CHAPTER 1

INTRODUCTION

Accurate traffic volume estimations are critical to allow the appropriate design of the geometric features and traffic control devices of roadway improvements to meet or exceed current and future traffic demands for a reasonable time period. In conducting traffic impact studies and reviews for planned roadway projects, a variety of traffic estimates are used by transportation engineers to evaluate the need for appropriate traffic control and geometric improvements.

What are the Appropriate Traffic Estimates To Use for Roadway Improvements and Traffic Control Solutions?

One of the primary dilemmas faced by governing traffic authorities in Nebraska is the design service traffic volume that should be used to determine the use of a traffic signal at suburban and fringe areas of towns and cities within the state. For example, the Design Hourly Volume (DHV) is sometimes used in the Peak Hour Warrant to justify the need for a traffic signal. However, the DHV is defined as the 30th highest hourly volume in the “design” year, whereas the peak hour volume (PHV) is defined as the highest hourly volume during an average day (1). Depending on the functional type of roadway, the PHV may be from 5 to 45 percent lower than the DHV. Consequently, implementation of the recommendations of traffic impact studies that use the DHV in the Peak Hour Warrant may result in the installation of unwarranted traffic signals. There is anecdotal evidence that some major chain developers believe a traffic signal will be beneficial to the success of their investment, whether or not it is warranted by the traffic volumes. This may result in the misuse of the intent of the traffic warrants for signalization defined in the Manual of Uniform Traffic Control Devices (MUTCD) (2).

Unwarranted traffic signals increase traffic delay, stops, and crashes. They also may be a tort liability risk and unnecessarily increase costs to developers. Variations in the use of traffic estimates to determine turning movement percentages in justifying auxiliary lanes may overstate the need for turning lanes and promote the unnecessary expenditure of highway and developer funds. On the other hand, variations in the use of traffic estimates may understate the need for traffic control and geometric improvements, resulting in unforeseen congestion, inconvenience, and additional expense.

The most commonly accepted traffic criteria for the design of the capacity of a roadway segment or intersection of roadways is the DHV, the 30th highest hourly volume in the design year (1). The definition infers that if a facility is to adequately serve throughout its life, its physical capacity will only be exceeded for about 30 hours out of the total 8,760 hours in the “design” year. The choice of the 30th highest hourly volume is a long-held concept which stems from research published in *A Policy on Geometric Design of Rural Highways* from the American Association of State Highway Officials (AASHO) in 1965 (pages 54-56). The concept is reproduced graphically in **FIGURE 1**. This figure is still used in the 2004 edition of the guidebook, which is commonly referred to as the Green Book (1).

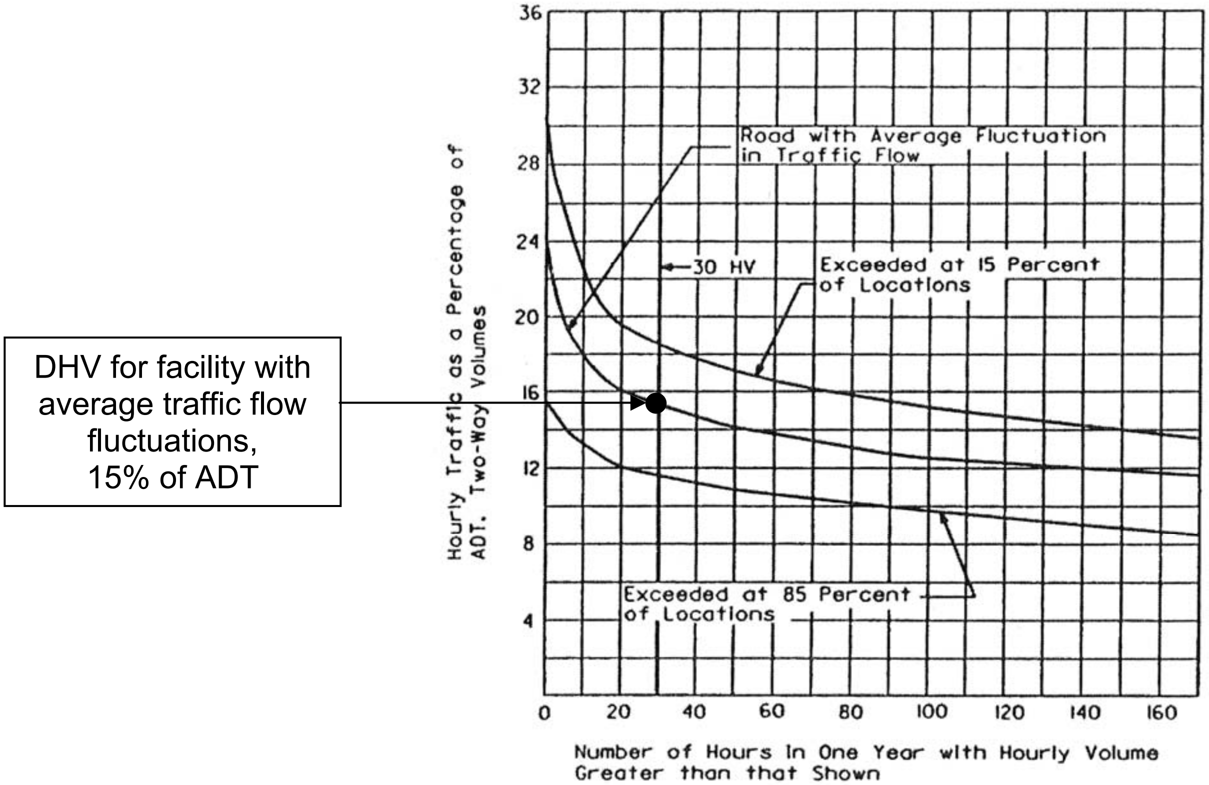


FIGURE 1 Relation Between Peak-Hour and Average Daily Traffic Volumes on Rural Arterials (p. 60, 1)

The data from which **FIGURE 1** was developed represent a multitude of rural arterials covering a wide range of volumes and geographic conditions. The horizontal axis of the figure indicates the 170 highest number of traffic hours in a typical year of 8,760 total hours. The vertical axis shows the value of the volume of traffic during these hours as a percentage of the average daily traffic (ADT) at the study locations. The vast amount of data points included in the study are bracketed by trend lines that capture the bulk of the results (70 percent as indicated by the curve labels), as shown by the upper and lower curves in the figure. The middle line represents sites that exhibited an average fluctuation in traffic flow. Visually comparing all of the trend lines together indicates that drastic traffic flow changes occurred near the 30th highest hour of the year, as the steepness of the curves indicates between the 1st highest hourly volume and the 30th. For the remainder of the hours between the 30th and the 170th, there is very little change in the slope of the curves, indicating that designing for that 30th hour would cover the expected traffic volume at almost any given hour in a given day of a given week in a given month of a given year.

If this concept is valid, one can estimate the average hourly volume that would be exceeded only 29 times per year on a facility with average traffic flow fluctuations by calculating 15 percent of the ADT, as shown by the caption in **FIGURE 1**. In an effort to reasonably balance desired level of service and practical economy, the 30th highest hour is traditionally seen as the pivot point of reasonable design.

Estimating a design hourly volume to plan the required number of lanes and appropriate traffic control devices in a suburban or urban situation is a challenge. In Nebraska, on a typical urban roadway, the 30^{th} highest hourly volume is generally about 9-10% of the ADT. This value has remained consistent over many years. Conversion equations between ADT and DHV developed from the last three years of available continuous traffic count data from the Nebraska Department of Roads (NDOR) are shown in **TABLE 1** (3, 4, 5).

TABLE 1 Relationship Between the 30^{th} Highest Hour of Yearly Traffic (DHV) and the ADT in Nebraska Derived from NDOR Continuous Traffic Count Data from the Years 2004-2006 (3, 4, 5)

Year of Continuous Traffic Count Data	Rural Highways Other Than Low Volume Rural Roads and Rural Interstates	Urban Highways and Streets Other Than Urban Interstates
2004	DHV = 6.89 + (0.1022)(ADT)	DHV = 96.44 + (0.0930)(ADT)
2005	DHV = 6.20 + (0.1025)(ADT)	DHV = 101.02 + (0.0927)(ADT)
2006	DHV = 4.21 + (0.1035)(ADT)	DHV = 105.46 + (0.0922)(ADT)

Example calculations using various representative ADT values which are realistic for daily traffic volumes on Nebraska roadways yield the following results shown for comparison purposes in **TABLE 2** for rural conditions and **TABLE 3** for urban conditions.

TABLE 2 Example Calculations to Estimate DHV on Rural Highways Other Than Low Volume Rural Roads and Rural Interstates Using NDOR Conversion Formulas Derived from Continuous Count Data for the Years 2004-2006 (3, 4, 5)

Year of Continuous Traffic Count	DHV Estimate using 100 ADT	DHV Estimate using 1,000 ADT	DHV Estimate using 10,000 ADT	DHV Estimate using 20,000 ADT
2004	18	110	1,029	2,051
2005	17	109	1,032	2,057
2006	15	108	1,040	2,075

TABLE 3 Example Calculations to Estimate DHV on Urban Highways and Streets Other Than Urban Interstates Using NDOR Conversion Formulas Derived from Continuous Count Data from the Years 2004-2006 (3, 4, 5)

Year of Continuous Traffic Count	DHV Estimate using 100 ADT	DHV Estimate using 1,000 ADT	DHV Estimate using 10,000 ADT	DHV Estimate using 20,000 ADT	DHV Estimate using 50,000 ADT
2004	106	190	1,027	1,957	4,747
2005	111	194	1,029	1,956	4,737
2006	115	198	1,028	1,950	4,716

Roadway improvements are normally based on volumes of traffic projected to a "design" year which is usually the time at which the facility will likely undergo a major reconstruction resulting in an opportunity to reassess the facility's function. The "design" year may be determined from the roadway network planning schedules (1-, 6-, and 20-year plans) of the appropriate governing agency, assuming funding mechanisms are similar to those upon which the plans are based. If the facility to be improved is designed to adequately accommodate the number of vehicles resulting from a reliable future traffic projection calculation and the design service volume is correctly estimated, the facility should be able to adequately function at a predetermined level of service 99.997% of the available hours in the "design" year. If the concept is valid, the assumptions appear to be very conservative, meaning there should be little complaint from the traveling public of frequent delays with over-capacity conditions as long as the level of service used to define "adequate" service is reasonable with respect to local user attitudes.

A suburban-urban surrogate estimate of design service volume is required to determine an equivalent DHV in less rural areas. Locations along the fringe of urban areas are the most contentious since the highest hourly volume of the typical 24-hour day occurs in the evening work-to-home peak. An understanding of the variability of the evening peak is necessary to properly assess a design service volume that will produce a geometric design configuration and traffic control device solution that will function appropriately for the "life" of the project. The design service volume is defined as the maximum hourly traffic volume that a roadway should be designed to serve without the quality of service falling below a predetermined level. The "life", in this less rural scenario, should exceed the generated traffic volume from a newly developed commercial attraction's ultimate build-out for several years. Practitioners agree that as a minimum, traffic volumes generated at the opening of a newly developed area should be accommodated with an acceptable level of service. A reasonable time period of similar acceptable level of service should be at least five years unless the land use in the area of the improvement drastically changes unpredictably. Choice of a suitable design service volume should be sensitive to the following points, highlighted by the Green Book:

Design should not be so economical that severe congestion results during peak periods. It may be desirable, therefore, to choose an hourly volume for design, which is about 50 percent of the volumes expected to occur during the few highest hours of the design year, whether or not that volume is equal to the 30th highest hour. Some congestion would be experienced by traffic during peak hours but the capacity would not be exceeded. A check should be made to ensure that the expected maximum hourly volume does not exceed the capacity **(p. 61, 1).**

The 30th Highest Hour Volume (HHV) criterion also applies in general to urban areas; however, where the fluctuation in traffic flow is markedly different from that on rural roadways, other hours of the year should be considered for the basis of design. A highest-hour-volume recommendation for these types of situations is given in the Green Book as follows:

In urban areas, an appropriate DHV may be determined from the study of traffic during normal daily peak periods. Because of the recurring morning and afternoon peak traffic flow, there is usually little difference between the 30th and the 200th highest hourly volume. For typical urban conditions, the highest hourly volume is found during the afternoon work-to-home travel peak. One approach for determining a suitable DHV is to select the highest afternoon peak traffic flow for each week and then average these values for the 52 weeks of the year. If the morning peak-hour volumes for each week of the year are all less than the afternoon peak volumes, the average of the 52 weekly afternoon peak-hour volumes would have about the same values as the 26th highest hourly volume of the year [assuming the average has half of the 52 peak hours higher than it and half lower]. If the morning peaks are equal to the afternoon peaks [a total of 52 x 2 = 104 hours], the average of the afternoon peaks would be about equal to the 52nd highest hourly volume [assuming the average has half of the 104 morning and afternoon peak hours higher than it and half lower].

The volumes represented by the 26th and 52nd highest hours of the year are not sufficiently different from the 30th highest hour value to affect design. Therefore, in urban design, the 30th highest hourly volume can also be assumed to be a reasonable representation of daily peak hours during the year **(p. 61*, 1*)*.***

The logic of the recommendation above should be compared to Nebraska traffic volume values to verify that it is similar to local traffic behaviors before accepting the assumptions.

There is a need for a better understanding of the local, commonly accepted use of traffic estimates to evaluate the need for intersection improvements in traffic impact studies and reviews of planned roadway projects. If the design hourly volume of every roadway improvement were estimated from a year's worth of manual or automated traffic counts at the improvement's location in question, error in the estimate of the 30th highest hourly volume would be small. However, the most commonly used method of determining peak-hour volumes involves an 8-hour count on a Tuesday, Wednesday or Thursday which are considered to be days of the week representing the most "average" traffic conditions. These types of counts are meaningful for many aspects of transportation engineering such as signal warrants but they are thought by practitioners to underestimate or overestimate the peak-hour volume, thereby resulting in physical improvements that will generate either over-capacity conditions more often than acceptable or unneeded auxiliary lanes.

RESEARCH PROJECT OBJECTIVES

The logic of the statements above is evident but does the logic fit reality closely enough to prevent the misinterpretation of commonly used traffic estimates? Key phrases alluding to assumptions of the logic fitting reality are shaded in the quotation above. One of the objectives of this research project is to compare these statements to reality based upon data collected at continuous counting sites operated and maintained by NDOR. If the assumptions aren't valid for user expectations and traffic conditions in Nebraska, the reality-based results should be used to appropriately represent more realistic estimates.

Traffic estimate accuracy is highly dependent on the applicability of the 30th highest hour assumption, as well as the appropriate choice of acceptable degree of congestion. Therefore, the following questions need to be answered before the design of any roadway improvement or traffic control solution can proceed:

1) **What is a politically acceptable and financially achievable degree of congestion in the design year, given the attitudes of the local traveling public?**

2) **What is a reasonably accurate estimate of the traffic volume standard selected to achieve relatively few opportunities of failure with respect to tolerable delay for local system users?**

Goals of the research project are to review traffic volume trends in Nebraska, determine if the trends follow the concept that has been thought to describe an appropriate estimate to determine the physical features of roadway systems for its "design" life, and develop a more realistic method of using traffic estimates to improve the quality and consistency of traffic impact studies and reviews of planned roadway projects. There exists a need to better plan for the future development along access points and standardize the methods used to determine the criteria for the selection of roadway improvements and traffic control solutions at those access points. Practitioners also find the notion of using a standard traffic volume unit for both design and operations desirable, if possible. For instance, NDOR uses DHV values to determine the appropriate number of lanes, pavement thickness, and turn-bay lengths yet warrants for signalization of intersections are based on peak hour, 4-hour vehicle volume and 8-hour counts. It would be desirable to be able to reliably estimate one traffic unit from another by understanding the relationships between them.

A better understanding of current traffic volume estimate relationships will promote consistency in their application. It will reduce the likelihood that unwarranted traffic signals and unnecessary roadway improvements will be installed at intersections on the state highway system. Likewise, it will reduce the likelihood that needed improvements are overlooked. Thus, the research results will promote greater safety and reduce unnecessary road user, highway, and developer costs.

CHAPTER 2

REVIEW OF REFERENCES TO ASSIST IN ANSWERING QUESTION 1: WHAT IS A POLITICALLY ACCEPTABLE AND FINANCIALLY ACHIEVABLE DEGREE OF CONGESTION IN THE DESIGN YEAR, GIVEN THE ATTITUDES OF THE LOCAL TRAVELING PUBLIC?

Traffic and design engineering practitioners have a plethora of guidebooks to choose from for advice on providing adequate service for road system users. As discussed earlier, the design service volume is the maximum hourly volume of traffic that a roadway should be designed to serve without the quality of service falling below a predetermined level. The roadway should be designed using a design hourly volume less than or equal to the design service volume.

Accepted Degrees of Congestion According to the 2004 Green Book

To meet the requirements described above, an understanding of "accepted" congestion needs to be defined. The following is an excerpt from the Green Book (p. 78, 1):

The degree of congestion that should not be exceeded during the design year on a proposed highway can be realistically assessed by:
1) determining the operating conditions that the majority of motorists will accept as satisfactory,
2) determining the most extensive highway improvement that the governmental jurisdiction considers practical, and
3) reconciling the demands of the motorist and the general public with the finances available to meet those demands.

This is an administrative process of high importance in meeting the expectations of the traveling public. The decision should first be made as to the degree of congestion that should not be exceeded during the design period. The appropriate design for a particular facility (such as number of lanes or optimal traffic control device) can then be estimated from the following foundational concepts (pp. 78-80, 1):

1. The highway should be so designed that, when it is carrying the design volume, the traffic demand will not exceed the capacity of the facility even during short intervals of time.
2. The design volume per lane should not exceed the rate at which traffic can dissipate from a standing queue (applicable primarily to freeways and high-type multilane highways).
3. Drivers should be afforded some choice of speed. The latitude in choice of speed should be related to the length of trip.
4. Operating conditions should be such that they provide a degree of freedom from driver tension that is related to or consistent with the length and duration of the trip.
5. There are practical limitations that preclude the design of an ideal freeway.

6. The attitude of motorists toward adverse operating conditions is influenced by their awareness of the construction and right-of-way costs that might be necessary to provide better service.

Level of Service (LOS) characterizes the operating conditions on a facility in terms of traffic performance measures related to speed and travel time, freedom to maneuver, traffic interruptions, and comfort and convenience. The transportation engineering profession has chosen to classify various service levels using a grading system from A through F. **TABLE 4** shows the alphabetic categories with short subjective phrases with respect to roadway segments characterizing general operation conditions for each of them.

TABLE 4 General Qualitative Definitions of Level of Service (p. 84, 1)

Level of Service	General Operating Conditions
A	Free flow
B	Reasonably free flow
C	Stable flow
D	Approaching unstable flow
E	Unstable flow
F	Forced or breakdown flow

The Green Book provides general guidance with respect to the appropriate level of service to which an improvement should perform. The guidance is reproduced in **FIGURE 2**. Recommendations are based on the variables of the functional class of the roadway, the location with respect to population concentration (rural, suburban, or urban), and the terrain type (level, rolling or mountainous). Service level recommendations are conscious of the expectations of drivers in the following ways:

1. Level of service decreases with decreased level of mobility in the functional hierarchy which has a direct impact on the design speed (and therefore upon the corresponding vertical and horizontal alignment) and the roadway cross section.
2. Level of service decreases with increasing cost of construction due to terrain.
3. Level of service decreases with increasing population density from rural to urban conditions.

	Appropriate level of service for specified combination of area and terrain type			
Functional Class	Rural level	Rural rolling	Rural mountainous	Urban and suburban
Freeway	B	B	C	C
Arterial	B	B	C	C
Collector	C	C	D	D
Local	D	D	D	D

FIGURE 2 Guidelines for Selection of Design Levels of Service (p. 85, 1)

Quantitative Recommendations for Design Service Volumes Related to Level of Service Given by the 2000 Highway Capacity Manual

The latest formal version of the guiding document for traffic analysis, planning and design is the 2000 Edition of the Highway Capacity Manual (6). This guidebook assigns general quantitative values to design service volume levels that bracket the A-F level of service categories recommended by the Green Book. Values are separated by the functional classes of

- Two-Lane Rural Highway, **FIGURE 3,**
- Multi-Lane Highways, **FIGURE 4,** and
- Basic Freeway Segments, **FIGURE 5.**

(SEE FOOTNOTE FOR ASSUMED VALUES)

FFS (mi/h)	Terrain	Service Volumes (veh/h)				
		A	B	C	D	E
65	Level	260	480	870	1460	2770
	Rolling	130	290	710	1390	2590
	Mountainous	N/A	160	340	610	1300
60	Level	260	480	870	1460	2770
	Rolling	130	290	710	1390	2590
	Mountainous	N/A	160	340	610	1300
55	Level	N/A	330	870	1460	2770
	Rolling	N/A	170	710	1390	2590
	Mountainous	N/A	110	340	610	1300
50	Level	N/A	N/A	330	1000	2770
	Rolling	N/A	N/A	170	790	2590
	Mountainous	N/A	N/A	110	420	1300
45	Level	N/A	N/A	N/A	330	2770
	Rolling	N/A	N/A	N/A	170	2590
	Mountainous	N/A	N/A	N/A	110	1300

Note:
Assumptions: 60/40 directional split; 20-, 40-, and 60-percent no-passing zones for level, rolling, and mountainous terrain, respectively; 14 percent trucks; and 4 percent RVs.
N/A = not achievable for the given condition
Source: Harwood et al. (7).

This table contains approximate values and is meant for illustrative purposes only. The values depend on the assumptions and should not be used for operational analyses or final design. This table was derived using assumed values listed in the footnote.

FIGURE 3 Design Service Volumes for Two-Lane Rural Highways (p. 12-19, 6)

(SEE FOOTNOTE FOR ASSUMED VALUES)

FFS (mi/h)	Number of Lanes	Terrain	Service Volumes (veh/h) A	B	C	D	E
60	2	Level	1120	1840	2650	3400	3770
		Rolling	1070	1760	2520	3240	3590
		Mountainous	980	1610	2310	2960	3290
	3	Level	1690	2770	3970	5100	5660
		Rolling	1610	2640	3790	4860	5390
		Mountainous	1470	2410	3460	4450	4930
50	2	Level	940	1540	2220	2910	3430
		Rolling	890	1460	2120	2780	3260
		Mountainous	820	1340	1940	2540	2990
	3	Level	1410	2310	3340	4370	5140
		Rolling	1340	2200	3180	4170	4900
		Mountainous	1230	2010	2910	3810	4480

Notes:
Assumptions: highway with 60-mi/h FFS has 8 access points/mi; highway with 50-mi/h FFS has 25 access points/mi; lane width = 12 ft; shoulder width > 6 ft; divided highway; PHF = 0.88; 5 percent trucks; and regular commuters.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions and should not be used for operational analyses or final design. This table was derived from the assumed values listed in the footnote.

FIGURE 4 Design Service Volume for Multi-lane Highways, (p. 12-11, 6)

(SEE FOOTNOTE FOR ASSUMED VALUES)

	Number of Lanes	FFS (mi/h)	Service Volumes (veh/h) for LOS A	B	C	D	E
Urban	2	63	1230	2030	2930	3840	4560
	3	65	1900	3110	4500	5850	6930
	4	66	2590	4250	6130	7930	9360
	5	68	3320	5430	7820	10,070	11,850
Rural	2	75	1410	2310	3340	4500	5790
	3	75	2110	3460	5010	6750	8680
	4	75	2820	4620	6680	9000	11,580
	5	75	3520	5780	8350	11,250	14,470

Notes:
Assumptions: Urban: 70-mi/h base free-flow speed, 12-ft-wide lanes, 6-ft-wide shoulders, level terrain, 5 percent heavy vehicles, no driver population adjustment, 0.92 PHF, 1 interchange per mile.
Rural: 75-mi/h base free-flow speed, 12-ft-wide lanes, 6-ft-wide shoulders, level terrain, 5 percent heavy vehicles, no driver population adjustment, 0.88 PHF, 0.5 interchanges per mile.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values given in the footnote.

FIGURE 5 Design Service Volumes for Basic Freeway Segments, (p. 13-13, 6)

The 2000 Highway Capacity Manual also gives quantitative criteria for levels of service A-F for intersections with various types of traffic control. Since traffic flow may be interrupted at intersections, service levels are quantified by units of user delay time in seconds. Separate criteria is given for the following traffic control types:

- Two-Way Stop Control, **FIGURE 6,**
- Signal Control, **FIGURE 7,** and
- All-Way Stop Control, **FIGURE 8.**

Level of Service	Average Control Delay (s/veh)
A	0–10
B	> 10–15
C	> 15–25
D	> 25–35
E	> 35–50
F	> 50

FIGURE 6 Delay Level of Service Criteria for Two-Way Stop-Controlled Intersections (p. 16-2, 6)

LOS	Control Delay per Vehicle (s/veh)
A	≤ 10
B	> 10–20
C	> 20–35
D	> 35–55
E	> 55–80
F	> 80

FIGURE 7 Delay Level of Service Criteria for Signalized Intersections (p. 17-2, 6)

Level of Service	Control Delay (s/veh)
A	0–10
B	> 10–15
C	> 15–25
D	> 25–35
E	> 35–50
F	> 50

FIGURE 8 Delay Level of Service Criteria for All Way Stop-Controlled Intersections (p. 17-32, 6)

Practitioner Guides for Volume Studies from the 2000 Institute of Transportation Engineers (ITE) Manual of Transportation Engineering Studies (7)
The quality of operations and level of performance of roadway segments and intersections cannot be evaluated unless two things are known:

1) the capacity of the segment or intersection and
2) the volume of traffic using the facility at a given point in time.

For planning purposes, the traffic volume anticipated for the design hour must be estimated using traditional methods so a suitable geometric configuration and traffic control system may be used successfully.

Although it would be advantageous to have all state, county and city roadway networks monitored with continuous counting devices, technology limits and budget constraints have not made this possible at this point in time. Therefore, traffic counts must be made to sample actual volumes for various periods of time to estimate design service volumes. Sample periods and sample methods depend upon the ultimate use to which the volume data will be put (p. 20, 6).

***Count Periods for Volume Studies,* (p. 20, 7)**
The counting period selected for a given location depends on the planned use of the data and the methods available for collecting the data. The count period should be representative of the time of day, day of week, or month of year that is of interest in the study. Saturday counts are sometimes needed for shopping areas. Typical count periods for turning movements, sample counts, vehicle classifications, and pedestrians include:

- ***2 hours; peak period***
- ***4 hours; morning and afternoon peak periods***
- ***6 hours; morning, midday, and afternoon peak periods***
- ***12 hours; daytime (say 7:00 am to 7:00 pm)***

Count intervals are typically 5 or 15 minutes.
Capacity analysis purposes: 15-minute counts are adequate.
Peak-hour factor determination: 5-minute counts are preferable.
Automatic counts: 1-hour counts are commonly used.

***Traffic Access and Impact Studies* (p. 146, 6)**
Studies will frequently include the following:

- ***Peak-period turning movements for site and street***
- ***Adjustment factors to relate count data to design period***
- ***Machine counts to verify peaking characteristics***

***Suggested Background Traffic Volume Data for Review* (p. 147, 6)**
The following recommendations are made for traffic volume data to be collected:

- ***Current and historic daily and hourly volume counts***
- ***Recent intersection turning movement counts***
- ***Seasonal variations***
- ***Projected volumes from previous studies or regional plans***
- ***Relationship of count day to both average and design days***

The time period(s) that provides the highest cumulative directional traffic demands should be used to assess the impact of site traffic on the adjacent street system and to define the roadway configurations and traffic control measure changes needed in the study area.

Typical Peak Traffic Flow Hours for Selected Land Use
FIGURE 9 is reproduced from the ITE guide.

Land Use	*Typical Peak Hours*[a]	*Peak Direction*
Residential	7:00–9:00 A.M. weekdays	Outbound
	4:00–6:00 P.M. weekdays	Inbound
Regional shopping center	5:00–6:00 P.M. weekdays	Total[b]
	12:30–1:30 P.M. Saturdays	Inbound
	2:30–3:30 P.M. Saturdays	Outbound
Office	7:00–9:00 A.M. weekdays	Inbound
	4:00–6:00 P.M. weekdays	Outbound
Industrial	Varies with employee shift schedule	
Recreational	Varies with activity type	

[a]Hours may vary based on local conditions.
[b]Period of maximum weekday traffic impact.
Source: ITE, 1987b.

FIGURE 9 Typical Peak Traffic Flow Hours for Selected Land Uses (p. 152, 7)

For uses that do not demonstrate substantial weekly or seasonal variations, select average days for the analysis. For developments that exhibit major seasonal variations, design days (approximating the 30th highest hour) should be selected.

Techniques for Projecting On-Site Traffic
FIGURE 10 is reproduced from the ITE guide.

Build-up	Appropriate in areas of moderate growth Usually used when project has horizon of 10 years or less Often the best method when there is good local information on development approvals
Transportation plan	Often used with large, regional projects that will develop over a long period Often appropriate for areas of high growth Locally credible transportation plan data that are adaptable to the study year must be available
Growth rates	Typically used for small projects that will be built within a year or two Local record keeping of traffic counts must be good At least 5 years of data showing stable growth should be available Simple, straightforward approach Not appropriate for long-range horizons May result in over- or undercounting nonsite traffic growth

FIGURE 10 Techniques for Projecting On-Site Traffic (p. 159, 7)

CHAPTER 3

USE OF AN ACCEPTABLE PROCEDURE FOR THE ANALYSIS OF ALTERNATIVES IN THE CHOICE OF OPTIMAL GEOMETRIC AND TRAFFIC CONTROL SOLUTIONS

The National Cooperative Highway Research Project 457 (*NCHRP 457): Evaluating Intersection Improvements: An Engineering Study Guide* (8) was developed specifically to define the steps involved in an engineering study of a problem intersection or intersection improvement, beginning with identifying the problem and viable alternatives to address the given situation. The document also illustrates how to use capacity analysis and traffic simulation models to determine the most effective operational traffic movement given the geometric configuration and choice of traffic control device. The report analyzed difficulties commonly faced when using traffic signal warrants to determine the appropriateness of traffic control signals and identified methods of determination for operational effectiveness that should be considered in the assessment of intersection improvements. The report then provides a step-by-step process for the execution of an engineering study for those improvements.

IT IS HIGHLY RECOMMENDED THAT THIS DOCUMENT GOVERN THE PROCESS REQUIRED BY NEBRASKA TO DETERMINE OPTIMAL INTERSECTION IMPROVEMENTS OF A FACILITY SINCE IT CONSIDERS BOTH GEOMETRIC AND TRAFFIC CONTROL DEVICE OPTIONS IN THE DETERMINATION OF AN OPTIMAL SOLUTION. An Internet version of the report includes internal hyperlinks between different parts of the report and external links to the most recent source material commonly used by practitioners. This Internet version also includes 17 interactive worksheets that can be helpful in using the guide.

A list of traffic control and geometric alternatives normally considered for problem locations or facility improvements from NCHRP 457 was reviewed and the required traffic estimates data for each alternate was compiled to get an idea of the typical uses of the data. **TABLE 5** lists required traffic data requirements for traffic control device options and **TABLE 6** lists traffic data requirements for geometric alternatives.

TABLE 5 Use of Traffic Estimates in Typical Traffic Control Alternatives for Optimal Operations Improvements at Intersections (8)

Traffic Control Alternative	Traffic Estimate Data Required
Add Flash Mode to Signal Control	1. Major-road and minor-road approach volumes for each hour of the average day 2. Major-road and minor-road approach through-lane count
Convert to Traffic Signal Control	Major-road and minor-road peak-hour, 4-hour and 8-hour counts.
Convert to Multi-way Stop Control	Minimum Volumes: 1. The vehicular volume entering the intersection from the major-street approaches (total of both approaches) averages at least 300 vehicles per hour for any 8 hours of an average day. 2. The combined vehicular, pedestrian and bicycle volume entering the intersection from the minor-street approaches (total of both approaches) averages at least 200 units per hour for the same 8 hours, with an average delay to minor-street vehicular traffic of at least 30 seconds per vehicle during the highest hour. 3. If the 85th-percentile approach speed of the major-street traffic exceeds 40 mph, the minimum vehicular volume warrants are 70 percent of the above values.
Convert to Two-Way Stop or Yield Control	Major- and minor-road approach volumes for the peak hour of the average day.
Prohibit On-Street Parking	1. Major- and minor-road approach volumes for 8 or more hours on the average day. 2. Major- and minor-road approach through-lane count.
Prohibit Left-Turn Movements	No traffic volumes required.

TABLE 6 Use of Traffic Estimates in Typical Geometric Alternatives for Optimal Operations at Intersections (8)

Geometric Alternative	Traffic Estimate Data Required
Convert to Roundabout	1. Major- and minor-road approach volumes for average day. 2. Major- and minor-road turn movement volumes for the average day (used to compute average left-turn percentage). 3. Major- and minor-road approach sight distance. 4. Major- and minor-road pedestrian, bicycle, and heavy vehicle volumes for the average day.
Add a Second Lane on the Minor Road	1. Major-road approach volume for the peak hour of the average day. 2. Minor-road turn movement volume for the peak hour of the average day (used to compute right-turn percentage).
Add a Left-Turn Bay on the Major Road	1. Major-road turn movement volume for the peak hour of the average day. 2. Major-road 85^{th}-percentile speed (posted speed can be substituted if data are unavailable).
Add a Right-Turn Bay on the Major Road	1. Major-road turn movement volume for the peak hour of the average day. 2. Major-road 85^{th}-percentile speed (posted speed can be substituted if data are unavailable).
Increase Length of Turn Bay	1. Major- and minor-road turn movement volumes for the peak hour of the average day. 2. Major-road 85^{th}-percentile speed (posted speed can be substituted if data are unavailable). 3. Major- and minor-road bay lengths (taper length should be excluded).
Increase the Right-Turn Radius	1. Heavy vehicle volume during the peak hour of the average day. 2. Major- and minor-road functional classification. 3. Major- and minor-road right-turn radius, measured to the edge of the traveled way.

CHAPTER 4

DETERMINING PHV-DHV AND PHV-AADT RELATIONSHIPS USING NDOR CONTINUOUS COUNT DATA IN NEBRASKA

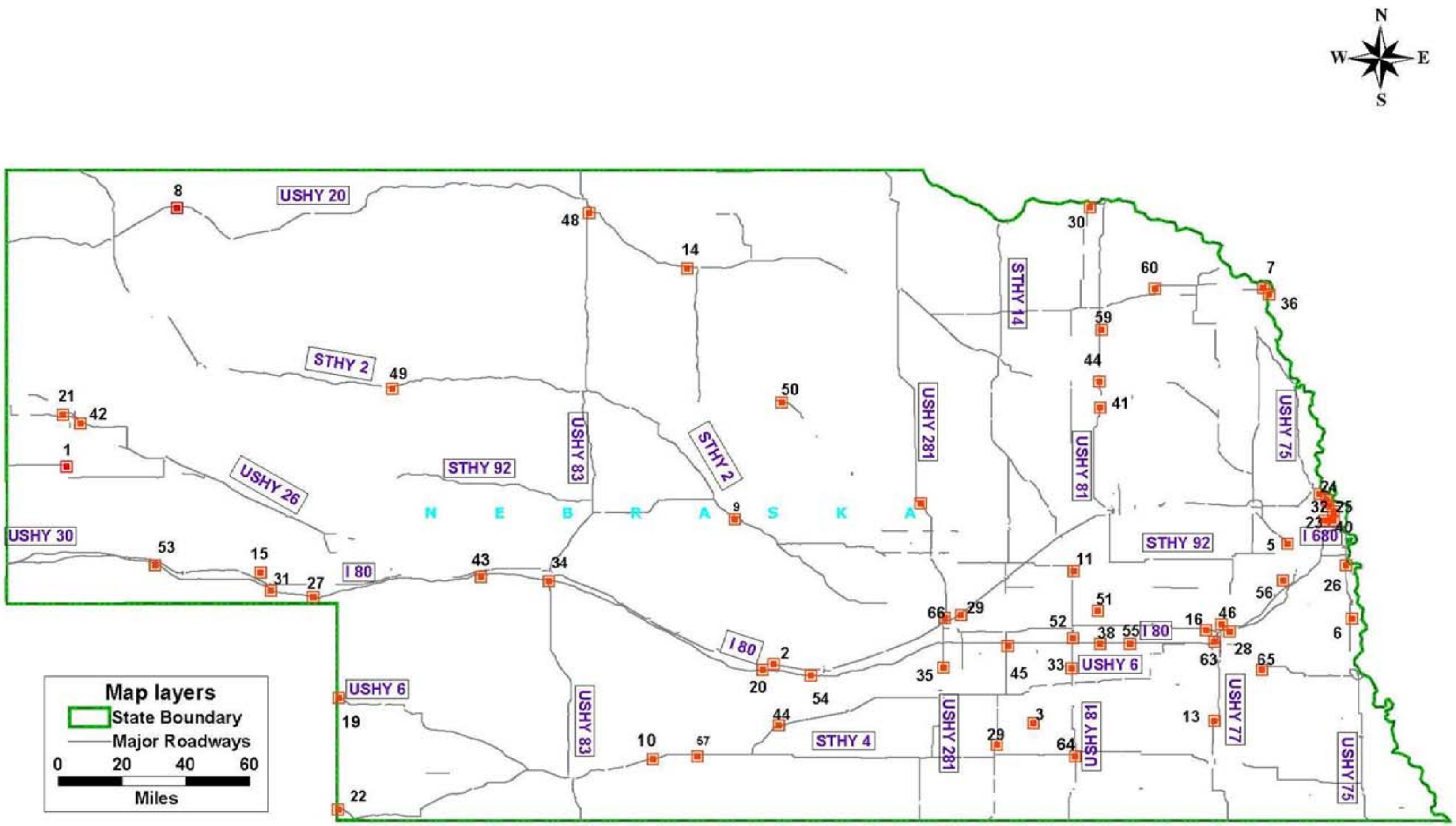
In conducting traffic impact studies and reviews for planned roadway projects, a variety of traffic estimates are used by transportation engineers to evaluate the need for traffic control and geometric improvements on the state highway system. This may lead to inconsistency (overstatement or understatement of needs) in the construction of roadway improvements and may result in unnecessary spending or increased delay. An analysis was completed to find best-fit equations to estimate the relationship between the three typical traffic estimates listed below for both urban and rural functional classifications in Nebraska:

- Peak Hour Volume (PHV) and Design Hourly Volume (DHV)
- Peak Hour Volume (PHV) and Average Annual Daily Volume (AADT)

There were 65 continuous traffic counter stations in Nebraska as of 2003, of which 63 were active in 2003. Traffic count data were not available at two count stations since they were discontinued by the year 2002. Counter stations were separated into five road categories:

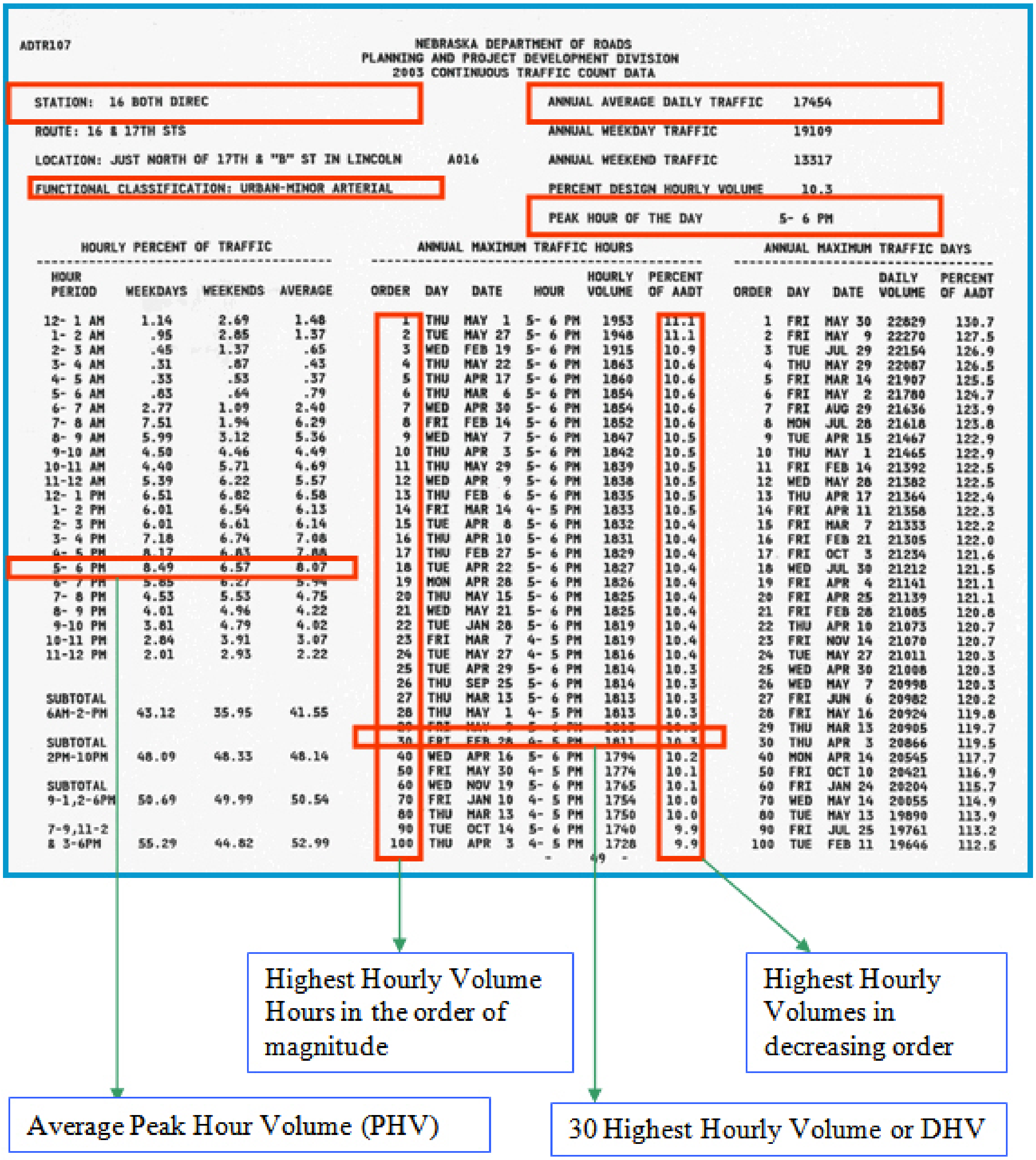
- Rural Interstate (11 stations)
- Other Rural Highways (30 stations)
- Low Volume Rural Roads (8 stations)
- Urban Interstate (5 stations)
- Other Urban Highways and Streets (9 stations)

A Nebraska state map with locations of these counter stations is shown in **FIGURE 11**. **FIGURE 12** shows the typical annual data collected in 2003 by using Continuous Counter Station 16 traffic in both directions (16th and 17th and “B” Streets in Lincoln, NE as an example.



Automatic Traffic Recoeders (ATR)

FIGURE 11 Locations of ATR Counter Stations in Nebraska



ADTR107

NEBRASKA DEPARTMENT OF ROADS
PLANNING AND PROJECT DEVELOPMENT DIVISION
2003 CONTINUOUS TRAFFIC COUNT DATA

STATION: 16 BOTH DIREC
ROUTE: 16 & 17TH STS
LOCATION: JUST NORTH OF 17TH & "B" ST IN LINCOLN A016
FUNCTIONAL CLASSIFICATION: URBAN-MINOR ARTERIAL

ANNUAL AVERAGE DAILY TRAFFIC 17454
ANNUAL WEEKDAY TRAFFIC 19109
ANNUAL WEEKEND TRAFFIC 13317
PERCENT DESIGN HOURLY VOLUME 10.3
PEAK HOUR OF THE DAY 5- 6 PM

HOURLY PERCENT OF TRAFFIC

HOUR PERIOD	WEEKDAYS	WEEKENDS	AVERAGE
12- 1 AM	1.14	2.69	1.48
1- 2 AM	.95	2.85	1.37
2- 3 AM	.45	1.37	.65
3- 4 AM	.31	.87	.43
4- 5 AM	.33	.53	.37
5- 6 AM	.83	.64	.79
6- 7 AM	2.77	1.09	2.40
7- 8 AM	7.51	1.94	6.29
8- 9 AM	5.99	3.12	5.36
9-10 AM	4.50	4.46	4.49
10-11 AM	4.40	5.71	4.69
11-12 AM	5.39	6.22	5.57
12- 1 PM	6.51	6.82	6.58
1- 2 PM	6.01	6.54	6.13
2- 3 PM	6.01	6.61	6.14
3- 4 PM	7.18	6.74	7.08
4- 5 PM	8.17	6.83	7.88
5- 6 PM	8.49	6.57	8.07
6- 7 PM	5.85	6.27	5.94
7- 8 PM	4.53	5.53	4.75
8- 9 PM	4.01	4.96	4.22
9-10 PM	3.81	4.79	4.02
10-11 PM	2.84	3.91	3.07
11-12 PM	2.01	2.93	2.22
SUBTOTAL 6AM-2-PM	43.12	35.95	41.55
SUBTOTAL 2PM-10PM	48.09	48.33	48.14
SUBTOTAL 9-1,2-6PM	50.69	49.99	50.54
7-9,11-2 & 3-6PM	55.29	44.82	52.99

ANNUAL MAXIMUM TRAFFIC HOURS

ORDER	DAY	DATE	HOUR	HOURLY VOLUME	PERCENT OF AADT
1	THU	MAY 1	5- 6 PM	1953	11.1
2	TUE	MAY 27	5- 6 PM	1948	11.1
3	WED	FEB 19	5- 6 PM	1915	10.9
4	THU	MAY 22	5- 6 PM	1863	10.6
5	THU	APR 17	5- 6 PM	1860	10.6
6	THU	MAR 6	5- 6 PM	1854	10.6
7	WED	APR 30	5- 6 PM	1854	10.6
8	FRI	FEB 14	5- 6 PM	1852	10.6
9	WED	MAY 7	5- 6 PM	1847	10.5
10	THU	APR 3	5- 6 PM	1842	10.5
11	THU	MAY 29	5- 6 PM	1839	10.5
12	WED	APR 9	5- 6 PM	1838	10.5
13	THU	FEB 6	5- 6 PM	1835	10.5
14	FRI	MAR 14	4- 5 PM	1833	10.5
15	TUE	APR 8	5- 6 PM	1832	10.4
16	THU	APR 10	5- 6 PM	1831	10.4
17	THU	FEB 27	5- 6 PM	1829	10.4
18	TUE	APR 22	5- 6 PM	1827	10.4
19	MON	APR 28	5- 6 PM	1826	10.4
20	THU	MAY 15	5- 6 PM	1825	10.4
21	WED	MAY 21	5- 6 PM	1825	10.4
22	TUE	JAN 28	5- 6 PM	1819	10.4
23	FRI	MAR 7	4- 5 PM	1819	10.4
24	TUE	MAY 27	4- 5 PM	1816	10.4
25	TUE	APR 29	5- 6 PM	1814	10.3
26	THU	SEP 25	5- 6 PM	1814	10.3
27	THU	MAR 13	5- 6 PM	1813	10.3
28	THU	MAY 1	4- 5 PM	1813	10.3
29	FRI	MAY 9	5- 6 PM	[illegible]	10.3
30	FRI	FEB 28	4- 5 PM	1811	10.3
40	WED	APR 16	5- 6 PM	1794	10.2
50	FRI	MAY 30	4- 5 PM	1774	10.1
60	WED	NOV 19	5- 6 PM	1765	10.1
70	FRI	JAN 10	4- 5 PM	1754	10.0
80	THU	MAR 13	4- 5 PM	1750	10.0
90	TUE	OCT 14	5- 6 PM	1740	9.9
100	THU	APR 3	4- 5 PM	1728	9.9

ANNUAL MAXIMUM TRAFFIC DAYS

ORDER	DAY	DATE	DAILY VOLUME	PERCENT OF AADT
1	FRI	MAY 30	22829	130.7
2	FRI	MAY 9	22270	127.5
3	TUE	JUL 29	22154	126.9
4	THU	MAY 29	22087	126.5
5	FRI	MAR 14	21907	125.5
6	FRI	MAY 2	21780	124.7
7	FRI	AUG 29	21636	123.9
8	MON	JUL 28	21618	123.8
9	TUE	APR 15	21467	122.9
10	THU	MAY 1	21465	122.9
11	FRI	FEB 14	21392	122.5
12	WED	MAY 28	21382	122.5
13	THU	APR 17	21364	122.4
14	FRI	APR 11	21358	122.3
15	FRI	MAR 7	21333	122.2
16	FRI	FEB 21	21305	122.0
17	FRI	OCT 3	21234	121.6
18	WED	JUL 30	21212	121.5
19	FRI	APR 4	21141	121.1
20	FRI	APR 25	21139	121.1
21	FRI	FEB 28	21085	120.8
22	THU	APR 10	21073	120.7
23	FRI	NOV 14	21070	120.7
24	TUE	MAY 27	21011	120.3
25	WED	APR 30	21008	120.3
26	WED	MAY 7	20998	120.3
27	FRI	JUN 6	20982	120.2
28	FRI	MAY 16	20924	119.8
29	THU	MAR 13	20905	119.7
30	THU	APR 3	20866	119.5
40	MON	APR 14	20545	117.7
50	FRI	OCT 10	20421	116.9
60	FRI	JAN 24	20204	115.7
70	WED	MAY 14	20055	114.9
80	TUE	MAY 13	19890	113.9
90	FRI	JUL 25	19761	113.2
100	TUE	FEB 11	19646	112.5

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FIGURE 12 Traffic Count Data at Counter Station 16 Located North of "B" Street on 16th and 17th Streets in Year 2003 (9)

Data used for the following analyses were collected from Nebraska Department of Roads (NDOR) continuous traffic count books for the years 2001, 2002, and 2003 (9, 10, 11). By using the data from the continuous count stations, the purest comparison can be made between average peak hour volumes and DHV and AADT since these are the most precise estimates available for this type of information.

After filtering out traffic count stations with incomplete or missing data, a total of 320 sets of data for 65 traffic counter stations were collected for all types of roadways included in the count data. This data is compiled in Appendix A.

A spreadsheet database was developed with the following information as column headings for each data set:

- Counter Station Number
- Functional Classification
- AADT Volume
- Peak Hour Percentage (weekday, weekend, and average)
- Peak Hour Volume (weekday, weekend, and average)
- DHV Percentage
- DHV Volume
- Day of DHV Occurrence
- Day of 1st, 10th, 20th, and 30th highest maximum traffic days and
- Percentage of AADT 1st, 10th, 20th, and 30th highest maximum traffic days

The data are further categorized by urban and rural functional types as shown in **TABLE 7**.

TABLE 7 Functional Roadway Categories Used in the NDOR Continuous Traffic Count Data Publications from 2001-2003 (9, 10, 11)

Category	Functional Type
Urban	Urban-Collector
	Urban-Minor Arterial
	Urban-Principal Arterial-Other
	Urban-Principal Arterial-Interstate
Rural	Rural-Major Arterial
	Rural-Minor Collector
	Rural-Major Collector
	Rural-Minor Arterial
	Rural-Principal Arterial-Other
	Rural-Principal Arterial-Interstate

Methodology
Relationships for PHV versus DVH and PHV versus AADT were plotted for each of the above functional classes except for 3 functional types which do not have sufficient data sets to form a relationship. The 3 functional types omitted from the analysis were: Urban-Collector, Rural-Major Arterial and Rural-Minor Collector. Each relationship was found to best fit a linear equation with a high value of R^2. Both best-fit lines with no constraints and lines intercepting zero were established. Figures showing these relationships are shown in Appendix B.

The data sets for each functional class were ranked from the lowest traffic volume to the highest for urban and rural area types with each correspondent relationship named accordingly, as shown in **TABLE 8** and **TABLE 9**.

TABLE 8 Rank of Urban Functional Classes and Name Convention for Relationship

Functional Class	Rank	Name Convention of Relationship
Urban-Minor Arterial	1	U1
Urban-Principal Arterial-Other	2	U2
Urban-Principal Arterial-Interstate	3	U3

TABLE 9 Rank of Rural Functional Classes and Name Convention for Relationship

Functional Class	Rank	Name Convention of Relationship
Rural-Major Collector	1	R1
Rural-Minor Arterial	2	R2
Rural-Principal Arterial-Other	3	R3
Rural-Principal Arterial-Interstate	4	R4

For the urban type, relationships between the first two ranks were compared at the 95 percent confidence level. Upper and lower 95 percent confidence bounds were plotted around the fitted regression lines for both relationships in one figure. The complete overlapping of two 95 percent confidence intervals indicated that the two relationships were not significantly different; therefore the data for these two functional classes could be combined to get a new linear relationship. The new relationship was then compared with the relationship of the third rank functional class at the 95 percent confidence level to see if they were significantly different. A flow chart describing the comparison process is presented in **FIGURE 13**.

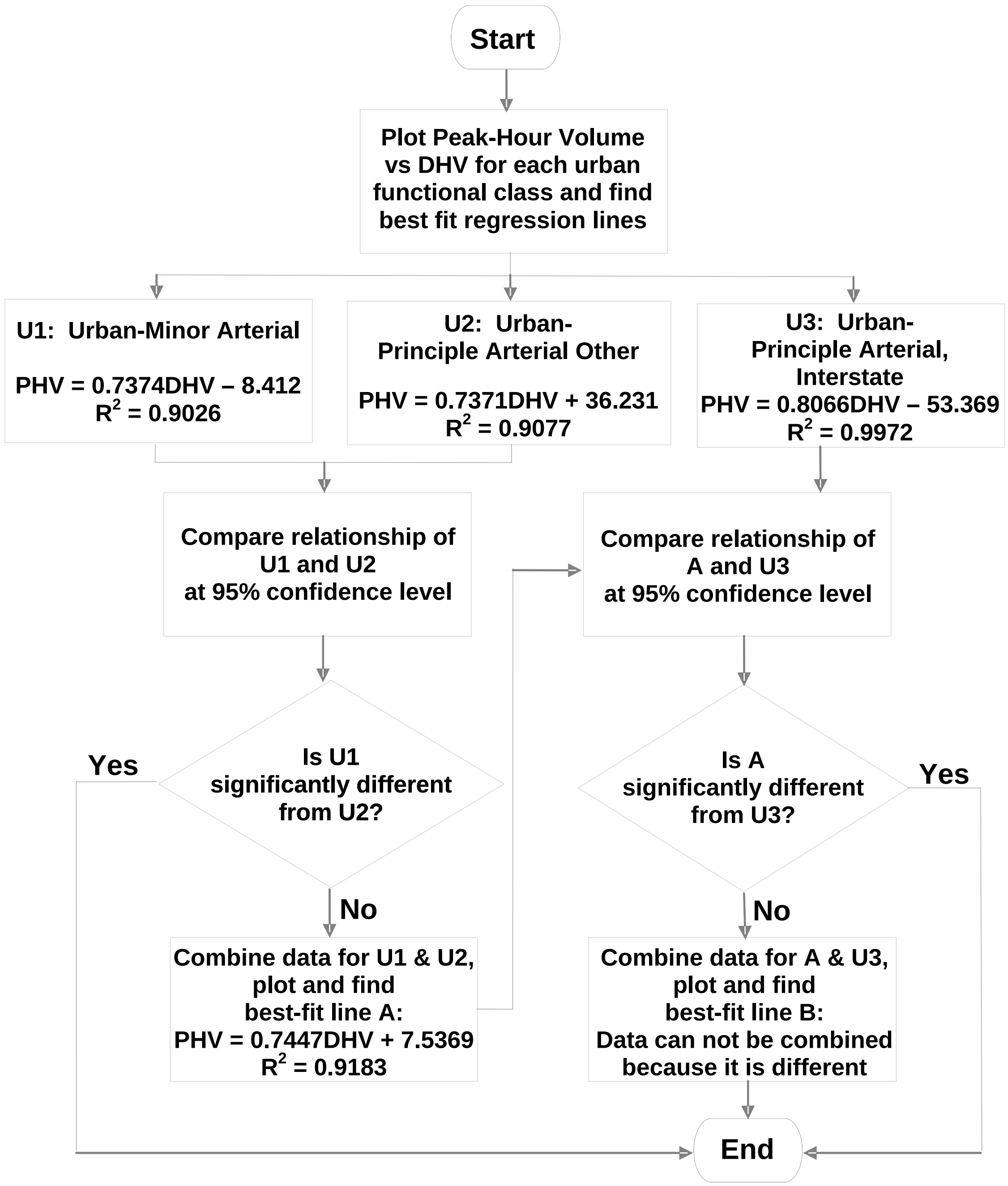

FIGURE 13 Flow Chart Showing Statistical Analysis Comparison of PHV-DHV Relationship Amongst Urban Functional Type Roadways

For rural roadway types, relationships for the last two ranks were compared first, then combined and compared with the 2nd rank data, then with the 1st rank data. **FIGURE 14** shows the comparison process for the rural ranked data relationship between PHV and AADT. For urban roadway types, a similar process was used.

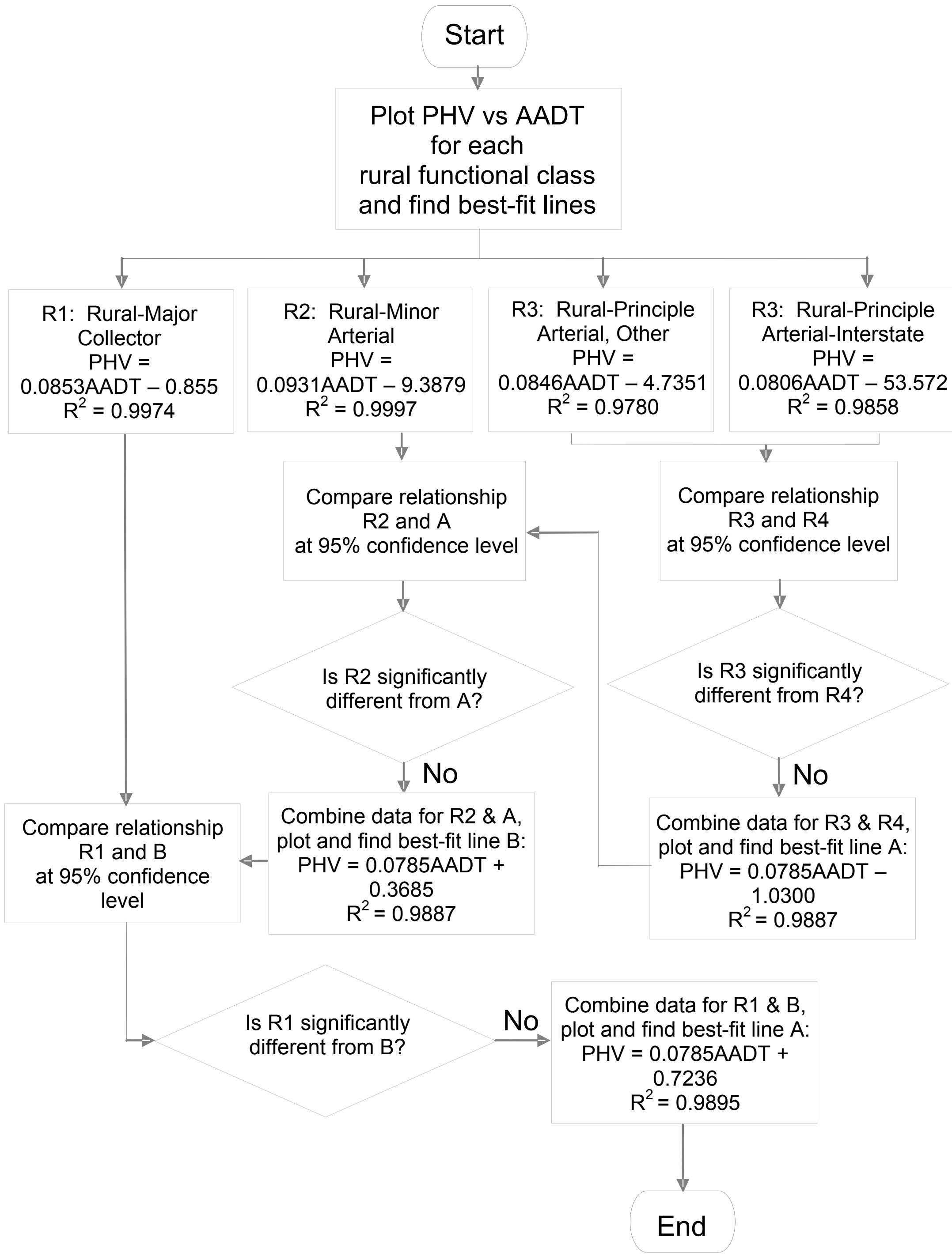

FIGURE 14 Flow Chart Showing Statistical Analysis Comparison of PHV-AADT Relationship Amongst Rural Functional Type Roadways

Finally, relationships between the combination of all rural classes and combination of all urban classes were compared at the 95 percent confidence level in order to find out if there was a common relationship that exists for all functional classes.

Findings

Strong linear relationships were found for both PHV versus DHV and PHV versus AADT for each functional class. **TABLE 10** and **TABLE 11** show the summaries of R^2 details.

TABLE 10 Summary of R^2 for Peak Hour Volume vs DHV Volume

Type	Relationship	R^2 (no constraint)	R^2 (intercept zero)
Urban	U1	0.9026	0.9025
	U2	0.9077	0.9072
	U3	0.9972	0.9971
	A (U1+U2)	0.9183	0.9183
Rural	R1	0.7784	0.7763
	R2	0.9974	0.9970
	R3	0.9874	0.9864
	R4	0.9340	0.9263
	A (R3+R4)	0.9563	0.9557
	B (R2+R3+R4)	0.9579	0.9574
	C (R1+R2+R3+R4)	0.9610	0.9604
Rural and Urban	(U1+U2+R1+R2+R3+R4)	0.9541	0.9536

TABLE 11 Summary of R^2 for Peak Hour Volume vs AADT Volume

Type	Relationship	R^2 (no constraint)	R^2 (intercept zero)
Urban	U1	0.9409	0.9402
	U2	0.9474	0.9467
	U3	0.9988	0.9986
	A (U1+U2)	0.9532	0.9529
Rural	R1	0.9974	0.9971
	R2	0.9997	0.9991
	R3	0.9780	0.9778
	R4	0.9858	0.9842
	A (R3+R4)	0.9887	0.9887
	B (R2+R3+R4)	0.9887	0.9887
	C (R1+R2+R3+R4)	0.9895	0.9895
Rural and Urban	(U1+U2+R1+R2+R3+R4)	0.9808	0.9807

The R^2 value measured the correlation between the PHV and DHV/AADT. A value of 1.0 would be a perfect fit of the regressed line to the data, meaning that the PHV and DHV or AADT relationships could be perfectly described linearly. Ideally, R^2 values which exceed 0.75 would indicate a strong relationship and higher confidence. Low R^2 values can indicate a weak relationship between the data points and the regression line used for traffic estimates.

Results showed that all the R^2 values exceeded 0.75, with most of them exceeding 0.90, which confirmed the strength in the relationship between PHV and DHV/AADT for all functional classes analyzed. The R^2 values for lines intercepting zero were found to be equal or slightly less than the values for those linear lines with no constraints.

Only one R^2 value of 0.7784 (0.7783 for intercepting zero) for the relationship between PHV and DHV for Rural-Major Collector appeared to be relatively lower than the others. It was observed that the DHV percentage for Station 15 and 50 in all collection years were consistently above 27 percent and as high as 34.7 percent, while the remaining DHV percentages were found to be around 12-18%, which is the typical percentage for rural roads. The high DHV percentages probably reflected a unique traffic pattern at the two stations. Since the data sets for these two stations took about half of the sample size in this functional class, they were believed to have affected the overall correlation coefficient between PHV and DHV. The relationship between PHV and AADT showed a very high R^2 value of 0.9974, which also supported the explanation. When combining all rural data into to one analysis, the linear relationships were found to be quite strong.

For urban functional classifications, the relationship for Urban-Principal Arterial-Interstate was found to be significantly different from the other two functions. This is likely due to the special nature of this type of urban roadway, i.e. high speed, high volume and total access control. Although it couldn't be further grouped with other functional classes, it had its own relationships between PHV and DHV/AADT, which also fit into nearly perfect regression lines.

TABLE 12 PHV Estimate Equations as a Function of DHV

Equation Functional Type	Regressed Equation	Equation Number	R^2
Urban (No Constraint)	PHV = 0.7447DHV + 7.5369	1	0.9183
Urban (Intercept Zero)	PHV = 0.7481DHV	2	0.9183
Rural (No Constraint)	PHV = 0.7321DHV – 20.872	3	0.9610
Rural (Intercept Zero)	PHV = 0.7197DHV	4	0.9604
Rural and Urban (No Constraint)	PHV = 0.7402DHV – 20.029	5	0.9541
Rural and Urban (Intercept Zero)	PHV = 0.7292DHV	6	0.9536

TABLE 13 PHV Estimate Equations as a Function of AADT

Equation Functional Type	Regressed Equation	Equation Number	R^2
Urban (No Constraint)	PHV = 0.0844AADT – 22.859	7	0.9532
Urban (Intercept Zero)	PHV = 0.0832AADT	8	0.9529
Rural (No Constraint)	PHV = 0.0785AADT + 0.7236	9	0.9895
Rural (Intercept Zero)	PHV = 0.0785AADT	10	0.9895
Rural and Urban (No Constraint)	PHV = 0.0801AADT – 4.5399	11	0.9808
Rural and Urban (Intercept Zero)	PHV = 0.0801AADT	12	0.9807

Examples of PHV Traffic Estimation Using Generated Relationships With DHV

The relationships established in this project between PHV and DHV can be used for PHV traffic volume estimation for different functional classes. Some examples are shown below. **TABLE 14** combines the results of the examples to view their similarity.

Example 1 Urban (Urban-Minor Arterial and Urban-Principal Arterial-Other Con.)

Given: **DHV of 2500 vph**
Estimate: **Peak Hour Volume**

Using linear (no constraint) equation:
PHV = 0.7447DHV + 7.5369 = 0.7747(2500) + 7.5369 = **1869 vph** **EQUATION 1**

Using linear (intercept zero) equation:
PHV = 0.7481DHV = 0.7481(2500) = **1870 vph** **EQUATION 2**

Example 2 Rural

Given: **DHV of 2500 vph**
Estimate: **Peak Hour Volume**

Using linear (no constraint) equation:
PHV = 0.7321DHV - 20.872 = 0.7321(2500) - 20.872 = **1809 vph EQUATION 3**

Using linear (intercept zero) equation:
PHV = 0.7197DHV = 0.7197(2500) = **1799 vph** **EQUATION 4**

Example 3 Rural and Urban

Given: **DHV of 2500 vph**
Estimate: **Peak Hour Volume**

Using linear (no constraint) equation:
PHV = 0.7402DHV - 20.029 = 0.7402(2500) - 20.029 = **1830 vph EQUATION 5**

Using linear (intercept zero) equation:
PHV = 0.7292DHV = 0.7292(2500) = **1823 vph** **EQUATION 6**

TABLE 14 Comparison of Results of Using Urban, Rural and Combined Rural-Urban Formulas Using DHV

Type	Estimated PHV
Urban (No Constraint)	1869
Urban (Intercept Zero)	1870
Rural (No Constraint)	1809
Rural (Intercept Zero)	1799
Rural and Urban (No Constraint)	1830
Rural and Urban (Intercept Zero)	1823
Low Estimate:1799 Average Estimate:1833 High Estimate:1870	

Examples of PHV Traffic Estimation Using Generated Relationships With AADT
The relationships established in this project between PHV and AADT can be used for PHV traffic volume estimation for different functional classes. Some examples are shown below. **TABLE 15** combines the example results to view their similarity.

<u>Example 1 Urban (Urban-Minor Arterial and Urban-Principal Arterial-Other Con.)</u>

Given: AADT of 25000 vpd
Estimate: Peak Hour Volume

Using linear (no constraint) equation:
PHV= 0.0844AADT - 22.859 = 0.0844(25000) - 22.859 = **2087 vph EQUATION 7**

Using linear (intercept zero) equation:
PHV = 0.0832AADT = 0.0832(25000) = **2080 vph** **EQUATION 8**

<u>Example 2 Rural</u>

Given: AADT of 25000 vpd
Estimate: Peak Hour Volume

Using linear (no constraint) equation:
PHV= 0.0785AADT + 0.7236= 0.0785(25000) + 0.7236= **1963 vph EQUATION 9**

Using linear (intercept zero) equation:
PHV = 0.0785AADT = 0.0785(25000) = **1963 vph** **EQUATION 10**

<u>Example 3 Rural and Urban</u>

Given: AADT of 25000 vpd
Estimate: Peak Hour Volume

Using linear (no constraint) equation:
PHV= 0.0803AADT - 4.5399= 0.0803(25000) - 4.5399= **2003 vph EQUATION 11**

Using linear (intercept zero) equation:
PHV = 0.0801AADT = 0.0801(25000) = **2003 vph** **EQUATION 12**

TABLE 15 Comparison of Results of Using Urban, Rural and Combined Rural-Urban Formulas Using AADT

Type	Estimated PHV
Urban (No Constraint)	2087
Urban (Intercept Zero)	2080
Rural (No Constraint)	1963
Rural (Intercept Zero)	1963
Rural and Urban (No Constraint)	2003
Rural and Urban (Intercept Zero)	2003
Low Estimate:1963 Average Estimate:2016 High Estimate:2087	

Conversely, if the average PHV is determined from a field study, DHV and AADT volumes can be estimated as well, which is important when converting a field-count peak hour value to an estimate of DHV or AADT.

Summary of Estimating PHV from DHV and AADT
Continuous traffic count data were collected for the 3 years from 2001-2003 and analyzed to establish methods for estimating traffic volume based on the relationships between PHV and DHV as well as PHV and AADT. Strong linear relationships were found in all functional classes. The relationships among urban and rural groups were compared statistically at the 95 percent confidence level and further grouped. Common relationships were finally established for the 3 categories: urban, rural and rural/urban combined. Methods for estimating PHV from DHV/AADT were also demonstrated in the examples.

Verifying Estimate Equations with Newer Field Data
Since the equations were derived from the years 2001-2003, it was necessary to apply them to newer data and determine if the relationships were still valid. Field data included in the 2004, 2005, 2006 Continuous Traffic Count Data was used to compare estimates made of PHV from 2004-2006 DHV and AADT data from all the counter stations. **TABLE 16** gives a summary of the validity of the equations using the newer data.

TABLE 16 Summary of Ability of Regressed Equations to Estimate Actual PHV

Predictability Range for Estimate of Actual PHV from Regressed Equations	**Continuous Count Data Years**		
	2004	**2005**	**2006**
± 5 percent	68 percent of counts	59 percent of counts	63 percent of counts
± 10 percent	95 percent of counts	82 percent of counts	84 percent of counts
± 15 percent	99 percent of counts	91 percent of counts	93 percent of counts
± 20 percent	100 percent of counts	100 percent of counts	98 percent of counts
± 30 percent	Not applicable	Not applicable	100 percent of counts
Average Predictability of Equations	Within ±3.9% of actual PHV value	Within ±5.1% of actual PHV value	Within ±5.2% of actual PHV value

For example, the regression equations were able to predict 59 percent of the 128 count entries available in the 2005 Continuous Count Data Book within ±5% of the actual average PHV, and 82 percent of the 128 count entries with ±10 percent of the actual average PHV. The average prediction rate for 2005 was within ±5.1% of the actual average PHV for that year. The range of predictability shown in **TABLE 16** indicates the estimate equations will be adequate over time.

CHAPTER 5

A CLOSER LOOK AT DHV AND THE 30TH HIGHEST HOUR CRITERIA: DOES THE LONG-HELD DEFINITION FOR AN APPROPRIATE DESIGN SERVICE VOLUME FIT TRAFFIC CHARACTERISTICS IN NEBRASKA?

As mentioned previously, the 2004 Green Book states that the hourly traffic volume that should generally be used for design is the 30th highest hourly volume (30HHV) of the year and that it may be used as the design service volume criteria for both rural and urban roadways (1). It further states that there is a significant break in the relationship between highest hour rank and percent of AADT near the 30th highest hour (1). The relationship is shown graphically in **FIGURE 1** (1). However, t-test comparisons of continuous count data in Nebraska show that the most significant break commonly occurs between the 14th and 24th highest hourly volumes in rural areas.

Appropriateness of 30th Highest Hourly Volume as the Design Service Volume in Nebraska

An analysis was conducted of 2001-2003 NDOR continuous count data to assess whether the 30th highest hour volume criteria is appropriate to use for a design service volume in Nebraska. Regression analysis was used to test the appropriateness of the use of 30 HV for the design of geometric features and traffic control solutions and to examine the design service volume criteria for each functional class of roadways in Nebraska.

Determination of the Significant Break Point for Each Functional Class of Roadway

To determine the significant break point for each functional class of roadway, the hourly volumes expressed as a percent of AADT were aggregated for all similar roadway types after checking that they were not significantly different from one year to the other using a t-test for two samples of unequal variances. The average hourly volumes over three years for each functional class were treated as separate data sets. These hourly volumes were plotted against the number of hours on the horizontal axis and a best-fit regression line was fit to the data points. Log regression was used for this analysis based on its high R^2 values when compared with linear or ordinary least squares (OLS) regression.

FIGURE 15 shows the best-fit regression curve for the functional class of Rural Major Collectors. The goal was to determine the location of the first significant break in the curve before or after the 30HHV, as 30HHV may not be appropriate for use as a representative design service volume in Nebraska.

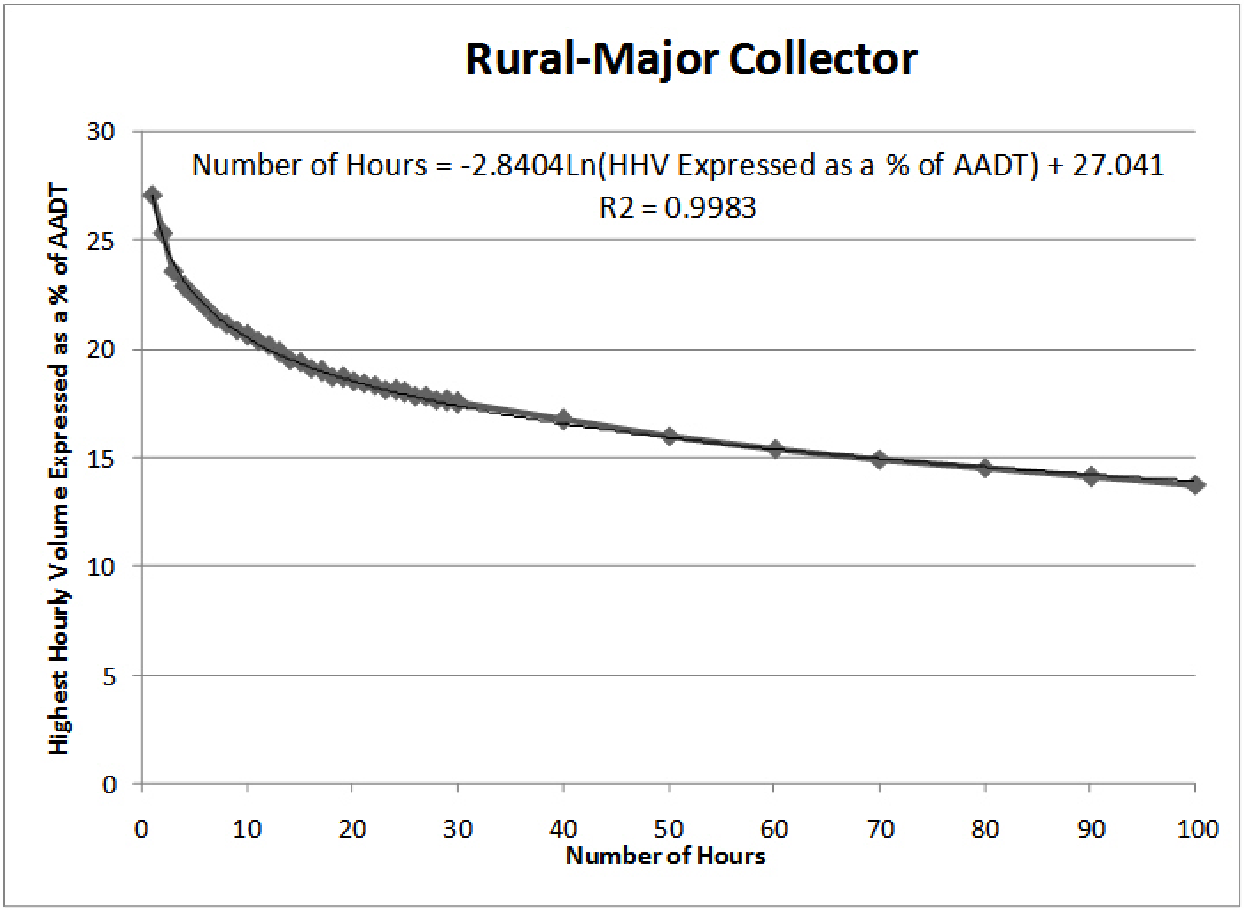

FIGURE 15 Best-Fit Curve for Highest Hourly Volumes for the Functional Category of Rural Major Collector in Nebraska Using 2001-2003 Continuous Count Data (9, 10, 11)

To find the point of significant change in the data, a scatter plot using logarithmic axes was created so the resulting best-fit curve would be linear as shown in **FIGURE 16**. The test was to locate the first significant change in linearity before 30HV. To find the significant change or break point, the data were divided into two separate data sets, the first one comprising values from the 30th highest hour to the 100th highest hour, and the second one from the 1st highest hour to the 29th highest hour. The R^2 of both the data sets and their difference was noted. This process was repeated, adding the last point in first set to the second set and eliminating that point from the first set. The break point was considered to be the point at which the difference in R^2 values between the two data sets was greatest taking into consideration the linearity of the complete data set.

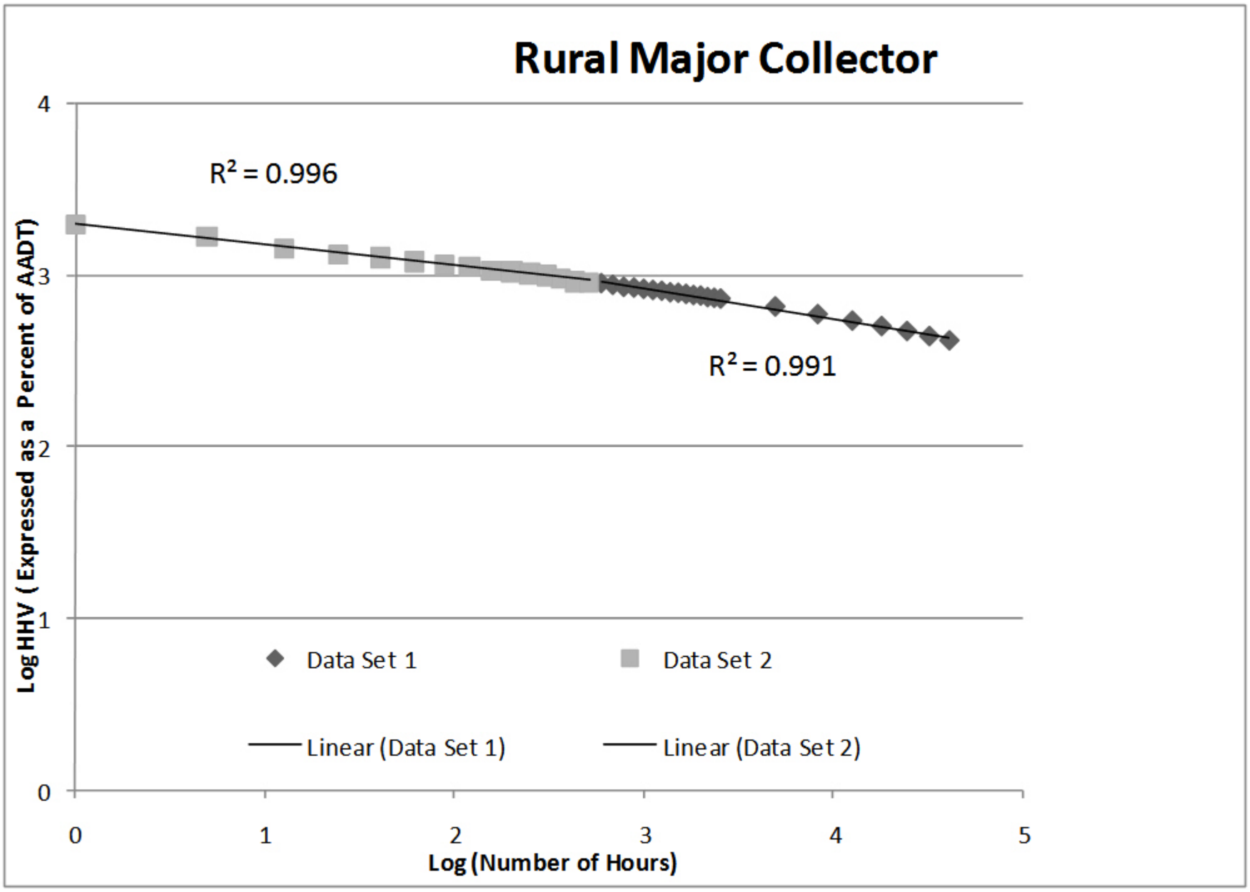

FIGURE 16 Comparison of Data Sets to Find Significant Change in DHV as a Percent of AADT Relationship of Rural Major Collectors (9, 10,11)

In **FIGURE 16**, the break point occurred at the 17th highest hour for Rural Major Collectors. Similarly, the break points were determined for all functional classes of roadways available. Related figures for other functional classes are shown in Appendix C and summarized in **TABLE 17**.

TABLE 17 Determination of the Point of Significant Change in the Highest Hourly Volume Curve Based on Roadway Functional Classification

Roadway Functional Class	Highest Hourly Volume Indicating Significant Slope Change (Break Point)
Rural Major Collector	17
Rural Minor Arterial	14
Rural Principle Arterial-Other	24
Rural Principle Arterial Interstate	20
Urban Minor Arterial	30
Urban Principle Arterial-Other	28
Urban Principle Arterial Interstate	23

It appears from the results that the rural roadways deviate more from the 30th highest hour criteria than do the urban roadways, which tend to closely match the 30th highest hour break point.

Determination of the Hourly Volume that the Average Peak Hour Volume Represents

Knowing the number of times the PHV will be exceeded annually is important to assist in evaluating the level of service at which a roadway segment or intersection will perform in high traffic situations. To determine the number of hours in a year with hourly volume greater than the PHV expressed as a percent of AADT, a basic linear regression equation was used of the form given below:

$$y = \beta_0 + \beta_1 x + \varepsilon \qquad \textbf{EQUATION 13}$$

where,

y = hourly traffic expressed as a percent of AADT,
x = number of hours with hourly volume greater than average PHV,
β_0 = estimated parameter for the constant,
β_1 = estimated parameter for the coefficient of regression, and
ε = random error term

In **EQUATION 13** the independent variable was the hourly traffic expressed as a percentage of AADT and the dependent variable was the number of hours. Linear regression equations were developed for the graphs as shown in **FIGURE 18** which were used to extrapolate the number of hours in a year that have more volume than the average peak hour volume in that given year.

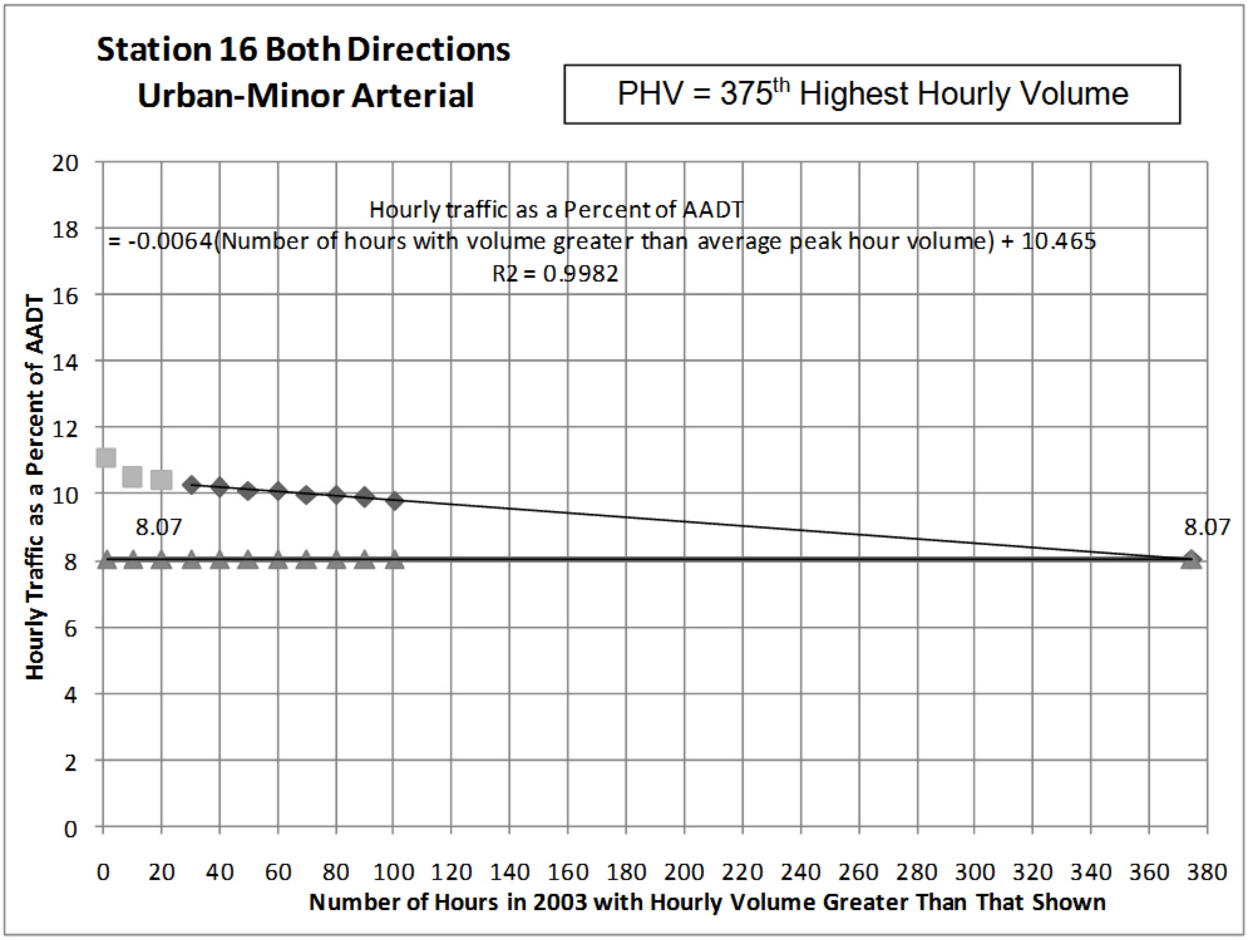

FIGURE 17 Estimating the Hourly Volume Which the Average Peak Hour Volume Represents by Extrapolation of NDOR Continuous Count Data, 2001-2003 (9, 10, 11)

To determine how many hours in a year have more volume than the peak hour traffic expressed as a percentage of AADT at a particular count station, the independent variable y in the regression equation developed was replaced with average hourly volume expressed as a percent of AADT (8.07 at this particular count station). The result (i.e., extrapolated hour) determines how many hours have more volume than the average peak hour volume in a year.

In **FIGURE 17**, it can be seen that for Count Station 16, there are 374 hours during the year 2003 with more traffic volume than the average peak hour volume expressed as a percent of AADT (8.07% of the AADT). Data from all available count stations for the period 2001-2003 were analyzed using the same technique. The extrapolated hours were then further divided by functional classification and by AADT for analyzing the results. The extrapolated hour changed, based on the functional type of roadway and the AADT traversing it.

From the data analysis, it was found that the average peak hour was equivalent to the 270th HHV of the year for all counter stations from 2001-2003. The Average Peak Hour Volume-Highest Hour Equivalent (APHV-HHE) for all roadways studied was 259, 272 and 281 for the years 2001, 2002 and 2003 respectively. These values show the average number of hours during which the average peak hour volume was exceeded for that specific year. Therefore, use of PHV as an estimate for design may not be appropriate for Nebraska or at least must be understood as a value that can be exceeded many times during the year.

Classification of APHV-HHE by Functional Type of Roadway
APHV-HHE values for each of the functionally classified roadways for the years 2001-2003 is shown in **FIGURE 18**. The value of APHV-HHE is greater on urban roadways than on rural roadways by about 100, due to the morning and afternoon peak hours in densely populated areas.

FIGURE 19 shows the maximum, minimum, mean and median statistics of the three-year APHV-HHEs. These statistics were compared to check for the variability and the standard deviation in results. Urban roadways showed more variability than rural roadways either because of the fluctuation of traffic conditions on urban roadways or due to the fact that fewer number of urban roadway counter stations were available for use in the analysis. Only 20 percent of all the roadways studied were located in urban settings and the large deviation in the maximum and minimum three-year APHV-HHEs may be attributed to this fact.

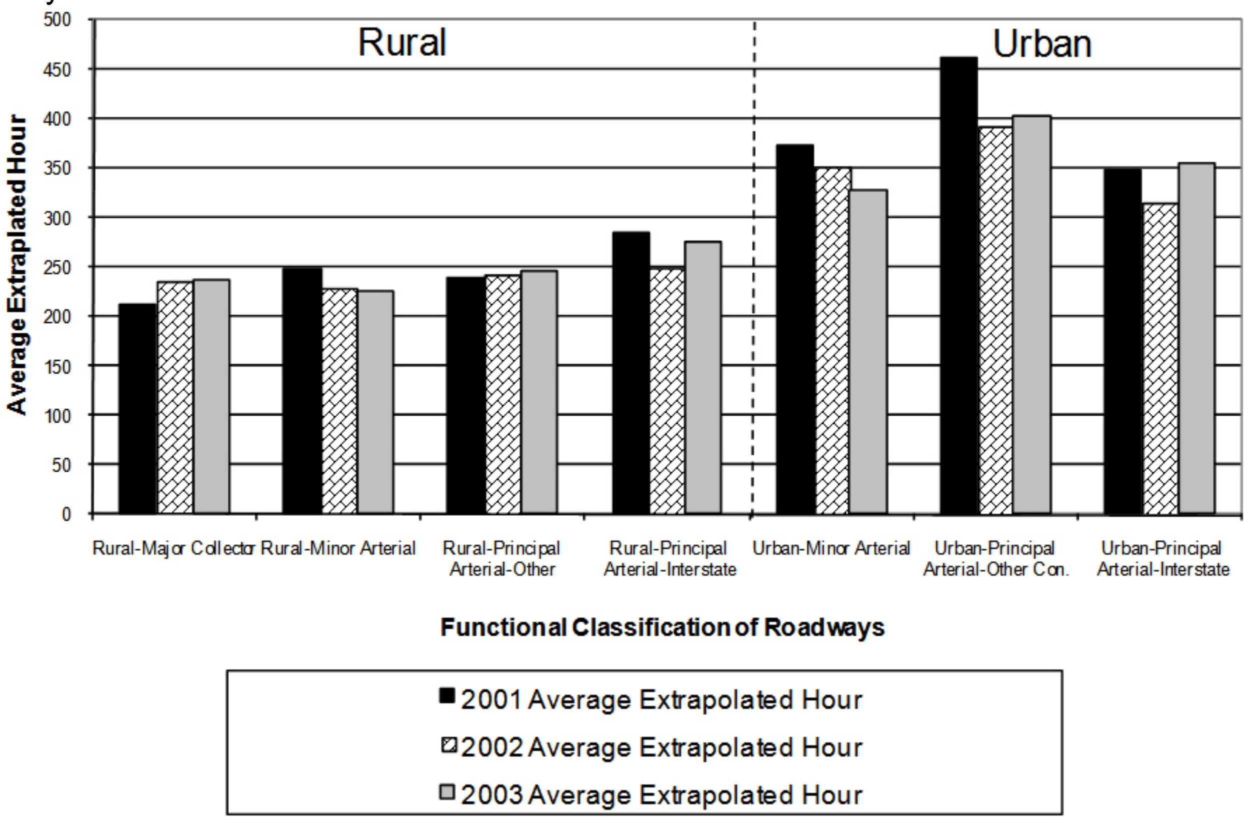

FIGURE 18 Average PHV Highest Hour Equivalent Classified by Roadway Functional Type, 2001-2003 (9, 10, 11)

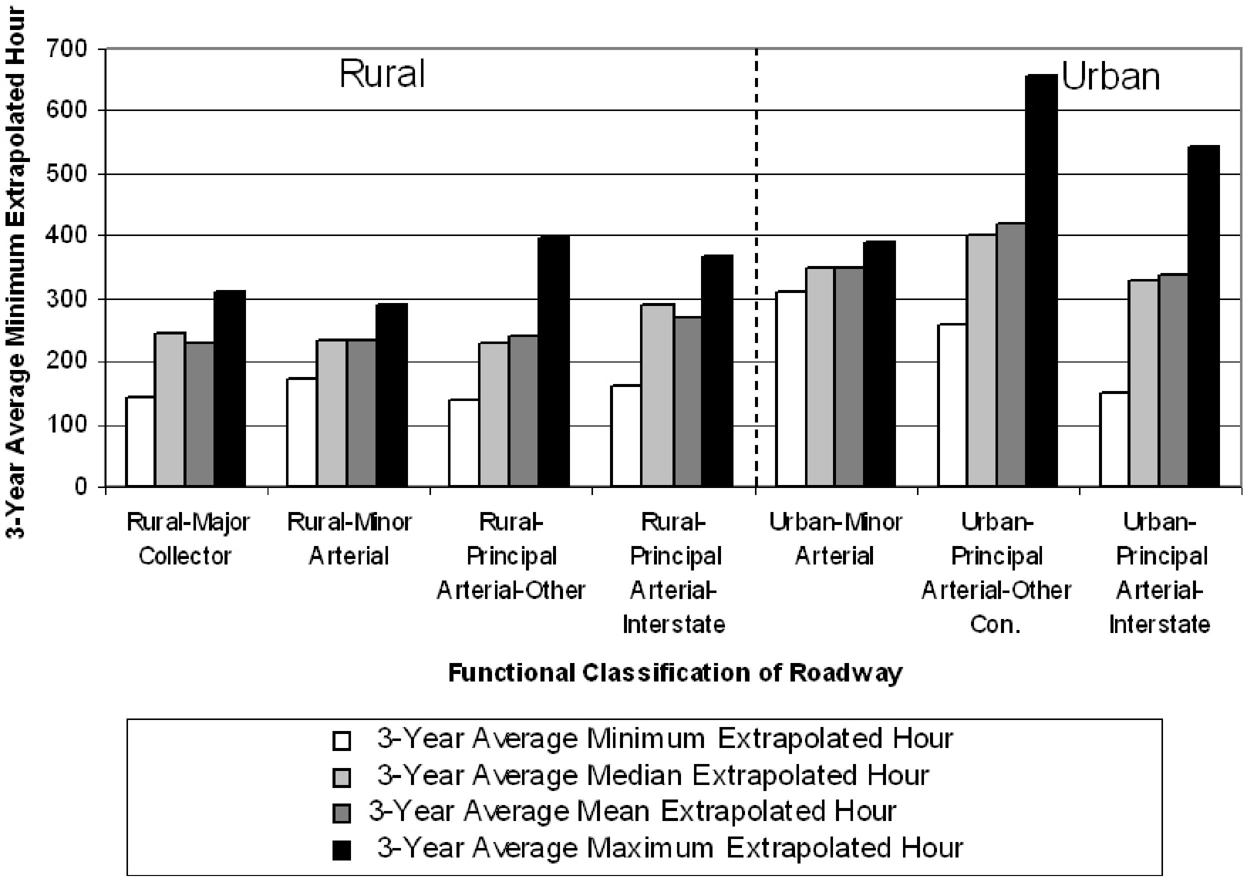

FIGURE 19 Descriptive Statistics for the Three-Year (2001-2003) Average PHV Highest Hour Equivalent Classified by Roadway Functional Type (9, 10, 11)

Classification of APHV-HHE by AADT of Roadway

The counter station data was categorized by AADT and the APHV-HHEs were determined for each data grouping. This type of classification was conducted mainly to understand the variation in the results with the AADT and with the functional classification. **FIGURE 20** shows that the APHV-HHEs increase with an increase in AADT except for the group of roadways with AADT between 8000-10000 vpd. This may be explained by the fact that only 2 counter stations represent this category.

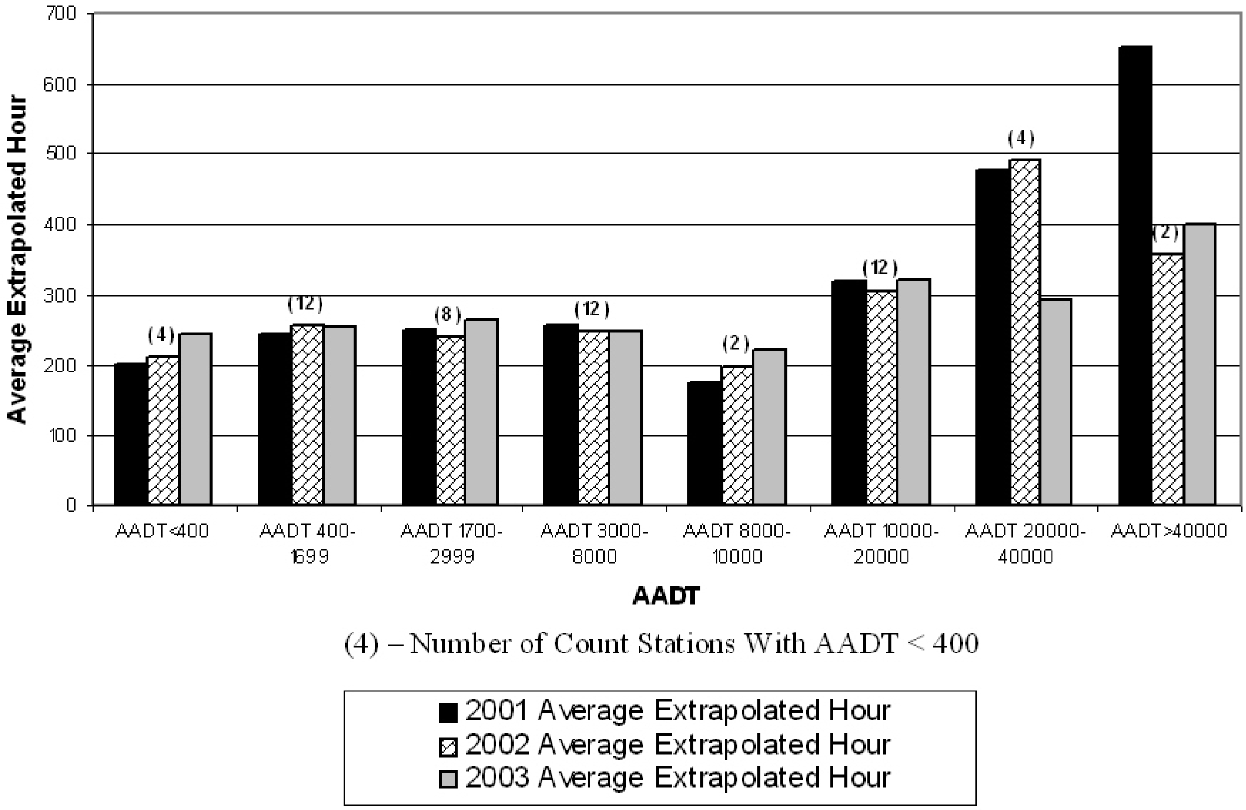

FIGURE 20 Three-Year (2001-2003) Average PHV Highest Hour Equivalent Based on AADT (9, 10, 11)

In **FIGURE 20**, it is evident that the 3-year APHV-HHEs with AADT less than 10,000 is between the 200th to 250th highest hour. To generalize the nature of the data, the roadways are classified into four groups of AADT ranges. **FIGURE 21** depicts the mean, maximum, minimum and median statistics of APHV-HHEs. These values estimate the variability and standard deviation of the APHV-HHEs based upon the AADT on those roadways.

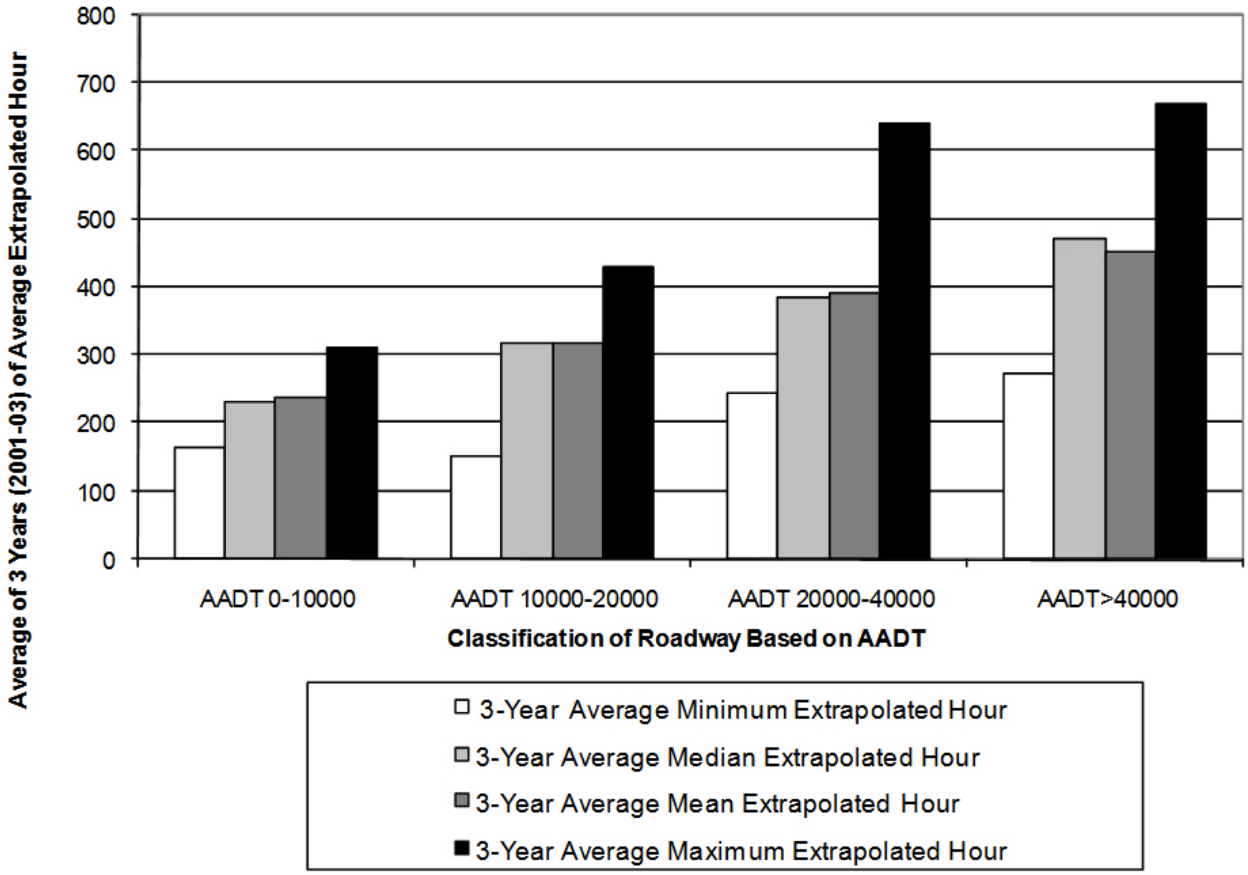

FIGURE 21 Descriptive Statistics for Three-Year (2001-2003) Average PHV Highest Hour Equivalents Classified by AADT (9, 10, 11)

TABLE 18 lists a best-fit equation developed from the AADT groupings shown in **FIGURE 21**. Given the AADT category of a facility and a desired highest hourly volume, one can estimate the volume which would approximate the design service volume required to match that desired highest hourly volume.

TABLE 18 Regressed Equations for Estimating Service Volumes Equivalent to Given Highest Hourly Volume.

AADT Category, Vehicles per Day	Service Volume Estimate for Given Highest Hourly Volume, Percent of AADT	Equation Number	R^2
1 to 10,000	y = - 0.021x + 12.99	14	0.95
10,000 to 20,000	y = - 0.013x + 11.28	15	0.94
20,000 to 40,000	y = - 0.011x + 11.27	16	0.94
Greater than 40,000	y = - 0.005x + 10.06	17	0.90

x = Desired Highest Hourly Volume, ranging from 1 to 8,760 hours in a year
y = Hourly Traffic Volume as a Percentage of AADT, vehicles per hour

Examples of Service Volume Estimation for Desired Highest Hourly Volume Using Generated Relationships Given AADT

The relationships established in this project between highest hourly volume and AADT can be used for service volume estimation for different AADT traffic volume categories.

<u>Example 1 Design Service Volume Estimate Based on Demand Exceeding Capacity of an Acceptable Level of Service Once or Twice per Week in the Design Year</u>

Given:

AADT of 16,000 vpd in the Design Year

Estimate:

Service volume estimate of the 52^{nd} highest hour (assumes that the system capacity meets the design service volume all hours except 1 hour per week) and 104^{th} highest hour (assumes that the system capacity meets the design service volume all hours except 2 hours per week) of the design year.

$y = -0.013x + 11.28$ **EQUATION 15**

Percentage of $AADT_{52nd\ HHV} = -0.013\ (52) + 11.28 = 10.604\%$ of 16000 = **1697 vph**

Percentage of $AADT_{104th\ HHV} = -0.013\ (104) + 11.28 = 9.928\%$ of 16000 = **1589 vph**

If the facility were designed for a peak volume of 1697 vph, it is likely the design capacity would be exceeded 1 hour each week during the year (say one afternoon peak per week). If the facility were designed for a peak volume of 1589 vph, it is likely the design capacity would be exceeded 2 hours each week during the year (say one morning and one afternoon peak or 2 afternoon peaks per week).

By using the equations in **TABLE 18**, one could estimate the number of hours during the year that a specific design volume would be exceeded to evaluate different performance levels for a given design and to evaluate cost differences for those performance levels based on the construction costs of the given geometry and traffic control devices used.

CHAPTER 6

CONCLUSIONS

Summary of Results

Traffic volume estimating is critical for planning, designing and maintaining a reasonable quality of service along surface transportation facilities. Reliable estimation of traffic volumes is needed to realistically assess problems and determine appropriate solutions that meet the expectations of the traveling public. Regression equations were developed in this project to find the relationship between average peak hour volume and the design hourly volume/average annual daily traffic to ensure the appropriate design of geometric and traffic control improvements that best fit traffic characteristics in Nebraska.

Comparisons using t-test analyses were conducted to check for the significant change in the relationships developed between average peak hour volumes and the highest hourly volumes. The results of the t-test comparisons indicated that the significant volume break occurs between 14th and 24th highest hourly volumes, depending on the functional type of the roadway and not at the 30th highest hourly volume for the analyzed data which is commonly accepted. Urban data fit the 30th HHV criteria fairly well as shown in **TABLE 17**.

Basic linear regression equations were used to extrapolate the number of hours in a year that have more volume than the average peak hour volume in that given year. From the data analysis, it was found that the average peak hour was equivalent to the 270th HHV of the year for all counter stations from 2001-2003. The Average Peak Hour Volume-Highest Hour Equivalent (APHV-HHE) for all roadways studied was 259, 272 and 281 for the years 2001, 2002 and 2003 respectively. These values show the average number of hours during which the average peak hour volume was exceeded for that specific year. Therefore, use of average PHV as an estimate for design may not be appropriate for Nebraska or at least must be understood as a value that can be exceeded many, many times during the year.

The traditional definitions of average peak hour volume (PHV), design hourly volume (DHV) described as the 30th highest traffic hour volume of the year, and the average annual daily traffic (AADT) were verified by using continuous traffic count data from NDOR. The study resulted in the following conclusions.

- The average peak hourly volume can be reasonably estimated if the DHV (defined as the 30th highest hourly volume of the year) or the AADT volume is known.
- Conversely, if the average PHV is established from an actual traffic count, the DHV or AADT can be reasonably estimated.
- Nebraska traffic characteristics indicate that a significant change in the rate of traffic increase as a percent of the AADT occurs between the 14th and 24th highest hours of the year or 0.16 or 0.27 percent of the total number of annual hours for rural type roadway which represents 47 to 67 percent of the 30 hour criteria. This differs from the commonly excepted value of the 30th highest hourly volume as the point where there is a significant change in volume which represents about 0.34 percent of the total annual hours.

- The location of significant change on urban type roadways closely approximates the 30th highest hour criteria representing 77 to 100 percent of the 30 hour criteria.
- The average peak hourly volume may be exceeded between 200 to 400 hours annually, depending on the functional classification of the roadway. Assuming that these 200 to 400 hours would likely be during the weekday morning or evening peak hours (which would be a total of 5 days per week multiplied by 2 peaks per day multiplied by 52 weeks or 520 hours annually), using the average peak hourly volume for geometric and traffic control design purposes would mean the volume of traffic would exceed the design service volume 38 to 77 percent of the total number of peak hours in the year. If the goal was for the design of the facility to only be exceeded 30 hours in the design year (about 0.34 percent of the total annual hours in a future year), the design would fall severely short of its goal. The result would be the appearance that the improvement was ineffective, poorly designed and a source of frustration to the traveling public.

The Need for Consistency

Although there are many aids for design and traffic engineers to analyze conditions to provide suitable solutions, the process of arriving at those solutions may take many forms, depending on the individual who is responsible for the analysis. It is highly recommended that the knowledge gained by this research project be used with the process defined in NCHRP 457 to define a more realistic range of traffic volume data that can be used in a consistent methodical process to determine the optimal geometric and traffic control solution for a given performance level in a given design year.

Defining a specific procedure to follow that is the result of a Transportation Research Board (TRB) project supported by the American Association of State Highway and Transportation Officials (AASHTO) will allow NDOR to review analysis results for all situations in a consistent format that may allow a more predictable performance product than previously experienced.

Limitations

The traffic data used for analysis was in the form of averages. The use of aggregate data reduces the total variability and nature of variability associated with the statistical relationship (12). Predictions from models based on aggregate data may appear to be more precise than they truly are. Data aggregation may also affect the prediction measures. Therefore, results from this research should be used knowing this limitation which is mainly due to the use of continuous traffic count data, which is presented as aggregate data.

One way to quantify the error in this research is to use disaggregate data for doing similar analysis and comparing those results with those from this research. However, disaggregate data were not available for this research. Further analysis is needed using disaggregate data in the future to quantify the error in this research and to validate the results obtained from this research.

CHAPTER 7
A PROCEDURE FOR THE OPTIMAL CHOICE OF GEOMETRIC AND TRAFFIC CONTROL SOLUTIONS FOR ROADWAY IMPROVEMENTS

***Choice of Reasonable Level of Service*:**
The procedure should begin with the recommendations for level of service provided by the 2004 Green Book shown in **FIGURE 2** to determine a “desirable” segment or intersection performance level depending on the functional type of roadway, terrain and population density area type.

***Choice of Design Year*:**
A reasonable design year should be chosen based on the authorized agency’s planning documents. If the situation is that of a new development adjacent to an existing facility, practitioners agree that geometric and traffic control improvements should exceed the traffic demand at the opening of the development and should provide what is agreed upon by stakeholders to be a reasonable level of service about 5 years beyond the predicted ultimate build-out of the development. The proximity of the location with respect to fringe areas of growing communities should be carefully considered as these areas can grow quickly at rates which are difficult to predict.

Consistent Methodology Through the Use of NCHRP 457:
FIGURE 22 is reproduced from NCHRP 457. It shows the process of assessing viable alternatives, narrowing the field of solutions and selecting the best alternative for improvement. A selection of candidate alternatives should be compiled before traffic data is collected to make sure that the appropriate field data is available for later analysis.

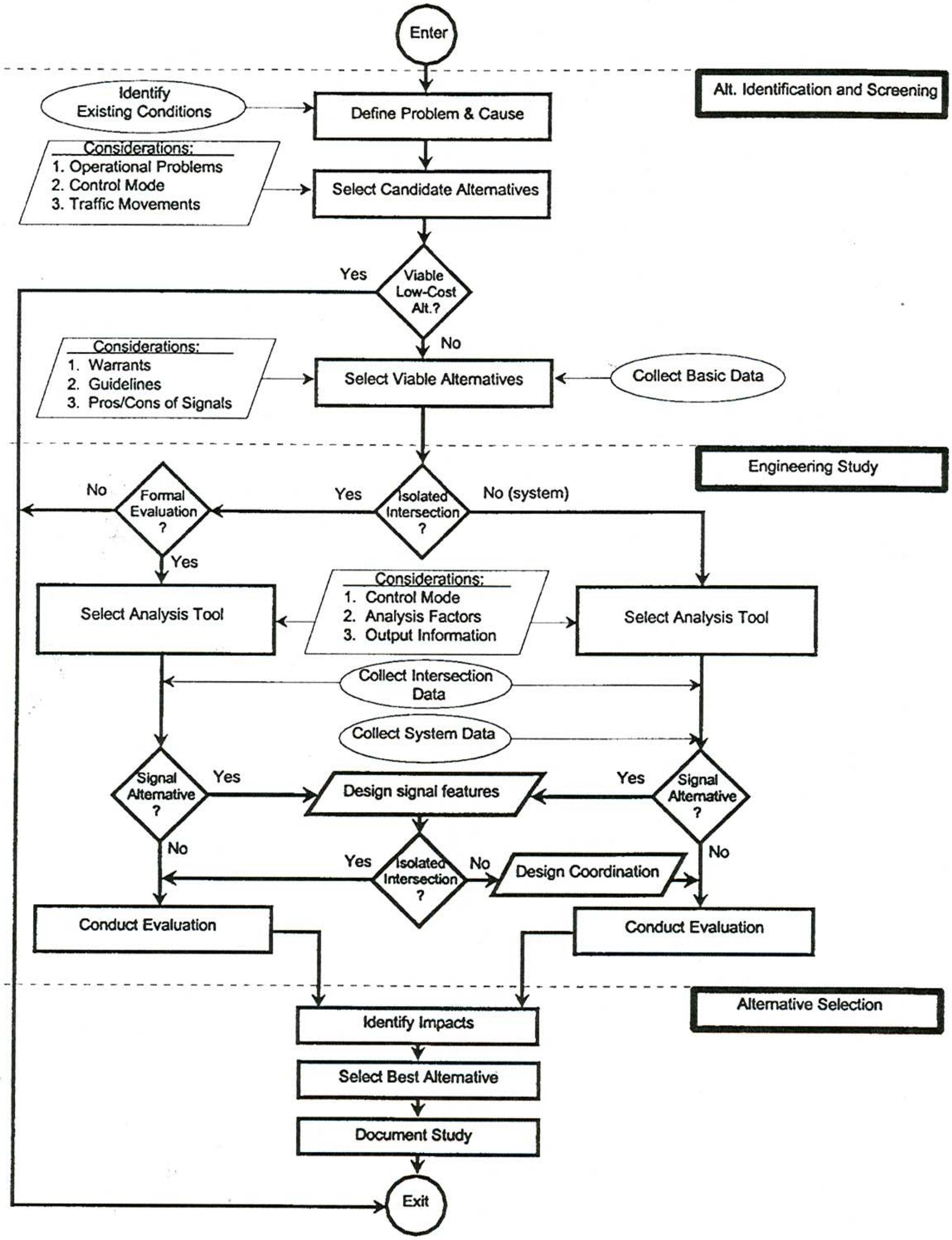

FIGURE 22 Flow Chart of the NCHRP 457 Assessment Process (Page 3, 8)

Traffic estimates required for different alternatives given in NCHRP 457 are listed in **TABLES 5 and 6**.

Including Project Recommendations in the NCHRP 457 Process:
Traffic data recommended in **TABLES 5 and 6** should be gathered using accepted traffic engineering procedures. Suggestions are given in Chapter 1. Once traffic counts are made and the peak hour volume determined, the DHV and AADT should be estimated from the developed regression equations in this project and compared to the best DHV and AADT information currently available. If reliable DHV and/or AADT for the segment is available, the PHV estimate should be calculated from the developed equations and compared to the traffic count PHV. A range of values should be used for analysis using **EQUATIONS 5, 6, 11 AND 12**: low, average and high if such distinctions appear in the field and estimated data. Simulations should be run using all possible volume ranges to see how the system would operate given the possibility that the field peak hour volume is inaccurate.

If an AADT is known for a facility, an estimate of the service volume for given hourly volumes can be estimated with **EQUATIONS 14-17** to give an idea of the performance level of a given design.

It should be noted that the NCHRP 457 process does not include a safety impact assessment. The expected safety of the optimal solution choice should be evaluated in some way, whether it is an informal subjective assessment or a formal quantitative evaluation. Suggestions are given in examples shown in NCHRP 457.

Evaluation of Cost of Desirable Level of Service:
Evaluate the cost of attaining the desirable level of service once an optimal solution is found and determine if it is economically feasible, given budgetary constraints of the funding agency.

Revise Expectations to Better Match Funding Capabilities:
If funding is not available to provide the desired level of service, reduce performance expectations to a more affordable range and iterate design and traffic control options until a reasonable level of service is balanced with available funding.

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