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Methodology Update for Estimating Volume to Service Flow Ratio

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KTC-15-27/PL22-13-1F

Methodology Update for Estimating Volume to Service Flow Ratio

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in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

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16. Abstract
Volume/service flow ratio (VSF) is calculated by the Highway Performance Monitoring System (HPMS) software as an indicator of peak hour congestion. It is an essential input to the Kentucky Transportation Cabinet's (KYTC) key planning applications, such as highway adequacy rating. After the Federal Highway Administration (FHWA) reassessed the HPMS, the standalone HPMS software was no longer supported. This conclusion was based on the 2000 Highway Capacity Manual (HCM2000). As a result, the peak capacity and VSF estimates based on the submittal file are no longer available to KYTC. The calculation of VSF ratios requires AADT, K factor, directional factor, and peak capacity as inputs. Previously, peak capacity was calculated by the HPMS software. The calculation method had limitations; it did not take into consideration certain information that may affect the VSF, such as the availability of a truck-climbing lane. Researchers in this study developed a Microsoft Access-based tool for KYTC that estimated peak capacity and VSF. They adapted the approach to ensure compliance with HCM2010. The tool required pre-processing the HIS extract to eliminate empty cells by either excluding segments with missing data or by imputing the values of the missing item(s). Often, a short segment was misclassified as a multilane highway or an urban two-lane or three-lane facility. Analyses indicated that VSF can provide a modest amount of information on a segment's level of traffic congestion. However, it does not reflect variations in travel time (or speed), which may greatly contribute to user experience on the highway. A post-processing step may also be needed to mitigate the impact of short segments in the input data. The future of VSF as the sole measure of congestion performance should be evaluated.

17. Key Words Volume/service flow (VSF) ratio, Peak hour congestion, Highway Performance monitoring System (HPMS), Highway adequacy rating, Microsoft Access-based tool, Traffic congestion		**18. Distribution Statement**	
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EXECUTIVE SUMMARY

Volume/service flow ratio (VSF) is calculated by the Highway Performance Monitoring System (HPMS) software as an indicator of peak hour congestion. It is an essential input to the Kentucky Transportation Cabinet's (KYTC) key planning applications, such as highway adequacy rating. After the Federal Highway Administration (FHWA) reassessed the HPMS, the standalone HPMS software, based on the 2000 Highway Capacity Manual (HCM2000), is no longer supported, and the peak capacity and VSF estimates based on the submittal file are no longer available to KYTC.

The calculation of VSF ratios requires AADT, K factor, directional factor, and peak capacity as input. Previously, peak capacity was calculated by the HPMS software based upon several HPMS items including cross section, lane configuration, access control, traffic control, and vehicular volume and classification. The procedure has certain limitations in that it does not take into consideration some information that may affect the VSF, such as the availability of a truck-climbing lane.

This study developed a Microsoft Access-based tool by adapting the peak capacity and VSF estimation approach (*1*) to ensure compliance with HCM2010. A process to extract its required input items from KYTC's Highway Information System (HIS) was established. The tool requires pre-processing the HIS extract to eliminate empty cells by either excluding segments with missing data or imputing the values of the missing item(s). A post-processing step may also be needed to mitigate the impact of short segments in the input data. A step-by-step guide to use the tool was included in the Appendix.

Analyses indicated that VSF can provide a modest amount of information on a segment's level of traffic congestion. However, it does not reflect variations in travel time (or speed), which may greatly contribute to user experience on the highway. The future of VSF as the sole measure of congestion performance should be evaluated.

CHAPTER 1   BACKGROUND

Volume/service flow ratio (VSF) is calculated by Highway Performance Monitoring System (HPMS) software as an indicator of peak hour congestion. It is an essential input to the Kentucky Transportation Cabinet's (KYTC) key planning applications, such as the highway adequacy rating. After the Federal Highway Administration (FHWA) reassessed the HPMS, the standalone HPMS software (based on the 2000 Highway Capacity Manual (HCM2000)) is no longer supported, and the peak capacity and VSF estimates based on the submittal file are no longer available to KYTC.

The calculation of VSF ratios requires AADT, K factor, directional factor, and peak capacity as input. Previously, peak capacity was calculated by the HPMS software based upon several HPMS items including cross section, lane configuration, access control, traffic control, and vehicular volume and classification. The procedure has certain limitations in that it does not take into consideration some information that may affect the VSF such as the availability of a truck-climbing lane.

KYTC requires a tool to estimate capacity and VSF to continue generating essential input to its key planning applications. This study developed a Microsoft Access-based tool by adapting the capacity estimation module from an earlier version of HPMS software(*1*) so that it reflects the changes in HCM2010 as well as data items available at KYTC.

This report documents the methodology and adaptation of the capacity estimation procedure. The input data pre-processing and interpretation of the results are also included. The Appendix includes a user's guide to the capacity and VSF estimation tool.

CHAPTER 2   METHODOLOGY

The HPMS's methodology used to estimate VSF requires, first, the estimation of peak capacity. Appendix N of the HPMS Field Manual documents the approach to estimating peak capacity (*1*). The VSF is then determined based on annual average daily traffic (AADT), K factor, and D factor.

The procedure first assigns the facility a type, based on its geometric features and the flow chart shown in Figure 2-1. The HPMS Field Manual outlines detailed procedures for estimating each segment's capacity using standard HPMS data items (*1*).

Figure 1   Capacity Hierarchy
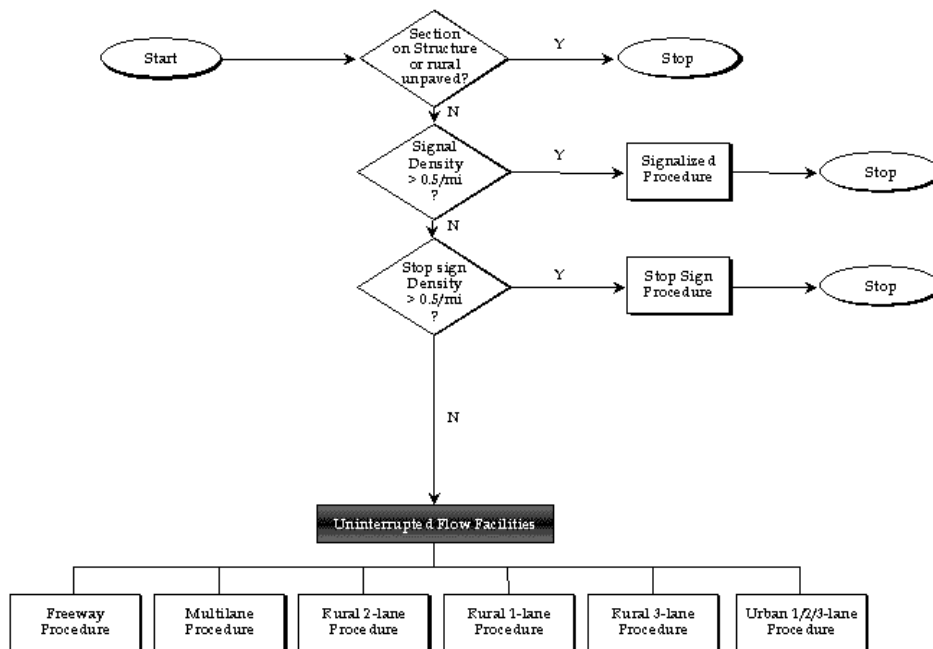
Figure 2-1 Assignment of Facility Type (source:(*1*))

2.1   Existing Method Testing

The research team programmed the methodology (see Appendix N) of the HPMS Field Manual(*1*) into a Microsoft Access database. The team then performed a test to compare the VSF generated by this program to that by the HPMS software using the 2012 submittal file (with 2011 data) provided by KYTC. For all facility types, the difference largely remains within 1%, except for a very small fraction (about 1%) of segments, where the difference can be larger. For example, on one urban street segment controlled by signals, the KTC program generated a peak

capacity that is 17% lower than the HPMS software's estimate. Further investigation was unable to pinpoint the reason for such a large difference. However, such occurrences were rare.

In addition, the research team observed some strange results of the HPMS software. For approximately 50 segments in the submittal file, it appeared that the HPMS software uses the procedure for signal-controlled facilities to estimate capacity — a one-way capacity. But when the software calculates VSF, it still applies the D factor, which is equivalent to treating the capacity as a two-way value. The HPMS software's source code was unavailable for researchers to examine and identify the potential cause.

The study advisory committee (SAC) approved the KTC program in 2014 and directed the team to proceed with adapting the program to achieve HCM2010 compliance. Meanwhile, due to changes to the HPMS submittal process and KYTC's enterprise database, a custom data extraction scheme was developed using KYTC's Transportation Intelligence Gateway (TIG) tool by Bentley Systems.

2.2 Methodology Update

HCM2010 made several updates to the analysis procedure(*2*), which resulted in changes to the capacity estimation method. The capacity estimation procedure discussed in this section was adapted from Appendix N of HPMS Field Manual(*1*). The variables used to calculate estimates follow the convention of the Appendix N and HCM2010, and the exhibits referred to are from the HCM2010.

2.2.1 Freeway

HCM2010 updated the free-flow speed (FFS) estimation method as follows:

$$\text{FFS} = 75.4 - f_{LW} - f_{LC} - 3.22\text{TRD}^{0.84} \tag{1}$$

Where:
FFS = Free-flow speed of the freeway segment (mi/h);
f_{LW} = adjustment for lane width (mi/h);
f_{LC} = adjustment for right-side lateral clearance (mi/h);
TRD = total ramp density (ramps/mi). Since only interchange information is available in the HIS extract, it is assumed that every interchange has two ramps.

The base capacity (pc/h/ln) can be calculated as:
$$\text{BaseCapacity} = 1700 + 10\,\text{FFS; for FFS}\le70 \tag{2}$$
$$= 2400; \text{for FFS}>70$$

Next, the capacity can be determined with the following equation.
$$\text{Capacity} = \text{BaseCapacity} * N * \text{PHF} * f_{HV} * f_p \tag{3}$$

Where:
N = number of lanes in one direction;
PHF = peak hour factor;
f_{HV} = adjustment factor for heavy vehicles;

f_p = adjustment factor for driver population.

2.2.2 Multilane Highway

For multilane highways, the Base FFS calculation equation is slightly different, as shown below.
$$\text{BFFS} = \text{SpeedLimit} + 7, \text{ for posted speed limit} < 50 \text{ mi/h,} \tag{4}$$
$$= \text{SpeedLimit} + 5, \text{ otherwise.}$$

Then, the FFS is calculated as
$$\text{FFS} = \text{BFFS} - f_{LW} - f_{LC} - f_M - f_A \tag{5}$$
Where:

f_{LW} = adjustment for lane width (mi/h);

f_{LC} = adjustment for right-side lateral clearance (mi/h);

f_M = adjustment for median type (mi/h);

f_A = adjustment for access-point density (mi/h).

Base Capacity = 1000 + 20 FFS; for FFS<=60
            = 2200; otherwise.

The capacity (veh/h) can be estimated using the same equation as in freeway procedure (equation (3)).

2.2.3 Rural Two-Lane Highway

HCM 2010 performs capacity estimates for rural two-lane highways separately for each direction. AADT, K factor and D factor are used to evaluate demand volume for each direction, which is subsequently used to determine adjustment factors involved in the procedure.

The impact of passing zones on the capacity of rural two-lane highway is accounted for by including a no-passing zone adjustment component in the estimation equation.
$$\text{One-way Capacity} = 1700 * \text{PHF} * f_G * f_{HV} - V_{NP} \tag{6}$$
Where:
PHF = peak hour factor;

f_G = adjustment for grades;

f_{HV} = adjustment factor for heavy vehicles;

V_{NP} = volume adjustment for no-passing zones (veh/h), it is calculated as $f_{NP} / 0.00776$.

f_{NP} = adjustment for the effect of no-passing zones on average travel speed (mi/h), which can be obtained from Exhibit 15-15.

In order to calculate f_{NP}, the free-flow speed has to be determined first. The FFS calculation equation is given below.

$$FFS = BFFS - f_{LS} - f_A \tag{7}$$

Where:

BFFS = Base FFS (mi/h), which is estimated as the posted speed limit plus 10 mi/h;

f_{LS} = adjustment for lane and shoulder width (mi/h);

f_A = adjustment factor for access-point density (mi/h).

In order to account for the effects of the truck-climbing lane, the percentage of no-passing zone is assumed to be 0 in the uphill direction wherever a climbing lane is present (as per the HIS extract). For the opposite direction, the percentage of no-passing zone for this segment in the HIS extract will be used.

Researchers followed the procedure described above to estimate capacity for each direction, and then the two-way capacity was calculated as the sum of two directional capacities. It should be noted that the two-way capacity should be capped at 3,200 vehicles per hour, which is the upper limit defined by HCM 2010.

2.2.4 Signal Controlled Facility

The following equation is used to calculate the capacity for signalized intersections.

$$Capacity = \sum_i s_i \frac{g_i}{C} \tag{8}$$

Where

s_i = saturation flow rate for lane group i (veh/h), which can be determined by equation(9).

$\frac{g_i}{C}$ = the green ratio for lane group i.

$$s = s_0 * N * PHF * f_w * f_{HV} * f_g * f_p * f_{bb} * f_a * f_{LU} * f_{LT} * f_{RT} * f_{Lpb} * f_{Rpb} \tag{9}$$

Where

s_0 = base saturation flow rate per lane (pc/h/ln);

N = number of lanes in lane group;

PHF = peak hour factor;

f_w = adjustment for lane width;

f_{HV} = adjustment for heavy vehicles;

f_g = adjustment for approach grade;

f_p = adjustment for effect of parking activity adjacent to lane group;

f_{bb} = adjustment for blocking effect of buses that stop within intersection area;

f_a = adjustment for area type;

f_{LU} = adjustment for lane utilization;

f_{LT} = adjustment for left turns in lane group;

f_{RT} = adjustment for right turns in lane group;

f_{Lpb} = pedestrian-bicycle adjustment factor for left turn movements;

f_{Rpb} = pedestrian-bicycle adjustment factor for right turn movements.

2.2.5 Stop Sign Controlled Facility

The following equation is used to determine the capacity for stop-controlled intersections.

$$\text{Capacity} = \frac{\sum_x V_x}{\sum_x \left(\dfrac{V_x}{C_{p,x}}\right)} \tag{10}$$

Where

V_x = flow rate of the x movement in the shared lane (veh/h);

$C_{p,x}$ = potential capacity of the x movement in the shared lane (veh/h) from the following equation.

$$C_{p,x} = CV_{c,x} \frac{e^{-V_{c,x}t_{c,x}/3600}}{1 - e^{-V_{c,x}t_{f,x}/3600}} \tag{11}$$

Where

$CV_{c,x}$ = conflicting flow rate for each movement x (veh/h);

$t_{c,x}$ = critical gap (seconds) for each movement x

$\quad = t_{c,base} + (P_{HV} * t_{c,HV})$

$t_{c,base}$ = base critical gap (seconds) from Exhibit 19-10;

P_{HV} = percent of heavy vehicles in traffic stream during the peak period;

$t_{c,HV}$ = 1.0 for one or two through lane roads; 2.0 otherwise;

$t_{f,x}$ = follow-up time (seconds) for each movement x;

$\quad = t_{f,base} + (P_{HV} * t_{f,HV})$

$t_{f,base}$ = base follow-up gap (seconds) from Exhibit 19-11;

$t_{f,HV}$ = 0.9 for one or two through lane roads; 1.0 otherwise.

2.2.6 Urban 1/2/3 Lane Street

The following equation is used to calculate the capacity for urban 1/2/3-lane streets.

$$\text{Capacity} = s_0 * N * PHF * f_w * f_{HV} * f_p * f_a \tag{12}$$

Where

s_0 = base saturation flow rate per lane (pc/h/ln);

N = number of peak lanes;

PHF = peak hour factor;

f_w = adjustment for lane width;

f_{HV} = adjustment for heavy vehicles;

f_p = adjustment for existence of parking activity;

f_a = adjustment factor for area type

CHAPTER 3   DATA ANALYSIS

The methodology described in Chapter 2 mostly relies on standard HPMS data items, which can be extracted from KYTC's HIS database. In order to preserve the most detailed information available from the HIS, the research team determined that roadway sections should be segmented in a way to ensure the homogeneity of attribute values on a section. Therefore, while the extract contains mostly data items in HPMS format, the segments are usually much shorter than HPMS segments. The required data items for capacity and VSF estimation for different facility types are marked by a dot in Table 3-1.

Table 3-1 Required Data Items for Peak Capacity and VSF Estimation

Items	Facility Type	Freeway	Multilane	Rural One/Two Lane	Rural Three Lane	Signal	Stop	Urban 1/2/3-Lane
Pavement type	•							
Facility type	•	•	•	•	•	•		•
Area type	•	•				•	•	
At grade signal	•							
At grade stop	•		•	•	•			
At grade other			•	•	•			
Section length	•	•	•		•			
Through lanes	•	•	•		•	•	•	•
Median type	•		•		•			
Median width	•		•		•			
Access control	•							
Terrain type		•	•	•	•			
AADT		•	•	•	•	•	•	
K factor		•	•	•	•	•	•	
D factor		•	•	•	•	•	•	
Functional system						•	•	
Peak lanes		•	•			•	•	•
Lane width		•	•	•	•	•		•
Shoulder width		•	•	•	•			
Peak truck percentage		•	•	•	•	•	•	•
Daily truck percentage				•	•			
Interchanges		•						
Speed limit			•		•			
Percent of passing sight distance				•	•			
Truck climbing lane				•	•			
Turning lanes						•	•	
Peak parking						•		•
Green ratio						•		

3.1 Input Pre-processing

Since all data items are not available for all segments of Kentucky highways, the statewide HIS extract contains a number of empty cells, which mostly appeared in K factor, D factor, and truck percentages. The segments with some items unavailable account for approximately 65.8% of all segments in the extract.

After consulting KYTC, the research team developed a set of procedures to impute the extract's missing values as follows:

1. Use the value of the data item on the adjacent upstream or downstream segments, if available, to fill in the missing data item.
2. If no data are available from adjacent segments, either the average of available data by functional classes or the HCM-recommended default is used for imputation. Detailed information can be found in Table 3-2 and Table 3-3.

Table 3-2 Data Source for Imputed Values

Item	Default
AADT	HIS average
K factor	HIS average
D factor	HIS average
Perc_Single_Unit	HIS average
Avg_Single_Unit	HIS average
Perc_Combination	HIS average
Avg_Combination	HIS average
Shoulder_Width	6 ft
Perc_Sight	80, leveled terrain; 50, rolling terrain; 20, mountainous terrain

Access control was among the data items that are unavailable for a number of segments. Roadway characteristics should be examined carefully before filling in an imputed value. In the statewide HIS extract used for this study, 145 segments were missing access control data after implementing the first step above. Because all of these segments were either arterials or collectors, researchers did not assign an access control to them.

Perc_Green (i.e., green time to cycle length ratio) is another item that may be unavailable. If the first step failed to produce an estimated green ratio, 50 was chosen as the default.

Table 3-3 Statewide Default Values

F_ System	AADT	K factor	D factor	Perc_ Single_ Unit	Avg_ Single_ Unit	Perc_ Combination	Avg_ Combination
1	36593	9.02	56.11	0.327	4.649	0.975	19.967
2	7278	8.42	56.08	0.455	6.264	0.525	8.592
6	4659	9.18	57.21	0.539	6.436	0.341	4.763
7	2480	9.72	57.98	0.525	6.07	0.254	2.978
8	834	11.03	56.79	0.575	5.723	0.296	2.494
9	398	12.9	56.64	0.742	7.932	0.289	4.311
11	98876	8.97	56.12	0.257	3.589	0.508	9.08
12	26465	8.33	55.67	0.29	4.791	0.454	8.255
14	18892	9.04	61.11	0.368	4.495	0.328	4.373
16	10034	9.38	59.05	0.425	5.141	0.248	2.767
17	4131	10.34	57.28	0.447	4.94	0.202	2.022
19	1259	11.83	56.59	0.555	4.988	0.346	3.226

After imputation, the Microsoft Access tool can calculate peak capacity and VSF for each segment in the extract. The output file contains identifiers of those segments whose attributes were imputed. The Appendix includes a detailed user's guide for the tool.

3.2 Output Analysis

Results from the HCM2010 compliant program were compared to those generated by the previous version using the same statewide extract (with imputed data). Due to changes in procedure, there were some notable changes to the estimated capacities and VSFs.

3.2.1 Freeway

Differences in segment capacity ranged from -1% to 3.5% for those classified as freeways. Given the change in method of estimating free-flow speed, this difference was expected and considered as reasonable. However, the HCM2010 approach for free-flow speed estimation may not yield realistic results for urban freeway segments with lower speed limits. For example, a segment of I-65 in Louisville area, with LRS_ID 056-I-0065-000 from mile point at 136.631 to mile point 137.318, has a speed limit of 50mph. The FFS calculated based on HCM2010 is 66 mph, while the 85th percentile speed derived from NAVTEQ data was only 51.9mph. The higher HCM-based FFS results in capacity overestimates.

3.2.2 Multilane Highway

Differences in the capacities of most multilane segments were minor, ranging from -3.2% to 3.7%, compared to the results from the previous method. There were also a few segments with a

more significant difference. Major discrepancies in capacity estimates were caused by changes in the methods used to estimate BFFS for segments with a speed limit less than 40 mph. Using the previous method, the BFFS was 40 mph for a multilane facility with a speed limit of 25 mph, but the updated method calculated it at 32 mph.

3.2.3   Rural Two-Lane Highway

The updated procedures generated estimated capacities on some segments that differed significantly from those obtained from the previous procedure (in terms of percentage of change). The change in peak capacity was between -643 and 982 vehicles per hour, while the percentage change ranged from -23% to 165%. Some segments showed significant capacity increases after applying the updated procedure. Most of these segments were located in mountainous areas. The main reason for this is that the new procedure estimates capacity based on direction. As a result, the grade and heavy vehicle adjustment factors for the downgrade direction are greater than those for the upgrade direction. In contrast, the HPMS Appendix N method applies a similar procedure as is applied to the upgrade direction in the HCM2010 approach, but it is for two-way capacity estimation.

Some segments had obvious declines in capacity. These segments typically have very low travel demand, which did not trigger the no-passing adjustment in the previous method. However, the HCM2010 procedure calls for an adjustment value even when the opposing demand flow rate is low.

3.2.4   Signal Controlled Facility and Urban 1/2/3-Lane Streets

The difference in capacity estimates before and after the update ranges from -10% to 10%. The major reason for this is HCM2010's change in lane-width adjustment methods.

3.2.5   Stop Sign Controlled Facility

There was little difference in capacity estimates between the two versions of the methodology.

3.3   Sensitivity Analysis

Sensitivity analysis was used to evaluate the significance of several input variables. Since the calculation involves a number of lookup tables, it is impractical to analytically measure the sensitivity of an input variable. The approach adopted was to use the current value of an input as the baseline and then measure the relative change in peak capacity as a result of this change in the input variable while holding others constant. Critical input variables, such as numbers of through lanes and peak lanes, were not part of this analysis because of their obvious impact on capacity. Table 3-4 lists the input variables analyzed for each facility type. Facilities controlled by stop signs were not listed here because none of the input variables significantly impacted peak capacity.

**Table 3-4 Input Variables for Sensitivity Analysis**

Variable	Freeway	Multilane	Rural Two-Lane	Signalized
AADT	•	•	•	•
K factor	•	•	•	•
D factor	•	•	•	•
Lane width	•	•	•	•
Peak truck percentage	•	•	•	•
Shoulder width	•	•	•	
Interchanges	•			
Percent of passing sight distance			•	
Truck climbing lane			•	
Green ratio				•
Peak parking				•
Left turning lanes				•

3.3.1 Freeway

Figure 3-1 shows the distribution of the relative change in peak capacity as a result of change in several input items. AADT, K factor, and D factor were grouped together because their impacts are of a similar magnitude. As Figure 3-1 (a) and (b) illustrates, a 10% increase or decrease in these factors would produce less than 0.5% increase or decrease in peak capacity for over 95% of the freeway segments.

Figure 3-1 (c) shows the distribution of the change in peak capacity as a result of a 5 point increase in peak truck percentage. Also, according to Figure 3-1 (d), increasing the number of interchanges by two per segment would noticeably decrease peak capacity for many freeway segments.

Other factors such as lane width and shoulder width were also examined. Their impact on peak capacity was minimal.
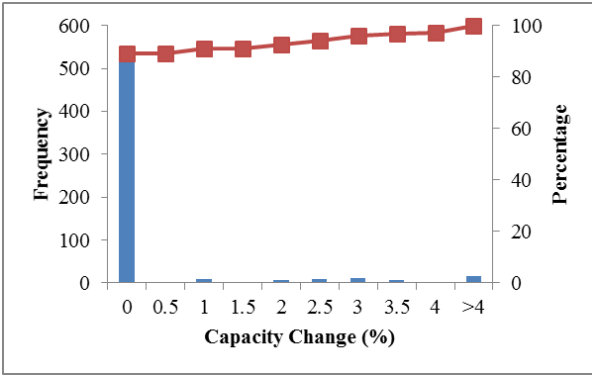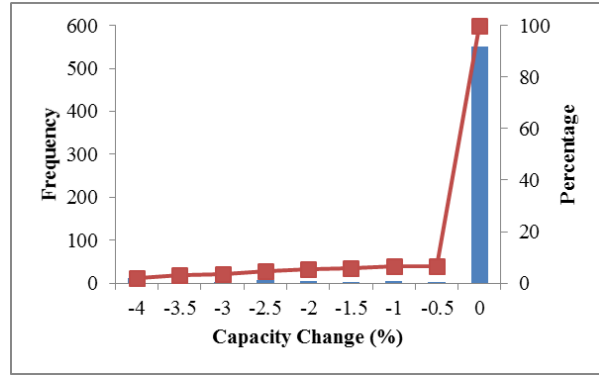
(a) AADT/K factor/D factor increased by 10%   (b) AADT/K factor/D factor decreased by 10%
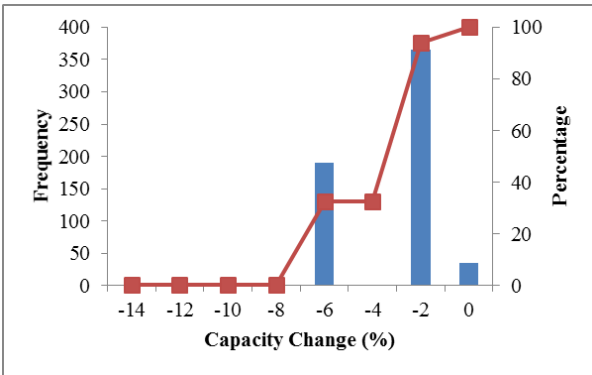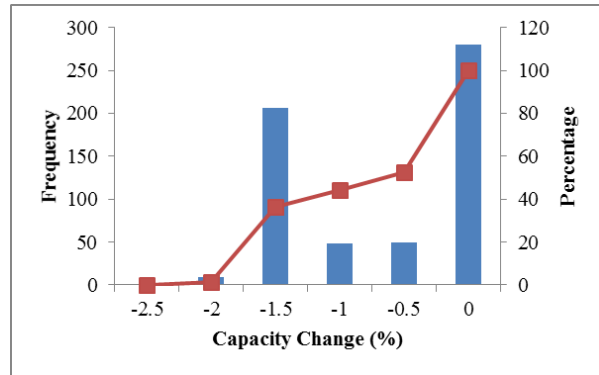
(c) Truck percentage added by 5 percentage point   (d) Interchanges per segment added by 2

Figure 3-1 Sensitivity Analysis on Freeways

3.3.2 *Multilane Highway*

Figure 3-2 illustrates the sensitivity of the peak capacity to changes in several input items on multilane highways. According to Figure 3-2 (a), a 5 point increase in truck percentage would cause at least 2% decline in peak capacity on almost all multilane segments. Also, Figure 3-2 (b) indicates that increasing lane width by 10% would have no impact on peak capacity for over 90% of multilane highways.

As Figure 3-2 (c) reveals, increasing both left-shoulder width and right-shoulder width by 3 ft, yielded a relative change in peak capacity of approximately 1% for over 90% of multilane segments. If both left shoulder width and right-shoulder width were increased by 6 ft, changes in peak capacity would be more dramatic, as seen in Figure 3-2 (d). AADT, K factor and D factor did not significantly influence peak capacity on multilane highways.

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(a) Truck percentage added by 5 percentage point

(b) Lane width increased by 10%

(c) Shoulder width increased by 3 ft

(d) Shoulder width increased by 6 ft
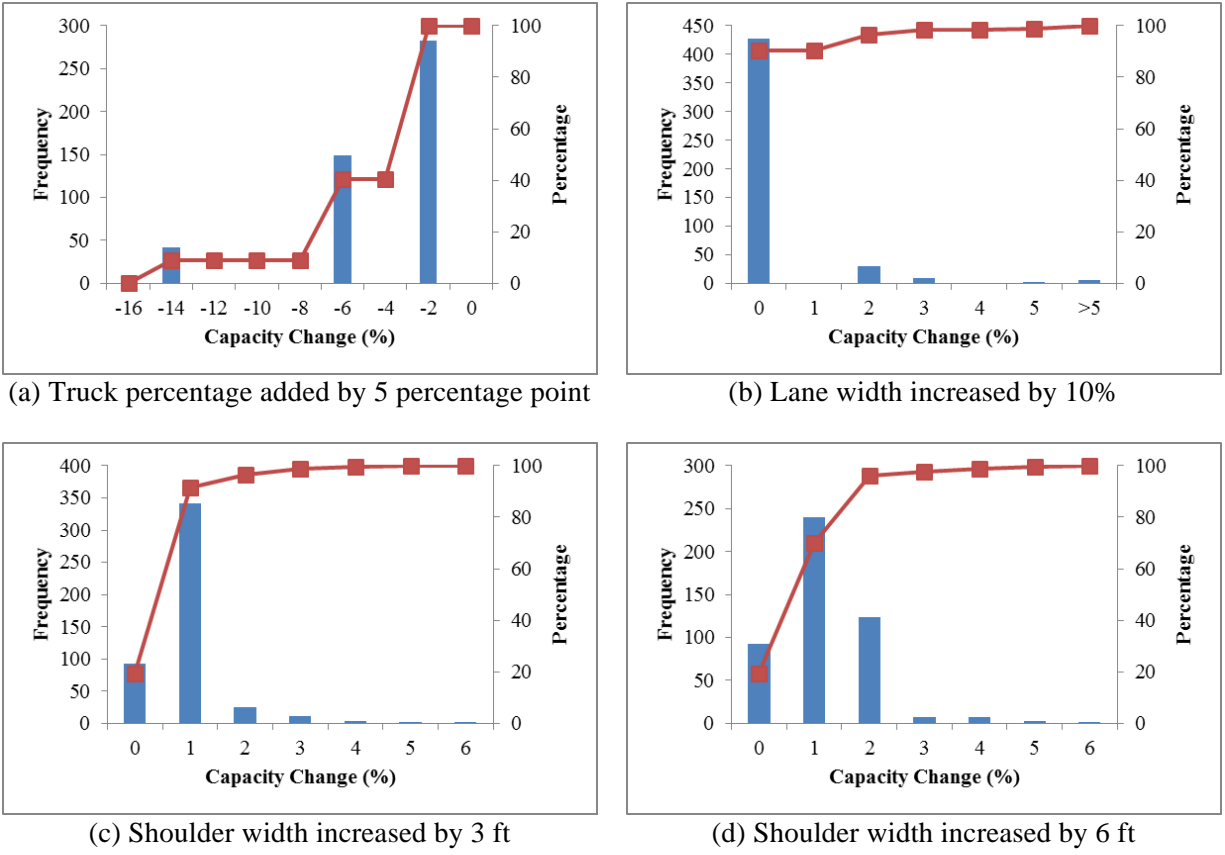
Figure 3-2 Sensitivity Analysis on Multilane Highways

3.3.3 *Rural Two-Lane Highway*

Figure 3-3 shows the effects of adjusting the values of different input items on peak capacity for rural two-lane highways. As with freeways and multilane highways, AADT, K factor and D factor were also grouped together. As shown in Figure 3-3 (a) and (b), a 10% increase or decrease in these factors would increase peak capacity on some segments while decreasing peak capacity on other segments. The research team attributed this to a significant change in the value of the no-passing zone adjustment factor, where the demand flow rate crosses a particular threshold. Nevertheless, vast majority of segments saw change in peak capacity within 1%.

Also, according to Figure 3-3 (c) and (d), the peak capacity of over 60% of the rural two-lane segments would remain unchanged if the percent of passing sight distance were increased or decreased 10%. However, a 5 percentage point increase in peak truck volume would produce a significant decline in peak capacity on rural two-lane highways, as shown in Figure 3-3 (e).

Figure 3-3 (f) and (g) indicate that increasing lane width by 10% or increasing shoulder width by 3 ft could result in either an increase or decrease of peak capacity. The reason for this was traced back to the no-passing zone adjustment factor look-up table. An increase in lane width or shoulder width will generally increase FFS, which may result in a larger or a smaller no-passing zone adjustment factor, depending on the combination of FFS, demand flow rate, and percent of no-passing zones.
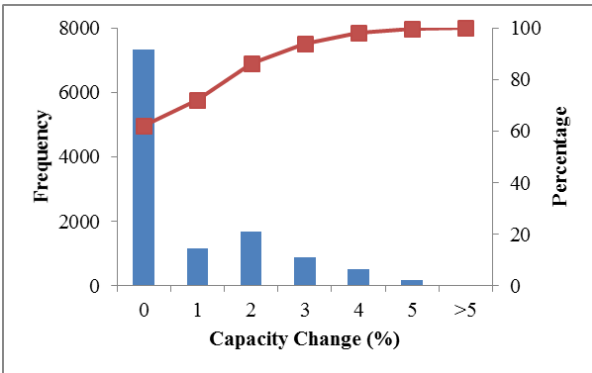
In addition, as shown in Figure 3-3 (h), adding a truck-climbing lane to existing segments currently lacking one would produce at least 10% increase in peak capacity for over 80% of the rural two-lane segments.
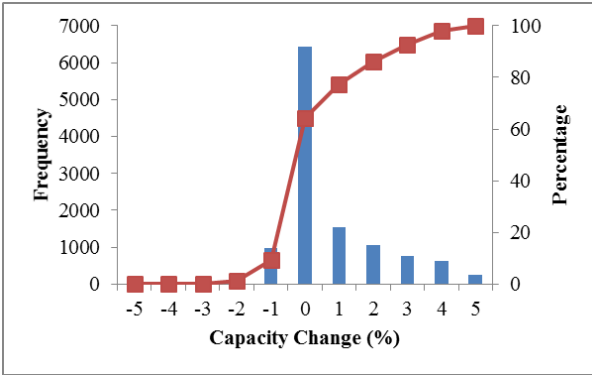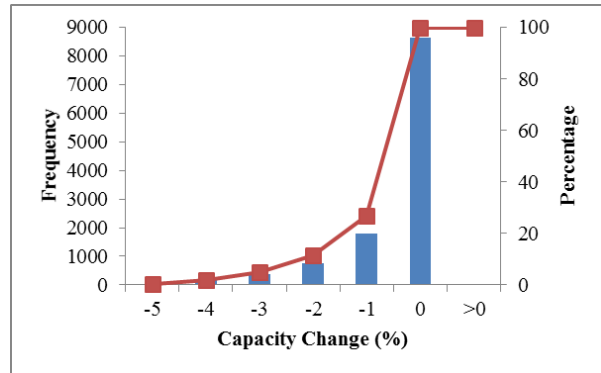

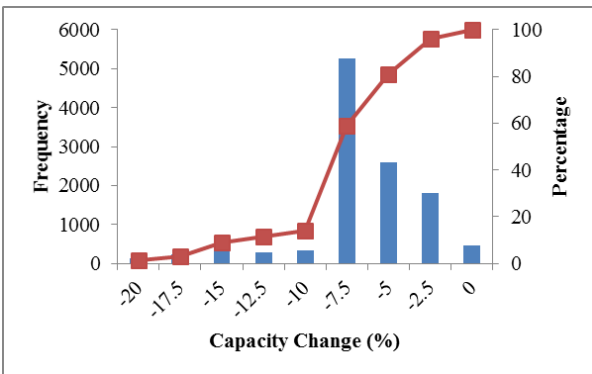(a) AADT/K factor/D factor increased by 10%
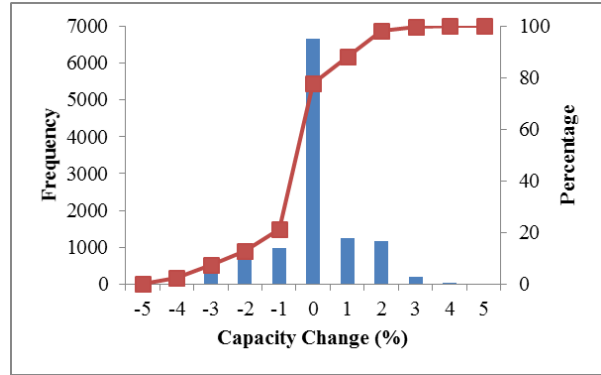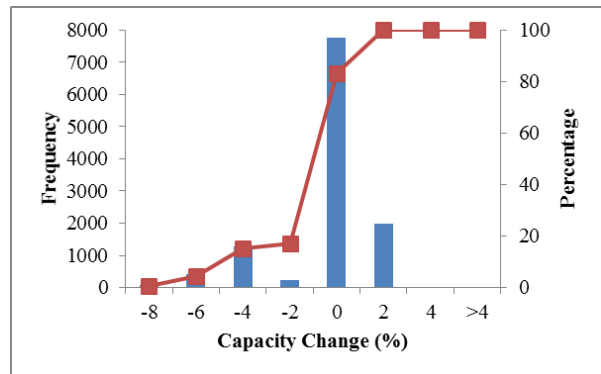
(b) AADT/K factor/D factor decreased by 10%
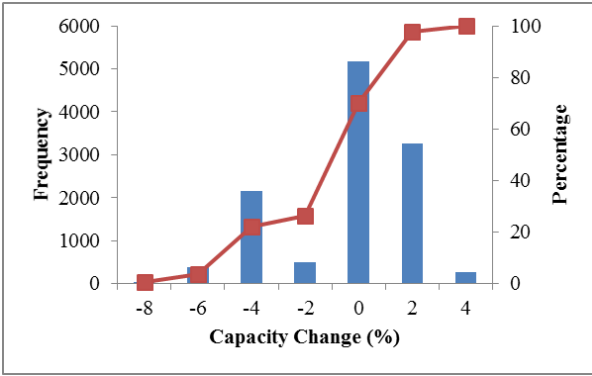
(c) Percent of sight distance increased by 10%
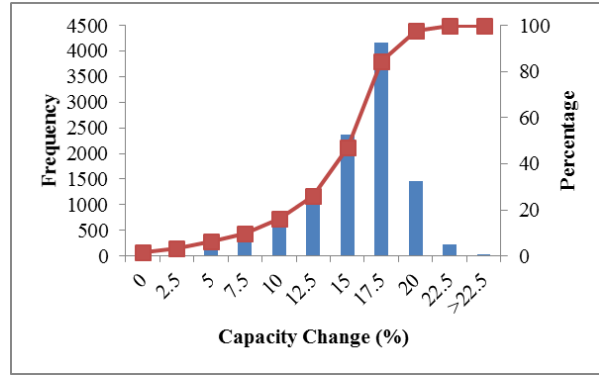
(d) Percent of sight distance decreased by 10%

(e) Truck percentage added by 5 percentage point

(f) Lane width increased by 10%

(g) Shoulder width increased by 3 ft

(h) Truck climbing lane added

Figure 3-3 Sensitivity Analysis on Rural Two-Lane Highways

### 3.3.4 Signal Controlled Facility

Figure 3-4 shows the distribution of the relative change in peak capacity due to changing several input items on signal controlled facilities. As illustrated in Figure 3-4 (a) and (b), increasing or decreasing the green ratio by 10% would significantly change the peak capacity for signalized segments. Figure 3-4 (c) indicates a 5 percentage point increase in peak truck percentage would decrease peak capacity by 4-5% on all the signalized segments. Also, increasing lane width by 10% would increase the peak capacity by 4-5% on about half of the signalized segments, which is shown in Figure 3-4 (d).

Figure 3-4 (e)-(h) capture the effects of different parking activities on peak capacity. As Figure 3-4 (e) and (f) reveal, switching from a no parking scenario to either one-sided or two-sided parking would significantly lower peak capacity. The effect of switching from one-sided parking to two-sided parking would be less significant, as Figure 3-4 (g) indicates. Figure 3-4 (h) shows that over 90% of the segments would gain more than 6% of peak capacity by moving from the existing one-side parking to prohibiting it.

Figure 3-4 (i)-(k) demonstrate how left-turn signals impact peak capacity on signalized segments. As shown in Figure 3-4 (i), permitting left turns at signalized intersections where they were not previously allowed would cause a less than 5% decrease in peak capacity. Adding an exclusive left-turn lane would significantly increase peak capacity (see Figure 3-4s (j) and (k)).
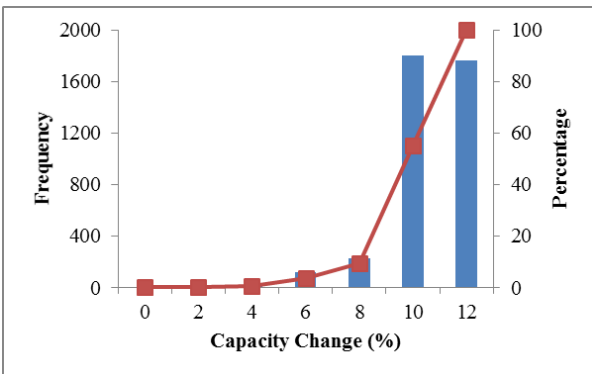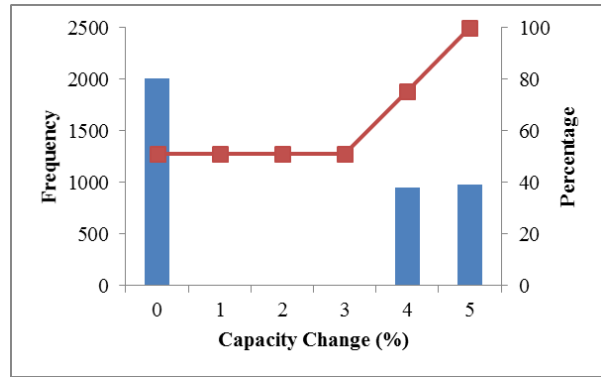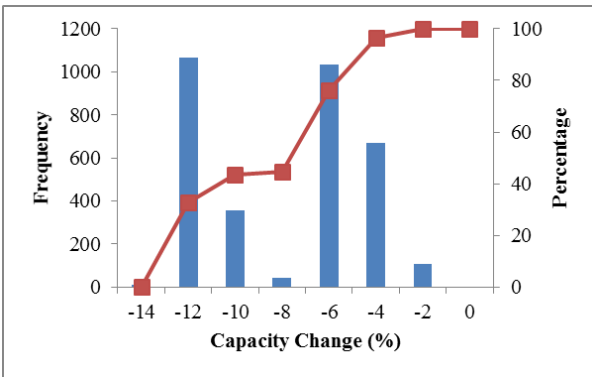

(a) Green ratio increased by 10%
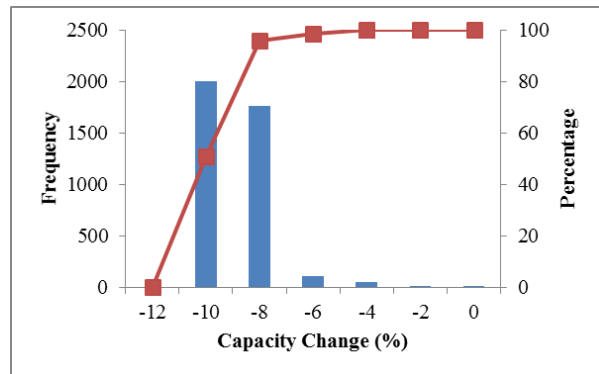
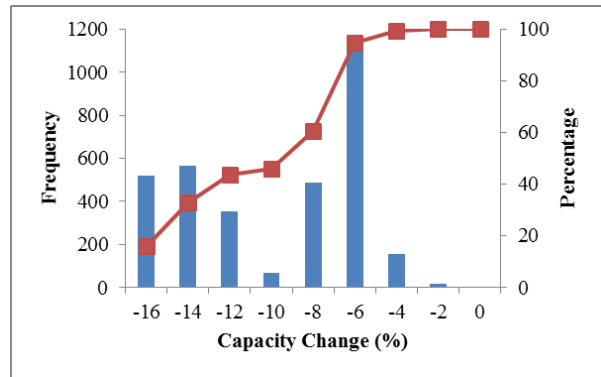
(b) Green ratio decreased by 10%

16
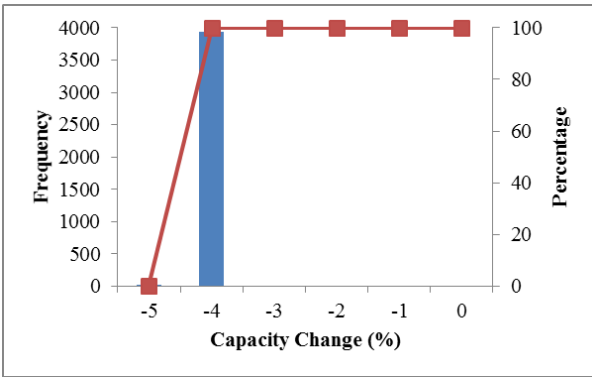
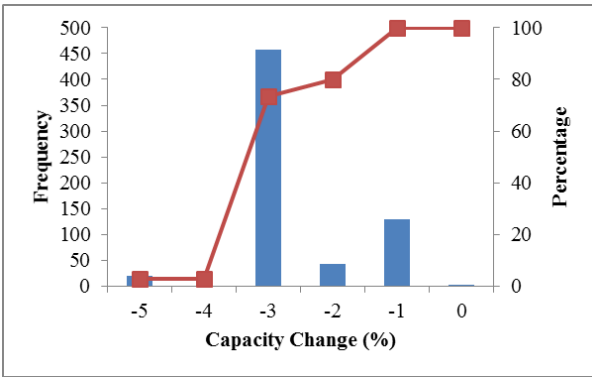(c) Truck percentage added by 5 percentage point
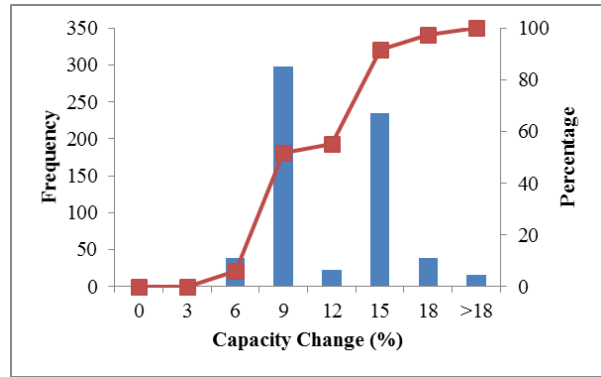

(d) Lane width increased by 10%

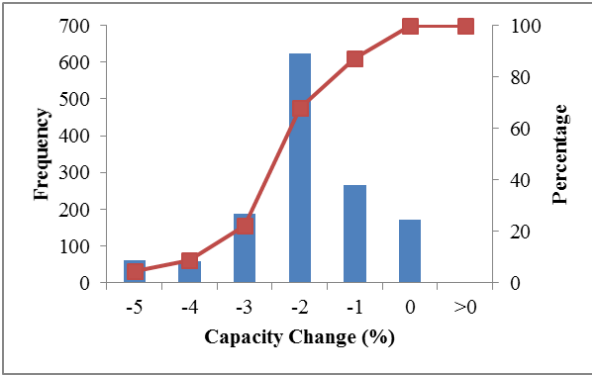(e) Change from no parking allowed to one side parking

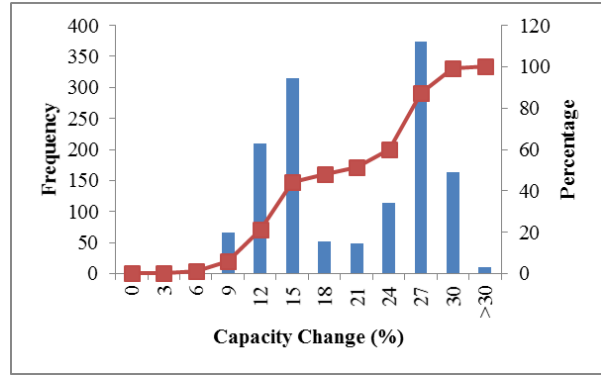(f) Change from no parking allowed to two side parking

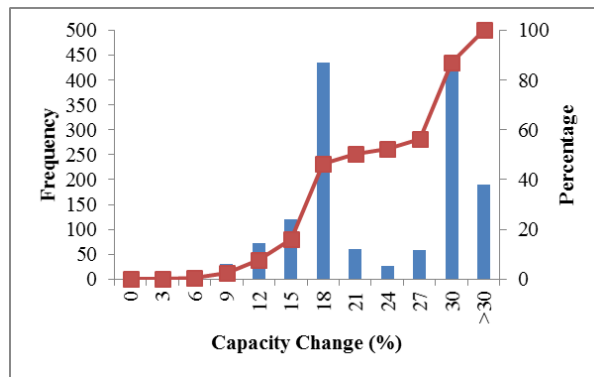(g) Change from one side parking to two side parking

(h) Change from one side parking to no parking allowed

(i) Change from no left turn permitted to left turn permitted

(j) Change from no left turn permitted to left turn permitted with an exclusive left turn lane

(k) Change from left turn permitted to left turn permitted with an exclusive left turn lane

Figure 3-4 Sensitivity Analysis on Signal Controlled Facilities

3.4   Impact of Segment Length and Post-processing

A consequence of integrating multiple asset features and preserving the homogeneity of feature values was the creation of very short segment lengths. Using short segment lengths increases the probability of misclassifying a facility type when following the procedure delineated in Figure 2-1.

For example, the North Broadway corridor in Lexington is an urban arterial with four through lanes which has a number of signal-controlled intersections. There are four segments, with lengths ranging from 0.01 mile to 0.29 mile, that lack traffic control devices. The method outlined in Figure 2-1 would classify them as multilane highways (shown in red color Figure 3-5). In reality, the operation on these segments is contingent on the effects signalized intersections have on adjacent segments.
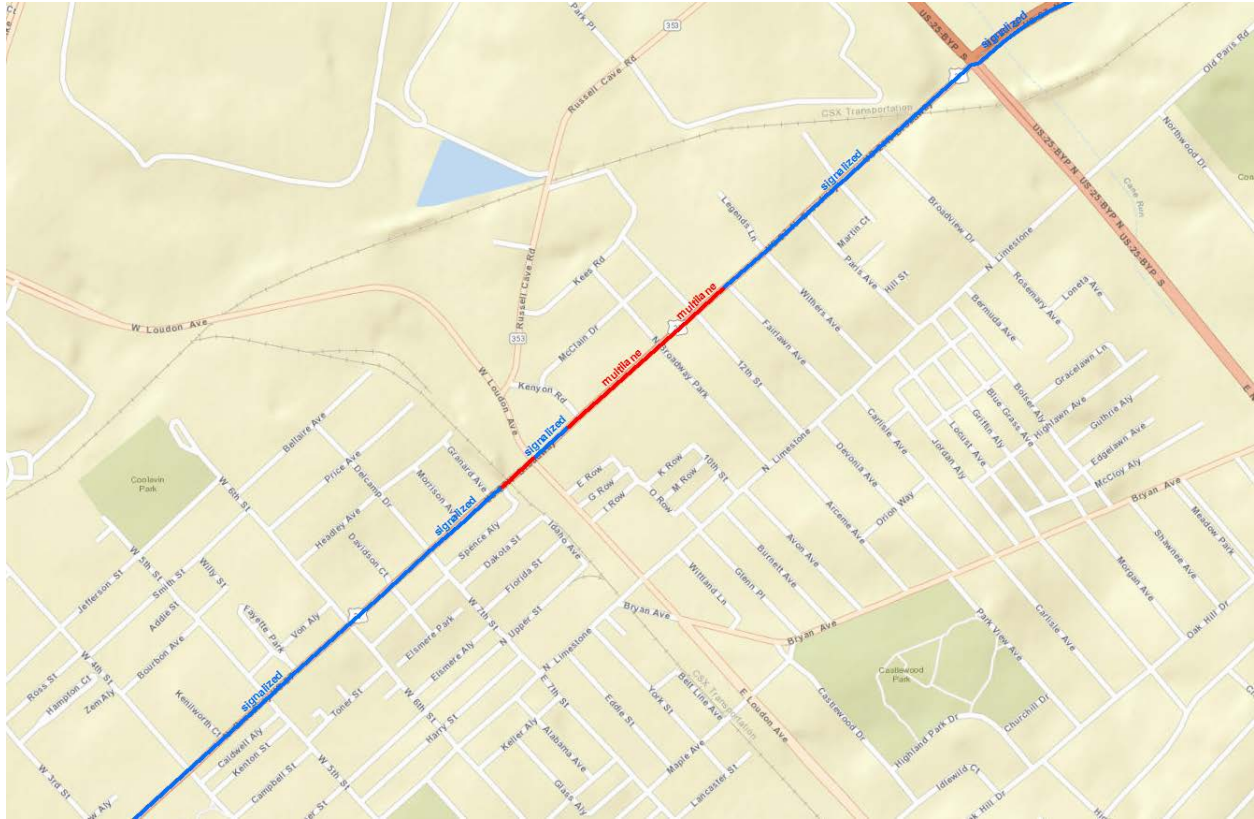
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Figure 3-5 North Broadway segment classification

To mitigate the effects of misclassifications, the research team developed a post-processing procedure based on HCM guidelines related to segment length. HCM indicates that the spacing of traffic control devices, including signals and stop signs, should be at least two miles to ensure the facility will be classified as multilane or rural two-lane highways. Based on this rule, segments like those shown in Figure 3-5 were reclassified as signal (or stop-sign) controlled facilities. The capacity of their adjacent intersections then replaced the estimated capacity at the current segments. Applying this rule impacted approximately 50% of segments previously classified as multilane highway and 8% of those previously classified as rural two-lane highway. KYTC officials noted that customizing the HIS extract process will significantly reduce, but not eliminate, misclassifications.

3.5 VSF as a Performance Measure

VSF has been traditionally used in planning application to measure congestion. With the availability of travel time data increasing, there is a concern over whether VSF establishes a complete picture of congestion. Using the level of service (LOS) ranges defined in the HCM, researchers mapped the relationship between VSF and LOS for uninterrupted flows based on NCHRP Report 387 (*3*).

Meanwhile, a simple mapping between the travel time index (TTI) and LOS was developed for facilities carrying uninterrupted flows. This was based on the assumption of linear speed-density relationship specified in Greenshield's model (*4*):

$$u = u_f \left(1 - \frac{k}{k_j} \right) \tag{13}$$

$$k_{cap} = \frac{k_j}{2} \tag{14}$$

Where:
u = average speed (mi/h);
u_f = the free-flow speed (mi/h);
k = density (pc/mi/ln);
k_j = the jam density (pc/mi/ln);
k_{cap} = the density when at capacity (pc/mi/ln).

TTI is defined as the ratio between the average travel time and free-flow travel time. This can also be written as the ratio between the free-flow speed and average speed, (i.e., u/u_f). Therefore,

$$TTI = \frac{2k_{cap}}{2k_{cap} - k} \tag{15}$$

The threshold density value of 45 pc/mi/ln between LOS E and LOS F, which is used in the HCM, can be treated as k_{cap}. Based on all threshold density values between LOS, the TTI ranges corresponding to each LOS can be estimated. For example, according to the HCM, the threshold density value between LOS A and LOS B is 11 pc/mi/ln on freeways. Therefore, substituting k with 11 in equation (15) renders the threshold value for TTI-based LOS A and LOS B as $2 \times 45 / (2 \times 45 - 11) = 1.14$.

For rural two-lane highways, the HCM's definition of percent of FFS (i.e., ratio between average speed and free-flow speed) can be used to obtain the TTI threshold corresponding to the LOS definition. Since the definition of LOS F is when demand exceeds capacity — which is same for freeways and multilane highways — the same TTI value is used as the threshold for LOS F.

Table 3-5 Linking Measures of Congestion

LOS	HCM Measures				VSF	TTI	
	Freeway	Multilane		Rural two lane	Uninterrupted facilities	Freeway/ Multilane	Rural two lane
	Density (pc/mi/ln)	FFS (mi/h)	Density (pc/mi/ln)	Percent of FFS			
A	<=11	All	<=11	>91.7	<0.6	<1.14	<1.09
B	>11-18	All	>11-18	>83.3-91.7	>=0.6-0.7	>=1.14-1.25	>=1.09-1.2
C	>18-26	All	>18-26	>75.0-83.3	>=0.7-0.8	>=1.25-1.41	>=1.2-1.33
D	>26-35	All	>26-35	>66.7-75.0	>=0.8-0.9	>=1.41-1.64	>=1.33-1.5
E	>35-45	60	>35-40	<=66.7	>=0.9-1.0	>=1.64-2	>=1.5-2
		55	>35-41				
		50	>35-43				
		45	>35-45				
F	>45 or v/c ratio>=1	v/c ratio >=1 or		v/c ratio>=1	>=1.0	>=2	>=2
		60	>40				
		55	>41				
		50	>43				
		45	>45				

3.5.1 Freeway Example

Analysis of selected corridors indicated that while VSF and LOS were largely in agreement, there were some concerns about whether VSF accurately reflects the level of congestion. For example, a 6-lane segment on I-264 between mile point 22.366 and mile point 22.696 in Louisville area has an AADT of 58,500 vehicles and a peak capacity of 6,457 vehicles per hour. Given a VSF value of 0.45, the segment's peak hour LOS would be A. However, the TTI-based LOS is E. Figure 3-6 shows — for this segment — speed distribution on a typical weekday. The distribution is bimodal. The left-hand peak represents a congested/unstable state, while the right-hand peak represents an uncongested state. The large speed differential indicates unreliable travel time. However, VSF-based performance analysis does not capture these dynamics.
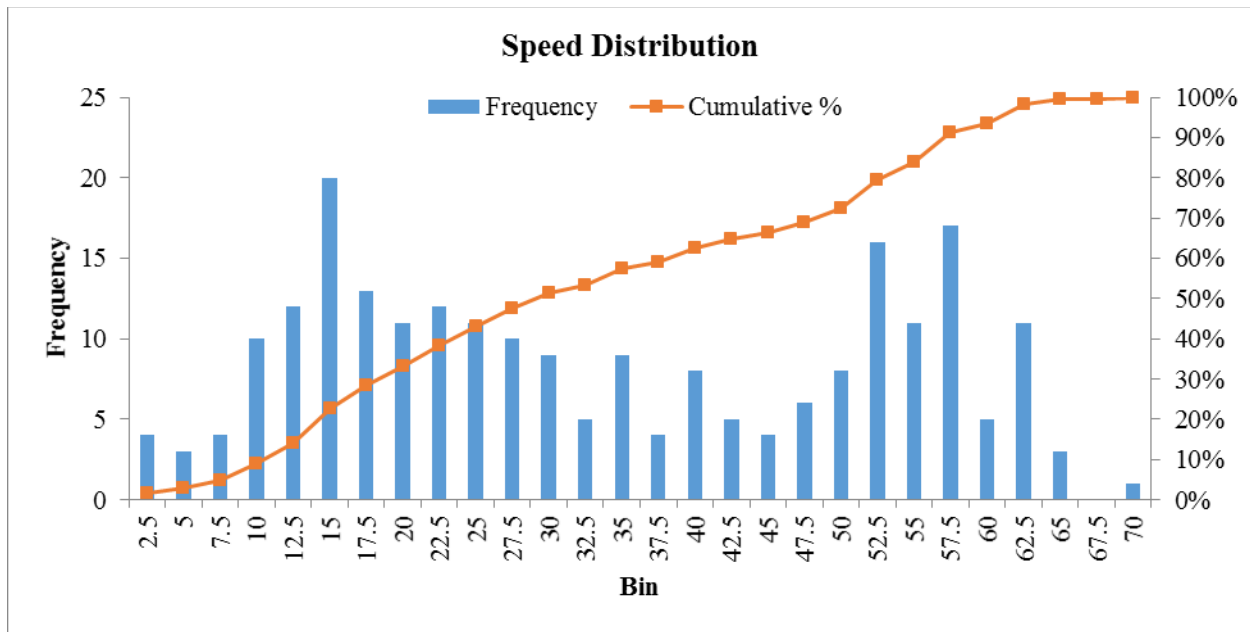
Figure 3-6 I-264 Speed Distribution (MP22.366-MP22.696)

3.5.2 Multilane Highway Example

Researchers selected a section of US 60 in Lexington between mile points 0 and 1.91 as an example of a multilane highway. The FFS of the segment is 58.71 mph and peak capacity is 3,803 vehicles per hour. With an AADT of 38,407 vehicles, the peak period VSF would be 0.58. Looking at Table 3-5, this segment's LOS would be A. However, if one follows the HCM procedure, the LOS would be C because the peak period traffic density is 20.86 passenger cars per mile per lane. While the VSF-based approach only began to differentiate the service quality when flow rate exceeds 60% of capacity, the HCM-based LOS analysis was more suited to distinguishing service quality at relatively low flow levels.

3.5.3 Rural Two-Lane Highway Example

On rural two-lane highways, LOS is defined based on PFFS, which indirectly reflects the flow rate. According to the HCM methodology, LOS is defined more conservatively — the HCM assumes service quality deteriorates quicker on rural two-lane highways even when demand is far less than capacity. For instance, the LOS on a segment of KY-70 in Edmonson County between mile points 11.748 and 12.388 would be A based on a VSF of 0.36. However, it would be LOS D according to HCM analysis.

The FFS on this segment is 39.66 mph, and its peak capacity is 2,302 vehicles per hour. The bi-directional demand flow rate is 962 passenger cars per hour. Accordingly, the adjustment of flow rate for average travel speed is 7.47 mph. Taking the no-passing adjustment into account, the calculated average travel speed is 29.59 mph, and PFFS is 0.75, which means the LOS is D when following the HCM. Moreover, the NAVTEQ speed data indicated that TTI on this segment

should be 1.202, and therefore, the service quality would be C based on TTI ranges shown in Table 3-5.

3.5.4 Signal Controlled Facility Example

For the signal controlled corridor, a different approach was used to compare the VSF with the NAVTEQ speed data. The Akcelik function was used to convert the calculated VSF to average speed so that the comparison could be performed between the same data type. The conversion equation is (*5*):

$$S = L / \left[L/FFS + 0.25T\left(x - 1 + \sqrt{(x-1)^2 + \frac{16*J*x*L^2}{T^2}} \right) \right] \qquad (16)$$

Where:
S = the average segment speed;
L = segment length;
FFS = free-flow speed, which uses speed limit in this case;
X = demand-to-capacity ratio;
T = duration of analysis period, which is set to 1 hour in this study;
J = the delay parameter, which can be calibrated if the speed at capacity is known.

Main Street in Frankfort, KY was selected as an example. The corridor map is shown in Figure 3-7.
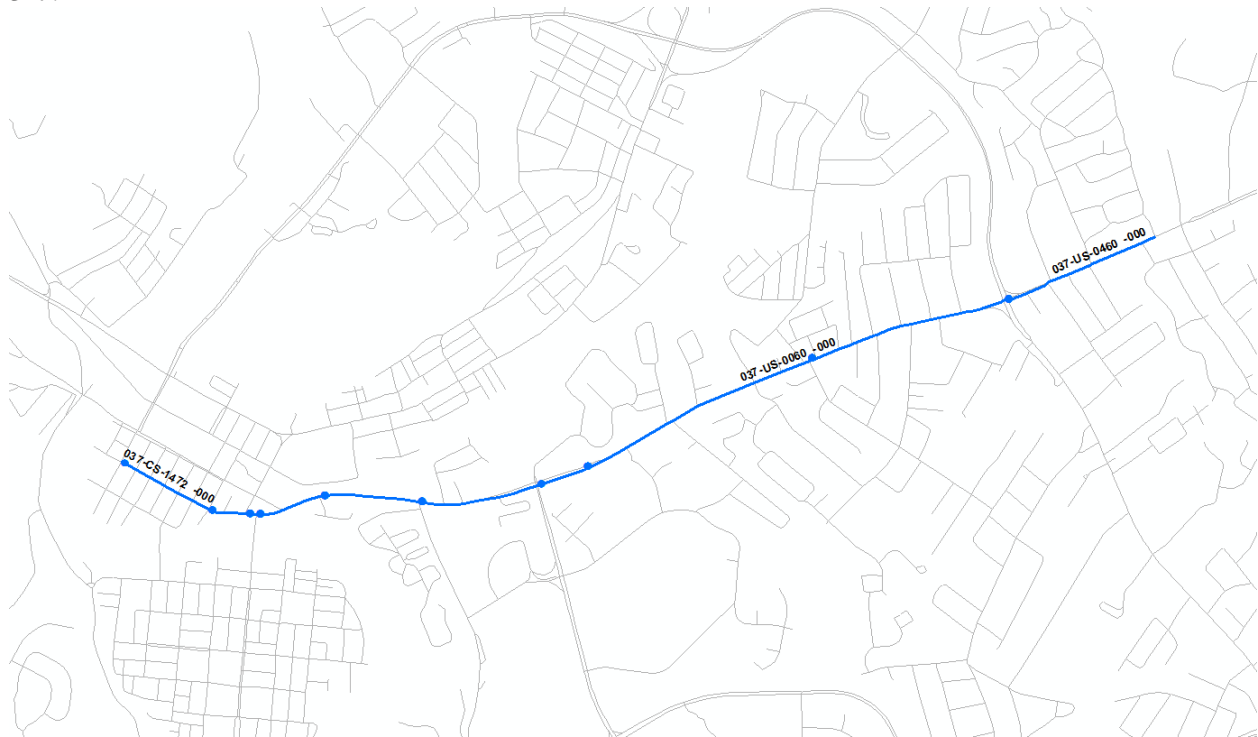

Figure 3-7 Main Street in Frankfort KY

The comparison between estimated speeds using the Akcelik function and the measured speeds based on NAVTEQ data is shown in Figure 3-8. For segments with low VSF values, speeds estimated by the Akcelik function using VSF were higher than the measured speeds. However, as the VSF approaches or exceeds 1 — indicating oversaturated conditions — the estimated speeds are lower than measured speeds. For example, the VSF on the segment bounded by Ann Street and High Street is 1.36, and the estimated speed is only 0.8 mph according to the Akcelik equation. In contrast, the actual speed measured by NAVTEQ is 11 mph.
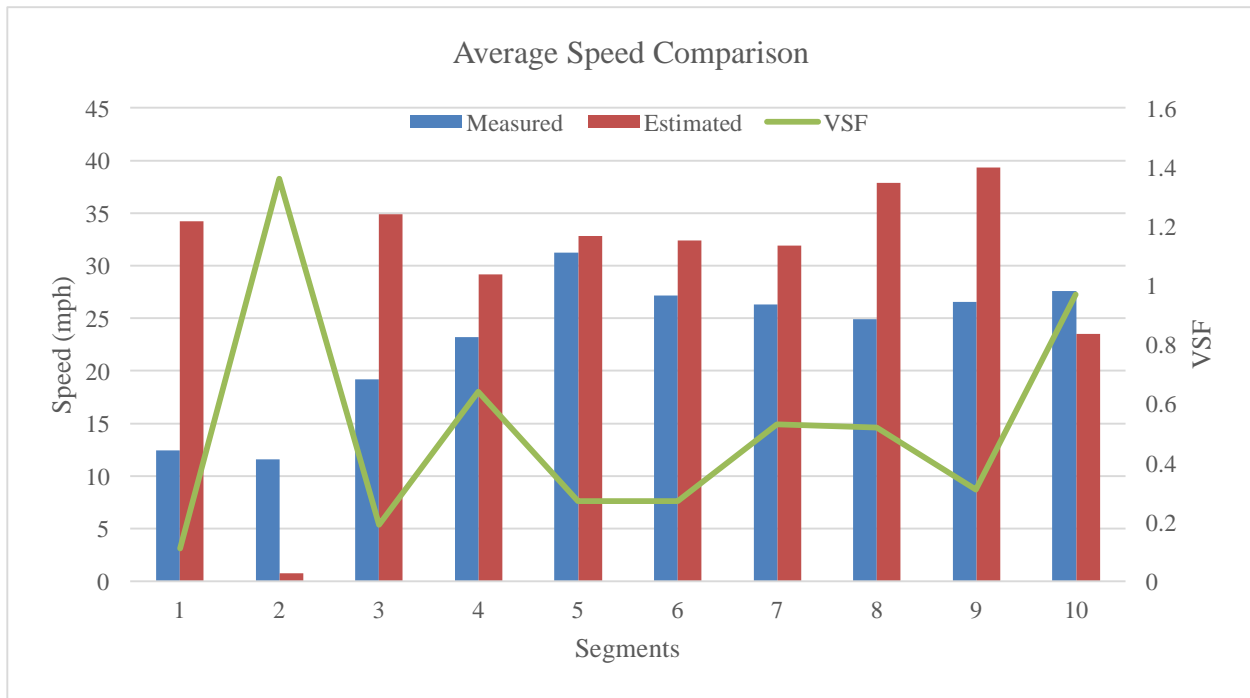

Figure 3-8 Estimated and Measured Speeds on Main Street in Frankfort KY

The VSF measure was developed decades ago for planning application based on very limited field data. The examples above briefly illustrate the limitations associated with using VSF as the sole measure of congestion. Given the increasing availability of measured speed data, the research team recommends that future research should examine how it could influence measures of congestion.

CHAPTER 4   CONCLUSIONS

This study developed a Microsoft Access-based tool by adapting the peak capacity and VSF estimation approach (*1*) to achieve HCM2010 compliance. The tool requires pre-processing of the extract to eliminate empty cells by either omitting segments with missing data or imputing values of the missing item(s). The Appendix contains a step-by-step user's guide.

To ensure the attribute values were homogenous, the HIS segments used as input were often very short. The consequence of this is that some segments may be misclassified. The most common type of misclassification occurred when a short segment, which may not contain a signalized intersection but is located very close to a traffic signal, was categorized as multilane highway or an urban two-lane or three-lane facility. These segments were identified during post-processing and appropriate corrective action was taken. However, using longer segment length in planning applications may largely avoid this problem. The flip side of this is that it requires aggregating attributes from shorter segments, which may be a source of inaccuracies. Sensitivity analysis can be used to gauge the impact of input variables on capacity and VSF estimates, and thus help in the design of a fitting aggregation method for a given planning application.

Analyses showed that VSF provides some information about the level of congestion on a segment. However, it does not reflect variations in travel time (or speed) that greatly influence user experiences on the highway. Examples of how VSF compared with other measures of congestion, including LOS as defined by the HCM and measured speeds by probe vehicles, shed light on its limitations. The research team suggests that future investigations should examine whether VSF could be used as the sole measure of congestion performance.

REFERENCES

[1] Highway Performance Monitoring System Field Manual. Office of Highway Policy Information, Federal Highway Administration, 2005.

[2] Highway Capacity Manual 2010. Transportation Research Board (TRB), National Research Council, Washington D.C., 2010.

[3] Dowling, R., W. Kittelson, J. Zegeer, and A. Skabardonis. Planning Techniques to Estimate Speeds and Service Volumes for Planning Applications. Transportation Research Board, 1997. p. 151 p.

[4] Fred L. Mannering, and S. S. Washburn. *Principles of Highway Engineering and Traffic Analysis 5th Edition* John Wiley & Sons, Inc., 2012.

[5] Highway Capacity Manual 2000. Transportation Research Board (TRB), National Research Council, Washington D.C., 2000.

APPENDIX   VSF CALCULATION TOOL USER'S GUIDE

This user's guide provides guidance for use the tool to generate peak capacity and VSF estimates. KYTC can use these estimates in adequacy rating and/or other planning applications. The tool contains two main parts. One is the HPMS procedure, which is described in Appendix N of the HPMS Field Manual.   The other is the updated procedure to reflect changes made in HCM2010. Following is the step-by-step instruction:

Step 1: Prepare the input file. Attribute names should match the field names in the table "HIS". *Note: All the attribute names in the table are self-explanatory, except for a few columns whose name starts with "A_" which indicates whether the data are available from the extract. The possible value of these columns is 0 or 1. A 0 means the data item is not originally available from the extract and is thus imputed, while 1 means the data are directly from the extract. This information should be added to the input file during pre-processing.*

Step 2: Copy and paste the Access database file and rename it appropriately (e.g., Capacity_and_VSF_2012).

Note: All tables in the database may contain data from the last run. So we will run the delete query first to empty the table so that new information can be stored.
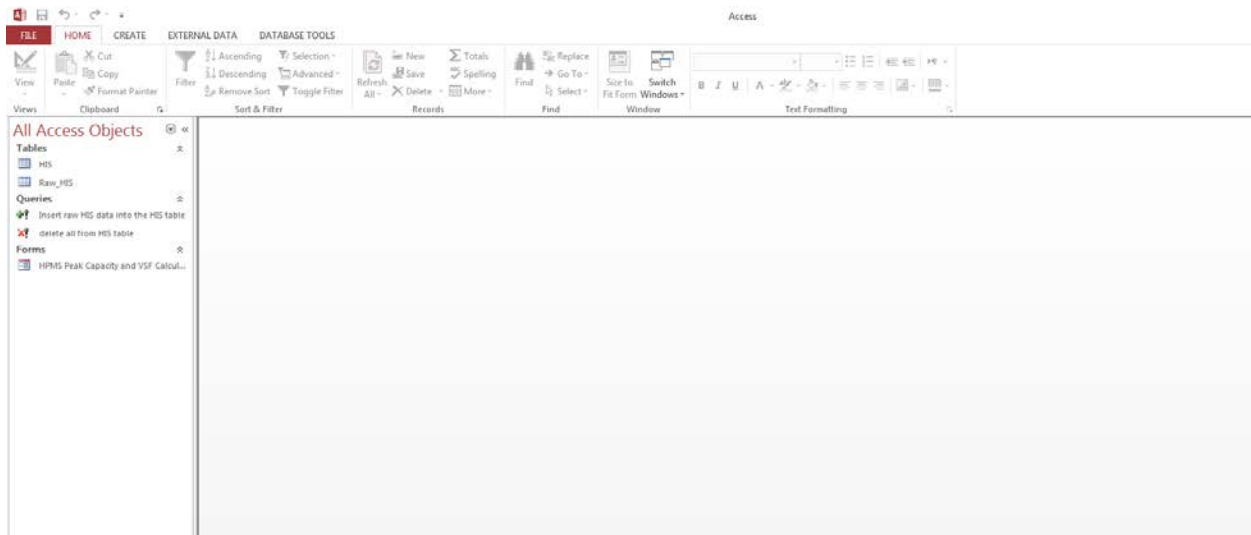
Step 3: Open the Access DB file, as shown in Figure A-1.

Figure A-1 Screenshot of the Microsoft Access Database

Step 4: Under the **Queries** object group, double click "delete all from HIS table." This executes the query. Then, click *Yes* from the pop-up window, as shown in Figure A-2. Tables may contain data from the last run, so this step deletes all records in table "HIS".
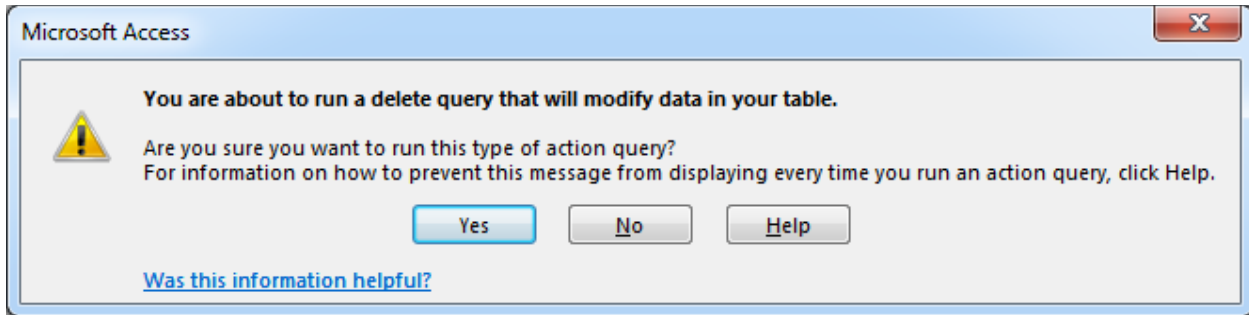
Figure A-2 Delete all from HIS Table Query

Step 5: From the EXTERNAL DATA tab, click Excel to open up the import window, as shown in Figure A-3 . Click Browse to locate the input table and then follow onscreen guidance to load the input table into the MS Access database. Name the table "Raw_HIS".
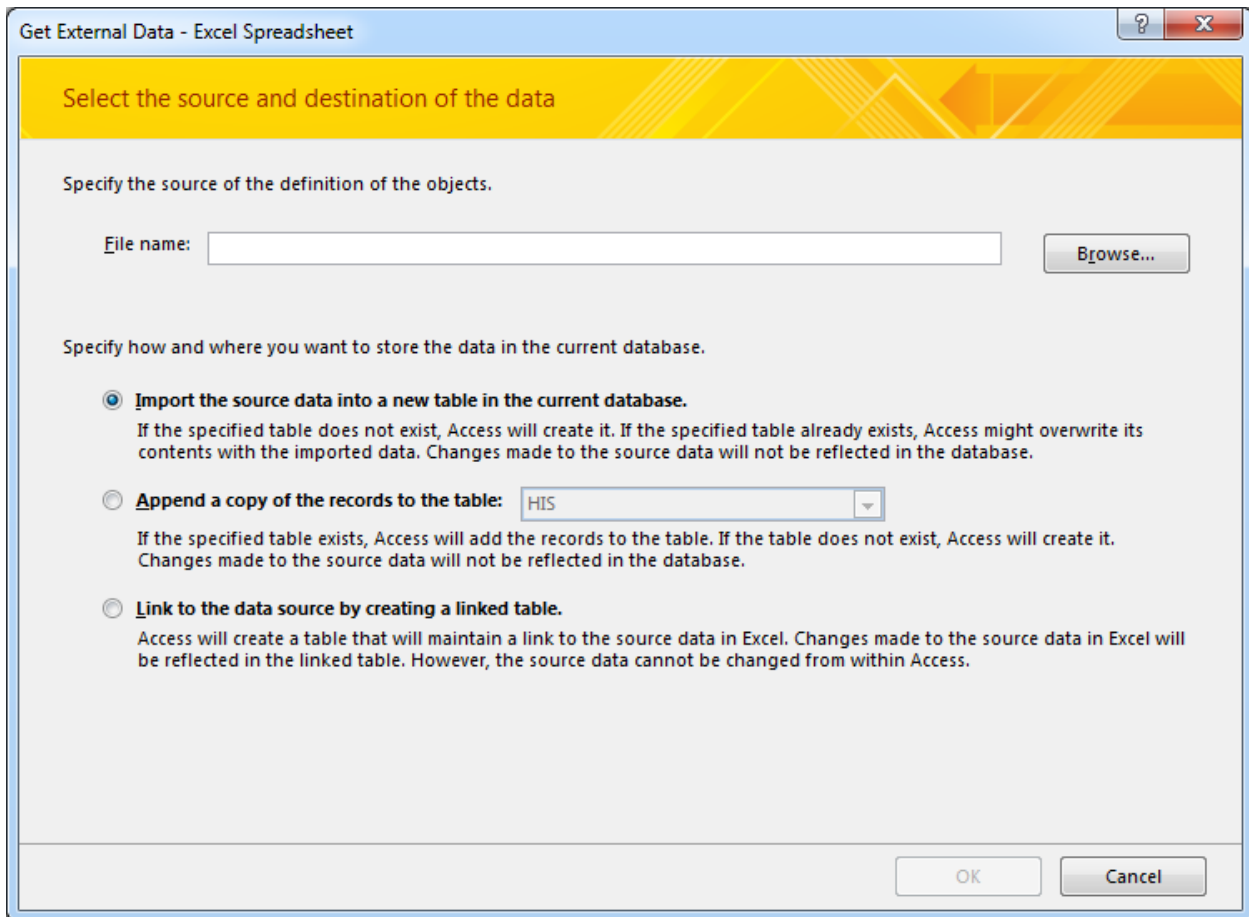

Figure A-3 Import Excel Data Table

Step 6: Under the **Queries** object group, double click the "Insert raw HIS data into the HIS table." This appends all the data records from the raw HIS table to the HIS table. Next, click *Yes* on the pop-up window.

Step 7: Under the **Forms** object group, double click the "Peak Capacity and VSF Calculation." This opens up the capacity and VSF calculation panel (Figure A-4). There are two options available. If a user clicks the top option, the calculation will be based on the HPMS Appendix N procedure The results of this calculation will be stored in the Peak_Capacity_Old, Highway_Type_Old, and Estimated_VSF_Old columns in the HIS table.

If a user clicks the bottom option, the calculation will be based on the updated HCM 2010 procedure. The result of this calculation will be stored in the Peak_Capacity_New, Highway_Type_New, and Estimated_VSF_New columns in the HIS table.

The two options are standalone programs that run independently of one another. Both sets of results are provided for comparison purpose. We do not recommend a specific order to run the methods — users may run the programs in whatever order they prefer.
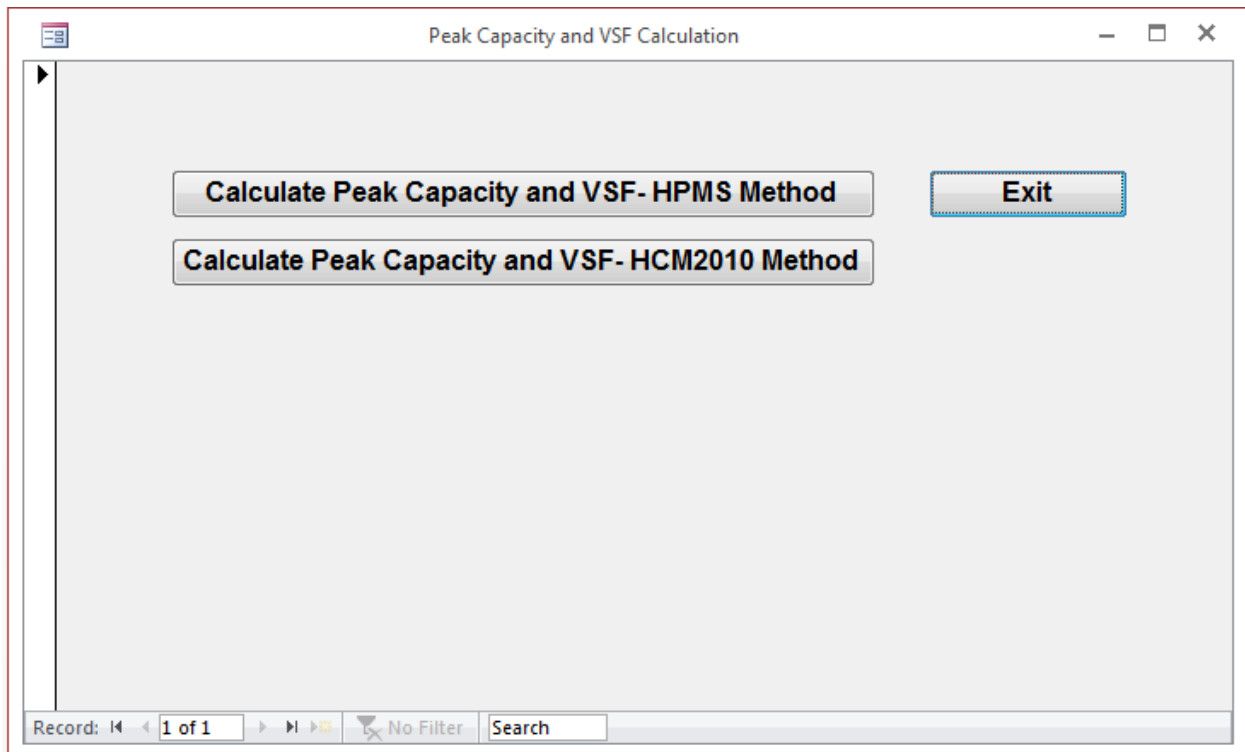

Figure A-4 Peak Capacity and VSF Calculation Panel