CHAPTER 8

TRAFFIC CHARACTERISTICS

CONTENTS

EXHIBITS

I. VEHICLE AND HUMAN FACTORS

Three major components affect driving: the vehicle, the roadway/environment, and the driver. In this chapter, vehicle and driver characteristics and how they are affected by the environment and physical properties of the roadway are identified. The characteristics and performance of motor vehicles play a major role in defining the fundamentals of traffic flow and capacity. Human behavior also contributes to the characteristics of traffic flow on a facility.

MOTOR VEHICLE CHARACTERISTICS

Characteristics of various roadway users

This section summarizes the operating characteristics of motor vehicles that should be considered in analyzing a facility. The major considerations are vehicle types and dimensions, turning radii and offtracking, resistance to motion, power requirements, acceleration performance, and deceleration performance. Motor vehicles include passenger cars, trucks, vans, buses, recreational vehicles, and motorcycles. These vehicles have unique weight, length, size, and operational characteristics. The forces that must be overcome by motor vehicles if they are to move are rolling, air, grade, curve, and inertial resistance. The weight/power ratios are useful for indicating the overall performance in overcoming these forces. Exhibit 8-1 summarizes typical motor vehicle weight and power for different vehicle types.

EXHIBIT 8-1. MOTOR VEHICLE WEIGHT AND POWER

Motor Vehicles	Empty Weight with Driver (kg)	Nominal Power (kW)	Weight-to-Power Ratio (kg/kW)
Passenger car	1540	78	19.7
Large pickup truck	1905	130	14.7
Two-axle, six-tire truck	4535	130	34.9
Tractor-semitrailer	11,340	242	46.9

Source: *Traffic Engineering Handbook* (*1*).

Vehicle acceleration and deceleration rates are factors in designing traffic signal timing, computing fuel economy and travel time values, and estimating how normal traffic flow is resumed after a breakdown. Vehicle acceleration rates of passenger cars accelerating after a stop range between 1 and 4 m/s^2, while passenger car deceleration rates range between 2 and 8 m/s^2 (*1*).

DRIVER CHARACTERISTICS

Driving is a complex task involving a variety of skills. The most important of these skills involve taking in and processing information and making quick decisions based on this information. Driver tasks are categorized into three main elements: control, guidance, and navigation. Control involves the driver’s interaction with the vehicle in terms of speed and direction (accelerating, braking, and steering). Guidance refers to maintaining a safe path and keeping the vehicle in the proper lane. Navigation means planning and executing a trip.

The perception and processing of information are important driver characteristics. About 90 percent of the information a driver receives is visual. A significant component in the successful processing and use of information is the speed with which this processing is done. One of the parameters that is used to quantify the speed of processing information is perception-reaction time, which represents how quickly a driver can respond to an emergency situation. The parameter called sight distance is directly associated with reaction time. There are three types of sight distance: stopping, passing, and decision. This parameter is used to determine geometric features of transportation facilities.

Other factors like nighttime driving, fatigue, driving under the influence of alcohol and drugs, elderly drivers, and police enforcement contribute to driver behavior on a transportation facility. All these factors can affect the operational parameters of speed, delay, and density.

PEDESTRIAN CHARACTERISTICS

Pedestrian speed is probably the most important characteristic of a pedestrian facility that is affected by individual pedestrian behavior and habit. Among several factors that influence walking speed are density, gender, size of platoon, percentage of elderly population, handicapped pedestrian population, and child pedestrian population. An average walking speed of 1.2 m/s is appropriate for typical groups of pedestrians. The amount of space required by a queued or standing pedestrian is 0.75 m^2. At signalized intersections pedestrian crossings must be assigned an amount of effective green time based on average walking speed.

BICYCLE CHARACTERISTICS

The bicycle and the bicyclist have very different characteristics and exhibit different operation than do drivers of motor vehicles. The typical speed of bicycles is about 25 km/h. Among other factors that affect bicycles are the type of bicycle, the bike path surface type, weather conditions, the grade of the path, and the mix of other nonmotorized users on the bike path.

BUS AND LIGHT RAIL CHARACTERISTICS

Bus and light rail capacity are affected by vehicle type, loading area performance, and dwell time of the bus or light rail vehicle. Each bus requires a certain amount of service time at stops that varies with the number of boarding and alighting passengers, door configuration, and fare collection method. The minimum safe spacing between buses in motion and the number of loading areas available at any stop also influence the total number of buses and persons that a given facility can carry.

The total passenger flow rate varies with bus capacity and the trade-off between seated capacity and standees. The largest number of seats and lowest number of standees should occur on longer suburban bus routes or on intercity bus routes where higher levels of comfort are essential. A typical 14-m urban transit bus can normally seat 43 passengers and carry up to 37 standees if the aisle circulation space is filled. Similarly, an 18-m articulated bus can carry 65 seated passengers and 55 standees. However, bus operator policy often limits the number of standees to levels below this theoretical capacity.

II. DEMAND AND VOLUME

In this manual, demand is the principal measure of the amount of traffic using a given facility. Thus, the term demand relates to vehicles arriving, while the term volume relates to vehicles discharging.

Traffic demand varies by month of the year, day of the week, hour of the day, and subhourly interval within the hour. These variations are important if highways are to effectively serve peak demands without breakdown. The effects of a breakdown may extend far beyond the time during which demand exceeds capacity and may take up to several hours to dissipate. Thus, highways minimally adequate to handle a peak-hour demand may be subject to breakdown if flow rates within the peak hour exceed capacity.

Seasonal peaks in traffic demand are also of importance, particularly for recreational facilities. Highways serving beach resort areas may be virtually unused during much of the year, only to be subject to oversaturated conditions during peak summer periods.

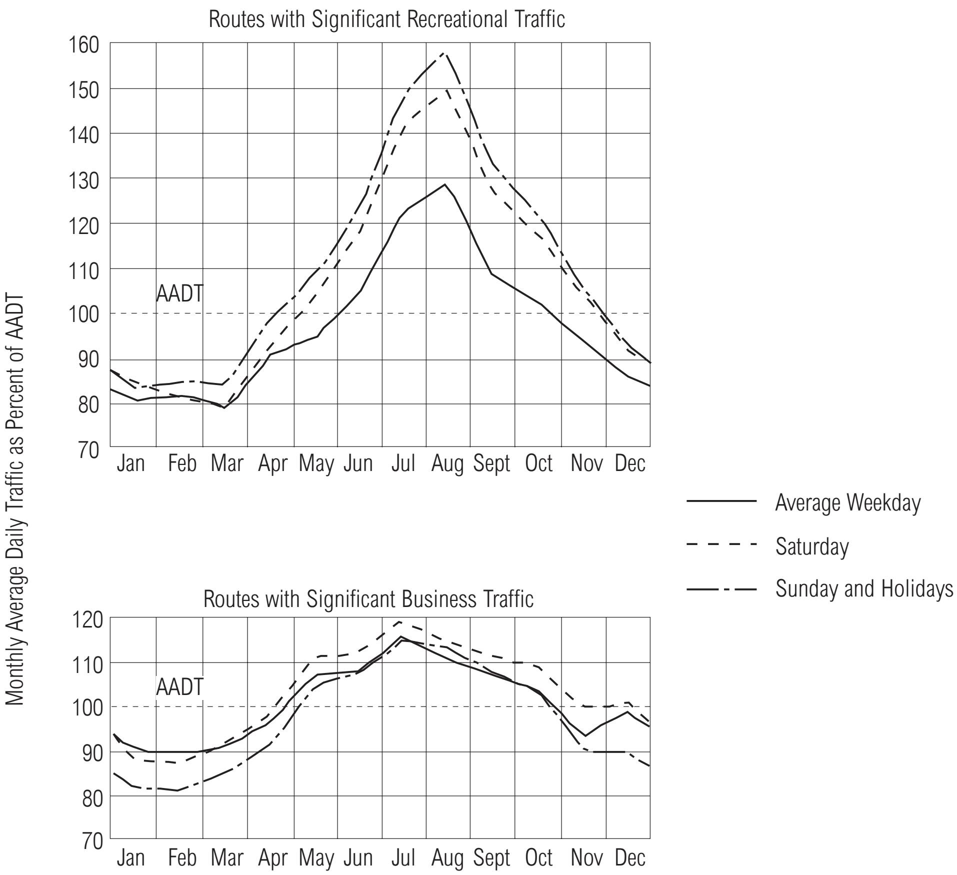
SEASONAL AND MONTHLY VARIATIONS

Seasonal fluctuations in traffic demand reflect the social and economic activity of the area being served by the highway. Exhibit 8-2 shows monthly variation patterns observed in Minnesota. Several significant characteristics are apparent:

- Monthly variations are more severe on rural routes than on urban routes,
- Monthly variations are more severe on rural routes serving primarily recreational traffic than on rural routes serving primarily business traffic, and
- Daily traffic patterns vary by month of year most severely for recreational routes.

Volume variations by month and type of day

EXHIBIT 8-2. EXAMPLES OF MONTHLY TRAFFIC VOLUME VARIATIONS FOR A FREEWAY

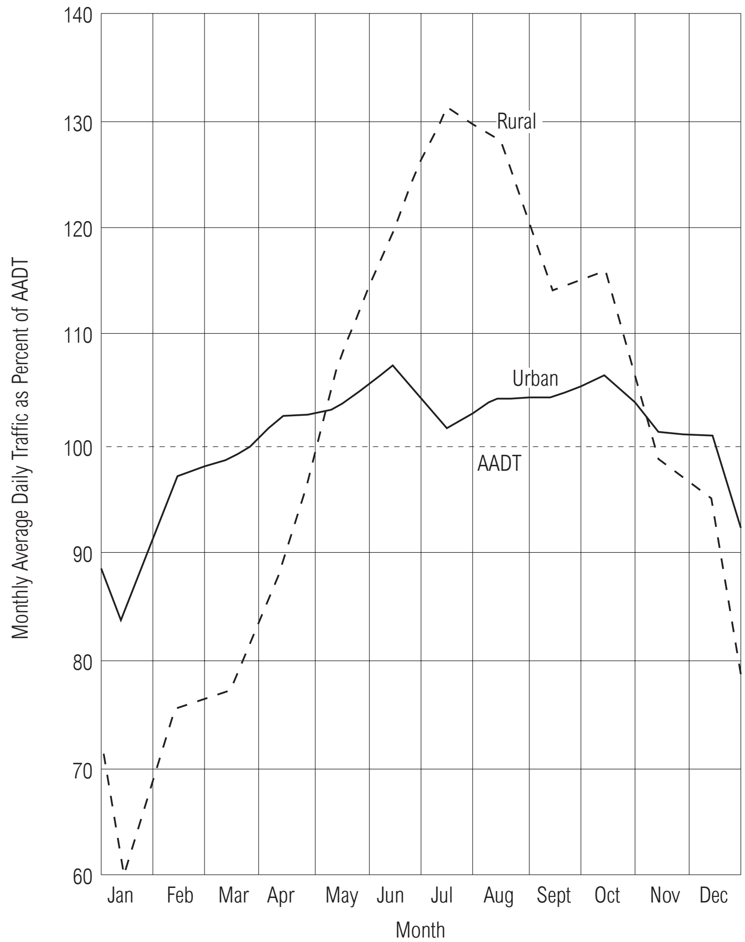


Source: Minnesota Department of Transportation.

These observations lead to the conclusion that commuter and business-oriented travel occurs in more uniform patterns and that recreational traffic creates the greatest variation in volume patterns.

The data for Exhibit 8-3 were collected on the same Interstate route. One segment is within 1.6 km of the central business district of a large metropolitan area. The other segment is within 80 km of the first but serves a combination of recreational and intercity business travel. The wide variation in seasonal patterns for the two segments underscores the effect of trip purpose and may also reflect capacity restrictions on the urban section.

EXHIBIT 8-3. EXAMPLES OF MONTHLY TRAFFIC VOLUME VARIATIONS FOR THE SAME INTERSTATE HIGHWAY (RURAL AND URBAN SEGMENTS)



Source: Muranyi (*2*).

Time of peak demand will vary according to highway type

DAILY VARIATIONS

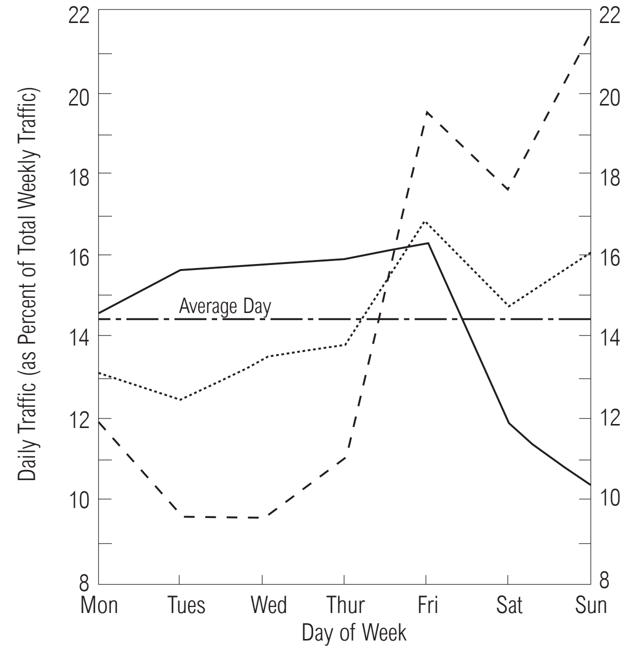
Volume variations by day of the week are also related to the type of highway on which observations are made. Exhibit 8-4 shows that weekend volumes are lower than weekday volumes for highways serving predominantly business travel, such as urban freeways. In comparison, peak traffic occurs on weekends on main rural and recreational highways. Furthermore, the magnitude of daily variation is highest for recreational access routes and lowest for urban commuter routes.

Exhibit 8-5 shows the variation in traffic by vehicle type for the shoulder lane of an urban freeway. Although the values shown in Exhibits 8-4 and 8-5 are typical of patterns that may be observed, they are not meant to substitute for local studies and analyses. The average daily traffic averaged over a full year is referred to as the annual average daily traffic, or AADT, and is often used in forecasting and planning.

HOURLY VARIATIONS

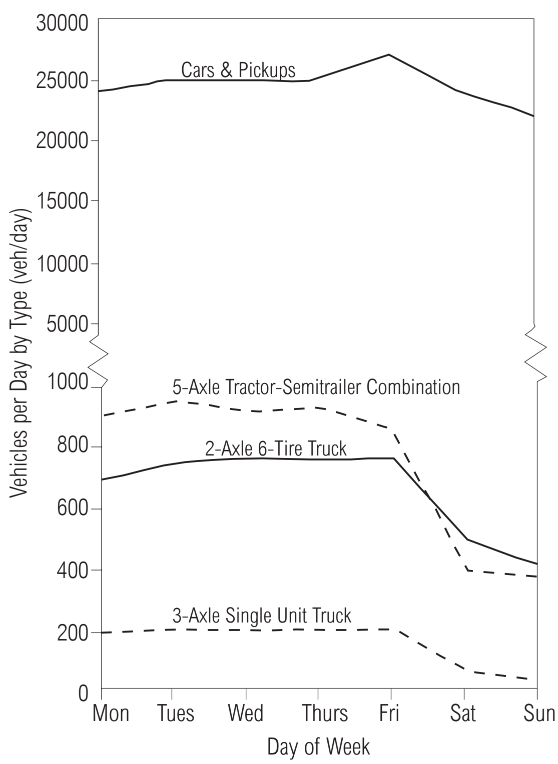
Typical hourly variation patterns are shown in Exhibit 8-6, where the patterns are related to highway type and day of the week. The typical morning and evening peak hours are evident for urban commuter routes on weekdays. The evening peak is generally somewhat more intense than the morning peak, as shown in Exhibit 8-6. On weekends, urban routes show a peak that is less intense and more spread out, occurring in early to midafternoon.

EXHIBIT 8-4. EXAMPLES OF DAILY TRAFFIC VARIATION BY TYPE OF ROUTE



Main rural route I-35, Southern Minnesota, AADT 10,823, 4 lanes, 1980.
Recreational access route MN 169, North-Central Lake Region, AADT 3,863, 2 lanes, 1981.
Suburban freeway, four freeways in Minneapolis-St. Paul, AADTs 75,000-130,000, 6-8 lanes, 1982.
Average day.
Source: Minnesota Department of Transportation.

EXHIBIT 8-5. DAILY VARIATION IN TRAFFIC BY VEHICLE TYPE FOR RIGHT LANE OF AN URBAN FREEWAY

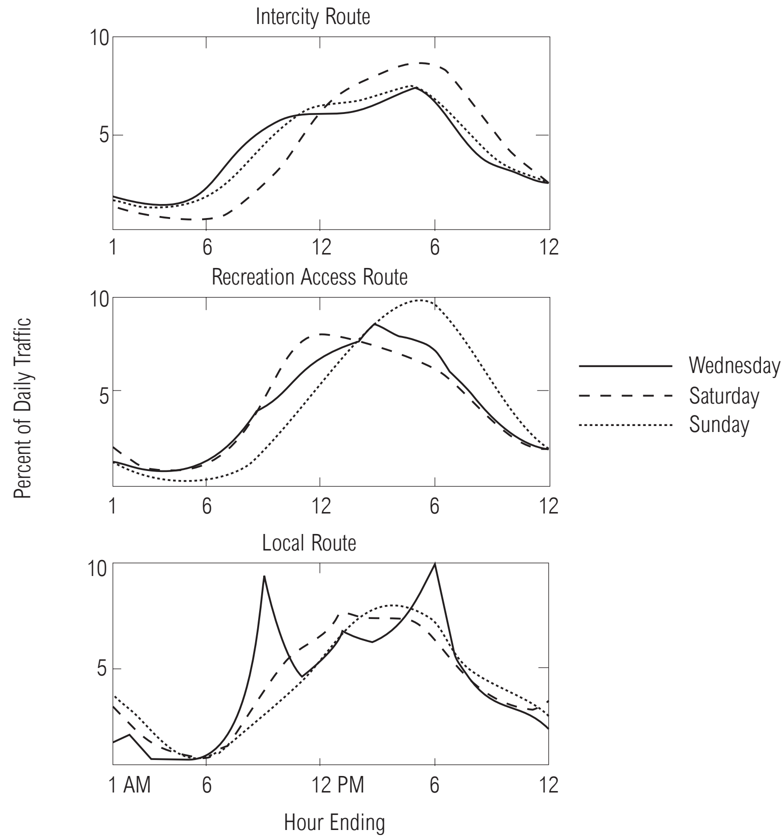


Data were collected on I-494, 4 lanes, in Minneapolis-St. Paul.
Source: Minnesota Department of Transportation.

Variation by day of week and route type for various types of vehicles

Traffic variation during the day by day of week

EXHIBIT 8-6. EXAMPLES OF HOURLY TRAFFIC VARIATIONS FOR RURAL ROUTES



Source: *Transportation and Traffic Engineering Handbook* (*3*).

Recreational routes also have single daily peaks. Saturday peaks on such routes tend to occur in the late morning or early afternoon (as travelers go to their recreational destination) and in late afternoon or early evening on Sundays (as they return home).

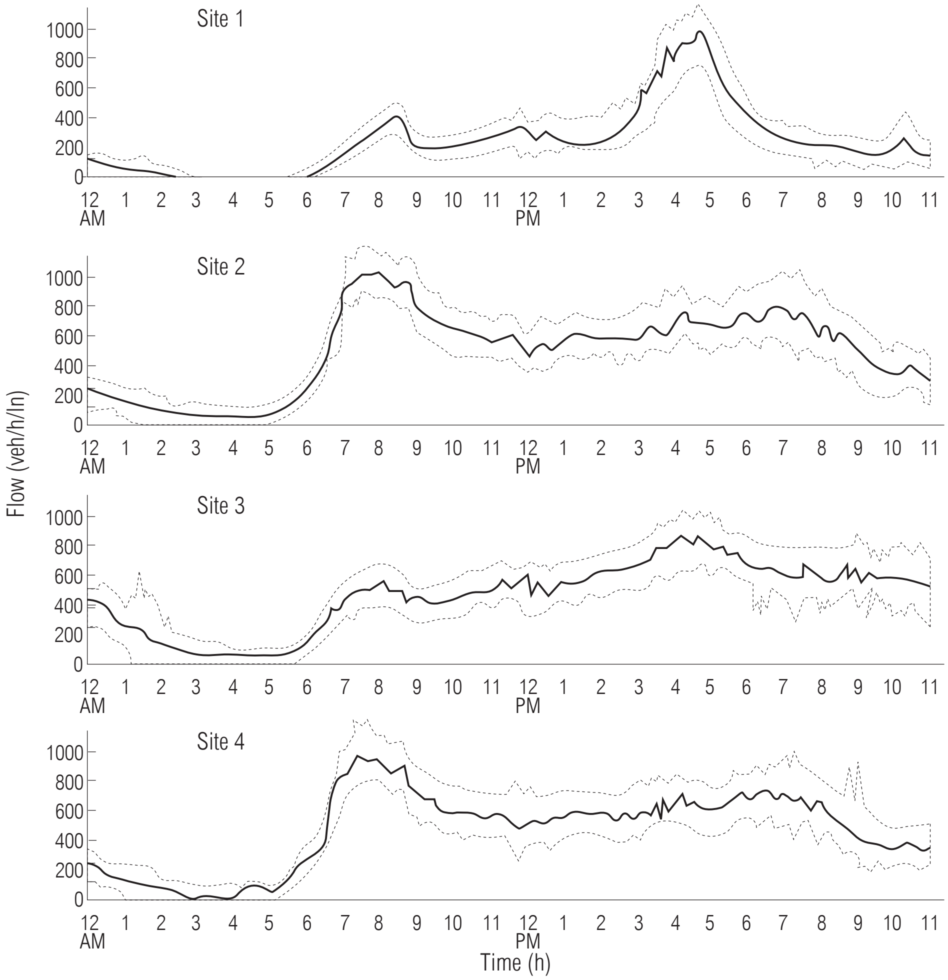
The repeatability of hourly variations is of great importance. The stability of peak-hour demand affects the feasibility of using such values in design and operational analysis of highways and other transportation facilities. Exhibit 8-7 shows data obtained in metropolitan Toronto. The area between the dotted lines indicates the range within which one can expect 95 percent of the observations to fall. Whereas the variations by hour of the day are typical for urban areas, the relatively narrow and parallel fluctuations among the days of the study indicate the repeatability of the basic pattern. The observations shown were obtained from detectors measuring traffic in one direction only, as evidenced by the single peak hour shown for either morning or afternoon.

It is again noted that the data of Exhibits 8-6 and 8-7 are typical of observations that can be made. The patterns illustrated, however, vary in response to local travel habits and environments, and these examples should not be used as a substitute for locally obtained data.

PEAK HOUR AND ANALYSIS HOUR

Capacity and other traffic analyses focus on the peak hour of traffic volume, because it represents the most critical period for operations and has the highest capacity requirements. The peak-hour volume, however, is not a constant value from day to day or from season to season.

EXHIBIT 8-7. REPEATABILITY OF HOURLY TRAFFIC VARIATIONS FOR URBAN STREETS



Note:
a. Sites 2 and 4 are one block apart on same street, in same direction.
b. All sites are two moving lanes in one direction.
Source: McShane and Crowley (*4*).

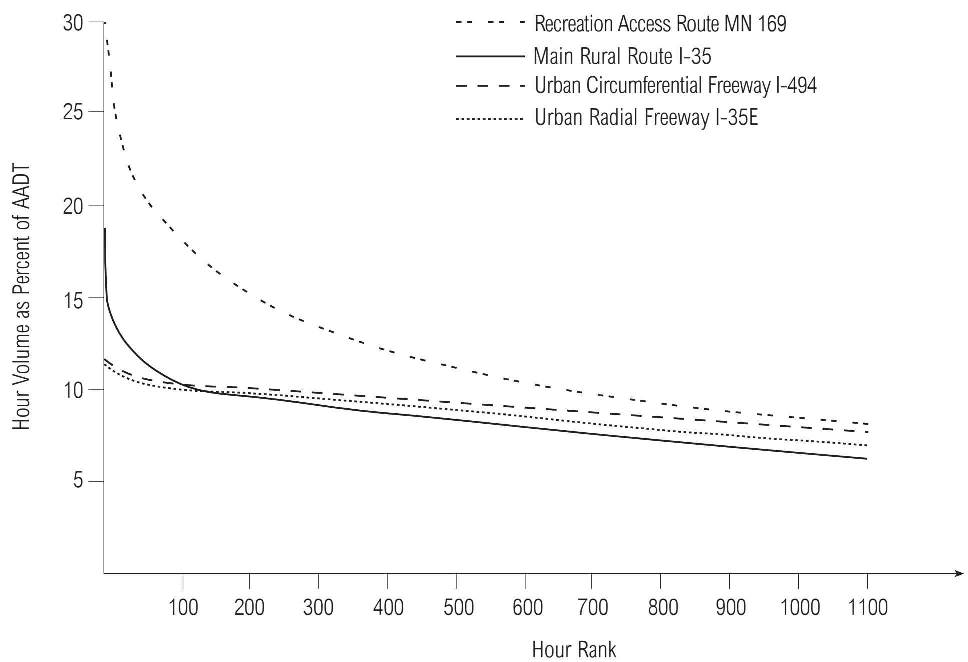
Repeatability of hourly patterns

Concept of peak hour and analysis hour

If the highest hourly volumes for a given location were listed in descending order, a large variation in the data would be observed, depending on the type of facility. Rural and recreational routes often show a wide variation in peak-hour volumes. Several extremely high volumes occur on a few selected weekends or in other peak periods, and traffic during the rest of the year is at much lower volumes, even during the peak hour. Urban streets, on the other hand, show less variation in peak-hour traffic. Most users are daily commuters or frequent users, and occasional and special event traffic are at a minimum. Furthermore, many urban routes are filled to capacity during each peak hour, and variation is therefore severely constrained.

Exhibit 8-8 shows hourly volume relationships measured on a variety of highway types in Minnesota. Recreational facilities show the widest variation in peak-hour traffic. Their values range from 30 percent of AADT in the highest hour of the year to about 15.3 percent of AADT in the 200th-highest hour of the year and 8.3 percent in the 1,000th-highest hour of the year. Main rural facilities also display a wide variation. The highest hour comprises 17.9 percent of the AADT, decreasing to 10 percent in the 100th-highest hour and 6.9 percent in the 1,000th-highest hour. Urban radial and circumferential facilities show far less variation. The range in percent of AADT covers a narrow band, from approximately 11.5 percent for the highest hour to 7 to 8 percent for the 1,000th-highest hour. Exhibit 8-8 is based on all hours, not just peak hours of each day, and shows only the highest 1,100 hours of the year.

EXHIBIT 8-8. RANKED HOURLY VOLUMES



Source: Minnesota Department of Transportation.

Selection of a peak demand usually implies that a small portion of the demand during a year will not be adequately served

The selection of an appropriate hour for planning, design, and operational purposes is a compromise between providing an adequate level of service (LOS) for every (or almost every) hour of the year and economic efficiency. Customary practice in the United States is to base rural highway design on an hour between the 30th- and the 100th-highest hour of the year. This range generally encompasses the knee of the curve (the area in which the slope of the curve changes from sharp to flat). For rural highways, the knee has often been assumed to occur at the 30th-highest hour, which is often used as the basis for estimates of design-hour volume. For urban roadways, a design hour for the repetitive weekday peak periods is common.

Past studies (*5,6*) have emphasized the difficulty in locating a distinct knee on hourly volume curves. These curves illustrate the point that arbitrary selection of an analysis hour between the 30th- and the 100th-highest hours is not a rigid criterion and indicate the need for local data on which to base informed judgments.

The selection of analysis hour must consider the impact on design and operations of higher-volume hours that are not accommodated. The recreational access route curve of Exhibit 8-8 shows that the highest hours of the year have more than twice the volume of the 100th hour, whereas the highest hours of an urban radial route are only about 15 percent higher than the volume in the 100th hour. Use of a design criterion set at the 100th hour would create substantial congestion on a recreational access route during the highest-volume hours but would have less effect on an urban facility. Another consideration is the LOS objective. A route designed to operate at LOS B can absorb larger amounts of additional traffic than a route designed to operate at LOS D during those hours of the year with higher volumes than the design hour. As a general guide, the most repetitive peak volumes may be used for the design of new or upgraded facilities. The LOS during higher-volume periods should then be tested as to the acceptability of the resulting traffic conditions.

The proportion of AADT occurring in the analysis hour is referred to as the K-factor, expressed as a decimal fraction:

K-factor defined

- The K-factor generally decreases as the AADT on a highway increases;
- The reduction rate for high K-factors is faster than that for lower values;
- The K-factor decreases as development density increases; and
- The highest K-factors generally occur on recreational facilities, followed by rural, suburban, and urban facilities, in descending order.

The K-factor should be determined, if possible, from local data for similar facilities with similar demand characteristics. Exhibit 8-9 presents an example of K-factors developed for Florida (*7*).

EXHIBIT 8-9. TYPICAL K-FACTORS

Area Type	K-Factor
Urbanized	0.091
Urban	0.093
Transitioning/Urban	0.093
Rural Developed	0.095
Rural Undeveloped	0.100

Source: Florida Department of Transportation (*7*).

The area types in Exhibit 8-9 are defined as follows:

- Urbanized areas are those designated by the U.S. Bureau of the Census.
- Urban areas are places with a population of at least 5,000 not already included in an urbanized area.
- Transitioning areas are the areas outside of, or urbanized areas expected to be included in, an urbanized area within 20 years.
- Rural areas are whatever is not urbanized, urban, or transitioning.

SUBHOURLY VARIATIONS IN FLOW

Volume forecasts for long-range planning studies are frequently expressed in terms of AADT (vehicles per day), subsequently reduced to hourly volumes. The analysis of LOS is based on peak rates of flow occurring within the peak hour. Most of the procedures in this manual are based on peak 15-min flow rates. Exhibit 8-10 shows the substantial short-term fluctuation in flow rate that can occur during an hour.

Variation in traffic within the hour

In Exhibit 8-10 the maximum 5-min rate of flow is 2,232 veh/h, whereas the maximum rate of flow for a 15-min period is 1,980 veh/h. The full hour volume is only 1,622 veh/h. A design for a peak 5-min flow rate would result in substantial excess capacity during the rest of the peak hour. A design for the peak-hour volume would result in oversaturated conditions for a substantial portion of the hour.

Consideration of these peaks is important. Congestion due to inadequate capacity occurring for only a few minutes can take substantial time to dissipate because of the dynamics of breakdown flow. Fifteen-min flow rates have been selected as the basis for most procedures of this manual. The relationship between the peak 15-min flow rate and the full hourly volume is given by the peak-hour factor (PHF). Whether the design hour is measured, established from the analysis of peaking patterns, or based on modeled demand, the PHF is applied to determine design-hour flow rates.

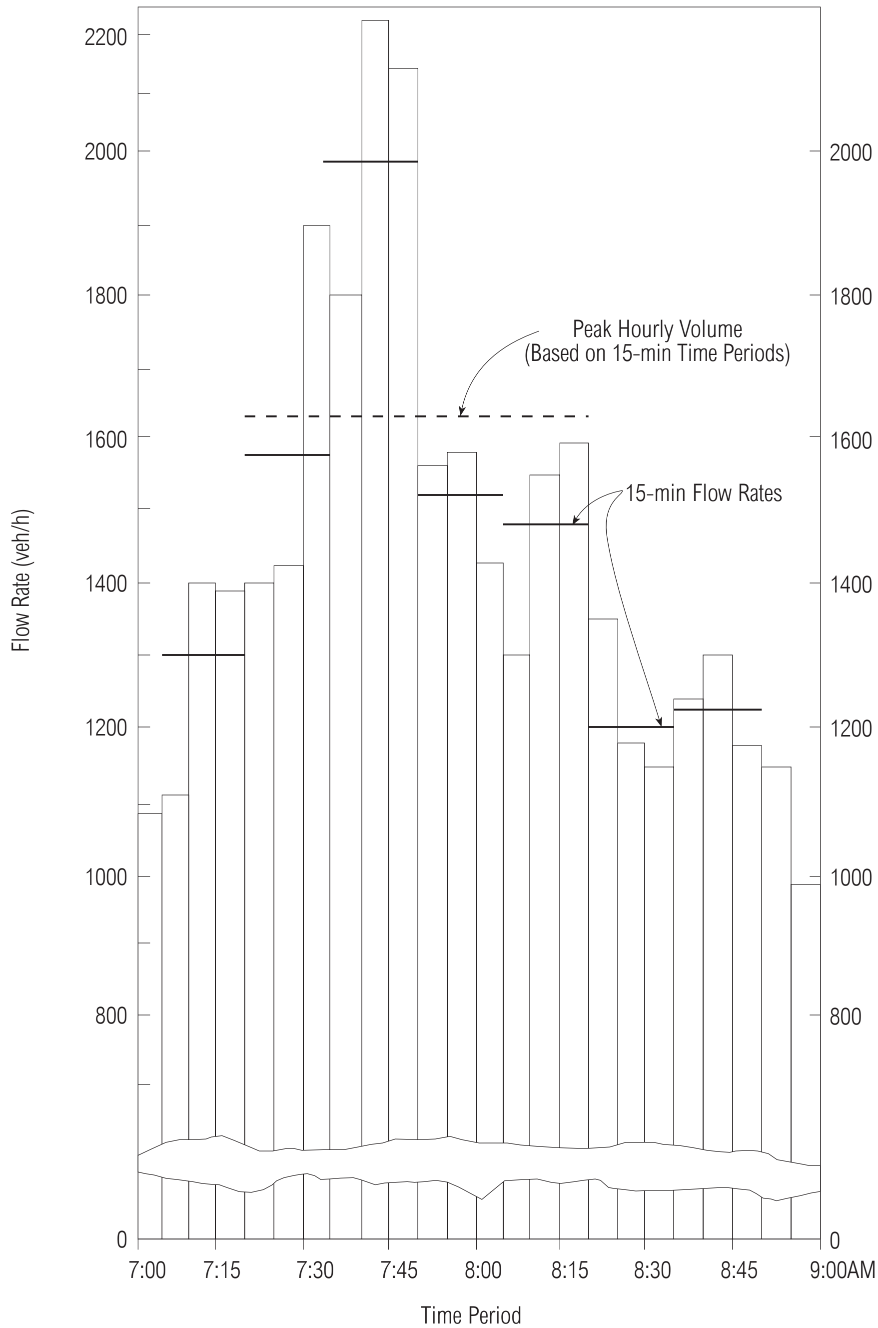
PHFs in urban areas generally range between 0.80 and 0.98. Lower values signify greater variability of flow within the subject hour, and higher values signify less flow variation. PHFs over 0.95 are often indicative of high traffic volumes, sometimes with capacity constraints on flow during the peak hour.

TEMPORAL DISTRIBUTIONS

The proportion of total daily traffic that occurs in the peak hour is defined by the K-factor. For many rural and urban highways, this factor falls between 0.09 and 0.10.

For highway sections with high peak periods and relatively low off-peak flows, the K-factor may exceed 0.10. Conversely, for highways that demonstrate consistent and heavy flows for many hours of the day, K-factors lower than 0.09 are often observed.

EXHIBIT 8-10. RELATIONSHIP BETWEEN SHORT-TERM AND HOURLY FLOWS



Source: Minnesota Department of Transportation.

SPATIAL DISTRIBUTIONS

Traffic volume varies in space as well as time. The two critical spatial characteristics in capacity analysis are directional distribution and volume distribution by lane. Volume may also vary longitudinally along various segments of a facility, but this does not explicitly affect capacity analysis computation because each facility segment serving different traffic demands is analyzed separately.

Directional Distribution

Concept of directional distribution

During any particular hour, traffic volume may be greater in one direction than in the other. An urban radial route, serving strong directional demands into the city in the morning and out at night, may display as much as a 2:1 imbalance in directional flows. Recreational and rural routes may also be subject to significant directional imbalances, which must be considered in analyses. Exhibit 8-11 gives the directional distribution on various highway types in Minnesota.

EXHIBIT 8-11. DIRECTIONAL DISTRIBUTION CHARACTERISTICS

	Percentage of Traffic in Peak Direction		
	Type of Facility		
Highest Hour of the Year	Urban Circumferential	Urban Radial	Rural
1st	53	66	57
10th	53	66	53
50th	53	65	55
100th	50	65	52

Source: Minnesota Department of Transportation.

Directional distribution is an important factor in highway capacity analysis. This is particularly true for two-lane rural highways. Capacity and level of service vary substantially on the basis of directional distribution because of the interactive nature of directional flows on such facilities. Procedures for two-lane highways include explicit consideration of directional distribution.

Whereas there is no explicit consideration of directional distribution in the analysis of multilane facilities, the distribution has a dramatic effect on both design and LOS. As indicated in Exhibit 8-11, up to two-thirds of the peak-hour traffic on urban radial routes has been observed to be moving in one direction. Unfortunately, this peak occurs in one direction during the morning and in the other in the evening. Thus, both directions of the facility must be adequate for the peak directional flow. This characteristic has led to the use of reversible lanes on some urban streets.

Directional distribution is not a static characteristic. It changes from year to year and by hour of the day, day of the week, and season. Development in the vicinity of highway facilities often induces traffic growth that changes the existing directional distribution.

The proportion of traffic moving in the peak direction of travel during peak hours is denoted as D. The K-factor, the proportion of AADT occurring in the analysis hour, was discussed previously. These two factors are used to estimate the peak-hour traffic volume in the peak direction using Equation 8-1:

$$DDHV = AADT * K * D \tag{8-1}$$

where

$DDHV$ = directional design-hour volume (veh/h),
$AADT$ = annual average daily traffic (veh/day),
K = proportion of AADT occurring in the peak hour, and
D = proportion of peak-hour traffic in the peak direction.

The product of the factors K and D is given for a number of facilities in Exhibit 8-12. The product gives the proportion of AADT occurring in the maximum direction of the peak hour.

EXHIBIT 8-12. OBSERVED VALUES OF K AND D ON SELECTED FREEWAYS AND EXPRESSWAYS

City and 1990 Urbanized Area Population	Facility	Year Count Taken	Number of Lanes	Annual Avg. Daily Traffic (2-Way)	Volumes in Peak Direction		Avg. Volume Per Lane (veh/h/ln)
					Vehicles (1-Way)	% 2-Way AADT (K * D)	
Atlanta, GA 2,157,806	I-20 E. of CBD at Moreland Ave.	1984	8	99,900	7794	7.8	1948
	I-20 at Martin Luther King Jr. Drive	1984	8	91,200	5198[a]	5.7[a]	1299[a]
	I-75 N. of CBD at University Ave.	1984	8	146,050	8179	5.6[a]	2045[a]
	I-75 N. of CBD (N. of I-85)	1984	8	82,830	5135	6.2[a]	1284[a]
	I-85 N. of I-75 at Monroe Dr.	1984	8	95,300	6765	7.1	1641
Boston, MA 2,775,370	I-93 N. of I-495	1984	6	76,500	5200	6.8	1733
	SE Expressway at Southampton St.	1982	6	143,300	6860	4.8	2286
	I-95 E. of Rt. 128 N. of Middlesex	1984	8	125,050	7282	5.8	1823
Denver, CO 1,517,977	I-25 S. of I-70	1984	8	175,000	7500	4.3	1875
	I-70, Colorado Blvd. to Dahlia	1984	6	114,000	4650	4.1	1550
	U.S. 6 W. of Federal Blvd.	1985	6	112,000	5835	5.2	1945
Detroit, MI 3,697,529	I-96 Jeffers Freeway at Warren	1980	8	67,600	6270	9.3	1568
	Lodge at E. Grant Blvd.	1981	6	111,450	5660	4.2	1558
Houston, TX 2,901,851	I-10 E. of Taylor St.	1985	10	151,000	7600	5.0	1520
	I-10 E. of McCarty	1985	8	110,200	7530	6.8	1882
	I-610 at Ship Channel	1985	10	103,200	5540	5.4	1108
Milwaukee, WI 1,226,293	N.-S. Freeway at Wisconsin	1984	8	118,080	5730	4.5	1342
	N.-S. Freeway at Greenfield	1984	8	110,050	6380	5.8	1595
	E.-W. Freeway at 26th St.	1984	6	121,150	5700	4.7	1900
	Zoo Freeway at Wisconsin	1984	6	110,730	4760	4.3	1581
	Airport Freeway at 68th	1984	6	81,020	3940	4.9	1313
New York, NY 16,044,012	Holland Tunnel	1982	4	73,200	2700	3.7	1350
San Francisco, CA 3,629,516	I-80 Oakland Bay Bridge	1984	10	223,000	8898	4.0	1780
Washington, D.C. 3,363,061	I-66 Theodore Roosevelt Bridge	1984	6	86,200	7413[a]	8.6[a]	2471[a]
	Anacostia Freeway at Howard Rd.	1984	6	121,700	6085[a]	5.0[a]	2028[a]

Note:
a. Values are based on K * D value for 1975.
Source: Levinson (*8*).

If average annual daily traffic is not known, it can be estimated from average weekday traffic using Equation 8-2 derived from the Highway Performance Monitoring System (HPMS) (*9*).

$$AADT = \frac{AWDT}{1.07} \tag{8-2}$$

where

$AADT$ = annual average daily traffic (veh/day), and
$AWDT$ = average weekday daily traffic (veh/day).

Lane Distribution

Concept of lane distribution

When two or more lanes are available for traffic in a single direction, the distribution in lane use varies widely. The volume distribution by lane depends on traffic regulations, traffic composition, speed and volume, the number and location of access points, the origin-destination patterns of drivers, the development environment, and local driver habits.

Because of these factors, there are no typical lane distributions. Data indicate that the peak lane on a six-lane freeway, for example, may be the shoulder, middle, or median lane, depending on local conditions.

Exhibit 8-13 gives daily lane distribution data for various vehicle types on selected freeways. The data are illustrative and are not intended to represent typical values.

EXHIBIT 8-13. LANE DISTRIBUTION BY VEHICLE TYPE

Highway	Vehicle Type	Percent Distribution By Lane		
		Lane 1[b]	Lane 2	Lane 3
Lodge Freeway, Detroit	Light[a]	29.2	38.4	32.4
	Single-Unit Trucks	30.8	61.5	7.7
	Combinations	88.5	2.9	8.6
	All Vehicles	30.9	37.8	31.3
I-95, Connecticut Turnpike	Light[a]	34.6	40.9	24.5
	All Vehicles	37.1	40.4	22.5
I-4, Orlando, Florida	All Vehicles	29.9	31.7	38.4

Notes:
a. Passenger cars, panel trucks, and pickup trucks.
b. Lane 1 = shoulder lane; lanes numbered from shoulder to median.
Source: Huber and Tracy (*10*); Florida Department of Transportation, 1993.

The trend indicated in Exhibit 8-13 is reasonably consistent throughout North America. Heavier vehicles tend toward the right-hand lanes, partially because they may operate at lower speeds than other vehicles and partially because of regulations prohibiting them from using leftmost lanes.

III. MEASURED AND OBSERVED VALUES

The methodologies in this manual are based on calibrated national average traffic characteristics observed over a range of facilities. Observations of these characteristics at specific locations will vary somewhat from national averages because of unique features of the local driving environment.

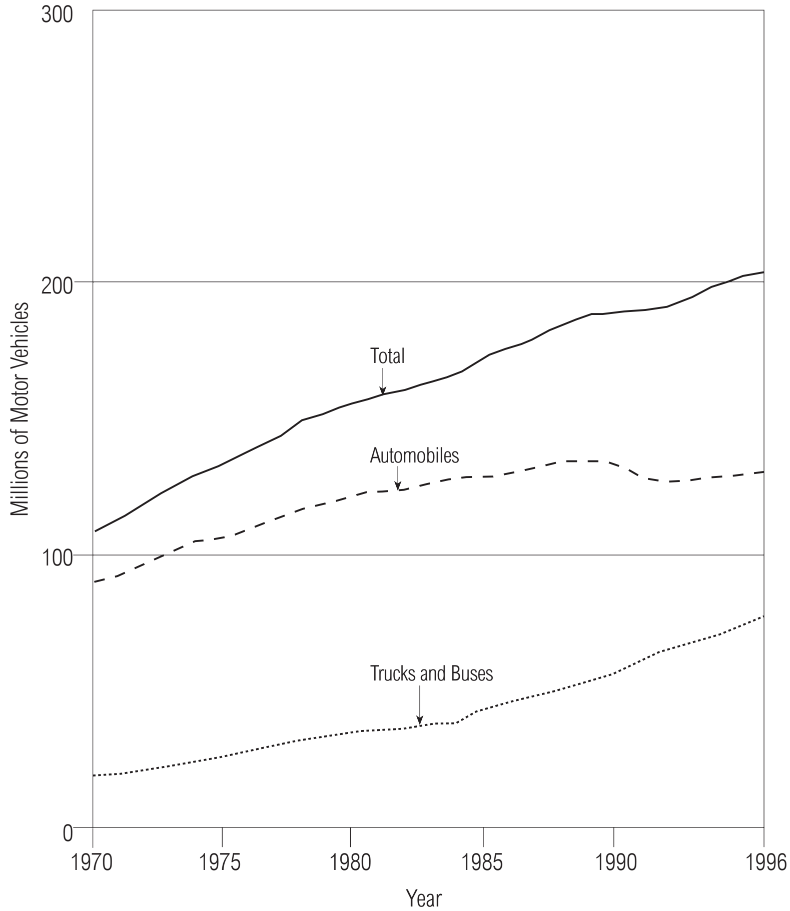
The levels and characteristics of traffic demand are constantly changing

The number of motor vehicles in the United States has been steadily increasing, with over 200 million registered vehicles in 1996. The increase during the 10-year period from 1986 was more than 17 percent (Exhibit 8-14). The number of passenger cars decreased during that period by 0.3 million, and the number of trucks grew by almost 30 million, with most of them in the light truck category. The number of motorcycles decreased from 5.2 million to 3.9 million.

On the rural Interstate system, automobiles and light trucks and buses account for 77 percent of average daily traffic volumes, with heavy trucks and buses representing the remainder (Exhibit 8-15). Annual travel on the roadways of the United States reached an estimated 4.0 trillion vehicle-km, or about three times the level reported in 1960, as shown in Exhibit 8-16. Travel grew about 47 percent during the 1960s, another 38 percent in the 1970s, and another 41 percent in the 1980s. Travel in urban areas accounted for 2.4 trillion vehicle-km in 1996, or 61 percent of the total, compared with 44 percent in 1960. The amount of travel in urban areas increased by almost 45 percent in the 1980s, faster than in rural regions, where growth was still very significant at 27 percent.

Exhibit 8-17 lists percentages of traffic distribution based on (a) vehicle classification data collected by states and compiled by the Federal Highway Administration and (b) information from the Bureau of the Census' Truck Inventory and Use Survey (TIUS) on the use of light trucks (*11*). The percentages in Exhibit 8-17 are daily values with peak-hour conditions being half or less than half of the numbers.

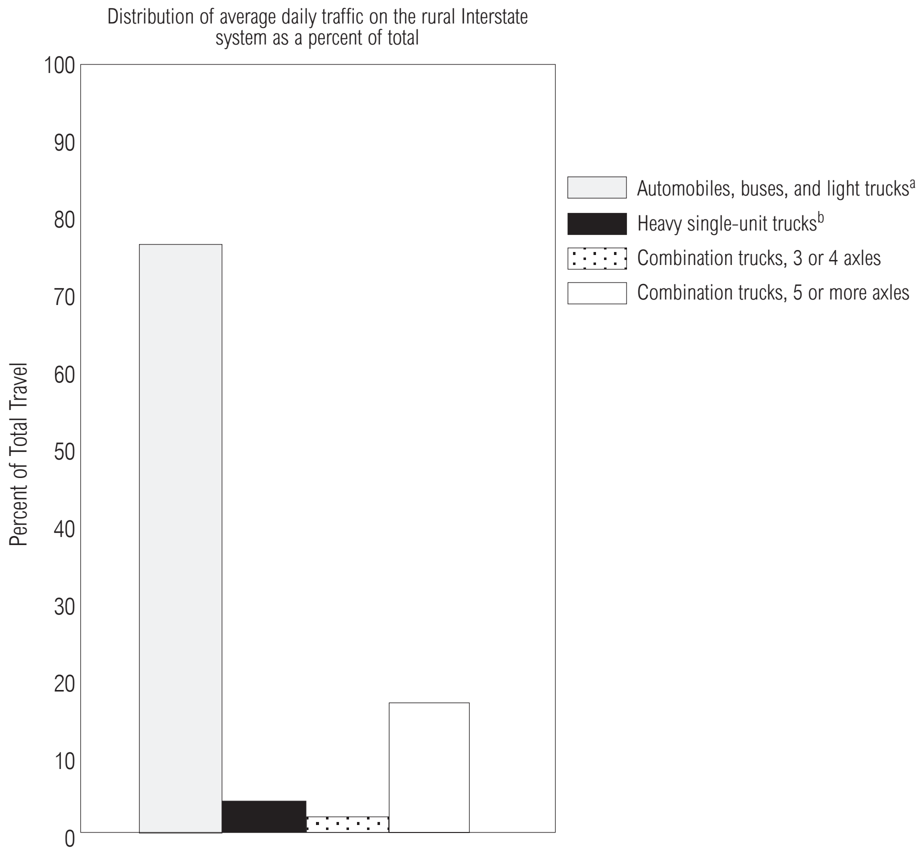
EXHIBIT 8-14. MOTOR VEHICLE REGISTRATION



Vehicle Type	1986	1996	Percent Change 1986 to 1996
Automobiles	130.0	129.7	-0.2
Buses	0.6	0.7	16.7
Trucks	45.1	75.9	68.3
P&C Light Trucks[a]	38.8	67.9	75.0
P&C Truck Tractors[a]	1.1	1.4	27.3
Other Single-Unit Trucks and Publicly Owned Trucks	5.2	6.6	26.9
Total	175.7	206.4	17.5
Motorcycles[b]	5.2	3.9	-25.0

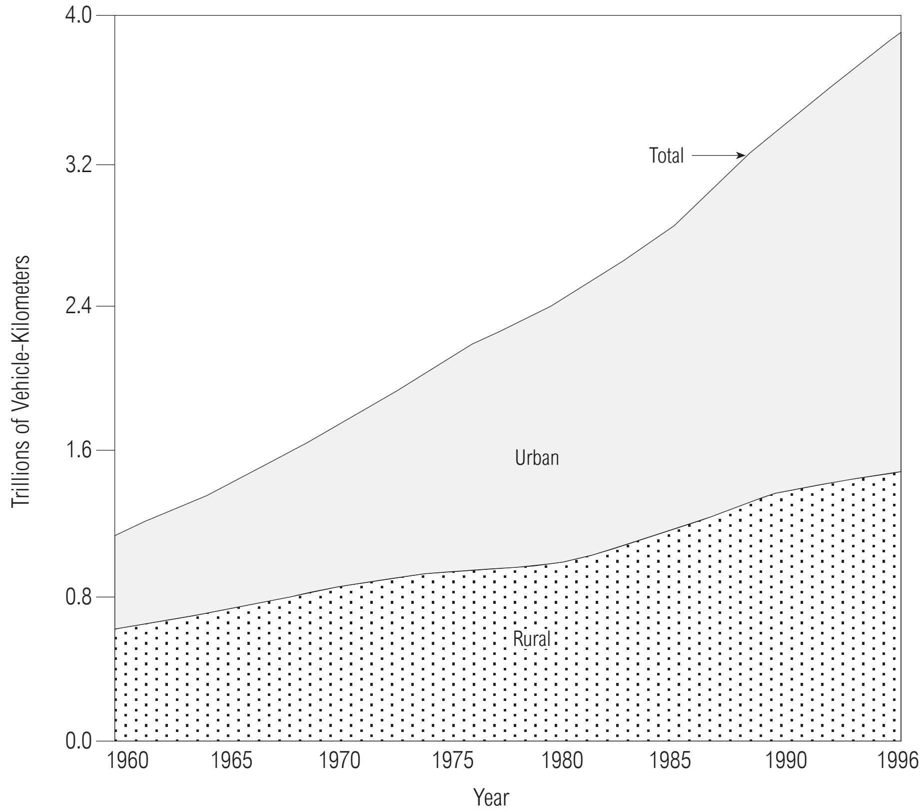
Notes:
a. Private and commercial.
b. Motorcycles not included in total.
Source: *Our Nation's Highways, Selected Facts and Figures*, Federal Highway Administration, 1996.

EXHIBIT 8-15. RURAL INTERSTATE TRAVEL BY VEHICLE TYPE



Notes:
a. All 2-axle, 4-tire trucks. Includes pickup trucks, panel trucks, vans, and other vehicles (campers, motor homes, etc.)
b. All vehicles on a single frame having either 2 axles and 6 tires or 3 or more axles (including camping and recreational vehicles and motor homes).
Source: *Our Nation's Highways, Selected Facts and Figures*, Federal Highway Administration, 1996.

EXHIBIT 8-16. ANNUAL VEHICLE KILOMETERS OF TRAVEL



Source: ***Our Nation's Highways, Selected Facts and Figures***, Federal Highway Administration, 1996.

EXHIBIT 8-17. PERCENT DISTRIBUTION OF TRAFFIC BY VEHICLE CLASS

Functional Class	Noncommercial Vehicles (%)	Commercial Vehicles Four-Tire (%)	Commercial Vehicles Single-Unit (%)	Commercial Vehicles Combination (%)	Total (%)
Rural					
Interstate	81.6	3.3	2.9	12.2	100
Other principal arterials	87.2	4.7	3.2	4.9	100
Minor arterial, collector and local	88.5	5.3	3.6	2.6	100
Average - rural	86.6	4.7	3.4	5.3	100
Urban					
Interstate	88.2	5.5	1.8	4.5	100
Other freeways and expressways	90.5	5.5	1.7	2.3	100
Other principal arterials	89.5	6.6	1.7	2.2	100
Minor arterials	90.4	6.4	1.7	1.5	100
Collectors	90.3	6.4	1.8	1.5	100
Local	91.0	6.4	1.8	0.8	100
Average - urban	89.8	6.2	1.7	2.3	100

Source: ***Quick Response Freight Manual*** (*11*).

VOLUMES AND FLOW RATES

Capacity is defined in terms of the maximum flow rate that can be accommodated by a given traffic facility under prevailing conditions. The determination of capacity involves the observation of highways of various types operating under high-volume conditions.

The direct observation of capacity is difficult to achieve for several reasons. The recording of a high, or even a maximum, volume or flow rate for a given facility does not ensure that a higher flow could not be accommodated at another time. Furthermore, capacity is sometimes not a stable operating condition.

The highest reported volumes and flow rates on facilities throughout the United States and Canada are identified in the following sections. The observations may or may not represent the absolute capacities of the subject highways and reflect prevailing conditions at the locations in question. These observations are a sample of high volumes recorded by state and local highway agencies and do not suggest that there are no other facilities experiencing similar, or even higher, volumes. In some cases, auxiliary lanes may be present, resulting in lower actual flows per lane than shown in the figures.

The data were collected from the literature and from surveys conducted by the Committee on Highway Capacity and Quality of Service of the Transportation Research Board and by the Federal Highway Administration over a number of years.

Freeways

The reported average annual daily traffic volumes on selected Interstate highways are given in Exhibit 8-18. Most of these high-volume freeways are found in the largest metropolitan areas. Daily traffic volumes on these heavily used highways exceed 200,000 veh/day. Exhibit 8-19 contains a sample of the maximum reported hourly one-way volumes and the average volumes per lane on rural and urban freeways in the United States. Most volumes in this table exceed 2,000 veh/h/ln, with several freeways featuring average lane volumes of more than 2,400 veh/h/ln. The highest reported lane volumes on selected freeways are given in Exhibit 8-20.

Freeway capacity analysis procedures of this manual use a rate of flow of 2,400 pc/h/ln for freeways with free-flow speeds of 120 km/h and 2,300 pc/h/ln for freeways with free-flow speeds of 110 km/h as the capacity under base conditions. Exhibit 8-19 contains observations of values higher than this standard, but these are the maximums reported on a given freeway and are not expected to be achieved on most other freeway segments.

Multilane Highways

The observation of multilane rural highways operating under capacity conditions is difficult, because such operations rarely occur. Exhibit 8-21, however, contains some data for four-lane, six-lane, and eight-lane highways in suburban settings operating with uninterrupted-flow conditions.

EXHIBIT 8-18. MAXIMUM ANNUAL AVERAGE DAILY TRAFFIC REPORTED ON SELECTED INTERSTATE ROUTES (1990)

Location	Section Length (km)	Annual Average Daily Traffic (veh/day)	Average Daily Traffic Per Lane (veh/day/ln)
14-Lane Routes			
I-405, Los Angeles-Long Beach, California	4.072	328,500	23,464
I-95, New Jersey Turnpike, NE New Jersey	0.982	270,491	19,321
I-95, George Washington Bridge, New York	0.756	270,400	19,314
12-Lane Routes			
I-5, Los Angeles-Long Beach, California	0.805	304,000	25,333
I-405, Los Angeles-Long Beach, California	3.154	288,200	24,017
I-90, Chicago, Illinois	1.658	275,883	22,990
I-5, Seattle-Everett, Washington	2.028	254,172	21,181
I-8, San Diego, California	2.028	253,600	21,133
I-15, San Diego, California	4.635	219,300	18,275
I-280, San Francisco-Oakland, California	3.026	208,900	17,408
I-95, Northeastern New Jersey	3.042	208,768	17,379
10-Lane Routes			
I-10, Los Angeles-Long Beach, California	5.552	330,600	33,060
I-405, Los Angeles-Long Beach, California	5.633	314,000	31,400
I-5, Los Angeles-Long Beach, California	3.380	263,600	26,360
I-80, San Francisco-Oakland, California	7.564	242,000	24,200
I-210, Los Angeles-Long Beach, California	8.272	231,200	23,120
I-95, Northeastern New Jersey	2.607	222,229	22,223
I-395, Washington, District of Columbia	0.772	220,455	22,046
I-610, Houston, Texas	2.181	216,390	21,639
H-1, Honolulu, Hawaii	2.720	209,158	20,916
8-Lane Routes			
I-5, Los Angeles-Long Beach, California	4.329	280,700	35,088
I-94, Chicago, Illinois	4.828	258,800	32,350
I-580, San Francisco-Oakland, California	2.816	250,000	31,250
I-10, Los Angeles-Long Beach, California	9.382	241,000	30,125
I-90, Chicago, Illinois	2.897	224,600	28,075
I-285, Atlanta, Georgia	0.338	212,060	26,508
I-635, Dallas-Fort Worth, Texas	7.612	210,497	26,312
I-395, Northern Virginia	2.849	208,590	26,074
6-Lane Routes			
I-880, San Francisco-Oakland, California	4.667	223,200	37,200
I-610, Houston, Texas	0.489	216,390	36,065
I-680, San Francisco-Oakland, California	0.644	210,000	35,000

Source: Federal Highway Administration.

EXHIBIT 8-19. REPORTED MAXIMUM HOURLY ONE-WAY VOLUMES ON SELECTED FREEWAYS

Location	Total Volume (veh/h)	Avg. Vol. Per Lane (veh/h/ln)
4–Lane Freeways		
I-66, Fairfax, Virginia	5301	2650
U.S. 71, Kansas City, Missouri	5256	2628
I-59, Birmingham, Alabama	4802	2401
I-35W, Minneapolis, Minnesota	4690	2345
I-225, Denver, Colorado	4672	2336
I-287, Morris Co., New Jersey	4624	2312
I-295, Washington, D.C.	4480	2240
I-235, Des Moines, Iowa	4458	2229
I-71, Louisville, Kentucky	4446	2223
I-55, Jackson, Mississippi	4436	2218
I-35, Kansas City, Kansas	4398	2199
CA 4, Contra Costa County, California	4342	2171
I-45, Houston, Texas	4240	2120
I-64, Charleston, West Virginia	4152	2077
U.S. 4/NH 16, Newington, New Hampshire	4083	2041
I-564, Norfolk, Virginia	3962	1982
Northern State Parkway, New York	3840	1920
I-93, Windham, New Hampshire	3804	1902
6–Lane Freeways		
I-495, Montgomery Co., Maryland	7495	2498
U.S. 6, Denver, Colorado	7378	2459
I-5, Portland, Oregon	7188	2396
I-35W, Minneapolis, Minnesota	6909	2303
CA 17, San Jose, California	6786	2262
Texas 121, Bedford, Texas	6673	2224
I-35E, Dallas, Texas	6611	2203
Garden State Parkway, New Jersey	6608	2203
I-5 Seattle-Everett, Washington	6533	2177
I-15, Salt Lake City, Utah	6357	2119
I-24, Nashville, Tennessee	6280	2093
NJ 3, Secaucus, New Jersey	6251	2083
I-287, Somerset Co., New Jersey	6151	2050
I-290, Hillside, Illinois	6149	2047
I-90, Northwest Tollway, Illinois	6120	2040
I-80, Omaha, Nebraska	6113	2038
I-40, Nashville, Tennessee	6104	2035
Southern State Parkway, New York	5610	1870
8–Lane Freeways		
I-635, Dallas, Texas	9090	2272
Garden State Parkway, New Jersey	8911	2228
I-495, Montgomery Co., Maryland	8793	2198
I-25, Denver, Colorado	8702	2175
I-495, Fairfax, Virginia	8610	2152
I-405, Los Angeles, California	8360	2090
I-5, Seattle, Washington	8295	2073
U.S. 50, Sacramento, California	8284	2071
U.S. 59, Houston, Texas	8268	2067
I-35W, Minneapolis, Minnesota	8168	2042
I-80, W. Paterson, New Jersey	6851	1712
I-71, Columbus, Ohio	6682	1670
Tunnels		
I-279, Fort Pitt Tunnel, Pittsburgh, Pennsylvania (4-lane)	4278	2139
I-376, Squirrel Hill Tunnel, Pittsburgh, Pennsylvania (4-lane)	3922	1961
I-895, Harbor Tunnel, Baltimore, Maryland (4-lane)	3166	1584
SR 1A, Callahan Tunnel, Boston, Massachusetts (2-lane, half of one-way pair)	3059	1530
I-95, Fort McHenry Tunnel, Baltimore, Maryland (8-lane)	5840	1460

Source: HCQS Survey, Federal Highway Administration.

EXHIBIT 8-20. REPORTED MAXIMUM LANE VOLUMES ON SELECTED FREEWAYS

Location	Avg. Volume Per Lane (veh/h/ln)	Volume In Peak Lane (veh/h/ln)
4-Lane Freeways		
I-70, Wheeling, West Virginia	-	2552
I-55, Jackson, Mississippi	2218	2542
I-235, Des Moines, Iowa	2229	2466
6-Lane Freeways		
I-40, Nashville, Tennessee	2035	2664
I-5, Seattle, Washington	2177	2630
I-24, Nashville, Tennessee	2093	2500
8-Lane Freeways		
I-5, Seattle, Washington	2073	2596
I-70, Columbus, Ohio	-	2298
I-71, Columbus, Ohio	1670	2088

Source: HCQS Survey and Federal Highway Administration.

EXHIBIT 8-21. REPORTED MAXIMUM ONE-WAY VOLUMES FOR SELECTED MULTILANE HIGHWAYS

Location	Total Volume (veh/h)	Avg. Volume Per Lane (veh/h/ln)
4-Lane Highways		
U.S. 101, Sonoma County, California	4124	2062
Utah 201, Salt Lake City, Utah	3989	1995
SR 17, Bergen County, New Jersey	3776	1888
U.S. 301, Prince Georges County, Maryland	3304	1652
6-Lane Highways		
U.S. 46, Passaic County, New Jersey	5596	1865
SR 3, Passaic County, New Jersey	5348	1783
U.S. 1, Essex County, Massachusetts	4776	1592
8-Lane Highways		
Almaden Expressway, San Jose, California	5428	1357

Source: HCQS Survey, Federal Highway Administration.

Rural Two-Way, Two-Lane Highways

Two-lane, two-way rural highways in the United States and Canada rarely operate at volumes approaching capacity, and thus the observation of capacity operations for such highways in the field is difficult.

A sampling of high-volume observations is given in Exhibit 8-22, but it is emphasized that none may be taken to represent capacity for the facilities shown. Observations on two-lane, two-way rural highways in Europe have been reported at far higher volumes. Volumes of more than 2,700 veh/h have been observed in Denmark, more than 2,800 in France, more than 3,000 in Japan, and more than 2,450 in Norway. Some of these volumes have contained significant numbers of trucks, some as high as 30 percent of the traffic stream (*12*).

EXHIBIT 8-22. REPORTED MAXIMUM VOLUMES ON SELECTED TWO-LANE RURAL HIGHWAYS

Location	Total Volume (veh/h)	Peak Dir. Volume (veh/h)	Off-Peak Dir. Volume (veh/h)
Highways			
Madera-Olsen Rd., Simi Valley, California	3107	1651	1456
Madera-Olsen Rd., Simi Valley, California	3027	1839	1188
Hwy. 1, Banff, Alberta, Canada	2450	-	-
Hwy. 35/115, Kirby, Ontario, Canada	2250	-	-
Wasatch Blvd., Salt Lake City, Utah	2198	1504	694
Hwy. 35, Kirby, Ontario, Canada	2050	-	-
U.S. 50, Lake Tahoe, California	1796	1386	410
NJ 50, Cape May Co., New Jersey	1714	1445	269
Hwy. 1, Banff-Yoho, Alberta-British Columbia, Canada	1517	-	-
Hwy. 4, Contra Costa, California	3350	1920	1430
Bridges/Tunnels			
U.S. 158, Nags Head, North Carolina	3195	-	-
Midtown Tunnel, Norfolk/Portsmouth, Virginia	2920	1827	1093
Sagamore Bridge, Hudson, New Hampshire	2701	-	-
TH 15, St. Cloud, Minnesota	2242	1146	1096
Underwood Bridge, Hampton, New Hampshire	1960	1041	919
Staley Viaduct, Decatur, Illinois	1919	971	948

Source: HCQS Survey and Federal Highway Administration.

Urban Streets

Since flow on urban streets is uninterrupted only on segments between intersections, the interpretation of high-volume observations is not as straightforward as for uninterrupted-flow facilities. Signal timing plays a major role in the capacity of such facilities, limiting the portion of time that is available for movement along the urban street at critical intersections. The volumes reported in Exhibit 8-23 are shown with the green to cycle time ratios in effect for the subject segments. Flow rates in vehicles per hour are estimated by taking the reported volumes and dividing by the reported green to cycle ratio. The prevailing conditions on urban streets may vary greatly, and such factors as curb parking, transit buses, lane widths, upstream intersections, and other factors may substantially affect operations and observed volumes.

Note that the comparison of maximum flow rates in vehicles per hour per lane varies widely. These observations did not include such factors as left- and right-turn lanes, which may enhance the capacity of the intersection approach, nor were other prevailing conditions cited. Capacity of the urban street is generally limited by the capacity of signalized intersections, with segment characteristics seldom playing a major role in the determination of capacity.

SPEED

Trends

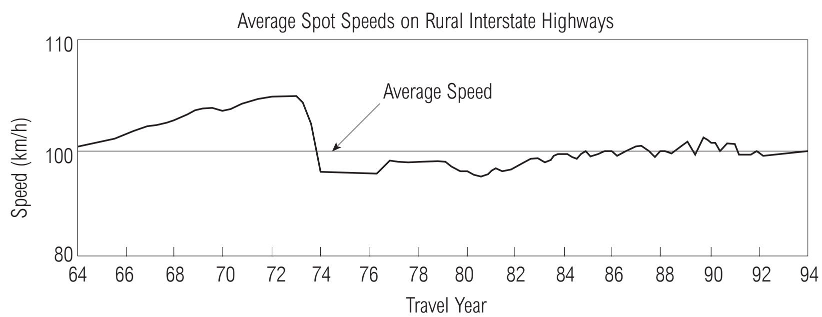
Nationwide speed trends though 1994 are shown in Exhibit 8-24 for Interstate rural highways. In 1973–1974, in response to a severe fuel shortage, the 55-mi/h (88.5-km/h) national speed limit was introduced, and a sharp decline in speeds was observed. Exhibit 8-25 confirms the increasing speed trends on highways in the United States. All of the highways referenced in Exhibit 8-25 had a 55-mi/h (88.5-km/h) speed limit when the data were reported.

EXHIBIT 8-23. REPORTED MAXIMUM DIRECTIONAL VOLUMES ON SELECTED URBAN STREETS

Location	Total Volume (veh/h)	Avg. Volume Per Lane (veh/h/ln)	g/C Ratio	Total Flow Rate (veh/h)	Avg. Flow Rate Per Lane (veh/h/ln)
4-Lane Urban Streets					
Ill. 83, DuPage Co., Illinois	3819	1910	0.80	4774	2387
So. Virginia St. (US 395), Reno, Nevada	2831	1415	0.62	4566	2282
Tara Blvd., Clayton, Georgia	2137	1068	0.47	4547	2272
Dougall Ave. SB, Windsor, Ontario, Canada	2240	1120	0.60	3733	1867
Antoine, Houston, Texas	2310	1155	0.65	3553	1777
Woodway WB, Houston, Texas	2156	1078	0.76	2836	1418
5-Lane Urban Streets					
North Shepard NB, PM, Houston, Texas	2100	1050	0.60	3500	1750
6-Lane Urban Streets					
Col. 2, Denver, Colorado	3435	1145	0.50	6870	2290
US 74/NC 27, Charlotte, North Carolina	4882	1627	0.80	6102	2034
Almaden Expressway, San Jose, California	3960	1320	0.66	6000	2000
Ygnacio Valley Road, Walnut Creek, California	3790	1263	0.65	5831	1943
Southwest Trafficway, Kansas City, Missouri	3492	1164	0.60	5820	1940
U.S. 19, Clearwater, Florida	4305	1435	0.75	5740	1913
Ward Parkway, Kansas City, Missouri	3477	1159	0.61	5700	1900
Seward Highway, Anchorage, Alaska	3177	1059	0.70	4538	1513
8-Lane Urban Streets					
Telegraph Rd., Detroit, Michigan	4400	1100	0.60	7333	1833
FM 1093, Houston, Texas	4500[a]	1125	0.70	6429	1607
FM 1093, Houston, Texas	4268[a]	1067	0.70	6097	1524

Note:
a. 2.7-m-wide lanes.
Source: HCQS Survey, Federal Highway Administration, Case Studies in Access Management, Draft Final Report, 1992.

EXHIBIT 8-24. SPEED TRENDS ON RURAL INTERSTATE HIGHWAYS



Note:
The data from 1965 to 1979 represent free-moving traffic on level, uncongested sections of the rural Interstate system. Beginning with 1980, the data represent all vehicle travel on the rural Interstate system.
Source: 1994 revision of Chapters 1 and 2 of the *Highway Capacity Manual*.

EXHIBIT 8-25. NATIONAL SPOT SPEED TRENDS FOR 90-km/h FACILITIES

Fiscal Year	Average Speed (km/h)	Median Speed (km/h)	85th Percentile Speed (km/h)	Percent > 90 km/h
Urban Interstate Highways				
1985	92.1	92.4	103.0	64.1
1987	93.3	93.3	104.3	67.4
1989	94.8	94.9	106.4	71.3
1991	94.6	94.6	106.4	69.8
Rural Interstate Highways				
1985	95.8	95.6	106.4	75.4
1987	96.1	96.8	107.0	73.7
1989	96.7	97.0	108.1	76.8
1991	96.4	95.6	108.1	75.5
Rural Streets				
1985	88.4	88.8	99.3	50.5
1987	90.0	90.3	101.1	54.3
1989	90.4	90.8	101.5	56.0
1991	90.8	90.6	101.5	54.5
Urban Principal Streets				
1985	86.1	86.3	97.4	42.1
1987	86.9	87.1	97.7	44.7
1989	87.9	88.7	98.7	47.7
1991	86.9	86.7	97.8	42.2

Note:
All highways have 90-km/h speed limit.
Source: Highway Statistics, Federal Highway Administration, 1992.

Aside from the general interest in the speed limit issue, these speed trends affect the procedures presented in this manual. Uninterrupted-flow procedures incorporate national average speed-flow and speed-density trends. The exact shape of these curves and the calibration of speeds (especially at the free-flow end of the relationships) reflect current trends. Curves used in this manual allow for average speeds of up to 120 km/h in response to the observed increase in driver-selected speeds under free-flow conditions.

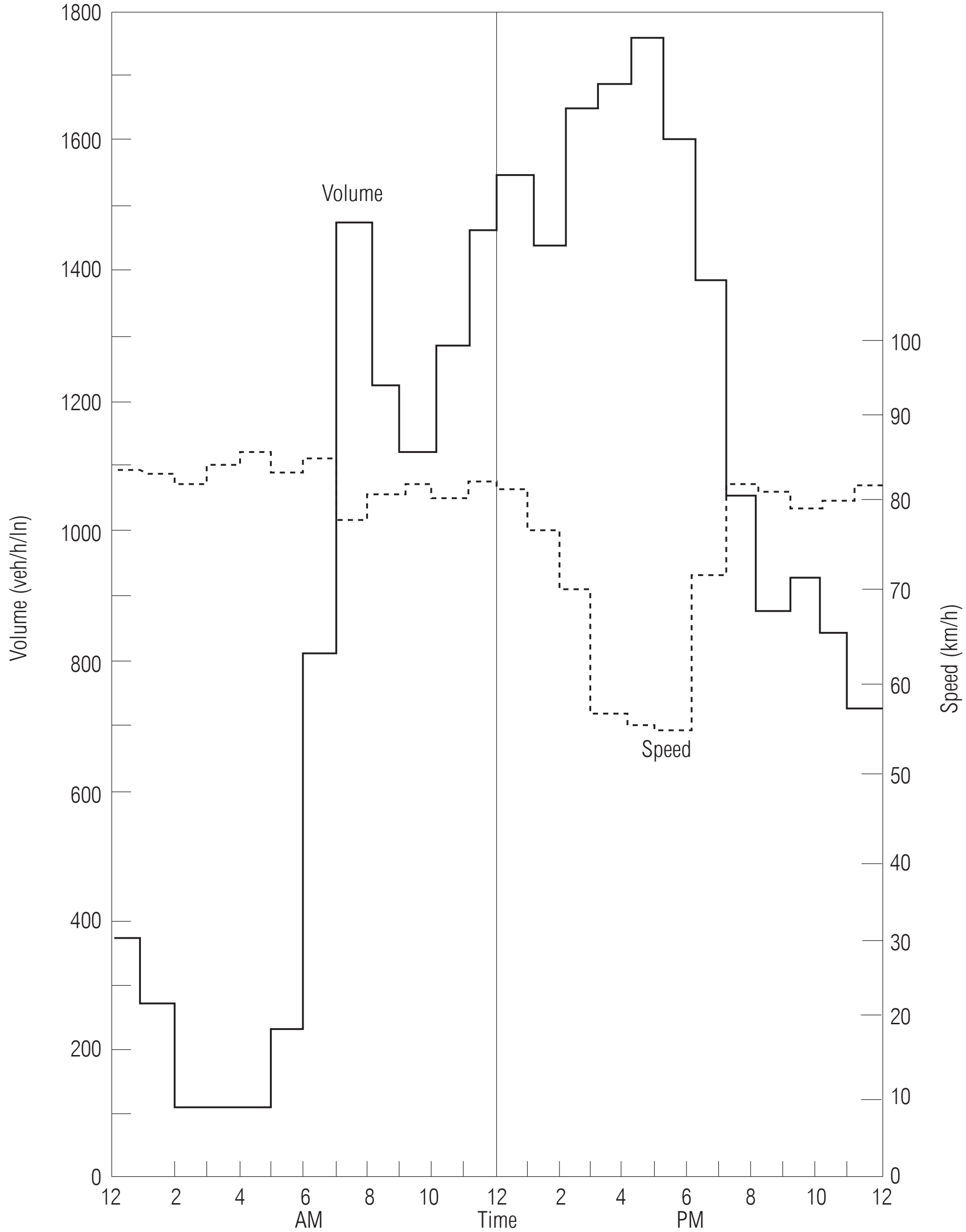
Speed Variation by Time of Day

Speed is not significantly affected by volume over a wide range of demand

Exhibits 8-26 and 8-27 show variations of speed with time of day, along with hourly volume variations, over a 24-h period for I-35W in Minneapolis. Exhibit 8-26 shows the weekday pattern, whereas Exhibit 8-27 shows a similar distribution for Saturdays.

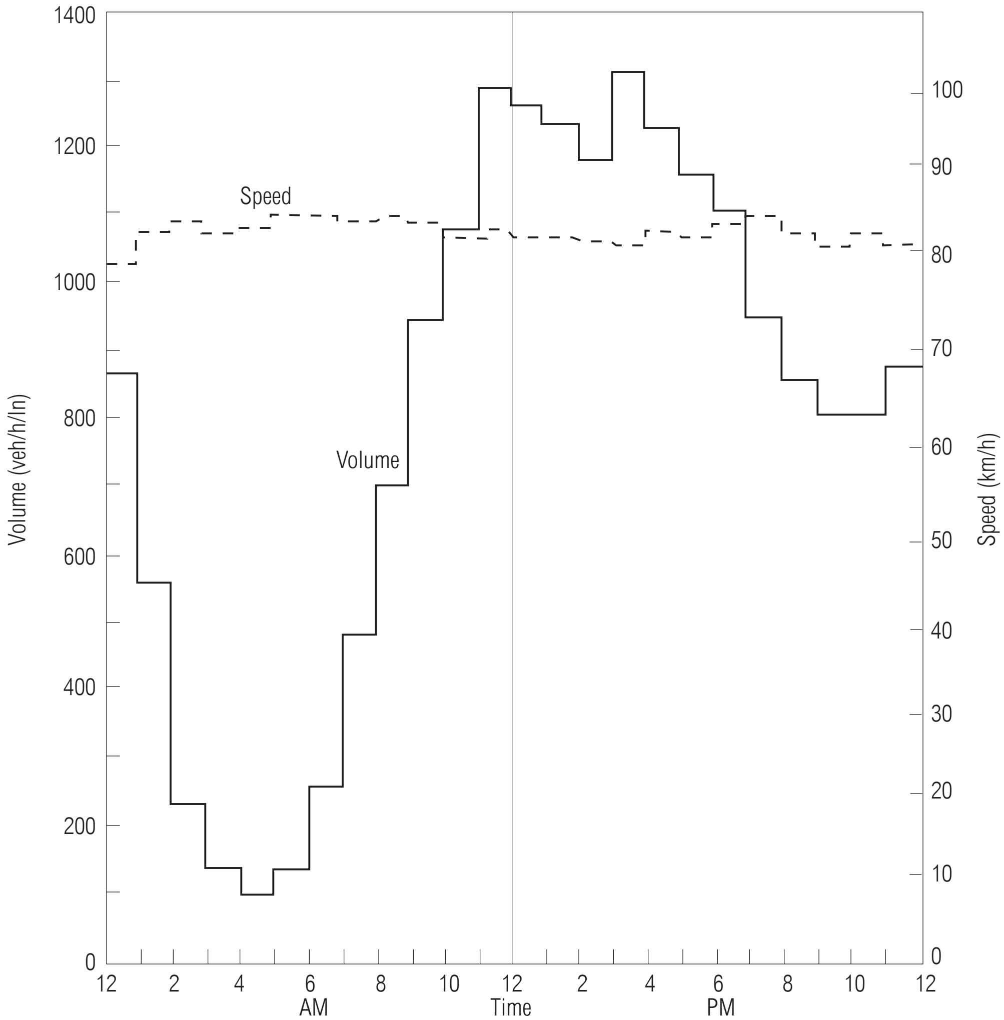
In these exhibits note that speed remains relatively constant despite significant changes in volume. In Exhibit 8-26, speed shows a marked response to volume increases only when the volume exceeds approximately 1,600 veh/h/ln. This trend is illustrated later and is an important characteristic in the procedures of this manual. If speed does not vary with flow rate over a broad range of flows, it becomes difficult to use speed as the sole measure for defining level of service. This important characteristic is the major reason why such measures as density have been introduced as primary measures of effectiveness for uninterrupted-flow facilities, with speed playing a secondary role.

EXHIBIT 8-26. SPEED AND VOLUME VARIATIONS BY HOUR OF DAY (WEEKDAYS)



Source: Minnesota Department of Transportation.

EXHIBIT 8-27. SPEED AND VOLUME VARIATIONS BY HOUR OF DAY (SATURDAYS)



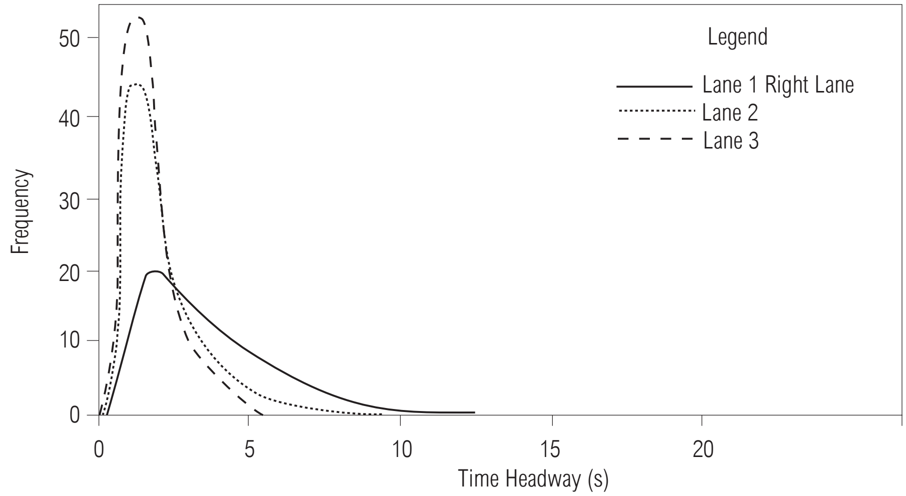
Source: Minnesota Department of Transportation.

HEADWAY DISTRIBUTIONS AND RANDOM FLOW

The average headway in a lane is the reciprocal of the flow rate. Thus, at a flow of 1,200 veh/h/ln, the average headway is 3,600/1,200 or 3 s. Vehicles do not, however, travel at constant headways. Vehicles tend to travel in groups, or platoons, with varying headways between successive vehicles. An example of the distribution of headways observed on the Long Island Expressway is shown in Exhibit 8-28. Lane 3 has the most uniform headway distribution, as evidenced by the range of values and the high frequency of the modal value, which is the peak of the distribution curve. The distribution of Lane 2 is similar to that of Lane 3, with slightly greater scatter (range from 0.5 to 9.0 s). Lane 1 shows a much different pattern: it is more dispersed, with headways ranging from 0.5 to 12.0 s, and the frequency of the modal value is only about one-third of that for the other lanes. This indicates that flow rate in the shoulder lane is usually lower than flow rates in the adjacent lanes when the total flows are moderate to high on the facility.

Exhibit 8-28 shows relatively few headways less than 1.0 s. A vehicle traveling at 95 km/h (26 m/s) would have a spacing of 26 m with a 1.0-s headway, and only 13 m with a 0.5-s headway. This effectively reduces the space between vehicles (rear bumper to front bumper) to only 7.5 to 9.0 m. This spacing (also called gap) would be extremely difficult to maintain.

EXHIBIT 8-28. TIME HEADWAY DISTRIBUTION FOR LONG ISLAND EXPRESSWAY



Source: Berry and Gandhi (*13*).

Drivers react to this intervehicle spacing, which they perceive directly, rather than to measures of headway. The latter include the length of the vehicle, which became smaller for passenger cars in the vehicle mix of the 1980s. In the 1990s, a larger vehicle mix is observed due to the popularity of sport utility vehicles. If drivers maintain essentially the same intervehicle spacing and car lengths continue to increase, some decreases in capacity could conceivably result.

If traffic flow were truly random, small headways (less than 1.0 s) could theoretically occur. Several mathematical models have been developed that recognize the absence of small headways in most traffic streams (*14*).

SATURATION FLOW AND LOST TIME AT SIGNALIZED INTERSECTIONS

Saturation flow rates have been increasing with time

The basic concepts of saturation headway, saturation flow rate, and start-up and change interval lost times were introduced elsewhere in this chapter. Exhibit 8-29 summarizes the results of representative past and recent studies. The table indicates that saturation headways have been shortening in the last decade, and consequently saturation flow rates have been increasing. This trend has been observed by both practicing professionals and researchers (*28*). In the table, saturation headway ranges from a low of 1.8 s to a high of 2.4 s, corresponding to a range of saturation flow rates of 2,000 to 1,500 pc/h/ln.

Exhibit 8-30 shows vehicle headway by position in the queue resulting from several past studies. It shows that, in most studies, the saturation headway does not become established until the sixth or seventh vehicle in the queue, indicating that the first five or six vehicles experience some start-up lost time. In discussing the results of Exhibit 8-30 (*28*), it was noted that the variation in discharge headways of the first several vehicles depended on the choice of a screenline for measuring headways rather than any real difference in the observed headways. Stop lines or curb lines have been used in combination with the front bumper, front or rear axles, or rear bumper. Caution is therefore advisable in comparing values of discharge headways from different studies. This manual uses the stop line as the screenline and the front wheels (or axle) as the measurement benchmark. Some other national practices apply different definitions or measurement techniques of saturation flow (*15, 28–32*). For that reason, the values quoted in international literature are not quite comparable (*30, 31*). The Canadian survey technique (*31*), however, allows the estimation of saturation flow rates for situations with queues as short as four to five vehicles. Saturation flow rates cited in various sources may also be influenced by the choice of vehicle positions and by the definition of lost time (*16*).

EXHIBIT 8-29. OBSERVED SATURATION FLOW RATES AT SIGNALIZED INTERSECTIONS

Date of Study	City or State	Sample Size	Saturation Flow Measurement Starting With Queue Position Number	Start-Up Lost Time (s)	Saturation Flow Rate (pc/h/ln)	Saturation Headway (s)
1967	Los Angeles, Santa Monica, California	6 Int.	5	2.05	1470	2.45
1971	Ames, Iowa	4 Int.	4	0.75	1572	2.29
1976	Nationwide		5	-	1682	2.14
1983	Lexington, Kentucky		4	1.40	1651	2.18
1986	Lawrence, Kansas		5	3.04	1827	1.97
1986	Austin, Dallas, Houston	8 h 3 Int.	6	-	2000	1.8
1986	Chicago, Houston, Los Angeles	7 Int.		1.31	1875	1.92
1987	Houston (peak)	30 h 2 Int.	5	-	1896	1.9
1987	Houston (off peak)	30 h 2 Int.	5	-	1832	1.97
1987	Los Angeles (peak)	34 h 2 Int.	5	-	1936	1.86
1987	Los Angeles (off peak)	34 h 2 Int.	5	-	1785	2.02
1988	Chicago	6.25 h 10 Int.	4	-	2000	1.8
1988	California, New York, Texas (single lane)	5 Int.	5	-	1791	2.01
1988	California, Illinois, New York, Texas (multilane)	7 Int.	5	-	1937	1.86
1989	College Station, Texas	30 h 2 Int.	4	1.31	1905	1.89
1991	Dallas	25 h 4 Int.	5	-	1910	1.88
1992	Florida	16 Int.	-	-	1840	1.96

Sources: References 15–27.

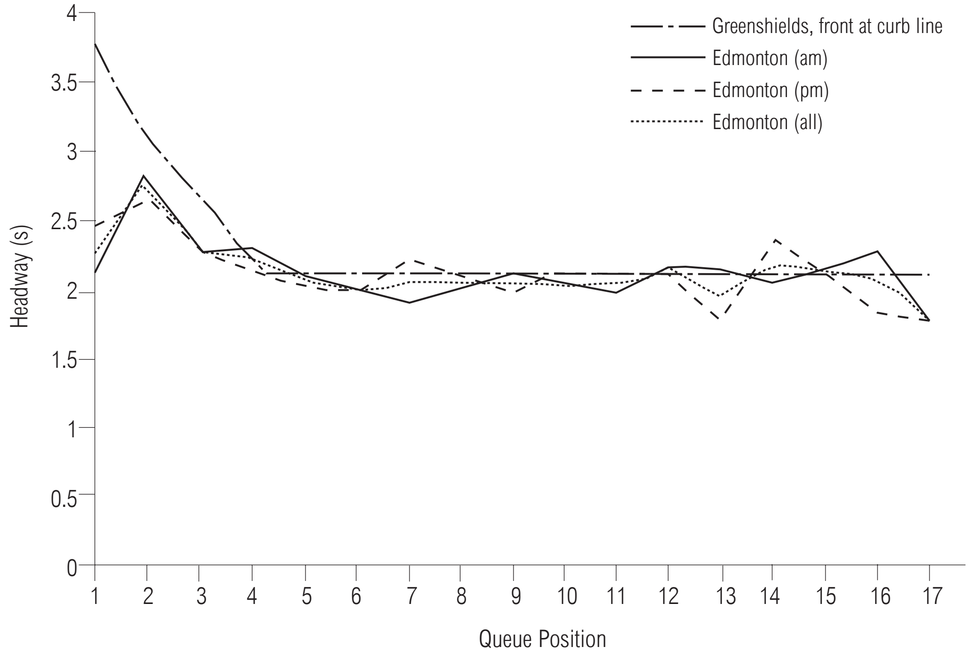
Saturation flow rates may decrease with long green times

Although most studies of intersection discharge headways have focused on the observation of the first 10 to 12 vehicles, there is some indication that the saturation headway may increase somewhat when green time becomes quite long. This effect implies that green phases longer than 40 or 50 s may not be proportionally as efficient as shorter phases (*31*).

Research (*33*) has shown the significance of prevailing conditions of lane width, parking, transit interference, pedestrian interference, turning movements, flow composition, signal progression, and other factors, all of which influence saturation flow values. For base conditions, including 3.6-m lanes, all through vehicles, all passenger cars, no parking, no transit interference, and low pedestrian volumes, the procedures of Chapter 16 recommend a saturation flow rate of 1,900 pc/h/ln, corresponding to a saturation flow headway of 1.9 s.

Start-up lost times were also measured during the studies identified in Exhibit 8-29 and other research projects (*34*) for a variety of conditions, including city size (population), location within the city, signal timing, speed limit, and other factors. Typical observed values range from 1.0 s to about 2.0 s. The variation in the data in Exhibit 8-29 and the importance of prevailing conditions suggest that local data collection be performed to determine saturation flow rates and lost times, which can lead to more accurate computations.

EXHIBIT 8-30. RESEARCH RESULTS ON QUEUE DISCHARGE HEADWAYS



Source: Teply and Jones (*28*).

SATURATION FLOW AT UNSIGNALIZED INTERSECTIONS

Saturation flow at a stop line on all-way stop-controlled intersections depends on the presence of vehicles on other approaches. When no traffic is present on other intersection approaches, the saturation flow rate on a single-lane approach is reported at 1,100 veh/h (*35*). For an intersection with four evenly loaded approaches and base conditions, 2,000 veh/h is reported (*35*).

BUS AND PASSENGER FLOWS

Exclusive busway volumes

The highest bus volumes experienced in a transit corridor in North America, 735 buses per hour through the Lincoln Tunnel and on the Port Authority Midtown Bus Terminal access ramps in the New York metropolitan area, are achieved on exclusive rights-of-way where buses make no stops (and where an 800-berth bus terminal is provided to receive these and other buses) (*36*). Where bus stops or layovers are involved, reported bus volumes are much lower. Exhibit 8-31 shows bus flow experience for North American cities.

Bus malls

When intermediate stops are made, bus volumes rarely exceed 120 buses per hour. However, volumes of 180 to 200 buses per hour are feasible where buses may use two or more lanes to allow bus passing. An example is Hillside Avenue in New York City. Two parallel bus lanes in the same direction, such as along Madison Avenue in New York, and the 5th and 6th Avenue Transit Mall in Portland, Oregon, also achieve this flow rate. Up to 45 buses one-way in a single lane in 15 min (a flow rate of 180 buses per hour) were observed on Chicago's State Street Mall; however, this flow rate was achieved by advance marshaling of buses into platoons of three buses and by auxiliary rear-door fare collection during the evening peak hours to expedite passenger loading.

Several downtown streets where there are two or three boarding positions per stop and where passenger boarding is not concentrated at a single stop carry bus volumes of 80 to 100 buses per hour. This frequency corresponds to about 5,000 to 7,500 passengers per hour, depending on passenger loads.

EXHIBIT 8-31. OBSERVED PEAK-DIRECTION, PEAK-HOUR PASSENGER VOLUMES ON U.S. AND CANADIAN BUS TRANSIT ROUTES—1995 DATA

Location	Facility	Peak-Hour, Peak-Direction Buses	Peak-Hour, Peak-Direction Passengers	Average Passengers per Bus
New Jersey	Lincoln Tunnel	735[a]	32,600	44
New York City	Madison Avenue	180	10,000	55
New York City	Long Island Expy.	165	7840	48
New York City	Gowanus Expy.	150	7500	35
Northern Virginia	Shirley Highway	160	5000	35
San Francisco	Bay Bridge	135	5000	37
Ottawa	West Transitway	225	11,100	49
Pittsburgh	East Busway	105	5400	51
Portland, OR	6th Avenue	175	8500	50
Newark	Broad Street	150	6000	40

Note:
a. No stops.
Source: Levinson and St. Jacques (*36*).

Historic streetcar volumes

These bus volumes provide initial capacity ranges that are suitable for general planning purposes. They compare with maximum streetcar volumes on city streets in the 1920s, which approached 150 cars per track per hour, under conditions of extensive queuing and platoon loading at heavy stops (*37*). However, the streetcars had two operators and large rear platforms where boarding passengers could assemble. Peak-hour bus flows observed at 13 major bus terminals in the United States and Canada range from 2.5 buses per berth at the George Washington Bridge Terminal in New York to 19 buses per berth at the Eglinton Station, Toronto.

Buses occupy loading areas at bus terminals for much longer periods of time than they occupy loading areas at on-street bus stops

The high berth productivity in Toronto reflects the special design of the terminal (with multiple positions in each berthing area), the wide doors on the buses using the terminal, and other factors. The relatively low productivity at the New York terminals reflects the substantial number of intercity buses using the terminals (which occupy berths for longer periods of time) and the single entrance doors provided on many suburban buses. This experience suggests an average of 8 to 10 buses per berth per hour for commuter operations. Intercity berths typically can accommodate one or two buses per hour.

The operating experience for typical light rail transit and streetcar lines in the United States and Canada is given in Exhibit 8-32. This exhibit lists typical peak-hour, peak-direction passenger volumes, service frequencies, and train lengths for principal U.S. and Canadian light rail transit lines.

EXHIBIT 8-32. OBSERVED U.S. AND CANADIAN LIGHT RAIL TRANSIT PASSENGER VOLUMES, PEAK HOUR AT THE PEAK POINT FOR SELECTED LINES (1993–1996 DATA)

City	Location (May be Trunk with Several Routes)	Trains/h	Cars/h	Avg. Headway (s)	Pass/Peak Hour Direction	Pass/m of Car Length
Calgary	South Line	11	33	320	4950	6.8
Denver	Central	12	24	300	3000	4.7
Edmonton	Northeast LRT	12	36	300	3220	4.0
Los Angeles	Blue Line	9	18	400	2420	5.4
Boston	Green Line Subway[a]	45	90	80	9600	5.3
Newark	City Subway	30	30	120	1760	4.6
Philadelphia	Norristown	8	8	450	480	3.3
Philadelphia	Subway-Surface[a]	60	60	60	4130	5.0
San Francisco	Muni Metro[a]	23	138	156	13,100	4.8
Sacramento	Sacramento LRT	4	12	900	1310	4.9
Toronto	Queen at Broadway[a]	51	51	70	4300	6.1
Portland, OR	Eastside MAX	9	16	400	1980	5.1

Note:
a. Trunks with multiple-berth stations.
In a single hour a route may have different lengths of trains and/or trains with cars of different lengths or seating configurations. Data represent the average car. In calculating the passengers per meter of car length, the car length is reduced by 9% to allow for space lost to driver cabs, stairwells, and other equipment.
Source: Parkinson and Fisher (*38*).

IV. REFERENCES

1. *Traffic Engineering Handbook*, 4th ed. Prentice-Hall, Englewood Cliffs, N.J., 1992.
2. Muranyi, T. C. A Method of Estimating Traffic Behavior on All Routes in a Metropolitan County. In *Highway Research Record 41*, HRB, National Research Council, Washington, D.C., 1963, pp. 61–78.
3. *Transportation and Traffic Engineering Handbook*, 2nd ed. Prentice-Hall, Englewood Cliffs, N.J., 1982.
4. McShane, W. R., and K. W. Crowley. Regularity of Some Detector-Observed Arterial Traffic Volume Characteristics. In *Transportation Research Record 596*, TRB, National Research Council, Washington, D.C., 1976, pp. 33–37.
5. Crabtree, J. D., and J. A. Deacon. Highway Sizing. In *Transportation Research Record 869*, TRB, National Research Council, Washington, D.C., 1982, pp. 6–14.
6. Werner, A., and T. Willis. Cost-Effective Level of Service and Design Criteria. In *Transportation Research Record 699*, TRB, National Research Council, Washington, D.C., 1979, pp. 1–7.
7. *Florida's Level of Service Standards and Guidelines Manual for Planning*. Systems Planning Office, Florida Department of Transportation, Tallahassee, 1995.
8. Levinson, H. *Characteristics of Urban Transportation Demand—A Handbook for Transportation Planners*. Urban Mass Transportation Administration, U.S. Department of Transportation, 1978.
9. *Development of Diurnal Traffic Distribution and Daily, Peak, and Off-Peak Vehicle Speed Estimation Procedures for Air Quality Planning*. Final Report, Work Order B-94-06. Comsis, Federal Highway Administration, Washington, D.C., April 1996.

10. Huber, M. J., and J. L. Tracy. Operating Characteristics of Freeways. In *NCHRP Report 60: Effects of Illumination on Operating Characteristics of Freeways*, HRB, National Research Council, Washington, D.C., 1968, pp. 3–48.
11. *Quick Response Freight Manual, Travel Model Improvement Program.* U.S. Department of Transportation, 1996.
12. *Two-Lane Rural Roads: Design and Traffic Flow*. Organization for Economic Cooperation and Development, Paris, France, 1972.
13. Berry D. S., and P. K. Gandhi. Headway Approach to Intersection Capacity. In *Highway Research Record 453*, HRB, National Research Council, Washington, D.C., 1973, pp. 56–60.
14. Gerlough, D. L., and M. J. Huber. *Special Report 165: Traffic Flow Theory: A Monograph*. TRB, National Research Council, Washington, D.C., 1975.
15. King, G. F., and M. Wilkinson. Relationship of Signal Design to Discharge Headway, Approach Capacity, and Delay. In *Transportation Research Record 615*, TRB, National Research Council, Washington, D.C., 1976, pp. 37–44.
16. Zegeer, J. D. Field Validation of Intersection Capacity Factors. In *Transportation Research Record 1091*, TRB, National Research Council, Washington, D.C., 1986, pp. 67–77.
17. Gerlough, D. L., and F. A. Wagner. *NCHRP Report 32: Improved Criteria for Traffic Signals at Individual Intersections*. HRB, National Research Council, Washington, D.C., 1967.
18. Carstens, R. L. Some Parameters at Signalized Intersections. *Traffic Engineering*, Aug. 1971.
19. Agent, K., and J. Crabtree. *Analysis of Lost Times at Signalized Intersections*. Kentucky Transportation Research Program, University of Kentucky, Lexington, 1983.
20. Lee, J., and R. L. Chen. Engineering Headway at Signalized Intersections in a Small Metropolitan Area. In *Transportation Research Record 1091*, TRB, National Research Council, Washington, D.C., 1986, pp. 117–126.
21. Molina, C. J., C. J. Messer, and D. B. Fambro. *Passenger Car Equivalencies for Large Trucks at Signalized Intersections*. TTI Research Report 397-2. Texas Transportation Institute, Texas A&M University, College Station, 1987.
22. Fambro, D. B., E. C. P. Chang, and C. J. Messer. *NCHRP Report 339: Effects of the Quality of Traffic Signal Progression on Delay*. TRB, National Research Council, Washington, D.C., 1991.
23. Prevedouros, P. D., and P. P. Jovanis. Validation of Saturation Flows and Progression Factors for Traffic-Actuated Signals. In *Transportation Research Record 1194*, TRB, National Research Council, Washington D.C., 1988, pp. 147–159.
24. Roess, R. P., J. M. Papayannoupoulis, J. M. Ulerio, and H. S. Levinson. *Levels of Service in Shared-Permissive Left-Turn Lane Groups at Signalized Intersections*. Report DTFH 61-87-C-00012. Transportation Training and Research Center, Polytechnic University, Brooklyn, N.Y., 1989.
25. Short, J. T. *Effects of Dips and Bumps on Saturation Flow Rates at Signalized Intersections*. Thesis. Texas A&M University, College Station, 1989.
26. Gaston, G. D. *An Operation Analysis of Protected Lead-Lag Left Turn Phasing*. Thesis. Texas A&M University, College Station, 1991.
27. Zegeer, J. *Data Collected for Florida Department of Transportation*. 1992.
28. Teply, S., and A. M. Jones. Saturation Flow: Do We Speak the Same Language? In *Transportation Research Record 1320*, TRB, National Research Council, Washington, D.C., 1991, pp. 144–153.
29. Akcelik, R. *Traffic Signal: Capacity and Timing Analysis*. Research Report ARR 123. Australian Road Research Board, Victoria, 1981.

30. Teply, S. (ed.). *Canadian Capacity Guide for Signalized Intersections*, 1st ed. Institute of Transportation Engineers, District 7, Canada, and University of Alberta, Edmonton, Alberta, Canada, 1984.
31. Teply, S., D. Allingham, D. Richardson, and B. Stephenson. *Second Edition of the Canadian Capacity Guide for Signalized Intersections*. Institute of Transportation Engineers, District 7, Canada, 1995.
32. Richtlinien fuer Lichtsignalanlagen, Forschungsgesellschaft fuer Strassen- und Verkehrswesen. Köln, Germany, 1992.
33. Agent, K., and J. Crabtree. *Analysis of Saturation Flow at Signalized Intersections*. Kentucky Transportation Research Program, University of Kentucky, Lexington, Feb. 1983.
34. Bonneson, J. Change Interval Timing and Lost Time for Single Point Urban Interchanges. *Journal of Transportation Engineering*, Vol. 118, No. 5, 1992.
35. *Transportation Research Circular 373: Interim Materials on Unsignalized Intersection Capacity*. TRB, National Research Council, Washington, D.C., 1991.
36. Levinson, H. S., and K. R. St. Jacques. Bus Lane Capacity Revisited. In *Transportation Research Record 1618*, TRB, National Research Council, Washington, D.C., 1998, pp. 189–199.
37. Blake, H. W., and W. Jackson. *Electric Railway Transportation*. McGraw-Hill Book Company, New York, 1924.
38. Parkinson, T., and I. Fisher. *TCRP Report 13: Rail Transit Capacity*. TRB, National Research Council, Washington, D.C., 1996.