CONCURRENT FLOW LANES – PHASE II

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CONCURRENT FLOW LANES - PHASE II

FINAL REPORT

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Maryland State Highway Administration

By

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STATE HIGHWAY ADMINISTRATION

RESEARCH REPORT

CONCURRENT FLOW LANES – PHASE II

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16. Abstract

This report provides the findings from a research effort designed to ascertain whether or not a chosen simulation software platform, the VISSIM micro-simulation platform, provides a suitable environment for modeling and analyzing traffic operations, including the specific details associated with modeling concurrent flow lanes with designated access points, along significant portions of the Maryland freeways.

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EXECUTIVE SUMMARY

This report provides the findings from a research effort designed to ascertain whether or not a chosen simulation software platform, the VISSIM micro-simulation platform, provides a suitable environment for modeling and analyzing traffic operations, including the specific details associated with modeling concurrent flow lanes with designated access points, along significant portions of the Maryland freeways. Details of the roadway geometry, traffic volume, vehicle composition, and vehicle occupancy required for the development of the existing conditions and proposed alternative models of the study roadway segment are provided. Techniques developed to ensure smooth transitioning between lanes and across links in both existing and proposed designs, provide continuous or limited access as required to managed lanes for only a subset of the classes, and ensure consistency in acceleration and deceleration lanes are presented.

In addition to data preparation and modeling work, parameters of the VISSIM simulation software must be set so that traffic measures from the simulation best match actual measurements taken from the field. The process of determining the optimal set of parameters for the existing conditions model so as to minimize error is known as calibration. Initial runs were conducted using default parameter settings. Results from these runs show that mean travel times estimated by the simulation model using default parameters were statistically significantly different from observed mean travel times. Thus, calibration of the parameters is essential. Results of runs of the developed and calibrated existing conditions simulation model show, through a comparison of mean travel times by roadway segment, that the VISSIM simulation platform using described modeling techniques is a suitable tool for modeling the I-270 roadway segment with concurrent flow lane operations. In fact, once calibrated, no significant statistical difference was found between mean segment travel times produced by the simulation and those recorded in

a traffic study on the actual roadway facility for all segments of the study area. Given this ability to match real-world operations using one set of parameters for multiple segments, it is anticipated that the developed models will have significant utility for future simulation studies of the I-270 corridor. Moreover, findings from the calibration effort will provide input of broad utility for VISSIM simulation models of freeways.

Parameters chosen through the calibration effort were employed in additional simulation experiments designed to assess the potential benefits of proposed alternative HOT lane facility designs under 2030 demand estimates. This report describes additional modeling effort required to replicate vehicular behavior in the presence of limited access HOT lane facilities with one or two HOT lanes.

Several findings from the analysis of proposed alternative HOT lane facility designs are suggested from this study. First, the results indicate that the chosen simulation platform and developed modeling techniques can sufficiently replicate traffic under given concurrent flow lane design alternatives. Second, the traffic performance in terms of delay, travel time, density, and fuel consumption is expected to significantly degrade under 2030 demand estimates given no facility upgrade as compared with existing 2006 operations, supporting the argument that capacity expansion is required under the predicted increase in demand. Third, conversion of the existing HOV lane to a single lane (Alternative 1) or double lane (Alternative 5) HOT lane facility results in improved roadway performance as compared with both 2030 No Build and 2006 existing facility design performance under associated demand estimates. Thus, even with increased demand for the I-270 roadway segment, overall performance improves with the conversion. Likewise, despite forecasts of significantly greater throughput, simulation run results indicate that the performance of Alternative 5 is on par with, or at least not much worse than that of Alternative 1. Finally, it was noted that the developed models can be used to identify possible flaws in access design.

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Chapter 1 Introduction

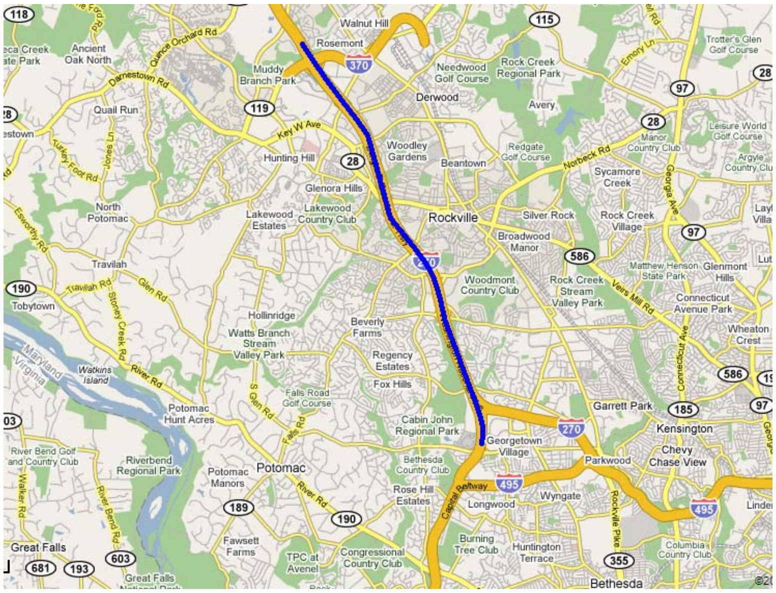
As has been demonstrated in various regions within the United States, the use of Express Toll Lanes (ETLs) or similarly functioning High Occupancy Toll (HOT) lanes can lead to more effective use of existing roadway capacity, improved traffic flow along general purpose lanes and additional revenue to support much needed transportation improvements. This report describes outcomes and efforts taken in the second phase of a multi-phase research effort to develop an application of a simulation model for the analysis of managed lanes adjacent to general purpose lanes (concurrent flow lanes).

Phase I of this project sought to develop a comprehensive understanding of the current state-of-the-art in modeling and analysis of nonbarrier separated electronic/high occupancy toll (HOT) lane and other concurrent flow lane operations as reported in (Miller-Hooks, Tarnoff, Chen and Chou, 2008). As part of the initial effort, information was gathered through interviews conducted with project managers of existing and proposed HOT lane facilities, modelers and other domain experts and review of related reports and literature. Details of models employed, and analytical tools used, to evaluate the impact of proposed HOT lanes on traffic operations and potential revenue; supplemental analysis tools; lane configurations; tolling strategies; High Occupancy Vehicle (HOV) restrictions; types of separation; how weaving is addressed; and design alternatives for ingress and egress between the HOT and general purpose lanes were provided. Knowledge pertaining to model calibration and validation was gleaned from the interview and literature review processes. Potential data sources for calibrating developed models were also identified. Finally, a proof-of-concept was developed to illustrate how details associated with violation modeling can be handled in the selected modeling framework, the VISSIM simulation platform, which was proposed for use in this and additional subsequent phases of this research effort. The VISSIM micro-simulation platform was chosen over other traffic simulators, because this platform had been successfully employed in modeling the impact of proposed HOT lane facilities on traffic operations in several studies conducted across the country as described in (Miller-Hooks, Tarnoff, Chen and Chou, 2008). While nearly all of these models treated the HOT lane facility as a separate link, effectively modeling a barrier separated facility, preliminary work within the platform indicated that this platform could also successfully be used to model nonbarrier separated facilities.

The primary purpose of this second phase of this research effort was to ascertain whether or not the chosen simulation software platform, the VISSIM simulation platform, and modeling methodologies provide a suitable framework for modeling and analyzing traffic operations, including the specific details associated with modeling concurrent flow lanes with designated access points, along significant portions of the Maryland freeways. While the intended use of these lanes is for non-intrusive (barrierless) tolling, the model will also be useful for studying the performance of HOV lane operations. An additional goal of this research phase was to gain initial insight into the performance of proposed HOT lane facility designs.

To complete this assessment, simulation models of four managed lane design alternatives associated with a 7-mile stretch of I-270 within the State of Maryland were developed: two existing condition models with mostly continuous access HOV lane operations under 2006 and 2030 traffic demand and two alternative models with limited access HOT lane facilities under 2030 demand. The study area, a segment of Southbound I-270 from I-370 to the Spur, is depicted in Figure 1. The study period is from 6:00 a.m. through 9:00 a.m., i.e. the morning peak hours. Parameters of the existing conditions model with 2006 traffic demand were calibrated based on actual traffic measurements. This report describes the developed simulation models, data employed within the modeling and calibration efforts, efforts taken to calibrate the existing conditions model, and results and findings from the assessment of the calibration effort and evaluation of proposed design alternatives.

Figure 1 – Study Area: Southbound lanes of I-270 from I-370 to the Spur



Data related to roadway geometry, traffic volume, vehicle composition, and vehicle occupancy are required for the development of the VISSIM model of the 7-mile stretch of I-270. Details associated with the preparation of these required input data are given in Chapter 2. In Chapter 3, the general approach to modeling the study roadway segment and specific implementation details are presented. Difficulties that arose in the modeling effort are described and measures taken to overcome these difficulties are provided. Once created, preferred parameters for use in the VISSIM model of the study roadway segment under existing conditions were identified through extensive calibration efforts as described in Chapter 4. In Chapter 5, proposed alternative designs for the nonbarrier separated HOT lane facility that would replace the HOV lane facility were described. The approach employed within this effort to model access points to the HOT lane facilities is presented and results of analysis of the proposed design alternatives are given. Finally, findings from this research effort, including an assessment of the simulation tool's adequacy in replicating actual traffic on managed lanes, and evaluation of various concurrent flow lane design alternatives are summarized in Chapter 6.

Chapter 2 Input Data

Data related to roadway geometry, traffic volume, vehicle composition, and vehicle occupancy are required for the development of the existing conditions and proposed alternative VISSIM models of the 7-mile stretch of I-270. Details associated with the preparation of these required input data are given next.

2.1 Roadway Geometry

The geometry of the study roadway segment, including characteristics of the interchanges, and general purpose, HOV and collector-distributor (CD) lanes, were extracted from maps available through GoogleMap. A scale of 1:100 meters was employed for this purpose. The study roadway segment consists of six interchanges connecting I-270 with local roads, including I-370 freeway, Shady Grove Road, Montgomery Avenue (MD 28), Falls Road (MD 189), Montrose Road, and the Spur connection to I-495. The interchanges involve eight on-ramps from local roads to CD lanes, five off-ramps from the CD lanes to the local roads, four slip ramps from CD lanes to general purpose (GP) lanes, and two slip ramps from GP lanes to CD lanes.

The I-270 facility hosts a single HOV lane in the southbound direction. This lane spans the entirety of the seven-mile study segment and beyond. The HOV lane splits at the Spur, connecting to I-495 Southbound and Eastbound. Continuous-access to the HOV lane is permitted from the northern-most point of the study roadway segment (at the I-370 interchange) to one mile north of the Spur, at which point access is closed via solid striping.

The study roadway can be divided into three segments with constant cross section, the latter two of which are depicted in Figure 2: I-370 to Shady Grove Road, Shady Grove Road to Montrose Road and Montrose Road to the Spur. There are three, rather than two, southbound CD lanes from I-370 to Shady Grove Road. All lanes within the study roadway segment have 12-foot widths.

Figure 2a – Typical Cross Section – Existing (Shady Grove Road to Montrose Road)

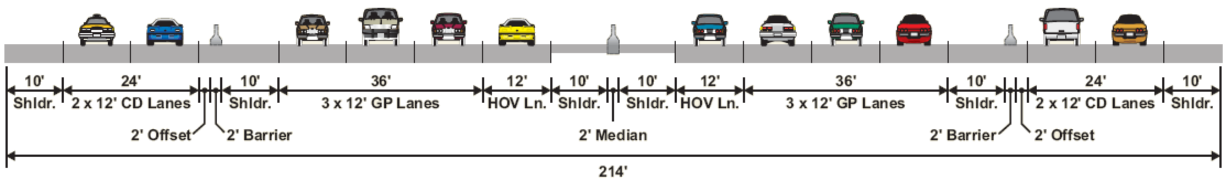
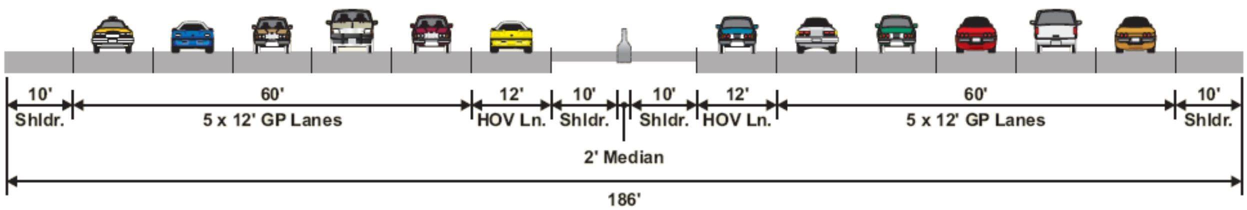


Figure 2b – Typical Cross Section – Existing (Montrose Road to Spur)



Two alternative HOT lane facility designs were considered in this study. The first, known as Alternative 1, employs the existing road layout converting the HOV lane facility to a single, limited access non-barrier separated HOT lane. The second, known as Alternative 5, converts the HOV lane facility to a limited access non-barrier separated HOT lane facility with two HOT lanes. This design accommodates two HOT lanes by converting the inside shoulder (and reducing the shoulder width), as well as the HOV lane, and restriping. Cross sections for these alternatives for portions of the study roadway segment are illustrated in Figures 3 and 4.

Figure 3a – Typical Cross Section – Alternative 1 (Shady Grove Road to Montrose Road)

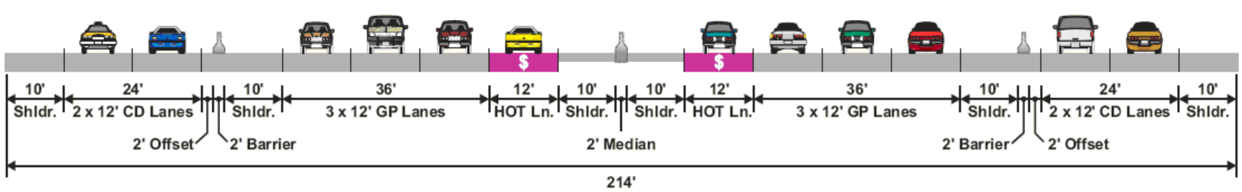


Figure 3b– Typical Cross Section – Alternative 1 (Montrose Road to Spur)

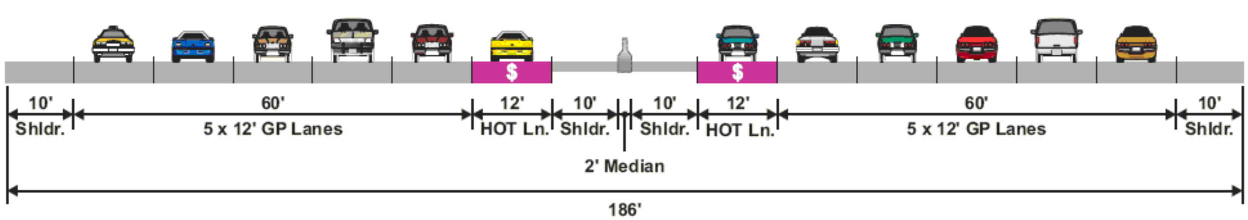


Figure 4a – Typical Cross Section – Alternative 5 (Shady Grove Road to Montrose Road)

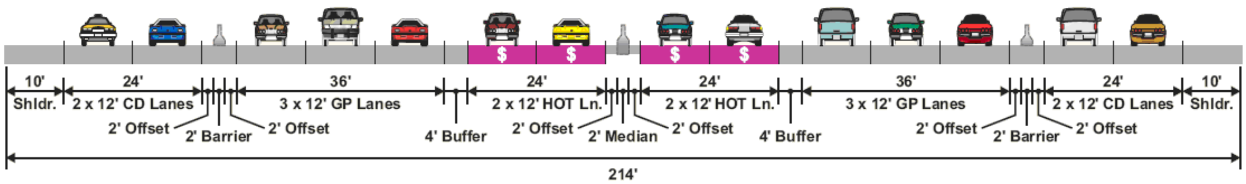
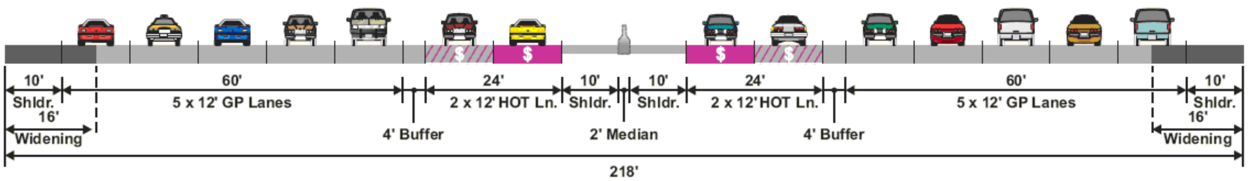


Figure 4b – Typical Cross Section – Alternative 5 (Montrose Road to Spur)



Source from: Maryland State Highway Administration (SHA)

The number of CD, GP and managed lanes for each portion of the study roadway segment are given in Table 1.

Table 1 – Number of Lanes in Existing and Alternatives Designs

Segment of I-270	I-370 to Shady Grove Road			Shady Grove Road to Montrose Road			Montrose Road to Spur		
Type of Lane	CD Lane	GP Lane	HOV/ HOT	CD Lane	GP Lane	HOV/ HOT	CD Lane	GP Lane	HOV/ HOT
Existing	3	3	1 HOV	2	3	1 HOV	0	5	1 HOV
Alternative 1	3	3	1 HOT	2	3	1 HOT	0	5	1 HOT
Alternative 5	3	3	2 HOT	2	3	2 HOT	0	5	2 HOT

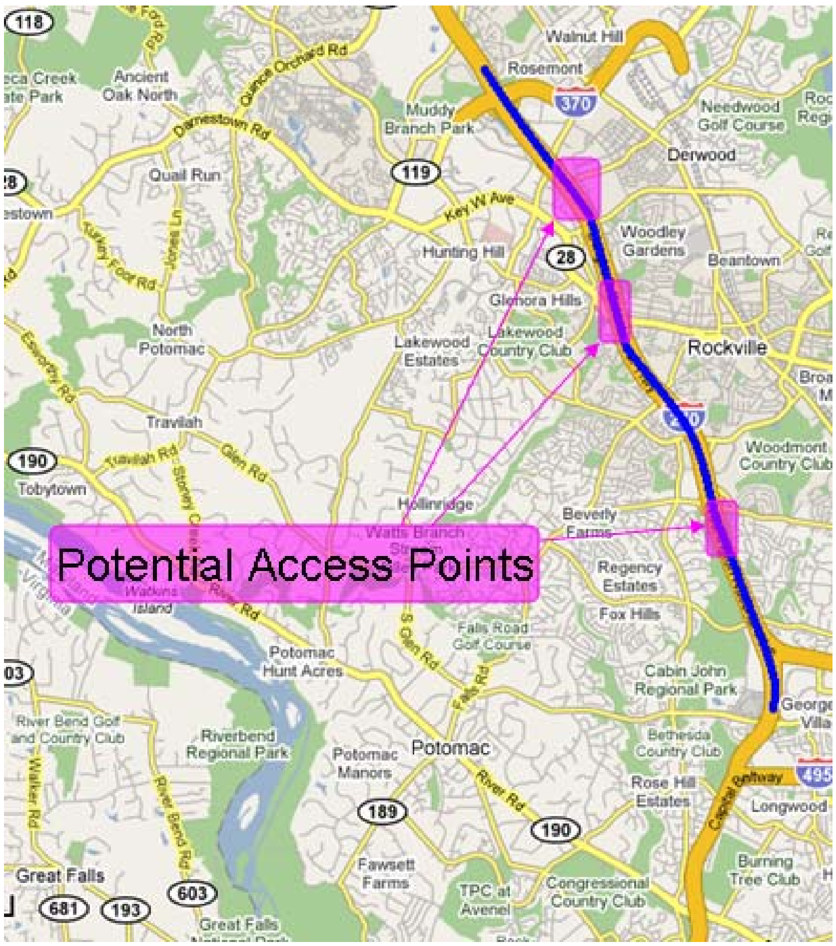
For each alternative HOT lane facility design, three access points to the facility were designated (depicted in Figure 5):

Access Point 1: 3,000 feet long; beginning 0.5 mile south of I-370 and ending at Shady Grove Road; allowing access from managed lanes to slip ramp to CD lanes located south of Shady Grove Road.

Access Point 2: 3,000 feet long; beginning 2,000 feet north of MD 28 and ending 1,000 feet south of MD 28; allowing vehicles in CD lanes to access managed lanes from slip ramp located south of Shady Grove Road and vehicles from managed lanes to access slip ramp to CD lanes located north of MD 189.

Access Point 3: 2,500 feet long; beginning 400 feet north of Montrose Road and ending 2,100 feet south of Montrose Road; allowing vehicles in CD lanes to access managed lanes from slip ramp located north of Montrose Rd.

Figure 5 – Access Point Locations for Alternative Designs



2.2 Traffic Volume

The VISSIM simulation software platform permits the input of traffic volume data (i.e. the demand) to be provided in either of two formats: as an origin-destination (O-D) matrix (indicating the number of vehicles that desire to travel between each O-D pair) or using turning percentages at interchanges and between CD lanes, GP lanes and other concurrent flow lanes. With either format, vehicle movements can be simulated between origins and destinations over the simulation period. Additionally, the O-D matrix and turning percentages may be dynamic, i.e. they may vary over the course of the simulation period. For example, O-D matrices may be created for every 15-minute interval. In this study, turning percentages are used to direct traffic between lanes and to potential destinations (specifically, exits from I-270). Traffic demand is assumed to be constant over the study period and, thus, only a single set of turning percentages is employed. Traffic demand and related turning percentages throughout the study roadway segment for existing conditions and the proposed alternatives were set based on two main sources of data:

1. Balanced morning-peak average hourly traffic volumes and turning rates of on- and off-ramps at interchanges employed within the Maryland SHA Western Mobility Study 2006 (Appendix A).
2. Maryland SHA CORSIM model estimates of turning rates on slip ramps between CD and GP lanes.

The 2006 existing condition traffic volumes were computed using data collected from the field. Traffic volume predictions were also completed for 2030 for each of three possible roadway geometries: No Build, Alternative 1 and Alternative 5. Traffic volumes and turning percentages for 2006 existing conditions and 2030 predictions obtained from these data sources are synopsized in Table 2 and Figures 6 through 9. These data were collected for each of five segments along the entire study roadway segment, depicted in Figure 10. Note that the middle three segments together constitute the Shady Grove Road to Montrose Road segment of the three-segment study roadway depiction used in Section 2.1 to describe roadway geometry.

Turning percentages along the study roadway segment for the 2030 forecast year were set to ensure consistency in flow across the segments. 2006 slip ramp usage rates were employed with some modifications that were applied to ensure consistency with 2030 demand estimates, which were given by lane classification (CD, GP or managed lanes).

Figure 6 – Synopsis of 2006 Traffic Volume and Turning Rates

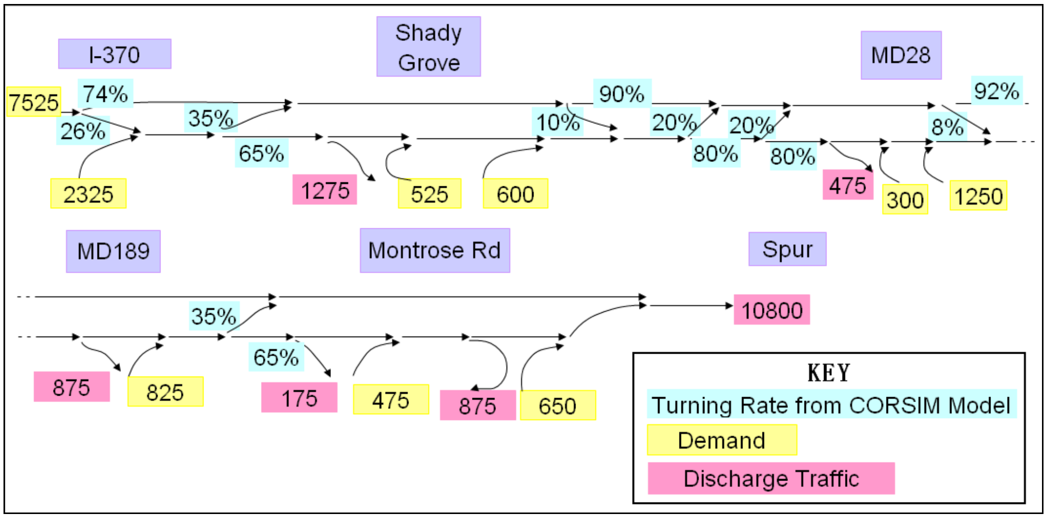


Figure 7– Synopsis of 2030 No Build Traffic Volume and Turning Rates

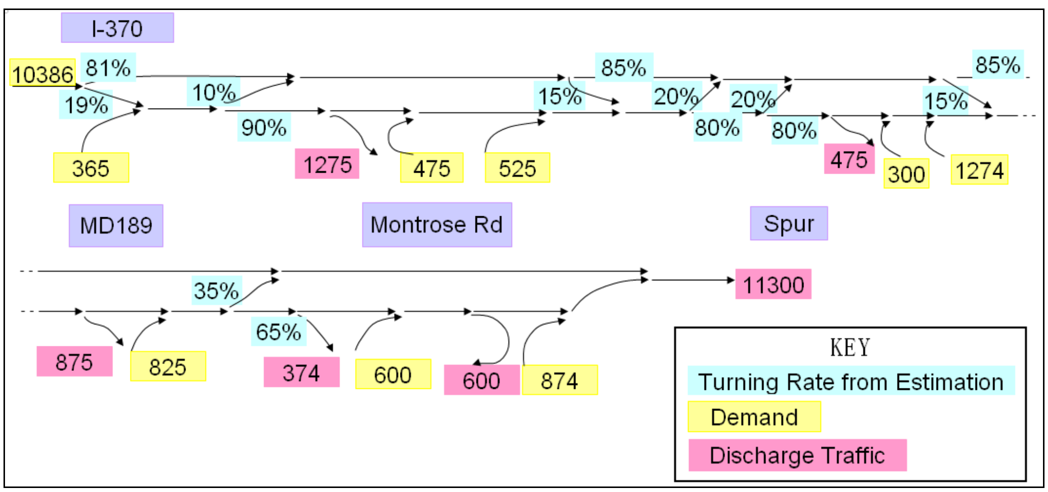


Figure 8 – Synopsis of 2030 Alternative 1 Traffic Volume and Turning Rates

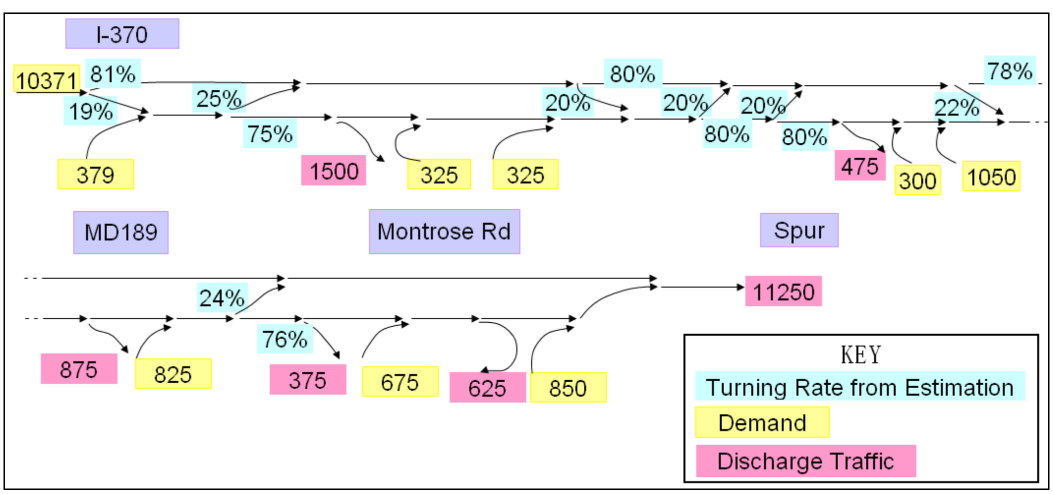


Figure 9 – Synopsis of 2030 Alternative 5 Traffic Volume and Turning Rates

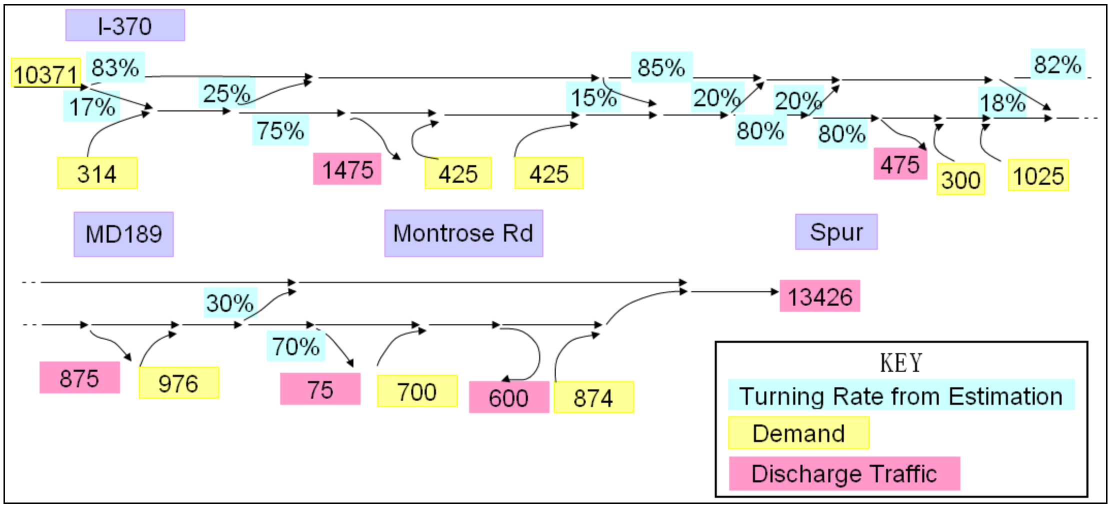
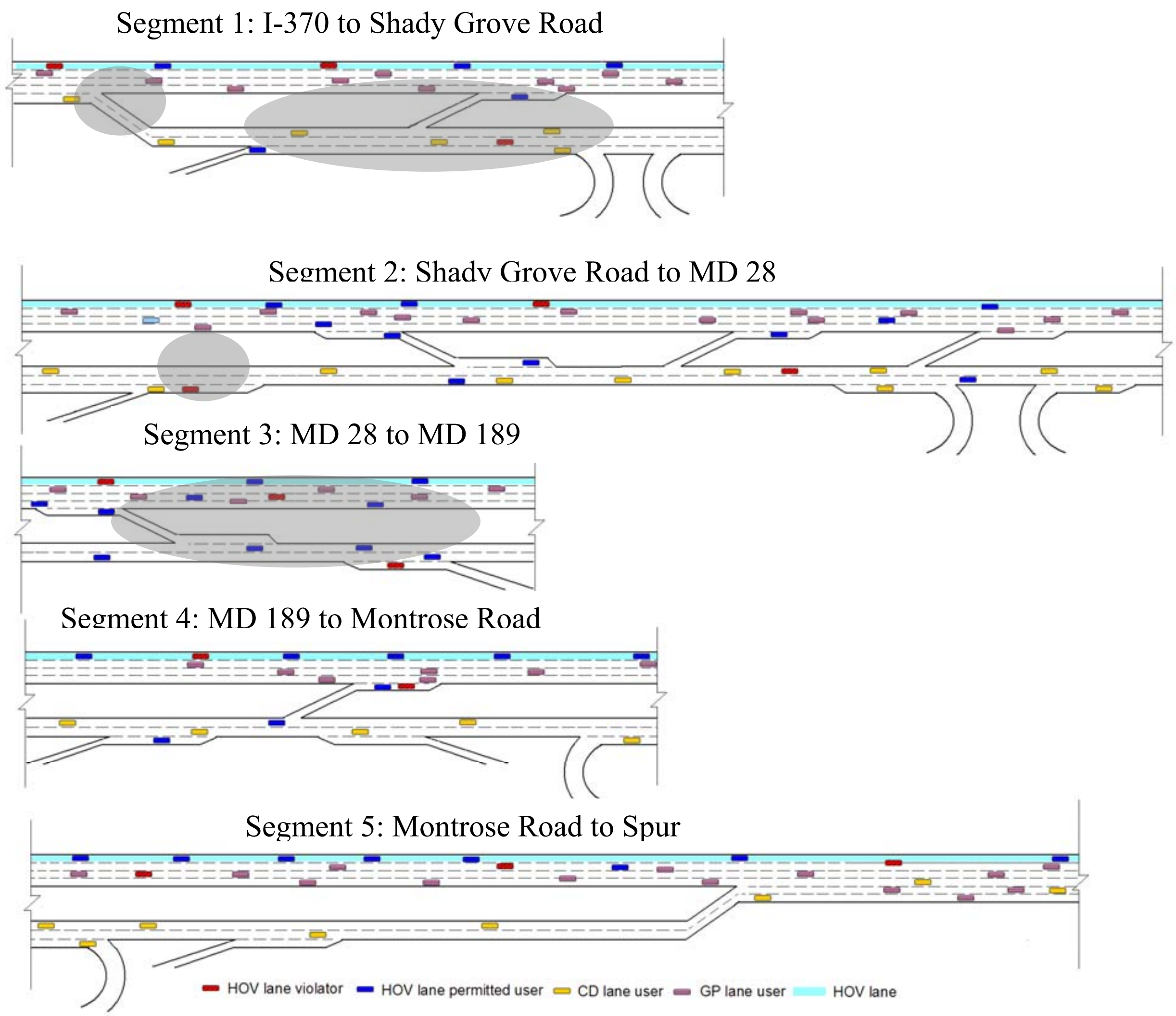


Table 2 – The Traffic Volume per Lane of Existing and Alternatives by Segment

2006 Existing										
Segment	**HOV**			**GP**			**CD**			**Total Volume**
	# of lanes	**volume**	**%**	**# of lanes**	**volume**	**%**	**# of lanes**	**volume**	**%**	
1	1	1478	15	3	6206	63	3	2167	22	9851
2	1	1261	13	3	5626	58	2	2813	29	9700
3	1	1293	12	3	6034	56	2	3448	32	10775
4	1	1287	12	3	5363	50	2	4076	38	10726
5	1	1404	13	3	9396	87	--	--	--	10800
2030 No Build										
Segment	**HOV**			**GP**			**CD**			**Total Volume**
	# of lanes	**volume**	**%**	**# of lanes**	**volume**	**%**	**# of lanes**	**volume**	**%**	
1	1	1613	15	3	6773	63	3	2365	22	10751
2	1	1268	13	3	5655	58	2	2828	29	9751
3	1	1197	12	3	5586	56	2	3192	32	9975
4	1	1251	12	3	5213	50	2	3962	38	10426
5	1	1469	13	3	9831	87	--	--	--	11300
2030 Alternative 1										
Segment	**HOV**			**GP**			**CD**			**Total Volume**
	# of lanes	**volume**	**%**	**# of lanes**	**volume**	**%**	**# of lanes**	**volume**	**%**	
1	1	1600	15	3	6771	63	3	2379	22	10750
2	1	1600	16	3	5561	56	2	2739	28	9900
3	1	1600	16	3	4731	48	2	3569	36	9900
4	1	1600	15	3	5775	56	2	2975	29	10350
5	1	1600	14	3	9650	86	--	--	--	11250

2030 Alternative 5										
Segment	HOV			GP			CD			Total Volume
	# of lanes	volume	%	# of lanes	volume	%	# of lanes	volume	%	
1	2	3300	27	3	6586	54	3	2314	19	12200
2	2	3300	29	3	5544	48	2	2731	24	11575
3	2	3325	29	3	4688	41	2	3537	31	11550
4	2	3325	27	3	6023	48	2	3103	25	12451
5	2	3225	24	3	10200	76	--	--	--	13425

Figure 10 – I-270 7-mile Roadway Stretch



The data from Figures 6 through 9 was employed in computing the traffic volume to be loaded into the links of the VISSIM model and turning percentages to be employed between CD and GP lanes and at interchanges. This information is depicted in the traffic flow chart presented in Figure 11. Similar figures are provided in Figures 12 through 14 for the 2030 options.

Figure 11 – Traffic Flow Chart for Existing 2006 VISSIM Model

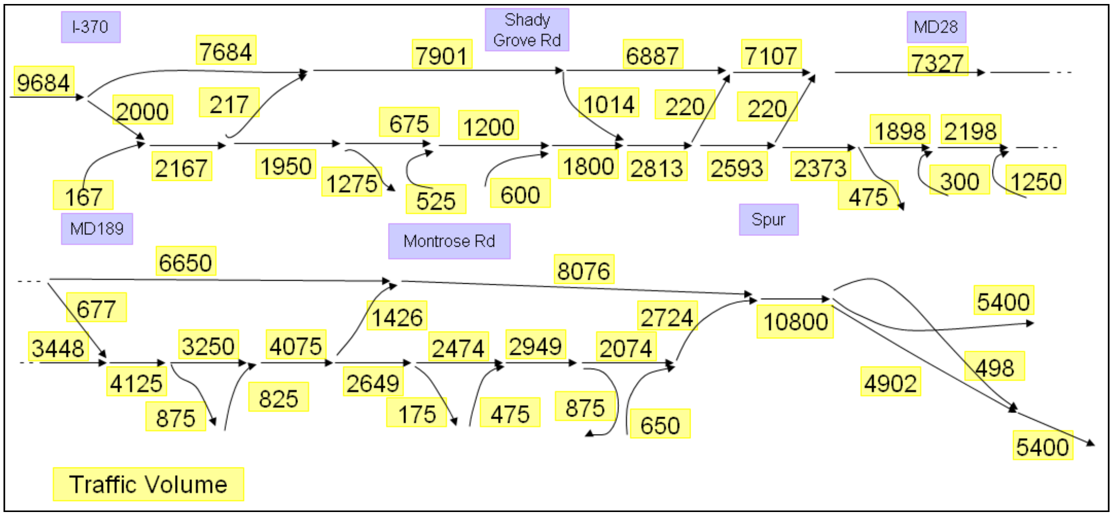


Figure 12 – Traffic Flow Chart for Existing 2030

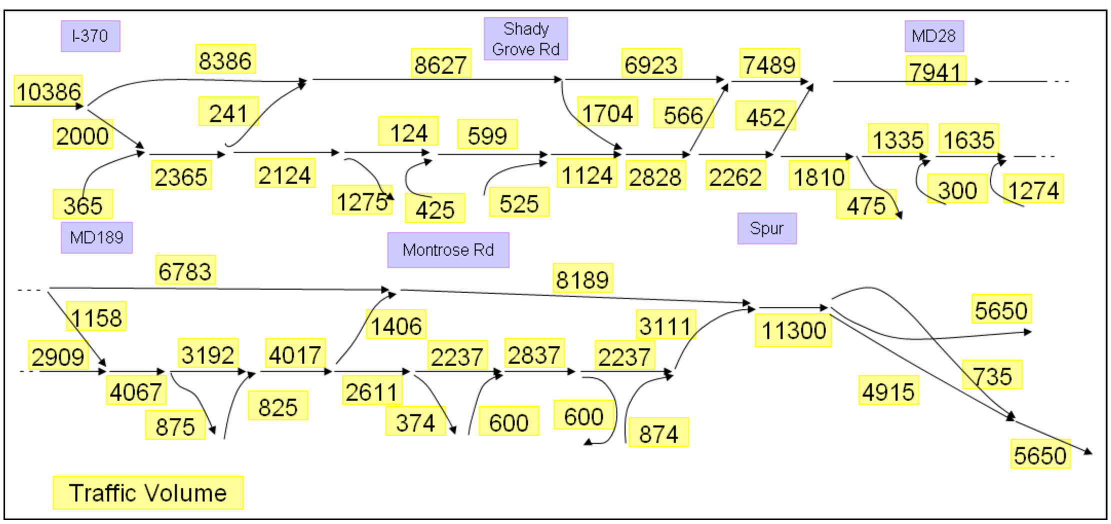


Figure 13 – Traffic Flow Chart for Alternative 1 2030

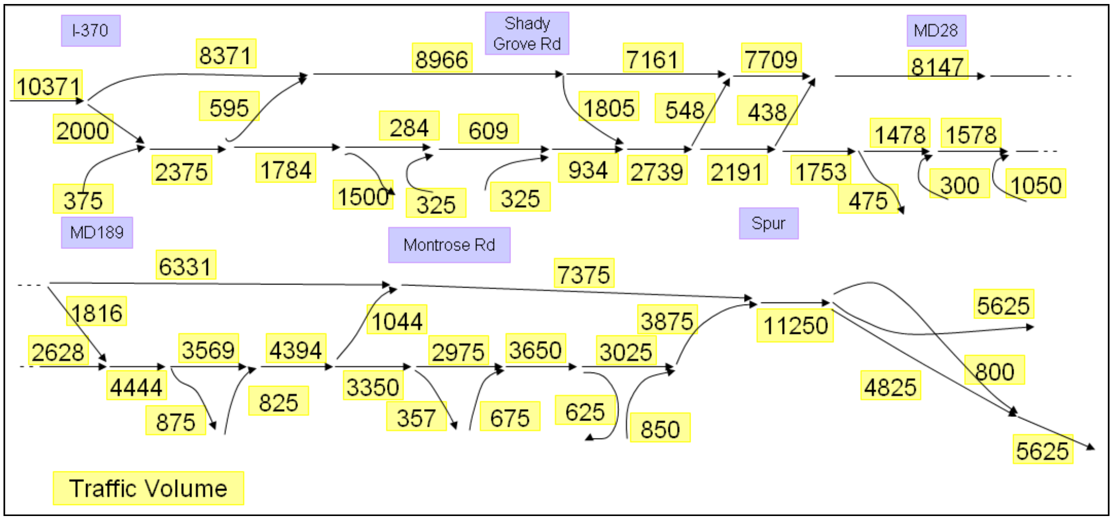
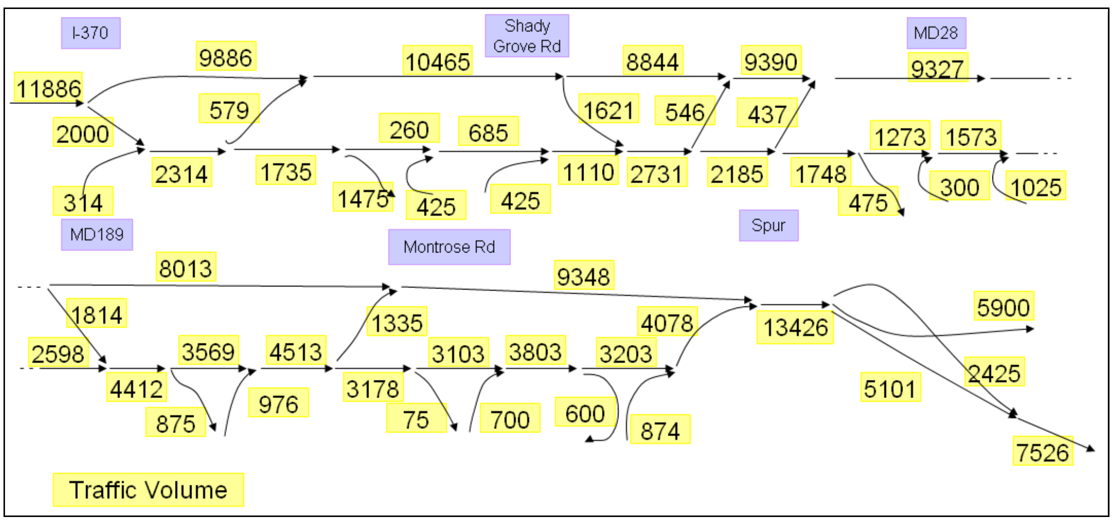


Figure 14 – Traffic Flow Chart for Alternative 5 2030



2.3 Vehicle Occupancy and Composition

Vehicle occupancy, i.e. the number of occupants (including the driver) riding in each vehicle, is a significant characteristic in terms of describing a vehicle's type in the context of this managed lane study. That is, a vehicle will be permitted to use the HOV lane in the existing conditions model during the study period only if that vehicle contains 2 or more occupants. Likewise, a vehicle will be permitted to use the HOT lane(s) in the alternative roadway configurations considered in this study if that vehicle contains 2 or more occupants or is a suitably equipped HOT lane user.

Average morning peak-hour hourly vehicle occupancy data employed within this study was based on data obtained via a survey conducted between 6:00 a.m. and 9:00 a.m. on May 23, 2006. This survey was conducted at one hour intervals at two stations, one located north of Democracy Boulevard and the second located south of Shady Grove Road. Vehicles were categorized as one of several types: personal cars with a single occupant (the driver), personal cars with a driver and one or more passengers, buses (assumed to carry 20 passengers), and trucks. Each lane was counted separately and the average per lane hourly occupancies were computed. The relevant average morning peak-hour number of vehicles per lane per hour by occupancy category is shown in Table 3.

Table 3 – Average Hourly Vehicle Occupancy during A.M. Peak in 2006

Lane	Vehicle Type			
	1*	2+**	Buses	Trucks
Southbound I-270 Spur North of Democracy Boulevard				
Lane 1 – GP	659	24	1	13
Lane 2 – GP	1607	134	3	123
Lane 3 – GP	1969	76	0	44
Lane 4 – HOV	161	484	5	10
Southbound I-270 South of Shady Grove Road				
Lane 1 – CD	1278	124	8	92
Lane 2 – CD	1587	98	3	70
Lane 3 – GP	709	43	2	24
Lane 4 – GP	1535	227	4	125
Lane 5 – GP	1693	13	0	39
Lane 6 – HOV	278	1128	17	9

* Passenger cars or vans with occupancy equals to one.
** Passenger cars or vans with occupancy higher than one.

The fraction within each category (i.e. the number of vehicles within each category as a fraction of the total number of vehicles in the roadway segment) is presented in Table 4. Note that it was assumed that this fraction is constant over the entire segment.

Table 4 – Fraction within each Vehicle Occupancy Category (2006)

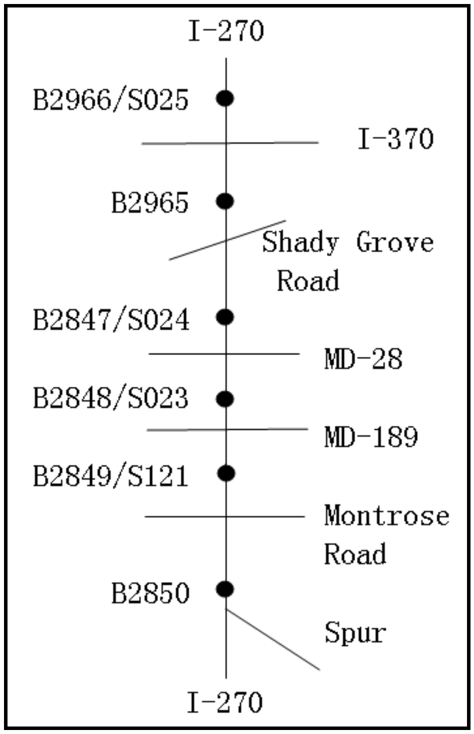
Segment	Lane	Total	1*		2+*		Buses		Trucks	
I-370 to Montrose Road	GP	5315	4235	79.7%	238	4.5%	4	0.1%	181	3.4%
	HOV		161	3.0%	490	9.2%	5	0.1%	10	0.2%
Montrose Road to Spur	CD+ GP	9106	6803	74.7%	521	5.7%	16	0.2%	350	3.8%
	HOV		278	3.0%	1145	12.6%	17	0.2%	9	0.1%

* Passenger cars or vans with occupancy of one.
** Passenger cars or vans with occupancy higher than one.

Additional survey data (provided by the Maryland SHA) obtained from six survey stations shown in Table 5 were available for use in this study. The location of survey stations are shown in Figure 15. For the five survey stations located between I-370 and Montrose Road, a 48-hour vehicle composition survey was taken each year during August of 2005 and April, May and August of 2007. For survey stations located between Montrose Road and the Spur, the 48-hour vehicle composition survey was taken in August of 2005 only. Traffic counts by vehicle class were recorded at one hour intervals. The following classes were considered.

- Class 1 – Motorcycles (MC);
- Class 2 – Passenger Cars;
- Class 3 – Light Trucks;
- Class 4 – Buses;
- Classes 5-9 – Single-Trailer Trucks; and
- Classes 10-13 – Multi-Trailer Trucks.

Figure 15 – Vehicle Composition Survey Station Locations



The fraction of vehicles falling within each category was obtained by dividing the number of vehicles of a given class by the total number of vehicles counted. For consistency with other sources of input data, all taken from 2006, where available, the average of the 2005 and 2007 fractions was computed for each vehicle type. Table 5 shows the average vehicle composition fractions computed from this second data source for each station.

Table 5 – Vehicle Composition 2005 to 2007

Station*	Year	Truck**	Bus	Car/MC/Light Truck
B2966/S025	2005	6.01%	0.40%	93.59%
	2006	6.18%	0.39%	93.43%
	2007	6.34%	0.39%	93.27%
B2965	2005	5.04%	0.46%	94.50%
	2006	4.27%	0.46%	95.27%
	2007	3.49%	0.46%	96.05%

Station*	Year	Truck**	Bus	Car/MC/Light Truck
B2847/S024	2005	5.03%	0.51%	94.47%
	2006	5.82%	0.52%	93.65%
	2007	6.62%	0.54%	92.84%
B2848/S023	2005	5.42%	0.50%	94.08%
	2006	5.90%	0.52%	93.59%
	2007	6.37%	0.54%	93.10%
B2849/S121	2005	5.73%	0.51%	93.76%
	2006	5.82%	0.50%	93.68%
	2007	5.92%	0.49%	93.59%
B2850	2005	6.07%	0.66%	93.27%
	2006	6.07%	0.66%	93.27%
	2007	N/A	N/A	N/A

* Refer to Figure 15 for station numbering.
** Trucks include Classes 5-13.
MC=motorcycle

In Table 6, the average vehicle composition over all relevant stations is given for each roadway segment. A number of assumptions were required in finalizing the composition values:

1. Vanpools shown in the raw occupancy data were treated as passenger cars with two or more vehicle occupants.

2. Several different types of trucks are found in the raw vehicle composition data. Light trucks are counted and treated in the model as passenger cars and all other truck types are classified under the truck category, i.e. assuming that they are heavy vehicles.

3. No trucks are allowed in the HOV lane.

4. Motorcycles are modeled as single passenger cars and are not permitted to use the HOV lane.

Table 6 – Average Vehicle Composition

Road Segment	Truck**	Bus	Car/MC/Light Truck
I-370 to Montrose Road	5.60%	0.48%	93.92%
Montrose Road to the Spur	6.07%	0.66%	93.27%

*** Trucks include Class 5-13.*

This second source of vehicle composition data, while studied, was not employed as input to the VISSIM model constructed for this study. The data source described in the previous section was obtained for 2006 directly and provided the additional required occupancy data. It is worth noting that a larger percentage of vehicles fall in the Car/Motorcycle/Light Truck category as obtained from the 2006 occupancy data than from this second data source. In both sources, it is assumed that vehicle composition does not change over time or as a function of volume or other traffic characteristics.

No vehicle occupancy and composition data were provided for 2030 traffic volume. Thus, the vehicle classification as input to the VISSIM alternatives models would be computed from the existing occupancy and composition data as shown above. This will be discussed in Section 5.1.

Chapter 3 Modeling the Existing Facility

A VISSIM simulation model was developed (using version 4.3) to replicate the existing facility along I-270 in the study area, including current traffic patterns, volumes, and driver behavior. While traffic conditions, including traffic volume, vehicle composition, and vehicle occupancy vary over the morning peak (6:00 a.m. to 9:00 a.m.), i.e. the study period, it was assumed that conditions were static over the period. Thus, a static modeling approach was employed.

In Section 3.1, details of the construction of the model with respect to the facility design are given. This is followed by a description of traffic modeling in Section 3.2. Additional modeling efforts required to perfect the existing conditions model are presented in Section 3.3. Rather than articulate generic techniques employed in creating a VISSIM model, details given in this chapter focus on major decisions taken in the modeling effort and nonstandard modeling techniques employed to better reflect real-world traffic movements.

3.1 Modeling the Physical Facility

In constructing the VISSIM model of the existing physical facility, two parallel links with connecting links were employed. One of the parallel links was used to model the CD lanes and the other was used to model the GP and HOV lanes. Separate links were used to connect the CD lanes with the GP lanes and neighboring roads with the CD lanes. The HOV lane was modeled using the same link as was used to model the GP lane to provide continuous access between the lanes as needed (i.e. between I-370 and Tuckerman Lane, which is one mile north of the Spur). All links employed in the model are of the “freeway” type as categorized in the simulation platform. This implies that model parameters associated with the “freeway” category were set identically for the entire facility.

3.2 Modeling Traffic

A number of considerations must be taken in modeling traffic within the simulation model. First, vehicles must be loaded into the network model. Vehicles are classified within one of eight categories that represent both vehicle type (e.g. truck, bus, passenger car) and whether or not the vehicle is eligible to and will use the HOV lane. The volume within each vehicle classification must be consistent with the traffic composition and occupancy data obtained from actual traffic conditions as determined from the input data described in Chapter 2. Second, the number of vehicles within each category of classification must be set for each origin and destination. Finally, smooth transitioning of traffic between the CD, GP and HOV lanes must be facilitated. Details of each of these components of modeling the traffic are described in the following subsections.

Each run of the VISSIM model entailed 5,400 seconds of simulation time, the first 1,800 seconds of which was considered as the warm-up period. Average results when provided in this report, unless otherwise specified, are hourly averages based on the 3,600 seconds of simulation run time.

3.2.1 Vehicle Loading by Classification

Vehicles are classified so that vehicles falling within the same class have similar characteristics. For example, vehicles in the same class are assumed to have similar physical features (e.g. same length and weight class), acceleration/deceleration rates (i.e. distributions), occupancies and desired speeds. Additionally, eligibility and desire to use HOV lanes is considered. Only those vehicles falling in classes 2, 4, 6, and 7 use the HOV lane.

Eight classes of vehicles were created for use in the existing conditions model:

(1) trucks that use only CD and GP lanes
(2) trucks that use only CD, GP and HOV lanes (i.e. HOV lane violators)
(3) buses that use only CD and GP lanes
(4) buses that use CD, GP and HOV lanes
(5) single occupancy vehicles (SOVs), i.e. passenger cars or vans with only one

passenger onboard, that use only CD and GP lanes

(6) single occupancy vehicles (SOVs), i.e. passenger cars or vans with only one passenger onboard, that use CD, GP and HOV lanes (i.e. HOV lane violators)

(7) HOVs (passenger cars or vans with more than one person on board) that use CD, GP and HOV lane

(8) HOVs (passenger cars or vans with more than one person on board) that use only CD and GP lanes

Classes 2 and 6 model trucks and passenger cars that violate the occupancy and vehicle classification restrictions of the HOV lane facility. These violators are assumed to behave similarly to comparable vehicles permitted to legally use the HOV lane in all other respects. Note that there is a short segment of solid striping in the southern most portion of the study roadway segment. A vehicle that crosses the solid striped line would commit an alternative form of violation. Only violations associated with vehicle occupancy and classification restrictions are considered in this study.

The composition in terms of these eight classes used in creating the existing conditions model is given in Table 6. The values shown in Table 7 were obtained from the composition and occupancy data described in Chapter 2.

Table 7 – Vehicle Class Composition - Existing 2006

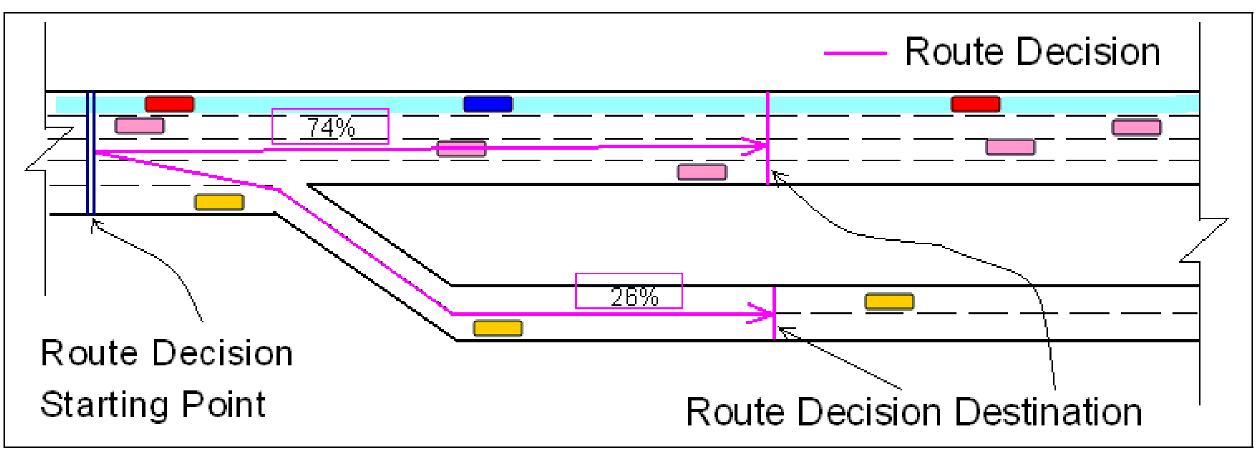
Class	Type	Occupancy	Using HOV?	Composition (%)	
				I-370 to Montrose Road	Montrose Road to Spur
Class 1	truck	1	no	3.4	3.8
Class 2	truck	1	yes	0.2	0.1
Class 3	bus	2+	no	0.1	0.2
Class 4	bus	2+	yes	0.1	0.2
Class 5	passenger car	1	yes	3.0	3.0
Class 6	passenger car	1	no	79.6	74.6
Class 7	passenger car	2+	yes	9.2	12.5
Class 8	passenger car	2+	no	4.4	5.6

For trucks and buses, i.e. Classes 1 through 4 employed within the alternatives VISSIM models, the desired speed was set to 43.5 mph, ranging from 42.3 to 48.5 mph. For Classes 5 through 8 employed within the alternatives VISSIM models, the desired speed was set to 50 mph, ranging from 47 to 68 mph. The default linear distribution of speeds was employed and default acceleration and deceleration rates by vehicle type were used.

3.2.2 Origin-Destination Modeling

VISSIM permits two methods for controlling vehicle destinations as mentioned previously. In creating the existing conditions model of the study roadway segment, the VISSIM methodology that employs turning percentages at all major decision points to achieve required volume exiting at each destination was used. This methodology permits the modeler to set specific decision points from which two or more choices for travel destinations are available. A destination in this context may be an exit from the facility or it may be the decision to travel between CD and GP lanes via slip ramps. The use of turning percentages in this context is illustrated in Figure 16 for a single vehicle classification. For this vehicle class, 74% of the vehicles reaching the bar (i.e. the route decision starting point) at the left end of the figure will continue in the mainstream, while 26% will follow the slip ramp as depicted to access the CD lanes. The bars at downstream of the roadway indicate a destination for the decision. Turning percentages may vary across vehicles classes.

Figure 16 – Vehicle Route Decision at Slip Ramp



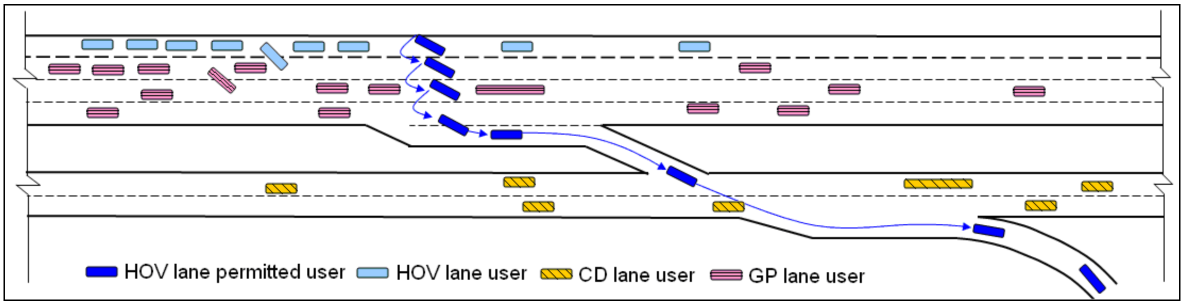
3.2.3 Smooth Transitioning between Lanes and Links

Additional modeling effort is required to: prevent vehicles from taking very late decisions that, for example, might call for the vehicle to abruptly cross multiple lanes to exit the facility; prevent vehicles from stopping in a lane while waiting for an appropriate gap to change lanes; prohibit certain vehicle classes from using the HOV lane, while simultaneously allowing other vehicle classes to have continuous access to that lane for the majority of the study roadway segment length; and facilitate smooth transitions between connected links in the model. The first two issues associated with smooth lane changing movements are addressed in Subsection 3.2.3.1. The modeling techniques used to simulate continuous access to the HOV lane for a subset of vehicle classes is described in Subsection 3.2.3.2. This is followed by a description of the methodology for ensuring smooth transitions at network model connections in Subsection 3.2.3.3.

3.2.3.1 Lane Changing Movements

Upon first running the created VISSIM model for the study roadway segment, vehicles in the model would often abruptly cross multiple lanes to exit or enter various portions of the segment. This abrupt action involved stopping of vehicles in the middle of a stretch to switch lanes. The stopping behavior occurred as the vehicle waited for a suitable gap for maneuvering to a neighboring lane. That is, when such a gap did not arise upon the vehicle's decision to switch lanes, the vehicle stopped to wait for such a gap. Figure 17 provides an example of such behavior. In the figure, an HOV vehicle (Class 7) attempted to exit the facility from the HOV lane, requiring the vehicle to cross three GP lanes, enter and exit the slip ramp, cross two CD lanes and enter the off-ramp. The vehicle did not take a decision to exit until it reached a location that was very close to the deceleration lane; thus, it was not possible to cross the GP lanes smoothly without passing the slip ramp and an abrupt crossing action was depicted. Such behavior required that the vehicle stop to wait for an appropriate gap to change lanes. Other vehicles were interrupted and lanes of the freeway became blocked as a consequence of this behavior.

Figure 17 – Vehicle Abruptly Crossing GP lanes to Exit Mainstream Lanes



While some drivers may behave as depicted in Figure 17, most do not. Most vehicles prepare for such decisions through lane changing behavior that facilitates a smoother transition. This desirable and more realistic behavior is depicted in Figure 18. Figure 18 illustrates vehicles maneuvering from an on-ramp to the HOV lane via CD and GP lanes, using a slip ramp, as well as from the HOV lane to an off-ramp via GP and CD lanes, using a slip ramp. The behaviors depicted in this figure show smooth transitions between the HOV lane and the on- and off-ramps. Lanes 1 through 3 are GP lanes, Lane 4 is classified as an HOV lane, and Lanes 5 and 6 are CD lanes.

Figure 18a – Smooth Transitioning to Enter the Freeway North of Shady Grove Rd

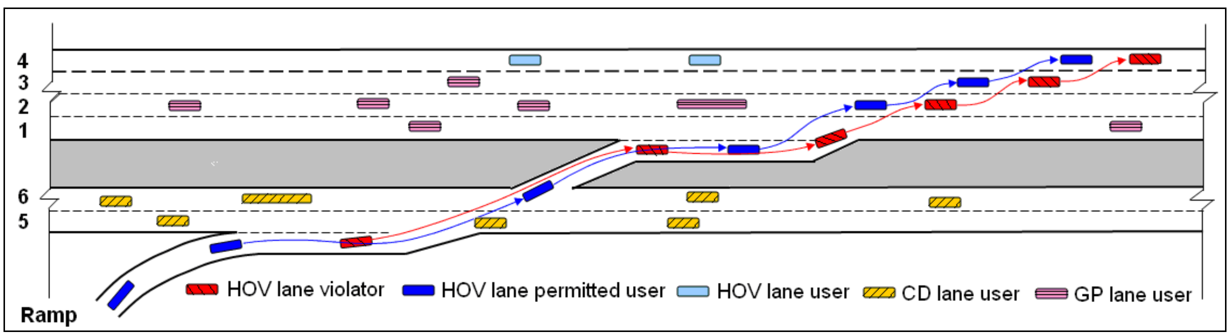
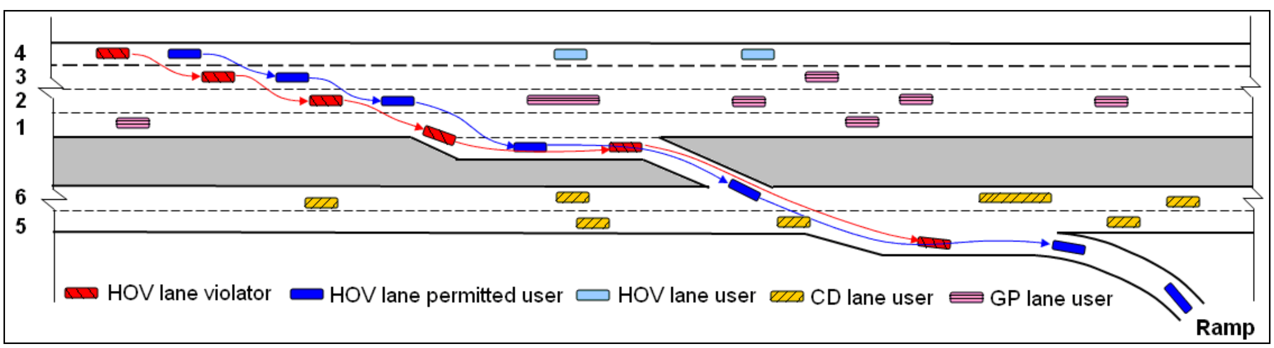


Figure 18b –Smooth Transitioning to Exit the Freeway North of MD 189



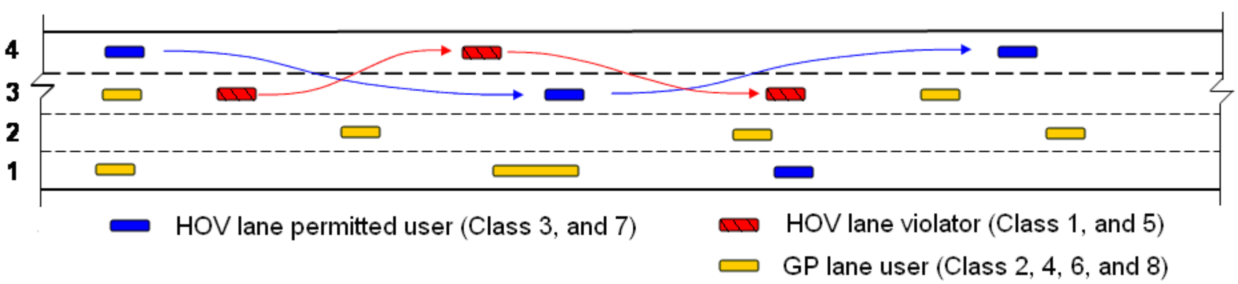
To achieve the smooth transitioning of vehicles as depicted in Figure 18, two major actions were taken:

1. The length of the Decision Route (defined in Figure 16) is extended or contracted to ensure that vehicles exiting or entering the facility or a portion thereof will have enough time to smoothly change lanes if required.
2. The Look Back Distance associated with any connector (discussed in Section 3.2.3.3) that is used with an exit or entrance is extended such that the vehicles are able to recognize the exit or entrance prior to arriving at the connector.

3.2.3.2 HOV Lane Access Control

The HOV lane was modeled as a separate lane, as opposed to separate facility (i.e. link). This modeling approach allows HOV users to move freely between the GP and HOV lanes in portions where continuous access is permitted, as is the case between I-370 and Montrose Road under existing conditions. Figure 19 depicts this free movement of HOV users between lanes. Continuous access allows eligible HOV lane users to choose between the HOV and GP lanes as traffic conditions change. While some violators, as depicted, may illegally use the HOV lanes, most single occupant or otherwise non-HOV users will not use the HOV lane. Some action was required to prevent these non-HOV users from moving into the HOV lane within the model.

Figure 19 – Modeling of Continuous Access



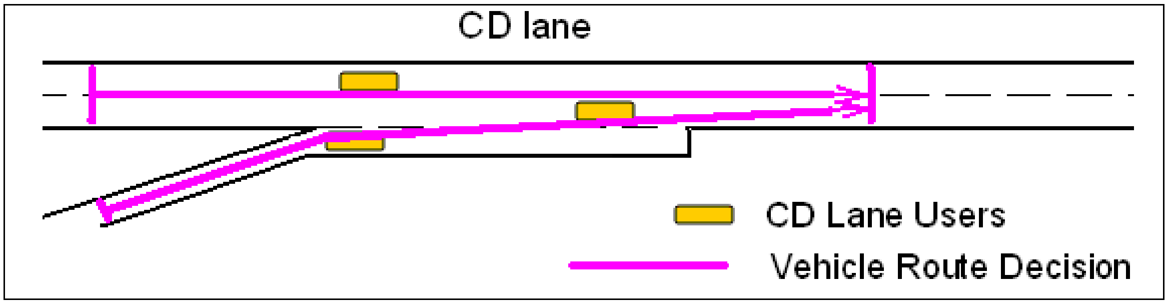
To prevent non-HOV users from using the HOV lane, the lane closure property of the HOV lane is set to “closed” for the non-HOV users, but to “open” for the HOV users and violators. GP lanes are open to all the vehicle classes.

3.2.3.3 Transitioning between Model Links via Acceleration and Deceleration Lane Connectors

A typical approach to modeling acceleration/deceleration as required at connections between the CD lanes and the local roads or GP lanes (via on-, off- or slip ramps) is to use connectors (so-called tapering). Thus, for example, to model acceleration between an on-ramp and the CD lanes, a connector would connect the on-ramp link with the link representing the CD-lanes. To make this connection, the link representing the CD-lanes would need to be broken at the site of the on-ramp. Thus, a second connector would be required to connect the two portions of the CD lane link. This modeling approach, however, results in a conflict between vehicles approaching from the upstream CD lanes and the on-ramp. Since the two connectors are operated separately, the behavior of the vehicles at this connection location will be haphazard and may even result in two vehicles being present at the same location at the same point in time.

Rather than this more typical modeling methodology for connecting the on-ramp with the CD lanes (or making other similar connections required within the model), only one connector is used. Without additional modeling work, the use of one connector may result in the sudden loss of vehicles from the model, because the vehicles are not told to switch lanes. Thus, route decision points were added to the model at the intersection of the on-ramp and the CD lanes (or other similar connections) as depicted in Figure 20.

Figure 20 – Connecting Acceleration Lane to Freeway at MD 189 On-Ramp

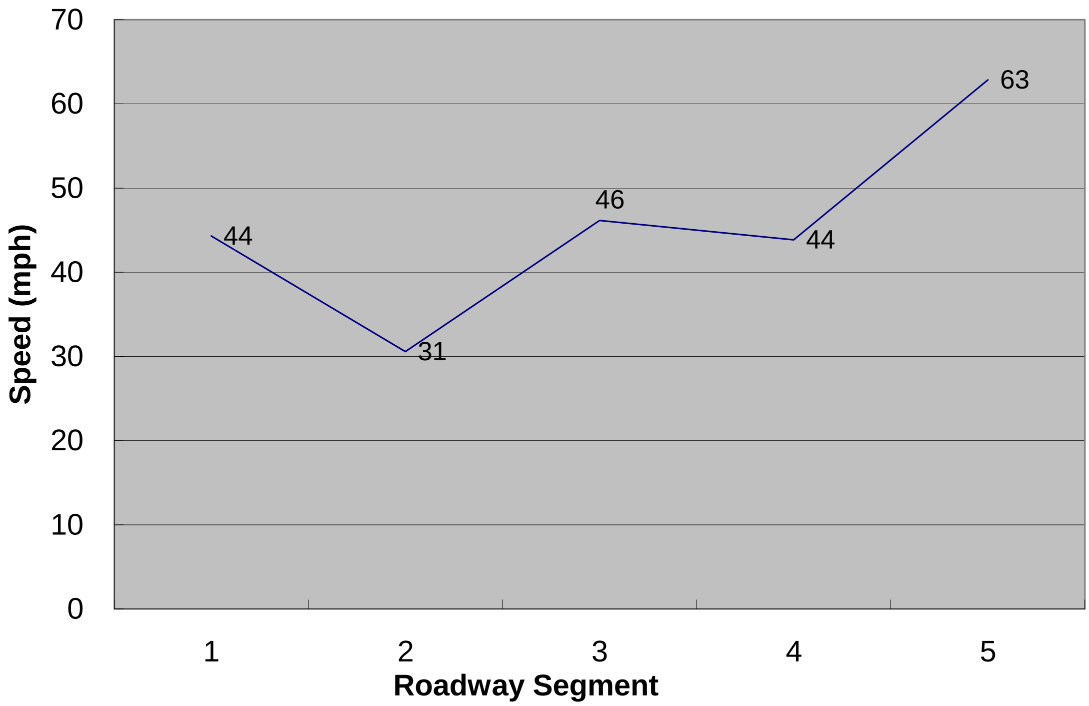


3.3 Additional Modeling Efforts to Perfect the Existing Conditions Model

Once traffic was loaded into the network model (employing modeling techniques described in prior subsections), runs were made to assess whether or not traffic was replicated in a way that mimics reality. A major consideration in assessing how well the model did after its creation was its ability to replicate conditions at bottlenecks. To evaluate the model with respect to bottlenecks, two steps were followed. First, bottlenecks in the model were identified using speed data from the input data of actual traffic conditions. Speed differentiation across segments was studied. Second, for each discovered bottleneck, the cause of that bottleneck was surmised based on the roadway geometry. The simulation model was suitably modified to more accurately reflect the bottleneck conditions once identified. Modifications primarily involved changes in Route Decision length. Model parameters, such as the Look-Back Distance parameter mentioned previously, were also adjusted accordingly.

Average speed data were taken from a vehicle travel time survey conducted by the Maryland SHA during several morning peak periods (from 6:00 a.m. to 9:00 a.m.) in April of 2004. 13 samples were provided after removing those data associated with abnormal conditions (e.g. in the event of a traffic incident). The travel time data were given by roadway segment, where speed for each segment (segmentation depicted in Figure 10) was as shown in Figure 21. Segment speeds were calculated from the segment travel times and lengths. The average speed for each segment was taken over all 13 sample speeds (given in Figure 21).

Figure 21 – Average Actual 2004 Segment Speed



Three categories of congestion (as suggested by Gomes et al., 2004b) are defined based on the average speeds: Severely Congested (average speed less than 40 mph), Moderately Congested (average speed between 40 and 50 mph) and Uncongested (average speed no lower than 50 mph). As depicted in Figure 21, Segments 1, 3 and 4 are considered to be moderately congested, Segment 2 is severely congested and Segment 5 is uncongested.

Based on the congestion designations, Segments 1 through 4 were considered further. It was assumed that no bottleneck exists within Segment 5. The next step in identifying bottlenecks was to consider the roadway geometry and average hourly traffic volume per lane. To do so, the details as shown in Figure 11 were studied. That is, if the geometry allows for a decision, such as to exit the facility, and simultaneously the traffic volume is found to be high in a nearby location, a bottleneck is suspected. Five such bottlenecks were identified along the study roadway segment, as depicted in Figure 22. Note that the average hourly traffic volume per lane was computed from the traffic volumes given in Figure 11 of Section 2.2 divided by the associated number of lanes.

Figure 22 – Bottleneck and Hourly Traffic Volume per Lane

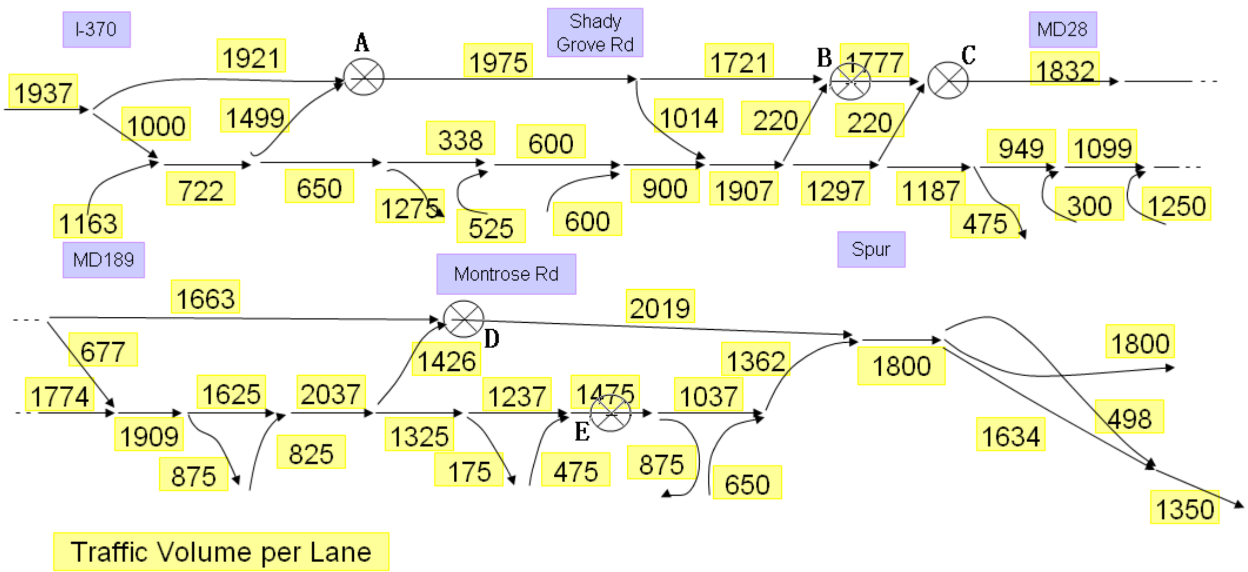


Table 8 – Identification of Bottlenecks

Bottleneck	Location
A	Slip ramp to GP lane from CD lane before Shady Grove Road
B	Slip ramp to GP lane from CD lane between Shady Grove Road and MD 28
C	Slip ramp to GP lane from CD lane between Shady Grove Road and MD 28
D	Slip ramp to GP lane from CD lane north of Montrose Road
E	On CD lane between on- and off-ramps at Montrose Road

Bottlenecks cannot necessarily be identified by comparing nominal traffic volumes. For example, the high average hourly traffic volume per lane (2,019 vehicles per lane per hour (vplph)) occurs between Montrose Road and the Spur. Moreover, the average speed for this segment is 63 mph. This segment (Segment 5), thus, is considered to be uncongested. On the contrary, average hour traffic volume per lane in Segment 2, which is considered to be Severely Congested, is not particularly high as compared with similar measurements in other portions of the study roadway segment.

The average speed for this segment is 31 mph. Thus, the average hourly traffic volume is lower in value on Segment 2 than Segment 5 as a consequence of the lower speed due to increased congestion. If only volume were considered, one might mistakenly rate Segment 2 as uncongested. To improve the accuracy in determining whether or not a segment of roadway is congested and whether or not a bottleneck exists, speed and roadway geometry must be considered in addition to average hourly traffic volume. Moreover, roadway geometry must be studied to identify the bottleneck cause.

Specific geometric considerations, including splits and merges, on- and off-ramps, and slip ramps, and maximum hourly volume per lane and speed are given in Table 9 for each segment.

Table 9 – Comparison of Segment Geometry, Maximum Volume and Speed

Road Segment	I-370 to Shady Grove Rd (1)	Shady Grove Rd to MD 28 (2)	MD 28 to MD 189 (3)	MD 189 to Montrose Rd (4)	Montrose Rd to Spur (5)
Splits and Merges	1	0	0	0	1
No. of Ramps	3	3	2	3	2
No. of Slip Ramps	1	3	1	1	0
Total No. of Ramps, Splits and Merges	5	6	3	4	3
Max. Volume per CD Lane	722	1907	1774	2037	1475
Max. Volume per Mainstream (GP and HOV) Lane	1975	1777	1832	1663	2019
Average Segment Speed (mph)	44	31	46	44	63

As shown in Table 9, the average segment speed is directly correlated with the number of slip ramps. Less strong correlation exists between average segment speed and maximum hourly volume per mainstream lane. Thus, it was concluded that the congestion at bottlenecks is due to a combined effect of geometry, especially number of slip ramps, and traffic volume per hour per lane in the GP and HOV lanes.

Chapter 4 Calibration

A myriad of software products exist for simulating vehicular traffic. These models can replicate many of the characteristics of vehicular behavior. Results from such simulation runs are often used to make decisions pertaining to operational and design changes. In this study, conversion of a HOV lane to a HOT lane facility is considered. Before one makes decisions based on the outcomes from the simulation runs, one must be sure that the simulation adequately replicates traffic conditions and vehicular behavior. In addition to the modeling issues described in Chapter 3, parameters of the VISSIM simulation software can be set so that traffic measures from the simulation best match actual measurements taken from the field. Initial runs were conducted using default parameter settings as described in Section 4.1. Results from these runs show that mean travel times estimated by the simulation model using default parameters were statistically significantly different from observed mean travel times. Thus, calibration of the parameters is essential. In this study, the parameters are calibrated based on average segment travel time.

In Section 4.2, relevant model parameters, along with their ranges and default values are presented. In Section 4.3, results of sensitivity analysis in which the parameters were set to their extreme values and simulation runs were conducted are presented. Such experiments provide additional insight into the impact of each parameter on travel time. Results of these preliminary runs, as well as input from PTV America, Inc. (PTV) modelers and the literature were used to identify five parameters as having the greatest impact on model calibration.

Even if only a few potential values were chosen for each of these five parameters, the number of runs that would be required to consider all parameter setting combinations would be very large. Thus, effort was taken to design the experiments and conduct a more limited set of runs. In Section 4.4, findings from a limited set of runs chosen based on factorial design are given. Results of these runs provide information about interactions between parameters, as well as the potential impact of specific parameter values. For example, in which direction (i.e. below or above a chosen value) the parameter should be set to obtain a particular behavior (e.g. lower or higher segment travel time) can be observed from the run results. The information concerning parameter interactions aided in choosing a subset of parameter combinations for the final set of runs. Intuition gleaned from results of the runs based on the factorial design is employed in final calibration runs. Final results of the calibration are also provided in Section 4.5.

4.1 Quality of Simulation Results given Default Parameter Settings

The VISSIM model of the study area was constructed with existing highway geometry and traffic demand as described in Chapter 2. Initial simulation experiments were conducted using default driving behavior parameter settings to ascertain how well the model does in replicating traffic given that default parameter settings are used as is often done in practice. Mean segment travel times obtained from the simulation results were compared with mean segment actual travel times obtained from the field. A comparison of mean travel times for small sample size (i.e. a t-test) was completed. It was assumed that travel times are normally distributed. The small sample size test was employed, because significantly fewer than 30 travel time samples were obtained through field observations for each roadway segment. Results of the analysis are given in Tables 10 and 11 and Figures 23 and 24.

Table 10 – Travel Times for Existing Conditions given Default Parameter Settings

Segment	GP				HOV			
	Simulated		Survey		Simulated		Survey	
	Ave	SD*	Ave	SD*	Ave	SD*	Ave	SD*
1	63	22	214	67	62	25	90	48
2	125	86	312	59	124	24	259	45
3	60	40	145	81	60	39	89	32
4	83	60	193	39	82	75	129	39
5	82	70	163	62	83	53	91	12
Total	412	--	668	--	410	--	668	--

** SD = Standard Deviation*

Figure 23 – Comparison of Survey GP Lane Travel Times with Simulated Travel Times given Default Parameter Settings and Existing Conditions

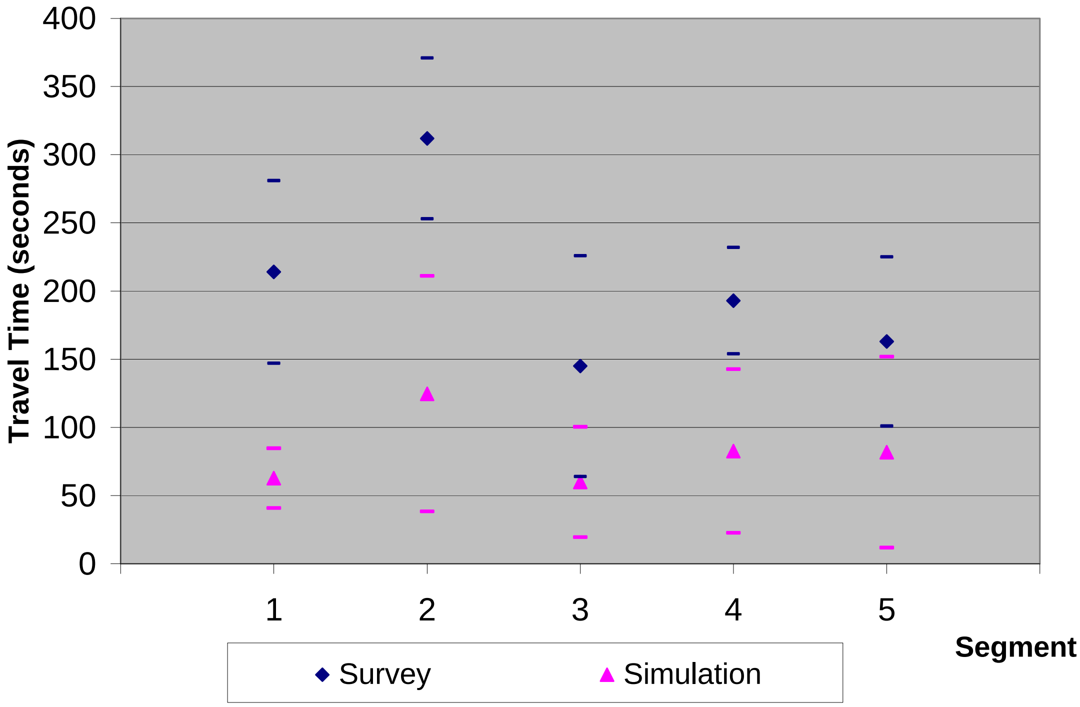


Figure 24 – Comparison of Survey HOV Lane Travel Times with Simulated Travel Times given Default Parameter Settings and Existing Conditions

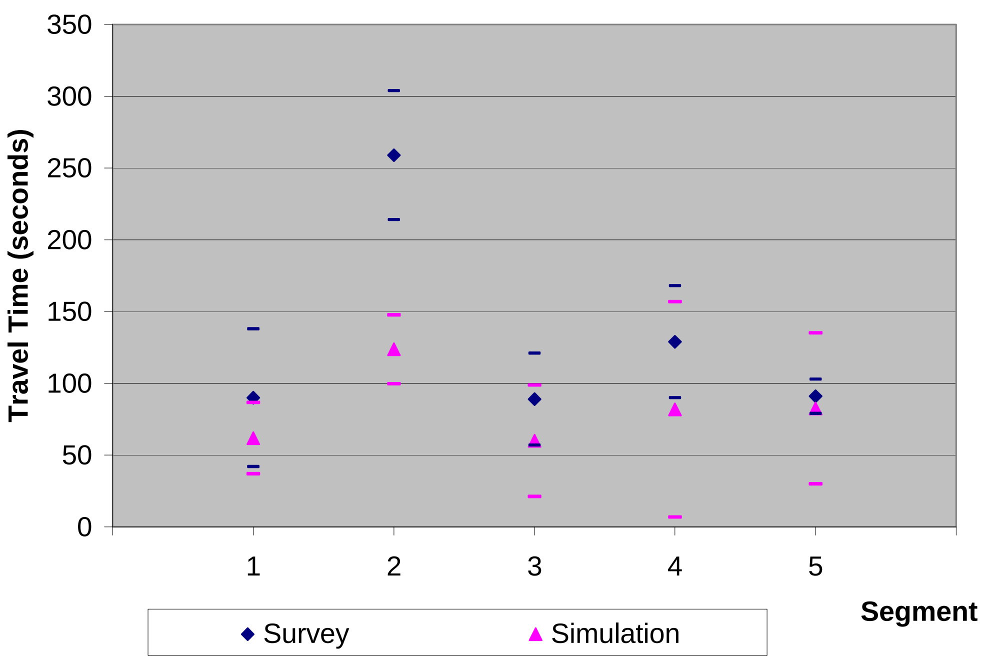


Table 11 – Statistical Analysis of Existing Condition Simulation Results given Default Parameter Settings

Segment	GP Lane				HOV Lane			
	t value	v value	T (0.025,v)	If -T<t<T, accepted	t value	v value	T (0.025,v)	If -T<t<T, accepted
1	7.818	11.005	2.145	Rejected	1.946	10.033	2.145	Accepted
2	10.970	11.105	2.110	Rejected	9.961	10.037	2.145	Rejected
3	3.634	11.013	2.145	Rejected	2.997	10.048	2.120	Rejected
4	9.702	11.451	2.052	Rejected	3.962	10.512	2.093	Rejected
5	4.520	11.153	2.101	Rejected	2.200	12.635	2.009	Rejected

V value is the degree of freedom of the sample.

The results indicate that the mean segment travel times obtained from the simulation using default parameter settings are statistically different from mean segment travel times obtained through field observations with one exception. Thus, it is concluded that calibration of the parameters is required.

4.2 Details of Relevant Model Parameters

Two categories of parameters exist in the VISSIM software platform:

(1) parameters that control physical attributes of the vehicles (e.g. acceleration and deceleration properties of a given vehicle class) and

(2) parameters that affect behavior associated with vehicle movement (e.g. car following behavior).

Five parameters are chosen for consideration in this study based on results of preliminary runs described in Section 4.2, as well as information gleaned from the literature and through conversations with PTV America, Inc. modelers. These five parameters are described next. Note that all five parameters fall under the second classification.

4.2.1 Parameters Impacting Physical Attributes of Vehicles

Eight vehicle classes were created in developing the VISSIM model for this study as defined in Chapter 3, Section 3.2.1. All vehicles falling into a given class will have identical attributes in terms of minimum and maximum acceleration, minimum and maximum deceleration, weight, power and length. Parameters of the physical attribute type pertain to each class. The default parameters for each class are employed in all model runs.

4.2.2 Parameters Affecting Behavior Associated with Movement

The VISSIM software package, like many others, implements accepted car-following and lane-changing models to capture the details of interactions between vehicles. Because the parameters associated with traffic movement are set by link-type, and all links in the developed model are of the freeway type, any change to a single parameter affects all links of the model. There are tens of parameters within this classification. A complete list of parameters falling within this category can be found in (PTV Guideline 2007). The five selected parameters that will be considered in the simulation runs associated with this calibration effort are described next. These parameters fall under two subclassifications: lane changing and car following behavior.

Parameters Associated with Lane Changing Behavior

Parameters associated with lane changing behavior dictate how far in advance a driver will adapt his/her behavior (specifically, change lanes) in anticipation of a change in roadway geometry (e.g. upcoming desired exit) or in reaction to information about improved travel conditions in a nearby lane. These parameters indicate how aggressive each driver will act in terms of lane changing decisions, i.e. the length of the acceptable gap between vehicles in the neighboring lane. Four parameters associated with lane changing behavior were calibrated in this study: Waiting Time Before Diffusion (WTBD), Safety Distance Reduction Factor (SDRF), Look-Back Distance (LBD) (also referred to as Lane Change Distance (LCD)), and Emergency Stop Distance (ESD).

Table 12 – Parameters Associated with Lane Changing Behavior

Parameter	Definition	Default value	Range
WTBD	maximum amount of time a vehicle can wait at the emergency stop position in anticipation of a gap sufficiently wide enough to change lanes in order to stay on its route	60 seconds	$(0, \infty)$
SDRF	effects safety distance during lane changing, calculated as follows: original safety distance × reduction factor	0.6	(0.1, 0.9)
LBD	a property of link connector, defines the distance at which vehicles will begin to attempt to change lanes	200 meters	400-1,000
ESD	Minimum distance permitted between vehicles	5 meters	$(0, \infty)$

Note that parameters of Look-Back Distance and Emergency Stop Distance are the only driver behavior parameters that can be specified for each link regardless of link type. Also note that the Look-Back Distance parameter's name has been changed to the Lane Change parameter in the most recent version of the VISSIM simulation platform.

Parameters Associated with Vehicle Following Behavior

Car-following models define the interaction between leading and lagging vehicles. There are a variety of existing car-following models, some of which focus on the acceleration function of the lagging vehicle and consider such measures as gap distance, vehicle speed, and speed difference between two cars. Other models focus on safety distance, where it is assumed that the following vehicle will maintain an appropriate safety distance. Remaining models are classified as psycho-physical models. Such models apply a minimum speed difference threshold for following and leading vehicles. The model adopted in VISSIM is a psycho-physical car following model of longitudinal vehicle movement developed by Wiedemann (Olstam and Tapani, 2004). A rule-based algorithm is employed for lateral movements. Briefly, drivers of vehicles are classified into types: free driving, approaching, following, and braking. Two model options are available: “Wiedemann 74” and “Wiedemann 99.” The former model is most appropriate for modeling urban traffic; whereas, the latter model was developed for interurban and freeway traffic modeling. Thus, the Wiedemann 99 approach was employed in this study. Numerous car-following parameters are designated in the VISSIM software, five of which were considered for calibration in this study: Standstill Distance (CC0), Headway Time (CC1), "Following" Variation (CC2), and Upper and Lower "Following" Thresholds (CC4 and CC5)

Table 13 – Selected Parameters Associated with Vehicle Following Behavior

Parameter	Definition	Default value	Range
CC0	Standstill Distance: the desired distance between stopped cars.	1.5 meters	--
CC1	Headway Time: higher value, more cautious driver.	0.9	0.2~1.5
CC2	"Following" Variation: desired safety following distance.	4 meters	5~20
CC4	Lower "Following" Threshold.	-0.35 mph	-0.1~2.0
CC5	Upper "Following" Threshold. CC5=-CC4	0.35 mph	0.1~2.0

CC1 (Headway Time) is computed based on the minimum distance that a driver will maintain between his/her vehicle and the vehicle ahead. The higher the value, the more cautious the driver. According to PTV (AG 2007), CC1 is considered to be the parameter with the greatest influence on roadway capacity. Headway Time is given in units of seconds. One can compute the safety distance, dx_safe = CC0 + CC1 × v, where v is the vehicle’s velocity given in meters per second. Given that (dx_safe + vehicle length) × capacity = free flow speed (AG 2007), one can determine dx_safe and, thereby Headway Time, if CC0 is preset.

CC4 and CC5 (Following Thresholds) control the speed differences between leader and follower during the ‘Following’ state. Smaller absolute values result in a more sensitive reaction of drivers to accelerations or decelerations of the preceding vehicle. It is recommended in PTV Manual 2007 that these two parameters have opposite signs and equal absolute values.

4.3 Sensitivity Analysis of Key Model Parameters

Recall from Section 4.2 that there are a very large number of parameters that can be calibrated in the VISSIM simulation platform. It would not be feasible to calibrate all of them. As discussed in the literature (Sensitivity of Simulated Capacity to Modification of VISSIM Driver Behavior Parameters), CC0-9, LBD, LAD, WTBD, and SDRF parameters are considered to be the key parameters in controlling traffic characteristics in the simulation model. Only a subset of these 14 parameters will be calibrated. To choose this subset, preliminary tests were undertaken. In each simulation conducted within these preliminary tests, all parameters, but one, were set to their default values. A chosen parameter was set to one of its extreme values. The extreme values employed in the preliminary tests are given in Table 14. We have assumed that driver behavior is not correlated with vehicle class, but instead with the position of the driver/vehicle in the freeway.

Table 14 – Extreme Values of Parameters for Preliminary Test

No.	Parameter	Level	Value	Units
CC0	Stopped Condition Distance	Low High	2.0 10.0	feet
CC1	Headway Time	Low High	0.20 1.50	seconds
CC2	"Following" Variation	Low High	5.00 20.00	feet
CC3	Threshold for entering "following"	Low High	-4 -15	feet
CC4&5	Upper and lower "Following" thresholds	Low High	0.1 2.0	mph
CC6	Speed Dependency of oscillation	Low High	2.00 20.00	--
CC7	Oscillation acceleration	Low High	0.50 1.50	ft/s^2
CC8	Stopped Condition Acceleration	Low High	6.4 10.0	ft/s^2

No.	Parameter	Level	Value	Units
CC9	Acceleration at 50 mph	Low High	2.10 7.50	ft/s^2
LBD	Look-back distance	Low High	50 1000	feet
LAD	Look Ahead Distance (min and max)	Low High	0 250	feet
WTBD	Waiting Time Before Diffusion	Low High	1 9999	second
SDRF	Safety Distance Reduction Factor	Low High	0.4 0.6	--

CC6 through CC9 of the car following parameters are associated with characteristics of acceleration. CC6 sets the level of correlation between speed oscillation and distance from the preceding vehicle. The higher the value of CC6, the stronger the relationship between oscillation speed and distance. CC7 is the acceleration rate of the oscillation process in feet per second squared (ft/s^2). CC8 is the desired acceleration rate (in ft/s^2) when starting from standstill (limited by the maximum acceleration rate defined by the acceleration distribution discussed previously). Finally, CC9 is the desired acceleration rate (in ft/s^2) when traveling at 50 mph (also limited by the maximum acceleration rate defined by the acceleration distribution.

To assess the impact of each parameter, its sensitivity, i.e. its impact on travel time or delay in seconds per vehicle given a one unit change in parameter value, was computed (graphed in Figures 25 and 26). The higher the parameter's sensitivity, the greater the impact of the parameter on simulated traffic performance.

Figure 25 – The Sensitivity of Parameters with respect to Travel Time

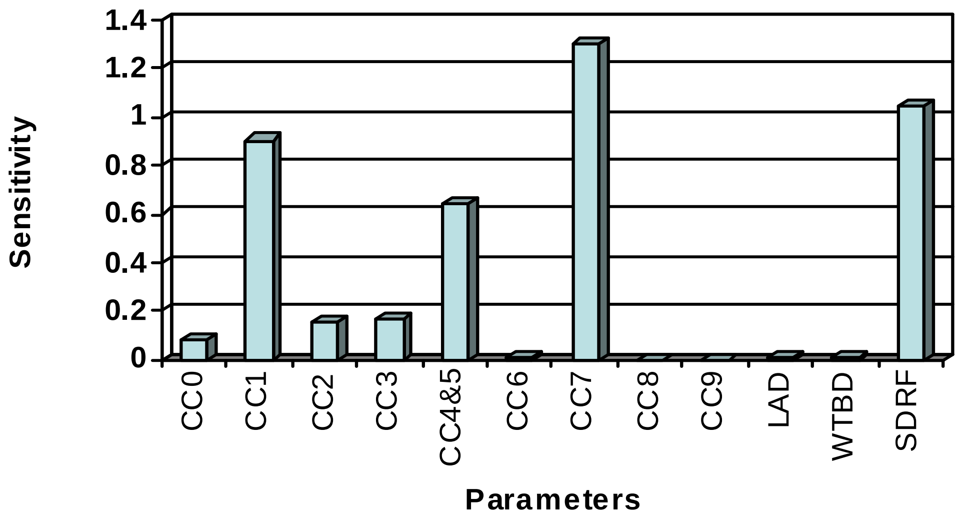
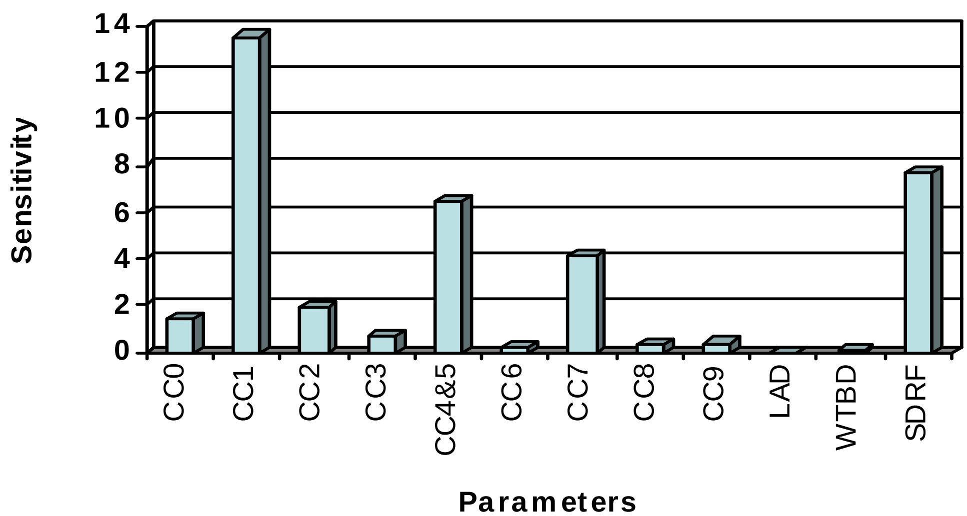


Figure 26 – The Sensitivity of Parameters with respect to Delay



The results of the preliminary experiments indicate that four parameters (CC1, CC4&5, CC7 and SDRF) have significantly more influence on travel time and delay than do others.

In addition to the sensitivity analysis conducted through these preliminary experiments, an in-depth literature review was conducted and conversations were held with experts at PTV. The literature review and conversations with experts provided additional insight into the choice of key parameters and their values.

Specific settings for several of the CC parameters were suggested in (Lownes and Machemehl, 2006) in the context of using of VISSIM to model freeway operations. These settings are given in Table 15.

Table 15 – Suggested to CC-Parameters by Literature

Link Type	CC0	CC1	CC4
Freeway	1.7	0.9	-1
Soft Curve	1.7	1.1	-1
Hard Curve	1.7	1.4	-1
Freeway Merge	1.7	0.9	-1
Soft Curve Merge	1.7	1.1	-1
Hard Curve Merge	1.7	1.4	-1

Source from Lownes and Machemehl, 2006 and Park and Qi, 2005

The settings of parameter CC1 given in the table were employed as initial parameter values in the calibration work described in Chapter 4.1. While CC7 was found to be a sensitive parameter in the sensitivity analysis, experts from PTV advised that it be left at its default setting. Thus, the CC7 parameter was not further considered.

As advised by experts from PTV, the WTBD parameter was changed to 9999 seconds. The WTBD parameter sets the length of time that a vehicle remains in the network if it is stopped, regardless of the reason (e.g. waiting to change lanes) for it being stopped. By setting this parameter to a high value, all vehicles remain in the network.

One additional parameter was chosen for inclusion in the calibration that could not be tested through the sensitivity analysis: LBD. (Park and Qi, 2005) suggests that the LBD parameter be set between 500 and 1,000 meters for freeways with relatively high traffic volumes. Note that its default value is 200 meters.

4.4 Factorial Design

Parameters CC1, CC2, CC4&5 SDRF and LBD can be set to nearly any number on half (i.e. the positive or negative side of) the real number line. Thus, a discrete set of potential values to be considered in the calibration must be selected or the experiment would require an infinite number of parameter settings. Even if only five discrete settings for each parameter were selected, if all 3,125 combinations of these parameters were to be applied in simulation runs, 15,625 runs (using 5 seed values for each run set) would be required. It was estimated that using one computer, 390 days of continuous simulation runs would be required to complete these runs, assuming perfect computer performance. Thus, an experimental design that could reduce the number of parameter combinations to be tested was desired.

A 2^k factorial design was employed to provide an initial estimate of how each factor (i.e. parameter) affects the results (i.e. estimated mean segment travel time) and whether or not there are interactions among the factors. This design requires that only two parameter settings be chosen for each parameter. Thus, for k=5, 25 runs will be required. The parameter settings associated with each run is referred to as a design point. The factorial design created with the 5 chosen parameters (assuming CC4&5 are treated together) is given in Table 16. In the design, for each parameter, the default parameter setting and a single suggested setting (as per the literature or PTV guideline) were employed. For the LBD, the 200 meters default value was employed as one possible parameter value. The second setting was based on values given in Table 17. That is, more than one setting was used over the entire study roadway segment. The LBD setting can vary from one link connector to another. Thus, which value was used for each link connector within this second setting (Set 2) depended on the link connector characteristics. Note that the default value was employed for all link connectors not associated with a route decision within runs associated with both the calibration and alternatives runs.

Table 16 – 2^k Factorial Design Points

Design Point	CC1	CC2	CC4&5	SDRF	LBD	Design Point	CC1	CC2	CC4&5	SDRF	LBD
0	0.9	4	0.35	0.6	200	16	0.9	4	0.35	0.6	Set 2
1	1.5	4	0.35	0.6	200	17	1.5	4	0.35	0.6	Set 2
2	0.9	15	0.35	0.6	200	18	0.9	15	0.35	0.6	Set 2
3	1.5	15	0.35	0.6	200	19	1.5	15	0.35	0.6	Set 2
4	0.9	4	2	0.6	200	20	0.9	4	2	0.6	Set 2
5	1.5	4	2	0.6	200	21	1.5	4	2	0.6	Set 2
6	0.9	15	2	0.6	200	22	0.9	15	2	0.6	Set 2
7	1.5	15	2	0.6	200	23	1.5	15	2	0.6	Set 2
8	0.9	4	0.35	0.4	200	24	0.9	4	0.35	0.4	Set 2
9	1.5	4	0.35	0.4	200	25	1.5	4	0.35	0.4	Set 2
10	0.9	15	0.35	0.4	200	26	0.9	15	0.35	0.4	Set 2
11	1.5	15	0.35	0.4	200	27	1.5	15	0.35	0.4	Set 2
12	0.9	4	2	0.4	200	28	0.9	4	2	0.4	Set 2
13	1.5	4	2	0.4	200	29	1.5	4	2	0.4	Set 2
14	0.9	15	2	0.4	200	30	0.9	15	2	0.4	Set 2
15	1.5	15	2	0.4	200	31	1.5	15	2	0.4	Set 2

Table 17 – Look Back Distance Values

Set 2	Mainstream	Off-ramp Slip Ramp	On-ramp	CD Lane	Spur
LBD	800	600	400	600	1000

4.5 Calibration Results

Results of the runs based on the 2^k factorial design from the previous section provided useful insight into the impact of changes to any single parameter, as well interactions between parameters. This insight was employed in designing approximately 130 additional experiments from which the final parameter values were obtained. These experiments were hand-designed using the insight gleaned from the factorial design, sensitivity analysis (Section 4.3), and expert advice (PTV experts and literature). The final chosen parameter values, i.e. those for which the resulting simulated mean segment travel times best matched the observed mean segment travel times once the calibration was complete, are given in Table 18.

Table 18 – Calibrated VISSIM Parameters

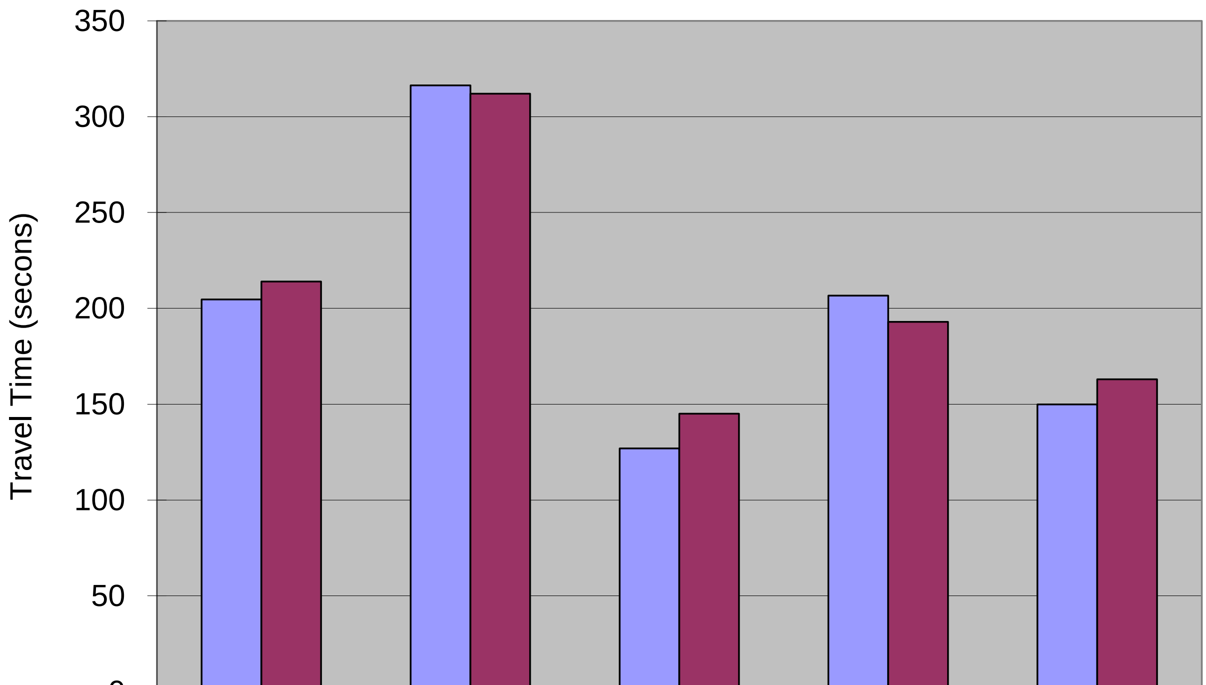
Parameter	Values
CC1	0.9 second (default)
CC2	12 feet
CC4&5	1.4 mph
SDRF	0.4
WTBD	9999 second
LBD	Mainstream: 800 m Off-ramp/Slip Ramp: 800 m On-ramp: 400 m CD Lane: 600 m Spur: 1000 m

Note that while the CC1 parameter remained at its default setting, other parameters were ultimately set to values that differed greatly from their default values. The fact that the optimal setting for CC1 is its default value implies moderate driving behavior and that the roadway is operating at its designed capacity. The chosen CC2 value is significantly larger than the default value. This infers that the safety distance is more variable than would have been modeled using the default value. The chosen value of SDRF as compared with the default value signifies the presence of aggressive lane changing behavior, where the safety distance employed by vehicles in the calibrated model is 33% of that suggested by the default value. The setting of WTBD to 9999 seconds guarantees that no vehicle will be removed from the model as discussed in Section 4.3. The values selected for LBD are consistent with the suggested range of values given in (Park and Qi, 2005) and are often set significantly higher than the default value. Table 19 and Figures 27 and 28 provide results in terms of estimated mean segment travel times using the final parameter settings resulting from the calibration effort (i.e. as given in Table 18).

Table 19 – Average Travel Time of Calibrated Existing Model from VISSIM Model

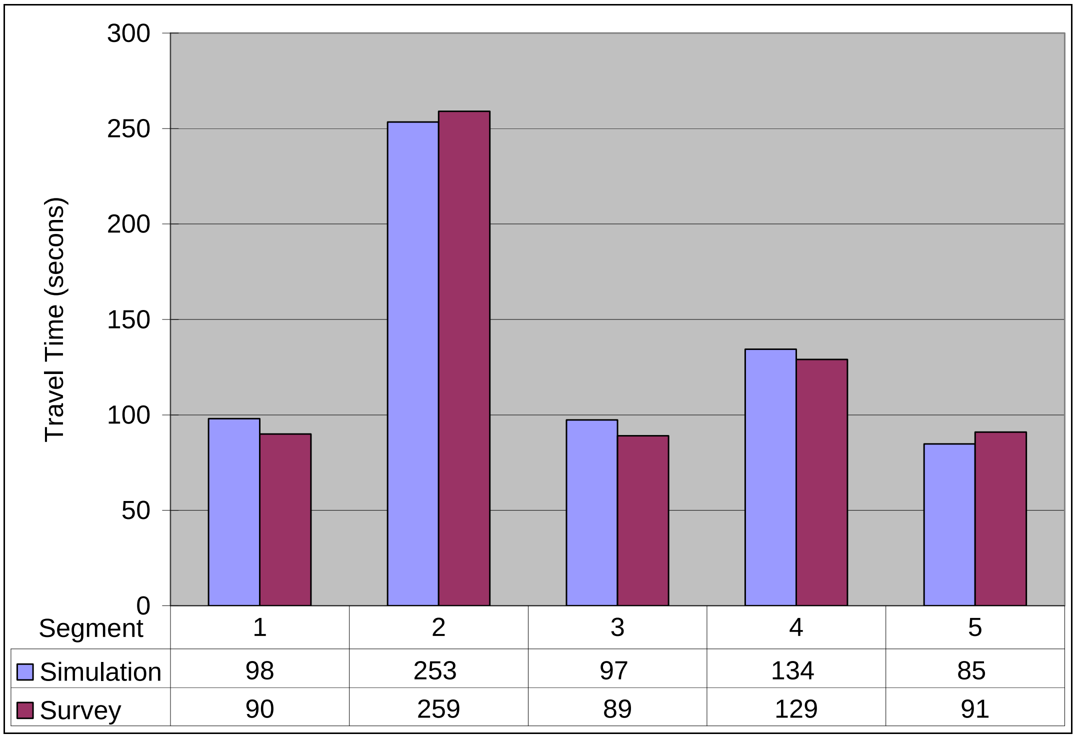
Segment	GP Lane		HOV Lane	
	Simulated	Survey	Simulated	Survey
1	205	214	98	90
2	316	312	253	259
3	127	145	97	89
4	207	193	134	129
5	150	163	85	91
Total	1004	1027	668	658

Figure 27 – Comparison of Calibration and Survey Average Travel Time on GP Lanes



Segment	1	2	3	4	5
Simulation	205	316	127	207	150
Survey	214	312	145	193	163

Figure 28 – Comparison of Calibration and Survey Average Travel Time on HOV Lane



The t-test indicates that the mean segment travel times obtained from the simulation using calibrated parameters are not statistically different from mean segment travel times obtained through field observations, assuming a confidence level of 95% level. It is also worth considering the fact that the same parameter values were employed in all segments; that is, the parameters were not chosen so as to produce only locally good results. The calibration is successively completed.

Table 20 – Statistical Analysis of the Calibrated Existing Condition

Segment	GP Lane				HOV Lane			
	t value	v value	T (0.025,v)	If -T<t<T, accepted	t value	v value	T (0.025,v)	If -T<t<T, accepted
1	0.486	11.005	2.201	accepted	-0.552	10.033	2.228	accepted
2	-0.251	11.105	2.201	accepted	0.410	10.037	2.228	accepted
3	0.773	11.013	2.201	accepted	-0.859	10.048	2.228	accepted
4	-1.201	11.451	2.201	accepted	-0.452	10.512	2.228	accepted
5	0.732	11.153	2.201	accepted	1.610	12.635	2.179	accepted

Chapter 5 Alternatives Models

In this chapter, techniques employed in modeling the two alternative HOT lane facility designs, Alternatives 1 and 5, are described and results of experiments designed to demonstrate VISSIM's capability to replicate traffic conditions associated with managed lane facilities with limited access are provided. Additional experiments were run to evaluate the performance, including travel time and delay, of these proposed managed lane alternatives, results from which are presented. 2030 estimates of demand under No Build, Alternative 1 and Alternative 5 designs were employed in these experiments.

5.1 Additional Data Input

Details concerning data required for developing and running the 2030 forecast year demand scenarios and alternative designs for all segments of the study roadway were provided in Chapter 2. A few additional details concerning the 2030 demand data and associated modeling are given here, as well as in succeeding subsections.

A comparison of 2030 demand estimates with 2006 demand data indicate an expected overall increase in demand for the study roadway segment and increased usage in terms of portion of traffic in managed lanes. Thus, it was necessary to re-estimate vehicle composition by vehicle class for the 2030 runs. These estimates are given in Table 21.

Table 21 –Vehicle Classification of Alternatives for 2030 Demand Estimates

Alternative 1					
Segment	**1**	**2**	**3**	**4**	**5**
Class 1	0.057	0.054	0.052	0.055	0.057
Class 2	0.002	0.003	0.003	0.003	0.004
Class 3	0.001	0.001	0.001	0.001	0.001
Class 4	0.002	0.003	0.003	0.003	0.001
Class 5	0.037	0.043	0.048	0.042	0.034
Class 6	0.701	0.673	0.648	0.679	0.758
Class 7	0.150	0.175	0.198	0.170	0.103
Class 8	0.050	0.048	0.046	0.049	0.042
Alternative 5					
Segment	**A**	**B**	**C**	**D**	**E**
Class 1	0.047	0.044	0.041	0.045	0.050
Class 2	0.004	0.005	0.005	0.004	0.006
Class 3	0.001	0.001	0.001	0.001	0.001
Class 4	0.004	0.005	0.005	0.004	0.002
Class 5	0.064	0.072	0.080	0.068	0.058
Class 6	0.577	0.543	0.507	0.558	0.671
Class 7	0.261	0.292	0.325	0.279	0.175
Class 8	0.041	0.039	0.036	0.040	0.037
No Build					
Segment	**A**	**B**	**C**	**D**	**E**
Class 1	0.056	0.057	0.058	0.056	0.058
Class 2	0.002	0.002	0.002	0.002	0.003
Class 3	0.001	0.001	0.001	0.001	0.001
Class 4	0.002	0.002	0.002	0.002	0.001
Class 5	0.037	0.035	0.034	0.037	0.031
Class 6	0.700	0.708	0.714	0.699	0.768
Class 7	0.151	0.143	0.138	0.152	0.095
Class 8	0.050	0.051	0.051	0.050	0.043

Desired speeds, as well as acceleration and deceleration rates, are set as in the existing conditions model described in Section 3.2.1. An additional HOT lane was employed in the Alternative 5 VISSIM model. Since vehicles running in VISSIM are permitted to choose a lane in which to travel based on lane speeds, vehicles in the HOT category will distribute themselves over the two HOT lanes.

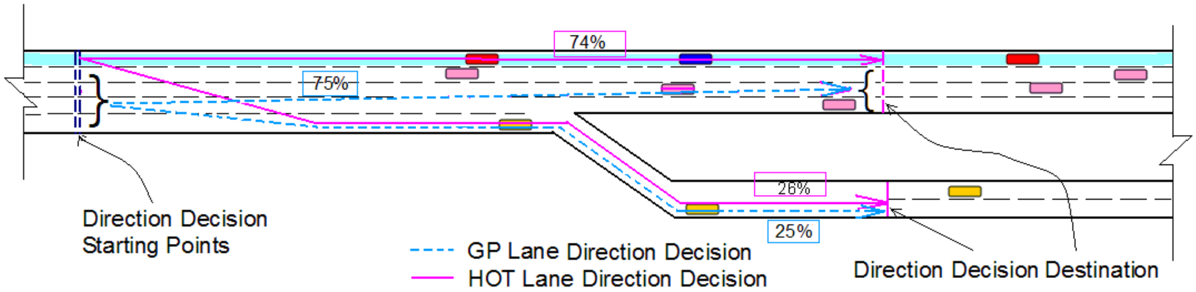
5.2 Alternative Modeling Details and VISSIM's Suitability

In this section, the realism with which VISSIM replicates traffic in managed lane facilities with limited access HOT lane(s) is evaluated. Section 5.2.1 introduces a lane-based OD control technique, referred to as Direction Decision. This technique is employed in conjunction with the link-based Vehicle Routing Decision technique discussed in Section 3.2.2. It is required for access control to the limited access HOT lane(s). The joint application of lane closure, Vehicle Routing Decision and Direction Decision enabling the VISSIM micro-simulation tool to replicate complicated vehicle weaving behavior between HOT, GP and CD lanes particularly at HOT lane access points is described in Section 5.2.2. Modeling techniques described in Section 3.2 to facilitate smooth transitioning between lanes are employed in the alternatives models. In Section 5.2.3, notable traffic behavior at particular points in the study segment is discussed.

5.2.1 Origin-Destination Modeling

Similar to the OD control method employing Route Decision illustrated in Section 3.2.2 for the existing conditions model, vehicle route control is required in the alternatives models. In addition to Route Decision, Direction Decision is employed. In VISSIM, Route Decision can only be applied on links (i.e. allowing decisions to exit HOV and GP lanes to enter the CD lanes, for example, but not to change lanes within a link); whereas, the Direction Decision permits lane-based O-D control. Direction Decision permits the turning percentage to be set by lane, rather than link. This is depicted in Figure 29, where the turning percentage (i.e. those to exit the mainstream facility from the HOV lane to enter the CD lanes) is 26% (i.e. 74% continue on) in the HOV lane, while it is 25% in the GP lanes.

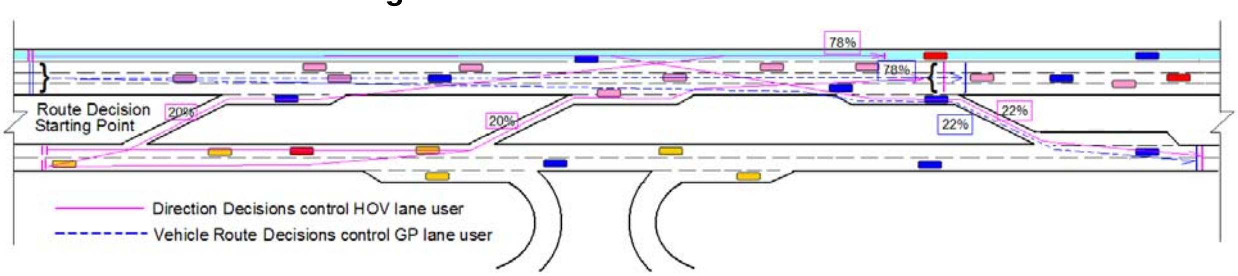
Figure 29 – Direction Decision at Slip Ramp for Alternatives



5.2.2 Access Control

To control access to the HOT lane facility so that vehicles in Classes 1, 3, 6, and 8 do not use the HOT lane facility and vehicles in Classes 2, 4, 5 and 7 will be forced into or out of the HOT lane facility at the access points, lanes are open or closed by user class within the model. To control the vehicle turning rate and weaving behavior at access points for both HOT and GP lane users in the alternatives models, Vehicle Route Decision and Direction Decision were employed jointly. Vehicle Route Decision was used to control GP lane users, i.e. Classes 1, 3, 6, and 8, directing them into the GP lanes and Direction Decision was used to direct HOT lane users, i.e. Classes 2, 4, 5, and 7, directing them into or out of the HOT lane facility. For example, at Access Point 2 (shown in Figure 30), HOT lane users present at the first slip ramp from the CD lanes were directed to enter the HOT lane(s) and HOT lane users at the second slip ramp were not permitted to enter the HOT lane(s) at that second access point. Lane closure in conjunction with Direction Decision enabled this latter prohibition. HOT lane users in the HOT lane(s) were permitted to exit the main facility via the third slip ramp. Note that the Vehicle Route Decision and Direction Decision cannot be used simultaneously for the same user classes.

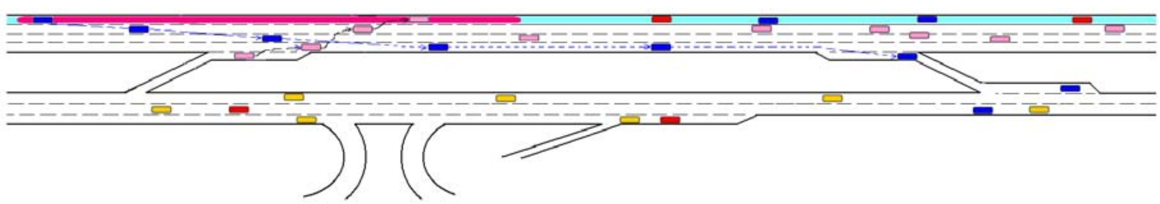
Figure 30 – Access Point 2 Control



5.2.3 Smooth Transitioning

To facilitate smooth transitioning between lanes, modeling techniques described in Section 3.2 for creating the existing conditions model were employed in creating the alternatives models. While smooth transitioning was generally noted across the roadway segment in runs of the alternatives models, performance was observed to degrade at Access Points 1 and 2. This degradation in performance appears to be a consequence of aggressive weaving behavior by vehicles entering the main facility from the CD lanes via the slip ramp wishing to enter the HOT lane facility at Access Points 1 and 2 as depicted in Figure 31. Alternatives designs were created assuming that such behavior could be prohibited; however, no physical barriers are put in place to prevent this behavior.

Figure 31 – Unpermitted Weaving at Access Point



While an alternative design is suggested by the findings of the simulation runs, i.e. where there is little or no room for entering vehicles to access the HOT lane facility from the slip ramp, modeling steps aligned with the desired outcome of the designs were taken to prevent this behavior. Direction Decision techniques were employed to force vehicles entering at these slip ramps to continue directly to the GP lanes. An alternative method may be to adjust driving behavior-related parameters.

5.3 Performance of Alternative Managed Lane Designs for 2030

In this section, the performance of Alternatives 1 and 5 in terms of travel time, delay and density is evaluated. To assess this performance, four models were run for simultaneous comparison:

1. The existing conditions model using 2006 traffic volume and composition data;

2. The existing conditions model using 2030 traffic volume and composition data, referred to as the No Build alternative;
3. The proposed Alternative 1 model using 2030 traffic volume and composition data, referred to as Alternative 1; and
4. The proposed Alternative 5 model using 2030 traffic volume and composition data, referred to as Alternative 5.

Five runs of each model were conducted employing five randomly selected seeds (identical for all four models). Each set of five runs required approximately three hours for completion on a Dell Optiplex GX520 Pentium 4 personal computer with a dual core processor, 3.20 gigahertz, and two gigabyte ram, running the Windows XP operating system.

Average travel time and hourly delay per segment and average per vehicle travel time and hourly delay for the entire study roadway length were computed over each set of five runs of the four models. All runs employed the parameter values identified in the calibration effort as given in Table 18 of Chapter 4.

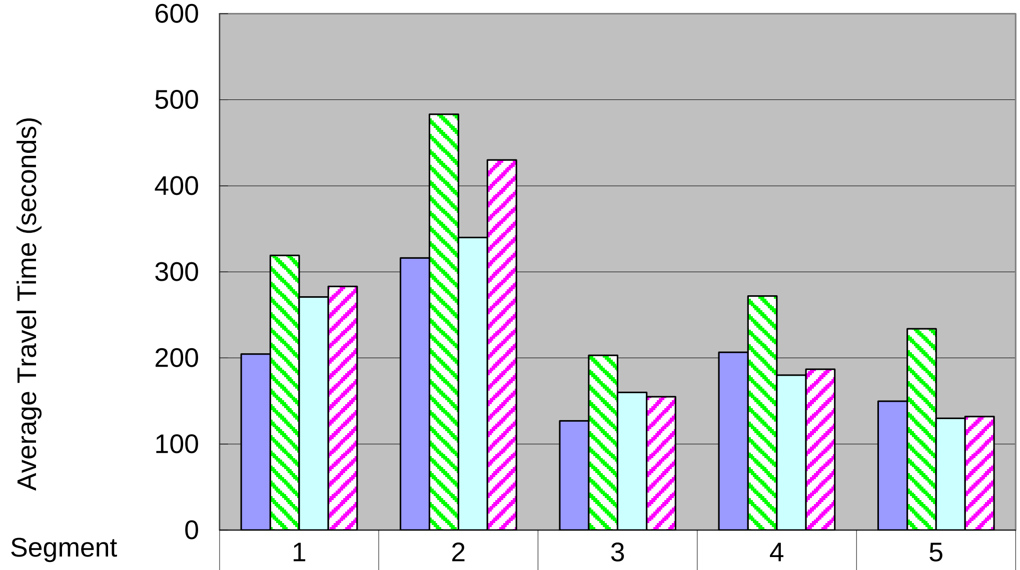
5.3.1 Evaluation of Segment Travel Times, Delays and Densities

Average travel time and hourly delay by segment and lane classification (managed and GP lanes) are reported and compared in Figures 32 through 35. Average speed by segment is provided in Table 22.

Table 22 – Average Speed of Models by Segment

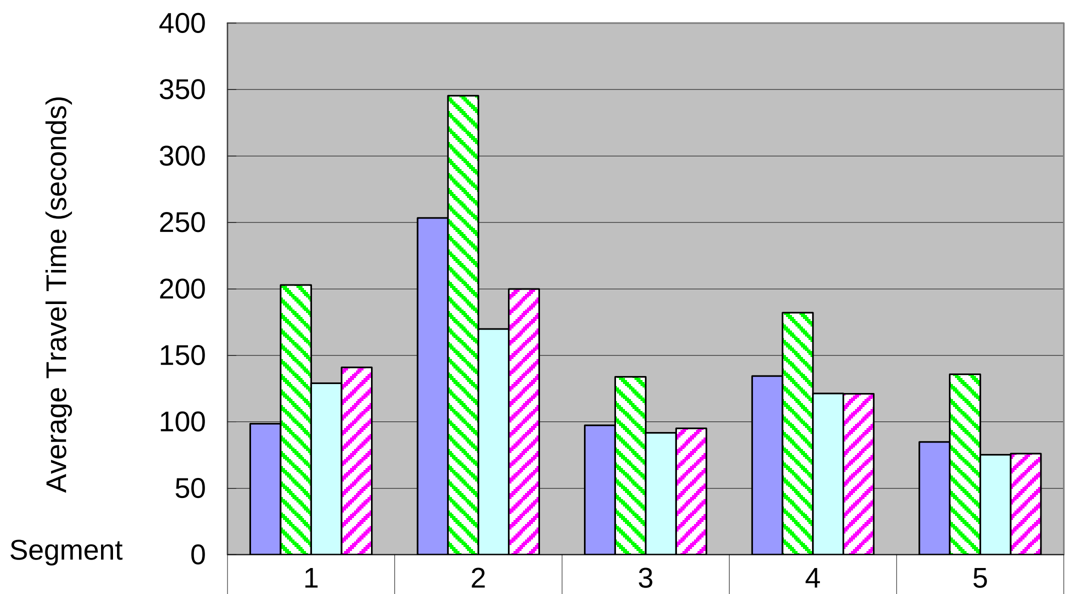
	Existing		No Build		Alternative 1		Alternative 5	
Segment	GP	HOV	GP	HOV	GP	HOV	GP	HOV
1	19.5	40.5	12.5	19.6	14.7	30.9	14.1	28.3
2	25.0	31.2	16.4	22.9	23.3	46.6	18.4	39.6
3	32.4	42.2	20.2	30.7	25.7	44.8	26.5	43.2
4	27.4	42.1	20.8	31.1	31.4	46.6	30.3	46.7
5	38.2	67.4	24.4	42.1	44.0	76.1	43.3	75.1

Figure 32 – Average Travel Time on GP Lanes



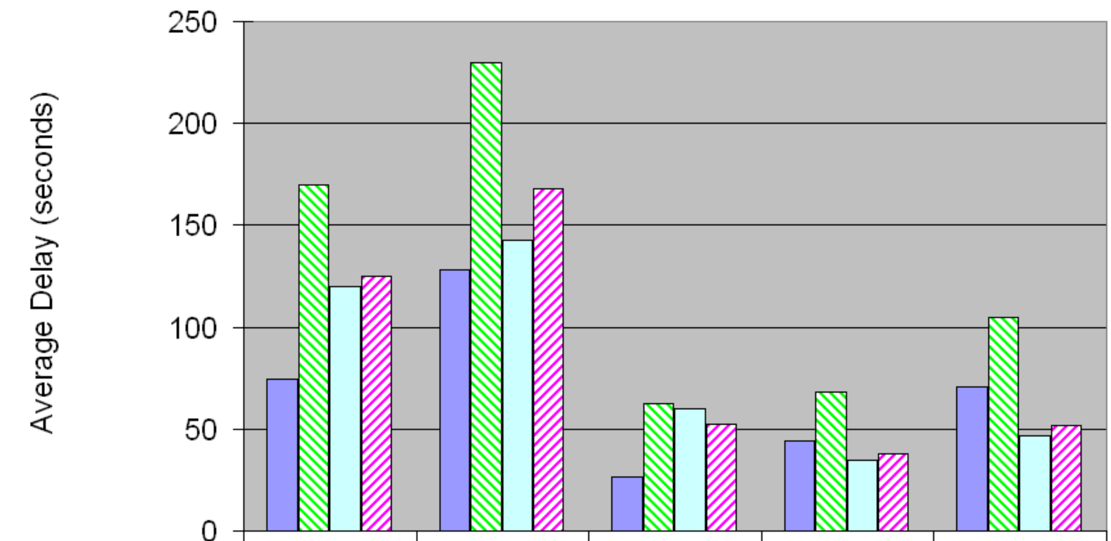
	1	2	3	4	5
Existing 2006	205	316	127	207	150
No Build 2030	319	483	203	272	234
Alternative 1 2030	271	340	160	180	130
Alternative 5 2030	283	430	155	187	132

Figure 33 – Average Travel Time on Managed Lanes



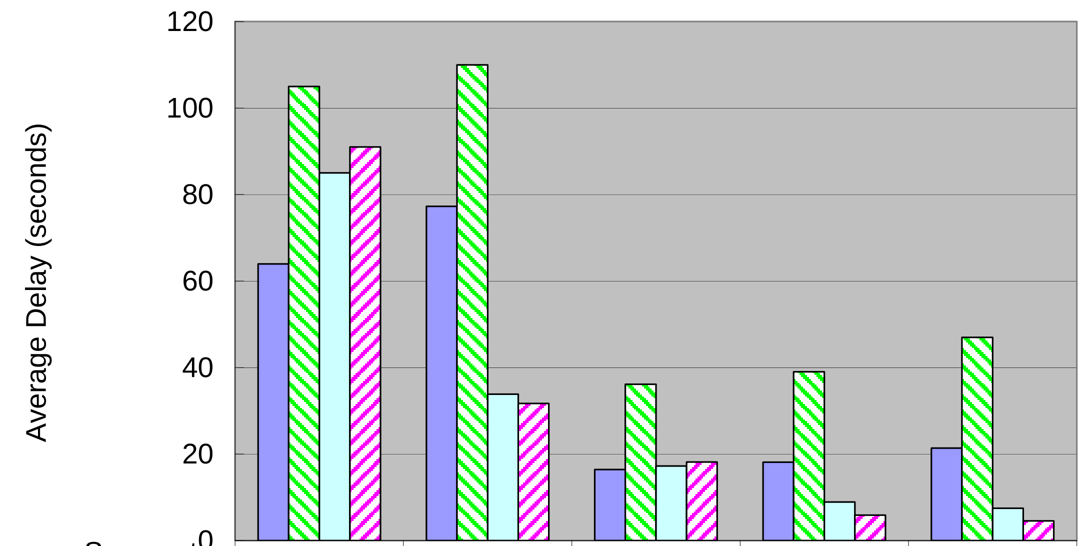
	1	2	3	4	5
Existing 2006	99	253	97	134	85
No Build 2030	203	345	134	182	136
Alternative 1 2030	129	170	92	121	75
Alternative 5 2030	141	200	95	121	76

Figure 34 – Average Delay on GP Lanes



Segment	1	2	3	4	5
Existing 2006	74	128	27	44	71
No Build 2030	170	230	63	68	105
Alternative 1 2030	120	143	60	35	47
Alternative 5 2030	125	168	53	38	52

Figure 35 – Average Delay on Managed Lanes



Segment	1	2	3	4	5
Existing 2006	64	77	16	18	21
No Build 2030	105	110	36	39	47
Alternative 1 2030	85	34	17	9	7
Alternative 5 2030	91	32	18	6	5

A statistical comparison was conducted to test the hypothesis that with 95% confidence the mean segment travel times for each lane classification were equivalent under both Alternatives 1 and 5. A series of z-tests were performed for this purpose. It was assumed that travel times are normally distributed. Results of this analysis are given in Table 23. The hypothesis is rejected for all but one segment (HOV lane only).

Table 23 -- Statistical Comparison of Alternatives 1 and 5

Segment	GP Lane		HOV Lane	
	z-value	If -1.96<z<1.96 Accepted*,	z-value	If -1.96<z<1.96, accepted
1	11.71416	Rejected	302.5007	Rejected
2	63.48207	Rejected	183.5604	Rejected
3	-8.85319	Rejected	3.627247	Rejected
4	151.9538	Rejected	-9.11718	Rejected
5	88.69442	Rejected	-0.28647	Accepted

These results indicate the following:

1. With no capacity improvements (i.e. the No Build scenario on both GP and managed lanes), average travel times increase by approximately 50% between 2006 and 2030 for both GP and managed lanes and average hourly delays increase by approximately 71% in GP lanes and 85% in managed lanes between 2006 and 2030 averaged over the entire roadway segment. Note that traffic volume in terms of total inflow is expected to increase by approximately 8% in 2030 as compared to 2006.

2. Average travel time and delay on the GP lanes of segments 1 and 2 increased by 17 and 37% (in terms of average travel time) and 29 and 45% (in terms of average delay) under Alternatives 1 and 5, respectively, as compared with existing 2006 GP lane travel times and delay, reflecting both increased traffic volume and speed reduction at Access Points 1 and 2.

3. Average travel time and delay on the GP lanes of segments 4 and 5 were reduced by 10 and 13% (average travel time) and 21 and 29% (average

delay), under Alternatives 1 and 5, respectively, as compared with existing 2006 GP lane travel times and delay, reflecting that despite increased traffic volume, the managed lane concept can help to increase roadway capacity, even with no additional physical added capacity as in Alternative 1.

4. Average travel time and delay on the HOT lanes of Segment 1 increased by 30 and 43% (average travel time) and 32 and 42% (average delay) under Alternatives 1 and 5, respectively, as compared with both existing 2006 and 2030 No Build HOV lane travel times and delay, respectively, as a consequence of increased traffic volume and increased weaving behavior between managed and GP lanes at Access Point 1.

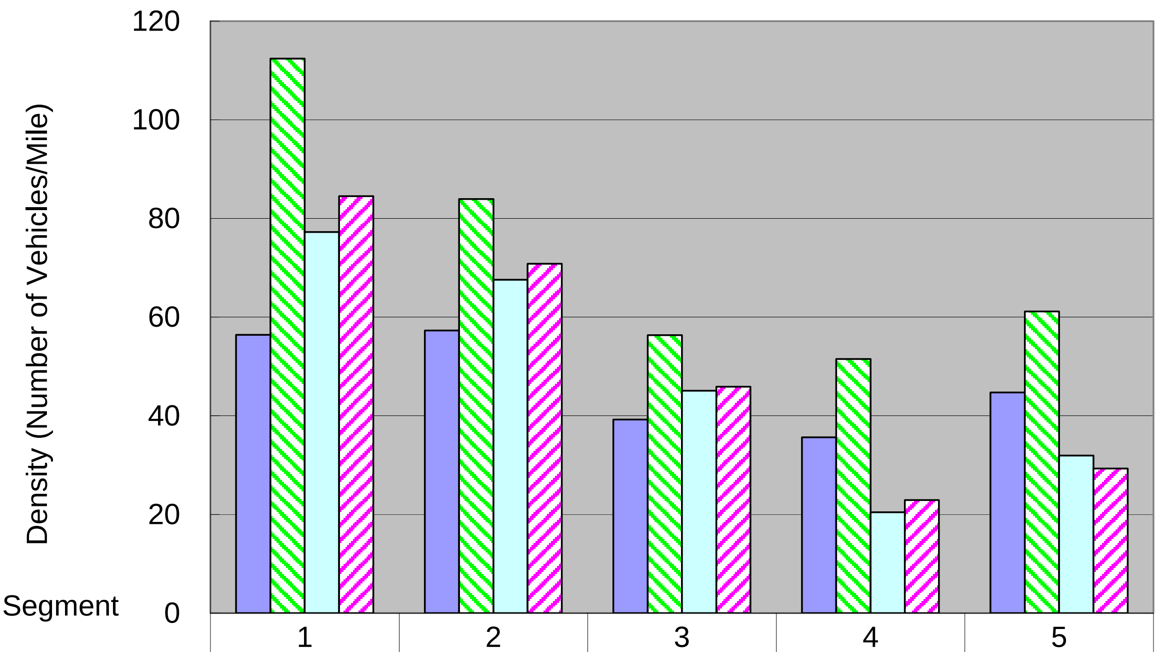
5. Average travel time and delay on the HOT lanes of segments 2 through 5 decreased by 14 and 20% (average travel time) and 49 and 54% (average delay) under Alternatives 1 and 5, respectively, as compared with both existing 2006 and 2030 No Build HOV lane travel times and delay, respectively, illustrating the potential benefits of managed lanes and the additional capacity of the second HOT lane in Alternative 5.

6. Predicted average travel time and delay for both HOT and GP lanes of Alternative 1 were reduced by 5 and 8% as compared with Alternative 5, respectively. While one might expect better performance for Alternative 5 as compared with Alternative 1, because Alternative 5 is designed with greater capacity, 2030 predicted demand for use of the HOT lanes is larger (by 13%) under Alternative 5 as compared with Alternative 1 as discussed in Section 2.2. With the need to support increased traffic volume under Alternative 5 as compared with Alternative 1, there is a potential for increased weaving behavior between HOT and GP lanes, as well as between the two HOT lanes available under Alternative 5.

7. Average speed in the managed lanes was consistently significantly higher than average speed observed in the simulation on the GP lanes in all four models.

The VISSIM model link evaluation function was employed to obtain segment densities (i.e. the number of vehicles per unit length of roadway) for the hour simulation period for each of the four models. The simulation platform reports average densities (over time) for each 10 meter segment of roadway for the simulation period. For each segment, the average density over the hour was computed, as reported in Figure 36. Similar results to that noted for average travel time and hourly delay are noted in comparing average density across the four models.

Figure 36 – Average Density on I-270 Mainstream Lanes by Segment



Segment	1	2	3	4	5
Existing 2006	56	57	39	36	45
No Build 2030	112	84	56	51	61
Alternative 1 2030	77	68	45	20	32
Alternative 5 2030	85	71	46	23	29

Additional assessment was performed, results from which are provided in Appendices B and C. Specifically, results of runs of the existing 2006 model are compared with results of runs of Alternatives 1 and 5 models using the 2006 demand data. These runs were necessary to allow comparison of the alternative designs while keeping demand constant. Note that the No Build scenario would be equivalent to the existing 2006 model for a run using the 2006 demand data. Results of these runs indicate that the alternative designs provide significantly improved service in both GP and managed lanes as compared with the existing design. This improvement is likely due to reduced weaving between the GP and managed lanes. The nearly similar performance predicted for Alternatives 1 and 5 is expected, because the managed lanes are not congested and, thus, there is no significant benefit of a second HOT lane. It is also worth considering whether or not the additional weaving that will occur between the two HOT lanes will reduce the performance of Alternative 5.

Two sets of additional simulation runs were conducted, results from which are given in Appendix C. In the first set, 2030 traffic volumes and turning percentages predicted for Alternative 1 were run under the Alternative 5 design. In the second set, 2030 traffic volumes and turning percentages predicted for Alternative 5 were run under the Alternative 1 design. If the simulation performs well, it was expected that the results would find decreased performance under the first set of runs and improved performance under the second. These runs produced results consistent with expectations, further confirming that the VISSIM micro-simulation platform is a reasonable platform for replicating traffic on facilities with concurrent flow lanes.

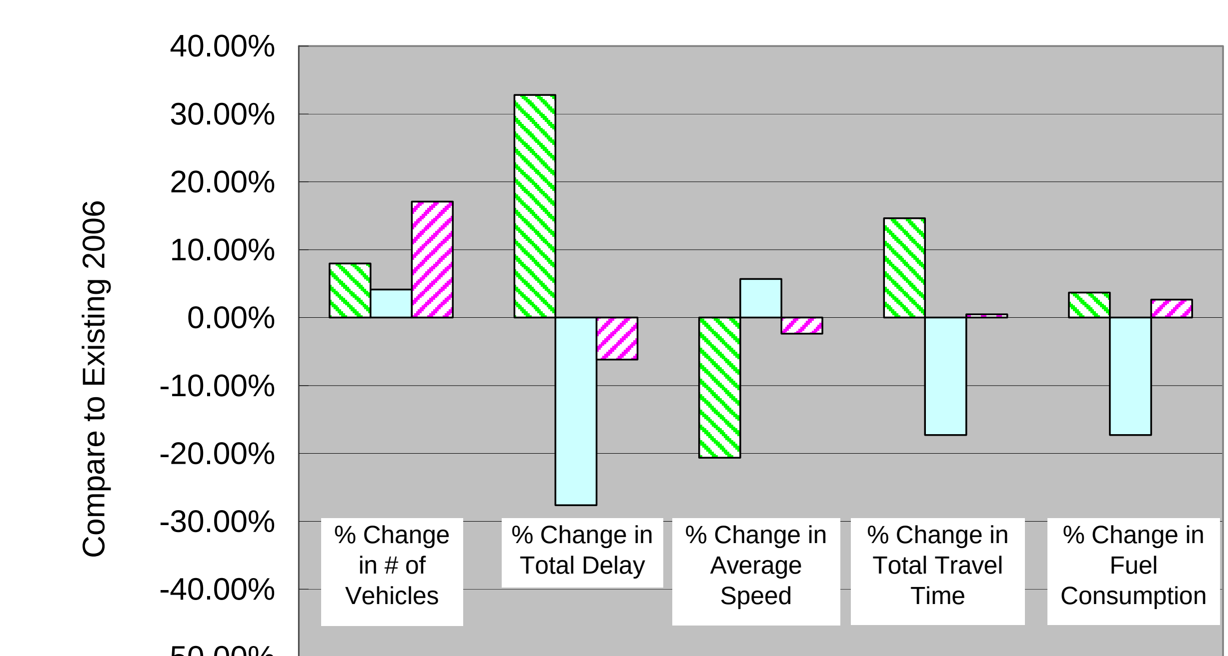
5.3.2 Evaluation of Network Travel Times and Delays

In addition to the segment-based traffic performance analysis described in Section 5.3.1, performance of the entire study roadway, aggregating predictions for GP and managed lanes, was investigated, results from which are provided in Table 22 and Figure 37.

Table 24 – Comparison of the Overall Performance for Entire Study Roadway

Scenarios	Existing	No Build	Alternative 1	Alternative 5
Total Segment Demand for Study Period	14480	15637	15079	16955
% Change in # of Vehicles	baseline	7.99%	4.14%	17.09%
% Change in Total Delay	baseline	32.78%	-27.60%	-6.17%
% Change in Total Travel Time	baseline	14.65%	-17.28%	0.49%
% Change in Emission and Fuel Consumption	baseline	3.67%	-20.66%	2.65%

Figure 37 – Comparison of Overall Performance for Entire Study Roadway



	1	2	3	4	5
No Build 2030	7.99%	32.78%	-20.63%	14.65%	3.67%
Alternative 1 2030	4.14%	-27.60%	5.70%	-17.28%	-17.28%
Alternative 5 2030	17.09%	-6.17%	-2.36%	0.49%	2.65%

Using results of the 2006 Existing Conditions model as a baseline, results of this more holistic comparison suggest that:

1. For the No Build alternative, performance in terms of both average speed and total delay significantly degrades as compared with existing conditions; thus, supporting the argument that capacity expansion is required under the predicted increase in demand.

2. The conversion of the HOV lane to a HOT lane facility, i.e. through Alternative 1, even with increased demand, leads to significantly improved performance.

3. Alternative 5 supports significantly more traffic with travel time and speed nearly equivalent to that observed under 2006 existing conditions, which

could not be accomplished under the No Build scenario.

Corresponding to findings from comparisons based on speed, the percent change in travel time and fuel consumption increases under the No Build scenario, decreases under Alternative 1 and slightly increases under Alternative 5 as compared with the 2006 existing conditions results. Note that the identical percent change in emissions (in terms of CO, NO_x and VOC) as measured within the VISSIM simulation platform to that of fuel consumed was determined. This is because the emissions were estimated as a linear function of fuel consumed.

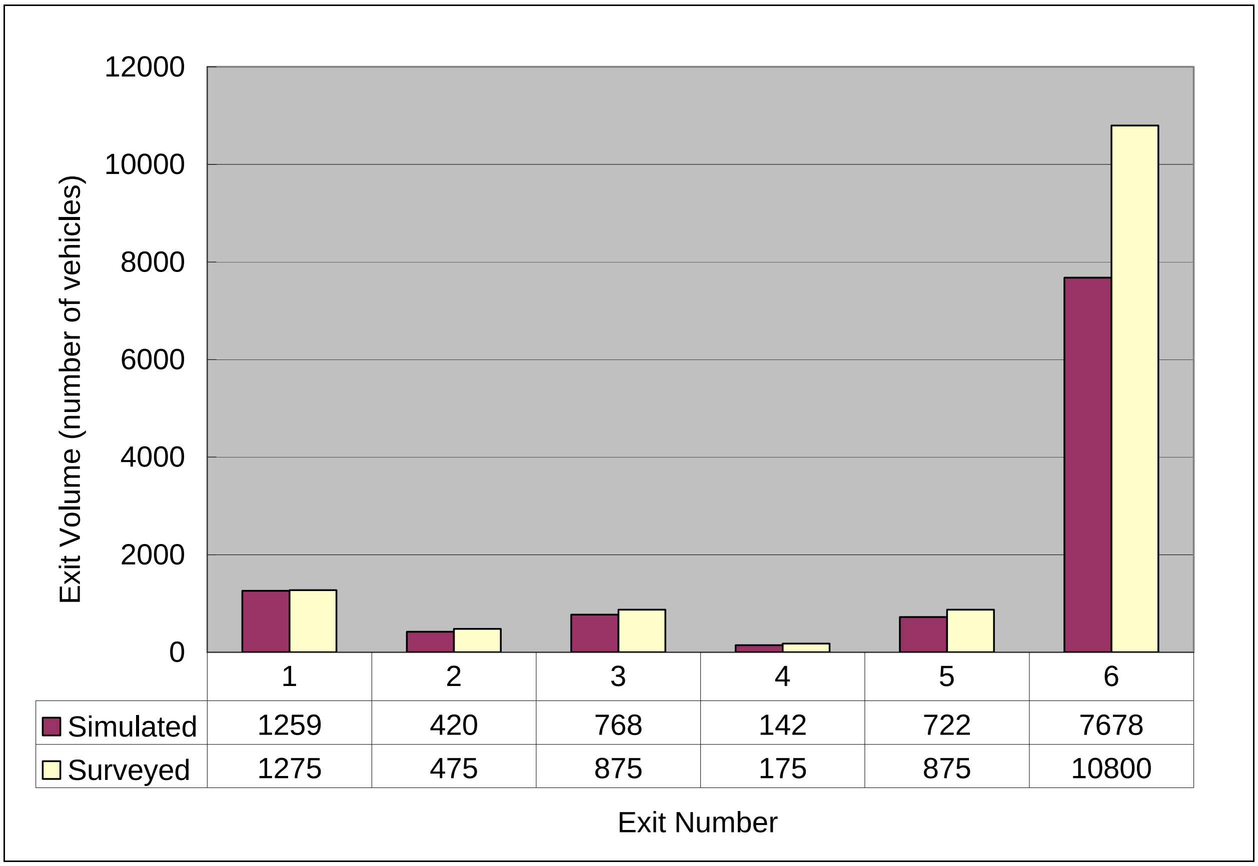
It is noteworthy that in absolute quantities, fuel consumed under the No Build Scenario is 279,296 gallons as compared with 269,399, 213,748, and 232,064 gallons consumed under the existing conditions, Alternative 1, and Alternative 5 scenarios, respectively.

To verify the reasonableness of the VISSIM simulation platform for modeling concurrent flow lane operations, the traffic inflow was compared to the traffic outflow for a single randomly chosen run of one hour duration for all four models. The results shown in Table 23 indicate a small difference between inflow and outflow, as is required to account for those vehicles that have entered, but have not yet exited the roadway network upon termination of the simulation runs. Given that the total travel time from one end of the study roadway to the other is approximately 1,000 seconds, and given demand rates for the network, one would expect approximately 4,000 to 4,700 vehicles to remain in the network, almost exactly what was noted from results of the experiments. Additionally, one can note that the inflow produced in the simulation of the existing conditions model is nearly identical to that of the surveyed inflow (14,480 compared with 14,476) used as input to the model (see Figure 11). For completeness, exiting volumes produced by one run of the existing conditions model are depicted in Figure 38.

Table 25 – Inflow and Outflow Volume

Scenarios	Inflow	Outflow	Difference
Existing 2006	14480	10459	4021
No Build	15637	11297	4340
Alternative 1 2030	15079	10885	4194
Alternative 5 2030	16955	12254	4701

Figure 38 – Outflow by Exit for a Run of the Existing Conditions Model



Exit numbering 1 through 5 in the figure corresponds with segments 1 through 5 (there is one off-ramp in each segment). Exit 6 corresponds with the outflow at the southern-most end of the simulated roadway segment.

Significant queuing was noted through observations of the resulting animation at three main locations in runs of the existing (and no build) model:

1) Slip ramp from CD lanes to GP lanes just north of Shady Grove Road;

2) Slip ramp from CD lanes to GP lanes just north of Montrose Road; and
3) Merge of CD lanes with mainstream lanes just south of Montrose Road.

Similar queuing at the slip ramps (1 and 2, above) was noted in the resulting animation of runs of the alternatives models. Additionally, queuing increased at Access Points 1 and 2 (most significantly, though, at Access Point 1).

5.4 Conclusions

Results of analyses conducted to evaluate VISSIM's capability to replicate concurrent flow lane operations, specifically nonbarrier separated HOT lane facilities with limited access, indicate that the VISSIM simulation platform is an appropriate tool for this purpose. In a comparison of results of runs designed to replicate four scenarios (existing conditions under 2006 demand, No Build under 2030 demand, and two alternative concurrent flow lane designs under 2030 demand), it was found that traffic performance will likely substantially degrade by 2030 as a consequence of increased demand should no changes to the facility be made. Moreover, this performance could be improved by converting the existing HOV lane to a limited access HOT lane or pair of lanes. In fact, the improved performance is expected under Alternative 5 despite predictions of greater demand and greater concurrent flow lane use by percent should such a conversion take place.

Chapter 6 Findings

The VISSIM micro-simulation platform was employed to replicate existing and proposed concurrent flow lane operations on a 7-mile stretch of I-270 in Maryland. Four models were constructed (existing conditions under 2006 demand, No Build under 2030 demand, and two alternative limited access HOT lane facility designs under 2030 demand as predicted under the given alternative). In this report, data related to roadway geometry, traffic volume, vehicle composition, and vehicle occupancy required for the development of the existing conditions and proposed alternative models of the study roadway segment after processing are presented, as well as techniques required to adequately model existing and expected vehicular behavior. Eight vehicular classifications were developed to model various vehicle classes and concurrent flow lane usage. Techniques were developed, as described herein, to ensure smooth transitioning between lanes and across links in both existing and proposed designs, provide continuous or limited access as required to managed lanes for only a subset of the classes, and ensure consistency in acceleration and deceleration lanes.

In addition to data preparation and modeling work, parameters of the VISSIM simulation software must be set so that traffic measures from the simulation best match actual measurements taken from the field. The process of determining the optimal set of parameters so as to minimize error is known as calibration. Initial runs were conducted using default parameter settings. Results from these runs show that mean travel times estimated by the simulation model using default parameters were statistically significantly different from observed mean travel times. Thus, it was shown that calibration of the parameters is essential. In this study, the parameters were calibrated based on average segment travel times as obtained from field observations. It was found that average segment travel times estimated from the calibrated simulation model of existing conditions matched average surveyed segment travel times with 95% confidence.

Parameters chosen through the calibration effort were employed in additional simulation experiments designed to assess the potential benefits of proposed alternative HOT lane facility designs under 2030 demand estimates. Several findings from this analysis of proposed alternatives are suggested from this study.

The results of the evaluation of the alternative HOT lane facility designs indicate that traffic performance, in terms of delay, travel time, traffic density and fuel consumption, significantly degrades under 2030 demand estimates given no facility upgrade as compared with existing 2006 operations. Conversion of the existing HOV lane to a single lane (Alternative 1) or double lane (Alternative 5) HOT lane facility results in improved roadway performance as compared with both 2030 No Build and 2006 existing facility design performance under associated demand estimates. Thus, even with increased demand for the I-270 roadway segment, overall performance improves with the conversion. Even with significantly greater forecasted traffic volume, simulation run results indicate that the performance of Alternative 5 is on par with, or perhaps slightly worse than, that of Alternative 1. This indicates that the cost of adding an additional lane in the HOT lane facility design may be warranted. One must trade-off the additional cost of facility construction and related maintenance with the added demand that can be served with comparable level of service, as well as resulting revenues in assessing the benefits of Alternative 5 in comparison to Alternative 1.

The improved performance of the limited access HOT lane facility alternatives as compared with the existing continuous access HOV lane facility may be due to the limited access design of the HOT lane facility. Limitations on access restrict lane changing decisions to only short roadway segments, thus, reducing weaving behavior between managed and GP lanes. It appears that the benefits of reducing lane changing options along the study roadway segment outweigh the degradation incurred as a result of merging behavior at the limited ingress and egress points.

The simulation results also indicate a possible concern as it relates to the proposed access point design for the limited access HOT lane facility alternatives. Specifically, abrupt weaving behavior is noted by vehicles seeking to enter the HOT lane facility at two of the three access points from the CD lanes. The alternative designs were created assuming that such behavior could be prohibited; however, no physical barriers are put in place to prevent this behavior. Such behavior could greatly degrade the traffic performance at these access points. Measures were taken to prevent such movements in the experiments and results are based on the assumption that such access could be likewise prohibited in reality. It is recommended that these access point locations be reconsidered. Alternatively, additional simulation runs can be conducted to predict performance, where such abrupt weaving behavior is permitted should the alternatives be implemented as designed.

Appendix A

AM Peak Period - Southbound (2006 Existing)

HOV: #'s in Red are from Field Counts #'s in Black are assumed

GP Lanes: #'s in Red are from Field Counts, #'s in Black are assumed

Interchange	Total Volume	HOV # Lanes	HOV % of Tot (2006 Count)	HOV Resulting Vol.	HOV HCS Adj. Capacity	HOV v/c	HOV LOS	GP Lanes # Lanes	GP Lanes % of Tot (2006 Count)	GP Lanes Resulting Vol.	GP Lanes HCS Adj. Capacity	GP Lanes v/c	GP Lanes LOS	CD Lanes # Lanes	CD Lanes % of Tot (2006 Count)	CD Lanes Resulting Vol.	CD Lanes HCS Adj. Capacity	CD Lanes v/c	CD Lanes LOS
I-370																			
	9,850	1	15%	1,478	2,233	0.66	C	3	63%	6,206	6,841	0.91	E	3	22%	2,167	6,522	0.33	B
Shady Grove Road																			
	9,700	1	13%	1,261	2,233	0.56	C	3	58%	5,626	6,841	0.82	D	2	29%	2,813	4,350	0.65	C
MD 28																			
	10,775	1	12%	1,293	2,233	0.58	C	3	56%	6,034	6,841	0.88	D	2	32%	3,448	4,350	0.79	D
MD 189																			
	10,725	1	12%	1,287	2,233	0.58	C	3	50%	5,363	6,841	0.78	D	2	38%	4,076	4,350	0.94	E
Montrose Road																			
	10,800	1	13%	1,404	2,233	0.63	C	5	87%	9,396	11,401	0.82	D						
I-270 Y-Split																			
	5,400	1	13%	702	2,233	0.31	B	3	87%	4,698	6,841	0.69	C						
West Lake Terrace																			
	5,400	1	13%	702	2,233	0.31	B	3	87%	4,698	6,841	0.69	C						
Democracy Boulevard																			
	5,400	1	13%	702	2,233	0.31	B	2	87%	4,698	4,561	1.03	F						
I-495/I-270 West Spur																			
	9,650							5	100%	9,650	11,401	0.85	D						
MD 190																			
	9,700							4	100%	9,700	9,121	1.06	F						
Cabin John Parkway																			
	8,050							4	100%	8,050	9,121	0.88	D						
Clara Barton Parkway																			
	9,025							5	100%	9,025	11,401	0.79	D						
George Washington																			
	8,100							4	76%	6,156	9,121	0.67	C	3	24%	1,944	6,522	0.30	B
VA 193																			
I-270 Y-Split																			
		1	12%	-	2,233	0.00	A	2	88%	-	4,561	0.00	A						
Rockledge Drive																			
		1	12%	-	2,233	0.00	A	2	88%	-	4,561	0.00	A						
MD 187																			
		1	12%	-	2,233	0.00	A	2	88%	-	4,561	0.00	A						
I-495																			

AM Peak Period - Southbound (2030 No Build)

HOV: #'s in Red are from Field Counts #'s in Black are assumed

GP Lanes: #'s in Red are from Field Counts, #'s in Black are assumed

Interchange	Total Volume	HOV # Lanes	HOV % of Tot (2006 Count)	HOV Resulting Vol.	HOV HCS Adj. Capacity	HOV v/c	HOV LOS	GP Lanes # Lanes	GP Lanes % of Tot (2006 Count)	GP Lanes Resulting Vol.	GP Lanes HCS Adj. Capacity	GP Lanes v/c	GP Lanes LOS	CD Lanes # Lanes	CD Lanes % of Tot (2006 Count)	CD Lanes Resulting Vol.	CD Lanes HCS Adj. Capacity	CD Lanes v/c	CD Lanes LOS
I-370																			
	10,750	1	15%	1,613	2,233	0.72	D	3	63%	6,773	6,644	1.02	F	3	22%	2,365	6,522	0.36	B
Shady Grove Road																			
	9,750	1	13%	1,268	2,233	0.57	C	3	58%	5,655	6,644	0.85	D	2	29%	2,828	4,350	0.65	C
MD 28																			
	9,975	1	12%	1,197	2,233	0.54	C	3	56%	5,586	6,644	0.84	D	2	32%	3,192	4,350	0.73	D
MD 189																			
	10,425	1	12%	1,251	2,233	0.56	C	3	50%	5,213	6,644	0.78	D	2	38%	3,962	4,350	0.91	E
Montrose Road																			
	11,300	1	13%	1,469	2,233	0.66	C	5	87%	9,831	11,073	0.89	D						
I-270 Y-Split																			
	6,125	1	13%	796	2,233	0.36	B	3	87%	5,329	6,676	0.80	D						
West Lake Terrace																			
	6,125	1	13%	796	2,233	0.36	B	3	87%	5,329	6,676	0.80	D						
Democracy Boulevard																			
	5,325	1	13%	692	2,233	0.31	B	2	87%	4,633	4,451	1.04	F						
I-495/I-270 West Spur																			
	10,050							5	100%	10,050	11,019	0.91	E						
MD 190																			
	10,550							4	100%	10,550	8,815	1.20	F						
Cabin John Parkway																			
	8,350							4	100%	8,350	8,815	0.95	E						
Clara Barton Parkway																			
	10,350							5	100%	10,350	11,019	0.94	E						
George Washington																			
	10,125							4	76%	7,695	8,815	0.87	D	3	24%	2,430	6,522	0.37	B
VA 193																			
I-270 Y-Split																			
		1	12%	-	2,233	0.00	A	2	88%	-	4,429	0.00	A						
Rockledge Drive																			
		1	12%	-	2,233	0.00	A	2	88%	-	4,429	0.00	A						
MD 187																			
		1	12%	-	2,233	0.00	A	2	88%	-	4,429	0.00	A						
I-495																			

AM Peak Period - Southbound (2030 Alternative 1)

		ETL					GP Lanes							CD Lanes						
Interchange	Total Volume	# Lanes	Forecast Vol.	HCS Adj. Capacity	v/c	LOS	# Lanes	% of GP+CD (2006 Count)		Resulting Vol.	HCS Adj. Capacity	v/c	LOS	# Lanes	% of GP+CD (2006 Count)		Resulting Vol.	HCS Adj. Capacity	v/c	LOS
I-370																				
	10,750	1	1,600	2,168	0.74	D	3	#'s in Red are from Field Counts, #'s in Black are assumed	74%	6,771	6,644	1.02	F	3	#'s in Red are from Field Counts, #'s in Black are assumed	26%	2,379	6,522	0.36	B
Shady Grove Road																				
	9,900	1	1,600	2,168	0.74	D	3		67%	5,561	6,644	0.84	D	2		33%	2,739	4,350	0.63	C
Gude Drive																				
	9,900	1	1,600	2,168	0.74	D	3		64%	5,312	6,644	0.80	D	2		36%	2,988	4,350	0.69	D
MD 28																				
	9,900	1	1,600	2,168	0.74	D	3		57%	4,731	6,644	0.71	C	2		43%	3,569	4,350	0.82	D
MD 189																				
	10,350	1	1,600	2,168	0.74	D	3		66%	5,775	6,644	0.87	D	2		34%	2,975	4,350	0.68	D
Montrose Road																				
	11,350	1	1,700	2,168	0.78	D	5		100%	9,650	11,073	0.87	D							
I-270 Y-Split																				
	6,525	1	1,475	2,168	0.68	C	3		100%	5,050	6,676	0.76	D							
West Lake Terrace																				
	5,700	1	650	2,168	0.30	A	3		100%	5,050	6,676	0.76	D							
Democracy Boulevard																				
	5,800	1	650	2,168	0.30	A	2		100%	5,150	4,451	1.16	F							
I-495/I-270 West Spur																				
	10,750	1	650	2,158	0.30	A	5		100%	10,100	11,019	0.92	E							
At-Grade Access																				
	10,750	1	1,600	2,158	0.74	D	5		100%	9,150	11,019	0.83	D							
MD 190																				
	11,425	1	1,600	2,158	0.74	D	4		100%	9,825	8,815	1.11	F							
Cabin John Parkway																				
	9,275	1	1,600	2,158	0.74	D	4		100%	7,675	8,815	0.87	D							
Clara Barton Parkway																				
	10,675	1	1,775	2,158	0.82	D	5		100%	8,900	11,019	0.81	D							
George Washington																				
	10,825						5		76%	8,227	11,019	0.75	D	3		24%	2,598	6,522	0.40	B
VA 193																				

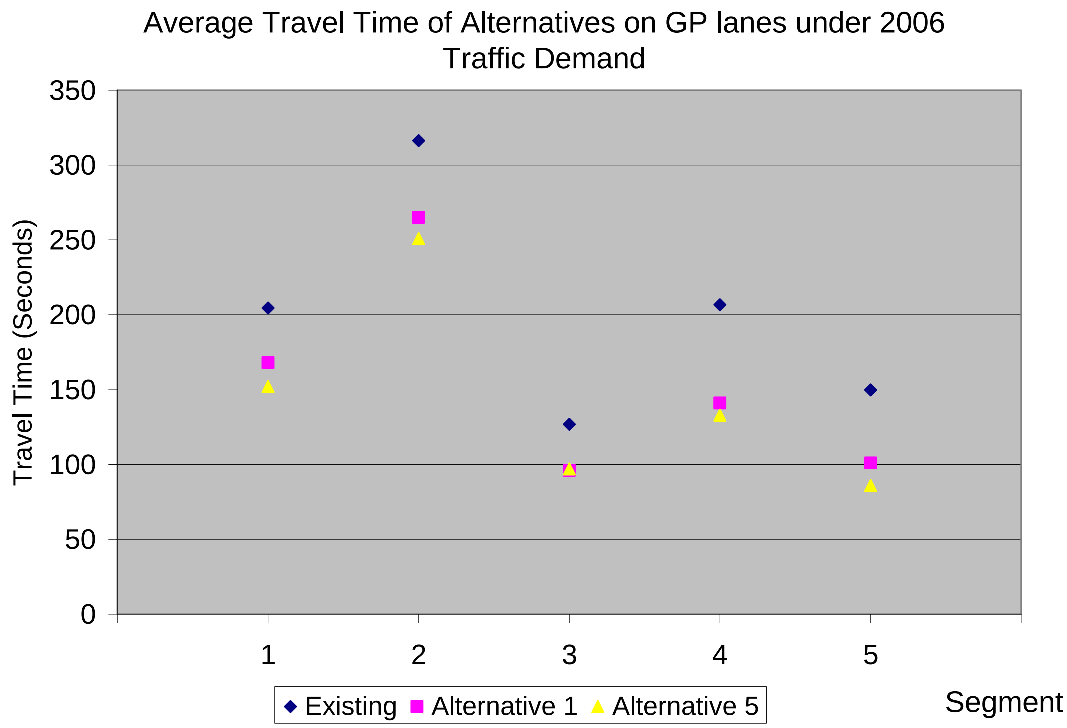
I-270 Y-Split																				
		1		2,158	0.00	A	2		100%	-	4,429	0.00	A							
Rockledge Drive																				
		1		2,158	0.00	A	2		100%	-	4,429	0.00	A							
MD 187																				
		1		2,158	0.00	A	2		100%	-	4,429	0.00	A							
I-495																				

AM Peak Period - Southbound (2030 Alternative 5)

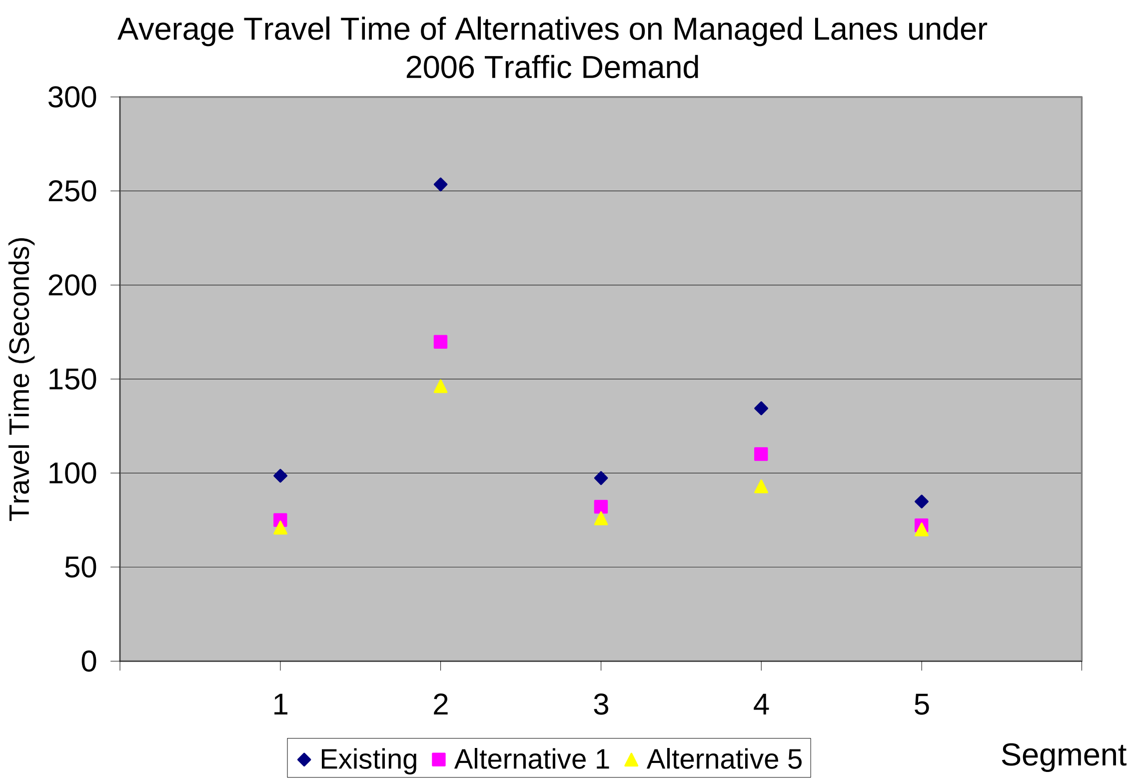
		ETL					GP Lanes							CD Lanes						
Interchange	Total Volume	# Lanes	Forecast Vol.	HCS Adj. Capacity	v/c	LOS	# Lanes	% of GP+CD (2006 Count)		Resulting Vol.	HCS Adj. Capacity	v/c	LOS	# Lanes	% of GP+CD (2006 Count)		Resulting Vol.	HCS Adj. Capacity	v/c	LOS
I-370																				
	12,200	2	3,300	4,337	0.76	D	3	#'s in Red are from Field Counts, #'s in Black are assumed	74%	6,586	6,644	0.99	E	3	#'s in Red are from Field Counts, #'s in Black are assumed	26%	2,314	6,522	0.35	B
Shady Grove Road																				
	11,575	2	3,300	4,337	0.76	D	3		67%	5,544	6,644	0.83	D	2		33%	2,731	4,350	0.63	C
Gude Drive																				
	11,600	2	3,325	4,337	0.77	D	3		64%	5,296	6,644	0.80	D	2		36%	2,979	4,350	0.68	D
MD 28																				
	11,550	2	3,325	4,337	0.77	D	3		57%	4,688	6,644	0.71	C	2		43%	3,537	4,350	0.81	D
MD 189																				
	12,450	2	3,325	4,337	0.77	D	3		66%	6,023	6,644	0.91	E	2		34%	3,103	4,350	0.71	D
Montrose Road																				
	13,425	2	3,225	4,337	0.74	D	5		100%	10,200	11,073	0.92	E							
I-270 Y-Split																				
	6,850	2	3,000	4,337	0.69	C	3		100%	3,850	6,676	0.58	C							
West Lake Terrace																				
	5,450	2	1,600	4,337	0.37	B	3		100%	3,850	6,676	0.58	C							
Democracy Boulevard																				
	6,025	2	1,600	4,337	0.37	B	2		100%	4,425	4,451	0.99	E							
I-495/I-270 West Spur																				
	10,750	2	1,600	4,316	0.37	B	5		100%	9,150	11,019	0.83	D							
At-Grade Access																				
	10,750	2	3,200	4,316	0.74	D	4		100%	7,550	8,815	0.86	D							
MD 190																				
	11,175	2	3,200	4,316	0.74	D	4		100%	7,975	8,815	0.90	E							
Cabin John Parkway																				
	9,050	2	3,200	4,316	0.74	D	3		100%	5,850	6,611	0.88	D							
Clara Barton Parkway																				
	11,075	2	3,075	4,316	0.71	C	4		100%	8,000	8,815	0.91	E							
George Washington																				
	10,825						5		76%	8,227	11,019	0.75	D	3		24%	2,598	6,522	0.40	B
VA 193																				

I-270 Y-Split																				
		1		2,158	0.00	A	3		100%	-	6,611	0.00	A							
Rockledge Drive																				
		1		2,158	0.00	A	2		100%	-	4,408	0.00	A							
MD 187																				
		1		2,158	0.00	A	2		100%	-	4,408	0.00	A							
I-495																				

Appendix B



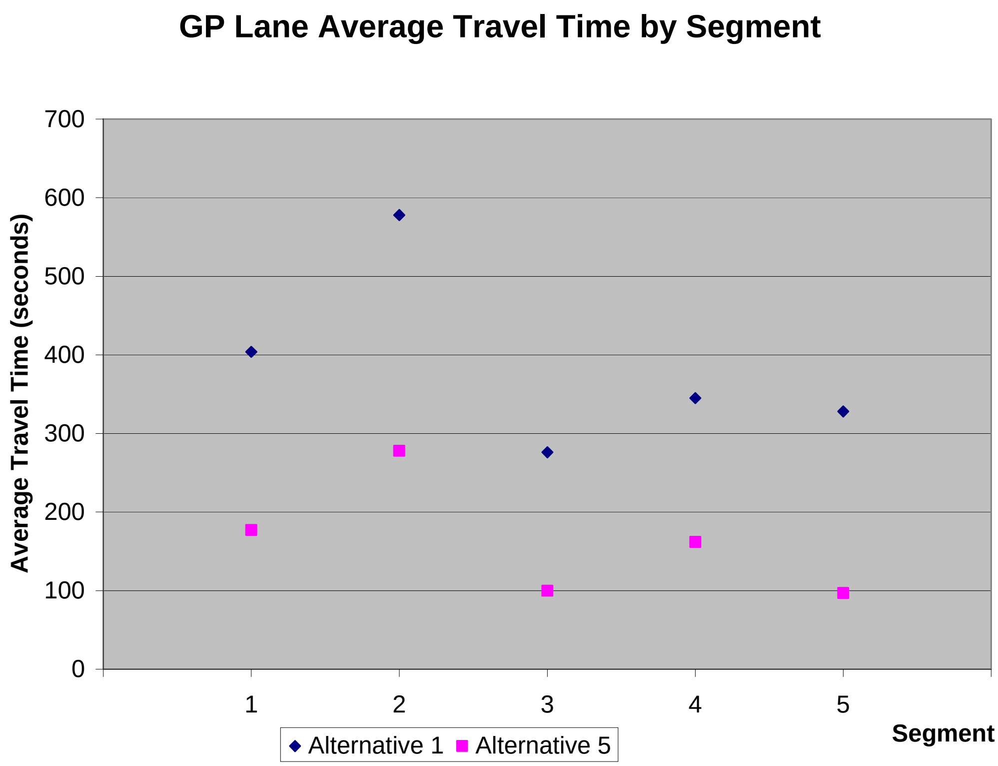
Average Travel Time of Alternatives on GP lanes under 2006 Traffic Demand
Travel Time (Seconds)
350
300
250
200
150
100
50
0
1
2
3
4
5
Existing
Alternative 1
Alternative 5
Segment



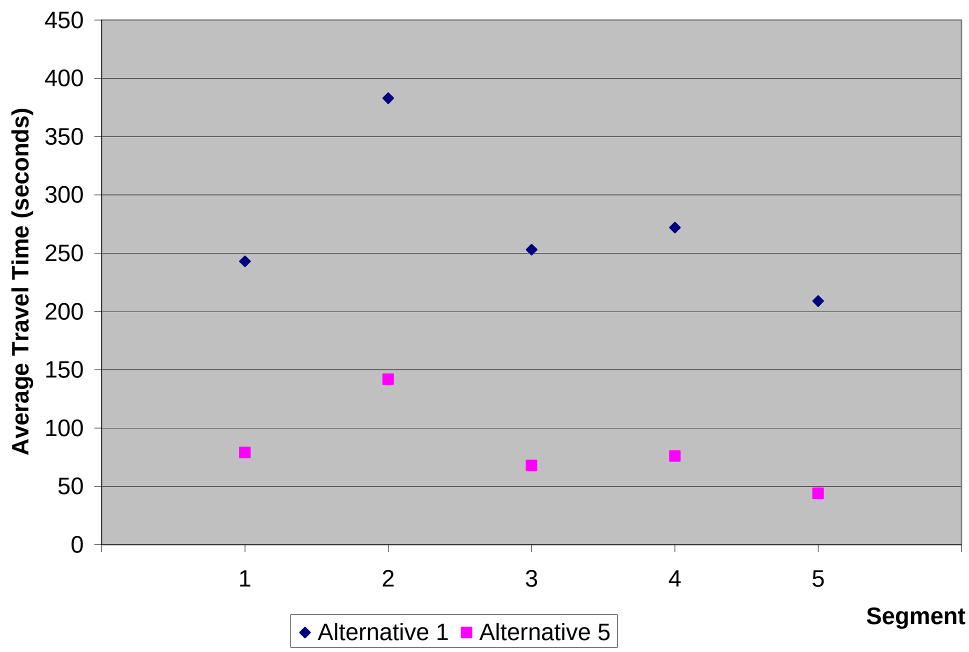
Average Travel Time of Alternatives on Managed Lanes under 2006 Traffic Demand
Travel Time (Seconds)
300
250
200
150
100
50
0
1
2
3
4
5
Existing
Alternative 1
Alternative 5
Segment

Appendix C

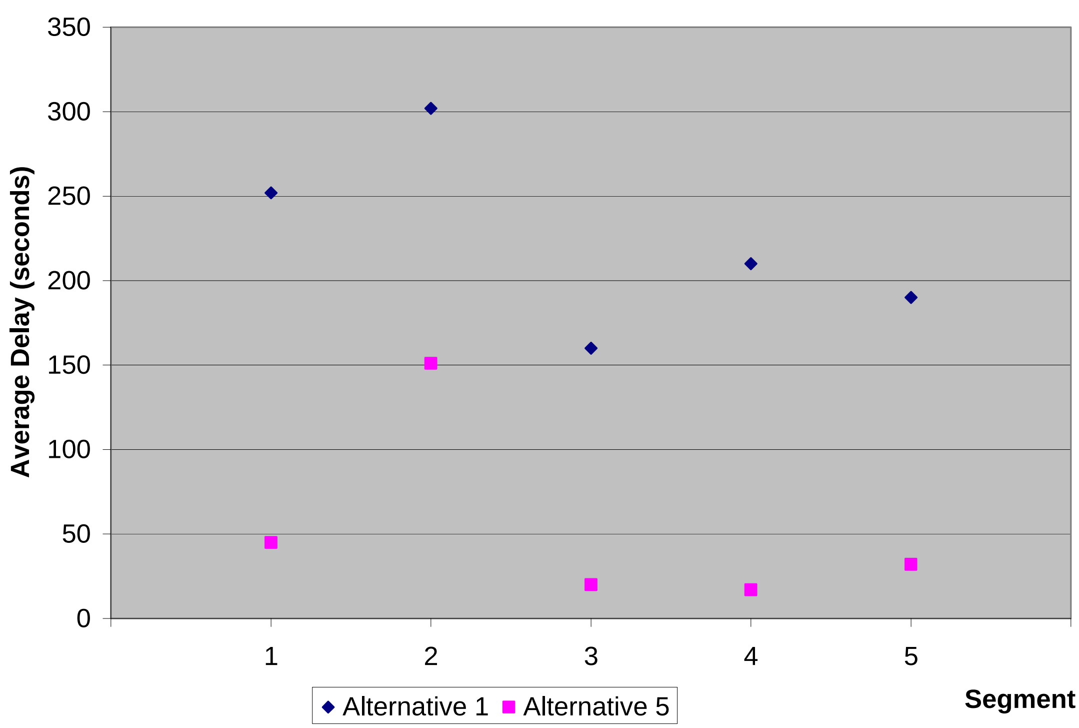
The results shown below is the Alternative 1 model running with Alternative 5 2030 traffic volume and Alternative 5 model running with Alternative 1 2030 traffic volume

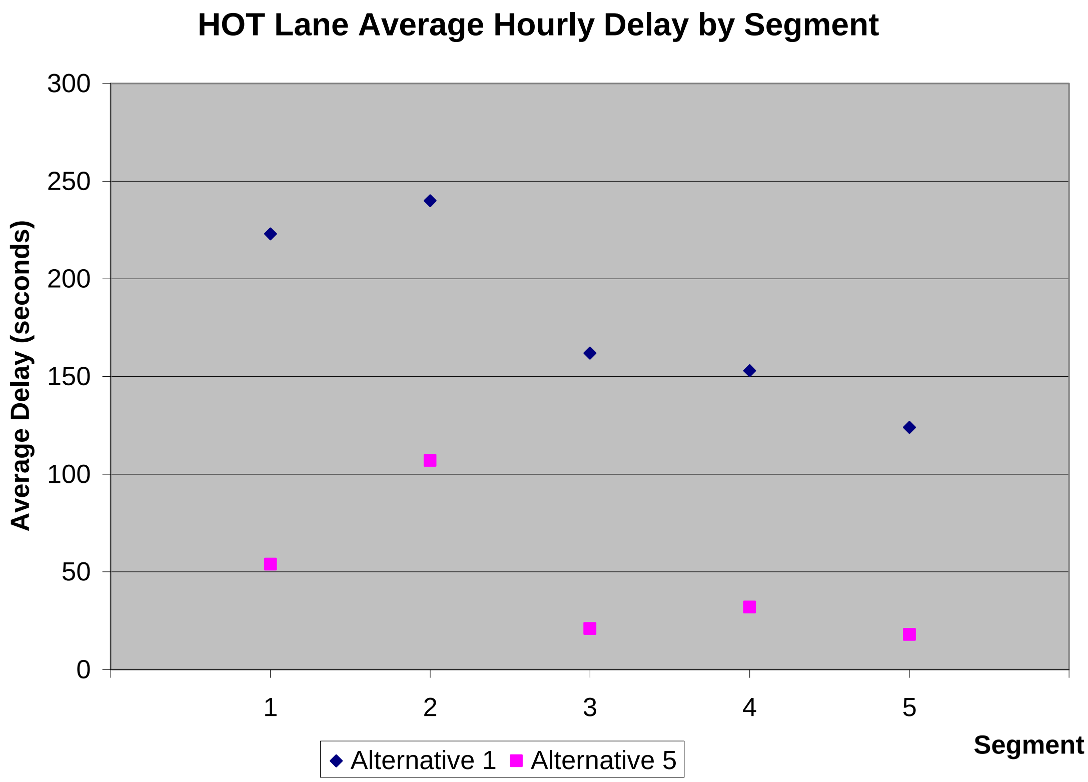


HOT Lane Average Travel Time by Segment



GP Lane Average Hourly Delay by Segment




HOT Lane Average Hourly Delay by Segment
Average Delay (seconds)
300
250
200
150
100
50
0
1
2
3
4
5
Segment
Alternative 1
Alternative 5

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