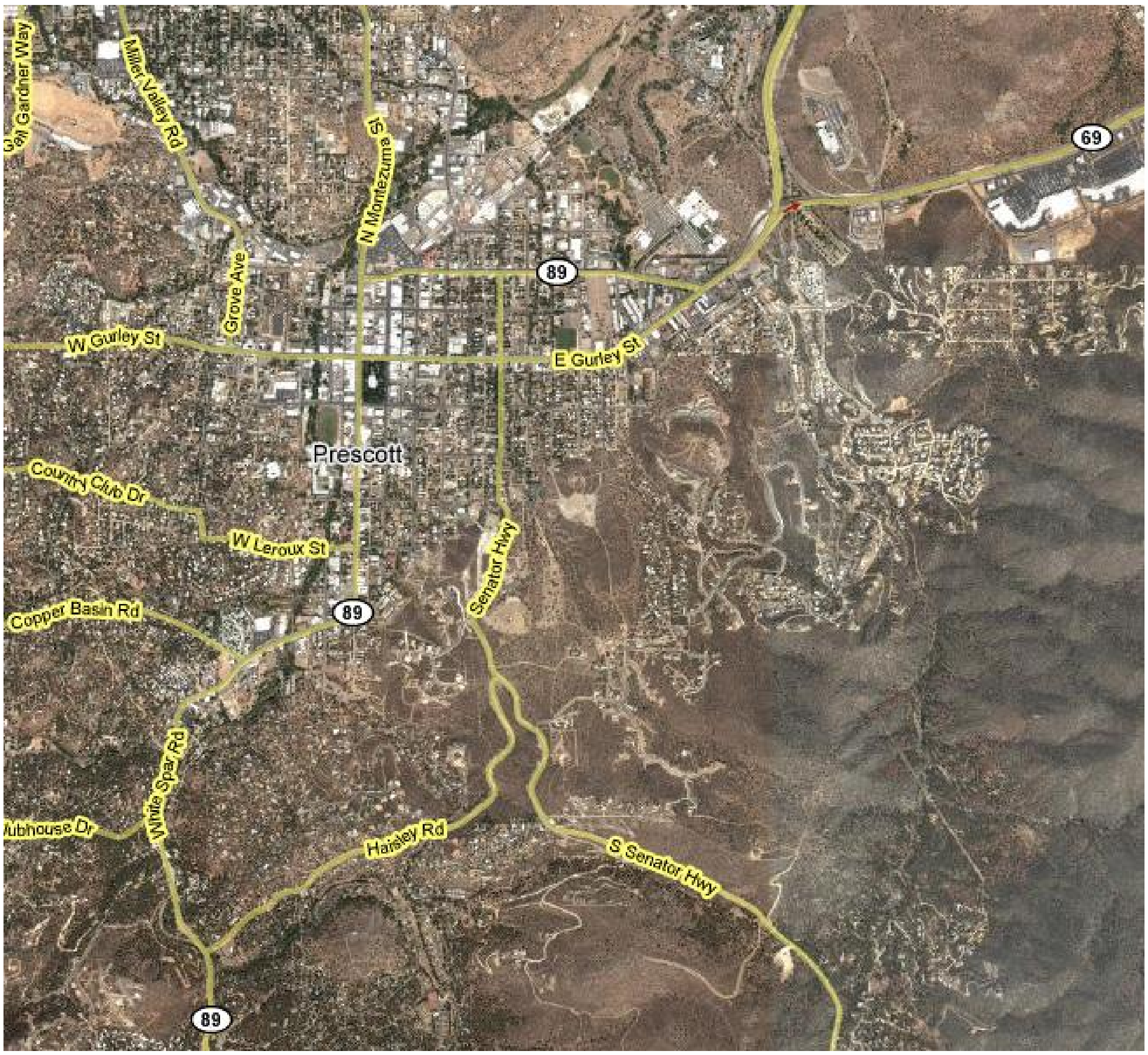
Prescott Southside Traffic Circulation Enhancement Project



FINAL REPORT

FEBRUARY 2008

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INTRODUCTION

Study Overview and Purpose

The City of Prescott General Plan (May 2004) envisions substantial growth with associated increases in traffic volumes on the existing and future street networks. This growth combined with infill and redevelopment will create street infrastructure and traffic management needs in various areas of the City at a neighborhood level that are not addressed by the Central Yavapai Metropolitan Planning Organization (CYMPO) 2030 Regional Transportation Plan adopted in December 2006. Other previous studies including the Year 2018 Transportation Plan and Southeast Prescott Circulation Element Traffic Analysis (BRW, 1994), could be used as reference, but do not include the current travel patterns in the study area reflecting the rapid growth of the last decade.

The purpose of this study is to assist the city in developing a series of recommendations related to improving transportation circulation in the South Prescott study area, for the short and long range timeframe seeking to balance both internal and through traffic conveyance with quality of life in the various neighborhood areas. The study area is bounded by Montezuma/White Spar on the west; Gurley and SR89/SR69 interchange on the north, White Spar/Haisley intersection to the south, and Robinson Road to the east. Figure 1 depicts the study area.

Organization of the Report

This report documents the method and results of the study and presents a recommended transportation alternative. The report will contain summary of findings with the corroborating data included in 5 Sections and three Appendices. Section I will present the current transportation conditions while Section II presents the capacity analysis of the current conditions. Section III summarizes the future transportation and traffic conditions. Section IV presents the roadway improvements and modifications with the recommended improvements and preliminary costs. Section V recaps the public involvement effort.

I. TRANSPORTATION PLANNING – 2006 CONDITIONS

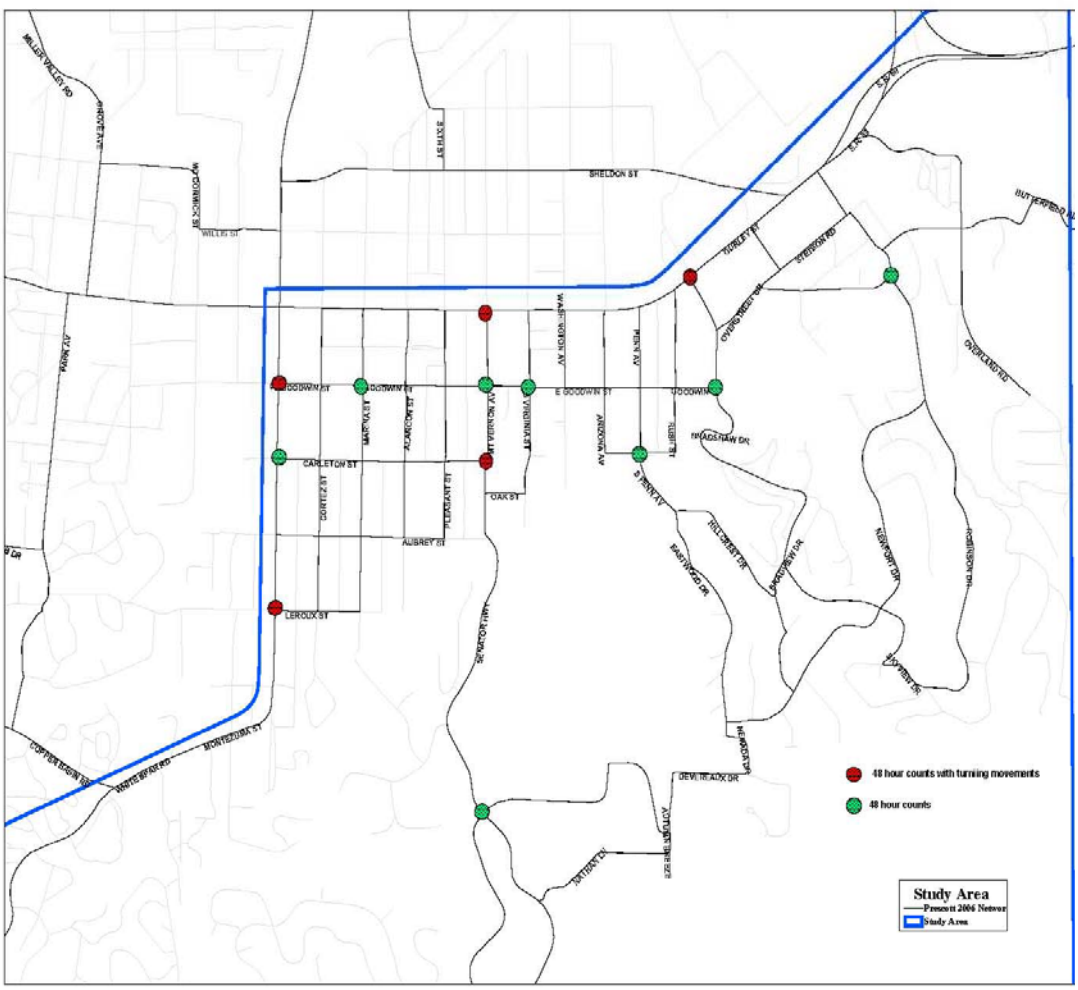
Transportation Planning

The transportation planning effort concentrated on developing data to be used in subsequent analysis tasks. Traffic count data collected in 2006 and early 2007 and the CYMPO model were used to ascertain the travel pattern and the current travel demand characteristics in the study area. Additionally, origin and destination (OD) travel surveys were conducted to identify the usage of Mt. Vernon Avenue as a potential alternative route to Gurley Street.

Traffic Count Data

A traffic count effort was conducted on February 13 and Feb 14, 2007 within the study area to collect 48 hour counts at specific locations and AM and PM peak period turning movement counts at selected intersections. The turning movement data was collected from 7 a.m. to 10 a.m. and from 3 p.m. to 6 p.m. Figure 1 depicts the 48 hour and turning movement count locations. Results from this effort are displayed in the traffic analysis section of this report.

Figure 1. Count Locations



Origin Destination Study

An origin and destination (OD) survey was conducted on February 15, 2007 to ascertain the trip characteristics along Mt. Vernon on the following locations:

- On Mt. Vernon south of Carleton
- Mt. Vernon and Goodwin Intersection
- Mt. Vernon and Carleton Intersection

The survey was conducted between the hours of 7 a.m. and 9 a.m. and 4 p.m. and 6 p.m. People traveling along those routes were stopped and asked specific questions regarding their trip characteristics. Appendix A contains the survey instrument used in the OD study in Tables A1-A3. Questions regarding trip origin, trip destination, purpose of the trip and probable routes used in the trip making were the questions of choice. Additional visual information was collected regarding the vehicle type and auto occupancy.

Figures 2 through 4 depict the results of the OD with respect to the route characteristics represented in the percent of the trip beginning at a specific location and traveling along Mt. Vernon to reach their destination. Both AM and PM conditions are presented. More detailed survey results are located in Appendix A, Figures A5-A28.

Figure 2. O-D Results – Goodwin Street

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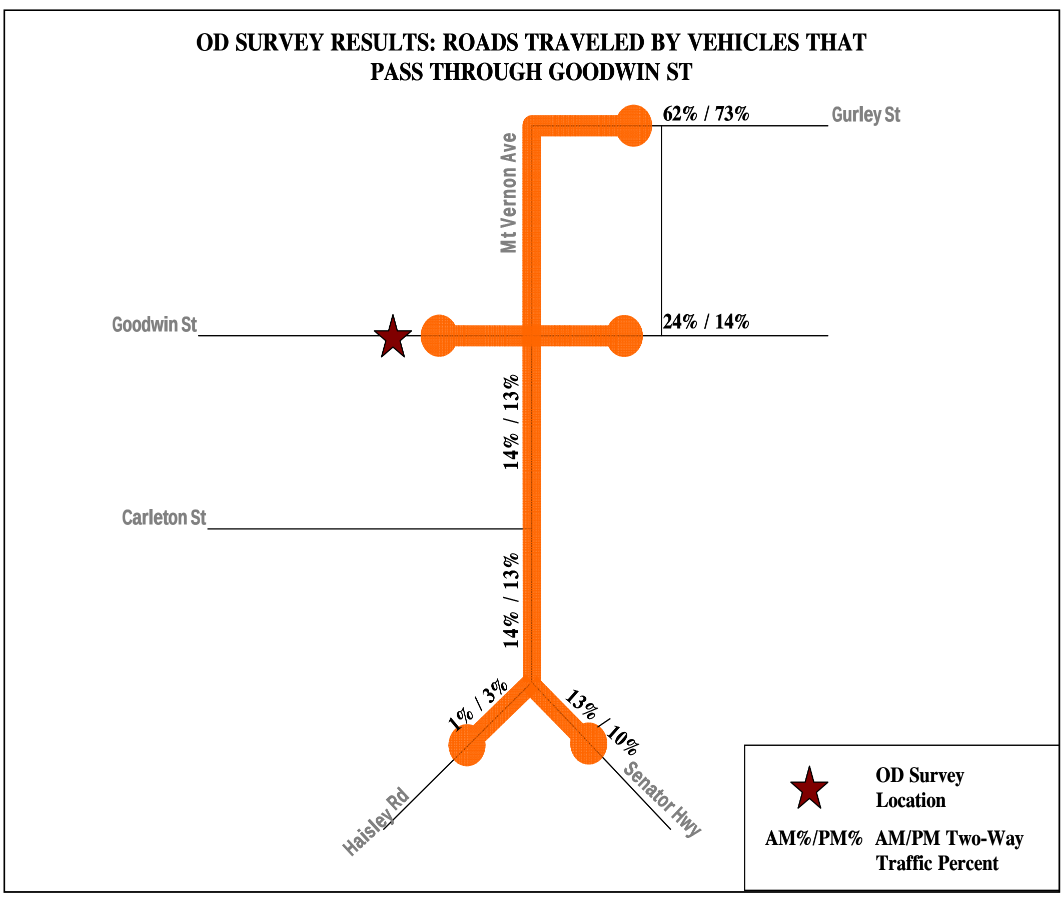


Figure 3. O-D Results – Carleton Street

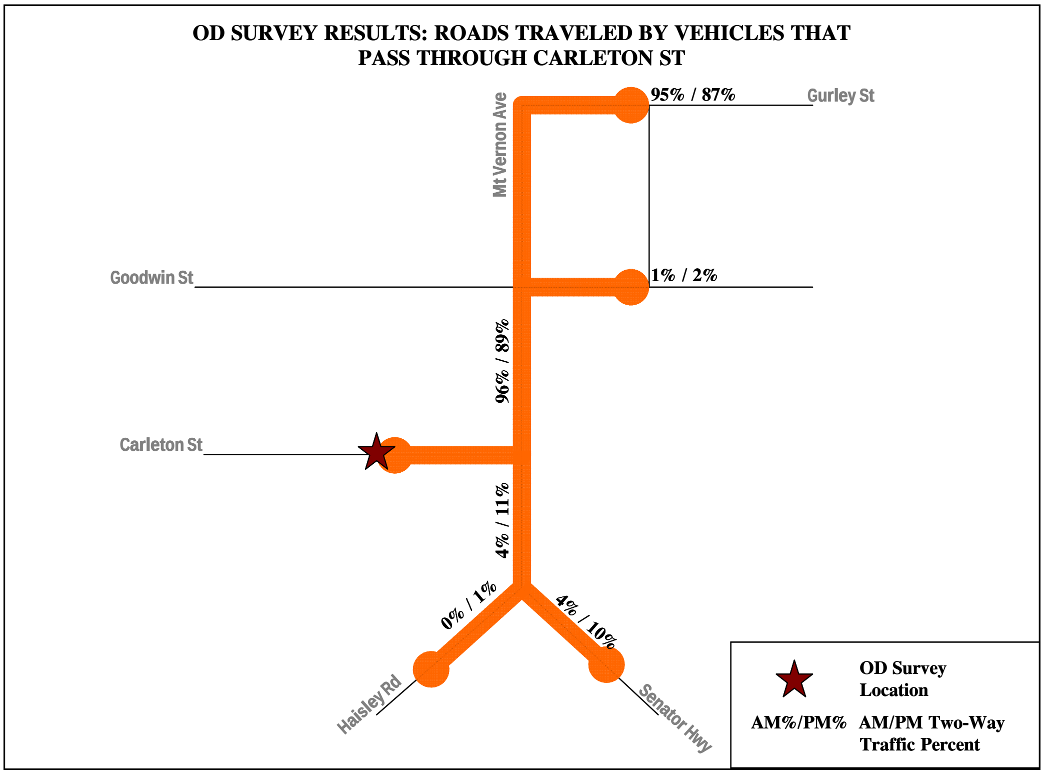
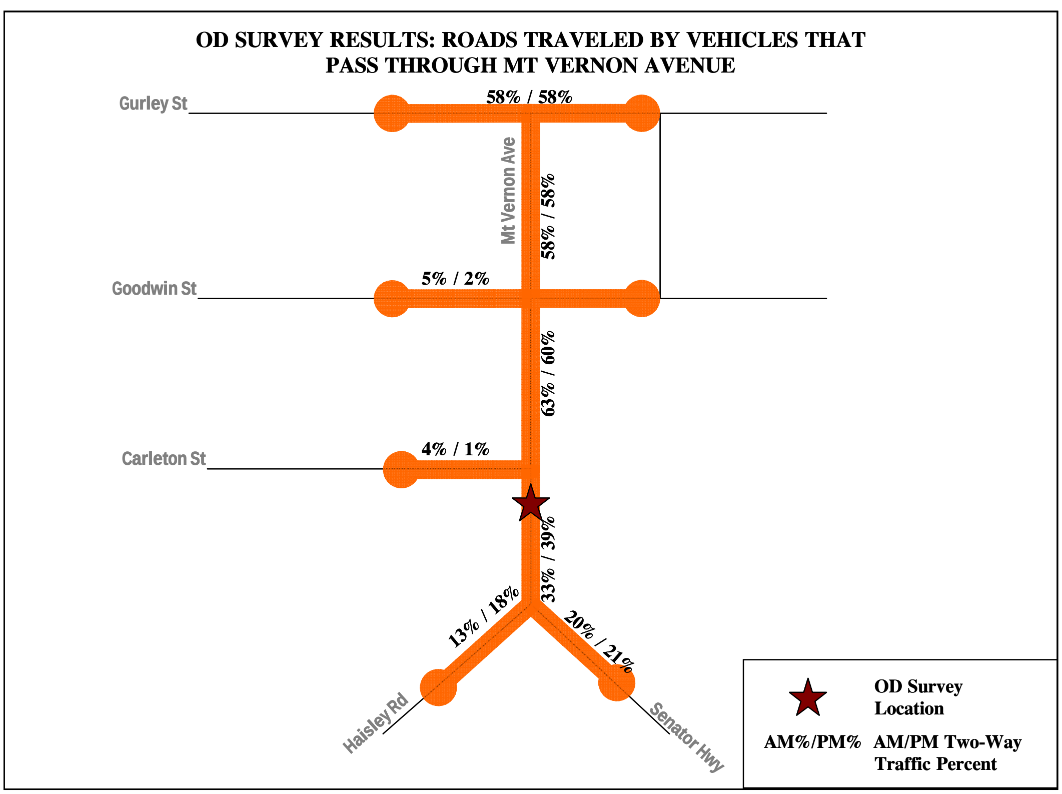


Figure 4. OD Results - Mt. Vernon Avenue



The OD surveys showed that Prescott residents are the most frequent users of Mt. Vernon Avenue for a variety of reasons, but predominantly for work in the morning, while there is a more equal distribution of trip purposes during the afternoon period.

The majorities of the travelers were using automobiles and were single occupant. However, the Mt Vernon location exhibited a large percentage of pick-up trucks in both the morning and afternoon time periods. Goodwin, instead, showed a large number of Vans in the afternoon period. Travelers intercepted at the Goodwin Street and Carleton Street locations, confirmed that they used Mt Vernon Avenue at least once a day or several times during the day.

II. CAPACITY ANALYSIS

CYMPO Travel Demand Model – 2006 Conditions

The 2004 CYMPO travel demand model was used as base for the traffic forecasting effort for the area shown in Figure 6. The figure also displays the roadway system considered in this analysis with the total number of lanes identified in blue for a four-lane facility and in green for a two lane facility. The model was revised in the study area to refine the traffic analysis zones (TAZs) structure and revise the socioeconomic data to 2006 conditions. Twenty-five (25) additional TAZs were created to better represent the travel pattern distribution in the study. Additionally, a more defined roadway network was also added to the regional system. The roadway mid-link level of service is an indicator of the facility performance with respect to the available capacity. Figure 5 provides a depiction of the levels of service for arterial roadways. Figure 7 displays the validation effort results based on recent, as well as, past traffic counts and the roadway level of service.

Figure 5. Roadway Segment Level of Service

Figure 6. Study Area and Roadway Number of Lanes

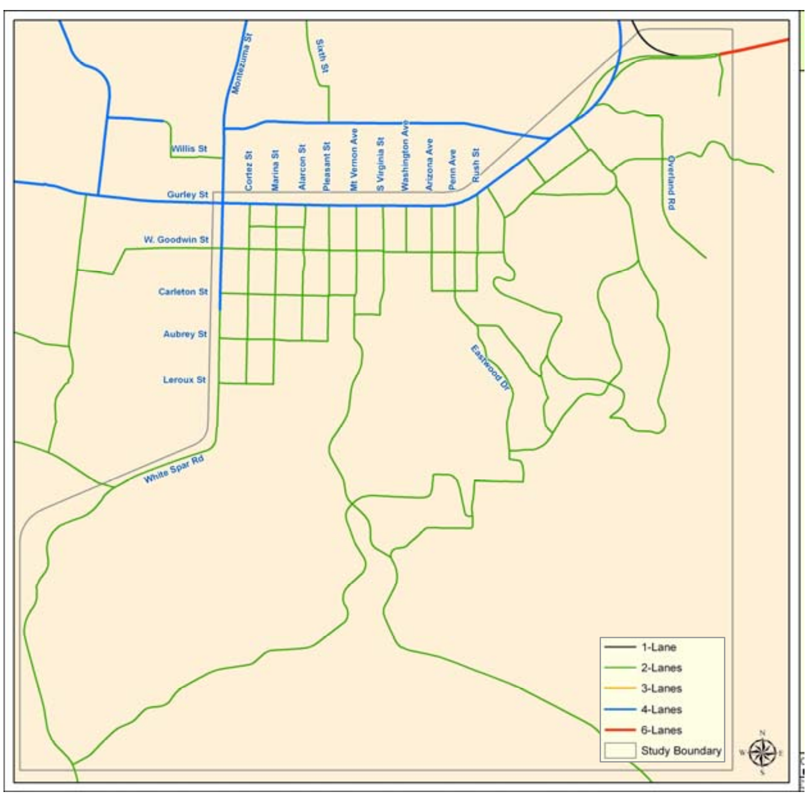
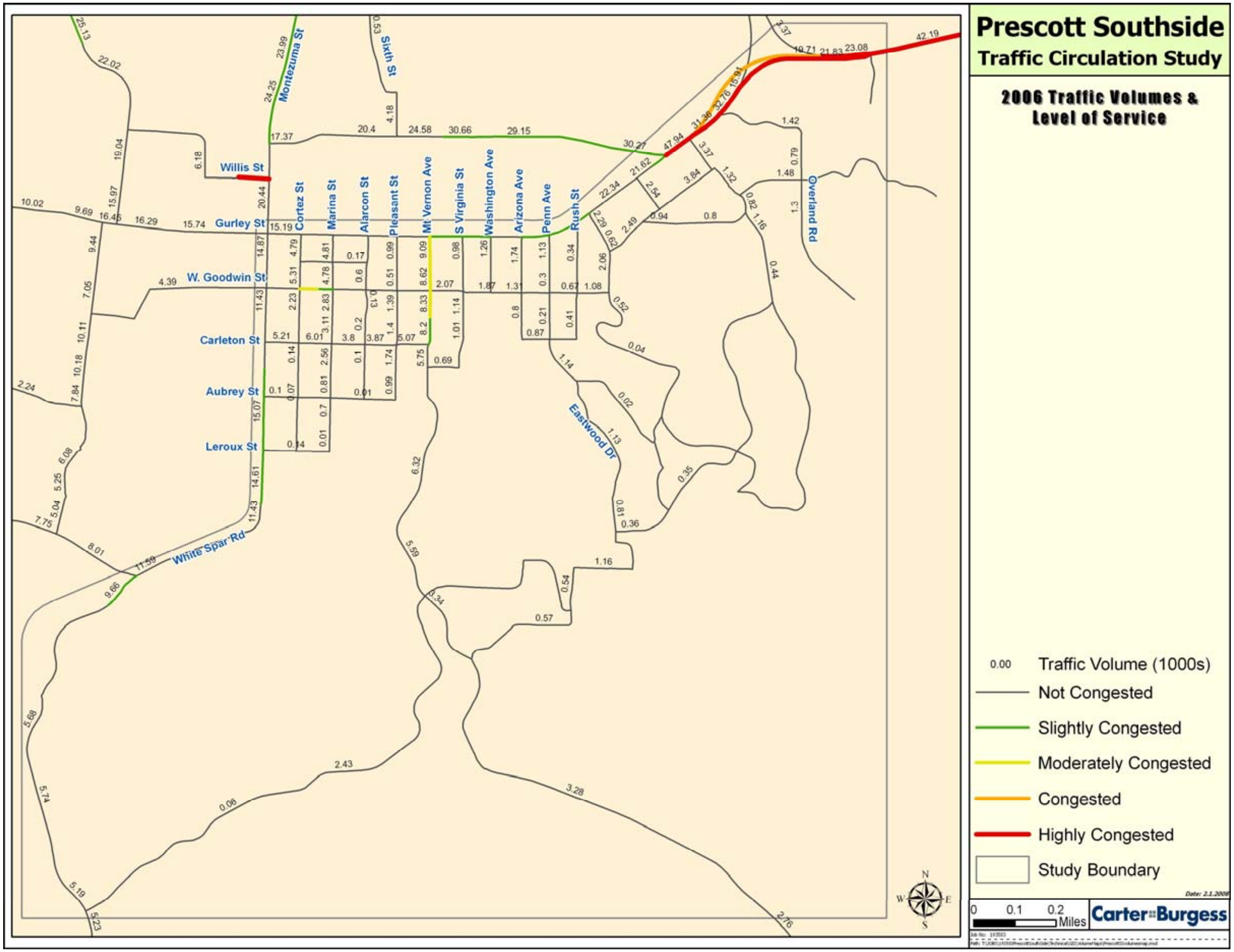


Figure 7. 2006 Daily Levels of Service and Traffic Volumes



2006 – 2007 Intersections Level of Service

The existing 2006 and 2007 conditions at specific intersections were analyzed to determine the level of service for each intersection. The ability of a transportation system to transmit the transportation demand is characterized as its level of service (LOS). Level of Service is a rating system from "A", representing the best operation to "F", representing the worst operation. Typically, level of service "D" is considered the minimum acceptable operation. Figure 8 shows the intersection locations considered in the study and Table 1 identifies the signalized intersections and Table 2, the unsignalized intersections.

Figure 8. Intersection Locations

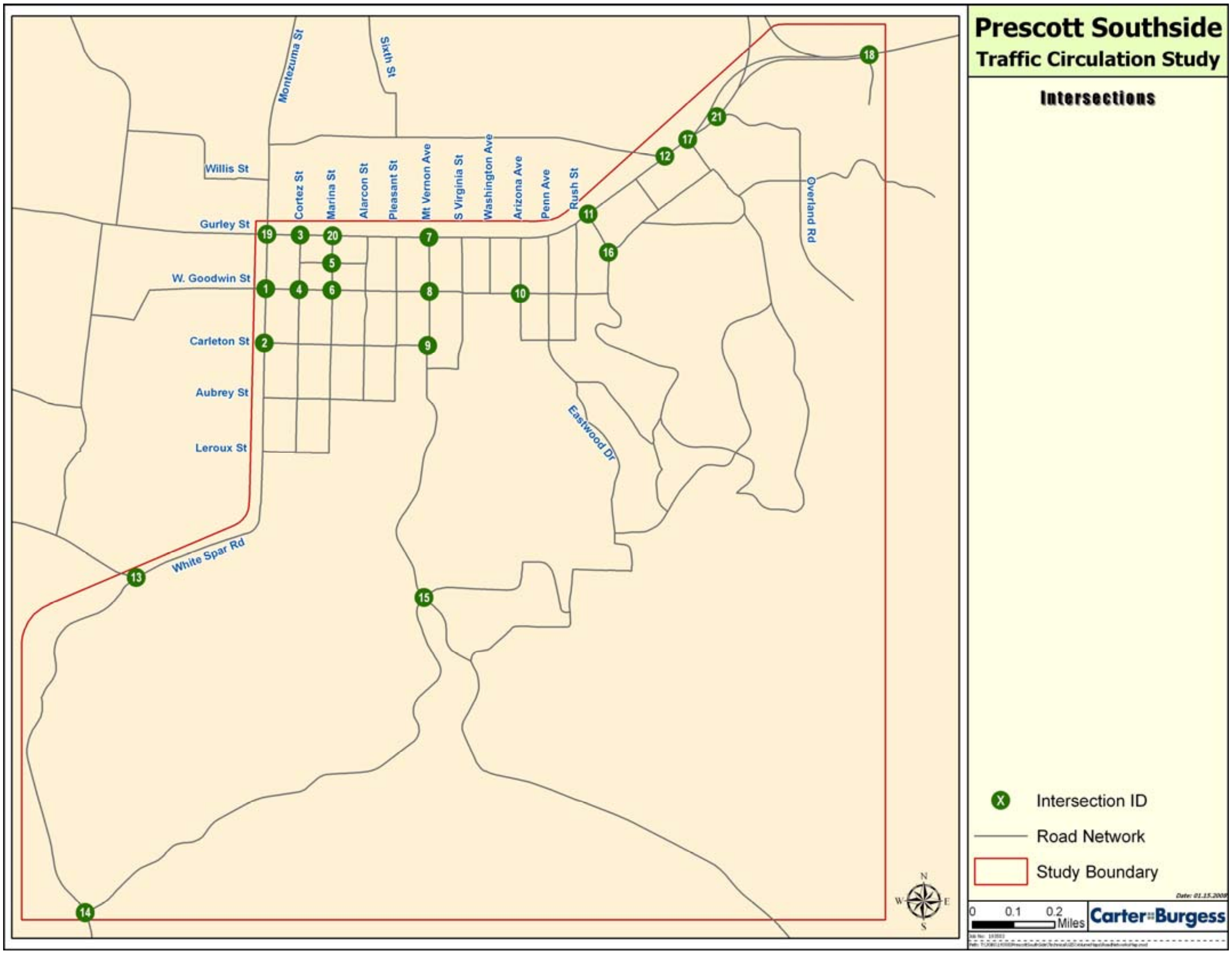


Table 1. Signalized Intersections

White Spar @ Copper Basin	Gurley @ Marina
Montezuma @ Carleton	Gurley @ Mt. Vernon
Montezuma @ Goodwin	Gurley @ Bradshaw
Montezuma @ Gurley	Sheldon @ Gurley (State)
Goodwin @ Cortez Gurley @ Cortez	SR 69 @ Heather Heights (State for bypass alternate)

Table 2. Unsignalized Intersections

White Spar @ Haisley	Butterfield @ Gurley
Haisley @ Mt. Vernon	Union @ Marina
Mt. Vernon @ Goodwin	Goodwin @ Marina
Mt. Vernon @ Carleton	Bradshaw @ Stetson
Robinson @ Gurley	

The appropriate reference for level of service operation is the Highway Capacity Manual, published by the Transportation Research Board. This manual considers the average delay per vehicle as the measure to determine the level of service of a signalized intersection. The delay and level of service are calculated for the intersection, each approach and each turning movement. For unsignalized intersections, the level of service is defined for each minor movement for two-way stop controls and is not defined for the intersection as a whole. For all-way stop controls, level of service is defined for the intersection, each approach and for each turning movement. Table 3 lists the level of service criteria for signalized and unsignalized intersections as stated in the Highway Capacity Manual.

Table 3. Level of Service Criteria for Signalized Intersections

LEVEL-OF-SERVICE	AVERAGE DELAY (seconds/vehicle)	
	SIGNALIZED	UNSIGNALIZED
A	≤ 10	≤ 10
B	> 10 to 20	> 10 to 15
C	> 20 to 35	> 15 to 25
D	> 35 to 55	> 25 to 35
E	> 55 to 80	> 35 to 50
F	> 80	> 50

To analyze the study area, Intersection Signal Timing information was obtained for each of the study intersections from the City of Prescott Transportation Services Division and is included in Appendix A. Synchro software was then used, utilizing the current turning movements at each study intersection and corresponding signal timing, to determine the delay and level of service.

The results from these analyses are provided in tables, figures and graphs located in the Appendix. Table 4 provides the level of service summary for each counted intersection for both the morning and evening peak periods. Table 5 provides the level of service for the remaining study intersections for both morning and evening peak periods. The delay and level of service for the study intersections, for each 15-minute interval in which the data was available, are provided in graphical form in the Appendix.

Table 4. Morning and Evening Peak Hour Level of Service

INTERSECTION	PEAK HOUR LEVEL OF SERVICE			
	MORNING		EVENING	
Montezuma Street & Goodwin Street	7:30 AM - 8:30 AM	B	4:15 PM - 5:15 PM	B
Montezuma Street & Carelton Street	7:30 AM - 8:30 AM	B	3:00 PM - 4:00 PM	B
Cortez Street & Gurley Street	9:00 AM - 10:00AM	B	4:15 PM - 5:15 PM	B
Mount Vernon Avenue & Gurley Street	7:30 AM - 8:30 AM	A	4:15 PM - 5:15 PM	B
Mount Vernon Avenue & Goodwin Street	7:30 AM - 8:30 AM	A	4:30 PM - 5:30 PM	A
White Spar Road & Copper Basin Road	7:45 AM - 8:45 AM	A	4:15 PM - 5:15 PM	A
Montezuma Street & Gurley Street	8:00 AM - 9:00 AM	B	4:15 PM - 5:15 PM	C
Sheldon Street & Gurley Street	7:30 AM - 8:30 AM	F	4:30 PM - 5:30 PM	F
Bradshaw Street & Gurley Street	7:30 AM - 8:30 AM	F	4:30 PM - 5:30 PM	F
Marina Street & Gurley Street	8:00 AM - 9:00 AM	B	4:30 PM - 5:30 PM	B

Table 5. Morning and Evening Peak Hour Level of Service – Estimated Hourly Volumes

INTERSECTION	PEAK HOUR LEVEL OF SERVICE	
	MORNING	EVENING
Cortez Street & Goodwin Street	B	B
Marina Street & Goodwin Steet	A	B
Arizona Avenue & Goodwin Street	A	A
White Spar Road & Haisley Road	A	B

The analyses reveal that with the exception of the intersections of Sheldon/Gurley and Bradshaw/Gurley, the study intersections are operating at acceptable levels of service. Furthermore, the levels of service at which these signalized intersections operate remain relatively constant throughout the day. However, the delay for the intersection Sheldon Street/Gurley Street appears to be much higher during the evening peak hour than during the morning peak hour.

Signal Warrant and Multi-way Stop Analysis

The Manual on Uniform Traffic Control Devices (MUTCD) as published by the United States Department of Transportation is the reference for determining the need for traffic signal installation throughout the United States. This document established eight separate, related sets of criteria termed “warrants”. If none of the eight warrants are satisfied, then a signal should not be installed. If one or more of the warrants are satisfied, then a signal might be appropriate. Table 6 shown below, provides the names of the primary signal warrants, while

Table 7 summarized the results of the analyses of the primary signal warrants for the 9 additional intersections.

Table 6. Signal Warrant Names

WARRANT	NAME
1A	Minimum Vehicular Volume
1B	Interruption of Continuous Traffic
1A and 1B	Combination of Warrants
2	Four-Hour Vehicular Volume
3B	Peak Hour Volume

Table 7. Existing Signal Warrant Analyses Summary

	ACTUAL NUMBER OF HOURS MET					
WARRANT	**1A**	**1B**	**1A & 1B**	**2**	**3B**	**WARRANT SATISFIED?**
REQUIRED HOURS MET	8	8	8	4	1	
Marina & Union	0	0	0	0	0	NO
Marina & Goodwin	1	0	0	0	0	NO
Mount Vernon & Goodwin	4	0	2	0	0	NO
Mount Vernon & Carleton	1	0	0	0	0	NO
Arizona & Goodwin	0	0	0	0	0	NO
White Spar & Haisley	0	0	0	0	0	NO
Mount Vernon & Haisley	0	0	0	0	0	NO
Bradshaw & Stetson	0	0	0	0	0	NO
Robinson & Gurley	0	0	0	0	0	NO

These analyses indicate that none of the intersections satisfy the warrants for signal installation.

The Manual of Uniform Traffic Control Devices (MUTCD) also established four separate, related sets of criteria to assist the determination of the need for stop signs on each approach to an intersection.

The first multi-way stop warrant is Warrant A, which indicates that a multi-way stop may be temporarily appropriate if a traffic signal is warranted, until it is installed. Warrant B suggests the installation of stop signs for each approach to an intersection if the intersection has been the site of five (5) or more collisions of a type potentially preventable by multi-way stop signs in a twelve-month period. Beyond the 10 signalized whose operations were analyzed, 9 additional unsignalized intersections were considered to determine if signalization or multi-way stop control is warranted.

Because none of the 9 intersections satisfied the signal warrants, each intersection was analyzed to see if the multi-way stop control warrants were satisfied. Warrant A was not satisfied for any of the intersections.

Collision data for the intersections was not available, so Warrant B was not considered in determining if multi-way stop control was appropriate. Warrants C and D did not consider the vehicle delay portion of the warrants since approach and departure counts and not turning movements counts were obtained for these 9 locations. The 8 hour average volumes were considered for Warrants C and D, and the results are summarized below in Table 8. These analyses indicate that the intersection of Marina/Goodwin and Mt. Vernon/Goodwin satisfy the traffic volume portion of the multi-way stop warrants.

Table 8. Multi-way Stop Warrant Analyses Summary

	ACTUAL NUMBER OF HOURS MET						
WARRANT	**C: 8-Hour Volumes**		**C - Delay**	**D: 8-Hour Volumes**		**D - Delay**	**WARRANT SATISFIED?**
	Major	Minor	(sec/veh)	Major	Minor	(sec/veh)	
CRITERIA	300	200	30	240	160	24	
Marina & Union	314	97	Not Measured	314	97	Not Measured	NO
Marina & Goodwin	464	251	Not Measured	464	251	Not Measured	YES
Mount Vernon & Goodwin	578	244	Not Measured	578	244	Not Measured	YES
Mount Vernon & Carleton	464	154	Not Measured	464	154	Not Measured	NO
Arizona & Goodwin	143	26	Not Measured	143	26	Not Measured	NO
White Spar & Haisley	301	103	Not Measured	301	103	Not Measured	NO
Mount Vernon & Haisley	264	83	Not Measured	264	83	Not Measured	NO
Bradshaw & Stetson	175	99	Not Measured	175	99	Not Measured	NO
Robinson & Gurley	4,076	61	Not Measured	4,076	61	Not Measured	NO

III. FUTURE CONDITIONS

Future Land Use Conditions

The year 2030 was chosen as the future horizon year to assess the travel demand needs in the study area. Additionally, the CYMPO model used in the current condition evaluation was applied in this task. Figure 9 depicts the anticipated population densities in the study area in 2006 and 2030, while Table 9 summarizes the population and employment growth in the study area. The socioeconomic data presented in future projections is reflective of jurisdictional general land use plans currently adopted by the individual agencies. The population and employment projections were reviewed and adopted by the jurisdictional agencies within the study area. Although a specific year was chosen, the growth could be experienced before or after the chosen horizon year depending on the economic conditions of the area. Hence a periodic update of the socioeconomic conditions and the projected future travel conditions should be conducted to help in guiding the implementation timeframe of any long term recommended improvements.

Table 9. Area Growth Summary

Horizon Year	2006	2030
Population	7,272	24,641
Dwelling Units	3,230	10,240
Employment	4,293	5,186

As Figure 9 shows, the area growth is quite substantial and contains a large increase in the County portion of the area of influence. Yavapai County, after reviewing the available land under their jurisdiction, noted that potentially the projections are reflective of the built conditions for the area. For this study, surrounding area growth was included because it will have a direct impact on the future mobility of Prescott South Side.

Future Transportation Conditions

To assess the future transportation needs, the CYMPO travel demand model was revised to include the new TAZ structure for the area and the 2030 adopted regional transportation plan. Figure 10 depicts the 2030 roadway system forecasted traffic volumes and corresponding levels of service at the segment level for the study area. As can be noticed, the roadways leading into Prescott from the south are projected to be highly congested in the year 2030 and the downtown area exhibits various levels of congestion primarily from moderately congested to highly congested.

Figure 9. Population Densities

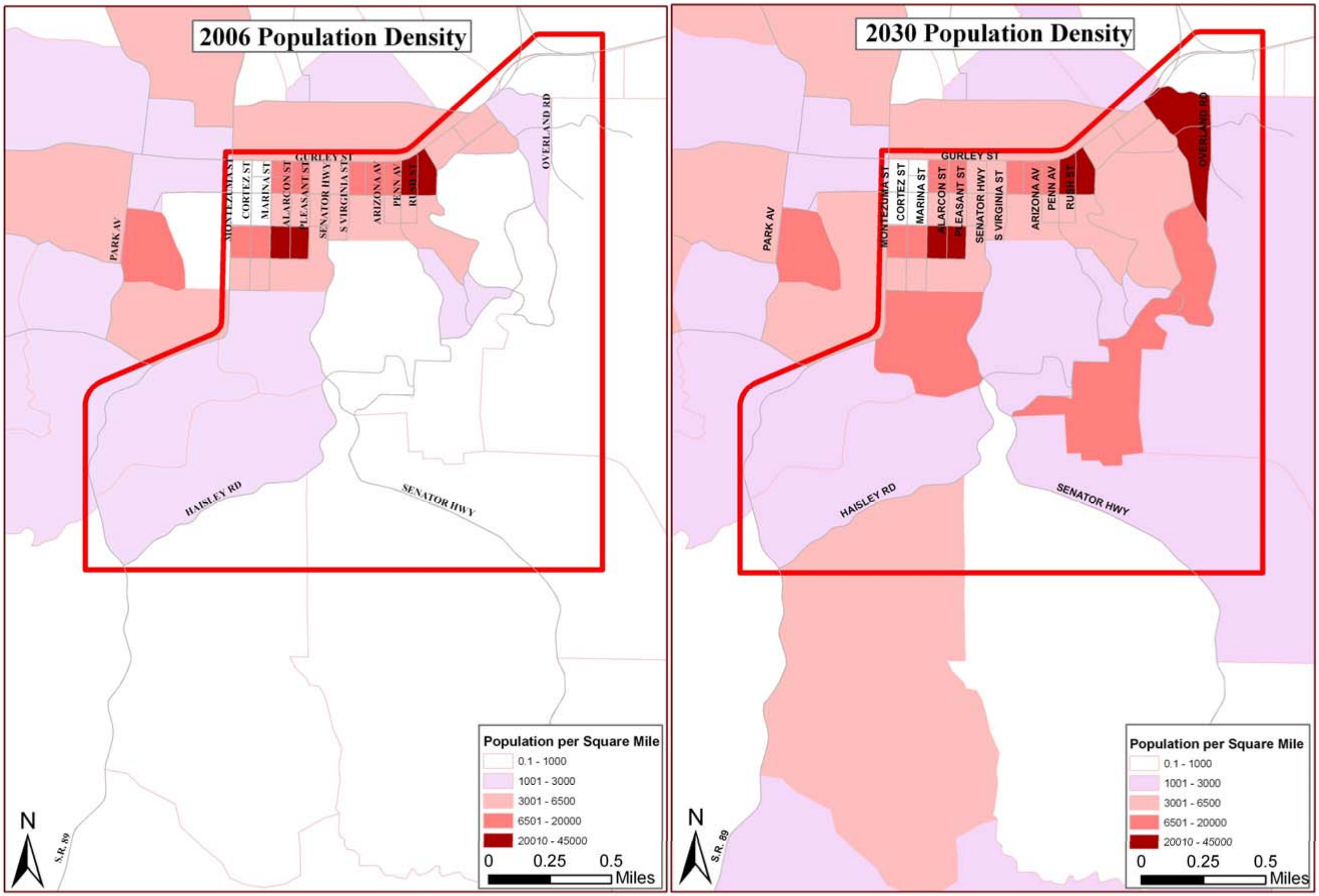
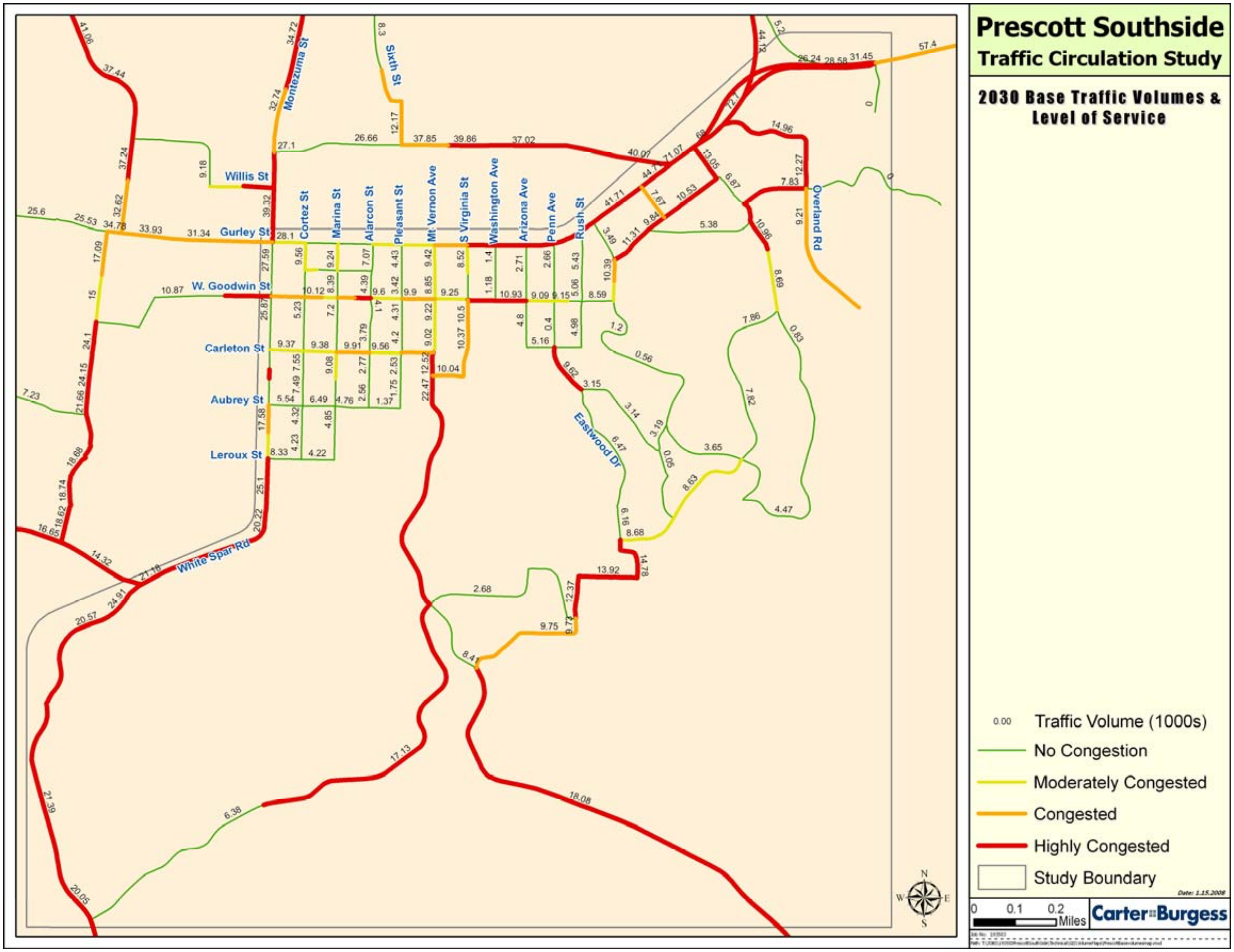


Figure 10. 2030 Base Traffic Conditions



Future Intersection Conditions - 2030 Level of Service

The anticipated 2030 traffic volumes at the study intersections were analyzed to determine the level of service for each intersection. The criteria used in the 2007 intersection level of service analysis were applied for the 2030 conditions.

Synchro software was used with the predicted 2030 peak hour turning movements to determine the delay and level of service at each study intersection. The intersections were first analyzed with the existing lane configurations and traffic control, and then with the recommended improvements. Detailed results for the unimproved intersection analyses are presented in Appendix B.

Synchro software provided calculated delays exceeding 120 seconds per vehicle. The equations for these delay calculations are accurate only for delays less than 60 seconds. The equations provide reasonably accurate results for delays between 60 and 120 seconds. Calculated delay greater than 120 seconds are much exaggerated. Therefore, all calculated delays greater than 120 seconds per vehicle were reduced to 120 seconds per vehicle. This adjustment becomes particularly meaningful when a specified movement is calculated to experience very high delay and thereby greatly exaggerates the corresponding approach delay and intersection delay. Tables of these adjustments are provided in the Appendix following the Synchro output reports.

The peak hour factor is the ratio of total traffic occurring during the peak hour to the peak 15-minute flow rate (4 times the maximum 15 minute volume) within the peak hour. A peak hour factor of 0.92 to 0.95 indicates a relatively high degree of uniformity of flow during the peak traffic hour. The peak hour factors for each movement and each approach were calculated from the existing 2007 counts obtained for this report and where appropriate were used at the corresponding intersection as a required input for the 2030 Synchro analysis. However, at low volumes the peak hour factor can become exaggerated, and is no longer appropriate for use in predicting future traffic patterns. When unacceptably low peak hour factors were calculated from existing counts, a default peak hour factor of 0.92 was utilized for the 2030 analyses to more realistically predict the behavior of future traffic. The inputs used for the peak hour factors are available in the Synchro output reports that are included in the Appendix of this report.

Intersections Improvements

The recommended lane configurations and traffic control for the study intersections are summarized in Table 10 and shown in Figure 11 through Figure 15. These recommendations produce instances where the 2030 levels of service are below “D”. Table 10 contains the list of improvements identified by the intersection ID number displayed in the figures. Right of way and constructability limitations eliminate potential improvements that would provide level of service “D” or better.

The lane configurations presented in Table 10 are only feasible if the necessary roadway widening improvements are in place to accommodate them. Currently, the widening of Montezuma, Goodwin, and Carleton are not planned. Hence the implementation of these suggested improvements is subject to potential future roadway widening.

Table 10. 2030 Intersection Improvements

ID	Location	Improvements
1	Montezuma St & Goodwin St	Exclusive left turn lane, thru lane, and shared thru-right turn lane at both NB & SB approaches.
2	Montezuma St & Carleton St	Exclusive left turn lane, thru lane, and shared thru-right turn lane at both NB & SB approaches.
5	Marina St & Union St	Signalize intersection.
6	Marina St & Goodwin St	Exclusive left turn lane and shared thru-right turn lane at all approaches.
7	Mt. Vernon Ave & Gurley St	Exclusive left turn lane, dual thru lanes, and exclusive right turn lane at the EB & SB approaches.
8	Mt. Vernon Ave & Goodwin St	Signalize intersection with exclusive left turn lane and shared thru-right turn lane at all approaches.
9	Mt. Vernon Ave & Carleton St	Stop control intersection with exclusive left turn lane and shared thru-right turn lane at both NB & SB approaches, and separate left turn and right turn lane at the EB approach.
12	Gurley St & Sheldon St	Dual left turn lanes, dual thru lanes, and a shared thru-right turn lane at the EB approach, and exclusive left turn lane, three thru lanes, and exclusive right turn lane.
13	White Spar Rd & Copper Basin Rd	Exclusive left turn lane, thru-lane, and shared thru-right turn lane at the SB approach. Dual left turn lanes, thru lane, and shared thru-right turn lane at the NB approach. The EB approach includes dual left turn lanes and a shared thru-right turn lane while the WB approach includes an exclusive left turn lane and shared thru-right turn lane.
14	White Spar Rd & Haisley Rd	Exclusive left turn lane and thru lane at SB approach.
15	Senator Highway & Haisley Rd	Signalize intersection with exclusive left turn lane and shared thru-right turn lane at all approaches.
19	Montezuma St & Gurley St	Exclusive left turn lane, dual thru lanes, and exclusive right turn lane at the NB & SB approach. The EB approach includes exclusive left-turn lane, thru lane, and shared thru-right turn lane while the WB approach includes an exclusive left turn lane, dual thru lanes, and exclusive right turn lane.

Figure 11. 2030 LOS with Improvements – AM and PM Peak Hours (1 of 5)

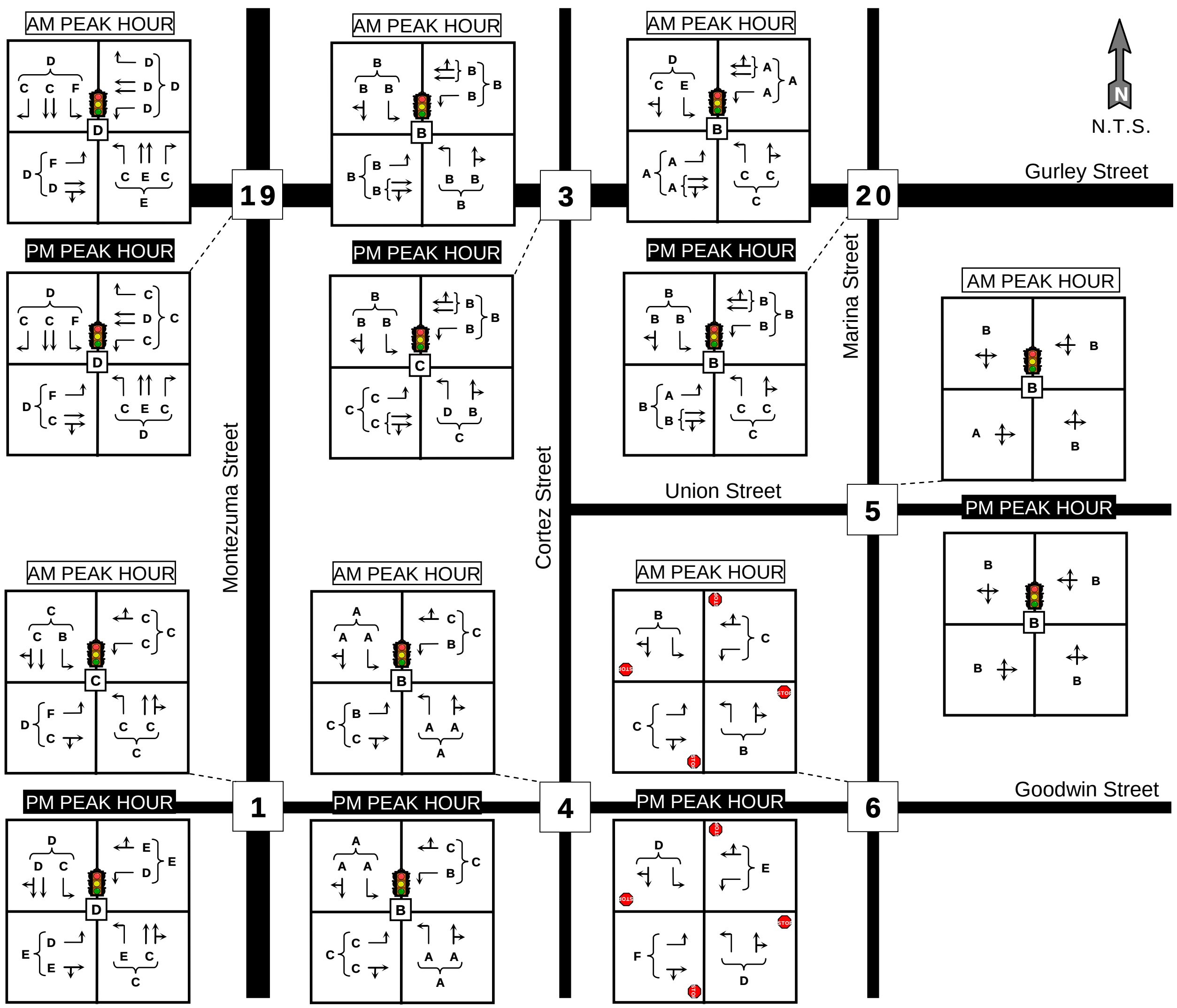


Figure 12. 2030 LOS with Improvements – AM and PM Peak Hours (2 of 5)

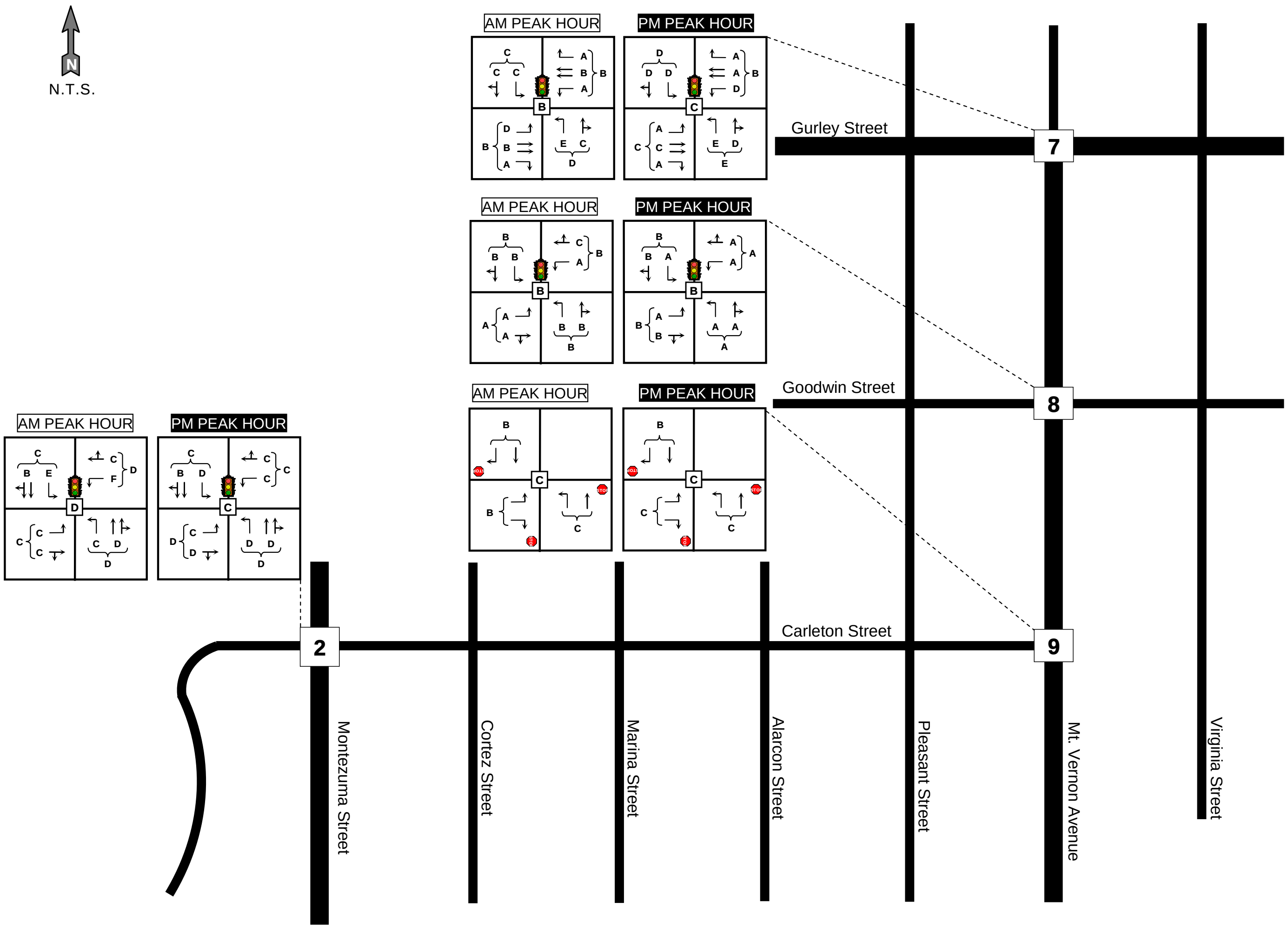


Figure 13. 2030 LOS with Improvements – AM and PM Peak Hours (3 of 5)

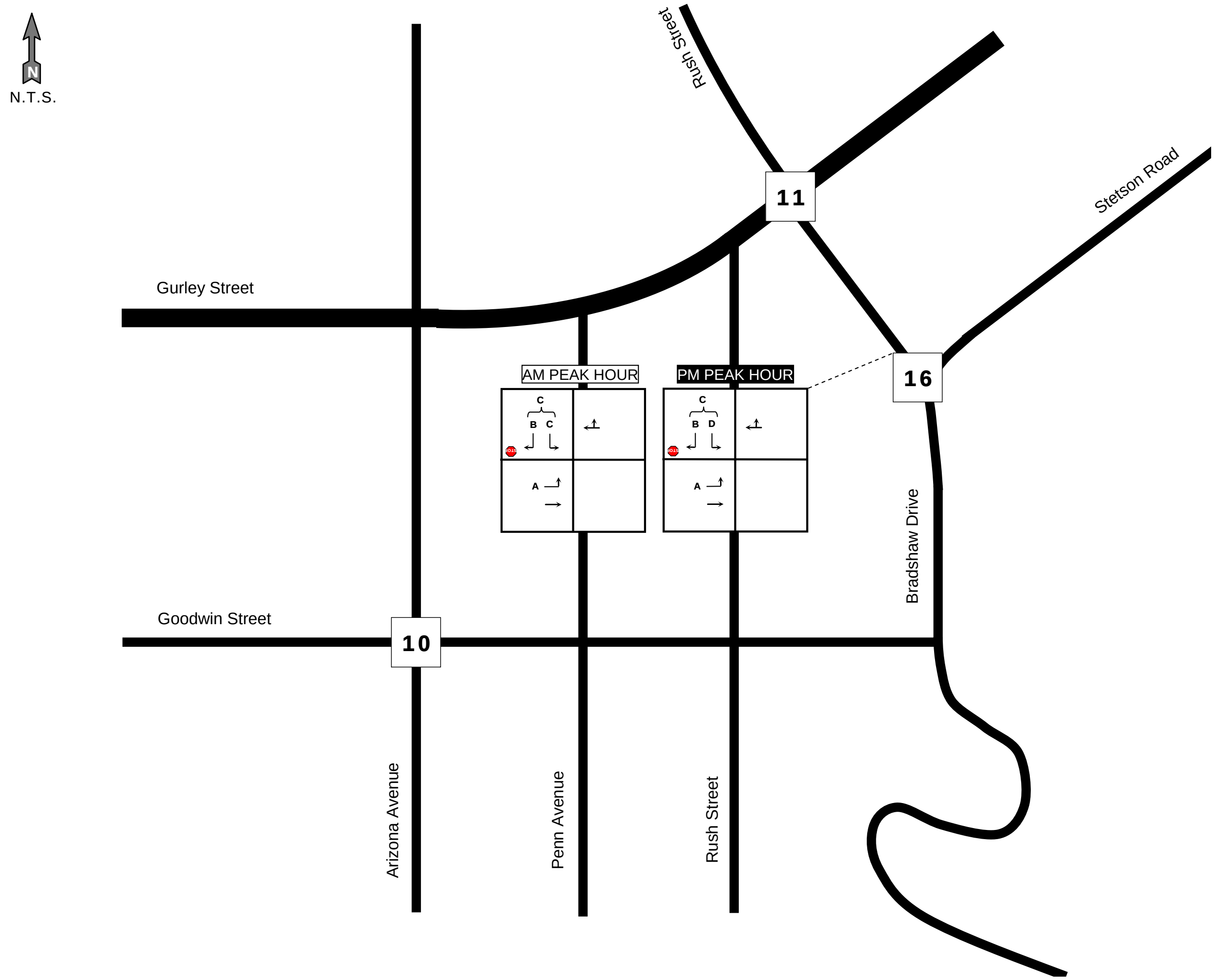


Figure 14. 2030 LOS With Improvements – AM and PM Peak Hours (4 of 5)

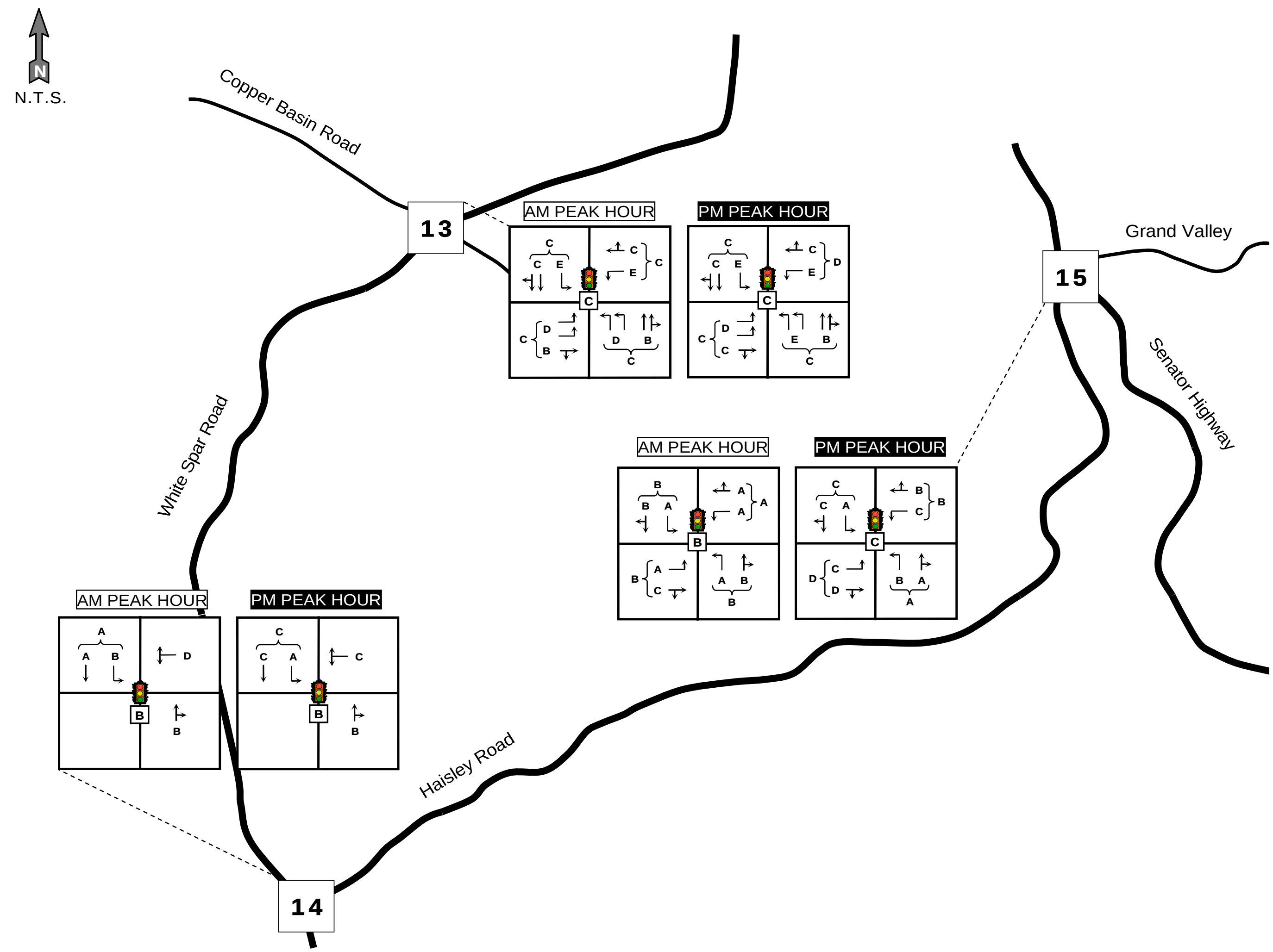
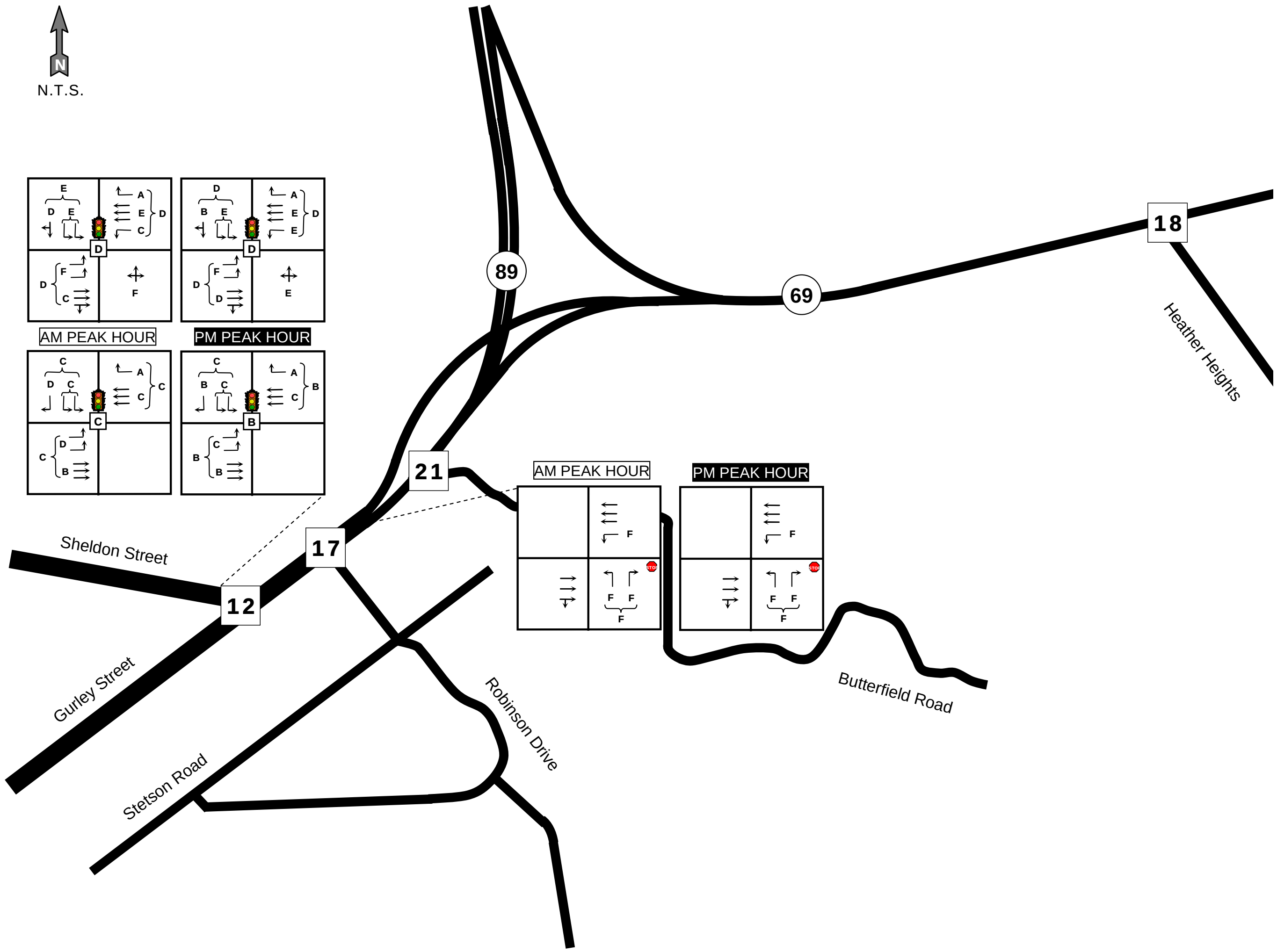


Figure 15. 2030 LOS with Improvements – AM and PM Peak Hours (5 of 5)



The daily traffic volumes for the year 2030, as shown in the planning model, indicate anticipated left-turn volumes at the intersection of Robinson Drive and Gurley Street (Intersection 17). This includes the southwest bound left-turn from Gurley Street onto Robinson Drive, and the northwest bound left-turn from Robinson Drive onto Gurley Street. Accordingly, these volumes were considered in generating morning and evening peak hour volumes and the intersection was analyzed with these left-turn movements.

As indicated in the level of service figures, both left-turns operate at "F" in both the morning and evening peak hours. In addition to high delay, permitting these left turns also magnifies several other problems. First inclusion of the left-turns decreases the capacity of the through movements on Gurley Street and increases the likelihood of vehicle collisions on Gurley Street. Furthermore, due to the proximity of the intersection of Robinson Drive and Gurley Street to the merging of SR89 and SR69, weaving issues will be compounded by permitting left-turns at Robinson Drive. For these reasons, it is recommended that both left-turn movements be prohibited at this intersection.

Intersection 16, the intersection of Bradshaw Drive and Stetson Road, was modified from its existing geometry for both the 2030 and the 2030 with improvements level of service analyses. The traffic model predicts the heaviest volumes on the current southern leg of the intersection (on Bradshaw Drive) and on the east leg of the intersection (Stetson Road). These traffic patterns suggest constructing the intersection so that the principal through street would be a connection of south Bradshaw Drive and east Stetson Road. North Bradshaw Drive would then be a minor roadway which would intersect the new Bradshaw-Stetson alignment and be stop controlled. This proposed configuration was used for the 2030 and the 2030 with improvements scenarios because the existing roadway configuration did not lend itself to an operations analysis with the predicted 2030 volumes.

Intersection 12, the intersection of Sheldon Avenue and Gurley Street, was considered with two possible recommendations: including the southern leg of the intersection and omitting the southern leg of the intersection. Figure 15 contains the resulting levels of service for both scenarios. Elimination of the southern leg is recommended because the intersection level of service improves significantly from D to C in the morning peak and D to B in the evening peak hour.

IV. ROADWAY IMPROVEMENTS AND MODIFICATIONS

Future Alternatives

After reviewing the future travel demand in the study area, several improvement alternatives were developed to address the study area congestion. Potential new existing roadway configurations as well as not physically constrained alternatives were considered. The purpose of testing these alternatives was to identify and quantify the effects of the various improvements on the future travel demand patterns.

Special consideration was given to traffic calming devices that could be utilized to reduce travel speed along Mt. Vernon Avenue. As the OD surveys have shown, the facility is a preferred alternate route to Gurley Street. An additional observation is the limited availability of alternate roadways trucks could use to access Senator Highway. Hence, when considering the diverse types of travel Mt. Vernon is currently accommodating, the traffic circle was the device identified as the appropriate one to accomplish the objective of speed reduction. Also, a traffic management strategy was included in conjunction with the traffic calming device in order to provide a better redistribution of traffic in the study area. The strategy consists of prohibiting the left-turn movements from Carleton Street and Goodwin Street onto Mt. Vernon Avenue. After review of many potential ideas, the following alternative emerged and were coded into the 2030 CYMPO travel demand model and tested for performance.

Alternative 1 Connection of White Spar Road and/or Senator Highway with a bypass extending to Robinson Drive, then run parallel to SR69 below the Yavapai-Prescott Indian Tribe Reservation jurisdictional boundaries and connect to SR69 in the vicinity of Holiday Drive. Limited access 4 lane arterial (could also consider parkway/expressway) with at grade intersection at White Spar Road, Senator Highway, Bradshaw Drive Robinson Drive, Butterfield Road and SR69.

Alternative 2 Mt. Vernon Avenue and Virginia Street as a one-way pair from Gurley Street to Aubrey Street.

Alternative 3 Remove access from Goodwin Street and Carleton Street to Mt. Vernon Avenue.

Alternative 4 Provide a parallel route to Mt. Vernon Avenue as the south bound continuation of Virginia Street around Acker Park to Haisley Road.

Alternative 5 Extend Aubrey Street to Virginia Street with prohibited left turns from Carleton Street and Goodwin Street to Mt. Vernon Avenue. Traffic circles at the intersection of Carleton Street and Goodwin Street at Mt. Vernon Avenue.

These alternatives include comments and suggestions presented by the public during the two public meetings conducted during the project.

Figures 16 through 20 depict the resulting 2030 traffic volumes and LOS produced by each alternative.

Figure 16. Alternative 1

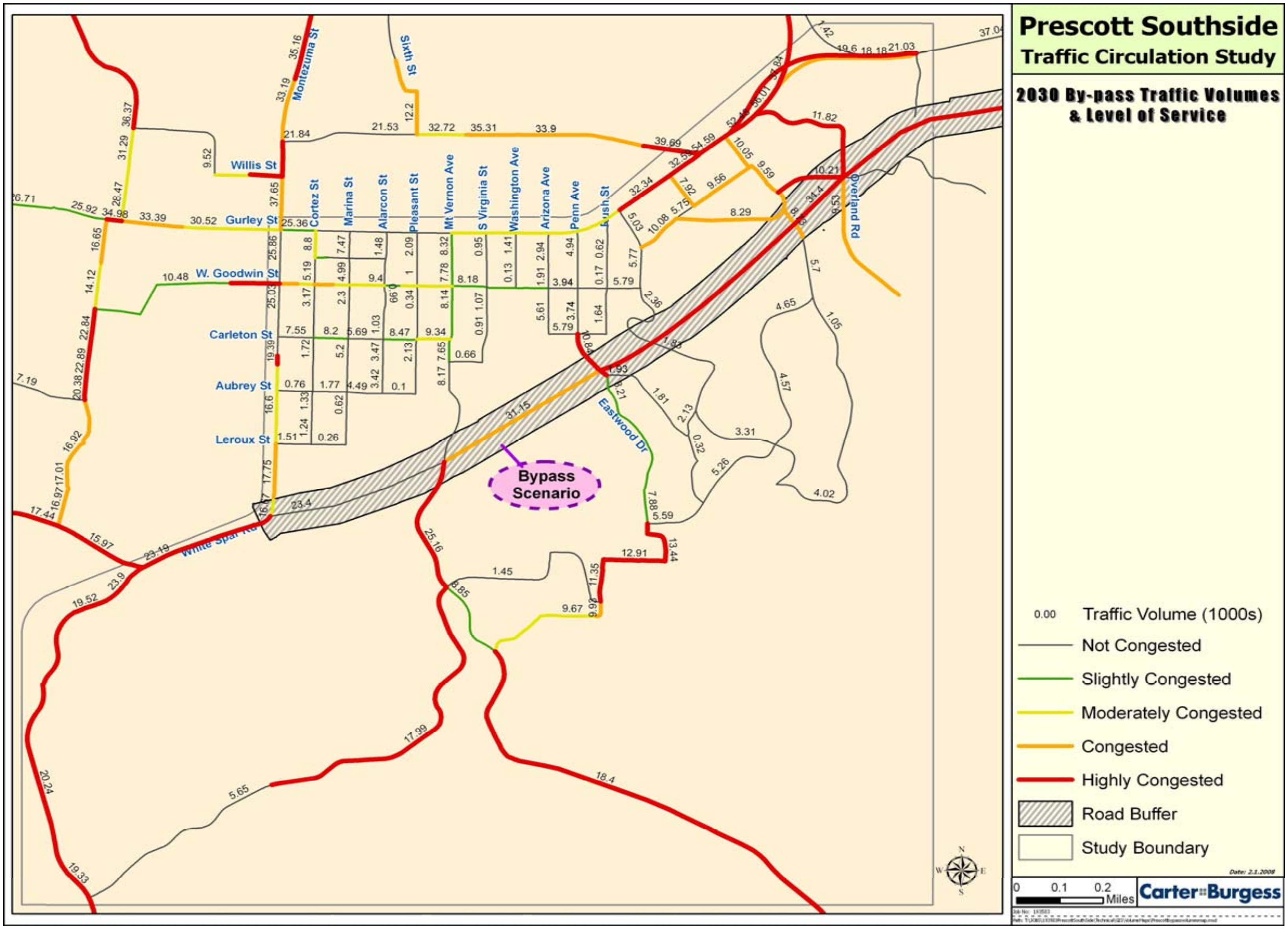


Figure 17. Alternative 2

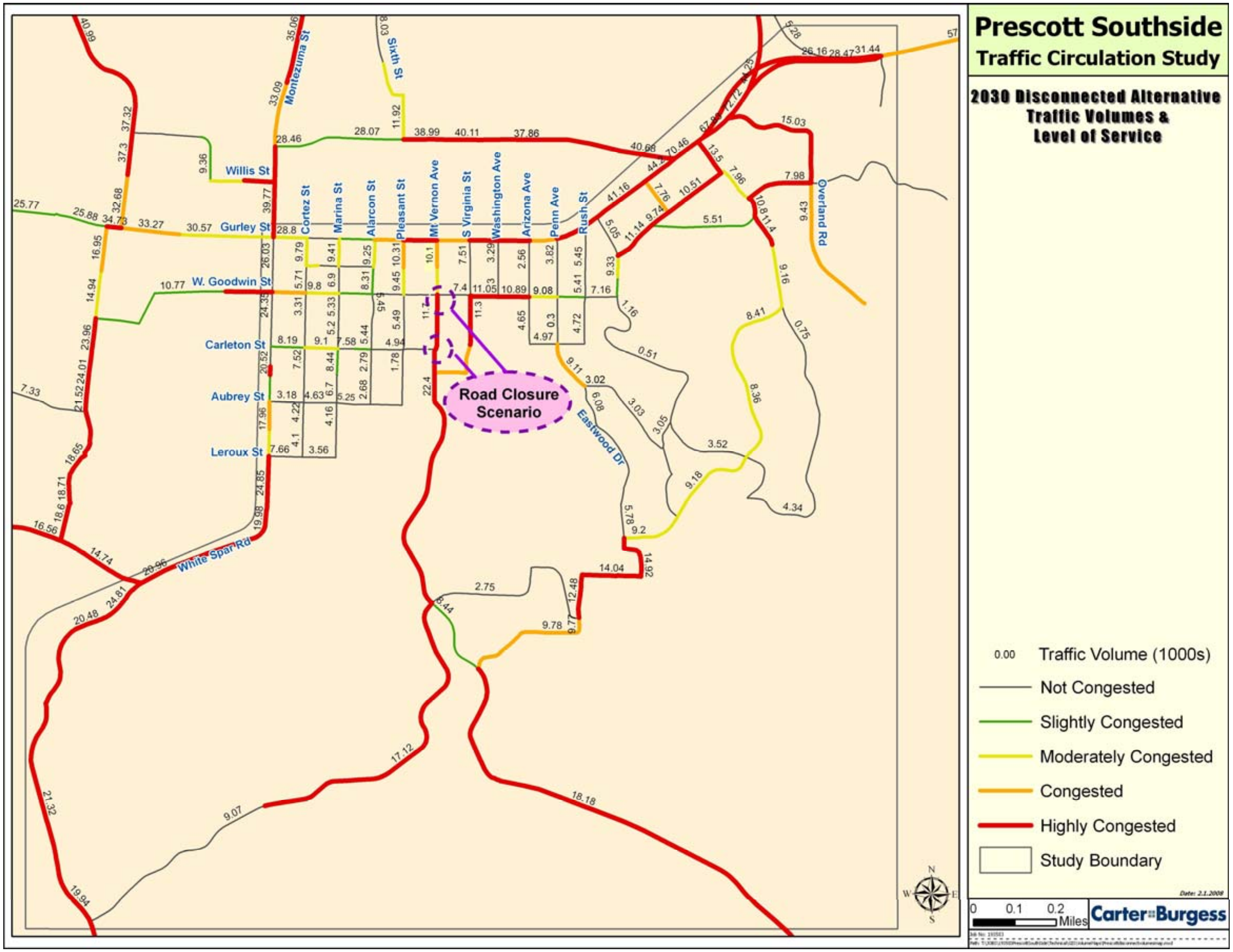


Figure 18. Alternative 3

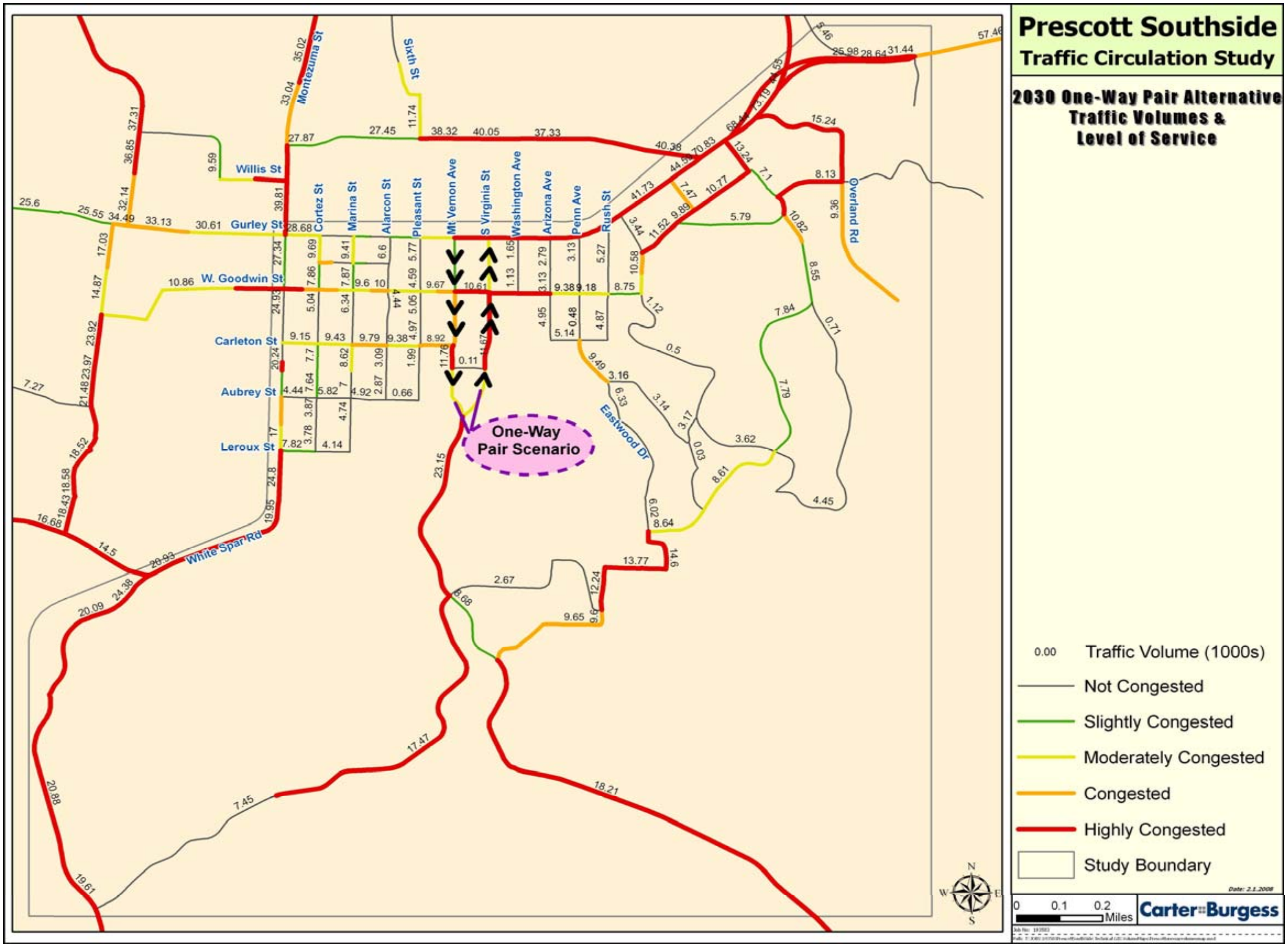


Figure 19. Alternative 4

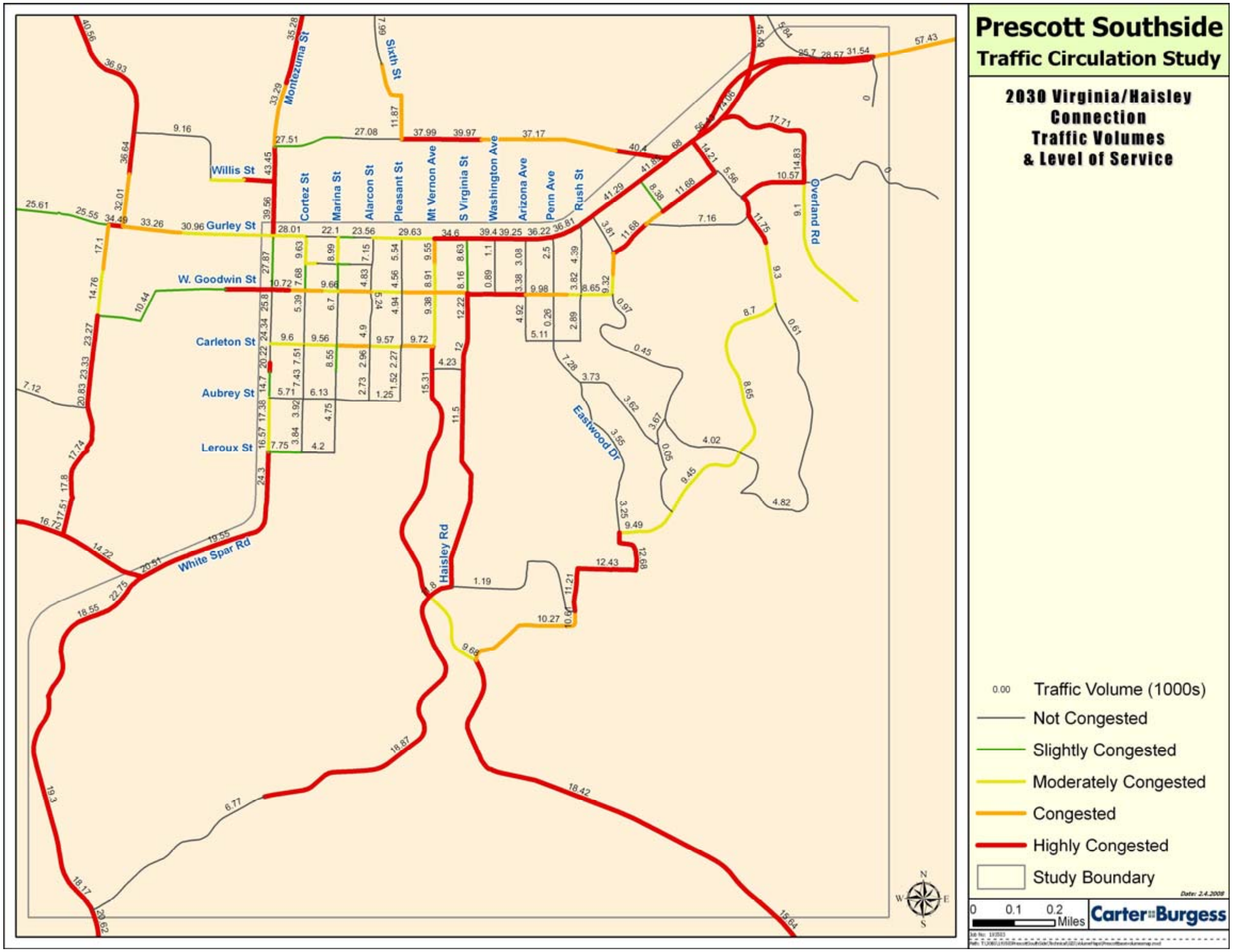
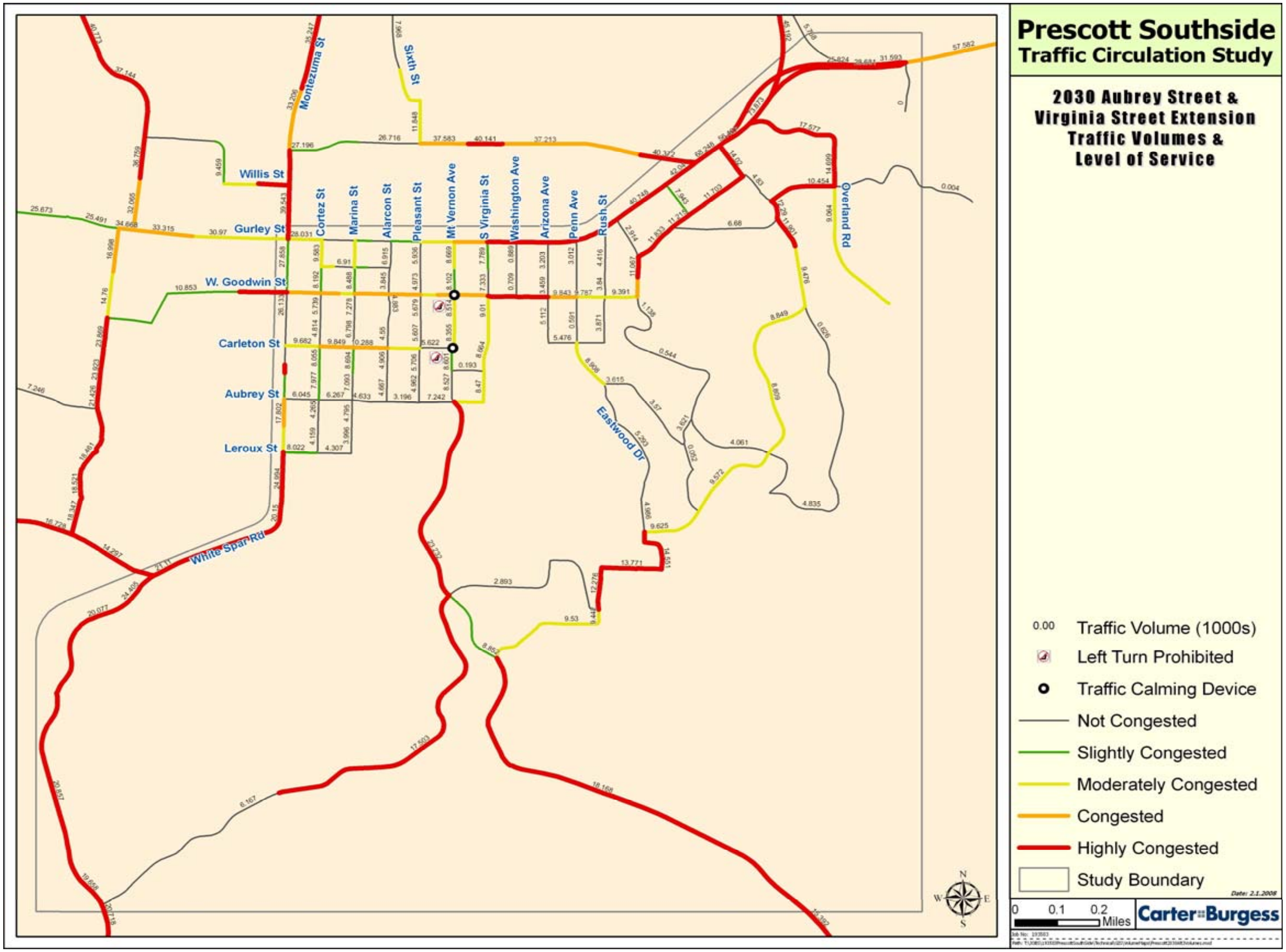


Figure 20. Alternative 5



Conclusions

The alternative analysis findings based on traffic flow characteristics can be summarized as follows:

Alternative 1 is the only alternative that could provide relief to the traffic congestion in the Prescott South Side area for long term.

Alternative 2, 3 and 4 will only redistribute the traffic between Mt. Vernon Avenue and Virginia Street without alleviating the overall congestion in the study area.

Alternative 5 brings a more balanced distribution of traffic among all north/south parallel facilities, but will not alleviate the congestion in the study area for the long term.

All alternatives do not relieve congestion on Senator Highway or White Spar Road south of Leroux Street.

Consideration was then given to potential fiscal and physical constraints. The alternatives were evaluated for right of way requirements, natural barriers, and cost of improvements.

Table 11 lists the assumption of construction cost for specific improvements items. Since final design plans for the alternatives have not been developed, the preliminary cost estimate will present an approximation of the potential construction cost of the improvement, not including right of way and earthwork expenditures. It must be noted that earthwork costs and right of way costs could increase the cost of an improvement tremendously. The unit costs are based on 2007 dollars and are not adjusted for inflation. Table 12 summarizes the preliminary physical constraints and construction cost by alternative.

Table 11. Facility Improvement Construction Unit Costs

Item	Unit	Cost
Arterial Roadway	Lane mile (construction)	$ 1,300,000.00
Local Roadway	Lane mile (construction)	$ 7000,000.00
Bridge	Square Foot	$ 250.00
Traffic Signal (4 legs)	Traffic Signal and Installation	$ 275,000.00
Traffic Circle	15-20 ft diameter (construction)	$ 50,000.00
Right-of-Way	Vacant Land – One Acre	$ 325,000.00

Table 12. Physical and Fiscal Evaluation

Scenarios	Constraints	ROW Acquisition Required From Multiple Property Owners Adjacent to Acker Park	Preliminary Construction Cost *	Alleviates Congestion
Alternative 1	Potentially many of physical and environmental nature. Unable to determine at this time due to the unknown final alignment of the facility	Unable to determine at this time	Facility Length = 3 miles Facility = 4 lanes arterial Signalized Intersections = 6 Cost = $17,250,000.00	Yes, north of Leroux Street
Alternative 2	N/A	N/A	N/A	No. Increased congestion on Mt Vernon and Virginia
Alternative 3	Acker Park and terrain in the area of the potential Virginia Road extension	No sufficient ROW near Acker Park to construct a new facility as a continuation of Virginia Street	Facility Length = 1000 Ft Facility = 2 lanes local street Signalized Intersections = 1 Cost = $ 550,000.00	No. Congestion on Mt Vernon and Virginia
Alternative 4	Acker Park and topography in the area of the potential Virginia Road extension and the IOOF Cemetery	No sufficient ROW near Acker Park to construct a new facility as a continuation of Virginia Street	Facility Length = 1 mile Facility = 2 lanes local street Cost = $ 1,400,000.00	No. Congestion on Mt Vernon and Virginia
Alternative 5	Aubrey extension east to Mt. Vernon is unfeasible due to topography	Potential needs of additional ROW at the intersections for the traffic calming device. Lack of ROW through Acker park to make the east and west connection from Virginia Street to Mt. Vernon	Facility Length = 1000 Ft Facility = 2 lanes local street Traffic Calming Devices =2 Cost = $ 300,000.00	Some. Better redistribution of traffic along all north – south facilities in the study area

Recommended Improvements

This particular study was conducted for the future horizon year 2030. However, it is important to mention that sometimes due to unforeseen economic, natural, or political reasons the growth anticipated in this study could be reached earlier or later than 2030. Hence the recommendations presented are based on those assumptions, and the area growth should be monitored for any deviations in order to expedite or retard the implementation of the improvements.

As presented above, Alternative 1 is the only strategy that could address the future travel demand in the study area. Due to environmental and terrain constraints the potential facility will not resemble the schematics show on Alternative 1. For the purpose of this study, the main focus was to determine what transportation improvements will be needed in the study area in order to address future travel demand. This study recommends that further investigation be conducted to determine the feasibility of such a facility to address future area growth and potential congestion.

To address the short term improvements, a combination of traffic calming strategies were formulated and tested to ascertain their performance. The best results were obtained by eliminating left turn from Carleton Street and Goodwin Street onto Mt Vernon Avenue. Additionally two traffic circles, located at the intersection of Goodwin Street and Carleton Street with Mount Vernon Avenue would help reduce the traveling speed on Mt Vernon Avenue.

Figure 21 presents a schematic of the recommended improvements along Mt Vernon for the short term scenario. The traffic calming improvements were tested on the 2006 scenario and the 2030 and are displayed in Figures 22 and 23 respectively.

As can be seen the strategy applied to the 2006 condition diverted the traffic onto Virginia Street and Pleasant Street in moderate amounts, but enough to improve the level of service on Mt Vernon Avenue. It must be mentioned that trucks needing to travel to Senator Highway could still use Mt Vernon Avenue, but they will have to reduce the travel speed when approaching the traffic circles. Depending on the area growth rate, this strategy should help alleviate the traffic on Mt Vernon Avenue for the next decade.

For the 2030 scenario, the traffic calming strategy helped improve the level of service on Mt Vernon Avenue and Virginia Street for a portion of those facilities. However, congestion reprieve is only local and does not address the overall future mobility of the study area.

Based on the above results, this study recommends exploring the feasibility of implementing the set of traffic calming strategies presented in Figure 21.

Figure 21. Recommended Short Term Traffic Calming Improvements

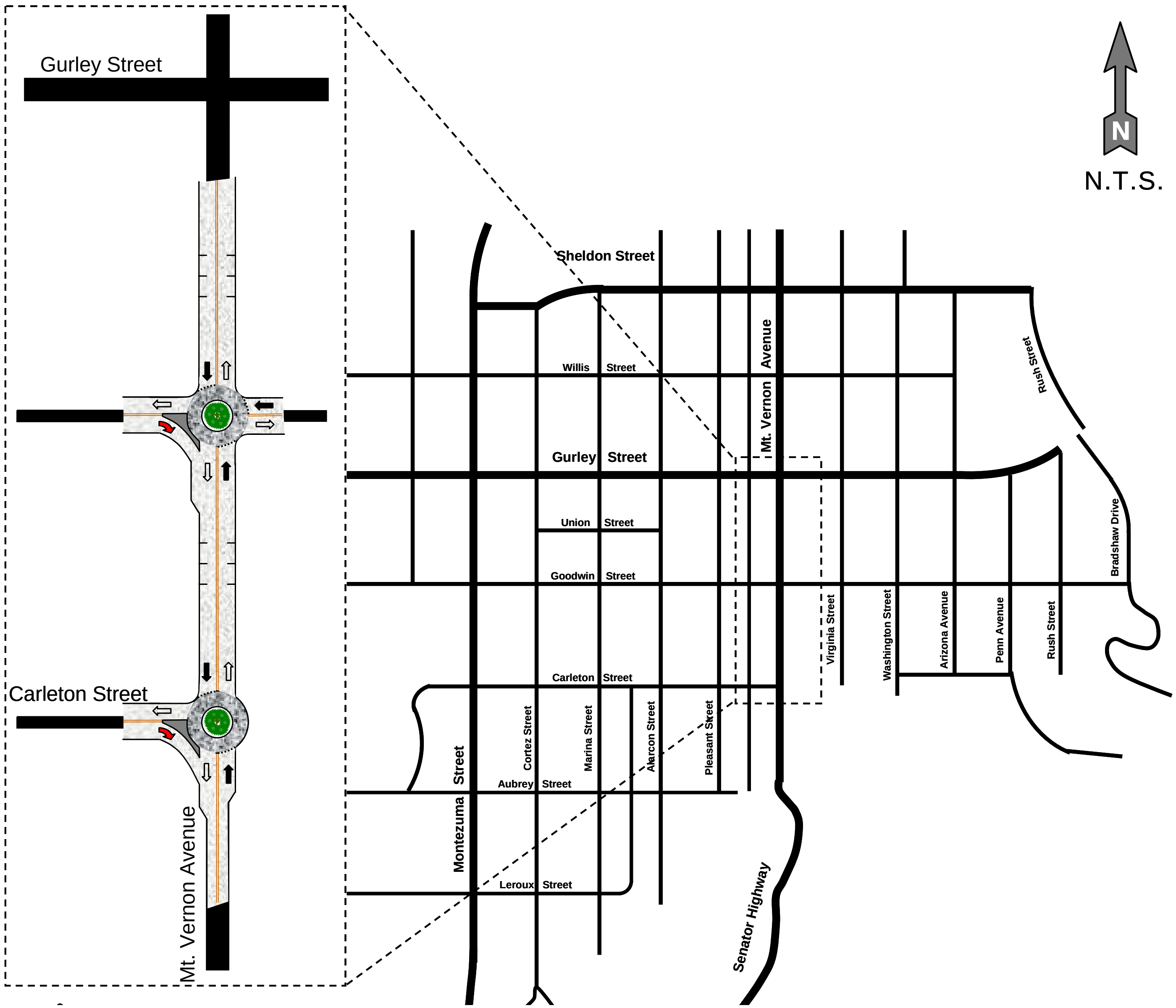


Figure 22. 2006 With Recommended Traffic Calming Improvements

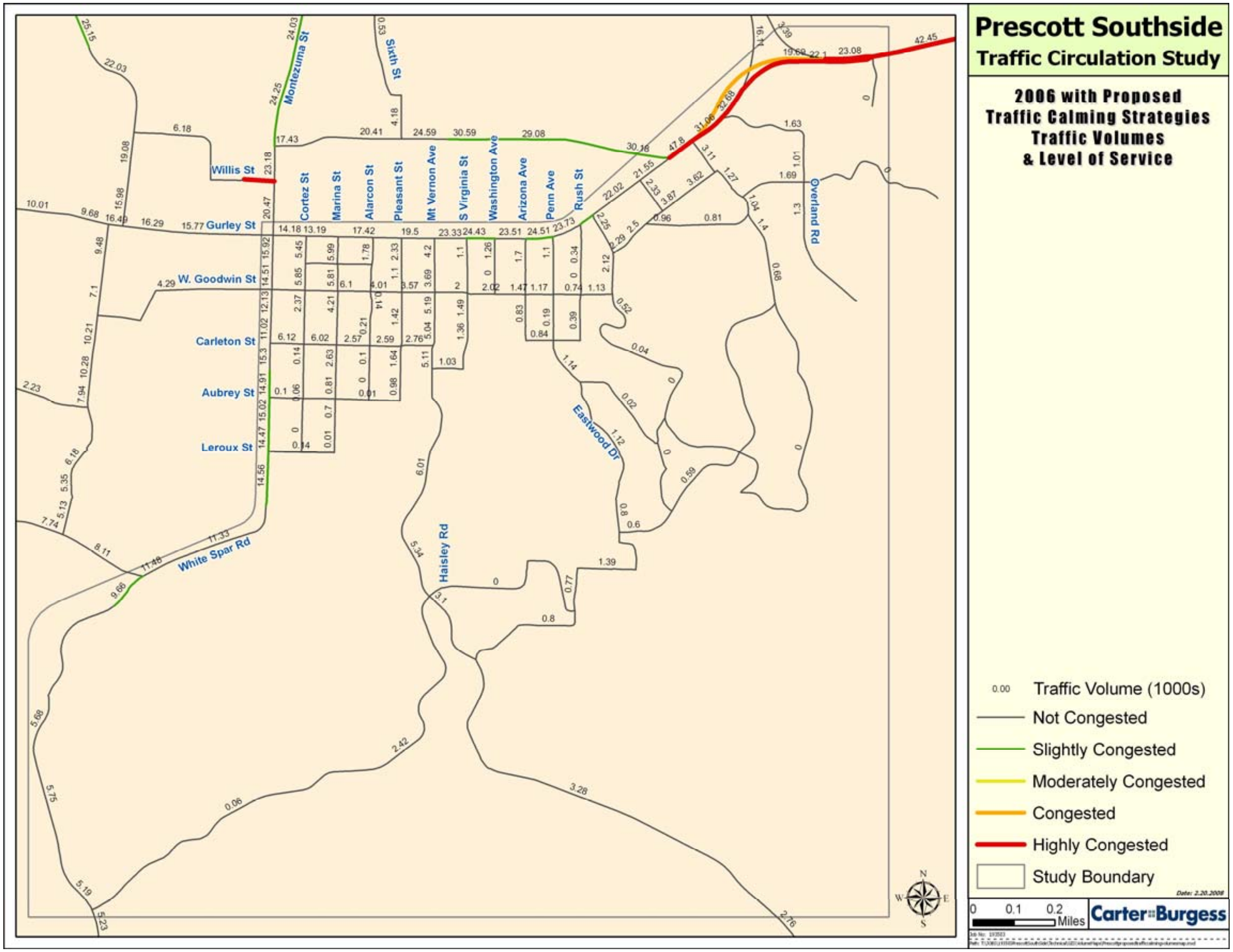
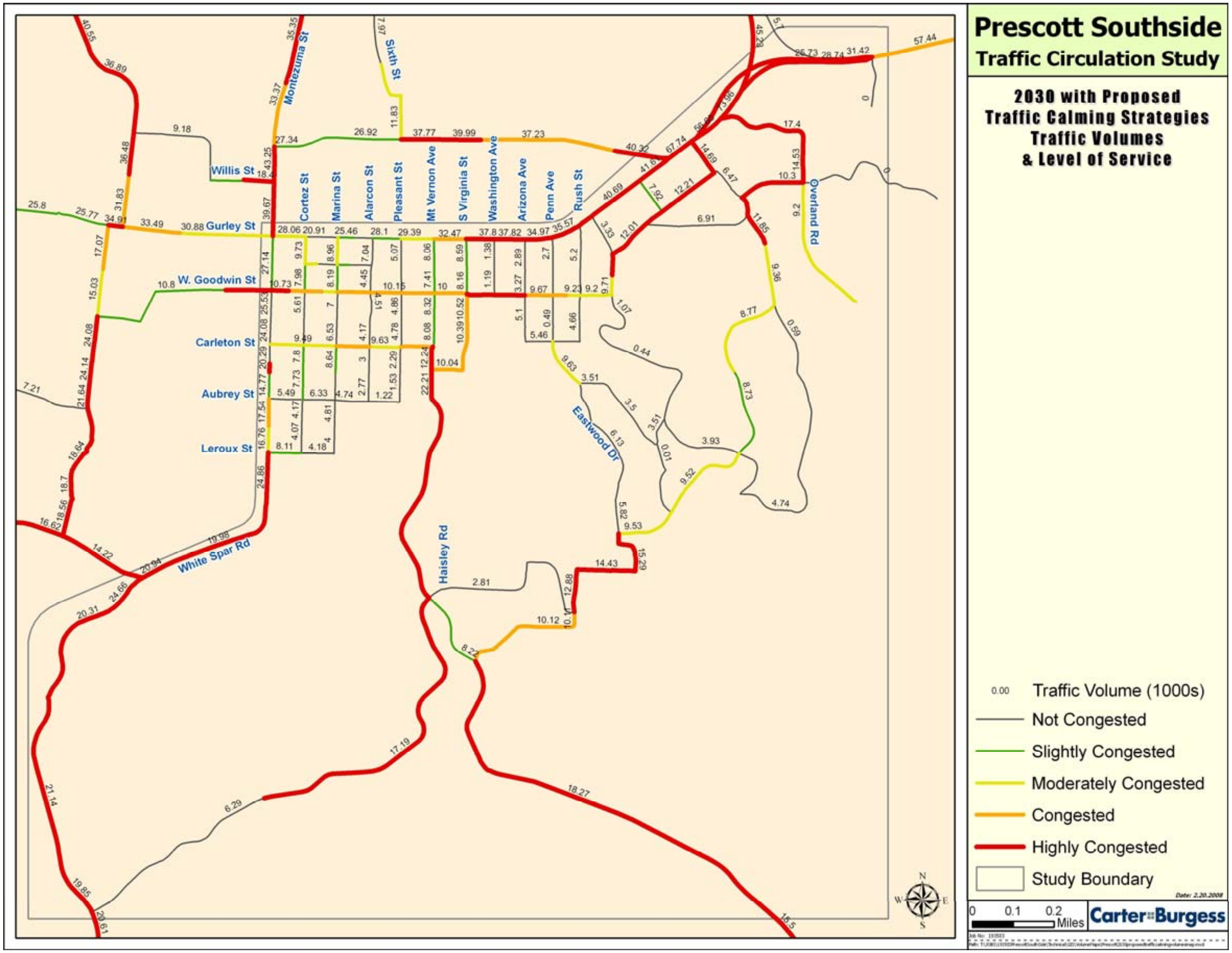


Figure 23. 2030 With Recommended Traffic Calming Improvements



V. PUBLIC INVOLVEMENT

Public Information Meetings

Two series of public meetings were held during the study to seek public input and comments. The first meeting was held to announce the beginning of the study and provided the public with an opportunity to voice their concerns and issues regarding the study area travel conditions. The meeting was conducted on February 1, 2007 at the Washington Elementary School Auditorium located at 300 E. Gurley in Prescott, AZ. Approximately 50 residents attended.

The majority of the public comments were in reference to the traffic traveling on Mt. Vernon Avenue and the impact of the vehicles on the historic neighborhood. Speeding and safety were the top concerns voice by the public. In addition, the public was concerned about the cut-through traffic utilizing the road to bypass Gurley Street.

The second public meeting was held on July 26, 2007 at Mile High Middle School, Hendricks Auditorium located at 300 S. Granite Street in Prescott, AZ. Approximately 60 residents attended the meeting. During the second public meeting, the current and future travel demand conditions findings were presented. In addition, three potential improvement concepts were presented for comment and discussion.

The citizens voiced concerns about the future population projection in the County portion of the study area. They are strongly opposed to any type of facility near Acker Park or facilities that would destroy the character of the community. They also provided feed back and ideas on the presented improvement concepts. Again, concern was expressed regarding the travel speed on Mt. Vernon Avenue.

APPENDIX A – ORIGIN- DESTINATION DATA

Traffic Count Data and OD study

A traffic count effort was conducted on February 13 and 14, 2007 within the study area to collect 48hr counts at specific locations and am and pm peak period turning movements counts at selected intersections. The turning movement data was collected from 7am to 10 am and from 3pm to 6pm. Figure 1 depicts the 48 hours and turning movement count locations. Results from this effort are displayed in the final report.

An origin and destination (OD) survey was conducted on February 15, 2007 to ascertain the trip characteristics along Mt Vernon at the following locations:

1 On Mt Vernon south of Carleton
2 Mt Vernon and Goodwin intersection
3 Mt Vernon and Carleton Intersection

The survey was conducted during the hours of 7am and 9 am and between 4 pm and 6 pm. People traveling along those routes were stopped and asked specific questions regarding their trip characteristics. Tables 1-3 show the survey instrument used in the OD study. Questions regarding trips origin, trip destination, purpose of the trip, and probable routes used in the trip making, were the questions of choice. Additional visual information was collected regarding vehicle type and auto occupancy.

Figures 2-4 depicts the results of the OD with respect to the route characteristics represented in the percent of the trips beginning at a specific location and traveling along Mt Vernon to reach their destination. Both am and pm conditions are presented. Figures 5-28 display trip making characteristics associated with the O-D survey.

FIGURE 1. COUNT LOCATIONS

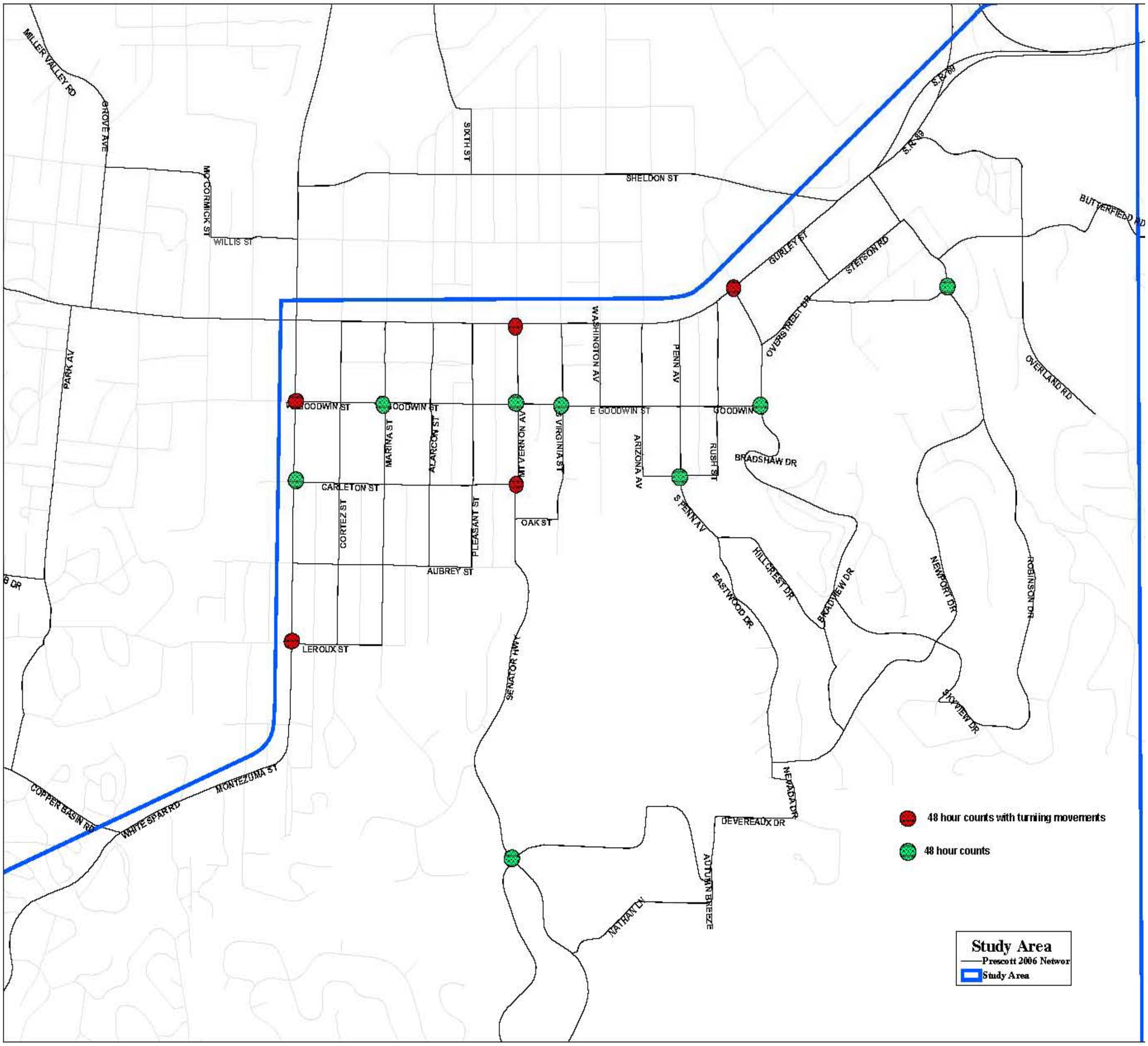


TABLE 1. ORIGIN-DESTINATION QUESTIONNAIRE
MT VERNON AVENUE - PRESCOTT, AZ
February 15, 2007

Mt Vernon Avenue			
Time of Survey		☐AM ☐PM	**Roads traveled or will travel** ☐Haisley Road ☐Gurley ☐Goodwin ☐Carlton ☐Senator Hwy
Number of Passengers ☐One person ☐Three persons ☐Two persons ☐More than 3 persons			**Type of Vehicle** ☐Car ☐Motorcycle ☐Light truck ☐Van ☐Pick-up truck ☐Heavy truck

1. **Where did this trip begin, list cross streets or nearest landmark**.

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey-Humboldt ☐Senator HWY
☐Spring Valley ☐Iron Springs/Copper Basin ☐Williamson Valley ☐Lonesome Valley

2. **What is your destination, list cross streets or nearest landmark**.

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey-Humboldt ☐Senator Hwy
☐Spring Valley ☐Iron Springs/Copper Basin ☐Williamson Valley ☐Lonesome Valley

3. **What is the purpose of the trip?**

☐Work ☐Business ☐Social ☐Shopping
☐Personal (i.e. medical trip, pick-up children) ☐Other ____________

4. **Where do you live most of the year?**

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey/Humboldt
☐Mayer ☐Williamson Valley ☐Phoenix ☐Other ____________

5. **How often do you use Mt Vernon to make trips?**

☐At least once a day ☐At least once a week ☐Less than once per week
☐Several times a day ☐A few times per week ☐Almost never

TABLE 2. ORIGIN-DESTINATION QUESTIONNAIRE
MT VERNON AVENUE - PRESCOTT, AZ
February 15, 2007

Goodwin Street			
Time of Survey		☐AM ☐PM	**Roads traveled or will travel** ☐Haisley Road ☐Gurley/Mt. Vernon ☐Goodwin ☐Senator Hwy
Number of Passengers ☐One person ☐Three persons ☐Two persons ☐More than 3 persons			**Type of Vehicle** ☐Car ☐Motorcycle ☐Light truck ☐Van ☐Pick-up truck ☐Heavy truck

1. **Where did this trip begin, list cross streets or nearest landmark.**

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey-Humboldt ☐Senator HWY
☐Spring Valley ☐Iron Springs/Copper Basin ☐Williamson Valley ☐Lonesome Valley

2. **What is your destination, list cross streets or nearest landmark.**

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey-Humboldt ☐Senator Hwy
☐Spring Valley ☐Iron Springs/Copper Basin ☐Williamson Valley ☐Lonesome Valley

3. **What is the purpose of the trip?**

☐Work ☐Business ☐Social ☐Shopping
☐Personal (i.e. medical trip, pick-up children) ☐Other ____________________

4. **Where do you live most of the year?**

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey/Humboldt
☐Mayer ☐Williamson Valley ☐Phoenix ☐Other ____________________

5. **How often do you use Mt Vernon to make trips?**

☐At least once a day ☐At least once a week ☐Less than once per week
☐Several times a day ☐A few times per week ☐Almost never

TABLE 3. ORIGIN-DESTINATION QUESTIONNAIRE
MT VERNON AVENUE - PRESCOTT, AZ
February 15, 2007

Carlton Street			
Time of Survey		☐AM ☐PM	**Roads traveled or will travel** ☐Haisley Road ☐Gurley/Mt Vernon ☐Goodwin/Mt Vernon ☐Senator Hwy
Number of Passengers ☐One person ☐Three persons ☐Two persons ☐More than 3 persons			**Type of Vehicle** ☐Car ☐Motorcycle ☐Light truck ☐Van ☐Pick-up truck ☐Heavy truck

1. **Where did this trip begin, list cross streets or nearest landmark.**

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey-Humboldt ☐Senator HWY
☐Spring Valley ☐Iron Springs/Copper Basin ☐Williamson Valley ☐Lonesome Valley

2. **What is your destination, list cross streets or nearest landmark.**

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey-Humboldt ☐Senator Hwy
☐Spring Valley ☐Iron Springs/Copper Basin ☐Williamson Valley ☐Lonesome Valley

3. **What is the purpose of the trip?**

☐Work ☐Business ☐Social ☐Shopping
☐Personal (i.e. medical trip, pick-up children) ☐Other ______________________

4. **Where do you live most of the year?**

☐Prescott ☐Prescott Valley ☐Chino Valley ☐Dewey/Humboldt
☐Mayer ☐Williamson Valley ☐Phoenix ☐Other ______________

5. **How often do you use Mt Vernon to make trips?**

☐At least once a day ☐At least once a week ☐Less than once per week
☐Several times a day ☐A few times per week ☐Almost never

FIGURE 2

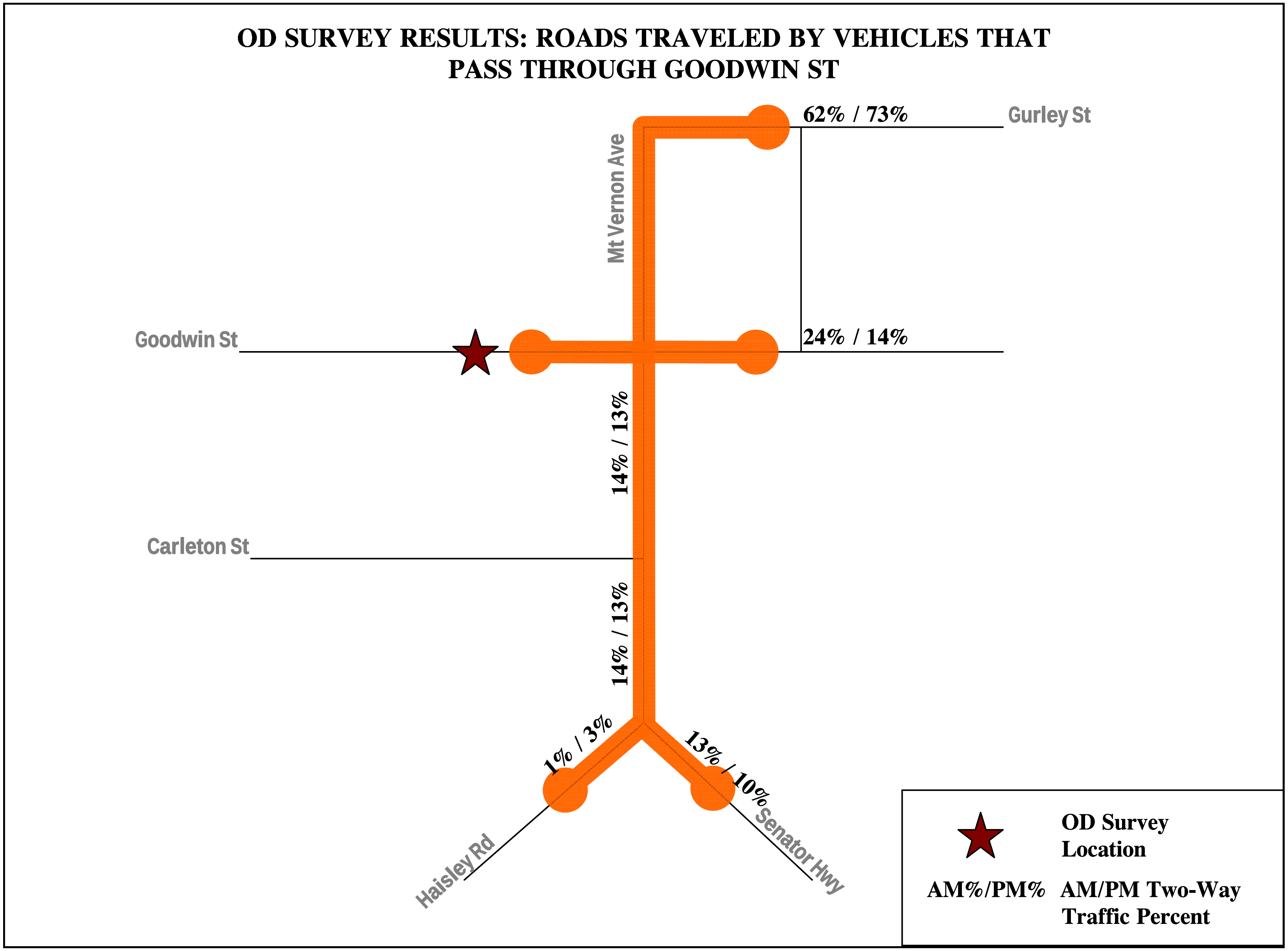
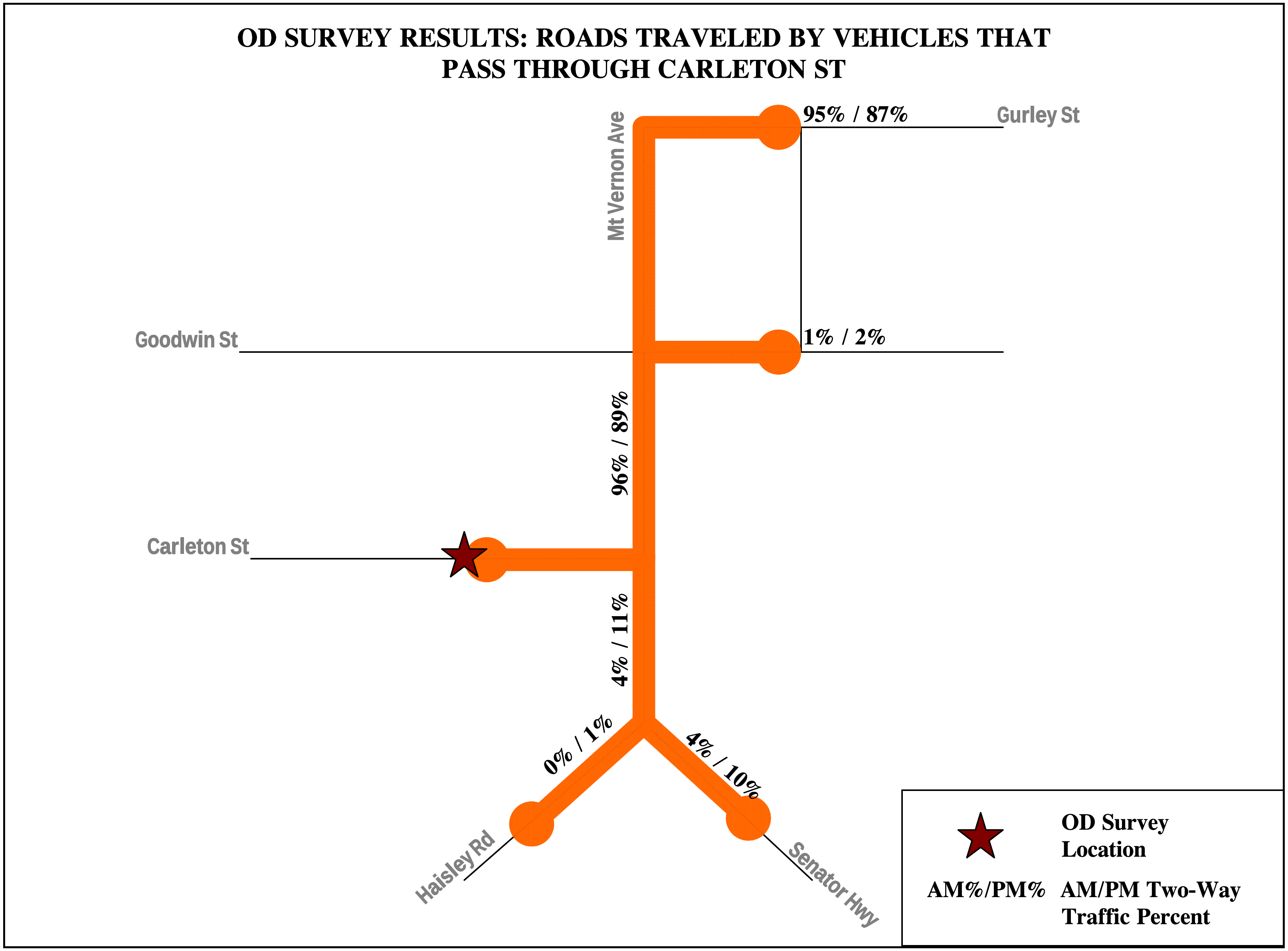

OD SURVEY RESULTS: ROADS TRAVELED BY VEHICLES THAT PASS THROUGH GOODWIN ST
62% / 73%
Gurley St
Mt Vernon Ave
Goodwin St
24% / 14%
14% / 13%
Carleton St
14% / 13%
1% / 3%
13% / 10%
Haisley Rd
Senator Hwy
OD Survey Location
AM%/PM%
AM/PM Two-Way Traffic Percent

FIGURE 3



OD SURVEY RESULTS: ROADS TRAVELED BY VEHICLES THAT PASS THROUGH CARLETON ST
95% / 87%
Gurley St
Mt Vernon Ave
Goodwin St
1% / 2%
96% / 89%
Carleton St
4% / 11%
0% / 1%
4% / 10%
Haisley Rd
Senator Hwy
OD Survey Location
AM%/PM%
AM/PM Two-Way Traffic Percent

FIGURE 4

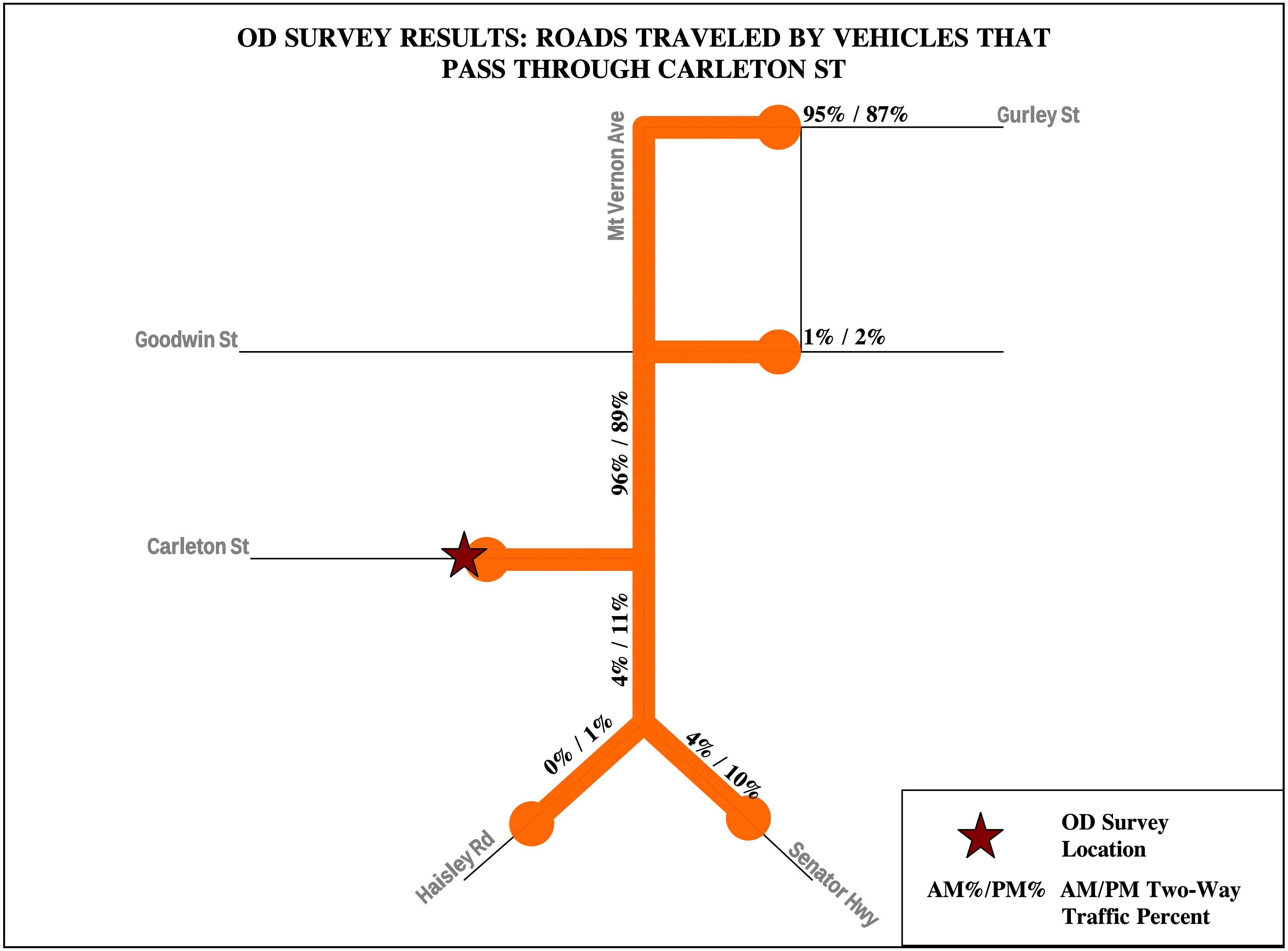

OD SURVEY RESULTS: ROADS TRAVELED BY VEHICLES THAT PASS THROUGH CARLETON ST
Mt Vernon Ave
95% / 87%
Gurley St
Goodwin St
1% / 2%
96% / 89%
Carleton St
4% / 11%
0% / 1%
4% / 10%
Haisley Rd
Senator Hwy
OD Survey Location
AM%/PM%
AM/PM Two-Way Traffic Percent

FIGURE 5. MT. VERNON - ROADWAYS TRAVELED

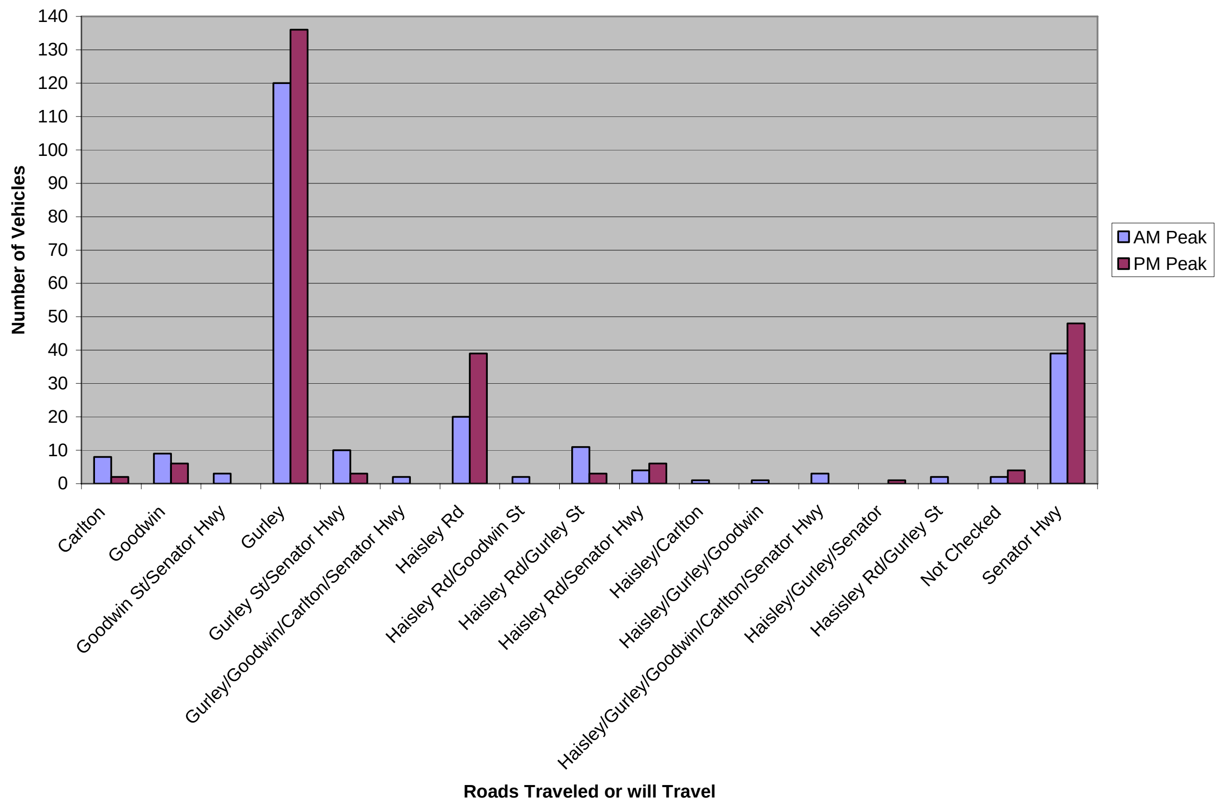


FIGURE 6. MT VERNON - AUTO OCUUPANCY

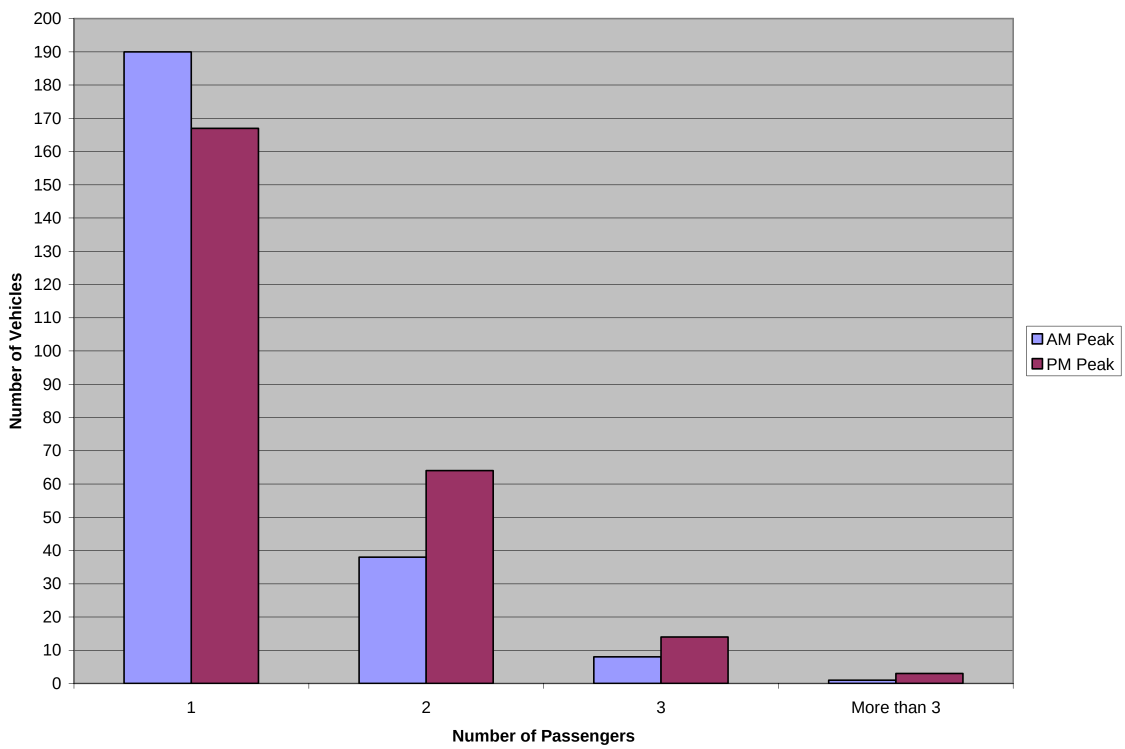


FIGURE 7. MT VERNON – VEHICLE TYPE

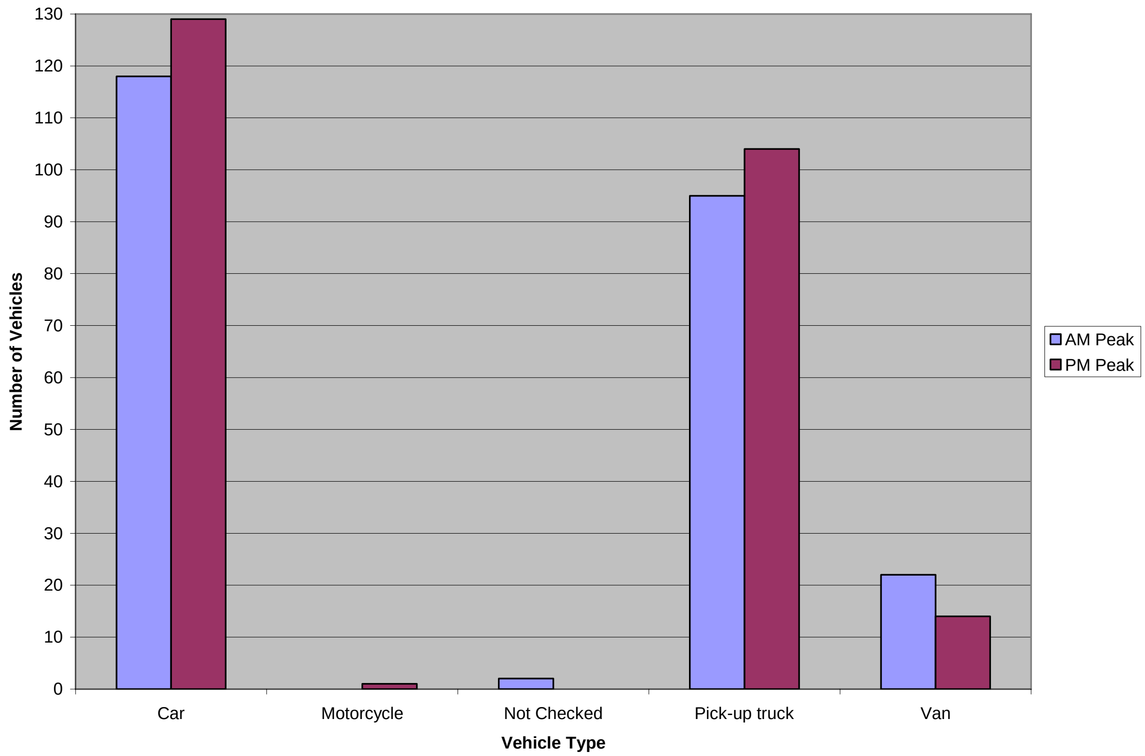


FIGURE 8. MT VERNON - TRIP ORGINS

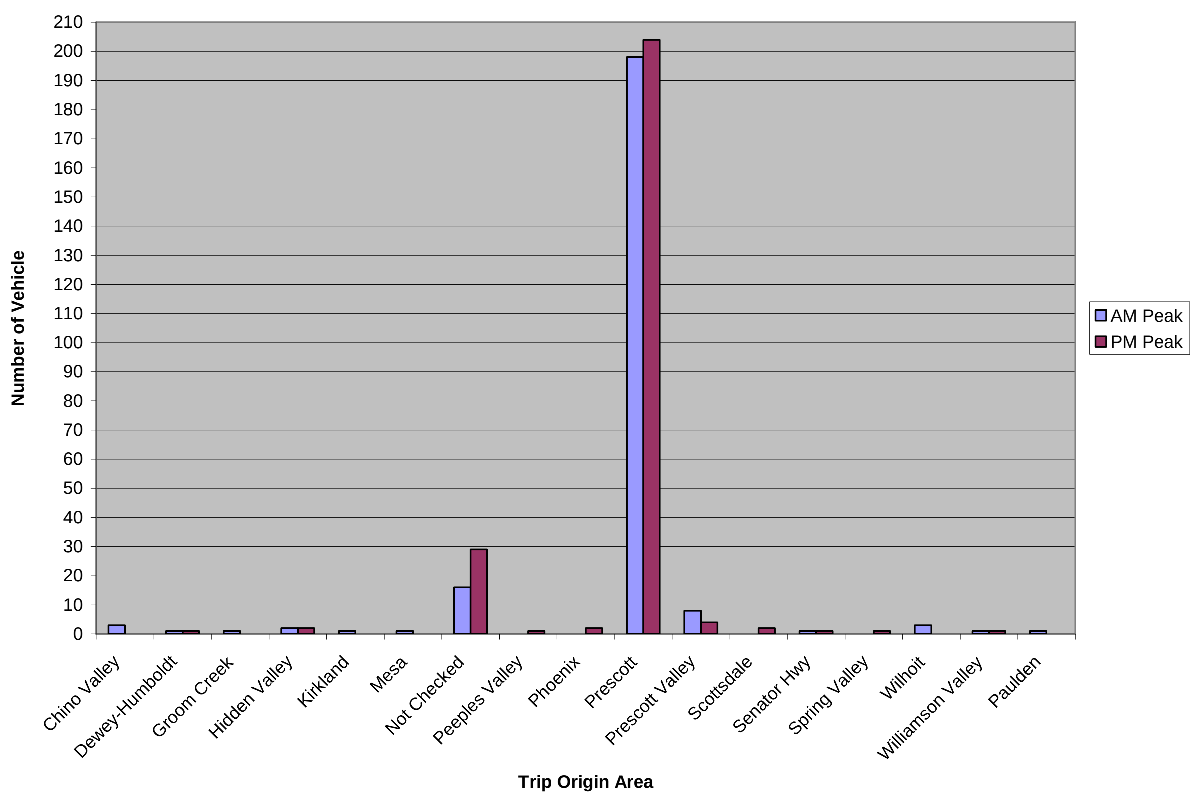


FIGURE 9. – MT VERNON - TRIP DESTINATIONS

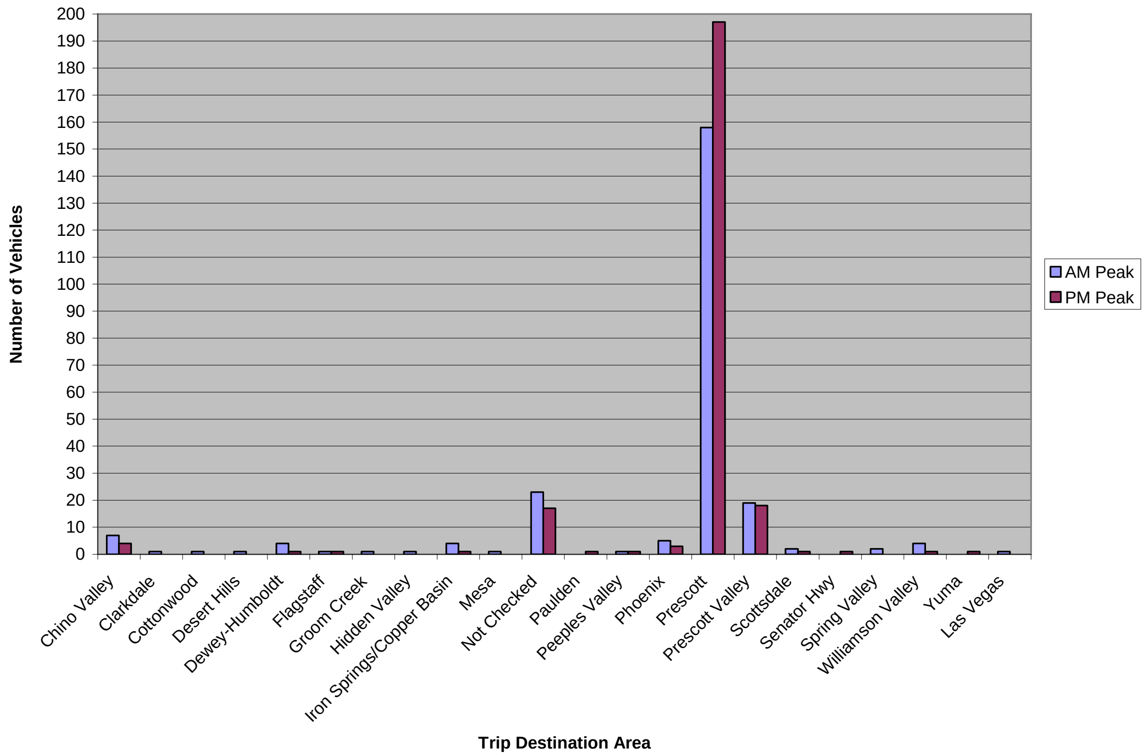


FIGURE 10. MT VERNON – TRIP PURPOSE

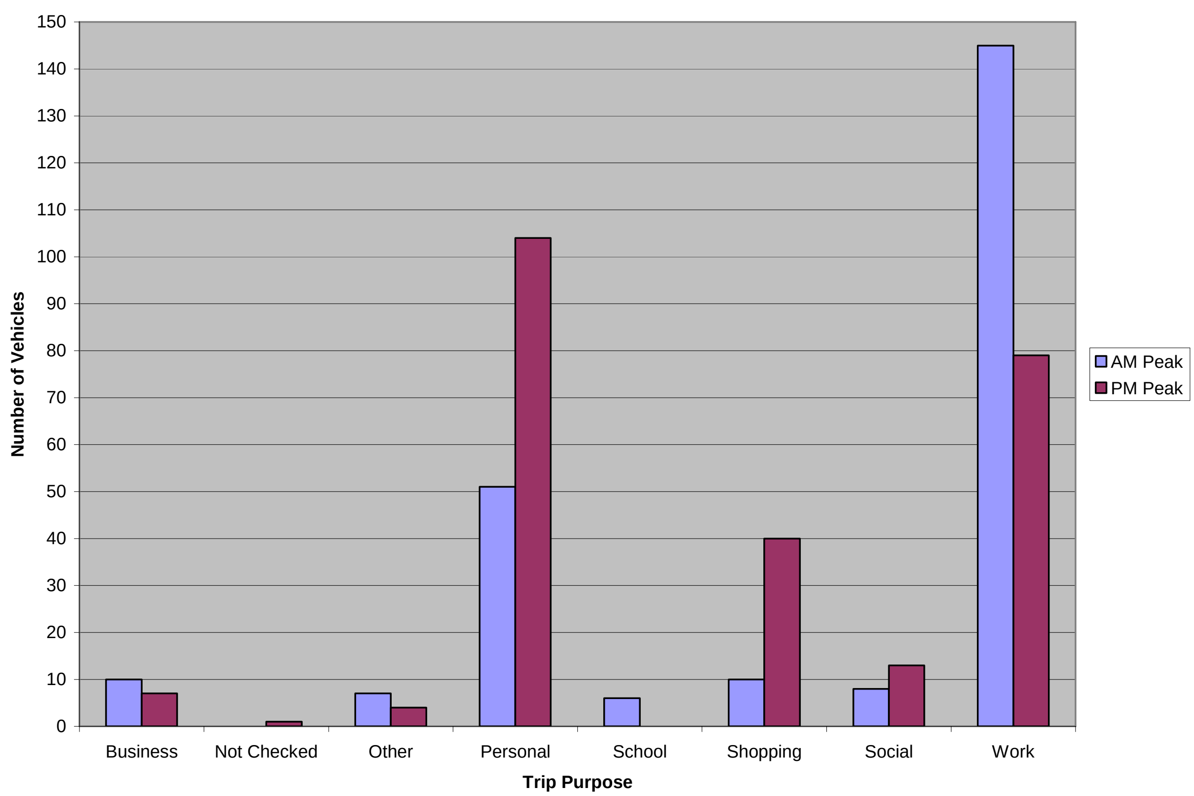


FIGURE 11. MT VERNON – PLACE OF RESIDNCE

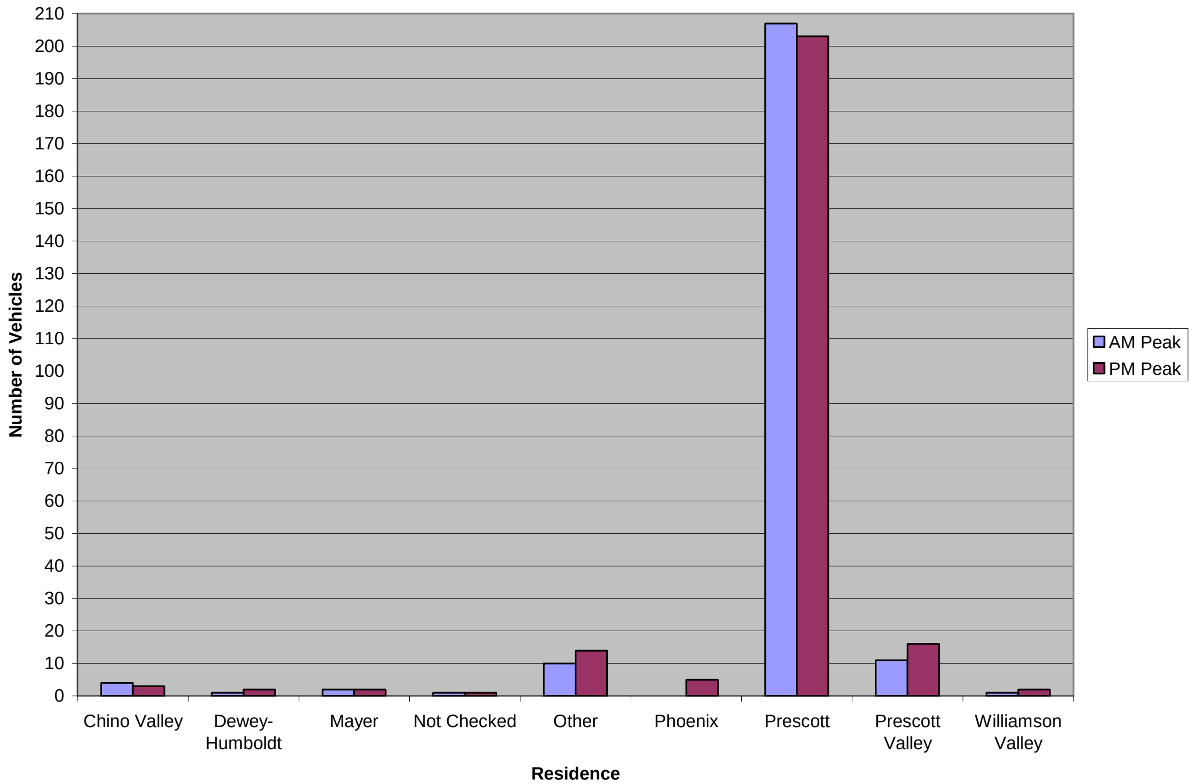


FIGURE 12. – GOODWIND – VEHICLE OCCUPANCY

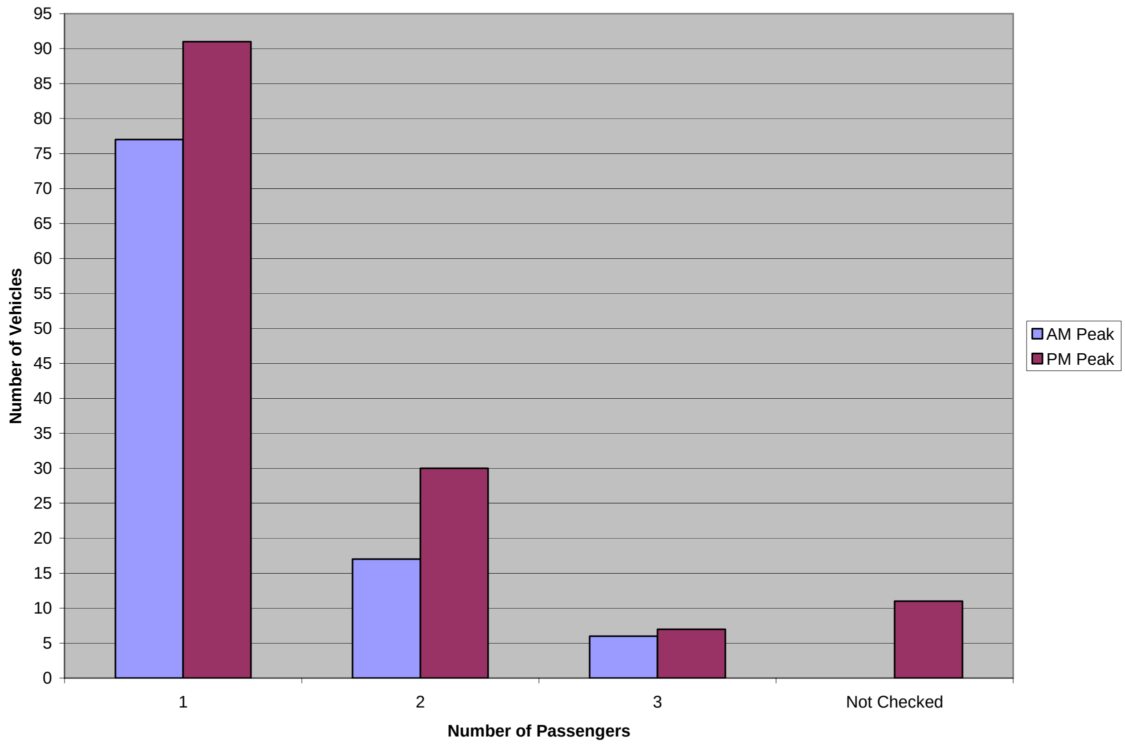


FIGURE 13. – GOODWIN ROADS TRAVELED

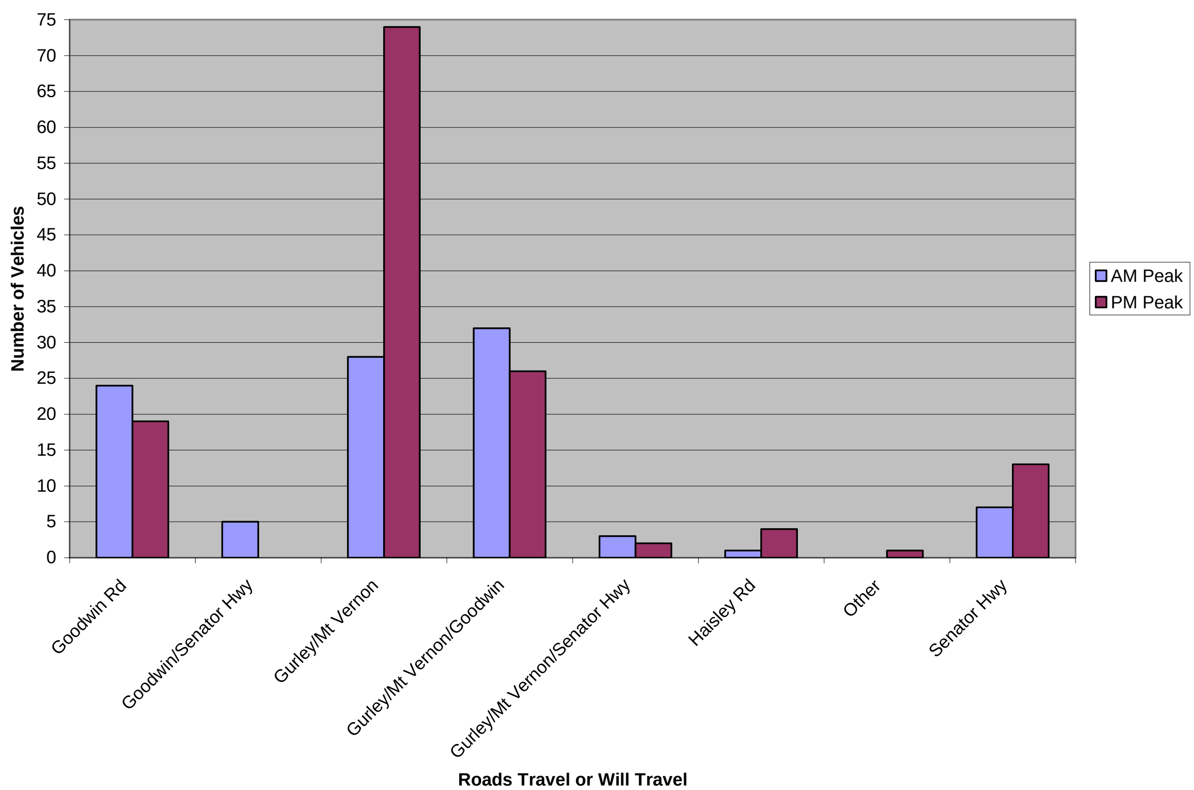


FIGURE 15. GOODWIN – VEHICLE TYPE

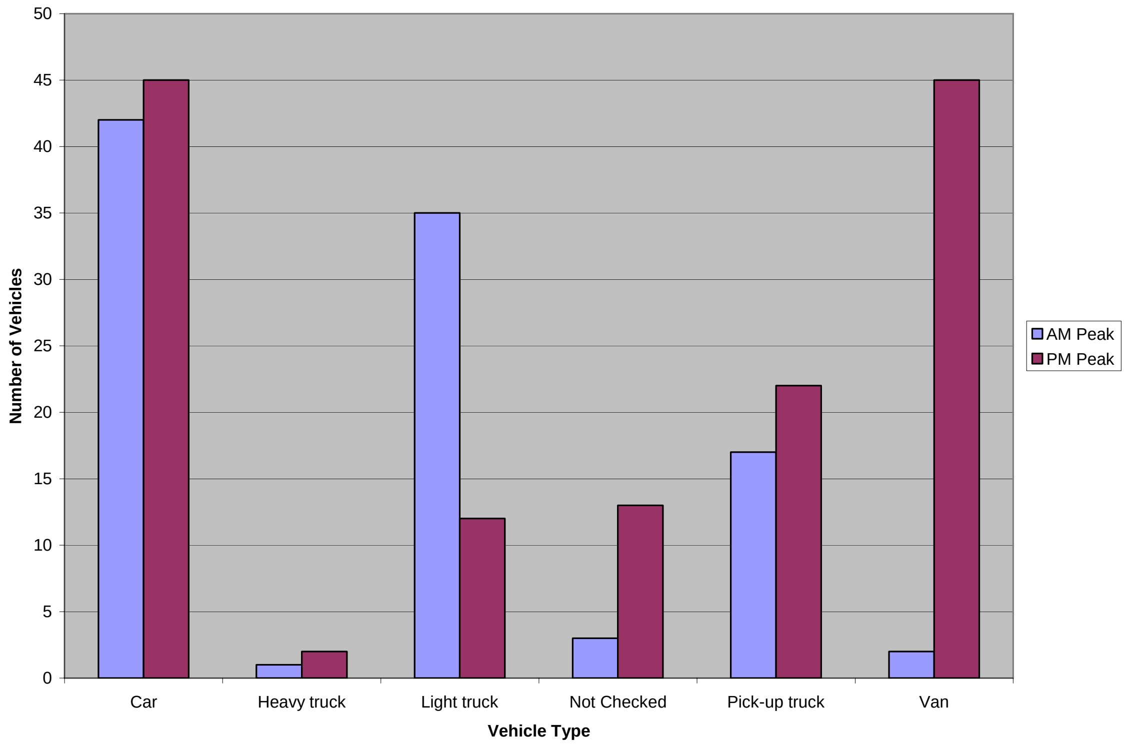


FIGURE 16. GOODWIN – TRIP ORIGIN

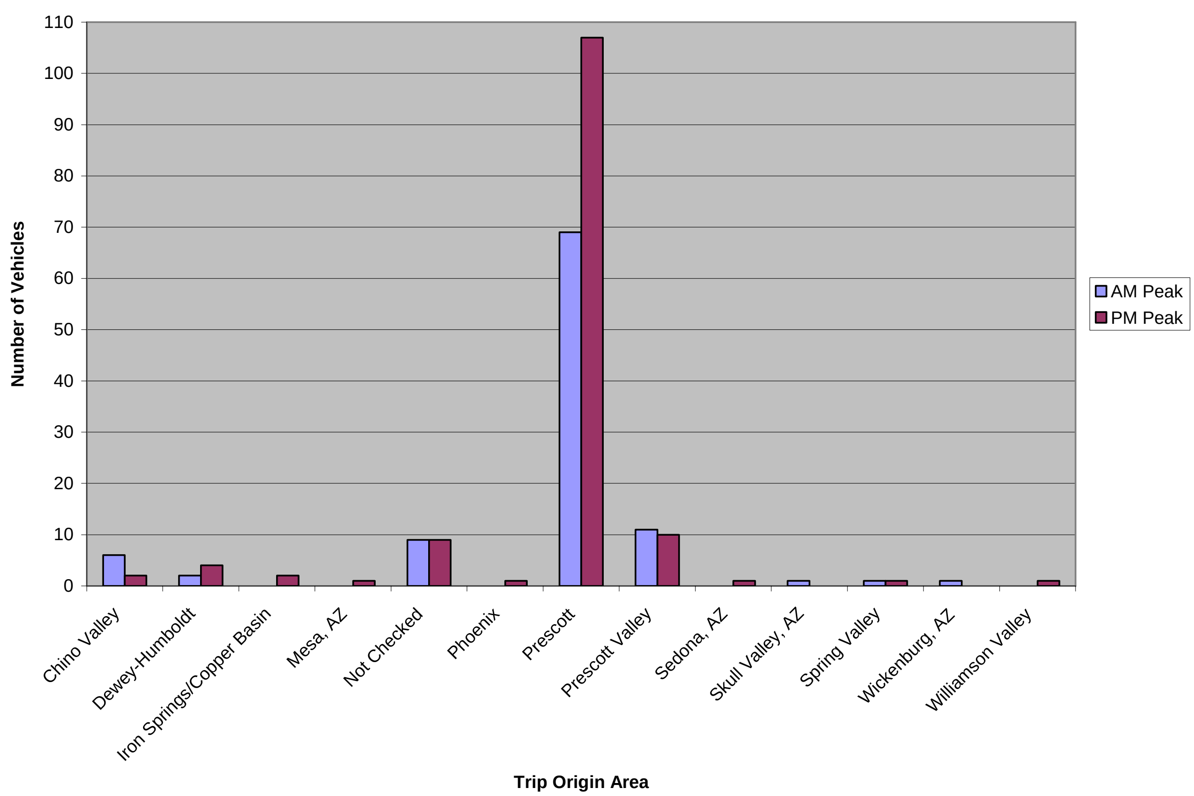


FIGURE 17. GOODWIN – TRIP DESTINATIONS

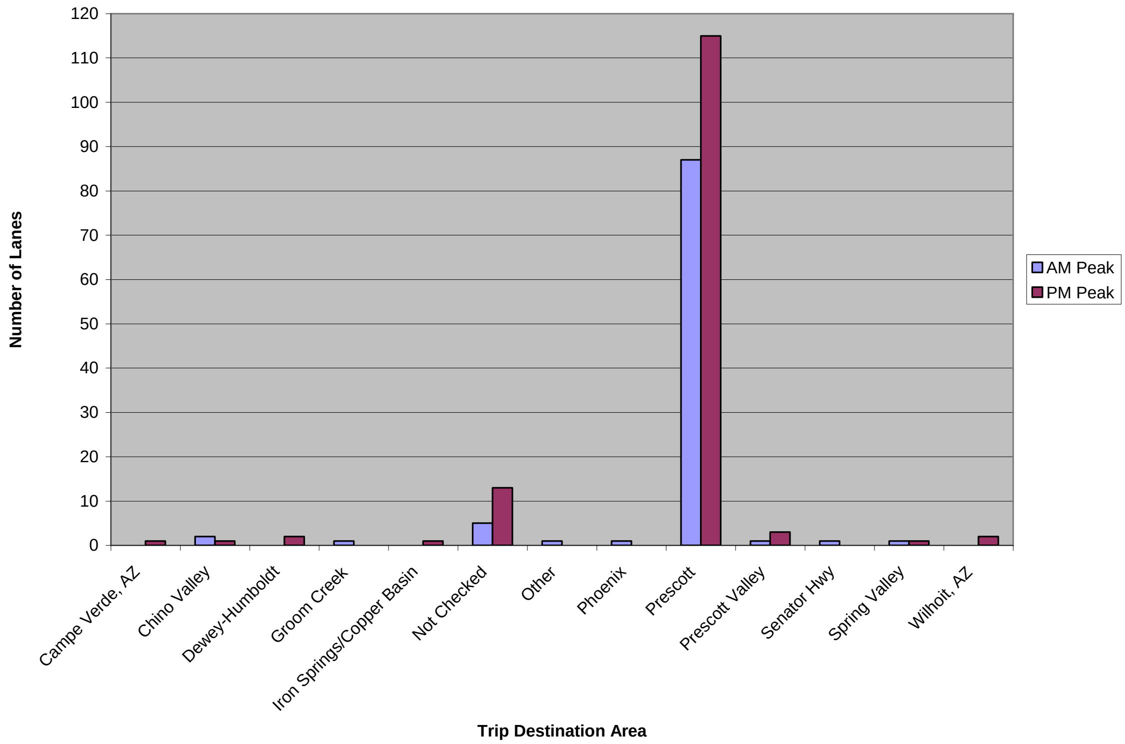


FIGURE 18. GOODWIN – TRIP PURPOSE

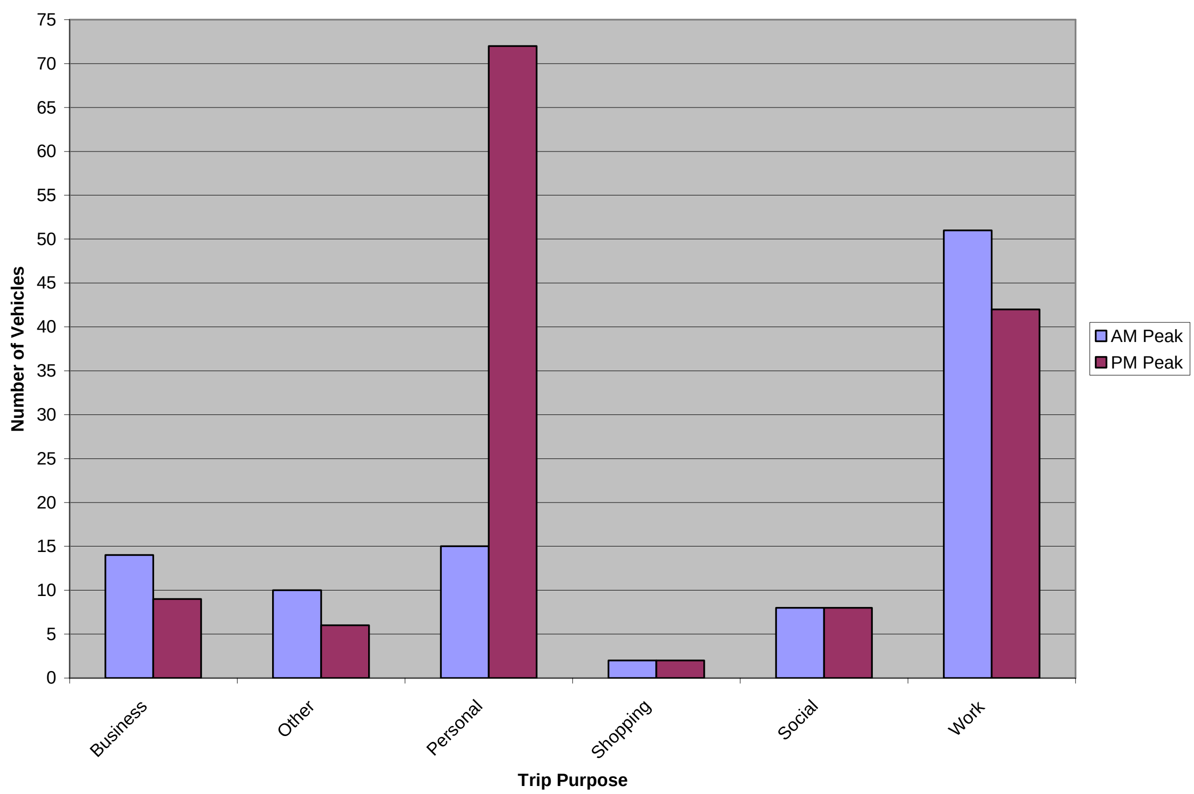


FIGURE 19. GOODWIN – PLACE OF RESIDENCE

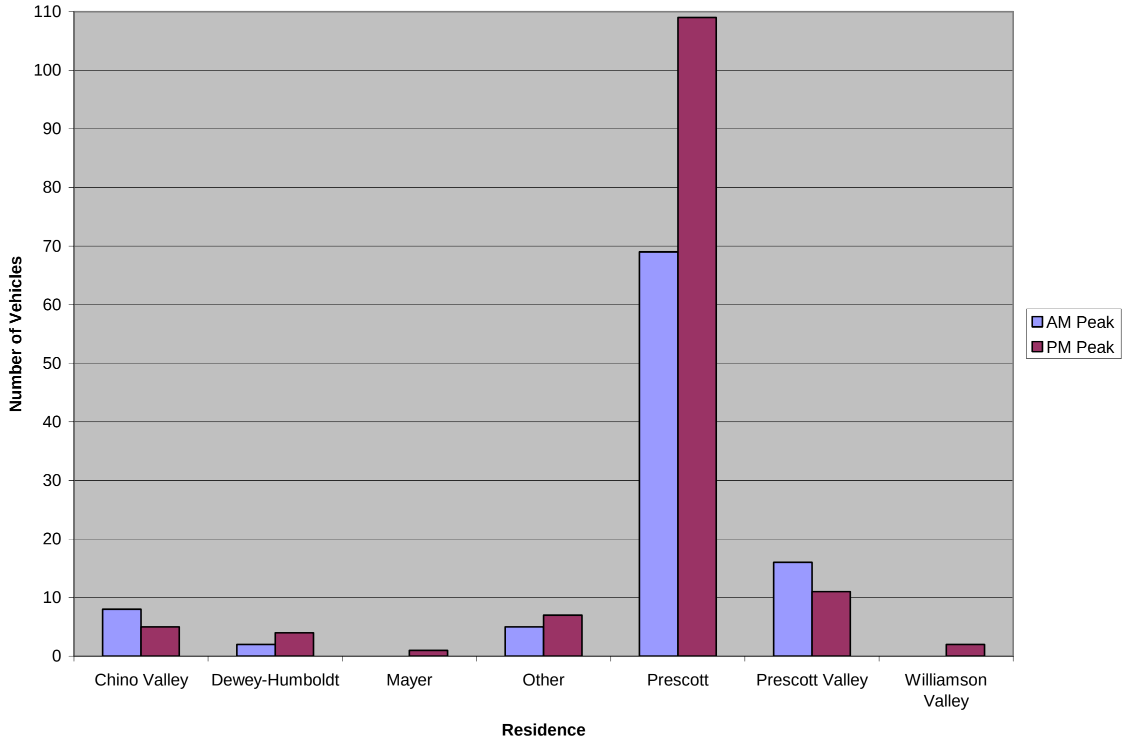


FIGURE 20. GOODWIN – FREQUENCY OF TRIPS ON MT. VERNON

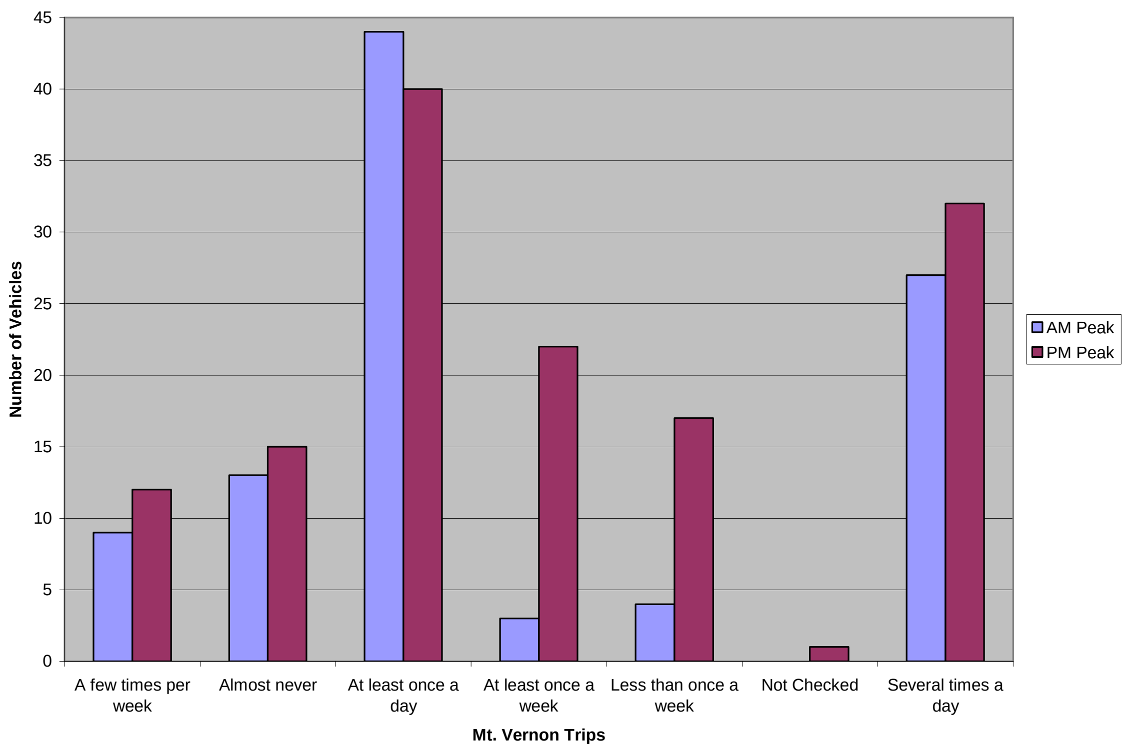


FIGURE 21. CARLTON – ROADS TRAVELED

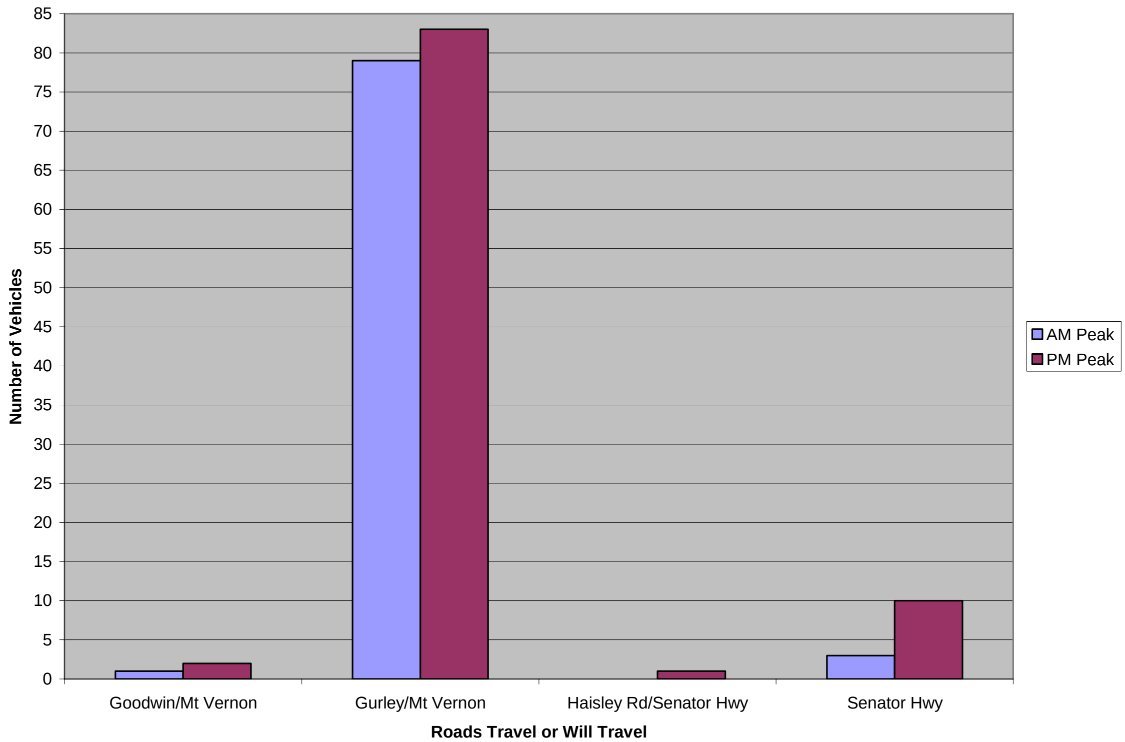


FIGURE 22. CARLTON – AUTO OCCUPANCY

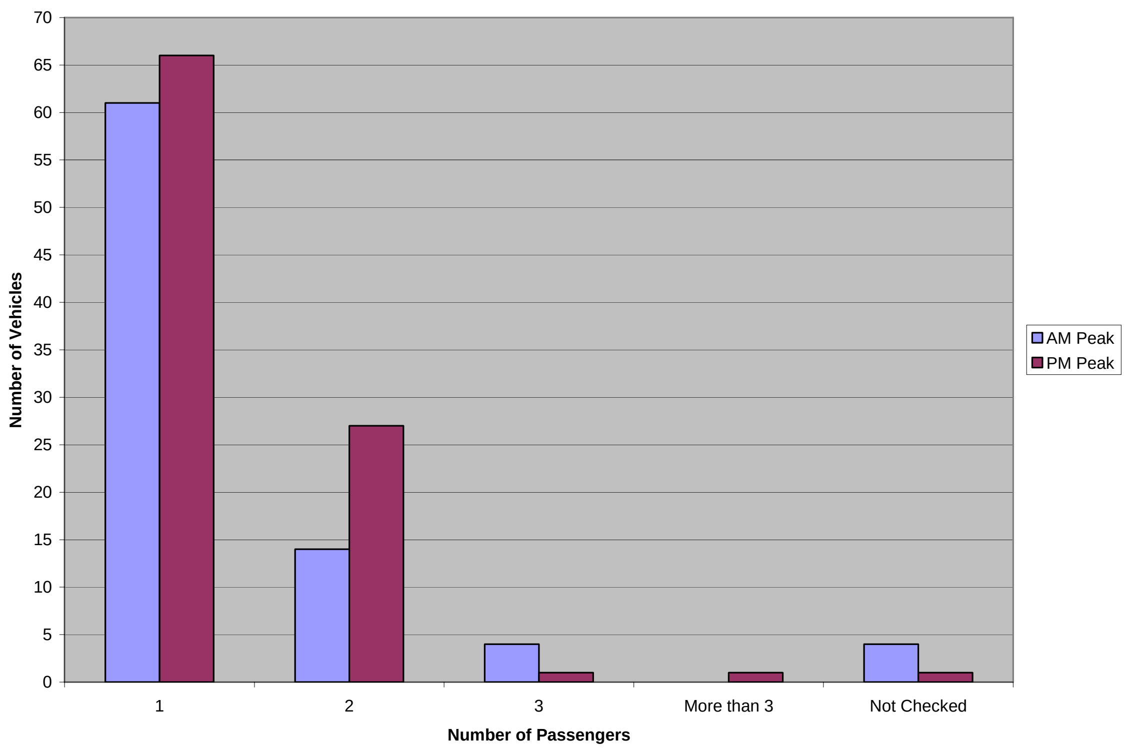


FIGURE 23. CARLTON - VEHICLE TYPE

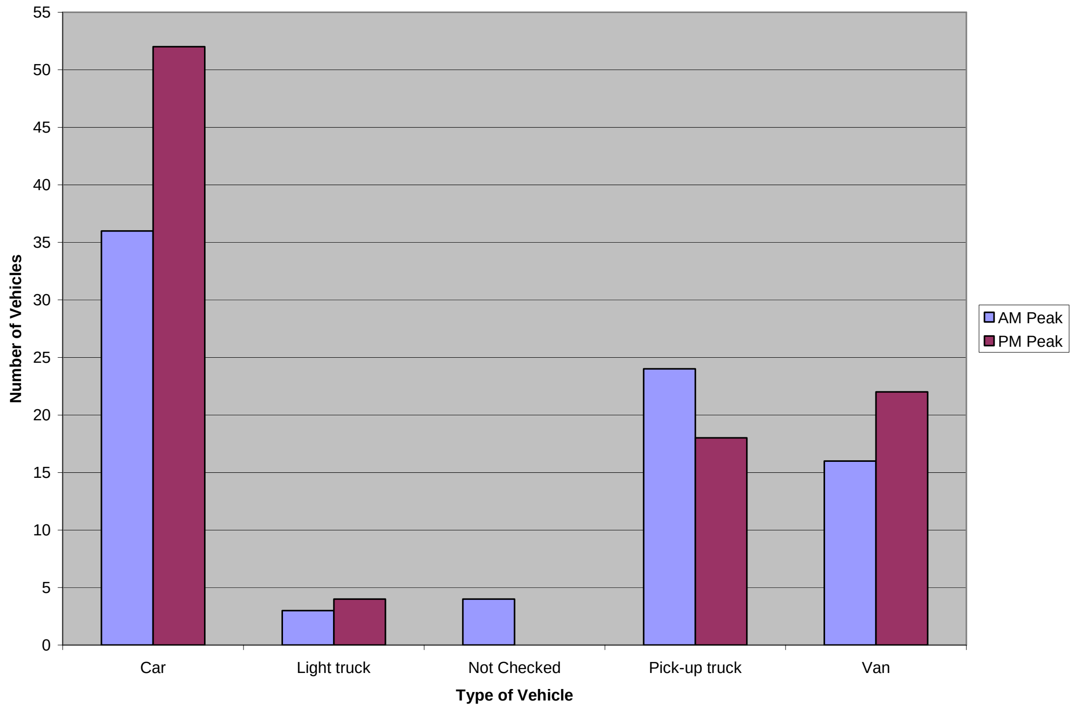


FIGURE 24. CARTON - TRIP ORIGINS

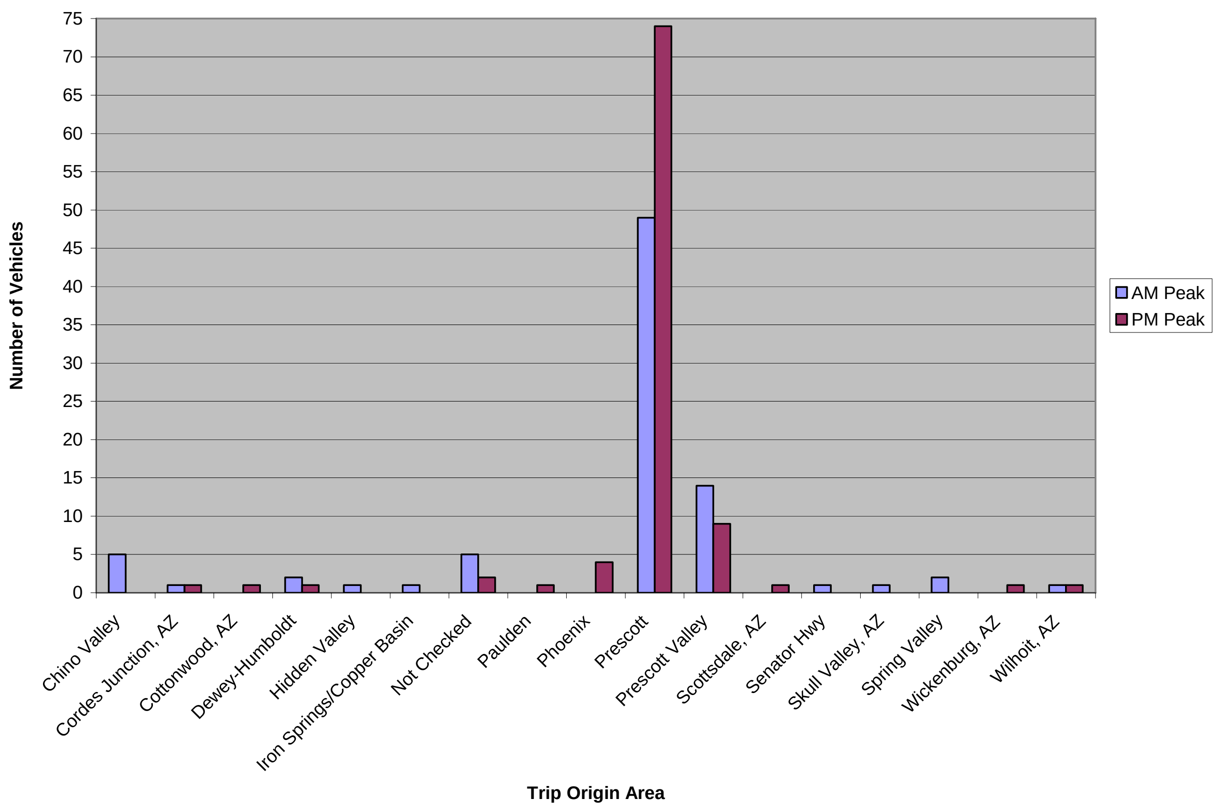


FIGURE 25. CARLTON - TRIP DESTINATIONS

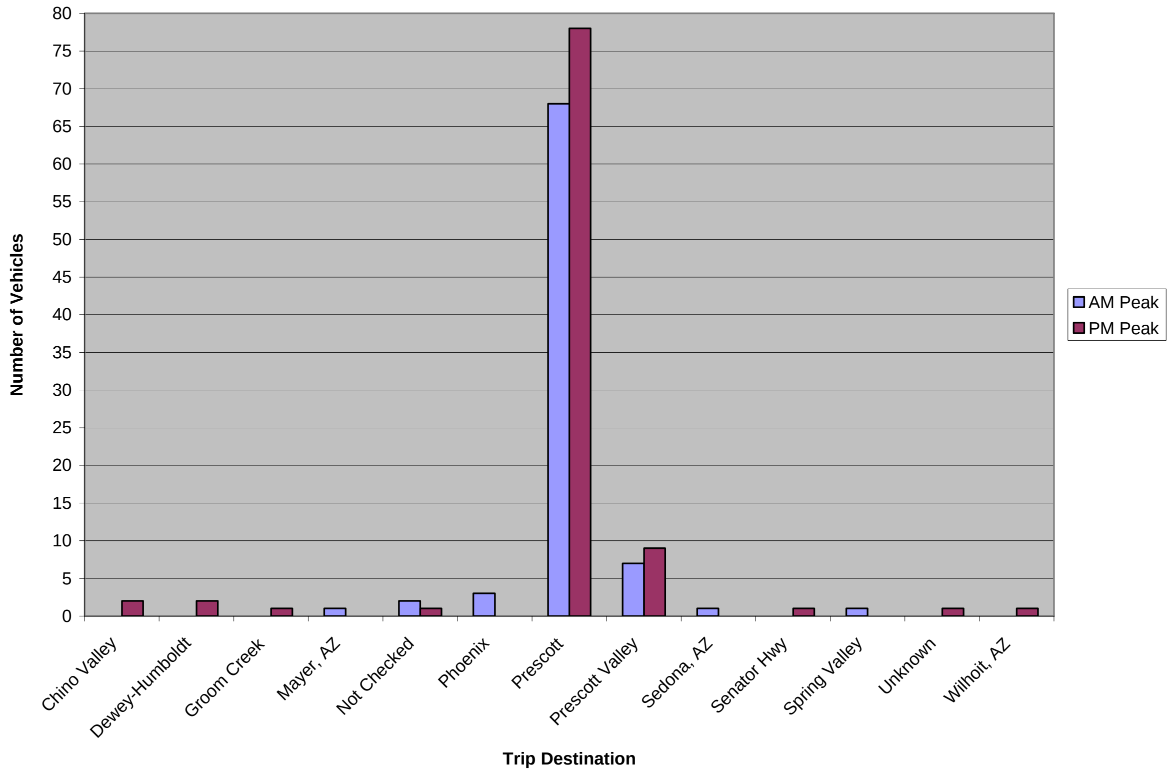


FIGURE 26. CARLETON - TRIP PURPOSES

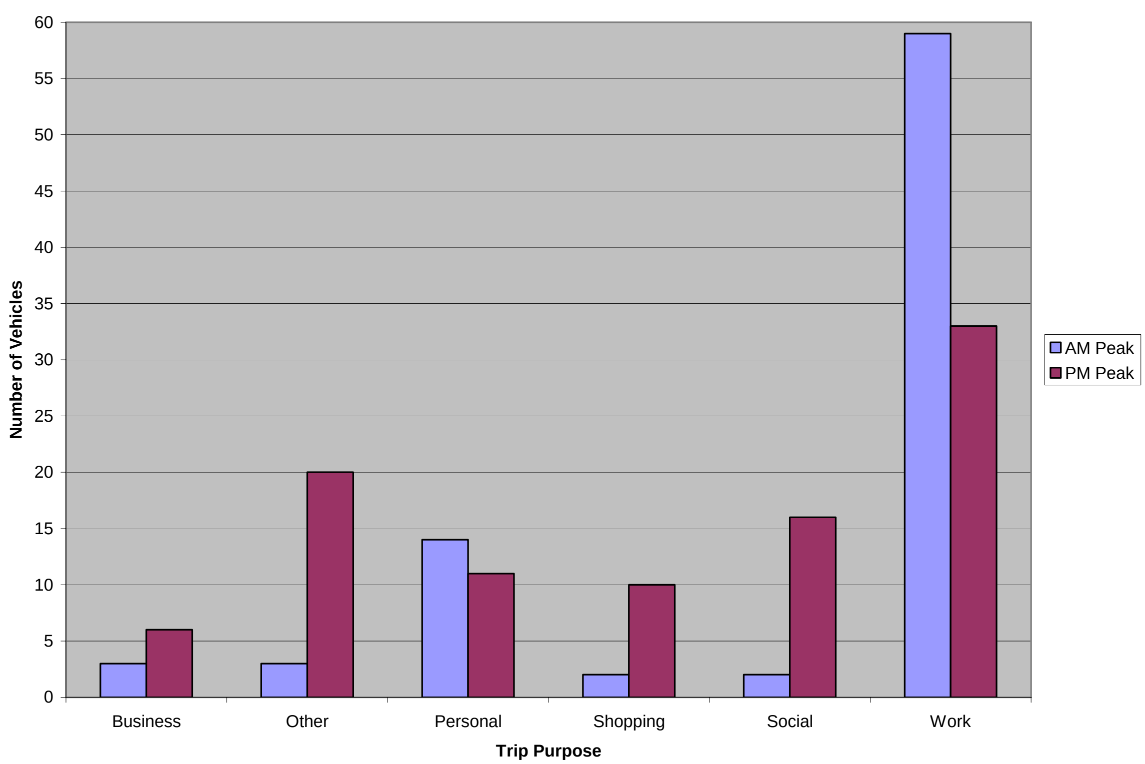


FIGURE 27. CARLTON - PLACE OF RESIDENCE

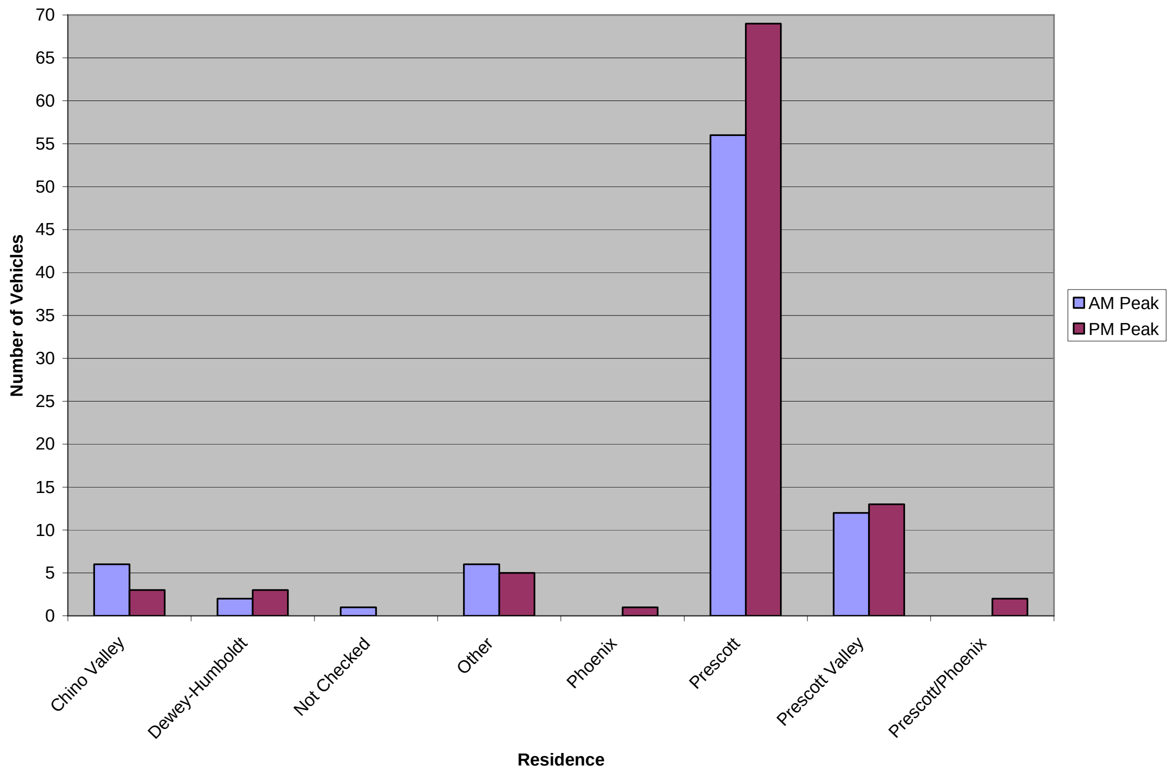
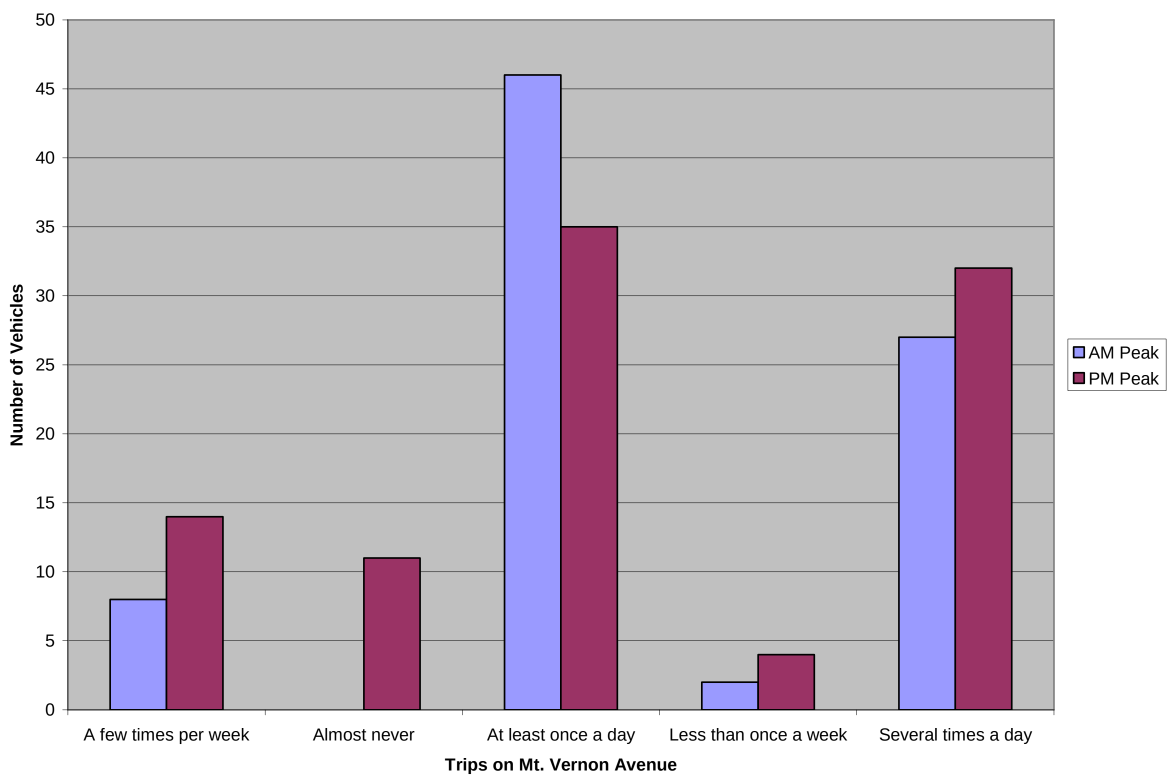


FIGURE 28. CALRTON - FREQUENCY OF TRIPS ON MT. VERNON



APPENDIX B – 2006 TRAFFIC DATA

Prescott, Arizona

Southside
2006 Traffic

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Existing Traffic Counts

Traffic Research and Analysis counted current traffic volumes at a portion of the intersections in the study area on Wednesday, 14 February 2007 for this study through contract with Morrison-Maierle. Additionally, traffic volumes previously counted by Field Data Services on Tuesday, 6 June, and Wednesday, 14 June, 2006 were utilized. Also traffic volumes previously counted by Traffic Research and Analysis on Tuesday, 19 March 2002, Wednesday, 20 March 2002, Wednesday, 29 May 2002, Thursday, 30 May 2002, Tuesday, 25 May 2004, and Wednesday, 26 May 2004, were utilized. The previous traffic counts were increased with a 4.0% annual rate for the appropriate time interval to achieve equivalent 2007 traffic volumes. The locations of the study intersections are shown on the following page in **Figure 1**.

The daily directional volumes for the study intersections are shown on the subsequent pages in **Figure 2** through **Figure 6**. The directional split for the morning and evening peak hour traffic volumes are shown in **Figure 12** and **Figure 15**. The turning movement volumes for selected study intersections are shown in **Figure 7** through **Figure 11**.

The turning movement volumes shown in **Figure 7** through **Figure 11** include the counts that were taken at select study intersections, and estimated turning movement volumes for those intersections where approach volumes were measured, but turning movement counts were not taken.

The estimated turning movement volumes were determined using the daily directional distribution for each leg of the intersection, and the hourly percentages of daily traffic for each roadway segment arriving during the peak hour (k-factor). The turning movement volumes were determined through an automated mathematical iteration process. This process assumed turning movement volumes for each approach, and then compared the resulting predicted departing volumes to the departing values calculated by summing the appropriate turning movement volumes. The turning movement percentages that resulted in the lowest value of the sum of the squares of the differences between the departing volumes were selected.

The k-factors that were used in calculating the peak hourly volumes are presented in **Table 1** and **Table 2.** **Table 1** includes the k-factors that were calculated from the 24-hour approach volumes, and the peak hour turning movement counts. **Table 2** includes the k-factors which were estimated for selected study intersections based on the directional split of adjacent roadways, existing traffic patterns, and the adjacent intersections' k-factors. In each case a minimum k-factor of 6%, and a maximum k-factor of 12% were used imposed on the study intersections to eliminate the propagation of extreme peak hour volume estimates.

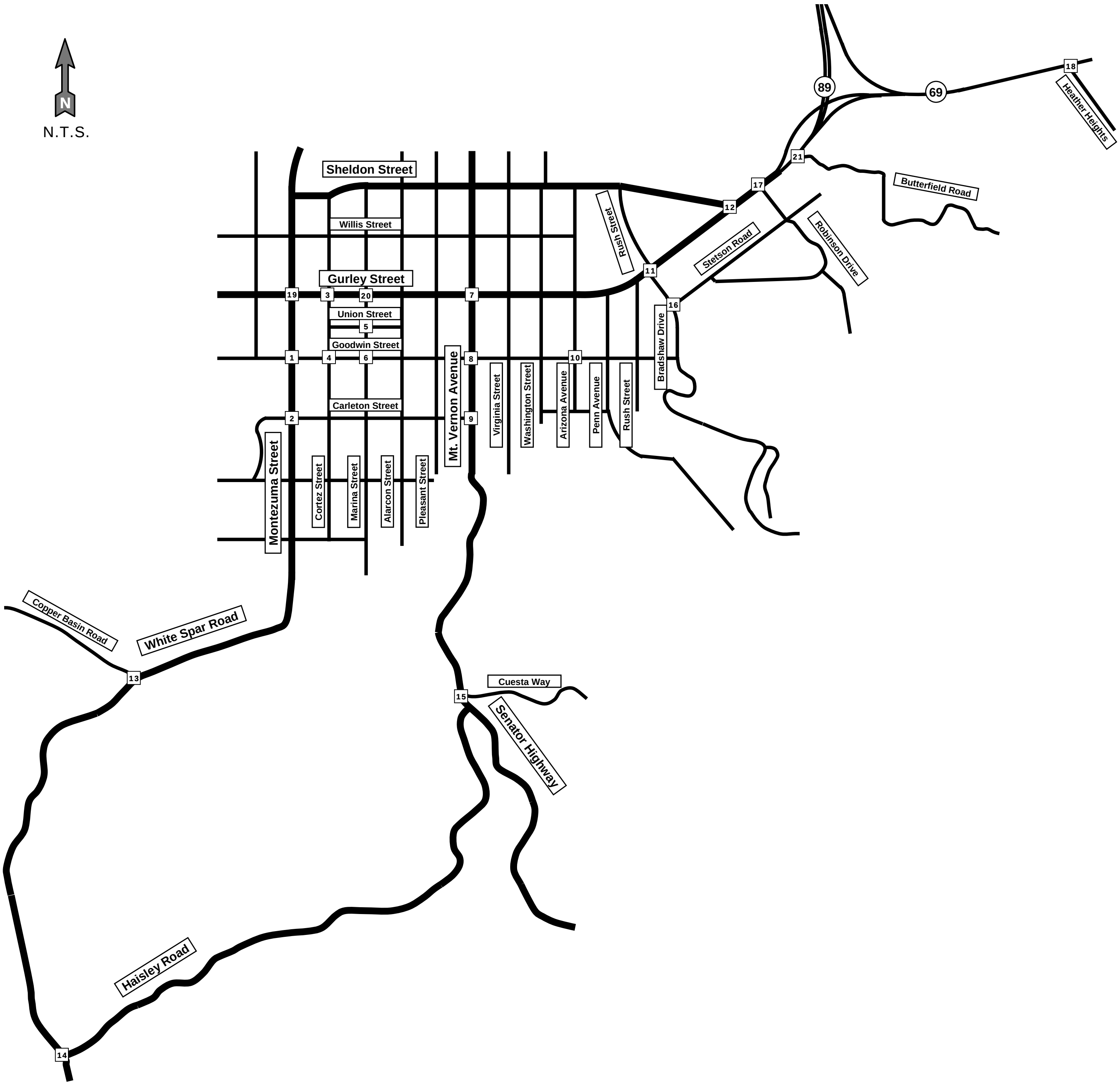

Figure 1: Location of Study Intersections

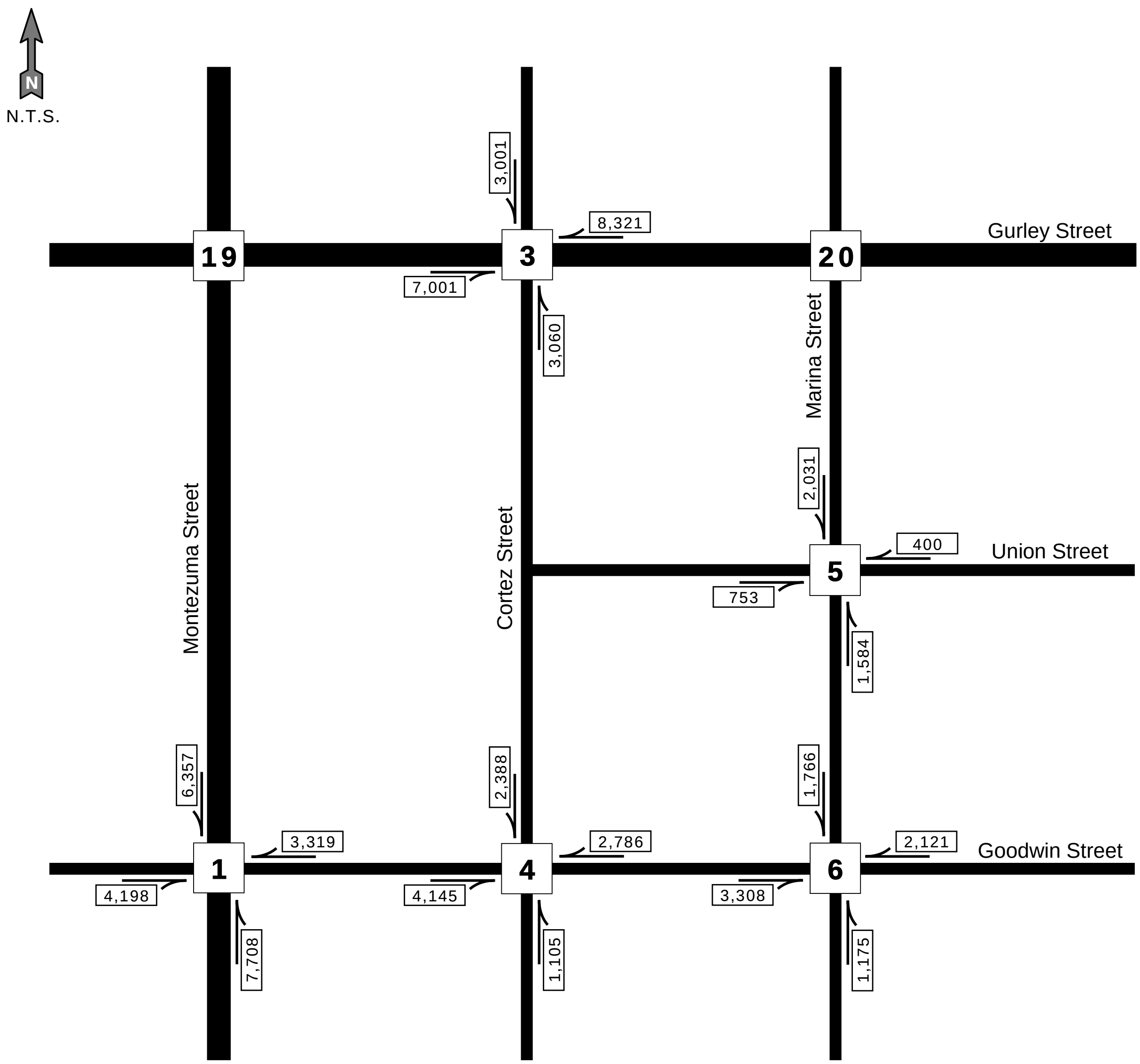

Figure 2: Directional Volumes for Intersections 1, 3, 4, 5, 6, 19, and 20 – Day

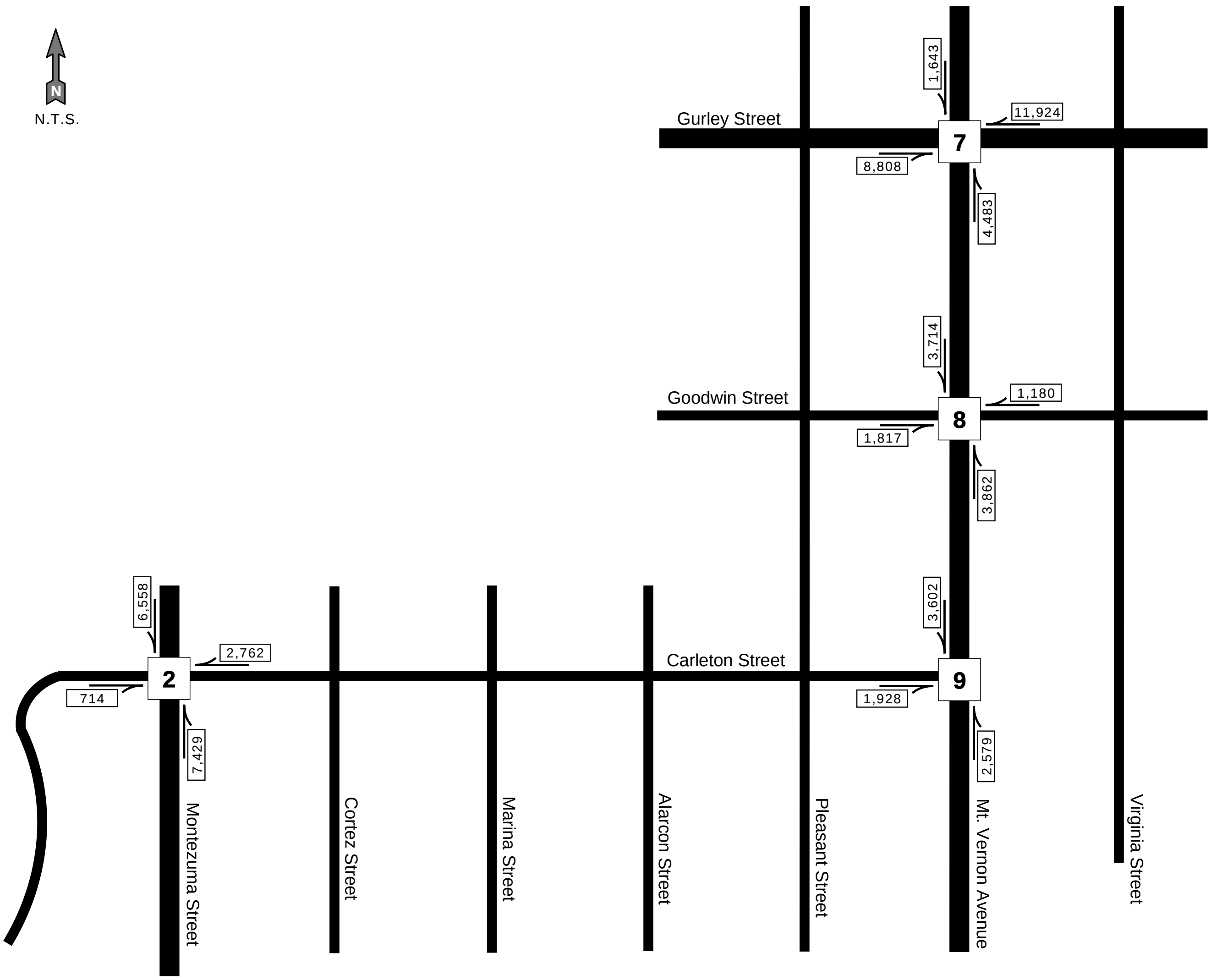

Figure 3: Directional Volumes for Intersections 2, 7, 8, and 9 – Day

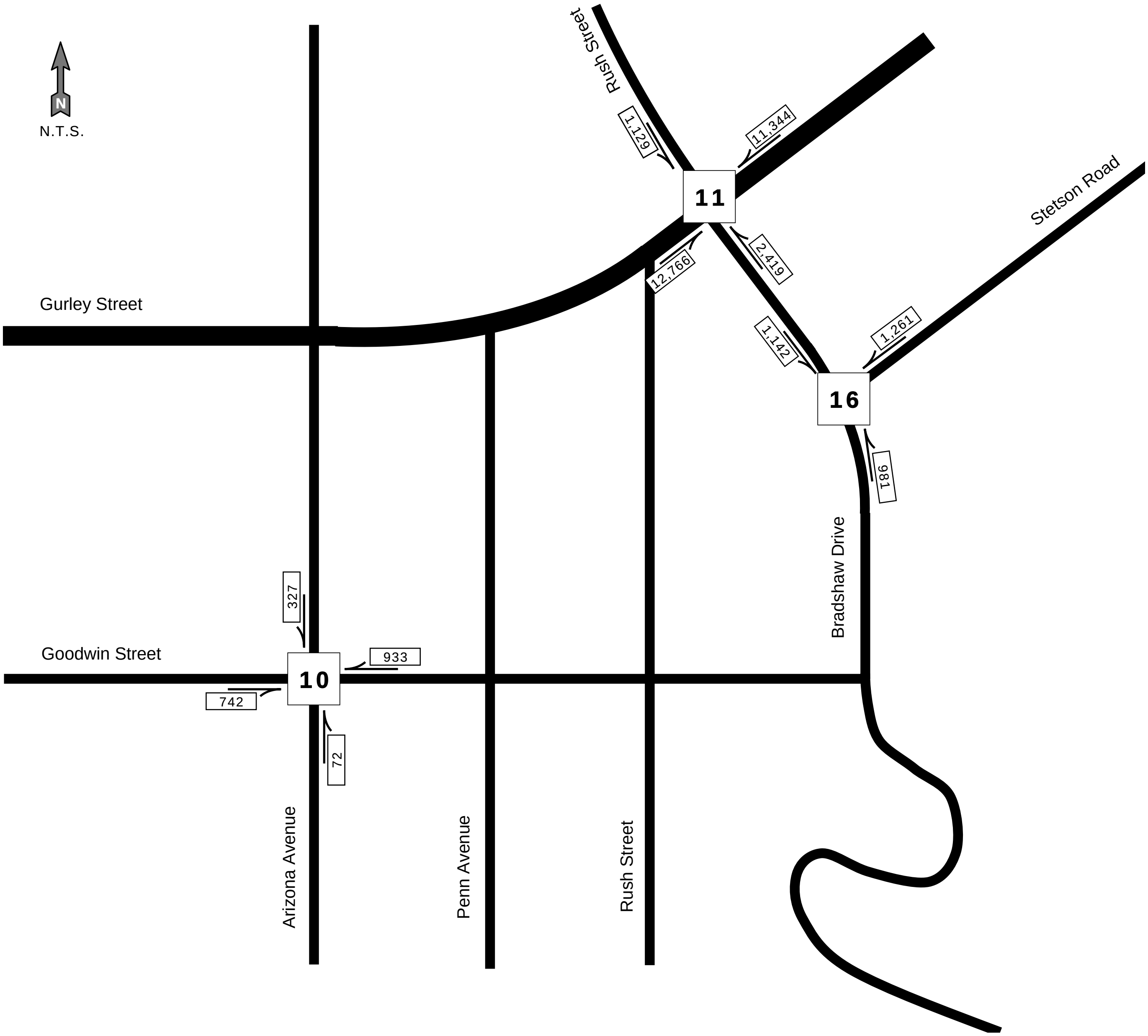

Figure 4: Directional Volumes for Intersections 10, 11, and 16 – Day

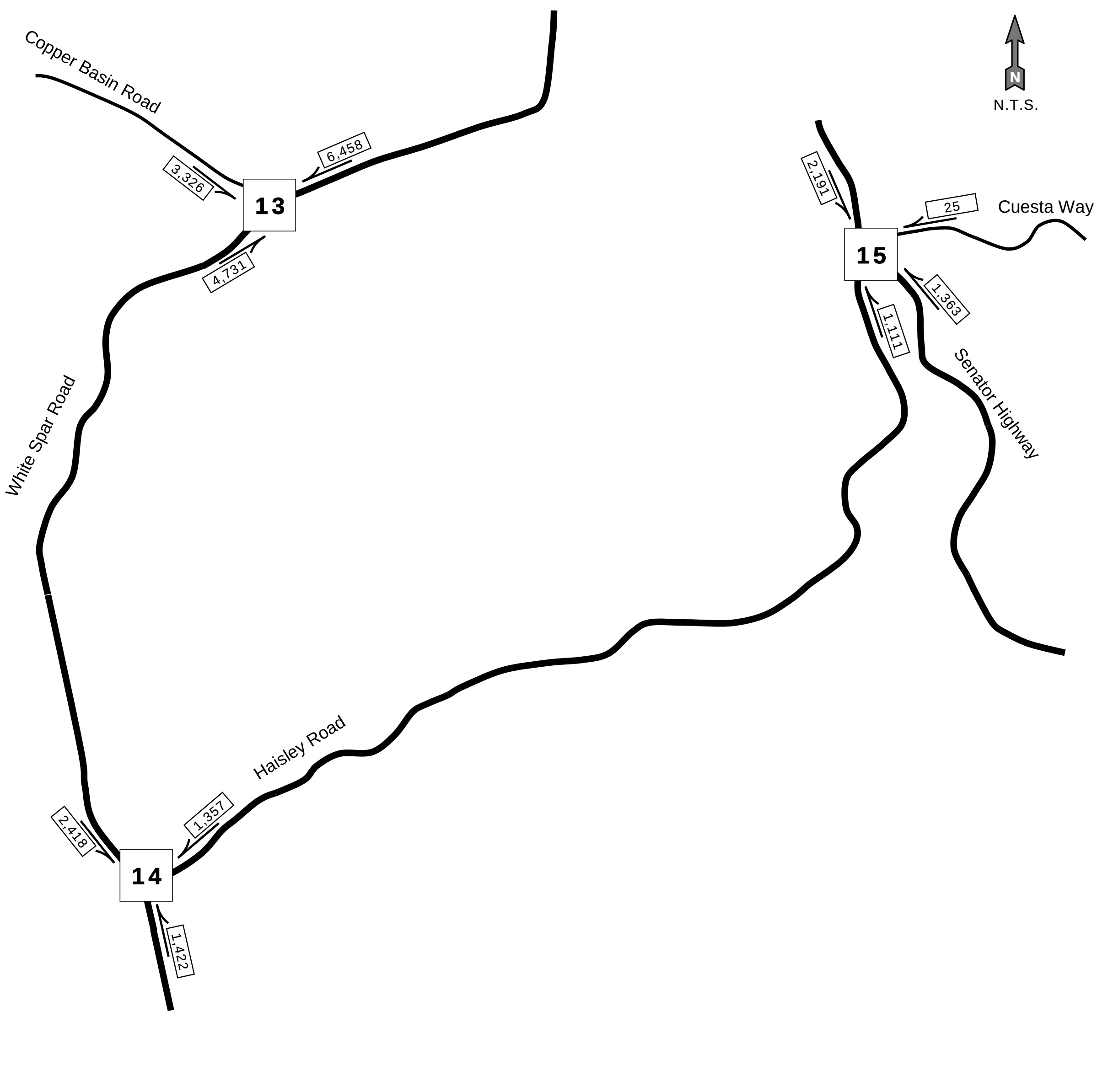

Figure 5: Directional Volumes for Intersections 13, 14, and 15 – Day

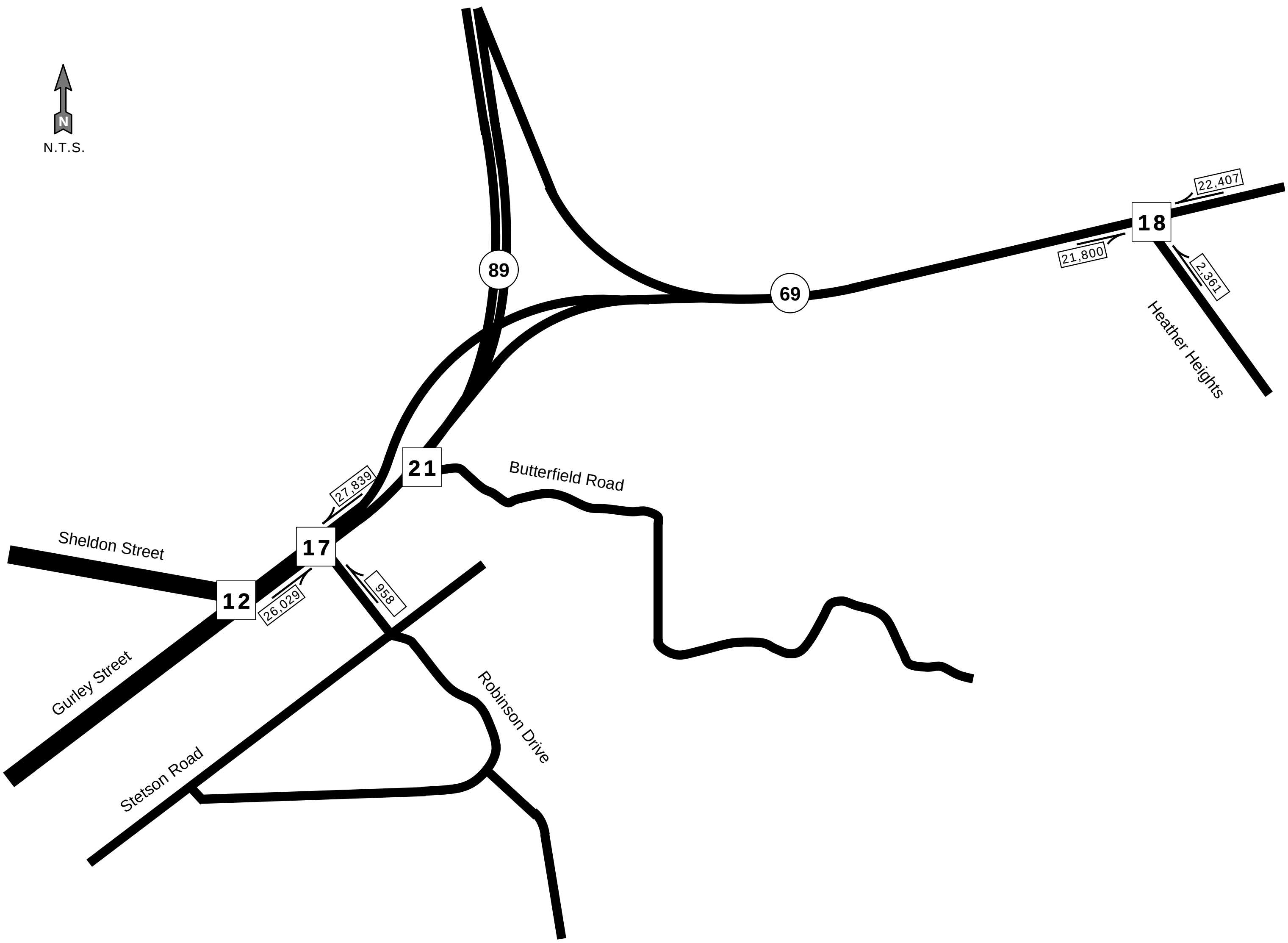

Figure 6: Directional Volumes for Intersections 12, 17, 18 and 21 – Day

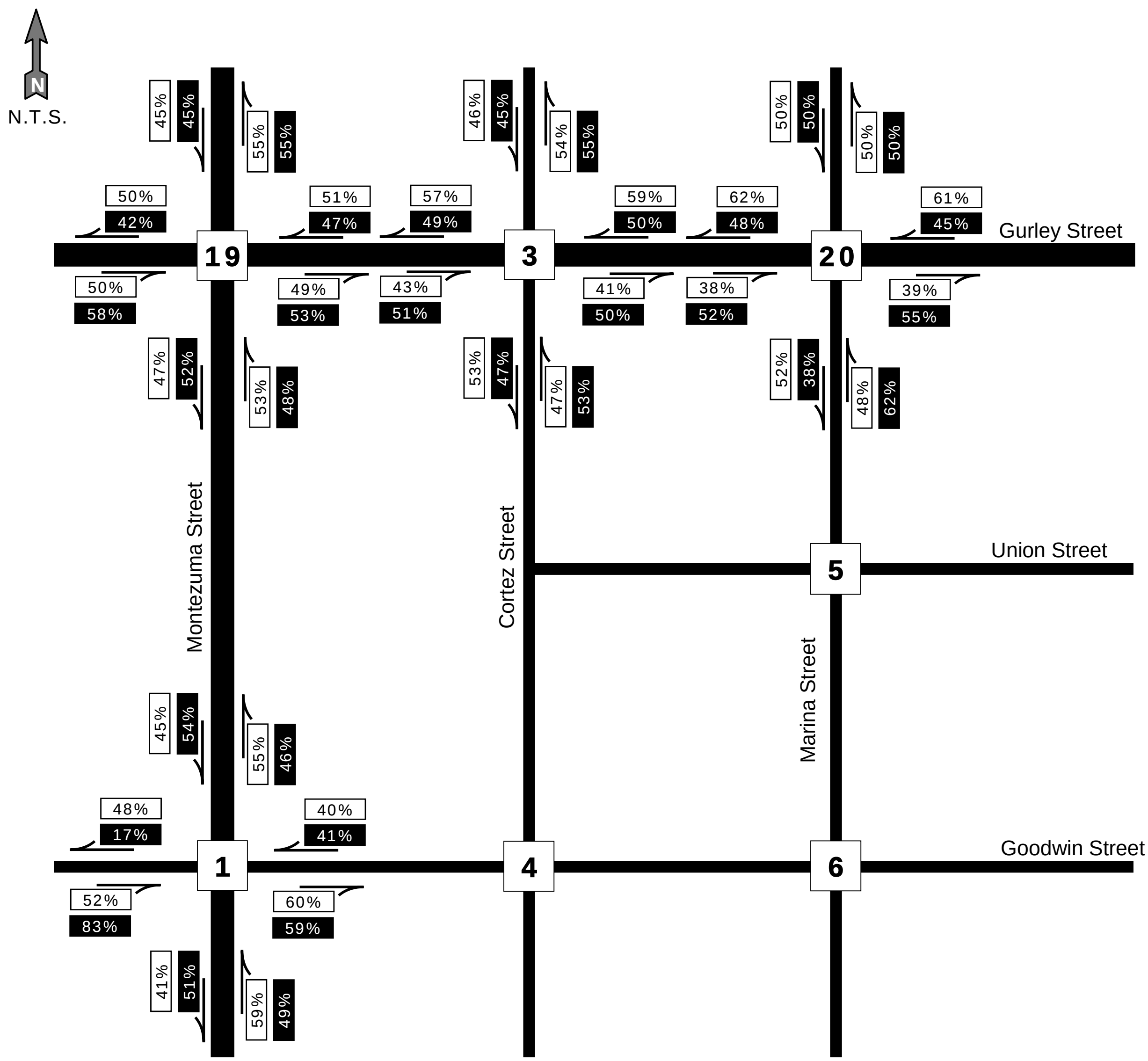

Figure 7: Directional Split for Intersections 1, 3, 4, 5, 6, 19, and 20 – Day

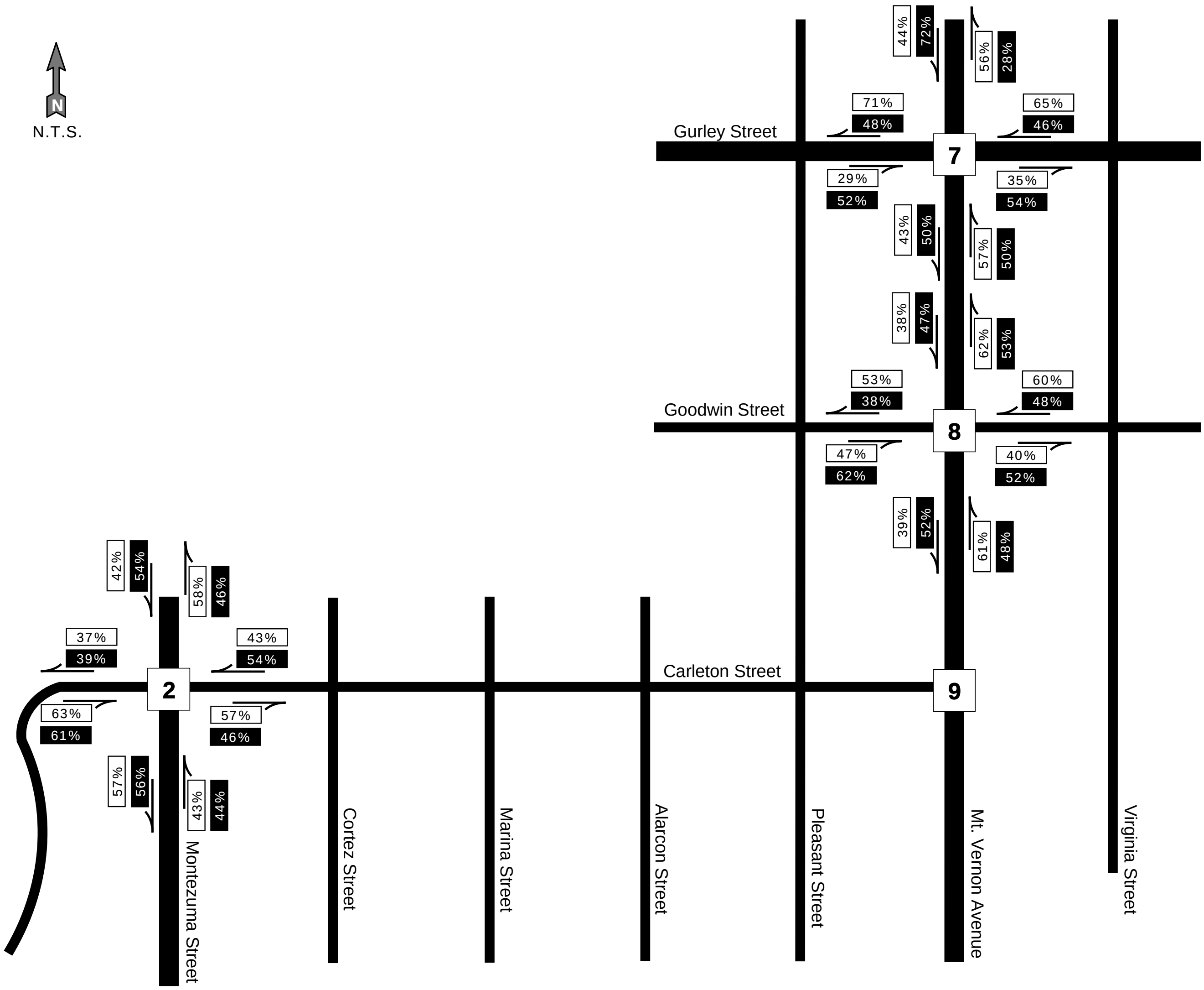

Figure 8: Directional Split for Intersections 2, 7, 8, and 9 – Day

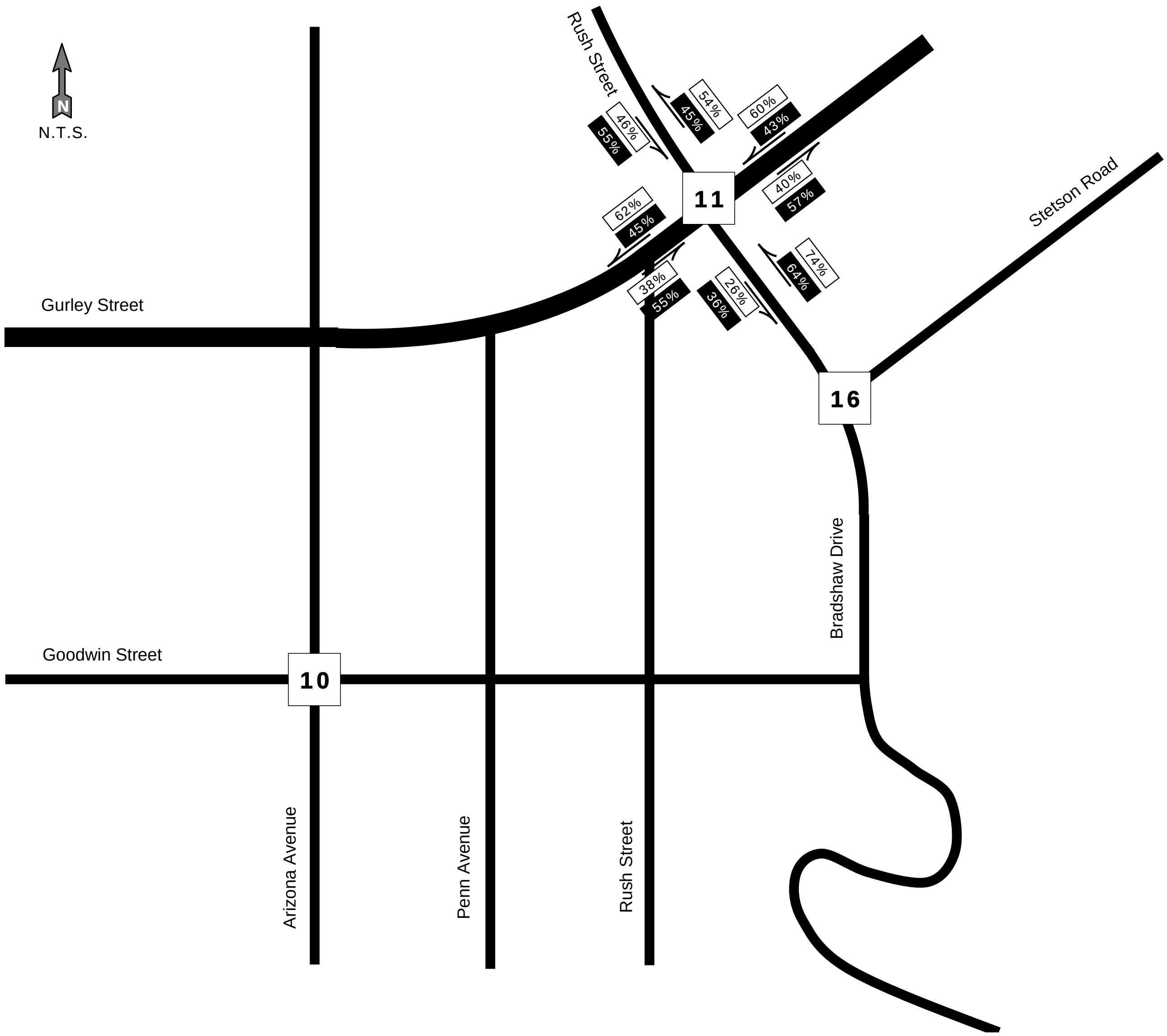

Figure 9: Directional Split for Intersections 10, 11, and 16 – Day

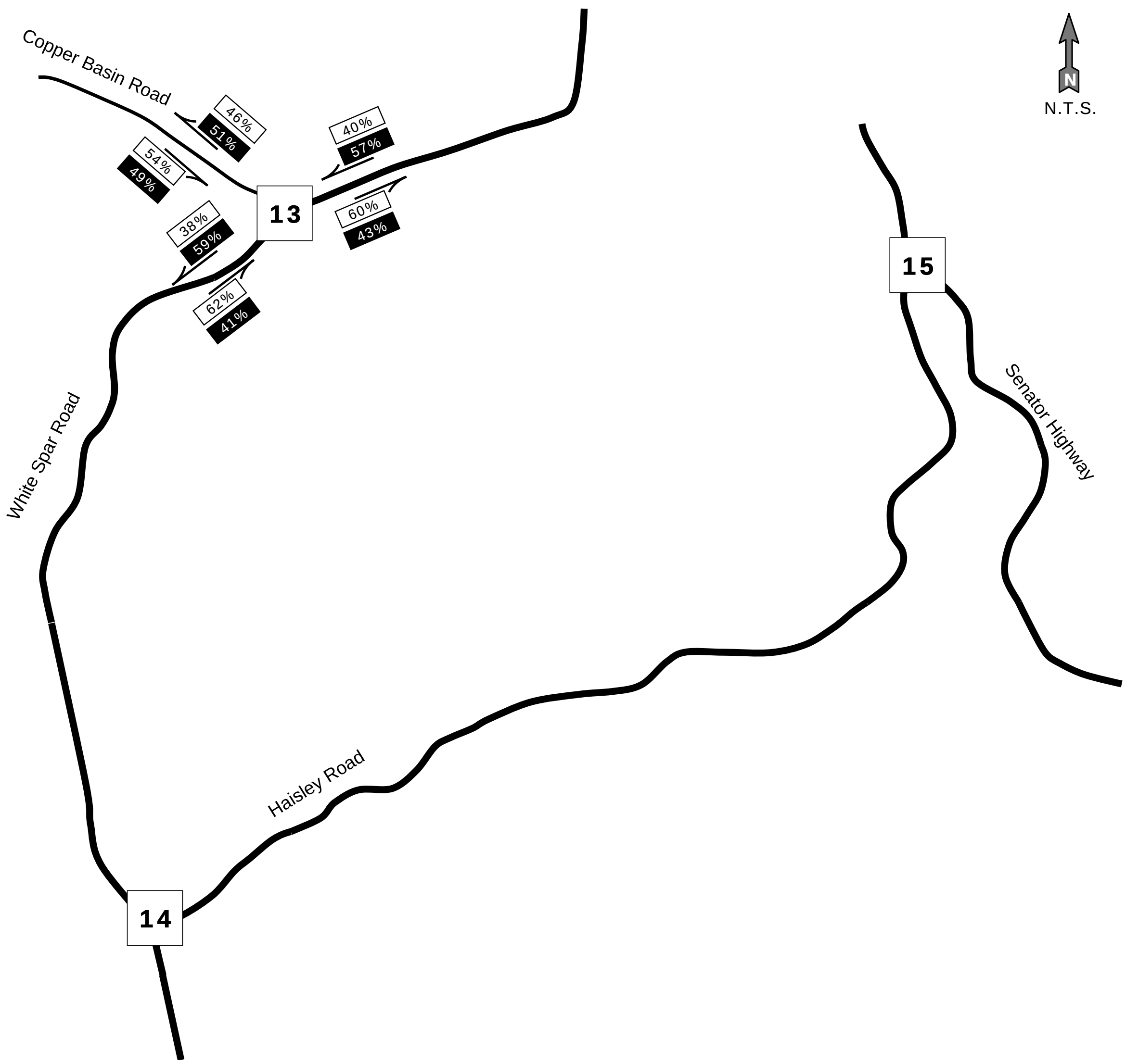

Figure 10: Directional Split for Intersections 13, 14, and 15 – Day

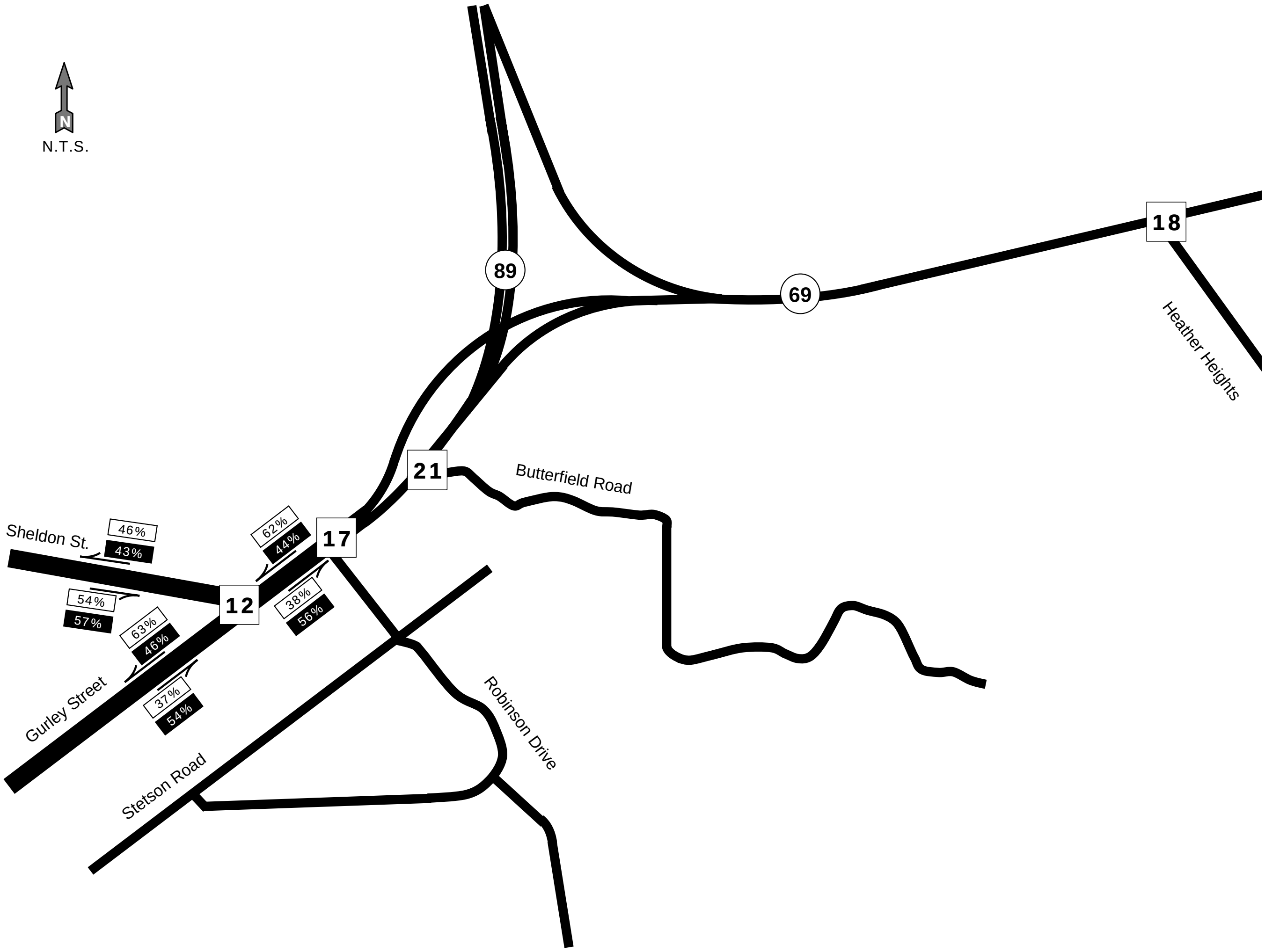

Figure 11: Directional Split for Intersections 12, 17, 18 and 21 – Day

Table 1: Calculated K-Factors

CALCULATED K-FACTORS

ID	STREET	LOCATION	DIRECTION	PORTION OF DAILY TRAFFIC AM Peak Hour	PORTION OF DAILY TRAFFIC PM Peak Hour
17	ROBINSON DRIVE	South of GURLEY STREET	Northbound	7%	6%
	GURLEY STREET	East of ROBINSON DRIVE	Westbound	8%	7%
	GURLEY STREET	West of ROBINSON DRIVE	Eastbound	6%	9%
5	MARINA STREET	North of UNION STREET	Southbound	7%	8%
	MARINA STREET	South of UNION STREET	Northbound	9%	9%
	UNION STREET	East of MARINA STREET	Westbound	12%	12%
	UNION STREET	West of MARINA STREET	Eastbound	10%	11%
16	BRADSHAW DRIVE	North of STETSON ROAD	Southbound	6%	10%
	BRADSHAW DRIVE	South of STETSON ROAD	Northbound	7%	8%
	STETSON ROAD	East of BRADSHAW DRIVE	Westbound	10%	8%
4	CORTEZ STREET	North of GOODWIN STREET	Southbound	6%	8%
	CORTEZ STREET	South of GOODWIN STREET	Northbound	6%	9%
	GOODWIN STREET	East of CORTEZ STREET	Westbound	7%	9%
	GOODWIN STREET	West of CORTEZ STREET	Eastbound	6%	7%
6	MARINA STREET	North of GOODWIN STREET	Southbound	6%	10%
	MARINA STREET	South of GOODWIN STREET	Northbound	6%	9%
	GOODWIN STREET	East of MARINA STREET	Westbound	8%	8%
	GOODWIN STREET	West of MARINA STREET	Eastbound	7%	9%
10	ARIZONA AVENUE	North of GOODWIN STREET	Southbound	10%	10%
	ARIZONA AVENUE	South of GOODWIN STREET	Northbound	6%	8%
	GOODWIN STREET	East of ARIZONA AVENUE	Westbound	9%	9%
	GOODWIN STREET	West of ARIZONA AVENUE	Eastbound	6%	10%
9	MT. VERNON AVENUE	North of CARLETON STREET	Southbound	6%	9%
	MT. VERNON AVENUE	South of CARLETON STREET	Northbound	9%	7%
	CARLETON STREET	West of MT. VERNON AVENUE	Eastbound	8%	10%
14	WHITE SPAR ROAD	North of HAISLEY ROAD	Southbound	6%	11%
	WHITE SPAR ROAD	South of HAISLEY ROAD	Northbound	11%	8%
	HAISLEY ROAD	East of WHITE SPAR ROAD	Westbound	8%	8%
15	SENATOR HIGHWAY	North of CUESTA WAY	Southbound	6%	10%
	SENATOR HIGHWAY	South of HAISLEY ROAD	Northbound	11%	6%
	CUESTA WAY	East of SENATOR HIGHWAY	Westbound	6%	12%
	HAISLEY ROAD	West of SENATOR HIGHWAY	Eastbound	9%	7%

Table 2: Estimated K-Factors

ESTIMATED K-FACTORS					
				PORTION OF DAILY TRAFFIC	
ID	STREET	LOCATION	DIRECTION	AM Peak Hour	PM Peak Hour
17	ROBINSON DRIVE	South of GURLEY STREET	Southbound	6%	10%
	GURLEY STREET	East of ROBINSON DRIVE	Eastbound	6%	9%
	GURLEY STREET	West of ROBINSON DRIVE	Westbound	8%	7%
5	MARINA STREET	North of UNION STREET	Northbound	9%	9%
	MARINA STREET	South of UNION STREET	Southbound	7%	8%
	UNION STREET	East of MARINA STREET	Eastbound	10%	11%
	UNION STREET	West of MARINA STREET	Westbound	12%	12%
16	BRADSHAW DRIVE	North of STETSON ROAD	Northbound	8%	8%
	BRADSHAW DRIVE	South of STETSON ROAD	Southbound	6%	10%
	STETSON ROAD	East of BRADSHAW DRIVE	Eastbound	8%	10%
4	CORTEZ STREET	North of GOODWIN STREET	Northbound	6%	9%
	CORTEZ STREET	South of GOODWIN STREET	Southbound	6%	8%
	GOODWIN STREET	East of CORTEZ STREET	Eastbound	6%	7%
	GOODWIN STREET	West of CORTEZ STREET	Westbound	7%	9%
6	MARINA STREET	North of GOODWIN STREET	Northbound	7%	7%
	MARINA STREET	South of GOODWIN STREET	Southbound	6%	10%
	GOODWIN STREET	East of MARINA STREET	Eastbound	6%	11%
	GOODWIN STREET	West of MARINA STREET	Westbound	7%	9%
10	ARIZONA AVENUE	North of GOODWIN STREET	Northbound	7%	8%
	ARIZONA AVENUE	South of GOODWIN STREET	Southbound	6%	10%
	GOODWIN STREET	East of ARIZONA AVENUE	Eastbound	6%	10%
	GOODWIN STREET	West of ARIZONA AVENUE	Westbound	11%	8%
9	MT. VERNON AVENUE	North of CARLETON STREET	Northbound	9%	7%
	MT. VERNON AVENUE	South of CARLETON STREET	Southbound	6%	9%
	CARLETON STREET	West of MT. VERNON AVENUE	Westbound	8%	9%
14	WHITE SPAR ROAD	North of HAISLEY ROAD	Northbound	8%	8%
	WHITE SPAR ROAD	South of HAISLEY ROAD	Southbound	6%	11%
	HAISLEY ROAD	East of WHITE SPAR ROAD	Eastbound	9%	7%
15	SENATOR HIGHWAY	North of CUESTA WAY	Northbound	9%	7%
	SENATOR HIGHWAY	South of HAISLEY ROAD	Southbound	6%	10%
	CUESTA WAY	East of SENATOR HIGHWAY	Eastbound	6%	12%
	HAISLEY ROAD	West of SENATOR HIGHWAY	Westbound	8%	8%

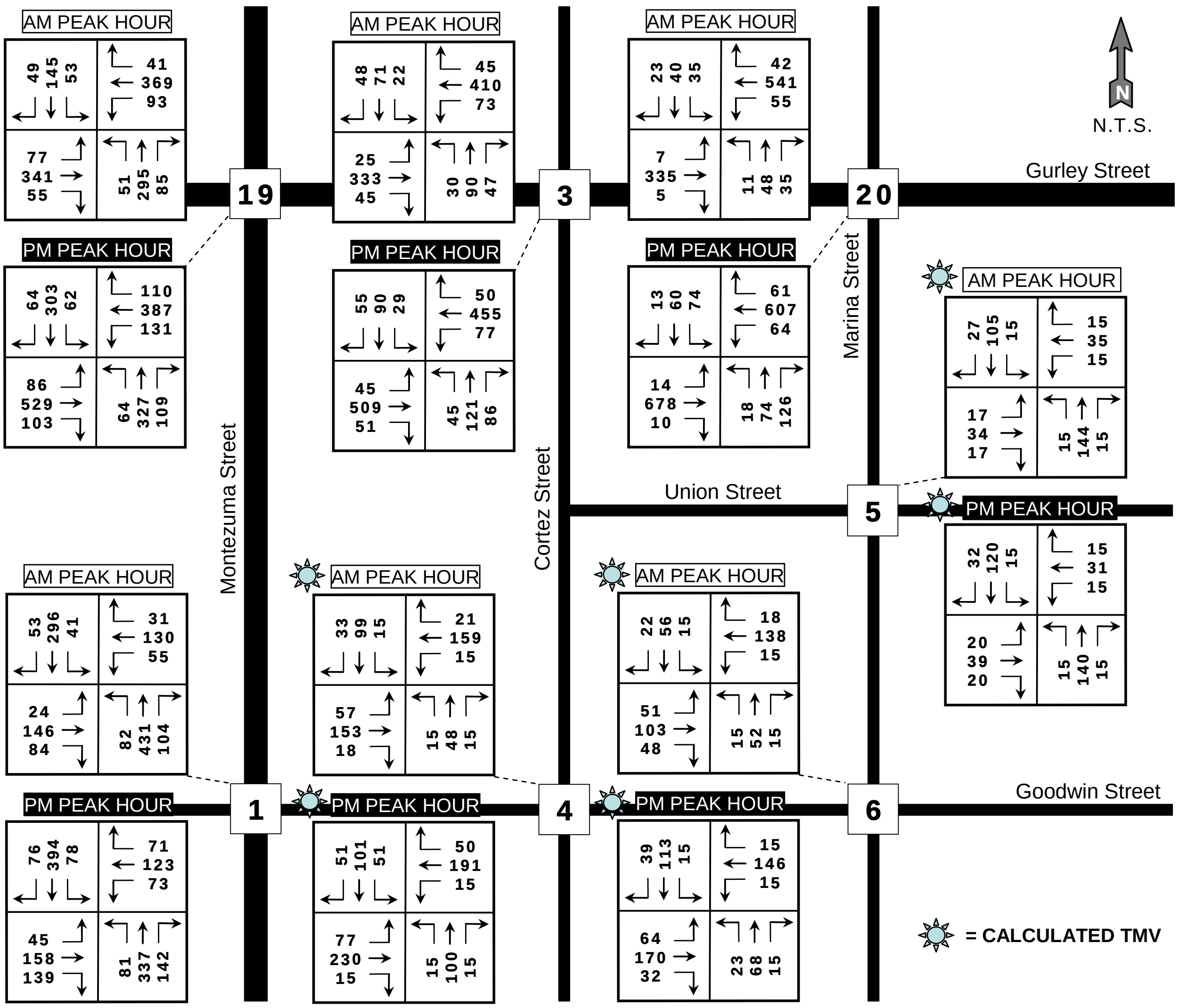

Figure 12: TMC for Intersections 1, 3, 4, 5, 6, 19, and 20 – AM and PM Peak Hours

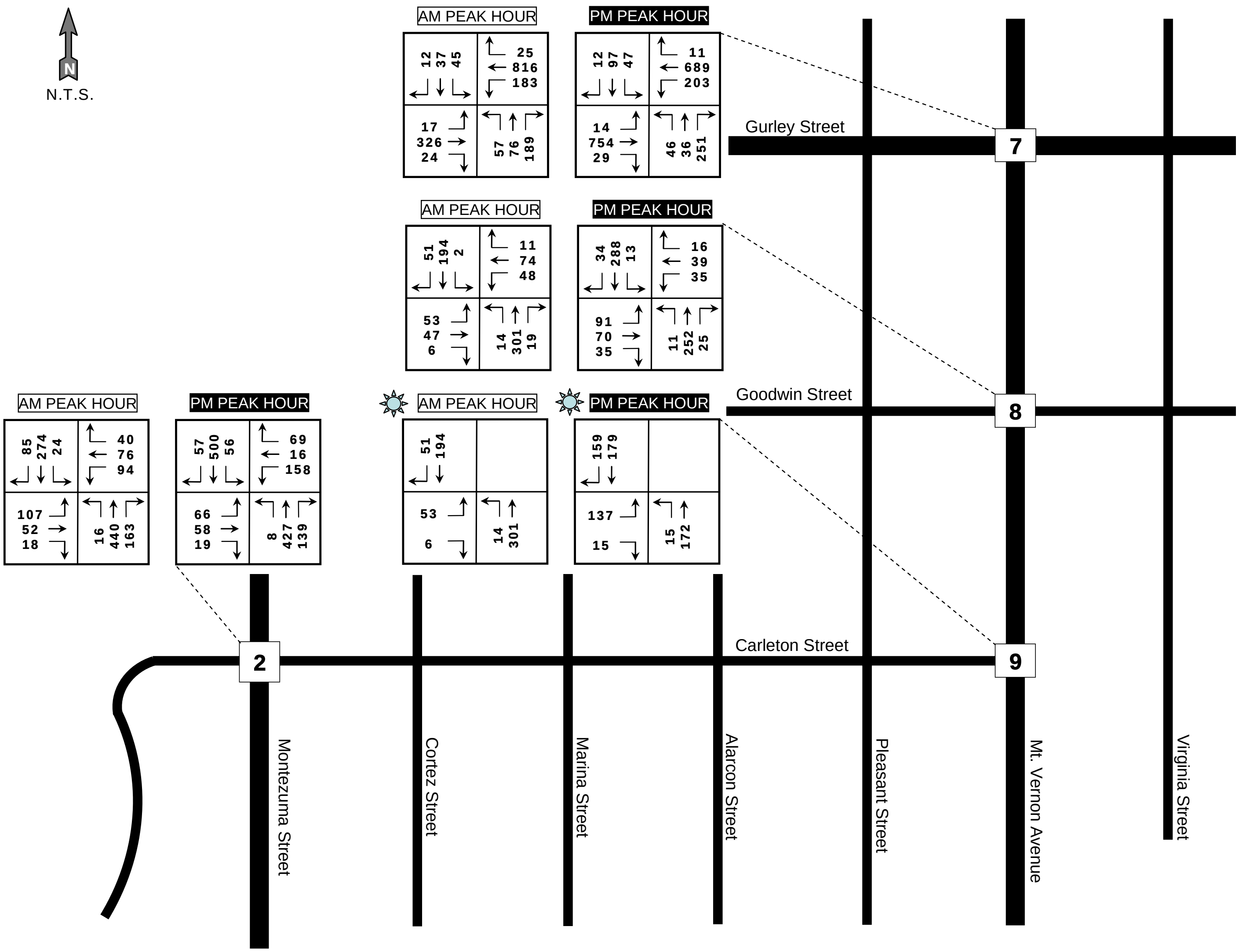

Figure 13: TMC for Intersections 2, 7, 8, and 9 – AM and PM Peak Hours

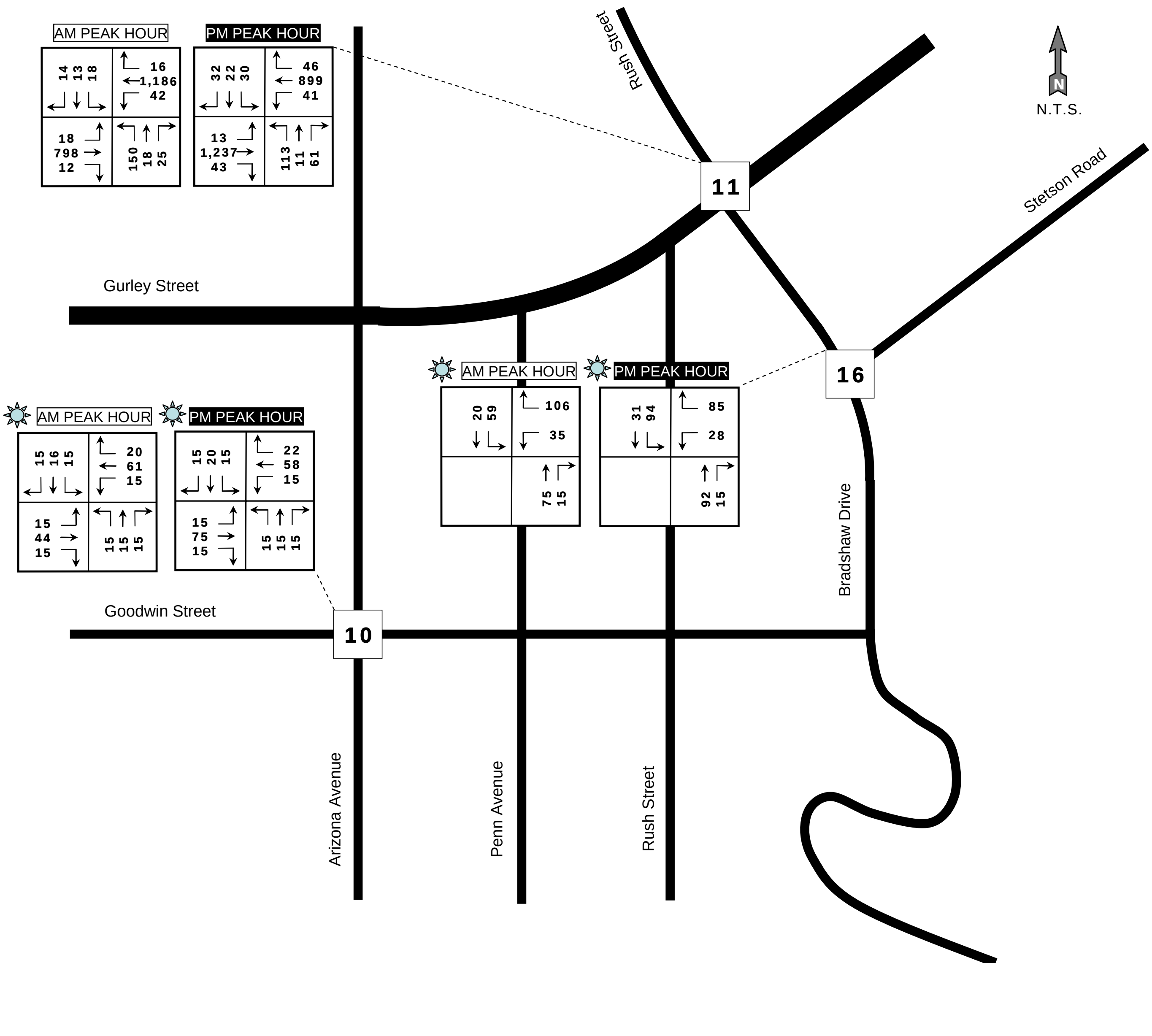

Figure 14: TMC for Intersections 10, 11, and 16 – AM and PM Peak Hours

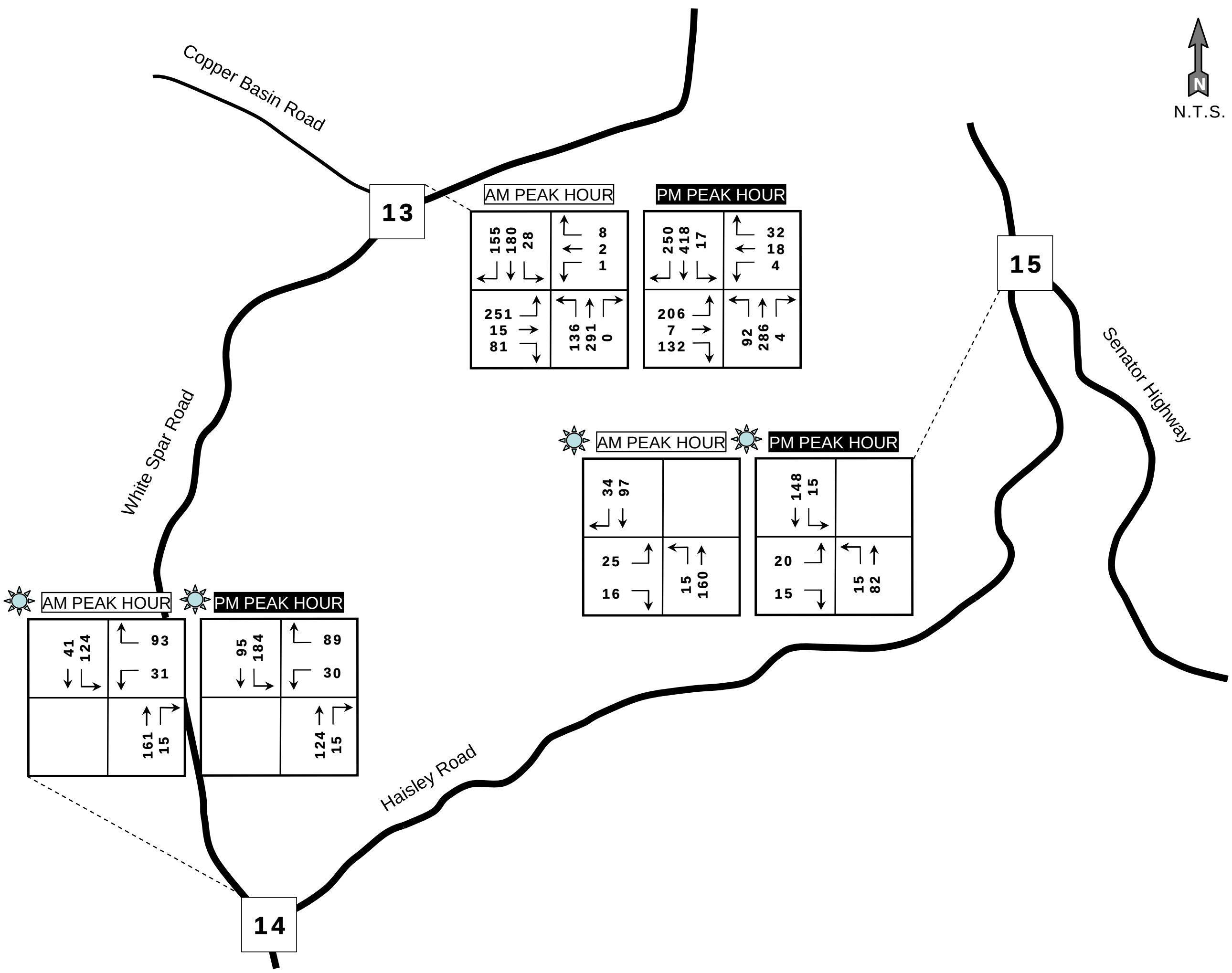

Figure 15: TMC for Intersections 13, 14, and 15 – AM and PM Peak Hours

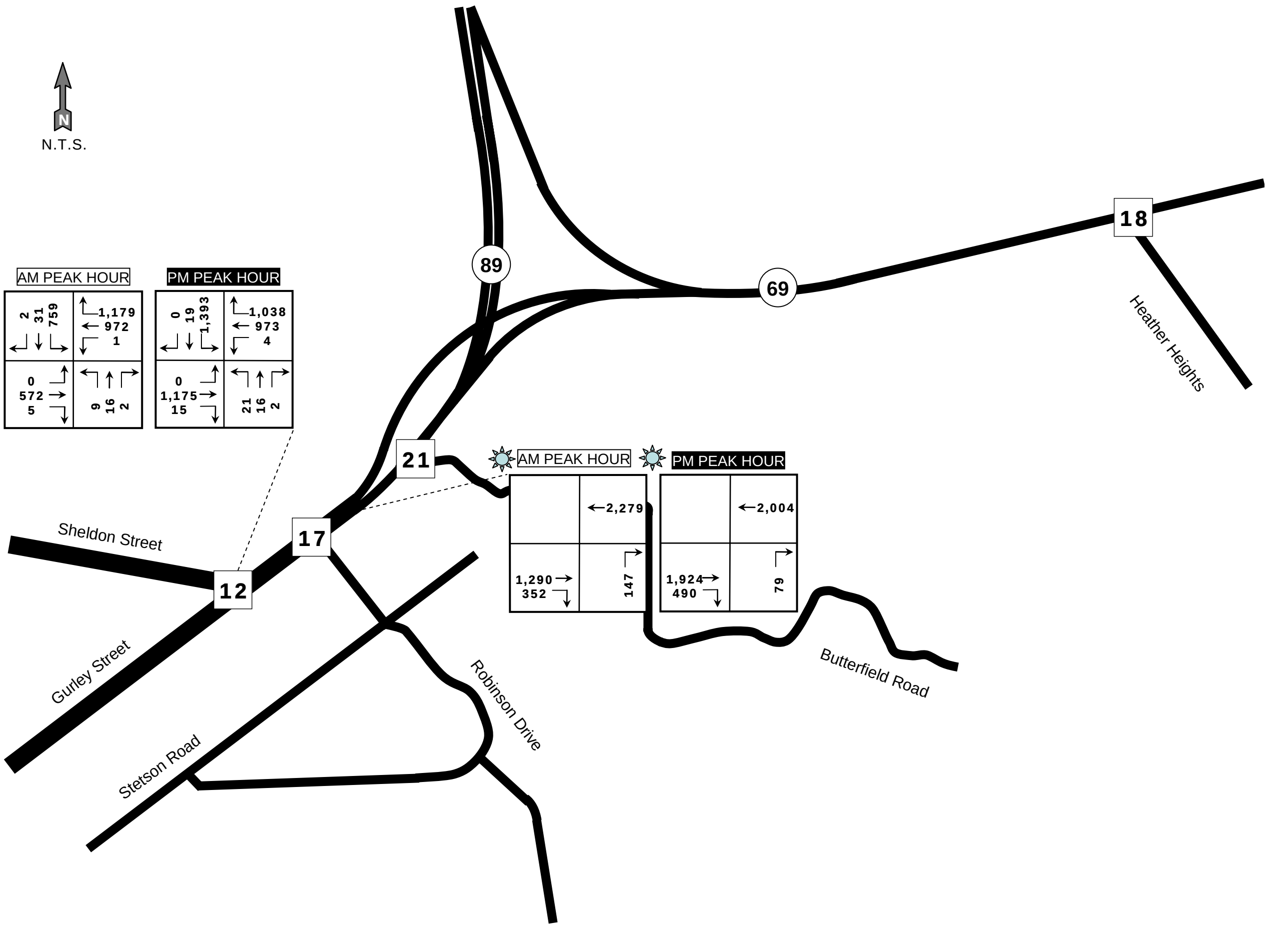

Figure 16: TMC for Intersections 12, 17, 18, and 21 – AM and PM Peak Hours

Level of Service

The existing 2007 conditions at the ten study intersections were analyzed to determine the level-of-service for each intersection. The ability of a transportation system to transmit the transportation demand is characterized as its level-of-service (LOS). Level-of-service is a rating system from “A”, representing the best operation, to “F”, representing the worst operation. Typically, level-of-service “D” is considered the minimum acceptable operation. The appropriate reference for level-of-service operation is the *Highway Capacity Manual*, published by the Transportation Research Board.

This manual considers the average delay per vehicle as the measure to determine the level-of-service of a signalized intersection. The delay and level-of-service are calculated for the intersection, each approach, and each turning movement. For unsignalized intersections the level-of-service is defined for each minor movement for two-way stop controls, and is not defined for the intersection as a whole. For all-way stop controls, level-of-service is defined for the intersection, each approach, and for each turning movement. **Table 3** lists the level-of-service criteria for signalized and unsignalized intersections as stated in the *Highway Capacity Manual*.

Table 3: Level-of-Service Criteria for Signalized Intersections

LEVEL-OF-SERVICE	AVERAGE DELAY (seconds/vehicle)	
	SIGNALIZED	UNSIGNALIZED
A	≤ 10	≤ 10
B	> 10 to 20	> 10 to 15
C	> 20 to 35	> 15 to 25
D	> 35 to 55	> 25 to 35
E	> 55 to 80	> 35 to 50
F	> 80	> 50

To analyze the study area, Intersection Signal Timing information was obtained for each of the study intersections from Ian Mattingly of the City of Prescott Transportation Services Division on 9 April 2007 and is included as **Appendix A**. Synchro Software was then used, utilizing the current turning movements at each study intersection and corresponding signal timing, to determine the delay and level-of-service for each of the study intersections. The results from these analyses are provided in tables, figures, and graphs. **Table 4** provides the level of service for each counted intersection for both the morning and evening peak periods. **Table 5** provides the level of service for the remaining study intersection for both the morning and evening peak periods. The level-of-service and intersection location are provided for the counted intersections in **Figure 17** through **Figure 21**. The delay and level-of-service for the study intersections, for each 15-minute interval in which count data was available, are provided in graphical form in **Figure 22** through **Figure 31**.

These analyses reveal that, with the exception of the intersections of White Spar/ Copper Basin, Sheldon/Gurley and Bradshaw/Gurley, the study intersections are operating at acceptable levels-of-service. Furthermore, the levels-of-service at which these signalized intersections operate remain relatively constant throughout the day. However, the delay for the intersection of Sheldon Street and Gurley Street appears to be much higher during the evening peak hour than during the morning peak hour as indicated in **Figure 22**.

Table 4: Morning and Evening Peak Hour Level-of-Service – Counted Intersections

INTERSECTION	PEAK HOUR LEVEL OF SERVICE			
	MORNING		EVENING	
Montezuma Street & Goodwin Street	7:30 AM - 8:30 AM	B	4:15 PM - 5:15 PM	B
Montezuma Street & Carelton Street	7:30 AM - 8:30 AM	B	3:00 PM - 4:00 PM	B
Cortez Street & Gurley Street	9:00 AM - 10:00AM	B	4:15 PM - 5:15 PM	B
Mount Vernon Avenue & Gurley Street	7:30 AM - 8:30 AM	A	4:15 PM - 5:15 PM	A
Mount Vernon Avenue & Goodwin Street	7:30 AM - 8:30 AM	A	4:30 PM - 5:30 PM	A
White Spar Road & Copper Basin Road	7:45 AM - 8:45 AM	F	4:15 PM - 5:15 PM	F
Montezuma Street & Gurley Street	8:00 AM - 9:00 AM	B	4:15 PM - 5:15 PM	C
Sheldon Street & Gurley Street	7:30 AM - 8:30 AM	F	4:30 PM - 5:30 PM	F
Bradshaw Street & Gurley Street	7:30 AM - 8:30 AM	F	4:30 PM - 5:30 PM	F
Marina Street & Gurley Street	8:00 AM - 9:00 AM	B	4:30 PM - 5:30 PM	B

Table 5: Morning and Evening Peak Hour Level-of-Service – Estimated Hourly Volumes

INTERSECTION	PEAK HOUR LEVEL OF SERVICE	
	MORNING	EVENING
Cortez Street & Goodwin Street	B	B
Marina Street & Goodwin Steet	A	B
Arizona Avenue & Goodwin Street	A	A
White Spar Road & Haisley Road	A	B

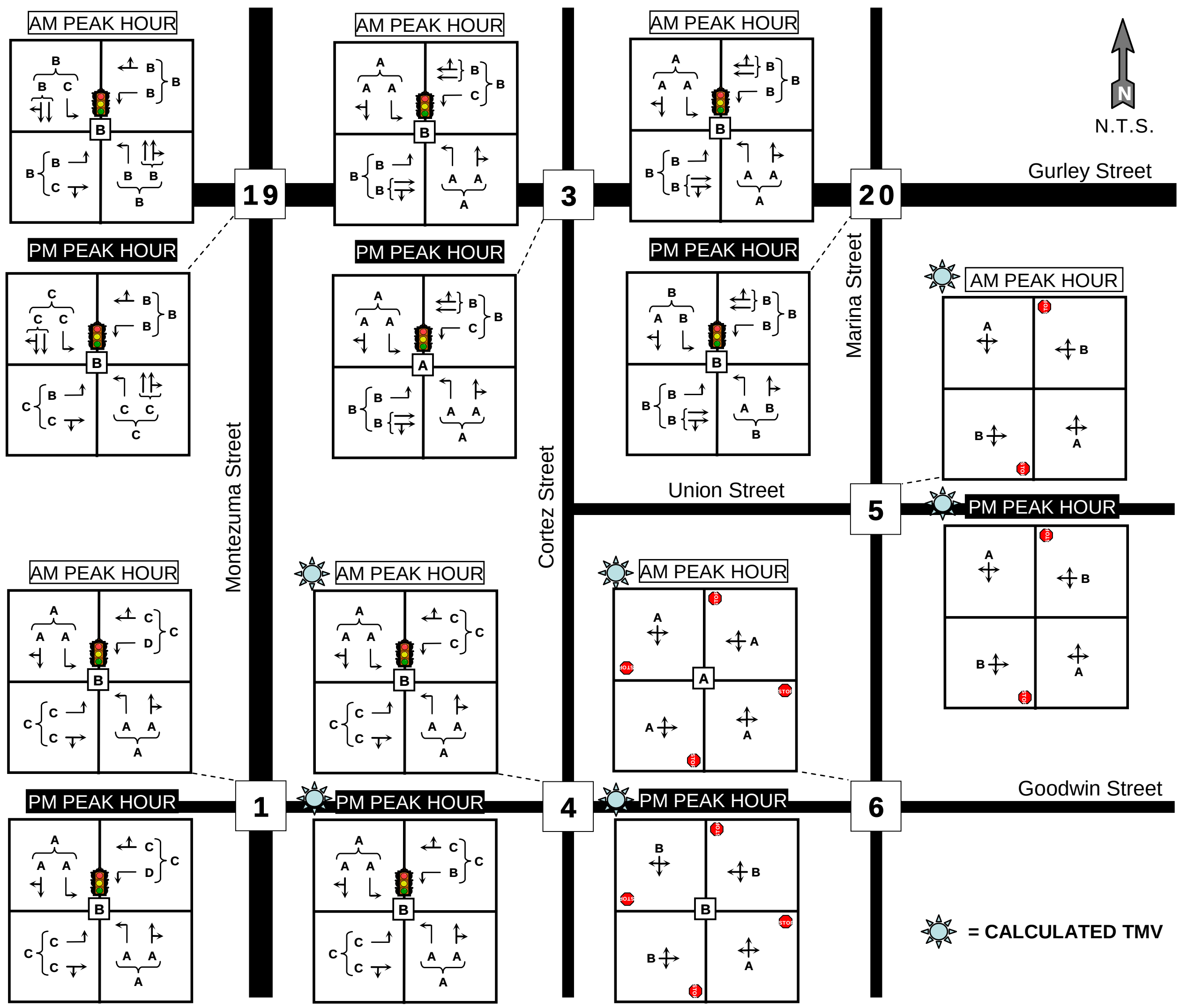

Figure 17: 2007 LOS for intersections 1, 3, 4, 5, 6, 19, and 20 – AM and PM Peaks

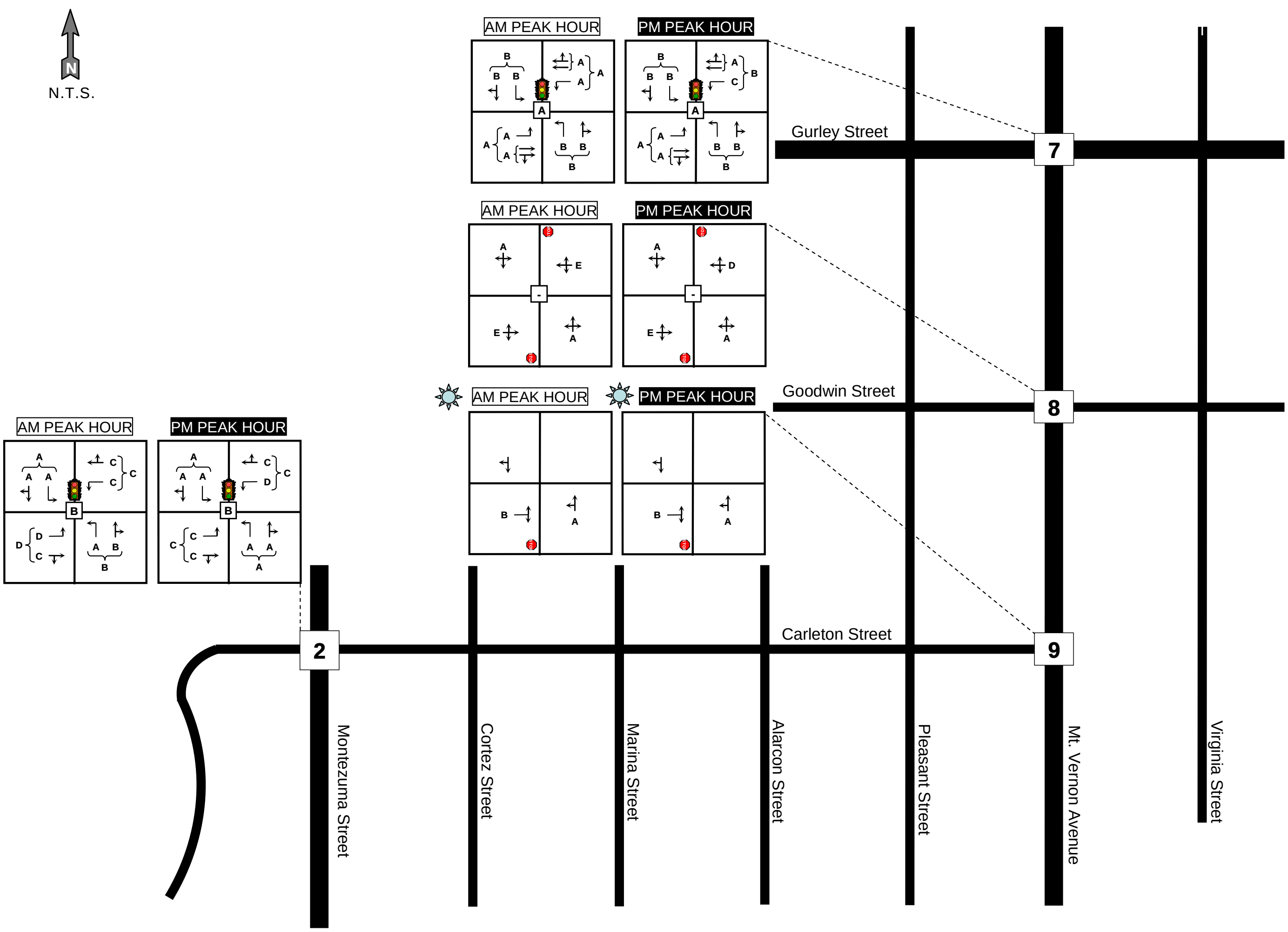

Figure 18: 2007 LOS for intersections 2, 7, 8, and 9 – AM and PM Peaks

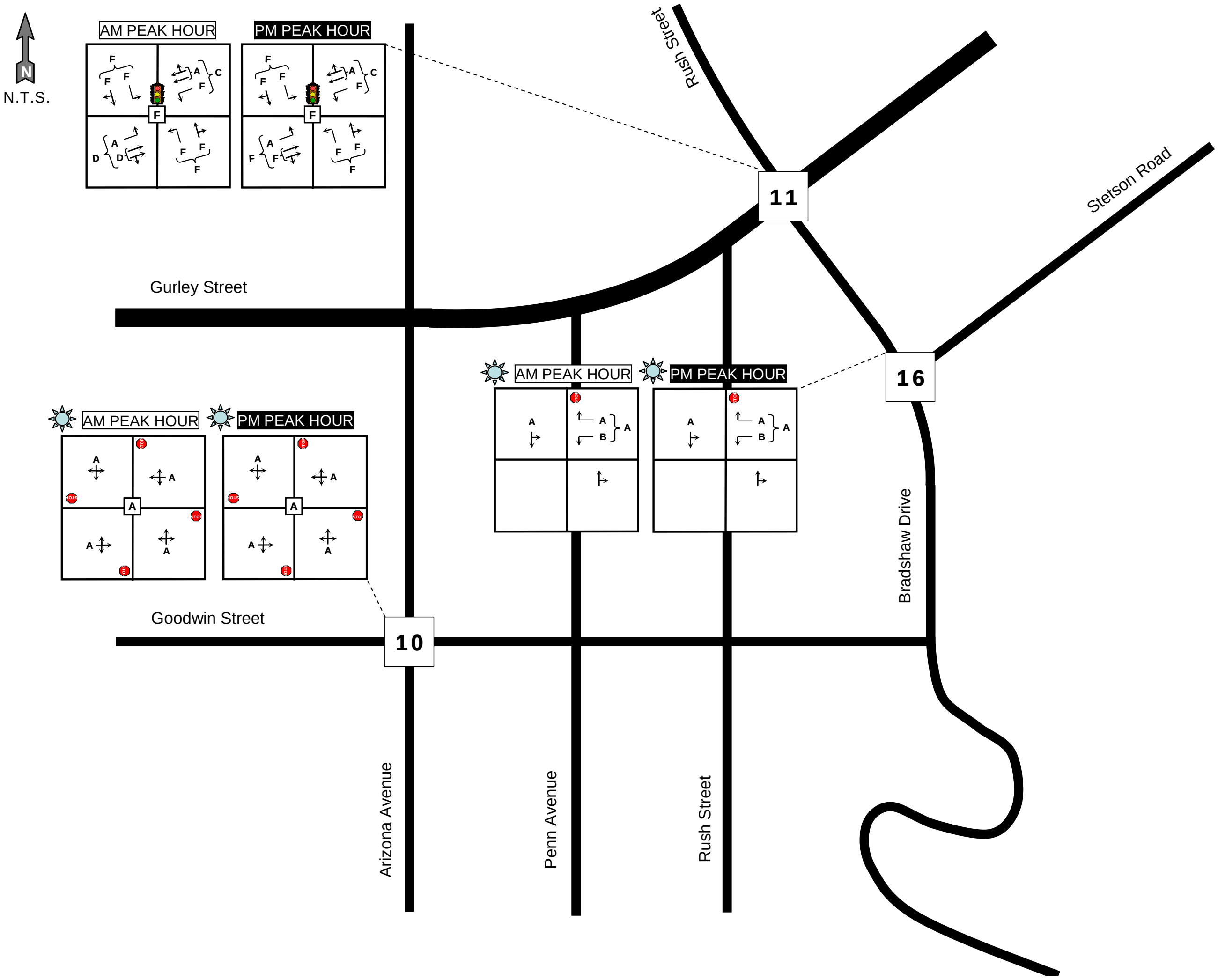

Figure 19: 2007 LOS for intersections 10, 11, and 16 – AM and PM Peaks

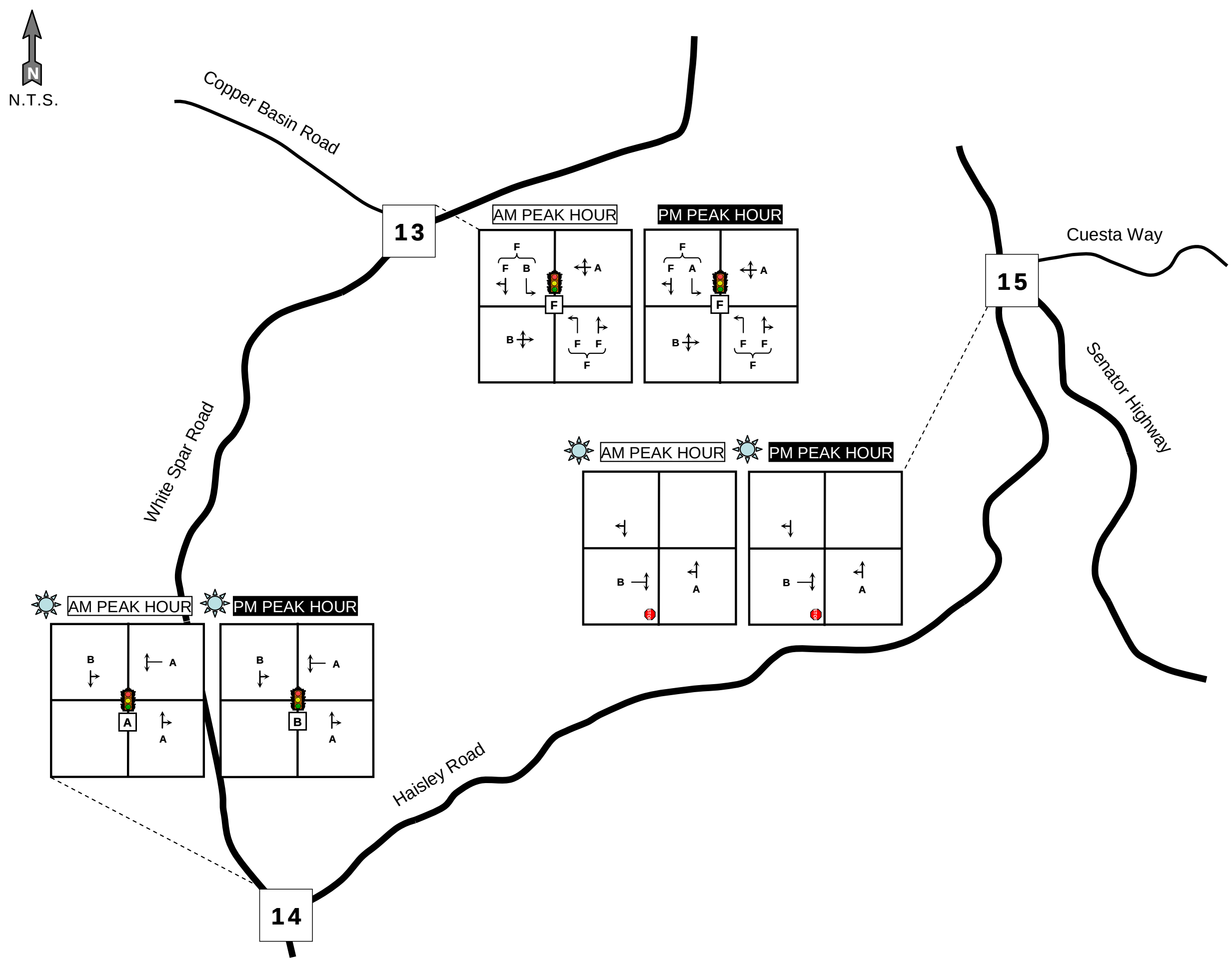

Figure 20: 2007 LOS for intersections 13, 14, and 15 – AM and PM Peaks

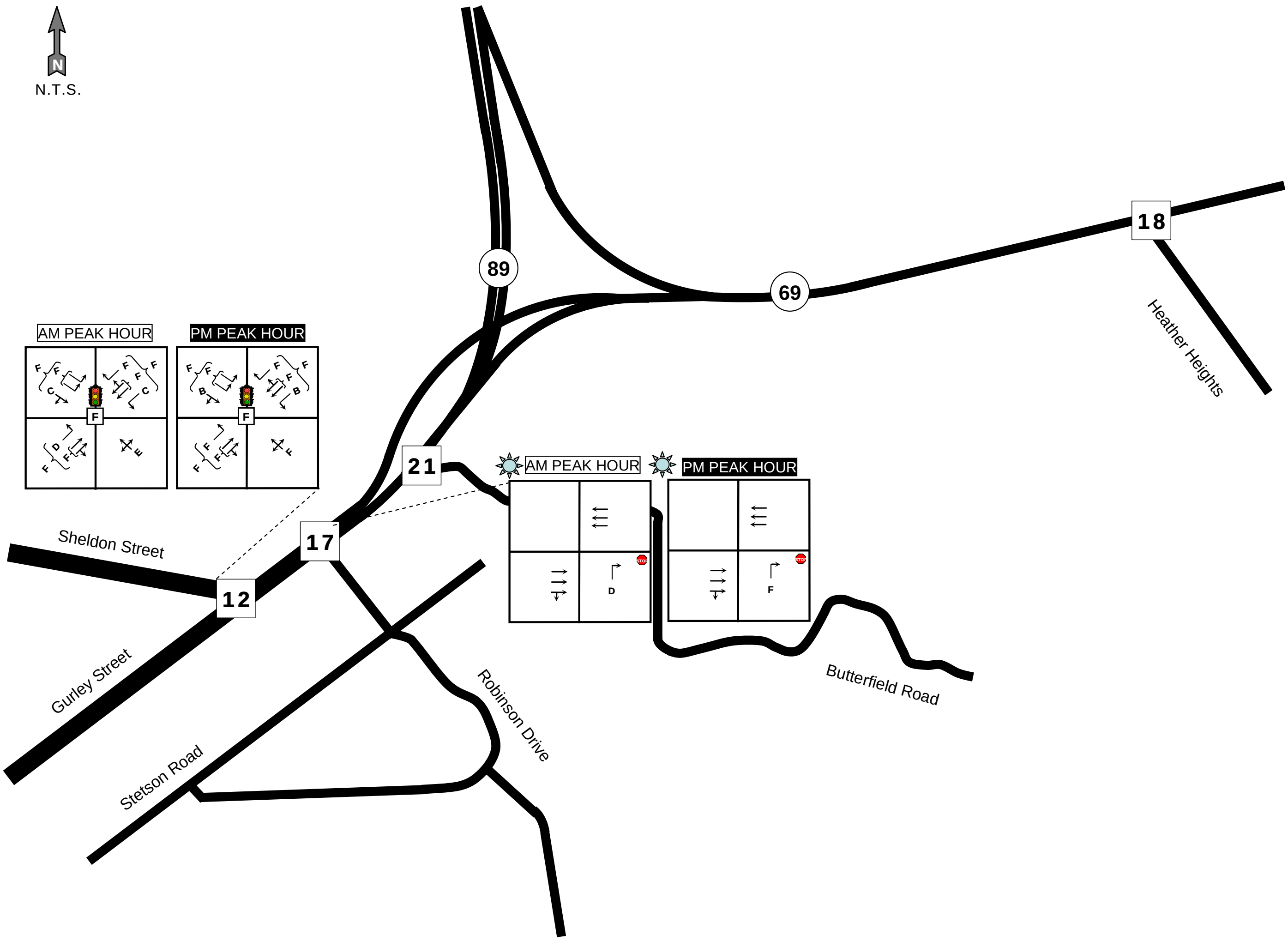

Figure 21: 2007 LOS for intersections 12, 17, 18, and 21 – AM and PM Peaks

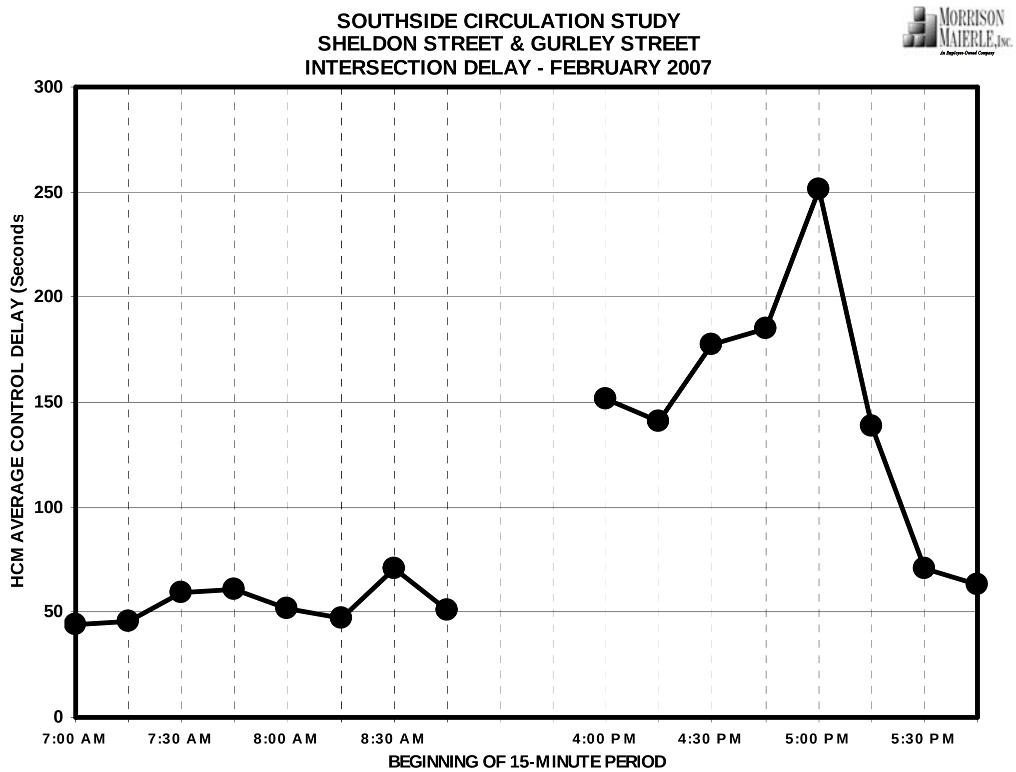

Figure 22: Sheldon Street & Gurley Street – 15-minute Delay Measurements

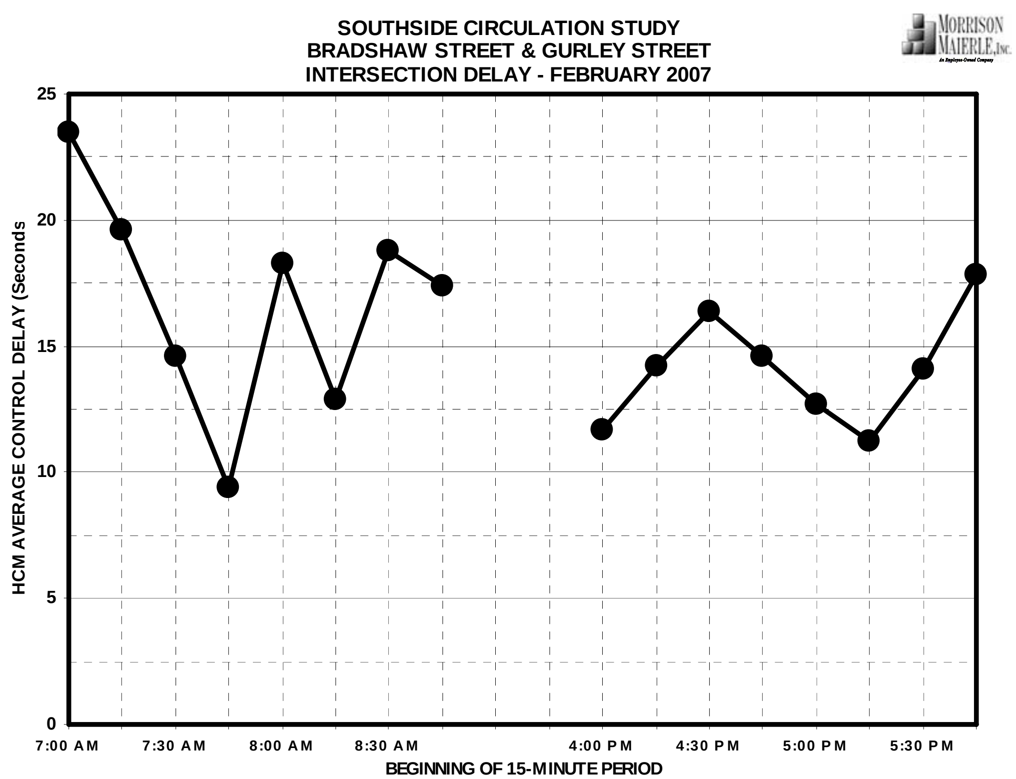

Figure 23: Bradshaw Street & Gurley Street – 15-minute Delay Measurements

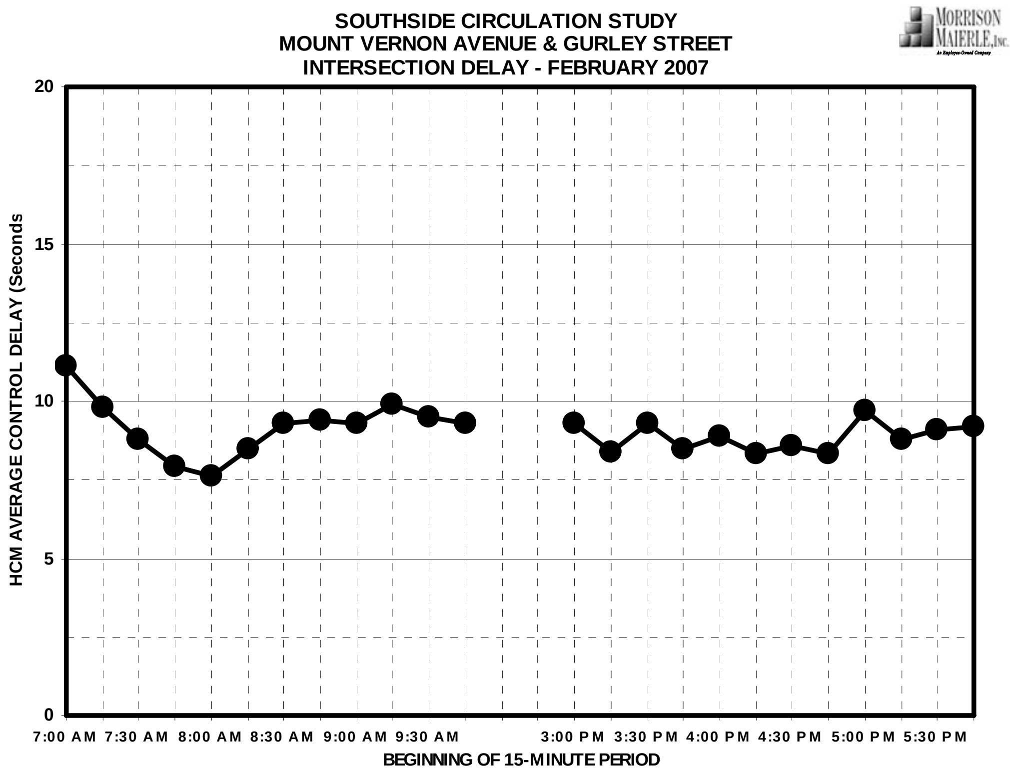

Figure 24: Mount Vernon Avenue & Gurley Street – 15-minute Delay Measurements

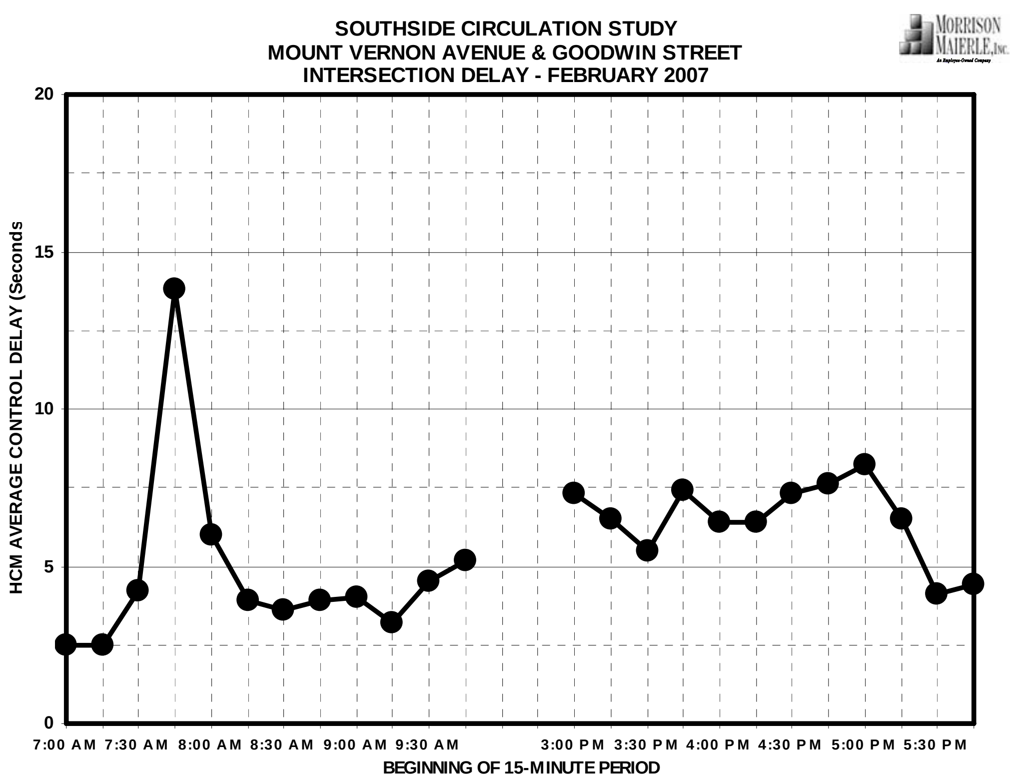

Figure 25: Mount Vernon Avenue & Goodwin Street – 15-minute Delay Measurements

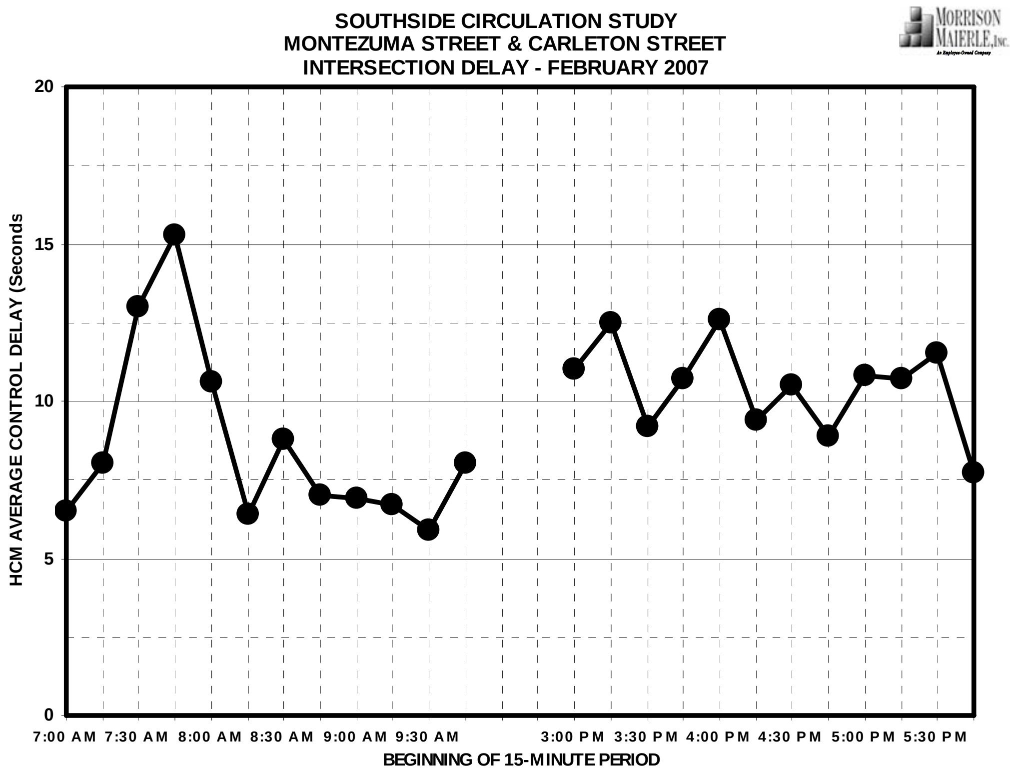

Figure 26: Montezuma Street & Carleton Street – 15-minute Delay Measurements

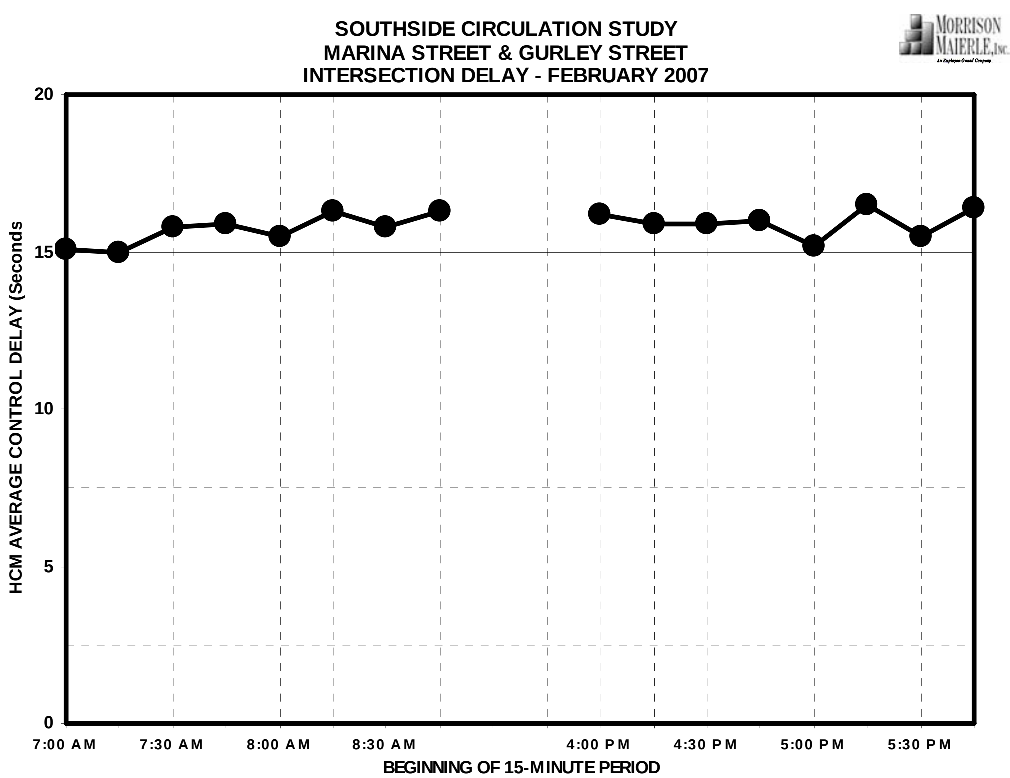

Figure 27: Marina Street & Gurley Street – 15-minute Delay Measurements

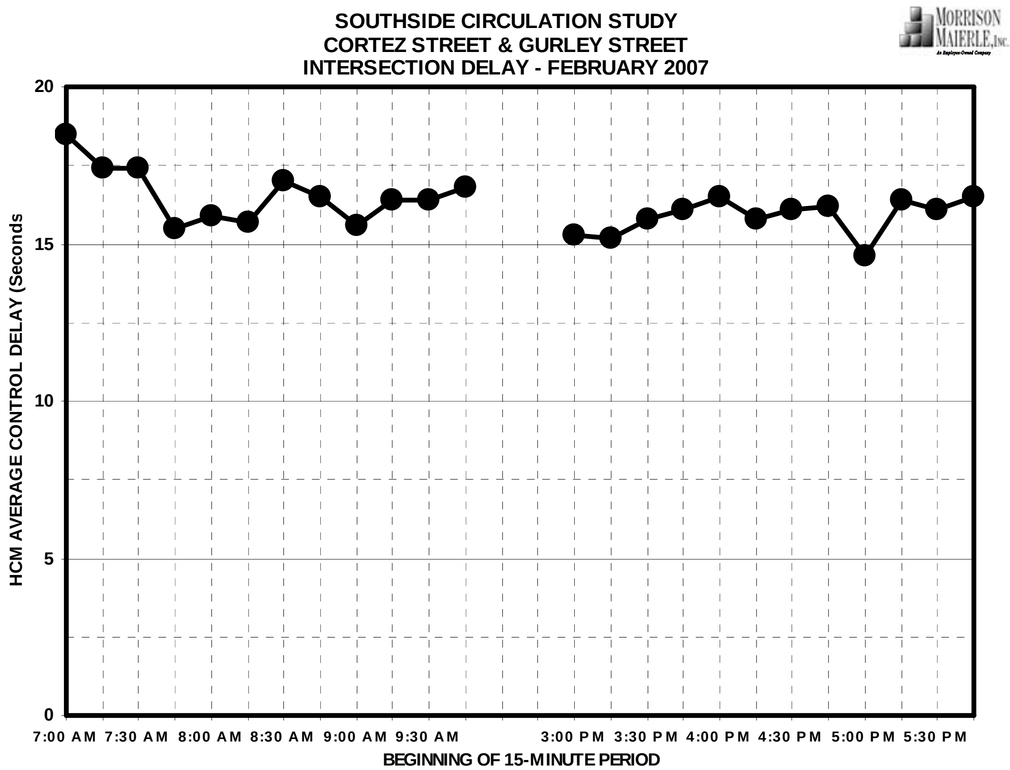

Figure 28: Cortez Street & Gurley Street – 15-minute Delay Measurements

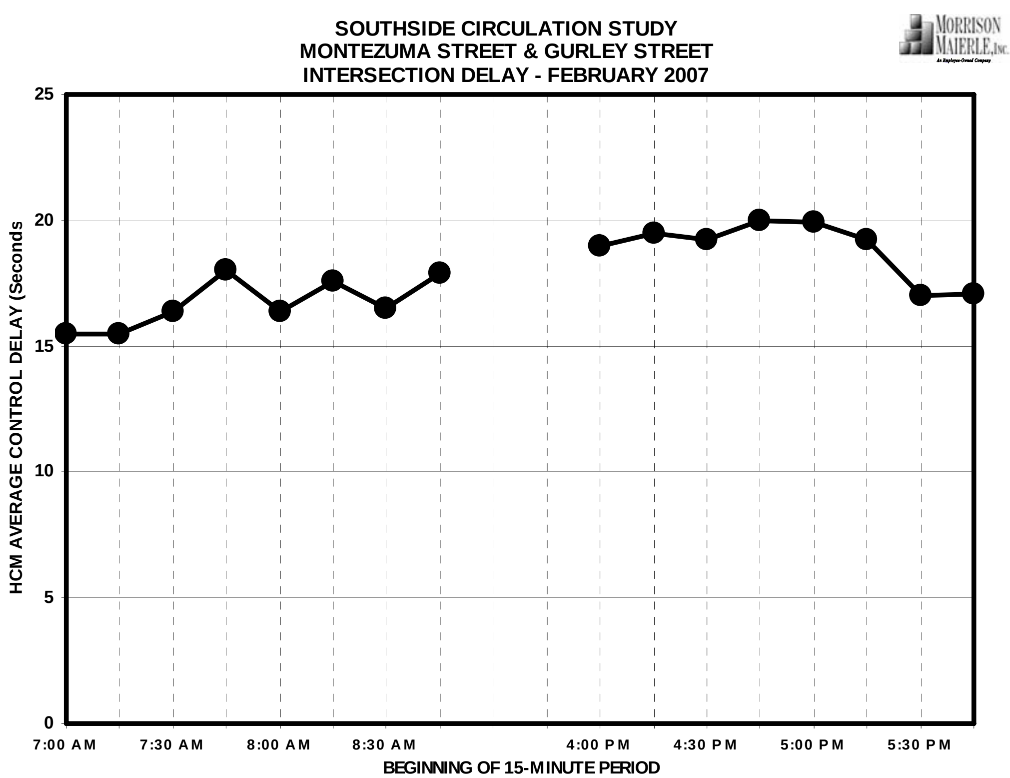

Figure 29: Montezuma Street & Gurley Street – 15-minute Delay Measurements

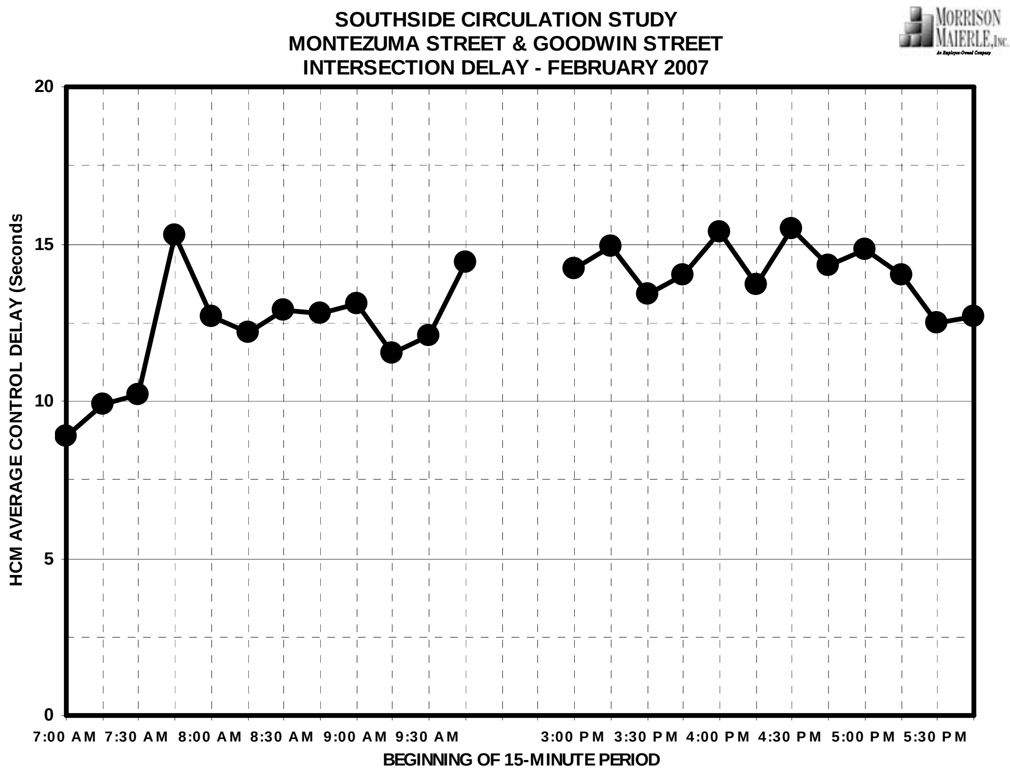

Figure 30: Montezuma Street & Goodwin Street – 15-minute Delay Measurements

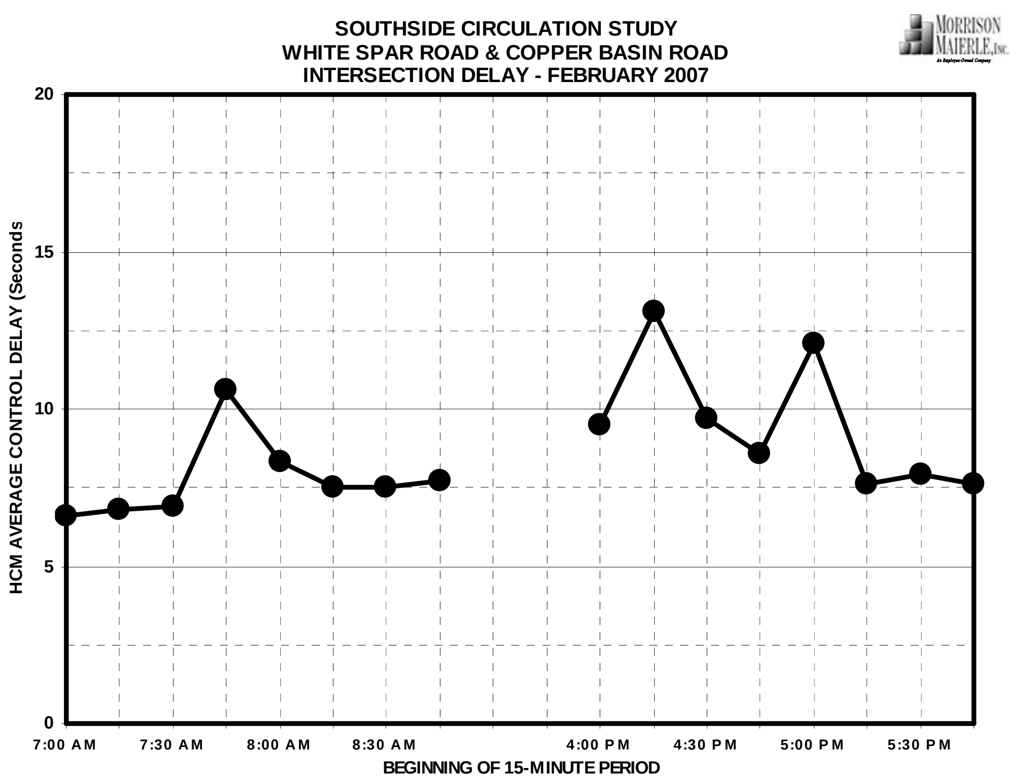

Figure 31: White Spar Road & Copper Basin Road – 15-minute Delay Measurements

Signal Warrant and Multiway Stop Analysis

The *Manual on Uniform Traffic Control Devices (MUTCD)* as published by the United States Department of Transportation is the reference for determining the need for traffic signal installation throughout the United States. This document establishes eight separate, related sets of criteria termed “warrants”. If none of the eight warrants are satisfied, then a signal should not be installed. If one or more of the warrants are satisfied, then a signal might be appropriate.

Table 6, shown below, provides the names of the primary signal warrants. **Table 7** on the following page summarizes the results of the analyses of the primary signal warrants for the 9 additional intersections.

Table 6: Signal Warrant Names

WARRANT	NAME
1A	Minimum Vehicular Volume
1B	Interruption of Continuous Traffic
1A and 1B	Combination of Warrants
2	Four-Hour Vehicular Volume
3B	Peak Hour Volume

Table 7: Existing Signal Warrant Analyses Summary

	ACTUAL NUMBER OF HOURS MET					
WARRANT	**1A**	**1B**	**1A & 1B**	**2**	**3B**	**WARRANT SATISFIED?**
REQUIRED HOURS MET	8	8	8	4	1	
Marina & Union	0	0	0	0	0	NO
Marina & Goodwin	1	0	0	0	0	NO
Mount Vernon & Goodwin	4	0	2	0	0	NO
Mount Vernon & Carleton	1	0	0	0	0	NO
Arizona & Goodwin	0	0	0	0	0	NO
White Spar & Haisley	0	0	0	0	0	NO
Mount Vernon & Haisley	0	0	0	0	0	NO
Bradshaw & Stetson	0	0	0	0	0	NO
Robinson & Gurley	0	0	0	0	0	NO

These analyses indicate that none of the intersections satisfy the warrants for signal installation.

The *Manual on Uniform Traffic Control Devices (MUTCD)* also establishes four separate, related sets of criteria to assist in the determination of the need for stop signs on each approach to an intersection. The first multi-way stop warrant is warrant A which indicates that a multi-way stop may be temporarily appropriate if a traffic signal is warranted, until it is installed. Warrant B suggests the installation of stop signs for each approach to an intersection if the intersection has been the site of five (5) or more collisions of a type potentially preventable by multi-way stop signs in a twelve-month period. Beyond the 10 signalized intersections whose operations were analyzed, 9 additional unsignalized intersections were considered to determine if signalization or multiway stop control was warranted.

Because none of the 9 intersections satisfied the signal warrants, each intersection was analyzed to see if the multiway stop control warrants were satisfied. Warrant A was not satisfied for any of the intersections. Collision data for the intersections was not available, so Warrant B was not considered in determining if multiway stop control was appropriate. Warrants C and D did not consider the vehicle delay portion of the warrants since approach and departure counts and not turning movement counts were obtained for these 9 locations. The 8 hour average volumes were considered for Warrants C and D, and the results are summarized below in **Table 8.**

Table 8: Multiway Stop Warrant Analyses Summary

	ACTUAL NUMBER OF HOURS MET						
WARRANT	C: 8-Hour Volumes		C - Delay	D: 8-Hour Volumes		D - Delay	WARRANT SATISFIED?
	Major	Minor	(sec/veh)	Major	Minor	(sec/veh)	
CRITERIA	300	200	30	240	160	24	
Marina & Union	314	97	Not Measured	314	97	Not Measured	NO
Marina & Goodwin	464	251	Not Measured	464	251	Not Measured	YES
Mount Vernon & Goodwin	578	244	Not Measured	578	244	Not Measured	YES
Mount Vernon & Carleton	464	154	Not Measured	464	154	Not Measured	NO
Arizona & Goodwin	143	26	Not Measured	143	26	Not Measured	NO
White Spar & Haisley	301	103	Not Measured	301	103	Not Measured	NO
Mount Vernon & Haisley	264	83	Not Measured	264	83	Not Measured	NO
Bradshaw & Stetson	175	99	Not Measured	175	99	Not Measured	NO
Robinson & Gurley	4,076	61	Not Measured	4,076	61	Not Measured	NO

These analyses indicate that the intersections of Marina / Goodwin and Mount Vernon / Goodwin satisfy the traffic volume portion of the multiway stop warrants.

APPENDIX C – 2030 TRAFFIC DATA

Prescott, Arizona

Southside
2030 Traffic

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Projected 2030 Traffic Volumes

Future daily traffic volumes for the year 2030 were generated at the study intersections by the model produced by Carter-Burgess. The boundaries of the study, as well as the locations of the study intersections are shown in **Figure 1**. The daily approach and departure volumes for the study intersections are shown on the subsequent pages in **Figure 2** through **Figure 6**.

In some cases, the traffic model did not predict future traffic volumes for one of the legs of a study intersection. This occurred when a roadway which serves as a leg to a study intersection was not sufficiently significant to include in the roadway network of the model. This occurred at the north leg of intersections 3, 7, and 20; the west leg of intersection 2 and the east leg of intersection 13. Volumes for these missing legs were estimated by calculating a ratio of traffic for the missing leg from the existing 2007 counts and applying that ratio to the known volumes for 2030. These approach and departure volumes are also included in **Figure 2** through **Figure 6**.

The estimated peak hour turning movement volumes at the study intersections were determined using (1) the daily approach and departure volumes, and (2) the percentage of daily traffic arriving during the peak hour (the k-factor) by leg, for each of the study intersections. The turning movement volumes were determined through an automated mathematical iteration process. This process assumed turning movement volumes for each approach, and then compared the resulting predicted departing volumes to the departing values calculated by summing the appropriate turning movement volumes. The iteration minimizes the sum of the squares of the differences between the two calculation procedures for the departing volumes. For the 2030 volumes, a minimum volume of 50 vehicles per hour and a minimum increase of 35% from 2007 were utilized.

The k-factors that were used in calculating the peak hourly volumes in 2007 for the study intersections were also used for calculating the hourly volumes at the corresponding intersections in 2030. These k-factors are presented in **Table 1** and **Table 2. Table 1** includes the k-factors that were calculated from the 24-hour approach volumes, and the peak hour turning movement counts. **Table 2** includes the k-factors which were estimated for selected study intersections based on the directional split of adjacent roadways, existing traffic patterns, and k-factors of the adjacent intersections. In each case a minimum k-factor of 6%, and a maximum k-factor of 12% were imposed on the study intersections to eliminate the propagation of extreme peak hour volume estimates.

Table 3 and **Table 4** contain additional k-factors which were necessary for predicting future 2030 turning movement volumes but were not needed for the 2007 calculations. Approach k-factors were calculated for intersections 1, 2, 3, 7, and 8. **Table 3** includes the additional k-factors that were calculated for the additional intersections from the 24-hour approach volumes, and the peak hour turning movement counts, but were not included in **Table 1**. **Table 4** includes the new k-factors which were estimated for select study intersections based on the directional split of adjacent roadways, existing traffic patterns, and the k-factors of adjacent intersections, which were not included in **Table 2**. **Table 4** also includes new k-factors for intersection 13 which better correspond with future 2030 traffic patterns. In each case a minimum k-factor of 6%, and a maximum k-factor of 12% were used imposed on the study intersections to eliminate the propagation of extreme peak hour volume estimates.

The resulting morning and evening peak hour turning movement volumes for all the study intersections are shown in **Figure 7** through **Figure 11**.

Figure 1: Location of Study Intersections

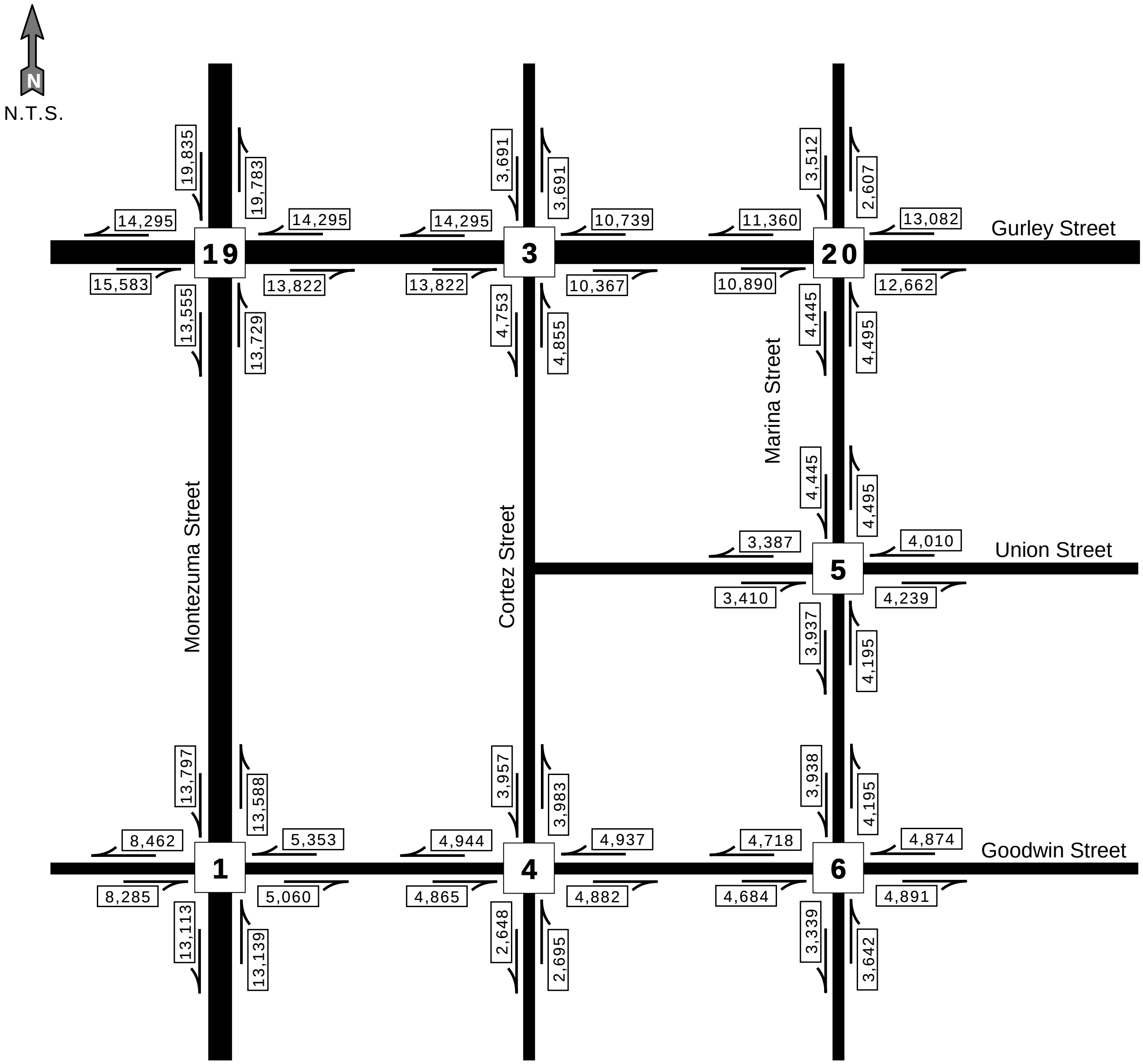

Figure 2: 2030 Directional Volumes for Intersections 1, 3, 4, 5, 6, 19, and 20 – Day

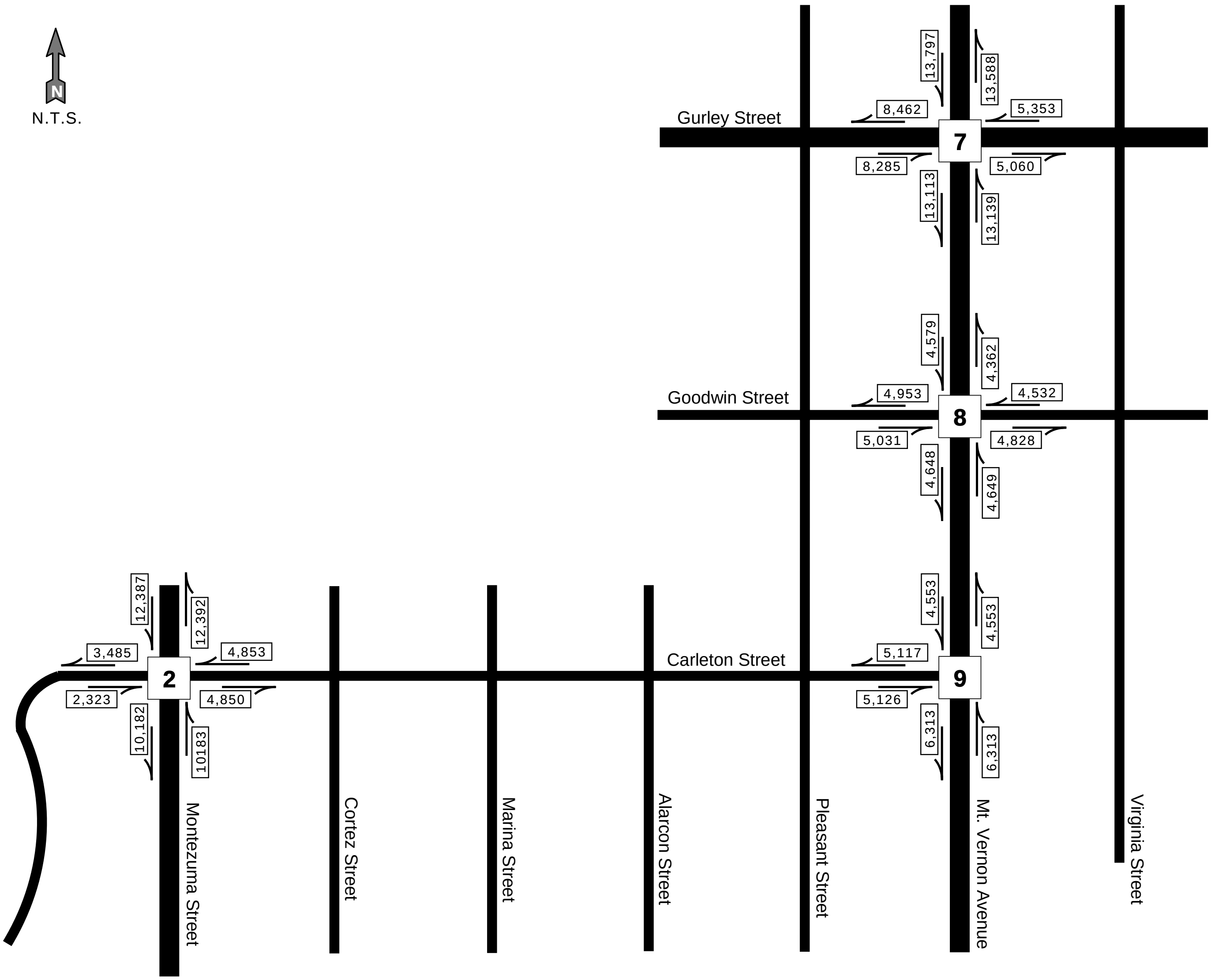

Figure 3: 2030 Directional Volumes for Intersections 2, 7, 8, and 9 – Day

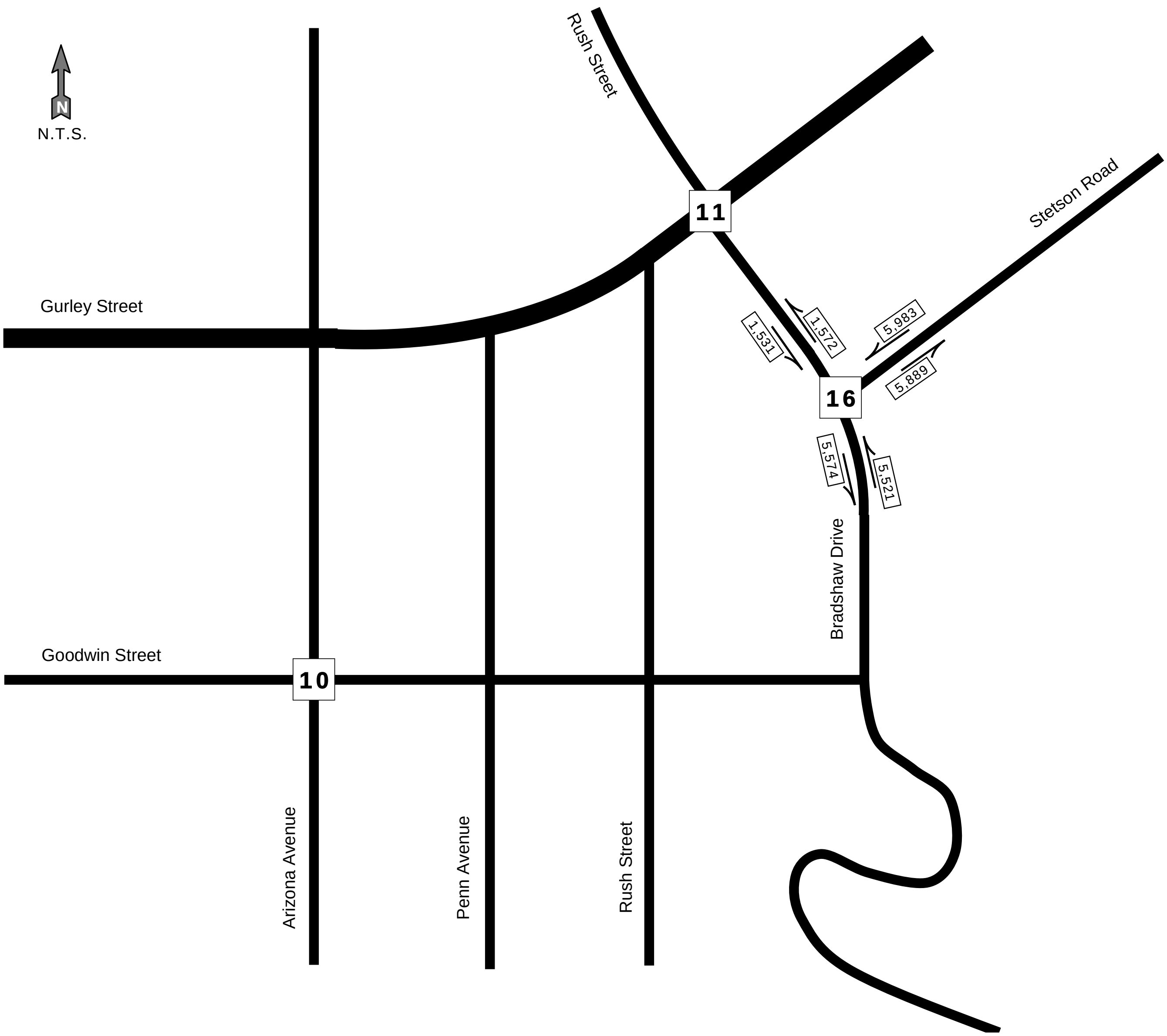

Figure 4: 2030 Directional Volumes for Intersection 16 – Day

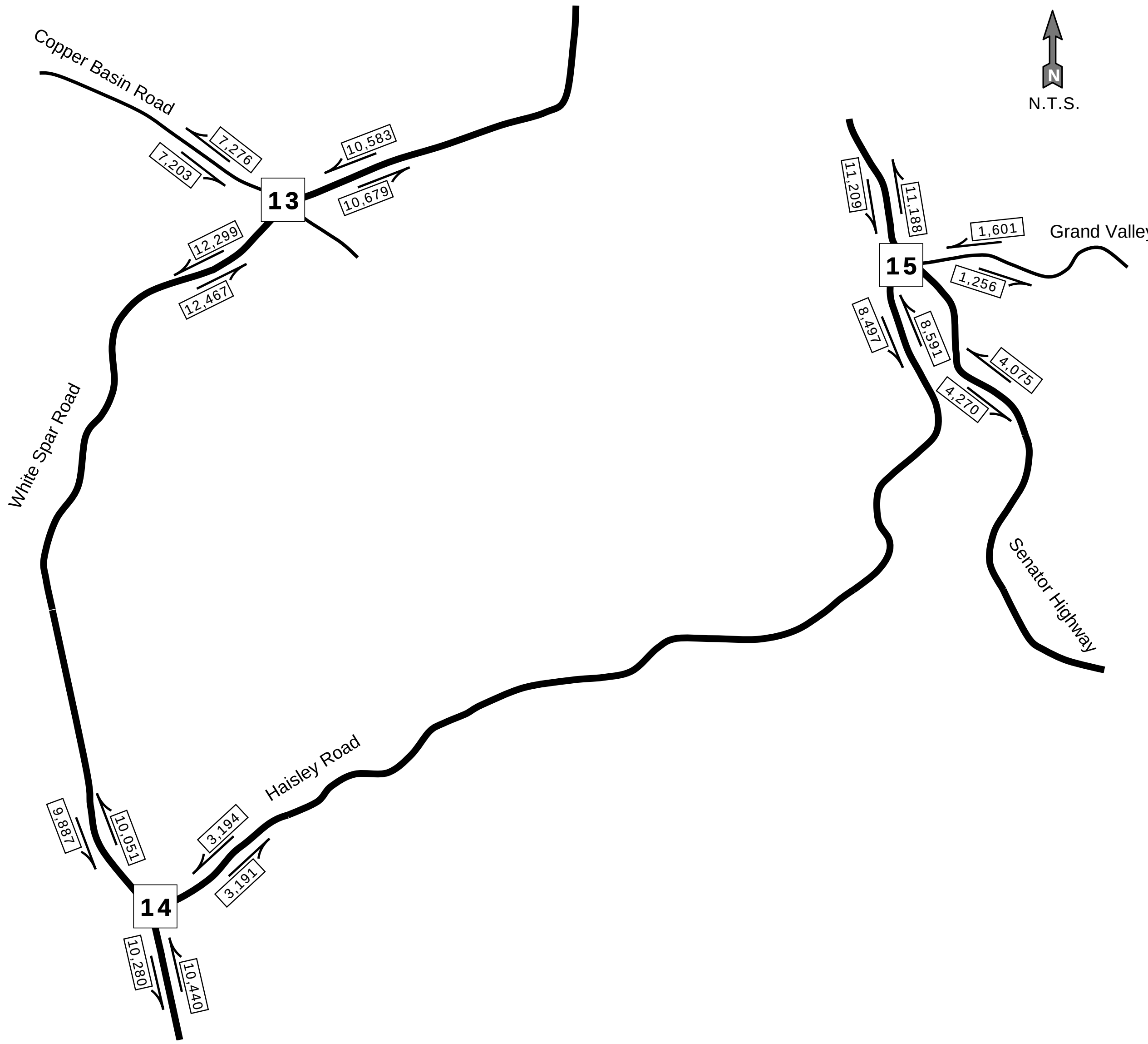

Figure 5: 2030 Directional Volumes for Intersections 13, 14, and 15 – Day

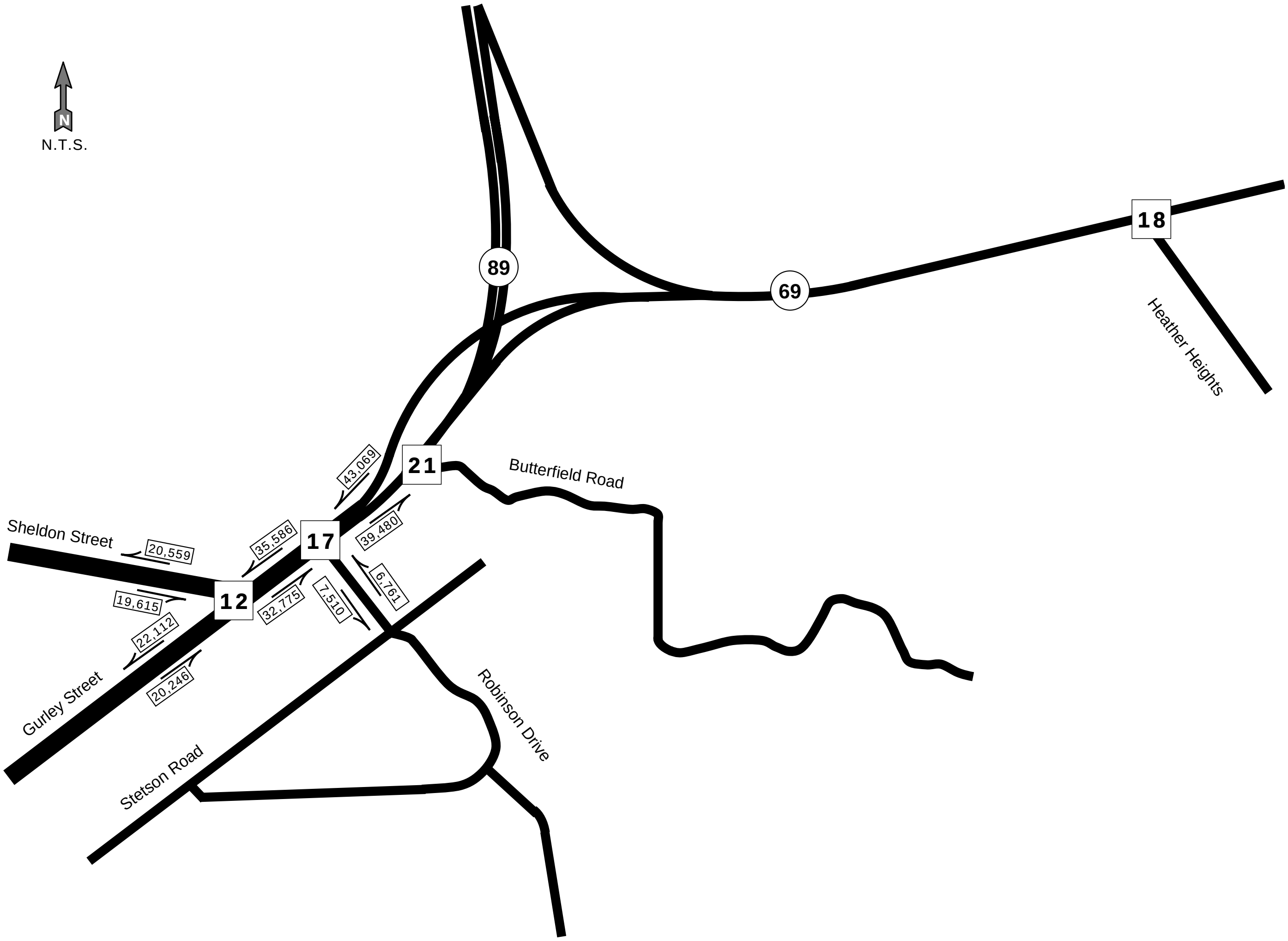

Figure 6: 2030 Directional Volumes for Intersections 12, and 17 – Day

Table 1: Calculated K-Factors from 2007

CALCULATED K-FACTORS

ID	STREET	LOCATION	DIRECTION	PORTION OF DAILY TRAFFIC	
				AM Peak Hour	PM Peak Hour
17	ROBINSON DRIVE	South of GURLEY STREET	Northbound	7%	6%
	GURLEY STREET	East of ROBINSON DRIVE	Westbound	8%	7%
	GURLEY STREET	West of ROBINSON DRIVE	Eastbound	6%	9%
5	MARINA STREET	North of UNION STREET	Southbound	7%	8%
	MARINA STREET	South of UNION STREET	Northbound	9%	9%
	UNION STREET	East of MARINA STREET	Westbound	12%	12%
	UNION STREET	West of MARINA STREET	Eastbound	10%	11%
16	BRADSHAW DRIVE	North of STETSON ROAD	Southbound	6%	10%
	BRADSHAW DRIVE	South of STETSON ROAD	Northbound	7%	8%
	STETSON ROAD	East of BRADSHAW DRIVE	Westbound	10%	8%
4	CORTEZ STREET	North of GOODWIN STREET	Southbound	6%	8%
	CORTEZ STREET	South of GOODWIN STREET	Northbound	6%	9%
	GOODWIN STREET	East of CORTEZ STREET	Westbound	7%	9%
	GOODWIN STREET	West of CORTEZ STREET	Eastbound	6%	7%
6	MARINA STREET	North of GOODWIN STREET	Southbound	6%	10%
	MARINA STREET	South of GOODWIN STREET	Northbound	6%	9%
	GOODWIN STREET	East of MARINA STREET	Westbound	8%	8%
	GOODWIN STREET	West of MARINA STREET	Eastbound	7%	9%
10	ARIZONA AVENUE	North of GOODWIN STREET	Southbound	10%	10%
	ARIZONA AVENUE	South of GOODWIN STREET	Northbound	6%	8%
	GOODWIN STREET	East of ARIZONA AVENUE	Westbound	9%	9%
	GOODWIN STREET	West of ARIZONA AVENUE	Eastbound	6%	10%
9	MT. VERNON AVENUE	North of CARLETON STREET	Southbound	6%	9%
	MT. VERNON AVENUE	South of CARLETON STREET	Northbound	9%	7%
	CARLETON STREET	West of MT. VERNON AVENUE	Eastbound	8%	10%
14	WHITE SPAR ROAD	North of HAISLEY ROAD	Southbound	6%	11%
	WHITE SPAR ROAD	South of HAISLEY ROAD	Northbound	11%	8%
	HAISLEY ROAD	East of WHITE SPAR ROAD	Westbound	8%	8%
15	SENATOR HIGHWAY	North of CUESTA WAY	Southbound	6%	10%
	SENATOR HIGHWAY	South of HAISLEY ROAD	Northbound	11%	6%
	CUESTA WAY	East of SENATOR HIGHWAY	Westbound	6%	12%
	HAISLEY ROAD	West of SENATOR HIGHWAY	Eastbound	9%	7%

Table 2: Estimated K-Factors from 2007

ESTIMATED K-FACTORS					
				PORTION OF DAILY TRAFFIC	
ID	STREET	LOCATION	DIRECTION	AM Peak Hour	PM Peak Hour
17	ROBINSON DRIVE	South of GURLEY STREET	Southbound	6%	10%
	GURLEY STREET	East of ROBINSON DRIVE	Eastbound	6%	9%
	GURLEY STREET	West of ROBINSON DRIVE	Westbound	8%	7%
5	MARINA STREET	North of UNION STREET	Northbound	9%	9%
	MARINA STREET	South of UNION STREET	Southbound	7%	8%
	UNION STREET	East of MARINA STREET	Eastbound	10%	11%
	UNION STREET	West of MARINA STREET	Westbound	12%	12%
16	BRADSHAW DRIVE	North of STETSON ROAD	Northbound	8%	8%
	BRADSHAW DRIVE	South of STETSON ROAD	Southbound	6%	10%
	STETSON ROAD	East of BRADSHAW DRIVE	Eastbound	8%	10%
4	CORTEZ STREET	North of GOODWIN STREET	Northbound	6%	9%
	CORTEZ STREET	South of GOODWIN STREET	Southbound	6%	8%
	GOODWIN STREET	East of CORTEZ STREET	Eastbound	6%	7%
	GOODWIN STREET	West of CORTEZ STREET	Westbound	7%	9%
6	MARINA STREET	North of GOODWIN STREET	Northbound	7%	7%
	MARINA STREET	South of GOODWIN STREET	Southbound	6%	10%
	GOODWIN STREET	East of MARINA STREET	Eastbound	6%	11%
	GOODWIN STREET	West of MARINA STREET	Westbound	7%	9%
10	ARIZONA AVENUE	North of GOODWIN STREET	Northbound	7%	8%
	ARIZONA AVENUE	South of GOODWIN STREET	Southbound	6%	10%
	GOODWIN STREET	East of ARIZONA AVENUE	Eastbound	6%	10%
	GOODWIN STREET	West of ARIZONA AVENUE	Westbound	11%	8%
9	MT. VERNON AVENUE	North of CARLETON STREET	Northbound	9%	7%
	MT. VERNON AVENUE	South of CARLETON STREET	Southbound	6%	9%
	CARLETON STREET	West of MT. VERNON AVENUE	Westbound	8%	9%
14	WHITE SPAR ROAD	North of HAISLEY ROAD	Northbound	8%	8%
	WHITE SPAR ROAD	South of HAISLEY ROAD	Southbound	6%	11%
	HAISLEY ROAD	East of WHITE SPAR ROAD	Eastbound	9%	7%
15	SENATOR HIGHWAY	North of CUESTA WAY	Northbound	9%	7%
	SENATOR HIGHWAY	South of HAISLEY ROAD	Southbound	6%	10%
	CUESTA WAY	East of SENATOR HIGHWAY	Eastbound	6%	12%
	HAISLEY ROAD	West of SENATOR HIGHWAY	Westbound	8%	8%

Table 3: Calculated K-Factors for 2030

CALCULATED K-FACTORS USED FOR 2030

ID	STREET	LOCATION	DIRECTION	PORTION OF DAILY TRAFFIC AM Peak Hour	PORTION OF DAILY TRAFFIC PM Peak Hour
1	MONTEZUMA STREET	North of GOODWIN STREET	Southbound	6%	9%
	MONTEZUMA STREET	South of GOODWIN STREET	Northbound	8%	7%
	GOODWIN STREET	East of MONTEZUMA STREET	Westbound	7%	8%
	UNION STREET	West of MONTEZUMA STREET	Eastbound	6%	8%
2	MONTEZUMA STREET	North of CARLETON STREET	Southbound	6%	9%
	MONTEZUMA STREET	South of CARLETON STREET	Northbound	8%	8%
	CARLETON STREET	East of MONTEZUMA STREET	Westbound	8%	9%
	CARLETON STREET	West of MONTEZUMA STREET	Eastbound	12%	12%
3	CORTEZ STREET	North of GURLEY STREET	Southbound	6%	6%
	CORTEZ STREET	South of GURLEY STREET	Northbound	6%	8%
	GURLEY STREET	East of CORTEZ STREET	Westbound	6%	7%
	GURLEY STREET	West of CORTEZ STREET	Eastbound	6%	9%
7	MT. VERNON AVENUE	North of GURLEY STREET	Southbound	6%	9%
	MT. VERNON AVENUE	South of GURLEY STREET	Northbound	7%	7%
	GURLEY STREET	East of MT. VERNON AVENUE	Westbound	9%	8%
	GURLEY STREET	West of MT. VERNON AVENUE	Eastbound	6%	9%
8	MT. VERNON AVENUE	North of GOODWIN STREET	Southbound	7%	9%
	MT. VERNON AVENUE	South of GOODWIN STREET	Northbound	9%	7%
	GOODWIN STREET	East of MT. VERNON AVENUE	Westbound	11%	8%
	GOODWIN STREET	West of MT. VERNON AVENUE	Eastbound	6%	11%

Table 4: Estimated K-Factors for 2030

ESTIMATED K-FACTORS USED FOR 2030					
				PORTION OF DAILY TRAFFIC	
ID	STREET	LOCATION	DIRECTION	AM Peak Hour	PM Peak Hour
1	MONTEZUMA STREET	North of GOODWIN STREET	Northbound	8%	7%
	MONTEZUMA STREET	South of GOODWIN STREET	Southbound	6%	9%
	GOODWIN STREET	East of MONTEZUMA STREET	Eastbound	6%	8%
	UNION STREET	West of MONTEZUMA STREET	Westbound	7%	8%
2	MONTEZUMA STREET	North of CARLETON STREET	Northbound	8%	8%
	MONTEZUMA STREET	South of CARLETON STREET	Southbound	6%	9%
	CARLETON STREET	East of MONTEZUMA STREET	Eastbound	12%	12%
	CARLETON STREET	West of MONTEZUMA STREET	Westbound	8%	9%
3	CORTEZ STREET	North of GURLEY STREET	Northbound	6%	8%
	CORTEZ STREET	South of GURLEY STREET	Southbound	6%	6%
	GURLEY STREET	East of CORTEZ STREET	Eastbound	6%	9%
	GURLEY STREET	West of CORTEZ STREET	Westbound	6%	7%
7	MT. VERNON AVENUE	North of GURLEY STREET	Northbound	7%	7%
	MT. VERNON AVENUE	South of GURLEY STREET	Southbound	6%	9%
	GURLEY STREET	East of MT. VERNON AVENUE	Eastbound	6%	9%
	GURLEY STREET	West of MT. VERNON AVENUE	Westbound	9%	8%
8	MT. VERNON AVENUE	North of GOODWIN STREET	Northbound	9%	7%
	MT. VERNON AVENUE	South of GOODWIN STREET	Southbound	7%	9%
	GOODWIN STREET	East of MT. VERNON AVENUE	Eastbound	6%	11%
	GOODWIN STREET	West of MT. VERNON AVENUE	Westbound	11%	8%
12	MARINA STREET	North of UNION STREET	Northbound	8%	8%
			Southbound	7%	6%
	UNION STREET	East of MARINA STREET	Eastbound	6%	9%
			Westbound	8%	7%
	UNION STREET	West of MARINA STREET	Eastbound	6%	10%
			Westbound	11%	9%
13	WHITE SPAR ROAD	North of COPPER BASIN ROAD	Northbound	8%	8%
			Southbound	8%	8%
	WHITE SPAR ROAD	South of COPPER BASIN ROAD	Northbound	8%	8%
			Southbound	8%	8%
	COPPER BASIN ROAD	West of WHITE SPAR ROAD	Eastbound	8%	8%
			Westbound	8%	8%
	COPPER BASIN ROAD	East of WHITE SPAR ROAD	Eastbound	8%	8%
			Westbound	8%	8%
19	MARINA STREET	North of UNION STREET	Northbound	8%	7%
			Southbound	6%	9%
	MARINA STREET	South of UNION STREET	Northbound	8%	7%
			Southbound	6%	9%
	UNION STREET	East of MARINA STREET	Eastbound	6%	9%
			Westbound	6%	7%
	UNION STREET	West of MARINA STREET	Eastbound	6%	9%
			Westbound	6%	7%
20	MARINA STREET	North of UNION STREET	Northbound	8%	7%
			Southbound	6%	7%
	MARINA STREET	South of UNION STREET	Northbound	8%	7%
			Southbound	6%	9%
	UNION STREET	East of MARINA STREET	Eastbound	6%	9%
			Westbound	9%	6%
	UNION STREET	West of MARINA STREET	Eastbound	6%	9%
			Westbound	6%	9%

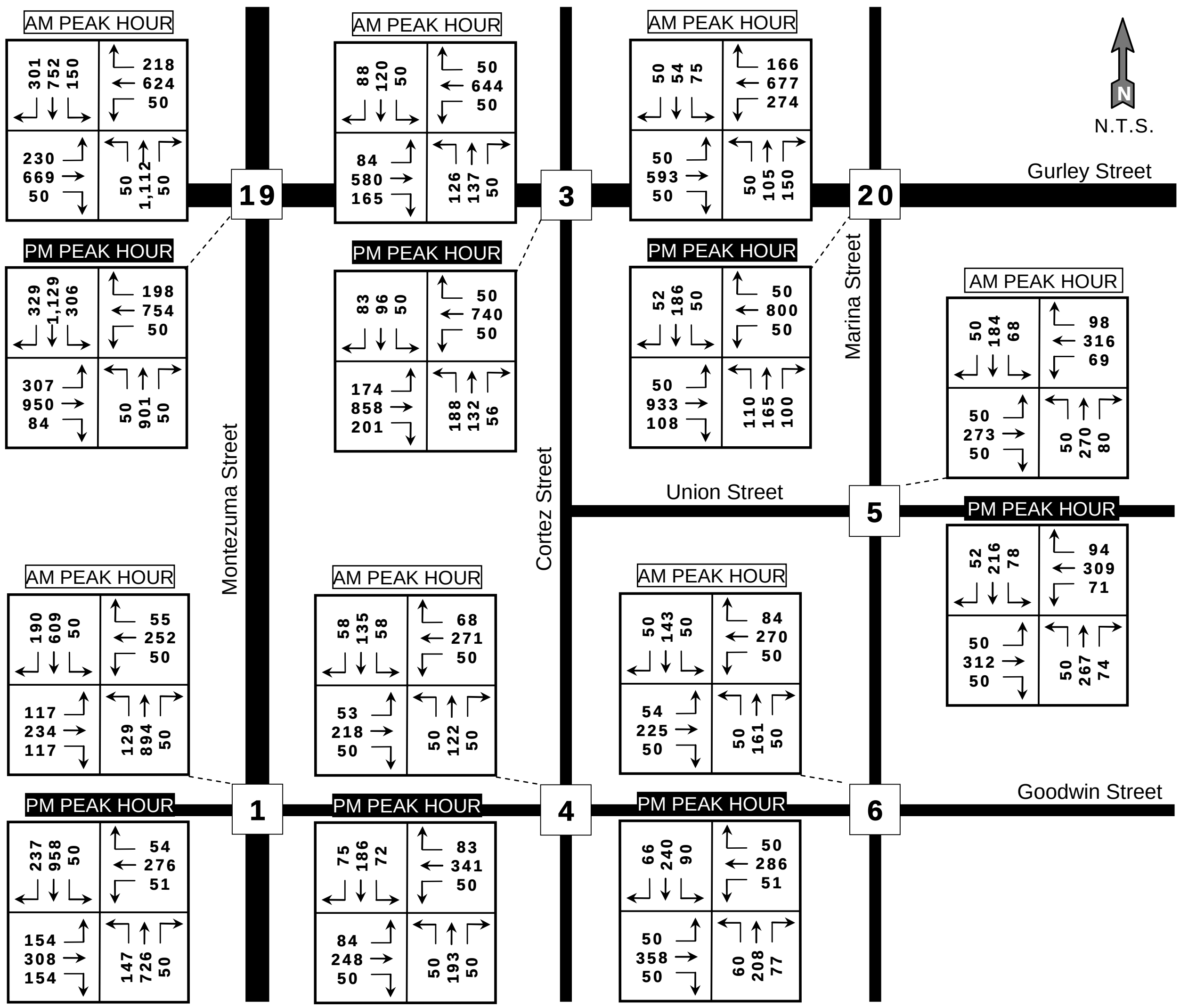

Figure 7: 2030 TMV for Intersections 1, 3, 4, 5, 6, 19, and 20 – AM and PM Peak Hours

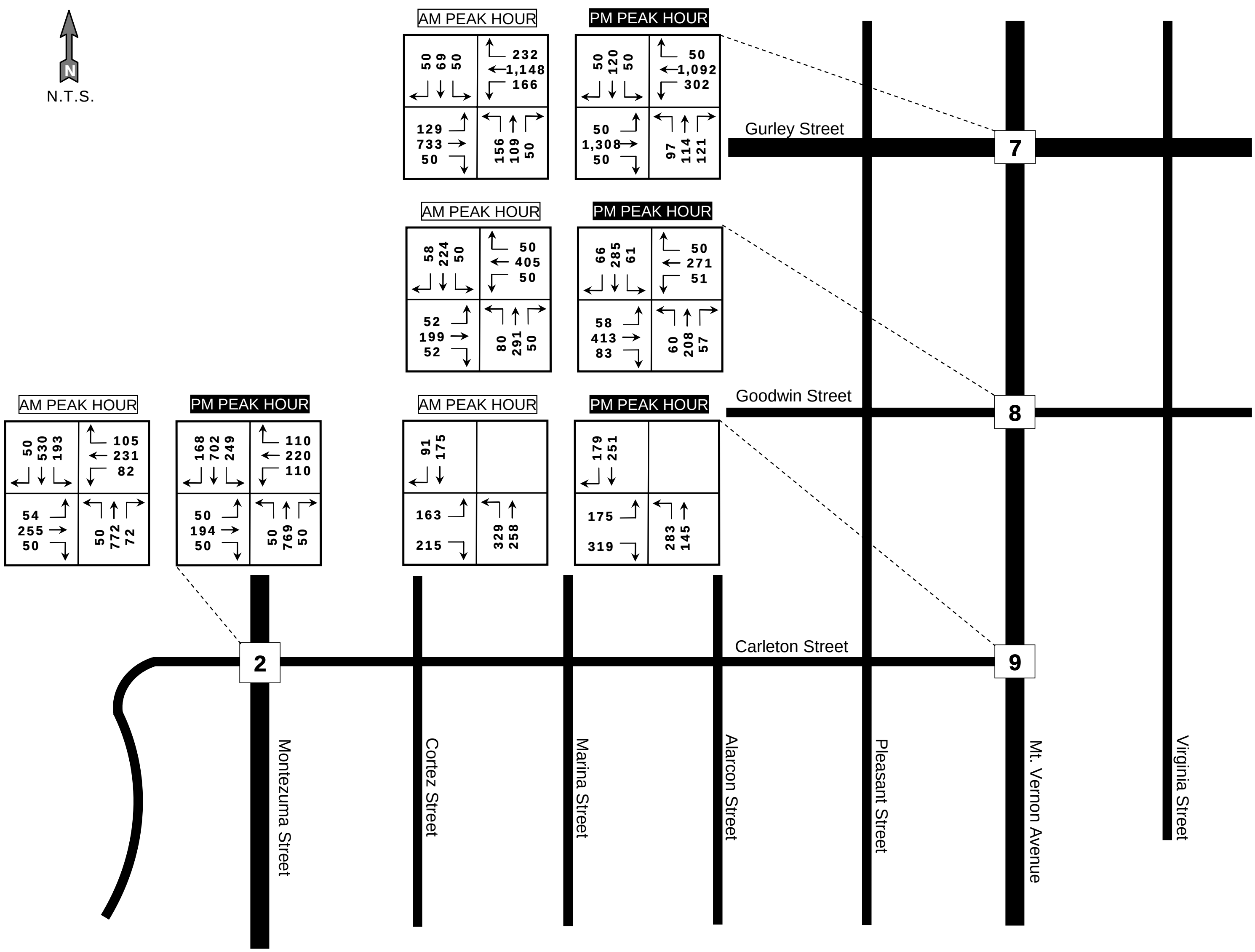

Figure 8: 2030 TMV for Intersections 2, 7, 8, and 9 – AM and PM Peak Hours

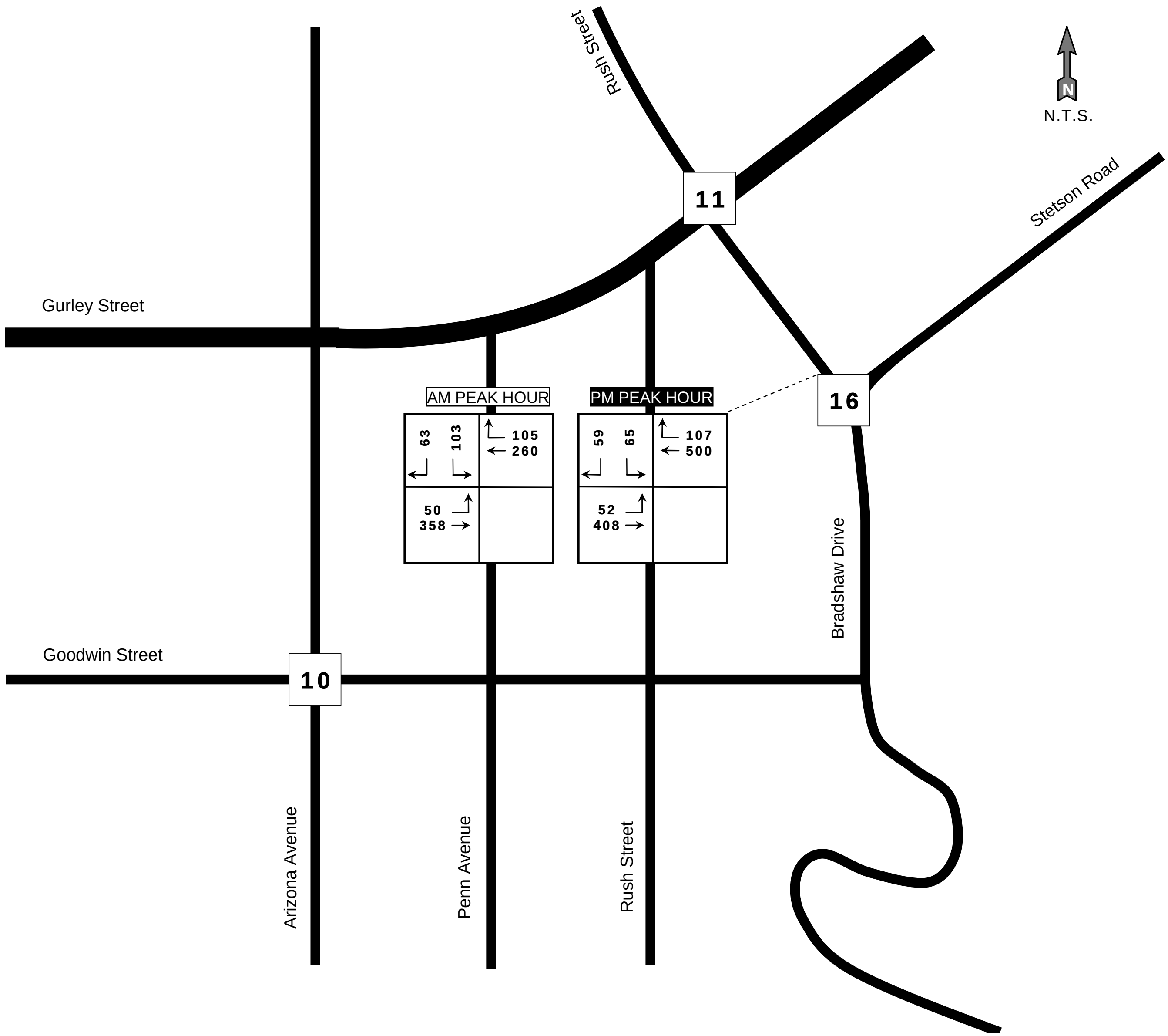

Figure 9: 2030 TMV for Intersection 16 – AM and PM Peak Hours

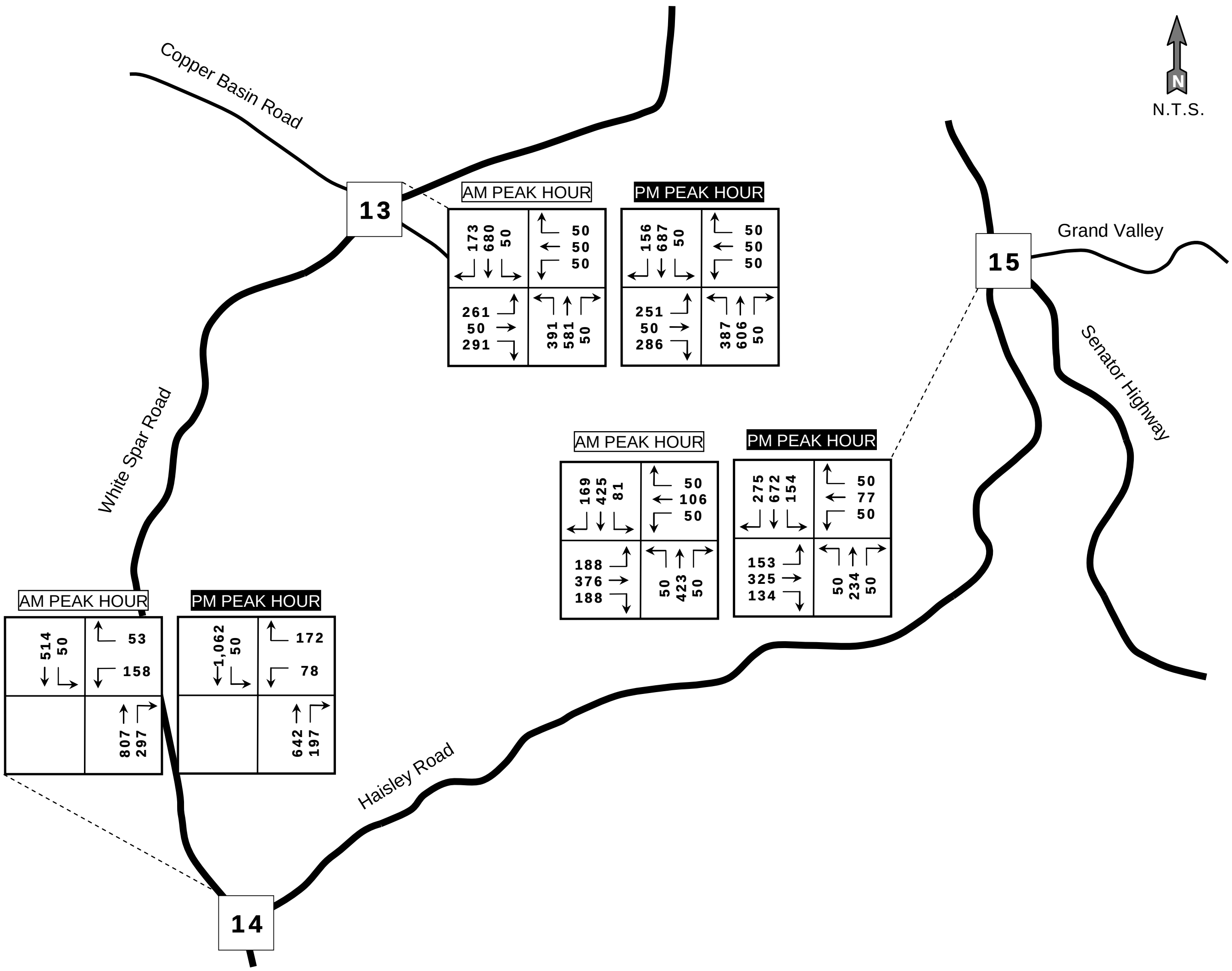

Figure 10: 2030 TMV for Intersections 13, 14, and 15 – AM and PM Peak Hours

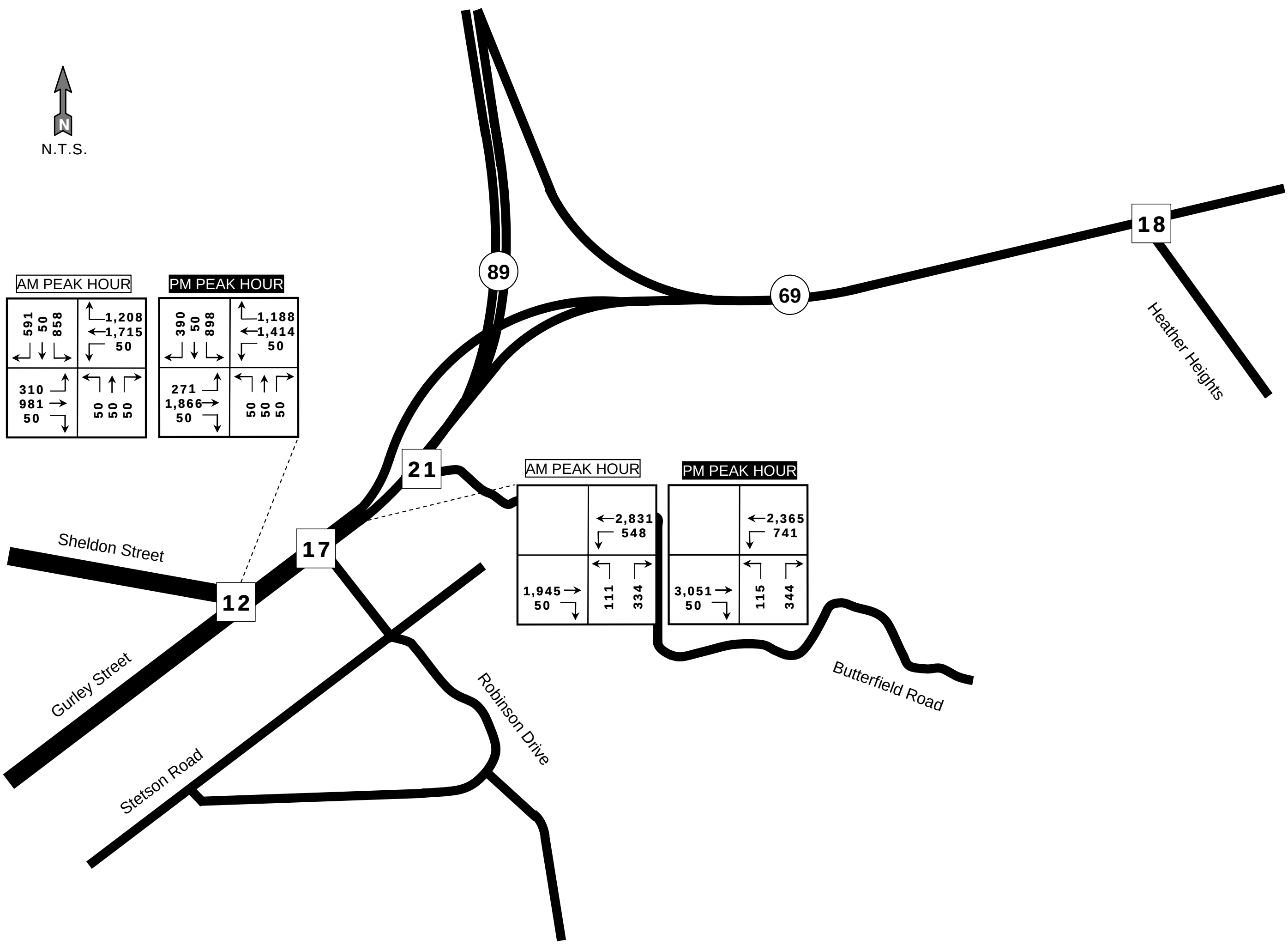

Figure 11: 2030 TMV for Intersections 12, and 17 – AM and PM Peak Hours

2030 Level of Service

The anticipated 2030 traffic volumes at the seventeen study intersections were analyzed to determine the level-of-service for each intersection. The ability of a transportation system to transmit the transportation demand is characterized as its level-of-service (LOS). Level-of-service is a rating system from "A", representing the best operation, to "F", representing the worst operation. Typically, level-of-service "D" is considered the minimum acceptable operation. The appropriate reference for level-of-service operation is the *Highway Capacity Manual*, published by the Transportation Research Board.

This manual considers the average delay per vehicle as the measure to determine the level-of-service of a signalized intersection. The delay and level-of-service are calculated for the intersection, each approach, and each turning movement. For unsignalized intersections the level-of-service is defined for each minor movement for two-way stop controls, and is not defined for the intersection as a whole. For all-way stop controls, level-of-service is defined for the intersection, each approach, and for each turning movement. **Table 5** lists the level-of-service criteria for signalized and unsignalized intersections as stated in the *Highway Capacity Manual*.

Table 5: Level-of-Service Criteria for Signalized Intersections

LEVEL-OF-SERVICE	AVERAGE DELAY (seconds/vehicle)	
	SIGNALIZED	UNSIGNALIZED
A	≤ 10	≤ 10
B	> 10 to 20	> 10 to 15
C	> 20 to 35	> 15 to 25
D	> 35 to 55	> 25 to 35
E	> 55 to 80	> 35 to 50
F	> 80	> 50

Synchro Software was used with the predicted 2030 peak hour turning movements to determine the delay and level-of-service at each study intersection. The intersections were first analyzed with the existing lane configurations and traffic control, and then with recommended improvements. The results from these analyses are presented in **Figure 12** through **Figure 16**, and **Figure 17** through **Figure 21** for the unimproved, and improved intersections respectively.

Synchro software provides calculated delays exceeding 120 seconds per vehicle. The equations for these delay calculations are accurate only for delays less than 60 seconds. The equations provide reasonably accurate results for delays between 60 and 120 seconds. Calculated delay greater than 120 seconds are very exaggerated. Therefore, all calculated delays greater than 120 seconds per vehicle were reduced to 120 seconds per vehicle. This adjustment becomes particularly meaningful when a specific movement is calculated to experience very high delay and thereby greatly exaggerates the corresponding approach delay and intersection delay. Tables of these adjustments are provided in the appendix following the Synchro output reports.

The peak hour factor is the ratio of total traffic occurring during the peak hour to the peak 15-minute flow rate (4 times the maximum 15 minute volume) within the peak hour. A peak hour factor of 0.92 to 0.95 indicates a relatively high degree of uniformity of flow during the peak traffic hour. The peak hour factors for each movement and each approach were calculated from the existing 2007 counts obtained for this report, and where appropriate, were used at the corresponding intersection as a required input for the 2030 Synchro analysis. However, at low volumes the peak hour factor can become exaggerated, and is no longer appropriate for use in predicting future traffic patterns. In some cases existing peak hour factors at lower volume study intersections were as low as 0.29. When unacceptably low peak hour factors were calculated from existing counts, a default peak hour factor of 0.92 was utilized for the 2030 analyses to more realistically predict the behavior of future traffic. The inputs used for the peak hour factors are available in the Synchro output reports that are included in the appendix of this report.

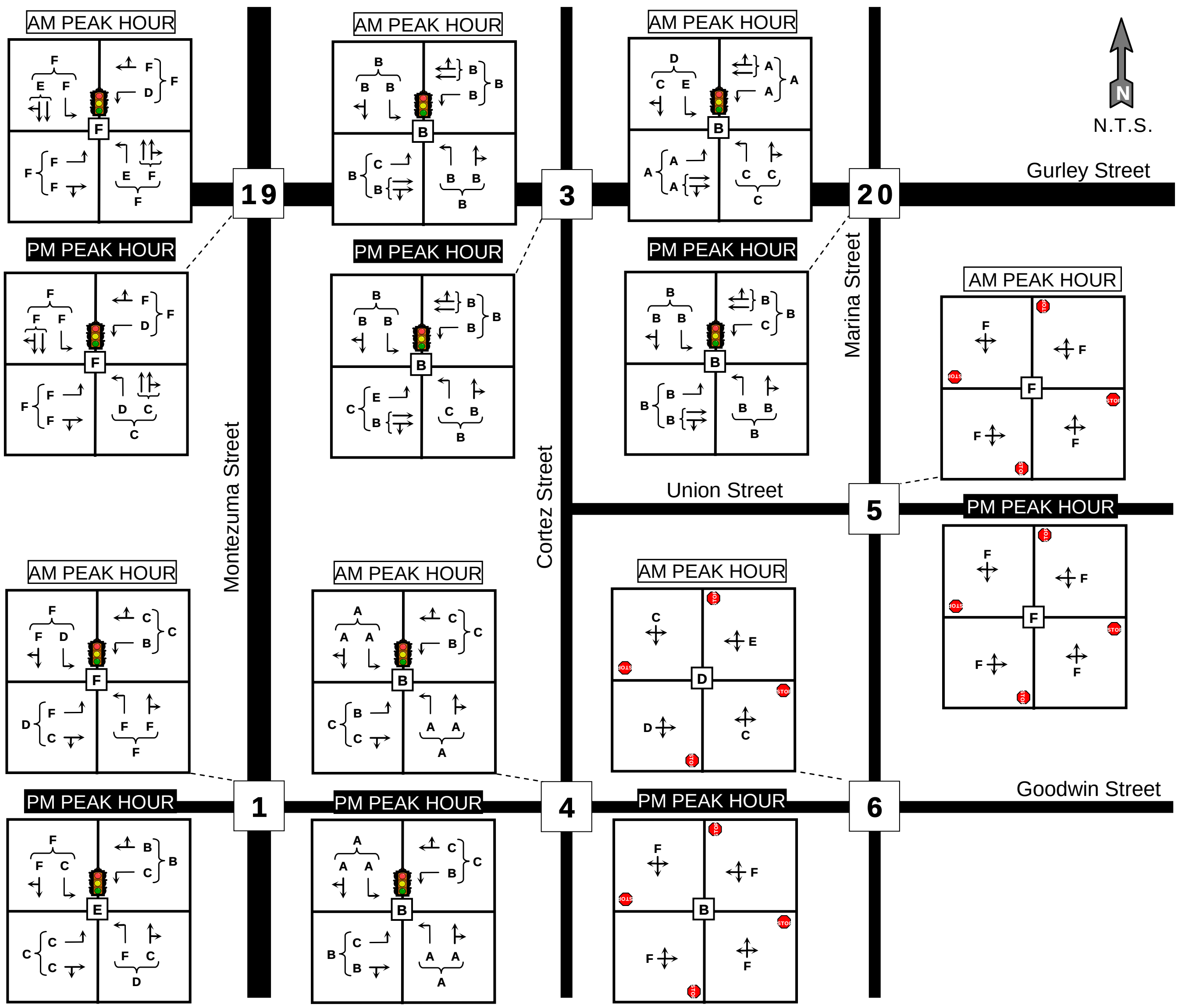

Figure 12: 2030 LOS with No Improvements – AM and PM Peak Hours (1 of 5)

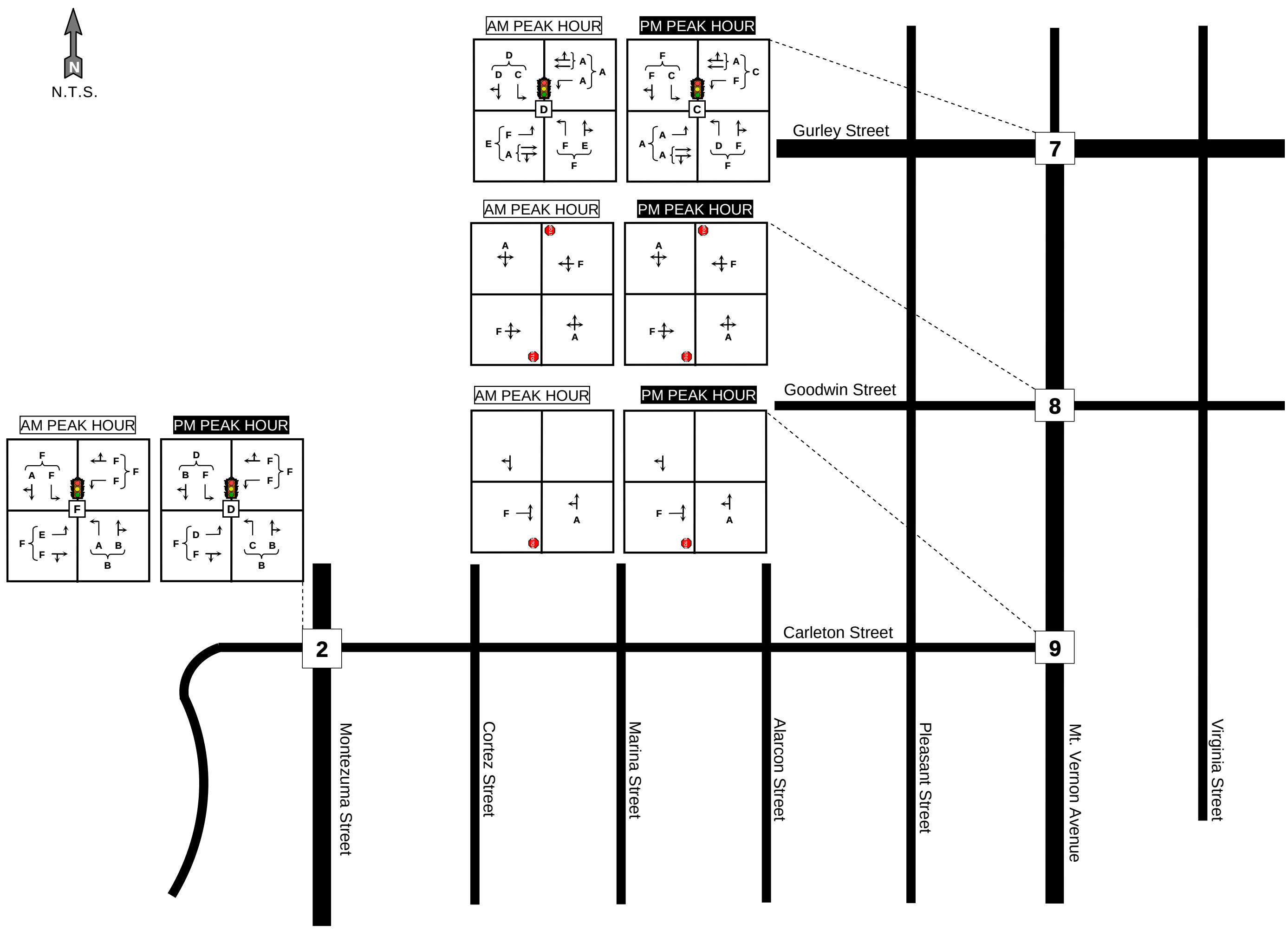

Figure 13: 2030 LOS with No Improvements – AM and PM Peak Hours (2 of 5)

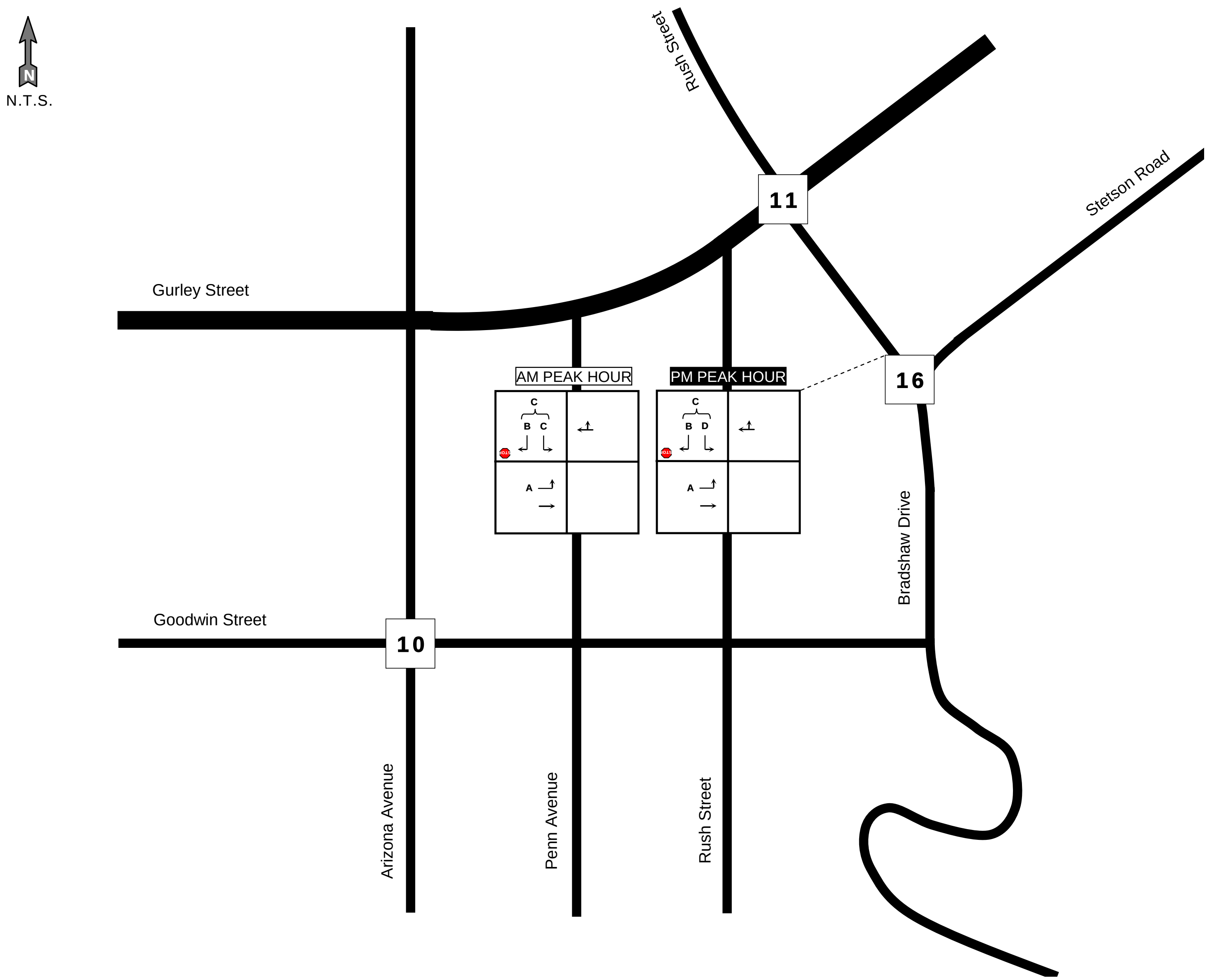

Figure 14: 2030 LOS with No Improvements – AM and PM Peak Hours (3 of 5)

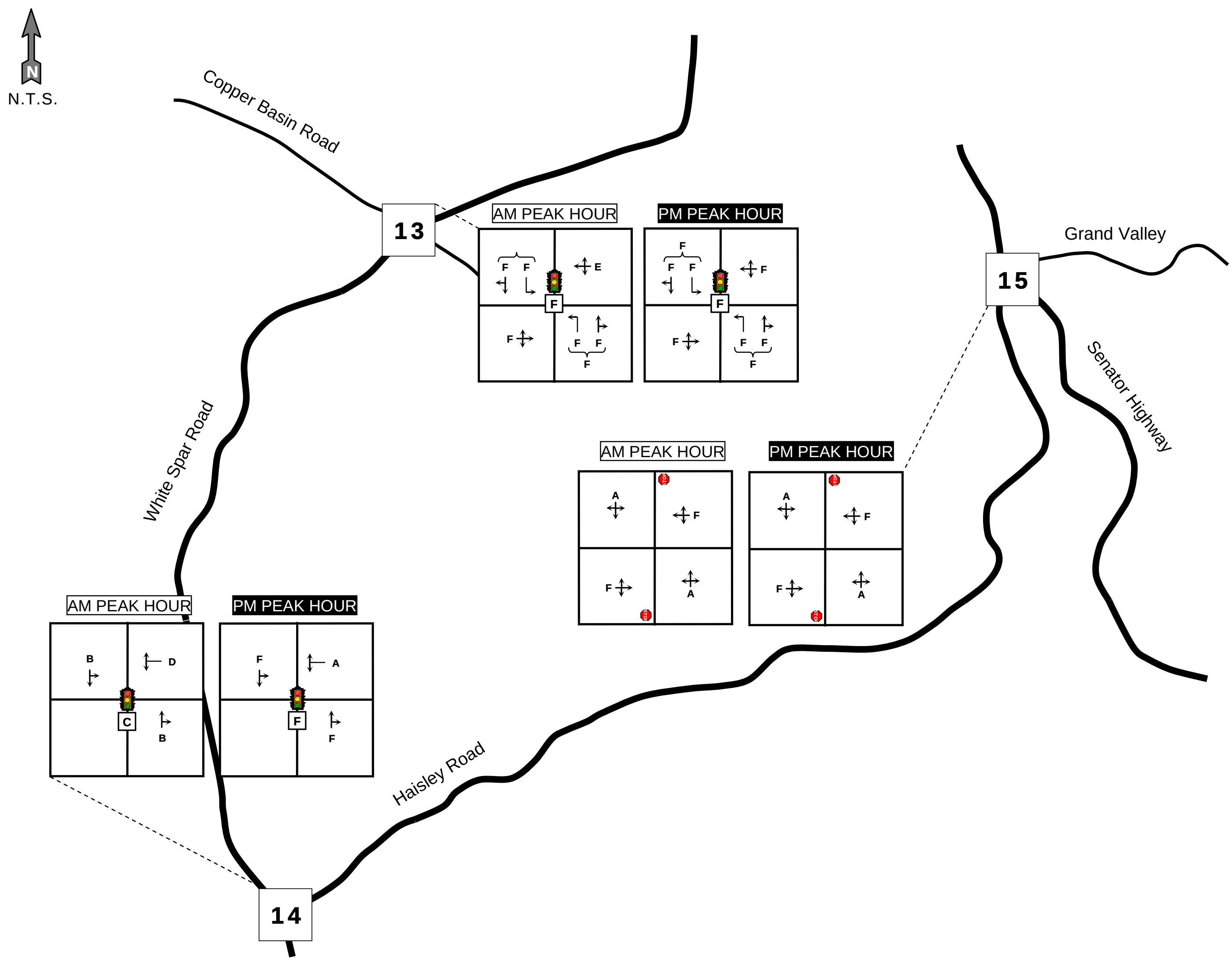

Figure 15: 2030 LOS with No Improvements – AM and PM Peak Hours (4 of 5)

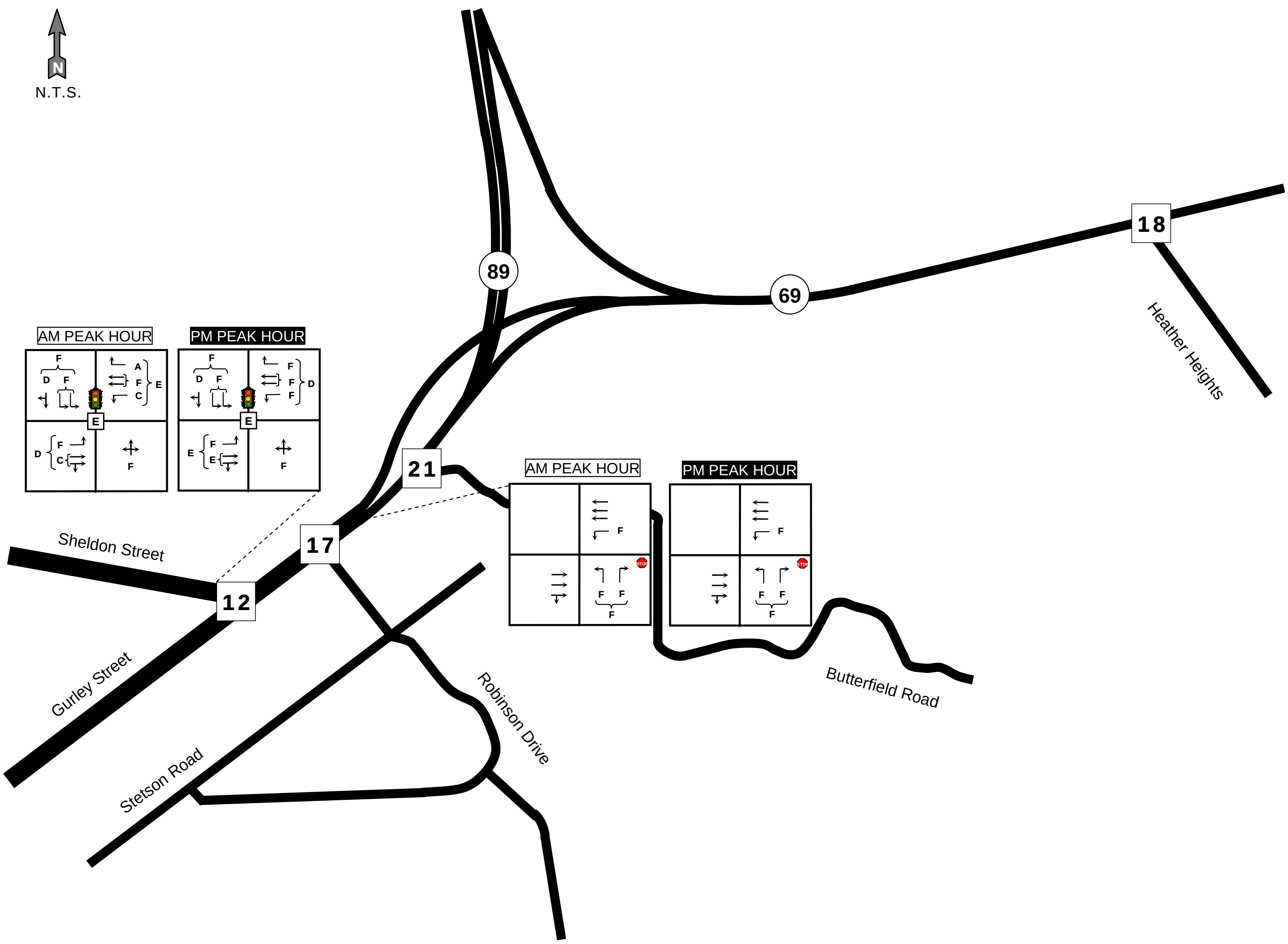

Figure 16: 2030 LOS with No Improvements – AM and PM Peak Hours (5 of 5)

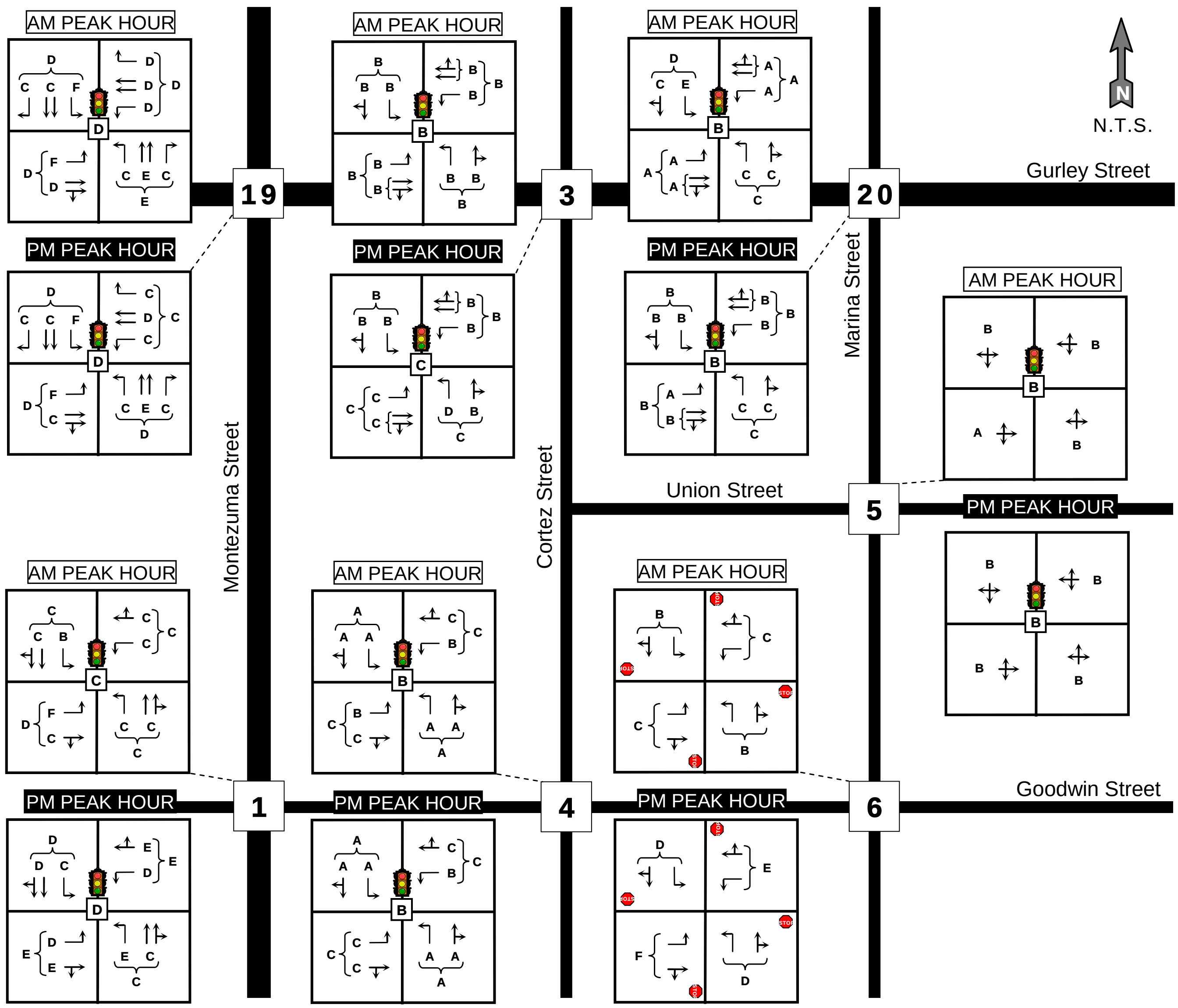

Figure 17: 2030 LOS with Improvements – AM and PM Peak Hours (1 of 5)

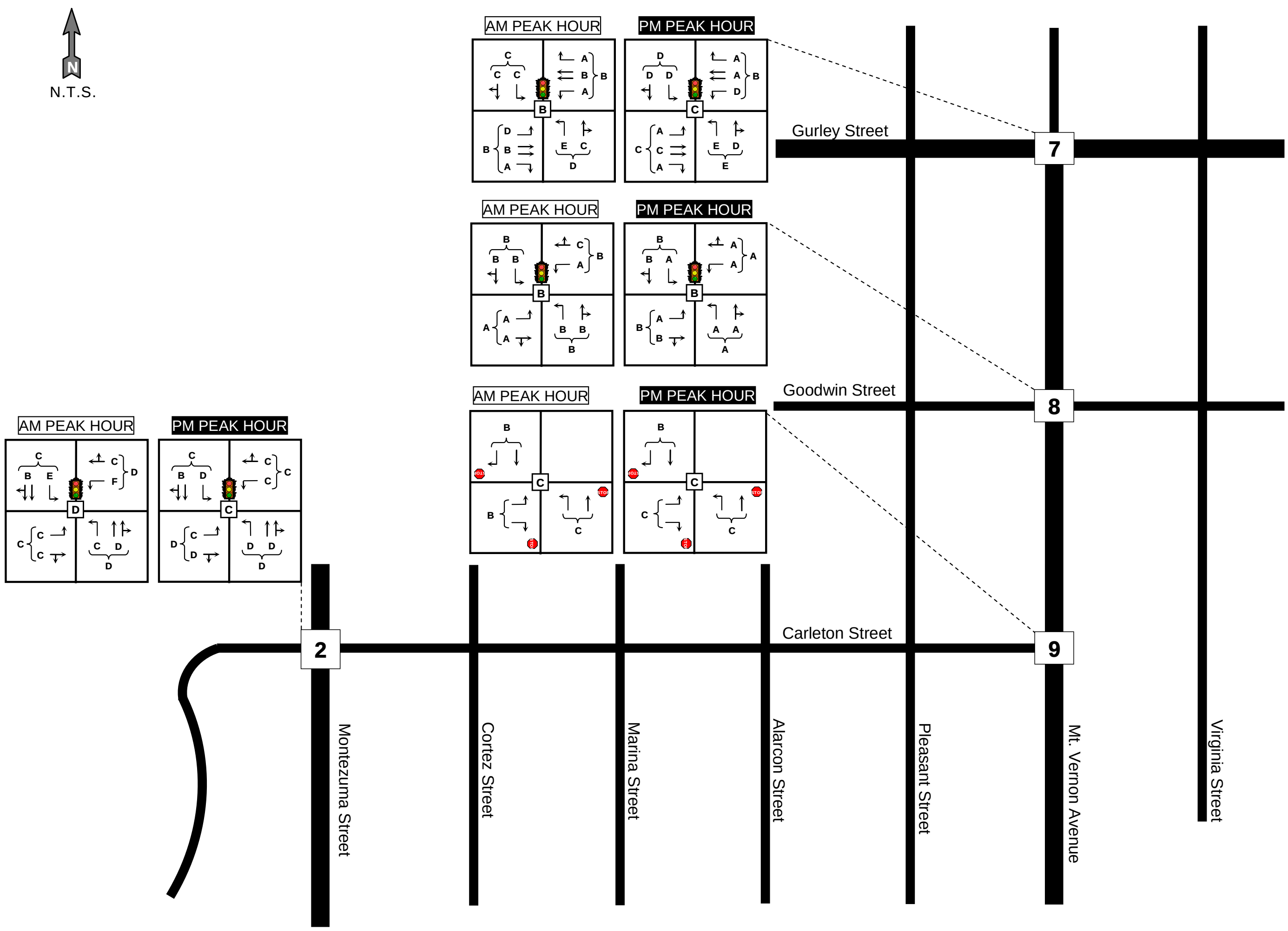

Figure 18: 2030 LOS with Improvements – AM and PM Peak Hours (2 of 5)

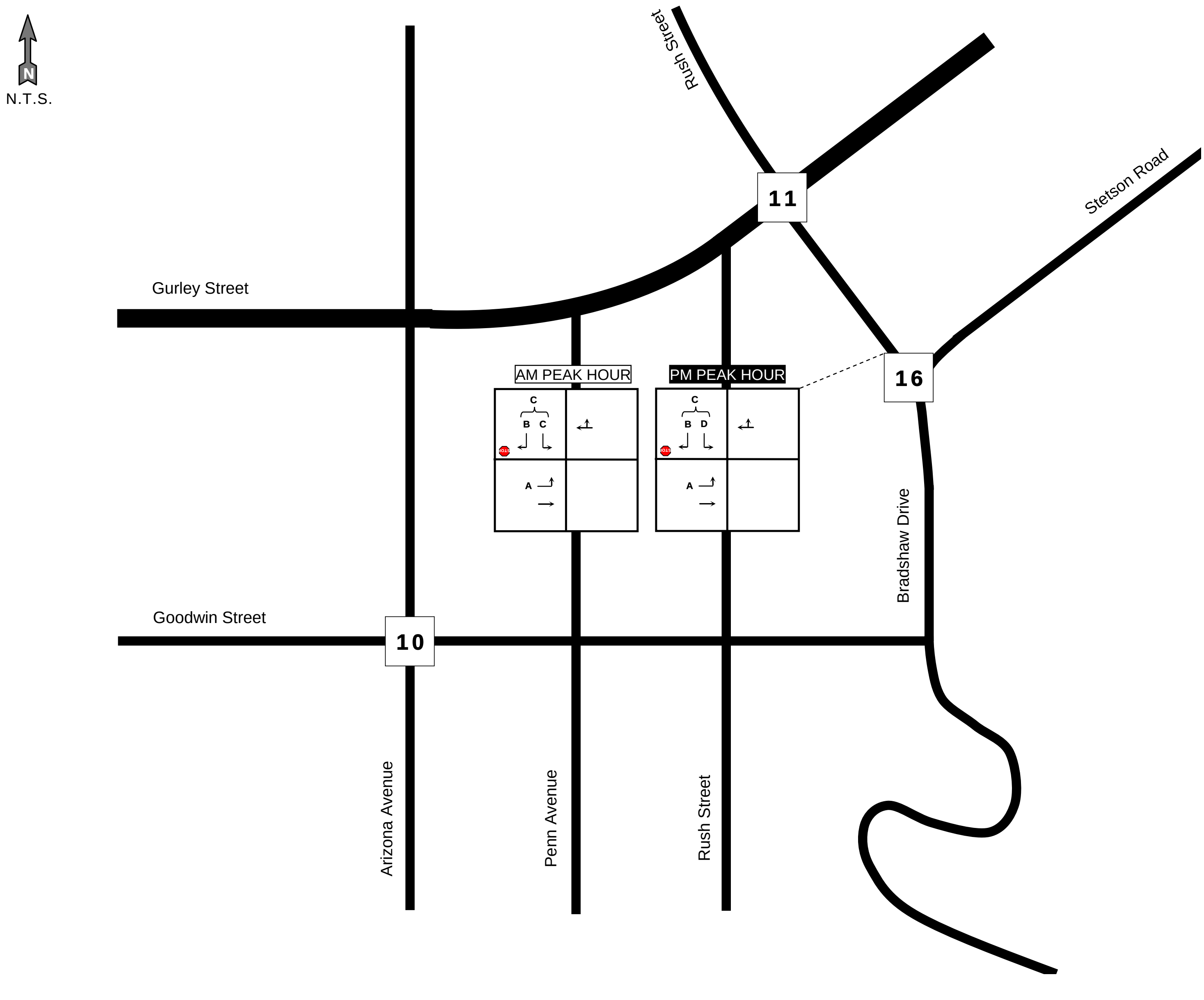

Figure 19: 2030 LOS with Improvements – AM and PM Peak Hours (3 of 5)

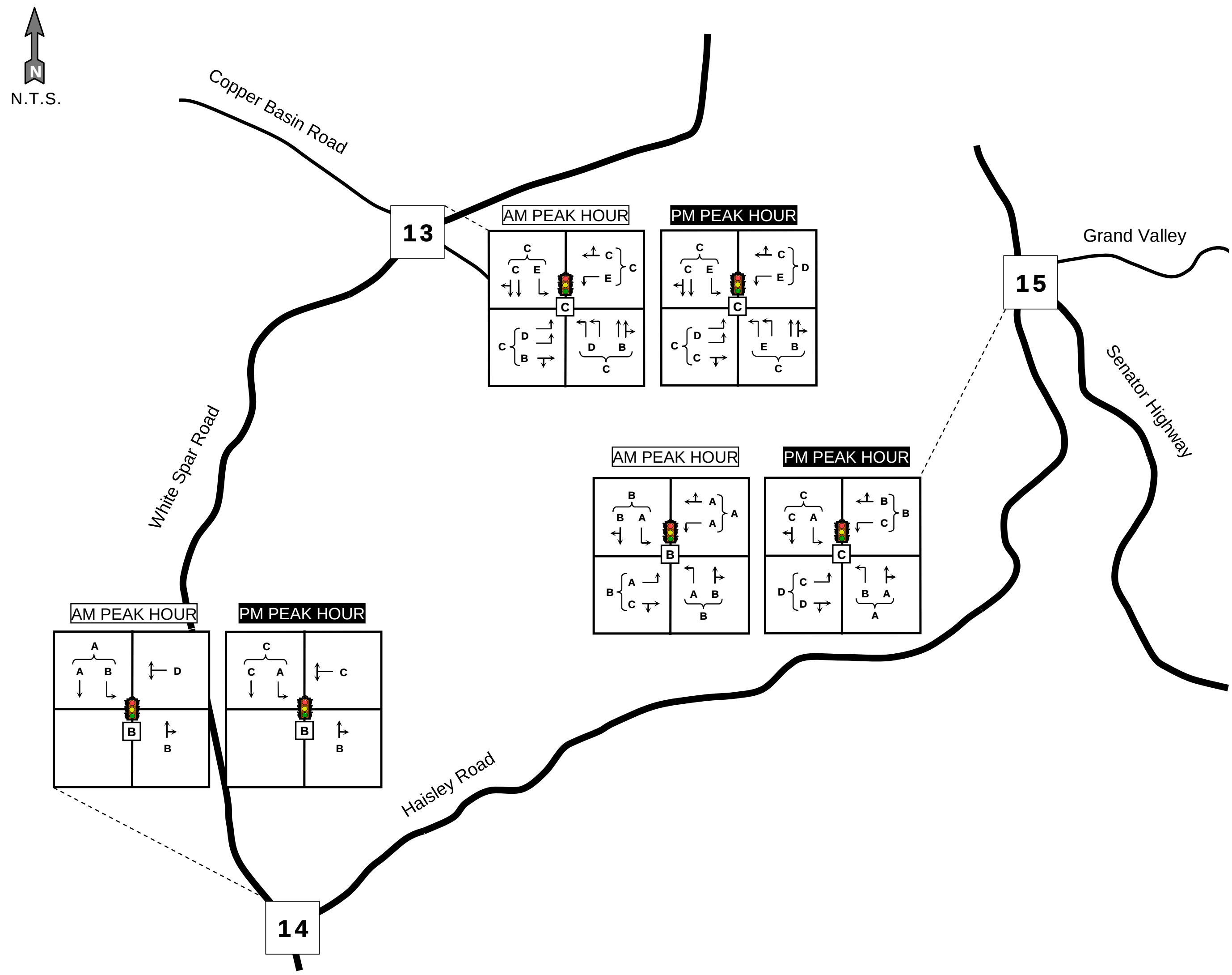

Figure 20: 2030 LOS with Improvements – AM and PM Peak Hours (4 of 5)

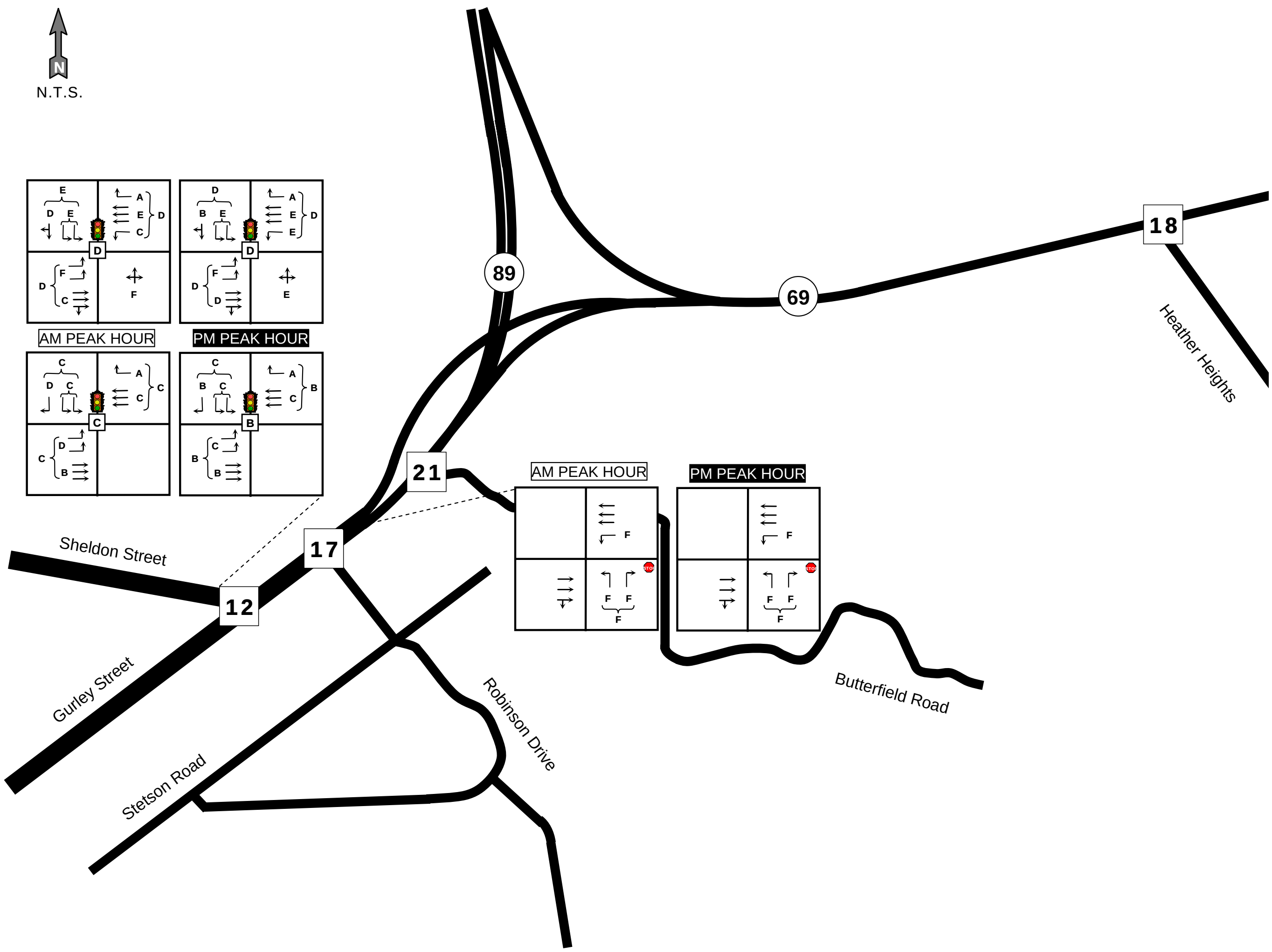

Figure 21: 2030 LOS with Improvements – AM and PM Peak Hours (5 of 5)

Recommendations and Conclusions

The recommended lane configurations and traffic control for the study intersections are shown in **Figure 17** through **Figure 21**, as previously presented in this report. These recommendations produce instances where the 2030 levels-of-service are below “D”. Right-of-way and constructability limitations eliminate potential improvements that would provide level-of-service “D” or better. Additionally, several other improvements, not specifically referenced in the level-of-service figures are recommended below.

The daily traffic volumes for the year 2030, as shown in the planning model, indicate anticipated left-turn volumes at the intersection of Robinson Drive and Gurley Street (Intersection 17). This includes the southwestbound left-turn from Gurley Street onto Robinson Drive, and the northwestbound left-turn from Robinson Drive onto Gurley Street. Accordingly, these volumes were considered in generating morning and evening peak hour volumes, and the intersection was analyzed with these left-turn movements. As indicated in the level-of-service figures, both left-turns operate at “F” in both the morning and evening peak hours.

In addition to high delay, permitting these left-turns also magnifies several other problems. First, inclusion of the left-turns decreases the capacity of the through movements on Gurley Street, and increases the likelihood of vehicle collisions on Gurley Street. Furthermore, due to the proximity of the intersection of Robinson Drive and Gurley Street to the merging of SR-89 and SR-69, weaving issues will be compounded by permitting left-turns at Robinson Road. For these reasons, it is recommended that both left-turns movements be prohibited at this intersection.

Intersection 16, the intersection of Bradshaw Drive and Stetson Road, was modified from its existing geometry for both the 2030 and the 2030 with improvements level-of-service analyses. The traffic model predicts the heaviest volumes on the current southern leg of the intersection (on Bradshaw Drive), and on the east leg of the intersection (Stetson Road.) These traffic patterns suggest constructing the intersection so that the principle through street would be a connection of south Bradshaw Drive and east Stetson Road. North Bradshaw drive would then be a minor roadway which would intersect the new Bradshaw-Stetson alignment, and be stop controlled. This proposed configuration was used for the “2030” and the “2030 with Improvements” scenarios because the existing roadway configuration did not lend itself to an operations analysis with the predicted 2030 volumes.

Intersection 12, the intersection of Sheldon Avenue and Gurley Street, was considered with two possible recommendations: including the southern leg of the intersection, and omitting the southern leg of the intersection. **Figure 21** contains the resulting levels-of-service for both scenarios. Elimination of the southern leg is recommended because the intersection level-of-service improves significantly (from D to C in the morning peak hour and D to B in the evening peak hour) with the closure of the south leg.