

Application of Highway Capacity Manual 2010 Level-of-Service Methodologies for Planning Deficiency Analysis

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DEDICATION

I dedicate this work to my family and the love of my life, thank you for your support through it all.

Philippians 4:13

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ABSTRACT

Influence on investment in the infrastructure is critical for decision-making by State planning departments. Judgments which are made for funding purposes and are based on limited data analysis by states can result in economic disasters for the state and nation. The Tennessee Department of Transportation (TDOT) uses the Evaluation of Roadway Efficiency (EVE) database to prioritize the importance of roadway projects in the state. In addition, EVE is also used as a method to provide deficiency analysis results for roadways in the state. This study will add support to the methodology of the current decision-making process in regards to Tennessee roadways for planning. Updating the current deficiency analysis program (EVE) with the methodologies of the *Highway Capacity Manual* (HCM) 2010 is the foundation for improving the decision-making abilities of those using EVE for such efforts. New algorithms have been developed based upon the procedures in the HCM 2010 for the following sections: Freeway, Two-Lane Highways, Multilane Highways, and Urban Arterials. Using these algorithms and data from Washington County in Tennessee, level-of-service results were analyzed and compared to previous deficiency analysis results. Some of these results were found to be better and other were found to be worse than the previous. The new method of the HCM 2010 applied in this procedure changed 60% of the level of service ratings based on the comparable data. This proves to show the update of the deficiency analysis method currently used will be beneficial in the assistance of the decision-making process.

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Chapter I – Introduction

The many state transportation organizations maintain an understanding of current and future roadways quality of service. These understanding may be based on the use of deficiency analysis. Deficiency analysis is used to determine weaknesses found in a system. This is found true in many types of systems, especially in roadway systems. Deficiency analysis is used by some as a method of appropriating funding for projects. This is the case when it comes to the Tennessee Department of Transportation. In order to provide assistance to TDOT's efforts for deficiency analysis, an improvement of the model they currently use will be established in this study. Using the *Highway Capacity Manual* (HCM) 2010 and the Tennessee Roadway Information Management System (TRIMS) developed by TDOT, this study will modernize the program for better Planning-Level analysis and quality of service. Planning-level analysis is beneficial for gaining understanding of roadway operations without the use of detailed information. Many developers use planning-level analysis to visualize necessary design ideas. Planning-level analyses are common in the transportation industry. Many engineers utilize the *Highway Capacity Manual* (HCM) methods of level-of service determination to assist them in planning efforts.

Level-of-service (LOS) is the best known measure for qualitatively assessing the quality of a roadway. In order to assist TDOT in projecting the level-of-service (LOS) of roadways in the state of Tennessee, updates have to be made to the structure of their current deficiency analysis program. The program developed by TDOT used for deficiency analysis is known as the Evaluation of Roadway Efficiency, EVE.

The most recent version of EVE is based on the methodologies of the HCM 2000. In order for EVE to remain most effective in its analysis, the methodologies used as a basis need to remain updated with those of the most current version of the HCM. The changes from HCM 2000 to HCM 2010 are a factor in the modification of EVE methodologies and will be presented in this study.

Planning-level analysis is beneficial for the appropriation of funding within the state as well as other forms of decision-making. Since the EVE database is used to prioritize projects in the state by level of importance, the methodologies must be kept up-to-date. The connection of the HCM and EVE is vitally important to maintain the principles on which EVE has been based. With the use of the algorithms from HCM 2010 Level of Service (LOS) analysis, planning efforts are made to help improve the accuracy of the deficiency analysis in Tennessee Roadway Systems. The new methods will be tested on data from Washington County in Tennessee and compared to results from the same location provided by TDOT researchers. The proposal of a newer version of the EVE database will be presented and suggested based on the results of the findings.

Chapter II – Literature Review

This chapter reviews previous research conducted in areas related to this thesis. Those areas are separated into sections for planning analysis using the *Highway Capacity Manual* (HCM), improvements of levels of service, and Investment decision for highways. This chapter is to provide an idea of the issues this thesis desires to address. The research described below provides important perspective to the ideas formulated in this thesis.

HCM for Planning

Using the HCM for planning level analysis is the basis of this study. It is important to understand the ideas of previous researchers and their use of the *Highway Capacity Manual* for planning or operational analysis. Researchers believe the HCM is a “principal guide in transportation decision-making, planning, and design. (Kittelson & Roess)” Research shows the importance of using the HCM for planning; however they understand the difficulties in this as well.

Washburn and McLeod studied the planning methods of HCM 2000 in order to create a with planning-analysis capabilities for two-lane and multilane highways. The software developed by the Florida Department of Transportation (FDOT) is known as HIGHPLAN. Creating this software based on the methods of the HCM is a difficult task. Difficulty arose when attempting to remain connected to the specific methods in the HCM while also having to make necessary assumptions for planning analysis.

Washburn and McLeod identified areas of the HCM needing changes as those of base information calculations and Two-Lane highway classification. The changes in base calculations suggested were the estimation of free flow speed, median adjustments, and capacity. With these changes addressed, the results of their research grasped how sensitive the planning application would be due to simplifications and assumptions that had been made. The authors stress the importance of knowing the restrictions of using a planning application based on assumed and simplified values.(Washburn, McLeod, & Courage, 2002)

Further research in the use of simplified values suggest caution is to be made when assuming input values in planning analysis based on the HCM. Zegeer and Blogg performed a study to identify the relevance of the default values suggested in the HCM 2000 analysis. This study is the basis of NCHRP Report 599. The research plan was to determine if the suggested default values used in the HCM analysis were relevantly related to values measured in the field for the same conditions. A sensitivity analysis was conducted to identify the relevance of the default values. Of the default input parameters analyzed, nineteen of the defaults returned values of high sensitivity. Results also showed one-third of the parameters have an associated default values of significance to the measure of service. Using default values can impact the service measure more than initially estimated.(Zegeer, Blogg, Nguyen, & Vandehey, 2008)

Further research shows the HCM can be used for multiple planning level analyses. Guttenplan and Davis studied the connection of the planning-level analysis and multimodal analysis. This study was formed as a result of the Urban Infill and Redevelopment Act of 1999. This Act encouraged the use of alternative transportation modes instead of the automobile. In an attempt to boost this wanted change, “multimodal transportation districts” (MMTD) were created. These districts made it easier for other modes to flourish such as sidewalks and bicycle lanes.

Research revealed a connection between land use development and transportation modes. The evaluation of the MMTD was based on level-of-service techniques and the application of those techniques in the HCM. The results from this study provided the government with a developed template for current and future development enhancement of multimodal planning.(Guttenplan, Davis, Steiner, & Miller, 2003)

The use of the HCM in planning also provides its fair amount of doubt in the uncertainty of the results. There is a certain level of uncertainty that can be tolerated when using the HCM and other manuals for decision-making. Research shows those who use the HCM for analysis has some doubts in the estimation. Tarko and Songchitruksa conducted surveys of analyst who use the HCM for planning analysis to communicate with stakeholders and decision-makers. They also provide a proposal procedure for handling uncertainty.

Through a web-based survey Tarko and Songchitruksa wanted to identify the confidence levels of HCM users in the results produced by the HCM. Targets of the survey were members of the Institute of Transportation Engineers, family members and friends of the Highway Capacity and Quality of Service committee. The results of the survey were as follows:

- 70% believe the knowledge of the accuracy in the outputs would benefit the decision-making process
- 67% believe it is best to report the uncertainty in the HCM results
- Transportation planners want to handle uncertainty more than traffic and design engineers

Tarko and Songchitruksa discovered the majority of the engineers using the HCM support the idea of handling data related to uncertainty. These results are due to transportation engineers

believing the uncertainty reports will result in better decision-making based on the HCM analysis. They also heed that reporting uncertainty can be beneficial but first must be researched to identify the non-engineering impacts as well as formatted to the understanding of transportation decision-makers and stakeholders.(Tarko & Songchitruksa, 2003)

Level of Service Improvements

The next topic of research related to this study is improvements of level of service (LOS). The HCM is used to provide the best measure of quality of service for a roadway with given conditions. The HCM methods for estimating level of service are directly related decision-making purposes based on quality analysis. Meaning, any efforts at improving the level of service methods can results in improvements in the planning analysis of the HCM.

Research shows the HCM levels of service are based on different measures of effectiveness (MOEs). However, these MOEs are selected by the Highway Capacity and Quality of Service Committee (HCQSC) as to which would best represent the LOS for the type of roadway. Many researchers have found that travelers perceive the LOS for roadway differently than what the HCQS believe.

Washburn, Ramlackhan, and McLeod performed a study to identify new measures of quality of service. Previous measures were based on the Highway Capacity Quality of Service Committee (HCQSC) and what they deemed was suitable to measure quality. Some committee members felt it was necessary for an alternative measure related to traveler perceptions. Therefore, a survey was conducted for travelers near a rural freeway in Florida. The idea of the study was to

gain information regarding the trip, personal information of the travelers, and overall traveler opinion of the trip on the freeway.

Washburn, Ramlackhan, and McLeod identified what the study refers to as the “cruise-control factor” to be the most important factor based on traveler opinion. This factor is related to the ability for travelers to maintain their desired travel speeds with deceleration. Other important factors were found to be freeway density, pavement quality, the percent of travel at the free flow speed, etiquette of fellow drivers on the road, and construction zone presence. Research also showed that there roadway quality is not solely based on one factor. Many factors were found to be significant, some of these factors are related to traffic and some are not. However, it is important to take each of these factors into account when assessing a roadway. (Washburn, Ramlackhan, & McLeod, 2004)

Another study conducted by Pechaux, Pietrucha, and Jovanis displayed the assessment of LOS at signalized intersections based on the HCM LOS methods from a user perspective and factors affecting user LOS perceptions. A model was created to estimate the LOS based on user perceptions. This model included factors pertaining to time-of-day, location, trip putrpose, personal characteristics, value of time and time use, temporal urgency, delay, signal characteristics, and user experience expectations. Results showed that users were more adherent to delay than the HCM originally believed. The reuslts showed that user classified their LOS beliefs into four levels of service as opposed to six. The estimation of delay for users at signalized intersections were found to “accurate but variable.” Results further determined that traveler

perceptions of LOS at signalized intersections were sensitive to factors other than delay. These factors could be addressed in future determination of LOS.

Researchers also believed that some parts of the level of service determination could be further analyzed for reclassification of levels. Azimi and Zhang 2010, report of using statistical methodology in order to provide better classification and categories for freeway flow conditions. The basis of their study was to improve quality of service assignments in comparison to those of the HCM. The HCM's main focus of analysis is of under saturated conditions. Their focus for the clustering was not only based on under saturated conditions, but oversaturated conditions as well. According to the authors, "Clustering is an unsupervised learning method that assigns observations into different groups or clusters" (Azimi & Zhang). The clustering methods used to identify the best classification for the given freeway were: K-means, fuzzy C-means, and CLARA. The data used in this study is from a loop detector in Austin, TX from 2004. Clusters were analyzed based on flow, speed, and density. Results of the study for flow and speed clustering showed that K-means and CLARA both produced results with only one oversaturated cluster but fuzzy C-means produced two. In order to validate the procedure, data from Minneapolis, MN was used from 2008 to 2009. From the conclusion of results, the idea arose to sub cluster oversaturated conditions. The LOS Class F, unlike that of the HCM, would be divided into three classes for better understanding of traffic during oversaturated conditions. They proposed the HCM differentiate between both conditions instead of just the under saturated condition. (Azimi & Zhang, 2010)

Similar to Azimi and Zhang, Brilon and Estel focused on subdividing the LOS Class F based on the German *Highway Capacity Manual*. The authors believed that in order to identify the worst areas of congestion that occur, the oversaturated conditions needed to be analyzed further. A

proposal of dividing the LOS Class F into four classes (F1-F4) was given. The study was done using freeway analysis. Given that the results are sensitive, the use of software is suggested to evaluate the methods further. The purpose of this study is to better identify the most problematic areas for congestion. The identification of bottlenecks is very important to planning procedures. If results are provided in the range of those similar to bottlenecks, planning adjustment could be made to the roadway to help improve the quality. (Brilon & Estel, 2010)

Highway Investment Decisions

The final topic of research related to this study is those of highway investment decisions. The ultimate scope of this study is for the development and use of a deficiency analysis program as a selector for highway investment. Sharma, Al-Hussein, Safouhi, and Bouferguene conducted a study of the use of an asset LOS (ALOS) determination methodology with the inclusion of a decision support system (DSS) to assess a roadway infrastructure. The model is based on the analytical hierarchy method and is believed to determine LOS in regards to more qualitative measures in comparison to quantitative. This model returned a combined LOS determination for an urban road based on vehicle, pedestrian, and bicyclist users, as well as information regarding safety and appearance of the road. The resulted framework for ALOS determination is primarily believed to provide assistance in decision-making regarding the infrastructure. This research further shows that use of current methods of level of service for decision-making purposes need to be modified in order to fully grasp all of the appropriate service measures. Calculating the asset LOS in this study provided a more general result of analysis based not

only on quantitative measure but qualitative measure as well.(Sharma, Al-Hussein, Safouhi, & Bouferguene, 2008)

Chapter III - Highway Capacity Manual

The *Highway Capacity Manual* (HCM) is a guide used to determine the quality of service and capacity for given roadway segments and facilities. The structural basis of the deficiency analysis method in this study is from the HCM 2010. The ideologies in the method are taken directly from the HCM. Before diving into the methods of the HCM 2010, here is a brief history of the HCM.

Evolution of HCM (History)

The *Highway Capacity Manual* was first published by the Transportation Research Board in 1950. There have been five major editions of the manual published in 1950, 1965, 1985, 2000, and 2010. Every edition brought forth something that the previous had not. It began as a way of determining the capacity and quality of service for roadways. “The first edition of the *Highway Capacity Manual* in 1950 did not contain any significant material on uninterrupted flow facilities. Freeways were not common, as the National System of Interstate and Defense Highways had not yet been initiated” (Roess, 2011). Therefore the 1965 edition included freeway chapters in addition to modification of previous methods in the first edition in 1950. The next major edition in 1985 introduced more focus into signalized analysis utilizing data collection from the early 80’s. The update to the 1985 manual came in 1994 with the addition of new chapters following the addition of Multilane Highway analysis previously. The HCM 2000 was focused more into the connection of micro simulation and

software application. The HCM 2010 added new analysis sections regarding multimodal facilities and roundabouts. Each edition provided something new based on research studies conducted. The most beneficial studies that brought about changes in the HCM were those of the National Cooperative Highway Research Program (NCHRP) project reports. The *Highway Capacity Manual* provides the best and most effective methods for calculating the appropriate results given specific conditions. As the nature of transportation grew, so has the ideology of the manual. The earlier models were not as detail specific as the one today and the future editions will be more complex in comparison to those today.

HCM 2000

The methodologies taken from the HCM 2000 for Multilane Highway, Two-Lane Highway, Freeway, and Urban Street chapters have been previously used for deficiency analysis programs for TDOT. The deficiency analysis program known as EVE was developed according to the HCM 2000 just as the program is being developed in this method is based from the HCM 2010.

HCM 2010

The HCM 2010 is the cornerstone of this entire study. For the purpose of this study specific chapters of the HCM 2010 were targeted. Those chapters, just like the ones from previous analysis, are: Freeway Segments, Multilane Highways, Two-Lane Highways, and Urban Street Segments. The algorithms in each section of the HCM 2010 provided the guideline for proper estimation of the quality of service amongst other values. Each chapter's algorithm can be found

below. Each of the figures below are from the HCM 2010 (National Research Council .
Transportation Research (2010).

This study focuses on segment analysis of the HCM 2010. Even though the manual contains sections for both segments and facilities, segment analysis was selected due to constraints of the data by TDOT through the TRIMS database. Also, the previous version of the deficiency analysis program was segment based.

For this study it is necessary to identify the changes affecting the analysis. Each of the chapters used for analysis has been update from the 2000 edition to the 2010 edition. The changes for each of the chapters are provided in the sections below.

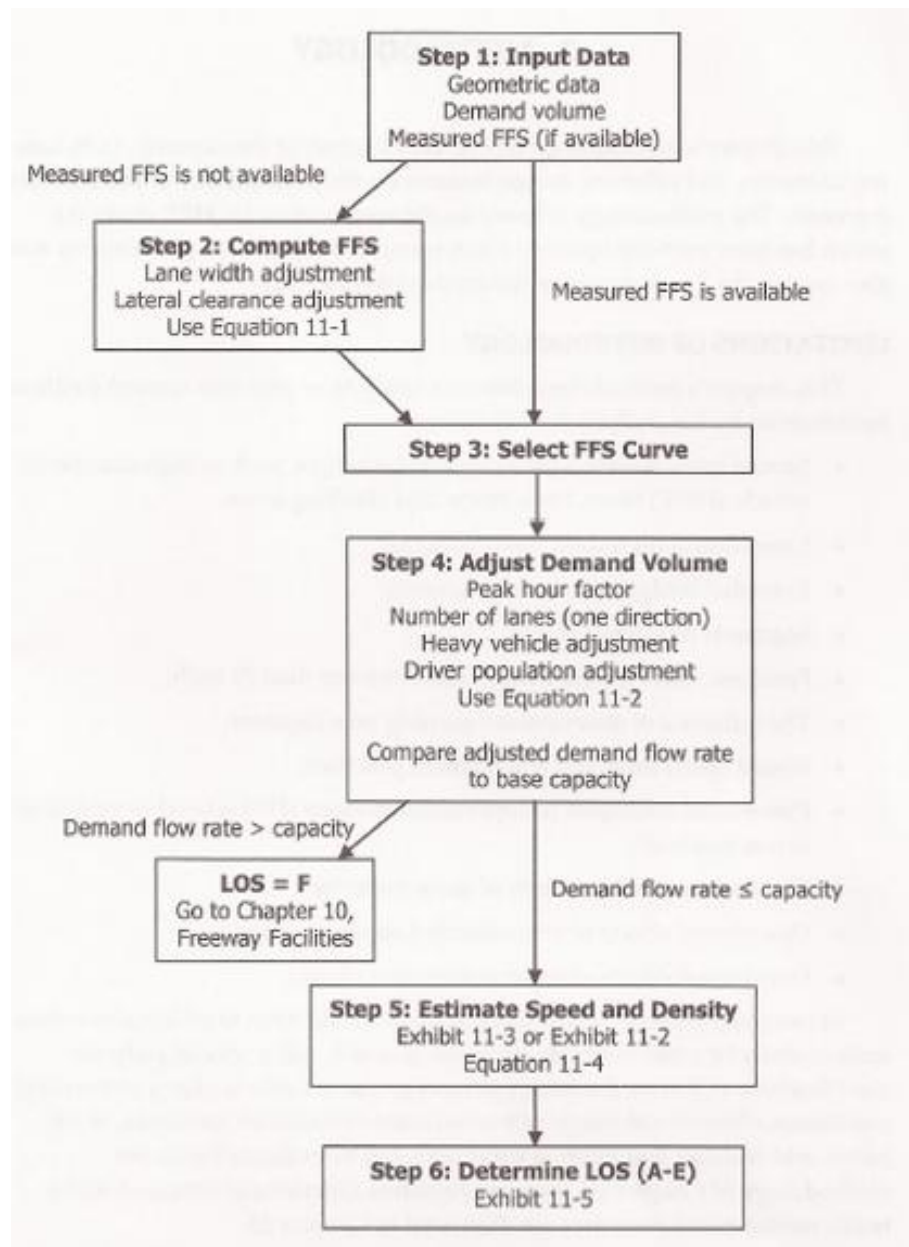


Figure 1: Basic Freeway Segment Algorithm (National Research Council . Transportation Research, 2010)

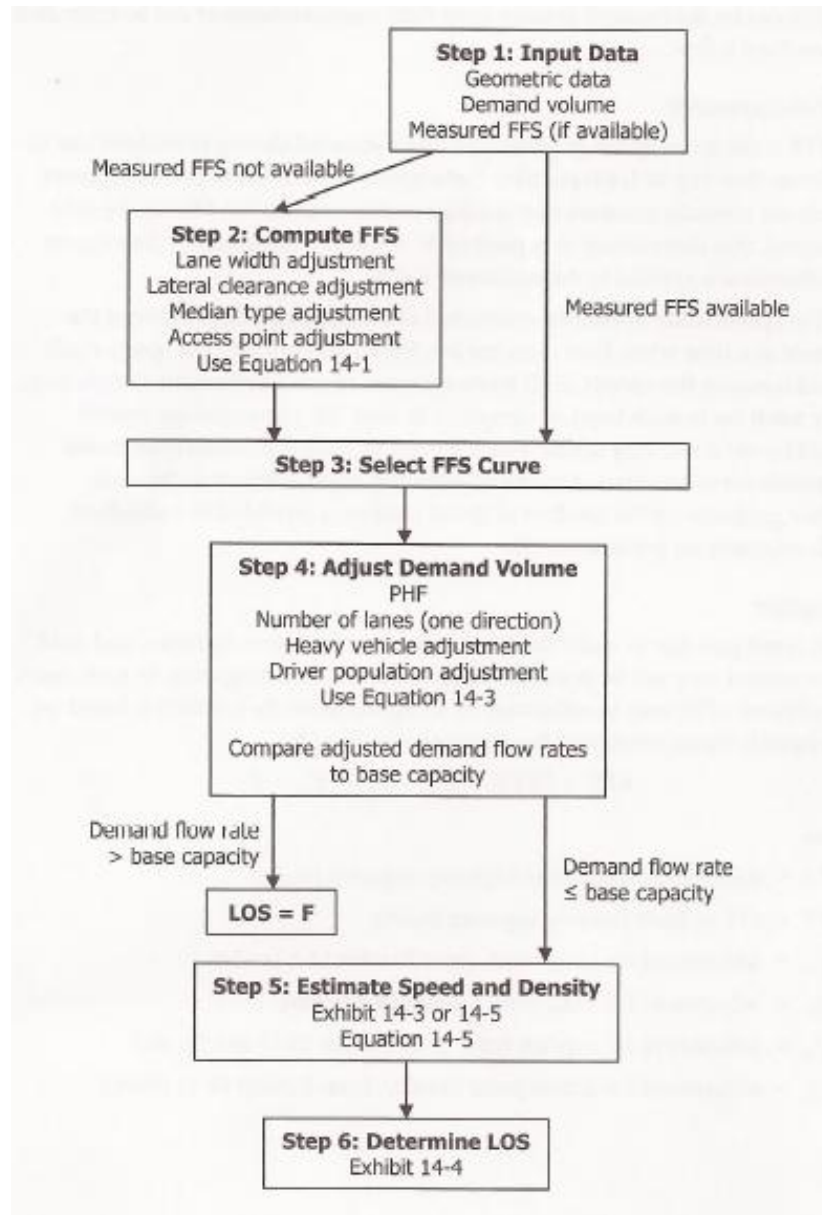


Figure 2: Multilane Highway Algorithm (National Research Council . Transportation Research, 2010)

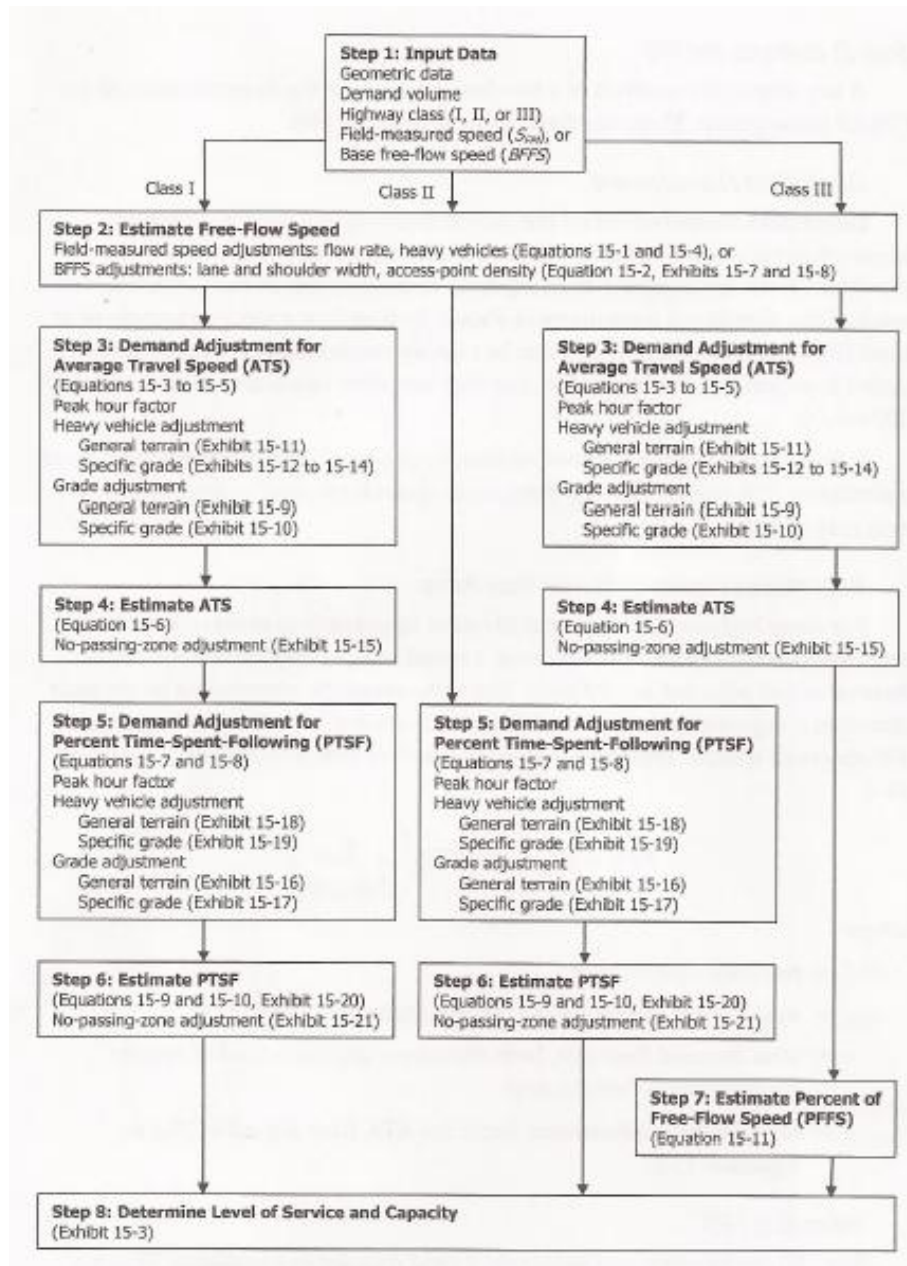


Figure 3: Two-Lane Highway Algorithm (National Research Council . Transportation Research, 2010)

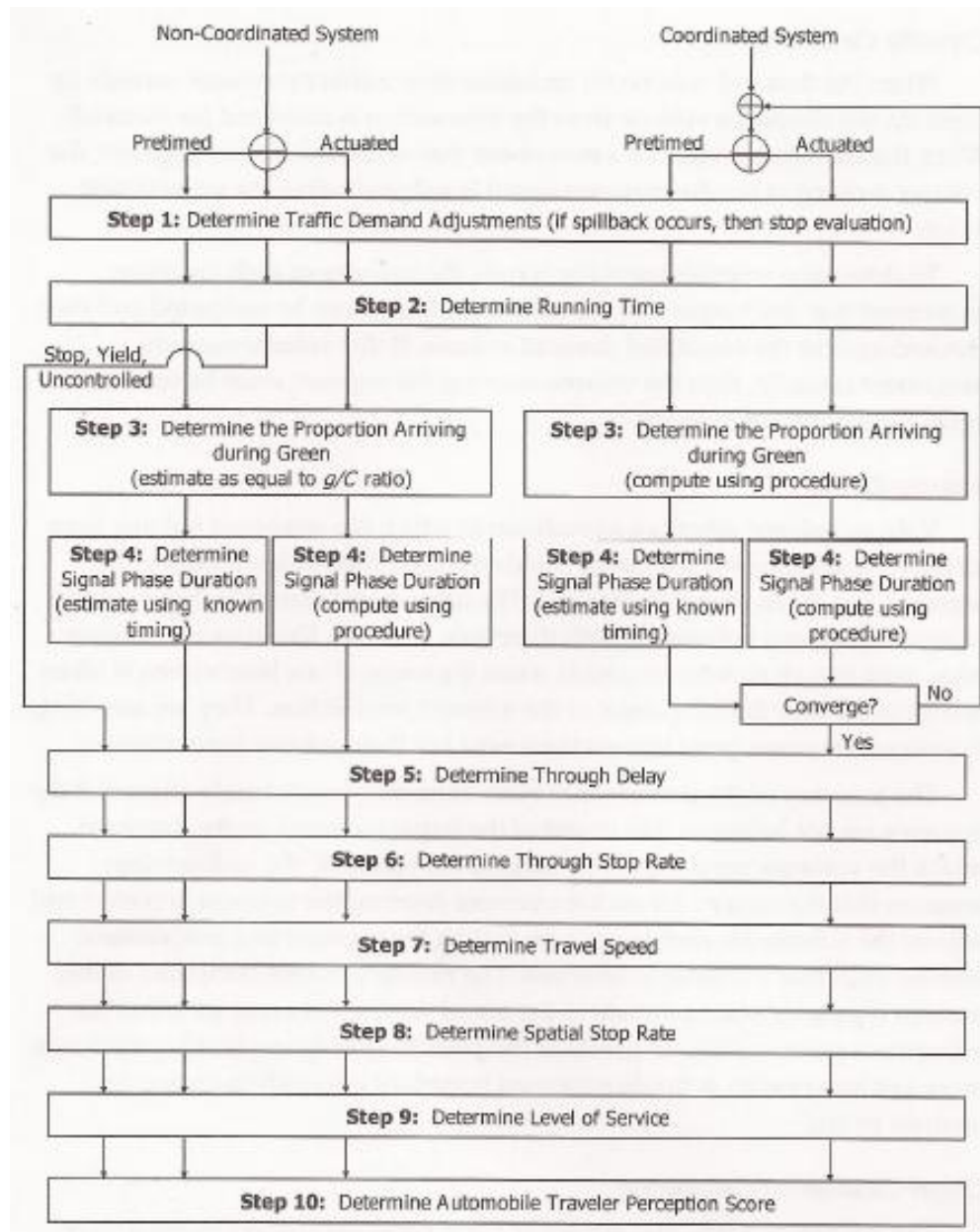
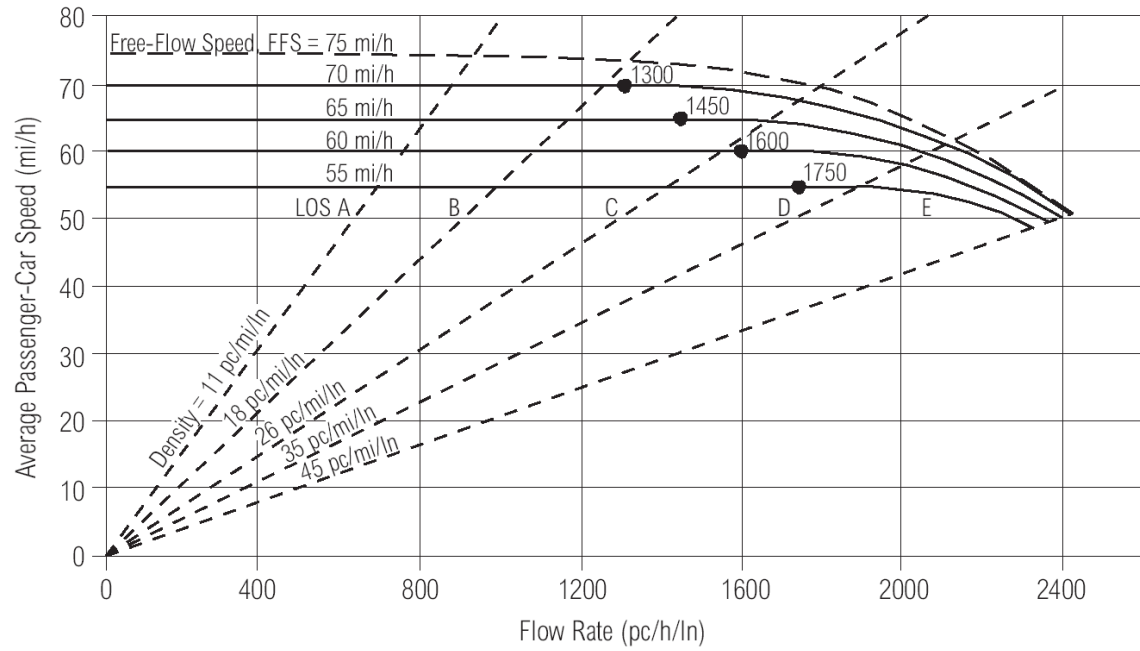


Figure 4: Urban Arterial Segment Algorithm (National Research Council . Transportation Research, 2010)

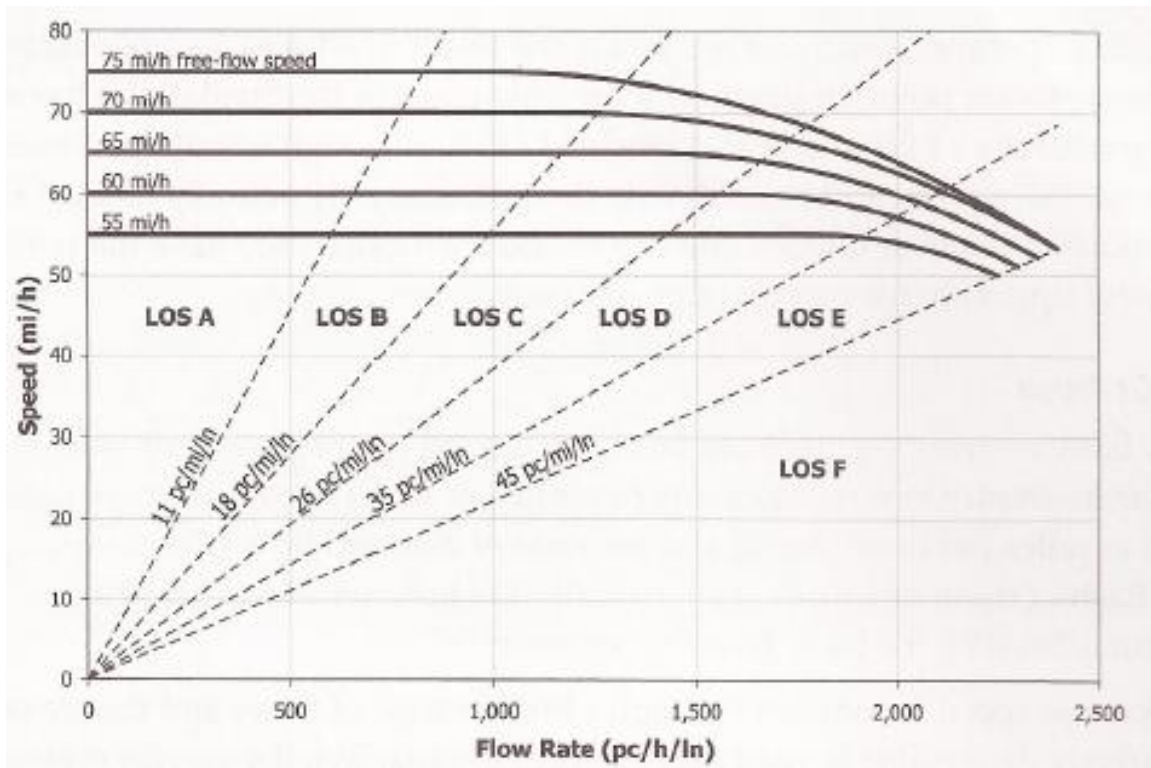
Freeway Segments

The methodology of the Freeway Segments remains similar to the HCM 2000. However, the major change is the modification of the speed-flow curve. Speed-flow curves have been used to depict the relationship of speed and flow rate for given free flow speeds (FFS). The curves are a critical part of level of service estimation. The speed-flow curve from the 2000 edition of the HCM is shown in Figure 5: HCM 2000 Basic Freeway Segments Speed-Flow Curve (National Research Council . Transportation Research, 2000). Figure 6: HCM 2010 Basic Freeway Segments Speed-Flow Curve (National Research Council . Transportation Research, 2010) shows the speed-flow curve provided in the HCM 2010. The differences in these curves are the breakpoints of each free flow speed curve. The 2000 edition shows its breakpoints on its curve; however the 2010 edition does not. The values of the breakpoints for the 2010 edition are 1000, 1200, 1400, 1600, and 1800 correlating to order of the curves starting with the 75 mph curve. The breakpoint changes affect the operation speed (S) for segments with free flow speeds within the range of 55-75 mph. the average FFS on a freeway segment falls within this range, therefore this methodology change is small.

Other changes from the HCM 2000 are found in the method for calculation of the Free Flow Speed (FFS). This change adds in the factor Total Ramp Density (TRD) which is similar to Interchange Density (ID) found in the 2000 edition. Whereas ID was used as a reduction factor, the new calculation uses TRD to directly estimate FFS. The previous FFS calculation also estimated the Base Free Flow Speed, but now in the 2010 edition an assumed value of 75.4 mph is used instead of a calculation based on the posted speed limit. The FFS calculation in the 2010 edition also removed the reduction due to lane width.



**Figure 5: HCM 2000 Basic Freeway Segments Speed-Flow Curve (National Research Council .
Transportation Research, 2000)**



**Figure 6: HCM 2010 Basic Freeway Segments Speed-Flow Curve (National Research Council .
Transportation Research, 2010)**

Multilane Highway Segments

There were no major changes in the methodology of multilane highways from 2000 to 2010. However, there was an addition Bicycle analysis for both Multilane and Two-Lane Highways. The quality of service for bicycle analysis is based on a travel-perception model. Meaning the level of service depends on the bicyclist perception of the segment. The level of service is determined by the calculation of a Bicycle Level of Service (BLOS) score. This score requires the input of traffic and geometric data for automobiles on the Multilane and Two-Lane Highway segments.

Urban Arterial Segments

The change to the Urban Street Segment analysis was the removal of Arterial Classification. The 2000 edition categorized for different types of arterials: High-Speed, Suburban, Urban, and Intermediate. All four of these arterial types are now analyzed using the same method.

Two-Lane Highway Segments

From 2000 to 2010 there was a significant change of the methodology of the Two-Lane Highway Analysis. The method in the 2010 HCM focuses on one directional analysis. This means two-lane roadways are now to be analyzed based on one direction of traffic and include inputs related to the opposing direction of traffic.

In addition to the methodology change, there was also an addition made to the Highway Classifications. The new method adopts a Class III Highway which is for developed areas that may travel through small towns. These highways also have a higher number of access points on the segment as compared to Class I and Class II highways.

Classification is a strong factor in the necessary level of service analysis. In order to understand how important a change in the highway class can be in the LOS results, a comparison was conducted. The comparison is of the data collected from two-lane highway segments in Washington County. Each segment is analyzed as if it is considered a Class I, Class II, or Class III highway. Since there are so many segments used in the analysis, samples of the results are shown in Figure 7: Sample of Two-Lane Highway Classification Comparison. Table 1: Two-Lane Highway Comparison Results shows the comparison of results by percentages. From the results, it shows that Class III provides the best results for LOS Classes A, B, and C. Class I

has the most results with LOS Classes D, E, and F. From this, consideration is made that Class III is the best for LOS results, next is Class II, and Class I is the worst in comparison to the other classes.

BLM	ELM	Road Name	AADT	Spd Limit	LOS if Class I	LOS if Class II	LOS if Class III
0.716	1.35	TELFORD RD.	2100	35	E	C	B
0	1.07	GLENDALE RD.	340	35	E	A	A
1.07	4.55	GLENDALE RD.	890	35	E	B	A
0	2.48	BUMPUS COVE RD.	420	20	E	A	B
0.96	0.99	OLD SR-34	1040	45	D	B	A
3	3.05	OLD SR-34	1040	45	C	B	A
4	4.138	OLD SR-34	1040	30	E	B	B
4.138	4.59	OLD SR-34	530	30	E	B	A
4.59	4.72	S. CLEAR SPRINGS RD.	530	30	E	B	A
0	1.538	BOWMANTOWN RD.	530	35	E	B	A
1.538	3.64	BOWMANTOWN RD.	540	35	E	B	A
1.94	3.34	ARNOLD RD.	920	40	E	C	A
8.511	8.559	DRY CREEK RD.	780	40	E	B	A
8.559	8.59	LONE OAK RD.	780	40	E	B	A
1.76	4.77	CONKLIN RD.	500	45	D	B	A
4.77	6.57	CONKLIN RD.	3420	45	D	D	B
0.19	0.36	W. MAIN ST.	6600	20	E	D	E
0.36	0.45	E. MAIN ST.	6600	20	E	D	E
0.49	0.548	FOX AVE.	3060	20	E	D	D
0.548	0.62	WOODROW AVE.	3060	20	E	D	E
1.047	1.126	S. CHEROKEE ST.	3060	20	E	D	E
1.438	1.462	OLD EMBREEVILLE RD.	3060	20	E	D	E
2.84	4	OLD EMBREEVILLE RD.	3060	35	E	D	C
4	5.68	BILL JONES RD.	500	30	E	B	A
4.14	4.19	OLD STAGECOACH RD.	460	40	E	B	A
8.347	8.38	HAIRETOWN RD.	1820	40	E	C	B
9.049	11.363	BOONES STATION RD.	1820	35	E	C	B
11.8	11.839	BOONES STATION RD.	90	35	E	A	A
11.839	12.08	LAKE PARK DR.	90	35	E	A	A
0.07	0.139	E. OAKLAND AVE.	5380	30	E	D	C
2.045	2.2	E. OAKLAND AVE.	5380	30	E	D	D
0	1.68	DUCKTOWN RD.	160	35	E	A	A
0	1.08	MAPLE SWAMP RD.	380	30	E	A	A
1.529	1.791	LIBERTY CHURCH DR.	380	30	E	A	A
1.791	2.83	HONEYSUCKLE LN.	380	30	E	A	A

Figure 7: Sample of Two-Lane Highway Classification Comparison

Table 1: Two-Lane Highway Comparison Results

Highway Class Type	Percent “Good” (Class A, B, C) (%)	Percent “Bad” (Class D, E, F) (%)
Class I	0.7	99.3
Class II	43.1	56.9
Class III	73.2	26.8

Chapter IV - Methodologies for Deficiency Analysis

Deficiency Analysis

Deficiency Analysis is a method used to determine weaknesses in a system. This is useful when it comes to analyzing roadway weaknesses. Many state transportation organizations run deficiency analyses in order to identify where appropriate funding should be going. This analysis can be run for current roadway conditions as well as future roadway conditions. With this in mind, the model to be presented is beneficial to the State of Tennessee's deficiency analysis of its roadways. Without the necessary change to the current deficiency analysis program, current methods will remain inconsistent with current state of the practice.

TDOT Deficiency Analysis

The Tennessee Department of Transportation (TDOT) maintains the state's current deficiency analysis program. With the assistance of TDOT, this study will provide reasonable and current methodologies for updating the deficiency analysis program.

TRIMS

In order to run an accurate deficiency analysis of Tennessee roadways, TDOT has provided data from its Tennessee Roadway Information Management System (TRIMS) database. The TRIMS database contains information regarding every roadway in the state. Much of the input data needed to run a proper analysis can be located within this database. Specific queries were run in

TRIMS in order to locate the appropriate data for the roadways. Data in TRIMS is separated by a combination of City or County and Route. From here one must select from data sets already presented in TRIMS. The information types are shown below in **Error! Reference source not found..**

The screenshot displays the 'Query' tab of the TRIMS application interface. The 'Simple' query type is selected. The 'Query Type' dropdown is set to 'Select Route By Name'. The 'Info Type' dropdown is expanded, showing a list of information types: Road Segment, Maintenance Feature, Maintenance Inventory, PPRM Projects, Road Segment, Roadway Description, Roadway Geometrics, Route Feature, Structure, and Traffic. The 'Traffic' option is highlighted. Below the dropdown, there are input fields for 'Special Case', 'County Seq', 'BLM', 'ELM', and 'Road Name' (with a 'Select a Road Name...' dropdown). A 'Locate' button is at the bottom. The interface also includes a sidebar with navigation icons and a top bar with tabs for 'Query', 'Route Info', 'Options', and 'Map Layers'.

Figure 8: TRIMS Information Type Selection

Of these information types, the most beneficial information for the analysis was located within the following sets:

- Roadway Segment
- Roadway Description
- Roadway Geometrics
- Route Feature
- Traffic

Any combination of these information types could be run in an advanced query. An example of the query ran for Washington County Road Segments is shown in Figure 9. From here the result of the query are tabulated.

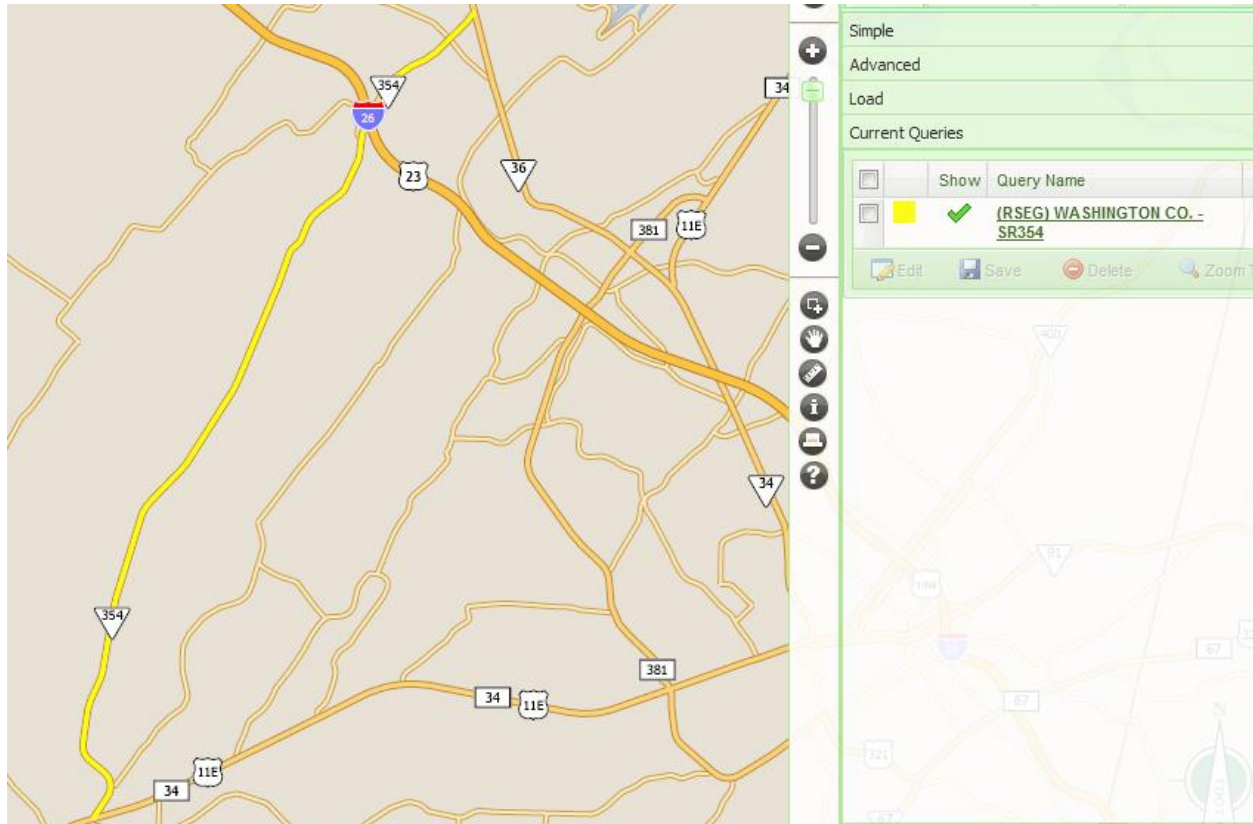


Figure 9: TRIMS - Washington County Road Segment Query

Using the advanced query, each information type above was selected and filtered to provide data for Washington County only. The data was then exported into excel and trimmed in order to establish all of the necessary parameters needed for the deficiency analysis.

EVE

Evaluation of Roadway Efficiency (EVE) is the program TDOT uses to run its Deficiency Analysis. This program was developed by Dr. Lee Han and Dr. Fred Wegmann of The University of Tennessee at Knoxville and adopted for use by TDOT. The current version of EVE is based on the methodologies of the 2000 *Highway Capacity Manual*. Since the version of the HCM used in EVE is now outdated, this study will provide a current version of its basic methodologies based now on the HCM 2010.

EVE takes input data from the TRIMS database and runs its deficiency analysis as shown in Figure 10. EVE in addition to providing current roadway analysis would also provide forecasted analysis for future conditions. Keeping this analysis as current as possible is the reason for this study.

Deficiency Summary and Recommendations

Reporting Options: ☐ Only list segments with LOS < All/Rur Urb Analysis Area: County: Sullivan Route: 10026 Refresh Data Scenario Builder

Segment Summary: Route 10026

Location: KINGSPORT Terrain: Mountainous Current Lanes: 4 Speed Limit: 65 Lane width: ☐ View PhotoLog

Land Use: Rural Road Type: Freeway Lane Width: 12 Year 1st Def: 2033 Speed: ☐ Map Segment

FClass: Interstate Features: 6.33-SR-347 ROCK SPRINGS RD. / CENTER OF OVERHEAD Expand List

Tot Crashes: 56 CrashRate: 0.64 CntRate: 0.01 ☒ High Cr HCr Ratio: 112.00 Worst Speed: 0 85% Speed: 0 Bridges

Computer Defined Capacity: 2,350 /Hour

County	SpclCse	CS	Beg Mile	End Mile	2012	2017	2022	2027	2032	2036
Sullivan	None	1	0.000	3.500	B	B	B	C	C	C
Sullivan	None	1	3.500	4.030	A	A	B	B	B	B
Sullivan	None	1	4.030	6.330	C	C	C	D	D	D
Sullivan	None	1	6.330	8.490	B	C	C	C	C	D

Comment: Apply

What If Effects on V/C Ratio and Recommendations

Terrain: Mountainous Current Lanes: 4 Width: 12 Speed Limit: 65

Peak %: 9 Dir. Dist.: 50 SU Truck %: 2 MU Truck %: 5

Year 1st Def: 2033 AADT: 36790 Capacity: 2,350 /Hour

Year	AADT	V/C	LOS	Gr. Fact.
2027	48528	.638	C	2.1%
2032	52440	.689	C	2.1%
2033	53223	.699	D	EST

Recommendation: Cost: \$

Comment: View Effects Apply Cancel

Improvement

Segment in Project:

Recommended: Cost: \$ 0.00

User-Defined: Cost: \$

Beg. Mile: End Mile:

Imp. Cur. LOS: Imp. Fut. LOS:

Comment: Apply to Project

Figure 10: Screenshot of EVE Summary

Chapter V - Procedures

Now with an understanding of deficiency analysis and the TRIMS system, the following procedure is used to estimate deficiency analysis based from the algorithms in the HCM 2010. This procedure has been developed to provide methodologies for future deficiency analysis programs, such as EVE. The procedure consists of Freeway Segments, Multilane Highways, Two-Lane Highways, and Urban Arterial Segments sections. As previously stated, the foundation of this procedure comes from the Capacity Analysis Procedure for Eve 4.0, which can be found in the Appendix.

Freeway Segment Analysis

The data inputs have collected are necessary for proper analysis of the freeway segments. Specific inputs to be taken from the TRIMS database for Freeway Segments are: Annual Average Daily Traffic (AADT), K Factor, D Factor, Peak Hour Factor (PHF), Lane Width, Lateral Clearance, Rural/Urban Classification, Terrain, Truck Percentage, and Posted Speed. The respective output results of the freeway segments are: Operational Speed (S), Capacity (c), v/c ratio, Density (D), and Level of Service (LOS). Each of these inputs and outputs are necessary to determine the density of the segments, which is needed to estimate the appropriate LOS.

This procedure is based on Chapter 11: Basic Freeway Segments of the HCM 2010. The heart of the methodology is the speed-flow relationship depicted in the figure below. The speed-flow curve depicted is used to determine the appropriate values of operation speed. Upon obtaining the flow rate and the operation speed of the vehicles on the segment, the LOS can be derived.

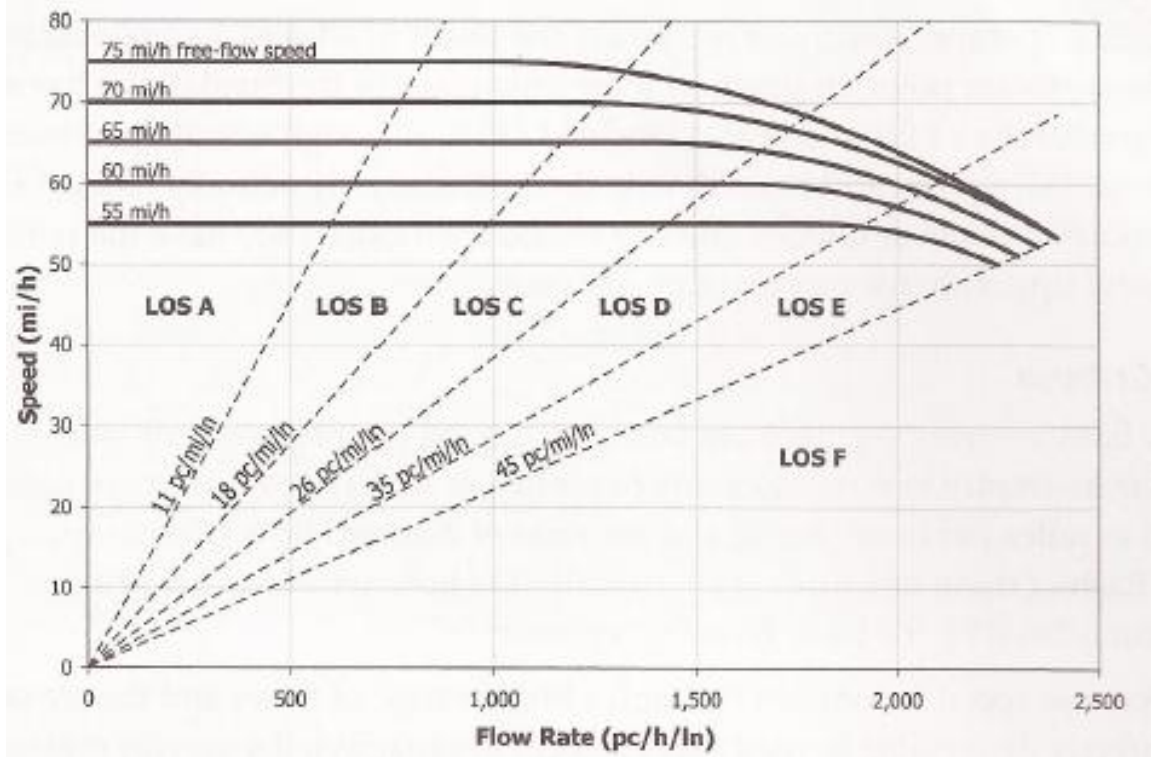


Figure 11: Freeway Segment Speed-Flow Curve (National Research Council . Transportation Research, 2010)

The first step in estimating the LOS of a segment, is the calculation of the flow rate (or, more appropriately, demand if the flow rate turns out to be higher than the capacity) for the segment.

Equation 1: Flow Rate

$$V_P = \frac{AADT \times K \text{ Factor} \times D \text{ Factor}}{PHF \times N \times f_{HV} \times f_P}$$

Equation 1: Flow Rate shows the flow rate calculation for freeway segments. The values for AADT, K and D factors are found in TRIMS data, as well as the PHF. However if the PHF is not

available in TRIMS, the PHF is 0.94 as a default. The value N is the number of lanes in one direction of traffic, and f_p is the driver population adjustment. This value is usually 1.0; however for drivers who are not mostly commuters, the value is 0.85. The reduction due to heavy vehicles (f_{HV}) can be calculated using Equation 2: Reduction due to Heavy Vehicles.

Equation 2: Reduction due to Heavy Vehicles

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

The inputs used in Equation 2: Reduction due to Heavy Vehicles are the percentage of multi- and single unit trucks from TRIMS (P_T and P_R). The values for E_T and E_R are determined based on the type of terrain for the segment. These values can be determined from the following table.

Table 2: E_T and E_R Values based on Terrain

Vehicle	Level	Rolling	Mountainous
E_T	1.5	2.5	4.5
E_R	1.2	2.0	4.0

In order to get the v/c ratio, first we must determine Capacity (c) of the segment. Capacity is a function of FFS, free-flow speed, according to the method in the HCM 2010. FFS can be

measured in the field. It is suggested that if FFS is measured in the field, then it is best to use the measured FFS. However if FFS is not directly measured in the field, which is often the case, use the new model suggested by HCM 2010 as shown in Equation 3: FFS Calculation.

Equation 3: FFS Calculation

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22TRD^{0.84}$$

The values in the FFS calculation are reductions in lane width (f_{LW}), lateral clearance, and Total Ramp Density. The values for lane width reduction are given in Table 3. The values for right-side lateral clearance are given in Table 4. Total ramp Density is described in the next section.

Table 3: Reduction due to Lane Width (National Research Council . Transportation Research, 2010)

Average Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11-12	1.9
≥10-11	6.6

**Table 4: Reduction due to Right-Side Lateral Clearance(National Research Council .
Transportation Research, 2010)**

Right-Side Lateral Clearance (ft)	<u>Lanes in One Direction</u>			
	2	3	4	≥5
≥6	0.0	0.0	0.0	0.0
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

Total Ramp Density (TRD) has a large impact on calculating FFS. It is defined as the number of on-ramps, off-ramps, and interchanges in one direction of travel over a certain length. The Total Ramp Density is based on a length of 6 miles. The TRD is determined in one of two methods. For segments longer than 6 miles, the TRD is the number of ramps within the length of the segment divided by the length of the segment (LS) in miles.

Equation 4: Total Ramp Density for Longer Segments

$$\text{TRD} = \frac{\# \text{ on} + \# \text{ off ramps} + \# \text{ interchange connections}}{\text{segment length}}$$

For segments shorter than 6 miles, the length of study areas upstream and downstream changes. Now, the number of ramps are those within upstream and downstream of $((6-LS)/2)$,

with LS being the segment length. Once the number of ramps in this section is determined, use the equation below.

Equation 5: Total Ramp Density for Shorter Segments

$$\text{TRD} = \frac{\# \text{ on} + \# \text{ off ramps} + \# \text{ interchange connections}}{6 \text{ miles}}$$

Once the FFS is obtained, the capacity per lane can be calculated. For segments with a FFS greater than 70 mph, use the value of 2400 vehicles per hour per lane. For segments with a FFS less than 70 mph, use Equation 6: Capacity Equation for FFS less than 70 mph.

Equation 6: Capacity Equation for FFS less than 70 mph

$$\text{Capacity} = 2400 - 10 \times (70 - \text{FFS})$$

Once the capacity is calculated, the v/c ratio is simply equal to v_P/c . Note that both flow rate (v_P) and Capacity (c) are on a per lane basis. It is necessary to make sure that their units match. For segments with the v/c ratio larger than 1.0, a LOS of F is assigned to the section and no further analysis is needed. If this happens, the operation speed (S) is then assumed to be 5 mph for the segment.

Operational Speed (S) is needed to calculate the density, which is a determinant of the LOS. The breakpoints of the speed-flow curve have changed in the HCM 2010. With the new breakpoints

of the curves, the ranges of flow rates (v_p) as well as the FFS are used to determine the appropriate Operational Speed (S). For segments with flow rates less than 1000 vphpl the operation speed is the same as the free flow speed. For segments with flow rates between 1000-2400 vphpl and have a FFS equal to 75 mph, use Equation 7: Freeway Operation Speed Calculation (FFS = 75mph). For segments with flow rates between 1200-2400 vphpl and have a FFS between 70-75 mph, use Equation 8: Freeway Operation Speed for FFS > 70 mph. For segments with flow rates between 1400-2350 vphpl and have a FFS between 65-70 mph, use Equation 9: Freeway Operation Speed for FFS > 65 mph. For segments with flow rates between 1600-2300 vphpl and have a FFS between 60-65 mph, use Equation 10: Freeway Operation Speed for FFS > 60 mph. For segments with flow rates between 1800-2250 vphpl and have a FFS between 55-60 mph, use Equation 11: Freeway Operation Speed for FFS > 55 mph.

Equation 7: Freeway Operation Speed Calculation (FFS = 75mph)

$$S = \text{FFS} - 0.00001107 (v_p - 1000)^2$$

Equation 8: Freeway Operation Speed for FFS > 70 mph

$$S = \text{FFS} - 0.00001160 (v_p - 1200)^2$$

Equation 9: Freeway Operation Speed for FFS > 65 mph

$$S = \text{FFS} - 0.00001418 (v_p - 1400)^2$$

Equation 10: Freeway Operation Speed for FFS > 60 mph

$$S = FFS - 0.00001816 (v_P - 1600)^2$$

Equation 11: Freeway Operation Speed for FFS > 55 mph

$$S = FFS - 0.00002469 (v_P - 1800)^2$$

Now with the flow rate, capacity, and operation speed (S) calculated the LOS can be determined. The density based on the lesser value of either the flow rate or the capacity. The density calculation is shown in Equation 12: Freeway and Multilane Highway Segment Density. To complete this analysis use Table 5 to determine the LOS. Once calculated, the analysis is complete.

Equation 12: Freeway and Multilane Highway Segment Density

$$\text{Density} = \text{Min} (v_P, c) / S$$

Table 5: Freeway Segment LOS Criteria

LOS	A	B	C	D	E	F
Density	0 - 11	>11 - 18	>18 - 26	>26 - 35	>35 - 45	>45

Multilane Highway Analysis

The analysis for Multilane Highways is similar to Freeway Segment analysis. The same input data is used from TRIMS just as the Freeway segment analysis. The outputs of the Multilane Highway analysis are also the same as the Freeway segment analysis. This Multilane Highway procedure is based on Chapter 14 of the HCM 2010. The speed-flow relationship shown below is the basis of this chapter methodology. After determining the Operational Speed of passenger cars and the flow rate, LOS can be derived.

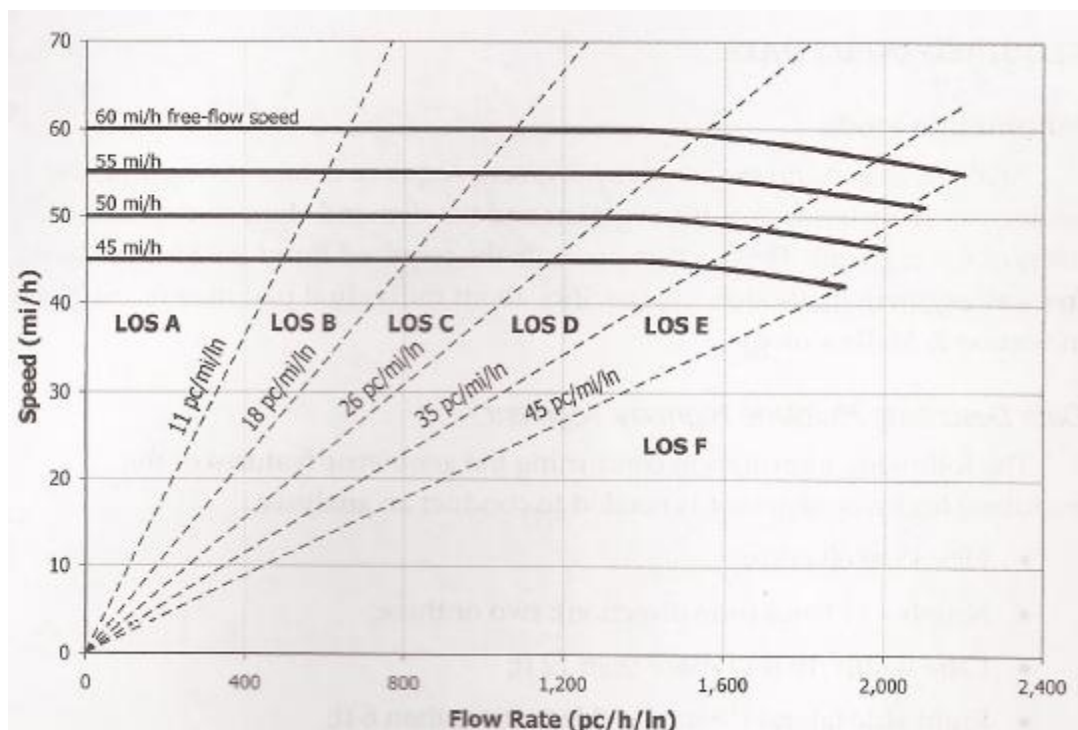


Figure 12: Multilane Highway Speed-Flow Curve (National Research Council .
Transportation Research, 2010)

Similar to the analysis procedure for freeway segments, the flow rate (or, more appropriately, demand if the flow rate turns out to be higher than the capacity) needs to be calculated first. This calculation is shown in Equation 1: Flow Rate. The difference between this calculation and the one for Freeway segments is the default value of for PHF is now 0.88. This is only to be used if TRIMS does not provide the PHF value. The remaining inputs are the same. This includes the calculation for f_{HV} and its respective inputs found in Table 6.

Table 6: E_T and E_R Values based on Terrain (National Research Council . Transportation Research, 2010)

Vehicle	PCE by Type of Terrain		
	Level	Rolling	Mountainous
Trucks and buses, E_T	1.5	2.5	4.5
RVs, E_R	1.2	2.0	4.0

Capacity, c , is to be found next in order to, eventually, calculate the v/c ratio. Capacity is a function of FFS, free-flow speed, according to HCM 2010. If FFS is already given from field measurements use it; however, if FFS needs to be estimated use Equation 13: *Multilane Highway FFS*. The base free flow speed (BFFS) is calculated depending on the posted speed limit. For Speed limits greater than 50 mph, the BFFS is the posted speed plus 5 mph. Otherwise the BFFS is the posted speed limit plus 7 mph. The reductions due to lane width, total lateral clearance, medians, and access point density are found in the following tables.

Equation 13: Multilane Highway FFS

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

Table 7: Reduction due to Lane Width (National Research Council . Transportation Research, 2010)

Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥ 12	0.0
$\geq 11-12$	1.9
$\geq 10-11$	6.6

$$\text{Total Lateral Clearance (TLC)} = L_{CR} + L_{CL}$$

Table 8: Reduction due to Total Lateral Clearance (National Research Council . Transportation Research, 2010)

Four-Lane Highways		Six-Lane Highways	
TLC (ft)	Reduction in FFS (mi/h)	TLC (ft)	Reduction in FFS (mi/h)
12	0.0	12	0.0
10	0.4	10	0.4
8	0.9	8	0.9
6	1.3	6	1.3
4	1.8	4	1.7
2	3.6	2	2.8
0	5.4	0	3.9

Table 9: Reduction due to Presence of Median (National Research Council . Transportation Research, 2010)

Median Type	Reduction in FFS, f_M (mi/h)
Undivided	1.6
TWLTL	0.0
Divided	0.0

The method for calculating access point density still remains the same from EVE 4.0. Use the following equation to determine the access points per mile if the information is provided in TRIMS. Continually, 20 access points per mile has to be used as a default value when data is unavailable. Once the Access Points per mile have been determined, use *Table 10*.

Equation 14: Access-Point Density Calculation

$$\text{Access Points/Mile} = \frac{\# \text{ of Access Points}}{\text{Length of the Segment}}$$

Table 10: Reduction due to Access-Point Density (National Research Council . Transportation Research, 2010)

Access-Point Density (access points/mi)	Reduction in FFS, f_A (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
≥40	10.0

Once the FFS is determined, the capacity, c , can be calculated. For segments with FFS greater than 60 mph, the capacity is 2200 vehicles per hour per lane. For segments with a FFS less than 60 mph, use Equation 15: Multilane Highway Capacity less than 60 mph.

Equation 15: Multilane Highway Capacity less than 60 mph

$$\text{Capacity} = 2200 - 20 \times (60 - \text{FFS})$$

Once capacity is determined, the v/c ratio can be calculated. This ratio is simply equal to v_P/c .

The flow rate and capacity are on an on per lane basis. At this point if the v/c ratio is larger than 1.0, we can readily assign a LOS of F to the section and move on and save calculation time.

In addition to this, the operation speed for a segment with a v/c ratio greater than 1.0 is 5 mph.

Next operation speed, S, is calculated for FFS between 45-60 mph. Each of the operation speed calculations are based on a flow greater than 1400 vphpl. For segments with a FFS > 55 mph use Equation 16: Multilane Operation Speed for FFS > 55 mph. For segments with a FFS between 50-55 mph use Equation 17: Multilane Operation Speed for FFS between 50-55 mph. For segments with a FFS between 45-50 mph use Equation 18: Multilane Operation Speed for FFS between 45-50 mph. For segments with a FFS equal to 45 mph, use Equation 19: Multilane Operation Speed for FFS = 45 mph. For segments with a FFS less than 45 mph, the operation speed is equal to the FFS. Upon determining the operation speed, the density can be calculated using Equation 12: Freeway and Multilane Highway Segment Density. The density is then used to determine the LOS for the segment from Table 11.

Equation 16: Multilane Operation Speed for FFS > 55 mph

$$S = \text{FFS} - \left[5.00 \times \left(\frac{vp - 1400}{800} \right)^{1.31} \right]$$

Equation 17: Multilane Operation Speed for FFS between 50-55 mph

$$S = FFS - [3.78 \times (\frac{vp-1400}{700})^{1.31}]$$

Equation 18: Multilane Operation Speed for FFS between 45-50 mph

$$S = FFS - [3.49 \times (\frac{vp-1400}{600})^{1.31}]$$

Equation 19: Multilane Operation Speed for FFS = 45 mph

$$S = FFS - [2.78 \times (\frac{vp-1400}{500})^{1.31}]$$

Table 11: LOS Criteria for Multilane Highways

LOS	A	B	C	D	E	F
Density	0 - 11	>11 - 18	>18 - 26	>26 - 35	>35 - D _{FFS}	>D _{FFS}

$$D_{FFS} = 0.00133333 FFS^3 - 0.2 FFS^2 + 9.566667 FFS - 102$$

Bicycle Analysis

An addition to Level of Service Analysis in the HCM 2010 is the analysis of the Bicycle Mode for Multilane and Two-Lane highways. The result of the LOS for bicycles is based on a Bicycle Level of Service score. This score is based on roadway geometrics and travel for automobiles on the segment. The score ranges from 0.5 to 6.5. Following the procedure below establishes the Level of Service for the segment.

In order to determine the bicycle score for the segment, the flow rate must first be determined. Using Equation 20: Flow Rate based on Outside Lane width, the flow rate is based on data taken from TRIMS.

Equation 20: Flow Rate based on Outside Lane width

$$V_{OL} = \frac{AADT \times K \times D}{PHF \times N}$$

Once the flow rate is determined, the effective width is estimated. The effective width depends on the width of the outside lane and shoulder width. The default value of the shoulder width is 6 feet, if unavailable in TRIMS. The effective width (W_e) is based on paved shoulder width (W_s) and directional hourly volume, as shown in the equations below. For segments with a flow greater than 160 vph, the effective width as a function of traffic volume is estimated in Equation 21: Width as a function of Traffic Volume for $V > 160$ vph. For all other segments, use Equation 22: Width as a function of Traffic Volume to determine the effective width as a function of traffic volume.

Equation 21: Width as a function of Traffic Volume for $V > 160$ vph

$$W_v = W_{ol} + W_s$$

Equation 22: Width as a function of Traffic Volume

$$W_v = (W_{ol} + W_s) \times (2 - 0.005V)$$

Next, the average effective width can be estimated. The average effective width is based on the percentage of highway parking, shoulder width (SW) and effective as a function of traffic volume. For segments with shoulder width greater than 8 feet, use Equation 23: Average effective width (SW > 8 feet). For segments with a SW between 4-8 feet, use Equation 24: Average effective width (4 ft. < SW < 8 ft.). For segments with a SW less than 4 feet, use Equation 25: Average Effective Width (SW < 4 ft.). Once the average effective speed is calculated for the segment, the Effective Speed Factor needs to be determined. The effective speed factor (Equation 26: Effective Speed Factor) is based solely on the posted speed limit for the segment. Upon calculating these values, the LOS can be determined.

Equation 23: Average effective width (SW > 8 feet)

$$W_e = W_v + W_s - (\%OHP \times 10ft)$$

Equation 24: Average effective width (4 ft. < SW < 8 ft.)

$$W_e = W_v + W_s - 2 \times (\%OHP (2ft + W_s))$$

Equation 25: Average Effective Width (SW < 4 ft.)

$$W_e = W_v + (\%OHP (2 ft. + W_s))$$

Equation 26: Effective Speed Factor

$$S_t = 1.1199 \ln (S_p - 20) + 0.8103$$

The Bicycle Level of Service is based upon a traveler-perception model. In order to determine the appropriate LOS for bicycles, the inputs are combined into a traveler score. The equation is shown in Equation 27: Bicycle Level of Service Score. This score is based on previous determined values, as well as the percentage of Heavy Vehicles, and the FHWA's 5-point surface rating. The percent of heavy vehicles on the roadway is set at a maximum of 50% for a flow less than 200 vph. The default value for the surface rating is set as 4.

Equation 27: Bicycle Level of Service Score

$$BLOS = 0.507 \times \ln (v_{OL}) + 0.1999S_t (1 + 10.38HV)^2 + 7.066 (1/P)^2 - 0.005(W_e)^2 + 0.057$$

Once the BLOS score is obtained use Table 12 to determine the appropriate LOS. This procedure is used for both Multilane Highways and Two-Lane Highways.

Table 12: Bicycle Level of Service Score Criteria (National Research Council . Transportation Research, 2010)

LOS	BLOS Score
A	≤ 1.5
B	$>1.5-2.5$
C	$>2.5-3.5$
D	$>3.5-4.5$
E	$>4.5-5.5$
F	>5.5

Urban Arterial Segment Analysis

This procedure is for the analysis of urban and suburban streets with traffic signal spacing of 2.0 miles or less. The analysis should be conducted for sections 2 miles or longer, and at least one mile in the urban area. The section we are referring to is defined as a portion of a roadway of which the AADT and/or roadway geometry do not change significantly. The analysis is meant to be performed for directional traffic. The inputs taken from the TRIMS database are as follows: AADT, K Factor, D Factor, PHF, Rural/Urban, Number of Lanes (N), Signals, Cross Streets, Functional Class, and Length of Segments. The outputs for Arterial segments are different than those of Freeway and Multilane Highway segments. The outputs to be determined are: Average Travel Speed (S_A or ATS), Running Time (T_R), Capacity (c), v/c ratio, Estimated Delay, and Level of Service (LOS).

The urban arterial analysis procedure is the most complicated analysis as compared to the other analysis procedures. It is based on Chapters 17 and 18 of HCM 2010. The nature and purpose of *Highway Capacity Manual* is for detailed operational assessment. Due to the planning emphasis of TRIMS, certain input parameters for HCM 2010 are not readily available. These parameters include:

- Signal Timing Plans
- Coordination of Signals along the Roadway
- Actuation/Detectors at Intersections
- Turning Movements
- Public Transit Demand and Service Information
- Parking Activities
- Pedestrian Demand

- Stop Controlled Intersection Information
- Roadside Assess Information
- Arrival Pattern at Intersections
- Queue Lengths

Due to the unavailability in TRIMS data related to signals assumptions must be made. For planning purposes, the procedure will use the following steps to analyze the LOS of urban arterials based on the HCM and assumptions.

Similar to the analysis procedure for freeway segments, we'll first calculate the flow rate (or, more appropriately, demand if the flow rate turns out to be higher than the capacity). The flow rate is calculated using Equation 28. This flow rate calculation is different from previous because it takes into account the percentage of flow turning left. The default value for the peak hour factor (PHF) is 0.9. The P_{LT} is the percent of left-turning vehicles with a default of 10% where applicable.

Equation 28: Urban Arterial Segment Flow Rate

$$v_p = \frac{AADT \times K \text{ Factor} \times D \text{ Factor} \times (1 - P_{LT})}{PHF}$$

Equation 29: Midsegment Volume

$$v_m = (AADT \times K \times D) / PHF$$

The flow rate, v_p , accounts for the volume of vehicles that travel through the intersection as well as turn from the direction of travel. It takes account for the left turning vehicles and combines the right turning vehicles into the account of the thru vehicles. The midsegment volume (v_m) is the volume of vehicles along the middle of the segment. It however does not account for turning vehicles. All vehicles in the midsegment calculation are assumed to travel through the intersection. The midsegment volume is calculated in Equation 29. Both the flow rate and volume are necessary for this analysis.

In the HCM 2000 the arterials were separated into sub categories. These subcategories were High-Speed, Suburban, Urban, and Intermediate arterials. The removal of the classifications allows the LOS determination method to be the same for all arterial types.

The next step in estimating the LOS, is determining the arterial segment the running time, T_R . This is to be estimated based on signal spacing, vehicle proximity, delay and FFS. Since FFS is rarely available in TRIMS, it must be calculated with Equation 30. Before FFS can be calculated, the inputs must first be determined.

Equation 30: Arterial FFS Calculation

$$FFS = (BFFS) \times f_{LS}$$

The FFS is based on the base free flow speed and the length of signal spacing. Equation 31 is used to calculate the BFFS for the segment, which is based on the posted speed, the adjustment due to access points, and the adjustment due to the cross section. The adjustment due to access points on the segment can be calculated using

Equation 33. The access point information is not found in TRIMS. Therefore, the default values are based on the functional code. The defaults are shown below (D_a). The adjustment due to the cross section can be calculated using Equation 33.

Equation 31: Arterial BFFS

$$\text{BFFS} = 25.6 + 0.47\text{PS} + f_{\text{CS}} + f_{\text{A}}$$

Equation 32: Arterial Access Point Adjustment

$$f_{\text{A}} = -0.078D_a/N_{\text{th}}$$

D_a :

Urban arterial = 34 points/mi

Urban collector = 61 points/mi

Suburban arterial = 21 points/mi

Suburban collector = 48 points/mi

Equation 33: Cross Section adjustment

$$f_{CS} = 1.5p_{rm} - 0.47p_{curb} - 3.7p_{curb}p_{rm}$$

p_{rm} = portion of link with restrictive median (total restrictive median portion / 2)

p_{curb} = portion of curbs present (total curbs portion / 2)

Once the BFFS has been determined, in order to estimate the FFS the adjustment of the signal spacing needs to be calculated using Equation 34. This is based on a default of 400 feet if the signal spacing cannot be determined from TRIMS.

Equation 34: Signal Spacing Adjustment

$$f_{LS} = 1.02 - \left(4.7 \times \frac{BFFS - 19.5}{\max(L_s, 400)}\right)$$

f_{LS} = signal spacing adjustment

L_s = distance between adjacent signals

The next step to determining the Running Time (T_R) is to account for traffic density. This is known as the vehicle proximity adjustment (f_v). The calculation is shown in Equation 35. The vehicle proximity adjustment is based solely on the midsegment volume (v_m).

Equation 35: Vehicle Proximity Adjustment

$$f_v = \frac{2}{1 + \left(1 - \frac{v_m}{52.8 \times N_{th} \times FFS}\right)^{0.21}}$$

Following this step, is the calculation of the delay caused by turning vehicles. This accounts for vehicles turning into the flow of traffic as well turn out of the flow of traffic. Based on the number of through of lanes and the midsegment volume for the segment, use the table below to determine the delay due to turning vehicles (d_{TV}).

Table 13: Through Vehicle Delay (National Research Council . Transportation Research, 2010)

Midsegment Volume (veh/h/ln)	Through Vehicle Delay (s/veh/pt) by Number of Through Lanes		
	1 Lane	2 Lanes	3 Lanes
200	0.04	0.04	0.05
300	0.08	0.08	0.09
400	0.12	0.15	0.15
500	0.18	0.25	0.15
600	0.27	0.41	0.15
700	0.39	0.72	0.15

Running time is based on running time, FFS, length of the segment, through movement control, and delays. For our analysis of arterial sections, we are to assume each segment in the analysis has a signalized intersection. Given the assumption, there are default values for through movement control, as well as start-up lost time.

Equation 36: Running Time Estimation

$$T_R = \frac{6 - l_1}{0.0025L} f_x + \frac{3600L}{5280(FFS)} f_v + d_{TV}$$

l_1 = start-up lost time (2.0 for signalized, 2.5 if stop of yield controlled)

f_x = control type adjustment (1.0 for signalized, 0 for uncontrolled, and $\min(v_{th}/c_{th}, 1.00)$ for yield controlled)

L = length of segment (ft)

The capacity, c , of a signalized intersection cannot be calculated without detailed signal timing information. For planning purposes, assumptions are made to make it possible for the analysis.

The major assumption of the capacity calculation below is there are no permitted left-turn movements allowed. This analysis is not for shared-lane movement groups.

Equation 37: Arterial Capacity

$$c = N \times S \times \frac{g}{C}$$

N is the number of through lanes in the travel direction. S is the saturation flow rate which is dependent upon the population. For larger cities such as Knoxville, Chattanooga, Nashville,

and Memphis the $S = 1900$ pcphpl only if the classification of the roadway is Urban. For all other roadways, $S = 1750$ pcphpl. The green ratio ($\frac{g}{C}$) is assumed to be 0.5 when the cross road is a state route and 0.67 otherwise. Given such, the capacity would be:

$$c = 950N \text{ or } 1273N \text{ (Urban in Knoxville, Nashville, Chattanooga, Memphis)}$$

$$c = 875N \text{ or } 1172.5N \text{ (otherwise)}$$

With the unit in vehicles/hour, one may also calculate X or the v/c ratio as:

Equation 38: v/c ratio

$$X = \frac{V_P}{c}$$

Signal delay at each intersection has to be estimated. The delay needed for this analysis is the through delay. For signalized intersections, the delay computed is the control delay. The through delay is the sum of control delay and geometric delay. Geometric delay can be disregarded since there is no effective change through intersections. For each intersection, the average delay per vehicle can be calculated as:

Equation 39: Average Delay per Vehicle:

$$d = d_1 + d_2 + d_3$$

d_1 = uniform delay (s/veh)

d_2 = incremental delay (s/veh)

d_3 = initial queue delay (s/veh)

Equation 40: Uniform Delay

$$d_1 = \frac{0.5C \left(1 - \frac{g}{C}\right)^2}{1 - [\min(1, X) \frac{g}{C}]}$$

Equation 41: Incremental Delay

$$d_2 = 900T[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}]$$

Equation 42: Initial Queue Delay

$$d_3 = \frac{3600}{vT} \left(ta \frac{Qb + Qe - Qeo}{2} + \frac{Qe^2 - Qeo^2}{2ca} - \frac{Qb^2}{2ca} \right)$$

The other values are shown above are described below.

Incremental Delay factor (k) = 0.5

Analysis Period (T) = 0.25 hour

Cycle Length (C) = 90 sec

Effective Green Time over cycle length (g/C) = 0.5 if the cross street is a state route and otherwise 0.67

Upstream Filtering Adjustment Factor (I) = 1.0 if signal is ≥ 0.6 miles from the nearest upstream signal and $I = 1.0 - 0.91X^{2.68}$ otherwise.

$d_3 = 0.0$ sec

The initial queue delay (d_3) is a new addition to the delay calculation in HCM 2010 for signalized intersections. However due to the lack of queue estimation information in TRIMS the value for d_3 is zero.

Since delay is incurred at all signalized intersections along the section, we define the cumulative delay per vehicle as:

$$d_S = \sum_{\text{All Signals}} d$$

The next calculation is the Average Travel Speed as shown in Equation 43. This is based on the segment length (in feet), Running Time, and estimated signal delay.

Equation 43: Average Travel Speed

$$S_t = \frac{3600L}{5280(T_R + d_s)}$$

Based on the travel speed determined previously and the base free flow speed of the section, the LOS can be determined from Table 14.

$$\% = \text{ATS} / \text{BFFS}$$

**Table 14: Level of Service Criteria Urban Arterial (National Research Council .
Transportation Research, 2010)**

Travel Speed as a Percentage of Base Free- Flow Speed (%)	LOS by Volume-to-Capacity Ratio ^a	
	≤1.0	> 1.0
>85	A	F
>67–85	B	F
>50–67	C	F
>40–50	D	F
>30–40	E	F
≤30	F	F

Two-Lane Highway Analysis

This final procedure of this entire analysis is of two-lane roads with no or only isolated traffic signals (spaced at least 2 miles apart). The updated HCM 2010 analysis method is for one-directional traffic as opposed to analyzing both directions. The inputs taken from the TRIMS database are: AADT, K Factor, D Factor, PHF, Rural/Urban, Terrain, Number of Lanes (N), Signals, Truck %, RV %, Functional Class, Length of Segments, Posted Speed, Directional Split, Grade, Lane Width, and Shoulder Width. The outputs that this analysis is estimating are: Average Travel Speed (ATS), Percent Time Spent Following (PTSF), Percent Free Flow Speed (PFFS), Capacity (c), v/c ratio, and Level of Service (LOS).

The two-lane highway analysis has been modified since the method used in the HCM 2000. The procedure in the HCM 2010 now provides a method for one directional analysis. The procedure to be implemented is based on Chapters 15 of HCM 2010. This procedure includes the addition of a new Highway Class. The main considerations of this procedure are the average travel speed, the percent time spent following, and the percent of the free flow speed on two lane highways. The presence of a passing lane or a directional climbing lane makes a difference to the LOS of a roadway segment and requires different analysis procedures. Both procedures can be found in this section, but the first procedure is for the segments without passing or climbing lanes.

It should be noted that two-lane roads exist in both rural and urban areas. HCM 2010 provides different standards and classifications for the three classes of such roadways as following.

Class I – These are two-lane highways on which motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary arterials connecting major traffic generators, daily commuter routes, or primary links in state or national highway networks generally are assigned to Class I. Class I facilities most often serve long-distance trips or provide connecting links between facilities that serve long-distance trips.

Class II – These are two-lane highways on which motorists do not necessarily expect to travel at high speeds. Two-lane highways that function as access routes to Class I facilities, serve as scenic or recreational routes that are not primary arterials, or pass through rugged terrain generally are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning and ending portions of longer trips, or trips for which sightseeing plays a significant role.

Class III – These are two-lane highways that serve as moderately developed areas. These highways may be portions of Class I and Class II highways that pass through small towns or developed recreational areas. On these segments, local traffic is mixed with through traffic; the density of un-signalized roadside access points is noticeably higher than in normal rural areas. Class III segments may be longer, passing through more spread-out recreational area, with increased roadside densities.

The first step is to identify the Two-Lane Highway. All of the following should be true to use this procedure.

- A total of two lanes for the two directions of the roadway(or 3 if the third lane is a passing lane or climbing lane) and
- The length of the segment between two signals is larger than 2 miles.

The next step is to classify the highway. In order to discover the appropriate LOS for the segment the classification of the highway is important. Because LOS considerations and analysis procedures are not exactly the same for the three classes of two-lane roads, it is essential to classify a roadway segment first. Being that some Class I and Class II highways can be designated as Class III Highways, we will classify Class I first. The descriptions of each highway class can be found in Table 15.

Table 15: Two-Lane Highway Classifications

Class I	Highway with relatively high speeds, major intercity routes, daily commuter routes, major links in highway networks, major traffic generator.
Class II	Not at high speeds, access routes, scenic or recreational routes, passing through rugged terrain, sight-seeing
Class III	Developed areas, local traffic mixed with through traffic, segments with higher access densities than in average rural areas

These steps must be followed in order to classify the highways accordingly.

1. A segment is classified as Class I if one of the following conditions is true:

- It has a SR or NHS designation;
 - It's FC code is 02, 06, 12, 14, or 16;
 - A speed limit greater than 45 mph
2. Class III segments for this classification are dependent of Access-Point Density.
 - If the access-point density is $\geq 12/\text{mi}$ for one side of traffic.
 3. Otherwise, we classify the segment as Class II.
 4. There might be exceptions based on the terrain, scenic route designation, and posted speed.

The next step is to determine the flow rate of the Two-Lane segment. Similar to other analysis procedures, we'll first calculate the flow rate for the section. This analysis requires a different flow rate calculation for Average Travel Speed and Percent Time Spent Following. Although the equations are similar, the factors within the equations correspond to different values.

Equation 44: ATS Flow Rate

$$v_{p, ATS} = \frac{AADT(K)}{PHF(f_{g, ATS})(f_{HV, ATS})}$$

v_p is total flow rate for the section in vph; include the "D-factor" to find the directional demand flow rate (d);

AADT and K factor should come from TRIMS;

PHF has default of 0.88 if unavailable from TRIMS;

$f_{g,ATS}$ is grade adjustment factor, method described below; and

$f_{HV,ATS}$ is heavy vehicle adjustment factor.

The grade adjustment factor for Average Travel Speed calculations, $f_{g,ATS}$, defined in HCM 2010 is separated into two sections. The first is based upon Level and Rolling Terrains and the other is based upon the grade percentage (G) of the segment. In order to find the adjustment factor for Level and Rolling Terrain sections, the one directional demand flow rate should be calculated as such:

Equation 45: Directional Demand Flow Rate

$$v_{vph} = V/PHF$$

The equation below is used to determine the $f_{g,ATS}$ for level and rolling terrains is given below. In order to find the $f_{g,ATS}$ for Mountainous terrains use Table 16.

$$f_{g,ATS} \text{ (level)} = 1.0$$

$$f_{g,ATS} \text{ (rolling)} = 0.274(v_{vph})^{0.1946}$$

Table 16: Reduction due to Grade for ATS

Grade (%)	$f_{g,ATS}$ Equation:
≥ 3	$0.76655 - 0.013924 * \text{GradeLength} + 0.00033188 * v_{vph}$
≥ 3.5	$0.694256 - 0.023495 * \text{GradeLength} + 0.00043896 * v_{vph}$
≥ 4.5	$0.582894 - 0.028148 * \text{GradeLength} + 0.00056958 * v_{vph}$
≥ 5.5	$0.461046 - 0.0268403 * \text{GradeLength} + 0.00069042 * v_{vph}$
≥ 6.5	$0.363503 - 0.032049 * \text{GradeLength} + 0.00076854 * v_{vph}$

The heavy vehicle adjustment factor for ATS, $f_{HV,ATS}$, is defined as the following in HCM 2010. It can be calculated using Equation 2: Reduction due to Heavy Vehicles, just as like the other sections. Use the equations and charts below to find the appropriate values for E_T and E_R . For segment with specific upgrades, use Table 17 for E_T and Table 18 for E_R .

E_T and E_R are:

Level: $E_T = 6.7057 v_{vph}^{-0.277}$

$E_R = 1.0$

Rolling: $E_T = .000001 v_{vph}^2 - 0.0026 v_{vph} + 2.8643$

$E_R = 1.1$

For segments with specific upgrades use the charts below:

E_T (ATS):

Table 17: E_T for specific Upgrades (ATS) (National Research Council . Transportation Research, 2010)

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h)								
		≤ 100	200	300	400	500	600	700	800	≥ 900
≥ 3 < 3.5	0.25	2.6	2.4	2.3	2.2	1.8	1.8	1.7	1.3	1.1
	0.50	3.7	3.4	3.3	3.2	2.7	2.6	2.6	2.3	2.0
	0.75	4.6	4.4	4.3	4.2	3.7	3.6	3.4	2.4	1.9
	1.00	5.2	5.0	4.9	4.9	4.4	4.2	4.1	3.0	1.6
	1.50	6.2	6.0	5.9	5.8	5.3	5.0	4.8	3.6	2.9
	2.00	7.3	6.9	6.7	6.5	5.7	5.5	5.3	4.1	3.5
	3.00	8.4	8.0	7.7	7.5	6.5	6.2	6.0	4.6	3.9
	≥ 4.00	9.4	8.8	8.6	8.3	7.2	6.9	6.6	4.8	3.7
≥ 3.5 < 4.5	0.25	3.8	3.4	3.2	3.0	2.3	2.2	2.2	1.7	1.5
	0.50	5.5	5.3	5.1	5.0	4.4	4.2	4.0	2.8	2.2
	0.75	6.5	6.4	6.5	6.5	6.3	5.9	5.6	3.6	2.6
	1.00	7.9	7.6	7.4	7.3	6.7	6.6	6.4	5.3	4.7
	1.50	9.6	9.2	9.0	8.9	8.1	7.9	7.7	6.5	5.9
	2.00	10.3	10.1	10.0	9.9	9.4	9.1	8.9	7.4	6.7
	3.00	11.4	11.3	11.2	11.2	10.7	10.3	10.0	8.0	7.0
	≥ 4.00	12.4	12.2	12.2	12.1	11.5	11.2	10.8	8.6	7.5
≥ 4.5 < 5.5	0.25	4.4	4.0	3.7	3.5	2.7	2.7	2.7	2.6	2.5
	0.50	6.0	6.0	6.0	6.0	5.9	5.7	5.6	4.6	4.2
	0.75	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
	1.00	9.2	9.2	9.1	9.1	9.0	9.0	9.0	8.9	8.8
	1.50	10.6	10.6	10.6	10.6	10.5	10.4	10.4	10.2	10.1
	2.00	11.8	11.8	11.8	11.8	11.6	11.6	11.5	11.1	10.9
	3.00	13.7	13.7	13.6	13.6	13.3	13.1	13.0	11.9	11.3
	≥ 4.00	15.3	15.3	15.2	15.2	14.6	14.2	13.8	11.3	10.0
≥ 5.5 < 6.5	0.25	4.8	4.6	4.5	4.4	4.0	3.9	3.8	3.2	2.9
	0.50	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
	0.75	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1
	1.00	10.3	10.3	10.3	10.3	10.3	10.3	10.3	10.2	10.1
	1.50	11.9	11.9	11.9	11.9	11.8	11.8	11.8	11.7	11.6
	2.00	12.8	12.8	12.8	12.8	12.7	12.7	12.7	12.6	12.5
	3.00	14.4	14.4	14.4	14.4	14.3	14.3	14.3	14.2	14.1
	≥ 4.00	15.4	15.4	15.3	15.3	15.2	15.1	15.1	14.9	14.8
≥ 6.5	0.25	5.1	5.1	5.0	5.0	4.8	4.7	4.7	4.5	4.4
	0.50	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8
	0.75	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8
	1.00	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.3	10.2
	1.50	12.0	12.0	12.0	12.0	11.9	11.9	11.9	11.8	11.7
	2.00	12.9	12.9	12.9	12.9	12.8	12.8	12.8	12.7	12.6
	3.00	14.5	14.5	14.5	14.5	14.4	14.4	14.4	14.3	14.2
	≥ 4.00	15.4	15.4	15.4	15.4	15.3	15.3	15.3	15.2	15.1

E_R (ATS):

Table 18: E_R for specific Upgrades (ATS) (National Research Council . Transportation Research, 2010)

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h)								
		≤100	200	300	400	500	600	700	800	≥900
≥3 <3.5	≤0.25	1.1	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0
	>0.25 ≤0.75	1.2	1.2	1.1	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75 ≤1.25	1.3	1.2	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>1.25 ≤2.25	1.4	1.3	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>2.25	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
≥3.5 <4.5	≤0.75	1.3	1.2	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75 ≤3.50	1.4	1.3	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>3.50	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
≥4.5 <5.5	≤2.50	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
≥5.5 <6.5	≤0.75	1.5	1.4	1.3	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75 ≤2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50 ≤3.50	1.6	1.5	1.4	1.3	1.2	1.1	1.0	1.0	1.0
	>3.50	1.6	1.6	1.6	1.5	1.5	1.4	1.3	1.2	1.1
≥6.5	≤2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50 ≤3.50	1.6	1.5	1.4	1.2	1.3	1.3	1.3	1.3	1.3
	>3.50	1.6	1.6	1.6	1.5	1.5	1.5	1.4	1.4	1.4

We will use TRIMS truck percentage information for following calculations:

$$P_T = (\text{TRIMS Combo Trucks \%}) + f_{RU} (\text{TRIMS Single Unit Trucks \%})$$

$$P_R = (1-f_{RU}) (\text{TRIMS Single Unit Truck \%})$$

Where:

f_{RU} is 1.00 for urban areas and 0.67 for rural areas.

Once this base information is determined for Average Travel Speed, the base information for the Percent Time Spent Following flow rate can be estimated using Equation 46.

Equation 46: PTSF Flow Rate

$$V_{P, PTSF} = \frac{AADT(K)}{PHF(f_{g, PTSF})(f_{HV, PTSF})}$$

where:

v_P is the total flow rate in vph;

AADT and K factor should come from TRIMS;

PHF has default of 0.88 if unavailable from TRIMS;

$f_{G, PTSF}$ is grade adjustment factor, method described below; and

$f_{HV, PTSF}$ is heavy vehicle adjustment factor.

The grade adjustment factor for the Percent Time Spent Following calculation, $f_{G, PTSF}$, is defined in HCM 2010 just as the method for ATS. As previously noted, the first is based upon Level and Rolling Terrains and the other is based upon the grade percentage (G) of the segment. In order to find the adjustment factor for Level and Rolling Terrain sections, the one directional demand flow rate should be calculated using Equation 45. Once determined, use the equations below to determine the grade adjustment based on terrain.

Level: $f_{g, PTSF} = 1.0$

Rolling: $f_{g, PTSF} = -0.0000005v_{vph}^2 + 0.0008v_{vph} + 0.6493$

For specific upgrades use the equations provided below:

Table 19: Reduction due to Grade for PTSF

Grade (%)	$f_{g,PTSF}$
≥ 3	$0.986265 + 0.011273 * \text{GradeLength} - 8.25E-5 * v_{vph}$
≥ 3.5	$0.988419 + 0.009734 * \text{GradeLength} - 4.375E-5 * v_{vph}$
≥ 4.5	$0.992222 + 0.035556 * \text{GradeLength} - 2E-5 * v_{vph}$
≥ 5.5	1.0

The heavy vehicle adjustment factor for PTSF, $f_{HV,PTSF}$, can be calculated using Equation 2. Use the charts below to find the appropriate values for E_T and E_R .

Level: $E_T = 1.1$ If $v_{vph} \leq 400$

$E_T = 1.0$ If $v_{vph} > 400$

$E_R = 1.0$

Rolling: $E_T = 0.000000005v_{vph}^3 - 0.000007v_{vph}^2 + 0.0013v_{vph} + 1.8048$

$E_R = 1.0$

For segments with specific upgrades use the charts below:

Table 20: E_T and E_R Values for Specific Grades (National Research Council . Transportation Research, 2010)

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, V_{vph} (veh/h)								
		≤100	200	300	400	500	600	700	800	≥900
Passenger Car Equivalents for Trucks (E_T)										
≥3 <3.5	≤2.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.5	1.3	1.3	1.2	1.0	1.0	1.0	1.0	1.0
	≥4.00	1.6	1.4	1.3	1.3	1.0	1.0	1.0	1.0	1.0
≥3.5 <4.5	≤1.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.50	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	2.00	1.6	1.3	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.8	1.4	1.1	1.2	1.2	1.2	1.2	1.2	1.2
≥4.00	2.1	1.9	1.8	1.7	1.4	1.4	1.4	1.4	1.4	
≥4.5 <5.5	≤1.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.50	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	2.00	1.7	1.6	1.6	1.6	1.5	1.4	1.4	1.3	1.3
	3.00	2.4	2.2	2.2	2.1	1.9	1.8	1.8	1.7	1.7
	≥4.00	3.5	3.1	2.9	2.7	2.1	2.0	2.0	1.8	1.8
≥5.5 <6.5	≤0.75	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.00	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2
	1.50	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.6
	2.00	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.8	1.8
	3.00	3.4	3.2	3.0	2.9	2.4	2.3	2.3	1.9	1.9
	≥4.00	4.5	4.1	3.9	3.7	2.9	2.7	2.6	2.0	2.0
≥6.5	≤0.50	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	0.75	1.0	1.0	1.0	1.0	1.1	1.1	1.1	1.0	1.0
	1.00	1.3	1.3	1.3	1.4	1.4	1.5	1.5	1.4	1.4
	1.50	2.1	2.1	2.1	2.1	2.0	2.0	2.0	2.0	2.0
	2.00	2.9	2.8	2.7	2.7	2.4	2.4	2.3	2.3	2.3
	3.00	4.2	3.9	3.7	3.6	3.0	2.8	2.7	2.2	2.2
	≥4.00	5.0	4.6	4.4	4.2	3.3	3.1	2.9	2.7	2.5
Passenger Car Equivalents for RVs (E_R)										
All	All	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

$$P_T = (\text{TRIMS Combo Trucks } \%) + f_{RU} (\text{TRIMS Single Unit Trucks } \%)$$

$$P_R = (1 - f_{RU}) (\text{TRIMS Single Unit Truck } \%)$$

where :

f_{RU} is 1.00 for urban areas and 0.67 for rural areas.

Analysis further in this section requires the use of directional split demands for both ATS and PTSF. Using the Directional Split provided in TRIMS, calculate the opposing demand of flow (v_o) and the analysis directional demand of flow (v_p). The TRIMS database contains the directional distribution as a percentage, assume the value given is the analysis directional demand and the remaining is the opposing demand. The equations below apply to both ATS and PTSF demand estimation. Use the appropriate values of v_p whether it is $v_{p,ATS}$ or $v_{p,PTSF}$.

Equation 47: Directional Flow Rate

$$v_d = v_p \times \text{D-factor}$$

Equation 48: Opposing Flow Rate

$$v_o = v_p - v_d$$

The next step in this procedure is to estimate the Free Flow Speed (FFS). In order to find the Average Travel Speed and Percent Time Spent Following, the FFS must be calculated.. If FFS is can be provided from direct field measurements, then it should be utilized. We'll use the following procedure from the HCM 2010 Chapter 15. The FFS is based on the base free flow speed, lane and shoulder widths, and access points. The base free flow speed is determined

using the posted speed and adding 10 mph. The factor f_{LS} , representing the reduction in FFS due to lane width and shoulder width restrictions, is found using Equation 50 where both inputs are in feet. The access point factor, f_A , which also causes reduction in FFS, is provided by Equation 51.

Equation 49: Two-Lane Highway FFS

$$FFS = BFFS - f_{LS} - f_A$$

Equation 50: Adjustment due to Lane Width and Shoulder Width

$$f_{LS} = 12.69 - 0.73(LW) - 0.70(SW)$$

Equation 51: Adjustment due to Access-Point Density

$$f_A = \frac{\text{Access - Points per Mile}}{4}$$

The default numbers for access-point density depend of highway classification. The access point reduction defaults for Class I and II are 4 and Class III is 8 for both directions. This is based upon the default values for access-point density being 8/mi and 16/mi for one directional traffic for Class I/II and Class III highways, respectively.

Upon determining the FFS, the Average Travel Speed can be obtained. Average Travel Speed, ATS, is a function of FFS, directional flow rate, and existence of passing zone. This is necessary for LOS determination of Class I and Class III highways. This value depends on the directional and opposing flow rate as well as the adjustment due to no-passing zones. Use the method described earlier to determine the values of $v_{d,ATS}$ and $v_{o,ATS}$.

Equation 52: Average Travel Speed (Two-Lane)

$$ATS = FFS - 0.00776(v_{d,ATS} + v_{o,ATS}) - f_{np,ATS}$$

The adjustment factor for no-passing zones, $f_{np,ATS}$, is specific to the direction of analysis. If the percentage of no-passing zone is known, Table 21 should be used. However, the percentage no-passing zone information is currently unavailable in TRIMS. For our analysis we are going to assume 20, 50 and 80% no-passing zones for Level, Rolling, and Mountainous segments. With the Table 21, use the values coordinated with the previously determined no-passing zone percentages for level, rolling, and mountainous and FFS to determine the appropriate adjustment factor.

Table 21: Reduction due to No-Passing Zones for ATS ($f_{np,ATS}$) (National Research Council .
Transportation Research, 2010)

Opposing Demand Flow Rate, v_o (pc/h)	Percent No-Passing Zones				
	≤ 20	40	60	80	100
FFS ≥ 65 mi/h					
≤100	1.1	2.2	2.8	3.0	3.1
200	2.2	3.3	3.9	4.0	4.2
400	1.6	2.3	2.7	2.8	2.9
600	1.4	1.5	1.7	1.9	2.0
800	0.7	1.0	1.2	1.4	1.5
1,000	0.6	0.8	1.1	1.1	1.2
1,200	0.6	0.8	0.9	1.0	1.1
1,400	0.6	0.7	0.9	0.9	0.9
≥1,600	0.6	0.7	0.7	0.7	0.8
FFS = 60 mi/h					
≤100	0.7	1.7	2.5	2.8	2.9
200	1.9	2.9	3.7	4.0	4.2
400	1.4	2.0	2.5	2.7	3.9
600	1.1	1.3	1.6	1.9	2.0
800	0.6	0.9	1.1	1.3	1.4
1,000	0.6	0.7	0.9	1.1	1.2
1,200	0.5	0.7	0.9	0.9	1.1
1,400	0.5	0.6	0.8	0.8	0.9
≥1,600	0.5	0.6	0.7	0.7	0.7
FFS = 55 mi/h					
≤100	0.5	1.2	2.2	2.6	2.7
200	1.5	2.4	3.5	3.9	4.1
400	1.3	1.9	2.4	2.7	2.8
600	0.9	1.1	1.6	1.8	1.9
800	0.5	0.7	1.1	1.2	1.4
1,000	0.5	0.6	0.8	0.9	1.1
1,200	0.5	0.6	0.7	0.9	1.0
1,400	0.5	0.6	0.7	0.7	0.9
≥1,600	0.5	0.6	0.6	0.6	0.7
FFS = 50 mi/h					
≤100	0.2	0.7	1.9	2.4	2.5
200	1.2	2.0	3.3	3.9	4.0
400	1.1	1.6	2.2	2.6	2.7
600	0.6	0.9	1.4	1.7	1.9
800	0.4	0.6	0.9	1.2	1.3
1,000	0.4	0.4	0.7	0.9	1.1
1,200	0.4	0.4	0.7	0.8	1.0
1,400	0.4	0.4	0.6	0.7	0.8
≥1,600	0.4	0.4	0.5	0.5	0.5
FFS ≤ 45 mi/h					
≤100	0.1	0.4	1.7	2.2	2.4
200	0.9	1.6	3.1	3.8	4.0
400	0.9	0.5	2.0	2.5	2.7
600	0.4	0.3	1.3	1.7	1.8
800	0.3	0.3	0.8	1.1	1.2
1,000	0.3	0.3	0.6	0.8	1.1
1,200	0.3	0.3	0.6	0.7	1.0
1,400	0.3	0.3	0.6	0.6	0.7
≥1,600	0.3	0.3	0.4	0.4	0.6

Once the Average Travel Speed is determined, the Percent Time Spent Following must be decided. A HCM 2010 based model is adopted for the estimation of PTSF (percent time-spent-

following) using Equation 53. This is only to be estimated for Class I and Class II highways.

The PTSF is calculated from inputs of base percent time spent following for the directional traffic, the presence of no-passing zones, and the directional and opposing flow rates.

Equation 53: Percent Time Spent Following (Two-Lane)

$$PTSF_D = BPTSF_D + f_{np} \left(\frac{V_{D, PTSF}}{V_{D, PTSF} + V_{O, PTSF}} \right)$$

Equation 54: Base Percent Time Spent Following (Two-Lane)

$$BPTSF = 100(1 - e^{av_d^b})$$

BPTSF is the base percent time-spent-following for the analysis direction of traffic. It can be estimated using Equation 54. The values for both "a" and "b" are based upon opposing demand flow rate (v_o). They can be estimated using the equations below.

$$a = -0.001461 - 3.4702E-6*v_o + 1.7708E-9*(v_o-900)^2$$

$$b = 0.9003701 - 7.3849E-5*v_o + 1.308E-7*(v_o-900)^2 - 8.049E-11*(v_o-900)^3$$

The combined adjustment factor for directional distribution and no-passing zone percentage or $f_{n/dp,PTSF}$ involves complicated procedures and calculations. Based on Exhibit 15-21 in the HCM 2010 we take assumptions to find the adjustment factor. Using 20, 50, and 80% as the no-passing zone percentage for level, rolling, and mountainous terrains and 60/40 as the directional split, the results are shown below. Use the values in Table 22 for flow rates less than 200vph.

The following equations will be used for the cases where the flow rate is greater than 200 vph:

Level Terrain:
$$f_{n/dp} = 45.96 - 0.018v_P - .00000497(v_P - 1142.86)^2 + .00000000594(v_P - 1142.86)^3$$

Rolling Terrain:
$$f_{n/dp} = 52.54 - 0.019v_P + .00000239(v_P - 1142.86)^2$$

Mountainous Terrain:
$$f_{n/dp} = 58.89 - 0.0218v_P + .0000032(v_P - 1142.86)^2$$

100% Passing:
$$f_{n/dp} = 18.24 - 0.0041v_P - .0000041(v_P - 1142.86)^2 + .0000000023(v_P - 1142.86)^3$$

100% No Passing:
$$f_{n/dp} = 68.05 - 0.0278v_P - .00000152(v_P - 1142.86)^2$$

The plots for the following equations can be found in the appendix.

Table 22: Reduction due to No-Passing Zone and Directional Distribution for PTSF

$(f_{np/dp,PTSF})$

Terrain:	$f_{np/dp,PTSF}$
Level	30.6
Rolling	46.1
Mountainous	52.3

The next step is to calculate the Percent Free Flow Speed (PFFS). This is necessary to determine to LOS for Class III highways. Use the equation below to determine PFFS, which is based on Average Travel Speed (ATS) and free flow speed (FFS). The ATS is based on the analysis direction, ATS_d .

Equation 55: Percent Free Flow Speed (Two-Lane)

$$PFFS = ATS_d / FFS$$

Prior to determining the v/c ratio one must first discover the capacity in the analysis direction. The v/c ratio is easily obtained as v_p/c where c is 3,200 vph for two-way directional traffic and 1,700 vph for one-way one-directional cases. Under non-ideal conditions the analysis direction capacity for ATS and PTSF is provided below. These equations are be use only if the demand adjustment factors are based upon a flow rate > 900 vehicles/hour.

$$C_{ATS} = 1700 \times f_{g,ATS} \times f_{HV,ATS}$$

$$C_{PTSF} = 1700 \times f_{g,PTSF} \times f_{HV,PTSF}$$

Once the ATS, PTSF, and PFFS are calculated the LOS can be estimated. It should be noted that whenever the v/c ratio is greater than 1.00, the LOS is considered F. Otherwise, LOS of the two-lane roadway segment may be determined depending on if it is a Class I, Class II, or Class III roadway. For Class I cases, both PTSF, percent time-spent-following, and ATS, average travel speed, values will be used to determine the level of service of the roadway segment. If the results differ and the ATS results as a LOS C and the PTSF results as LOS B, use the worse of the two LOS classes. Only LOS Classes A-E has been listed in the table, LOS F exists only when the volume exceeds the capacity or when v/c ratio is greater than 1.00. For Class II cases, only PTSF will be used. For Class III cases, the PFFS will be used. Again, only when v/c ratio is greater than 1.00 would LOS F result. The LOS can be estimated using Table 23.

**Table 23: Level of Service Criteria for Two-Lane Highways (National Research Council .
Transportation Research, 2010)**

LOS	Class I Highways		Class II Highways	Class III Highways
	ATS (mi/h)	PTSF (%)	PTSF (%)	PFFS (%)
A	>55	≤35	≤40	>91.7
B	>50–55	>35–50	>40–55	>83.3–91.7
C	>45–50	>50–65	>55–70	>75.0–83.3
D	>40–45	>65–80	>70–85	>66.7–75.0
E	≤40	>80	>85	≤66.7

Chapter VI – Findings and Discussion

Study Methodology

Task

Here is the timeline of tasks conducted to complete this study.

- Research the changes between HCM 2000 and HCM 2010
- Create new algorithms based from HCM 2010
- Collect Input Data from TRIMS for Analysis
- Discover which changes of the HCM are applicable to the analysis
- Regress and simplify the charts from the HCM 2010
- Run the analysis of the given segments
- Discuss the analysis results
- Identify the segments needing further analysis

Assumptions

Planning-level analyses are based on provided information as well as assumptions that are made for analysis. The assumptions made in this analysis are based on default values in the HCM 2010 as well as values believed to be beneficial to the methodology. These assumptions provide constraints to the newly created algorithms, and permit them to be planning-level analysis methods. Here is a list of assumptions and changes for this deficiency analysis:

- Base Free Flow Speed Calculations (Urban Arterial, Two-Lane Highways)

- Access-Point Densities (Urban Arterial, Two-Lane Highways)
- Signalized Intersections Delay Estimation (Urban Arterials)
- Saturation Flow Rates for larger cities (Urban Arterial)
- Initial Queue Delay (d_3) cannot be applicable due to lack of signal information (Urban Arterial)
- Class I Highways have travel speeds higher than 45 mph
- Class III Highways have 12 access-points/ mi or greater per side of traffic (Two-Lane)
- No-Passing Zone Percentages for Level, Rolling, and Mountainous Terrains are 20%, 50%, and 80% respectively

Newly Created Algorithms

The following algorithms are based on the algorithms provided in the HCM 2010, as shown in the previous chapter. A simplistic methodology for planning the quality of service for a roadway is obtained with each algorithm. Before the developed algorithms can be used, the input data for each segment must be known. Upon obtaining the input data, the algorithms then can be used for analysis. In addition to the roadway class specific algorithms, the algorithm for Bicycle Analysis on Two-Lane and Multilane Highways is provided.

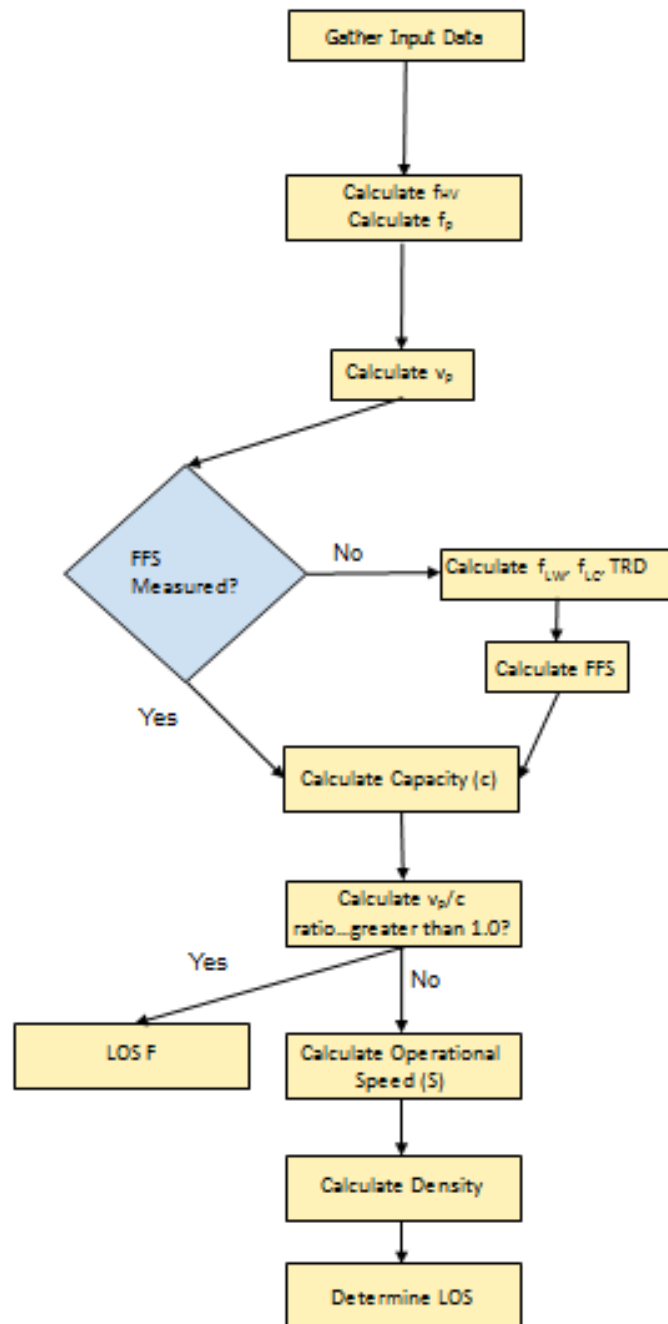


Figure 13: Freeway Segment Algorithm

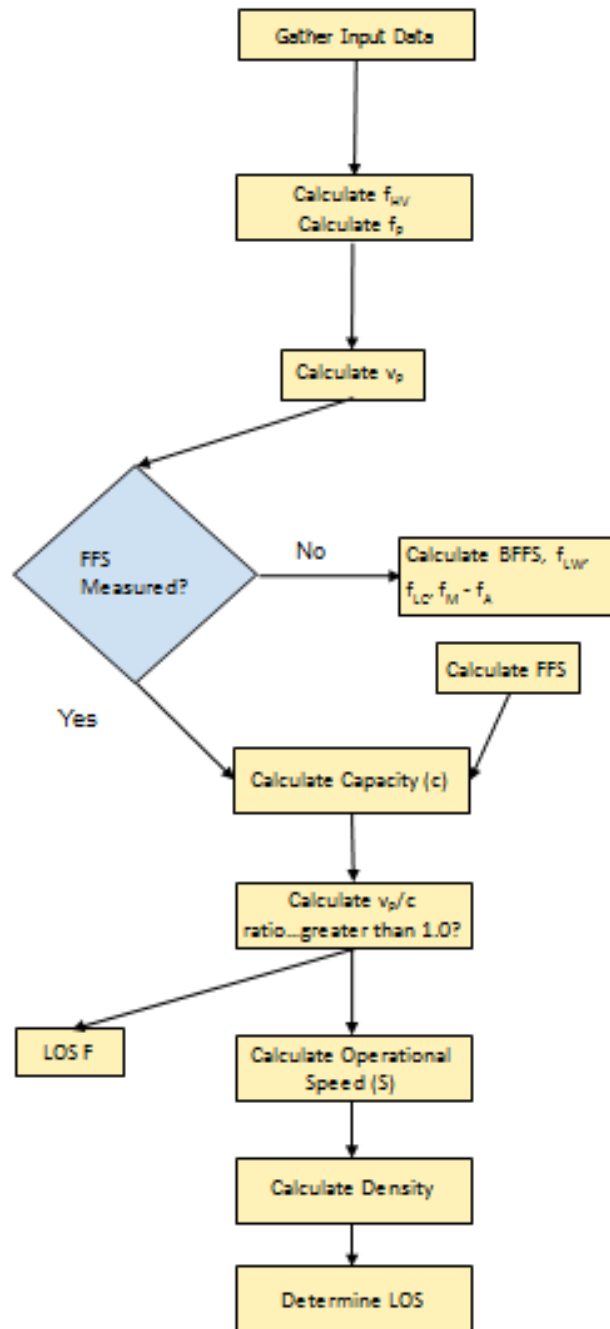


Figure 14: Multilane Highway Algorithm

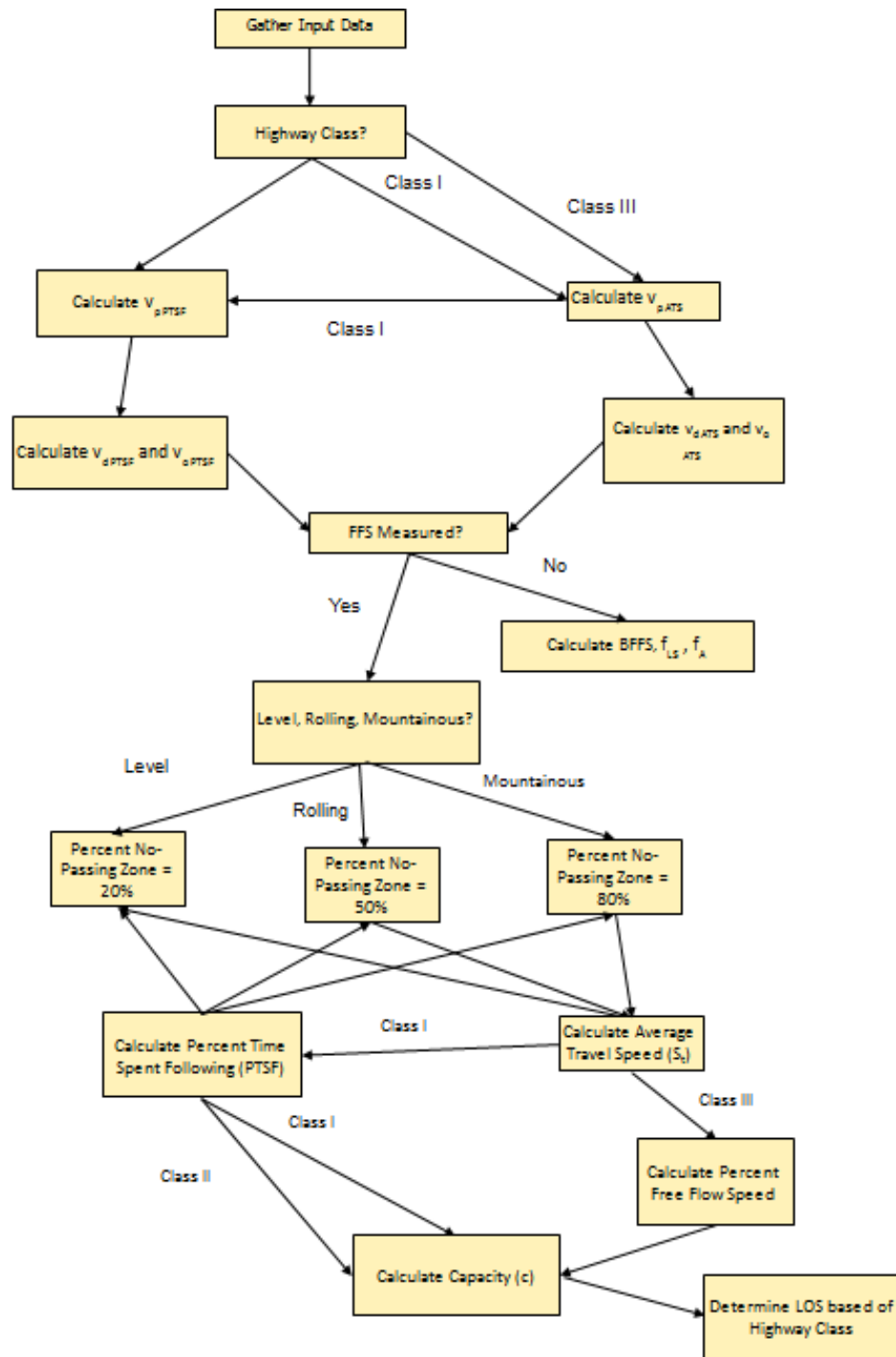


Figure 15: Two-Lane Highway Algorithm

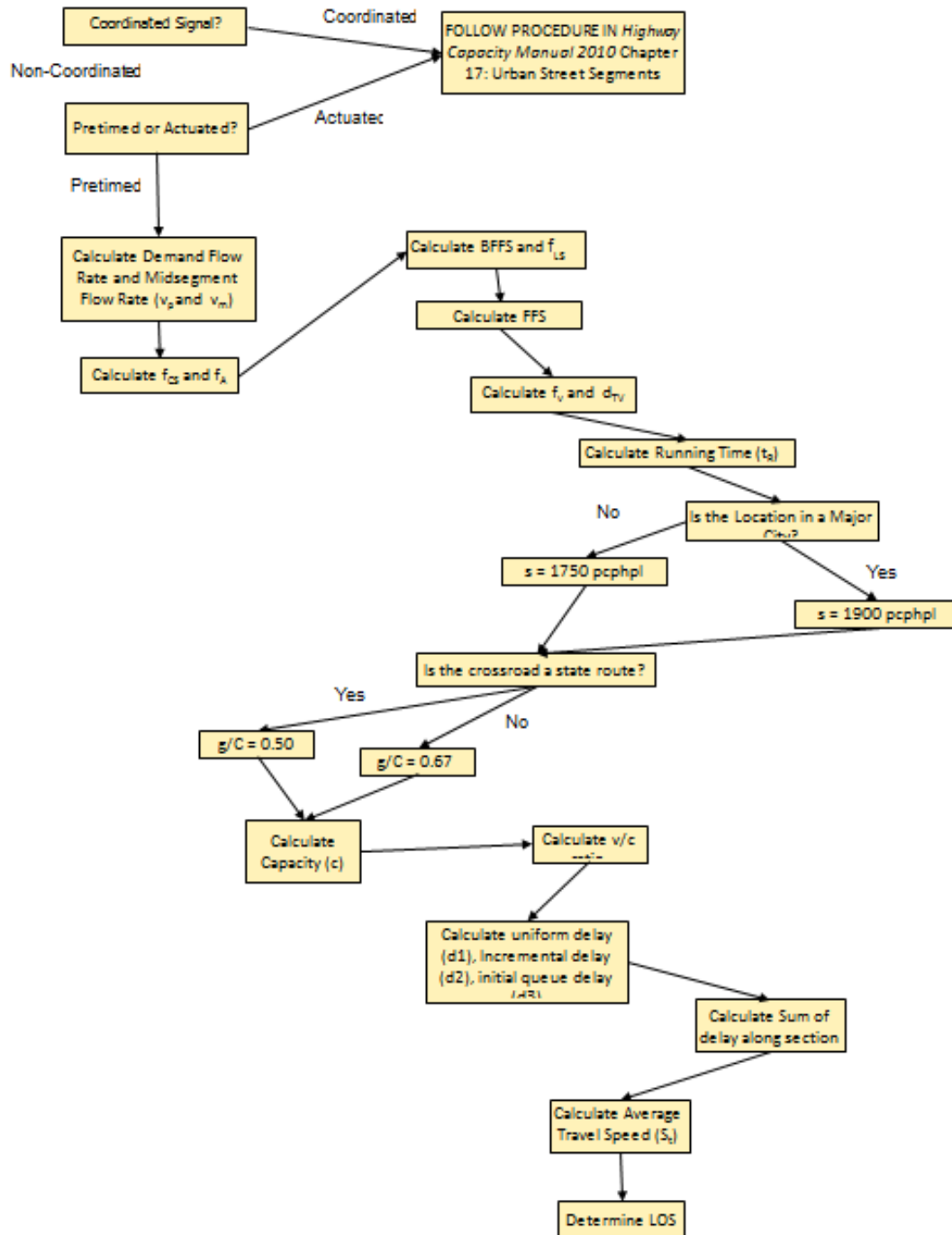


Figure 16: Urban Arterial Segment Algorithm

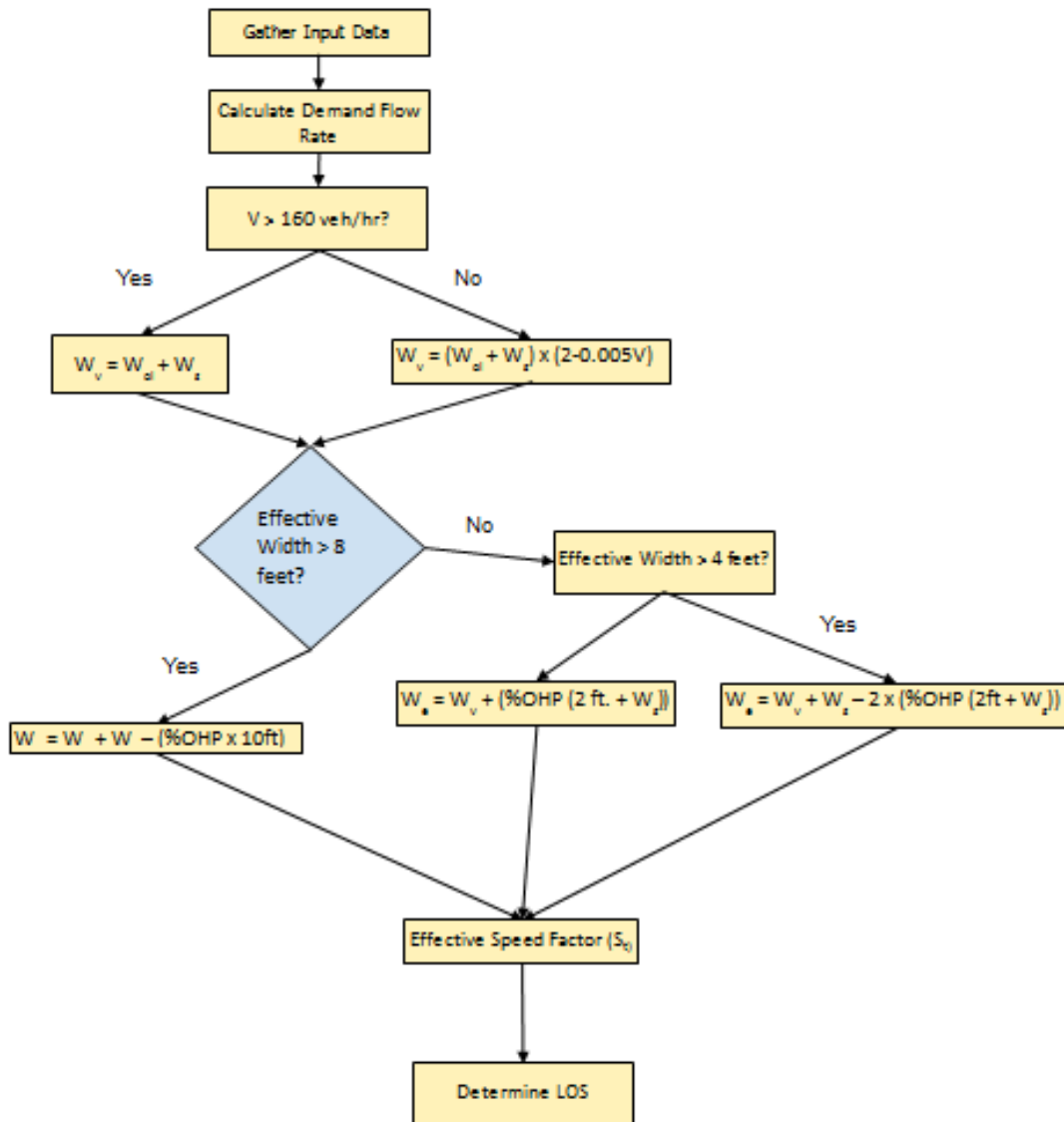


Figure 17: Bicycle Analysis Algorithm

Data Contributions

Making the analysis methods more manageable for programmers became is an important task.

Regression analyses were conducted for the charts within Chapter 15: Two-Lane Highways of the *Highway Capacity Manual* 2010. These charts are as follows:

- Exhibit 15-7
- Exhibit 15-9
- Exhibit 15-10
- Exhibit 15-11
- Exhibit 15-16
- Exhibit 15-17
- Exhibit 15-18
- Exhibit 15-20
- Exhibit 15-21

With the use of the software programs JMP and Microsoft Excel, statistical regression equations were formulated to simplify the charts. In order to determine whether some regressions were statistically stable, the mean absolute percentage error (MAPE) was calculated. For Exhibit 15-17, the MAPE values were all within 2% error; meaning the regression equation is only a 2% error away from the actual value. Devoid of a preset determinant for a sufficient MAPE value, the assumption was made that such a small error percentage is sufficient.

Example Calculations

Thousands of roadway segments were provided for the analysis. Given the amount of data for these segments, it would be difficult to demonstrate the calculations for each segment provided in this thesis. Therefore, this section contains example calculations using the deficiency analysis procedures developed in this study. Data used in the example calculations is the same data

from Washington County as the rest of the data used in the overall analysis. The calculations are separated by roadway class. The desired results of each section can be found in **bold print**.

Two-Lane Highway

Analysis Results:

- Average Travel Speed
- Percent Time Spent Following
- v/c ratio
- Percent Free Flow Speed
- Capacity
- Level of Service (LOS)

TRIMS DATA:

- Segment Length: 1.35 miles
- Road Name: Telford Rd.
- AADT: 2100
- K-Factor: 0.12
- D-Factor: 0.65
- PHF: 0.9
- P_T (Multi-Unit Trucks): 0%
- P_R (Single-Unit Trucks): 2%
- Right Shoulder Width: 2 feet
- Lane Width: 10 feet
- Number Lanes: 2
- Terrain: Rolling
- Speed Limit: 35 mph
- Functional Class: 08
- Rural

Assumptions:

- Access-Point Density (APD): 8 access-points/mile
- Based on APD, Highway Class Type: Class II
- f_{RU} : 0.67

Average Travel Speed Estimation:

$$V_{vph} = (AADT \times K \times D) / PHF = (2100 \times 0.12 \times 0.65) / 0.9 = 182 \text{ vph}$$

$$f_{g,ATS} = 0.274 \times (V_{vph})^{0.1946} = 0.274 \times (182)^{0.1946} = 0.754$$

$$E_T = 0.000001 \times (V_{vph})^2 - 0.0026 \times (V_{vph}) + 2.8643 = 0.000001 \times (182)^2 - 0.0026 \times (182) + 2.8643 = 2.424$$

$$E_R = 1.1$$

$$P_T = P_T + f_{RU} (E_R) = 0\% + 0.67 (2\%) = 0.134$$

$$P_R = (1 - f_{RU}) \times (E_R) = (1 - 0.67) \times (2\%) = 0.0066$$

$$f_{HV} = 1 / ((1 + P_T \times (E_T - 1) + P_R \times (E_R - 1))) = 1 / ((1 + 0.134 \times (2.424 - 1) + 0.0066 \times (1.1 - 1))) = 0.981$$

$$v_P = (AADT \times K) / (PHF \times f_{g,ATS} \times f_{HV}) = (2100 \times 0.12) / (0.9 \times 0.754 \times 0.981) = 378.52 \text{ vph}$$

$$v_D = 378.52 \times (D - \text{FACTOR}) = 378.52 \times (0.65) = 246.04 \text{ vph}$$

$$v_O = 378.52 - 246.04 = 132.48 \text{ vph}$$

$$BFFS = 35 \text{ mph} + 10 \text{ mph} = 45 \text{ mph}$$

$$f_{LS} = 12.69 - 0.73 \times (10 \text{ feet}) - 0.7 \times (2) = 3.99$$

$$f_A = \text{Maximum } (4, 8/4) = 4$$

$$FFS = BFFS - f_{LS} - f_A = 45 \text{ mph} - 3.99 - 4 = 37.01 \text{ mph}$$

$$f_{np} = 2.3$$

$$\mathbf{ATS = FFS - 0.00776 \times (v_P) - f_{np} = 37.01 \text{ mph} - 0.00776 \times (378.52 \text{ vph}) - 2.3 = 31.89 \text{ mph}}$$

Percent Time Spent Following *Estimation*:

$$V_{vph} = 182 \text{ vph}$$

$$f_{g,PTSF} = -0.0000005 \times (V_{vph})^2 + 0.0008 \times (V_{vph}) + 0.6493 = -0.0000005 \times (182)^2 + 0.0008 \times (182) + 0.6493 = 0.778$$

$$E_T = 0.000000005 \times (V_{vph})^3 - 0.000007 \times (V_{vph})^2 + 0.0013 \times (V_{vph}) + 1.8048 = 0.000000005 \times (182)^3 - 0.000007 \times (182)^2 + 0.0013 \times (182) + 1.8048 = 1.84$$

$$E_R = 1$$

$$P_T = 0.134$$

$$P_R = 0.0066$$

$$f_{HV} = 1 / ((1 + P_T \times (E_T - 1) + P_R \times (E_R - 1))) = 1 / ((1 + 0.134 \times (1.84 - 1) + 0.0066 \times (1 - 1))) = 0.989$$

$$v_P = (AADT \times K) / (PHF \times f_{g,PTSF} \times f_{HV}) = (2100 \times 0.12) / (0.9 \times 0.778 \times 0.989) = 363.79 \text{ v/h}$$

$$v_D = 363.79 \times (\text{D-factor}) = 363.79 \times 0.65 = 236.46 \text{ v/h}$$

$$v_O = 363.79 - 236.46 = 127.33 \text{ v/h}$$

$$\text{BFFS} = 45 \text{ mph}$$

$$F_{LS} = 3.99$$

$$F_A = 4$$

$$\text{FFS} = 37.01 \text{ mph}$$

$$a = -.00187$$

$$b = 1.01$$

$$\text{BPTSF}_d = 100 \times (1 - e^{-0.00187 \times (236.46)^{1.01}}) = 36.77 \%$$

$$f_{NP} = 47.08$$

$$\text{PTSF} (\%) = \text{BPTSF} + f_{NP} \times (v_d/v_p) = 36.77\% + 47.08 \times (236.46/363.79) = 67.37 \%$$

$$\text{PFFS} (\%) = \text{ATS}/\text{FFS} = 86.16 \%$$

$$\text{Capacity: } 1700 \text{ v/h}$$

$$V_{ats}/c = 378.52/1700 = 0.223$$

$$V_{ptst}/c = 363.79/1700 = 0.214$$

The LOS criterion for this segment is based on Percent Time Spent Following because it is considered a Class II Highway. The LOS determination for this segment is **LOS Class C**.

Multilane Highways:

Results:

- Density
- Operational Speed (S)
- Capacity (c)
- v/c ratio
- Level of Service (LOS)

TRIMS DATA:

- Segment Length: 4.73 miles
- Road Name: W. MOUNTCASTLE DR.
- AADT: 8500 vph
- K-Factor: 0.11
- D-Factor: 0.65
- PHF: 0.91
- P_T (Multi-Unit Trucks): 0%
- P_R (Single-Unit Trucks): 1%
- Total Lat. Clear. 2 feet
- Lane Width: 12 feet
- Total Number Lanes: 4
- Terrain: Rolling
- Speed Limit: 35 mph
- Functional Class: 16
- Urban
- Median: none

Assumptions:

- Access-Point Density (APD): 8 access-points/mile
- Population adjustment (f_p): 1

$$E_T = 2.5$$

$$E_R = 2$$

$$P_T = 0$$

$$P_R = 1\%$$

$$f_{HV} = 1 / (1 + P_T(E_T - 1) + P_R(E_R - 1)) = 1 / (1 + 0(2.5 - 1) + 0.01(2 - 1)) = 0.9901$$

$$V_P = AADT \times K \times D / PHF \times N \times f_{HV} \times f_g = (8500 \times 0.11 \times 0.65) / (0.91 \times 2 \times 0.9901 \times 1) = 337.27 \text{ v/h}$$

$$BFFS = 35 \text{ mph} + 7 = 42 \text{ mph}$$

$$f_{LW} = 0$$

$$f_{LC} = 1.3$$

$$f_M = 1.6$$

$$f_A = 8 \text{ access-point/mi} / 2 = 4$$

$$FFS = 42 - 0 - 1.3 - 1.6 - 4 = 32.8 \text{ mph}$$

$$\text{Capacity} = 2200 - 20 \cdot (60 - 32.8) = 1080 \text{ v/h}$$

$$\text{v/c ratio} = 337.27 / 1080 = 0.31$$

$$\text{Operational Speed} = \text{FFS} = 32.8$$

$$\text{Density} = 337.27 / 32.8 = 10.28 \text{ v/mi}$$

The criterion for LOS determination in the segment is based on Density. The resulting density of this segment describes as **LOS Class A**.

Urban Arterial Segment:

Results:

- Average Travel Speed (ATS)
- Estimated Delay (D_s)
- Running Time (T_R)
- Capacity (c)
- v/c ratio
- Level of Service (LOS)

TRIMS DATA:

- Segment Length: 0.28 miles
- Total Segment Length: 2.1 miles
- Road Name: King Springs Rd
- AADT: 2900
- K-Factor: 0.12
- D-Factor: 0.65
- PHF: 0.9
- P_T (Multi-Unit Trucks): 0%
- P_R (Single-Unit Trucks): 1%
- Right Shoulder Width: 1 foot
- Lane Width: 12 feet
- Number Lanes: 2
- Terrain: Rolling
- Speed Limit: 30 mph
- Functional Class: 16
- Urban

Assumptions:

- $D_a = 34$ access-point/mi
- $g/C = 0.67$
- $S = 1750$
- $D_3 = 0$
- $k = 0.5$
- $C = 90$ sec
- $T = 0.25$ hr.

$$V_m = (AADT \cdot K \cdot D) / PHF = (2900 \cdot 0.12 \cdot 0.65) / 0.9 = 251.33 \text{ v/h}$$

$$V_p = (AADT \cdot K \cdot D \cdot (1 - 0.1)) / PHF = (2900 \cdot 0.12 \cdot 0.65 \cdot (1 - 0.1)) / 0.9 = 226.2 \text{ B/H}$$

$$P_{CURB} = 0$$

$$P_{RM} = 0$$

$$F_{CS} = 0$$

$$F_a = -0.078 \cdot (D_a) / N = -0.078 \cdot (34) / 1 = -2.652$$

$$BFFS = 25.6 + 0.47 \cdot (30) + 0 + -2.652 = 37.05 \text{ mph}$$

$$F_{ls} = 1.02 - 4.7 \cdot ((BFFS - 19.5) / (0.28 \cdot 5280)) = 0.9642$$

$$FFS = BFFS \cdot f_{ls} = 37.05 \cdot 0.9642 = 35.72 \text{ mph}$$

$$F_v = 2 / (1 + (1 - (V_m / (52.8 \cdot N \cdot FFS)))^{0.21}) = 2 / (1 + (1 - (251.33 / (52.8 \cdot 1 \cdot 35.72)))^{0.21}) = 1.015$$

$$D_{TV} = (V_m - 300) \cdot 0.0007 + 0.08 = (251.33 - 300) \cdot 0.0007 + 0.08 = 0.0459$$

$$T_R = ((6 - 2) / (0.0025 \cdot LS \cdot 5280)) \cdot 1 + ((3600 \cdot (LS \cdot 5280)) / (FFS \cdot 35.72)) \cdot f_v + d_{tv} = ((6 - 2) / (0.0025 \cdot 0.28 \cdot 5280)) \cdot 1 + ((3600 \cdot (0.28 \cdot 5280)) / (5280 \cdot 35.72)) \cdot 1.015 + 0.0459 = 29.77 \text{ sec}$$

$$\text{Capacity (c)} = N \cdot S \cdot (g/C) = 1 \cdot 1750 \cdot 0.67 = 1172.5 \text{ v/h}$$

$$X = v/c \text{ ratio} = 226.2 / 1172.5 = 0.19$$

$$I = 1 - 0.91 \cdot (X^{2.68}) = 1 - 0.91 \cdot (0.19^{2.68}) = 0.989$$

$$D1 = k \cdot C \cdot (1 - (g/C)^2) / (1 - X) \cdot (g/C) = 0.5 \cdot 0.67 \cdot (1 - (0.67)^2) / (1 - 0.19) \cdot (0.67) = 5.628 \text{ sec}$$

$$D2 = 900 \cdot T \cdot ((X - 1) + \sqrt{(X - 1)^2 + ((8 \cdot k \cdot I \cdot X) / (c \cdot 0.25))}) = 900 \cdot 0.25 \cdot ((0.19 - 1) + \sqrt{(0.19 - 1)^2 + ((8 \cdot 0.5 \cdot 0.989 \cdot 0.19) / (1172.5 \cdot 0.25))}) = 0.3625 \text{ sec}$$

$$D_s = d1 + d2 + d3 = 5.628 + 0.3625 + 0 = 5.99 \text{ sec}$$

$$ATS = (3600 \cdot LS \cdot 5280) / (5280 \cdot (T_R + D_s)) = (3600 \cdot 0.28 \cdot 5280) / (5280 \cdot (29.77 + 5.99)) = 28.19 \text{ mph}$$

$$\text{Travel Speed} / BFFS = 28.19 / 37.05 = 0.761 = 76.1\%$$

The LOS criterion for this segment is based on the percentage of travel speed to base free flow speed. The LOS determination for this segment is **LOS Class B**.

Freeway Analysis:

Results:

- Density
- Operational Speed (S)
- Capacity (c)
- v/c ratio
- Level of Service (LOS)

TRIMS DATA:

- Segment Length: 3.22 miles
- Road Name: JAMES H. QUILLEN PKWY.
- AADT: 46050
- K-Factor: 0.11
- D-Factor: 0.51
- PHF: 0.91
- P_T (Multi-Unit Trucks): 5%
- P_R (Single-Unit Trucks): 3%
- Total Lateral Clearance: 2 feet
- Lane Width: 12 feet
- Total Number Lanes: 4
- Terrain: Rolling
- Speed Limit: 70 mph

Assumptions:

- Ramps: 4 ramps/mile
- $f_p = 1$

$$TRD = 4/6 = 0.6667$$

$$E_T = 2.5$$

$$E_R = 2$$

$$P_T = 5\%$$

$$P_R = 3\%$$

$$f_{HV} = 1 / (1 + P_T(E_T - 1) + P_R(E_R - 1)) = 1 / (1 + 0.05(2.5 - 1) + 0.03(2 - 1)) = 0.905$$

$$V_P = AADT \times K \times D / PHF \times N \times f_{HV} \times f_g = (46050 \times 0.11 \times 0.51) / (0.91 \times 2 \times 0.905 \times 1) = 1419.5 \text{ v/h}$$

$$f_{LW} = 0$$

$$f_{LC} = 0$$

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22 \times TRD^{0.84} = 75.4 - 0 - 0 - 3.22 \times (0.6667)^{0.84} = 73.11 \text{ mph}$$

$$\text{Capacity} > 70 \rightarrow c = 2400 \text{ v/h}$$

$$v/c \text{ ratio} = 1419.5 / 2400 = 0.59$$

$$\text{Operational Speed (S)} = FFS - 0.0000116 \times (v_p - 1200)^2 = 73.11 - 0.0000116 \times (1419.5 - 1200)^2 = 72.55 \text{ mph}$$

$$\text{Density} = v_p / S = 1419.5 / 72.55 = 19.56 \text{ v/mi}$$

The LOS criterion for this segment is based on density of the roadway. The LOS determination for this segment is **LOS Class C**.

RESULTS

Before the comparison part of the deficiency analysis, the results must first be analyzed for patterns and key changes to the LOS. Specific factors were found to affect the level of service more than others. For the Freeway segment analysis the important factor appeared to be Total Ramp density (TRD). The TRD affects the free flow speed, which is always relatively high for freeway segments. Since TRD is based on the number of ramps within six miles, a high or low number of ramps on the segment will not affect the LOS in a major way. Multilane Highway analysis and Freeway analysis are similar. The number of ramps and interchanges on a freeway

are similar to access points on a multilane highway. The results of the multilane highway analysis reveal the importance of access-point densities, which can result in a change of the Level of Service by an entire grade. Access point densities are also a factor in the Two-Lane Highway analysis, and also effect the LOS in the manner described for Multilane Highways.

The most important factor in Two-lane Highways analysis is highway classification. The level of service criterion is different for each of the three classes. In correctly classifying a highway can produce results that are inaccurate. The complexity of the Two-Lane Highway analysis is comparable to that of the Urban Arterial Segment analysis. Unlike two-lane analysis, segment length is important and can result in level of services grade changes. In order to improve the results of segments less than 0.1 miles during the analysis, said segments were added to adjacent segments with similar input data (i.e. AADT, Number of Lanes, etc.). Thus increasing the segment lengths of the adjacent segment and improving the overall level of service. Later in the comparison of results, the segments were reverted back to their original lengths.

Annual Average Daily Traffic (AADT) was very important for each of the analyses because it is a part of the flow rate calculation. Segments returning the LOS Class F results were due to the higher AADT values. Multilane Highways with very high AADT values can result the demand on segments exceeding the capacity, resulting in a LOS Class F. Two-lane Highways with a combination of high AADT values and shorter segment lengths results in lower LOS Classes. Two-Lane highway results also show that segments with more than 5,000 vph resulted in LOS Classes D, E, and F.

Other reasons two-lane highway segments resulted in low LOS classes are as follows. The change in functional class codes from an urban segment to a rural segment can result in different LOS results on the same segment for different links on the segment. This is because the

analysis calculations for rural and urban segments differ in specific values. Change in Speed Limits also affects the LOS Class. Highway Class I is based on high speeds and if the segment has a speed limit less than 45 mph then the highway classification is changed. This results in a change of LOS criterion as described previously. Now that these results have been analyzed, next is comparison section of the deficiency analysis.

Comparison

In order for this study to be considered a proper deficiency analysis, a comparison of current results and past results must occur to identify the changes. Using results from a previous EVE analysis, a comparison of the same roadway segments were conducted. The percentage of LOS changes in the results from EVE and the new method can be shown in Table 24. These percentages are based on a total of 256 comparable segments. As shown in the table, LOS Class A decreased by more than half, LOS Class B increased and produced the most values, LOS Class C remained about the same, LOS Class D increased by more than half, LOS Class E decreased, and LOS Class F increased from nothing previously.

The change of methodologies results in a higher percentage of bad LOS grades for the roadway segments as compared to EVE. There are 42 segments with the LOS Classes of D, E, and F. These segments will be highlighted and further analyzed to discover possible improvement methods.

Table 25 shows the percentages of the results that are either the same, better, or worse than those from EVE. There are 35 segments with better levels of service results than those provided by EVE. The reason for these results are because of the changes from the HCM

methodologies. This shows that the updated methods have improved the analysis compared to the past. The newer methods are more accurate for the given analysis.

Table 24: Portion of LOS Values

LOS	EVE Analysis (%)	New Analysis (%)
A	42.97	20.31
B	27.34	44.53
C	21.48	18.75
D	4.30	10.55
E	3.91	2.73
F	0	3.13

Table 25: New LOS Percentage Results

New LOS Results	Number	Percentage of Total (%)
Same as EVE	104	40.6
Better than EVE	35	13.7
Worse than EVE	117	45.7

Of the 45.7% of the segments with results worse than those of EVE 4.0, the majority of them were only a single LOS grade from the previous. Concern was shown for segments with more

than one LOS grade change, more importantly segments going from a “good” LOS grade (A-C) to a “poor” LOS grade (D-F). There were 16 segments fitting this description. Of these 16 segments, the most concerning was the Two-Lane segment that changed from a LOS Class B to a LOS Class E. This resulted from the roadway having a lane drop and changing the LOS analysis type. Segments prior to this one had 4 lanes and then this segment went down to 2 lanes, thus changing the analysis from Multilane to two-Lane analysis. The remaining 15 segments resulted as they are due to changes in Two-Lane Highway classification. Highway classification is very sensitive and as shown can result in changes in the LOS.

In order to gain a better understanding of the connection between EVE 4.0 results and result of the newly proposed method, bubble diagrams were used. The use of the diagrams is to show how much deviation from previous results had been made. As shown in Figure 18, the results of this new method are similar to those of the results of the EVE method.

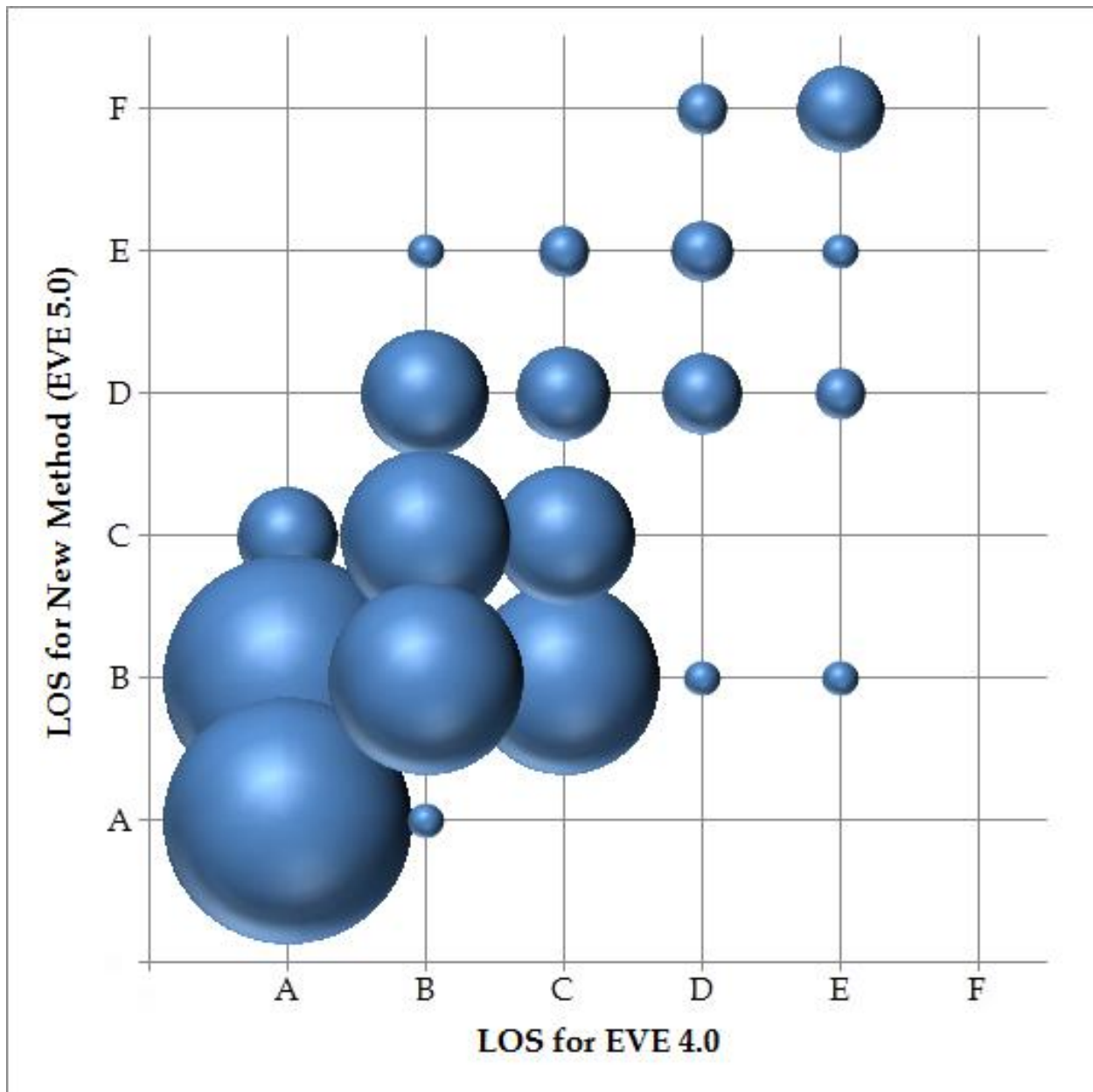


Figure 18: LOS Comparison results

For roadway class specific analysis the comparison data was separated into either of the following types: Two-Lane Highways, Multilane Highways, and Urban Street Segments. There were no Freeway Segments in the comparison data; therefore there are no results for

comparison. Out of the results provided above, roughly 86% of the results were Two-Lane Highways, 11% are Urban Arterial Segments, and the remaining 4% are Multilane Highways. The comparison results of each of these roadway classes are provided in the tables below. Table 26 shows the number of segments with results similar to that of EVE.

Table 26: Same as EVE Results by Roadway Class

Roadway Class	Number	Percentage Total Same as EVE (%)
Two-Lane Highways	86	82.7
Urban Arterial Segments	12	11.5
Multilane Highways	6	5.8

In comparison to EVE, the following figures show bubble diagrams according to roadway class. The results shown are based on 219 Two-lane highway segments, 28 Urban Arterial Segments, and 9 Multilane Segments. Of the segments for Two-Lane, Urban Arterial, and Multilane highways the percentages of values worse than EVE 4.0 are 47%, 40%, and 33% respectively. The values on the Urban Arterial and Multilane segments only change by one LOS Class. The values change on the Two-Lane Highway segments due to the reasons previously discussed.

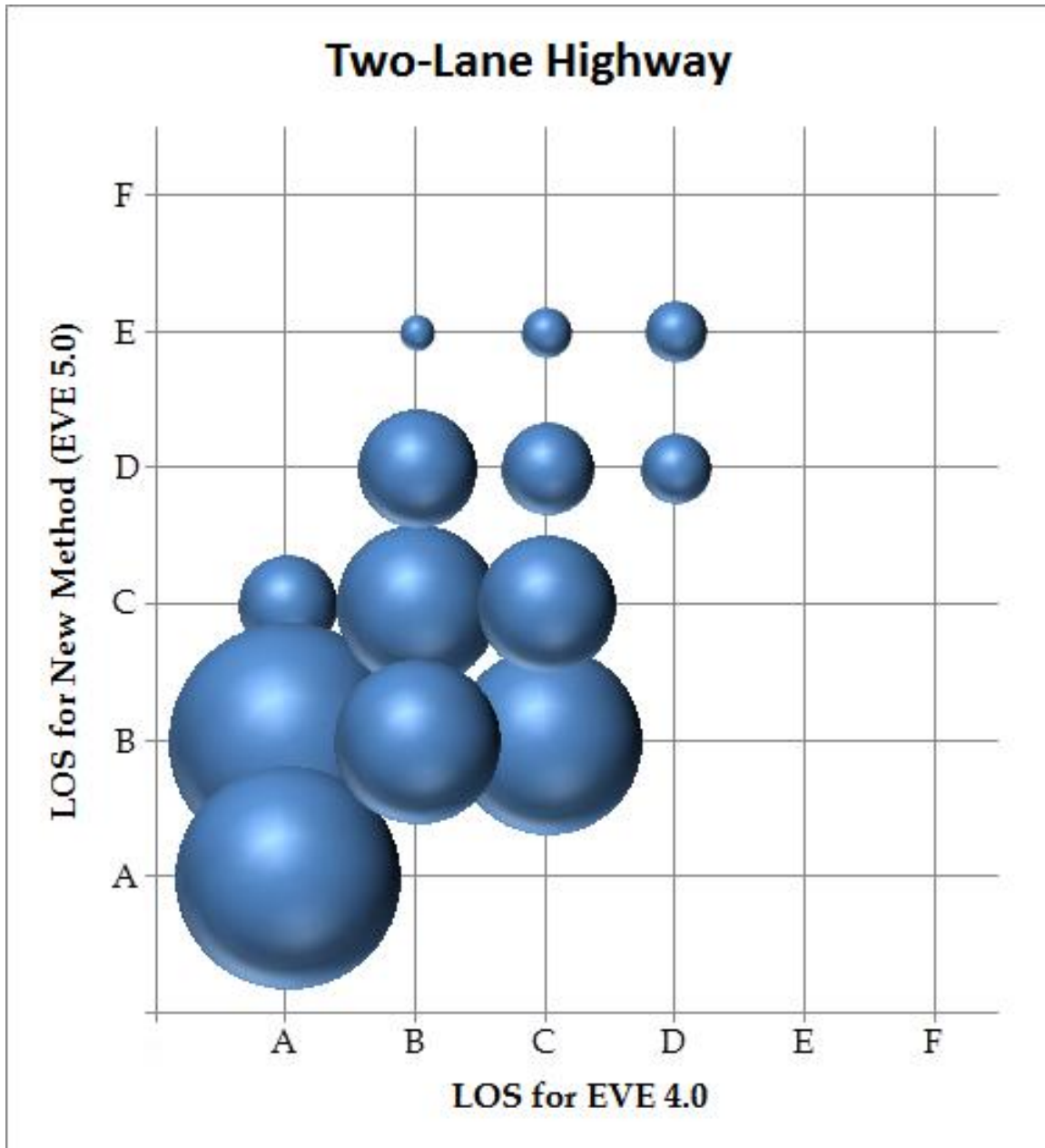


Figure 19: LOS Comparison Results for Two-Lane Highways

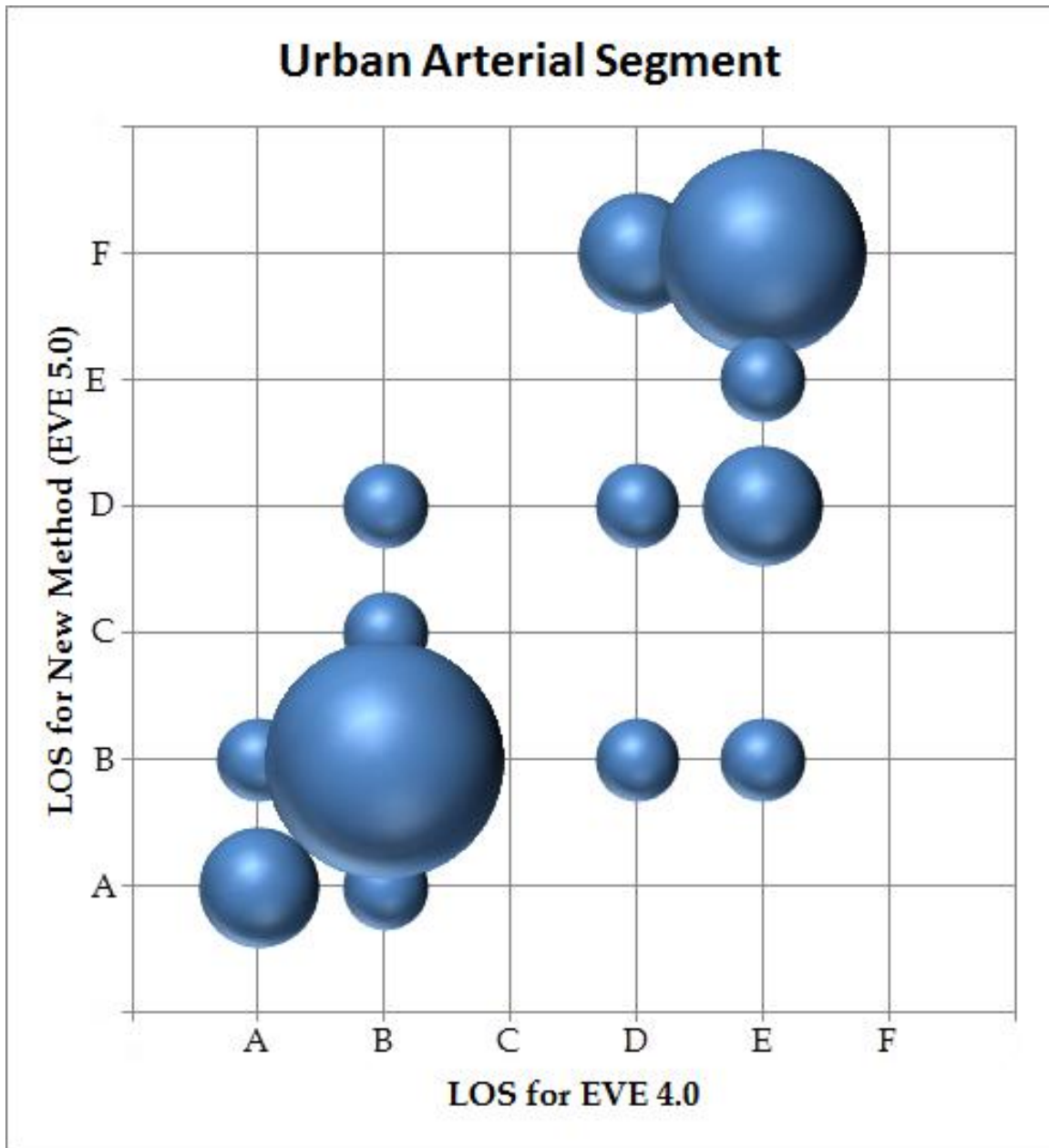


Figure 20: Urban Arterial Segment LOS Comparison Percentages

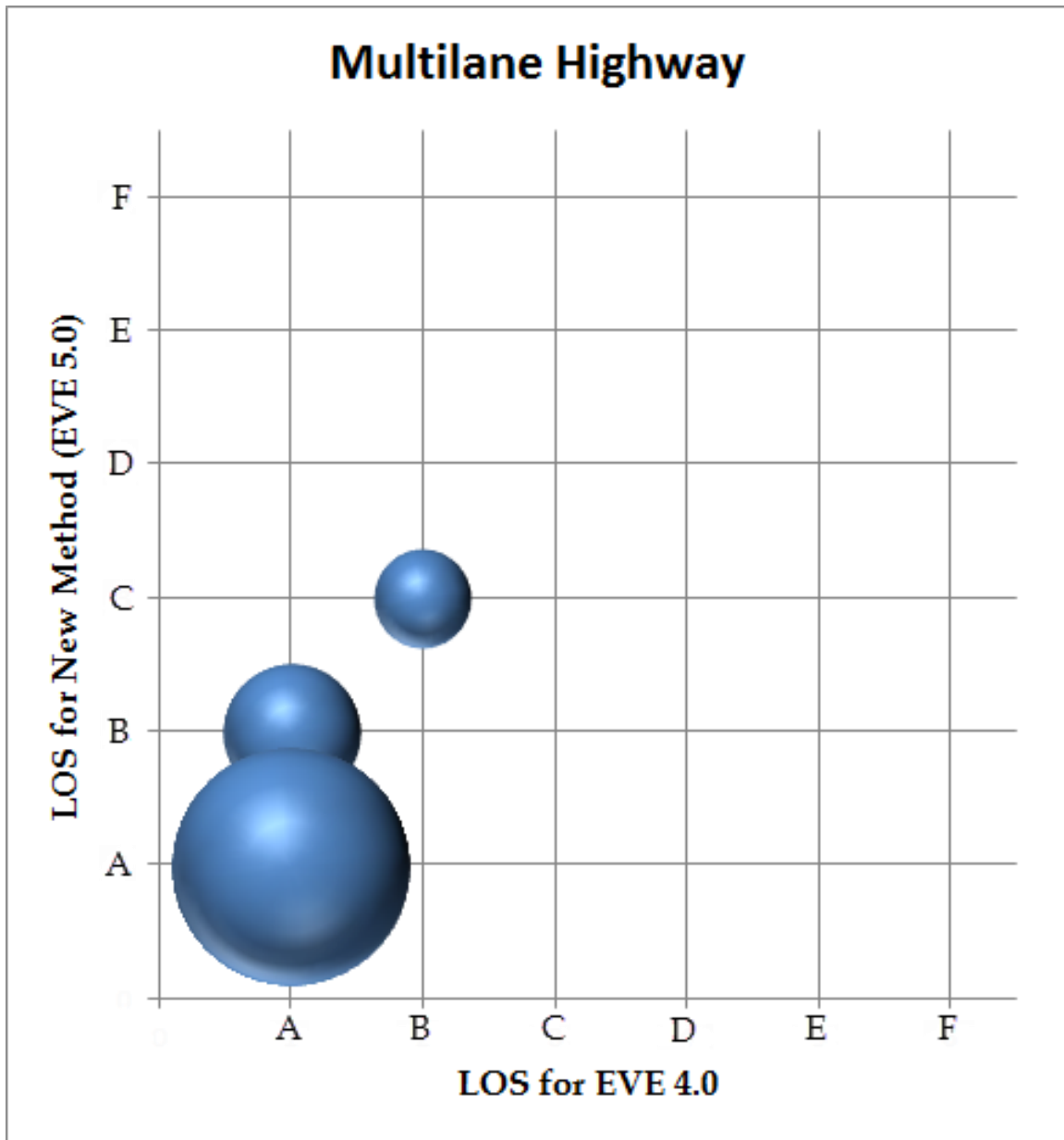


Figure 21: Multilane Highway LOS Comparison Percentages

Given these results, segments with LOS Classes D, E, and F need to be analyzed further to identify methods of improvement. There are 42 segments that need to be analyzed further out of this data set. Some things that may need improvement on these segments are lane width,

speed, segment length. The deficiency analysis program will analyze the roadways with poor LOS grades. From there it is up to the analyst to determine the next step.

Chapter VII – Conclusion

Summary

This study was conducted to benefit those using deficiency analyses for decision-making purposes in the transportation field. Since deficiency analysis is used for funding based decisions the method of the deficiency analysis for Tennessee had to be updated. This new method is based on the HCM 2010 methodologies and algorithms. With the use of the HCM 2010 a new procedure was established to determine the quality of service on Freeway, Multilane Highway, Two-Lane Highway, and Urban Arterial segments of roadway. The study was taking from a planning-level view for the analysis. Therefore, many assumptions were to be made if the data was not provided. This study used TRIMS, a database provide by TDOT for input data. The inputs that were not in TRIMS had to be assumed. Using data from Washington County, some of the findings of the analysis show the importance of access-point density on Multilane and Two-Lane highways as well as proper classification for Two-Lane highways. A comparison of Two-Lane highways was conducted and it shows that in comparison to the others, Class III highways provide the best LOS. Another comparison was conducted in order to identify problem areas in comparison to EVE database results. Of the 256 segments compared to EVE, 42 of these segments results in a LOS Class D, E, or F. These 42 results will be further analyzed to resolve any potential issues resulting in a poor LOS. The updating in the deficiency analysis method used for decision-making is beneficial for future project estimation and must be continually updated with information selected from the most recent edition of *Highway Capacity Manual*.

Suggestions

In order to provide the best results from this deficiency analysis procedure and future deficiency analyses it is believed that the following suggestions must be considered. Careful attention must be paid to the classification of Two-Lane highways. Each highway class bases its level of service determination on a different service measure. Thus an incorrect classification will result in an incorrect LOS determination. The use of a deficiency analysis program in association with an information database (such as TRIMS) must conduct analyses whenever there is new information in the database related to the program. In addition to running analyses, it is beneficial to develop methods of gathering information related to other modes of transportation into such information databases. The addition of the bicycle analysis for this program will provide a gateway for other alternative modes to be added. This addition of a multimodal analysis method to determine deficiency analysis is dependent upon the information database. If the information database does not contain the necessary information, the alternative modes cannot be assessed. Additional measures of effectiveness (MOEs) on a roadway should be considered for each roadway for future deficiency analysis efforts. Developing an analysis considering safety as a service measure will provide benefit to both the users as well as the planners. If safety is to be added a starting point of research should be NCHRP Report 546. The use of the most accurate input data will be beneficial to the analysis of the segment.

APPENDIX

Capacity Analysis Procedures for Eve 4.0

Roadway Class Sorting Procedure

INPUT: from TRIMS database:

*FC Code, Route Name, Number of Lanes, Direction, Lane Type, AADT,
Terrain*

OUTPUT: Roadway Class (RC) and, hence, procedures to be used for analysis.

*RC = 1 for Freeway, 2 for Arterial, 3 for Two-Lane Road, or 4 for
Multilane Road*

Step 1. Identify Interstate and Freeways

IF FC Code = 01 or 11 THEN

RC = 1 for freeway/Interstate

EXIT

ELSE IF Road Name = "Interstate ..." THEN

RC = 1

EXIT

END IF

Step 2. Identify Arterials

IF Signal Spacing is less or equal to 2.0 miles THEN

RC = 2 for arterial

EXIT

END IF

Step 3. Identify Two-Lane Roads

IF Number of Lanes = 2 THEN

IF Directions = 2 THEN

RC = 3 for two-lane road

EXIT

END IF

ELSE Number of Lanes = 3 THEN

IF Directions = 2 THEN

IF Pavement Width <= 28 THEN

RC = 3

EXIT

ELSE IF Lane Type = 0, 2, 3, 6, 8, or 12 THEN

RC = 3

EXIT

END IF

END IF

END IF

Step 4. The Rest Are Multilane Roads

RC = 4

Section I - Freeway Segment Analysis Procedure

INPUT: from TRIMS database:

*AADT, K Factor, D Factor, PHF, Lane Width, Lateral Clearance,
Rural/Urban, Terrain,
Truck Percentage,*

OUTPUT: Operational Speed (S)

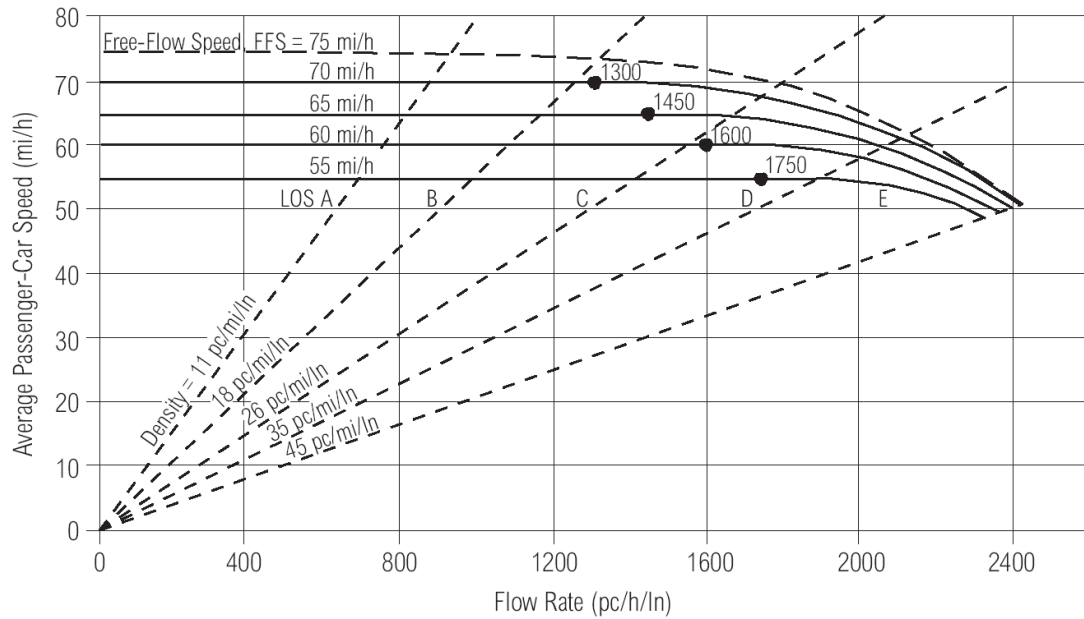
Capacity (c)

v/c ratio

Density (D)

Level of Service (LOS)

This procedure is based on Chapter 23 of HCM 2000. At the heart of the methodology is the speed-flow relationship depicted in the figure below. Once the flow rate and the average passenger-car speed are obtained, the LOS can be derived.



First calculate the flow rate (or, more appropriately, demand if the flow rate turns out to be higher than the capacity) entering the segment:

$$V_P = \frac{AADT \times K \text{ Factor} \times D \text{ Factor}}{PHF \times N \times f_{HV} \times f_P}$$

where:

AADT, *K* and *D* factors should come from *TRIMS*;

PHF is 0.9 by default but can be changed if *TRIMS* has it;

N is the number of lanes in one direction;

f_P is usually 1.0 but can be changed if the drivers are not mostly commuters;

f_{HV} can be estimated from the following procedure:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

where:

P_T and P_R are the percentage of multi- and single unit trucks from TRIMS; and

E_T and E_R are from the following table.

Factor	Type of Terrain		
	Level	Rolling	Mountainous
E_T (trucks and buses)	1.5	2.5	4.5
E_R (RVs)	1.2	2.0	4.0

Capacity, c , is the next we need to find in order to, eventually, calculate the v/c ratio. Capacity is a function of FFS, free-flow speed, according to HCM 2000. If FFS is not directly measured in the field, which is usually the case, the following model is suggested by HCM 2000:

$$FFS = BFFS - f_{LW} - f_{LC} - f_N - f_{ID}$$

where

$BFFS = \text{Min} (PS + 10, \text{Max} (0.88 PS + 14, 70 \text{ if urban and } 75 \text{ if rural}))$ in mph;

PS is the posted speed limit in mph; and

f_{LW} , f_{LC} , f_N , and f_{ID} can be derived from the following tables.

Lane Width (ft)	Reduction in Free-Flow Speed, f_{LW} (mi/h)
12	0.0
11	1.9
10	6.6

Right-Shoulder Lateral Clearance (ft)	Reduction in Free-Flow Speed, f_{LC} (mi/h)			
	Lanes in One Direction			
	2	3	4	≥ 5
≥ 6	0.0	0.0	0.0	0.0
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

Number of Lanes (One Direction)	Reduction in Free-Flow Speed, f_N (mi/h)
≥ 5	0.0
4	1.5
3	3.0
2	4.5

Note: For all rural freeway segments, f_N is 0.0.

Interchanges per Mile	Reduction in Free-Flow Speed, f_{ID} (mi/h)
0.50	0.0
0.75	1.3
1.00	2.5
1.25	3.7
1.50	5.0
1.75	6.3
2.00	7.5

In calculating f_{ID} we do have to figure out the number of Interchanges per Mile first, which may be tricky. Obviously it is not always easy to find out the number of

interchanges in a mile in TRIMS due to data “breakdown” at county boundaries, etc. The number of Interchanges per Mile, nevertheless, can be approximated as:

$$\frac{1}{\text{Distance between Adjacent Interchanges}}$$

If the segment been analyzed does not have any “artificial” boundaries, such as county lines, rural/urban changes, and lane add/drop cutting, then the length of the segment, or *LS*, is the same as the distance between two adjacent interchanges. Even if there were rural/urban or lane add/drop cases, we should be able to find the distance between adjacent interchanges. The only time we will have some challenges is when the county line cuts between two interchanges. For such cases, we would simplify the process by using the following defaults:

$$f_{ID} = 0.0 \text{ for rural and } 1.5 \text{ for urban}$$

We will not use Model 1 for FFS for now. The final FFS results from Models 2 and 3:

$$FFS = \text{Max (FFS from Model 2, FFS from Model 3)}$$

Once we have FFS, the capacity per lane can be calculated as:

$$\begin{aligned} \text{Capacity} &= 2400 \text{ pcphpl, if } FFS \geq 70 \text{ mph} \\ &= 2400 - 10 \times (70 - FFS) \text{ pcphpl, otherwise} \end{aligned}$$

Then the v/c ratio is simply equal to v_p/c . Note that both v_p and c here are on a per lane basis. Make sure they match. At this point if the v/c ratio is larger than 1.0, we can readily assign a LOS of F to the section and move on and save calculation time. Therefore:

IF $v_p/c > 1.0$ THEN

LOS = F

S = 5 mph

EXIT

END IF

We can also calculate the operation speed, S , according to the following procedure:

IF $(3400 - 30FFS) < v_p$ THEN

IF $70 < FFS$ THEN

$$S = FFS - \left[\left(FFS - \frac{160}{3} \right) \left(\frac{v_p + 30FFS - 3400}{30FFS - 1000} \right)^{2.6} \right]$$

ELSE

$$S = FFS - \left[\frac{1}{9} (7FFS - 340) \left(\frac{v_p + 30FFS - 3400}{40FFS - 1700} \right)^{2.6} \right]$$

END IF

ELSE

$$S = FFS$$

END IF

And the density can be calculated also as:

$$\text{Density} = \text{Min} (v_p, c)/S$$

With density, we can calculate the level of service, LOS:

LOS	A	B	C	D	E	F
Density	0 - 11	>11 - 18	>18 - 26	>26 - 35	>35 - 45	>45

Note though, if $v_p/c > 1.0$ then LOS automatically becomes F no matter what density, S, etc. are.

Section II - Multilane Segment Analysis Procedure

INPUT: from TRIMS database:

*AADT, K Factor, D Factor, PHF, Lane Width, Lateral Clearance,
Rural/Urban, Terrain,
Truck Percentage,*

OUTPUT: Operational Speed (S)

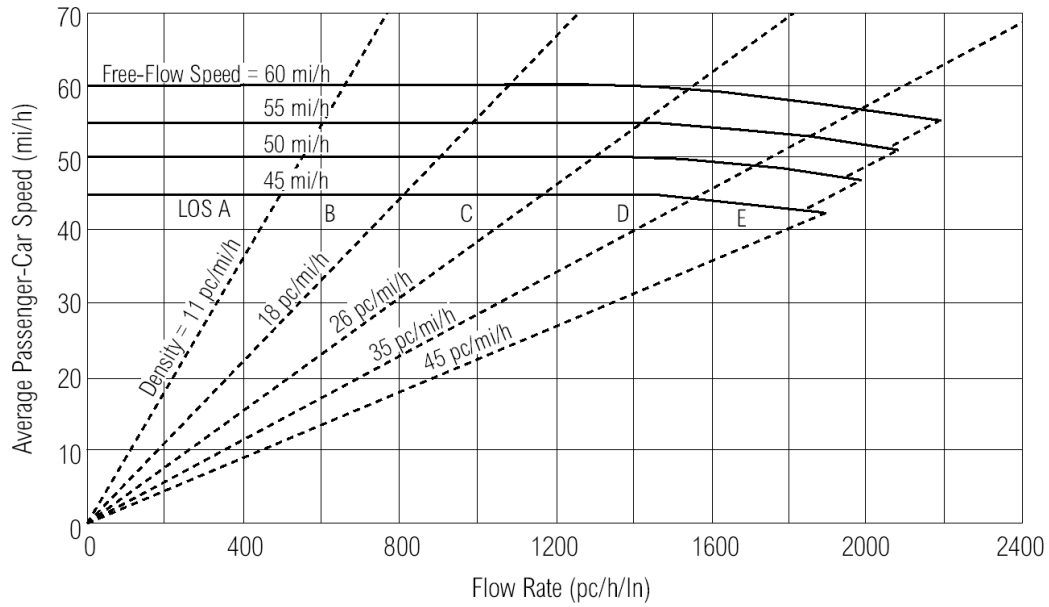
Capacity (c)

v/c ratio

Density (D)

Level of Service (LOS)

This procedure is based on Chapter 21 of HCM 2000. At the center of the methodology is the speed-flow relationship depicted in the figure below. Once the flow rate and the average passenger-car speed are obtained, the LOS can be derived.



Similar to the analysis procedure for freeway segments, we'll first calculate the flow rate (or, more appropriately, demand if the flow rate turns out to be higher than the capacity) entering the segment:

$$V_P = \frac{AADT \times K \text{ Factor} \times D \text{ Factor}}{PHF \times N \times f_{HV} \times f_P}$$

where:

AADT, *K* and *D* factors should come from *TRIMS*;

PHF is 0.9 by default but can be changed if *TRIMS* has it;

N is the number of lanes in one direction;

f_P is usually 1.0 but can be changed if the drivers are not mostly commuters;

f_{HV} can be estimated from the following procedure:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

where:

P_T and P_R are the percentage of multi- and single unit trucks from TRIMS; and

E_T and E_R are from the following table.

Factor	Type of Terrain		
	Level	Rolling	Mountainous
E_T (trucks and buses)	1.5	2.5	4.5
E_R (RVs)	1.2	2.0	4.0

Capacity, c , is the next we need to find in order to, eventually, calculate the v/c ratio. Capacity is a function of FFS, free-flow speed, according to HCM 2000. We have developed a model for implementing HCM 2000.

$$FFS = \text{Max} (45, \text{Min} (60, BFFS - f_{LW} - f_{LC} - f_M - f_A))$$

Where,

FFS is bound between 45 and 60 mph;

$BFFS = -8 \times 10^{-5} PS^3 + 0.0121 PS^2 + 0.3926 PS + 14.718$ mph; and

f_{LW} , f_{LC} , f_M , and f_A can be derived from the following tables.

Lane Width (ft)	Reduction in Free-Flow Speed, f_{LW} (mi/h)
12	0.0
11	1.9
10	6.6

Four-Lane Highways		Six-Lane Highways	
Total Lateral Clearance ^a (ft)	Reduction in FFS (mi/h)	Total Lateral Clearance ^a (ft)	Reduction in FFS (mi/h)
12	0.0	12	0.0
10	0.4	10	0.4
8	0.9	8	0.9
6	1.3	6	1.3
4	1.8	4	1.7
2	3.6	2	2.8
0	5.4	0	3.9

Note:

a. Total lateral clearance is the sum of the lateral clearances of the median (if greater than 6 ft, use 6 ft) and shoulder (if greater than 6 ft, use 6 ft). Therefore, for purposes of analysis, total lateral clearance cannot exceed 12 ft.

Median Type	Reduction in FFS (mi/h)
Undivided highways	1.6
Divided highways (including TWLTLs)	0.0

Access Points/Mile	Reduction in FFS (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
≥ 40	10.0

It is rather difficult to figure out the actual number of access points (intersections as well as driveways) per mile in a segment since such information is not always collected and/or coded in TRIMS. However, if such information is available, one could calculate it with the following equation. In the past, 20 access points per mile has been used when data is not available.

$$\text{Access Points/Mile} = \frac{\text{Nb of Access Points}}{\text{Length of the Segment}}$$

Once we have FFS, the capacity, c, can be calculated as:

$$\begin{aligned}\text{Capacity} &= 2200 \text{ pphpl, if FFS} \geq 60 \text{ mph} \\ &= 2200 - 20 \times (60 - \text{FFS}) \text{ pphpl, otherwise}\end{aligned}$$

Then the v/c ratio is simply equal to v_F/c . Note that both v_F and c are on per lane basis. At this point if the v/c ratio is larger than 1.0, we can readily assign a LOS of F to the section and move on and save calculation time. Therefore:

IF $v_F/c > 1.0$ THEN

LOS = F

S = 5 mph

EXIT

END IF

We can also calculate the operation speed, S, according to the following procedure:

IF $v_F > 1400$ THEN

IF FFS > 55 THEN

$$S = FFS - \left[\left(\frac{3}{10} FFS - 13 \right) \left(\frac{v_p - 1,400}{28FFS - 880} \right)^{131} \right]$$

ELSE IF 55 ≥ FFS > 50 THEN

$$S = FFS - \left[\left(\frac{34}{205} FFS - \frac{219}{41} \right) \left(\frac{v_p - 1,400}{\frac{171}{5} FFS - 1181} \right)^{131} \right]$$

ELSE IF 50 ≥ FFS > 45 THEN

$$S = FFS - \left[\left(\frac{10}{43} FFS - \frac{350}{43} \right) \left(\frac{v_p - 1,400}{33FFS - 1050} \right)^{131} \right]$$

ELSE

$$FFS = 45$$

$$S = FFS - \left[\left(\frac{1}{5} FFS - \frac{56}{9} \right) \left(\frac{v_p - 1,400}{36FFS - 1,120} \right)^{131} \right]$$

END IF

ELSE

$$S = FFS$$

END IF

And the density can be calculated also as:

$$Density = Min(v_p, c)/S$$

With density, we can calculate the level of service, LOS:

<i>LOS</i>	<i>A</i>	<i>B</i>	<i>C</i>	<i>D</i>	<i>E</i>	<i>F</i>
<i>Density</i>	<i>0 - 11</i>	<i>>11 - 18</i>	<i>>18 - 26</i>	<i>>26 - 35</i>	<i>>35 - D_{FFS}</i>	<i>>D_{FFS}</i>

$$D_{FFS} = 0.00133333 FFS^3 - 0.2 FFS^2 + 9.566667 FFS - 102$$

Note though, if $v_F/c > 1.0$ then LOS automatically becomes F no matter what density, S, etc. are.

Section III - Urban Arterial Analysis Procedure

This procedure is for the analysis of urban and suburban streets with traffic signal spacing of 2.0 miles or less. The analysis should be conducted for sections 2 miles or longer, and at least one mile in the downtown area¹. A section is defined as a portion of a roadway where the AADT and/or roadway geometry do not change significantly. The analysis is meant to be performed for directional traffic.

INPUT: from TRIMS database:

*AADT, K Factor, D Factor, PHF, Rural/Urban, Number of Lanes (N),
Signals, Cross*

Streets, Functional Class, Length of Segments

OUTPUT: Average Travel Speed (S_A)

Capacity (c)

v/c ratio

Level of Service (LOS)

The urban arterial analysis procedure is more complicated than the analysis procedures for freeways and multilane highways. It is based on Chapters 10, 15, and 16 of HCM 2000.

¹ The procedure may be applied to shorter segments also.

The nature and purpose of Highway Capacity Manual is for detailed operational assessment. Due to the planning emphasis of TRIMS, certain input parameters for HCM 2000 are not readily available. These parameters include:

Signal Timing Plans

Coordination of Signals along the Roadway

Actuation/Detectors at Intersections

Turning Movements

Public Transit Demand and Service Information

Parking Activities

Pedestrian Demand

Stop Controlled Intersection Information

Roadside Assess Information

Arrival Pattern at Intersections

For planning purposes, EVE will use the following steps to analysis the LOS of urban arterials.

Step 1. Demand Calculation/Estimation

Similar to the analysis procedure for freeway segments, we'll first calculate the flow rate (or, more appropriately, demand if the flow rate turns out to be higher than the capacity) entering the section:

$$V_P = \frac{AADT \times K \text{ Factor} \times D \text{ Factor} \times (1 - P_{LT})}{PHF}$$

where:

AADT, K and D factors should come from TRIMS;

PHF is 0.9 by default but can be changed if TRIMS has it; and

P_{LT} is the percent of left-turning vehicles with a default of 10% where applicable.

Step 2. Arterial Classification

HCM 2000 defines LOS for arterials based on the operational speed on different classes of roadways. Therefore, it is important to classify the arterial type at an early stage of the analysis procedure.

Criterion	Functional Category			
	Principal Arterial		Minor Arterial	
Mobility function	Very important		Important	
Access function	Very minor		Substantial	
Points connected	Freeways, important activity centers, major traffic generators		Principal arterials	
Predominant trips served	Relatively long trips between major points and through-trips entering, leaving, and passing through the city		Trips of moderate length within relatively small geographical areas	
	Design Category			
Criterion	High-Speed	Suburban	Intermediate	Urban
Driveway/access density	Very low density	Low density	Moderate density	High density
Arterial type	Multilane divided; undivided or two-lane with shoulders	Multilane divided; undivided or two-lane with shoulders	Multilane divided or undivided; one-way, two-lane	Undivided one-way, two-way, two or more lanes
Parking	No	No	Some	Significant
Separate left-turn lanes	Yes	Yes	Usually	Some
Signals/mi	0.5–2	1–5	4–10	6–12
Speed limit	45–55 mi/h	40–45 mi/h	30–40 mi/h	25–35 mi/h
Pedestrian activity	Very little	Little	Some	Usually
Roadside development	Low density	Low to medium density	Medium to moderate density	High density

Design Category	Functional Category	
	Principal Arterial	Minor Arterial
High-Speed	I	N/A
Suburban	II	II
Intermediate	II	III or IV
Urban	III or IV	IV

For the purpose of EVE analysis, we'll use TRIMS parameters to determine the functional category. The parameters from TRIMS include:

- *Functional Classification (FC)*
- *Speed Limit (SL)*

<i>FC Code</i>	<i>Procedure for Arterial Type (C_A)</i>
----------------	---

	<i>Determination</i>	
02	$C_A = 1$	
06	$C_A = 2$	
07	$C_A = 3$	
08	IF $SL > 30$ THEN $C_A = 3$ ELSE $C_A = 4$ END IF	Default = 4
09	$C_A = 4$	
12	IF $SL \geq 45$ THEN $C_A = 1$ ELSE $C_A = 2$ END IF	Default = 2
14	$C_A = 2$	
16	IF $SL \geq 40$ THEN $C_A = 2$ ELSE IF $SL \geq 35$ THEN $C_A = 3$ ELSE	Default = 4

	$C_A = 4$ END IF	
17	IF $SL \geq 30$ THEN $C_A = 3$ ELSE $C_A = 4$ END IF	Default = 4
19	$C_A = 4$	Default = 4

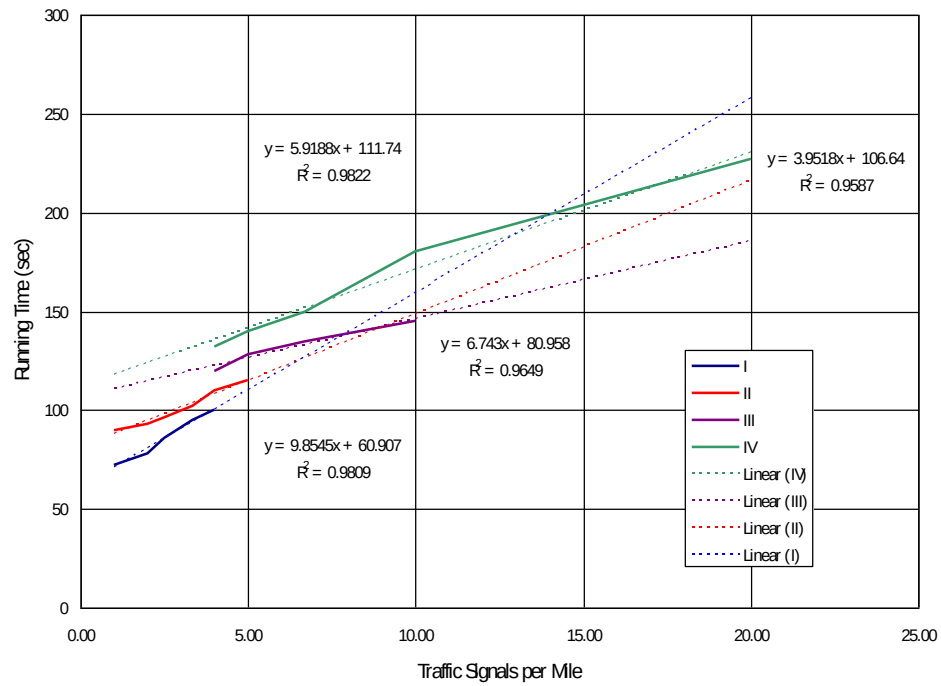
C_A is Arterial Class

Step 3. Average Running Time Estimation

For the arterial segment the running time, T_R , is to be estimated based on arterial class and FFS. Since FFS is rarely available in TRIMS, we'll use the following equations. Note though the Speed_Lmt, or speed limit, term should be greater than 0.0.

Variable SD is signal density within the section and can be calculated as the number of traffic signals divided by the length, L , of the section. Note that these equations were resultant from linear regressions with $R^2 > 0.95$ based on HCM 2000 methodology.

Arterial Class	Equation for T_R (sec)	
I	$T_R = L \times \text{Max}[3600/\text{Speed_Lmt}, (9.85455D+60.907)]$	Eq. Xa
II	$T_R = L \times \text{Max}[3600/\text{Speed_Lmt}, (6.74305D+80.958)]$	Eq. Xb
III	$T_R = L \times \text{Max}[3600/\text{Speed_Lmt}, (3.95185D+106.64)]$	Eq. Xc
IV	$T_R = L \times \text{Max}[3600/\text{Speed_Lmt}, (5.91885D+111.74)]$	Eq. Xd



Based on Akcelik model with Tennessee calibration, running speed for the segment is calculated as:

$$T_{RA} = 3,600 \left\{ \frac{T_R}{3600} + \left(\frac{v}{c} - 1 \right) + \sqrt{\left(\frac{v}{c} - 1 \right)^2 + \frac{v 8J}{c Q}} \right\}$$

where

$$J = 1.00 \text{ and}$$

$$Q = 900 \text{ vpl}$$

Step 4. Capacity Estimation

The capacity, c , of a signalized intersection cannot be calculated without detailed signal timing information. For planning purposes, assumptions are made to make it possible for the analysis.

$$c = N \times s \times \frac{g}{C}$$

N is the number of through lanes in the travel direction, s is the saturation flow rate (1,900 vph), and $\frac{g}{C}$ is green ratio, which is assumed to be 0.5 when the cross road is a state route and 0.65 otherwise. Given such, the capacity would be:

$$c = 1,235 \times N \text{ or } 950 \times N$$

with the unit of vehicles/hour. One may also calculate X or the v/c ratio as:

$$X = \frac{V_P}{C}$$

Step 5. Average Delay Estimation

Signal delay at each intersection has to be estimated. For each intersection, the average delay per vehicle can be calculated as:

$$d = d_1(PF) + d_2 + d_3$$

$$d_1 = \frac{0.5C \left(1 - \frac{g}{C}\right)^2}{1 - \left[\min(1, X) \frac{g}{C}\right]}$$

$$d_2 = 900T \left[(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}} \right]$$

$$d_3 = \frac{1800Q_0(1+u)}{cT}$$

These equations must be simplified, due to the lack of detailed local information, thru the use of following default values.

$$PF = 1.0$$

$$k = 0.5$$

$$I = 1.0 - 0.91X^{2.68}$$

$$T = 0.25 \text{ hour}$$

$$d_3 = 0.0 \text{ sec}$$

$$C = 90 \text{ sec}$$

$$g/C = 0.5 \text{ if the cross street is a state route and } 0.67 \text{ otherwise}$$

Therefore, if the signal is at an intersection with a state route (or higher class road), the average delay per vehicle, in seconds, at this intersection would be:

$$d = \frac{45}{4 - 2[\min(1, X)]} + 225 \left[X - 1 + \sqrt{(X - 1)^2 + \frac{I \times X}{60 \times N}} \right]$$

If the cross road has a lesser classification, then

$$d = \frac{15}{3 - 2[\min(1, X)]} + 225 \left[X - 1 + \sqrt{(X - 1)^2 + \frac{I \times X}{80 \times N}} \right]$$

where $I = 1.0$ if $X > 1.0$ and $I = 1.0 - 0.91X^{2.68}$ otherwise.

Since delay is incurred at all signalized intersections along the section, we define the cumulative delay per vehicle as:

$$d_S = \sum_{\text{All Signals}} d$$

Step 6: Average Travel Speed Estimation

$$S_A = \frac{3600L}{T_{RA} + d_S}$$

where S_A is the average speed in mph and L is the section length in miles.

Step 7: LOS Assessment

Based on the average travel speed and arterial class, LOS can be determined:

Urban Street Class	I	II	III	IV
Range of free-flow speeds (FFS)	55 to 45 mi/h	45 to 35 mi/h	35 to 30 mi/h	35 to 25 mi/h
Typical FFS	50 mi/h	40 mi/h	35 mi/h	30 mi/h
LOS	Average Travel Speed (mi/h)			
A	> 42	> 35	> 30	> 25
B	> 34–42	> 28–35	> 24–30	> 19–25
C	> 27–34	> 22–28	> 18–24	> 13–19
D	> 21–27	> 17–22	> 14–18	> 9–13
E	> 16–21	> 13–17	> 10–14	> 7–9
F	≤ 16	≤ 13	≤ 10	≤ 7

Section IV - Two-Lane Highway Analysis Procedure

This procedure is for the analysis of two-lane roads with no or only isolated traffic signals (spaced at least 2 miles apart). The analysis can be performed for two- or one-directional traffic.

INPUT: from TRIMS database:

AADT, K Factor, D Factor, PHF, Rural/Urban, Terrain, Number of Lanes (N), Signals,

Truck %, RV %, Functional Class, Length of Segments, Posted Speed, Safe Operational Speed (from IHSDM)

OUTPUT: Average Travel Speed (S_A)

Percent Time Spent Following

Capacity (c)

v/c ratio

Level of Service (LOS)

The two-lane highway analysis procedure is an improved and complicated one in comparison with its earlier versions prior to 1997. The procedure to be implemented is based on Chapters 12 and 20 of HCM 2000. The main considerations of this procedure are passing opportunities and the average travel speed. The presence of a passing lane or a directional climbing lane makes a marked difference to the LOS of a

roadway segment and requires different analysis procedures. The following is the procedure for the sections without passing or climbing lanes.

It should be noted that two-lane roads exist in both rural and urban areas. HCM 2000 provides different standards for the two classes of such roadways as following.

Class I – These are two-lane highways on which motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary arterials connecting major traffic generators, daily commuter routes, or primary links in state or national highway networks generally are assigned to Class I. Class I facilities most often serve long-distance trips or provide connecting links between facilities that serve long-distance trips.

Class II – These are two-lane highways on which motorists do not necessarily expect to travel at high speeds. Two-lane highways that function as access routes to Class I facilities, serve as scenic or recreational routes that are not primary arterials, or pass through rugged terrain generally are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning and ending portions of longer trips, or trips for which sightseeing plays a significant role.

Step 1. Identification of a Two-Lane Highway

All of the following should be true to use this procedure.

- *A total of two lanes for the two directions of the roadway (or 3 if the third lane is a passing lane or climbing lane) and*

- *The length of the segment between two signals is larger than 2 miles.*

Step 2. Roadway Classification

Because LOS considerations and analysis procedures are not exactly the same for the two classes of two-lane roads, it is essential to classify a roadway segment first.

Table 4.1

Class	Description
I	Highways on which motorists expect to travel at relatively high speeds, including major intercity routes, primary arterials, and daily commuter routes
II	Highways on which motorists do not necessarily expect to travel at high speeds, including access routes, scenic and recreational routes that are not primarily arterials, and routes through rugged terrain

1. A segment is classified as **Class I** if one of the following conditions is true:

- It has a SR or NHS designation;
- It's FC code is 02, 06, 12, 14, or 16;

2. Otherwise, we classify the segment as **Class II**.

3. There might be exceptions based on the terrain, scenic route designation, and posted speed.

Step 3. Demand Estimation

Similar to other analysis procedures, we'll first calculate the demand for the section:

$$v_p = \frac{AADT \times K \text{ Factor}}{PHF \times f_G \times f_{HV}}$$

where:

v_p is the two-way flow rate in pc/h;

AADT and K factor should come from TRIMS;

PHF is 0.88 for rural area and 0.92 for urban area if unavailable from TRIMS;

f_G is grade adjustment factor; and

f_{HV} is heavy vehicle adjustment factor.

The grade adjustment factor, f_G , is defined as the following in HCM 2000.

Table 4.2

Rang of v_p Two-Way Flow (pc/h)	Type of Terrain		
	Level	Rolling	Mountainous
>0000 - 0600	1.00	0.71	0.57
>0600 - 1200	1.00	0.93	0.85

>1200 - 0000	1.00	0.99	0.99
--------------	------	------	------

It should be noted that since v_p and f_g are both unknown an iterative process has to be used here. We probably start with an f_g value of 1.00, 0.93, or 0.85 depending on the terrain type and solve for v_p .

The heavy vehicle adjustment factor, f_{HV} , is defined as the following in HCM 2000.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Table 4.3

Vehicle Type	Rang of v_p Two-Way Flow (pc/h)	Type of Terrain		
		Level	Rolling	Mountainous
Trucks, E_T	>0000 - 0600	1.7	2.5	7.2
	>0600 - 1200	1.2	1.9	7.2
	>1200 - 0000	1.1	1.5	7.2
RVs, E_R	All	1.0	1.1	3.2

We will use TRIMS truck percentage information for following calculations:

$$P_T = (\text{TRIMS Combo Trucks \%}) + f_{RU} (\text{TRIMS Single Unit Trucks \%})$$

and

$$P_R = (1-f_{RU}) (\text{TRIMS Single Unit Truck \%})$$

where f_{RU} is 1.00 for urban areas and 0.67 for rural areas.

Again, an iterative process has to be used with the initial values for E_T being 1.2, 1.9, or 7.2 depending on the terrain type.

To simplify the iterative process and aid the convergence of f_{HV} , f_G , and v_P values, an alternative and perhaps more “programming friendly” approach is to modify Tables 4.2 and 4.3 into Tables 4.4 and 4.5.

Table 4.4

	Type of Terrain		
	Level	Rolling	Mountainous
$f_G =$	1.00	$1.00 - e^{-\frac{(0.0024v_P + 0.6092)}{P}}$	$1.00 - e^{-\frac{(0.0026v_P + 0.0017)}{P}}$

Table 4.5

Vehicle	Type of Terrain
---------	-----------------

Type	Level	Rolling	Mountainous
Trucks, E_T	$-0.0004v_p + 1.7458$	$-0.0007v_p + 2.6389$	7.2
RVs, E_R	1.0	1.1	3.2

Use the same iterative process with same initial values and stop when $\Delta v_p < 10$ vph

Step 4. Estimate Free Flow Speed (FFS)

Research has established relationships of FFS, posted speed limit, and the safe operational speed from geometric design perspective. We'll use the following procedure, which is a combination of research findings from HCM 2000, IHSDM, and UT studies.

$$FFS = \text{Max}(25, \text{Min}(65, V_{85} - f_{LS} - f_A))$$

The following pseudo-code is provided to explain the calculation process for V_{85} .

IF (PS is available in TRIMS) THEN

$$V_{85} = -8 \times 10^{-5} PS^3 + 0.0121 PS^2 + 0.3926 PS + 14.718 \text{ mph};$$

ELSE IF (D_c and G are available in TRIMS and $D_c \neq 0$) THEN

$$V_{85} = C_A - C_B/R \text{ or}$$

$$V_{85} = C_A - 0.00017455 C_B D_c$$

because $R = D_c/5,729.58$ or $0.00017455 D_c$

Grade Range	C_A	C_B
$-9\% \leq G < -4\%$	63.81	6309.73
$-4\% \leq G < -0\%$	66.24	7607.24
$-0\% \leq G < 4\%$	65.51	7329.62
$4\% \leq G < 9\%$	60.38	5643.43

ELSE

$V_{85} = 60$ mph for Class I or 45 mph for Class II segments

END IF

Where,

PS is posted speed limit in mph;

V_{85} is the 85th percentile speed or base free flow speed in mph;

G is grade in %;

R is radius in feet;

D_c is the degree of curvature, which is stored in TRIMS

C_A is a coefficient for speed prediction model; and

C_B is a coefficient for speed prediction model.

The factor f_{LS} , which represents the reduction in FFS due to lane width and shoulder width restrictions has the following values:

Lane Width (ft)	Reduction in FFS (mi/h)			
	Shoulder Width (ft)			
	$\geq 0 < 2$	$\geq 2 < 4$	$\geq 4 < 6$	≥ 6
$9 < 10$	6.4	4.8	3.5	2.2
$\geq 10 < 11$	5.3	3.7	2.4	1.1
$\geq 11 < 12$	4.7	3.0	1.7	0.4
≥ 12	4.2	2.6	1.3	0.0

The access point factor, f_A , which also causes reduction in FFS is tabulated below.

$$f_A = \frac{\text{Access Point per Mile}}{4}$$

where the default numbers of access points per mile in rural and urban areas is 0 and 10, respectively.

Step 5. Determine Average Travel Speed

Average Travel Speed, ATS , is a function of FFS, flow rate, and existence of passing zone.

$$ATS = FFS - 0.00776v_P - f_{np}$$

where f_{np} is the adjustment factor for no-passing zones. If the percentage of no-passing zone is known, the HCM 2000 table should be used. However, the

percentage no-passing zone information is currently unavailable in TRIMS. Therefore, the following presents an alternative set of default values for f_{np} based on terrain types.

Two-Way Demand Flow Rate, v_p (pc/h)	Type of Terrain		
	Level	Rolling	Mountainous
0	0.00	0.00	0.00
200	0.60	1.90	2.60
400	1.70	3.10	3.90
600	1.60	2.70	3.40
800	1.40	2.15	2.70
1000	1.10	1.80	2.20
1200	0.80	1.40	1.90
1400	0.60	1.05	1.40
1600	0.60	0.95	1.30
1800	0.50	0.85	1.10
2000	0.50	0.75	1.00
2200	0.50	0.75	0.90
2400	0.50	0.70	0.90
2600	0.50	0.70	0.90
2800	0.50	0.65	0.80
3000	0.50	0.65	0.70
3200	0.50	0.60	0.60

Step 6. Decide Percent Time Spent Following

A HCM 2000 based model is adopted for the estimation of PTSF (percent time-spent-following). Note that the range of PTSF is between 0 and 100.

$$PTSF = \text{Max}(BPTSF + f_{n/dp}, 0)$$

BPTSF is the base percent time-spent-following for both directions of traffic. It can be estimated as:

$$\begin{aligned} BPTSF &= 100 (1 - e^{-0.000879 v_p}) \text{ or} \\ &= 100 (1 - 1/e^{0.000879 v_p}) \end{aligned}$$

The combined adjustment factor for directional distribution and no-passing zone percentage or $f_{n/dp}$ involves complicated procedures and calculations. Based on HCM 2000 procedures and simplifications for the state of Tennessee with directional split being 60/40 during the peak hour and 20, 50, and 80% being the no-passing zone percentage for level, rolling, and mountainous terrains.

Note that if Medium_Width > 0, then 100% No Passing equation should be used. On the other hand if Number_of_Lane = 3 and Lane_Type = 0, then 100% Passing Zone equation below should be used.

The following equations will be used for the cases of:

Level Terrain: $f_{n/dp} = 13.422 - 0.0077 v_p + 0.000001 v_p^2$

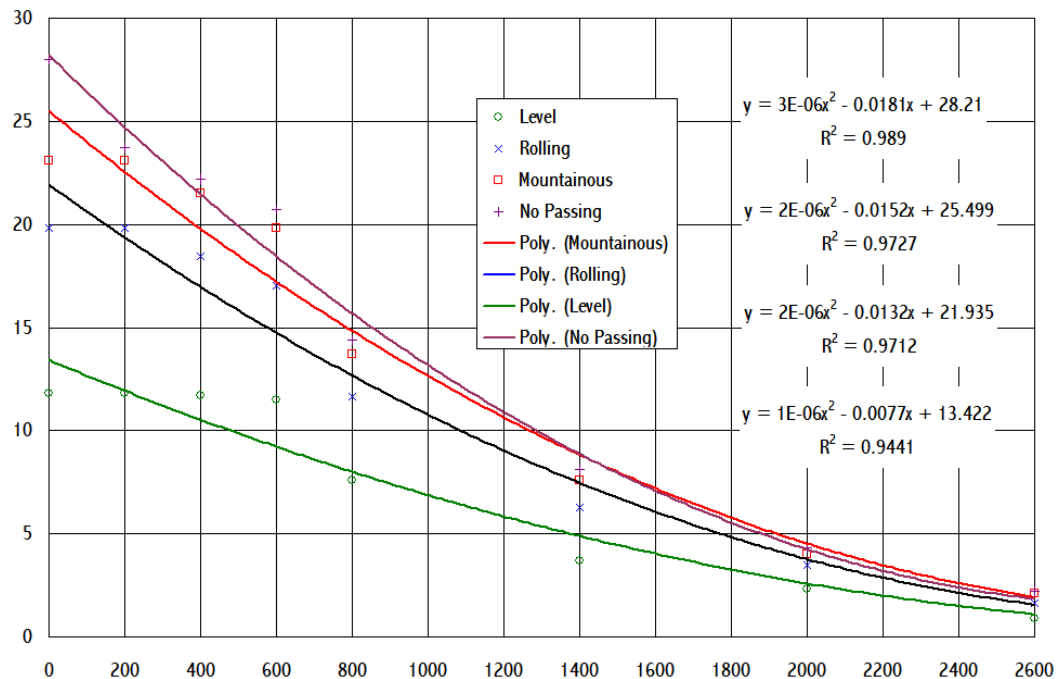
Rolling Terrain: $f_{n/dp} = 21.935 - 0.0132 v_p + 0.000002 v_p^2$

Mountainous Terrain: $f_{n/dp} = 25.499 - 0.0152 v_p + 0.000002 v_p^2$

100% No Passing: $f_{n/dp} = 28.210 - 0.0181 v_p + 0.000003 v_p^2$

100% Passing: $f_{n/dp} = 1.3422 - 0.0008 v_p + 0.0000001 v_p^2$

The following trend-line analysis plot is provided to show the approximation process in deriving these equations.



Step 7. Calculate v/c ratio

The v/c ratio is easily obtained as v_p/c where c is equal to 3,200 vph for two-way traffic and 1,700 vph for one-way one-direction cases.

Step 8. Determine LOS

It should be noted that whenever the v/c ratio is greater than 1.00, the LOS should be F. Otherwise, LOS of the two-lane roadway segment may be determined depending on if it is a Class I or Class II roadway.

For Class I cases, both PTSF, percent time-spent-following, and ATS, average travel speed, values will be used to determine the level of service of the roadway segment. Note that only LOS A through E are listed in the table. LOS F exists only when the volume exceeds the capacity or when v/c ratio is greater than 1.00.

LOS	Percent Time-Spent-Following	Average Travel Speed (mi/h)
A	≤ 35	> 55
B	> 35–50	> 50–55
C	> 50–65	> 45–50
D	> 65–80	> 40–45
E	> 80	≤ 40

For Class II cases, only PTSF will be used. Again, only when v/c ratio is greater than 1.00 would LOS F result.

LOS	Percent Time-Spent-Following
A	≤ 40
B	> 40–55
C	> 55–70
D	> 70–85
E	> 85

APPENDICIES

HIGHWAY FUNCTIONAL CLASS CODE

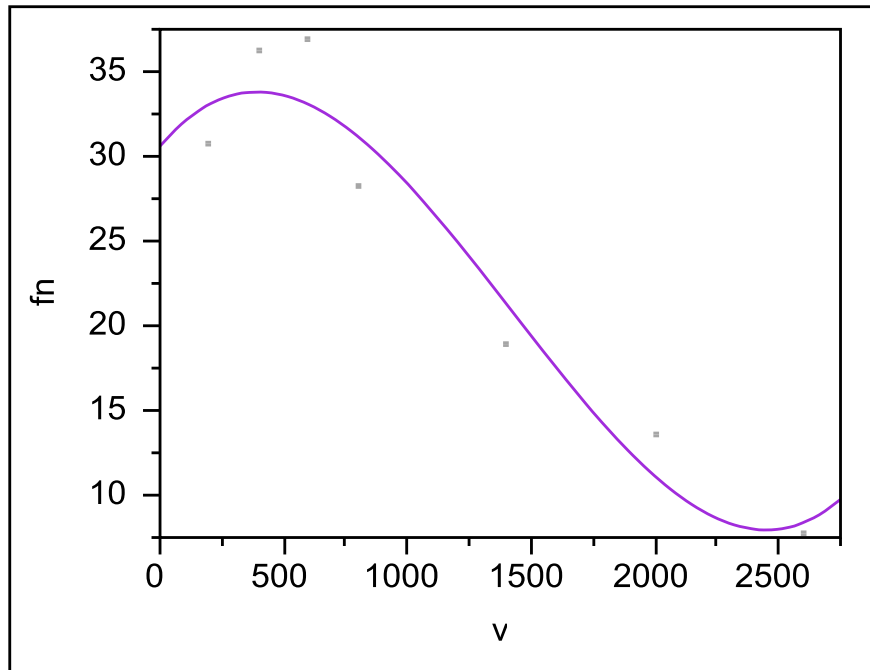
<i>FC Code</i>	<i>Functional Class</i>
<i>Rural Area</i>	
<i>01</i>	<i>Principal Arterial Interstate</i>
<i>02</i>	<i>Other Principal Arterial</i>
<i>06</i>	<i>Minor Arterial</i>
<i>07</i>	<i>Major Collector</i>
<i>08</i>	<i>Minor Collector</i>
<i>09</i>	<i>Local</i>
<i>Urban Area</i>	
<i>11</i>	<i>Principal Arterial Interstate</i>
<i>12</i>	<i>Principal Arterial, Other Freeways & Expressways</i>
<i>14</i>	<i>Other Principal Arterial</i>
<i>16</i>	<i>Minor Arterial</i>
<i>17</i>	<i>Collector</i>
<i>19</i>	<i>Local</i>

HIGHWAY FUNCTIONAL CLASS AADT STATISTICS FROM TRIMS

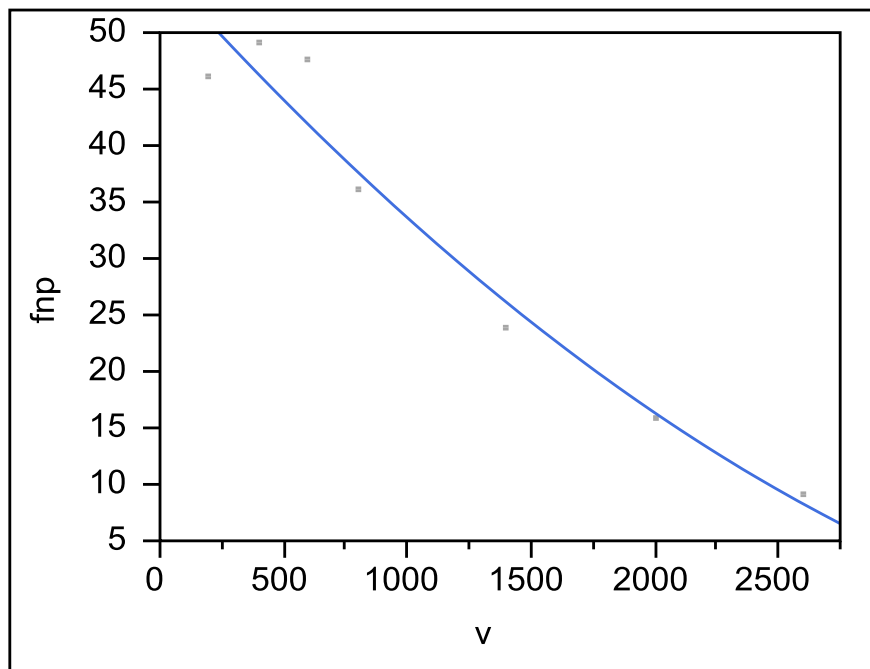
<i>FC</i>	<i># Rcd</i>	<i>% Rcd</i>	<i>Avg_AA DT</i>	<i>Min_AA DT</i>	<i>Max_AA DT</i>	<i>Std_AA DT</i>
<i>?</i>	<i>4</i>	<i>00.02%</i>	<i>3,235</i>	<i>0</i>	<i>6,470</i>	<i>3,735</i>
<i>1</i>	<i>0</i>	<i>00.00%</i>				
<i>2</i>	<i>1083</i>	<i>04.37%</i>	<i>6,274</i>	<i>0</i>	<i>45,490</i>	<i>3,604</i>
<i>6</i>	<i>3407</i>	<i>13.74%</i>	<i>4,820</i>	<i>0</i>	<i>21,680</i>	<i>3,153</i>
<i>7</i>	<i>4894</i>	<i>19.73%</i>	<i>2,313</i>	<i>20</i>	<i>19,530</i>	<i>2,400</i>
<i>8</i>	<i>7603</i>	<i>30.66%</i>	<i>980</i>	<i>0</i>	<i>13,010</i>	<i>1,237</i>
<i>9</i>	<i>18</i>	<i>00.07%</i>	<i>1,364</i>	<i>0</i>	<i>5,110</i>	<i>1,570</i>
<i>11</i>	<i>0</i>	<i>00.00%</i>				
<i>12</i>	<i>17</i>	<i>00.07%</i>	<i>3,304</i>	<i>0</i>	<i>15,480</i>	<i>5,005</i>
<i>14</i>	<i>753</i>	<i>03.04%</i>	<i>9,672</i>	<i>0</i>	<i>30,060</i>	<i>5,453</i>
<i>16</i>	<i>2877</i>	<i>11.60%</i>	<i>6,225</i>	<i>0</i>	<i>33,960</i>	<i>4,475</i>
<i>17</i>	<i>4145</i>	<i>16.71%</i>	<i>2,967</i>	<i>0</i>	<i>29,790</i>	<i>2,758</i>
<i>19</i>	<i>0</i>	<i>00.00%</i>				
<i>SR</i>	<i>11695</i>	<i>47.16%</i>	<i>4,491</i>	<i>0</i>	<i>45,490</i>	<i>3,960</i>
<i>NHS</i>	<i>913</i>	<i>03.68%</i>	<i>6,959</i>	<i>0</i>	<i>45,490</i>	<i>4,395</i>

$f_{n/dp,PTSF}$ Charts

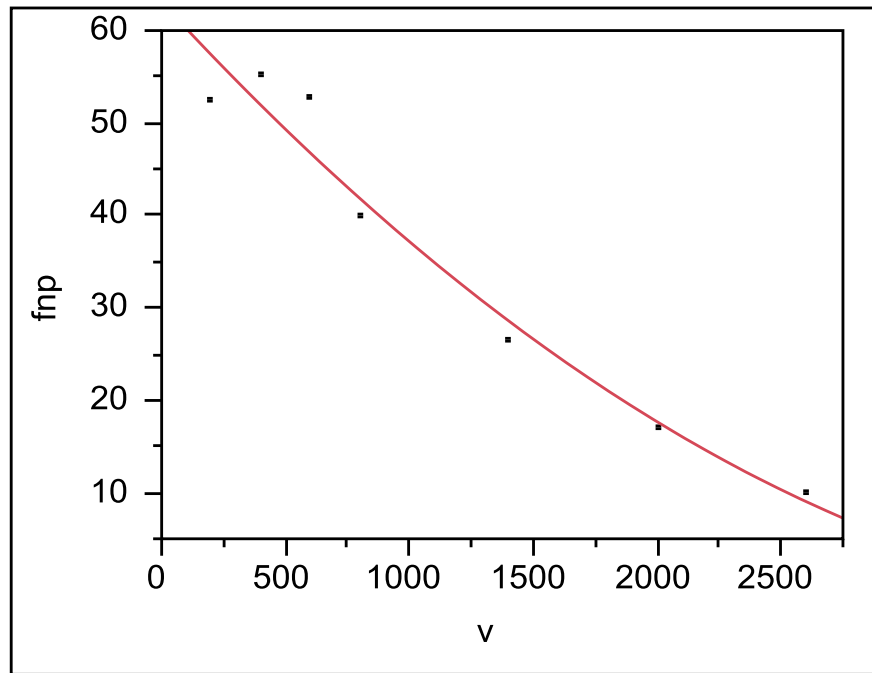
Level Terrain (20% No-Passing Zone)



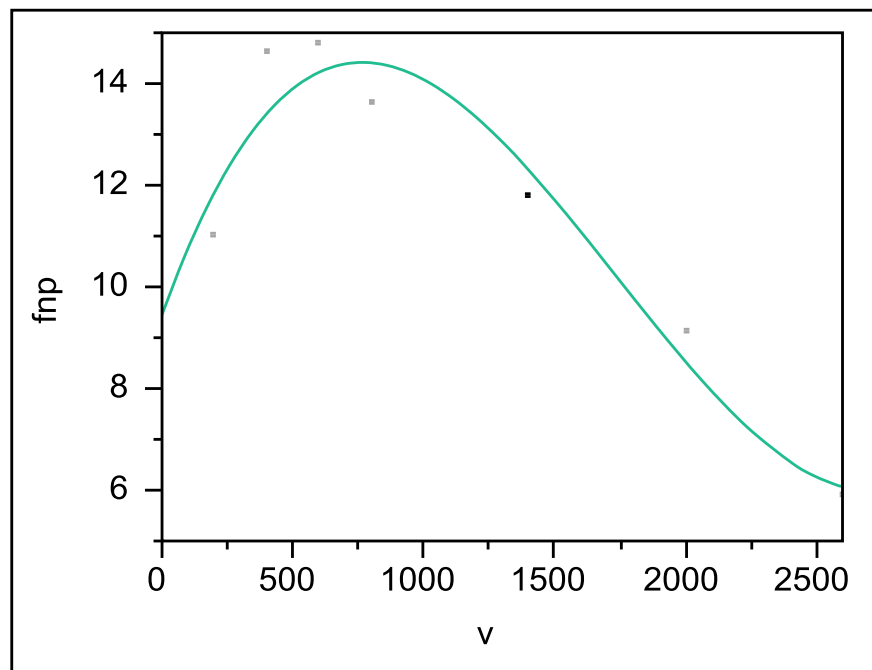
Rolling Terrain (50% No-Passing Zone)



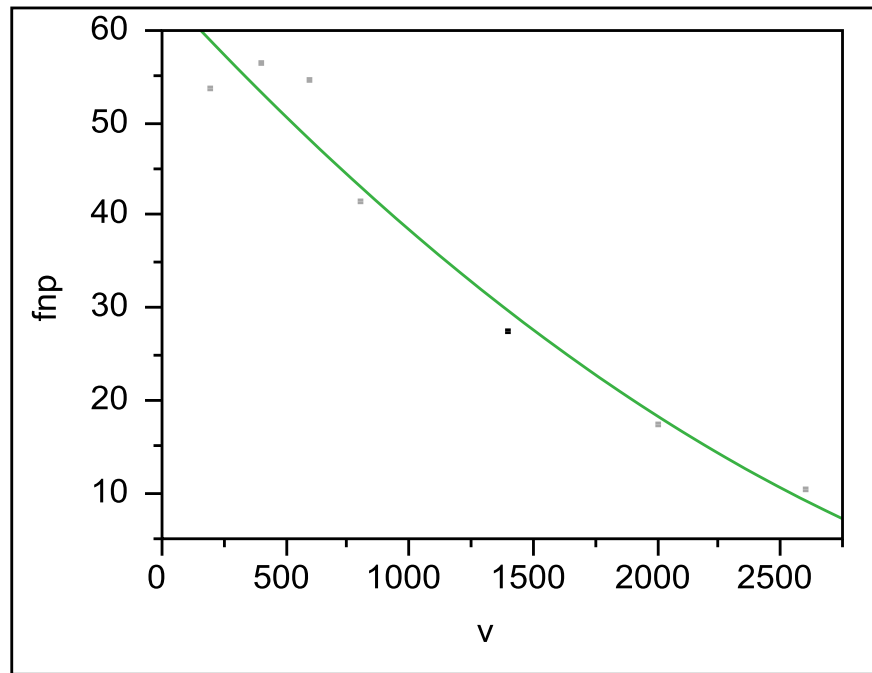
Mountainous Terrain (80% No-Passing Zone)



0% No-Passing Zone



100% No-Passing Zone



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VITA

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