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Traffic Flow on Two-Lane Highways An Overview

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by *R. Tapio Luttinen*

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Abstract

Highway capacity methods are based on both theoretical and empirical research. Two-lane highways present many challenges for traffic scientists that would be beneficial for capacity studies. This report gives an overview of the research of traffic flow on two-lane highways. The emphasis is on such theoretical research that may have some implications for further capacity studies.

The passing mechanism on two-lane highways is given a mathematical description, as well as the distribution of passing sight on a roadway. Models for platooning and average travel speeds are described as well. The impact of heavy vehicles is modeled. Finally the perceptions of Finnish drivers are briefly discussed.

It is suggested that directional analysis should be the foundation of capacity studies of two-lane highways. Two-way analysis should be based on a proper combination of directional results.

Speed-flow curves on two-lane highways appear to have a concave shape, although the speed-flow relationships at traffic conditions near and at capacity deserve further research. The slope of the speed-flow curve is steeper at low flow rates and high free-flow speeds.

The dynamics of heavy vehicle impact deserve further research. Because of the problems in defining passenger-car equivalencies, it is better to estimate the impact in terms of changes in percent time spent following and in average travel speed. The average travel speed of passenger cars is suggested as a service measure instead of the average travel speed of mixed traffic.

As well as traffic flow theories should be verified against empirical data, the evaluation of the results should be based on the perceptions of the road users. Accordingly, it is necessary to base the level-of-service definitions on the perceptions of road users.

Preface

After the new Highway Capacity Manual (HCM 2000) has been finished it has been necessary to look ahead. In order to visualize improvements for current methods or even new methodologies it is necessary to take a broader look at the issues. The feedback from users of the manual and perceptions of the users of facilities should be evaluated. It is also useful to make an overview of theoretical studies over the last decades.

This report presents an overview of theoretical studies of two-lane highways. The emphasis is on such research which supports the two-lane highway chapter in HCM 2000.

The first draft of the report was presented for comments to the two-lane roads subcommittee of the TRB Highway Capacity and Quality of Service Committee at the TRB Annual Meeting in January, 2001.

Lahti, August 10, 2001

R. TAPIO LUTTINEN

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1

Introduction

Two-lane highways represent the absolute majority of road networks in the world, even in the most developed countries. These roads have, however, not played a major role in recent research of traffic flow and highway capacity. More attention has been given to freeways and multilane highways, which carry the highest traffic flows. Multilane highways and freeways, along with urban streets, also have the highest degrees of congestion. The focus of research funding can, therefore, be understood.

The vehicle mileage on two-lane highways is, however, substantial. The passenger and goods traffic on two-lane highways have high economic implications. It is also necessary to estimate the performance of a two-lane highway in order to decide the economic feasibility of its improvement, even into a multilane highway or a freeway.

For theoretical research two-lane highways present challenges, which have not attracted the attention they would have deserved. The interaction between flows in both directions and the geometric properties presents many unexplored challenges to traffic scientists. Also such basic issues as the shape of the speed-flow curve and the nature of heavy vehicle impact are not sufficiently understood at present.

The purpose of this study is to present an overview of the theory of traffic flow on two-lane highways. The focus is on such issues which carry direct implications to capacity and level-of-service studies. Before the theoretical overview it is necessary to have a brief look into the history of capacity studies. The emphasis is more on qualitative than quantitative aspects of the methods. This way it is, hopefully, possible to raise issues in the capacity studies which can have further light from theoretical research. Some statistical issues will also be briefly discussed.

2

Capacity studies for two-lane highways

2.1 The early years

In the early decades of the twentieth century capacity (theoretical lane volume) was estimated from the formula (Hammond and Sorenson 1941)

$$c = \frac{5280v}{s}, \quad (2.1)$$

where c was the capacity of a single lane in cars per hour, v was the speed in miles per hour, and s was the average spacing in feet of moving vehicles, from center to center. The spacing factor was usually derived from considerations of driver reaction time, braking distances, and coefficients of friction (Evans 1950).

The minimum safe headway was first considered equal to the time (T_R) taken to apply brakes after the preceding vehicle had applied brakes, plus the time required to travel the length (L) of the vehicle with the current speed v

$$h_{\min} = T_R + \frac{L}{v}. \quad (2.2)$$

Clayton (1941) used $T_R = 1$ second. The maximum flow rate (Clayton called it saturation density) as a function of speed was, accordingly

$$q_{\max} = \frac{3600}{1 + \frac{L}{1.47V}} = \frac{5280V}{1.47V + L}, \quad (2.3)$$

where L was the length of the vehicle in feet (15 feet for “light cars”), and V denoted the speed, in miles per hour. Because the densities conforming to higher speeds on this curve appeared to be dangerous (figure 2.1), a term was added which made allowance to the distance required to pull up after brakes had been applied:

$$q_{\max} = \frac{5280V}{L + AV + \frac{V^2}{B}}. \quad (2.4)$$

The parameter A was estimated equal to 1.47 and the parameter B equal to 30 for light vehicles and 20 for heavy vehicles. Equation 2.4 has maximum flow rate 1,830 veh/h at speed 34 km/h (figure 2.1). According to Clayton, formulas of type (2.4) were generally accepted, although the constants and the power of V varied. The resulting one lane capacities were in the range of 1,050–2,900 vehicles per hour (Normann 1949).

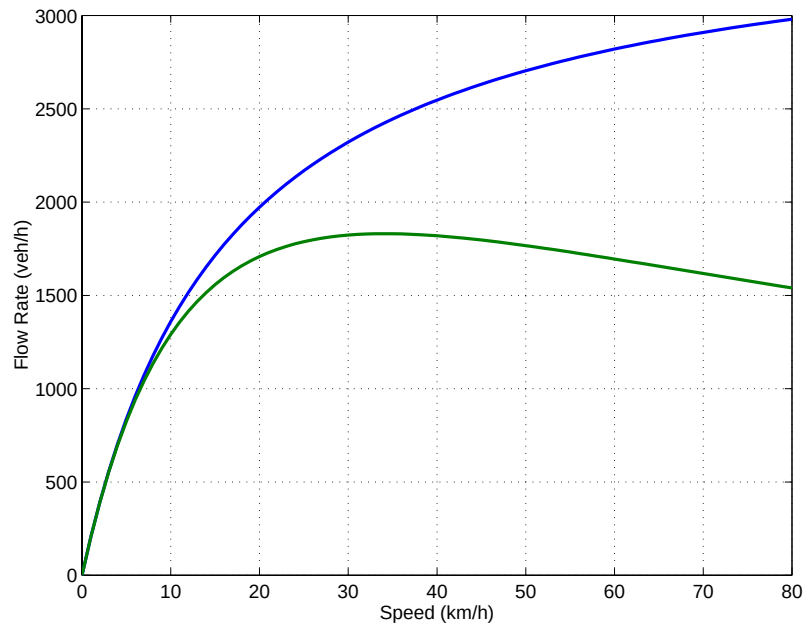


FIGURE 2.1 · Maximum flow rate according to equations 2.3 (upper curve) and 2.4 (lower curve)

The early theoretical derivations, obtained by assuming that all vehicles traveled at the same speed, resulted in extremely high and impractical flow rates. In order to get more reliable information, field studies were conducted from 1934 through 1939 by the U.S. Bureau of Public Roads under the direction of O. K. Normann.

Normann (1942) calculated the theoretical capacity of a highway as a function of speed and average spacing. He used empirically estimated average spacings of trailing vehicles at different speeds (Normann 1939). The theoretical capacity for vehicles traveling at same speed, based on this analysis, was 2,000 vehicles per hour in the daytime and about 1,800 at night. The speed at capacity was 33 miles per hour (53 km/h).

Normann (1939) introduced “practical capacity” as a quality indicator. It permitted “reasonable range of speeds and a reasonable amount of freedom from interference between vehicles.” Normann (1942) suggested 800 veh/h as the practical capacity for a two-lane “level tangent rural highway carrying few trucks.”

Normann (1942) also introduced measures to evaluate the quality of traffic flow on highways. In the light of later developments these measures are of special

interest:

1. The proportion of headways less than nine seconds
2. The ratio of actual passings to desired passings
3. The average number of passings per vehicle
4. Speed differences between successive vehicles.

Normann was especially concerned about “the time that slow-moving cars must be trailed” and “freedom of movement” as indicated by speed differences between successive drivers. In this article Normann presented many of the central ideas of traffic performance analysis.

The headway criterion, although with a five-second limit, and the (percent) *time spent following* slower vehicles were later adopted as service measures in the third edition of HCM (Transportation Research Board 1985). Wardrop (1952) as well as Morrall and Werner (1990) have suggested the use of the *overtaking ratio* as a service measure. The idea of *speed changes* while trailing slower vehicles was further developed by Greenshields (1961) and Drew (1968).

2.2 The first Highway Capacity Manual

The first edition of the U.S. *Highway Capacity Manual* (Bureau of Public Roads 1950) was based on the work of Normann (1939, 1942, 1949). The manual defined three levels of roadway capacity:

1. *Basic capacity* “is the maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained.”
2. *Possible capacity* “is the maximum number of vehicles that can pass a given point on a lane or a roadway during one hour, under the prevailing roadway and traffic conditions.”
3. *Practical capacity* “is the maximum number of vehicles that can pass a given point on a roadway or in a designated lane during one hour without the traffic density being so great as to cause unreasonable delay, hazard, or restriction to the drivers’ freedom to maneuver under the prevailing roadway and traffic conditions.”

Basic capacity of a two-lane highway was approximated equal to the basic lane capacity of a multilane highway, namely 2,000 passenger cars per hour. When traffic was evenly divided by directions, the passing restrictions were assumed to limit the flow in each direction approximately to 1,000 passenger cars per hour. Thus, the basic capacity for a two-lane highway in the 1950 HCM was 2,000 veh/h regardless of the directional distribution.

The practical capacity was not based on speed differences, as Normann (1942) had suggested, but on operating speeds. The practical capacity of a rural two-lane highway under ideal conditions was 900 passenger cars per hour regardless of the directional distribution. That provided an operating speed of 45 to 50 miles per hour (72–80 km/h). On high speed highways the practical capacity under ideal conditions was 600 passenger cars per hour. These highways had operating speeds 50–55 miles per hour. Within urban areas the practical capacity 1500 passenger cars per hour was based on operating speed 35 miles per hour (56 km/h).

2.3 The 1960's and the second Highway Capacity Manual

In the beginning of 1960's Greenshields (1961) developed a *quality index* for traffic flow based on “frustration factors.” He considered the frequency and amount of speed changes as undesirable factors, which irritated drivers and increased the cost of operation. The quality index was

$$Q = \frac{1000S}{\Delta_s \sqrt{f}}, \quad (2.5)$$

where S was the average speed (mi/h), Δ_s was the absolute sum of speed changes per mile, and f was the number of speed changes per mile. The data were collected either with an instrumented vehicle or by using aerial photographs. According to Greenshields, quality of flow could not be measured at a point.

In 1965 the second edition of HCM (Highway Research Board 1965) extended the idea of practical capacity to the well known six *levels of service* (A–F), still in use today. The level of service (LOS) on two-lane highways, was expressed in terms of the operating speed as the governing service measure and the traffic volume limitation as a supplementary service measure. Both the operating speed and maximum service volume limits had to be satisfied.

The “basic capacity” concept in the 1950 HCM (Bureau of Public Roads 1950) was replaced in the 1965 HCM by “capacity under ideal conditions.” *Ideal conditions* on two-lane highways included:

1. Uninterrupted flow free from side interferences of vehicles and pedestrians
2. Passenger cars only
3. Traffic lanes 12 ft (3.7 m) wide with adequate shoulders and lateral clearance at least 6 ft (1.8 m)
4. Average highway speed at least 70 mile/h (113 km/h)
5. No passing sight restrictions

The average highway speed was defined as “the weighted average of the design speeds within a highway section.” The capacity under ideal conditions was 2000 passenger cars per hour total for both directions; i.e., it was equal to the the

basic capacity in the 1950 HCM. Directional distribution was not considered to have any effect on operating conditions.

A few years later Drew (1968) extended the earlier ideas of Greenshields (1961). Instead of the frequency and amount of speed changes, Drew studied *acceleration noise*; i.e., standard deviation of accelerations, including decelerations as negative accelerations. According to Drew, acceleration noise could describe the discomfort experienced by a motorist, as well as operating costs and safety, better than travel time or average speed.

He also developed an *energy-momentum approach* to LOS. The total energy for a traffic stream over a section of a road was the sum of kinetic energy and internal energy, where acceleration noise represents internal energy. On a freeway the kinetic energy reached its maximum and internal energy its minimum when traffic volume was about 8/9 of possible capacity. This was used as a boundary condition between levels of service D and E. The upper limit for LOS A was 35% of possible capacity, at which volume kinetic energy was equal to internal energy.

Recently, Brilon (2000a) has used a similar approach in a study of traffic *efficiency*. He estimated that the efficiency reaches its maximum at a flow rate near 90% of capacity. The maximum is lower than capacity only on freeways and multi-lane highways, which have a convex speed-flow curve. On two-lane highways, where the speed-flow curve at uncongested conditions is linear or concave, the maximum efficiency is reached at capacity.

2.4 The third Highway Capacity Manual

In the development of the third edition of HCM (Transportation Research Board 1985) the mean speed was considered as an inadequate measure of the balance between passing demand and passing supply. The mean speed was also found to be less sensitive to traffic flow rate than was previously supposed (Harwood, May, Anderson, Leiman and Archilla 1999).

HCM 1985 adopted *percent time delay* (PTD) as the service measure. PTD was defined as “the average percent of time that all vehicles are delayed while traveling in platoons due to inability to pass.” This follows the suggestion of Normann (1942). Motorists were defined to be delayed when traveling behind a platoon leader at speeds less than their desired speed and at headways less than five seconds. The percent of vehicles traveling at headways less than five seconds could be used as a surrogate measure of PTD in field studies. Accordingly, the estimation of PTD was much easier than the estimation of Greenshields’s (1961) quality index or acceleration noise.—For specific grades the level-of-service measure was the average upgrade travel speed.

The 1985 HCM model adjusted the volume-to-capacity ratio for the combined effect of terrain type and percent no-passing zones. Service flow rate was adjusted for directional distribution, but the combined effect of no-passing zones and directional distribution was not considered.

The capacity of a two-lane highway under ideal conditions was 2,800 veh/h.

The speed at capacity was 45 mi/h; i.e., 72 km/h, which implied a critical density of 19.4 veh/km per lane.

Roadway conditions had no effect on capacity. The effect of directional split on capacity (C) was estimated by relation

$$C = 2000 + 1600(1 - d). \quad (2.6)$$

This gave the capacity of 2,800 pcu/h for a 50/50 directional split in ideal conditions. For an ideal highway with a 100/0 directional distribution, the HCM capacity was 2,000 pcu/h, which was the lane capacity estimate for freeways and multilane highways at the time when the two-lane model was being developed. If critical speed was still 72 km/h, the critical density was 28 veh/km.

2.5 Highway Capacity Manual 2000

The expression “percent time delay” in HCM1985 was a slight misnomer, because the criterion was not delay but time spent traveling in platoons. HCM2000 (Transportation Research Board 2000) uses a more descriptive term *percent time spent following* (PTSF), which is defined as the average percent of travel time that vehicles must travel in platoons behind slower vehicles due to inability to pass. Simulation studies indicated that PTSF can be estimated as the percentage of vehicles traveling at headways of *three* seconds or less (Harwood et al. 1999).

Other service measures were also considered, such as average travel speed, delay rate (s/veh-km), density, and passing ratio (Harwood et al. 1999). Delay rate and passing rate were considered impossible to measure in the field. PTSF was assumed to describe traffic conditions better than density, because density is less evenly distributed on two-lane highways than on multilane highways and freeways. The *average travel speed* (ATS) was selected as an auxiliary criterion for high-class highways, because it makes LOS sensitive to design speed and enables the use of the same criteria for both general and specific terrain segments. Specific upgrades and downgrades can be analyzed by a directional-segment procedure.

As compared with the third edition (Transportation Research Board 1985), HCM 2000 will decrease the capacity for each direction of travel from 2,000 pc/h to 1,700 pc/h, but the total two-way capacity is increased from 2,800 pc/h to 3,200 veh/h. For short distances a two-way capacity between 3,200 veh/h and 3,400 veh/h may be attained. Consequently, the capacity in major direction is 1,700 pc/h until traffic flow in minor direction reaches 1,500 veh/h. After that the two-way capacity is the determining factor. The capacity can be expressed mathematically as follows:

$$C = \min \left\{ \frac{1700}{P_d}, 3200 \right\}, \quad (2.7)$$

where P_d is the proportion of traffic in the major direction. The effect of directional distribution on capacity is displayed in figure 2.2. Two-way capacity is the sum of the one-way capacity and the opposing flow rate.

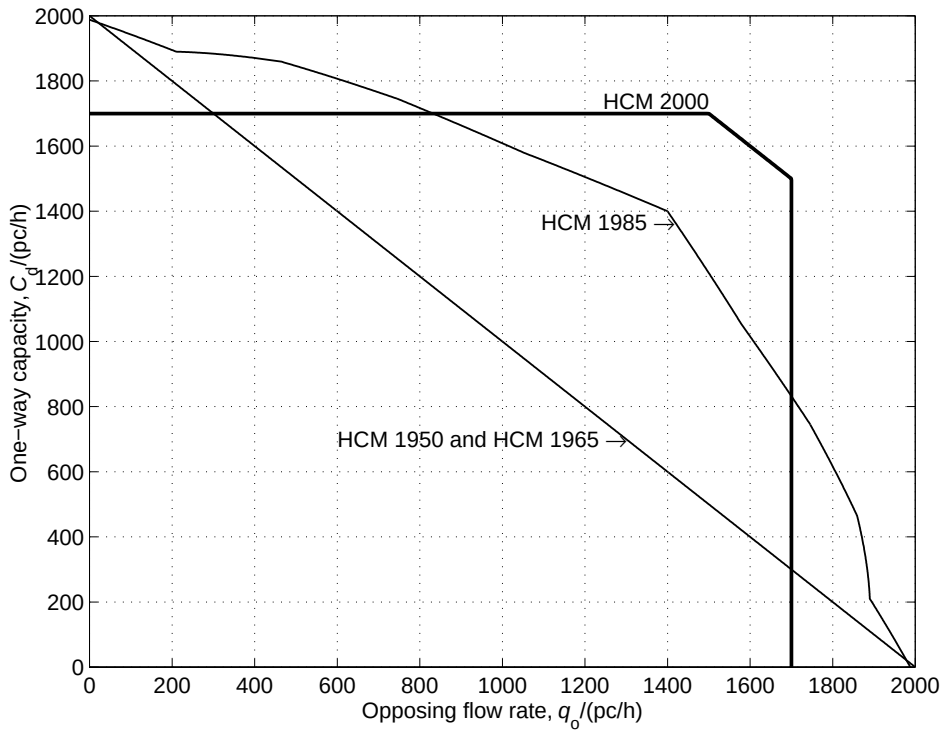


FIGURE 2.2 · The effect of opposing flow on one-way capacity

PTSF is a function of (passenger-car equivalent) two-way flow rate q_{pc} , and it is adjusted for the combined effect of directional distribution of traffic and percent no-passing zones ($f_{d,np}$):

$$P_W = 100 \left(1 - e^{-0.000879 q_{pc}} \right) + f_{d,np}. \quad (2.8)$$

ATS is estimated using free-flow speed, $v(0)$, and adjustments due to flow rate (pc/h) and percentage of no-passing zones. HCM 2000 assumes a linear relationship between speed and flow rate

$$\bar{v}_s(q) = v(0) - 0.0125 q_{pc} - f_{np}, \quad (2.9)$$

where $v(0)$ is free-flow speed, q_{pc} is the passenger-car equivalent flow rate (pc/h), and f_{np} is the adjustment for the percentage of no-passing zones. The model assumes that the speed difference under free-flow conditions on two two-lane highways with similar sight conditions is maintained at all flow rates, even at capacity; ie., speed-flow curves are parallel.

On directional segments ATS is (Harwood et al. 1999)

$$\bar{v}_s(q) = v(0) - 0.0125(q_d + q_o) - f_{np}. \quad (2.10)$$

The effect of opposing flow was estimated equal to the effect of the flow rate in the analyzed direction. Thus, the directional distribution does not have any effect on ATS.

3

Passing

3.1 Passing mechanism

On ordinary two-lane highways passing vehicles must use the lane reserved for oncoming traffic. Passing demand, as well as supply, occur at certain locations at certain times. Passing can occur, when passing demand and supply occur at the same time at the same location.

Passing demand is a consequence of speed differences between vehicles. Passing opportunity is supplied when there is a long enough headway in the *oncoming traffic* and a long enough *sight distance*. These features make traffic flow on two-lane highways difficult to analyze. Consequently, mathematical theories of two-lane highway traffic are not at the same level of development as freeway traffic flow models.

Drivers on a two-lane highway have different *desired speeds*, which they try to maintain. Desired speed is the speed of a vehicle if unimpeded by other vehicles. It depends on such factors as speed limit, roadway geometry, weather, road surface, trip purpose and length, properties of the vehicle, personality of the driver, and general traffic conditions (Morrall and Werner 1990, McLean 1989).¹ Desired speed is useful as a concept, but it cannot be directly observed (Taylor, Young and Bonsall 1996).

As a vehicle approaches a slower vehicle, it has three options:

1. It can pass the slower vehicle “on the fly.”
2. It can follow the slower vehicle until a passing opportunity is found.
3. It can follow the vehicle ahead with no passing intention.

¹McLean (1989) defines desired speed as “the speed at which drivers would choose to travel if unimpeded by other vehicles *or alignment constraints*.” The speed at which vehicles would traverse a road segment, if unimpeded by other traffic, he calls *unimpeded speed*. The definition of “desired speed” above is equivalent to the “unimpeded speed” of McLean. The chosen terminology adapts better to the discussion below.

The first option is possible, if the driver finds a passing opportunity when the slower vehicle is reached. If there is no passing opportunity at that moment, the driver must wait following the slower vehicle. It is, however, possible that the driver will stay as a follower because of high traffic and/or heavy-vehicle flow rate, or because the speed of the vehicle ahead is only slightly slower than the desired speed of the driver. In both cases the driver considers the difference in travel time not worth the safety risk of passing. This third option is usually ignored in theoretical models. All vehicles following a vehicle with a slower desired speed are assumed to pass at the first opportunity.

If vehicles cannot pass slower vehicles without delay, platoons begin to form. Platooning increases the proportion of relatively short (follower) headways and decreases the mean speed. In addition, in front of slower vehicles there are empty zones, which cannot be used effectively. This reduces the capacity of two-lane highways.

A good overview of traffic flow models for two-lane highways has been presented by McLean (1989). The discussion below considers microscopic models for both traffic flow and passing sights. The purpose of the discussion is to find the structure of the effects of opposing flow and passing sights on travel speeds and percent time spent following.

Let us consider traffic flow, where each vehicle maintains its desired speed as long as it is unimpeded by other vehicles. When a vehicle catches up a slower vehicle, it changes its speed (v) instantaneously to the speed (v_1) of the slower vehicle. When passing becomes possible, the faster vehicle instantaneously resumes its desired speed v .

In order to pass a slower vehicle, a faster vehicle must travel in the opposing lane a distance d_r relative to the slower vehicle. The time required to pass is

$$t_p = \frac{d_r}{v - v_1}. \quad (3.1)$$

The total distance traveled in the opposing lane during the passing maneuver is

$$d_p = v t_p = \frac{d_r v}{v - v_1}. \quad (3.2)$$

Tanner (1958) has proposed a model which includes a time t_a that a faster vehicle following a slower vehicle must wait after a passing opportunity has started. This adjusts the model for the extra time required by acceleration. The time required to pass and the distance traveled in the opposing lane are then

$$t_{p|a} = t_a + \frac{d_r}{v - v_1} \quad (3.3)$$

$$d_{p|a} = v t_{p|a}. \quad (3.4)$$

These models consider situations where a vehicle with a higher desired speed passes vehicles with lower desired speeds one at a time. It is not a realistic model for passings in a platoon. Jacobs (1974) has suggested a model based on queuing theory with slower vehicles as servers and faster vehicles as customers. Passing

mechanism follows the first-in-first-out (FIFO) queuing discipline. As compared to the model above, the Jacobs (1974) model serves to benefit slower vehicles to the disadvantage of faster vehicles, resulting in a lower overall mean speed McLean (1989). It is also possible that a vehicle passes several vehicles in a platoon in a single passing maneuver.

Tanner (1961) and Yeo (1964) have developed a model where slow vehicles move in platoons and each platoon is passed in a single maneuver. Miller (1961a, 1961b, 1962, 1963) assumed that vehicles moved in platoons, and faster traveling platoons passed slower platoons. Passings, however, occurred randomly and independently. Many of these models were based on the kinetic theory approach described by Newell (1955). In general, the passing mechanism in a platoon has been too complicated for traffic flow models and has been greatly simplified.

3.2 Passing demand

Let us consider a homogenous section of a two-lane highway having traffic in one direction only and no passing-sight limitations. Because passing supply is infinite, vehicles can maintain their desired speed at all times. If the distance from a fast vehicle with speed v_j to a slow vehicle with speed $v_i < v_j$ is d_{ij} , the faster vehicle will catch up the slower vehicle in time $d_{ij}/(v_j - v_i)$. If the density of vehicles with speed v_i is k_i , the passing rate per unit time of a vehicle traveling at speed v_j is

$$R_p = k_i(v_j - v_i). \quad (3.5)$$

Passing density of all vehicles with speed v_j is

$$\rho_{p|j} = k_j k_i (v_j - v_i), \quad (3.6)$$

where k_j is the density of vehicles with speed v_j . The equation can also be expressed in terms of flow rates (Normann 1942):

$$\rho_{p|j} = \frac{q_j q_i (v_j - v_i)}{v_j v_i} = q_j q_i \left(\frac{1}{v_i} - \frac{1}{v_j} \right), \quad (3.7)$$

where q_j and q_i are the flow rates of vehicles with speeds v_j and v_i , respectively.

Let the density function of the space distribution of (desired) speeds be $f(v)$. The flow rates and densities of vehicles in the infinitesimal speed ranges $(v_j, v_j + dv_j)$ and $(v_i, v_i + dv_i)$ are then

$$k_j = k f(v_j) dv_j \quad (3.8)$$

$$k_i = k f(v_i) dv_i \quad (3.9)$$

$$q_j = q f(v_j) dv_j \quad (3.10)$$

$$q_i = q f(v_i) dv_i, \quad (3.11)$$

where k is the total traffic density and q is the total flow rate in the observed direction. A vehicle of speed v_j passes a vehicles of speed v_i at rate (Newell 1955)

$$\rho_{p|j} = (v_j - v_i)k_i f(v_i) dv_i. \quad (3.12)$$

Total active passing density is obtained by integration over speeds v_j and $v_i \leq v_j$:

$$\begin{aligned} \rho_p &= k^2 \int_0^\infty \int_0^{v_j} (v_j - v_i) f(v_i) f(v_j) dv_i dv_j \\ &= q^2 \int_0^\infty \int_0^{v_j} \left(\frac{1}{v_i} - \frac{1}{v_j} \right) f(v_i) f(v_j) dv_i dv_j. \end{aligned} \quad (3.13)$$

Passing density can also be expressed from the standpoint of passive passings

$$\begin{aligned} \rho_p &= k^2 \int_0^\infty \int_{v_i}^\infty (v_j - v_i) f(v_j) f(v_i) dv_j dv_i \\ &= q^2 \int_0^\infty \int_{v_i}^\infty \left(\frac{1}{v_i} - \frac{1}{v_j} \right) f(v_j) f(v_i) dv_j dv_i. \end{aligned} \quad (3.14)$$

Because of symmetry, the upper double intergrals in equations (3.13) and (3.14) amount to one half of the *Gini's mean difference* $\gamma(v)$ (Stuart and Ord 1987, Johnson, Kotz and Balakrishnan 1994) of the speed distribution:

$$\gamma(v) = \int_0^\infty \int_0^\infty |v_j - v_i| f(v_i) f(v_j) dv_i dv_j = \frac{2\rho_p}{k^2}. \quad (3.15)$$

Passing density can then be expressed in terms of the mean difference of speeds:

$$\rho_p = \frac{k^2 \gamma(v)}{2} = \frac{q^2 \gamma(v)}{2 \bar{v}_s^2}. \quad (3.16)$$

As flow rate and speed variation increase, passing rate increases also. Assuming that the speed distribution is unsensitive to changes in traffic density, passing rate in unimpeded conditions increases with the square of flow rate. The result is intuitive: If flow rate doubles, the number of vehicles slower than v doubles, which doubles the passing demand of vehicles faster than v . In addition, the number of vehicles faster than v also doubles, which quadruples the total passing demand (Beckmann, McGuire and Winsten 1956).

When the space distribution of speeds is normal with mean $\bar{v}_s$ and standard deviation $\sigma_s(v)$, the mean difference of speeds is $\gamma(v) = 2\sigma_s(v)\pi^{-1/2}$ (Stuart and Ord 1987). Passing density is accordingly (Wardrop 1952):

$$\rho_p = \frac{k^2 \gamma(v)}{2} = \frac{k^2 \sigma_s(v)}{\sqrt{\pi}} \approx 0.564 k^2 \sigma_s(v) = 0.564 \frac{q^2 \sigma_s(v)}{\bar{v}_s^2}. \quad (3.17)$$

This ‘‘Wardrop catch-up rate formula’’ has been used as an estimate of passing demand. It is directly proportional to the standard deviation of space distribution of speeds and to squared traffic density.

Equations (3.13) and (3.17) apply only to situations with no passing delay. The model even allows the possibility of several passings of a vehicle at the same time. When passing opportunities are limited, passing delays reduce the catch-up rate. According to McLean (1989), equation (3.17) provides an upper bound for passing demand.

3.3 Passing opportunities

Passing is possible if sight distance is longer than a critical sight distance (d_c) and if a long enough distance (d_f) in the opposing lane is free of oncoming vehicles. The critical sight distance assures that an oncoming vehicle coming from behind a sight obstacle and traveling with an assumed maximum speed $v_{\max}$ does not cause a hazard to the passing vehicle.

If oncoming vehicle is approaching at speed v_o , during the time t_p that the passing vehicle occupies the opposing lane, the oncoming vehicle travels distance

$$d_o = v_o t_p = \frac{d_r v_o}{v - v_1}, \quad (3.18)$$

where d_r is the distance relative to the slower vehicle that the faster vehicle must travel in the opposing lane. The distance free of oncoming vehicles should, accordingly, be greater than

$$d_f = d_p + d_o = \frac{d_r(v + v_o)}{v - v_1}. \quad (3.19)$$

Correspondingly, the critical sight distance is

$$d_c = \frac{d_r(v + v_{\max})}{v - v_1}. \quad (3.20)$$

A safety margin should be included in d_r .

If a passing vehicle with speed v requires time t_p on the opposing lane, the necessary time gap t_o in opposing flow is (Normann 1942)

$$t_o = \frac{t_p(v_2 + v)}{v_2}, \quad (3.21)$$

where v_2 is the speed of the oncoming vehicle.

Normann (1942) found the following linear model for the ratio of observed to desired passings

$$P = 0.903 - 0.000467q, \quad (3.22)$$

where q is the total two-way flow rate. The number of observed passings becomes zero as flow rate reaches 1,934 veh/h.—It should be noted that the data of Normann shows considerable scatter and an apparent non-linear effect (McLean 1989).

Using equations (3.7) and (3.22) Normann calculated the number of actual passings. Figure 3.1 displays the number of required and actual passings based on equations (3.17) and (3.22), when 2/3 of the traffic flow travels to main direction, the space mean of free-flow speeds is 70 km/h, and the standard deviation is 10 km/h. The parameters were selected so that the distribution is similar to the one presented by Normann (1942), even though his distribution was not normal.

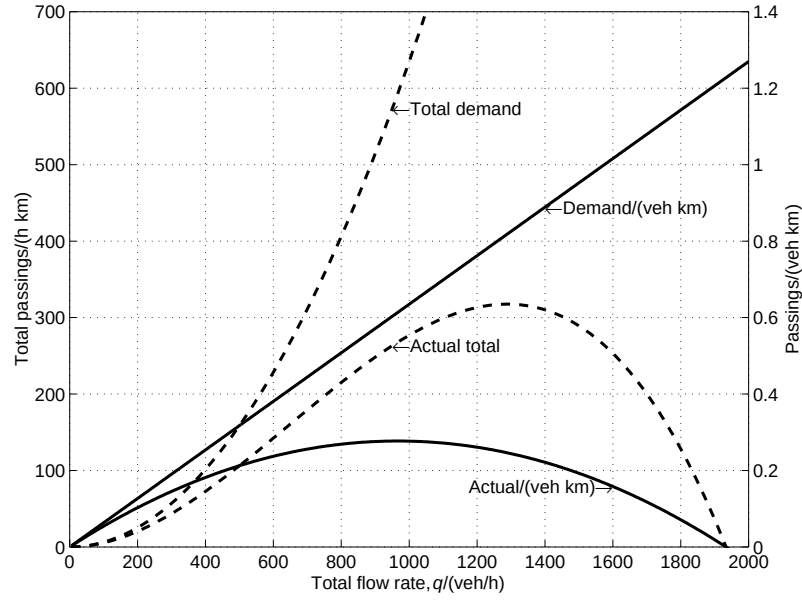


FIGURE 3.1 · Required (upper curve) and actual (lower curve) number of passings when 2/3 of total traffic travels to main direction, and free-flow speed distribution is normal with $\bar{v}_s = 70$ km/h and $\sigma_s(v) = 10$ km/h

According to Normann (1942) the average time in seconds that slow vehicles must be trailed whenever passing cannot be started immediately is

$$E[W_p] = \frac{1}{2} \left[\frac{3600(1 - P_{t_o})}{(1 - F(t_o))q_o \left(\frac{\bar{v}_o + v_1}{\bar{v}_o} \right)} \right], \quad (3.23)$$

where

$E[W_p]$ = average trailing time in seconds

P_{t_o} = proportion of time of long enough headways in opposing flow

$1 - F(t_o)$ = proportion of long enough headways in opposing flow

q_o = opposing hourly traffic volume

$\bar{v}_o$ = average speed of oncoming traffic (miles per hour)

v_1 = speed of the vehicle to be passed (miles per hour).

If the space mean of desired speeds is constant, passing demand increases as traffic flow and standard deviation of desired speeds increase. The supply for passing opportunities depends on the opposing flow and availability of passing sights.

German theoretical studies (Brilon 1977, Heidemann 1993) have indicated that the influence of the opposing flow does not change significantly as the opposing flow rises above 400–450 veh/h. According to Heidemann (1993), for opposing flows above 340 km/h the capacity of the major direction is 1720 veh/h. The “optimal” density on major direction is 35 veh/km with the mean speed of approximately 50 km/h.

3.4 Sight models

3.4.1 Gustavsson's model

Sight distances on a highway are restricted by vertical curvature and horizontal curvature connected with sight obstructions, such as trees or buildings beside the roadway. As a driver approaches an obstruction, such as a hill top, the sight distance decreases to a minimum. At the top sight almost instantaneously opens to the next obstruction. Figure 3.2 displays the saw-like shape of a sight curve. It decreases linearly as a vehicle approaches the obstruction, increases rapidly to a higher value as the vehicle passes the obstruction in order to decrease again.

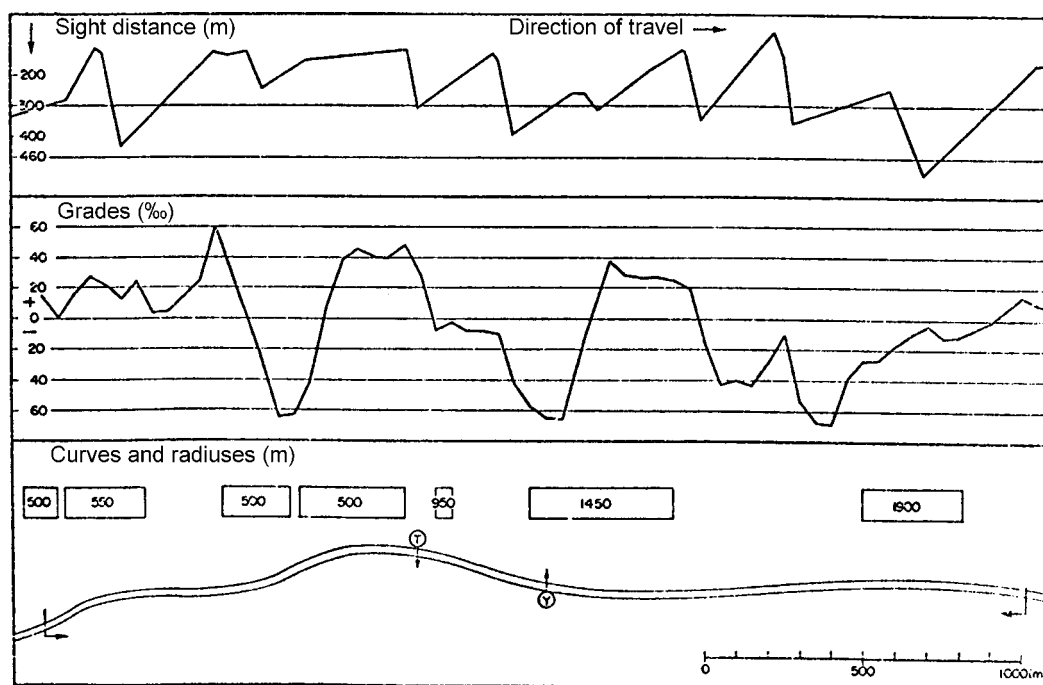


FIGURE 3.2 · Sight distances along a highway (note inverted scale) (Roine 1972)

Gustavsson (1967) has introduced a sight function, which in a simplified manner describes the shape of the sight curve. It is assumed that along an infinitely long highway there is a minimum sight distance d_0 . Sight distance decreases linearly to the minimum (d_0) as the distance to a sight obstruction decreases. That is, sight distance extends a distance d_0 beyond the obstruction. At the point of the obstruction sight distance is d_0 , but increases with a jump and starts to decrease to the minimum again. The obstructions and, consequently, the jumps are assumed to be randomly located along the roadway. The number of jumps on a given roadway length, accordingly, follows the Poisson distribution, and the

distance between jumps d_j follows the negative exponential distribution

$$P\{d_j \leq x\} = F_j(x) = \begin{cases} 1 - e^{-\rho_j x}, & \text{if } x \geq 0 \\ 0 & \text{if } x < 0, \end{cases} \quad (3.24)$$

where ρ_j is the density of jumps. This is the distribution function of sight distances above d_0 .

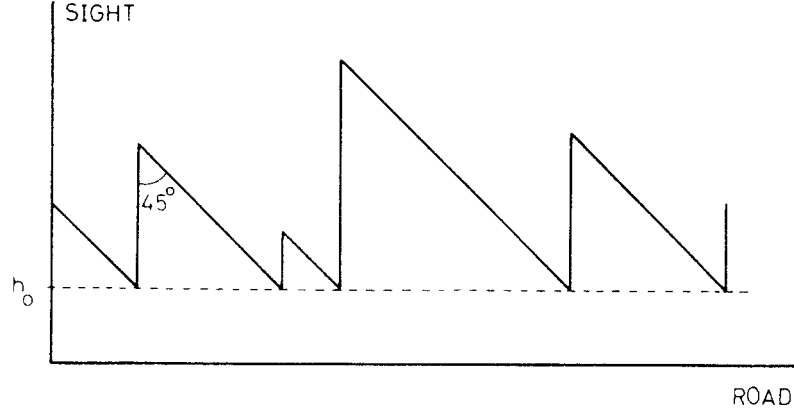


FIGURE 3.3 · Sight distances along a roadway according to the model of Gustavsson (1967)

3.4.2 Kallberg's model

The effect of sight distances has also been considered by Kallberg (1980). He recalls that the relative speed of the opposing traffic is $v + v_o$, and its traffic density is $k_o = q_o/v_o$. The opposing flow rate relative to a fast vehicle is

$$q_r = k_o(v + v_o) = \frac{q_o(v + v_o)}{v_o}. \quad (3.25)$$

If oncoming traffic has exponential headways and constant speeds the probability of a passing opportunity for one vehicle during a short time interval Δt is

$$P_p = q_r \Delta t \sum_{i=1}^{\infty} e^{-it_p q_r} = q_r \Delta t \frac{e^{-it_p q_r}}{1 - e^{-it_p q_r}}. \quad (3.26)$$

If sufficient sight distance exists at each point with probability P_s , the probability of a passing opportunity is

$$P_p = P_s q_r \Delta t \frac{e^{-it_p q_r}}{1 - e^{-it_p q_r}}. \quad (3.27)$$

This assumes that the probabilities of sight distances along a roadway are independent. The model of Gustavsson (1967) is more realistic in this respect (see figures 3.2 and 3.3).

4

Platooning

4.1 Distance spent following

Let us assume that vehicles in observer direction and opposing direction travel at constant speeds v_1 and v_o , respectively. Vehicle spacings are random; i.e., they follow negative exponential distributions

$$F_1(x) = \begin{cases} 1 - e^{-k_1 x}, & \text{if } x \geq 0 \\ 0 & \text{if } x < 0, \end{cases} \quad (4.1)$$

$$F_2(x) = \begin{cases} 1 - e^{-k_o x}, & \text{if } x \geq 0 \\ 0 & \text{if } x < 0, \end{cases} \quad (4.2)$$

where k and k_o are densities of traffic in observed and opposing directions, respectively. Sight distances above d_0 follow the distribution in equation (3.24). The probability for a sight distance greater than d is accordingly $1 - F_j(d - d_0)$. For simplicity, the sight distances can be measured as distances above d_0 , so that critical sight distance above d_0 is $d_{c0} = d_c - d_0$. The following notation is also adopted (Gustavsson 1967):

- d_s = sight distance
- d'_c = sight distance above d_c ; i.e., $(d_s - d_c)_+$
- d_v = distance from the fast vehicle to the first oncoming vehicle
- d_1 = traveling distance, at speed v_1 , up to the nearest oncoming vehicle ($c_1 d_v$)
- $c_1 = v_1 / (v_1 + v_o)$
- $c_2 = v / (v - v_1)$
- $c_3 = v / (v + v_o)$

When a faster vehicle traveling with speed v catches up a slower vehicle with speed v_1 , it must follow behind the slower vehicle a distance (W_d) depending on the sight distance (d_s) and distance (d_v) to the first oncoming vehicle. That is, W_d is (Gustavsson 1967):

- W_{00} , if sight distance is $d_s \leq d_c$ and distance to the next oncoming vehicle is $d_v \leq d_f$. This event has probability $P_{00} = F_j(d_{c0}) F_2(d_f)$. The faster vehicle must travel a distance $\max \{d_s, d_1\}$, after which there is no information about either the sight or the distance to the first oncoming vehicle. The distance spent following is, accordingly,

$$W_{00} = \max \{d_s, d_1\} + W \quad (4.3)$$

- W_{01} , if sight distance is $d_s \leq d_c$ and distance to the next oncoming vehicle is $d_v > d_f$. This event has probability $P_{01} = F_j(d_{c0}) [1 - F_2(d_f)]$. The expression for W_{01} is analogous to the case W_{10} below.
- W_{10} , if sight distance is $d_s > d_c$ and distance to the next oncoming vehicle is $d_v \leq d_f$. This event has probability $P_{10} = [1 - F_j(d_{c0})] F_2(d_f)$. This case has two subcases:

1. If $d_1 > d_c$ and the faster vehicle travels the distance d_1 , there is no information left about either the sight distance nor the distance to the first oncoming vehicle. The distance spent following can therefore be written as

$$W_{10} = d_1 + W. \quad (4.4)$$

2. If $d_1 \leq d_c$, the traveling distance to the first oncoming vehicle does not exceed d_c , and two cases must be further distinguished

- (a) If $d'_c \leq d_1$, the faster vehicle has to travel behind the slower one at least the distance $d_c + d'_c$, after which no information about sight or oncoming traffic conditions is left. Accordingly,

$$W_{10} = d_{c0} + d'_{c0} + W. \quad (4.5)$$

- (b) If $d_1 > d_{c0}$, the sight exceeds d_{c0} when the faster vehicle meets the oncoming vehicle. There are two further subcases:

- i. If the faster vehicle has traveled the distance d_1 further and $d_v > d_f$, passing is possible, and therefore

$$W_{10} = d_1. \quad (4.6)$$

- ii. If the faster vehicle has traveled the distance d_1 further and $d_v \leq d_f$, passing is not yet possible. The vehicle is in a position where assumptions are the same as have been assumed for W_{10} . If the further waiting distance is denoted by W'_{10} , the whole waiting distance is

$$W_{10} = d_1 + W'_{10}. \quad (4.7)$$

- W_{11} , if sight distance is $d_s > d_c$ and distance to the next oncoming vehicle is $d_v > d_f$. This event has probability $P_{11} = [1 - F_j(d_{c0})][1 - F_2(d_f)]$. Passing is possible without any delay, and the waiting distance is $W_{11} = 0$.

According to Gustavsson (1967), the mean distance spent following is

$$\begin{aligned}
 E[W_d] = & \frac{-1}{\frac{k_1}{c_2}} + \frac{\rho_j}{\frac{k_1}{c_2} \left(\rho_j + \frac{k_1}{c_2} \right)} e^{-(k_1/c_2)d_{c0}} + \frac{\rho_j + \frac{k_o}{c_3}}{\left(\rho_j + \frac{k_o}{c_1} \right) \left(\frac{k_1}{c_2} + \rho_j + \frac{k_o}{c_3} \right)} e^{\rho_j d_{c0} + k_o d_f} \\
 & + \frac{\frac{k_o}{c_1} \left(1 - \frac{c_1}{c_3} \right)}{\left(\rho_j + \frac{k_o}{c_1} \right) \left[\frac{k_1}{c_2} + \rho_j \left(1 - \frac{c_1}{c_3} \right) \right]} e^{\rho_j (d_{c0} - c_1 d_f)} \\
 & + \frac{\frac{c_1 k_1 k_o}{c_3^2 c_2}}{\left(\rho_j + \frac{k_1}{c_2} \right) \left[\frac{k_1}{c_2} + \rho_j \left(1 - \frac{c_1}{c_3} \right) \right] \left(\frac{k_1}{c_2} + \rho_j + \frac{k_o}{c_3} \right)} e^{-(k_1/c_2)c_3 d_f + \rho_j (d_{c0} - c_3 d_f)}. \quad (4.8)
 \end{aligned}$$

If $\rho_j = 0$, equation (4.8) gives the mean distance spent following assuming that there are no passing sight restrictions.

4.2 Short headways

In HCM (Transportation Research Board 1985) the proportion of headways less than five seconds $F(5)$ was used as an approximation of percent time delay (PTD). In HCM2000 (Transportation Research Board 2000) the corresponding service measure is called percent time spent following (PTSF). It can be approximated by the proportion of headways less than or equal to three seconds (Harwood et al. 1999). It will be shown that directional distribution has an effect on $F(5)$ and $F(3)$ even if there is no intervehicular interaction and, consequently, no platooning.

If the proportion of flow on the major direction is P_d , the proportion of headways less than five seconds is obtained by the weighted average of $F(5)$ on both lanes

$$F(5 | q, P_d) = P_d F\left(5 \mid P_d \frac{q}{3600}\right) + (1 - P_d) F\left(5 \mid (1 - P_d) \frac{q}{3600}\right), \quad (4.9)$$

where q is the flow rate (veh/h). If traffic flow is random, time headways follow the negative exponential distribution, and the proportion of headways less than five seconds is

$$F(5 | q, P_d) = P_d \left(1 - e^{-P_d q / 720}\right) + (1 - P_d) \left(1 - e^{-(1-P_d)q/720}\right) \quad (4.10)$$

Figure 4.1 displays $F(5)$ and the increase in $F(5)$ due to directional distribution when headways follow the negative exponential distribution. The increase is largest under medium flows (850–1,000 veh/h), where the gradient of the $F(5)$

curve is steepest. As exponential distribution describes a traffic flow with no vehicle interactions, figure 4.1 shows the effect of directional distribution with no vehicular interaction.

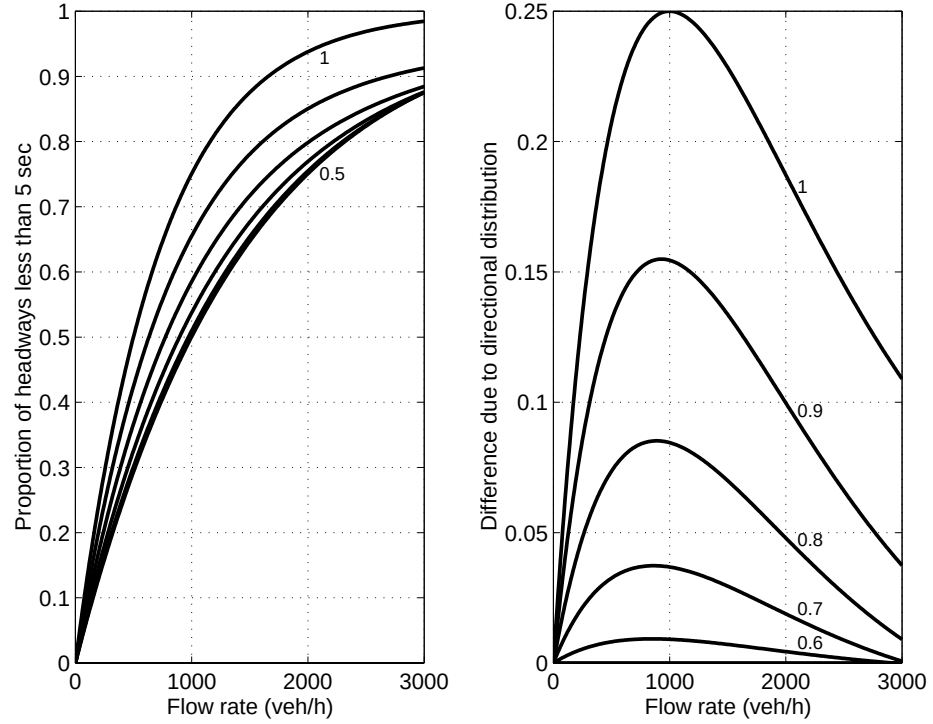


FIGURE 4.1 · The effect of directional distribution on the proportion of headways less than five seconds in a random flow

Figure 4.2 displays the effect of directional distribution on headways not larger than three seconds:

$$F(3 | q, P_d) = P_d (1 - e^{-P_d q / 1200}) + (1 - P_d) (1 - e^{-(1-P_d)q / 1200}). \quad (4.11)$$

The differences due to directional distribution have the same magnitudes, but maximum differences occur at higher flow rates (1,410–1,660 veh/h).

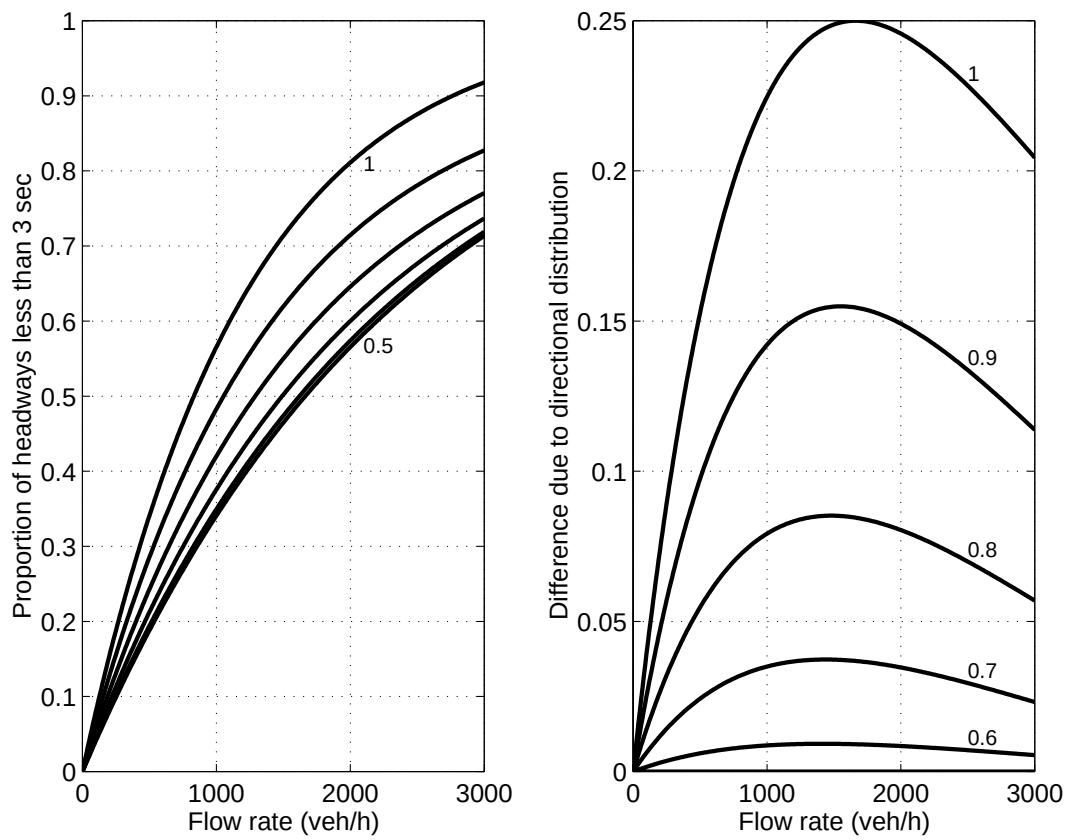


FIGURE 4.2 · The effect of directional distribution on the proportion of headways not larger than three seconds in a random flow

5

Average travel speed

5.1 Erlander's model

5.1.1 Theory and assumptions

Erlander (1967, 1969, 1971*a*, 1971*b*) applied the sight model of Gustavsson (1967, 1969) to the model of Carleson (1957), which assumed that desired speeds followed a space distribution ($F_v(x)$). There was both a lower ($v_{\min} > 0$) and an upper ($v_{\max} < \infty$) limit for desired speeds.

For the mean distance spent following ($E[W_d]$) Erlander applied Gustavsson's equation (4.8). Below it will be denoted by $W_d(v, v_1)$ so that the speeds of fast and slow vehicles can be explicitly given as parameters of the model. The interaction of the overtaken vehicle with other vehicles was not considered, but the overtaken vehicle maintained its average speed $E[v_1]$ and not its desired speed v_1 . The equation (3.20) for critical sight distance was adjusted similarly:

$$d_c = \frac{d_r(v + v_{\max})}{v - E[v_1]}. \quad (5.1)$$

It was, accordingly, assumed that the slower vehicle may already have been delayed when the faster vehicle caught up with it. According to Erlander (1971*a*), equation (4.8) cannot be expected to be a good approximation in dense traffic, where the distance traveled between successive passings is short, or when the oncoming traffic deviates considerably from the structure (Poisson process) assumed by Gustavsson (1967). The theory of Erlander (1967) is described below.

Let $E[v]$ be the average speed of a vehicle with desired speed v . A vehicle with desired speed v overtakes in a time unit on the average

$$R_p(v, v_1) = [E[v] - E[v_1]] k_1 dF_v(v_1) \quad (5.2)$$

vehicles with desired speed $v_1 < v$ (see section 3.2). If a vehicle with desired speed v has to travel on the average a distance $W_d(v, E[v_1])$ at speed $E[v_1]$ before

passing, the proportion of time spent driving in speed $E[v_1]$ is

$$\begin{aligned} P_W(v, E[v_1]) &= \frac{W_d(v, E[v_1]) R_p(v, v_1)}{E[v_1]} \\ &= \frac{W_d(v, E[v_1])}{E[v_1]} [E[v] - E[v_1]] k_1 dF_V(v_1). \end{aligned} \quad (5.3)$$

Considering all passed vehicles with speeds lower than v , the proportion of time spent following is

$$P_W(v) = \int_{v_{\min}}^v \frac{W_d(v, E[v_1])}{E[v_1]} [E[v] - E[v_1]] k_1 dF_V(v_1). \quad (5.4)$$

For the proportion of time $1 - P_W(v)$ the vehicle travels at its desired speed v . The average proportion of time spent following for all vehicles in the stream is

$$P_W = \int_{v_{\min}}^{v_{\max}} P_W(v) dF_V(v). \quad (5.5)$$

The average travel speed (average distance traveled in one time unit) of a vehicle with desired speed v is thus

$$\begin{aligned} E[v] &= v[1 - P_W(v)] + E[v_1] P_W(v) \\ &= v - \int_{v_{\min}}^v W_d(v, E[v_1]) [E[v] - E[v_1]] \left[\frac{v}{E[v_1]} - 1 \right] k_1 dF_V(v_1) \\ &= \frac{v + k_1 \int_{v_{\min}}^v W_d(v, E[v_1]) [v - E[v_1]] dF_V(v_1)}{1 + k_1 \int_{v_{\min}}^v W_d(v, E[v_1]) \frac{v - E[v_1]}{E[v_1]} dF_V(v_1)}, \quad (v > v_{\min}). \end{aligned} \quad (5.6)$$

This equation “cannot in general be solved by simple methods” (Erlander 1967).

The equation (5.6) for $E[v]$ has itself ($E[v_1]$) as a parameter, but the parameter v_1 of the latter is not greater than the original parameter v . This suggests that equation (5.6) could be solved recursively. A starting point for the method is obtained by observing that $E[v_{\min}] = v_{\min}$.

Erlander (1967) has proposed a discrete approximation method to estimate the integral equation (5.6). The interval $[v_{\min}, v_{\max}]$ is partitioned into n speed classes

$$v_{\min} = u_0 < u_1 < \cdots < u_n = v_{\max}. \quad (5.7)$$

All speeds $u_{i-1} < v \leq u_i$ are assigned to speed class u_i .

A vehicle has desired speed u_i with probability p_i defined as

$$\begin{aligned} p_0 &= F_V(u_0) \\ p_i &= F_V(u_i) - F_V(u_{i-1}), \quad i = 1, \dots, n. \end{aligned} \quad (5.8)$$

For this discrete speed distribution, equation (5.6) can be written as:

$$E[v] = \frac{v + k_1 \sum_{j=0}^{i-1} W_d(v, E[u_j]) [v - E[u_j]] p_j}{1 + k_1 \sum_{j=0}^{i-1} W_d(v, E[u_j]) \frac{v - E[u_j]}{E[v_1]} p_j}, \quad (5.9)$$

where

$$i = \{ \kappa : u_{\kappa-1} < v \leq u_{\kappa} \}. \quad (5.10)$$

The summation goes from 0 to $i - 1$. Because there is no queuing model, vehicles with equal desired speeds have no interaction and, consequently, $v - E[u_i] \approx 0$.

The mean speeds $E[u_i]$ of slower vehicles can now be solved recursively:

$$E[u_0] = u_0 \quad (5.11)$$

$$E[u_i] = \frac{u_i + k_1 \sum_{j=0}^{i-1} s(u_i, E[u_j])[u_i - E[u_j]]p_j}{1 + k_1 \sum_{j=0}^{i-1} s(u_i, E[u_j]) \frac{u_i - E[u_j]}{E[u_j]} p_j}. \quad (5.12)$$

This equation is a slightly modified version of equation (6) of Erlander (1967). The modification ($E[u_{i-1}] \rightarrow E[u_i]$) makes the equation less complicated and is based on the discussion above about no interaction between vehicles with equal desired speeds.

McLean (1989) observed some local instabilities in the method as the number of speed classes increased. As speed difference approached zero, the critical gap for passing tended to infinity. He adjusted the equation (5.1) for critical sight distance for greater than desired speeds during passing maneuvers:

$$d_c = \frac{d_r(v + \Delta v + v_{\max})}{v + \Delta v - E[v_1]}, \quad (5.13)$$

where Δv was the additional increment in speed a driver was willing to employ while passing. He found $\Delta v = 4$ km/h a suitable approximation.

The Erlander model has some important limitations:

- Opposing flow is considered random. The model does not consider the increase in passing opportunities due to platooning of the opposing flow (cf. Ashton, Buckley and Miller 1968).
- Platooning and passing in platoons are not considered properly.
- Instantaneous speed changes in the passing model are not realistic.
- The length of the observed highways segment is infinite. In real life homogeneous highways segments are typically not longer than a few kilometers.

These limitations are more severe at high traffic densities. Jacobs (1974) has also noted that the slower speeds in this model change alike to the changes of the faster speeds, which is contrary to the experience. It can, however, be assumed that the model gives some useful information about the effect of flow rates and no-passing zones on the average travel speeds (Luttinen 2000).

5.1.2 Numerical results

Numerical results (figs. 5.1, 5.2, and 5.3) were calculated by the Erlander (1967) method with slight modifications:

- Desired speed distribution was assumed normal with standard deviation 10% of mean desired speed.
- The minimum and maximum speeds were $v_{\min} = \bar{v} - 30 \text{ km/h}$ and $v_{\max} = \bar{v} + 30 \text{ km/h}$.
- The probabilities (p_i) of desired speed classes were defined using conditional distributions:

$$F_V(u | u \leq v_{\max}) = \frac{F_V(u)}{F_V(v_{\max})}. \quad (5.14)$$

This modification assured that $\sum_{i=0}^n p_i = 1$.

- The speed of oncoming flow was assumed constant. This speed was also used in the calculation of critical sight distance.
- The modification in equation (5.12) has been described above.
- Because the probability mass of each speed class is assigned to the upper limit of the class, the average travel speed is overestimated by one half of the class width $b = (u_u - u_{i-1})/2$. This bias is corrected by subtracting b from the average travel speed estimate of the model.

The HCM 2000 (Transportation Research Board 2000) passing sight criterion (300 m) has been used. The probability of a no-passing zone can then be calculated as

$$P_{np} = 1 - e^{-\rho_j(0.3-d_0)}. \quad (5.15)$$

Jump density ρ_j was, accordingly, defined as

$$\rho_j = \frac{\ln(1 - P_{np})}{0.3 - d_0}. \quad (5.16)$$

Minimum sight distance (d_0) was 150 m.

Desired speeds in the observed direction were assumed to follow normal distribution with mean 95 km/h and coefficient of variation 10%. The speed (v_o) of oncoming traffic at prevailing flow rate (q_o) was assumed to be constant

$$v_o = 95 - 0.0125q_o. \quad (5.17)$$

In passing opportunity calculations, the maximum speed of oncoming vehicles was assumed to be 30 % higher than v_o . Passing was assumed to be performed at desired speed. A higher passing speed would presumably have better described the actual passing maneuver. The applied model, however, reflects the possibility that a trailing vehicle does not necessarily very actively search for an immediate

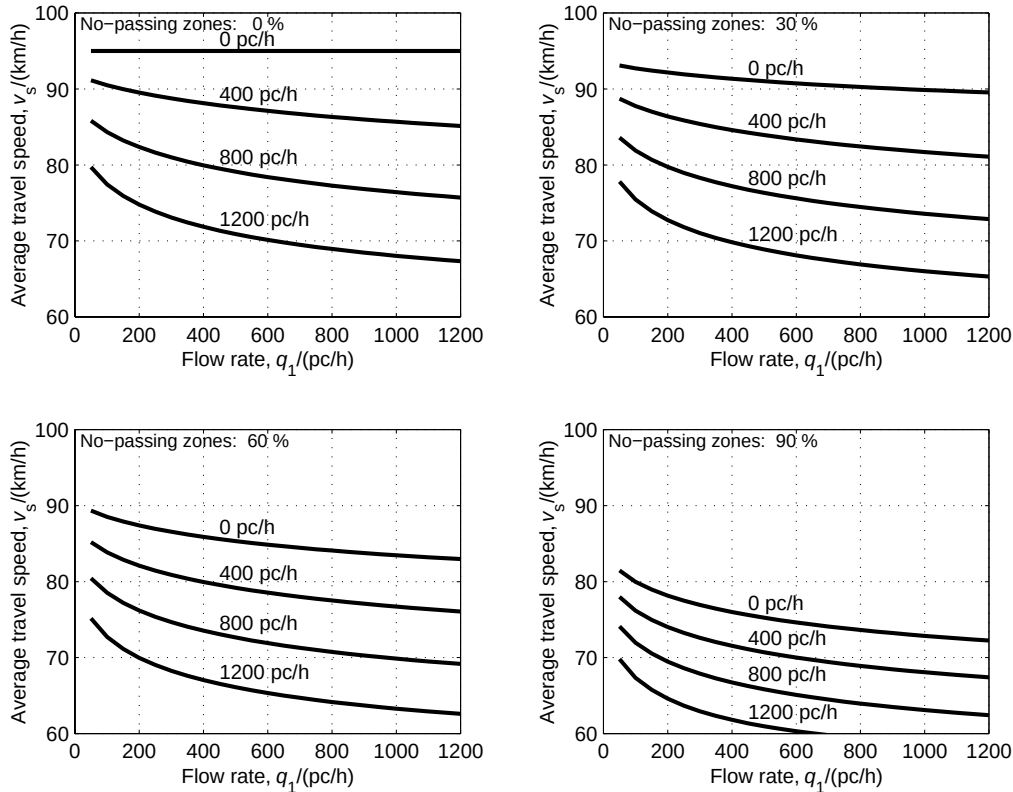


FIGURE 5.1 · Speed-flow diagrams for different levels of opposing flow

passing opportunity, if the desired speed of the vehicle ahead is only slightly lower than the desired speed of the trailing vehicle. The relative distance (d_T) moved during the passing maneuver was 50 m.

If there are no passing restrictions and no opposing traffic, the model assumes that vehicles travel at their desired speed and pass without slowing down (fig. 5.1). The average travel speed is thus equal to the average desired speed, irrespective of flow rate. It is evident that the model is not realistic at high flow rates.

Figure 5.1 demonstrates that the model produces concave speed-flow curves as flow rate in the observed direction increases and the flow rate in opposing direction is constant. Speed decrease is steeper at low flow rates. When both opposing flow rate (q_o) and percent no-passing zones (P_{np}) are high, the initial drop in average travel speed is very steep. Concave speed-flow curves were also produced by the theoretical model of (Jacobs 1974), which did not include the effects of sight conditions, but had an explicit model for queuing.

Average travel speed decreases almost linearly as opposing flow rate (q_o) increases (fig. 5.2). The model suggests that the effect of opposing flow is larger than the effect of the flow rate in the observed direction. Empirical research has, however, suggested just the opposite: The opposing flow has a smaller

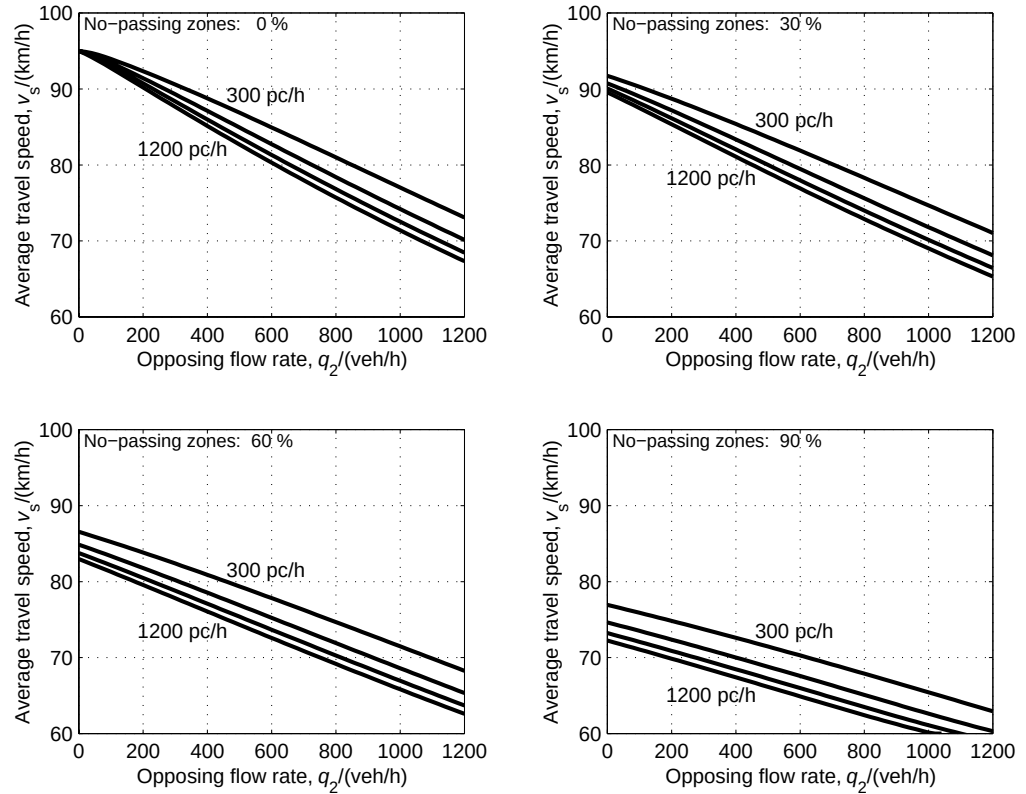


FIGURE 5.2 · The effect of opposing flow on average travel speeds for different levels of flow in the observed direction

effect on average travel speeds than the flow rate in the observed direction (Luttinen 2000, McLean 1989). The lack of a queuing model probably leads to an underestimation of the effect of flow rate in the observed direction and, consequently, overestimation of the effect of the opposing flow rate.

As the percentage of no-passing zones (P_{np}) increases the effect of opposing flow rate (q_o) decreases. This is intuitive: As one factor limits passing opportunities the effect of other similar factors diminishes. The curves for different flow rates in the observed direction also become more parallel. This is, however, due to the unrealism in the model under good sight conditions when q_o is low and q_1 is high.

The decrease in the average travel speed becomes steeper as the percentage of no-passing zones (P_{np}) increases (fig. 5.3). The effect of no-passing zones decreases as opposing flow (q_o) increases.—Because the model used traffic density as a parameter, flow levels in observed direction were obtained by interpolation from $q_1 = k_1 E[v_1]$. This made the curves slightly irregular.

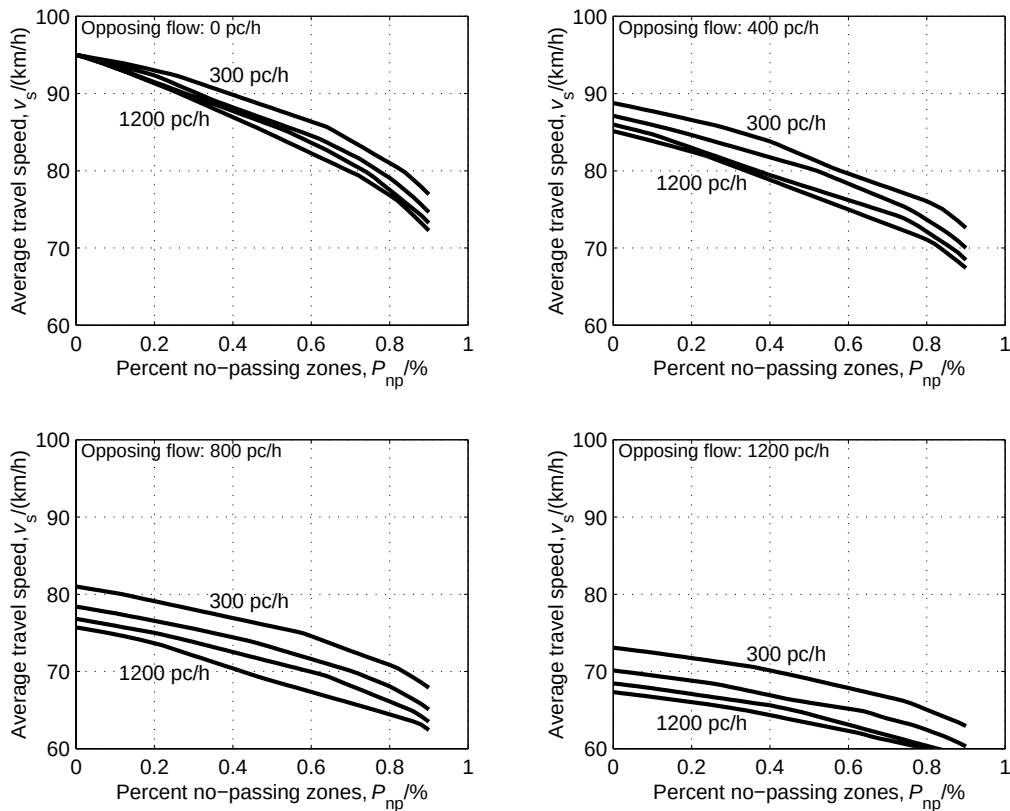


FIGURE 5.3 · The effect of no-passing zones on average travel speeds for different levels of flow in the observed direction

5.2 Standard deviation of average travel speeds

At very low flow conditions there is very little interaction between vehicles, and vehicles can travel almost all the time at the speed desired by the drivers. Such traffic conditions can be characterised as free-flow conditions and the speeds are called *free-flow speeds*.

Even under similar roadway conditions all vehicles do not travel at the same speed. Drivers and vehicles have different characteristics. There are also some inaccuracies in the perception of speed and control of the vehicle, which cause additional variation in the speeds.

This kind of variation can be approximated by the normal distribution. As the name of the distribution indicates, the populations of drivers and vehicles should be “normal”; i.e.; there should be no separate classes of vehicles or drivers. McLean (1989) has given an overview of free-speed studies on two-lane highways. He concluded that the desired speeds of cars can be “reasonably well represented by a normal distribution with mean of about 90 to 100 km/h and a coefficient of variation of about 0.11 to 0.14.”

Some drivers do not feel safe or comfortable when driving at high speeds. They

may have limited driving experience or there may be limitations in the vehicle performance. As the design speed or posted speed increases, the proportion of “slow drivers” is likely to increase. Accordingly, it can be expected that the standard deviation increases as the average free-flow speed increases. The increase in the standard deviation is even more pronounced in a mixed traffic flow of passenger cars and heavy vehicles.

As traffic flow increases, more and more of the vehicles are restricted by a vehicle ahead. Drivers with faster desired speeds follow slower vehicles at slower than desired speeds. This decreases both the mean and the standard deviation of travel speeds (Beckmann et al. 1956). Because the speeds are restricted at the lower end of the distribution only, the skewness of the speed distribution increases as indicated in figure 5.4.

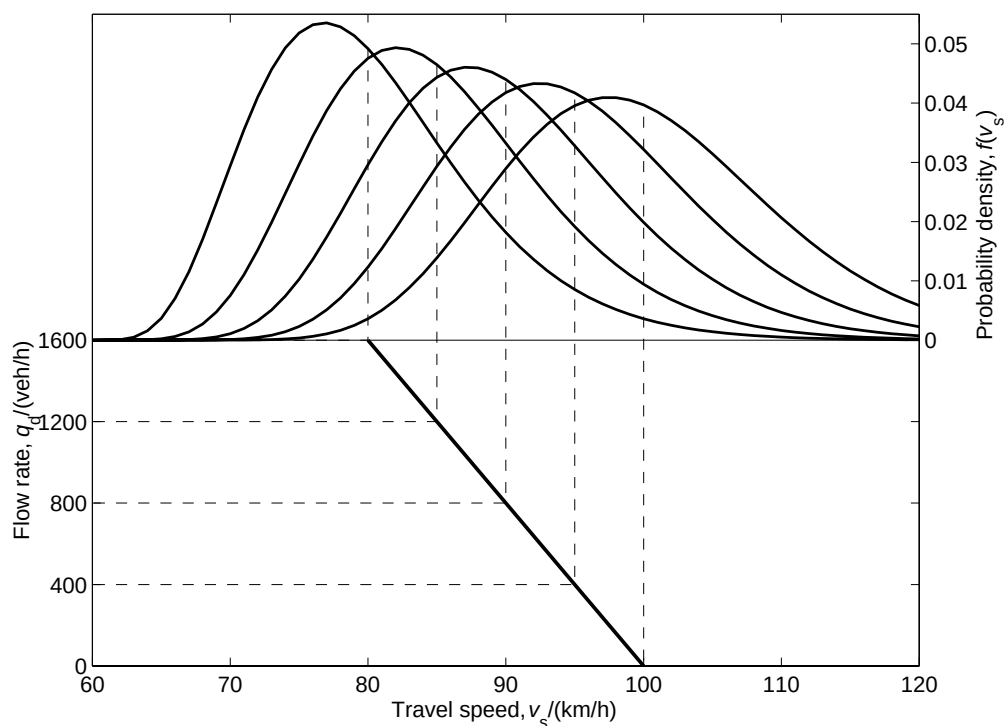


FIGURE 5.4 · Speed distributions and a speed flow curve

Figure 5.4 displays a linear speed-flow curve and travel speed density functions following a three-parameter gamma distribution (Luttinen 1996) with a minimum speed of 60 km/h. The slope of the speed-flow curve is 12.5 as suggested by the new HCM (Harwood et al. 1999). The coefficient of variation of the speed distribution is 0.1. It shows the change in the mean, standard deviation and skewness of the speed distribution as flow rate changes. Although the parameters of the distribution have not been statistically estimated, the basic shapes of the

distributions can be assumed to be realistic. The figure also indicates that the standard deviation of speeds is very large when compared to the speed-flow curve. Accordingly, some problems can be expected in the estimation of the parameters of the speed-flow model.

As the standard deviation of travel speeds increases the passing demand increases also (3.17). If other conditions remain the same, the increase in platooning reduces the ATS. Following the reasoning above, the decrease in the ATS is steeper if the standard deviation of free-flow speeds is large. Because high free-flow speeds are expected to have high standard deviation, it can also be expected that ATS decreases steeper on highways with higher free-flow speeds.

It was assumed that the increase in traffic flow and platooning decreases both the average and standard deviation of travel speeds. If the decrease in ATS is steeper when the standard deviation of travel speeds is large, the travel speeds decrease steeper at low flow rates than at high flow rates. The model of Erlander (figure 5.1 on page 37) also indicates a concave speed-flow curve. It can also be deduced intuitively: When platooning has decreased the ATS, the proportion of vehicles in a given flow increment having a desired speed lower than the ATS is smaller than at free-flow conditions. The impedance effect of the flow increment is then also smaller.

This reasoning has led to the following hypothesis: The speed-flow curves on two-lane highways have a concave shape so that the speed decrease is steepest at low flow rates and highways with high free-flow speeds. This hypothesis has found support in Finnish data (Luttinen 2000).

6

The impact of heavy vehicles

6.1 Types of impact

Heavy vehicles (trucks, buses, recreational vehicles, tractors, passenger cars and vans with trailers etc.) usually have greater impedance effects than passenger cars and vans. These effects are caused by slower speeds of heavy vehicles, difficulties in passing them, and larger roadspace requirements. On the other hand heavy vehicles in opposing stream increase the platooning rate, which increases passing opportunities in the observed direction. (McLean 1989)

Heavy vehicles generally operate at lower unimpeded speeds than passenger cars. Because of a lower power to weight ratio, the impact of heavy vehicles is especially large in grades. In steep upgrades heavy vehicles the speed of heavy vehicles may be extremely low (crawl speed). In steep downgrades heavy vehicles may operate at reduced speeds because of their poor braking performance (Brilon and Weiser 1998, Harwood et al. 1999).

The slower the speed of a slower vehicle the greater is the catch-up rate. On the other hand, passing is also easier. Because of a larger speed difference the passing maneuver takes less time, and it is possible to accept a shorter gap in the opposing flow (Ashton et al. 1968).

The additional length of heavy vehicles and the difficulty of seeing around them makes these vehicles more difficult to pass. About 30-65 % longer gaps in the opposing flow are required when passing a heavy vehicle as compared to passenger cars (McLean 1989). When heavy vehicles pass slower vehicles, they require larger gaps than passenger cars.

Because heavy vehicles are longer than passenger cars, they require more road space. According to McLean (1989), under “tight” following headway conditions the only additional road space is the direct effect of length. Under “relaxed” following conditions drivers may choose to compensate for the difficulty of seeing around a heavy vehicle by maintaining a longer following spacing. Heavy vehicles, however, maintain a following headway similar to passenger cars. The improved view of the road allows them to compensate for the poorer braking

performance. Similar results have been obtained on Finnish freeways (Mannan and Enberg 1998).

6.2 Passenger-car equivalency

Flow rate of mixed traffic of heavy vehicles, passenger cars, and vans is often expressed in terms of passenger-car units. Passenger-car equivalents (PCEs) are used to transform mixed flows to equivalent passenger-car flows.

The PCE of a heavy vehicle is the number of passenger cars having the same impedance effect as a single heavy vehicle of a given type under prevailing roadway, traffic, and control conditions. Assuming that the fraction of trucks and passenger cars in a mixed traffic flow $q = q_T + q_C$ as seen by a stationary observer (Daganzo 1997) is $P_T = q_T/q$, and the PCE is E_T , the equivalent passenger-car flow rate is

$$q_E = (1 - P_T)q + P_T q E_T = q [1 + P_T(E_T - 1)]. \quad (6.1)$$

The PCE is (see figure 6.1)

$$E_T = 1 + \frac{q_E - q}{P_T q}. \quad (6.2)$$

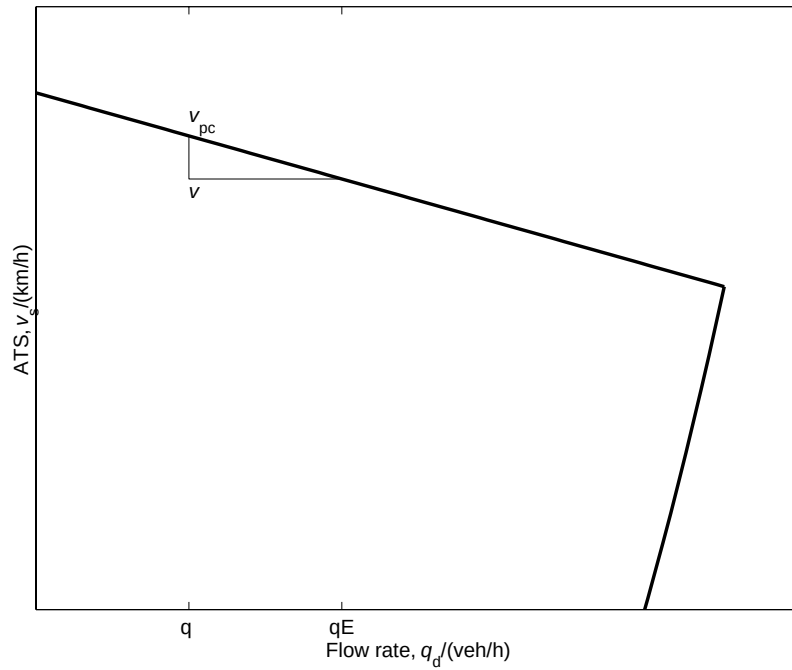


FIGURE 6.1 · Average travel speed as a criterion for passenger-car equivalency

If the PCEs for trucks, buses, and recreational vehicles are E_T , E_B , and E_R , respectively, and the proportions of respective vehicle classes in the traffic flow

are P_T , P_B , and P_R , the equivalent flow rate is (Werner and Morrall 1976)

$$q_E = q [1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)]. \quad (6.3)$$

The heavy vehicle adjustment factor

$$f_{hv} = \frac{q}{q_E} = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)} \quad (6.4)$$

converts the flow rate in passenger car units to a mixed flow rate with given percentages of heavy vehicles.

Let us assume that ATS $\bar{v}_s(q)$ is a function of flow rate q . If the ATS at a mixed flow rate q is v , the equivalent passenger-car flow rate is the value of inverse of the speed-flow function at v :

$$q_E = \bar{v}_s^{-1}(v). \quad (6.5)$$

If $\bar{v}_s(q)$ is a linear function with slope $\bar{v}_s'$ the equivalent passenger-car flow rate can be calculated using the notation in figure 6.1:

$$q_E = q + \frac{v - v_{pc}}{\bar{v}_s'}, \quad (6.6)$$

where v is the ATS of the mixed flow and v_{pc} is the ATS of an equal flow rate (q) of passenger cars. The PCE is then

$$E_T = 1 + \frac{v - v_{pc}}{\bar{v}_s' P_T q}. \quad (6.7)$$

6.3 Theoretical observations

St. John (1976) and St. John and Kobett (1978) used the average speed of passenger cars as the measure of equivalence. They demonstrated that heavy vehicle adjustment should be nonlinear. Each incremental addition of heavy vehicles has a smaller effect on average passenger car speed, because the former additions have already depressed the speeds (see also McLean 1989).

For the first increment the equivalent flow is

$$q_{E1} = q(1 + \Delta r), \quad (6.8)$$

where

$$r = \frac{P_T}{100}(\epsilon - 1), \quad (6.9)$$

$$\Delta r = \frac{\Delta P_T}{100}(\epsilon - 1), \quad (6.10)$$

and ϵ is the form of equivalency (equivalence kernel) associated with the incrementally added trucks. If the effective percentage of the second incremental addition is not ΔP_T but $\Delta P_T q / q_{E1}$, the equivalent flow rate is

$$q_{E2} = q \left(1 + \Delta r + \frac{q \Delta r}{q_{E1}} \right) = q \left(1 + \Delta r + \frac{\Delta r}{1 + \Delta r} \right). \quad (6.11)$$

The relative equivalent flow rate after the i th addition is

$$Q_i = \frac{q_{Ei}}{q} = Q_{i-1} + \frac{\Delta r}{Q_{i-1}}. \quad (6.12)$$

The change in the relative equivalent flow rate is

$$\Delta Q_i = Q_i - Q_{i-1} = \frac{\Delta r}{Q_{i-1}}. \quad (6.13)$$

The differential form of the equation is

$$dQ = \frac{dr}{Q}, \quad (6.14)$$

which has a general solution

$$Q = \frac{q_E}{q} = \sqrt{2r + c}, \quad (6.15)$$

where c is a constant. Because $q_E/q = 1$ when $r = 0$, the constant is $c = 1$, and the particular solution of the differential equation is (St. John 1976)

$$\frac{q_E}{q} = \sqrt{2r + 1}, \quad (6.16)$$

which gives a nonlinear adjustment factor

$$f_T = (2r + 1)^{-\frac{1}{2}} = \left(\frac{P_T}{50} (\epsilon - 1) + 1 \right)^{-\frac{1}{2}}. \quad (6.17)$$

The impact of heavy vehicles on ATS has been estimated by using the model of Erlander (see section 5.1.1). The ATS is the average travel speed of mixed traffic. The mean desired speed of passenger cars was assumed as 90 km/h and of heavy vehicles as 80 km/h. The standard deviation of desired speeds was 10% of the mean desired speed. The difference of ATS between passenger-car traffic and mixed traffic was calculated for 15 percent heavy vehicles (figure 6.2). The respective PCEs are displayed in figure 6.3. It should, however, be remembered that Erlander's model does not consider platooning, which is a serious limitation when considering the impact of heavy vehicles.

As the flow rate increases, the marginal impact of a given heavy vehicles decreases. The PCE is highest at low flow rates, decreases steeply, and then starts to increase slowly as the flow rate increases. The high PCE value at low flow rates is not caused by a large impact of heavy vehicles, but by the effect of slow vehicles to the statistical average speed. As opposing flow rate increases, the PCEs decrease. When the proportion of no-passing zones is very high, the PCEs are not sensitive to opposing flow rates.

An interesting feature of the model is that PCEs are high under good passing conditions; i.e., when flow rates in both directions are low and there are few no-passing zones. Under the assumptions of the model, high flow rates in both

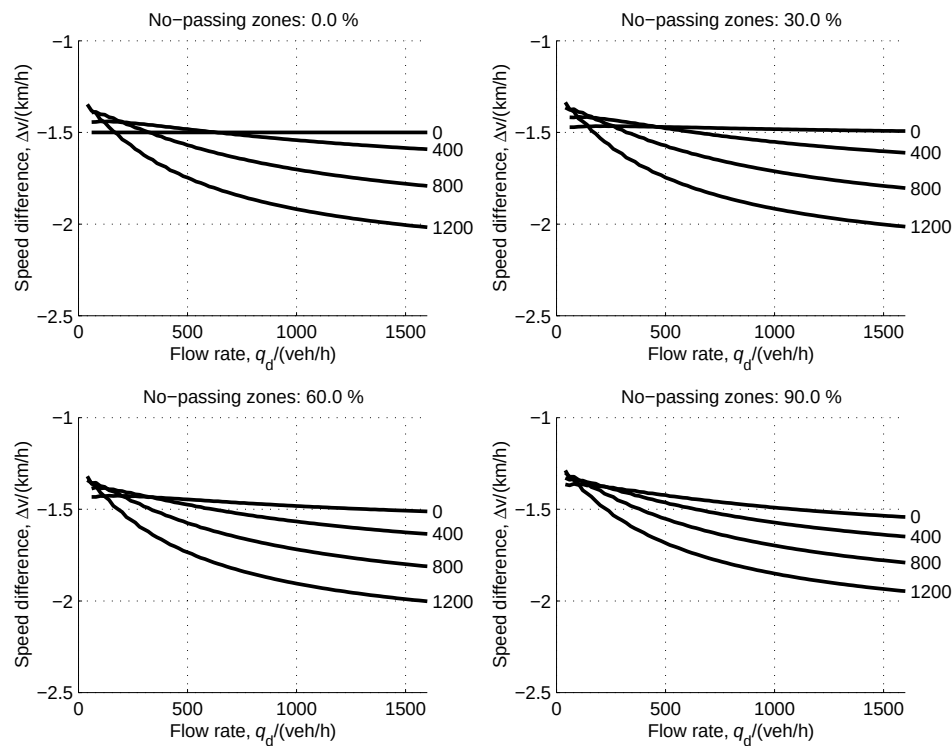


FIGURE 6.2 · The impact of 15 % heavy vehicles on ATS at opposing flow rates 0, 400, 800, and 1200 veh/h according to Erlander's model

directions and a high percentage of no-passing zones decrease the average speed of passenger-car flows in such a degree that the impact of heavy vehicles slightly decreases (figure 6.2), when compared to roadway conditions with no passing restrictions. This effect could be larger, if the platooning model was more realistic.

Jacobs (1974) used an integral equation approach similar to the model of Erlander (see section 5.1.1). His model did not consider no-passing zones, but the model was formulated to describe the effect of platooning behind slow vehicles. He used the average speed of passenger cars with the highest desired speed (130 km/h) as a measure for quality of service. The results indicated that the PCE values decreased as the truck percentage increased. He also observed that, if the speed approached the lowest desired speed of passenger cars, the PCE of trucks tended to infinity. For lower speeds the PCEs could not be defined.

This observation of Jacobs (1974) can be applied to average travel speeds: It is possible that under uncongested conditions no passenger-car flow rate has the average travel speed of a given mixed traffic flow, as figure 6.1 reveals. The figure also indicates that at congested conditions different methods should be used to estimate the PCEs.

McLean (1989) argues that heavy vehicle impedance is at a maximum for

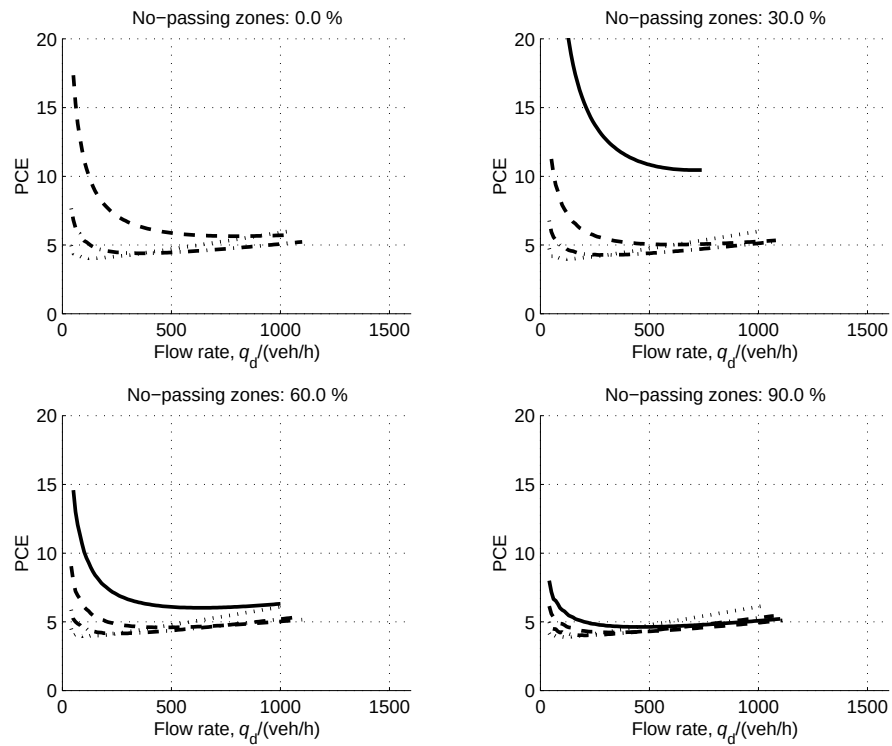


FIGURE 6.3 · Passenger-car equivalencies for 15 % heavy vehicles at opposing flow rates 0 (solid), 400 (dashed), 800 (dashed-dotted), and 1200 veh/h (dotted) according to Erlander's model

moderate flows, steep grades, and constrained passing opportunities. At low flows and ample passing opportunities, heavy vehicle impedance is relatively small because catch-ups are infrequent and passing delays are small (see also Huber 1982, Sumner, Hill and Shapiro 1984). This phenomenon is demonstrated in figure 6.2, although Erlander's model includes heavy vehicles in the ATS.

At moderate flows and constrained passing opportunities catch-ups are frequent and passing is difficult. At flow rates approaching capacity the heavy vehicle impedance is high in difficult terrain, when low speeds of heavy vehicle coupled with only a few passing opportunities lead to a marked reduction in capacity. In level terrain the impact of heavy vehicles should be less than at moderate conditions, because the passenger-car speeds are reduced regardless of the heavy vehicles.

Figure 6.2 indicates that the *increase* in heavy vehicle impact decreases at high flow rates, but Erlander's model does not indicate any decrease in the heavy vehicle impact. The results could be different, if the model considered platooning.

6.4 Estimation of passenger-car equivalencies

6.4.1 General considerations

Since the 1965 HCM (Highway Research Board 1965) the impact of heavy vehicles has been described in terms of passenger-car equivalencies (PCEs). In recent years this approach has been found more complex than originally assumed:

1. The PCEs should be defined in the same terms as the level of service. Accordingly, separate PCEs should be estimated for capacity, speed and platooning analyzes (van Aerde and Yagar 1984).
2. It is possible that at certain conditions PCEs are infinite or undefinable (Jacobs 1974). The quality of service in mixed traffic may be such that it does not occur at any flow rate of passenger cars only (see section 6).
3. PCEs vary not only with roadway conditions, such as terrain type, but also with traffic conditions, such as heavy vehicle percentage and even flow rate. This was the main reason, why the PCE concept has been rejected in the German guidelines (Brilon, Großmann and Blanke 1994, Brilon and Weiser 1998). The problem has also been acknowledged in the preparation of the HCM 2000 (Harwood et al. 1999).

In directional analysis the impact of heavy vehicles in the opposing flow should also be estimated. The estimation of PCEs on a directional basis was suggested by van Aerde and Yagar (1984). The impact of heavy vehicles may also be different depending on the free-flow speed or posted speed limit on the highway. Even in level terrain the differential speed limits reduce the speeds of heavy vehicles. There may also be interactions between various vehicle types (Sumner et al. 1984). In a vehicle mix of passenger cars, trucks, and recreational vehicles the impact of the latter vehicle type may be less than in an equivalent mix of passenger cars and recreational vehicles only.

HCM 2000 will have only two types of heavy vehicles: trucks and recreational vehicles (RVs) (Harwood et al. 1999). Buses can be considered as trucks. The impact of RVs on level and rolling terrain is very small. No impact is considered for PTSF. For average travel speed $E_R = 1.1$ in rolling terrain. In level terrain RVs can be considered as passenger cars.

6.4.2 Mixed traffic speeds vs. passenger-car speeds

HCM 2000 will use the ATS of mixed traffic as a level-of-service measure (Harwood et al. 1999). The German guidelines (Brilon et al. 1994), however, measure the ATS of passenger cars. The latter approach had been adopted also by St. John and Kobett (1978) and Huber (1982). Figure 6.4 displays the difference between these two methods. McLean (1989) points out a theoretical difficulty in the first approach: The PCEs are very large at low flow rates when impedance effects are negligible (see also Huber 1982). He does not, however, consider this as a great

problem provided that the method of derivation is clearly stated and the PCE values are applied consistently.

St. John (in Huber 1982) noted that, if the car-only form is used, there is no direct measure of the speed depression experienced by trucks due to a steep sustained upgrade. This is a problem especially at low-flow conditions, when trucks do not have a major impact on passenger-car speeds. If the speeds of heavy vehicles at low-flow conditions are a major concern, it is possible to make a separate model of heavy-vehicle speeds (Duncan 1974, Kehittämiskeskus 1995).

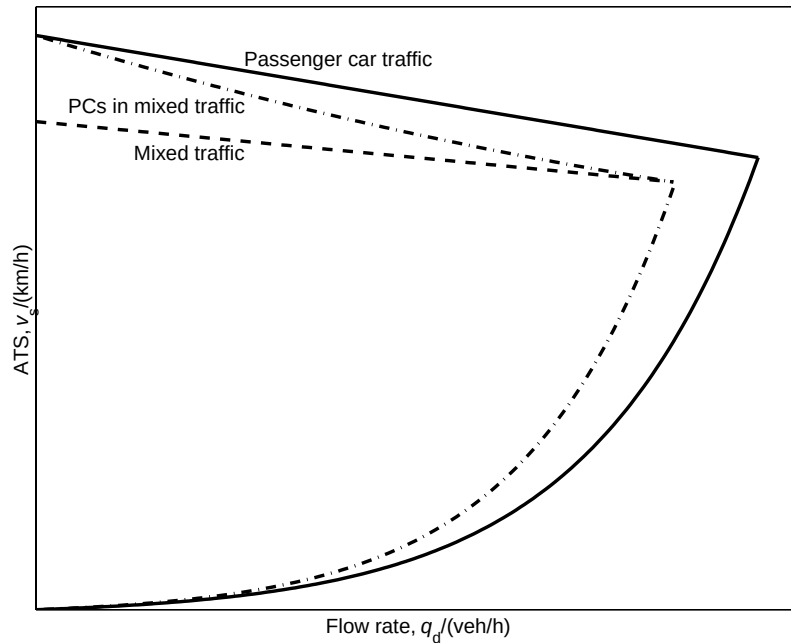


FIGURE 6.4 · Average travel speed of passenger cars, mixed traffic, and passenger cars in mixed traffic

The ATS of mixed traffic at low flow rates and a high percentage of heavy vehicles may be low because of the differential speed limits (see figure 6.2 and 6.3). If the ATS of all vehicles was used, this would indicate a high PCE (fig. 6.3) and a low level of service even though practically all vehicles traveled at their desired speed. The ATS used as a level-of-service measure should be the *average travel speed of passenger cars* in the traffic flow. The heavy vehicle adjustment for PTSF is naturally based on the PTSF in the mixed traffic flow.

6.4.3 Estimation methods

The data, collected from field measurements, does not allow the use of similar methods as in simulation experiments (Sumner et al. 1984, Elefteriadou, Torbic and Webster 1997). van Aerde and Yagar (1984) suggested multiple linear regres-

sion analysis, which can be expressed in terms of ATS and flow rates as:

$$\hat{v}_s = b_0 + b_1 q_C + b_2 q_T + b_3 q_R + b_4 q_O + b_5 q_o, \quad (6.18)$$

where b_i are regression coefficients (b_0 is the free-flow speed), q_C is the flow rate of passenger cars, and q_T , q_R , q_O , q_o are the flow rates of trucks, RVs, other vehicles, and opposing vehicles, respectively. PCEs were estimated as the ratio

$$E_i = \frac{b_i}{b_1}. \quad (6.19)$$

Considering the discussion about the impacts of heavy vehicles in section 6, this linear approach is not considered adequate.

The linear regression models of Duncan (1974) for the speeds of light vehicles (v_L) and heavy vehicles (v_H)

$$v_L = b_{0,L} + b_{1,L} q_{d,L} + b_{2,L} q_{d,H} + b_{3,L} q_{o,L} + b_{4,L} q_{o,H} \quad (6.20)$$

$$v_H = b_{0,H} + b_{1,H} q_{d,L} + b_{2,H} q_{d,H} + b_{3,H} q_{o,L} + b_{4,H} q_{o,H} \quad (6.21)$$

were considered unreliable. For two-lane highways the PCE varied from 31 to -22. This result supports the conclusion above.

Equation (6.2) on page 44 stated that

$$E_T = 1 + \frac{q_E - q}{P_T q}, \quad (6.22)$$

where q_E is the equivalent passenger-car flow rate of a mixed flow rate q with a fraction P_T of trucks. If sample j has a mixed flow rate q and a level-of-service measure α_j , the corresponding LOS measure at passenger-car flow rate q is $\alpha(q)$, and the slope (α') of the α -flow curve is constant, the PCE can be expressed as

$$E_T = 1 + \frac{\alpha_j - \alpha(q)}{\alpha' P_T q}, \quad (6.23)$$

which is essentially the equation (6.7). This equation can be applied also in the case that the measure of LOS is the ATS of passenger-cars.

The approach in the HCM 2000 is a direct application of the above equation with a correction for the effect of grade. The slope of the PTSF-flow curve is not constant, but for low heavy vehicle percentages and low PCEs the error due to nonconstant slope can be considered insignificant. This approach will be used below.

ATS and PTSF have separate passenger-car equivalents for trucks and recreational vehicles, as suggested by van Aerde and Yagar (1984). The passenger-car equivalency of trucks for average travel speed is (Harwood et al. 1999)

$$E_{T,v} = 1 + \frac{[\bar{v}_s(q, g, P_T) - \bar{v}_s(q, g)] f_{g,v}}{-0.0125 P_T q}, \quad (6.24)$$

where $\bar{v}_s(q, g, P_T)$ is the ATS of mixed flow q of passenger cars and trucks in grade g , and P_T is the proportion of trucks in the flow. The PCE for PTSF is

$$E_{T,W} = 1 + \frac{[P_W(q, g, P_T) - P_W(q, g)] f_{g,W}}{0.0879 P_T q e^{-0.000879 q / f_{g,W}}}, \quad (6.25)$$

where $P_W(q, g, P_T)$ is the PTSF of mixed flow q of passenger cars and trucks in grade g .

The speed-flow curve and PTSF-flow curve should, however, be estimated for each location. Consequently, for average travel speed the PCE of sample j at location i is:

$$E_{T,i,j}(q, P_T) = 1 + \frac{\bar{v}_{s,pc,i,j} - \bar{v}_{s,pc,i}(q)}{P_T q \bar{v}'_{s,pc,i}} \quad (6.26)$$

where

P_T = the fraction of trucks in the sample

q = flow rate of the sample

$\bar{v}_{s,pc,i}(q)$ = ATS of passenger-car flow rate q at location i

$\bar{v}_{s,pc,i,j}$ = ATS of passenger cars in the sample

$\bar{v}'_{s,pc,i}$ = slope of the speed-flow curve at location i .

The PTSF-flow curve $P_{W,pc}(q)$ in HCM 2000 is not linear, but the logarithmic transform $\phi_{pc}(q) = \ln[1 - P_{W,pc}(q)]$ is linear. This transform can be used to estimate the PCE:

$$E_{T,i,j}(q, P_T) = 1 + \frac{\phi_{i,j} - \phi_{pc,i}(q)}{P_T q \phi'_{pc,i}}, \quad (6.27)$$

where

$\phi_{pc,i}(q)$ = logarithmic transform of PTSF for PCE flow rate q at location i

$\phi_{i,j}$ = logarithmic transform of PTSF in the sample; i.e., $\ln(1 - P_{W,i,j})$

$\phi'_{pc,i}$ = slope of the $\phi_{pc,i}(q)$ curve.

7

Drivers' perception

This chapter discusses some empirical research conducted in Finland about drivers' perception of the quality of flow. Although this discussion does not have a direct connection with the discussion above, it has been included because of the increasing international interest in the subject.

Kiljunen and Summala (1996) studied the applicability of the Alert C protocol (free/heavy/slow/queueing/stationary traffic) that had been proposed for the RDS-TMC traffic information system. The Alert C traffic conditions can be defined as follows:

1. *Traffic flowing freely*—Average travel speed is greater than 90 % of free-flow speed.
2. *Heavy traffic*—Average travel speed is 75–90 % of free-flow speed.
3. *Slow traffic*—Average travel speed is 25–75 % of free-flow speed.
4. *Queueing traffic*—Average travel speed is 10–25 % of free-flow speed.
5. *Stationary traffic*—Average travel speed is less than 10 % of free-flow speed.

Kiljunen and Summala suggested a Finnish modification: fluent / platooning / slow / queueing / stationary. In addition, the “fluent” situation could be further divided into “free” and “fluent”.

Journey speed did not explain perceived traffic conditions in a better way than the parameters measured at one spot. Fifty percent of car drivers reported that their driving comfort decreased when the average headway was shorter than five seconds, or when the average spot speed was eight percent slower than free flow speed. The results suggested that traffic was not irritating because of growing density but because of driver and trip related factors and the behaviour of other drivers.(Kiljunen and Summala 1996)

According to Kiljunen and Summala (1998) drivers' perception of traffic conditions is connected with design standard of highways, speed levels, illumination, drivers' expectations and backgrounds. When drivers expect high traffic volumes

they also consider traffic more fluent under high volumes. During bad weather and in the dark the quality of traffic was considered lower. These observations make it difficult to directly connect "traffic quality" to the HCM levels of service.

With some qualifications they suggested that traffic condition *fluent* in the modified Finnish Alert C classification corresponds to levels of service A-C in the HCM. *Platooning* corresponds to levels of service D-E. Level of service F include traffic conditions *slow*, *queueing*, and *stationary* traffic. When spot speeds stayed above 77–78 km/h, more than half of the drivers considered traffic conditions "fluent". Fifty percent of car drivers considered the traffic "fluent" when the difference between target speed and driving speed was less than 10 km/h.

Luoma (1998) studied the estimation of transportation system efficiency. The best efficiency indicator for passenger car drivers seemed to be the difference between individual, condition dependent target speed and driving speed. The target speed was calculated using the driver estimated arrival time and the distance to destination. When the difference was more than 20 km/h, only 0–10% of drivers considered the transportation system efficient. For commercial vehicle operators the predictability of travel times was more important.

The predictability of truck travel times have been studied with the help of tachometers (Luoma and Jaatinen 1999). It was found that the Finnish IVAR calculation method, which follows the 1985 HCM (Transportation Research Board 1985), on average overestimates the travel times of trucks. The definition of confidence limits for travel times, however, requires further research.

In addition to driver related factors, the government is interested on total social costs of transportation. Finnra (Keskushallinto 1995) has studied the estimation of optimal travel speed; i.e., the speed that minimizes the social transportation costs under prevailing conditions. It was, however, found that transportation economic calculations do not provide an adequate basis for defining the optimal speed level.

Finnra (Kehittämiskeskus 1993) has defined a four stage quality classification for urban arterials. The classification is defined in terms of traffic quality and environmental quality. The traffic quality factors are shortly stated as follows:

1. *Good*—Traffic is fluent and safe. Travel speed is good.
2. *Fair*—Momentary congestion is possible and drivers must occasionally be especially on the alert. During congestion travel speeds decrease a little.
3. *Tolerable*—Traffic is congested during peak hours, and drivers must be especially on the alert. During congestion travel speeds are considerable lower than free-flow speeds.
4. *Bad*—Traffic becomes frequently congested and goes at crawl speed even at non-peak hours.

8

Conclusions and recommendations

8.1 Directional analysis of traffic flows

Total flow and directional split give an unequivocal description of traffic flow in both directions. For passing opportunities opposing flow rate is, however, a more important factor than directional distribution.

The effect of heavy vehicles depends on the direction of flow studied. As the proportion of heavy vehicles increases, passing demand and platooning in the same direction increases also. Platooning, however, provides more passing opportunities for the opposing flow. This twofold effect is hidden, if only total flow is analyzed.

Density is not a meaningful measure for two-way flows on two-lane highways with a skew directional distribution, because the major flow cannot effectively use the free space on the opposite lane. Let us consider an extreme example: If the directional split is 100/0 and traffic in one direction is congested, the density of the roadway is half of the density of the congested lane. At this density traffic should flow rather smoothly. Directional analysis avoids the averaging of the characteristics of traffic flow.

If two-way analysis is based on a proper combination of directional results (see section 4.2), the two approaches are compatible. Separate analyses with different mathematical models may produce incompatible results (see Luttinen 2001).

It is suggested that **the analysis of two-lane highways is based on directional analysis**. Directional analysis is also compatible with the multilane highway methodology in HCM 2000.

8.2 Percent time spent following

The exponential model (section 4.2) indicates that the directional distribution has an effect on two-way PTSF even in random traffic; i.e., with no vehicular interaction. The section also presents a proper method to combine directional PTSF

measures into two-way PTSF measures. The adjustment for directional distribution should, of course, keep the PTSF within the 0–100 % range.—For problems in the HCM 2000 method see Luttinen (2001).

8.3 *Speed-flow curves*

Theoretical results by (Erlander 1967) and (Jacobs 1974) have suggested that the speed-flow curve has a concave shape. This indicates that the marginal impact of each additional vehicle decreases as flow rate increases. Concave curves have also been obtained by empirical research in countries such as Finland (Lut:2000) and Germany (Brilon:1998).

The study of the standard deviation of average travel speeds in section 5.2 indicated that the speed-flow curve has a concave shape and also that the slope of the curve is steeper, if the free-flow speed is high. This leads to the following conclusion: **The speed-flow curves on two-lane highways have a concave shape so that the speed decrease is steepest at low flow rates and highways with high free-flow speeds.**

The proposed theoretical models cannot be expected to give good results at very high flow rates. Empirical data at congested conditions is also scarce. The speed-flow relationship at congested and near-congested conditions deserves further empirical and theoretical research.

8.4 *The impact of heavy vehicles*

If the ATS of all vehicles is used as a service measure, the ATS may be low even though practically all vehicles traveled at their desired speed. **The ATS used as a level-of-service measure should be the average travel speed of passenger cars in the traffic flow.** The heavy vehicle adjustment for PTSF can be based on the PTSF in the mixed traffic flow.

It is possible that a given ATS does not occur at any given uncongested passenger-car flow rate. In that case it is not possible to estimate the PCEs. Also, under low flow rates the PCE is not very descriptive. The impact of heavy vehicles on average passenger-car speeds is low, but so is also the impact of a few additional passenger cars. Consequently, the PCEs may be very high. It is suggested that **the impact of heavy vehicles is expressed in terms of changes in PTSF and the ATS of passenger cars.**

It is evident that the impact of heavy vehicles depend on the proportion (or number of) heavy vehicles, flow rate in the observed direction, flow rate in the opposing direction, hilliness of the road, and the percent no-passing zones. Theoretical studies suggest that

1. The marginal impact of heavy vehicles decreases as flow rate and/or the proportion of heavy vehicles increases.

2. The effect of opposing flow decreases as the percent passing zones increases, but the impact of no-passing zones is not very high.

The current state of knowledge is not, however, sufficient. **The dynamics of heavy-vehicle impact deserve further theoretical and empirical research.**

8.5 Drivers' perception

As well as traffic flow theories should be verified against empirical data, the evaluation of the results should be based on the perceptions of the road users. Even "correct" models for PTSF and ATS do not reveal an appropriate level of service classification neither the service measures nor the threshold values for the service measures. It is necessary to **base the level-of-service definitions on the perceptions of road users.**

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