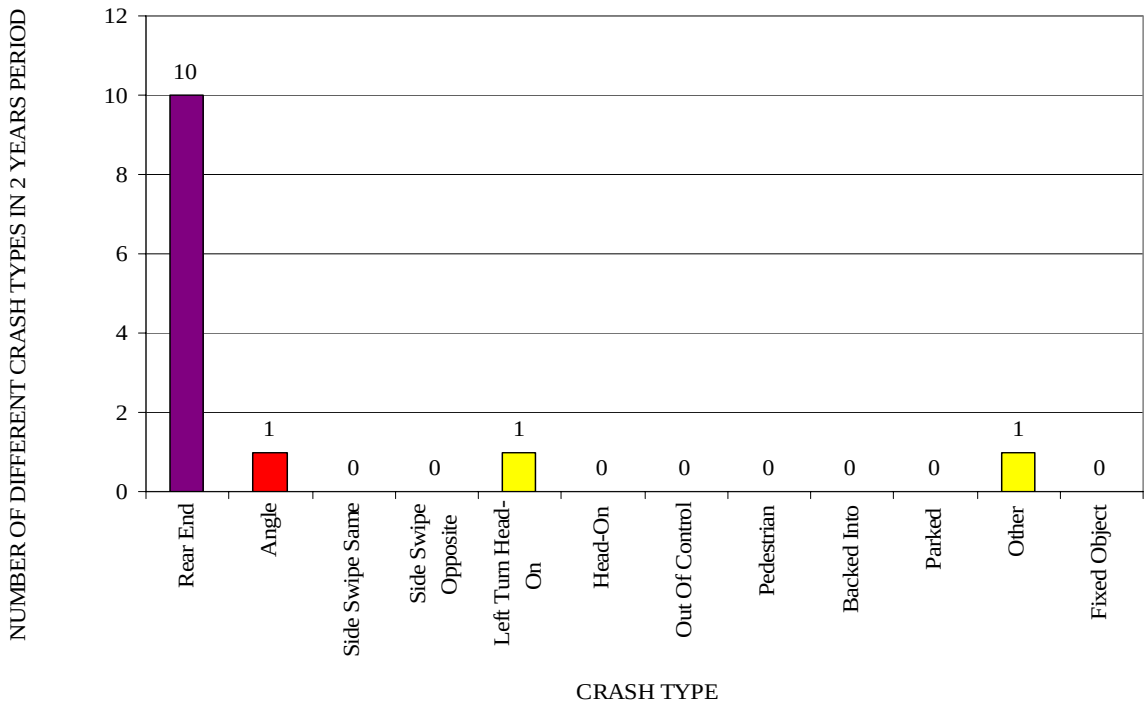


Figure 6.40.5 Two Year (2003-2004) crash experience at Swan Creek Road and South Graham Road Intersection

<u>Safety Deficiencies and Recommended Improvements</u>

Majority of the crashes occurring at the intersection of Swan Creek Road and South Graham Road are of the rear-end type, happening due to several possible reasons including restricted or poor visibility of traffic signals and improper clearance interval. The required clearance interval is computed and provided in the proposed signal-timing plan.

Left-turn phase is provided in westbound approach, as left-turn volume for this approach satisfies left-turn warrant. The large amount of westbound left-turning vehicles requires that a protected left-turn phase for Swan Creek Road be added. Because the left-turn volume is so large, the warrant for a protected left-turn phase is met.

The existing signal-timing should be modified with new cycle length of 60 seconds. The pavement markings should be upgraded for the all approaches.

To improve signal visibility, the existing span wire installation should be replaced by a mast-arm signal installation, as shown in the proposed condition diagram in Figure 6.40.6.

Additionally, speed limit signs should be posted downstream of the intersection for all directions of travel. These signs should be posted at locations where they do not currently exist.

Post-mounted traffic signals with "LEFT" illuminated signs should be provided at the northeast and southwest corners of the intersection to assist left turning vehicles.

The proposed diagram in Figure 6.40.6 shows the recommended improvements including mast-arm signal installation and post-mounted traffic signals for left-turns.

The existing yellow and all-red intervals are shown in Table 6.40.1, along with the proposed lengths of these intervals and their potential benefits.

Table 6.40.1. Existing and Proposed Clearance Intervals for the South Graham Road and Swan Creek Road Intersection

Interval/Phase	Existing Interval Length (sec)		Proposed Interval Length (sec)		Potential Benefit
	Yellow	All-Red	Yellow	All-Red	
N-S Through Phase	4.7	1.0	5.0	1.0	Less rear-end, left-turn head-on, angle, and injury crashes
E-W Through Phase	4.9	0.0	5.0	1.2	
E-W left turn Phase	-	-	5.0	1.2	

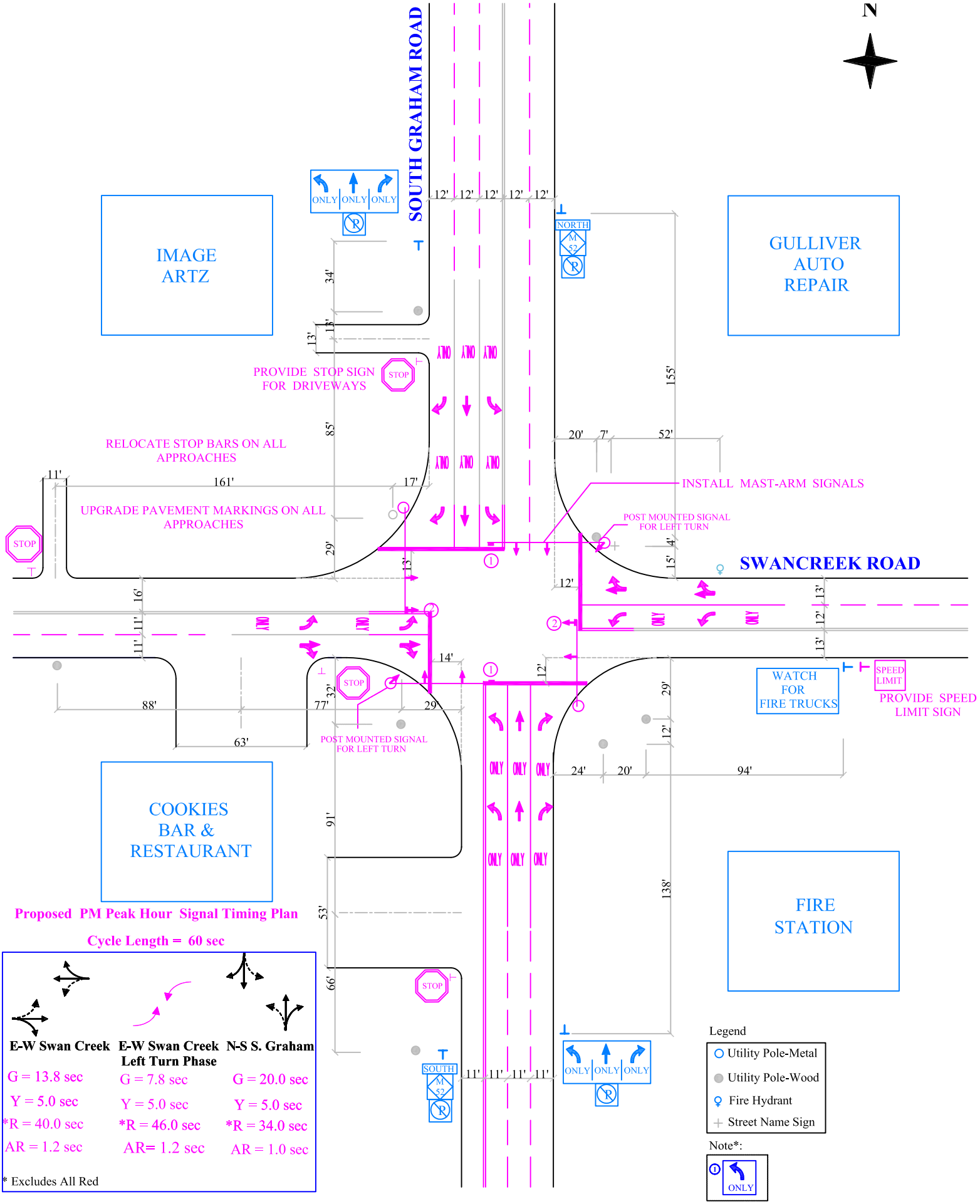
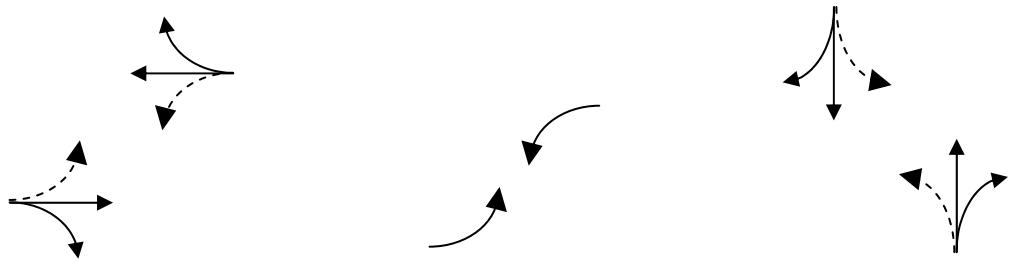


Figure 6.40.6 Proposed Diagram of South Graham Road and Swan Creek Road Intersection (Not to Scale)

The proposed PM peak hour signal timing plan based on a 60-second cycle length is shown below:

Proposed PM Peak Hour Signal Timing Plan
Cycle Length = 60 Seconds



E-W Swan Creek	E-W Swan Creek Left Turn Phase	N-S S. Graham
G = 13.8 sec	G = 7.8 sec	G = 20.0 sec
Y = 5.0 sec	Y = 5.0 sec	Y = 5.0 sec
*R = 40.0 sec	*R = 46.0 sec	*R = 34.0 sec
AR = 1.2 sec	AR= 1.2 sec	AR = 1.0 sec

* Excludes All Red

A list of the recommended improvements is summarized below:

- Increase the yellow interval to 5.0 seconds for all phases.
- Increase the all-red interval to 1.2 second for the Swan Creek Road phases.
- Provide an excusive left-turn phase for Swan Creek Road.
- Relocate stop bars at all approaches, as per MMUTCD guidelines.
- Upgrade the existing span-wire configuration to a mast-arm signal installation.
- Install post-mounted traffic signals with a “LEFT” illuminated sign at the northeast and southwest corners of the intersection for east and westbound left-turning vehicle.
- Upgrade pavement markings for all approaches.
- Post speed limit signs downstream of the intersection for all directions of travel. Signs should be posted at locations where they do not currently exist.

- Provide stop signs for all driveways located near the intersection. Landowners should provide these stop signs.
- Optimize signal timings for AM and off-peak hours.
- Implement the optimized PM signal timings.

Operational Evaluation

The results of the level of service analysis for the intersection of South Graham Road and Swan Creek Road indicate that, under existing conditions during the PM peak hour, the overall intersection level of service is A with a delay of 9.2 seconds per vehicle.

With the modifications to the signal timings and phasing and the noted improvements, the overall intersection level of service will change to B with a delay of 14.5 seconds per vehicle. Even though the level of service and delay for the intersection decreased after all proposed changes are made, mobility through the intersection occurs at a higher level of safety.

The level of service results for the existing conditions and with the proposed improvements is summarized in Table 6.40.2.

Table 6.40.2.
South Graham Road and Swan Creek Road
Level of Service Analysis
PM Peak Hour

Approach/Street	LOS for Existing Conditions		LOS with Proposed Improvements	
	Delay (Sec/Vehicle)	LOS	Delay (Sec/Vehicle)	LOS
EB Swan Creek Road	11.8	B	15.9	B
WB Swan Creek Road	15.5	B	13.6	B
NB S. Graham Road	4.8	A	14.6	B
SB S. Graham Road	5.1	A	14.7	B
Overall Intersection	**9.2**	**A**	**14.5**	**B**

7.0 EVALUATION PLAN

Effectiveness evaluation studies are performed to provide decision-makers with insightful information on the effectiveness of various projects and programs. "Evaluation is an assessment of the value of an activity as measured by its success or failure in achieving a predetermined set of goals or objectives" (*Highway Safety Evaluation- Procedural Guide*, Federal Highway Administration, 1981, FHWA-TS-81-219). An effectiveness evaluation is the statistical and economic assessment of the extent to which a project or program achieves its ultimate safety goal of reducing the number and severity of traffic crashes.

In general, there are two types of effectiveness evaluations that may be performed for safety projects and programs: crash-based evaluations and non-crash based evaluations (*Highway Safety Improvement Program*, Federal Highway Administration, 1980, FHWA-TS-81-218). The objective of a crash-based evaluation is to assess the effectiveness of a completed safety initiative. The purpose of a non-crash based evaluation is to assess the effectiveness of the intermediate measures, since it requires considerable time to accumulate an adequate amount of traffic crash data.

7.1 Crash-Based Evaluations

The purpose of an evaluation plan is to provide overall guidance for the evaluation study, since the actual evaluations may not take place for several years. Crash-based evaluation plans were developed as a part of this study and include the following elements:

- Projects for evaluation
- Project purpose
- Evaluation objectives and Measures of Effectiveness (MOEs)
- Experimental plans
- Data requirements

Projects for Evaluation

The study sites to be included in the evaluation are those where the recommended improvements have been implemented, and at least one year of traffic crash data is available for the 'after' period (period of time after the installation of the improvements). For the current study, this would consist of the 40 study intersections in Saginaw County.

Project Purpose

The purpose of the "Traffic and Safety Engineering Services to MPOs" project is:

- To reduce traffic crashes for the following crash types (in general):
  - Total crashes
  - Specific crash types based on the countermeasures implemented at the individual intersections
- To reduce the severity of traffic crashes
  - Injury crashes
  - Fatal crashes

Evaluation Objectives and MOEs

The objective of the evaluation is to determine the effect of the implemented countermeasures on total crashes, injury crashes and fatal crashes, and specifically on the targeted crash types of rear end crashes, right angle crashes, and left-turn-head-on crashes.

An economic analysis (cost-benefit analysis) should be included in the evaluation to assess the effectiveness of the projects in term of traffic and safety benefits of crash reductions, and the cost of the improvement project. This analysis should only be performed if a statistically significant reduction was observed for the project.

The following are the MOEs that should be included in the evaluation:

- Percent change in crash frequencies and rates (crashes per million entering vehicles) 'before' and 'after' the improvements are installed for various crash categories:
  - Total crashes
  - Injury crashes
  - Fatal crashes
  - Rear-End crashes
  - Right angle crashes
  - Left-turn head-on crashes

Experimental Plans

The evaluation of safety treatments can be performed using a variety of experimental plans. Among them, the 'before and after' (B&A) (Figure 7.1) and the 'before and after with control' (B&A with control) (Figure 7.2) are recommended. In addition, consideration should also be given to conduct the evaluation using the Empirical Bayes method. The difference in these evaluation procedures lies in the assumptions made in quantifying the number of crashes that would have occurred in the 'after' period, had the safety treatment not been implemented.

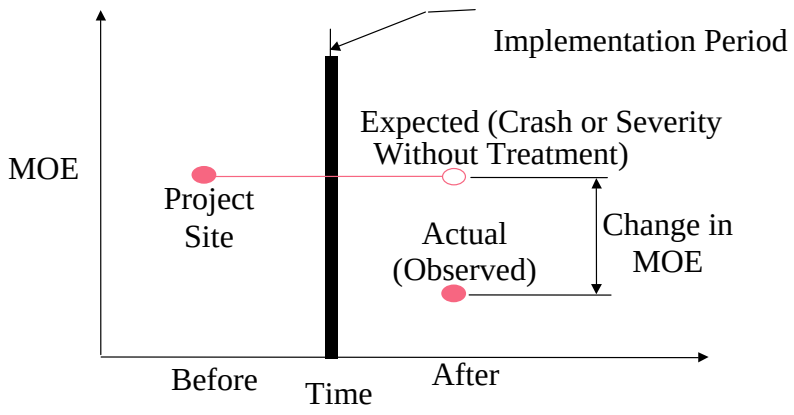


Figure 7.1. 'Before and After' Study Plan

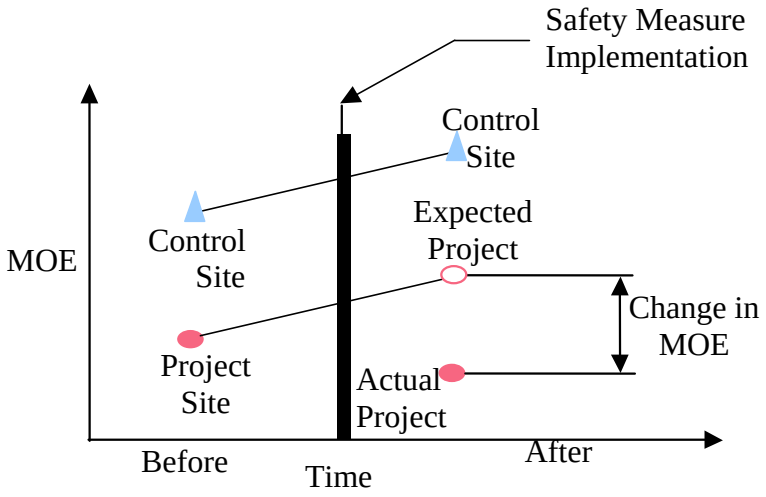


Figure 7.2. 'Before and After with Control' Study Plan

B&A Plan

Conceptually, the B&A study assumes that the expected 'after' crash MOEs without improvement is the same as the 'before' crashes. Therefore, the change in the crash MOEs is measured as the difference in the 'before' and 'after' crash MOEs.

B&A with Control Plan

The B&A with control study plan compares the percent change in the MOEs at the project site (test site) with the percent change in the MOEs at similar site(s) without improvement (control sites) for the same time period. An assumption is made that the test sites, in absence of the improvements, exhibit traffic crash experience similar to the control sites. Any difference between the crash experience at the test and control sites is attributed to the improvements.

Empirical Bayes Method

The Empirical Bayes method may be used in an evaluation to account for 'regression to the mean' bias that may be associated with 'before and after' evaluations. Some researchers believe that when countermeasures are installed at high crash locations, the results of a B&A study are biased. According to Persuad and Hauer ('Comparison of Two Methods for Debiasing Before-and-After Accident Studies', ***Transportation Research Record 975***, Washington, D.C., 1984), "the bias is caused by the erroneous assumption that the number of accidents on a system in the period before treatment is an unbiased estimate of what should be expected to occur on the system during an equivalent after period had the treatment not been applied". They also suggest "even if the treatment had not been applied, the number of crashes at high crash locations would reduce in the after period, due to regression to the mean effects" (Hauer, E., ***Observational Before-After Studies in Road Safety-Estimating the Effect of Highway and Traffic Engineering Measures on Road Safety***, Elsevier Science Ltd, New York, 1997). Thus, when simple B&A comparisons are made, some researchers claim that the safety treatments appear to be more effective than they actually are.

According to Hauer, the Empirical Bayes method increases the precision of estimation and corrects for the regression to the mean bias (Bauer, K.M. and Harwood, D.W., ***Statistical Models of At–Grade Intersection Accident-Addendum***, FHWA-RD-99-094, Federal Highway Administration Report, March 2000). In this method, traffic crash data at the test site and the crash experience expected at

similar sites are used along with weighted factors to calculate the expected number of crashes at the test sites without the applied treatment.

Data Requirements

For the 'before' period, traffic crash data should consist of a two-year period. The traffic crash data contained in this report may be used to quantify the crash experience of the 'before' period. Traffic crash data for the 'after' period should be obtained for the period beginning one month after the installation of the countermeasures, and collected for a one to two year period, as the data becomes available. The source of the data should be the UD-10 traffic crash reports, which may be obtained from the Michigan State Police. Traffic crash data for the same 'before' and 'after' period is needed for similar control sites that were not improved as a part of this study. Traffic volume data will be needed to calculate crash rates (number of crashes per million entering vehicles). The intersection volumes may be estimated using the mid block traffic volume data available from the MDOT.

7.2 Non-Crash-Based Evaluations

Surrogate measures are proxy-measures of variables that indicate the risks and hazards at a location, and may be used as a relative measure of safety. Traffic crashes may occur due to inadequate engineering design, improper driver behavior, or a combination of the two factors, as well as others. Thus, operational surrogate measures can be assessed in order to quantify driver behavior patterns at the study sites 'before' and 'after' the installation of the safety improvements. Examples of surrogate measures include traffic conflict, speeding, red light violations at signalized intersections, violation of driver expectancies and others.

Traffic conflict or 'near miss' is an evasive action taken by a driver to avoid an impending collision and may be observed by abrupt braking, vehicles swerving and others. In lieu of adequate 'after' crash data, traffic conflict and other studies may be conducted in order to assess the safety improvements. Another surrogate measure may include the frequency and magnitude of vehicles exceeding the speed limit at locations where mitigation strategies were implemented to alleviate speeding problems. These evaluations studies are typically supplemented by comprehensive evaluations of traffic crash data, based on the program's continuation in the future.

8.0 RECOMMENDATIONS

As a part of this study, two types of improvements are recommended including site-specific improvements, and policy-type improvements that can be implemented over time. The following are the recommended improvements to reduce crashes and injuries in Saginaw County:

Site-Specific Improvements

Engineering Strategies:

1. Detailed site-specific improvements have been recommended for all 40 of the study intersections and are presented in Sections 5.0 and 6.0 of this report.

Education/Enforcement Strategies

2. **Provide increased enforcement to identify drivers under the influence of alcohol near the intersection** of Bay Road and Davenport Avenue.

3. **Encourage local high schools to conduct seminars to emphasize the consequences of drinking and driving**.

4. **Provide increased enforcement against unbelted driving at the intersection** of Bay Road and Tittabawassee Road.

5. **Provide increased speed enforcement near the intersections** of Bay Road and Tittabawassee Road, Tittabawassee Road and NB I-675, Tittabawassee Road and Center Road, Tittabawassee Road and Mackinaw Road, Gratiot Road and Graham Road, and North Michigan Avenue and Davenport Avenue.

6. **Provide increased enforcement of red light running at the following intersections**: Bay Road and Davenport Avenue, Bay Road and State Street, Bay Road and Congress Avenue, Bay Road and Schust Road, Tittabawassee Road and Center Road, Tittabawassee Road and Mackinaw Street, Gratiot Road and Center Road, Gratiot Road and River Road, Gratiot Road and Wheeler Road, North Michigan Avenue and

Davenport Avenue, North Michigan Avenue and West Genesee Avenue, Michigan Avenue and Court Street.

Policy-Type Engineering Improvements

7. **Provide adequate clearance intervals (yellow and all-red intervals) at all signalized intersections**, which follow the Institute of Transportation Engineers (ITE) recommended practice. Please note that ITE's recommended practice was used as the basis for the *Revised Guidelines for Traffic Signal Vehicle Change Intervals*, published by MDOT on April 23, 2002.

   While reviewing the study intersections, it was observed that many of the signalized intersections had yellow intervals that exceeded the lengths recommended by ITE, and they had no all-red intervals, or all-red intervals with lengths shorter than that recommended by ITE. The existing lengths of the clearance intervals may be contributing to a large proportion of rear end crashes, right angle crashes and in some cases left-turn head-on crashes. A high occurrence of red light running was also identified from this study, a portion of which may be attributed to the excessive lengths of the yellow intervals and the lack of all-red intervals.

8. **Install actuated or semi-actuated traffic signals at intersections where one or more movements have low or fluctuating traffic flow**. This type of improvement will help improve traffic flow and safety, and may help reduce negative impacts to the environment related to vehicular emissions and air pollution due to unnecessary acceleration and deceleration cycles.

9. **Incorporate access management techniques into the planning process for future developments.** These strategies are a large concern for developments located on a corner property of an intersection. Such strategies include locating driveways as far away from the intersection as possible, limiting access of a corner property to one driveway on each of the adjacent roads, and maintaining an adequate separation distance between driveways, intersections and other nearby driveways. Strategic options may be reviewed

to implement the concept of shared driveways by adjacent commercial developments to reduce the total number of driveways in a commercial area around specific intersections. Additional information on access management strategies can be found in MDOT's manual entitled: *Reducing Traffic Congestion and Improving Traffic Safety in Michigan Communities: The Access Management Guidebook*, published in September 2001.

10. **Establish a program to review, evaluate operations and retime, if necessary, all signalized intersections** on a defined schedule (such as, every "x" years, not to exceed "x" years, etc). In general, proper signal timing is not universally achieved or maintained in numerous jurisdictions across the United States. Maintaining proper signal timing will help improve safety and flow and allow agencies to perform routine field inspections that are needed in order to maintain high levels of safety and efficiency.

11. **Install left-turn lanes where left-turn movements are allowed and adequate pavement width/right-of-way is available.** In the absence of an exclusive left-turn lane, vehicles must stop and wait to turn left in the through traffic lane. A stationary vehicle in the through lane increases the risk of traffic crashes. The stopped vehicle may also block the view of oncoming through traffic for opposing drivers turning left. Additionally, some drivers who attempt to pass a vehicle waiting to turn left, must enter the right-most lane close to the intersection. The sudden appearance of a through vehicle in the right lane may be unexpected by an opposing driver turning left across the path of the through vehicle, contributing to left-turn head-on crashes. Thus, consideration should be given to installing exclusive left-turn lanes at intersections that do not currently have them, and that could be accommodated without any widening.

12. **Improve signal visibility at signalized intersections** by increasing the size of the signal lenses to 12-inch diameters at all signalized intersections (regardless of the approach speeds), using LED signal lenses, installing back plates, providing additional post-mounted signals for left turning vehicles, placing signal heads at the far side of the intersection and providing one traffic signal for each lane of traffic. The latter two can be accomplished by using the mast arm signal support system.

13. **Implement an annual pavement marking and signage improvement/maintenance program.** Currently, pavement markings at some of the intersections and along some of the road segments are generally faded, increasing the difficulty for drivers to see and properly use the lanes as they were intended. It is recommended that the pavement markings, such as lane markings, stop bars, crosswalks, arrow symbols and text be reviewed and re-marked, as needed, on an annual basis.

14. **Install speed limit signs downstream of all intersections** for all directions of travel. These signs should be posted at locations where they do not currently exist.

15. **Improve on-street parking facilities by:**
    - Restricting on-street parking in close proximity to the intersection (100 to 150 feet from the center line of the intersection).
    - Providing better delineation and signage of the on-street parking facilities to improve sight distance and visibility for drivers.
    - Redesigning on-street angular parking to have a protective barrier to separate through traffic from parked vehicles. The protective barrier may consist of a narrow raised median. The degree of angular parking may need to be revised in order to accommodate the protective barrier.

General Education/Enforcement Strategies

16. **Participate in national public awareness campaigns to discourage red light running at signalized intersections**. The Federal Highway Administration sponsors an annual event entitled "Stop on Red" Week to discourage such driver behavior. More information can be found on the Internet at http://safety.fhwa.dot.gov/fourthlevel/pro_res_srlr_week.htm.

17. **Continue to participate in educational and enforcement programs** in conjunction with the Office of Highway Safety Planning/Michigan State Police, as done in the past years, such as the Drive Michigan Safely, Operation Nightcap, and SPOTLIGHT programs.